ATTACHMENT 13 ENGINEERING DOCUMENTS

(CONTINUED)

AECOM

J.K. SPRUCE POWER PLANT -

PLANT DRAINS POND -

Alternative Composite Liner Design Documentation

June 2022
AECOM Project 60566130

Delivering a better world

Prepared for:

CPS Energy Calaveras Power Station
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J.K. SPRUCE POWER PLANT
ALTERNATIVE COMPOSITE LINER DESIGN DOCUMENTATION
TAC Title 30, Part 1, § 352, Subchapter F, § 352.721 and 40 CFR § 257.72
PLANT DRAINS POND (PDP)

Liner Design Criteria	Liner Documentation
30 TAC §352.721 Liner Design Criteria for New and Lateral Expansions of Coal Combustion Residuals Surface Impoundments. *The commission adopts by reference 40 Code of Federal Regulations §257.72 (Liner design criteria for new CCR surface impoundments and any lateral expansion of a CCR surface impoundment) as amended through the April 17, 2015, issue of the Federal Register (80 FR 21301).* ***40 CFR § 257.72 (a) New CCR surface impoundments and lateral expansions of existing and new CCR surface impoundments must be designed, constructed, operated, and maintained with either a composite liner or an alternative composite liner that meets the requirements of § 257.70(b) or (c).*** § 257.70 (b) *A composite liner must consist of two components; the upper component consisting of, at a minimum, a 30-mil geomembrane liner (GM), and the lower component consisting of at least a two-foot layer of compacted soil with a hydraulic conductivity of no more than 1x10^{-7} centimeters per second (cm/sec). GM components consisting of high density polyethylene (HDPE) must be at least 60-mil thick. The GM or upper liner component must be installed in direct and uniform contact with the compacted soil or lower liner component. The composite liner must be* *(1) Constructed of materials that have appropriate chemical properties and sufficient strength and thickness to prevent failure due to pressure gradients (including static head and external hydrogeologic forces), physical contact with the CCR or leachate to which they are exposed, climatic conditions, the stress of installation, and the stress of daily operation;* *(2) Constructed of materials that provide appropriate shear resistance of the upper and lower component interface to prevent sliding of the upper component including on slopes;* *(3) Placed upon a foundation or base capable of providing support to the liner and resistance to pressure gradients above and below the liner to prevent failure of the liner due to settlement, compression, or uplift; and* *(4) Installed to cover all surrounding earth likely to be in contact with the CCR or leachate.*	Texas Administrative Code adopts by reference 40 CFR § 257.72. 40 CFR § 257.72 allows for a new surface impoundment to be constructed with a composite liner that meets the requirements of 40 CFR § 257.70 (b) or (c). This documentation demonstrates that the Plant Drains Pond (PDP) is designed with an alternative composite liner that meets the requirements of 40 CFR § 257.70 (c). The upper component of the alternative composite liner is a 60 mil HDPE geomembrane. The lower component is a CETCO Resistex 200 FLW9 geosynthetic clay liner (GCL), which is a dry-blended, polymer-treated GCL with a manufacturer-certified hydraulic conductivity (k) of 3 x 10^{-9} cm/sec (ASTM D5887) and a reported thickness (t) of 0.8 cm (CETCO, personal communication). When exposed to a composite leachate prepared from CPS Spruce Plant CCR and FGD, compatibility testing yielded the result of 7.59 x 10^{-10} cm/sec (ASTM D6766) after 858.2 hours and 3.2 pore volumes, at which time the test was terminated. Using Equation 1 in § 257.70 (c) (2), using the site-specific leachate compatibility testing result of 7.59 x 10^{-10} cm/s, the liquid flow rate (Q) was calculated as Q = 23.88 cm^3/sec for the CETCO Resistex 200 FLW9 GCL and was calculated as Q = 52.58 cm^3/sec for two feet of 1 x 10^{-7} cm/sec compacted soil; in the calculations, the pond surface area (A) was established as 2.83 acres, hydraulic conductivity (k) and thickness (t) for the GCL were obtained from manufacturer data and leachate-specific conductivity testing, 2 feet (60.96 cm) of compacted soil, and hydraulic head (h) acting on the two liners was specified as 7.19 feet (219.15 cm), which is height of the maximum normal pond operating level above the upper surface of the installed liner. The alternative composite liner is constructed of materials that have appropriate chemical properties and sufficient strength and thickness to prevent failure due to pressure gradients, physical contact with CCR or leachate to which they are exposed, climatic conditions, the stress of installation, and the stress of daily operation. The pond will have 3.5:1 side slopes, a gently sloping base, and a maximum side-slope height of approximately 9 feet. The configuration and application of liner materials in the PDP are well-demonstrated and conventional. In daily operation, the alternative composite liner system on the base of the pond will be subject to the weight of a12-inch sand protective layer, a 6-inch concrete working surface, and the loaders and trucks used to muck out the solids. The alternative composite liner is constructed of materials that provide appropriate shear resistance between the upper and lower components to prevent sliding on the 3.5:1 side slopes. The HDPE is textured to increase friction between the geomembrane and the GCL; both components are anchored by an anchor trench. Sliding of the liner components is not considered to be a possible failure mechanism. The alternate composite liner is founded on a minimum 1-foot thick over-excavated layer that is compacted to at least 95% of the maximum dry density as per Standard Proctor ASTM D698. In addition, a minimum of 1-foot below the over-excavation, subgrade is scarified and compacted to at least 95% of the maximum dry density as per Standard Proctor ASTM D698. The native formation below the compacted soil is fine-grained soils, which, in turn are underlain by clayey to silty clayey sands. The native soils are assessed to be competent and capable of supporting the loads and stresses of pond construction and operation. The alternative composite liner covers the entire surface impoundment surface and extends beyond the top of the embankments into an anchor trench. The height of the pond embankments allows for 2 feet of freeboard above the maximum normal operating level.
257.70 (c) If the owner or operator elects to install an alternative composite liner, all of the following requirements must be met: (1) An alternative composite liner must consist of two components; the upper component consisting of, at a minimum, a 30-mil GM, and a lower component, that is not a geomembrane, with a liquid flow rate no greater than the liquid flow rate of two feet of compacted soil with a hydraulic conductivity of no more than 1x10^{-7} cm/sec. GM components consisting of high density polyethylene (HDPE) must be at least 60-mil thick. If the lower component of the alternative liner is compacted soil, the GM must be installed in direct and uniform contact with the compacted soil. (2) The owner or operator must obtain certification from a qualified professional engineer that the liquid flow rate through the lower component of the alternative composite liner is no greater than the liquid flow rate through two feet of compacted soil with a hydraulic conductivity of 1x10^{-7} cm/sec. The hydraulic conductivity for the two feet of compacted soil used in the comparison shall be no greater than 1x10^{-7} cm/sec. The hydraulic conductivity of any alternative to the two feet of compacted soil must be determined using recognized and generally accepted methods. The liquid flow rate comparison must be made using Equation 1 of this section, which is derived from Darcy's Law for gravity flow through porous media.	

Certification Statement 30 TAC §352.721 and 40 CFR § 257.72(c) – Design of the Liner for a New CCR Surface Impoundment

CCR Unit: CPS Energy; Spruce Plant; Plant Drains Pond

I, Alexander W. Gourlay, being a Registered Professional Engineer in good standing in the State of Texas, do hereby certify, to the best of my knowledge, information, and belief, that the information contained in this certification has been prepared in accordance with the accepted practice of engineering. I certify, for the above-referenced CCR Unit, that the documentation as to whether the construction of the CCR Unit meets the requirements of 30 TAC §352.721 and 40 CFR § 257.72(a) is accurate.

Alexander W. Gourlay, P.E.

Printed Name

June 28, 2022

Date

ALEXANDER W. GOURLAY

143733

6/28/22

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GEOTECHNICAL ENGINEERING STUDY

FOR

J.K. SPRUCE –CALAVERAS LAKE POWER PLANT
PROPOSED NEW COAL COMBUSTION RESIDUAL PONDS
SAN ANTONIO, TEXAS

RABA KISTNER CONSULTANTS

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Project No. ASA17-096-00
February 5, 2018

Mr. Eric R. Olson
CPS Energy
c/o Mr. Steve Dean, P.E.
Pape-Dawson Engineers, Inc.
2000 NW Loop 410
San Antonio, Texas 78213

RE: Geotechnical Engineering Study
J.K. Spruce - Calaveras Lake Power Plant
Proposed Two New Coal Combustion Residual Containment Ponds
San Antonio, Texas

Dear Mr. Dean:

Raba Kistner Consultants Inc. (RKCI) is pleased to submit the report of our Geotechnical Engineering Study for the above-referenced project. This study was performed in accordance with RKCI Proposal No. PSA17-189-00, dated December 7, 2017. The purpose of this study was to drill borings within the approximate footprint of the proposed Coal Combustion Residual (CCR) containment ponds, to perform laboratory testing to classify and characterize subsurface conditions, perform a geophysical survey to evaluate the seismic response of the underlying geometrical and to prepare an engineering report presenting our findings and recommendations for the proposed CCR ponds.

We appreciate the opportunity to be of service to you on this project. Should you have any questions about the information presented in this report, or if we may be of additional assistance with value engineering or on the materials testing-quality control program during construction, please call.

Very truly yours,

RABA KISTNER CONSULTANTS, INC.

Samrawit Haile.

Sam Haile, E.I.T.
Graduate Engineer

ERIC J. NEUNER
116311

2/5/2018

Eric J. Neuner, P.E.
Associate | Manager, San Antonio Engineering

SH/EJN/kv

Attachments

Copies Submitted: Electronic - PDF

GEOTECHNICAL ENGINEERING STUDY

For

J.K. SPRUCE – CALAVERAS POWER PLANT
PROPOSED NEW COAL COMBUSTION RESIDUAL CONTAINMENT PONDS
SAN ANTONIO, TEXAS

Prepared for

PAPE-DAWSON ENGINEERS, INC.
San Antonio, Texas

Prepared by

RABA KISTNER CONSULTANTS, INC.
San Antonio, Texas

PROJECT NO. ASA17-096-00

February 5, 2018

RABAKISTNER

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ATTACHMENTS

PROJECT DESCRIPTION

We understand that two (2) new Coal Combustion Residual (CCR) containment ponds are proposed at the existing J.K. Spruce Power Plant. In general, the containment ponds will be located north and east of the existing power plant and west of Calaveras Lake, see Figure 1. Conceptually, the containment ponds will have dimensions of approximately 325 ft by 550 ft in plan view and the bottom may extend to depths of approximately 10 ft below the existing ground surface (or 5 feet above the upper limit of the observed groundwater surface). Currently, the existing ground surface slopes downward to the east and south with approximately 18 ft. of vertical relief.

The containment ponds will be lined and berms with maximum heights up to 6 ft are anticipated to extend above the lowest existing ground surface (approximately El 499 ft msl). We anticipate that the berms will be sloped at 1 Vertical (V) to 3 Horizontal (H), and an approximately 10-foot wide crest will be constructed. We assume that the berms will be tapered to accommodate the elevated grade change to the west.

We understand that CPS maintains the Calaveras Lake at a target pool elevation of El 485 ft msl with periodic fluctuations of plus or minus one foot. Levels above the target pool elevation are usually due to rainfall in the Calaveras Creek, Hondo Creek and Chupaderas Creek watersheds, and typically return to the target pool elevation within a few days of precipitation.

On the basis of historic aerial photographs, available from Google Earth, it appears that the site has been previously developed. Previous developments appeared to consist of a parking area, yard, and some other structures. Currently, the site appears to be covered with grass and a concrete slab. A water fill pond is present south and east of the proposed containment ponds.

RISK

The geotechnical engineering recommendations contained in this memorandum are intended to provide Pape-Dawson Engineers, Inc; CPS Energy; and the U.S. Environmental Protection Agency with information pertaining to the stability of the proposed CCR containment ponds at the referenced site.

The geotechnical properties of the soils encountered in this study involve variability. The selection of analysis parameters for this project was based on a review of the available geotechnical data, our knowledge of the project area, and design calculations using select surveyed geometries. The results of our analyses were then reviewed with respect to important trends and general concepts, keeping these conditions and limitations in mind. Our conceptual recommendations are based on a conservative approach as is warranted for the analyses.

BORINGS AND LABORATORY TESTS

Subsurface conditions at the site were evaluated by eleven borings drilled at the locations shown on the Boring Location Map, Figure 1. At seven of the boring locations, temporary monitoring wells (MW-series borings) were installed to observe groundwater levels over a relatively short time period (approximately 3 weeks after drilling) and to perform pump tests to calculate the underlying material hydraulic conductivity. The boring locations and elevations were surveyed by Pape-Dawson Engineers.

The surveyed ground surface elevation at each of the boring locations is listed in the table below as well as the approximate bottom elevation of each boring. Boring coordinates are provided on the provided boring logs.

Boring No.	Ground Surface Elevation (ft msl)	Approximate Boring Depth (ft)	Boring Bottom Elevation (ft, msl)
B-1	510.10	50	460.10
B-2	506.18	50	456.18
B-3	513.40	50	463.40
B-4	510.00	50	460.00
MW-1	513.91	35	478.91
MW-2	508.83	35	473.83
MW-3	516.86	35	481.86
MW-4	503.80	20	483.80
MW-5	503.36	35	468.36
MW-6	514.49	35	479.49
MW-7	500.22	35	465.22

The borings were drilled using a truck-mounted drilling rig. During drilling operations, Split-Spoon (with Standard Penetration Test), relatively undisturbed Shelby tube, and auger cutting samples were collected. Each sample was visually classified in the laboratory by a member of our geotechnical engineering staff. The geotechnical engineering properties of the strata were evaluated by the natural moisture content, Atterberg limits, swell, unconfined compression, sieve analysis with hydrometer tests, consolidation, hydraulic conductivities, triaxial and direct shear tests.

The results of the field and laboratory tests are presented in graphical or numerical form on the boring logs illustrated on Figures 2 through 12. A key to classification terms and symbols used on the logs is presented on Figure 13. The results of the laboratory and field testing are also tabulated on Figure 14 for ease of reference. Laboratory test results for the unconfined compression curves, one-dimensional consolidation, consolidated-undrained triaxial, and direct shear tests are presented on Figures 15, 16, 17, and 18, respectively.

Standard Penetration Test results are noted as “blows per ft” on the boring logs and Figure 14, where “blows per ft” refers to the number of blows by a falling hammer required for 1 ft. of penetration into the soil/weak rock (N-value). Where hard or dense materials were encountered, the tests were terminated at 50 blows even if one foot of penetration had not been achieved. When all 50 blows fall within the first 6 in. (seating blows), refusal “ref” for 6 in. or less will be noted on the boring logs and on Figure 14.

In addition, a Seismic Vs100 Geophysical Investigation was performed at the site to evaluate the average shear-wave velocity in the upper 100 ft of the geometrical to evaluate Seismic Site Class. The results of the geophysical investigation in presented in Appendix A.

GENERAL SITE CONDITIONS

GEOLOGY

A review of the *Geologic Atlas of Texas, San Antonio Sheet*, indicates that this site is naturally underlain with the soils/rocks of the Wilcox Group, which is composed of mudstone with varying amounts of sandstone and lignite. The Wilcox Group may weather to yellowish-brown clay, sandy clay, clayey sands, and sands.

The Wilcox Group grades downward into the Midway Group, which is composed of clay, silt, and sand, with some pebbles near its base. Glauconite is often encountered in these soils. **Key engineering considerations for development supported on the soils/rock of this formation typically include the presence of possible water-bearing layers, very hard mudstone/sandstone layers, and the expansive nature of the highly plasticity clays that can be present in this formation.**

STRATIGRAPHY

In general, the natural stratigraphy at this site consists of surficial sands that are underlain by fine-grained soils, which in turn are underlain by clayey to silty clayey sands. Exceptions include, Boring MW-1 where surficial sands were not observed, and Borings MW-6 and MW-7, where the fine-grained soil layer were not observed. **Cemented sands or sandstone were encountered at variable depths and intervals in our borings (annotated on our borings). In Boring MW-4, auger refusal on cemented sand/sandstone was encountered at a depth of 20 ft.** As previously discussed, the site has been previously developed. Although fill was not observed in our borings, remnants of past construction (localized fill materials that contain miscellaneous debris, utilities, abandoned foundations, rubble and other materials) should be anticipated during site grading.

Each stratum has been designated by grouping soils that possess similar physical and engineering characteristics. The boring logs should be consulted for more specific stratigraphic information. Unless noted on the boring logs, the lines designating the changes between various strata represent approximate boundaries. The transition between materials may be gradual or may occur between recovered samples. The stratification given on the boring logs, or described herein, is for use by RKCI in its analyses and should not be used as the basis of design or construction cost estimates without realizing that there can be variation from that shown or described.

The boring logs and related information depict subsurface conditions only at the specific locations and times where sampling was conducted. The passage of time may result in changes in conditions, interpreted to exist, at or between the locations where sampling was conducted.

GROUNDWATER

Groundwater observations are summarized in the following table.

Summary of Groundwater Observations

Boring No.	Ground Surface Elevation (ft msl)	Groundwater Elevation at Time of drilling (ft msl)	January 9, 2018 Groundwater Elevation (ft msl)	January 19, 2018 Groundwater Elevation (ft msl)	January 25, 2018 Groundwater Elevation (ft msl)
B-1	510.10	486.85	N/A	N/A	N/A
B-2	506.18	486.68	N/A	N/A	N/A
B-3	513.40	489.00	N/A	N/A	N/A
B-4	510.00	488.70	N/A	N/A	N/A
MW-1	513.91	488.71	489.41	488.51	489.19
MW-2	508.83	486.23	489.13	490.03	N/M
MW-3	516.86	489.36	490.96	490.96	490.72
MW-4	503.80	491.20	490.40	490.20	N/M
MW-5	503.36	486.56	487.46	488.16	486.89
MW-6	514.49	487.39	488.89	488.49	N/M
MW-7	500.22	488.32	489.02	488.62	488.79

N/A – Borings backfilled with grout after drilling.
N/R – Not measured.

As mentioned previously, this site is bounded to the west, south, and east by Calaveras Lake. The groundwater levels encountered at this site are most likely dominated by the surface water elevation of Calaveras Lake (El 485 ft msl). Fluctuations in groundwater levels are possible due to variations in rainfall and surface water run-off.

SEISMIC CONSIDERATIONS

Seismicity Discussion

In general, the site is located south and east of the Balcones Fault Zone (located generally north of the City of San Antonio). The Balcones Fault Zone extends approximately from the southwest part of the state near Del Rio, Texas to the north central region near Dallas, Texas along Interstate Highway 35 and consists of a northeast trending series of normal faults, which generally serves to contrast Upper Cretaceous rock formations in the southeast with Lower Cretaceous formations to the northwest. As a result of this large-scale, regional faulting, minor internal fault sequences and fractures exist throughout this zone that follow the same structural trend and accommodate localized displacement between rock units. The main tectonic events of the Balcones faulting are generally considered to have occurred during the Miocene epoch (27 to 12 million years ago), but there is considerable evidence that structural adjustments also took place during the earlier Cretaceous period, which ended approximately 66 million years ago (Abbott and Woodruff, 1986). On the basis of published literature, the Balcones Fault system has remained essentially inactive for nearly 15 million years, with the last major activity occurring during the Miocene. According to National Seismic Hazard maps developed by the U.S. Geological Survey (USGS, 2014), the Balcones Fault Zone is in one of the lowest-risk zones for earthquakes or other seismic hazards in the United States. Based on review of the 2014 USGS hazard map for the conterminous United States, the total number of earthquake-shaking events causing damage within the San Antonio and Austin regions, expected within a 10,000-year time period, is less than two. As San Antonio and Austin are fully contained within an "aseismic zone" as defined by the USGS, the probability that an earthquake of damage-causing magnitude will occur during the lifetime of structures presently being constructed is considered to be very low.

References:

1. Patrick Abbott and C. M. Woodruff, eds., *The Balcones Escarpment: Geology, Hydrology, Ecology* (San Antonio: Geological Society of America, 1986).

2. Edward Collins and Stephen Lauback, *Faults and Fractures in the Balcones Fault Zone* (Austin: Austin Geological Society, 1990).

3. Robert T. Hill, "The Geologic Evolution of the Non-Mountainous Topography of the Texas Region: An Introduction to the Study of the Great Plains," *American Geologist* 10 (August 1892).

4. E. H. Sellards, W. S. Adkins, and F. B. Plummer, *The Geology of Texas* (University of Texas Bulletin 3232, 1932).

5. *Grimshaw, Thomas W.; Charles Woodruff, Jr. (1986). "Structural Style in an En Echelon Fault System, Balcones Fault Zone, Central Texas: Geomorphologic and Hydrologic Implications". The University of Texas. Retrieved 2008-10-27.*

6. *"Peak Acceleration (%g) with 10% Probability of Exceedance in 50 Years". USGS. October 2002. Archived from the original on 2007-06-27.*

7. Balcones Escarpment from the *Handbook of Texas Online*. Retrieved 30 July 2015. Texas State Historical Association

8. Seismic-Hazard Maps for the Conterminous United States, 2014 (USGS Scientific Investigations Map 3325)

Developing Horizontal Peak Ground Acceleration

We understand that the CCR pond will be designed to withstand the peak ground acceleration with a 2% probability of exceedance (PE) in 50 years (mean return time of 2,475 years). The National Earthquake Hazards Reduction Program (NEHRP) interactive deaggregations models were used to obtain the probabilistic bedrock accelerations at the site. The NEHRP models consider ground motion from many sources surrounding the site location with the assumption that the site condition is rock with an average shear wave velocity of 2,500 ft/s. Bedrock spectral response acceleration at short periods (S_s), and at 1-second periods (S_1) of 0.091 g and 0.031 g, respectively, were obtained from the NEHRP models (Appendix B).

A detailed site-specific seismic hazard analysis was beyond our scope of services. The guidelines established by NEHRP were used to propagate the bedrock acceleration (2% PE in 50 years) to the ground surface (Per Section 11.4.2). On the basis of the average shear-wave velocity in the upper 100 ft (results presented in Appendix A), the geomaterial has a shear wave velocity ranging from 1,062 to 1,106 ft/second. Hence, the underlying soil profile within the upper 100 feet should be defined as Site Class D (Stiff Soil: Shear wave velocity range of 600 to 1,200 ft/second). Using Site Class D classification, the approximate surficial horizontal peak ground acceleration (HPGA) at this site is 0.075 g. The HPGA value of 0.075 g was used in our potential liquefaction analysis and berm global stability analysis for the seismic condition (presented later).

Liquefaction Potential

During an earthquake, sudden increases in pore water pressures can develop within saturated soil deposits due to seismic shaking. Where the increased pore water pressure exceeds the total overburden pressure loose and medium dense saturated sandy deposits may experience a sudden loss of strength, sometimes resulting in loss of bearing capacity, permanent lateral displacement, and/or settlement of the ground. This phenomenon is called soil liquefaction.

Based on the current subsurface exploration, loose to very dense sands are present below the upper cohesive soil layer. Groundwater is expected to be near the groundwater observations to date. For the liquefaction analyses, groundwater was considered to occur at El 491. The liquefaction potential assessment of sands was conducted using the "Simplified Procedure" developed by Seed and Idriss.[1,2] This method is based on extensive analyses of field data from sites that had been subjected to liquefaction from various earthquakes. The corrected blow count $(N_1)_{60}$ is a number standardized by hammer efficiency and normalized to an effective overburden pressure. A peak ground acceleration of 0.075g (as previously discussed) and estimated moment magnitude of 7.5 was used in the analyses.

SPT borings were drilled using a drill rig equipped with an automatic hammer. Based on documentation provided by EnviroCore Drilling, Inc., the drill rig hammer used at the site has an average efficiency of 86.9 percent. The efficiency of the automatic hammers was measured and evaluated by others. The provided efficiency of the automatic hammer was used in the liquefaction potential analyses.

A minimum factor of safety (FOS) of 1.1 between the computed and design Cyclic Stress Ratios (CSR) was used for liquefaction analysis. Based on the liquefaction analyses for Borings B-1 through B-4, presented in Appendix C, the site soils have a calculated FOS greater than the minimum target FOS of 1.1 (calculated FOS ranging from approximately 8 to 14). On the basis of these findings, RKCI believes the site soils have a very low risk of experiencing liquefaction due to an earthquake.

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[1] Seed, H.B. and Idriss, I.M. (1982). *Ground Motions and Soil Liquefaction During Earthquakes,* Earthquake Engineering Research Institute, CA.

[2] Seed, H.B., Tokimatsu, K., Harder, L.F. and Chung, R. H. (1985). "Influence of SPT Procedures in Soil Liquefaction Resistance Evaluation." *Journal of Geotechnical Engineering*, ASCE, Vol. 111, No.12, December, pp.1425-1455.

CCR POND DESIGN CONSIDERATIONS

ESTIMATED CCR POND BOTTOM

As discussed previously, the CCR Pond bottom may extend to depths of approximately 10 ft below the existing ground surface or 5 feet above the upper limit of the observed groundwater surface. On the basis of our groundwater observations to date, the highest groundwater reading was at approximately El 491 ft msl. For evaluation purposes, we assumed that the pond bottom may extend to approximately El 496. Therefore, we anticipate that excavations of approximately 4 to 21 ft may be required to construct the CCR pond. On the basis of the boring results and anticipated pond bottom, it appears the pond bottom (composite liner) may be founded on the underlying sand.

On the basis of the field pump tests performed on Borings MW-1, MW-3, MW-5, and MW-7 on January 25 and 26, 2018, the underlying sandy soils have field hydraulic conductivities ranging from $1.55x10^{-4}$ cm/sec to $9.56x10^{-4}$ cm/sec and are summarized in the following:

- MW-1: $9.56x10^{-4}$ cm/sec
- MW-3: $1.55x10^{-4}$ cm/sec
- MW-5: $5.31x10^{-4}$ cm/sec
- MW-7: $2.38x10^{-4}$ cm/sec

Collected intact Shelby tube samples tested in the laboratory had calculated hydraulic conductivities summarized in the following and annotated on the boring logs:

B-2 (depth 6 to 8 ft, sandy clay):	$1.88x10^{-7}$ cm/sec
B-3 (depth 3 to 5 ft, silty sand):	$2.05x10^{-6}$ cm/sec
MW-4 (depth 11 to 13 ft, silty sand):	$9.05x10^{-7}$ cm/sec

On the basis of the field and laboratory hydraulic conductivity tests, we anticipate that the lower component of the liner will need to consist of 2 ft of engineered fill capable of achieving a hydraulic conductivity of less than $1x10^{-7}$ cm/sec. Liner material considerations are presented in a later section.

ANTICIPATED MATERIAL FOR BERM CONSTRUCTION

Consideration may be given to using the onsite natural material to construct the berms. The natural materials are generally considered acceptable materials to use when constructing berms and slopes. In addition, the berms are not expected to be exposed to flowing water, other than rain that falls on the berm crest and berm slopes. The risk of berm failure due to erosion is considered to be very low. We recommend that vegetation be established on newly constructed slopes as quickly as possible. Care should be taken to prevent unnecessary disturbance to constructed slopes, as this can cause localized destabilization and erosion. Disturbance and/or erosion on finished slopes should be quickly repaired.

Excavation Equipment. In general, conventional excavation equipment is expected to be suitable for the excavation of the soils encountered in our borings. However, previous studies have encountered sandstone/cemented sand at varying depths in the vicinity of this site. **In Borings B-4, MW-1, and**

MW-6, sandstone/cemented sand material was encountered within or near the zone of the anticipated CCR pond bottom. Layers of mudstone, sandstone, and/or cemented sands/gravels are common in this area of San Antonio and therefore possible that these materials could be encountered during excavations. These layers are typically encountered at variable depths and with variable thicknesses. Although they can be massive, they are frequently present as isolated stringers or boulders. **Rock excavation equipment will be required where these layers are encountered.** Our boring logs are not intended for use in determining construction means and methods and may therefore be misleading if used for that purpose. We recommend that earth-work contractors interested in bidding on the work perform their own test in the form of test pits to determine the quantities of the different materials to be excavated, as well as the preferred excavation methods and equipment for this project.

UNSUITABLE ONSITE MATERIALS

Although not observed in our borings, localized fill materials that contain miscellaneous debris, rubble, remnants of past construction and other materials may be encountered. In addition, an existing concrete slab is located within the footprint of the northern pond. Consideration must be given to removing all vegetation, organic topsoil, existing structures, abandoned foundations, utilities, associated backfill, and other deleterious material. We recommend that these materials be entirely removed from below the pond bottom and proposed berms, if any.

EXPANSIVE SOIL-RELATED MOVEMENTS

With the exception of Boring MW-5, the CCR pond bottom is anticipated to be founded on sand. Expansive soil related movements for the natural sand material are not anticipated. However, in the vicinity of Boring MW-5, we estimate approximately 1 ft of potentially expansive soil may remain below the pond bottom in this areas. We anticipate that some of this material may be removed and replaced to construct the composite liner, and eventually be surcharged by CCR product. In addition, the existing potentially expansive soil is expected to remain below the proposed berms or the excavated side walls for the CCR Pond.

The anticipated ground movements due to swelling of the underlying expansive soils at the site were estimated using the empirical procedure, Texas Department of Transportation (TxDOT) Tex-124-E, Method for Determining the Potential Vertical Rise (PVR). Where the potentially expansive clays will be surcharged by berms and/or CCR product, PVR values of 1 in. or less were estimated for the stratigraphic conditions as previously discussed. However, where the clay will remain near the ground surface, cut slopes, or nominal berm fill will be placed, PVR values of on the order of 2 in. were estimated for the stratigraphic conditions as previously discussed. Once grading plans and berm configurations are developed, we recommend that the differential soil-related movements be further evaluated.

The TxDOT method of estimating expansive soil-related movements is considered an acceptable method for this project, and is based on empirical correlations utilizing the measured plasticity indices and assuming typical seasonal fluctuations in moisture content (an active zone of 15 ft, and dry moisture conditions were assumed in estimating the above PVR values).

SETTLEMENT DUE TO BERM FILL AND CCR MATERIAL

Berm fills with heights up to 6 ft are anticipated at this site. On the basis of our settlement models, we calculated settlements on the order of 1 inch for berm heights up to 6 ft. Typically, 50 percent of the total settlement will occur during construction of the fill. Settlement along the berm alignment is anticipated to decrease (to nominal) as the height of the berm fill decreases to the west. This potential settlement should be considered as differential (estimated on the order of 1/2 inch).

Cuts of approximately 4 to 21 ft are anticipated for the CCR pond. The weight of CCR material is expected to be less than the weight of soil/cemented materials to be replaced, and hence only nominal settlement is anticipated below the CCR Pond.

BERM GLOBAL STABILITY ANALYSIS

Global stability analysis of the anticipated cuts and berms was performed for Sections A-A' (cut slope), B-B' (berm), C-C' (cut slope), and D-D' (berm) as illustrated on Figure 1. The plotted sections were based on conceptual sections/elevations and the estimated CCR pond bottom elevation. The groundwater surface was assumed to occur near El 491 ft msl. Models for an empty CCR pond and "Maximum Pool," as modeled in our sections, were estimated.

Minimum Factor of Safety

Slope stability analysis consists of comparing the sliding and restraining forces along a possible slide plane and determining the factor of safety. Gravity (i.e. surcharge, soil weight and water in the slope) provides the driving force while shear strength of the soil provides the restraining force. When the driving force acting on the slope is greater than the restraining force, the slope will move. The factor of safety of the slope is the ratio of the restraining force divided by the driving force. Slides occur when the factor of safety is 1.0 or less. The target factor of safety for the short-term (end of construction), long-term condition, and pseudo-static conditions (i.e., seismic loading) are summarized in the following table.

Global Stability Minimum Target Factor of Safety

Condition	Minimum Target Factor of Safety
Short-Term, End of Construction	≥1.3
Long-Term, Maximum Pool	≥1.4 to 1.5
Seismic Loading	≥ 1.0

We consider a significant slope failure to involve a volume of slope material that is large enough to substantially impair the serviceability or operation of the berm or that could imperil human life. Shallow, sloughing slope failures that involve relatively little material or that can be repaired locally without substantially impacting the ash pond operations are considered to be minor slope failures and do not control the conclusions of our stability analyses.

Method of Analysis

While there are many different methods of stability analysis and numerous available computer programs, we have selected the program Slide version 6.014, a slope stability computer program, developed by Rocscience. The Spencer method with a non-circular sliding surface was utilized for the conditions being considered.

Loading Conditions

For satisfactory performance, an earth embankment should have an acceptable factor of safety during construction and throughout its projected service lifetime. Stability analyses should include variations in stress conditions brought on by construction practices and sequencing, external loadings, and any anticipated changes in hydraulic conditions. The following paragraphs discuss each stability condition analyzed in our study.

External Loads External loads for the roadways along the berm crest have also been modeled. A traffic loading of HS20 (modeled as an equivalent uniform surcharge of 100 psf) was applied to the crest of the berm.

CCR Material Load On the basis of our historic field density testing on typical CCR material (Circa 2014), the total weight of the material varied from 92 to 122 pounds per cubic foot (pcf). We have included a total weight of 120 pcf (modeled as no strength) for additional loads in the analyses conducted for the “maximum pool” of the berms. These loads account for the increase in pressure in the bottom of the ponds and along the berm slopes due to weight of the CCR material in the ponds. The increase in the pressure due to this material is modeled in our analysis.

Soil Properties

The soil properties used in our analyses are based on limited laboratory testing, index properties of the soil, empirical correlations, and our experience. The soil properties used in the models are summarized in the following table and are considered as conservative.

SOIL PROPERTIES USED IN THE GLOBAL STABILITY MODEL

Soil Type	Density (pcf)	End of Construction Cohesion (psf)	Long-Term Friction Angle (degrees)
Estimated Engineered Berm Fill	125	1,000	25[a]
Natural Cohesive Soil	125	1,000[b]	27[b]
Upper Natural Cohesionless Soil	120	0[d]	35[c]
Lower Natural Cohesionless Soil	130	0[d]	38[c]
CCR Material	120	No Strength	No Strength

[a] Estimated strength for compacted engineered material
[b] Estimated from laboratory tests and correlations
[c] Estimated from SPT correlations
[d] Friction angle used for this condition

Results of Analyses

The following table contains a summary of the results from our slope stability analyses for each static loading condition and slope configuration. In general, the point where a potential slide surface was permitted to intersect the slope face not allowed to occur (within relevant slope crest). This limitation was intended to reduce the occurrence of "non-critical" shallow failure surfaces resulting from the analyses. A graphical presentation of the most critical failure surface from our SLIDE iterations for each berm profile studied can be found in Appendix D.

Computed Factors of Safety – Static Condition

Slope Profile	End of Construction (Short-Term)	Pond Side (Long-Term)	Dry Side (Long-Term)	Maximum Pool on Pond Side (Long Term)	Maximum Pool on Dry Side (Long Term)
A-A'	>1.5 (A-1)	>1.5 (A-2)	N/A	>1.5 (A-4)	N/A
B-B'	>1.5 (B-1 & B-6)	>1.5 (B-7)	>1.5 (B-2)	>1.5 (B-9)	>1.5 (B-4)
C-C'	>1.5 (C-1)	>1.5 (C-2)	N/A	>1.5 (C-4)	N/A
D-D'	>1.5 (D-1 & D-6)	>1.5 (D-7)	>1.5 (D-2)	>1.5 (D-9)	>1.5 (D-4)

(Referenced Figure in Appendix D)

Pseudo-static (seismic) analyses were performed with soil behavior modeled using undrained soil strength values. A summary of the calculated factors of safety are presented in the following table.

Computed Factors of Safety – Pseudo-Static Condition (Seismic)
Horizontal Peak Ground Acceleration = 0.075g

Slope Profile	Pond Side	Dry Side	Maximum Pool on Pond Side	Maximum Pool on Dry Side
A-A'	>1.5 (A-3)	N/A	>1.5 (A-5)	N/A
B-B'	>1.5 (B-8)	>1.5 (B-3)	>1.5 (B-10)	>1.5 (B-5)
C-C'	>1.5 (C-3)	N/A	>1.5 (C-5)	N/A
D-D'	>1.5 (D-8)	>1.5 (D-3)	>1.5 (D-10)	>1.5 (D-5)

(Referenced Figure in Appendix D)

In general, the global stability analyses for the conditions evaluated resulted in calculated factors of safety greater than the targeted factor of safety for short-term, long-term, and seismic conditions. If steeper slopes are planned, CCR pond bottom elevation changes, or the berm configuration is altered, then additional evaluation will be required.

BERM CONSTRUCTION CONSIDERATIONS

Proposed berm fill materials should be further tested in the laboratory to evaluate that the proposed material has strength characteristics greater than those estimated in the global stability analysis. The laboratory testing should be performed on remolded samples compacted to a minimum of 95 or 90 percent of the maximum dry density as determined by the Standard Proctor (ASTM D698) or Modified Proctor (ASTM D1557), respectively. The strength tests (minimum of three tests) may consist of either:

- ASTM D3080/D3080M-11 Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions; or
- ASTM D4767-11 Standard Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils

The contractor will also be responsible for ensuring that the properties of all delivered berm fill materials are similar to those tested in the laboratory.

Consideration can be given to utilizing the excavated on-site natural material for the berm construction. However, cemented sand/sandstone may be encountered and processing of the excavated material may be required to reduce the maximum particle size to 4 in. in any dimension. Processed material larger than 4 inches should be discarded or processed to the maximum dimension. Care should be taken when placing the larger pieces so that they are not concentrated in a manner such that voids develop between nested pieces; a sufficient quantity of fines should be provided to reduce this risk. Furthermore, special care will be required during excavation activities to separate organics and any deleterious material.

Berm fill should be placed in maximum 8-inch thick loose lifts and compacted to the levels given in the following Compaction Summary. The fill should be placed at a moisture content compatible with the required density. Depending on the soil moisture at the time of construction, aeration or wetting may be required to achieve proper compaction. The fill should not be placed on soft or yielding materials.

COMPACTION SUMMARY

Category	**Minimum Compaction[a] (Percent)**	
	Standard Proctor	**Modified Proctor**
Prepared Subgrade and Berm Engineered Fill	**95 [b]**	**90 [b]**

[a] Measured as a percent of the maximum dry density as determined by the Standard or Modified Proctor test (ASTM D698 or D1557), respectively.

[b] Moisture content within 3% of optimum moisture content.

Please note that finished slopes have an increased potential for erosion and relatively shallow slip surface failures. Therefore, installation of erosion control measures and/or increased slope maintenance may be required until vegetation is established. Failures, if any, should be overexcavated beyond the failure plane and replaced with compacted fill placed in benches.

Fill slopes steeper than 1V:4H should be benched prior to placement of fill or a clay liner directly on them. Benching the fill/liner will help reduce the potential for sloughing or creating an artificial failure plane in which the material is being placed on. Bench shelves should be approximately 6 feet wide, but bench faces should not be higher than 2 feet. Fill/liner slopes should be constructed by extending the compacted fill beyond the planned profile of the slope and then trimming the slope to the desired configuration.

LINER MATERIAL CONSIDERATIONS

Consideration may be given to trying to use the onsite fine-grained soils as clay liner material. **However, the characteristics/variability of this material can change considerably in relatively short horizontal and vertical distances as evident in our boring logs, and additional evaluation of the onsite fine-grained soil as use of liner material is warranted.**

It has been our experience that compacted clay liners of a minimum of 24 in. are adequate to reduce water seepage to acceptable limits. Soils used as the liner material should be classified as fat clay (CH) or lean clay (CL) in accordance with ASTM D 2487-10 Unified Soil Classification System. In addition, soil liner material should adhere to the following specifications:

Soil Liner Specifications		
Property	**Unit**	**Specification**
Plasticity Index	%	≥ 20
Liquid Limit	%	≥ 45
% Passing (200 sieve)	%	≥ 50
Maximum Particle Size	in.	3/4*

* or minimum particle size specified by the geomembrane supplier.

Soils that adhere to the liner specifications presented above, typically have a saturated soil permeability less than 1 x 10^{-7}cm/sec. Compacted soil liner material should be free of refuse, roots, rocks, and other deleterious materials. Soil liner material should be placed in maximum 8-inch thick loose lifts and compacted to the levels given in the Compaction Summary under Section titled *Berm Construction Considerations*. Particles larger than 3/4 in. in dimension (or the maximum particle size specified by the Geomembrane supplier), roots, and deleterious material should not be permitted in the soil liner. Additional soil liner placement considerations can be provided when additional information and direction become available.

LIMITATIONS

This engineering report has been prepared in accordance with accepted Geotechnical Engineering practices in the region of south/central Texas and for the use of Pape-Dawson Engineers, Inc. (CLIENT) and its representatives for design purposes. This report may not contain sufficient information for purposes of other parties or other uses. This report is not intended for use in determining construction means and methods.

If this report is provided to prospective subcontractors, the client should make it clear that the information is provided for factual data only and not as a warranty of subsurface conditions included in this report. Unanticipated soil or rock conditions may require the expenditure of additional funds to attain a properly constructed project. Therefore, some contingency fund is recommended to accommodate such potential extra costs.

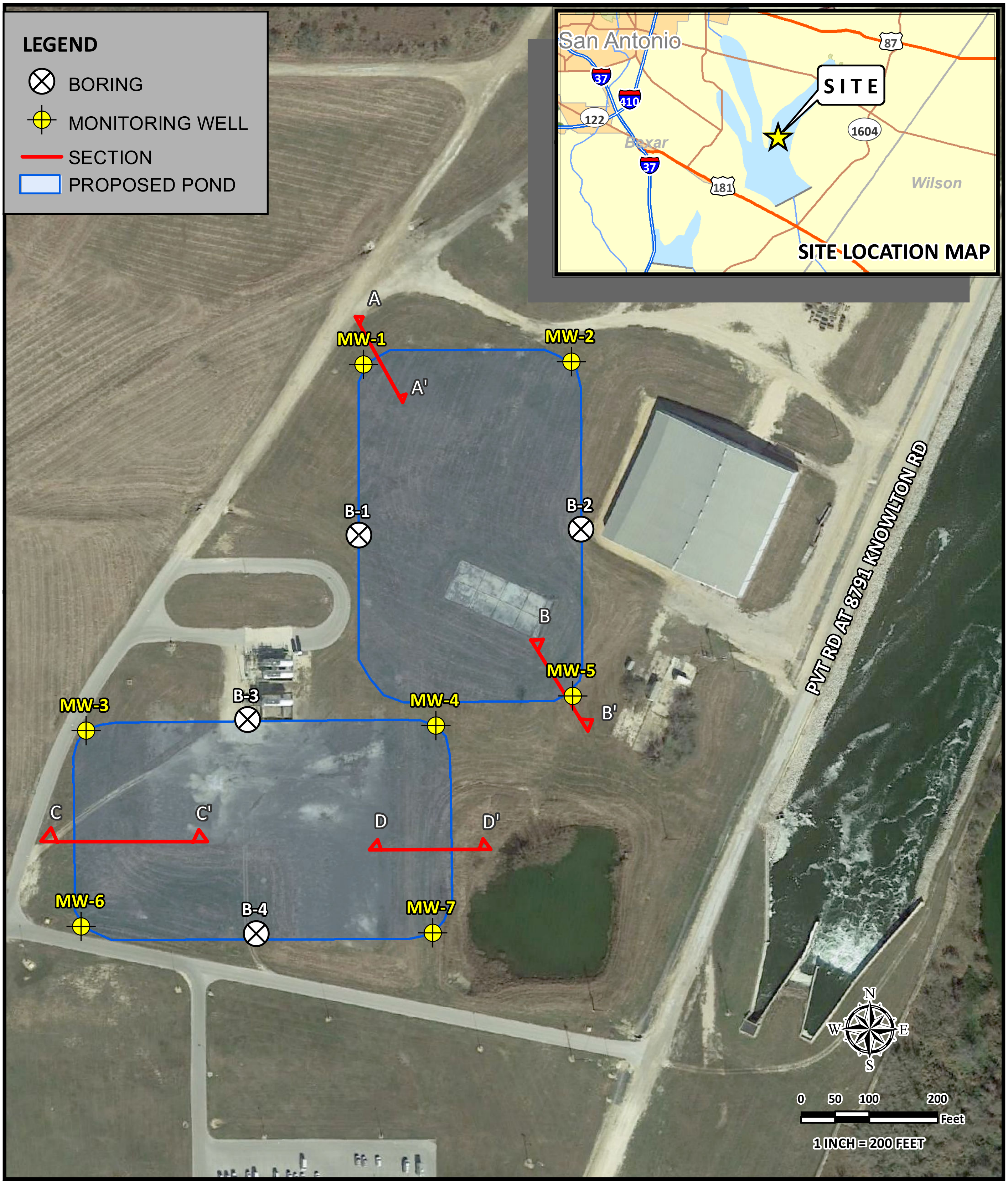
The recommendations submitted in this report are based on the data obtained from 11 borings drilled at this site and our understanding of the project information provided to us. If the project information described in this report is incorrect, is altered, or if new information is available, we should be retained to review and modify our recommendations.

The analyses, conclusions, and recommendations contained in this report are based on the data obtained from the subsurface exploration. The field exploration methods used indicate subsurface conditions only at the specific locations where samples were obtained, only at the time they were obtained, and only to the depths penetrated. Discrete sampling cannot be relied on to accurately reflect natural variations in stratigraphy that may exist between sample locations and/or intervals. This report may not reflect the actual variations of the subsurface conditions across the site. However, it is important to note that a significant portion of the apparent site variability is due to variation in the proportions of sand and clay in the native soils. These variations cause the soil classification to change between borings, while our experience indicates the behavior of these soils varies within a relatively narrow range.

The scope of our Geotechnical Engineering Study does not include an environmental assessment of the air, soil, rock, or water conditions either on or adjacent to the site. No environmental opinions are presented in this report.

* * * * * * * * * * * * * * * * * *

ATTACHMENTS



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TBPE Firm F-3257 / TBPG Firm #50220

SOURCE: Aerial Photography Obtained from Google Earth Pro - 2017

BORING LOCATION MAP

J.K. SPRUCE - CALAVERAS LAKE POWER PLANT
PROPOSED TWO NEW COAL COMBUSTION
RESIDUAL CONTAINMENT PONDS
SAN ANTONIO, TEXAS

REVISIONS:

No.	DATE	DESCRIPTION

PROJECT No.:
ASA17-189-00

ISSUE DATE:	12/12/2017
DRAWN BY:	KRB
CHECKED BY:	EJN
REVIEWED BY:	EJN

FIGURE

1

NOTE: This Drawing is Provided for Illustration Only, May Not be to Scale and is Not Suitable for Design or Construction Purposes

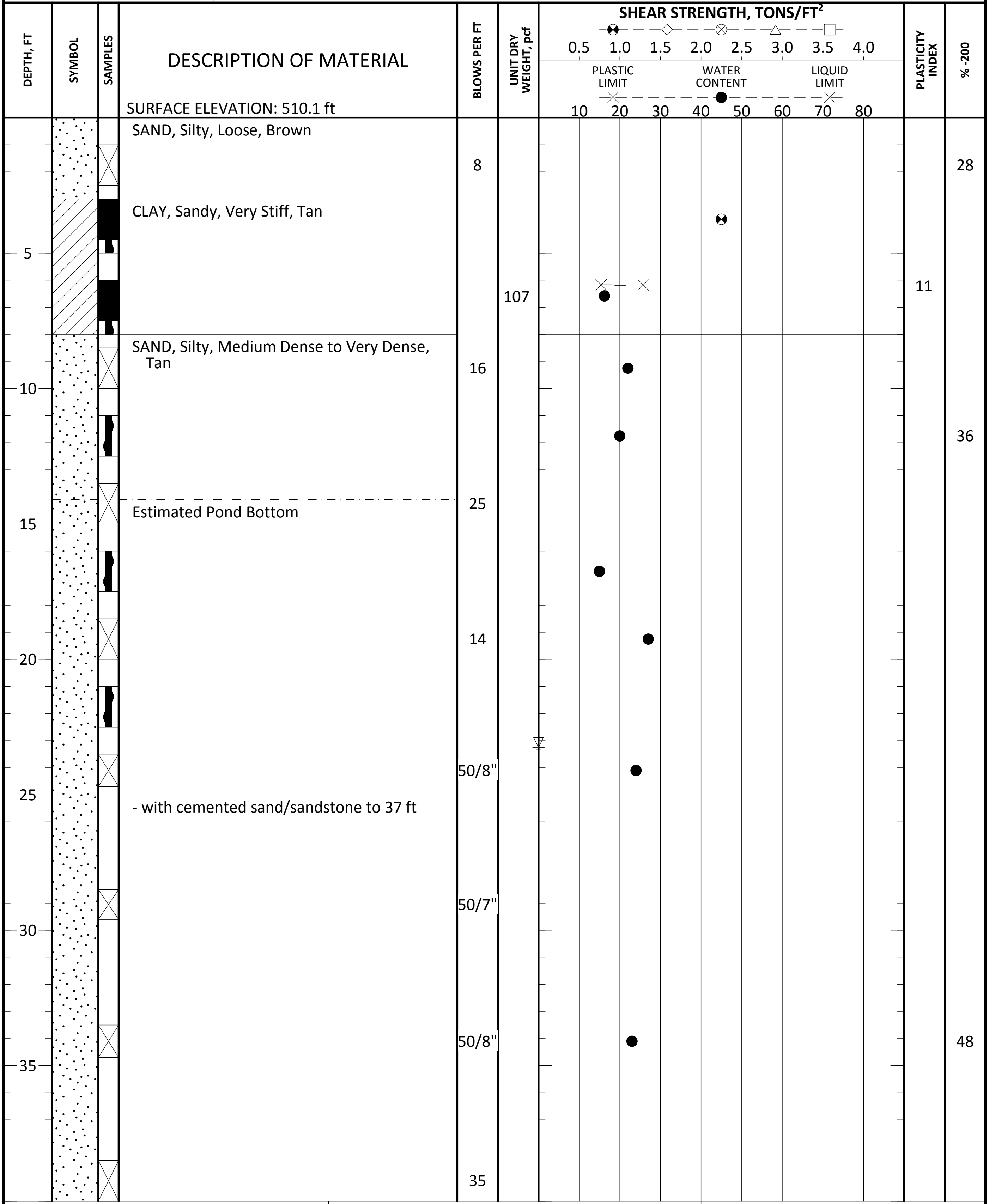
LOG OF BORING NO. B-1

J.K. Spruce - Calaveras Lake Power Plant
Proposed Two New Coal Combustion Residual Containment Ponds
San Antonio, Texas

TBPE Firm Registration No. F-3257

DRILLING METHOD: Hollow Stem Auger

LOCATION: N 29.31326; W 98.31708



SURFACE ELEVATION: 510.1 ft

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
	SAND, Silty, Loose, Brown	8			28
	CLAY, Sandy, Very Stiff, Tan				
5					
			107	11	
	SAND, Silty, Medium Dense to Very Dense, Tan	16			
10					
					36
	Estimated Pond Bottom	25			
15					
		14			
20					
		50/8"			
25					
	- with cemented sand/sandstone to 37 ft				
		50/7"			
30					
		50/8"			48
35					
		35			

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 50.0 ft
DATE DRILLED: 12/20/2017
DEPTH TO WATER: 23.25 ft
DATE MEASURED: 12/20/2017
PROJ. No.: ASA17-096-00
FIGURE: 2a

LOG OF BORING NO. B-1

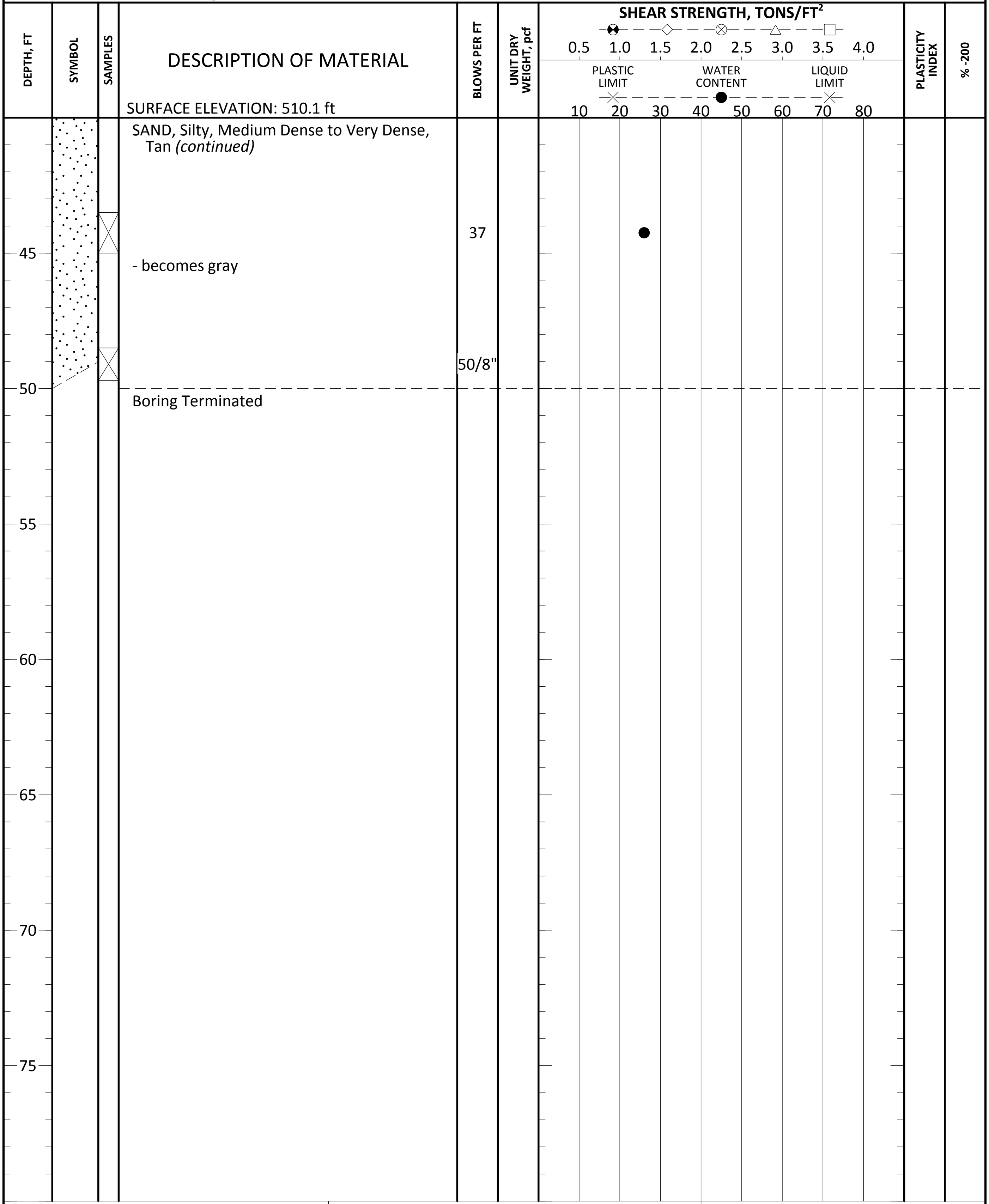
J.K. Spruce - Calaveras Lake Power Plant
Proposed Two New Coal Combustion Residual Containment Ponds
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Hollow Stem Auger

LOCATION: N 29.31326; W 98.31708

SURFACE ELEVATION: 510.1 ft



DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT
	SAND, Silty, Medium Dense to Very Dense, Tan *(continued)*	
		37
45	- becomes gray	
		50/8"
50	Boring Terminated	

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED:	50.0 ft	**DEPTH TO WATER:**	23.25 ft	**PROJ. No.:**	ASA17-096-00
DATE DRILLED:	12/20/2017	**DATE MEASURED:**	12/20/2017	**FIGURE:**	2b

LOG OF BORING NO. B-2

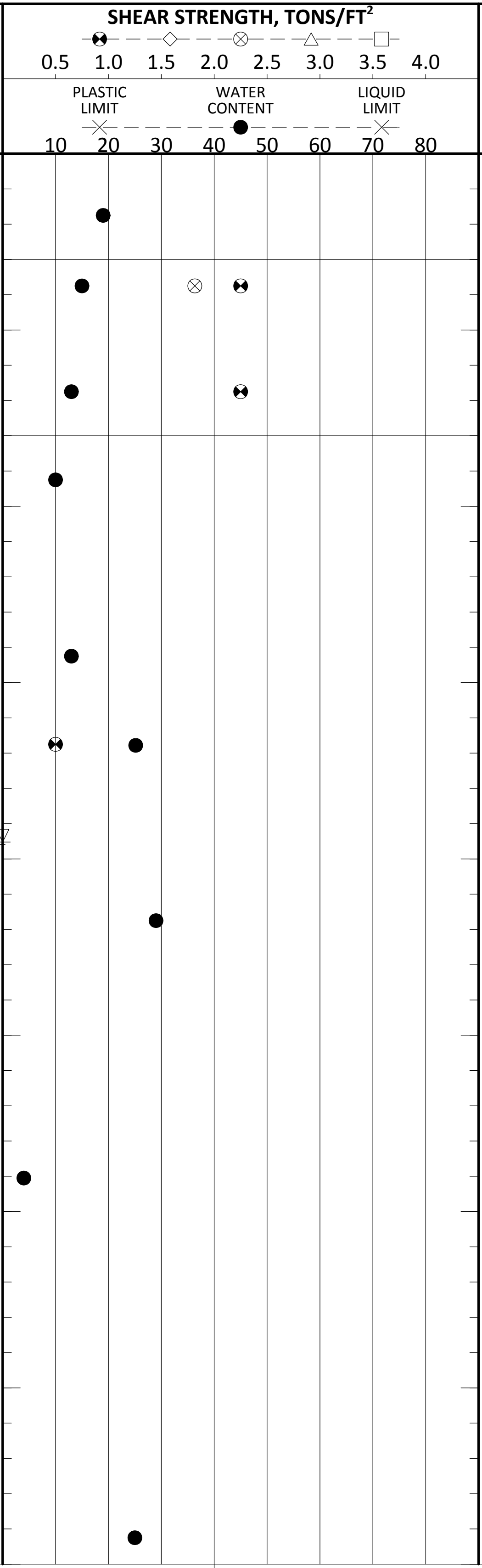
J.K. Spruce - Calaveras Lake Power Plant
Proposed Two New Coal Combustion Residual Containment Ponds
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Hollow Stem Auger

LOCATION: N 29.31328; W 98.31606

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
	SURFACE ELEVATION: 506.18 ft				
	SAND, Silty, Loose, Brown				
		7			
	CLAY, Sandy, Very Stiff, Tan		114		52
	- Hydraulic Conductivity = 1.88x10^{-7} cm/sec				
5					
			115		
	SAND, Silty, Medium Dense, Tan				
		15			
10					
	- Estimated Pond Bottm				
		28			
15					
			1		27
		44			
20					
		28			
25	- with cemented sand/sandstone to 30 ft				
30					
	- becomes gray	40			
35					
		50			



NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 50.0 ft
DATE DRILLED: 12/22/2017
DEPTH TO WATER: 19.52 ft
DATE MEASURED: 12/22/2017
PROJ. No.: ASA17-096-00
FIGURE: 3a

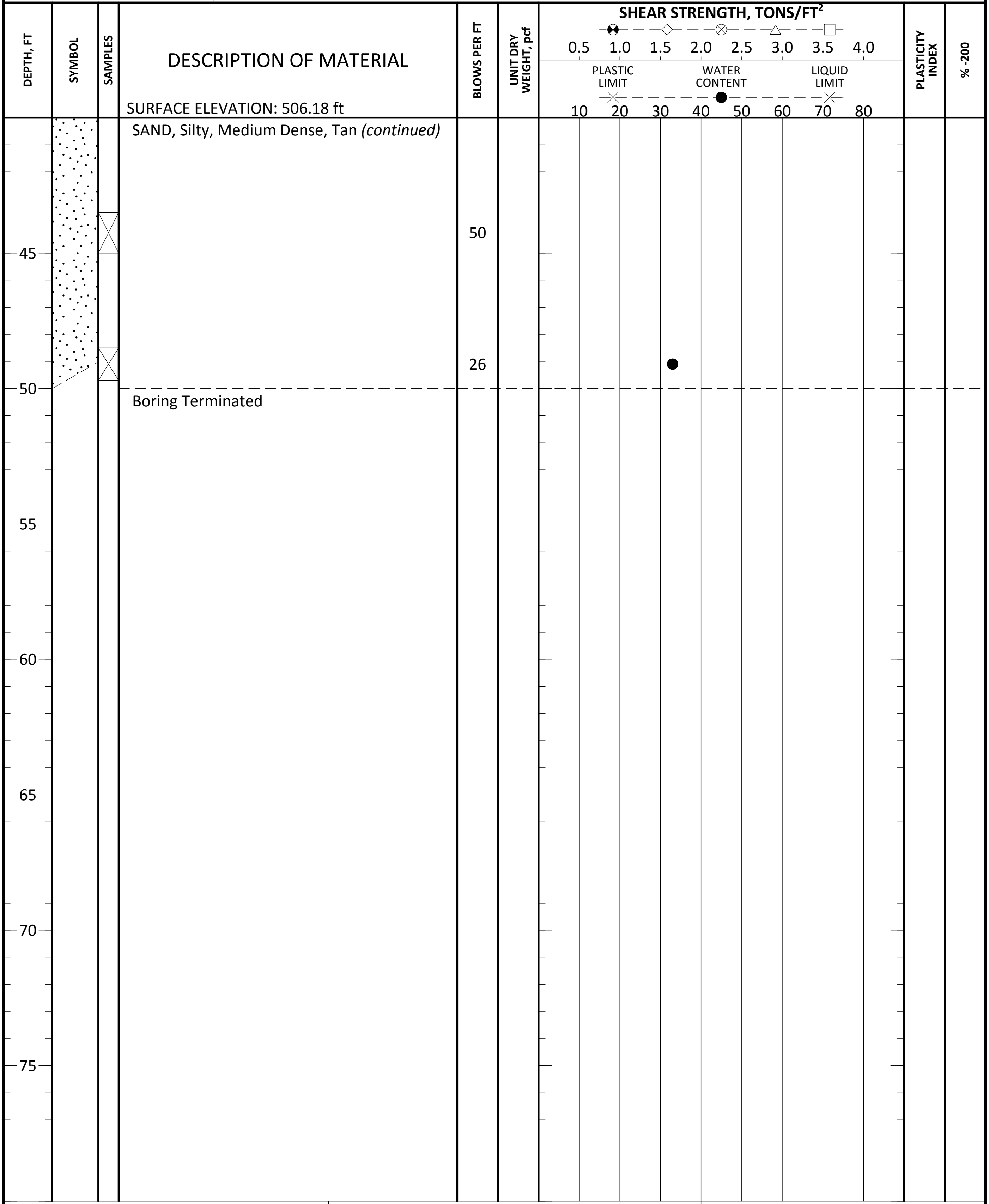
LOG OF BORING NO. B-2

J.K. Spruce - Calaveras Lake Power Plant
Proposed Two New Coal Combustion Residual Containment Ponds
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Hollow Stem Auger

LOCATION: N 29.31328; W 98.31606



DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT
	SAND, Silty, Medium Dense, Tan *(continued)*	
		50
		26
50	Boring Terminated	

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 50.0 ft
DATE DRILLED: 12/22/2017
DEPTH TO WATER: 19.52 ft
DATE MEASURED: 12/22/2017
PROJ. No.: ASA17-096-00
FIGURE: 3b

LOG OF BORING NO. B-3

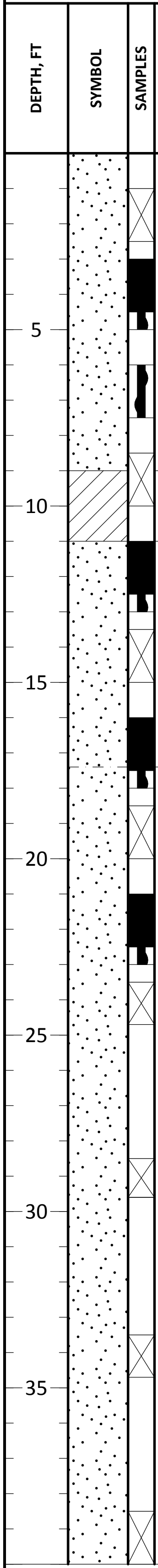
J.K. Spruce - Calaveras Lake Power Plant
Proposed Two New Coal Combustion Residual Containment Ponds
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

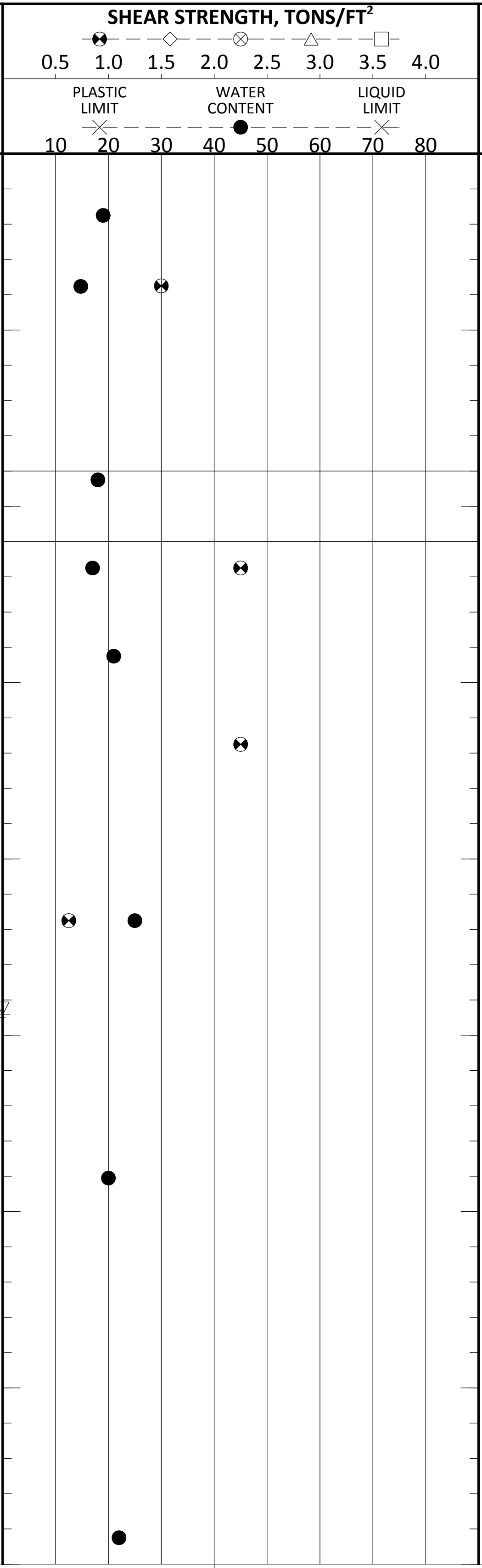
DRILLING METHOD: Hollow Stem Auger

LOCATION: N 29.31245; W 98.31760

SURFACE ELEVATION: 513.4 ft



DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf
SAND, Silty, Medium Dense, Tan and Dark Brown	16	
- Hydraulic Conductivity = 2.05×10^{-6} cm/sec		110
CLAY, Sandy, Very Stiff, Tan	19	
SAND, Silty, Medium Dense to Dense, Tan to Light Gray		101
	28	
- Estimated Pond Bottom		
	12	
	19	
	50	
	50	
	36	



PLASTICITY INDEX	% -200

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 50.0 ft
DATE DRILLED: 1/2/2018
DEPTH TO WATER: 24.42 ft
DATE MEASURED: 1/2/2018
PROJ. No.: ASA17-096-00
FIGURE: 4a

LOG OF BORING NO. B-3

J.K. Spruce - Calaveras Lake Power Plant
Proposed Two New Coal Combustion Residual Containment Ponds
San Antonio, Texas

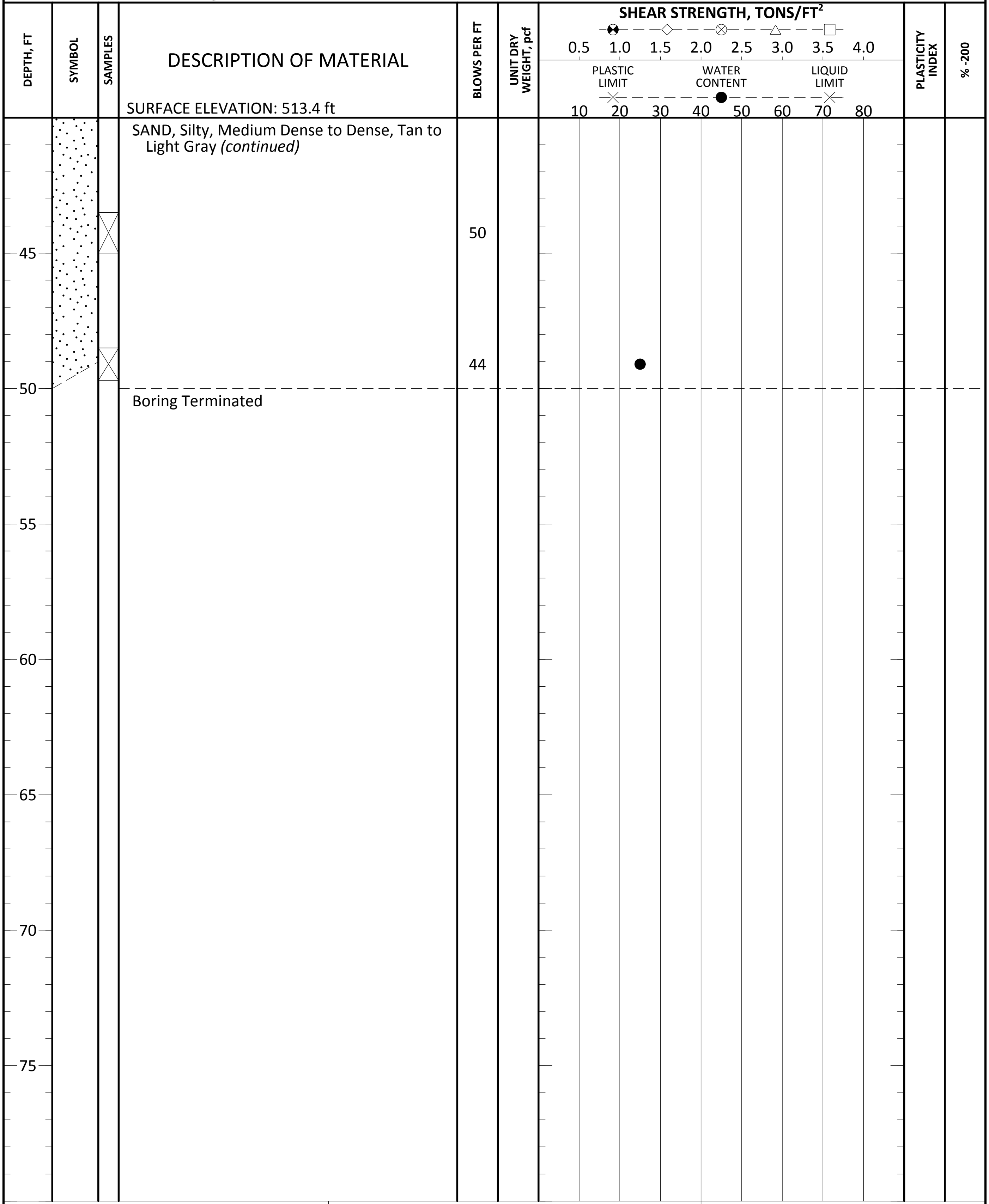
TBPE Firm Registration No. F-3257

DRILLING METHOD: Hollow Stem Auger

LOCATION: N 29.31245; W 98.31760

SURFACE ELEVATION: 513.4 ft

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	SHEAR STRENGTH, TONS/FT²	PLASTICITY INDEX	% -200
			SAND, Silty, Medium Dense to Dense, Tan to Light Gray *(continued)*					
45		☒		50				
		☒		44				
50			Boring Terminated					



NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 50.0 ft
DATE DRILLED: 1/2/2018

DEPTH TO WATER: 24.42 ft
DATE MEASURED: 1/2/2018

PROJ. No.: ASA17-096-00
FIGURE: 4b

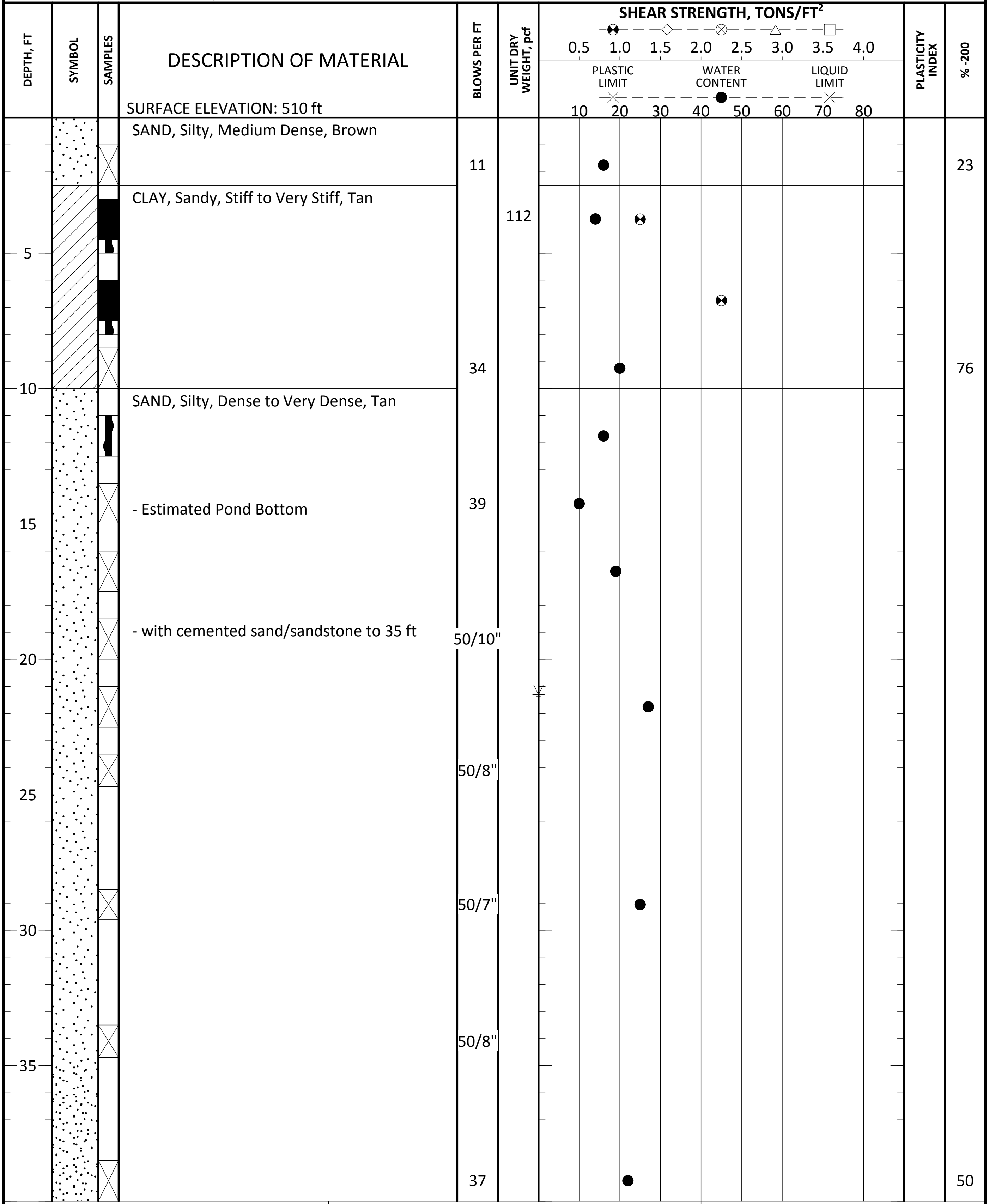
LOG OF BORING NO. B-4

J.K. Spruce - Calaveras Lake Power Plant
Proposed Two New Coal Combustion Residual Containment Ponds
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Hollow Stem Auger

LOCATION: N 29.31166; W 98.31756



SAND, Silty, Medium Dense, Brown	11		23
CLAY, Sandy, Stiff to Very Stiff, Tan		112	
	34		76
SAND, Silty, Dense to Very Dense, Tan			
- Estimated Pond Bottom	39		
- with cemented sand/sandstone to 35 ft	50/10"		
	50/8"		
	50/7"		
	50/8"		
	37		50

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 50.0 ft
DATE DRILLED: 12/20/2017
DEPTH TO WATER: 21.3 ft
DATE MEASURED: 12/20/2017
PROJ. No.: ASA17-096-00
FIGURE: 5a

LOG OF BORING NO. B-4

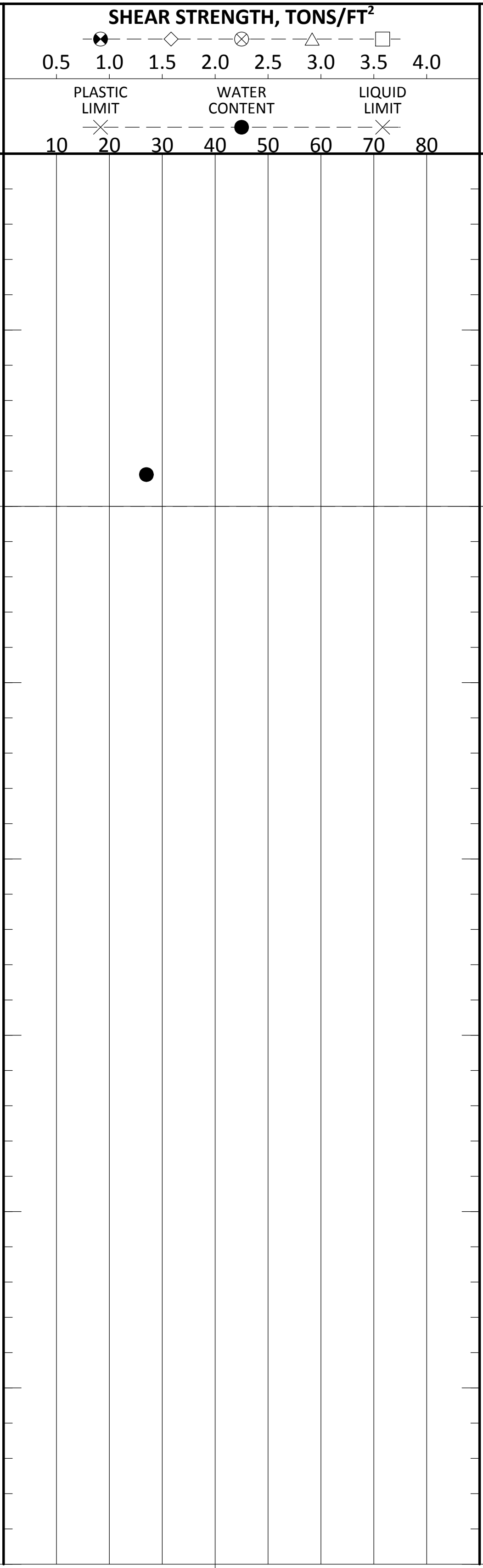
J.K. Spruce - Calaveras Lake Power Plant
Proposed Two New Coal Combustion Residual Containment Ponds
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Hollow Stem Auger

LOCATION: N 29.31166; W 98.31756

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL SURFACE ELEVATION: 510 ft	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
			SAND, Silty, Dense to Very Dense, Tan *(continued)* - becomes gray - becomes gray *(continued)*				
45		☒		50/10"			
		☒		50/8"			
50			Boring Terminated				
55							
60							
65							
70							
75							



NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 50.0 ft
DATE DRILLED: 12/20/2017
DEPTH TO WATER: 21.3 ft
DATE MEASURED: 12/20/2017
PROJ. No.: ASA17-096-00
FIGURE: 5b

LOG OF BORING NO. MW-1

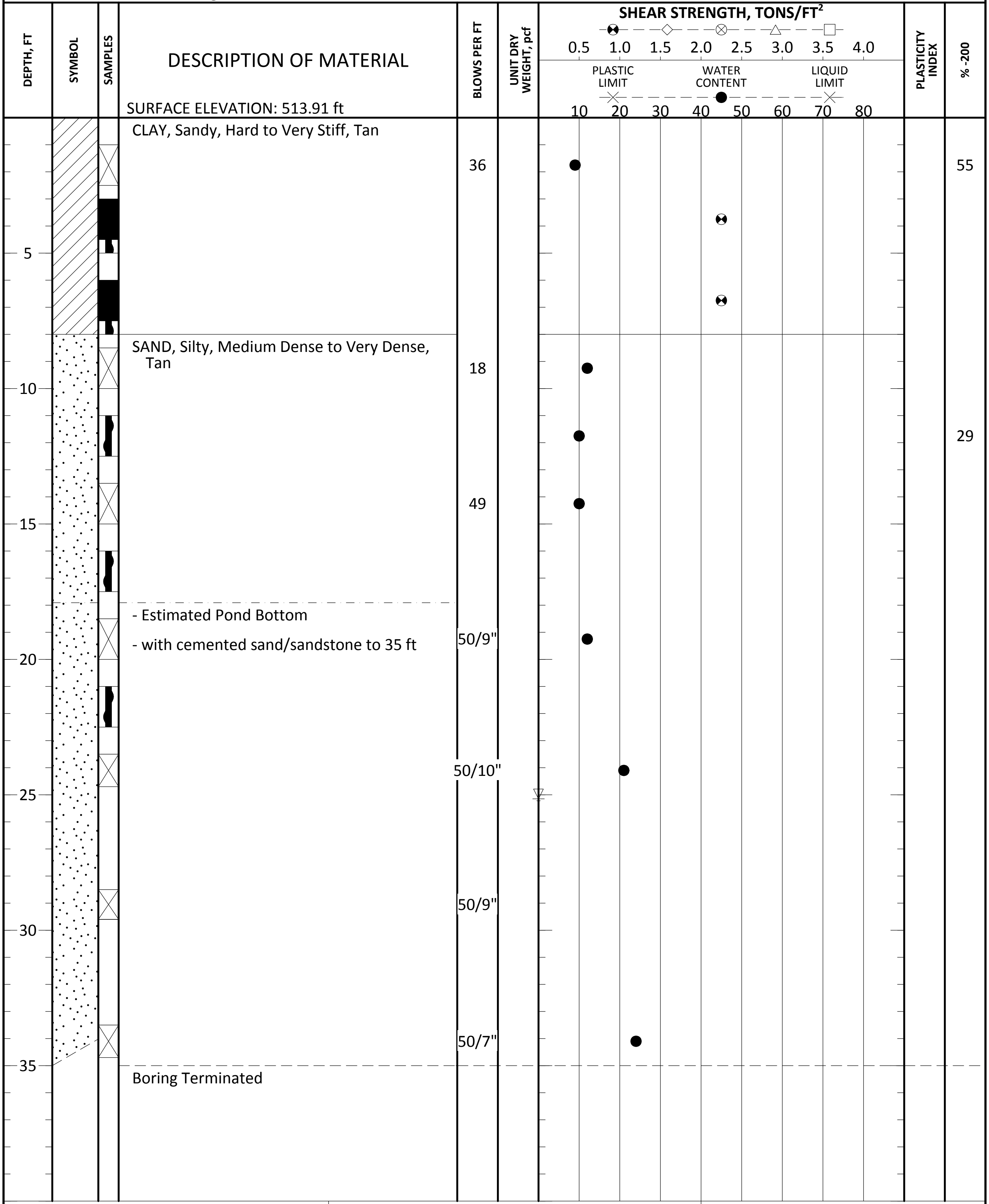
J.K. Spruce - Calaveras Lake Power Plant
Proposed Two New Coal Combustion Residual Containment Ponds
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Hollow Stem Auger

LOCATION: N 29.31395; W 98.31705

SURFACE ELEVATION: 513.91 ft



CLAY, Sandy, Hard to Very Stiff, Tan

SAND, Silty, Medium Dense to Very Dense, Tan

- Estimated Pond Bottom

- with cemented sand/sandstone to 35 ft

Boring Terminated

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 35.0 ft
DATE DRILLED: 12/20/2017

DEPTH TO WATER: 25.15 ft
DATE MEASURED: 12/20/2017

PROJ. No.: ASA17-096-00
FIGURE: 6

LOG OF BORING NO. MW-2

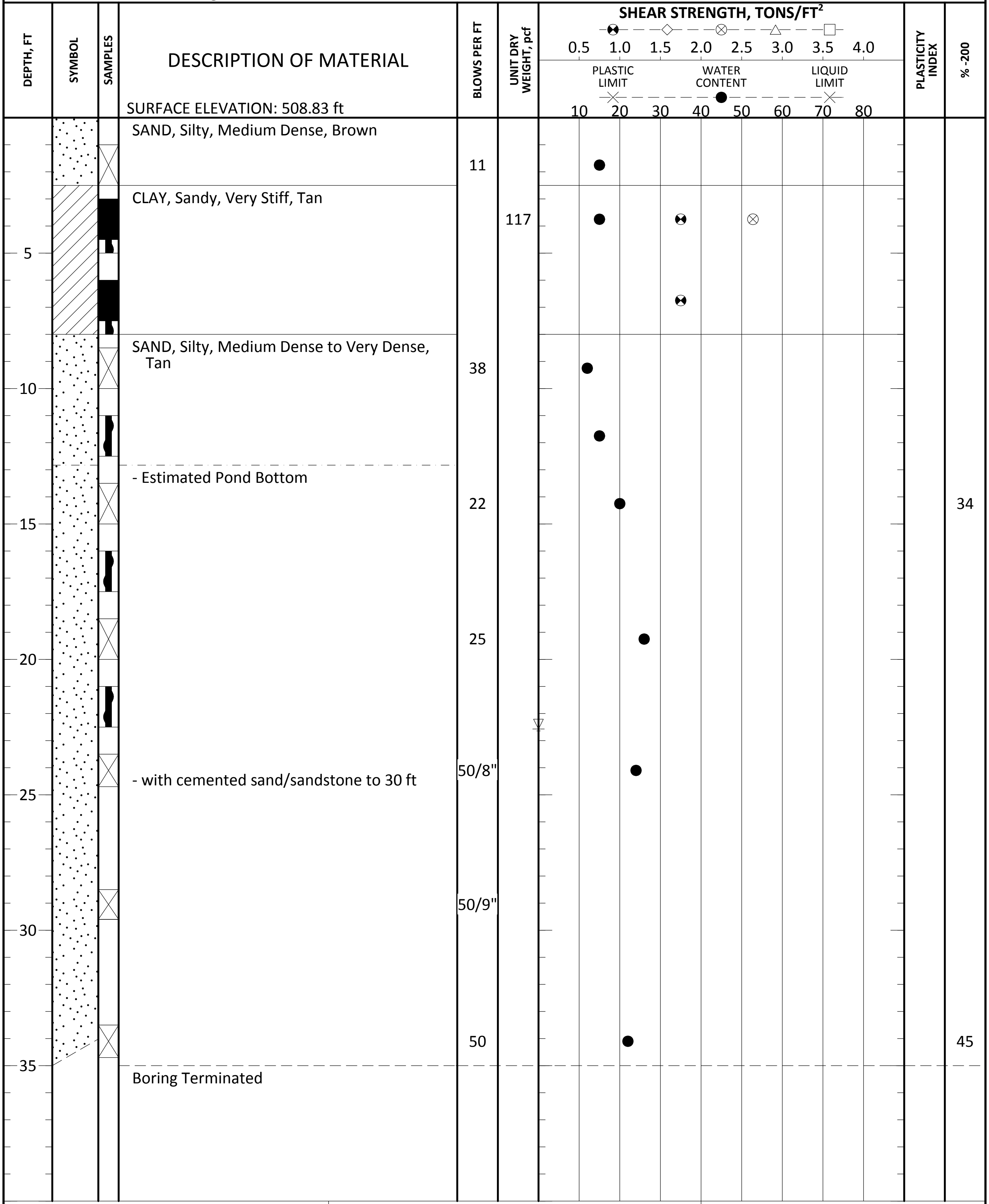
J.K. Spruce - Calaveras Lake Power Plant
Proposed Two New Coal Combustion Residual Containment Ponds
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Hollow Stem Auger

LOCATION: N 29.31395; W 98.31610

SURFACE ELEVATION: 508.83 ft



DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
SAND, Silty, Medium Dense, Brown	11			
CLAY, Sandy, Very Stiff, Tan		117		
SAND, Silty, Medium Dense to Very Dense, Tan	38			
- Estimated Pond Bottom	22			34
	25			
- with cemented sand/sandstone to 30 ft	50/8"			
	50/9"			
	50			45
Boring Terminated				

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 35.0 ft
DATE DRILLED: 12/20/2017
DEPTH TO WATER: 22.57 ft
DATE MEASURED: 12/20/2017
PROJ. No.: ASA17-096-00
FIGURE: 7

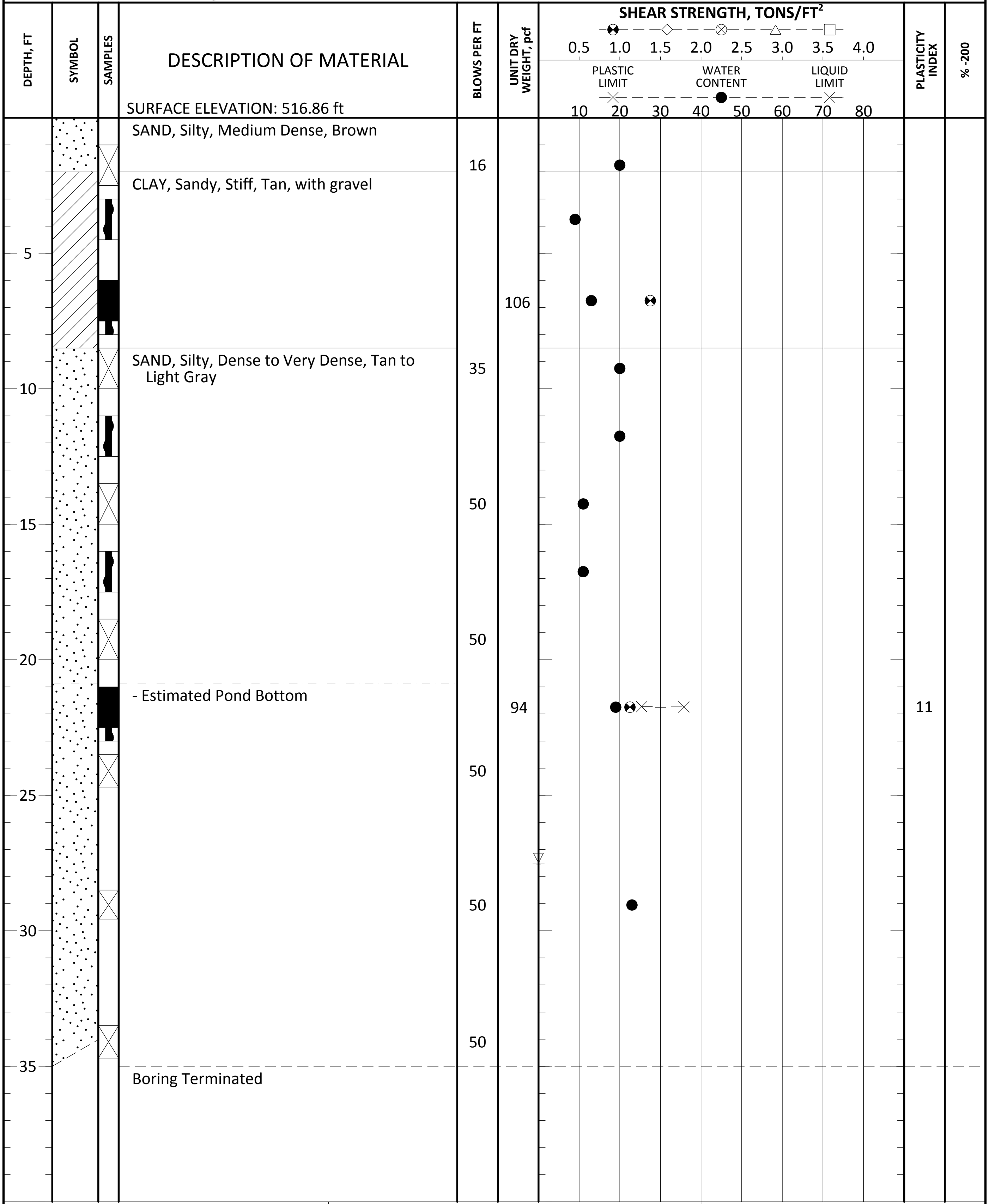
LOG OF BORING NO. MW-3

J.K. Spruce - Calaveras Lake Power Plant
Proposed Two New Coal Combustion Residual Containment Ponds
San Antonio, Texas

TBPE Firm Registration No. F-3257

DRILLING METHOD: Hollow Stem Auger

LOCATION: N 29.31249; W 98.31836



NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 35.0 ft
DATE DRILLED: 1/3/2018

DEPTH TO WATER: 27.5 ft
DATE MEASURED: 1/3/2018

PROJ. No.: ASA17-096-00
FIGURE: 8

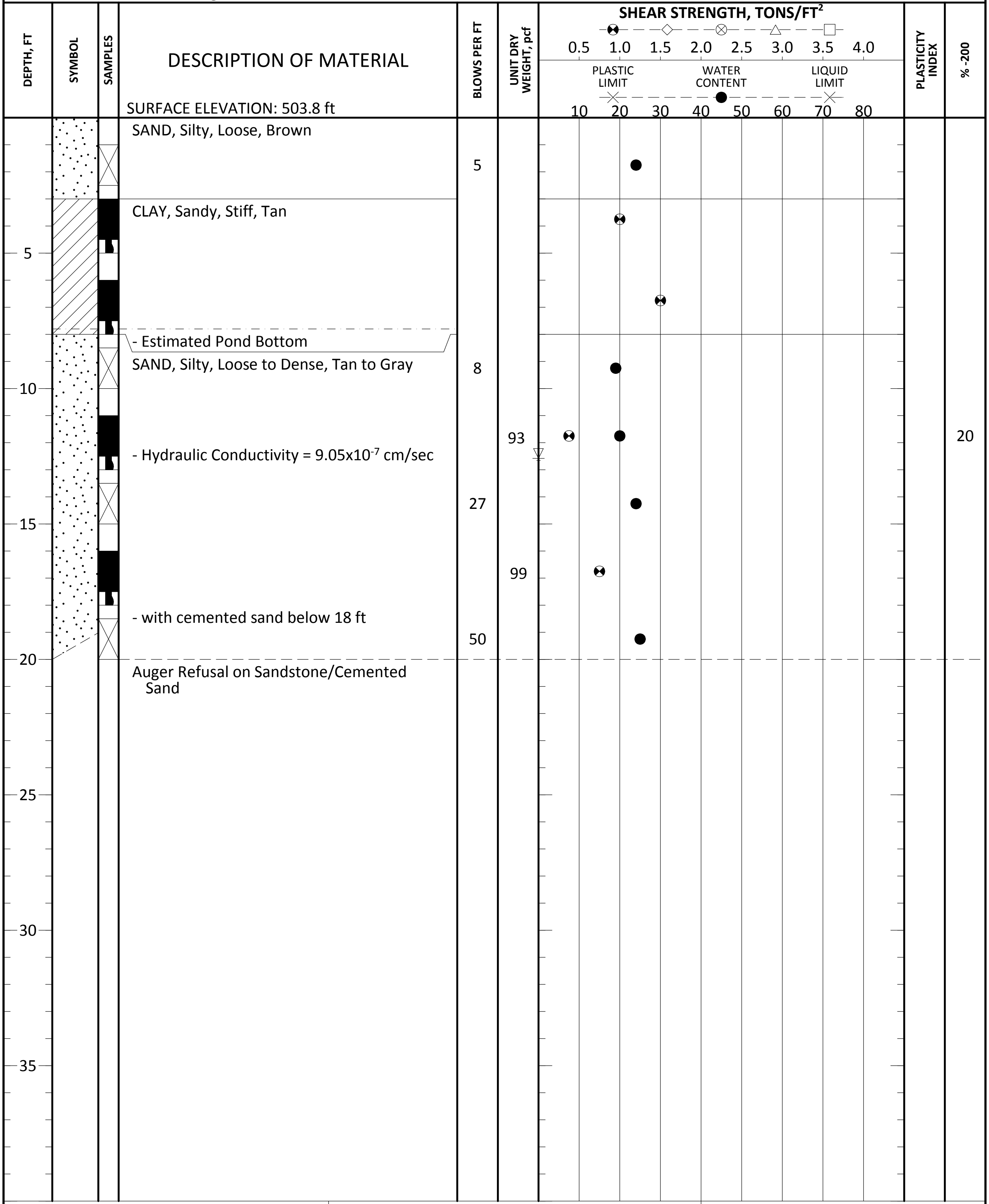
LOG OF BORING NO. MW-4

J.K. Spruce - Calaveras Lake Power Plant
Proposed Two New Coal Combustion Residual Containment Ponds
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Hollow Stem Auger

LOCATION: N 29.31250; W 98.31673



NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 20.0 ft
DATE DRILLED: 12/22/2017
DEPTH TO WATER: 12.58 ft
DATE MEASURED: 12/22/2017
PROJ. No.: ASA17-096-00
FIGURE: 9

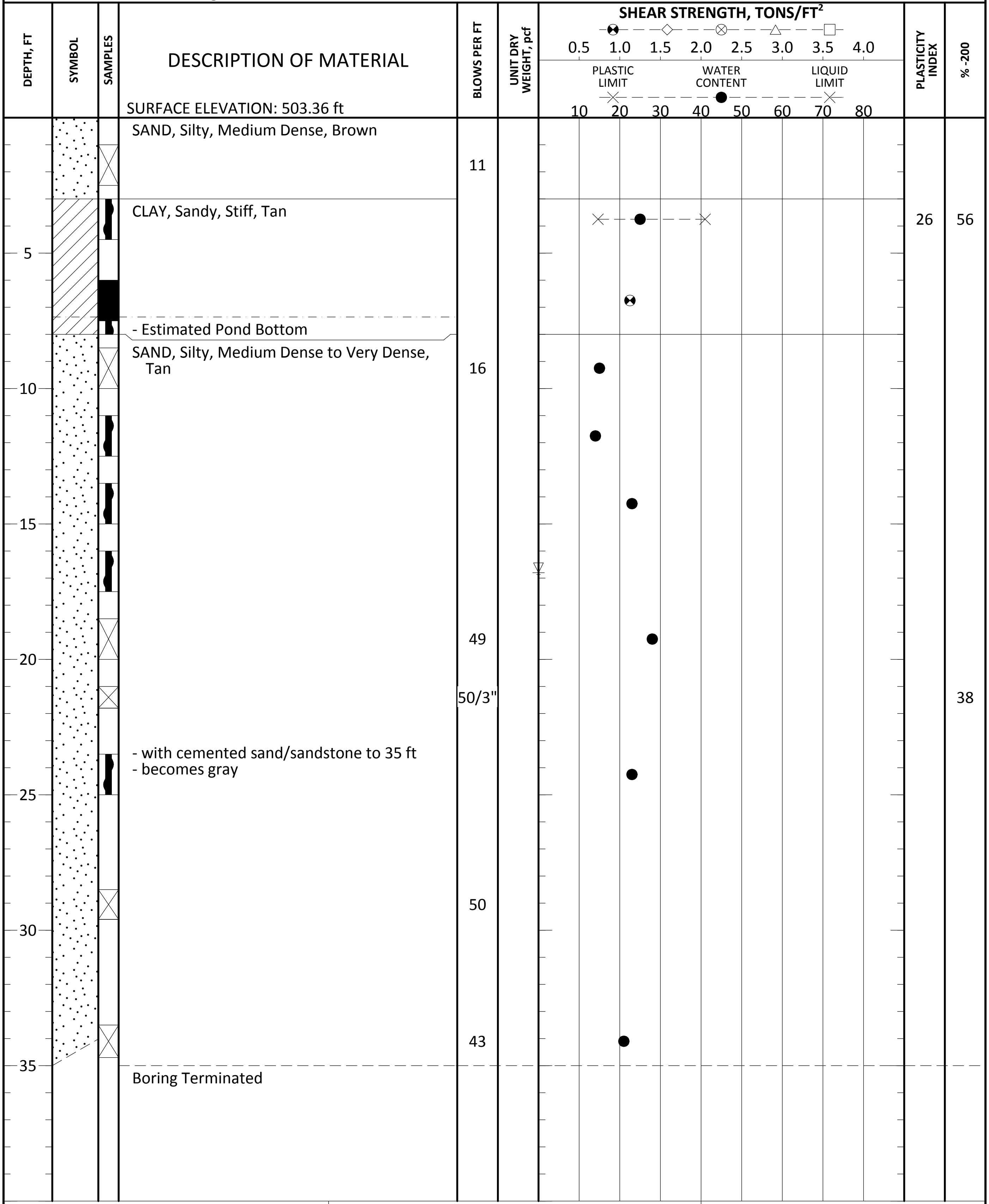
LOG OF BORING NO. MW-5

J.K. Spruce - Calaveras Lake Power Plant
Proposed Two New Coal Combustion Residual Containment Ponds
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Hollow Stem Auger

LOCATION: N 29.31261; W 98.31610



SURFACE ELEVATION: 503.36 ft

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
SAND, Silty, Medium Dense, Brown	11			
CLAY, Sandy, Stiff, Tan			26	56
- Estimated Pond Bottom				
SAND, Silty, Medium Dense to Very Dense, Tan	16			
	49			
	50/3"			38
- with cemented sand/sandstone to 35 ft - becomes gray				
	50			
	43			
Boring Terminated				

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 35.0 ft
DATE DRILLED: 12/21/2017

DEPTH TO WATER: 16.8 ft
DATE MEASURED: 12/21/2017

PROJ. No.: ASA17-096-00
FIGURE: 10

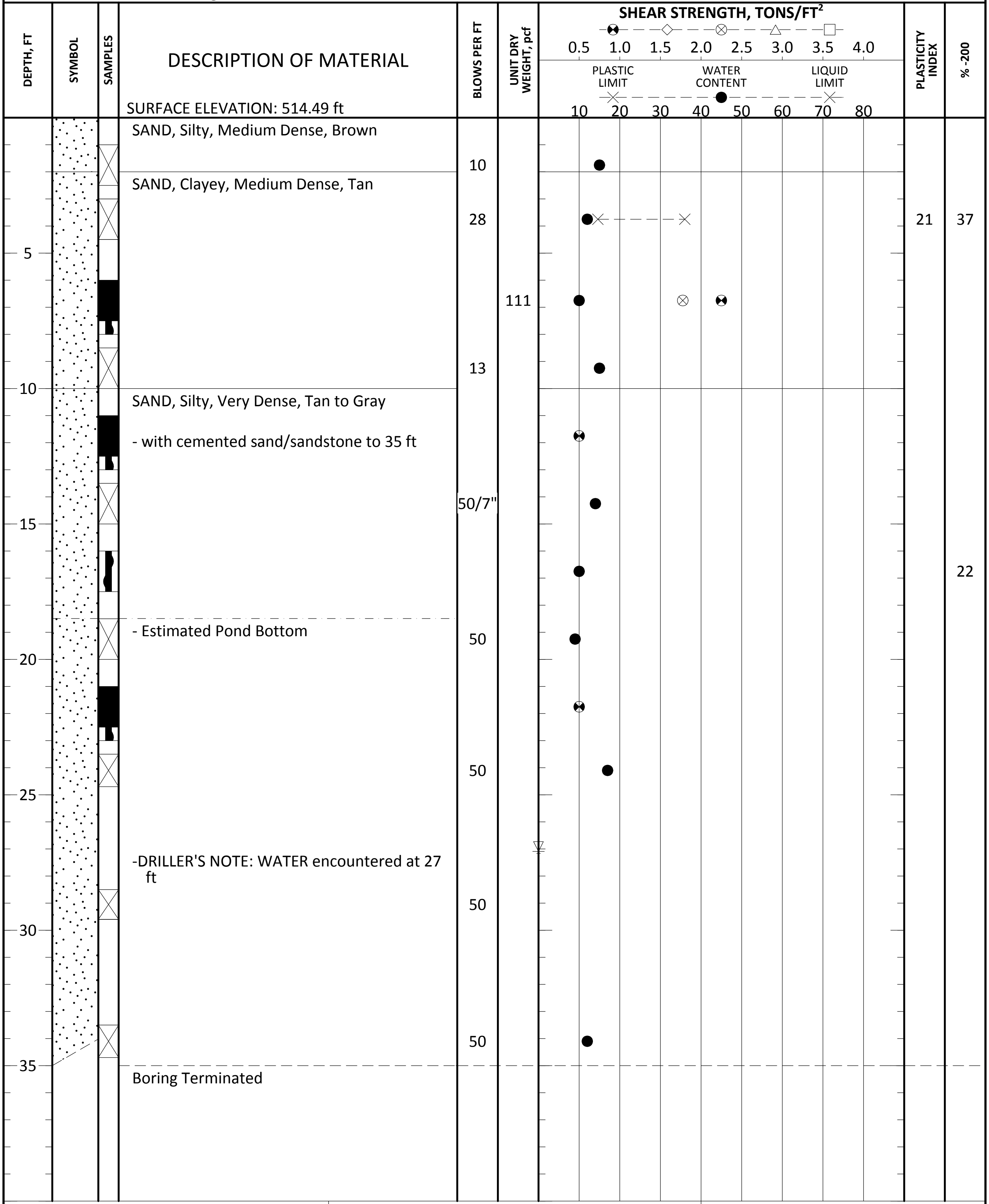
LOG OF BORING NO. MW-6

J.K. Spruce - Calaveras Lake Power Plant
Proposed Two New Coal Combustion Residual Containment Ponds
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Hollow Stem Auger

LOCATION: N 29.31177; W 98.31841



SURFACE ELEVATION: 514.49 ft

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
SAND, Silty, Medium Dense, Brown	10			
SAND, Clayey, Medium Dense, Tan	28		21	37
		111		
	13			
SAND, Silty, Very Dense, Tan to Gray				
- with cemented sand/sandstone to 35 ft				
	50/7"			
				22
- Estimated Pond Bottom	50			
	50			
-DRILLER'S NOTE: WATER encountered at 27 ft				
	50			
	50			
Boring Terminated				

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 35.0 ft
DATE DRILLED: 1/3/2018
DEPTH TO WATER: 27.09 ft
DATE MEASURED: 1/3/2018
PROJ. No.: ASA17-096-00
FIGURE: 11

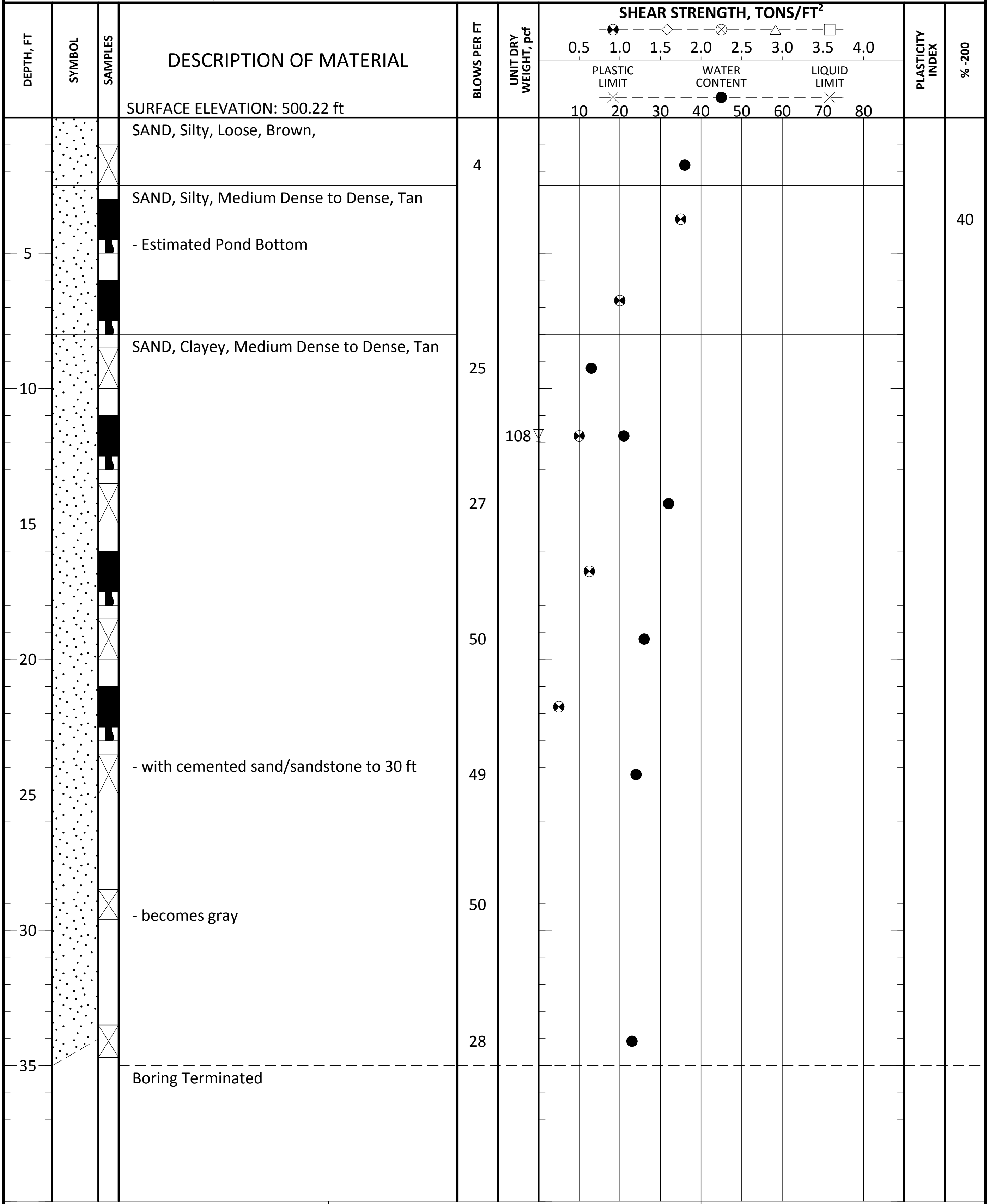
LOG OF BORING NO. MW-7

J.K. Spruce - Calaveras Lake Power Plant
Proposed Two New Coal Combustion Residual Containment Ponds
San Antonio, Texas

TBPE Firm Registration No. F-3257

DRILLING METHOD: Hollow Stem Auger

LOCATION: N 29.31166; W 98.31675



DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	% -200
	SURFACE ELEVATION: 500.22 ft			
	SAND, Silty, Loose, Brown,			
		4		
	SAND, Silty, Medium Dense to Dense, Tan			40
	- Estimated Pond Bottom			
5				
	SAND, Clayey, Medium Dense to Dense, Tan			
		25		
10				
			108	
		27		
15				
		50		
20				
	- with cemented sand/sandstone to 30 ft	49		
25				
	- becomes gray	50		
30				
		28		
35	Boring Terminated			

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 35.0 ft
DATE DRILLED: 12/22/2017
DEPTH TO WATER: 11.87 ft
DATE MEASURED: 12/22/2017
PROJ. No.: ASA17-096-00
FIGURE: 12

KEY TO TERMS AND SYMBOLS

MATERIAL TYPES

SOIL TERMS

- CALCAREOUS
- CALICHE
- CLAY
- CLAYEY
- GRAVEL
- GRAVELLY
- PEAT
- SAND
- SANDY
- SILT
- SILTY
- FILL

ROCK TERMS

- CHALK
- CLAYSTONE
- CLAY-SHALE
- CONGLOMERATE
- DOLOMITE
- IGNEOUS
- LIMESTONE
- MARL
- METAMORPHIC
- SANDSTONE
- SHALE
- SILTSTONE

OTHER

- ASPHALT
- BASE
- CONCRETE/CEMENT
- BRICKS / PAVERS
- WASTE
- NO INFORMATION

WELL CONSTRUCTION AND PLUGGING MATERIALS

- BLANK PIPE
- SCREEN
- BENTONITE
- CEMENT GROUT
- BENTONITE & CUTTINGS
- CONCRETE/CEMENT
- CUTTINGS
- GRAVEL
- SAND
- VOLCLAY

SAMPLE TYPES

- AIR ROTARY
- GRAB SAMPLE
- CORE
- GEOPROBE SAMPLER
- ROTOSONIC -DAMAGED
- MUD ROTARY
- NO RECOVERY
- NX CORE
- PITCHER
- ROTOSONIC -INTACT
- SHELBY TUBE
- SPLIT BARREL
- SPLIT SPOON
- TEXAS CONE PENETROMETER
- DISTURBED

STRENGTH TEST TYPES

- ⧓ POCKET PENETROMETER
- ◇ TORVANE
- ⊗ UNCONFINED COMPRESSION
- △ TRIAXIAL COMPRESSION UNCONSOLIDATED-UNDRAINED
- □ TRIAXIAL COMPRESSION CONSOLIDATED-UNDRAINED

NOTE: VALUES SYMBOLIZED ON BORING LOGS REPRESENT SHEAR STRENGTHS UNLESS OTHERWISE NOTED

PROJECT NO. ASA17-096-00

RABAKISTNER

KEY TO TERMS AND SYMBOLS (CONT'D)

TERMINOLOGY

Terms used in this report to describe soils with regard to their consistency or conditions are in general accordance with the discussion presented in Article 45 of SOILS MECHANICS IN ENGINEERING PRACTICE, Terzaghi and Peck, John Wiley & Sons, Inc., 1967, using the most reliable information available from the field and laboratory investigations. Terms used for describing soils according to their texture or grain size distribution are in accordance with the UNIFIED SOIL CLASSIFICATION SYSTEM, as described in American Society for Testing and Materials D2487-06 and D2488-00, Volume 04.08, Soil and Rock; Dimension Stone; Geosynthetics; 2005.

The depths shown on the boring logs are not exact, and have been estimated to the nearest half-foot. Depth measurements may be presented in a manner that implies greater precision in depth measurement, i.e 6.71 meters. The reader should understand and interpret this information only within the stated half-foot tolerance on depth measurements.

RELATIVE DENSITY

Penetration Resistance Blows per ft	Relative Density
0 - 4	Very Loose
4 - 10	Loose
10 - 30	Medium Dense
30 - 50	Dense
> 50	Very Dense

COHESIVE STRENGTH

Resistance Blows per ft	Consistency	Cohesion TSF
0 - 2	Very Soft	0 - 0.125
2 - 4	Soft	0.125 - 0.25
4 - 8	Firm	0.25 - 0.5
8 - 15	Stiff	0.5 - 1.0
15 - 30	Very Stiff	1.0 - 2.0
> 30	Hard	> 2.0

PLASTICITY

Plasticity Index	Degree of Plasticity
0 - 5	None
5 - 10	Low
10 - 20	Moderate
20 - 40	Plastic
> 40	Highly Plastic

ABBREVIATIONS

B = Benzene
T = Toluene
E = Ethylbenzene
X = Total Xylenes
BTEX = Total BTEX
TPH = Total Petroleum Hydrocarbons
ND = Not Detected
NA = Not Analyzed
NR = Not Recorded/No Recovery
OVA = Organic Vapor Analyzer
ppm = Parts Per Million

Qam, Qas, Qal = Quaternary Alluvium
Qat = Low Terrace Deposits
Qbc = Beaumont Formation
Qt = Fluviatile Terrace Deposits
Qao = Seymour Formation
Qle = Leona Formation
Q-Tu = Uvalde Gravel
Ewi = Wilcox Formation
Emi = Midway Group
Mc = Catahoula Formation
El = Laredo Formation
Kknm = Navarro Group and Marlbrook Marl
Kpg = Pecan Gap Chalk
Kau = Austin Chalk

Kef = Eagle Ford Shale
Kbu = Buda Limestone
Kdr = Del Rio Clay
Kft = Fort Terrett Member
Kgt = Georgetown Formation
Kep = Person Formation
Kek = Kainer Formation
Kes = Escondido Formation
Kew = Walnut Formation
Kgr = Glen Rose Formation
Kgru = Upper Glen Rose Formation
Kgrl = Lower Glen Rose Formation
Kh = Hensell Sand

KEY TO TERMS AND SYMBOLS (CONT'D)

TERMINOLOGY

SOIL STRUCTURE

Slickensided	Having planes of weakness that appear slick and glossy.
Fissured	Containing shrinkage or relief cracks, often filled with fine sand or silt; usually more or less vertical.
Pocket	Inclusion of material of different texture that is smaller than the diameter of the sample.
Parting	Inclusion less than 1/8 inch thick extending through the sample.
Seam	Inclusion 1/8 inch to 3 inches thick extending through the sample.
Layer	Inclusion greater than 3 inches thick extending through the sample.
Laminated	Soil sample composed of alternating partings or seams of different soil type.
Interlayered	Soil sample composed of alternating layers of different soil type.
Intermixed	Soil sample composed of pockets of different soil type and layered or laminated structure is not evident.
Calcareous	Having appreciable quantities of carbonate.
Carbonate	Having more than 50% carbonate content.

SAMPLING METHODS

RELATIVELY UNDISTURBED SAMPLING

Cohesive soil samples are to be collected using three-inch thin-walled tubes in general accordance with the Standard Practice for Thin-Walled Tube Sampling of Soils (ASTM D1587) and granular soil samples are to be collected using two-inch split-barrel samplers in general accordance with the Standard Method for Penetration Test and Split-Barrel Sampling of Soils (ASTM D1586). Cohesive soil samples may be extruded on-site when appropriate handling and storage techniques maintain sample integrity and moisture content.

STANDARD PENETRATION TEST (SPT)

A 2-in.-OD, 1-3/8-in.-ID split spoon sampler is driven 1.5 ft into undisturbed soil with a 140-pound hammer free falling 30 in. After the sampler is seated 6 in. into undisturbed soil, the number of blows required to drive the sampler the last 12 in. is the Standard Penetration Resistance or "N" value, which is recorded as blows per foot as described below.

SPLIT-BARREL SAMPLER DRIVING RECORD

Blows Per Foot	Description
25	25 blows drove sampler 12 inches, after initial 6 inches of seating.
50/7"	50 blows drove sampler 7 inches, after initial 6 inches of seating.
Ref/3"	50 blows drove sampler 3 inches during initial 6-inch seating interval.

NOTE: To avoid damage to sampling tools, driving is limited to 50 blows during or after seating interval.

PROJECT NO. ASA17-096-00

RESULTS OF SOIL SAMPLE ANALYSES

PROJECT NAME: J.K. Spruce - Calaveras Lake Power Plant
Proposed Two New Coal Combustion Residual Containment Ponds
San Antonio, Texas

FILE NAME: ASA17-096-00.GPJ 2/5/2018

Boring No.	Sample Depth (ft)	Blows per ft	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	USCS	Dry Unit Weight (pcf)	% -200 Sieve	Shear Strength (tsf)	Strength Test
B-1	1.0 to 2.5	8							28		
	3.0 to 4.5									2.25	PP
	4.5 to 5.0										
	6.0 to 7.5		16	27	16	11		107			
	7.5 to 8.0										
	8.5 to 10.0	16	22								
	11.0 to 12.5		20						36		
	13.5 to 15.0	25									
	16.0 to 17.5		15								
	18.5 to 20.0	14	27								
	21.0 to 22.5										
	23.5 to 24.7	50/8"	24								
	28.5 to 29.6	50/7"									
	33.5 to 34.7	50/8"	23						48		
	38.5 to 40.0	35									
	43.5 to 45.0	37	26								
	48.5 to 49.7	50/8"									
B-2	1.0 to 2.5	7	19								
	3.0 to 4.5		15					114	52	1.82	UC
	4.5 to 5.0										
	6.0 to 7.5		14					115		2.25	PP
	7.5 to 8.0										
	8.5 to 10.0	15	10								
	11.0 to 12.5										
	13.5 to 15.0	28	13								
	16.0 to 17.5		25					91	27	0.50	PP
	17.5 to 18.0										
	18.5 to 20.0	44									
	21.0 to 22.5		29								
	23.5 to 24.7	28									
	28.5 to 29.6		4								
	33.5 to 34.7	40									
	38.5 to 40.0	50	25								
	43.5 to 45.0	50									
	48.5 to 49.7	26	33								
B-3	1.0 to 2.5	16	19								
	3.0 to 4.5		15					110		1.50	PP
	4.5 to 5.0										
	6.0 to 7.5										

PP = Pocket Penetrometer TV = Torvane UC = Unconfined Compression FV = Field Vane UU = Unconsolidated Undrained Triaxial

CU = Consolidated Undrained Triaxial

RESULTS OF SOIL SAMPLE ANALYSES

PROJECT NAME: J.K. Spruce - Calaveras Lake Power Plant
Proposed Two New Coal Combustion Residual Containment Ponds
San Antonio, Texas

FILE NAME: ASA17-096-00.GPJ 2/5/2018

Boring No.	Sample Depth (ft)	Blows per ft	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	USCS	Dry Unit Weight (pcf)	% -200 Sieve	Shear Strength (tsf)	Strength Test
B-3	8.5 to 10.0	19	18								
	11.0 to 12.5		17					101		2.25	PP
	12.5 to 13.0										
	13.5 to 15.0	28	21								
	16.0 to 17.5									2.25	PP
	17.5 to 18.0										
	18.5 to 20.0	12									
	21.0 to 22.5		25							0.63	PP
	22.5 to 23.0										
	23.5 to 24.7	19									
	28.5 to 29.6	50	20								
	33.5 to 34.7	50									
	38.5 to 40.0	36	22								
	43.5 to 45.0	50									
	48.5 to 49.7	44	25								
B-4	1.0 to 2.5	11	16						23		
	3.0 to 4.5									1.25	PP
	4.5 to 5.0										
	6.0 to 7.5									2.25	PP
	7.5 to 8.0										
	8.5 to 10.0	34	20						76		
	11.0 to 12.5		16								
	13.5 to 15.0	39	10								
	16.0 to 17.5		19								
	18.5 to 20.0	50/10"									
	21.0 to 22.5		27								
	23.5 to 24.7	50/8"									
	28.5 to 29.6	50/7"	25								
	33.5 to 34.7	50/8"									
	38.5 to 40.0	37	22						50		
	43.5 to 45.0	50/10"									
	48.5 to 49.7	50/8"	27								
MW-1	1.0 to 2.5	36	9						55		
	3.0 to 4.5		13					112		2.25	PP
	4.5 to 5.0										
	6.0 to 7.5									2.25	PP
	7.5 to 8.0										
	8.5 to 10.0	18	12								
	11.0 to 12.5		10						29		

PP = Pocket Penetrometer TV = Torvane UC = Unconfined Compression FV = Field Vane UU = Unconsolidated Undrained Triaxial

CU = Consolidated Undrained Triaxial

RESULTS OF SOIL SAMPLE ANALYSES

PROJECT NAME: J.K. Spruce - Calaveras Lake Power Plant
Proposed Two New Coal Combustion Residual Containment Ponds
San Antonio, Texas

FILE NAME: ASA17-096-00.GPJ 2/5/2018

Boring No.	Sample Depth (ft)	Blows per ft	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	USCS	Dry Unit Weight (pcf)	% -200 Sieve	Shear Strength (tsf)	Strength Test
MW-1	13.5 to 15.0	49	10								
	16.0 to 17.5										
	18.5 to 20.0	50/9"	12								
	21.0 to 22.5										
	23.5 to 24.7	50/10"	21								
	28.5 to 29.6	50/9"									
	33.5 to 34.7	50/7"	24								
MW-2	1.0 to 2.5	11	15								
	3.0 to 4.5		15					117		2.64	UC
	4.5 to 5.0										
	6.0 to 7.5									1.75	PP
	7.5 to 8.0										
	8.5 to 10.0	38	12								
	11.0 to 12.5		15								
	13.5 to 15.0	22	20						34		
	16.0 to 17.5										
	18.5 to 20.0	25	26								
	21.0 to 22.5										
	23.5 to 24.7	50/8"	24								
	28.5 to 29.6	50/9"									
	33.5 to 34.7	50	22						45		
MW-3	1.0 to 2.5	16	20								
	3.0 to 4.5		9								
	6.0 to 7.5		13							1.38	PP
	7.5 to 8.0										
	8.5 to 10.0	35	20								
	11.0 to 12.5		20								
	13.5 to 15.0	50	11								
	16.0 to 17.5		11								
	18.5 to 20.0	50									
	21.0 to 22.5		19	35	24	11		94		1.13	PP
	22.5 to 23.0										
	23.5 to 24.7	50									
	28.5 to 29.6	50	23								
	33.5 to 34.7	50									
MW-4	1.0 to 2.5	5	24								
	3.0 to 4.5									1.00	PP
	4.5 to 5.0										
	6.0 to 7.5									1.50	PP

PP = Pocket Penetrometer TV = Torvane UC = Unconfined Compression FV = Field Vane UU = Unconsolidated Undrained Triaxial

CU = Consolidated Undrained Triaxial

RESULTS OF SOIL SAMPLE ANALYSES

PROJECT NAME: J.K. Spruce - Calaveras Lake Power Plant
Proposed Two New Coal Combustion Residual Containment Ponds
San Antonio, Texas

FILE NAME: ASA17-096-00.GPJ 2/5/2018

Boring No.	Sample Depth (ft)	Blows per ft	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	USCS	Dry Unit Weight (pcf)	% -200 Sieve	Shear Strength (tsf)	Strength Test
MW-4	7.5 to 8.0										
	8.5 to 10.0	8	19								
	11.0 to 12.5		20					93	20	0.38	PP
	12.5 to 13.0										
	13.5 to 15.0	27	24								
	16.0 to 17.5		28					99		0.75	PP
	17.5 to 18.0										
	18.5 to 20.0	50	25								
MW-5	1.0 to 2.5	11									
	3.0 to 4.5		25	41	15	26	CL		56		
	6.0 to 7.5									1.13	PP
	7.5 to 8.0										
	8.5 to 10.0	16	15								
	11.0 to 12.5		14								
	13.5 to 15.0		23								
	16.0 to 17.5										
	18.5 to 20.0	49	28								
	21.0 to 21.8	50/3"							38		
	23.5 to 25.0		23								
	28.5 to 29.6	50									
	33.5 to 34.7	43	21								
MW-6	1.0 to 2.5	10	15								
	3.0 to 4.5	28	12	36	15	21	SC		37		
	6.0 to 7.5		10					111		1.78	UC
	7.5 to 8.0										
	8.5 to 10.0	13	15								
	11.0 to 12.5									0.50	PP
	12.5 to 13.0										
	13.5 to 15.0	50/7"	14								
	16.0 to 17.5		10						22		
	18.5 to 20.0	50	9								
	21.0 to 22.5									0.50	PP
	22.5 to 23.0										
	23.5 to 24.7	50	17								
	28.5 to 29.6	50									
	33.5 to 34.7	50	12								
MW-7	1.0 to 2.5	4	36								
	3.0 to 4.5								40	1.75	PP
	4.5 to 5.0										

PP = Pocket Penetrometer TV = Torvane UC = Unconfined Compression FV = Field Vane UU = Unconsolidated Undrained Triaxial

CU = Consolidated Undrained Triaxial

FIGURE 14d

RESULTS OF SOIL SAMPLE ANALYSES

PROJECT NAME: J.K. Spruce - Calaveras Lake Power Plant
Proposed Two New Coal Combustion Residual Containment Ponds
San Antonio, Texas

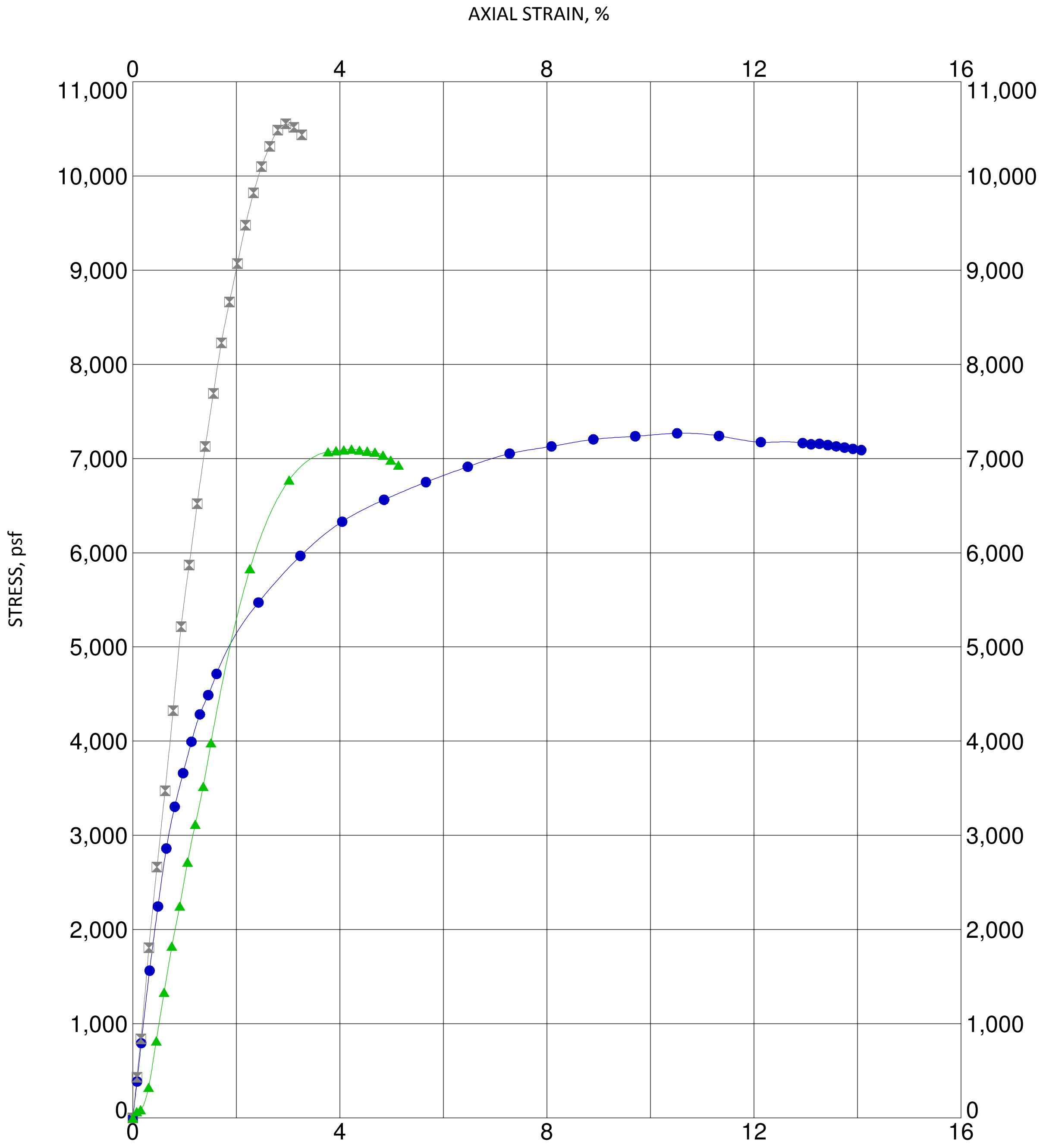
FILE NAME: ASA17-096-00.GPJ 2/5/2018

Boring No.	Sample Depth (ft)	Blows per ft	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	USCS	Dry Unit Weight (pcf)	% -200 Sieve	Shear Strength (tsf)	Strength Test
MW-7	6.0 to 7.5									1.00	PP
	7.5 to 8.0										
	8.5 to 10.0	25	13								
	11.0 to 12.5		21					108		0.50	PP
	12.5 to 13.0										
	13.5 to 15.0	27	32								
	16.0 to 17.5									0.63	PP
	17.5 to 18.0										
	18.5 to 20.0	50	26								
	21.0 to 22.5									0.25	PP
	22.5 to 23.0										
	23.5 to 25.0	49	24								
	28.5 to 29.6	50									
	33.5 to 34.7	28	23								

PP = Pocket Penetrometer TV = Torvane UC = Unconfined Compression FV = Field Vane UU = Unconsolidated Undrained Triaxial

CU = Consolidated Undrained Triaxial

PROJECT NO. ASA17-096-00



Specimen Identification		Classification	Shear Str. (tsf)	Failure Strain (%)	PI	Dry Unit Weight (pcf)	w (%)
● B-2	3 ft	Sandy Clay - CL	1.8	10.5		114.0	14.6
⧗ MW-2	3 ft	Sandy Clay - CL	2.6	3.0		117.5	14.5
▲ MW-6	6 ft	Clayey Sand - SC	1.8	4.2	21	111.4	10.4

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(210) 699-6426 fax
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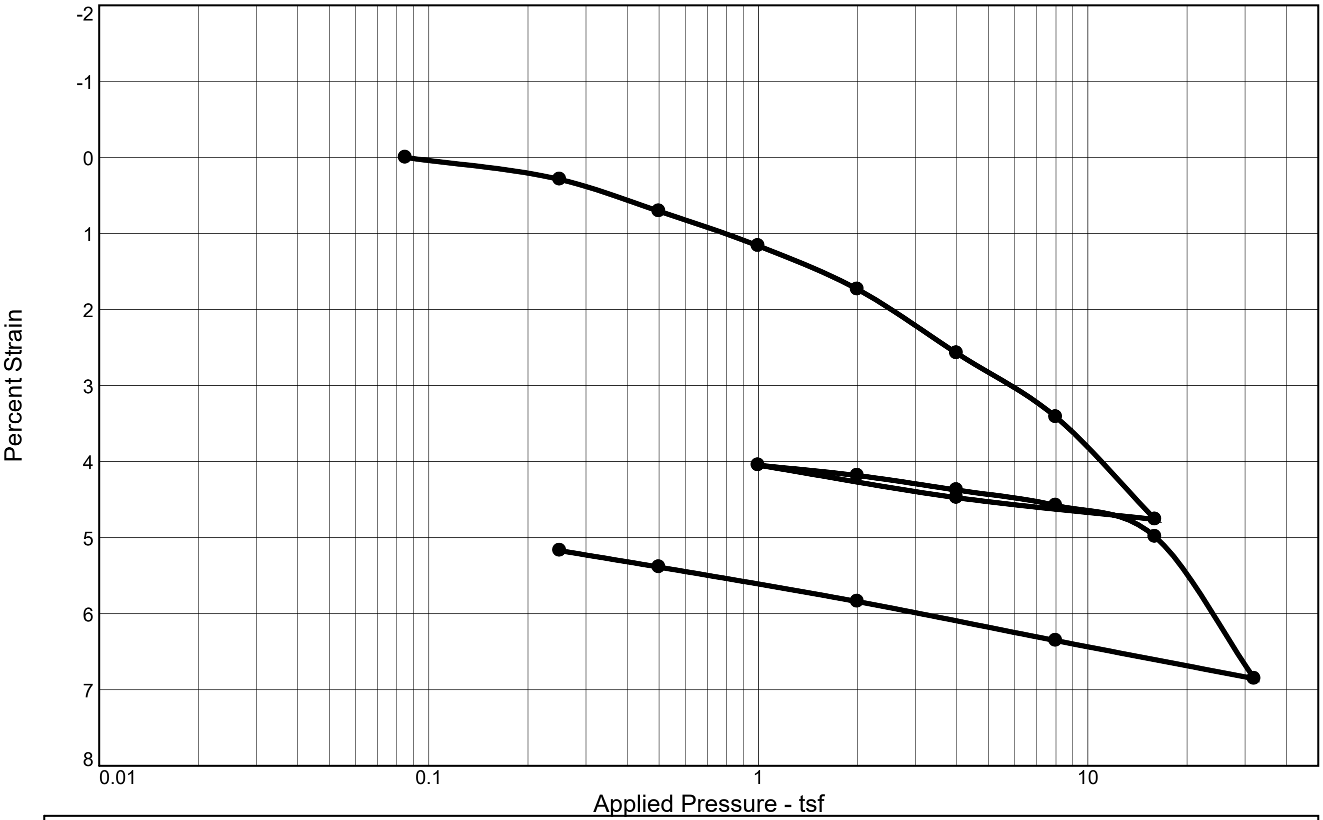
UNCONFINED COMPRESSION

J.K. Spruce - Calaveras Lake Power Plant
Proposed Two New Coal Combustion Residual
Containment Ponds
San Antonio, Texas

R-K UNCONFINED COMPRESSION ASA17-096-00.GPJ RKCI SAN ANTONIO-1.GDT 1/25/18

FIGURE 15

CONSOLIDATION TEST REPORT



Coefficients of Consolidation and Secondary Consolidation

No.	Load (tsf)	C_v (ft.^{2}/day)	C_α	No.	Load (tsf)	C_v (ft.^{2}/day)	C_α	No.	Load (tsf)	C_v (ft.^{2}/day)	C_α
2	0.25	3.638		9	4.00	6.829		16	8.00	3.188	
3	0.50	0.969		10	1.00	3.486		17	2.00	3.233	
4	1.00	0.901		11	2.00	14.265		18	0.50	1.630	
5	2.00	1.640		12	4.00	7.855		19	0.25	0.239	
6	4.00	0.893		13	8.00	0.470					
7	8.00	0.960		14	16.00	0.794					
8	16.00	1.672		15	32.00	0.347					

Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
82.1 %	25.3 %	91.1	N/A	N/A	2.65	0.97	0.8	0.05	0.03	0.816

MATERIAL DESCRIPTION	USCS	AASHTO
Silty Sand	SM	

Project No. ASA17-096 **Client:** Pape-Dawson Engineers

Project: CCR Containment Ponds- Calaveras Lake

Location: Boring 2 Sample 9 16-18ft **Depth:** 16-18 **Sample Number:** 9

Remarks:
ASTM D2435
estimated specific gravity
weight added to prevent swell after inundation=0.085tsf

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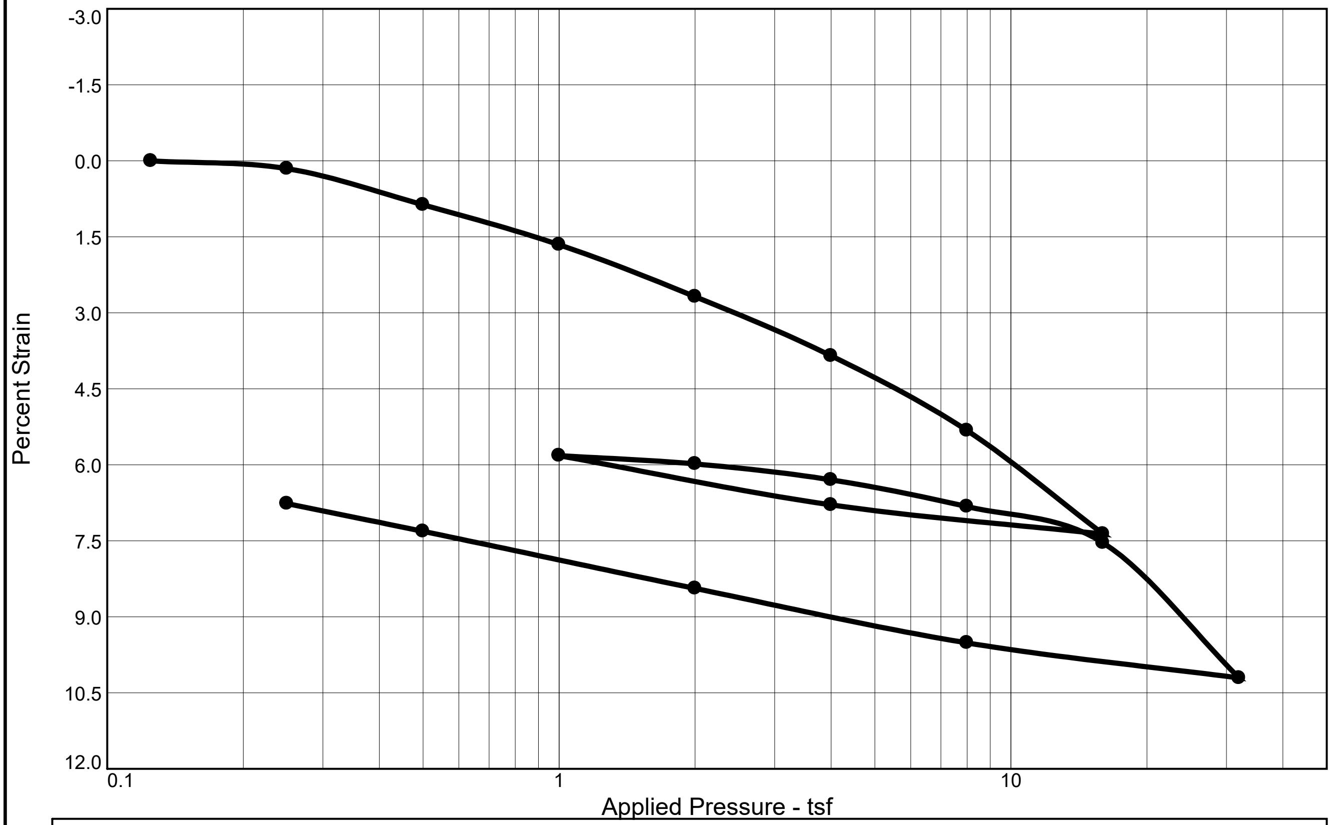
Figure 16a

CONSOLIDATION TEST REPORT



Coefficients of Consolidation and Secondary Consolidation

No.	Load (tsf)	C_v (ft.^{2}/day)	C_α	No.	Load (tsf)	C_v (ft.^{2}/day)	C_α	No.	Load (tsf)	C_v (ft.^{2}/day)	C_α
2	0.25	2.156		9	4.00	41.121		16	8.00	2.394	
3	0.50	0.937		10	1.00	0.440		17	2.00	0.781	
4	1.00	0.878		11	2.00	3.224		18	0.50	0.410	
5	2.00	0.896		12	4.00	2.967		19	0.25	0.043	
6	4.00	1.904		13	8.00	1.799					
7	8.00	2.991		14	16.00	3.851					
8	16.00	0.940		15	32.00	1.595					

Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
79.6 %	18.8 %	101.7	N/A	N/A	2.65	.72	0.3	0.04	0.05	0.627

MATERIAL DESCRIPTION	USCS	AASHTO
Silty Sand	SM	

Project No. ASA17-096 **Client:** Pape-Dawson Engineers

Project: CCR Containment Ponds- Calaveras Lake

Location: Boring 3 Sample 7 11-13ft **Depth:** 11-13 **Sample Number:** 7

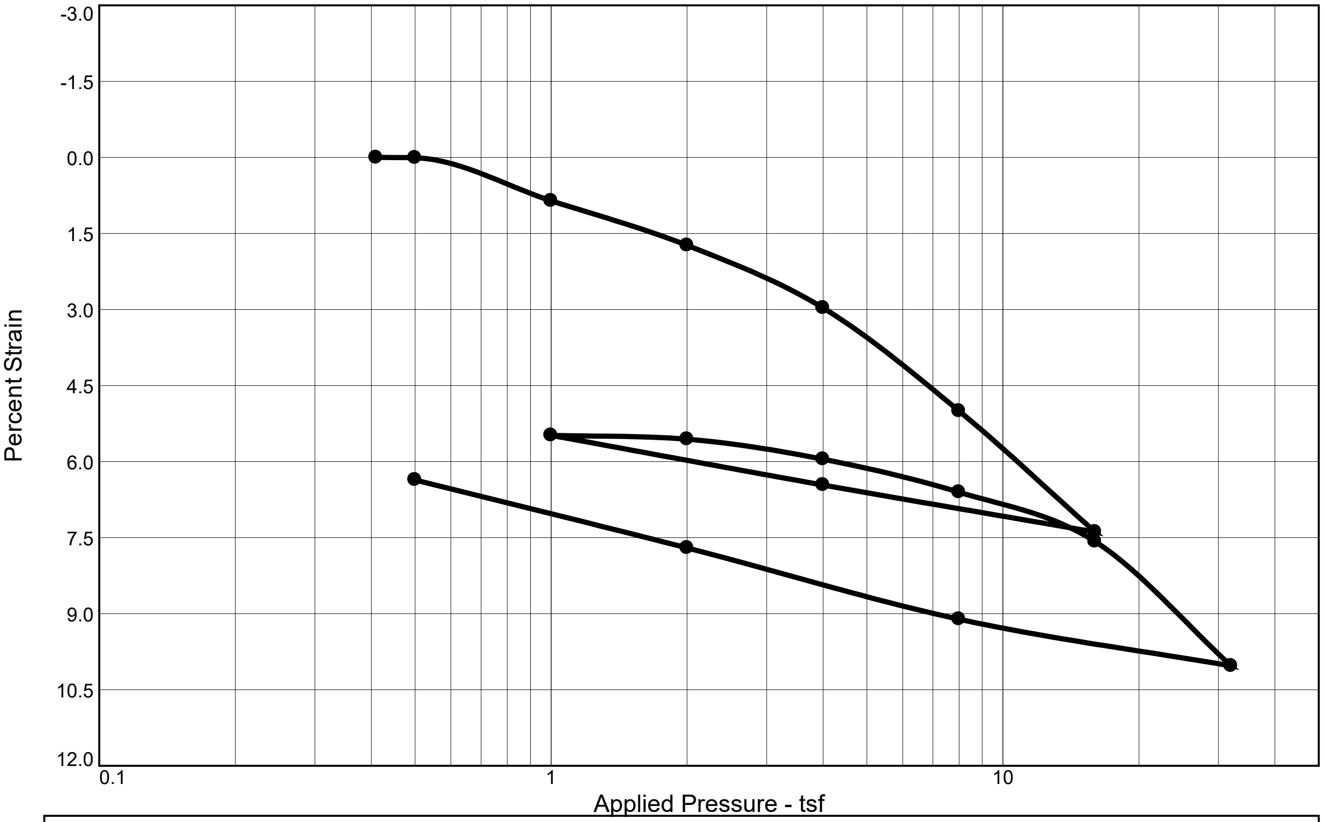
RABA KISTNER CONSULTANTS, INC.

Remarks:

ASTM D2435
estimated specific gravity
weight added to prevent swell after inundation=0.125tsf

Figure 16b

CONSOLIDATION TEST REPORT



Coefficients of Consolidation and Secondary Consolidation

No.	Load (tsf)	C_v (ft.^{2}/day)	C_α	No.	Load (tsf)	C_v (ft.^{2}/day)	C_α	No.	Load (tsf)	C_v (ft.^{2}/day)	C_α
2	0.50	0.596		9	1.00	0.083		16	2.00	0.023	
3	1.00	3.082		10	2.00	4.172		17	0.50	0.005	
4	2.00	2.028		11	4.00	1.426					
5	4.00	1.837		12	8.00	0.443					
6	8.00	6.282		13	16.00	0.388					
7	16.00	0.854		14	32.00	0.100					
8	4.00	1.454		15	8.00	1.404					

Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Initial Void Ratio
Saturation	Moisture									
78.5 %	13.9 %	112.5	N/A	N/A	2.65	0.26	0.6	0.05	0.04	0.471

MATERIAL DESCRIPTION	USCS	AASHTO
Sandy Clay	CL	

Project No. ASA17-096 **Client:** Pape-Dawson Engineers

Project: CCR Containment Ponds- Calaveras Lake

Location: Boring MW-1 Sample 2 3-5ft **Depth:** 3-5 **Sample Number:** 2

Remarks:
ASTM D2435
estimated specific gravity
weigth added to prevent swell after inundation=0.41tsf

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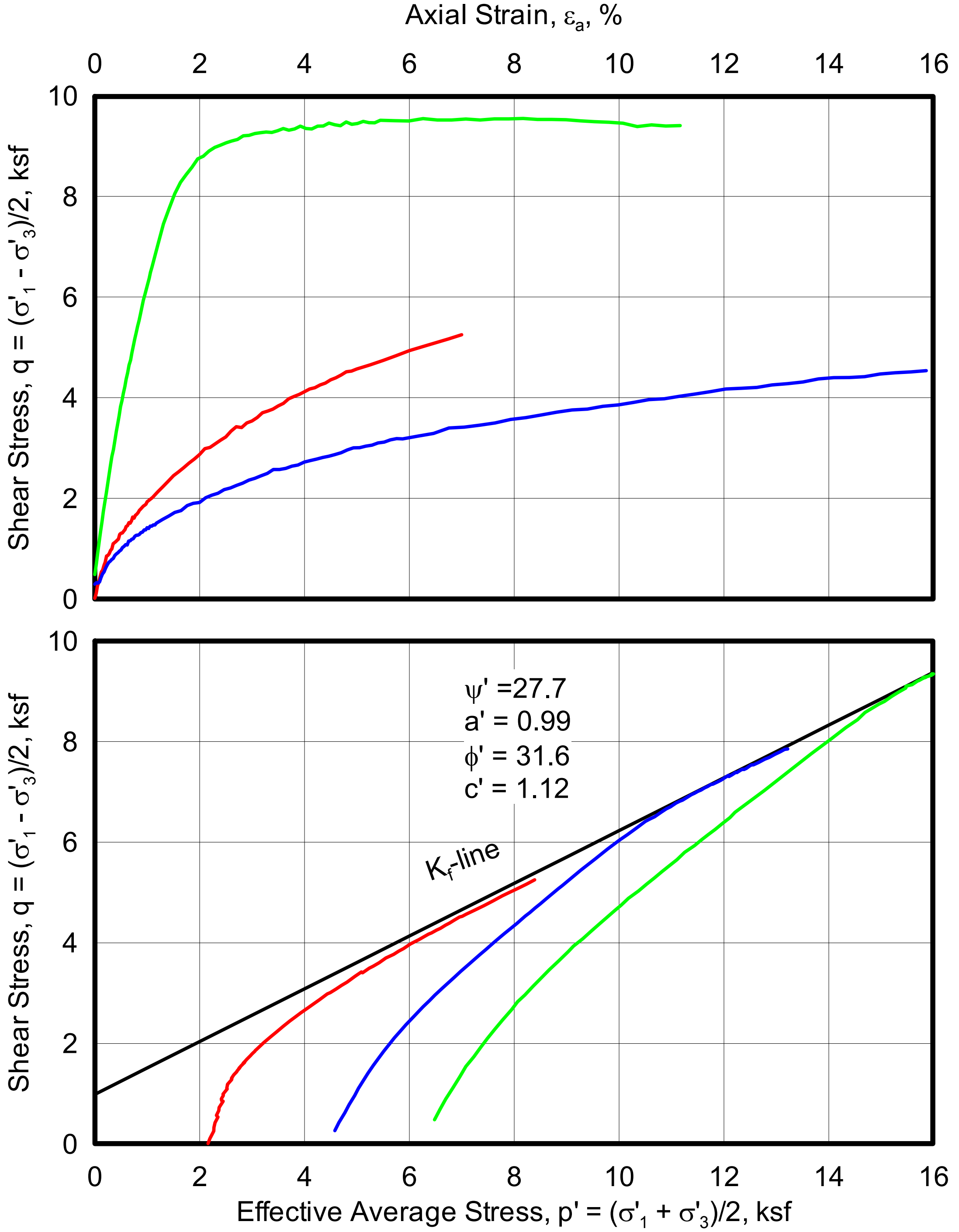
Figure 16c

PROJECT ASA17-096-00



MULTI STAGE TRIAXIAL UNDRAINEDCOMPRESSION TEST RESULTS

ISOTROPICALLY CONSOLIDATED - STRESS PATH
SINGLE SAMPLE MULTI- STAGE CU

MATERIAL: Silty Sand - SM
INITIAL WATER CONTENT: 27.97%
INITIAL DRY UNIT WEIGHT: 99.69 pcf
INITIAL VOID RATIO: 0.66
SPECIFIC GRAVITY: 2.65 (assumed)

FINAL WATER CONTENT: 27.42%
INITIAL DEGREE OF SATURATION: 92.6%
FINAL DEGREE OF SATURATION: 100.0%

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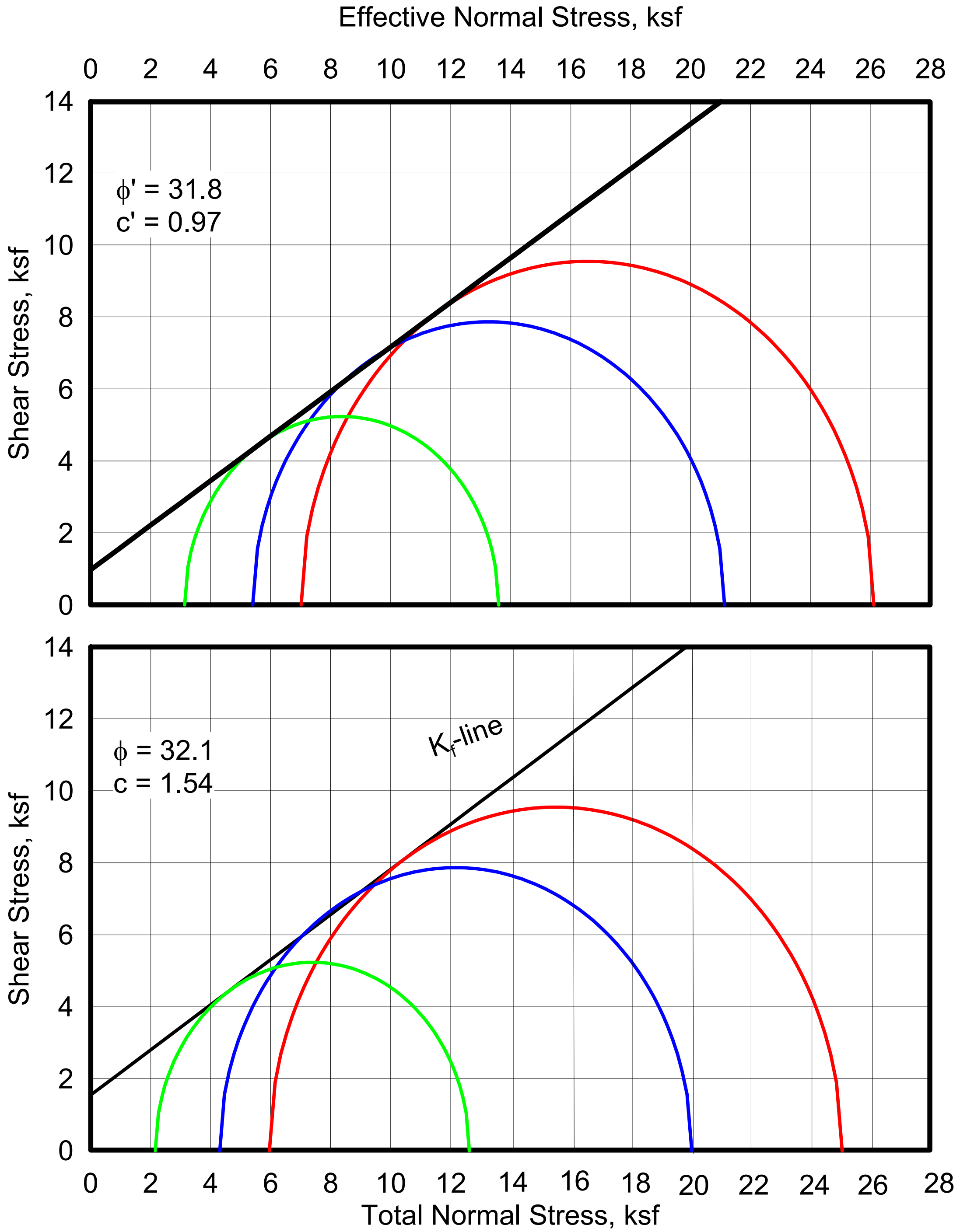
J.K. SPRUCE –CALAVERAS LAKE POWER PLANT
PROPOSED NEW COAL COMBUSTION RESIDUAL PONDS
SAN ANTONIO, TEXAS
STRESS PATH
BORING MW-4, DEPTH 16 TO 18 FT

FIGURE 17a

PROJECT ASA17-096-00



MULTI STAGE TRIAXIAL UNDRAINEDCOMPRESSION TEST RESULTS

ISOTROPICALLY CONSOLIDATED - STRESS PATH
SINGLE SAMPLE MULTI-STAGE CU

MMATERIAL: Silty Sand - SM
INITIAL WATER CONTENT: 27.97%
INITIAL DRY UNIT WEIGHT: 99.69 pcf
INITIAL VOID RATIO: 0.66
SPECIFIC GRAVITY: 2.65 (assumed)

FINAL WATER CONTENT: 27.42%
INITIAL DEGREE OF SATURATION: 92.6%
FINAL DEGREE OF SATURATION: 100.0%

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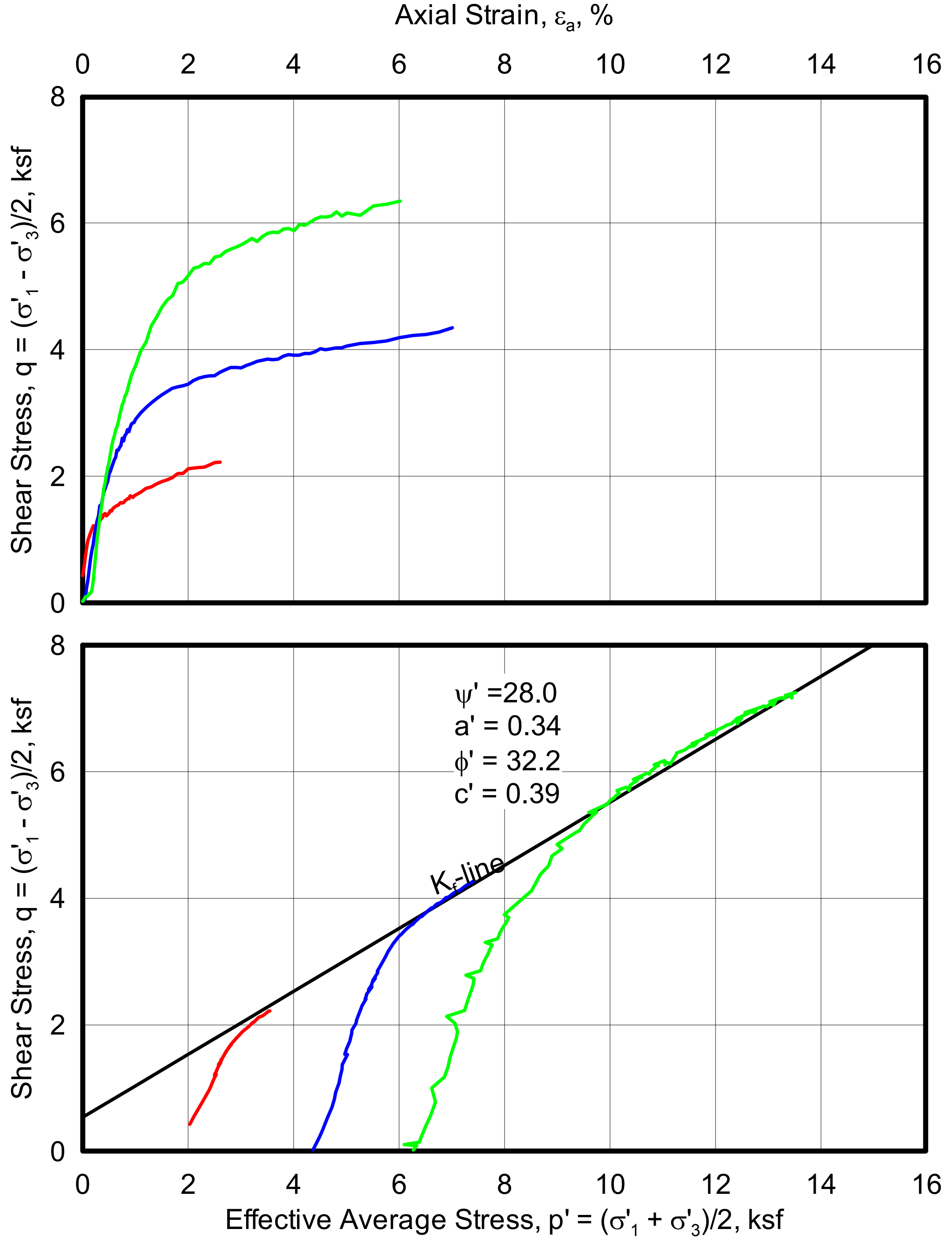
SAN ANTONIO, TEXAS
MOHR CIRLE
BORING MW-4, DEPTH 16 TO 18 FT

PROJECT NO. ASA17-096-00



MULTI STAGE TRIAXIAL UNDRAINEDCOMPRESSION TEST RESULTS

ISOTROPICALLY CONSOLIDATED - STRESS PATH
SINGLE SAMPLE MULTI STAGE CU

MATERIAL: Sandy Clay-(SC)
INITIAL WATER CONTENT: 16.29%
INITIAL DRY UNIT WEIGHT: 107.26 pcf
INITIAL VOID RATIO: 0.60
SPECIFIC GRAVITY: 2.74 (measured)

FINAL WATER CONTENT: 19.92%
INITIAL DEGREE OF SATURATION: 75.0%
FINAL DEGREE OF SATURATION: 100.0%
LL = 27 ; PL = 16 ; PI = 11

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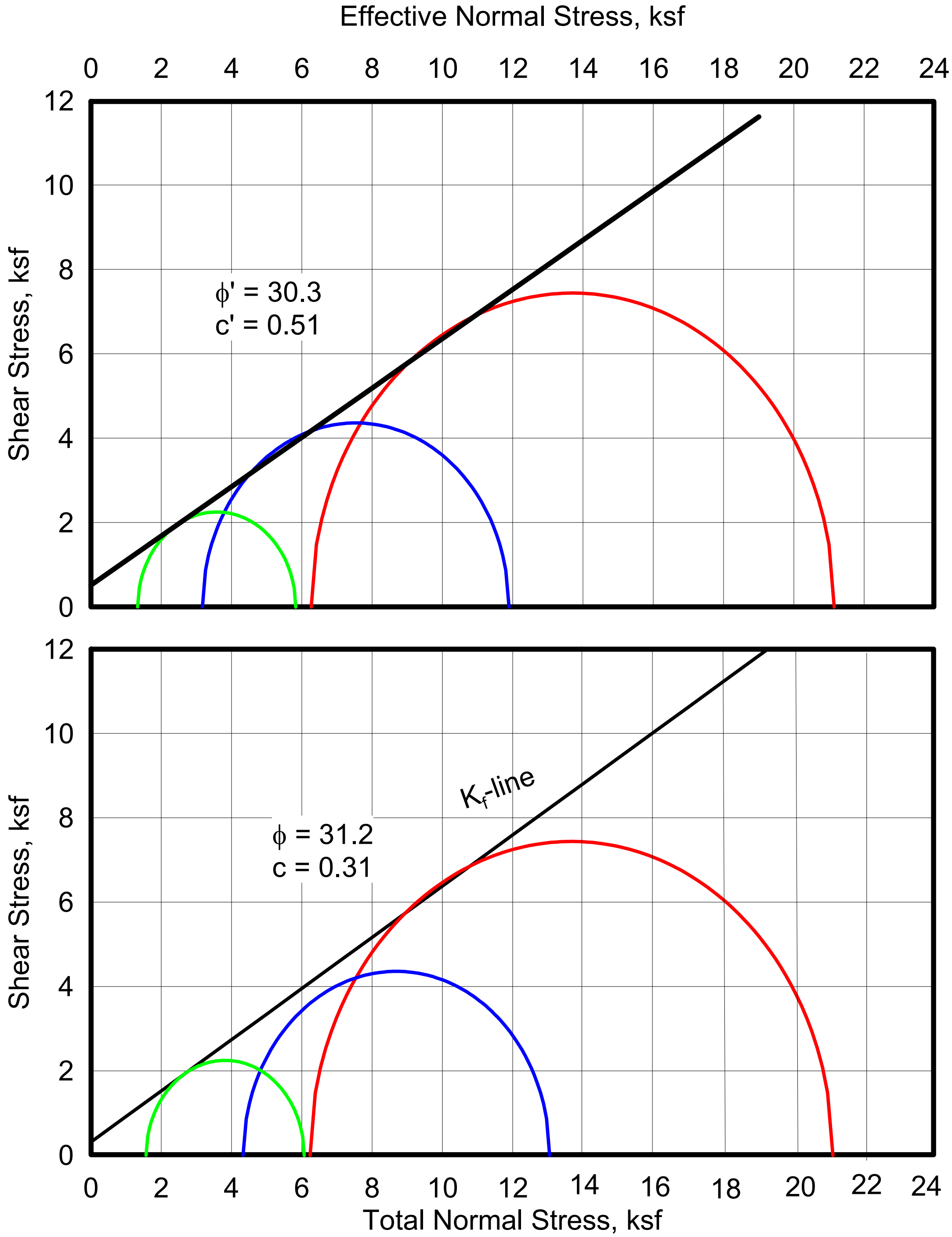
J.K. SPRUCE –CALAVERAS LAKE POWER PLANT PROPOSED NEW COAL COMBUSTION RESIDUAL PONDS SAN ANTONIO, TEXAS
STRESS PATH
BORING B-1, DEPTH 6 TO 8 FT

FIGURE 17c

PROJECT No. ASA17-096-00



MULTI STAGE TRIAXIAL UNDRAINEDCOMPRESSION TEST RESULTS

ISOTROPICALLY CONSOLIDATED - STRESS PATH
SINGLE SAMPLE MULTI STAGE CU

MMATERIAL: Reddish brown Clayey Sand (SC), w/ stone and clay layers

INITIAL WATER CONTENT: 16.29%	FINAL WATER CONTENT: 19.92%
INITIAL DRY UNIT WEIGHT: 107.26 pcf	INITIAL DEGREE OF SATURATION: 75.0%
INITIAL VOID RATIO: 0.60	FINAL DEGREE OF SATURATION: 100.0%
SPECIFIC GRAVITY: 2.74 (measured)	LL = 27; PL = 16; PI = 11

FIGURE

RABA KISTNER
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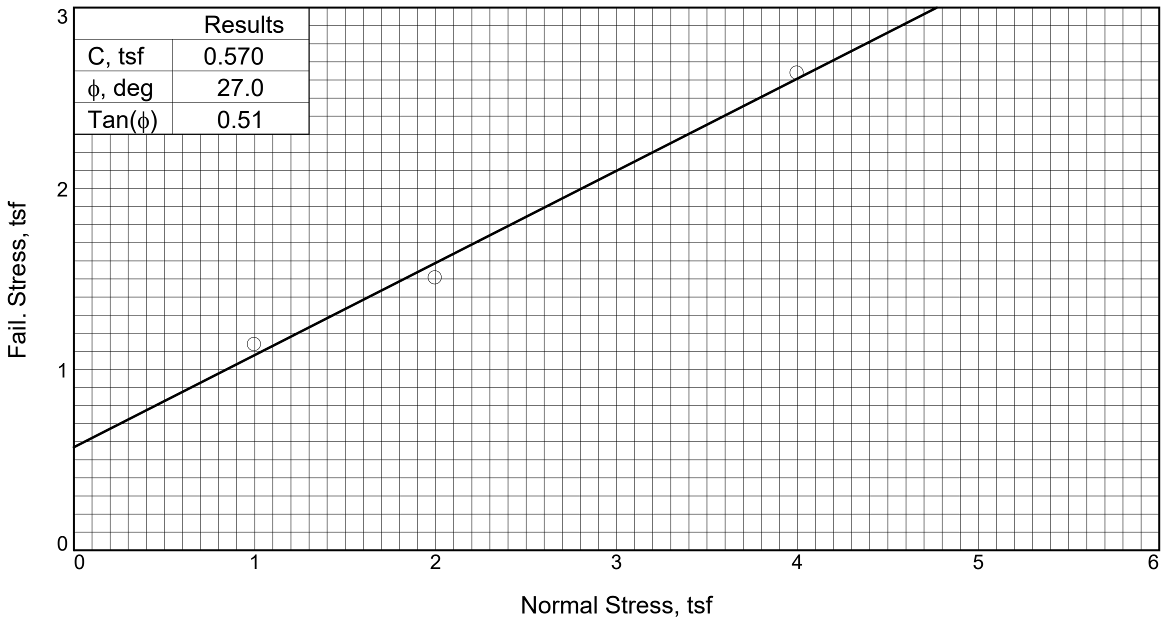
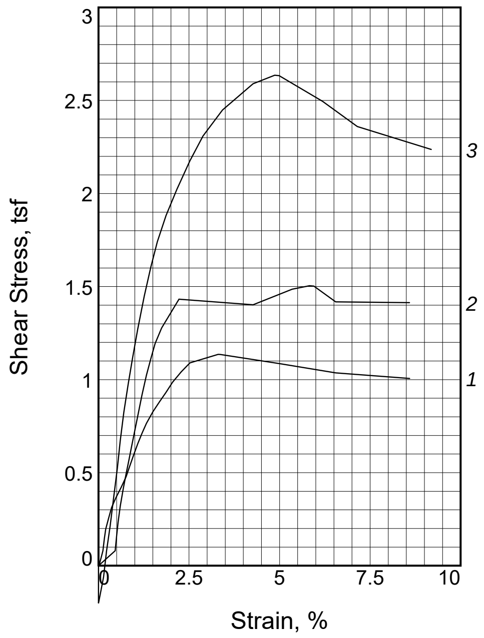
J.K. SPRUCE –CALAVERAS LAKE POWER PLANT PROPOSED NEW COAL COMBUSTION RESIDUAL PONDS
SAN ANTONIO, TEXAS
MOHR CIRLE
BORING B-1, DEPTH 6 TO 8 FT



	Results
C, tsf	0.570
ϕ, deg	27.0
Tan(ϕ)	0.51



Sample No.		1	2	3
Initial	Water Content, %	18.0	17.4	19.9
	Dry Density, pcf	94.6	94.2	94.9
	Saturation, %	64.0	61.4	71.2
	Void Ratio	0.7429	0.7503	0.7375
	Diameter, in.	2.50	2.50	2.50
	Height, in.	0.99	0.99	0.99
At Test	Water Content, %	31.4	31.2	31.4
	Dry Density, pcf	93.0	92.9	91.3
	Saturation, %	107.2	106.4	102.8
	Void Ratio	0.7734	0.7758	0.8071
	Diameter, in.	2.50	2.50	2.50
	Height, in.	1.01	1.01	1.03
Normal Stress, tsf		1.000	2.000	4.000
Fail. Stress, tsf		1.136	1.504	2.636
Strain, %		3.3	5.8	4.9
Ult. Stress, tsf				
Strain, %				
Strain rate, in./min.		0.00	0.00	0.00

Sample Type: Silty Sand - SM

Description: Tan to gray

LL= 35 **PL=** 24 **PI=** 11

Specific Gravity= 2.642

Remarks: MTE# 21-011

Figure 18

Client:

Project: J.K. SPRUCE –CALAVERAS LAKE POWER PLANT
PROPOSED NEW COAL COMBUSTION RESIDUAL PONDS
SAN ANTONIO, TEXAS

Location: MW-3

Sample Number: 12 **Depth:** 21-23FT

Proj. No.: ASA17-096-00 **Date Sampled:**

DIRECT SHEAR TEST REPORT

RABA-KISTNER CONSULTANTS, INC.

Tested By: Chain **Checked By:** JB

APPENDIX A
Seismic Vs100 Geophysical Investigation

GEOPHYSICS AND NDE FOR INFRASTRUCTURE ASSESSMENT

MEASURING CONCERNS • MITIGATING RISKS

Olson
ENGINEERING

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Wheat Ridge, CO 80033 USA
phone: 303.423.1212
fax: 303.423.6071

January 5, 2018

Raba Kistner Consultants, Inc.
12821 W Golden Lane
San Antonio, TX 78249

Attn: Eric Neuner, P.E.
Phone: 210.699.9090
Email: eneuner@rkci.com

Re: Seismic (Vs100) Geophysical Investigation
San Antonio CPS
San Antonio, TX
Olson Project No. 5966A

Olson Engineering, Inc. (Olson) conducted a geophysical investigation located at the CPS Energy Facility, southeast of San Antonio, TX (Figure 1). The objective of the survey was to obtain the one-dimensional (1D) vertical distribution of shear-wave velocities to a depth of 100 feet (~30 meters) to determine the IBC average shear-wave velocity; that is, the Vs100 (feet) or Vs30 (meters). To meet the objective, a geophysical survey was completed using the passive Multi-channel Analysis of Surface Waves (MASW) method.

The survey was performed based on the scope of work outlined in Olson Proposal No. P2017357.1PG. The field work was conducted on December 13th, 2017 by Olson geophysicist Miriam Moller. The following report presents results from the surface wave investigation and summarizes the site conditions, field methods, data acquisition and interpretation procedures. For further information regarding the intricacies of the MASW technique for determination of Vs100, Olson can submit an addendum to this report upon request.

Figure 1. A) Approximate site location indicated by red star; b) line locations indicated by red lines.

Data Acquisition

The geophysical lines were collected with 24 4.5 Hz geophones spaced 10 feet apart for a total length of 230 feet (*inset photo at right*). Seismic data were acquired using a Geometrics Geode 24-channel digital seismograph. This system utilizes a state-of-the-art, 24-bit seismograph connected to a field laptop via Ethernet cable. Analog data from the geophones are collected in the Geode seismograph where the data are digitized, transmitted to the laptop computer, and then recorded on the hard drive.

There are no predefined source points for passive-source surface seismic surveys. Instead, the method uses ambient noise, or vibrational energy, that exists at a site. Small-strain vibrations generated by vehicular motion and other activities create surface wave energy that propagates in all directions across a site. For this project, additional 'sources' of ambient noise were generated with a sledgehammer and moving vehicle off the end of the line to improve the signal-to-noise ratio. It is best to orient each array such that surface wave energy propogates along the array. When using the passive surface wave method, this 'ambient signal' is the wave-energy measured and recorded for analysis. A minimum of 12 unfiltered 32 second ambient vibrational energy records were recorded for each line using a 2 millisecond (ms) sample rate.

Figure 1b (above) shows the layout of the four lines where MASW seismic data were acquired at the site. Line numbering is purely sequential to the order of acquisition. Locations for the seismic lines were selected based on the site access, crew & equipment safety, and ability to collect quality data.

Data Processing

Passive MASW analysis consists of generating a frequency-velocity transform from surface waves, picking the transformed data to derive a dispersion curve, and inverting this dispersion curve to a layered Vs model. Figure 2 illustrates the dispersion curve picking approach used for passive MASW records, with a sample from Line 3 of this investigation. These steps are repeated for each sounding location using all 24 geophones at a time, resulting in a one-dimensional (1D) layered Vs sounding model. The program SurfSeis, version 5.3, by the Kansas Geological Survey was used to accomplish these steps. In addition to providing a 1D Vs sounding, the layer-weighted average Vs value is computed to a total depth of 100 feet (~30 meters) for each sounding site, in accordance with the IBC 2009 specifications. This approach is generally conservative, as velocity is much more likely to increase with depth than it is to stay constant or decrease. This computation yields the Vs100 foot (or Vs30 meter) value, detailed in Table 1613.5.5 of the 2009 International Building Code (IBC).

While four lines were collected, the results of Line 1 were of poor quality, and as such are not presented or used in the overall Vs100 calculation for the site. The dispersion curve which was generated was of poor quality and as such, so was the resultant 1D sounding.

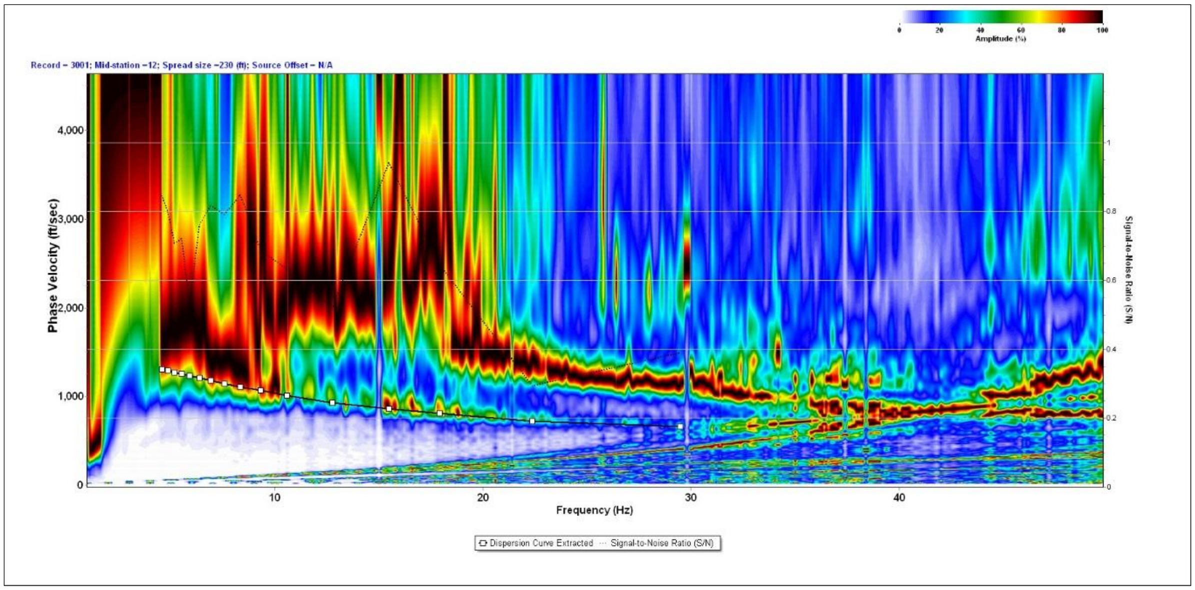


Figure 2. Example dispersion curve from Line 3 of this investigation.

Vs100 Results

The shear-wave velocity curves derived from the MASW method are presented in a single plot on Figure 3, and tabulated in Table 1. The 1D Vs graphs represent a seismic sounding centered at the middle of each line. Olson makes an attempt to collect multiple lines at any given site in order to show if any variation in the subsurface seismic conditions exist; as well as acquire records with ambient energy approaching the linear array of geophones from different angles.

The passive surface-wave data obtained at this site produced Vs100 values of (*using equation 16-40, IBC 2009, section 1613.5.5*):

Line 2 Vs100 = 1,080 ft/s
Line 3 Vs100 = 1,062 ft/s
Line 4 Vs100 = 1,106 ft/s

The average value for the three seismic lines at this site is **Vs100= 1,083 ft/s (330 meters/second)**. The results from the 1D Vs graph indicate generally increasing velocity values with depth. Vs100 values listed above, and presented in Figure 3, were computed in order to be used with Table 1613.5.5 of IBC 2009, or current equivalent, for determining the Site Class. Based on our experience, Vs100 results from passive surface-wave testing have been found to fall within 10 to 15% of Vs data obtained via more expensive crosshole or downhole seismic testing.

Table 1. Tabulated velocity results for three MASW lines.

Line 2 - Vs 100 = 1080 ft/s			
Depth Range (feet)			Vs (ft/s)
0.0	-	4.6	746
4.6	-	10.3	610
10.3	-	17.5	569
17.5	-	26.5	1311
26.5	-	37.7	893
37.7	-	51.7	895
51.7	-	69.2	1581
69.2	-	91.1	1629
91.1	-	100.0	1839

Line 3 - Vs 100 = 1062 ft/s			
Depth Range (feet)			Vs (ft/s)
0.0	-	3.8	731
3.8	-	8.7	617
8.7	-	14.7	670
14.7	-	22.2	1026
22.2	-	31.6	954
31.6	-	43.3	1019
43.3	-	58.0	1323
58.0	-	76.3	1393
76.3	-	99.2	1226
99.2	-	100.0	1564

Line 4 - Vs 100 = 1106 ft/s			
Depth Range (feet)			Vs (ft/s)
0.0	-	3.4	783
3.4	-	7.7	760
7.7	-	13.0	663
13.0	-	19.6	917
19.6	-	28.0	1084
28.0	-	38.4	1035
38.4	-	51.3	1086
51.3	-	67.6	1293
67.6	-	87.9	1346
87.9	-	100.0	1541

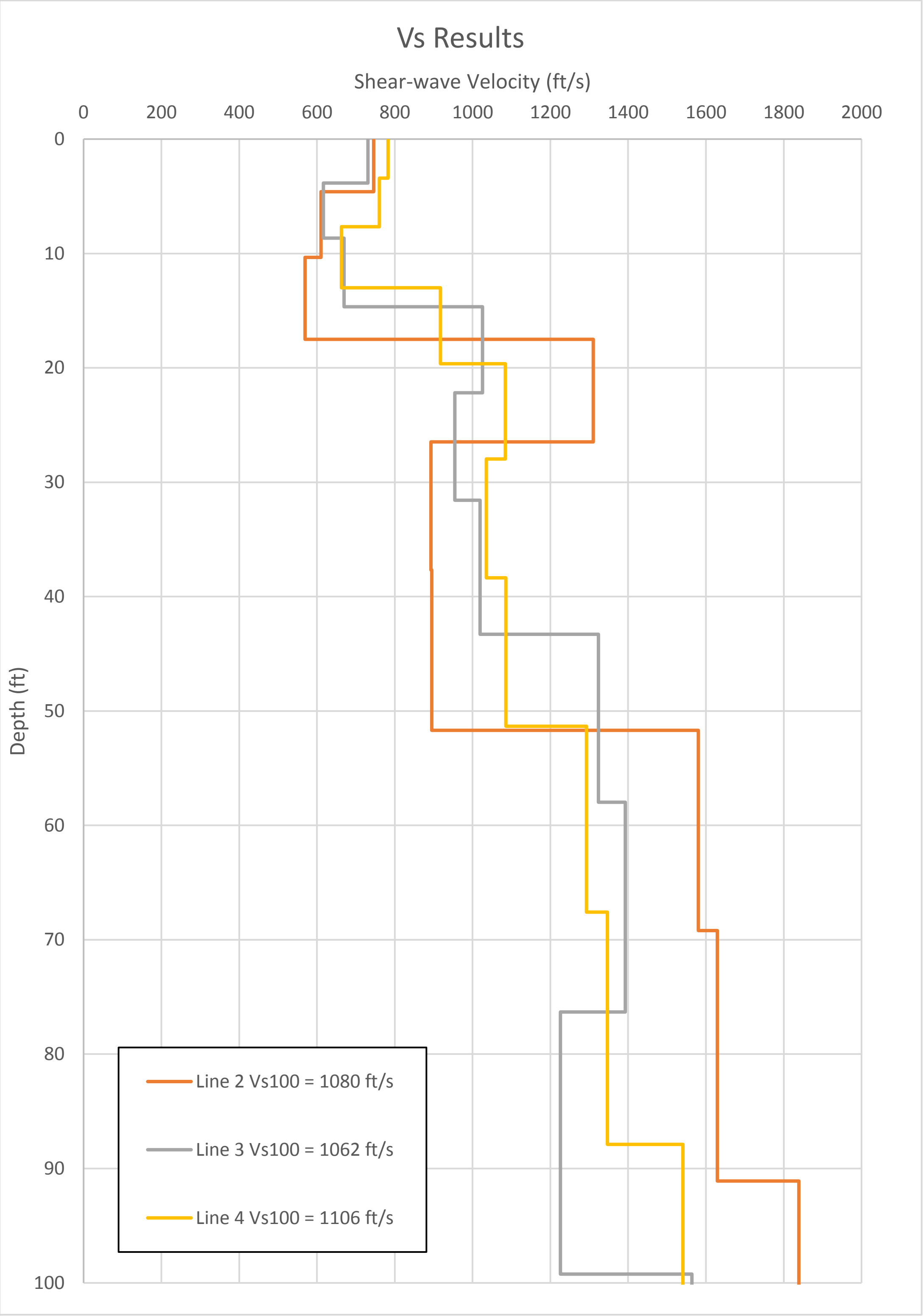


Figure 3. 1D Shear-wave velocity models for Lines 2 through 4.

Closure

The quality of the passive surface wave data was good for the three presented lines at this site. Based on the quality of the passive surface-wave data and the repeatability of the results, we have confidence that the 1D shear-wave velocity results and calculated Vs100 values are representative of the site conditions.

Olson Engineering does not assign a seismic site classification based on Vs measurements, because we are aware that other site factors may influence the classification. Site classification is an engineering judgment and decision; Olson is presenting Vs profiles and the resultant average shear-wave velocities in graphical and tabular format (computed according to IBC specifications) beneath each seismic line. Due caution and a conservative approach should be employed when evaluating site conditions as related to structural assessment and/or foundation design at any project site.

The geophysical methods and field procedures defined in this report were applicable to the project objectives and have been successfully applied by Olson to investigations of similar size and nature. However, sometimes field or subsurface conditions are different from those anticipated and the resultant data may not achieve the project objectives. Olson warrants that our services were performed within the limits prescribed for this project, with the usual thoroughness and competence of the geophysical profession. Olson conducted this project using the current standards of the geophysical industry and utilized in house quality control standards to produce a precise geophysical survey.

If you have any questions regarding the field procedures, seismic data analysis, or the Vs results presented herein, please do not hesitate to contact us. We appreciate working with you and look forward to providing Raba Kistner with geophysical or engineering services in the future.

Respectfully submitted,
Olson Engineering, Inc.

Miriam Moller
Staff Geophysicist

Nicole Pendrigh
Senior Geophysicist

(1 copy e-mailed PDF format)

APPENDIX B
NEHRP Seismic Provisions

Design Maps Detailed Report

2009 NEHRP Recommended Seismic Provisions (29.313°N, 98.316°W)

Site Class D – “Stiff Soil”, Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters and Risk Coefficients

Note: Ground motion values contoured on Figures 22-1, 2, 5, & 6 below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_{SUH} and S_{SD}) and 1.3 (to obtain S_{1UH} and S_{1D}). Maps in the Proposed 2015 NEHRP Provisions are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

Figure 22–1: Uniform–Hazard (2% in 50–Year) Ground Motions of 0.2-Second Spectral Response Acceleration (5% of Critical Damping), Site Class B

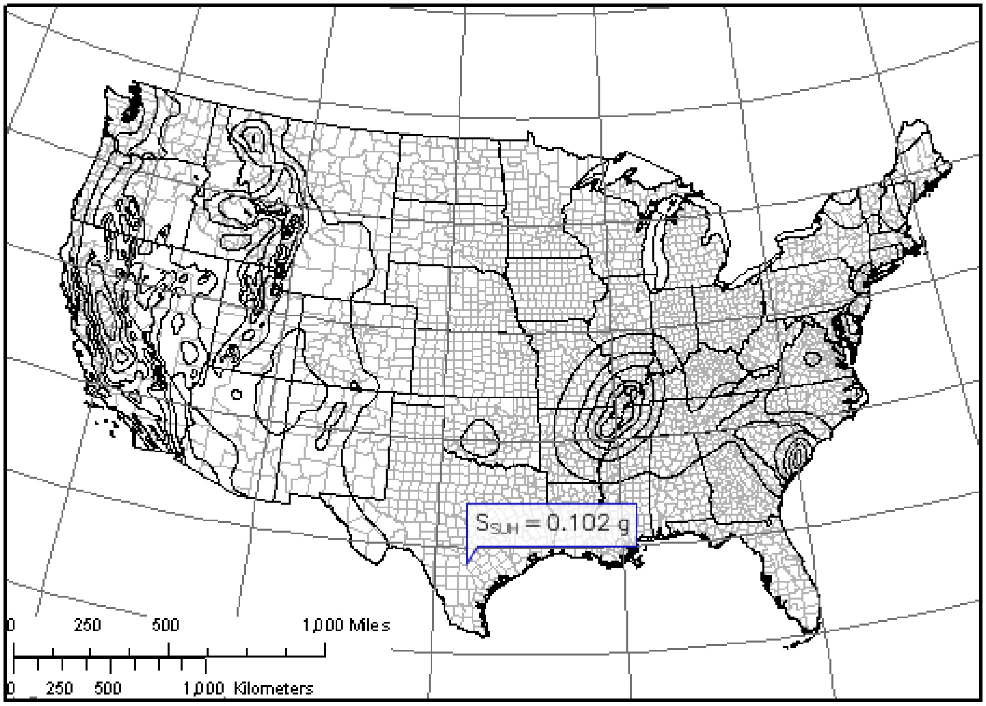


Figure 22–2: Uniform–Hazard (2% in 50–Year) Ground Motions of 1.0-Second Spectral Response Acceleration (5% of Critical Damping), Site Class B

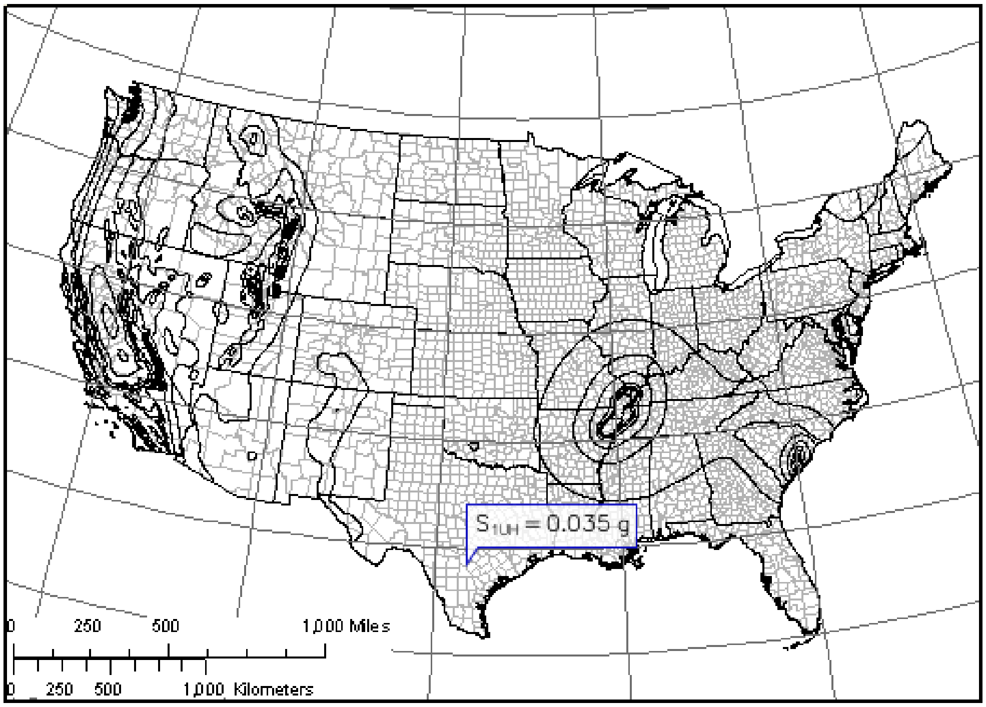

S_{1UH} = 0.035 g
0
250
500
1,000 Miles
0
250
500
1,000 Kilometers

Figure 22–3: Risk Coefficient at 0.2-Second Spectral Response Period

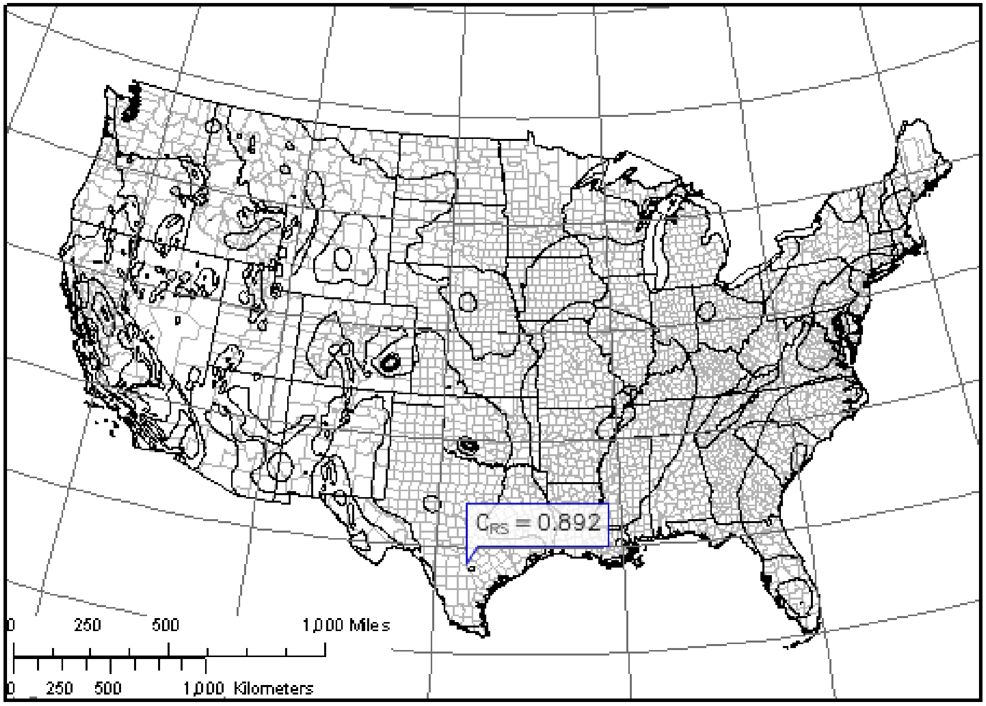


Figure 22–4: Risk Coefficient at 1.0-Second Spectral Response Period

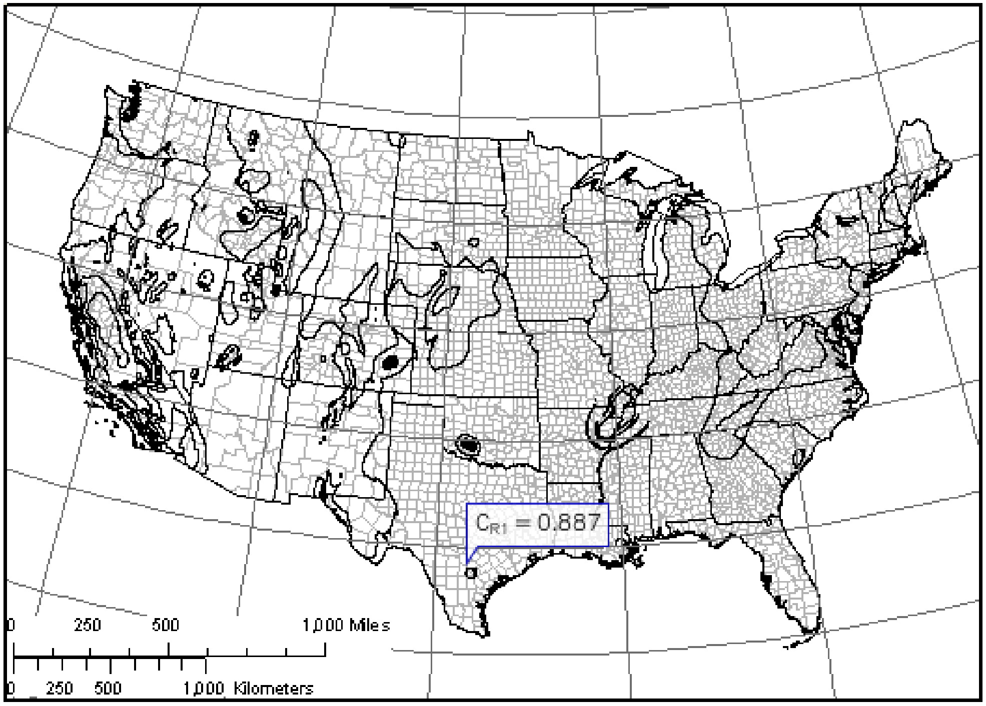


Figure 22–5: Deterministic Ground Motions of 0.2-Second Spectral Response Acceleration (5% of Critical Damping), Site Class B

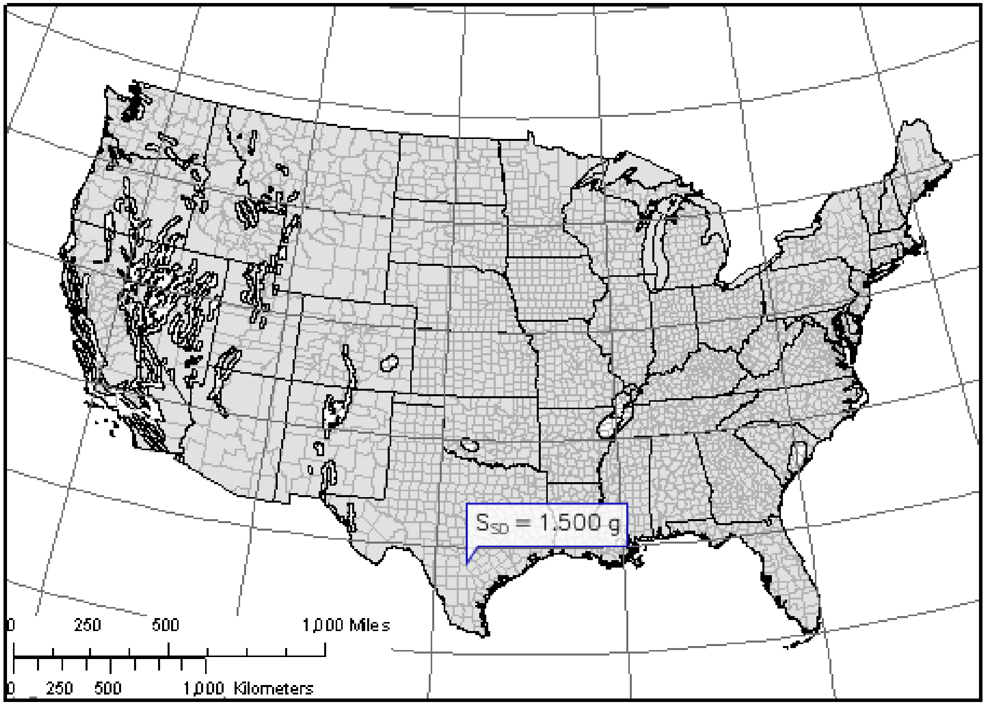
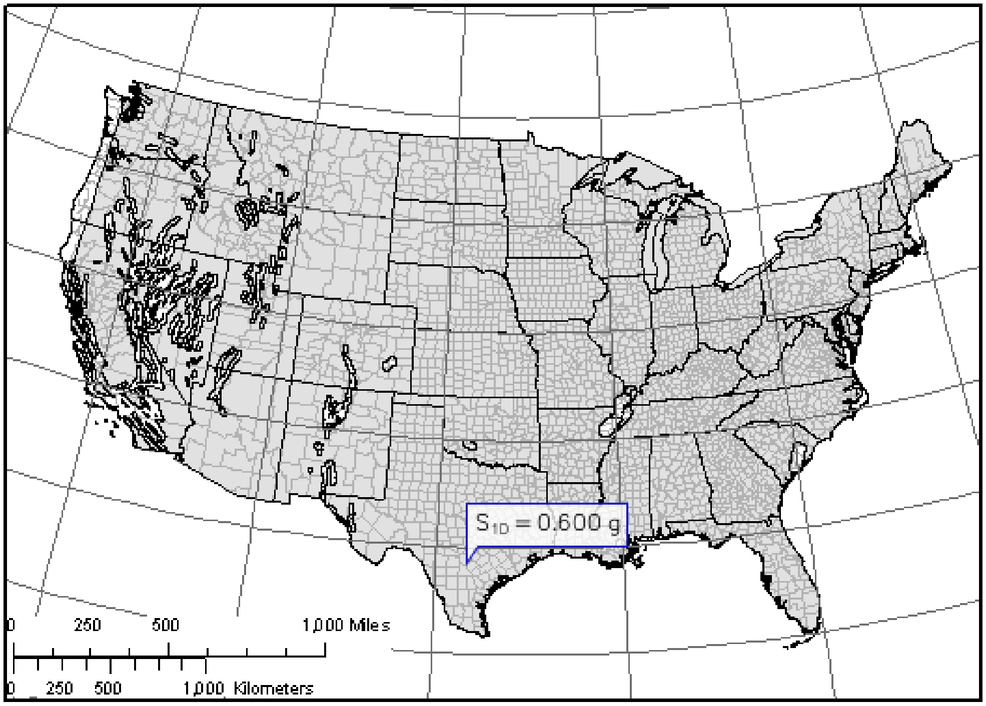


Figure 22–6: Deterministic Ground Motions of 1.0-Second Spectral Response Acceleration (5% of Critical Damping), Site Class B



Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index *PI* > 20, • Moisture content *w* ≥ 40%, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients, Risk Coefficients, and Risk–Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Equation (11.4–1): $C_{RS}S_{SUH}$ = 0.892 x 0.102 = 0.091 g

Equation (11.4–2): S_{SD} = 1.500 g

S_S ≡ “Lesser of values from Equations (11.4–1) and (11.4–2)” = 0.091 g

Equation (11.4–3): $C_{R1}S_{1UH}$ = 0.887 x 0.035 = 0.031 g

Equation (11.4–4): S_{1D} = 0.600 g

S_1 ≡ “Lesser of values from Equations (11.4–3) and (11.4–4)” = 0.031 g

Table 11.4–1: Site Coefficient F_a

Site Class	Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = D and S_S = 0.091 g, F_a = 1.600

Table 11.4–2: Site Coefficient F_v

Site Class	Spectral Response Acceleration Parameter at 1–Second Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.031 g, F_v = 2.400

Equation (11.4–5): $S_{MS} = F_aS_S = 1.600 \times 0.091 = 0.146$ g

---

Equation (11.4–6): $S_{M1} = F_vS_1 = 2.400 \times 0.031 = 0.075$ g

---

Section 11.4.4 — Design Spectral Acceleration Parameters

Equation (11.4–7): $S_{DS} = ⅔\ S_{MS} = ⅔ \times 0.146 = 0.097$ g

---

Equation (11.4–8): $S_{D1} = ⅔\ S_{M1} = ⅔ \times 0.075 = 0.050$ g

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Section 11.4.5 — Design Response Spectrum

Figure 22–7: Long–period Transition Period, T_L (s)

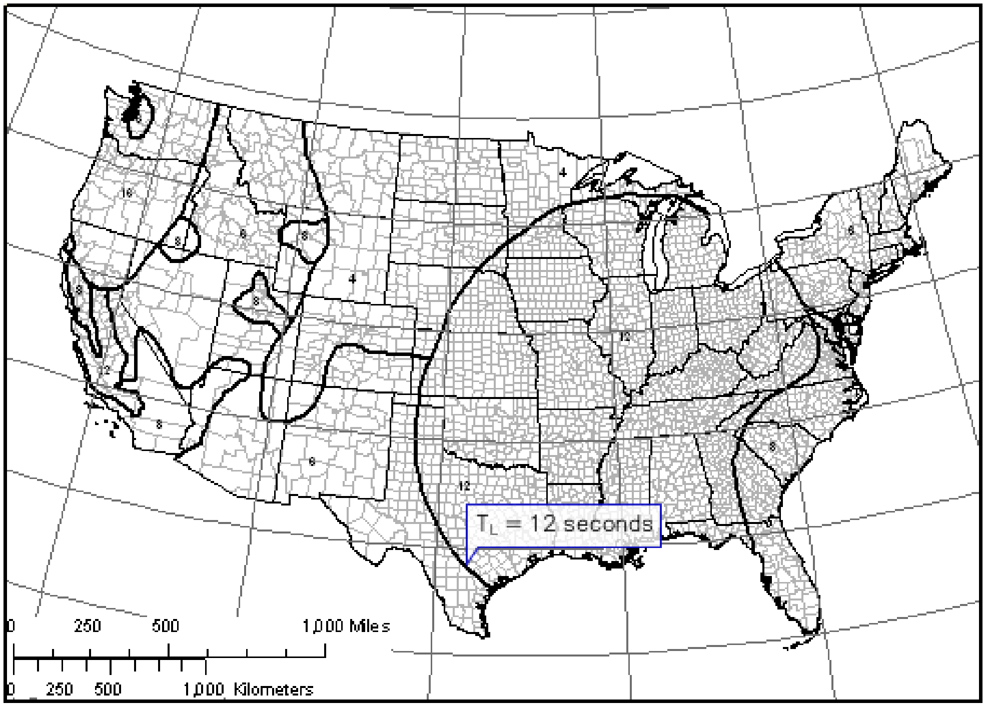
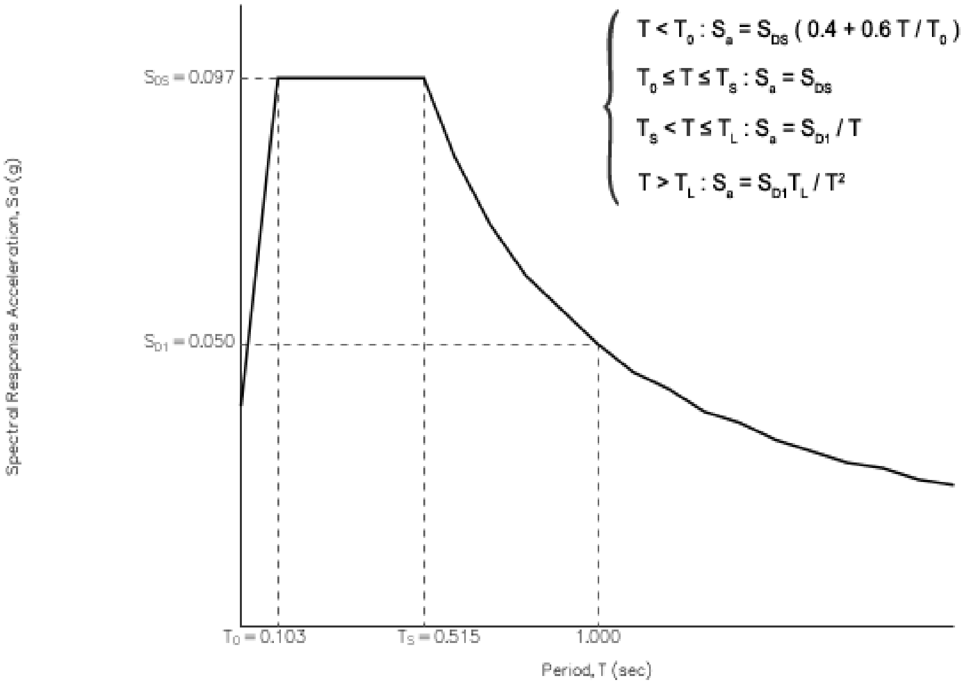
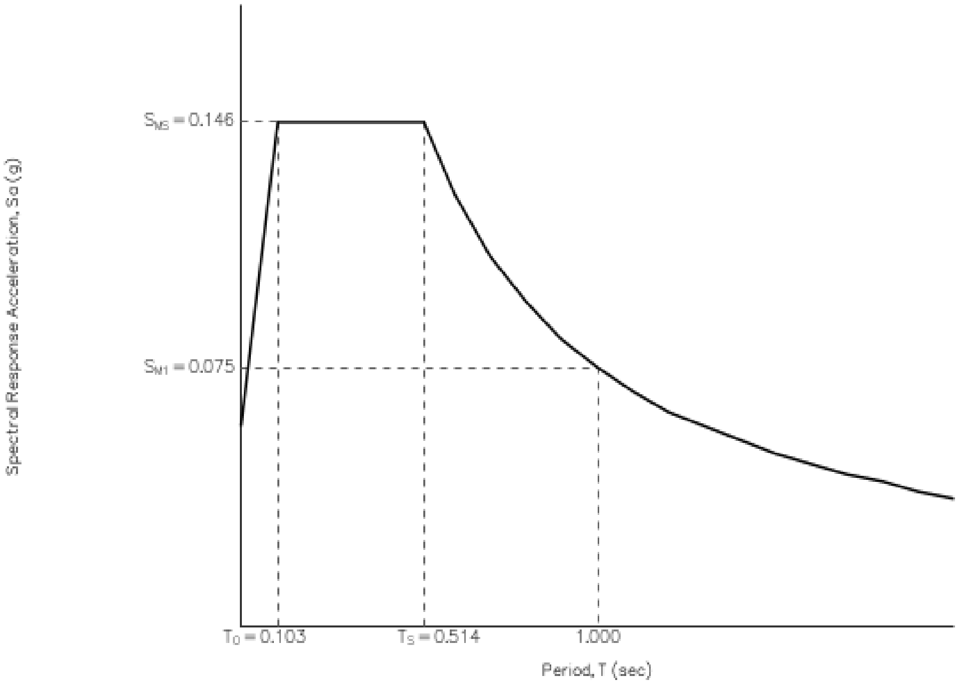


Figure 11.4–1: Design Response Spectrum



Section 11.4.6 — MCE_R Response Spectrum

The MCE_R response spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.047 g, F_{PGA} = 1.600

Mapped PGA PGA = 0.047 g

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.600 \times 0.047 = 0.075$ g

APPENDIX C
Liquefaction Analyses

Project Name: CPS CCR Ponds Boring No.: B-1

Job No.: ASA17-096-00

Total Depth: 50.0 ft Design Maximum Acceleration: 0.075 g

Water Level: 15 ft Design EQ Magnitude: 7.5

Depth (ft)	Thick-ness (ft)	Soil Type	Unit Weight (pcf)	Over-burden Stress (psf)	Pore Water Pressure (psf)	Effective Overburden Stress (psf)	γ_d (stress reduction coefficient)			Field SPT Value (N)	Correction Factor		SPT (Corrected)		Computed CSR for γ_d @			% of Fine	Expected CSR vs. N_{60}		Factor of Safety			Depth (ft)	EQ induced Volumetric Strain (%)			Post Liquefaction Settlement (in)		
							Upper Bound	Lower Bound	Average		CN	Er %	N_{70}	N_{60}	Upper Bound	Lower Bound	Average		@ given % Fine & M7.5	@ given % Fine & Magnitude	Upper Bound	Lower Bound	Average		Upper Bound	Lower Bound	Average	Upper Bound	Lower Bound	Average
0.0				0	0.0	0	1.000	1.000	1.000																					
3	3	Sand	115	345	0.0	345	0.998	0.992	0.995	8	2.408	87	24	28	Above GWT	Above GWT	Above GWT	28	Above GWT	Above GWT	>>1.00	>>1.00	>>1.00	3						
8	5	Clay	115	920	0.0	920	0.997	0.978	0.987	0	1.474	87	0	0	Above GWT	Above GWT	Above GWT	52	Above GWT	Above GWT	>>1.00	>>1.00	>>1.00	8						
10	2	Sand	120	1160	0.0	1160	0.996	0.972	0.984	16	1.313	87	26	30	Above GWT	Above GWT	Above GWT	36	Above GWT	Above GWT	>>1.00	>>1.00	>>1.00	10						
15	5	Sand	125	1785	0.0	1785	0.993	0.957	0.975	25	1.059	87	33	38	Above GWT	Above GWT	Above GWT	36	Above GWT	Above GWT	>>1.00	>>1.00	>>1.00	15						
20	5	Sand	125	2410	312.0	2098	0.989	0.938	0.963	14	0.976	87	17	20	0.055	0.053	0.054	36	0.394	0.520	9	10	10	20						
25	5	Sand	130	3060	624.0	2436	0.982	0.913	0.948	50	0.906	87	56	66	0.060	0.056	0.058	36	0.500	0.661	11	12	11	25						
30	5	Sand	130	3710	936.0	2774	0.971	0.880	0.926	50	0.849	87	53	61	0.063	0.057	0.060	36	0.500	0.661	10	12	11	30						
35	5	Sand	130	4360	1248.0	3112	0.957	0.834	0.895	50	0.802	87	50	58	0.065	0.057	0.061	48	0.500	0.661	10	12	11	35						
40	5	Sand	130	5010	1560.0	3450	0.939	0.773	0.856	35	0.761	87	33	39	0.066	0.055	0.061	48	0.500	0.661	10	12	11	40						
45	5	Sand	130	5660	1872.0	3788	0.919	0.699	0.809	37	0.727	87	33	39	0.067	0.051	0.059	48	0.500	0.661	10	13	11	45						
50	5	Sand	130	6310	2184.0	4126	0.897	0.618	0.758	50	0.696	87	43	50	0.067	0.046	0.056	48	0.500	0.661	10	14	12	50						
																												Total (in)		
																												0.00	0.00	0.00

Cyclic Ratio $= 0.65 \times \frac{a_{max}}{g} \times \frac{\sigma_o}{\overline{\sigma}_o}$ *(Seed & Idriss, 1982)*

Where

γ_d: *Stress Reduction Coefficient (Fig. 40, " Ground Motions and Soil Liquefaction During Earthquakes" , Seed & Idriss, 1982)*

$N_{corrected} = N_{field} \times C_N \times \frac{E_r}{60\%} \times \eta_2 \times \eta_3 \times \eta_4$ *(Bowles, " Foundation Analysis and Design" , 4th Edition)*

Where

$C_N = 1/\sqrt{\overline{\sigma}_0}$ $\overline{\sigma}_0$ *in tsf*

E_r: *% of Input Energy*

η_2: *Rod Length Correction*

η_3: *Sampler Correction*

η_4: *Borehole Diameter Correction*

Assumed : $\eta_2, \eta_3, \eta_4 = 1$

Post - Liquefaction Settlement

$S = \varepsilon_v \times H$ *(FHWA - SA - 97 - 076, ch8)*

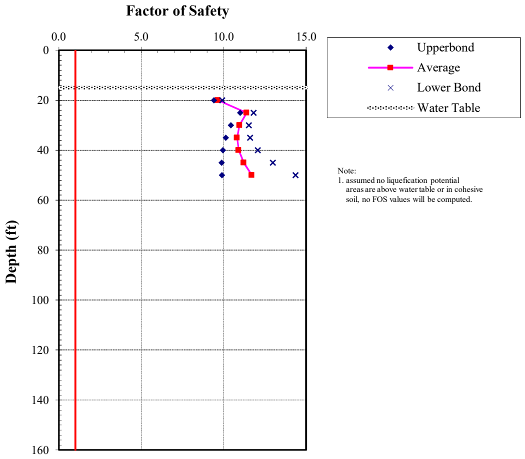
Where

ε_v: *Volumetic Strain for Different EQ Magnitude, (%) (Tokimatsu & Seed, 1987)*

H: *Thickness of Liquefiable Layer*

Ratios between M7.5 & Different EQ Magnitude (Seed & Idriss,1982)

M8.5	*0.89*
M7.5	*1.00*
M6.75	*1.13*
M6.0	*1.32*
M5.25	*1.50*



Note:
1. assumed no liquefaction potential areas are above water table or in cohesive soil, no FOS values will be computed.

Project Name: CPS CCR Ponds
Boring No.: B-2
Job No.: ASA17-096-00
Total Depth: 50.0 ft
Water Level: 15 ft
Design Maximum Acceleration: 0.075 g
Design EQ Magnitude: 7.5

Depth (ft)	Thickness (ft)	Soil Type	Unit Weight (pcf)	Over-burden Stress (psf)	Pore Water Pressure (psf)	Effective Overburden Stress (psf)	γ_d (stress reduction coefficient)			Field SPT Value (N)	Correction Factor		SPT (Corrected)		Computed CSR for γ_d @			% of Fine	Expected CSR vs. N_{60}		Factor of Safety			Depth (ft)	EQ induced Volumetric Strain (%)			Post Liquefaction Settlement (in)		
							Upper Bound	Lower Bound	Average		CN	Er %	N_{70}	N_{60}	Upper Bound	Lower Bound	Average		@ given % Fine & M7.5	@ given % Fine & Magnitude	Upper Bound	Lower Bound	Average		Upper Bound	Lower Bound	Average	Upper Bound	Lower Bound	Average
0.0				0	0.0	0	1.000	1.000	1.000																					
3	3	Sand	115	345	0.0	345	0.998	0.992	0.995	7	2.408	87	21	24	Above GWT	Above GWT	Above GWT	28	Above GWT	Above GWT	>>1.00	>>1.00	>>1.00	3						
8	5	Clay	115	920	0.0	920	0.997	0.978	0.987	0	1.474	87	0	0	Above GWT	Above GWT	Above GWT	52	Above GWT	Above GWT	>>1.00	>>1.00	>>1.00	8						
10	2	Sand	120	1160	0.0	1160	0.996	0.972	0.984	15	1.313	87	24	29	Above GWT	Above GWT	Above GWT	27	Above GWT	Above GWT	>>1.00	>>1.00	>>1.00	10						
15	5	Sand	125	1785	0.0	1785	0.993	0.957	0.975	28	1.059	87	37	43	Above GWT	Above GWT	Above GWT	27	Above GWT	Above GWT	>>1.00	>>1.00	>>1.00	15						
20	5	Sand	125	2410	312.0	2098	0.989	0.938	0.963	44	0.976	87	53	62	0.055	0.053	0.054	27	0.526	0.695	13	13	13	20						
25	5	Sand	130	3060	624.0	2436	0.982	0.913	0.948	28	0.906	87	31	37	0.060	0.056	0.058	27	0.526	0.695	12	12	12	25						
35	10	Sand	130	4360	1248.0	3112	0.957	0.834	0.895	40	0.802	87	40	46	0.065	0.057	0.061	27	0.526	0.695	11	12	11	35						
40	5	Sand	130	5010	1560.0	3450	0.939	0.773	0.856	50	0.761	87	47	55	0.066	0.055	0.061	27	0.526	0.695	10	13	11	40						
45	5	Sand	130	5660	1872.0	3788	0.919	0.699	0.809	50	0.727	87	45	53	0.067	0.051	0.059	27	0.526	0.695	10	14	12	45						
50	5	Sand	130	6310	2184.0	4126	0.897	0.618	0.758	26	0.696	87	22	26	0.067	0.046	0.056	27	0.526	0.695	10	15	12	50						
																														Total (in)		
																												0.00	0.00	0.00

Cyclic Ratio = $0.65 \times \frac{a_{max}}{g} \times \frac{\sigma_o}{\overline{\sigma}_o}$ *(Seed & Idriss, 1982)*

Where

γ_d: *Stress Reduction Coefficient (Fig. 40, " Ground Motions and Soil Liquefaction During Earthquakes" , Seed & Idriss, 1982)*

$N_{corrected} = N_{field} \times C_N \times \frac{E_r}{60\%} \times \eta_2 \times \eta_3 \times \eta_4$ *(Bowles, " Foundation Analysis and Design" , 4th Edition)*

Where

$C_N = 1 / \sqrt{\overline{\sigma}_0}$ $\quad \overline{\sigma}_0$ *in tsf*

E_r: *% of Input Energy*

η_2: *Rod Length Correction*

η_3: *Sampler Correction*

η_4: *Borehole Diameter Correction*

Assumed : $\eta_2, \eta_3, \eta_4 = 1$

Post - Liquefaction Settlement

$S = \varepsilon_v \times H$ *(FHWA - SA - 97 - 076, ch8)*

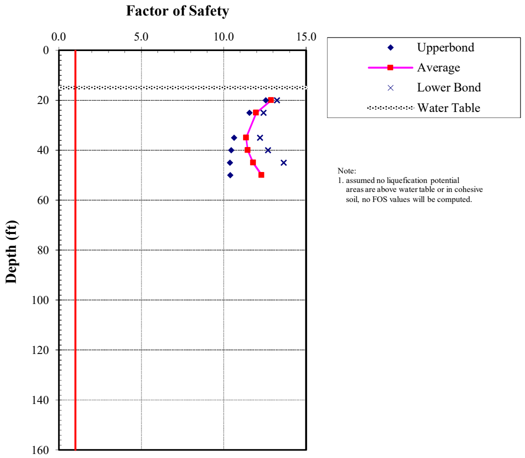
Where

ε_v: *Volumetic Strain for Different EQ Magnitude, (%) (Tokimatsu & Seed, 1987)*

H: Thickness of Liquefiable Layer

Ratios between M7.5 & Different EQ Magnitude (Seed & Idriss,1982)

M8.5	0.89
M7.5	1.00
M6.75	1.13
M6.0	1.32
M5.25	1.50



Note:
1. assumed no liquefication potential areas are above water table or in cohesive soil, no FOS values will be computed.

Project Name: CPS CCR Ponds Boring No.: B-3
Job No.: ASA17-096-00
Total Depth: 50.0 ft Design Maximum Acceleration: 0.075 g
Water Level: 22 ft Design EQ Magnitude: 7.5

Depth (ft)	Thickness (ft)	Soil Type	Unit Weight (pcf)	Over-burden Stress (psf)	Pore Water Pressure (psf)	Effective Overburden Stress (psf)	γ_d (stress reduction coefficient)			Field SPT Value (N)	Correction Factor		SPT (Corrected)		Computed CSR for γ_d @			% of Fine	Expected CSR vs. N_{60}		Factor of Safety			Depth (ft)	EQ induced Volumetric Strain (%)			Post Liquefaction Settlement (in)		
							Upper Bound	Lower Bound	Average		CN	Er %	N_{70}	N_{60}	Upper Bound	Lower Bound	Average		@ given % Fine & M7.5	@ given % Fine & Magnitude	Upper Bound	Lower Bound	Average		Upper Bound	Lower Bound	Average	Upper Bound	Lower Bound	Average
0.0				0	0.0	0	1.000	1.000	1.000																					
9	9	Sand	115	1035	0.0	1035	0.996	0.975	0.986	16	1.390	87	28	32	Above GWT	Above GWT	Above GWT	28	Above GWT	Above GWT	>>1.00	>>1.00	>>1.00	9						
11	2	Clay	115	1265	0.0	1265	0.996	0.969	0.982	0	1.257	87	0	0	Above GWT	Above GWT	Above GWT	52	Above GWT	Above GWT	>>1.00	>>1.00	>>1.00	11						
15	4	Sand	120	1745	0.0	1745	0.993	0.957	0.975	28	1.071	87	37	43	Above GWT	Above GWT	Above GWT	27	Above GWT	Above GWT	>>1.00	>>1.00	>>1.00	15						
20	5	Sand	125	2370	0.0	2370	0.989	0.938	0.963	12	0.919	87	14	16	Above GWT	Above GWT	Above GWT	27	Above GWT	Above GWT	>>1.00	>>1.00	>>1.00	20						
25	5	Sand	125	2995	187.2	2808	0.982	0.913	0.948	19	0.844	87	20	23	0.051	0.047	0.049	27	0.400	0.528	10	11	11	25						
30	5	Sand	130	3645	499.2	3146	0.971	0.880	0.926	50	0.797	87	49	58	0.055	0.050	0.052	27	0.516	0.682	12	14	13	30						
35	5	Sand	130	4295	811.2	3484	0.957	0.834	0.895	50	0.758	87	47	55	0.058	0.050	0.054	27	0.516	0.682	12	14	13	35						
40	5	Sand	130	4945	1123.2	3822	0.939	0.773	0.856	36	0.723	87	32	38	0.059	0.049	0.054	27	0.516	0.682	12	14	13	40						
45	5	Sand	130	5595	1435.2	4160	0.919	0.699	0.809	50	0.693	87	43	50	0.060	0.046	0.053	27	0.516	0.682	11	15	13	45						
50	5	Sand	130	6245	1747.2	4498	0.897	0.618	0.758	44	0.667	87	36	42	0.061	0.042	0.051	27	0.516	0.682	11	16	13	50						
																												Total (in)		
																												0.00	0.00	0.00

Cyclic Ratio $= 0.65 \times \frac{a_{max}}{g} \times \frac{\sigma_o}{\bar{\sigma}_o}$ *(Seed & Idriss, 1982)*

Where

γ_d: *Stress Reduction Coefficient (Fig. 40, " Ground Motions and Soil Liquefaction During Earthquakes" , Seed & Idriss, 1982)*

$N_{corrected} = N_{field} \times C_N \times \frac{E_r}{60\%} \times \eta_2 \times \eta_3 \times \eta_4$ *(Bowles, " Foundation Analysis and Design" , 4th Edition)*

Where

$C_N = 1 / \sqrt{\bar{\sigma}_0}$ $\quad \bar{\sigma}_o$ *in tsf*

E_r: *% of Input Energy*

η_2: *Rod Length Correction*

η_3: *Sampler Correction*

η_4: *Borehole Diameter Correction*

Assumed : $\eta_2, \eta_3, \eta_4 = 1$

Post - Liquefaction Settlement

$S = \varepsilon_v \times H$ *(FHWA - SA - 97 - 076, ch8)*

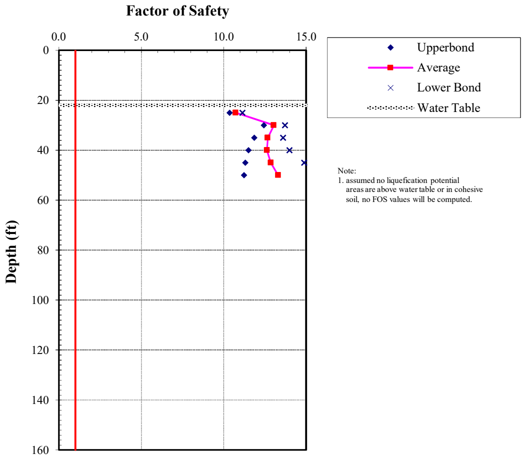
Where

ε_v: *Volumetic Strain for Different EQ Magnitude, (%) (Tokimatsu & Seed, 1987)*

H: Thickness of Liquefiable Layer

Ratios between M7.5 & Different EQ Magnitude (Seed & Idriss,1982)

M8.5	*0.89*
M7.5	*1.00*
M6.75	*1.13*
M6.0	*1.32*
M5.25	*1.50*



Note:
1. assumed no liquefication potential areas are above water table or in cohesive soil, no FOS values will be computed.

Project Name: CPS CCR Ponds
Boring No.: B-4
Job No.: ASA17-096-00
Total Depth: 50.0 ft
Design Maximum Acceleration: 0.075 g
Water Level: 19 ft
Design EQ Magnitude: 7.5

Depth (ft)	Thickness (ft)	Soil Type	Unit Weight (pcf)	Over-burden Stress (psf)	Pore Water Pressure (psf)	Effective Overburden Stress (psf)	γ_d (stress reduction coefficient)			Field SPT Value (N)	Correction Factor		SPT (Corrected)		Computed CSR for γ_d @			% of Fine	Expected CSR vs. N_{60}		Factor of Safety			Depth (ft)	EQ induced Volumetric Strain (%)			Post Liquefaction Settlement (in)		
							Upper Bound	Lower Bound	Average		CN	Er %	N_{70}	N_{60}	Upper Bound	Lower Bound	Average		@ given % Fine & M7.5	@ given % Fine & Magnitude	Upper Bound	Lower Bound	Average		Upper Bound	Lower Bound	Average	Upper Bound	Lower Bound	Average
0.0				0	0.0	0	1.000	1.000	1.000																					
2	2	Sand	115	230	0.0	230	0.998	0.995	0.996	11	2.949	87	40	47	Above GWT	Above GWT	Above GWT	23	Above GWT	Above GWT	>>1.00	>>1.00	>>1.00	2						
4	2	Clay	115	460	0.0	460	0.998	0.989	0.993	0	2.085	87	0	0	Above GWT	Above GWT	Above GWT	76	Above GWT	Above GWT	>>1.00	>>1.00	>>1.00	4						
10	6	Sand	120	1180	0.0	1180	0.996	0.972	0.984	34	1.302	87	55	64	Above GWT	Above GWT	Above GWT	27	Above GWT	Above GWT	>>1.00	>>1.00	>>1.00	10						
15	5	Sand	125	1805	0.0	1805	0.993	0.957	0.975	39	1.053	87	51	59	Above GWT	Above GWT	Above GWT	27	Above GWT	Above GWT	>>1.00	>>1.00	>>1.00	15						
20	5	Sand	125	2430	62.4	2368	0.989	0.938	0.963	50	0.919	87	57	67	0.049	0.047	0.048	27	0.516	0.682	14	15	14	20						
25	5	Sand	130	3080	374.4	2706	0.982	0.913	0.948	50	0.860	87	53	62	0.054	0.051	0.053	27	0.516	0.682	13	13	13	25						
30	5	Sand	130	3730	686.4	3044	0.971	0.880	0.926	50	0.811	87	50	59	%f>50	%f>50	%f>50	50	%f>50	%f>50	>>1.00	>>1.00	>>1.00	30						
35	5	Sand	130	4380	998.4	3382	0.957	0.834	0.895	50	0.769	87	48	56	0.060	0.053	0.057	27	0.516	0.682	11	13	12	35						
40	5	Sand	130	5030	1310.4	3720	0.939	0.773	0.856	37	0.733	87	34	39	0.062	0.051	0.056	27	0.516	0.682	11	13	12	40						
45	5	Sand	130	5680	1622.4	4058	0.919	0.699	0.809	50	0.702	87	44	51	0.063	0.048	0.055	27	0.516	0.682	11	14	12	45						
50	5	Sand	130	6330	1934.4	4396	0.897	0.618	0.758	50	0.675	87	42	49	0.063	0.043	0.053	27	0.516	0.682	11	16	13	50						
																													Total (in)		
																												0.00	0.00	0.00

Cyclic Ratio $= 0.65 \times \frac{a_{max}}{g} \times \frac{\sigma_o}{\overline{\sigma}_o}$ *(Seed & Idriss, 1982)*

Where

γ_d: *Stress Reduction Coefficient (Fig. 40, " Ground Motions and Soil Liquefaction During Earthquakes" , Seed & Idriss, 1982)*

$N_{corrected} = N_{field} \times C_N \times \frac{E_r}{60\%} \times \eta_2 \times \eta_3 \times \eta_4$ *(Bowles, " Foundation Analysis and Design" , 4th Edition)*

Where

$C_N = 1 / \sqrt{\overline{\sigma}_0}$ $\overline{\sigma}_0$ *in tsf*

E_r: *% of Input Energy*

η_2: *Rod Length Correction*

η_3: *Sampler Correction*

η_4: *Borehole Diameter Correction*

Assumed : $\eta_2, \eta_3, \eta_4 = 1$

Post - Liquefaction Settlement

$S = \varepsilon_v \times H$ *(FHWA - SA - 97 - 076, ch8)*

Where

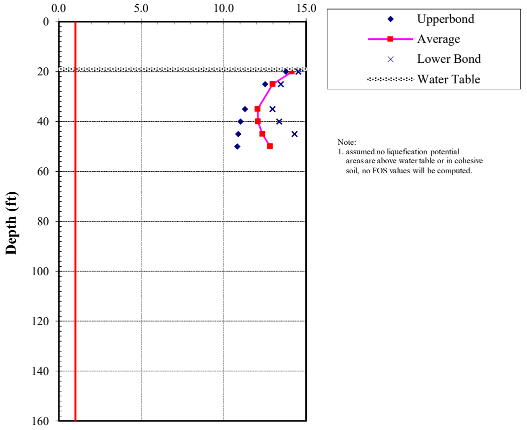
ε_v: *Volumetric Strain for Different EQ Magnitude, (%) (Tokimatsu & Seed, 1987)*

H: Thickness of Liquefiable Layer

Ratios between M7.5 & Different EQ Magnitude (Seed & Idriss,1982)

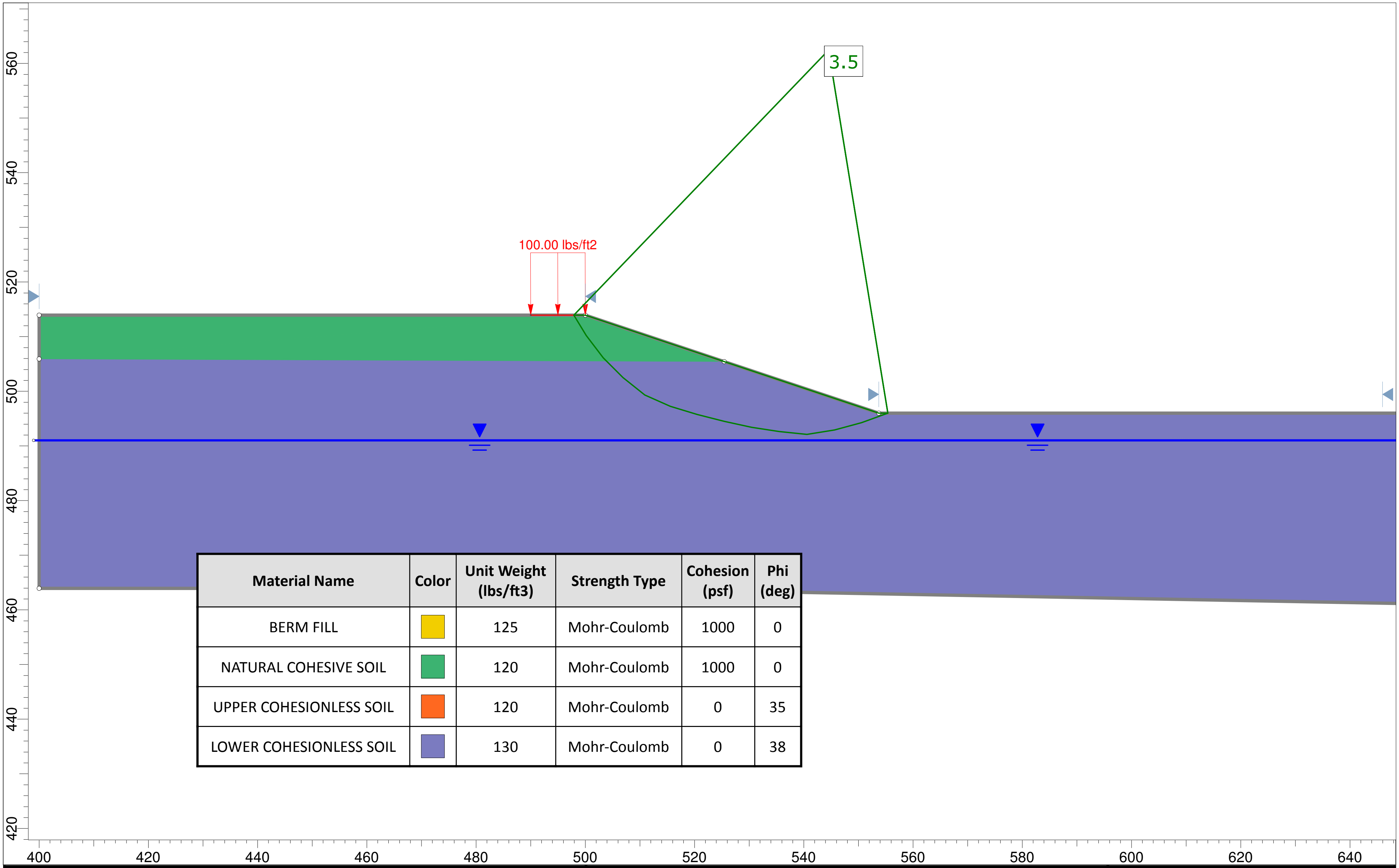
M8.5	*0.89*
M7.5	*1.00*
M6.75	*1.13*
M6.0	*1.32*
M5.25	*1.50*

Factor of Safety



Note:
1. assumed no liquefication potential areas are above water table or in cohesive soil, no FOS values will be computed.

APPENDIX D
Slope Stability Analysis



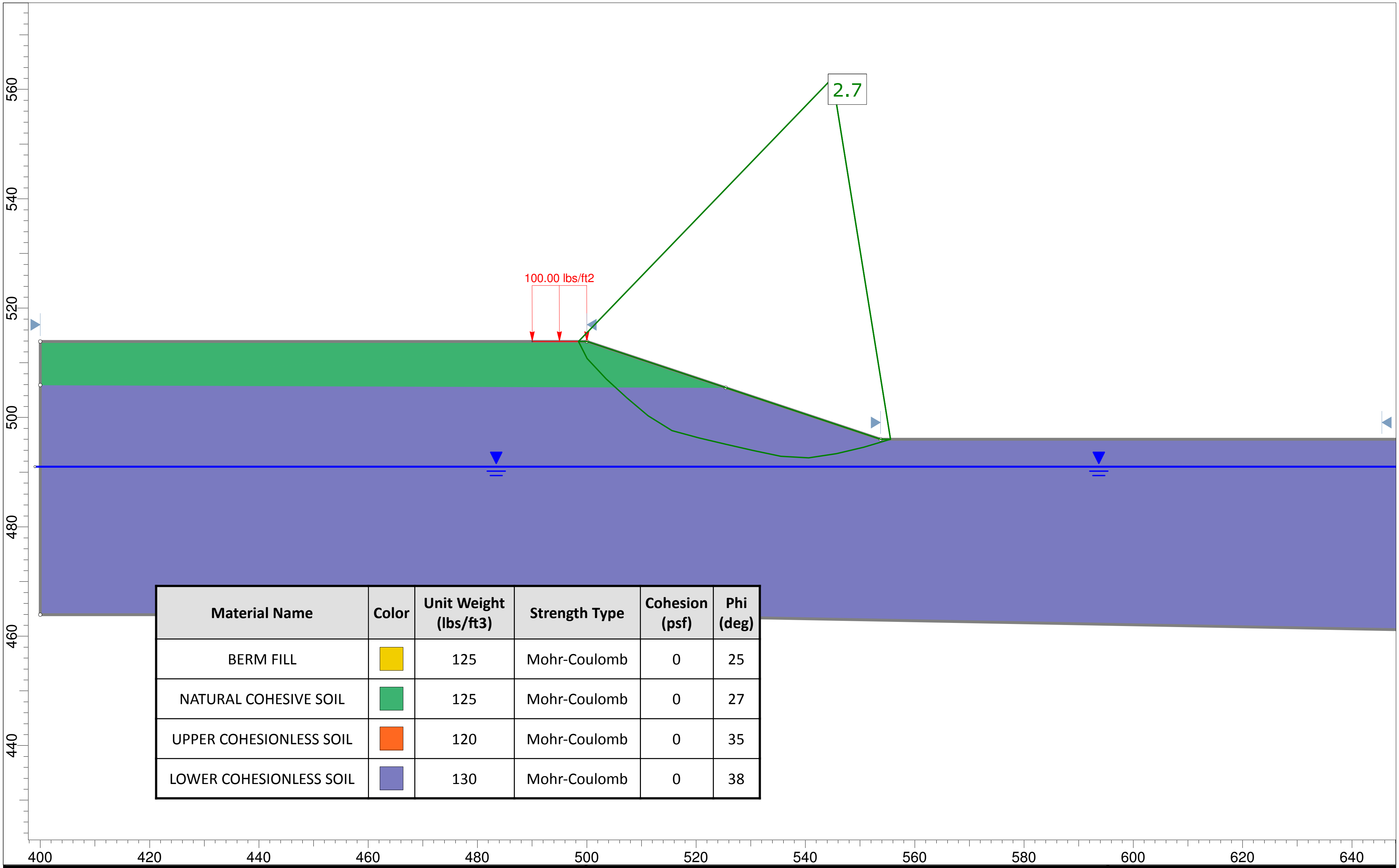
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
BERM FILL	🟨	125	Mohr-Coulomb	1000	0
NATURAL COHESIVE SOIL	🟩	120	Mohr-Coulomb	1000	0
UPPER COHESIONLESS SOIL	🟧	120	Mohr-Coulomb	0	35
LOWER COHESIONLESS SOIL	🟪	130	Mohr-Coulomb	0	38

ASA17-096-00
J.K. SPRUCE –CALAVERAS LAKE POWER PLANT
PROPOSED NEW COAL COMBUSTION RESIDUAL PONDS
SAN ANTONIO, TEXAS

GLOBAL STABILITY ANALYSIS
SECTION A-A
END OF CONSTRUCTION (SHORT TERM)

FIGURE A-1

RABA KISTNER CONSULTANTS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
BERM FILL	🟨	125	Mohr-Coulomb	0	25
NATURAL COHESIVE SOIL	🟩	125	Mohr-Coulomb	0	27
UPPER COHESIONLESS SOIL	🟧	120	Mohr-Coulomb	0	35
LOWER COHESIONLESS SOIL	🟪	130	Mohr-Coulomb	0	38

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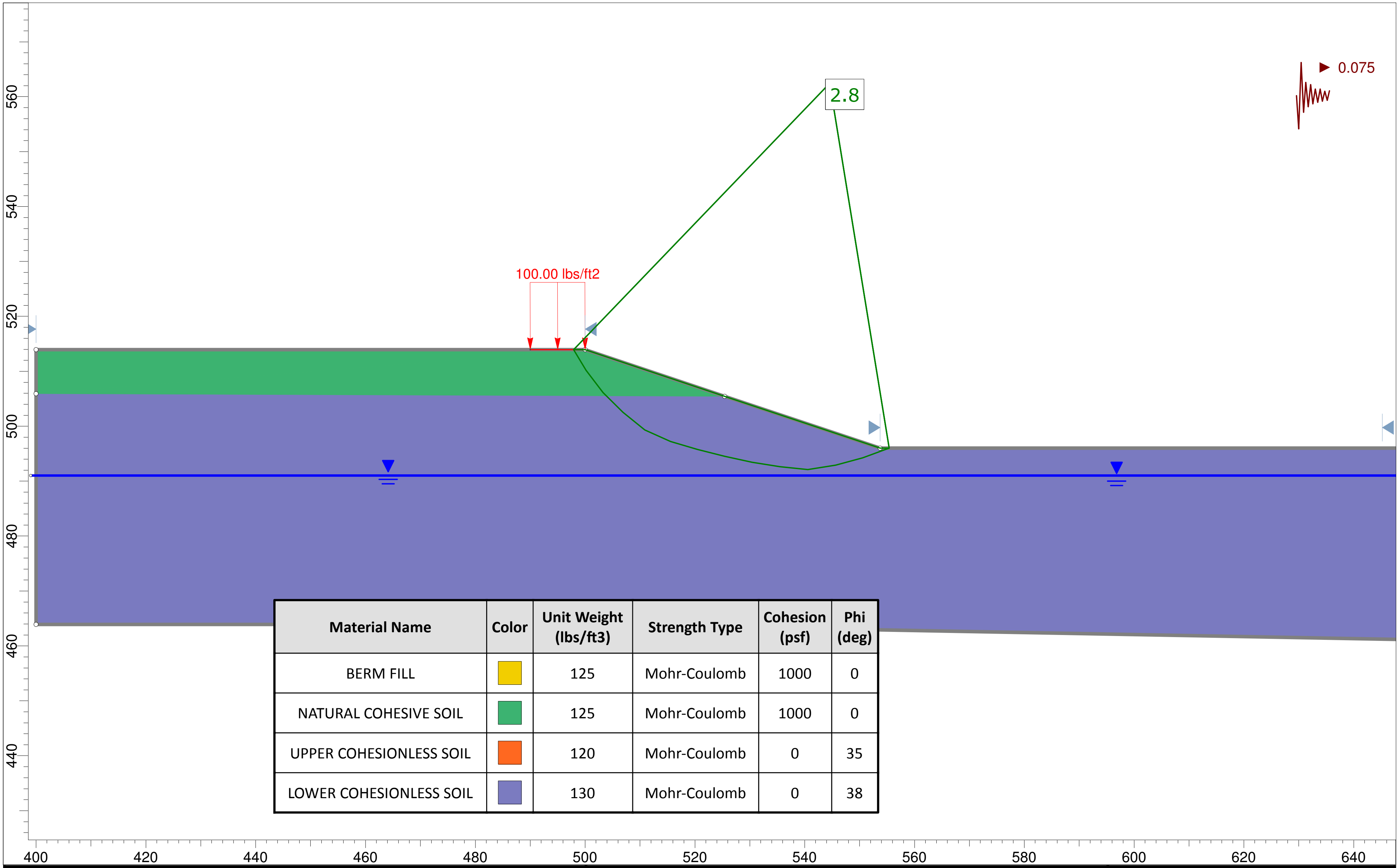
PROPOSED NEW COAL COMBUSTION RESIDUAL PONDS
SAN ANTONIO, TEXAS

GLOBAL STABILITY ANALYSIS
SECTION A-A
LONG TERM

FIGURE A-2

RABA KISTNER
CONSULTANTS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
BERM FILL	🟨	125	Mohr-Coulomb	1000	0
NATURAL COHESIVE SOIL	🟩	125	Mohr-Coulomb	1000	0
UPPER COHESIONLESS SOIL	🟧	120	Mohr-Coulomb	0	35
LOWER COHESIONLESS SOIL	🟪	130	Mohr-Coulomb	0	38

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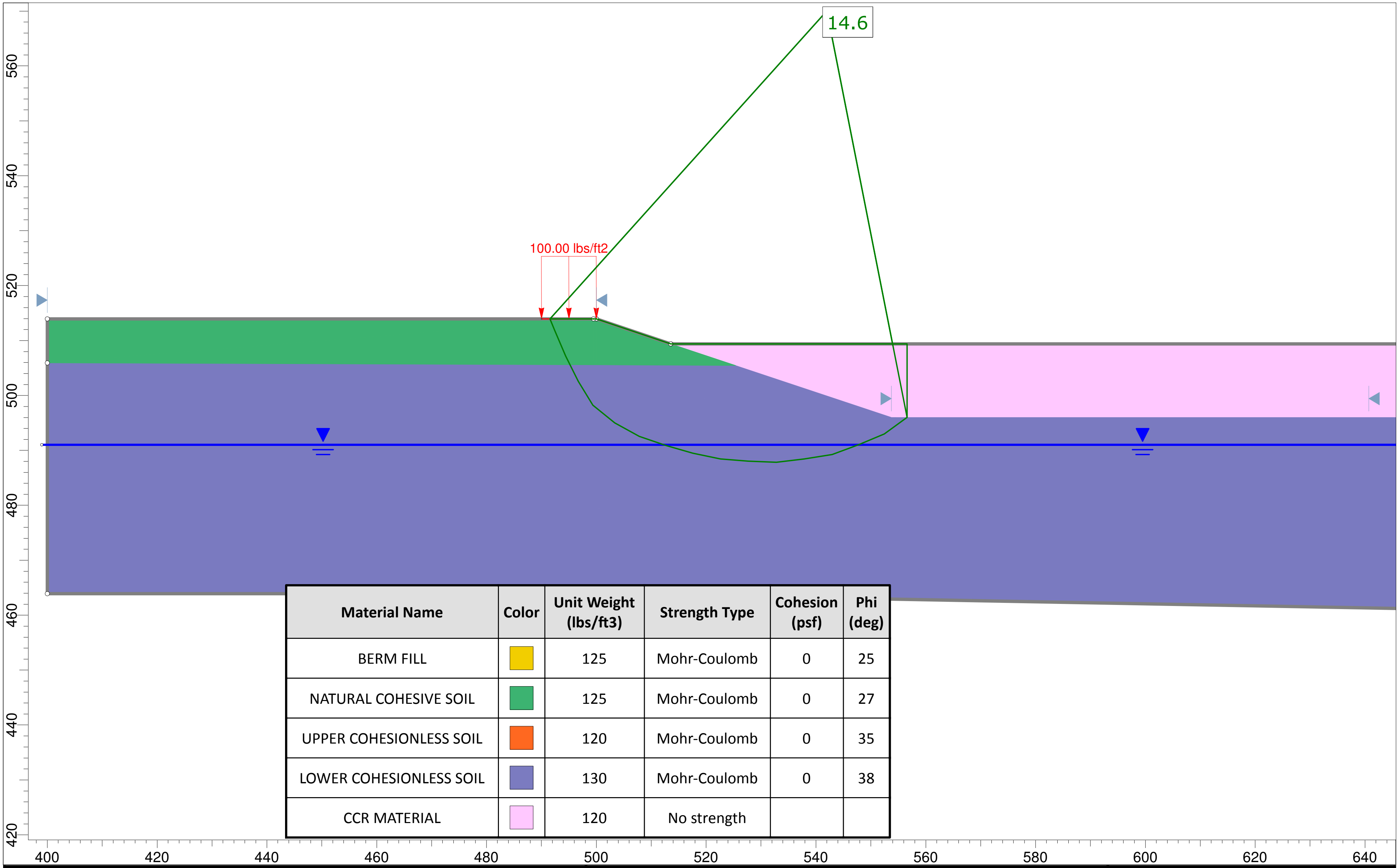
PROPOSED NEW COAL COMBUSTION RESIDUAL PONDS
SAN ANTONIO, TEXAS

GLOBAL STABILITY ANALYSIS
SECTION A-A
SEISMIC CONDITION

FIGURE A-3

RABA KISTNER
CONSULTANTS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
BERM FILL	🟨	125	Mohr-Coulomb	0	25
NATURAL COHESIVE SOIL	🟩	125	Mohr-Coulomb	0	27
UPPER COHESIONLESS SOIL	🟧	120	Mohr-Coulomb	0	35
LOWER COHESIONLESS SOIL	🟦	130	Mohr-Coulomb	0	38
CCR MATERIAL	🟪	120	No strength		

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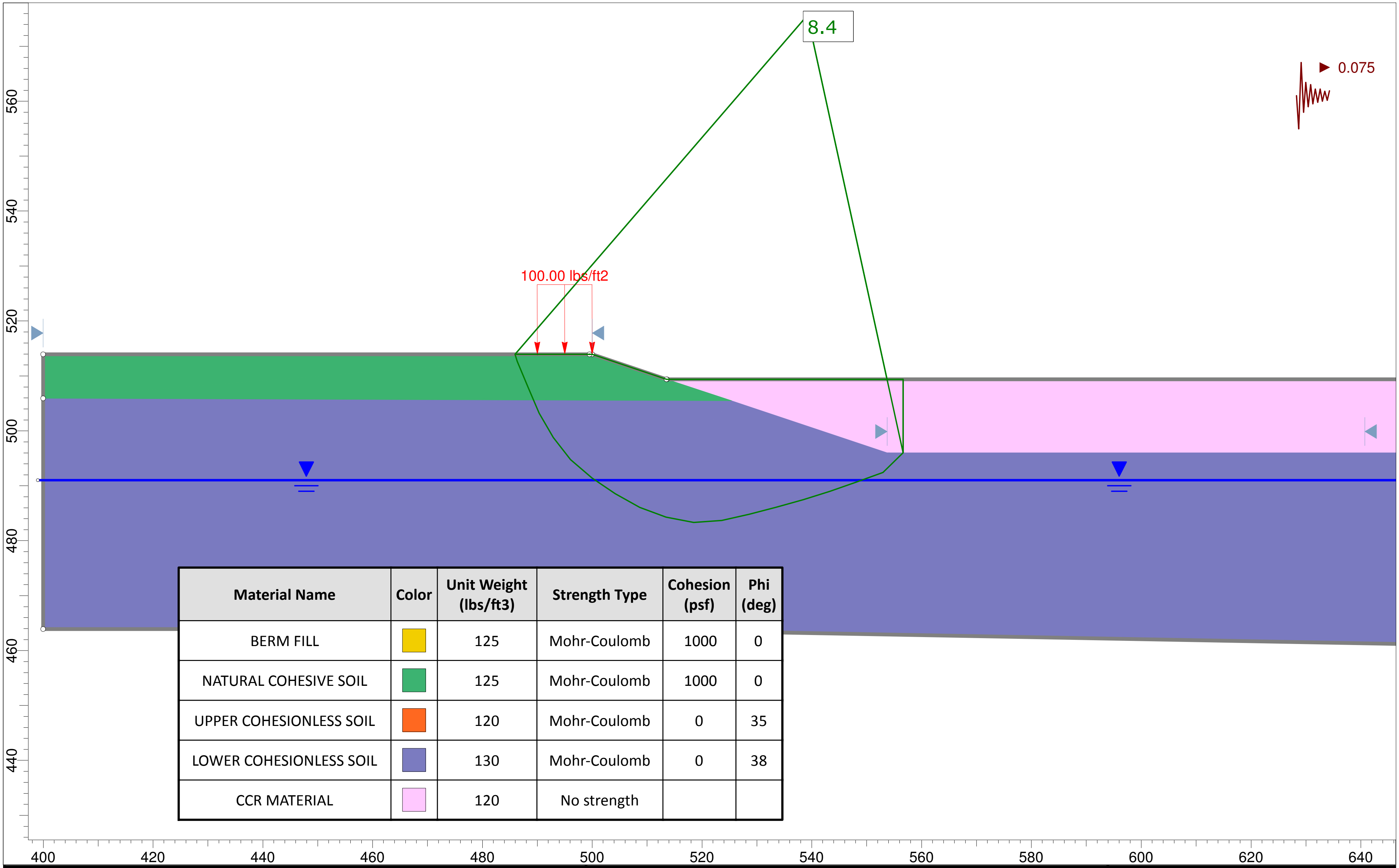
J.K. SPRUCE –CALAVERAS LAKE POWER PLANT
PROPOSED NEW COAL COMBUSTION RESIDUAL PONDS
SAN ANTONIO, TEXAS

GLOBAL STABILITY ANALYSIS
SECTION A-A
LONG TERM

FIGURE A-4

RABA KISTNER CONSULTANTS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
BERM FILL	🟨	125	Mohr-Coulomb	1000	0
NATURAL COHESIVE SOIL	🟩	125	Mohr-Coulomb	1000	0
UPPER COHESIONLESS SOIL	🟧	120	Mohr-Coulomb	0	35
LOWER COHESIONLESS SOIL	🟦	130	Mohr-Coulomb	0	38
CCR MATERIAL	🟪	120	No strength		

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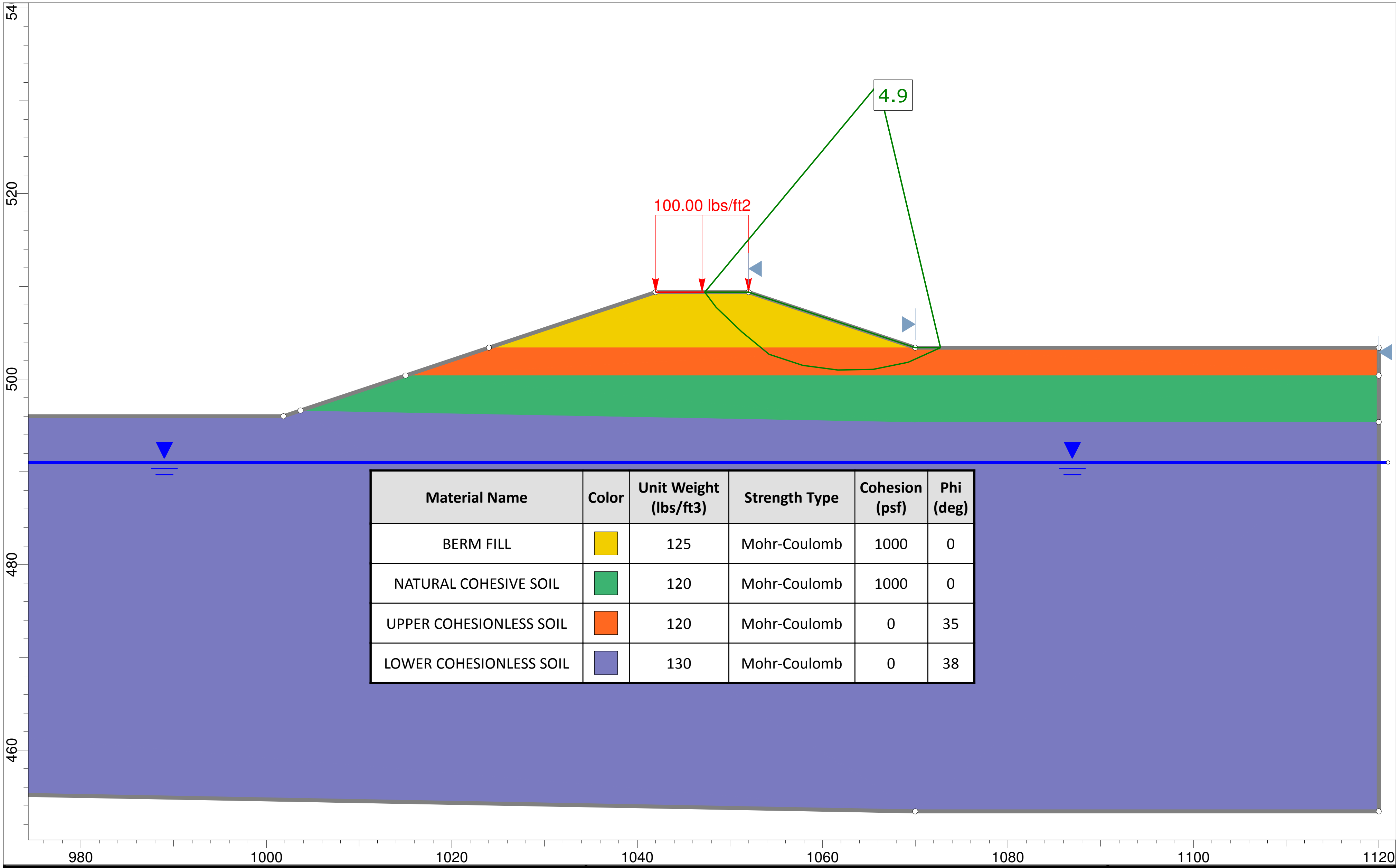
PROPOSED NEW COAL COMBUSTION RESIDUAL PONDS
SAN ANTONIO, TEXAS

GLOBAL STABILITY ANALYSIS
SECTION A-A
SIESMIC CONDITION

FIGURE A-5

RABA KISTNER CONSULTANTS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
BERM FILL	🟨	125	Mohr-Coulomb	1000	0
NATURAL COHESIVE SOIL	🟩	120	Mohr-Coulomb	1000	0
UPPER COHESIONLESS SOIL	🟧	120	Mohr-Coulomb	0	35
LOWER COHESIONLESS SOIL	🟪	130	Mohr-Coulomb	0	38

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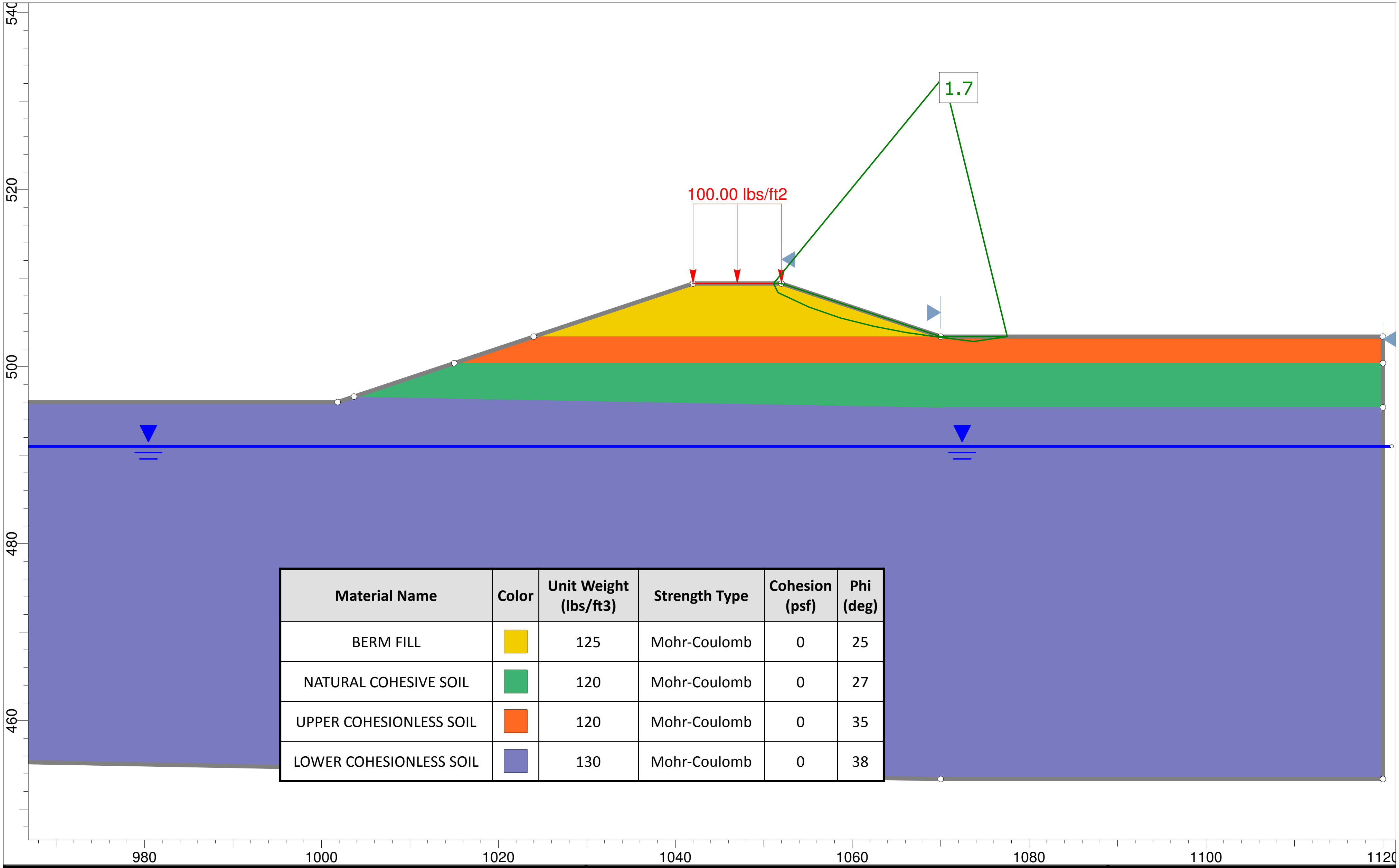
J.K. SPRUCE –CALAVERAS LAKE POWER PLANT
PROPOSED NEW COAL COMBUSTION RESIDUAL PONDS
SAN ANTONIO, TEXAS

GLOBAL STABILITY ANALYSIS
SECTION B-B' DRY SIDE
END OF CONSTRUCTION (SHORT TERM)

FIGURE B-1

RABA KISTNER
CONSULTANTS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
BERM FILL	🟨	125	Mohr-Coulomb	0	25
NATURAL COHESIVE SOIL	🟩	120	Mohr-Coulomb	0	27
UPPER COHESIONLESS SOIL	🟧	120	Mohr-Coulomb	0	35
LOWER COHESIONLESS SOIL	🟪	130	Mohr-Coulomb	0	38

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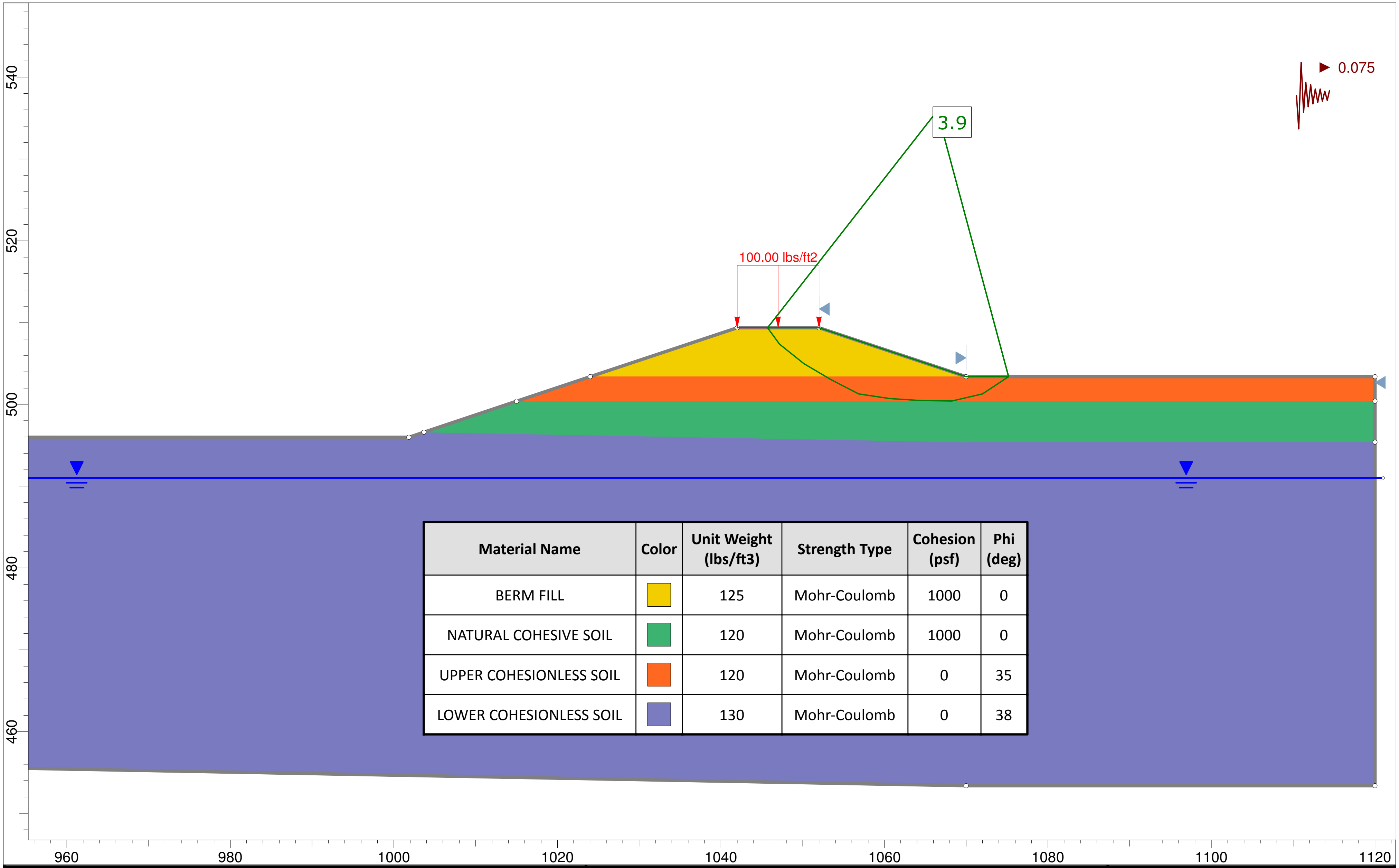
J.K. SPRUCE –CALAVERAS LAKE POWER PLANT
PROPOSED NEW COAL COMBUSTION RESIDUAL PONDS
SAN ANTONIO, TEXAS

GLOBAL STABILITY ANALYSIS
SECTION B-B' DRY SIDE
LONG TERM

FIGURE B-2

RABA KISTNER
CONSULTANTS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
BERM FILL	🟨	125	Mohr-Coulomb	1000	0
NATURAL COHESIVE SOIL	🟩	120	Mohr-Coulomb	1000	0
UPPER COHESIONLESS SOIL	🟧	120	Mohr-Coulomb	0	35
LOWER COHESIONLESS SOIL	🟪	130	Mohr-Coulomb	0	38

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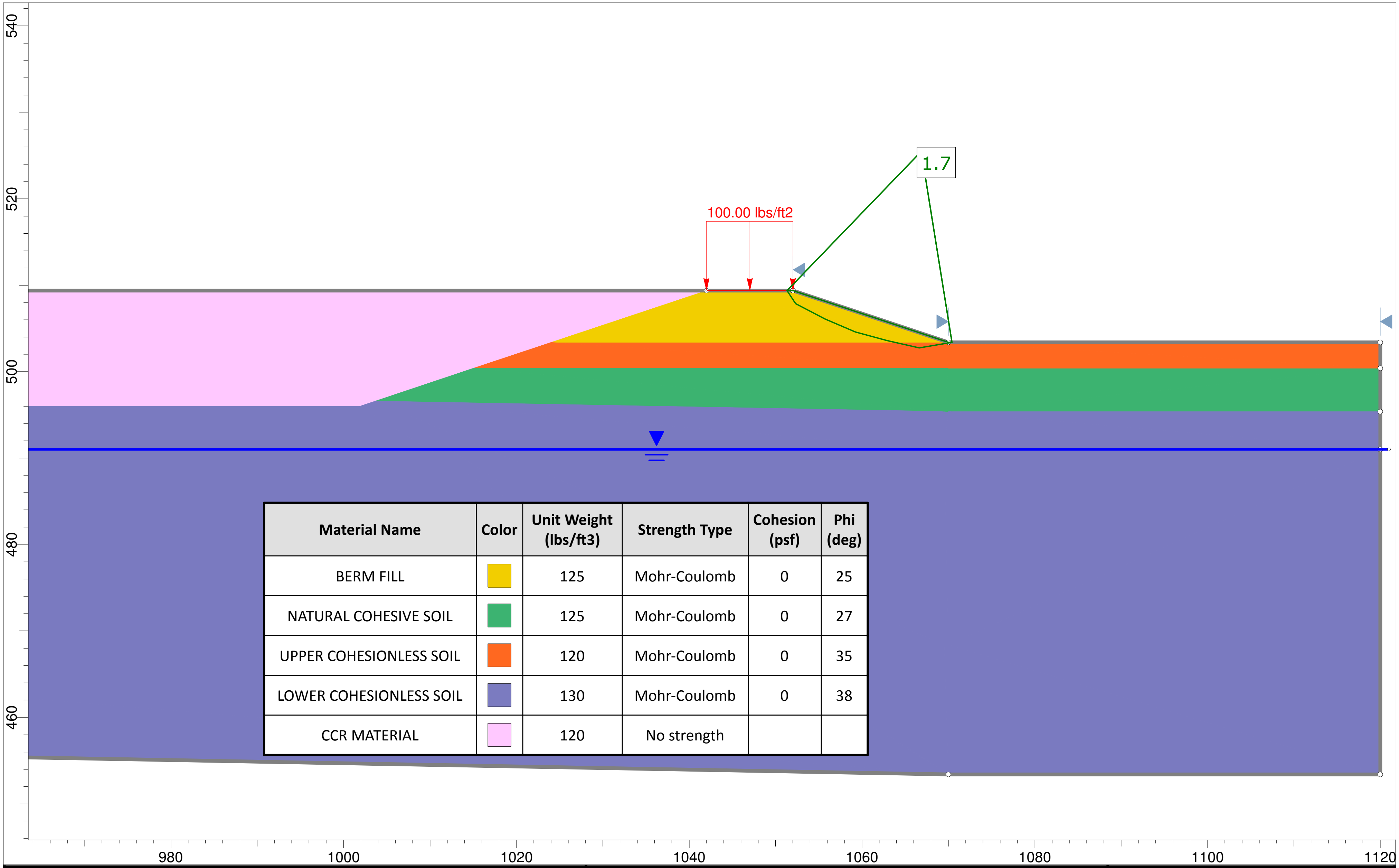
J.K. SPRUCE –CALAVERAS LAKE POWER PLANT
PROPOSED NEW COAL COMBUSTION RESIDUAL PONDS
SAN ANTONIO, TEXAS

GLOBAL STABILITY ANALYSIS
SECTION B-B' DRY SIDE
SEISMIC CONDITION

FIGURE B-3

RABA KISTNER
CONSULTANTS



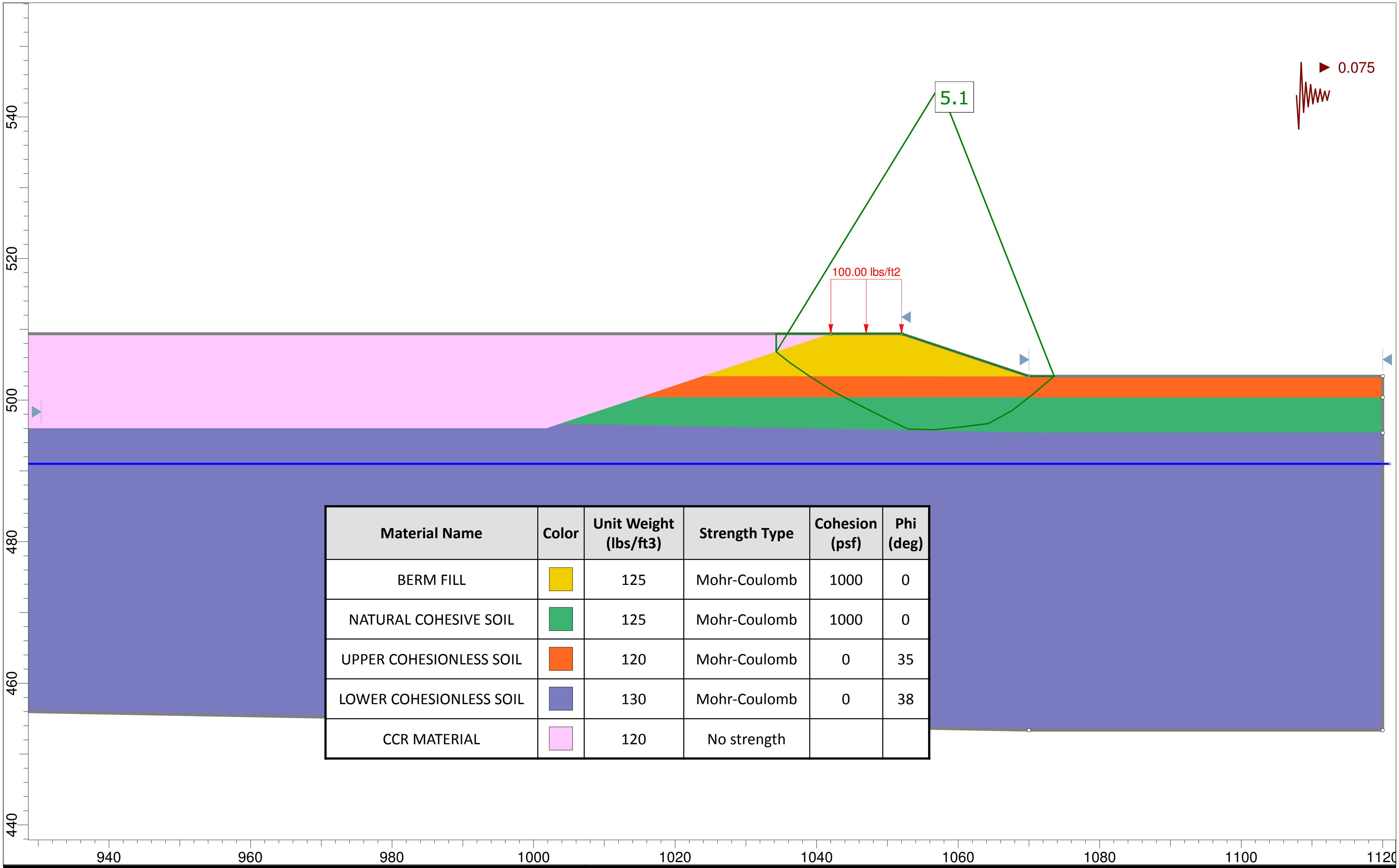
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
BERM FILL	🟨	125	Mohr-Coulomb	0	25
NATURAL COHESIVE SOIL	🟩	125	Mohr-Coulomb	0	27
UPPER COHESIONLESS SOIL	🟧	120	Mohr-Coulomb	0	35
LOWER COHESIONLESS SOIL	🟪	130	Mohr-Coulomb	0	38
CCR MATERIAL	▨	120	No strength		

ASA17-096-00
J.K. SPRUCE –CALAVERAS LAKE POWER PLANT
PROPOSED NEW COAL COMBUSTION RESIDUAL PONDS
SAN ANTONIO, TEXAS

GLOBAL STABILITY ANALYSIS
SECTION B-B' DRY SIDE
LONG TERM

FIGURE B-4

RABA KISTNER
CONSULTANTS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
BERM FILL	🟨	125	Mohr-Coulomb	1000	0
NATURAL COHESIVE SOIL	🟩	125	Mohr-Coulomb	1000	0
UPPER COHESIONLESS SOIL	🟧	120	Mohr-Coulomb	0	35
LOWER COHESIONLESS SOIL	🟦	130	Mohr-Coulomb	0	38
CCR MATERIAL	🟪	120	No strength		

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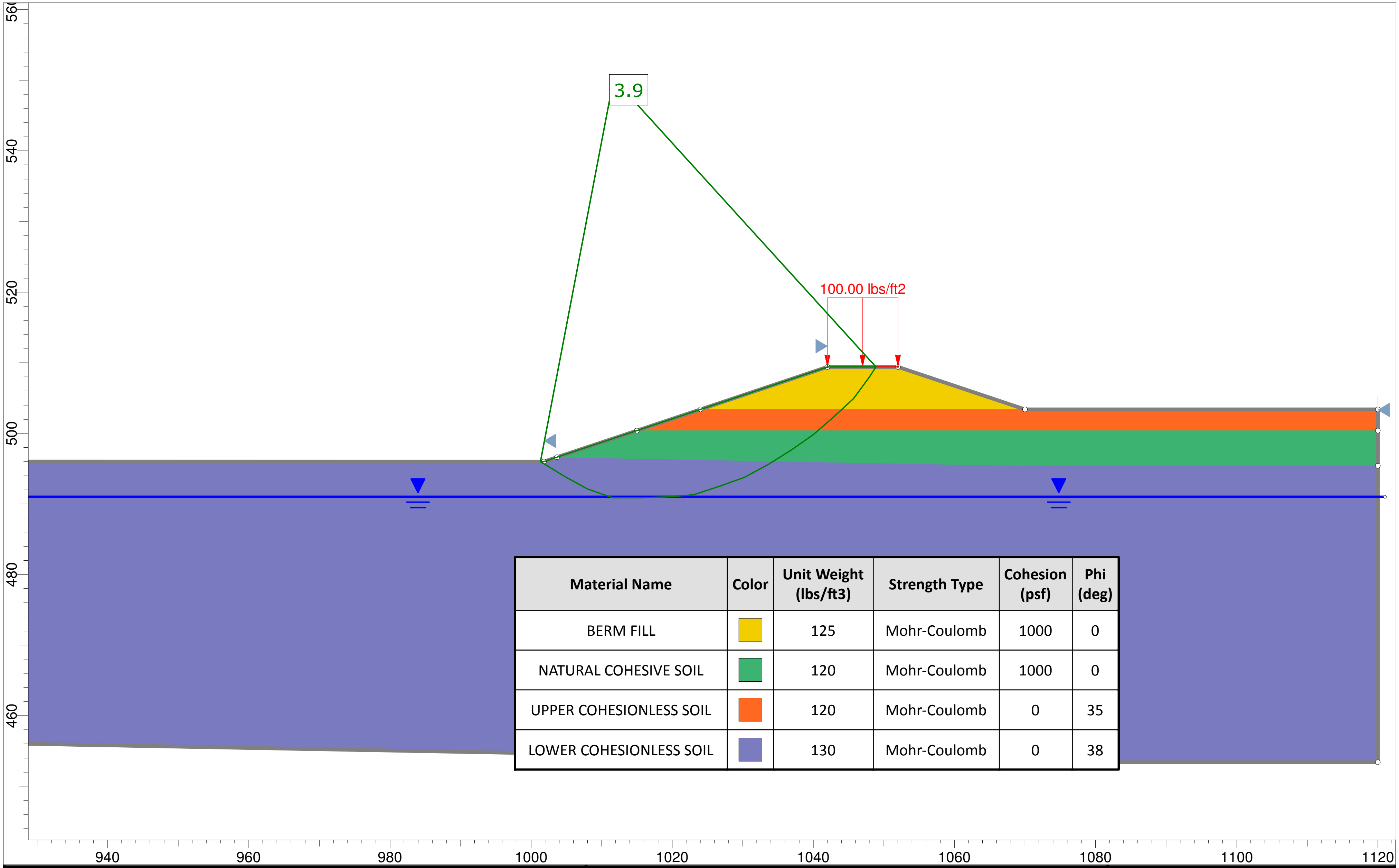
J.K. SPRUCE –CALAVERAS LAKE POWER PLANT
PROPOSED NEW COAL COMBUSTION RESIDUAL PONDS
SAN ANTONIO, TEXAS

GLOBAL STABILITY ANALYSIS
SECTION B-B' DRY SIDE
SEISMIC CONDITION

FIGURE B-5

RABA KISTNER
CONSULTANTS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
BERM FILL	🟨	125	Mohr-Coulomb	1000	0
NATURAL COHESIVE SOIL	🟩	120	Mohr-Coulomb	1000	0
UPPER COHESIONLESS SOIL	🟧	120	Mohr-Coulomb	0	35
LOWER COHESIONLESS SOIL	🟪	130	Mohr-Coulomb	0	38

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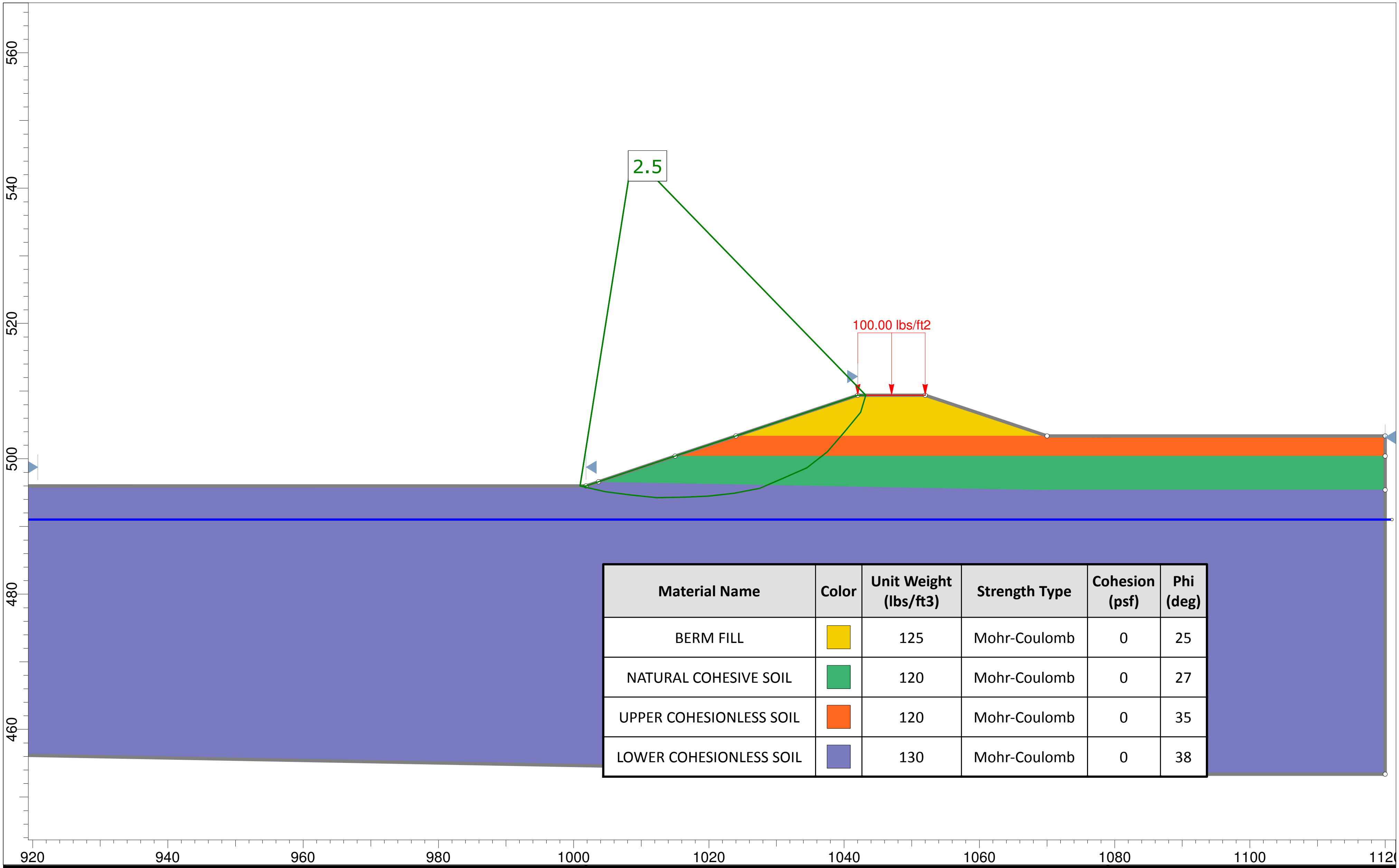
J.K. SPRUCE –CALAVERAS LAKE POWER PLANT
PROPOSED NEW COAL COMBUSTION RESIDUAL PONDS
SAN ANTONIO, TEXAS

GLOBAL STABILITY ANALYSIS
SECTION B-B' POND SIDE
END OF CONSTRUCTION (SHORT TERM)

FIGURE B-6

RABA
KISTNER
CONSULTANTS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
BERM FILL	🟨	125	Mohr-Coulomb	0	25
NATURAL COHESIVE SOIL	🟩	120	Mohr-Coulomb	0	27
UPPER COHESIONLESS SOIL	🟧	120	Mohr-Coulomb	0	35
LOWER COHESIONLESS SOIL	🟪	130	Mohr-Coulomb	0	38

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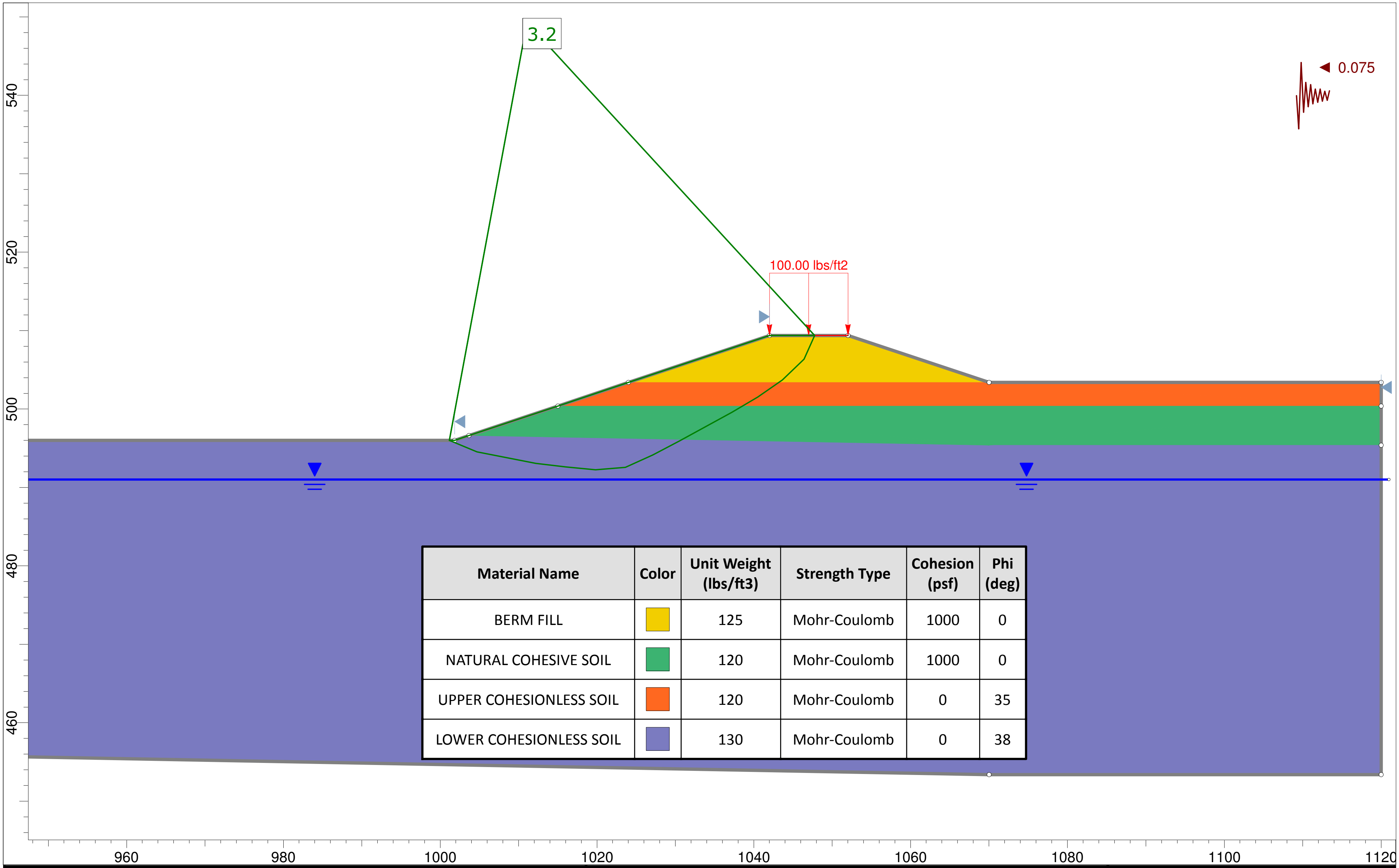
J.K. SPRUCE –CALAVERAS LAKE POWER PLANT
PROPOSED NEW COAL COMBUSTION RESIDUAL PONDS
SAN ANTONIO, TEXAS

GLOBAL STABILITY ANALYSIS
SECTION B-B' POND SIDE
LONG TERM

FIGURE B-7

RABA KISTNER
CONSULTANTS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
BERM FILL	🟨	125	Mohr-Coulomb	1000	0
NATURAL COHESIVE SOIL	🟩	120	Mohr-Coulomb	1000	0
UPPER COHESIONLESS SOIL	🟧	120	Mohr-Coulomb	0	35
LOWER COHESIONLESS SOIL	🟪	130	Mohr-Coulomb	0	38

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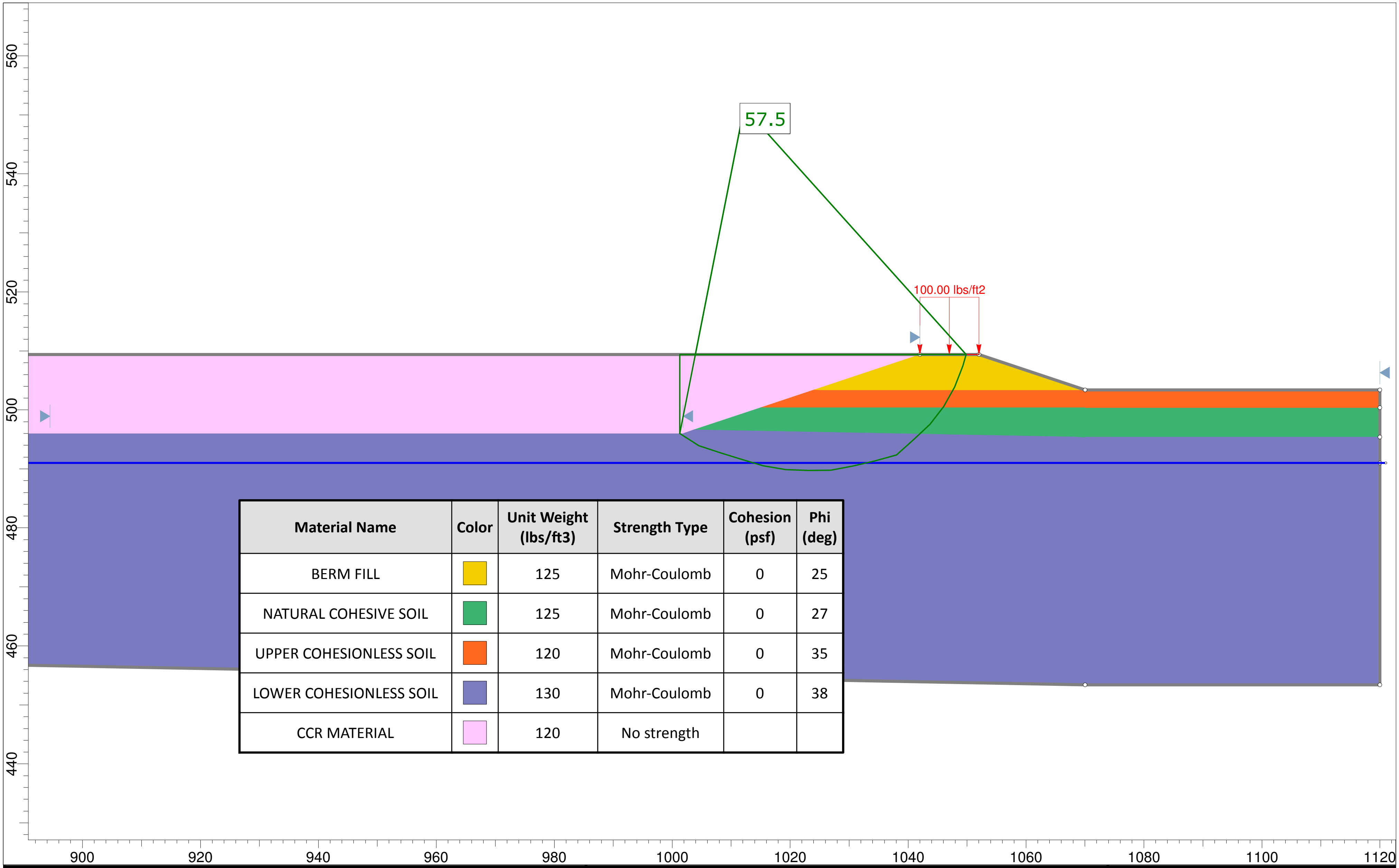
J.K. SPRUCE –CALAVERAS LAKE POWER PLANT
PROPOSED NEW COAL COMBUSTION RESIDUAL PONDS
SAN ANTONIO, TEXAS

GLOBAL STABILITY ANALYSIS
SECTION B-B' POND SIDE
SEISMIC CONDITION

FIGURE B-8

RABA KISTNER
CONSULTANTS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
BERM FILL	🟨	125	Mohr-Coulomb	0	25
NATURAL COHESIVE SOIL	🟩	125	Mohr-Coulomb	0	27
UPPER COHESIONLESS SOIL	🟧	120	Mohr-Coulomb	0	35
LOWER COHESIONLESS SOIL	🟦	130	Mohr-Coulomb	0	38
CCR MATERIAL	🟪	120	No strength		

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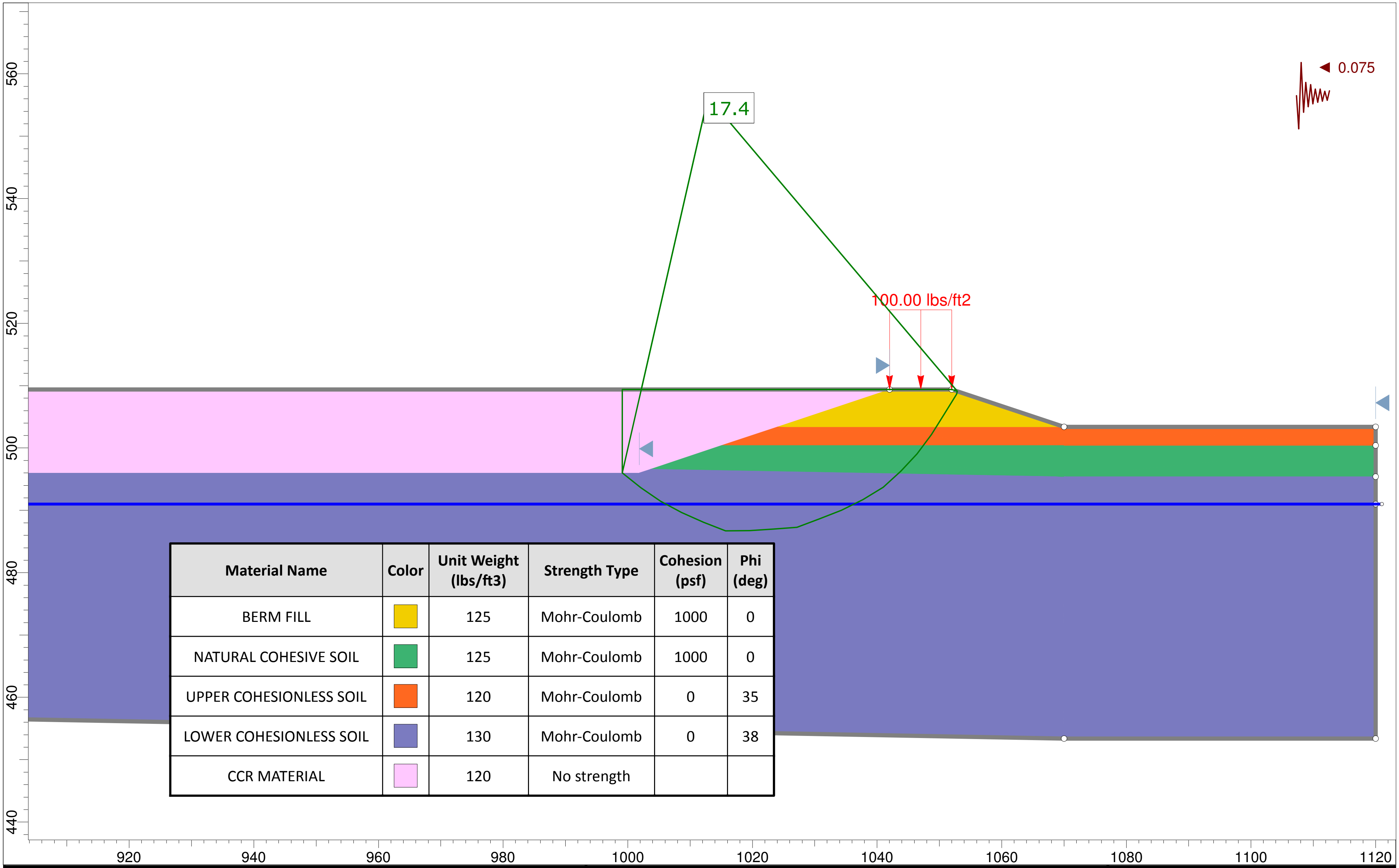
PROPOSED NEW COAL COMBUSTION RESIDUAL PONDS
SAN ANTONIO, TEXAS

GLOBAL STABILITY ANALYSIS
SECTION B-B' POND SIDE
LONG TERM

FIGURE B-9

RABA KISTNER CONSULTANTS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
BERM FILL	🟨	125	Mohr-Coulomb	1000	0
NATURAL COHESIVE SOIL	🟩	125	Mohr-Coulomb	1000	0
UPPER COHESIONLESS SOIL	🟧	120	Mohr-Coulomb	0	35
LOWER COHESIONLESS SOIL	🟦	130	Mohr-Coulomb	0	38
CCR MATERIAL	🟪	120	No strength		

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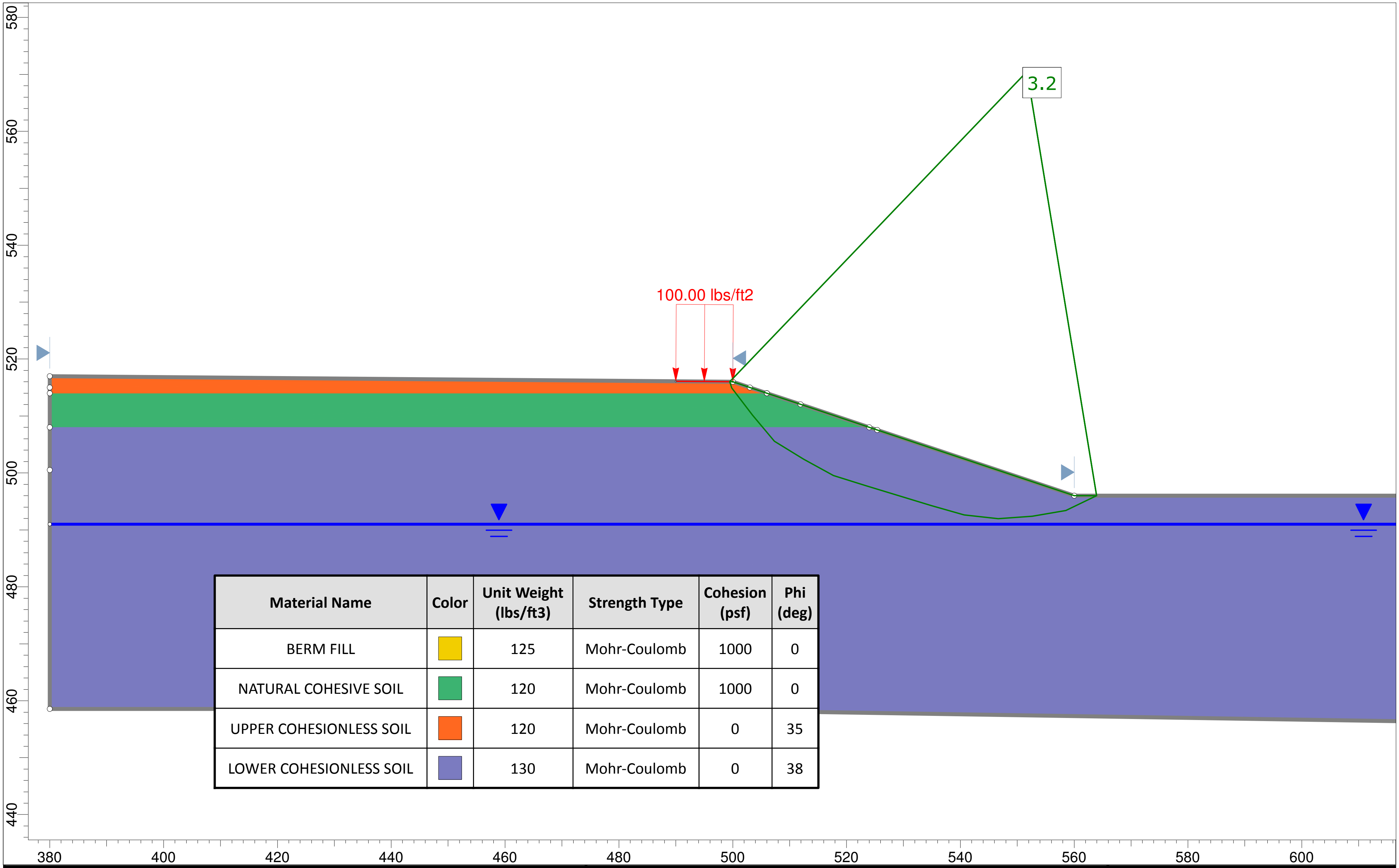
PROPOSED NEW COAL COMBUSTION RESIDUAL PONDS
SAN ANTONIO, TEXAS

GLOBAL STABILITY ANALYSIS
SECTION B-B' POND SIDE
SEISMIC CONDITION

FIGURE B-10

RABA KISTNER
CONSULTANTS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
BERM FILL	🟨	125	Mohr-Coulomb	1000	0
NATURAL COHESIVE SOIL	🟩	120	Mohr-Coulomb	1000	0
UPPER COHESIONLESS SOIL	🟧	120	Mohr-Coulomb	0	35
LOWER COHESIONLESS SOIL	🟪	130	Mohr-Coulomb	0	38

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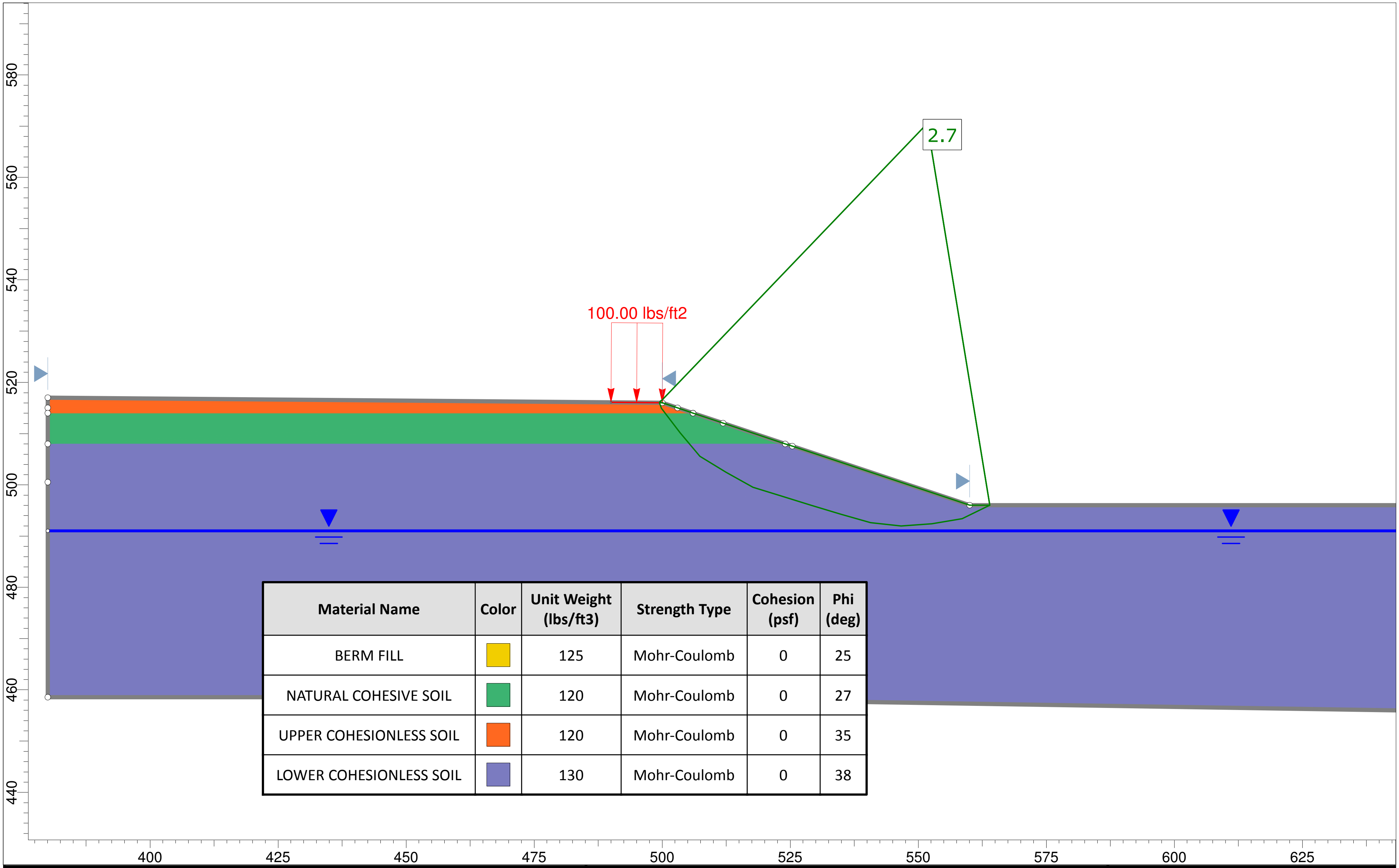
J.K. SPRUCE –CALAVERAS LAKE POWER PLANT
PROPOSED NEW COAL COMBUSTION RESIDUAL PONDS
SAN ANTONIO, TEXAS

GLOBAL STABILITY ANALYSIS
SECTION C-C'
END OF CONSTRUCTION (SHORT TERM)

FIGURE C-1

RABA
KISTNER
CONSULTANTS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
BERM FILL	🟨	125	Mohr-Coulomb	0	25
NATURAL COHESIVE SOIL	🟩	120	Mohr-Coulomb	0	27
UPPER COHESIONLESS SOIL	🟧	120	Mohr-Coulomb	0	35
LOWER COHESIONLESS SOIL	🟪	130	Mohr-Coulomb	0	38

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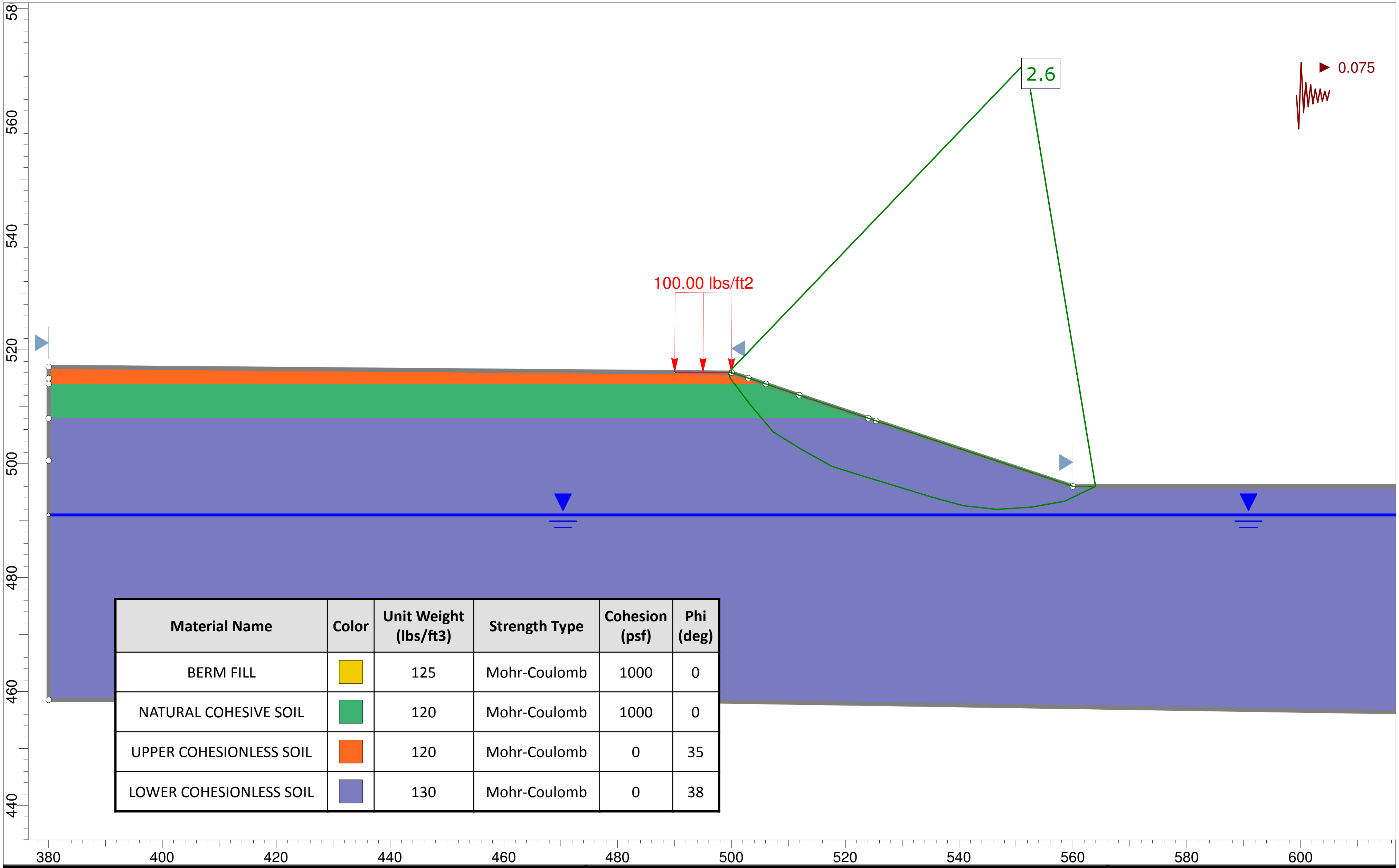
SAN ANTONIO, TEXAS

GLOBAL STABILITY ANALYSIS
SECTION C-C'
LONG TERM

FIGURE C-2

RABA KISTNER
CONSULTANTS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
BERM FILL	🟨	125	Mohr-Coulomb	1000	0
NATURAL COHESIVE SOIL	🟩	120	Mohr-Coulomb	1000	0
UPPER COHESIONLESS SOIL	🟧	120	Mohr-Coulomb	0	35
LOWER COHESIONLESS SOIL	🟪	130	Mohr-Coulomb	0	38

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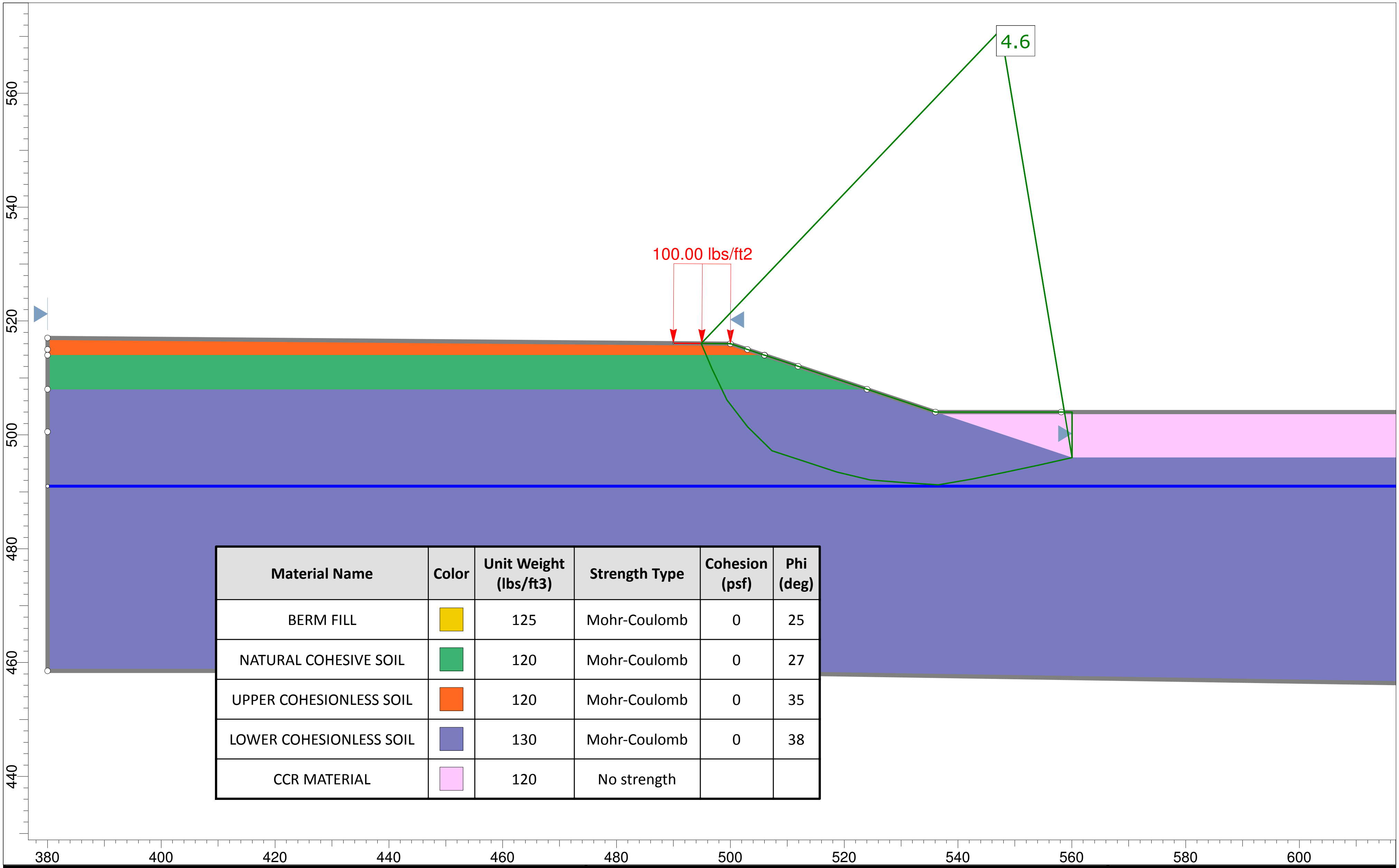
SAN ANTONIO, TEXAS

GLOBAL STABILITY ANALYSIS
SECTION C-C'
SEISMIC CONDITION

FIGURE C-3

RABA
KISTNER
CONSULTANTS



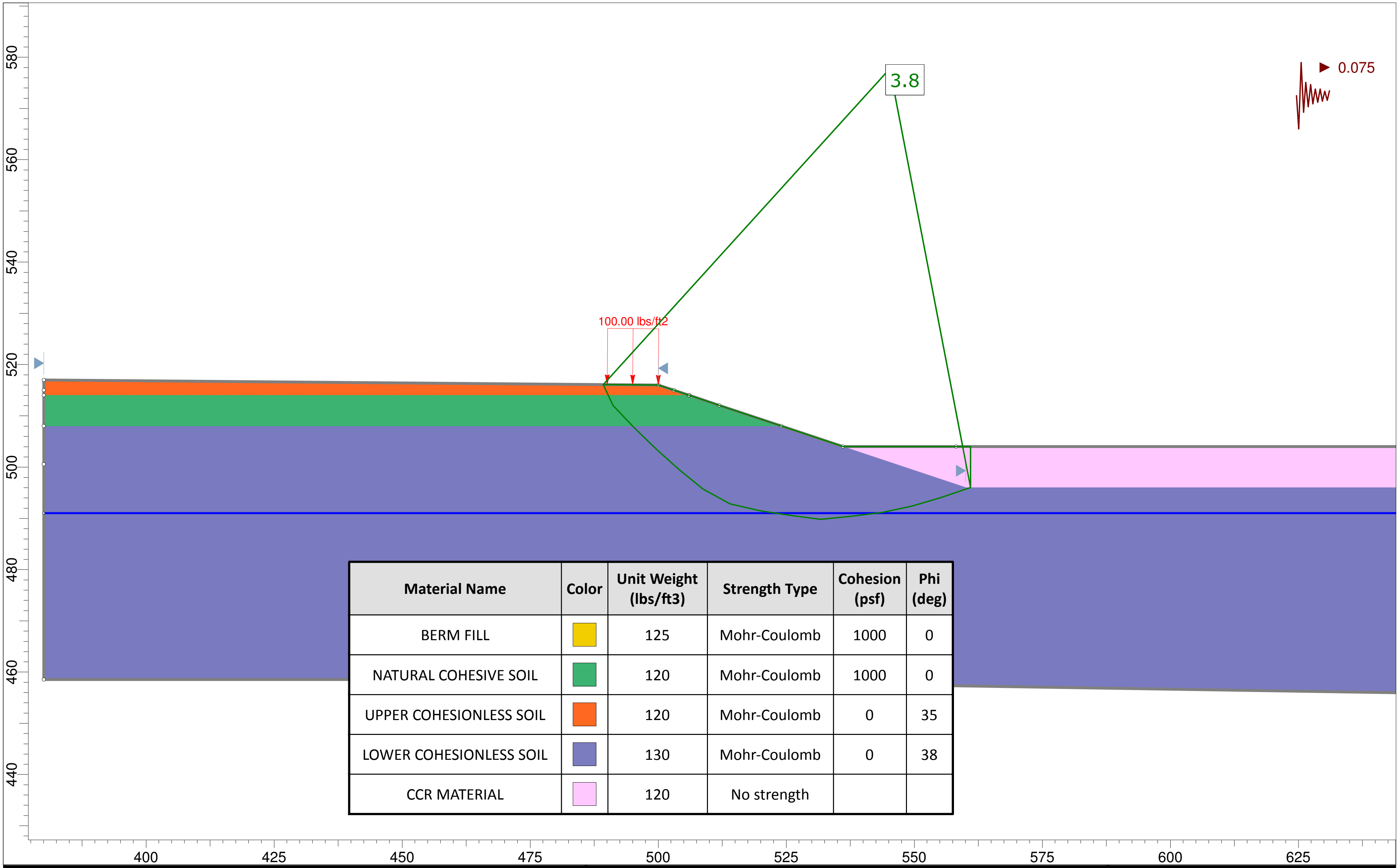
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
BERM FILL	🟨	125	Mohr-Coulomb	0	25
NATURAL COHESIVE SOIL	🟩	120	Mohr-Coulomb	0	27
UPPER COHESIONLESS SOIL	🟧	120	Mohr-Coulomb	0	35
LOWER COHESIONLESS SOIL	🟦	130	Mohr-Coulomb	0	38
CCR MATERIAL	🟪	120	No strength		

ASA17-096-00
J.K. SPRUCE –CALAVERAS LAKE POWER PLANT
PROPOSED NEW COAL COMBUSTION RESIDUAL PONDS
SAN ANTONIO, TEXAS

GLOBAL STABILITY ANALYSIS
SECTION C-C'
LONG TERM

FIGURE C-4

RABA KISTNER
CONSULTANTS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
BERM FILL	🟨	125	Mohr-Coulomb	1000	0
NATURAL COHESIVE SOIL	🟩	120	Mohr-Coulomb	1000	0
UPPER COHESIONLESS SOIL	🟧	120	Mohr-Coulomb	0	35
LOWER COHESIONLESS SOIL	🟦	130	Mohr-Coulomb	0	38
CCR MATERIAL	🟪	120	No strength		

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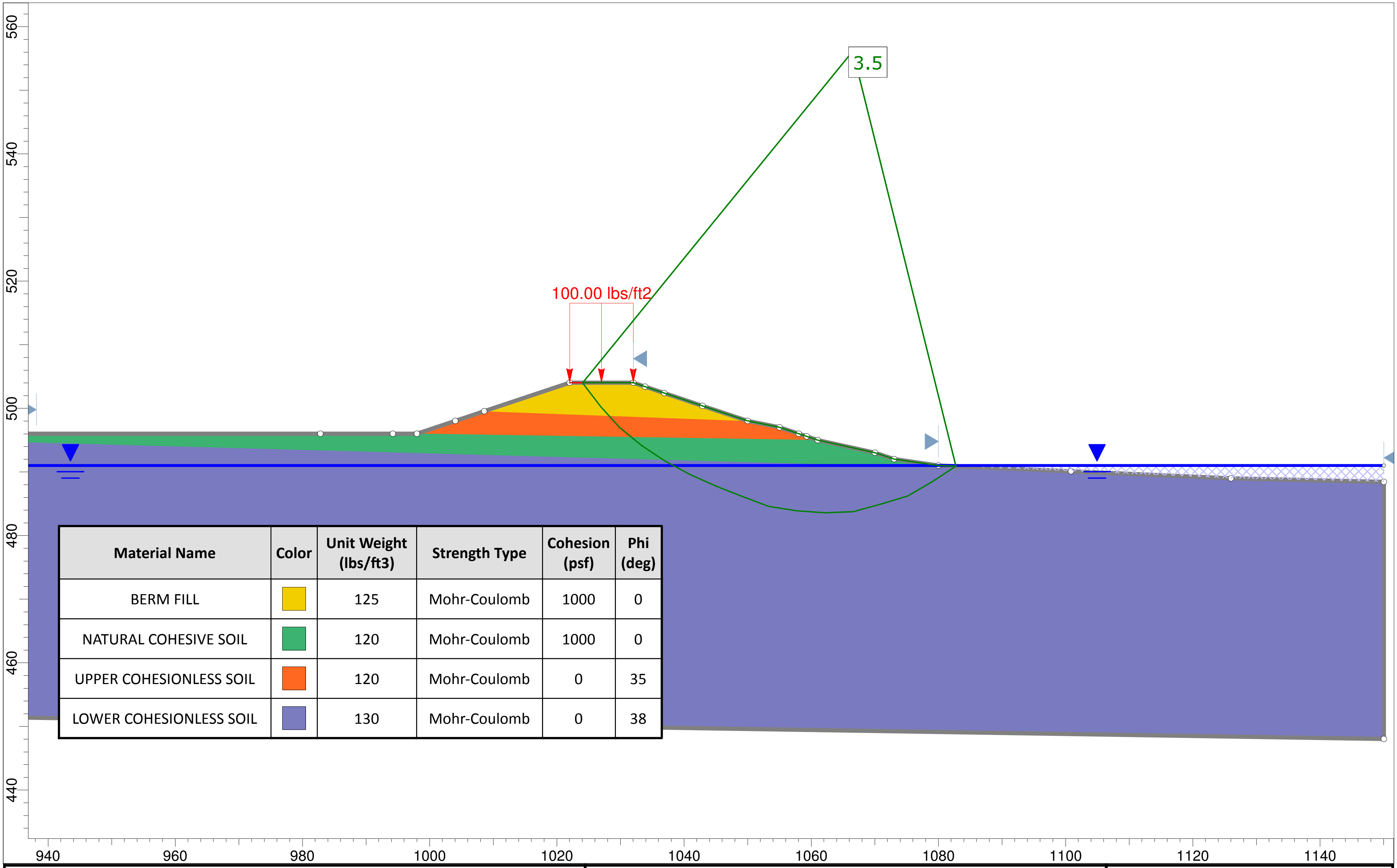
J.K. SPRUCE –CALAVERAS LAKE POWER PLANT
PROPOSED NEW COAL COMBUSTION RESIDUAL PONDS
SAN ANTONIO, TEXAS

GLOBAL STABILITY ANALYSIS
SECTION C-C'
SEISMIC CONDITION

FIGURE C-5

RABA
KISTNER
CONSULTANTS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
BERM FILL	🟨	125	Mohr-Coulomb	1000	0
NATURAL COHESIVE SOIL	🟩	120	Mohr-Coulomb	1000	0
UPPER COHESIONLESS SOIL	🟧	120	Mohr-Coulomb	0	35
LOWER COHESIONLESS SOIL	🟪	130	Mohr-Coulomb	0	38

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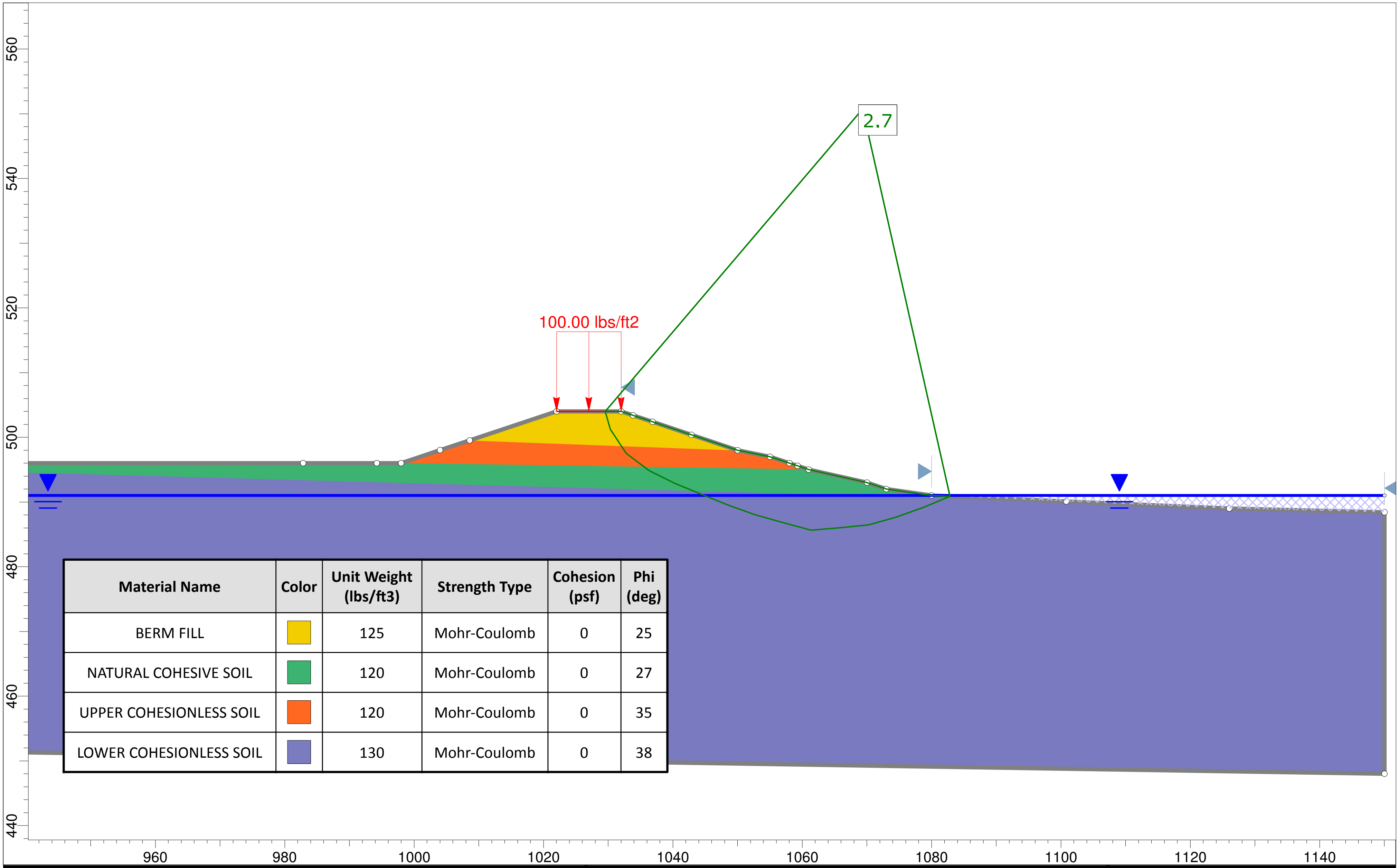
J.K. SPRUCE –CALAVERAS LAKE POWER PLANT
PROPOSED NEW COAL COMBUSTION RESIDUAL PONDS
SAN ANTONIO, TEXAS

GLOBAL STABILITY ANALYSIS
SECTION D-D' DRY SIDE
SHORT TERM

FIGURE D-1

RABA KISTNER CONSULTANTS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
BERM FILL	🟨	125	Mohr-Coulomb	0	25
NATURAL COHESIVE SOIL	🟩	120	Mohr-Coulomb	0	27
UPPER COHESIONLESS SOIL	🟧	120	Mohr-Coulomb	0	35
LOWER COHESIONLESS SOIL	🟪	130	Mohr-Coulomb	0	38

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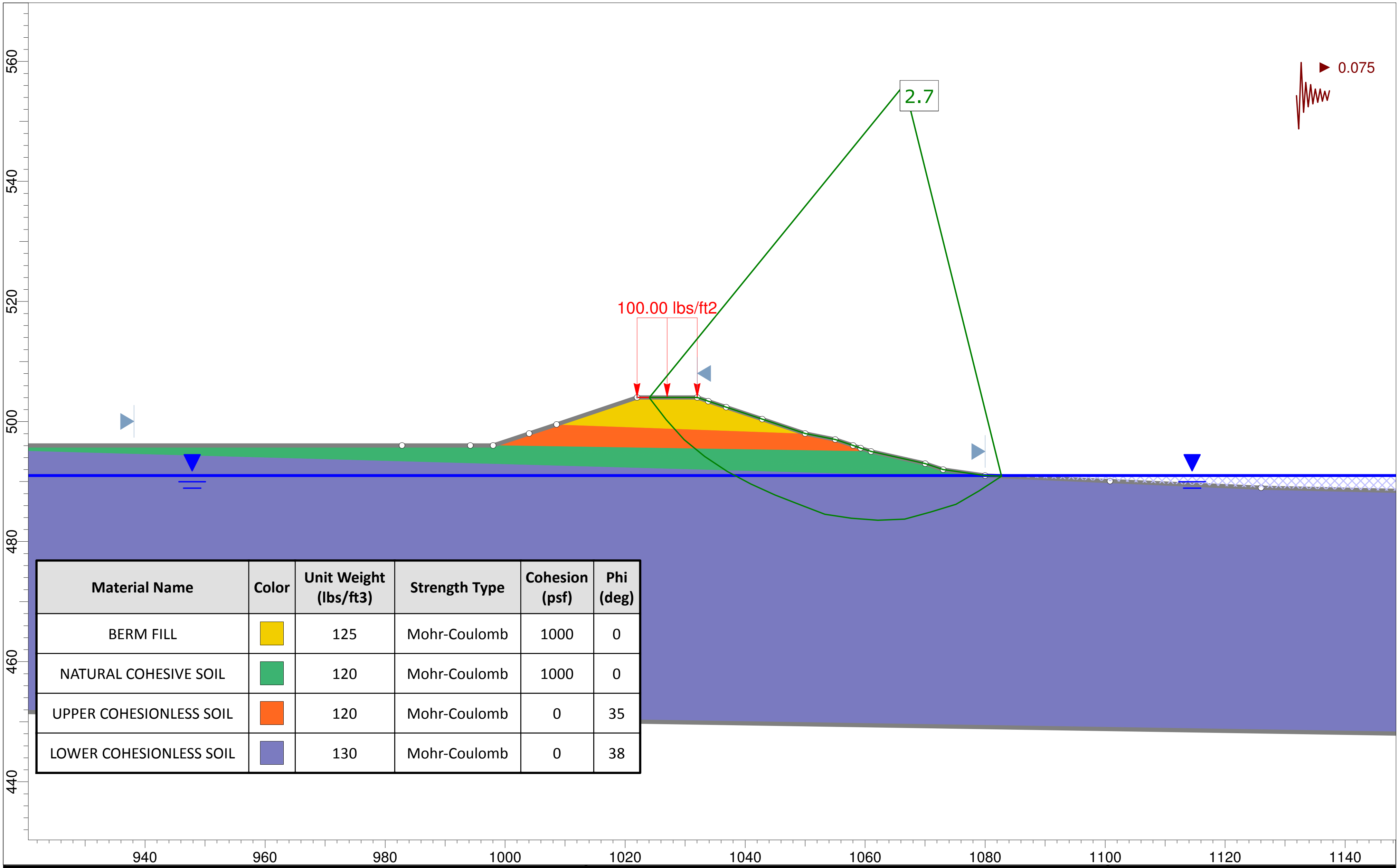
J.K. SPRUCE –CALAVERAS LAKE POWER PLANT
PROPOSED NEW COAL COMBUSTION RESIDUAL PONDS
SAN ANTONIO, TEXAS

GLOBAL STABILITY ANALYSIS
SECTION D-D' DRY SIDE
LONG TERM

FIGURE D-2

RABA KISTNER
CONSULTANTS



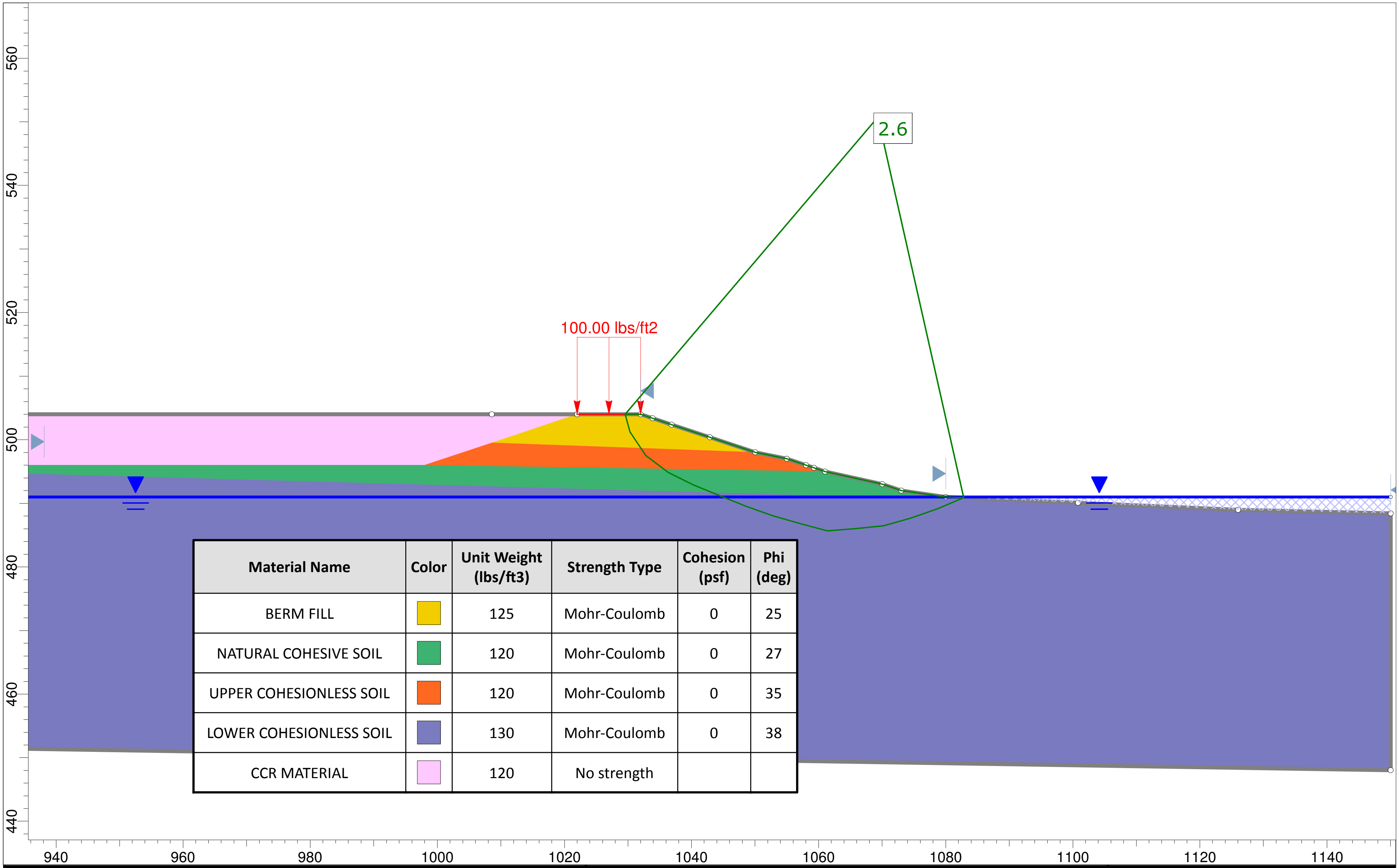
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
BERM FILL	🟨	125	Mohr-Coulomb	1000	0
NATURAL COHESIVE SOIL	🟩	120	Mohr-Coulomb	1000	0
UPPER COHESIONLESS SOIL	🟧	120	Mohr-Coulomb	0	35
LOWER COHESIONLESS SOIL	🟪	130	Mohr-Coulomb	0	38

ASA17-096-00
J.K. SPRUCE –CALAVERAS LAKE POWER PLANT
PROPOSED NEW COAL COMBUSTION RESIDUAL PONDS
SAN ANTONIO, TEXAS

GLOBAL STABILITY ANALYSIS
SECTION D-D' DRY SIDE
SEISMIC CONDITION

FIGURE D-3

RABA KISTNER CONSULTANTS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
BERM FILL	🟨	125	Mohr-Coulomb	0	25
NATURAL COHESIVE SOIL	🟩	120	Mohr-Coulomb	0	27
UPPER COHESIONLESS SOIL	🟧	120	Mohr-Coulomb	0	35
LOWER COHESIONLESS SOIL	🟦	130	Mohr-Coulomb	0	38
CCR MATERIAL	🟪	120	No strength		

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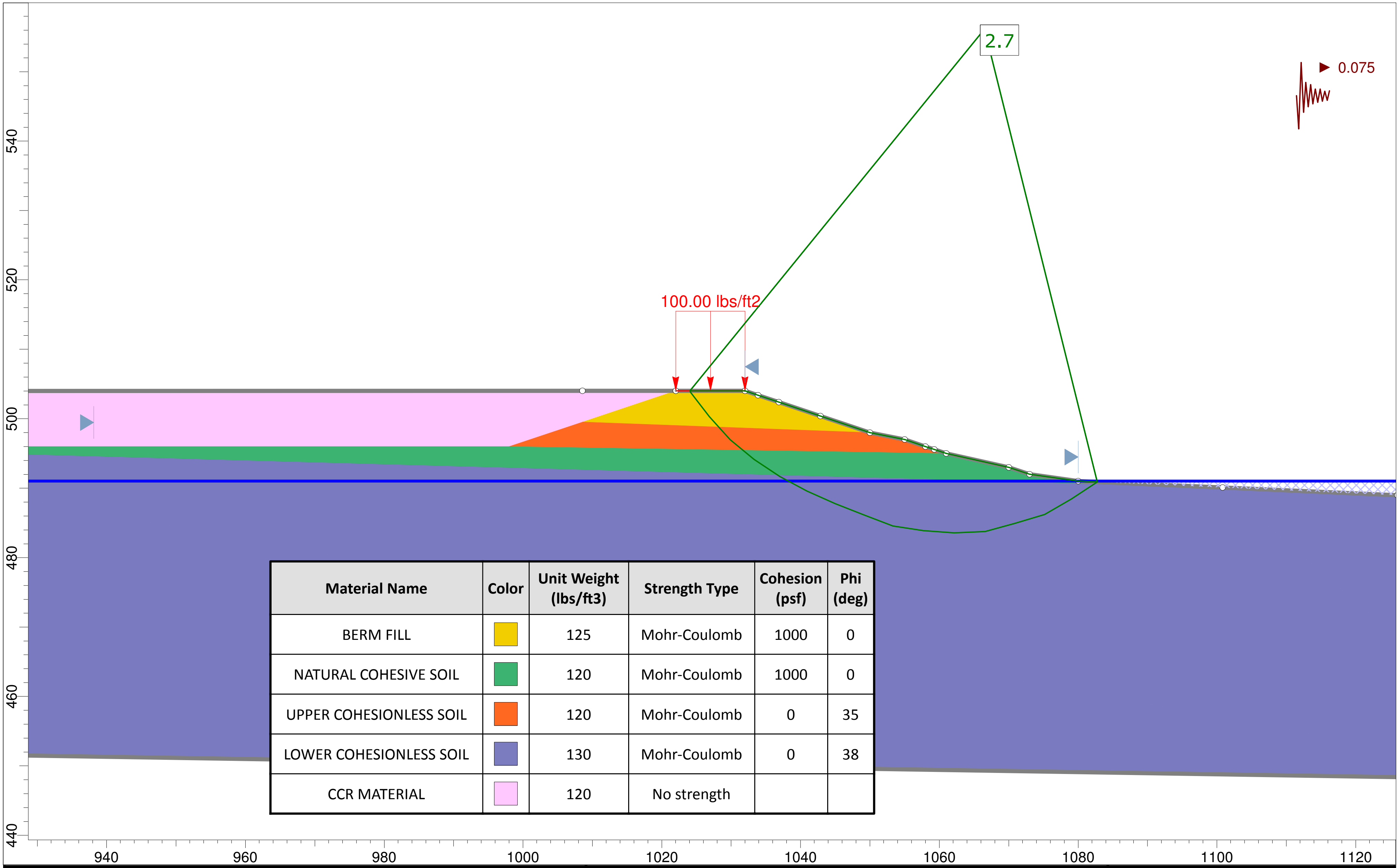
J.K. SPRUCE –CALAVERAS LAKE POWER PLANT
PROPOSED NEW COAL COMBUSTION RESIDUAL PONDS
SAN ANTONIO, TEXAS

GLOBAL STABILITY ANALYSIS
SECTION D-D' DRY SIDE
LONG TERM

FIGURE D-4

RABA KISTNER
CONSULTANTS



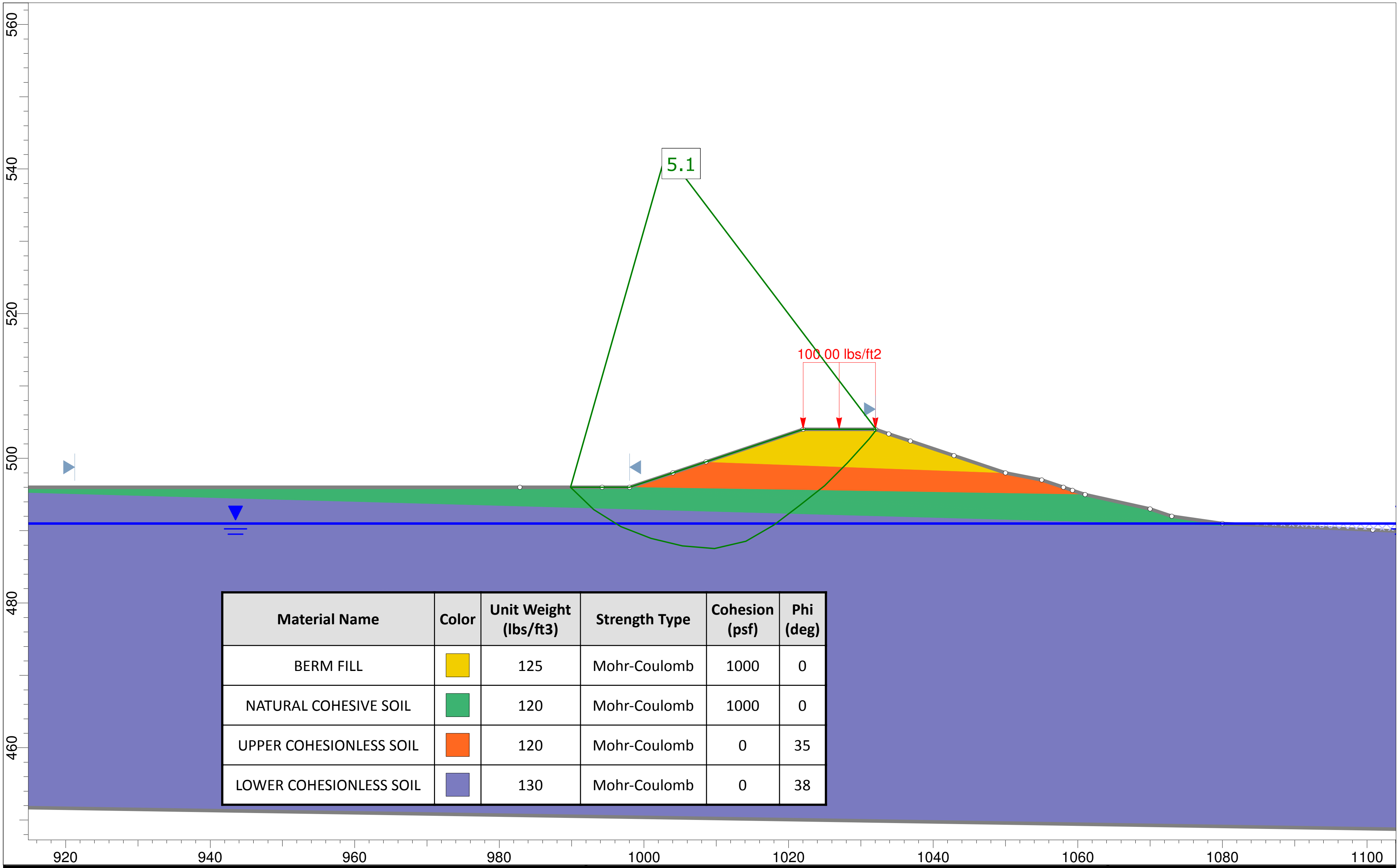
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
BERM FILL	🟨	125	Mohr-Coulomb	1000	0
NATURAL COHESIVE SOIL	🟩	120	Mohr-Coulomb	1000	0
UPPER COHESIONLESS SOIL	🟧	120	Mohr-Coulomb	0	35
LOWER COHESIONLESS SOIL	🟦	130	Mohr-Coulomb	0	38
CCR MATERIAL	🟪	120	No strength		

ASA17-096-00
J.K. SPRUCE –CALAVERAS LAKE POWER PLANT
PROPOSED NEW COAL COMBUSTION RESIDUAL PONDS
SAN ANTONIO, TEXAS

GLOBAL STABILITY ANALYSIS
SECTION D-D' DRY SIDE
SEISMIC CONDITION

FIGURE D-5

RABA KISTNER
CONSULTANTS



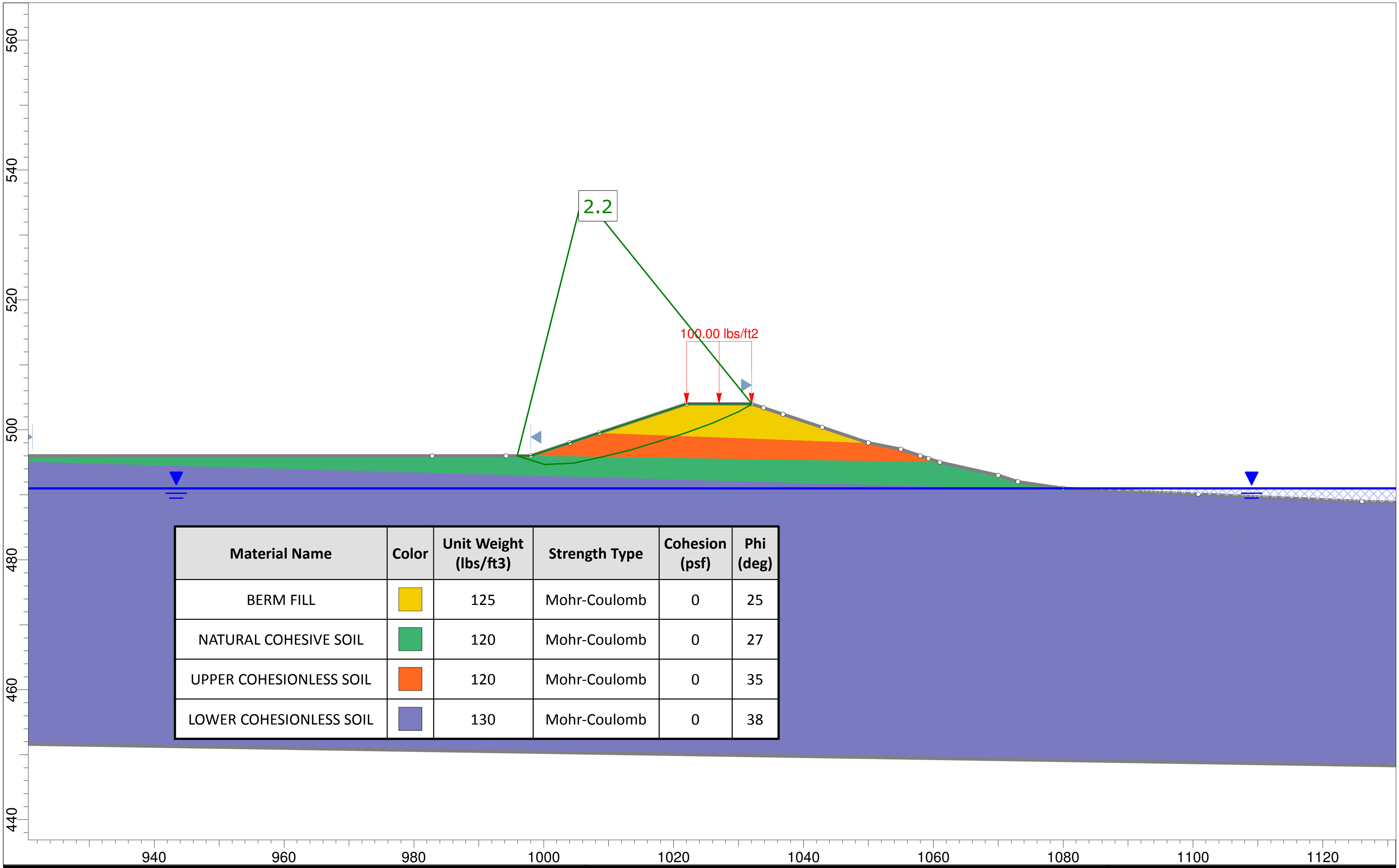
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
BERM FILL	🟨	125	Mohr-Coulomb	1000	0
NATURAL COHESIVE SOIL	🟩	120	Mohr-Coulomb	1000	0
UPPER COHESIONLESS SOIL	🟧	120	Mohr-Coulomb	0	35
LOWER COHESIONLESS SOIL	🟪	130	Mohr-Coulomb	0	38

ASA17-096-00
J.K. SPRUCE –CALAVERAS LAKE POWER PLANT
PROPOSED NEW COAL COMBUSTION RESIDUAL PONDS
SAN ANTONIO, TEXAS

GLOBAL STABILITY ANALYSIS
SECTION D-D' POND SIDE
END OF CONSTRUCTION (SHORT TERM)

FIGURE D-6

RABA KISTNER
CONSULTANTS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
BERM FILL	🟨	125	Mohr-Coulomb	0	25
NATURAL COHESIVE SOIL	🟩	120	Mohr-Coulomb	0	27
UPPER COHESIONLESS SOIL	🟧	120	Mohr-Coulomb	0	35
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J.K. SPRUCE –CALAVERAS LAKE POWER PLANT
PROPOSED NEW COAL COMBUSTION RESIDUAL PONDS
SAN ANTONIO, TEXAS

GLOBAL STABILITY ANALYSIS
SECTION D-D' POND SIDE
LONG TERM

FIGURE D-7

RABA KISTNER
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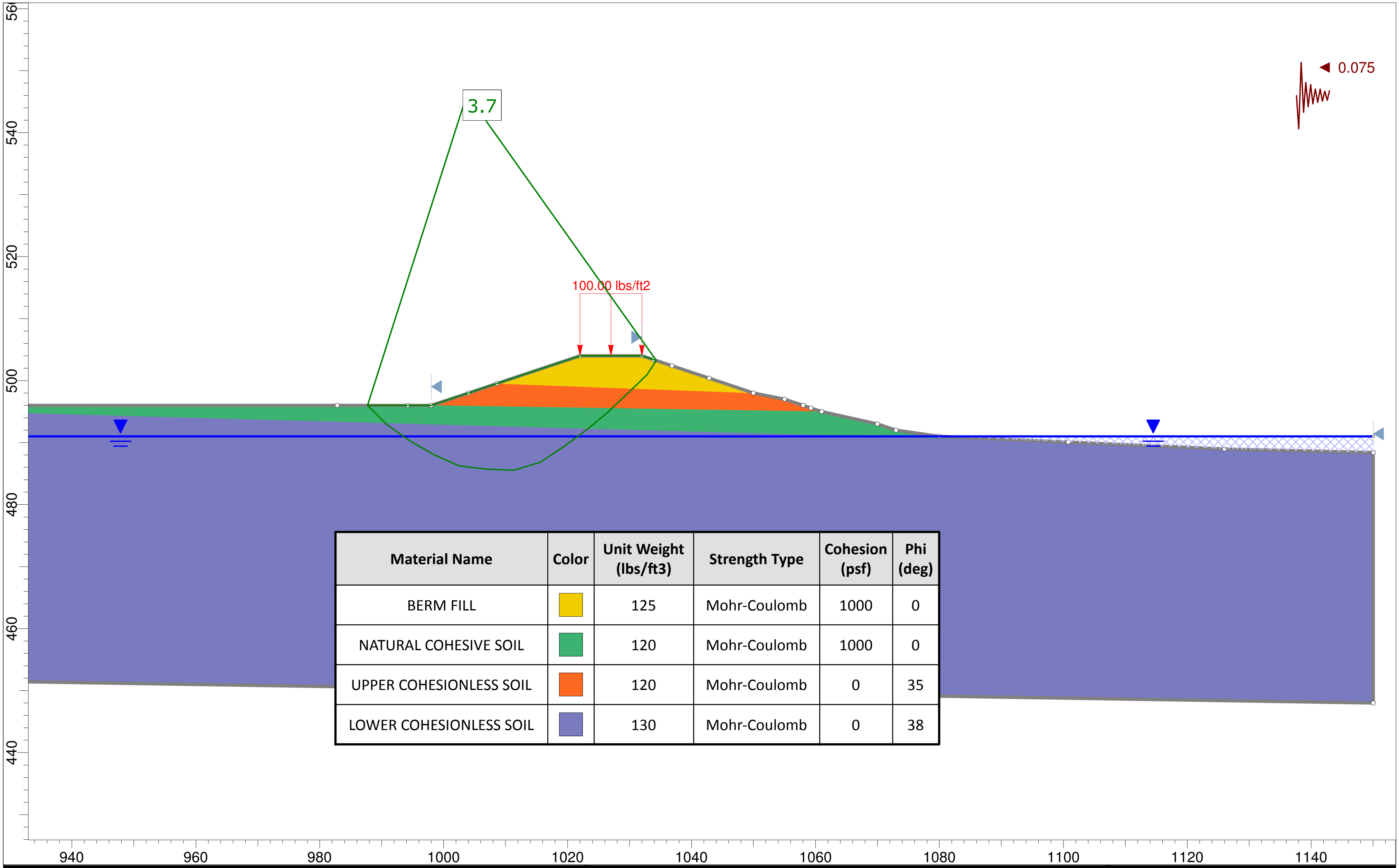


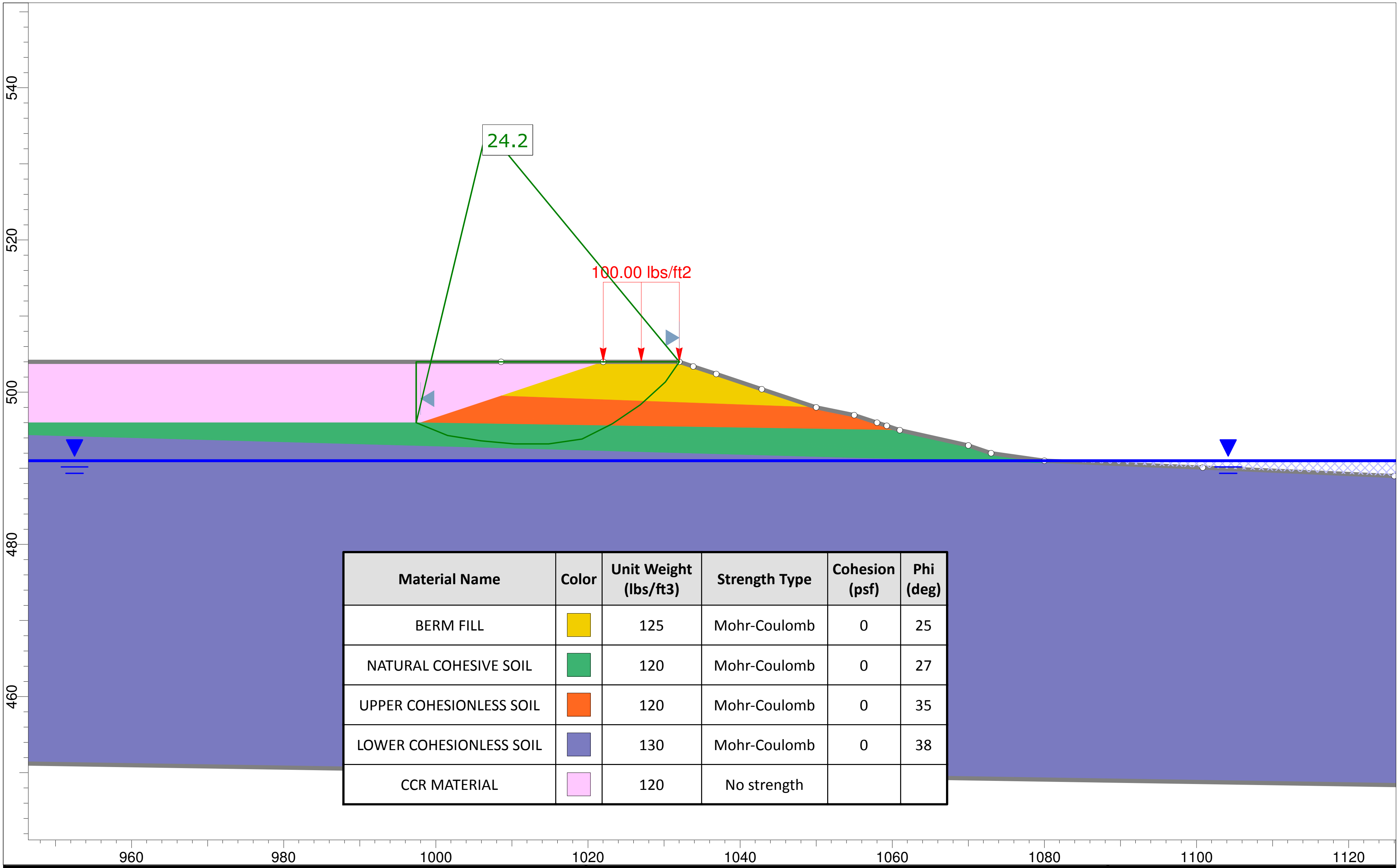
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
BERM FILL	🟨	125	Mohr-Coulomb	1000	0
NATURAL COHESIVE SOIL	🟩	120	Mohr-Coulomb	1000	0
UPPER COHESIONLESS SOIL	🟧	120	Mohr-Coulomb	0	35
LOWER COHESIONLESS SOIL	🟪	130	Mohr-Coulomb	0	38

ASA17-096-00
J.K. SPRUCE –CALAVERAS LAKE POWER PLANT
PROPOSED NEW COAL COMBUSTION RESIDUAL PONDS
SAN ANTONIO, TEXAS

GLOBAL STABILITY ANALYSIS
SECTION D-D' POND SIDE
SEISMIC CONDITION

FIGURE D-8

RABA KISTNER CONSULTANTS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
BERM FILL	🟨	125	Mohr-Coulomb	0	25
NATURAL COHESIVE SOIL	🟩	120	Mohr-Coulomb	0	27
UPPER COHESIONLESS SOIL	🟧	120	Mohr-Coulomb	0	35
LOWER COHESIONLESS SOIL	🟦	130	Mohr-Coulomb	0	38
CCR MATERIAL	🟪	120	No strength		

ASA17-096-00
J.K. SPRUCE –CALAVERAS LAKE POWER PLANT
PROPOSED NEW COAL COMBUSTION RESIDUAL PONDS
SAN ANTONIO, TEXAS

GLOBAL STABILITY ANALYSIS
SECTION D-D' POND SIDE
LONG TERM

FIGURE D-9

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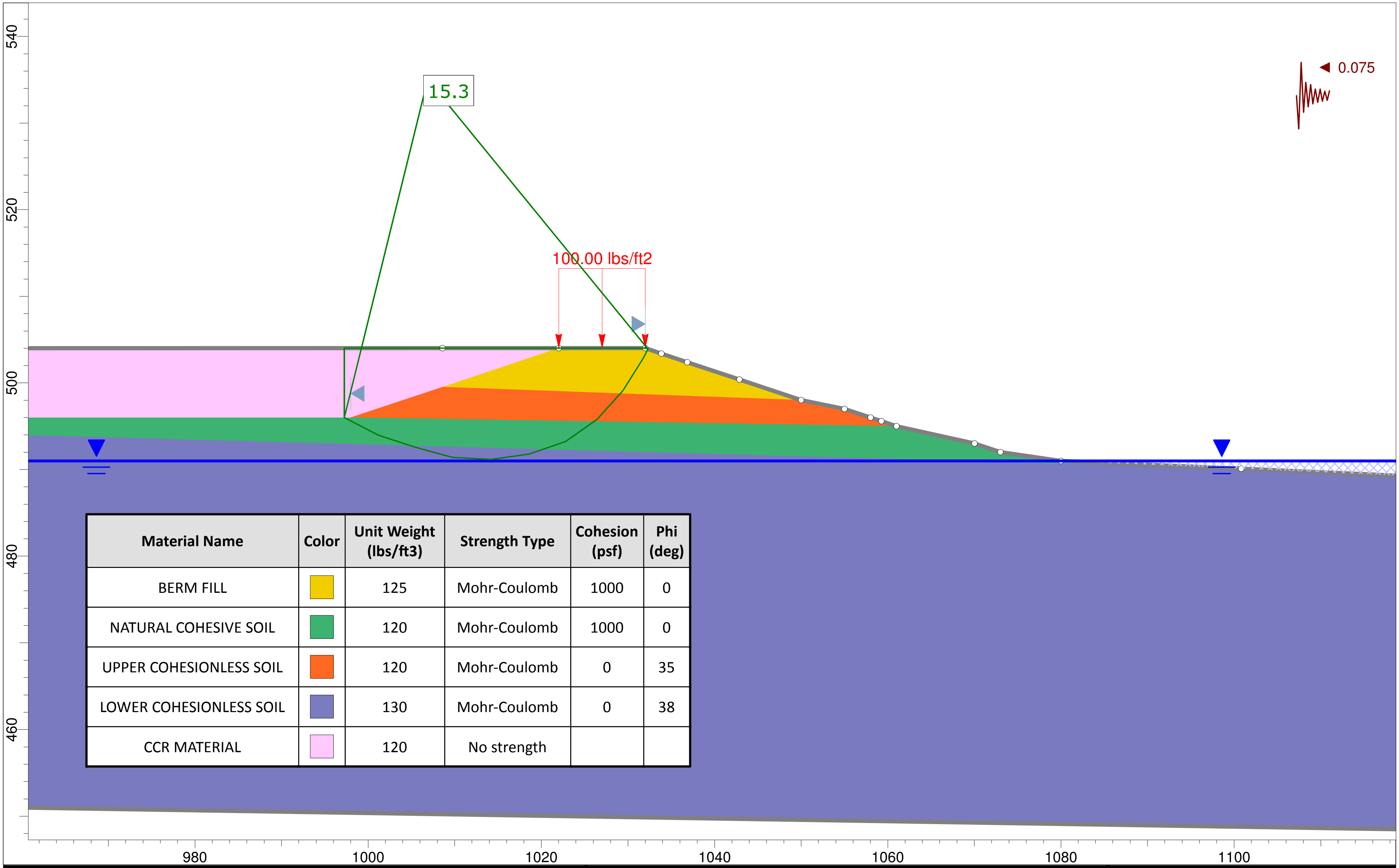

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
BERM FILL	🟨	125	Mohr-Coulomb	1000	0
NATURAL COHESIVE SOIL	🟩	120	Mohr-Coulomb	1000	0
UPPER COHESIONLESS SOIL	🟧	120	Mohr-Coulomb	0	35
LOWER COHESIONLESS SOIL	🟪	130	Mohr-Coulomb	0	38
CCR MATERIAL	◻	120	No strength		

ASA17-096-00
J.K. SPRUCE –CALAVERAS LAKE POWER PLANT
PROPOSED NEW COAL COMBUSTION RESIDUAL PONDS
SAN ANTONIO, TEXAS

GLOBAL STABILITY ANALYSIS
SECTION D-D' POND SIDE
SEISMIC CONDITION

FIGURE D-10

RABA KISTNER CONSULTANTS

GEOTECHNICAL ENGINEERING STUDY

FOR

CALAVERAS GEOTECHNICAL SURVEY
J.K. SPRUCE POWER PLANT
SAN ANTONIO, TEXAS

RABA
KISTNER

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San Antonio, TX 78249

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San Antonio, TX 78269

P 210.699.9090
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TBPE Firm F-3257

WWW.RKCI.COM

Project No. ASA20-044-00
September 24, 2020

Mr. Steven Dean, P.E., CFM
Pape-Dawson Engineers, Inc.
2000 NW Loop 410
San Antonio, Texas 78213

RE: **Geotechnical Engineering Study**
Calaveras Geotechnical Survey
J. K. Spruce Power Plant
San Antonio, Texas

Dear Mr. Dean:

RABA KISTNER, Inc. (RKI) is pleased to submit the report of our Geotechnical Engineering Study for the above-referenced project. This study was performed in accordance with RKI Proposal No. PSA20-089-00, dated July 15, 2020. The purpose of this study was to drill borings within or near the proposed structure footprints, to perform laboratory testing to classify and characterize subsurface conditions, and to prepare an engineering report presenting foundation design and construction recommendations for the proposed structures, as well as to provide pavement design and construction guidelines.

The following report contains our design recommendations and considerations based on our current understanding of the project information provided to us. There may be alternatives for value engineering of the foundation and pavement systems, and RKI recommends that a meeting be held with the Owner and design team to evaluate these alternatives.

We appreciate the opportunity to be of service to you on this project. Should you have any questions about the information presented in this report, or if we may be of additional assistance with value engineering or on the materials testing-quality control program during construction, please call.

Very truly yours,

RABA KISTNER, INC.

R. BLAKE WRIGHT
117753

Isaac Molina, P.E.
Project Engineer

R. Blake Wright, P.E.
Project Manager

RBW/IM/kv

Attachments

Copies Submitted: Above (Electronic)

GEOTECHNICAL ENGINEERING STUDY

For

CALAVERAS GEOTECHNICAL SURVEY
J. K. SPRUCE POWER PLANT
SAN ANTONIO, TEXAS

Prepared for

PAPE-DAWSON ENGINEERS, INC.
San Antonio, Texas

Prepared by

RABA KISTNER, INC.
San Antonio, Texas

PROJECT NO. ASA20-044-00

September 24, 2020

RABAKISTNER

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ATTACHMENTS

INTRODUCTION

RABA KISTNER, Inc. (RKI) has completed the authorized subsurface exploration for the proposed facility at the J.K. Spruce Power Plant adjacent to Calaveras Lake in San Antonio, Texas. This report briefly describes the procedures utilized during this study and presents our findings along with our recommendations for foundation design and construction considerations, as well as for pavement design and construction guidelines.

PROJECT DESCRIPTION

To be considered in this study is a new pond located at the J.K. Spruce Power Plant in San Antonio, Texas. The pond will be approximately 3 acres in total area, separated by a wall to form 2 ponds of approximately 1.5 acres each. The depth of the pond is not known at this time. The pond will include the following structures/elements:

- A concrete separator wall to divide the pond into two cells;
- A concrete sump;
- Slab-on-grade foundations for electrical equipment shelter (estimated load of 40,000 lbs) and a transformer (estimated load of 6,000 lbs);
- Two clarifiers with associated foundations and personnel access structures (estimated load of 150,000 lbs each); and
- New driveway pavements to access the pond and equipment.

LIMITATIONS

This engineering report has been prepared in accordance with accepted Geotechnical Engineering practices in the region of south/central Texas and for the use of the Pape-Dawson Engineers, Inc. (CLIENT) and its representatives for design purposes. This report may not contain sufficient information for purposes of other parties or other uses. This report is not intended for use in determining construction means and methods.

The recommendations submitted in this report are based on the data obtained from 13 borings drilled at this site, our understanding of the project information provided to us, and the assumption that site grading will result in only minor changes in the existing topography at the new structure locations. If the project information described in this report is incorrect, is altered, or if new information is available, we should be retained to review and modify our recommendations.

This report may not reflect the actual variations of the subsurface conditions across the site. This is particularly true of this site with respect to the variable depth of fill materials. The nature and extent of variations across the site may not become evident until construction commences. The construction process itself may also alter subsurface conditions. If variations appear evident at the time of construction, it may be necessary to reevaluate our recommendations after performing onsite observations and tests to establish the engineering impact of the variations.

The scope of our Geotechnical Engineering Study does not include an environmental assessment of the air, soil, rock, or water conditions either on or adjacent to the site. No environmental opinions are presented in this report.

If final grade elevations are significantly different from grades discussed herein (more than plus or minus 1 ft), our office should be informed about these changes. If needed and/or if desired, we will reexamine our analyses and make supplemental recommendations.

BORINGS AND LABORATORY TESTS

Subsurface conditions at the site were evaluated by 13 borings drilled at the locations shown on the Boring Location Map, Figure 1. These locations are approximate and distances were measured using tape, angles, pacing, etc. The recent borings were drilled to depths ranging from 10 to 50 ft below the existing ground surface using a truck-mounted drilling rig. During drilling operations split-spoon (with standard penetration test) and relatively undisturbed Shelby tube samples were collected at the depths annotated on our boring logs.

Each sample was visually classified in the laboratory by a member of our Geotechnical Engineering staff. The geotechnical engineering properties of the strata were evaluated by natural moisture content, Atterberg limits, direct shear (Figure 19), and sieve analysis tests.

The results of all laboratory tests are presented in graphical or numerical form on the boring logs illustrated on Figures 2 through 14. A key to classification terms and symbols used on the logs is presented on Figure 15. The results of the laboratory and field testing are also tabulated on Figure 16 for ease of reference. The results of the Dynamic Cone Penetrometer (DCP) tests are presented on Figure 17. Moisture-Density Relationship (Proctor) and California Bearing Ratio (CBR) test results are also presented on Figure 18.

Standard penetration test results are noted as “blows per ft” on the boring logs and Figure 16, where “blows per ft” refers to the number of blows by a falling hammer required for 1 ft of penetration into the soil/weak rock. Where hard or dense materials were encountered, the tests were terminated at 50 blows even if one foot of penetration had not been achieved. When all 50 blows fall within the first 6 in. (seating blows), refusal “ref” for 6 in. or less will be noted on the boring logs and on Figure 16.

Samples will be retained in our laboratory for 30 days after submittal of this report. Other arrangements may be provided at the request of the Client.

GENERAL SITE CONDITIONS

SITE DESCRIPTION

The project site is within the J.K. Spruce Power Plant adjacent to Calaveras Lake in San Antonio, Texas. Existing structures include buildings to the north and east, and pavements to the south and west. The site is currently grass covered. The topography generally slopes downward toward the east with vertical relief of about 5 ft across the site.

GEOLOGY

A review of the *Geologic* the *Atlas of Texas, Austin Sheet*, indicates that this site is naturally underlain with soils/rocks of the Wilcox Group, which is composed of mudstone with varying amounts of sandstone and lignite. The Wilcox Group may weather to yellowish-brown clay, sandy clay, and sands.

The Wilcox Group grades downward into the Midway Group, which is composed of clay, silt, and sand, with some pebbles near its base. Glauconite is often encountered in these soils. Key engineering considerations for development supported on the soils/rock of this formation typically include the presence of possible water-bearing layers, very hard mudstone/sandstone layers, and the expansive nature of the soil.

SEISMIC COEFFICIENTS

The following information has been summarized for seismic considerations associated with this site per ASCE 7-16 edition.

- Site Class Definition: **Class C**. Based on the soil borings conducted for this investigation and our experience in the area, the upper 100 ft of soil may be characterized as very dense soil and soft rock.
- Risk-Targeted Maximum Considered Earthquake Ground Motion Response Accelerations for the Conterminous United States of 0.2-Second Spectral Response Acceleration (5% Of Critical Damping): **S_s = 0.052g**.
- Risk-Targeted Maximum Considered Earthquake Ground Motion Response Accelerations for the Conterminous United States of 1-Second Spectral Response Acceleration (5% Of Critical Damping): **S_1 = 0.023g**.
- Values of Site Coefficient: **F_a = 1.3**
- Values of Site Coefficient: **F_v = 1.5**
- Where g is the acceleration due to gravity.

The Maximum Considered Earthquake Spectral Response Accelerations are as follows:

- 0.2 sec, adjusted: **S_{ms} = 0.068g**
- 1 sec, adjusted: **S_{m1} = 0.034g**

The Design Spectral Response Acceleration Parameters (SA) are as follows:

- 0.2 sec SA: **S_{DS} = 0.045g**
- 1 sec SA: **S_{D1} = 0.023g**

STRATIGRAPHY

Each stratum has been designated by grouping soils that possess similar physical and engineering characteristics. The boring logs should be consulted for more specific stratigraphic information. Unless noted on the boring logs, the lines designating the changes between various strata represent approximate boundaries. The transition between materials may be gradual or may occur between recovered samples. The stratification given on the boring logs, or described herein, is for use by RKI in its analyses and should not be used as the basis of design or construction cost estimates without realizing that there can be variation from that shown or described.

GROUNDWATER

During drilling, groundwater was encountered in some borings, as presented in the following table.

Boring Identifier	Approximate Observed Groundwater Elevation During Drilling (ft, msl)
B-4	490
B-5	484
B-9	483
B-10	482

It is possible for groundwater to exist beneath this site at shallow depths on a transient basis, particularly in granular stratums following periods of precipitation. Fluctuations in groundwater levels occur due to variation in rainfall and surface water run-off. The construction process itself may also cause variations in the groundwater level.

Based on the findings in our borings and on our experience in this region, we believe that groundwater seepage encountered during site earthwork activities and shallow foundation construction may be controlled using temporary earthen berm and conventional sump-and-pump dewatering methods. For excavations to depths greater than about 15 ft, provisions should be made to handle water entering excavations during construction. For deep foundation excavations, this could include the use of temporary casing to reduce groundwater seepage and sloughing of the in-situ soils.

FOUNDATION ANALYSIS

EXPANSIVE SOIL-RELATED MOVEMENTS

The anticipated ground movements due to swelling of the underlying soils at the site were estimated for slab-on-grade construction using the empirical procedure, Texas Department of Transportation (TxDOT) Tex-124-E, Method for Determining the Potential Vertical Rise (PVR). PVR values ranging from less than 1 to 2-1/4 in. were estimated for the stratigraphic conditions encountered in our borings. A surcharge load of 1 psi (concrete slab and sand cushion), an active zone of 10 to 15 ft, and dry moisture conditions were assumed in estimating the above PVR values.

The TxDOT method of estimating expansive soil-related movements is based on empirical correlations utilizing the measured plasticity indices and assuming typical seasonal fluctuations in moisture content. If desired, other methods of estimating expansive soil-related movements are available, such as estimations based on swell tests and/or soil-suction analyses. However, the performance of these tests and the detailed analysis of expansive soil-related movements were beyond the scope of the current study. It should also be noted that actual movements can exceed the calculated PVR values due to isolated changes in moisture content (such as due to leaks, landscape watering, etc.) or if water seeps into the soils to greater depths than the assumed active zone depth due to deep trenching or excavations.

Overexcavation and Select Fill Replacement

To reduce expansive soil-related movements in at-grade construction, a portion of the upper expansive subgrade soils can be removed by overexcavating and backfilling with a suitable select fill material. PVR values have been estimated for overexcavation and select fill replacement to various elevations below the existing ground surface and are summarized in the table below. Recommendations for the selection and placement of select backfill materials are addressed in a subsequent section of this report.

Transformers (Area of Borings B-4 and B-5)	
Overexcavation and Select Fill Replacement Elevation (ft, msl)*	**Estimated PVR (in.)**
513	Less Than 1

*We recommend that existing fill be remediated if fill depths extend below the overexcavation and select fill replacement depth.

Discharge Sump (Area of Boring B-6)	
Overexcavation and Select Fill Replacement Elevation (ft, msl)*	**Estimated PVR (in.)**
510	Less Than 1

*We recommend that existing fill be remediated if fill depths extend below the overexcavation and select fill replacement depth.

Separator Wall (Area of Borings B-7 and 8)	
Overexcavation and Select Fill Replacement Elevation (ft, msl)*	**Estimated PVR (in.)**
512	Less Than 1

*We recommend that existing fill be remediated if fill depths extend below the overexcavation and select fill replacement depth.

Clarifiers (Area of Borings B-9 and 10)	
Overexcavation and Select Fill Replacement Elevation (ft, msl)*	**Estimated PVR (in.)**
510	Less Than 1

*We recommend that existing fill be remediated if fill depths extend below the overexcavation and select fill replacement depth.

Drainage Considerations When overexcavation and select fill replacement is selected as a method to reduce the potential for expansive soil-related movements at any site, considerations of surface and subsurface drainage may be crucial to construction and adequate foundation performance of the soil-supported structures. Filling an excavation in relatively impervious plastic clays with relatively pervious select fill material creates a "bathtub" beneath the structure, which can result in ponding or trapped water within the fill unless good surface and subsurface drainage is provided.

Water entering the fill surface during construction or entering the fill exposed beyond the structure lines after construction may create problems with fill moisture control during compaction and increased access for moisture to the underlying expansive clays both during and after construction.

Several surface and subsurface drainage design features and construction precautions can be used to limit problems associated with fill moisture. These features and precautions may include but are not limited to the following:

- Installing berms or swales on the uphill side of the construction area to divert surface runoff away from the excavation/fill area during construction;
- Sloping of the top of the subgrade with a minimum downward slope of 1.5 percent out to the base of a dewatering trench located beyond the structure perimeter;
- Sloping the surface of the fill during construction to promote runoff of rain water to drainage features until the final lift is placed;
- Sloping of a final, well maintained, impervious clay or pavement surface (downward away from the structure) over the select fill material and any perimeter drain extending beyond the structure lines, with a minimum gradient of 6 in. in 5 ft;
- Constructing final surface drainage patterns to prevent ponding and limit surface water infiltration at and around the structure perimeter;
- Locating the water-bearing utilities, roof drainage outlets and irrigation spray heads outside of the select fill and perimeter drain boundaries; and
- Raising the elevation of the ground level floor slab.

Details relative to the extent and implementation of these considerations must be evaluated on a project-specific basis by all members of the project design team. Many variables that influence fill drainage considerations may depend on factors that are not fully developed in the early stages of design. For this reason, drainage of the fill should be given consideration at the earliest possible stages of the project.

FOUNDATION RECOMMENDATIONS

FOUNDATION CONSIDERATIONS

Review of the borings and test data indicate the factors discussed below will affect foundation design and construction at this site.

- Potential to encounter buried utilities and localized fills;
- Remediation of uncontrolled fills;
- Potential to encounter groundwater seepage;

- Sloughing of granular materials during excavation; and
- Potential for moderate-to-heavy foundation loads for the proposed improvements.

FOUNDATION OPTIONS

The following recommendations are based on the data obtained from our field and laboratory studies, our past experience with geotechnical conditions similar to those at this site, and our engineering design analyses.

The following alternatives are available to support the structures:

- Drilled, straight-shaft piers;
- Rigid-engineered beam and slab foundations;
- Shallow footing foundations.

The owner may select from these foundation systems depending on the performance criteria established for the structures. Cost analyses have not been conducted for any foundation system and are beyond the scope of this study.

SITE GRADING

A site plan with topographic information developed by AECOM and dated March 30, 2020, was used in our evaluation. We have prepared all foundation recommendations based on the provided site plan, and the stratigraphic conditions encountered at the time of our study. If site grading plans differ from those discussed in this report by more than plus or minus 1 ft, RKI must be retained to review the site grading plans prior to bidding the project for construction. This will enable RKI to provide input for any changes in our original recommendations that may be required as a result of site grading operations or other considerations.

EXISTING FILL

It should be noted that fill materials were encountered in 5 of 11 borings all within the top 1 ft of the existing ground surface. RKI is not aware of any documentation of the placement and compaction methods utilized in placement of the fill. With any undocumented fill material, there is a risk of potential settlement, the magnitude of which is not possible to predict without additional information.

The fill materials generally consisted of granular soils. Based on our observations, the existing fill materials are likely suitable for the support of the proposed structures. However, due to the apparent variability in the materials and in the comparative strength of the materials, some degree of isolated settlement should be anticipated for structures supported on the fill materials. It is not possible to accurately quantify the magnitude of potential settlement due to uncertainties regarding fill placement methods and control. Thus, there will be a degree of risk regarding the performance of structures supported on fill. **The only means by which this risk can be eliminated is through complete removal and recompaction of the existing fill materials.**

For shallow foundations or ground supported floor slabs, fill removal and recompaction or overexcavation and select fill replacement is recommended. The fill should be free of vegetation, root mass, organic topsoil, and particles larger than 4 in. Thus, excessive differential settlement-related risks associated with undocumented/uncontrolled fill will be reduced.

For other ancillary flatwork, such as sidewalks and pavements, these risks will remain in areas where existing fill is encountered. The only way to eliminate risk is to completely remove and recompact the existing fill materials, spoiling any oversized, organic, or otherwise deleterious and/or degradable materials. If this is not considered feasible, and settlement related risk in areas of flatwork is tolerable to the owner, consideration can be given to partial removal of the fill material. As a minimum, existing fill materials should be thoroughly proofrolled to identify weak or compressible zones in the near-surface material.

Based on the current information, the lateral extent of the fill materials is not known. Consideration may be given to additional exploration utilizing test pits to try and determine the lateral extent, the depth, and constituents of the existing fill materials.

DRILLED, STRAIGHT-SHAFT PIERS

Drilled, straight-shaft piers may also be considered to support the proposed structures using the values presented in the following tables. The provided values are based on a factor of safety of 2 for skin friction and 3 for end-bearing with respect to the design shear strength. These values may be increased by 1/3 for transient load conditions. Based on the 50-ft maximum depth of exploration, pier depths should not extend below an elevation of 465 ft msl.

Straight Shaft Pier Capacities – Transformers and Electrical Equipment Shelter		
Elevation* (ft, msl)	**Allowable Side Shear Resistance (ksf)**	**Allowable Axial End-Bearing (ksf)**
513 to 501	Neglect	3.4
501 to 465	1.0	12.4

*These recommendations should be reviewed if final foundation elevations differ from existing grade by more than +/- 1 ft.

Straight Shaft Pier Capacities - Clarifiers		
Elevation* (ft, msl)	**Allowable Side Shear Resistance (ksf)**	**Allowable Axial End-Bearing (ksf)**
511 to 496	Neglect	3.0
584 to 569	1.0	12.4

*These recommendations should be reviewed if final foundation elevations differ from existing grade by more than +/- 1 ft.

Final shaft depths will be based on interpretation of conditions in the field at the time of construction. Due to the variable conditions at this site, RKI must be present at the time of pier construction to verify the field conditions are similar to those assumed in the preparation of our recommendations. For bid purposes, the owner should anticipate that deeper piers will be required in some areas. Consequently, contractors bidding on the job should include unit costs for various depths of additional pier embedment. Unit costs should include those for both greater and lesser depth in both bedrock (i.e. sandstone) and soil.

Allowable Uplift Resistance

Resistance to uplift forces exerted on the drilled, straight-shaft piers will be provided by the sustained compressive axial force (dead load) plus the allowable uplift resistance provided by the soil. The resistance provided by the soil depends on the shear strength of the soils adjacent to the pier shaft and below the depth of the active zone. The allowable uplift resistance provided by the soils at this site may be estimated using 2/3 of the axial compressive side shear resistance provided in the *Straight Shaft Pier Capacity* tables. These values were evaluated using a factor of safety of 2.

Reinforcing steel will be required in each pier shaft to withstand a net force equal to the uplift force minus the sustained compressive load carried by that pier. We recommend that each pier be reinforced to withstand this net force or an amount equal to 1 percent of the cross-sectional area of the shaft, whichever is greater.

PIER SHAFTS

The pier shafts will be subject to potential uplift forces if the surrounding expansive soils within the active zone are subjected to alternate drying and wetting conditions. The maximum potential uplift force acting on the shaft may be estimated by:

$$F_u = 22*D$$

where:

F_u = uplift force in kips; and
D = diameter of the shaft in feet.

PIER SPACING

Where possible, we recommend that the piers be spaced at a center to center distance of at least three shaft diameters on-center for straight-shaft piers. Such spacing will not require a reduction in the load carrying capacity of the individual piers.

If design and/or construction restraints require that piers be spaced closer than the recommended three shaft diameters, RKI must re-evaluate the allowable bearing capacities presented above for the individual piers. Reductions in load carrying capacities may be required depending upon individual loading and spacing conditions.

FLOOR SLABS

Two alternatives are available to construct the floor slab systems for drilled pier foundations if chosen for the transformer and clarifier structures. The Owner may select the alternative best satisfying the required performance criteria.

> **Alternative No. 1:** Floor slabs which have high performance criteria or which are movement sensitive in nature, should be structurally suspended because of the anticipated ground movements. A positive void space of at least 4 in., preferably more, should be provided between the slab and the underlying soils (see also *Void Space Considerations*).
>
> **Alternative No. 2:** Floor slabs within the superstructure may be ground supported provided the anticipated movements discussed under the *Expansive Soil-Related Movements* section of this report will not impair the performance of the floor, frame, or roof systems.
>
> If differential movements between the slab and the structure are objectionable, soil-supported floor slabs could be dowelled to the perimeter grade beams. Dowelled slabs that are subjected to heaving will typically crack and develop a plastic hinge along a line which will be approximately 5 to 10 ft inside and parallel to the grade beams. Slabs cast independent of the grade beams, interior columns and partitions should experience minimum cracking, but may create difficulties at critical entry points such as doors and may impact interior partitions that are secured to exterior walls.
>
> We recommend that a vapor barrier comprised of polyethylene or polyvinyl chloride (PVC) sheeting be placed between the supporting select fill and the concrete floor slab.

GRADE BEAMS

For a deep foundation system, if chosen, we recommend that the grade beams interconnecting the piers be structurally suspended. A positive void space of at least 4 in., preferably more, should be provided between the soffits of grade beams and the underlying soils.

RIGID-ENGINEERED BEAM AND SLAB FOUNDATIONS

Rigid-engineered beam and slab foundations may be utilized for proposed structures, provided the selected foundation type can be designed to withstand the anticipated soil-related movements (see *Expansive Soil-Related Movements* and *Existing Fill*) without impairing either the structural or the operational performance of the structures. If a shallow foundation system is to be considered, we recommend that the existing fill be remediated and that the PVR reduction be utilized to reduce expansive soil-related movements.

Allowable Bearing Capacity

Shallow foundations founded on compacted native soil or select fill should be proportioned using the design parameters presented in the following table.

Minimum depth below final grade	18 in.
Minimum beam width	12 in.
Maximum allowable bearing pressure for grade beams	1,900 psf
Maximum allowable bearing pressure for widened beams	2,400 psf

The above presented maximum allowable bearing pressures will provide a factor of safety of about 3 with respect to the measured shear strength, provided that select fill is selected and placed as recommended in the *Select Fill* section of this report and the subgrade is prepared in accordance with the recommendations outlined in the *Site Preparation* section of this report.

BRAB Criteria

Beam and slab foundations are sometimes designed using criteria developed by the Building Research Advisory Board (BRAB). The recommended value for the Climatic Rating (C_w) for the project location is 16.

It should be noted that if the highest plasticity index (PI) value encountered in the subsurface profile occurs in the uppermost subsurface layer, BRAB criteria requires that this PI value be selected as the design PI. Such a standard design PI calculation/selection method does not allow the designer to account for the reduced expansion potential of a relatively thin, surficial clay veneer overlying a shallow less expansive formation. The BRAB design plasticity index, soil support index (C), and estimated unconfined compressive strength (q_u) presented in the following table may be utilized for the proposed structures. These design parameters apply for conditions encountered in our borings and for the grades existing at the time of our field exploration.

BRAB Criteria for Existing Site Conditions

		Parameters		
Improvement	**Associated Borings**	**Estimated Soil Unconfined Compressive Strength (q_u)**	**BRAB Design Plasticity Index**	**Soil Support Index (C)**
Transformers, Electrical Equipment Shelter, and Clarifiers	B-4, B-5, B-9, and B-10	2,000 psf	20	0.94

The design criteria will change if a select fill building pad is constructed for the proposed structures. If site grading operations alter the thickness of the on-site soil beneath the residence, then the criteria for the residence should be re-evaluated for the appropriate slab design parameters. If any overexcavation and select fill replacement is performed, then RKI must be retained to revise our original recommendations that may be required as a result.

AREA FLATWORK

It should be noted that ground-supported flatwork such as walkways, courtyards, etc. will be subject to the same magnitude of potential soil-related movements as discussed previously (see *Expansive Soil-Related Movement* and *Existing Fill* sections). Thus, where these types of elements abut rigid structure foundations or isolated/suspended structures, differential movements should be anticipated. As a minimum, we recommend that flexible joints be provided where such elements abut the main structure to allow for differential movement at these locations. Where the potential for differential movement is objectionable, it may be beneficial to consider methods of reducing anticipated movements or to consider structurally suspending critical areas to match the adjacent structure performance.

PERMANENT SLOPES

The stability of permanent slopes depends on many factors, including the height and geometry of the slopes, the types of materials contained in the slopes, effects of groundwater, and any surface pressures present. In general, permanent cut and fill slopes, constructed at 3H:1V (3 horizontal to 1 vertical) have been observed to perform satisfactorily. Therefore, it is our opinion that slopes should be constructed at 3H:1V or flatter. Fill slopes should be constructed by extending the compacted fill beyond the planned profile of the slope and then trimming the slope to the desired configuration.

Cut slopes can be designed similar to fill slopes. However, the potential for sloughing and/or general slope failure increases with an increase in the steepness and depth of cut, particularly if low strength soil occurs in or near the base of the slope.

If steeper slopes are anticipated, global stability analysis of proposed slopes should be evaluated. Depending on the acceptable factor of safety for stability for long-term condition, steeper slopes may need to be reinforced to increase stability (such as tiebacks, helical anchors, deadmen, soil nails, or other reinforcement systems).

RETAINING STRUCTURES

Retaining walls may be required to accommodate potential grade changes near the pond areas. The following sections provide general information for evaluating lateral earth pressures, backfill compaction, drainage, and the footings for the retaining walls, if any.

Global stability analyses have not been performed. If required by the City of San Antonio Information Bulletin 171, RKI should be retained to evaluate the global stability of the proposed retaining walls and proposed slopes. A global stability analysis for any system requires details regarding the wall/slope type, backfill, surcharge loading, and the specific site topography at the section location. When this information is available, RKI can be retained to perform the global stability analysis. However, the internal stability of the proposed retaining wall(s) should be checked by the wall designer. The general recommendations provided herein may require modification once additional information becomes available.

LATERAL EARTH PRESSURES

Equivalent fluid density values for computation of lateral soil pressures acting on walls were evaluated for various types of backfill materials that may be placed behind the walls. These values, as well as corresponding lateral earth pressure coefficients and estimated unit weights, are presented in the following.

Back Fill Type	Estimated Total Unit Weight (pcf)	Active Condition		At-Rest Condition	
		Earth Pressure Coefficient, k_a	Equivalent Fluid Density (pcf)	Earth Pressure Coefficient, k_o	Equivalent Fluid Density (pcf)
Washed Gravel	135	0.29	40	0.45	60
Crushed Limestone	145	0.24	35	0.38	55
Clean Sand	120	0.33	40	0.5	60
Pit Run Clayey Gravels or Sands	135	0.32	45	0.48	65
Inorganic Clays of Low to Medium Plasticity (Liquid Limit less than 40 percent)	120	0.40	50	0.55	65
Onsite Soil	120	0.59	70	0.74	90

The values tabulated above under "Active Conditions" pertain to flexible retaining walls free to tilt outward as a result of lateral earth pressures. For rigid, non-yielding walls the values under "At-Rest Conditions" should be used.

The "At-Rest" condition is present when the wall is not allowed to move. Once the wall moves outward a short distance, it relieves part of the horizontal stress. The horizontal movement required to reach the active condition may be estimated by using 0.01*H (where H is the wall height). For example, for a 10 ft. tall wall, horizontal movements up to 1.2 inches may be required to develop the active condition. Once the soil attains the active condition, the horizontal stress in the soil (and thus the pressure acting on the wall) will be reduced. Features/structures directly behind the wall may experience settlements similar to the horizontal movements. Where these types of movements are objectionable, the retaining wall should be designed using At-Rest Conditions.

For the provided values to be valid for sand or gravel backfill, the backfill should be placed in a wedge extending upward and away from the edge of the wall at a 45-degree angle or flatter. If sand and gravel are to be placed within a steeper wedge, the values for Pit Run Gravels/Sands, or Inorganic Clays provided above should be used. Further, any soft soil on the excavation slope should be removed prior to placement of backfill.

The values presented above assume the surface of the backfill materials to be level. Sloping the surface of the backfill materials will increase the surcharge load acting on the structures. The above values also do not include the effect of surcharge loads such as loading from construction equipment, vehicular loads (such as 250 psf), future storage near the structures or other loading/surcharge conditions. Nor do the values account for possible hydrostatic pressures resulting from groundwater seepage entering and ponding within the backfill materials. However, these surcharge loads and groundwater pressures should be considered in designing any structures subjected to lateral earth pressures.

The use of expansive clay soils as backfill against the proposed retaining structures is not recommended. Expansive soils generally provide higher design active earthen pressures, as indicated above, but may also exert additional active pressures associated with swelling. Controlling the moisture and density of these materials during placement will help reduce the likelihood and magnitude of future active pressures due to swelling, but this is no guarantee.

Wall Backfill Compaction

Placement and compaction of backfill behind the walls will be critical, particularly at locations where backfill will support adjacent near-grade foundations and/or flatwork. If the backfill is not properly compacted in these areas, the adjacent foundations/flatwork can be subject to settlement.

To reduce potential settlement of adjacent foundations/flatwork, the backfill materials should be placed and compacted as recommended in the *Select Fill* section of this report. Each lift or layer of the backfill should be tested during the backfilling operations to document the degree of compaction. Within at least a 5-ft zone of the wall backside, we recommend that compaction be accomplished using hand-guided compaction equipment capable of achieving the maximum density in a series of 3 to 5 passes. Thinner lifts may be required to achieve compaction.

Drainage

The use of drainage systems is a positive design step toward reducing the possibility of hydrostatic pressure acting against the retaining structures. Drainage may be provided by the use of a drain trench and pipe. The drain pipe should consist of a slotted, heavy duty, corrugated polyethylene pipe and should be installed and bedded according to the manufacturer’s recommendations. The drain trench should be filled with gravel (meeting the requirements of ASTM D 448 coarse concrete aggregate Size No. 57 or 67) and extend from the base of the structure to within 2 ft of the top of the structure. The bottom of the drain trench will provide an envelope of gravel around the pipe with minimum dimensions consistent with the pipe manufacturer’s recommendations. The gravel should be wrapped with a suitable geotextile fabric (such as Mirafi 140N or equivalent) to help minimize the intrusion of fine-grained soil particles into the drain system. The pipe should be sloped and equipped with clean-out access fittings consistent with state-of-the-practice plumbing procedures.

As an alternative to a full-height gravel drain trench behind the proposed retaining structures, consideration may be given to utilizing a manufactured geosynthetic material for wall drainage. A number of products are available to control hydrostatic pressures acting on earth retaining structures, including Amerdrain (manufactured by American Wick Drain Corp.), Miradrain (manufactured by Mirafi, Inc.), Enkadrain (manufactured by American Enka Company), and Geotech Insulated Drainage Panel (manufactured by Geotech Systems Corp.). The geosynthetics are placed directly against the retaining structures and are hydraulically connected to the gravel envelope located at the base of the structures.

Weepholes may be considered along the length of the proposed basement structures, if desired, in addition to one of the two alternative drainage measures presented above. Based on our experience, weepholes, as the only drainage measure, often become clogged with time and do not provide the required level of drainage from behind retaining structures.

Retaining Wall Foundations

Footings may be designed using the parameters provided in the section titled *Allowable Bearing Capacity.* To reduce the potential for differential settlement, we recommend extending the retaining wall foundations as may be necessary to bear on similar foundation materials along the length of any walls.

EXCAVATION SLOPING AND BENCHING

If utility trenches or other excavations extend to or below a depth of 5 ft below construction grade, the contractor or others shall be required to develop a trench safety plan to protect personnel entering the trench or trench vicinity. The collection of specific geotechnical data and the development of such a plan, which could include designs for sloping and benching or various types of temporary shoring, are beyond the scope of the current study. Any such designs and safety plans shall be developed in accordance with current OSHA guidelines and other applicable industry standards.

FOUNDATION CONSTRUCTION CONSIDERATIONS

SITE DRAINAGE

Drainage is an important key to the successful performance of any foundation. Good surface drainage should be established prior to and maintained after construction to help prevent water from ponding within or adjacent to the structure foundations and to facilitate rapid drainage away from the foundations. Failure to provide positive drainage away from the structure can result in localized differential vertical movements in soil supported foundations and floor slabs, which can in turn result in cracking in the sheetrock partition walls, and shifting of ceiling tiles, as well as improper operation of windows and doors.

Current ordinances, in compliance with the Americans with Disabilities Act (ADA), may dictate maximum slopes for walks and drives around and into new buildings. These slope requirements can result in drainage problems for buildings supported on expansive soils. We recommend that, on all sides of the building, the maximum permissible slope be provided away from the building.

Also to help control drainage in the vicinity of the structures, we recommend that roof/gutter downspouts and landscaping irrigation systems not be located adjacent to the foundation. Where a select fill overbuild is provided outside of the floor slab/foundation footprint, the surface should be sealed with an impermeable layer (pavement or clay cap) to reduce infiltration of both irrigation and surface waters. Careful consideration should also be given to the location of water bearing utilities, as well as to provisions for drainage in the event of leaks in water bearing utilities. All leaks should be immediately repaired.

Other drainage and subsurface drainage issues are discussed in the *Expansive Soil-Related Movements* section of this report and under *Pavement Construction Considerations*.

Furthermore, as discussed in a previous section of this report, it has been our past experience that shallow groundwater seepage may be encountered within the existing or remediated fill at the project site or within granular stratums. We recommend that any drainage related issues be thoroughly addressed by the design team.

SITE PREPARATION

Site preparation for this project will include removal of old foundation systems and utilities, if any. The requirements for specific areas will depend on the depth, size and loading of the facilities that must be constructed following any demolition activities. These activities and operations should be carefully considered and monitored to make sure that old foundation elements and abandoned utility lines do not result in post construction maintenance issues, problems, or allow influx of groundwater seepage.

Structure areas and all areas to support select fill should be stripped of all vegetation, root mass, organic topsoil, pavement section, utilities, structures, and associated backfill. Existing utilities and associated backfill, extending into excavations, be plugged/capped to reduce the potential for groundwater influx. We recommend all existing fill under proposed structures be remediated. Partial remediation under pavements may be considered, see *Existing Fill*. Furthermore, as discussed in a previous section of this report, we recommend that one of the PVR reduction options be utilized to reduce expansive soil-related movements to within acceptable structural and operational tolerances, or structurally suspended.

Exposed subgrades should be thoroughly proofrolled in order to locate weak, compressible zones. A fully-loaded tandem wheeled dump truck or a similar heavily-loaded piece of construction equipment should be used for planning purposes. Proofrolling operations should be observed by the Geotechnical Engineer or their representative to document subgrade condition and preparation. Weak or soft areas identified during proofrolling should be removed and replaced with suitable, compacted on-site clays, free of organics, oversized materials, and degradable or deleterious materials.

Upon completion of the proofrolling operations and just prior to fill placement or slab construction, the exposed subgrade should be moisture conditioned by scarifying to a minimum depth of 6 in. and recompacting to a minimum of 95 percent of the maximum density as determined by TxDOT Test Method TEX-114-E or ASTM D698. The moisture content of the subgrade should be maintained within the range of optimum moisture content to 3 percentage points above optimum moisture content until permanently covered.

ONSITE SOIL AND FILL

The use of onsite expansive soils may be a considered for general fill (outside of the structure footprints), if the potential vertical movements in excess of those discussed previously will not adversely impact either the structural or operational tolerances for the proposed improvements for which this material is being considered.

If existing soil and/or fill can be processed in order to meet the select fill requirements, then consideration can be given to using the material onsite as select fill.

SELECT FILL

Recommendations for preferred select fill materials are provided below.

Imported Crushed Limestone Base – Imported crushed limestone base materials should be crushed stone or gravel aggregate. We recommend that materials specified for use as select fill meet the TxDOT 2014 Standard Specifications for Construction and Maintenance of Highways, Streets and Bridges, Item 247, Flexible Base, Type A or B, Grades 1-2 or 3.

Recycled Materials – Recycled materials (i.e. concrete) are a viable alternative to crushed limestone to be used as fill, provided the recycled material is determined to be environmentally acceptable. We recommend that the recycled concrete material meet the requirements of TxDOT Item 247, Paragraph 2.13.2.1. prior to hauling to the site.

Recycled material may be used as fill if deleterious materials can be separated (i.e. rebar, soil, wood, metal, plastic, piping, conduit, etc). Oversized rubble should be processed to a well-graded material similar to the *Imported Crushed Limestone Base* with a maximum particle size of 4 inches. Rubble larger than 4 inches in any dimension should be discarded or processed to the maximum dimension. Care should be taken when placing the fill that the larger pieces are not concentrated in a manner such that voids develop between nested pieces; a sufficient quantity of fines should be provided to reduce this risk.

Recommendations for alternative select fill materials are provided below.

Granular Pit Run Materials – Granular pit run materials should consist of GC, SC & combination soils (clayey gravels), as classified according to the Unified Soil Classification System (USCS). Alternative select fill materials shall have a maximum liquid limit not exceeding 40, a plasticity index between 7 and 20, and a maximum particle size not exceeding 4 inch. In addition, if these materials are utilized, grain size analyses and Atterberg Limits must be performed during placement at a rate of one test each per 5,000 cubic yards of material due to the high degree of variability associated with pit-run materials.

Low PI Materials – Low PI materials should consist of CL clays, as classified according to the Unified Soil Classification System (USCS). Alternative select fill materials shall have a maximum liquid limit not exceeding 40, a plasticity index between 7 and 20, and a maximum particle size not exceeding 4 inch. In addition, if these materials are utilized, grain size analyses and Atterberg Limits must be performed during placement at a rate of one test each per 5,000 cubic yards of material due to the high degree of variability associated with these materials.

If the above-listed materials or alternative select fills are being considered for bidding purposes, the materials should be submitted to the Geotechnical Engineer for evaluation at a minimum of 10 working days or more prior to the bid date. Failure to do so will be the responsibility of the contractor. The contractor will also be responsible for ensuring that the properties of all delivered alternate select fill materials are similar to those of the pre-approved submittal. **It should also be noted that when using alternative fill materials such as *Granular Pit Run or Low PI Materials*, difficulties may be experienced with respect to moisture control during and subsequent to fill placement, as well as with erosion, particularly when exposed to inclement weather. This may result in sloughing of beam trenches and/or pumping of the fill materials.**

Granular Pit Run or Low PI Materials will be very susceptible to small changes in moisture content and to disturbance from foot traffic during the placement of steel reinforcement in beam trenches, particularly in periods of inclement weather. Disturbance from such foot traffic and from the accumulation of excess water can result in losses in bearing capacity and increased settlement. If inclement weather is anticipated at the time construction, consideration should be given to protecting the bottom of foundation excavations by placing a thin mud mat (layer of flowable fill or lean concrete) at the bottom of trenches immediately following excavation. This will reduce disturbance from foot traffic and will impede the infiltration of surface water. The side slopes of beam trench excavations may also need to be flattened to reduce sloughing in cohesionless soils. All necessary precautions should be implemented to protect open excavations from the accumulation of surface water runoff and rain.

Soils classified as CH, MH, ML, SM, GM, OH, OL and Pt under the USCS are not considered suitable for use as select fill materials at this site.

Select Fill Placement and Compaction

It is recommended that select fill be placed in loose lifts not exceeding 8 in. in thickness and compacted to at least 98 percent of maximum density as determined by ASTM D698. The moisture content of the fill should be maintained within the range of 2 percentage points below to 2 percentage points above the optimum moisture content until final compaction. For low PI and granular pit-run materials, the moisture content of the fill should be maintained within the range of optimum to plus 3 percentage points above the optimum moisture content until final compaction.

General Fill Placement and Compaction

The remaining fill (such as parking lot areas or green spaces) may be compacted to at least 95 percent of maximum dry density as determined by ASTM D698. The moisture content of the fill should be maintained within the range of optimum to plus 3 percentage points above the optimum moisture content until final compaction.

SHALLOW FOUNDATION EXCAVATIONS

Shallow foundation excavations should be observed by the Geotechnical Engineer or their representative prior to placement of reinforcing steel and concrete. This is necessary to verify that the bearing soils at the bottom of the excavations are similar to those encountered in our borings and that excessive loose materials and water are not present in the excavations. If soft pockets of soil are encountered in the foundation excavations, they should be removed and replaced with a compacted non-expansive fill material or lean concrete up to the design foundation bearing elevations.

It should also be noted that the some of the soils at this site are gravelly/sandy and cohesionless in nature; consequently, these soils will be very susceptible to small changes in moisture content and to disturbance from foot traffic during the placement of steel reinforcement in beam trenches, particularly in periods of inclement weather. Disturbance from such foot traffic and from the accumulation of excess water can result in losses in bearing capacity and increased settlement. If inclement weather is anticipated at the time construction, consideration should be given to protecting the bottoms of beam trenches by placing a thin mud mat (layer of flowable fill or lean concrete) at the bottom of trenches immediately following excavation. This will reduce disturbance from foot traffic and will impede the infiltration of surface water. The side slopes of beam trench excavations may also need to be flattened to reduce sloughing in cohesionless soils. All necessary precautions should be implemented to protect open excavations from the accumulation of surface water runoff and rain.

DRILLED PIERS

Each drilled pier excavation must be examined by an RKI representative who is familiar with the geotechnical aspects of the soil stratigraphy, the structural configuration, foundation design details and assumptions, prior to placing concrete. This is to observe that:

- The shaft has been excavated to the specified dimensions at the correct depth established by the previously mentioned criteria;
- The shaft has been drilled plumb within specified tolerances along its total length; and
- Excessive cuttings, buildup and soft, compressible materials have been removed from the bottom of the excavation.

Due to the presence of high blow count materials including, but not limited to, sandstone, high-powered, high-torque drilling equipment should be anticipated for drilled pier construction at this site (see also *Excavation Equipment*).

Reinforcement and Concrete Placement

Reinforcing steel should be checked for size and placement prior to concrete placement. Placement of concrete should be accomplished as soon as possible after excavation to reduce changes in the moisture content or the state of stress of the foundation materials. No foundation element should be left open overnight without concreting.

Temporary Casing

Groundwater seepage was observed in the test borings at elevations ranging from 482 to 490 ft at the time of our subsurface exploration. Groundwater seepage and/or side sloughing is likely to be encountered at the time of construction, depending on climatic conditions prevalent at the time of construction. Therefore, we recommend that the bid documents require the foundation contractor to specify unit costs for different lengths of casing that may be required.

EXCAVATION SLOPING AND BENCHING

If utility trenches or other excavations extend to or below a depth of 5 ft below construction grade, the contractor or others shall be required to develop a trench safety plan to protect personnel entering the trench or trench vicinity. The collection of specific geotechnical data and the development of such a plan, which could include designs for sloping and benching or various types of temporary shoring, are beyond the scope of the current study. Any such designs and safety plans shall be developed in accordance with current OSHA guidelines and other applicable industry standards.

To assist in preparing an excavation safety plan, we have classified the soils encountered at this site based on the data collected during this study. The natural soils encountered at this site are classified as Type C soils under current Occupational Safety and Health Administration (OSHA) regulations pertaining to excavations. In excavations penetrating these soils, the sloping and benching schemes specified for Type C soils under the OSHA regulations require that the excavation sidewalls be sloped no steeper than 1.5:1 (horizontal:vertical).

EXCAVATION EQUIPMENT

Our boring logs are not intended for use in determining construction means and methods and may therefore be misleading if used for that purpose. We recommend that earth-work and utility contractors interested in bidding on the work perform their own tests in the form of test pits to determine the quantities of the different materials to be excavated, as well as the preferred excavation methods and equipment for this site.

VOID SPACE CONSIDERATIONS

If the structurally suspended floor system described as Alternative No. 1 under the *Floor Slab* section of this report is selected, several special design issues should be considered for the resulting subfloor void space. These issues are discussed below.

Ventilation

Observations by members of our firm of open crawl spaces have indicated a need for adequate subfloor ventilation. Such ventilation helps promote evaporation of subgrade moisture which may accumulate in spite of special surface and subsurface drainage features. As a minimum, free flowing passive vents may need to be installed along the perimeter beam to provide cross ventilation. If structural configurations will limit the free flow of air through passive vents, forced air, power vents should be installed. All vents should be designed such that they will not allow the drainage of surface water into the void space.

A minimum clearance of 4 in. has been recommended between both the grade beams and floor slab and the underlying finished subgrade. Such a minimum clearance is also recommended between the subgrade and any utilities which may be suspended from the underside of the floor. This clearance will allow swell-related subgrade movements without damaging the utilities. It is recommended that the utility clearance not be provided by the addition of narrow trenches running parallel to and immediately below the utilities, unless proper slopes and drainage outlets are provided to prevent ponding of water in the trenches.

Drainage

As discussed throughout this report, positive drainage is a key factor in the long term performance of any foundation. This is not only critical around the perimeter of the structure, but also in any subfloor void spaces. Surface drainage should be established that will direct water away from and will prevent water from ponding adjacent to piers. This positive drainage should be maintained both prior to and after construction.

Compaction control of the backfill around the perimeter of the structure following the placement of soil retainer blocks is critical to the drainage away from the foundation following construction. Materials for the backfill around the perimeter of the structure should be the onsite soils. These materials should be compacted in uniformly thin lifts (8-inch maximum loose thickness) to at least 90 percent of the maximum dry density as determined by ASTM D698. These soils should be placed and compacted at optimum to plus 3 percent above optimum moisture content. Compaction by hand operated mechanical tampers will help to avoid damage to the soil retainer blocks. Following backfilling operations the soil retainer blocks should be checked to see that they have not been broken or collapsed during the compaction operations. Any soil retainer blocks that are broken or collapsed should be repaired or replaced.

Carton Forms

When carton forms are used to form subfloor void spaces, the forms often get wet or sometimes absorb water from humid air. This can result in collapse of the forms during the placement of concrete, thus diminishing the design void space. Conversely, if the carton forms are too strong and do not decompose sufficiently with time, they may not collapse as soil heave occurs, resulting in heave damage to the floor slab. Where there is sufficient moisture to cause the appropriate deterioration after construction, there may be a resulting moisture problem in the floor slab as a result of poor ventilation and the accumulation of condensation within the resulting unventilated void space. The lack of ventilation may also result in increased soil movements that will diminish the design void space. For these reasons, we recommend that where possible, consideration be given to methods other than the use of carton forms to form the recommended void space beneath floor slabs. If project specifics require the use of carton forms, then as a minimum, care should be taken to ensure that the carton forms are designed for use in the project location, and that carton forms are properly stored, protected, and installed during construction.

INTERIOR WALLS

It is not uncommon for cracking to occur in interior partition walls that are supported by a “floating” floor slab and structurally tied to either an interior column or an exterior wall supported by deep foundations. This should be taken into account during the design phase of the project if a “floating” slab foundation is used to support the proposed structure.

UTILITIES

Utilities which project through slab-on-grade, slab-on-fill, “floating” floor slabs, or any other rigid unit should be designed with either some degree of flexibility or with sleeves. Such design features will help reduce the risk of damage to the utility lines as vertical movements occur. These types of slabs will generally be constructed as monolithic, grid type beam and slab foundations or as a “floating” floor slab described as Alternative No. 2 under the *Floor Slab* section of this report.

Our experience indicates that significant settlement of backfill can occur in utility trenches, particularly when trenches are deep, when backfill materials are placed in thick lifts with insufficient compaction, and when water can access and infiltrate the trench backfill materials. The potential for water to access the backfill is increased where water can infiltrate flexible base materials due to insufficient penetration of curbs, and at sites where geological features can influence water migration into utility trenches. It is our belief that another factor which can significantly impact settlement is the migration of fines within the backfill into the open voids in the underlying free-draining bedding material.

To reduce the potential for settlement in utility trenches, we recommend that consideration be given to the following:

- All backfill materials should be placed and compacted in controlled lifts appropriate for the type of backfill and the type of compaction equipment being utilized and all backfilling procedures should be tested and documented.
- Curbs should completely penetrate base materials and be installed to a sufficient depth to reduce water infiltration beneath the curbs into the pavement base materials.
- Consideration should be given to wrapping free-draining bedding gravels with a geotextile fabric (similar to Mirafi 140N) to reduce the infiltration and loss of fines from backfill material into the interstitial voids in bedding materials.

PAVEMENT RECOMMENDATIONS

Recommendations for both flexible and rigid pavements are presented in this report. The Owner and/or design team may select either pavement type depending on the performance criteria established for the project. In general, flexible pavement systems have a lower initial construction cost as compared to rigid pavements. However, maintenance requirements over the life of the pavement are typically much greater for flexible pavements. This typically requires regularly scheduled observation and repair, as well as overlays and/or other pavement rehabilitation at approximately one-half to two-thirds of the design life. Rigid pavements are generally more "forgiving", and therefore tend to be more durable and require less maintenance after construction.

For either pavement type, drainage conditions will have a significant impact on long term performance, particularly where permeable base materials are utilized in the pavement section. Drainage considerations are discussed in more detail in a subsequent section of this report.

SUBGRADE CONDITIONS

We have assumed the subgrade in pavement areas will consist of recompacted onsite soils or fill, placed and compacted as recommended in the *Select Fill* section of this report. Based on laboratory California Bearing Ratio (CBR) test results, DCP results, and our experience with similar subgrade soils, we have assigned a design CBR value of 5 for use in pavement thickness design analyses.

DESIGN INFORMATION

The pavement section recommendations were prepared using the 1993 “Guide for the Design of Pavement Structures” by the American Association of State Highway and Transportation Officials (AASHTO). We have based our analysis on the following design parameters. **The Project Civil Engineer should review anticipated traffic loading and frequencies to verify that the assumed traffic loading and frequency is appropriate for the intended use of the facility.**

Pavement Design Parameters	Flexible Pavement	Rigid Pavement
Performance Period	20 years	
Design Traffic, 18-kip Equivalent Standard Axle Loads (ESALs) Light Duty Heavy Duty	 85,000(1) 292,400(2)	 77,500(3) 209,300(4)
California Bearing Ratio (CBR)	5.0(5)	
Initial Serviceability Index	4.2	4.5
Terminal Serviceability Index	2.0	
Overall Standard Deviation	0.45	0.35
Reliability	70	
Modulus of Subgrade reaction (k-value)	-	100 pci
28-day Concrete Modulus of Rupture	-	550 psi
28-day Concrete Elastic Modulus	-	4,000,000 psi
Load Transfer Coefficient	-	4.2
Drainage Coefficient	-	1.0
Roadbed Soil Resilient Modulus	7,500 psi	-

(1)Approximately equivalent to 4 tractor-trailer trucks per day.
(2)Approximately equivalent to 16 tractor-trailer trucks per day.
(3)Approximately equivalent to 2 tractor-trailer trucks per day.
(4)Approximately equivalent to 7 tractor-trailer trucks per day.
(5)The CBR was assigned based on our laboratory CBR test results, DCP test results, and our experience with similar soils.

RECOMMENDED PAVEMENT SECTIONS

Pavement sections recommended for this site are as listed in the table below.

Pavement Type	Flexible Pavement		Rigid Pavement	
Traffic	Light Duty	Heavy Duty	Light Duty	Heavy Duty
Portland Cement Concrete (in.)	-	-	5	6
Asphaltic Concrete Surface Course (in.)	2	3	-	-
Flexible (Granular) Base (in.)	8	8	-	-
Lime/cement Treated Subgrade (in.) (1)	6(1)	6(1)	6(1)	6(1)

(1)Cement or lime treated soils may be used as a working or construction platform only to help facilitate construction over clay or cohesionless subgrades, and considered as an **option** to enhance pavement performance. Consideration may also be given to incorporating geogrid at the bottom of the flexible base to enhance pavement performance.

Rigid Pavement Consideration

We recommend Jointed Plain Concrete Pavement (JPCP) be utilized for the rigid pavement sections. JPCP typically does not require distributed steel, micro- or macro-fibers, or any other "reinforcing" material. The following recommendations are based on ACI 330R-08 "Guide for the Design and Construction of Concrete Parking Lots."

Typical joint types in JPCP include: control (contraction) joints, isolation joints (sometimes called expansion joints), and construction joints. The recommended joint spacing is 30 times the thickness of the slab up to a maximum of 15 ft. The length of a slab or panel should not be more than 25% greater than its width. For pavements with a thickness of 7 in. or greater, dowels may be required along all control joints. Tie bars may be required at the first longitudinal joint from the pavement edge to keep the outside edge from separating from the pavement.

Isolation joints are used to separate concrete slabs from other structures or fixed objects within or abutting the paved area to offset the effects of expected differential horizontal and vertical movements. Such structures include, but are not limited to, buildings, light standard foundations, and drop inlets. Isolation joints are also used at "T" intersections to accommodate differential movement along the different axes. Isolations joints are sometimes referred to as expansion joints. However, they are rarely needed to accommodate concrete expansion so they are not typically recommended for use as regularly spaced joints.

We recommend a jointing layout plan be established and reviewed by all parties prior to construction. We also recommend avoiding jointing lines which create angles of less than 60 degrees, "T" joints, and interior corners.

Proper curing of the concrete pavement should be initiated immediately after finishing. All control joints should be formed or sawed to a depth of at least 1/4 the thickness of the concrete slab and should extend completely through monolithic curbs (if used). Sawing of control joints should begin as soon as the concrete will not ravel, preferably within 1 to 3 hours using an early entry saw or 4 to 8 hours with a conventional saw. Timing will be dictated by site conditions.

Flexible Pavement Consideration

Based on our experience, the reported flexible pavement sections often perform adequately; however, maintenance or an overlay is generally needed sooner than would be required for a thicker design section. Consideration could be given to adding additional asphalt (i.e. an additional 1 in.) or incorporating a geogrid below the flexible base. In our opinion, incorporating geogrid into the pavement section will enhance overall pavement performance and reduce the potential for cracking and maintenance in asphalt pavements.

Another option to help reduce the potential for cracking and maintenance in asphalt pavements is including reinforcing fibers, such as Forta-Fi®, into the Hot Mix Asphalt (HMA). These are options and are not required. The geogrid reinforcement should conform to TxDOT Type 2 geogrid, or an approved substitute. If geogrid or reinforcing fibers are used in the provided options, we do not recommend reducing the report sections without further discussion with the design team.

SUBGRADE TREATMENT OPTION

Some of the soils at this site are either plastic or cohesionless and can be difficult to work with, particularly during periods of inclement weather. To provide a suitable, weather-resistant working surface for construction activity, the upper 6 in. to 8 in. of the subgrade soils may be treated with hydrated lime or cement. This is an **option** and is **not** required as part of the pavement thickness design presented above. We do not recommend that the treated subgrade be considered as a structural pavement component. Recommendations for treatment are provided in the section of this report entitled *Treatment of Subgrade*.

PAVEMENT CONSTRUCTION CONSIDERATIONS

SUBGRADE PREPARATION

Areas to support pavements should be stripped of all vegetation and organic topsoil and the exposed subgrade should be proofrolled in accordance with the recommendations in the *Site Preparation* section under *Foundation Construction Considerations*.

After completion of the proofrolling operations and just prior to flexible base placement, the exposed subgrade should be moisture conditioned by scarifying to a minimum depth of 6 in. and recompacting to a minimum of 95 percent of the maximum density determined from the TxDOT Tex-114-E or ASTM D698. The moisture content of the subgrade should be maintained within the range of optimum moisture content to 3 percentage points above optimum until permanently covered.

DRAINAGE CONSIDERATIONS

As with any soil-supported structure, the satisfactory performance of a pavement system is contingent on the provision of adequate surface and subsurface drainage. Insufficient drainage which allows saturation of the pavement subgrade and/or the supporting granular pavement materials will greatly reduce the performance and service life of the pavement systems.

Surface and subsurface drainage considerations crucial to the performance of pavements at this site include (but are not limited to) the following:

1) Any known natural or man-made subsurface seepage at the site which may occur at sufficiently shallow depths as to influence moisture contents within the subgrade should be intercepted by drainage ditches or below grade French drains.
2) Final site grading should eliminate isolated depressions adjacent to curbs which may allow surface water to pond and infiltrate into the underlying soils. **Curbs should completely penetrate base materials and should be installed to sufficient depth to reduce infiltration of water beneath the curbs**.
3) Pavement surfaces should be maintained to help minimize surface ponding and to provide rapid sealing of any developing cracks. These measures will help reduce infiltration of surface water downward through the pavement section.

ONSITE SOIL FILL (PAVEMENTS)

As discussed previously, the pavement recommendations presented in this report were prepared assuming that onsite soils will be used for fill grading in proposed pavement areas. Existing fill remediation is recommended to control settlement, see *Existing Fill*. We recommend that onsite soils be placed in loose lifts not exceeding 8 in. in thickness and compacted to at least 95 percent of the maximum density as determined by TxDOT Tex-114-E or ASTM D698. The moisture content of the fill should be maintained within the range of optimum water content to 3 percentage points above the optimum water content until permanently covered. We recommend that fill materials be free of roots and other organic or degradable material. We also recommend that the maximum particle size not exceed 4 in. or one half the lift thickness, whichever is smaller.

TREATMENT OF SUBGRADE

Lime or cement treatment of the subgrade soils, if utilized, should be in accordance with the TxDOT Standard Specifications, Item 260 or Item 275, respectively. A sufficient quantity of hydrated lime or cement should be mixed with the subgrade soils to reduce the soil plasticity index to 20 or less. Based on our experience with similar soils, we recommend that at least 4 percent hydrated lime or cement treatment by weight be used to increase the pH of the subgrade clays to 12.4 or higher. For construction purposes, we recommend that the optimum lime or cement content of the subgrade soils be determined by laboratory testing with representative samples of the subgrade materials being used for this project. Treated subgrade soils should be compacted to a minimum of 95 or 98 percent of the maximum density at a moisture content within the range of optimum moisture content to 3 percentage points above the optimum moisture content as determined by Tex-113-E.

We recommend that during site grading operations additional laboratory testing be performed to determine the concentration of soluble sulfates in the subgrade soils. If present, the sulfate in the soil may react with calcium-based stabilizers such as lime or cement. The adverse reaction, referred to as sulfate-induced heave, has been known to cause cohesive subgrade soils to swell in short periods of time, resulting in pavement heaving and possible failure.

FLEXIBLE BASE COURSE

The flexible base course should be crushed limestone conforming to TxDOT Standard Specifications, Item 247, Type A, Grade 1-2. Base course should be placed in lifts with a maximum thickness of 8 in. and compacted to a minimum of 95 percent of the maximum density at a moisture content within the range of 2 percentage points below to 2 percentage points above the optimum moisture content as determined by Tex-113-E.

ASPHALTIC CONCRETE SURFACE COURSE

The asphaltic concrete surface course should conform to TxDOT Standard Specifications, Item 340, Type C or D. The asphaltic concrete should be compacted to a minimum of 92 percent of the maximum theoretical specific gravity (Rice) of the mixture determined according to Test Method Tex-227-F. Pavement specimens, which shall be either cores or sections of asphaltic pavement, will be tested according to Test Method Tex-

207-F. The nuclear-density gauge or other methods which correlate satisfactorily with results obtained from project roadway specimens may be used when approved by the Engineer. Unless otherwise shown on the plans, the Contractor shall be responsible for obtaining the required roadway specimens at their expense and in a manner and at locations selected by the Engineer.

PORTLAND CEMENT CONCRETE

The Portland cement concrete should have a minimum 28-day compressive strength of 4,000 psi. A liquid membrane-forming curing compound should be applied as soon as practical after broom finishing the concrete surface. The curing compound will help reduce the loss of water from the concrete. The reduction in the rapid loss in water will help reduce shrinkage cracking of the concrete.

CONSTRUCTION RELATED SERVICES

CONSTRUCTION MATERIALS TESTING AND OBSERVATION SERVICES

As presented in the attachment to this report, *Important Information About Your Geotechnical Engineering Report*, subsurface conditions can vary across a project site. The conditions described in this report are based on interpolations derived from a limited number of data points. Variations will be encountered during construction, and only the geotechnical design engineer will be able to determine if these conditions are different than those assumed for design.

Construction problems resulting from variations or anomalies in subsurface conditions are among the most prevalent on construction projects and often lead to delays, changes, cost overruns, and disputes. These variations and anomalies can best be addressed if the geotechnical engineer of record, RKI is retained to perform construction observation and testing services during the construction of the project. This is because:

- RKI has an intimate understanding of the geotechnical engineering report's findings and recommendations. RKI understands how the report should be interpreted and can provide such interpretations on site, on the client's behalf.
- RKI knows what subsurface conditions are anticipated at the site.
- RKI is familiar with the goals of the owner and project design professionals, having worked with them in the development of the geotechnical workscope. This enables RKI to suggest remedial measures (when needed) which help meet the owner's and the design teams' requirements.
- RKI has a vested interest in client satisfaction, and thus assigns qualified personnel whose principal concern is client satisfaction. This concern is exhibited by the manner in which contractors' work is tested, evaluated and reported, and in selection of alternative approaches when such may become necessary.
- RKI cannot be held accountable for problems which result due to misinterpretation of our findings or recommendations when we are not on hand to provide the interpretation which is required.

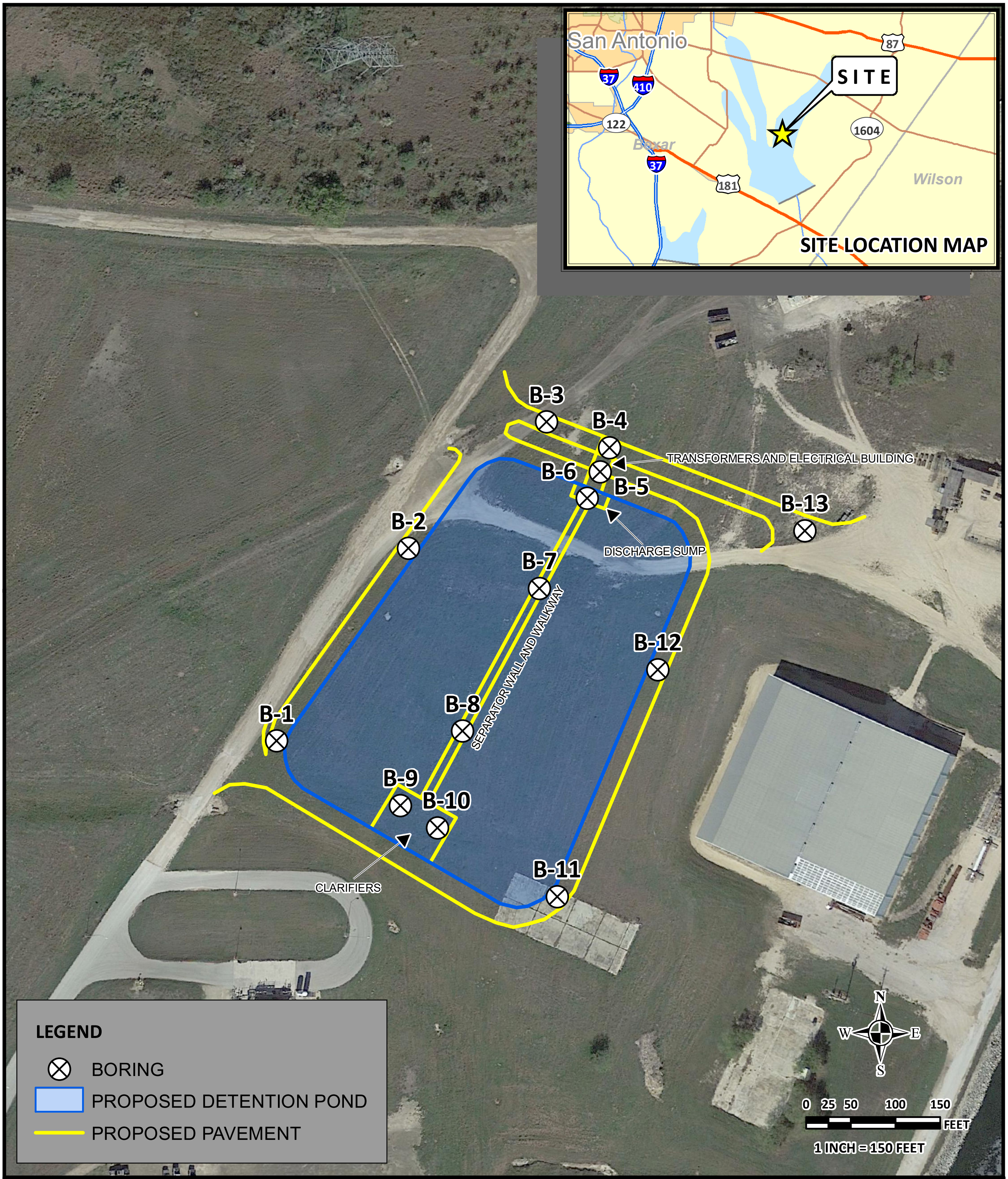
BUDGETING FOR CONSTRUCTION TESTING

Appropriate budgets need to be developed for the required construction testing and observation activities. At the appropriate time before construction, we advise that RKI and the project designers meet and jointly develop the testing budgets, as well as review the testing specifications as it pertains to this project.

Once the construction testing budget and scope of work are finalized, we encourage a preconstruction meeting with the selected contractor to review the scope of work to make sure it is consistent with the construction means and methods proposed by the contractor. RKI looks forward to the opportunity to provide continued support on this project, and would welcome the opportunity to meet with the Project Team to develop both a scope and budget for these services.

* * * * * * * * * * * * * * * * * *

ATTACHMENTS



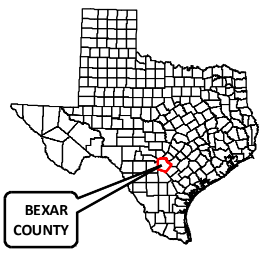
RABA KISTNER

12821 West Golden Lane
San Antonio, Texas 78249
(210)699-9090 TEL
(210)699-6426 FAX
www.rkci.com
TBPE Firm Number 3257

SOURCE: Aerial Photography Obtained from Google Earth Pro - 2019

BORING LOCATION MAP

CALAVERAS GEOTECHNICAL SURVEY
J.K. SPRUCE POWER PLANT
SAN ANTONIO, TEXAS



PROJECT No.: ASA20-044-00

ISSUE DATE:	09/03/2020
DRAWN BY:	KRB
CHECKED BY:	IM
REVIEWED BY:	RBW

FIGURE

1

NOTE: This Drawing is Provided for Illustration Only, May Not be to Scale and is Not Suitable for Design or Construction Purposes

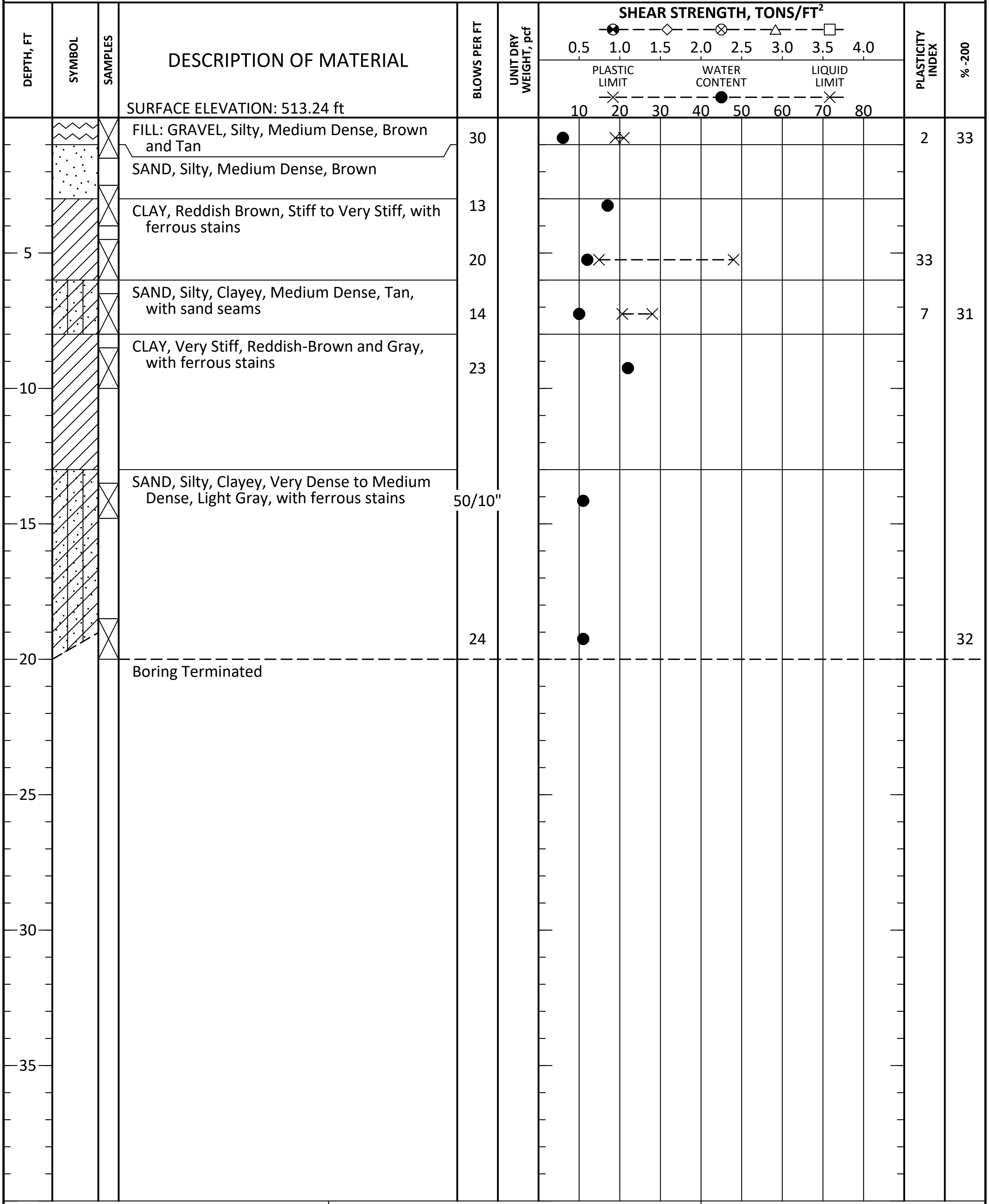
LOG OF BORING NO. B-1

Calaveras Geotechnical Survey
J.K. Spruce Power Plant
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 13664500.53; E 2186398.74



DESCRIPTION OF MATERIAL	BLOWS PER FT	PLASTICITY INDEX	% -200
FILL: GRAVEL, Silty, Medium Dense, Brown and Tan	30	2	33
SAND, Silty, Medium Dense, Brown			
CLAY, Reddish Brown, Stiff to Very Stiff, with ferrous stains	13 20	33	
SAND, Silty, Clayey, Medium Dense, Tan, with sand seams	14	7	31
CLAY, Very Stiff, Reddish-Brown and Gray, with ferrous stains	23		
SAND, Silty, Clayey, Very Dense to Medium Dense, Light Gray, with ferrous stains	50/10" 24		32
Boring Terminated			

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED:	20.0 ft	**DEPTH TO WATER:**	DRY	**PROJ. No.:**	ASA20-044-00
DATE DRILLED:	7/30/2020	**DATE MEASURED:**	7/30/2020	**FIGURE:**	2

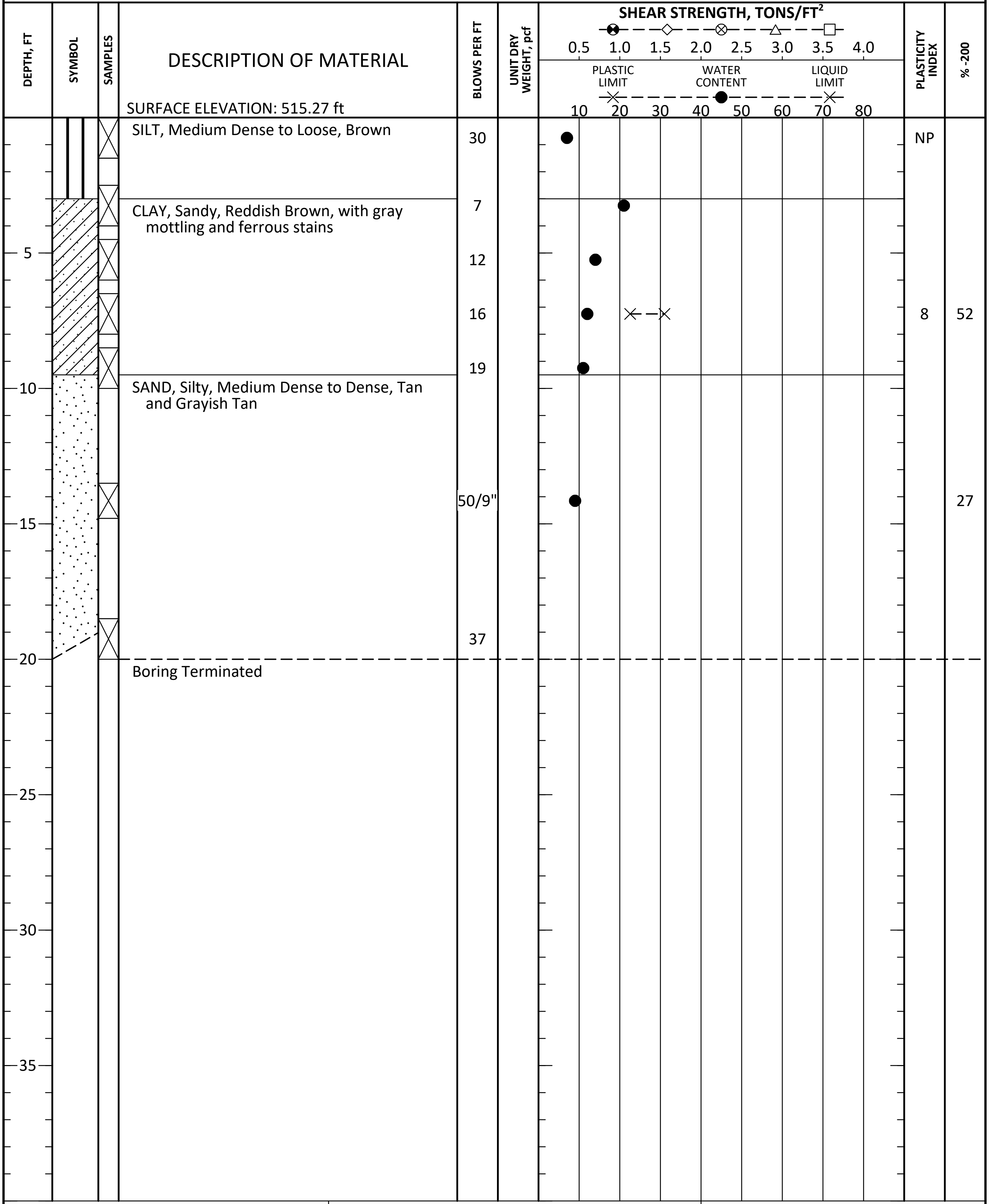
LOG OF BORING NO. B-2

Calaveras Geotechnical Survey
J.K. Spruce Power Plant
San Antonio, Texas

TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 13664707.14; E 2186527.09



SURFACE ELEVATION: 515.27 ft

DESCRIPTION OF MATERIAL	BLOWS PER FT	PLASTICITY INDEX	% -200
SILT, Medium Dense to Loose, Brown	30	NP	
CLAY, Sandy, Reddish Brown, with gray mottling and ferrous stains	7		
	12		
	16	8	52
	19		
SAND, Silty, Medium Dense to Dense, Tan and Grayish Tan	50/9"		27
	37		
Boring Terminated			

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED:	20.0 ft	**DEPTH TO WATER:**	DRY	**PROJ. No.:**	ASA20-044-00
DATE DRILLED:	7/29/2020	**DATE MEASURED:**	7/29/2020	**FIGURE:**	3

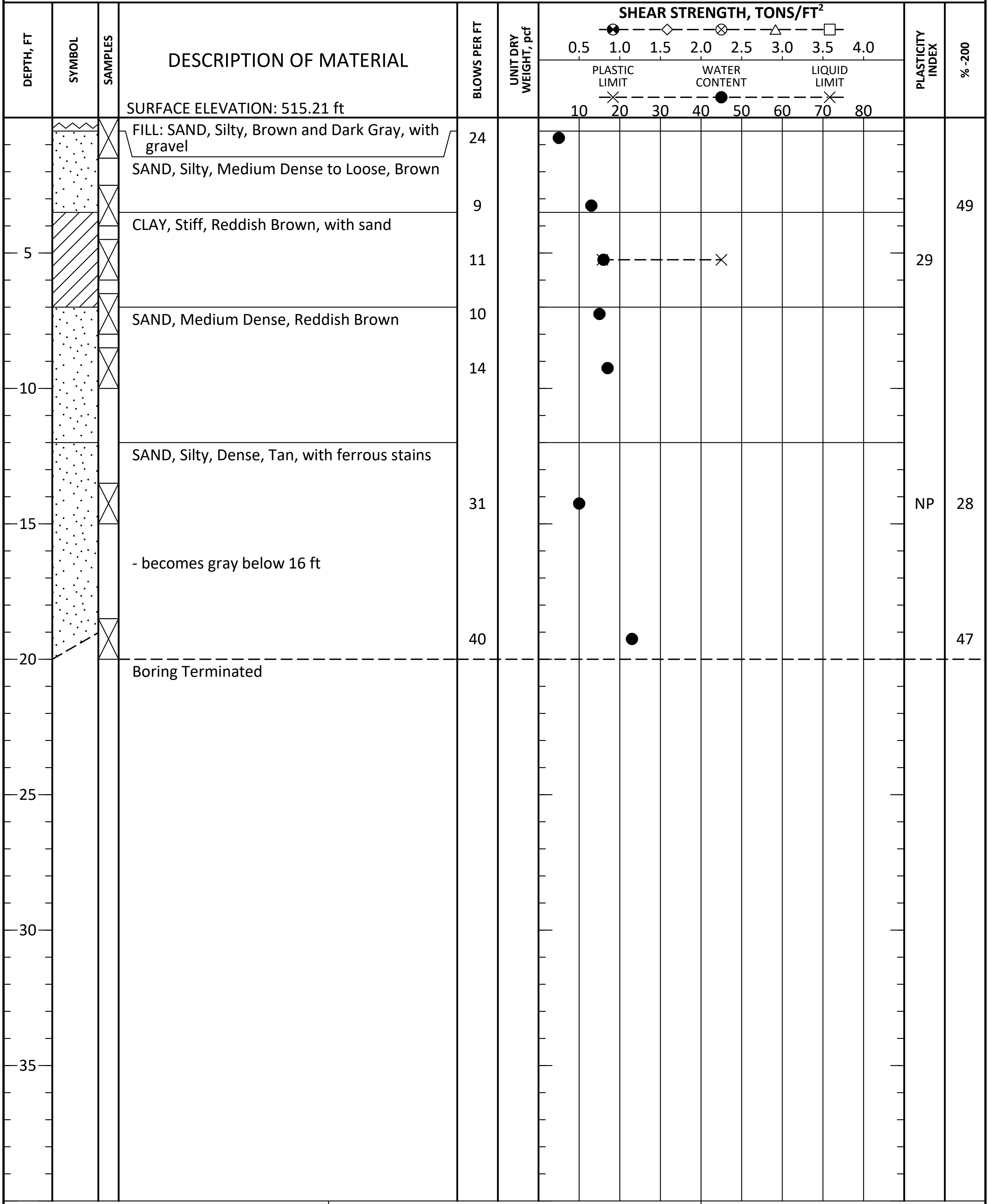
LOG OF BORING NO. B-3

Calaveras Geotechnical Survey
J.K. Spruce Power Plant
San Antonio, Texas

TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 13664866.46; E 2186692.50



DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
FILL: SAND, Silty, Brown and Dark Gray, with gravel	24			
SAND, Silty, Medium Dense to Loose, Brown	9			49
CLAY, Stiff, Reddish Brown, with sand	11		29	
SAND, Medium Dense, Reddish Brown	10			
	14			
SAND, Silty, Dense, Tan, with ferrous stains	31		NP	28
- becomes gray below 16 ft	40			47
Boring Terminated				

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED:	20.0 ft	**DEPTH TO WATER:**	DRY	**PROJ. No.:**	ASA20-044-00
DATE DRILLED:	7/30/2020	**DATE MEASURED:**	7/30/2020	**FIGURE:**	4

LOG OF BORING NO. B-4

Calaveras Geotechnical Survey
J.K. Spruce Power Plant
San Antonio, Texas

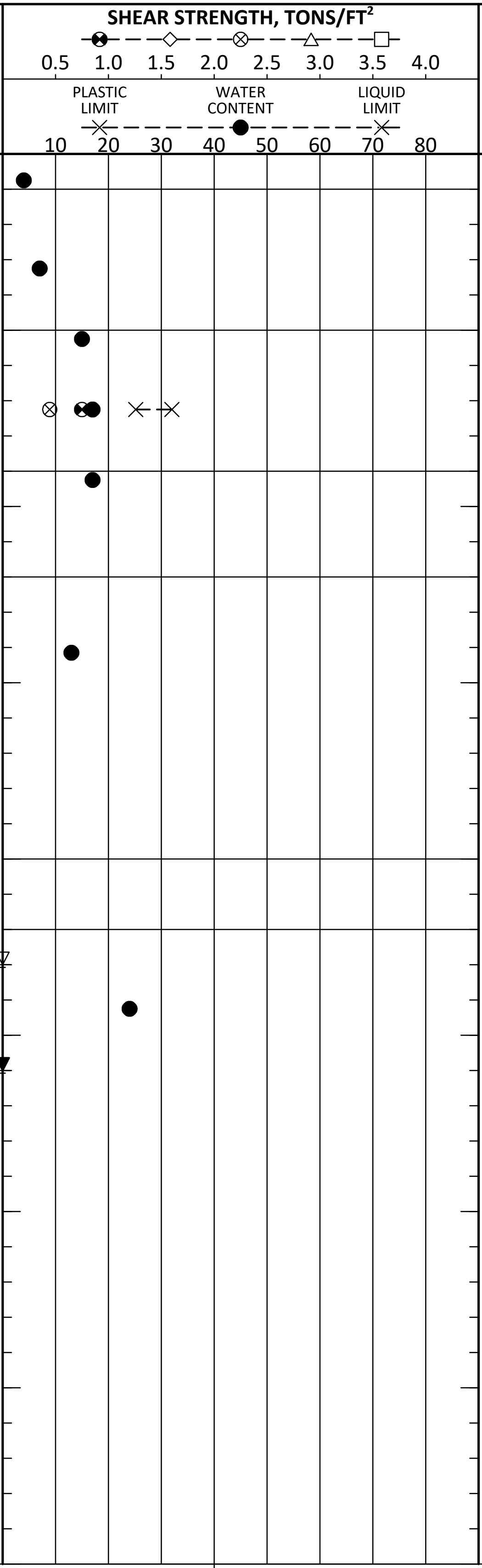
RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 13664824.62; E 2186764.51

SURFACE ELEVATION: 513.09 ft

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
	FILL: SAND, Silty, Medium Dense, Dark Brown, with gravel	13			21
	SAND, Silty, Loose, Reddish Brown, with dark brown seams	8			23
5	CLAY, Sandy, Silty, Stiff, Reddish Brown	9			
	- with black stains to 7 ft		110	7	
	SAND, Clayey, Medium Dense, Reddish Brown	24			34
10					
	SAND, Very Dense, Tan to Grayish Tan, with ferrous stains	50/10"			
15					
		50			
20	CLAY, Tan, with ferrous stains				
	CLAY, Sandy, Hard, Tan DRILLER'S NOTE: WATER encountered at 23 ft	45			57
25					
		50/7"			
30					
		50/8"			
35					
		50/11"			



NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 49.4 ft
DATE DRILLED: 7/29/2020

DEPTH TO WATER: 26 ft
DATE MEASURED: 7/29/2020

PROJ. No.: ASA20-044-00
FIGURE: 5a

LOG OF BORING NO. B-4

Calaveras Geotechnical Survey
J.K. Spruce Power Plant
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 13664824.62; E 2186764.51

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL SURFACE ELEVATION: 513.09 ft	BLOWS PER FT	UNIT DRY WEIGHT, pcf	SHEAR STRENGTH, TONS/FT^2 (0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0) PLASTIC LIMIT — WATER CONTENT — LIQUID LIMIT (10 20 30 40 50 60 70 80)	PLASTICITY INDEX	% -200
	⁙		SAND, Dense to Very Dense, Gray, Brown, and Dark Brown, with ferrous stains					
45	⁙	☒		33				
	⁙	☒		50/5"				
50			Boring Terminated					
55								
60								
65								
70								
75								

DEPTH DRILLED:	49.4 ft	**DEPTH TO WATER:**	26 ft	**PROJ. No.:**	ASA20-044-00
DATE DRILLED:	7/29/2020	**DATE MEASURED:**	7/29/2020	**FIGURE:**	5b

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

LOG OF BORING NO. B-5

Calaveras Geotechnical Survey
J.K. Spruce Power Plant
San Antonio, Texas

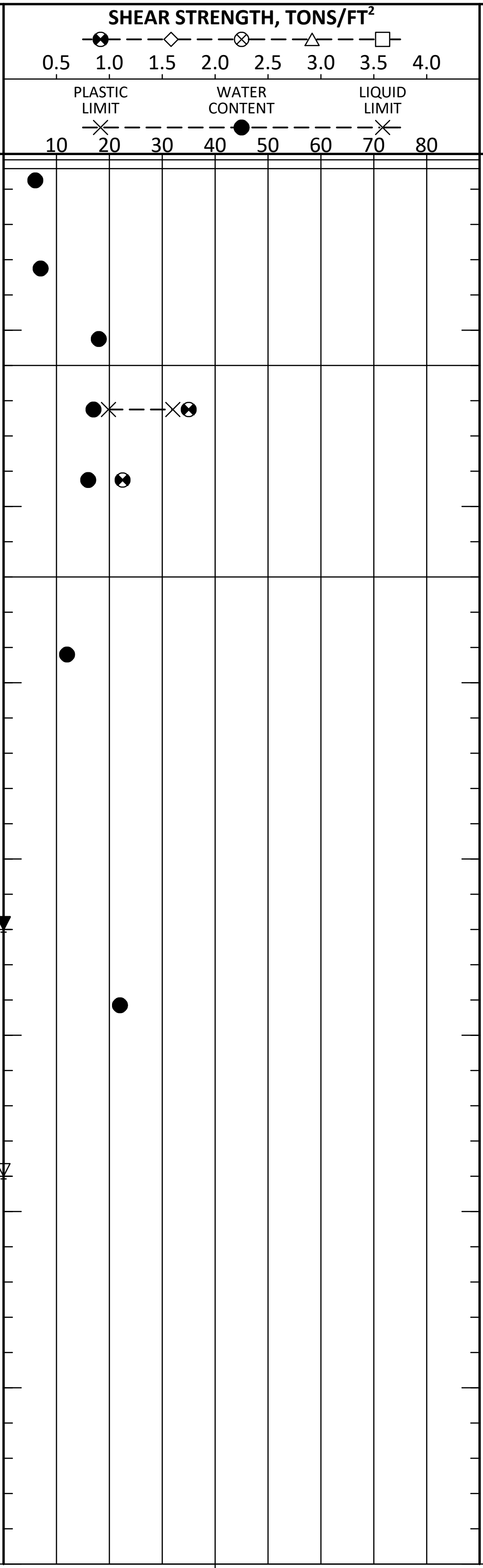
RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 13664804.06; E 2186757.52

SURFACE ELEVATION: 512.79 ft

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
BASE MATERIAL (2 in.)				
SILT, Sandy, Dark Gray, with trace gravel	26			
SAND, Medium Dense to Loose, Reddish Brown	14			
	9			
SAND, Clayey, Reddish Brown, with ferrous stains			12	31
SAND, Dense to Very Dense, Tan, with ferrous stains	50/11"			
	31			
- with gray silt and silty clay seams from 20 to 25 ft				
	50/10"			41
DRILLER'S NOTE: WATER encountered at 29 ft	50/9"			
	50/7"			
	44			



NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 49.3 ft
DATE DRILLED: 7/29/2020

DEPTH TO WATER: 22 ft
DATE MEASURED: 7/29/2020

PROJ. No.: ASA20-044-00
FIGURE: 6a

LOG OF BORING NO. B-5

Calaveras Geotechnical Survey
J.K. Spruce Power Plant
San Antonio, Texas

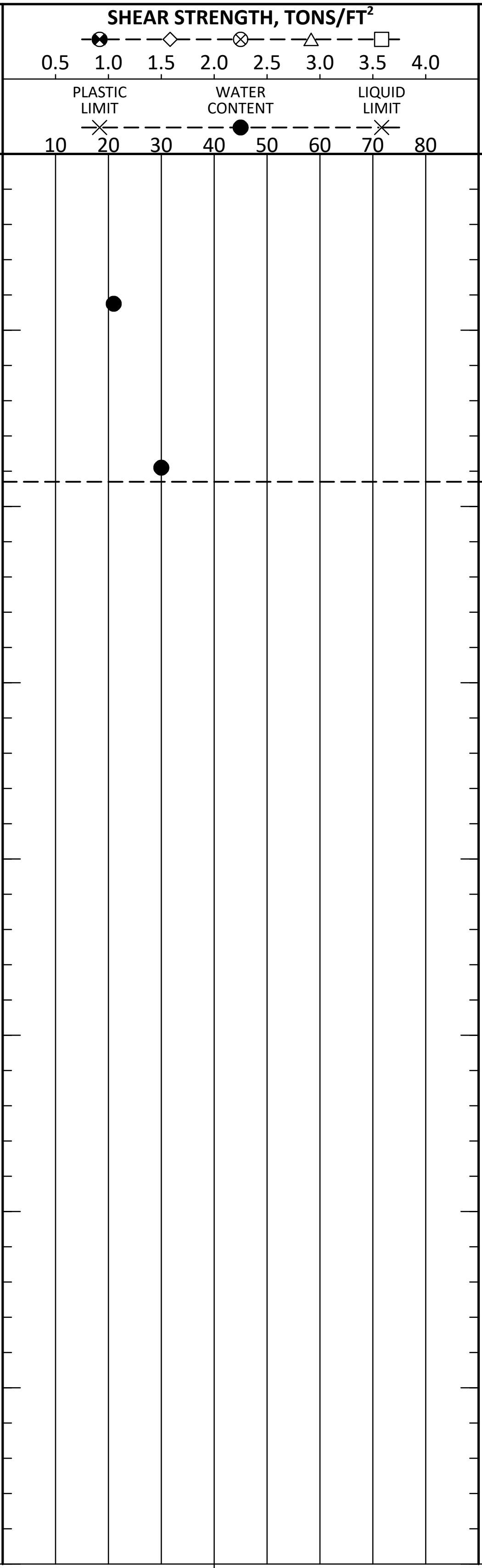
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 13664804.06; E 2186757.52

SURFACE ELEVATION: 512.79 ft

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
			CLAY, Sandy, Very Stiff to Hard, Light Gray, with ferrous stains				
		☒		21			59
45			- with dark gray below 45 ft				
		☒		50/4"			
50			Boring Terminated				
55							
60							
65							
70							
75							



NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED:	49.3 ft	**DEPTH TO WATER:**	22 ft	**PROJ. No.:**	ASA20-044-00
DATE DRILLED:	7/29/2020	**DATE MEASURED:**	7/29/2020	**FIGURE:**	6b

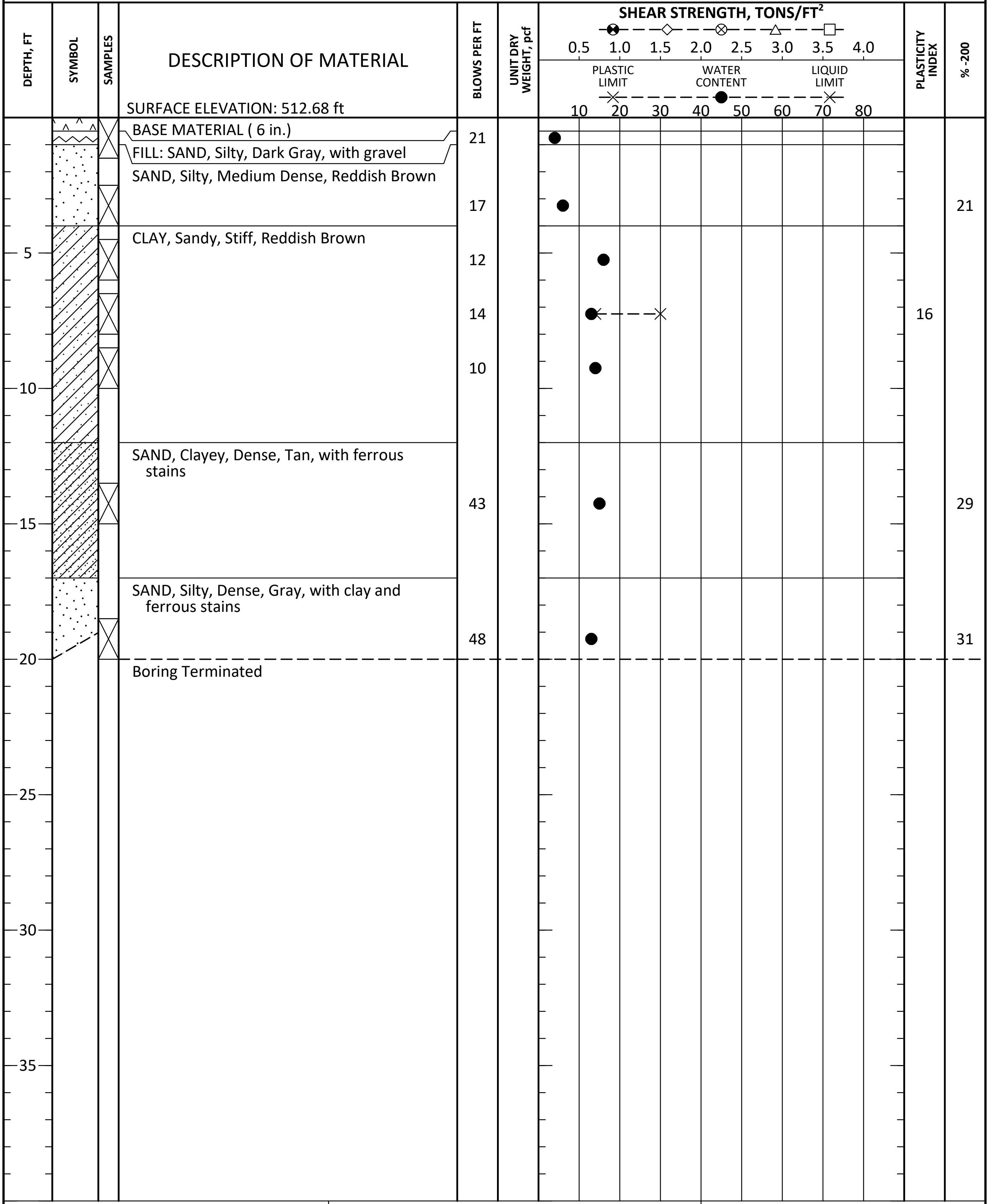
LOG OF BORING NO. B-6

Calaveras Geotechnical Survey
J.K. Spruce Power Plant
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 13664772.98; E 2186738.39



DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
BASE MATERIAL (6 in.)	21			
FILL: SAND, Silty, Dark Gray, with gravel				
SAND, Silty, Medium Dense, Reddish Brown	17			21
CLAY, Sandy, Stiff, Reddish Brown	12			
	14		16	
	10			
SAND, Clayey, Dense, Tan, with ferrous stains	43			29
SAND, Silty, Dense, Gray, with clay and ferrous stains	48			31
Boring Terminated				

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 20.0 ft
DATE DRILLED: 7/30/2020
DEPTH TO WATER: DRY
DATE MEASURED: 7/30/2020
PROJ. No.: ASA20-044-00
FIGURE: 7

LOG OF BORING NO. B-7

Calaveras Geotechnical Survey
J.K. Spruce Power Plant
San Antonio, Texas

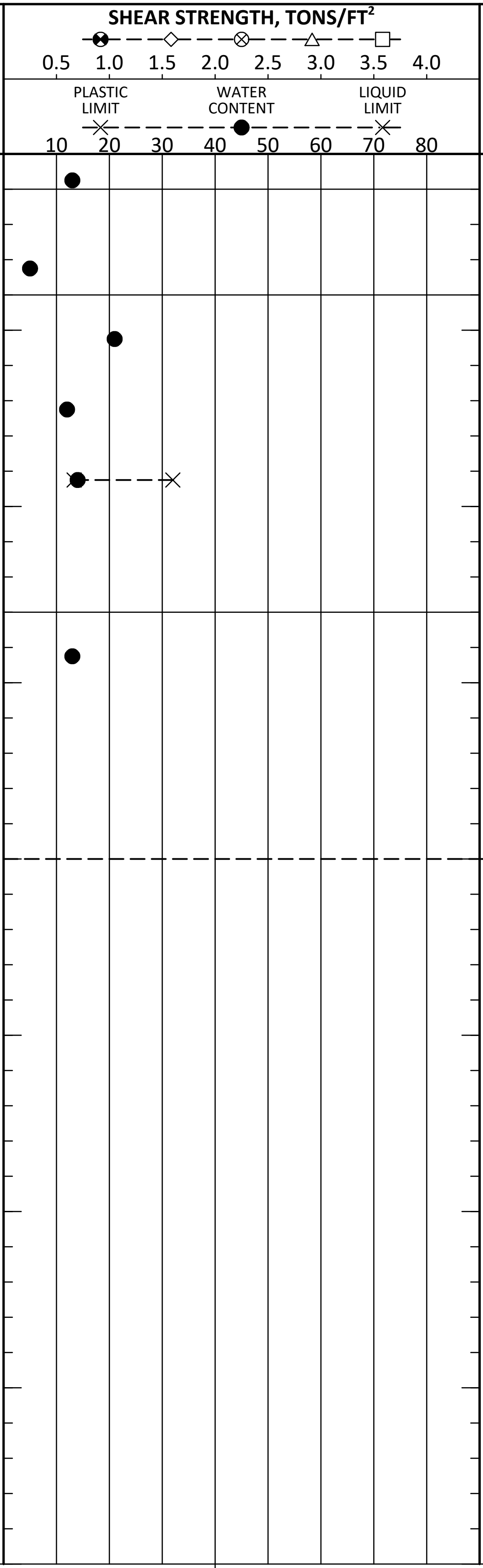
RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 13664677.95; E 2186684.56

SURFACE ELEVATION: 512.72 ft

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
FILL: SILT, Dense, Gray, with gravel	32		NP	
SAND, Silty, Medium Dense, Reddish Brown	23			21
CLAY, Sandy, Stiff, Reddish Brown, with ferrous stains	9			
	12			
	14		19	
SAND, Dense, Tan, with ferrous stains	49			
	42			
Boring Terminated				



DEPTH, FT: 5, 10, 15, 20, 25, 30, 35

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 20.0 ft
DATE DRILLED: 7/30/2020
DEPTH TO WATER: DRY
DATE MEASURED: 7/30/2020
PROJ. No.: ASA20-044-00
FIGURE: 8

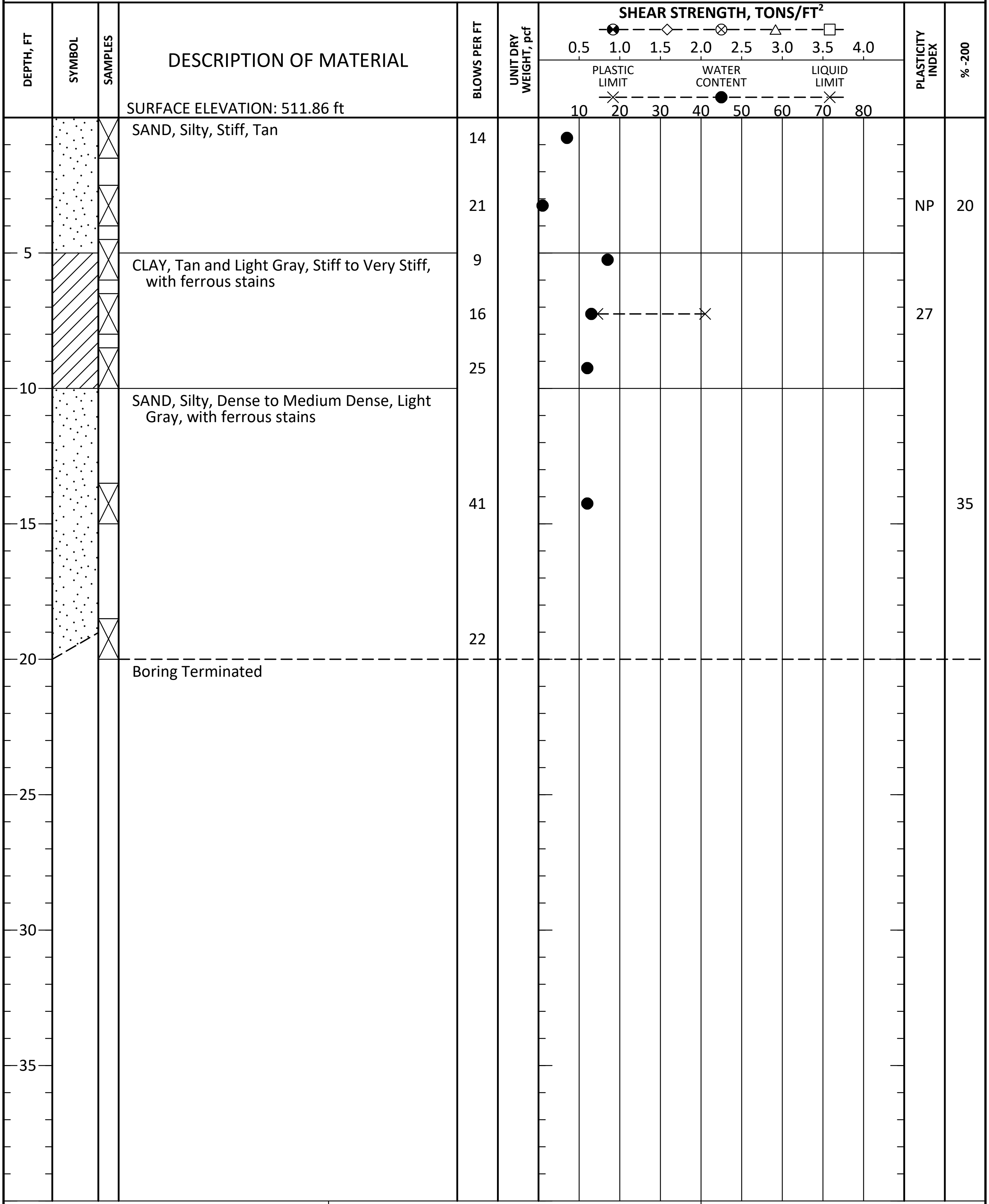
LOG OF BORING NO. B-8

Calaveras Geotechnical Survey
J.K. Spruce Power Plant
San Antonio, Texas

TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 13664520.77; E 2186596.21



SURFACE ELEVATION: 511.86 ft

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
SAND, Silty, Stiff, Tan	14			
	21		NP	20
CLAY, Tan and Light Gray, Stiff to Very Stiff, with ferrous stains	9			
	16		27	
	25			
SAND, Silty, Dense to Medium Dense, Light Gray, with ferrous stains	41			35
	22			
Boring Terminated				

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 20.0 ft
DATE DRILLED: 7/30/2020

DEPTH TO WATER: DRY
DATE MEASURED: 7/30/2020

PROJ. No.: ASA20-044-00
FIGURE: 9

LOG OF BORING NO. B-9

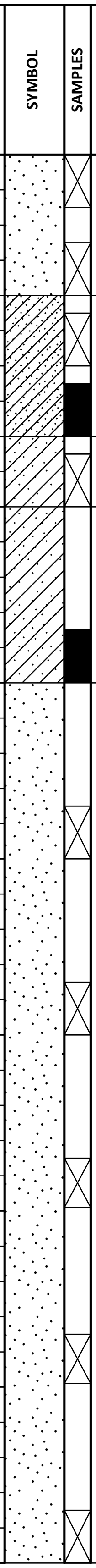
Calaveras Geotechnical Survey
J.K. Spruce Power Plant
San Antonio, Texas

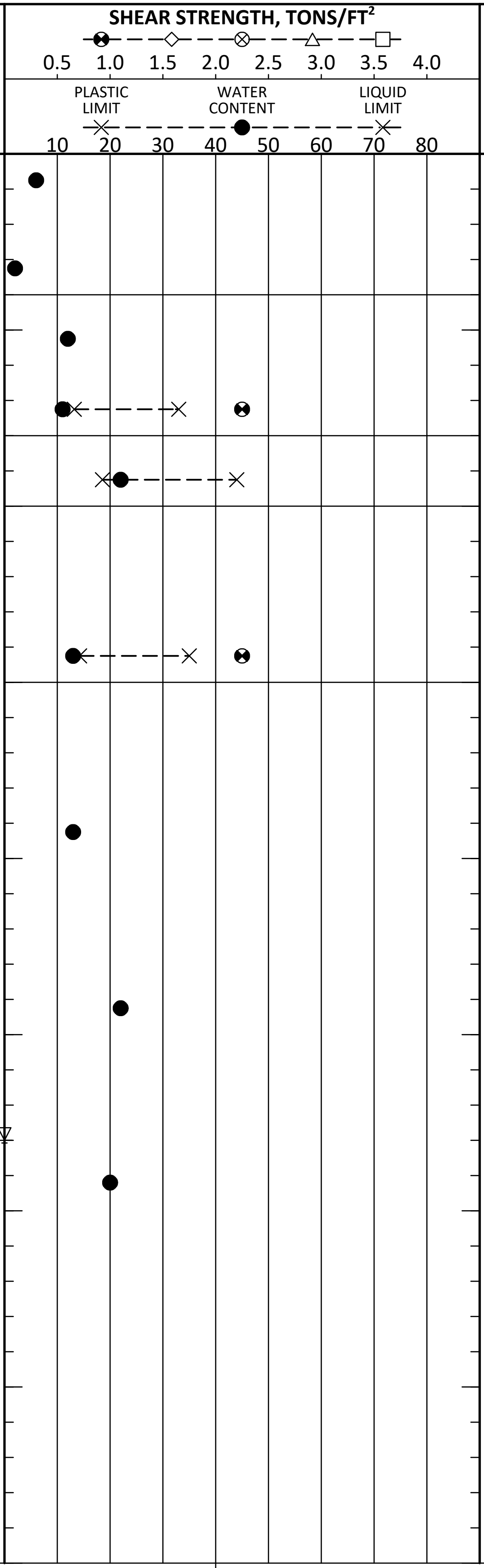
RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 13664428.32; E 2186524.83

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
	SURFACE ELEVATION: 511.19 ft				
	SAND, Silty, Medium Dense, Tan	22			
		19			
5	SAND, Clayey, Medium Dense, Reddish Brown, with sand	12			
				20	41
	CLAY, Sandy, Stiff, Tan and Light Gray	12		25	
10	CLAY, Sandy, Stiff to Hard, Reddish Brown, with sand				
				21	60
15	SAND, Medium Dense to Very Dense, Tan and Light Gray, with ferrous stains				
		29			
20					
		47			
25	- clayey seams below 25 ft				
	DRILLER'S NOTE: WATER encountered at 28 ft	50/11"			
30					
		50/11"			
35					
		40			





NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 50.0 ft
DATE DRILLED: 7/30/2020
DEPTH TO WATER: 28 ft
DATE MEASURED: 7/30/2020
PROJ. No.: ASA20-044-00
FIGURE: 10a

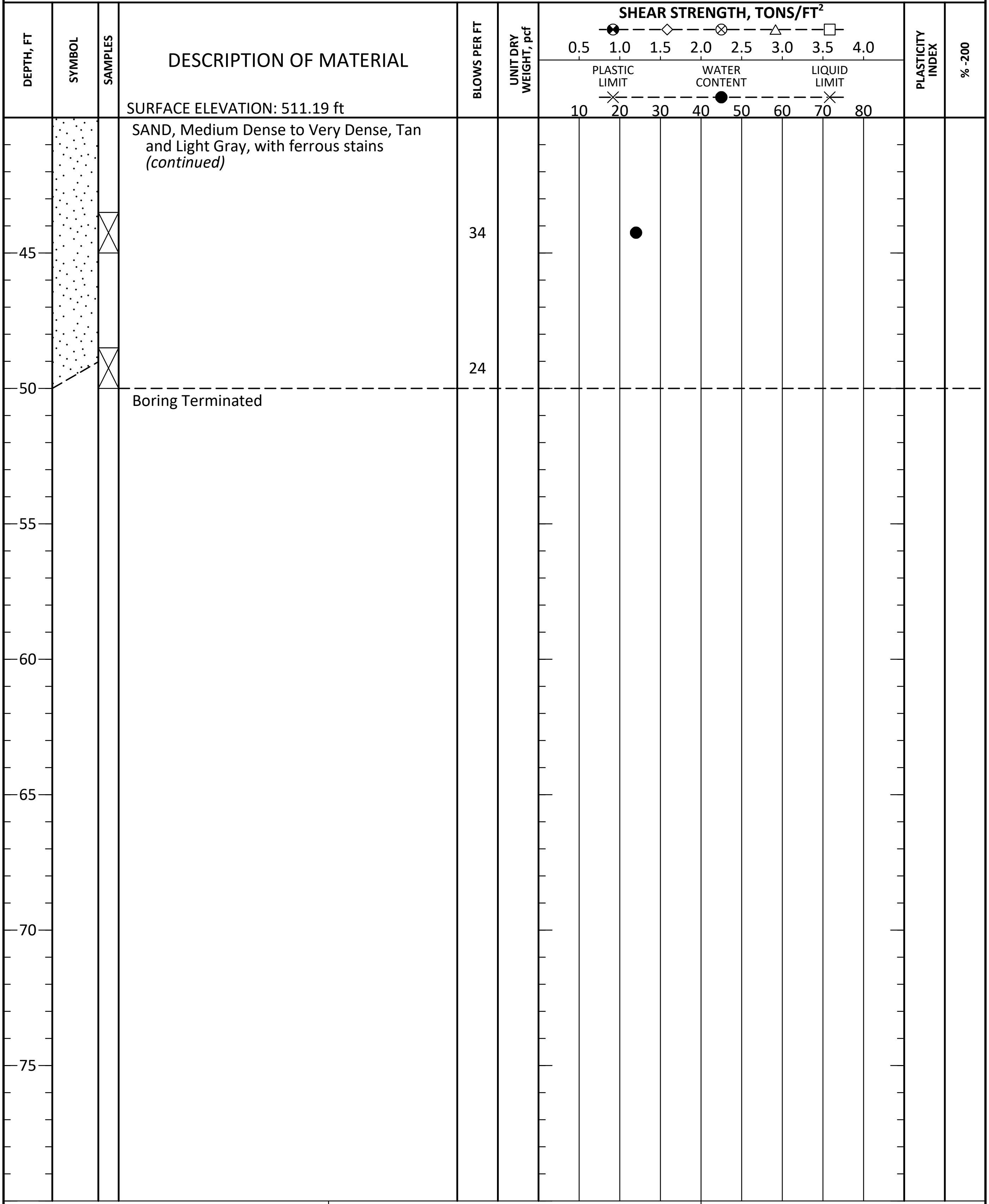
LOG OF BORING NO. B-9

Calaveras Geotechnical Survey
J.K. Spruce Power Plant
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 13664428.32; E 2186524.83



SAND, Medium Dense to Very Dense, Tan and Light Gray, with ferrous stains *(continued)*

Boring Terminated

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED:	50.0 ft	**DEPTH TO WATER:**	28 ft	**PROJ. No.:**	ASA20-044-00
DATE DRILLED:	7/30/2020	**DATE MEASURED:**	7/30/2020	**FIGURE:**	10b

LOG OF BORING NO. B-10

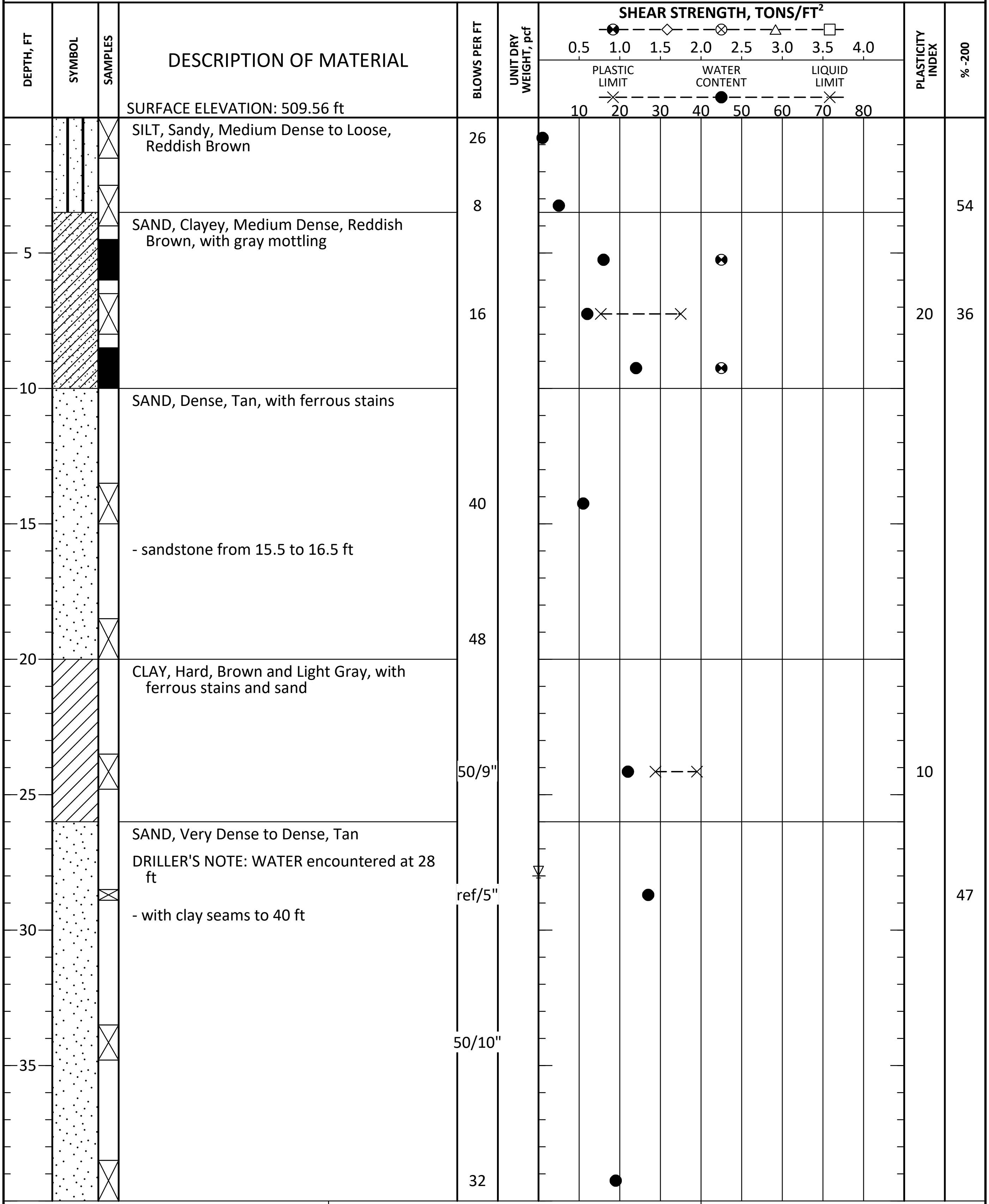
Calaveras Geotechnical Survey
J.K. Spruce Power Plant
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 13664415.18; E 2186568.33

SURFACE ELEVATION: 509.56 ft



DESCRIPTION OF MATERIAL	BLOWS PER FT	PLASTICITY INDEX	% -200
SILT, Sandy, Medium Dense to Loose, Reddish Brown	26		
	8		54
SAND, Clayey, Medium Dense, Reddish Brown, with gray mottling			
	16	20	36
SAND, Dense, Tan, with ferrous stains	40		
- sandstone from 15.5 to 16.5 ft			
	48		
CLAY, Hard, Brown and Light Gray, with ferrous stains and sand			
	50/9"	10	
SAND, Very Dense to Dense, Tan DRILLER'S NOTE: WATER encountered at 28 ft - with clay seams to 40 ft	ref/5"		47
	50/10"		
	32		

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 50.0 ft
DATE DRILLED: 7/31/2020
DEPTH TO WATER: 28 ft
DATE MEASURED: 7/31/2020
PROJ. No.: ASA20-044-00
FIGURE: 11a

LOG OF BORING NO. B-10

Calaveras Geotechnical Survey
J.K. Spruce Power Plant
San Antonio, Texas

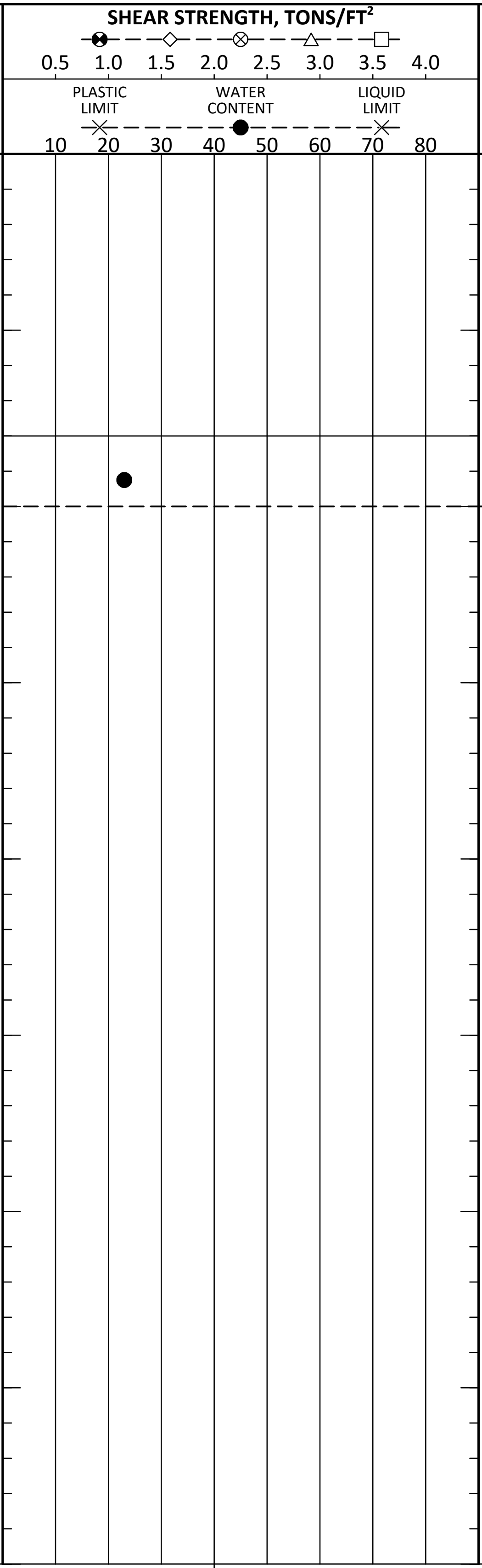
RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 13664415.18; E 2186568.33

SURFACE ELEVATION: 509.56 ft

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
43–48	⁙	☒	SAND, Very Dense to Dense, Tan *(continued)*	40			
48–50	▨	☒	CLAY, Sandy, Hard, Brown to Dark Brown, with ferrous stains	46			70
50			Boring Terminated				



NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 50.0 ft
DATE DRILLED: 7/31/2020
DEPTH TO WATER: 28 ft
DATE MEASURED: 7/31/2020
PROJ. No.: ASA20-044-00
FIGURE: 11b

LOG OF BORING NO. B-11

Calaveras Geotechnical Survey
J.K. Spruce Power Plant
San Antonio, Texas

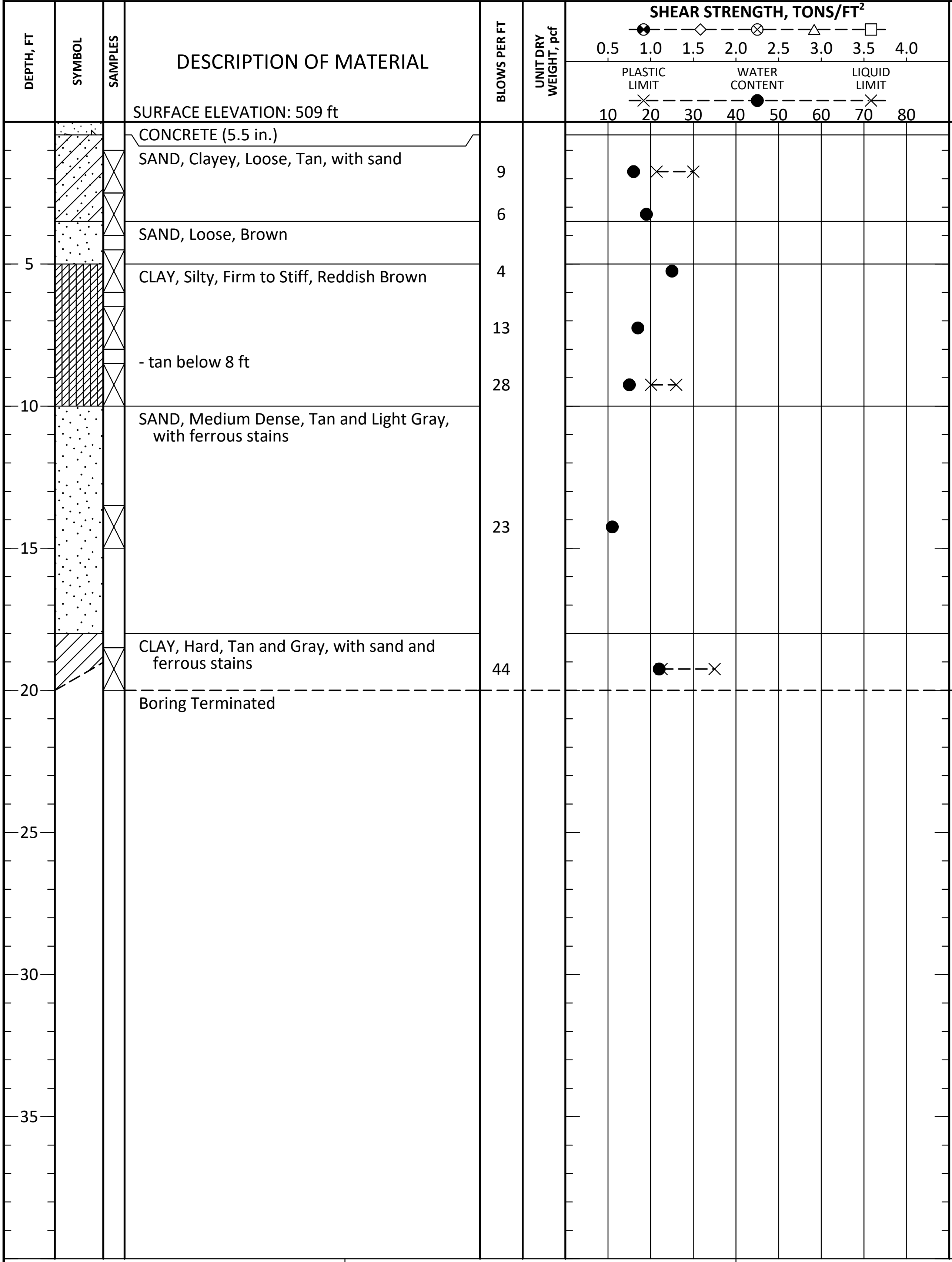
RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 13664330.40; E 2186703.75

SURFACE ELEVATION: 509 ft

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
CONCRETE (5.5 in.)				
SAND, Clayey, Loose, Tan, with sand	9		9	44
	6			60
SAND, Loose, Brown				
CLAY, Silty, Firm to Stiff, Reddish Brown	4			
	13			
- tan below 8 ft	28		6	
SAND, Medium Dense, Tan and Light Gray, with ferrous stains	23			
CLAY, Hard, Tan and Gray, with sand and ferrous stains	44		12	
Boring Terminated				



NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 20.0 ft
DATE DRILLED: 7/30/2020
DEPTH TO WATER: DRY
DATE MEASURED: 7/30/2020
PROJ. No.: ASA20-044-00
FIGURE: 12

LOG OF BORING NO. B-12

Calaveras Geotechnical Survey
J.K. Spruce Power Plant
San Antonio, Texas

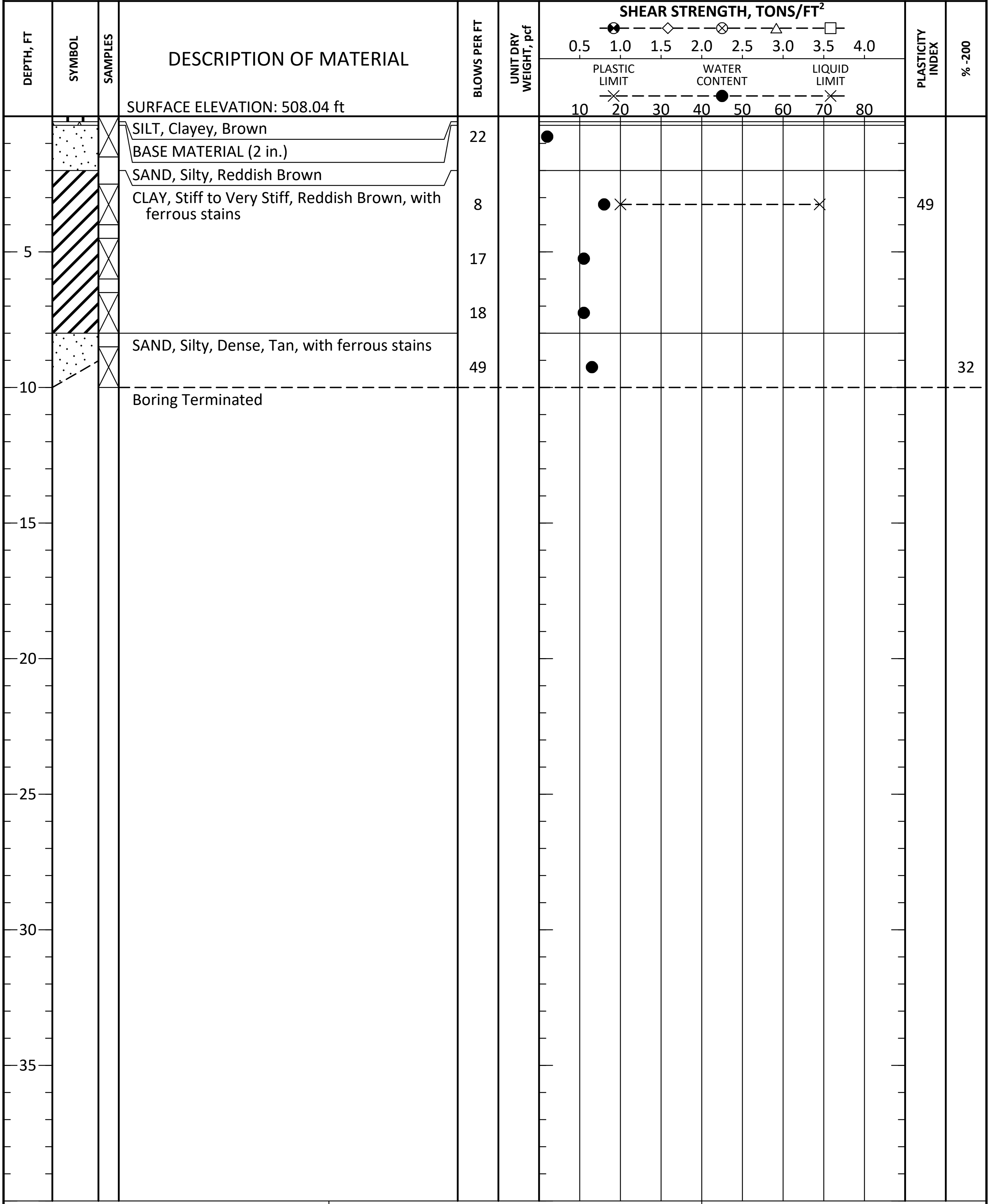
RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 13664578.44; E 2186816.01

SURFACE ELEVATION: 508.04 ft

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
SILT, Clayey, Brown	22			
BASE MATERIAL (2 in.)				
SAND, Silty, Reddish Brown				
CLAY, Stiff to Very Stiff, Reddish Brown, with ferrous stains	8		49	
	17			
	18			
SAND, Silty, Dense, Tan, with ferrous stains	49			32
Boring Terminated				



NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 10.0 ft
DATE DRILLED: 7/30/2020
DEPTH TO WATER: DRY
DATE MEASURED: 7/30/2020
PROJ. No.: ASA20-044-00
FIGURE: 13

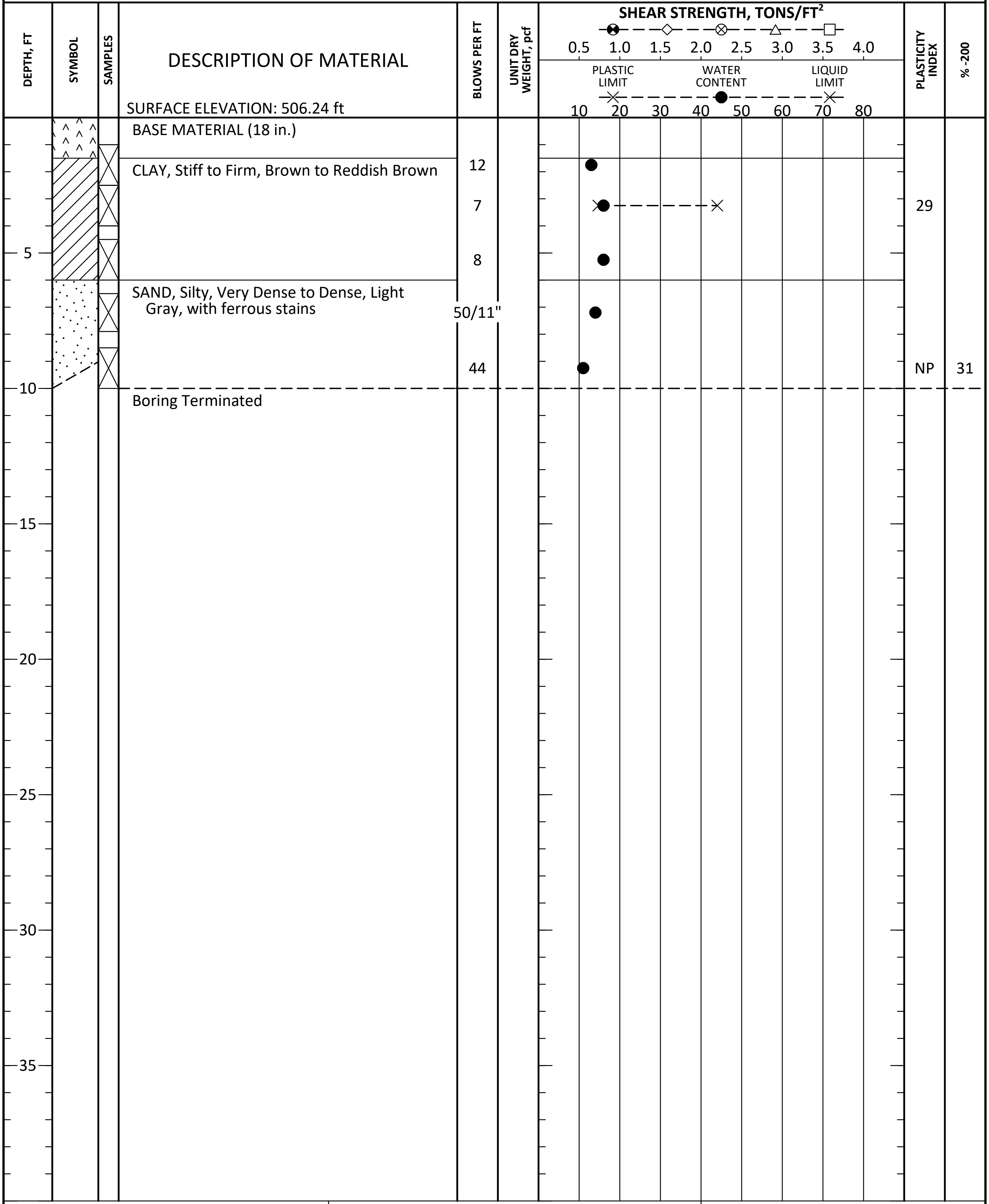
LOG OF BORING NO. B-13

Calaveras Geotechnical Survey
J.K. Spruce Power Plant
San Antonio, Texas

TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 13664738.20; E 2186975.35



SURFACE ELEVATION: 506.24 ft

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
BASE MATERIAL (18 in.)				
CLAY, Stiff to Firm, Brown to Reddish Brown	12			
	7		29	
	8			
SAND, Silty, Very Dense to Dense, Light Gray, with ferrous stains	50/11"			
	44		NP	31
Boring Terminated				

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPTH DRILLED: 10.0 ft
DATE DRILLED: 7/30/2020
DEPTH TO WATER: DRY
DATE MEASURED: 7/30/2020
PROJ. No.: ASA20-044-00
FIGURE: 14

KEY TO TERMS AND SYMBOLS

MATERIAL TYPES

SOIL TERMS

- CALCAREOUS
- CALICHE
- CLAY
- CLAYEY
- GRAVEL
- GRAVELLY
- PEAT
- SAND
- SANDY
- SILT
- SILTY
- FILL

ROCK TERMS

- CHALK
- CLAYSTONE
- CLAY-SHALE
- CONGLOMERATE
- DOLOMITE
- IGNEOUS
- LIMESTONE
- MARL
- METAMORPHIC
- SANDSTONE
- SHALE
- SILTSTONE

OTHER

- ■ ASPHALT
- BASE
- CONCRETE/CEMENT
- BRICKS / PAVERS
- WASTE
- □ NO INFORMATION

WELL CONSTRUCTION AND PLUGGING MATERIALS

- BLANK PIPE
- SCREEN
- BENTONITE
- CEMENT GROUT
- BENTONITE & CUTTINGS
- CONCRETE/CEMENT
- CUTTINGS
- GRAVEL
- SAND
- VOLCLAY

SAMPLE TYPES

- AIR ROTARY
- GRAB SAMPLE
- CORE
- GEOPROBE SAMPLER
- ROTOSONIC -DAMAGED
- MUD ROTARY
- NO RECOVERY
- NX CORE
- PITCHER
- ROTOSONIC -INTACT
- ■ SHELBY TUBE
- SPLIT BARREL
- SPLIT SPOON
- TEXAS CONE PENETROMETER
- DISTURBED

STRENGTH TEST TYPES

- ⊗ POCKET PENETROMETER
- ◇ TORVANE
- ⊗ UNCONFINED COMPRESSION
- △ TRIAXIAL COMPRESSION UNCONSOLIDATED-UNDRAINED
- □ TRIAXIAL COMPRESSION CONSOLIDATED-UNDRAINED

NOTE: VALUES SYMBOLIZED ON BORING LOGS REPRESENT SHEAR STRENGTHS UNLESS OTHERWISE NOTED

PROJECT NO. ASA20-044-00

RABAKISTNER

KEY TO TERMS AND SYMBOLS (CONT'D)

TERMINOLOGY

Terms used in this report to describe soils with regard to their consistency or conditions are in general accordance with the discussion presented in Article 45 of SOILS MECHANICS IN ENGINEERING PRACTICE, Terzaghi and Peck, John Wiley & Sons, Inc., 1967, using the most reliable information available from the field and laboratory investigations. Terms used for describing soils according to their texture or grain size distribution are in accordance with the UNIFIED SOIL CLASSIFICATION SYSTEM, as described in American Society for Testing and Materials D2487-06 and D2488-00, Volume 04.08, Soil and Rock; Dimension Stone; Geosynthetics; 2005.

The depths shown on the boring logs are not exact, and have been estimated to the nearest half-foot. Depth measurements may be presented in a manner that implies greater precision in depth measurement, i.e 6.71 meters. The reader should understand and interpret this information only within the stated half-foot tolerance on depth measurements.

RELATIVE DENSITY

Penetration Resistance Blows per ft	Relative Density
0 - 4	Very Loose
4 - 10	Loose
10 - 30	Medium Dense
30 - 50	Dense
> 50	Very Dense

COHESIVE STRENGTH

Resistance Blows per ft	Consistency	Cohesion TSF
0 - 2	Very Soft	0 - 0.125
2 - 4	Soft	0.125 - 0.25
4 - 8	Firm	0.25 - 0.5
8 - 15	Stiff	0.5 - 1.0
15 - 30	Very Stiff	1.0 - 2.0
> 30	Hard	> 2.0

PLASTICITY

Plasticity Index	Degree of Plasticity
0 - 5	None
5 - 10	Low
10 - 20	Moderate
20 - 40	Plastic
> 40	Highly Plastic

ABBREVIATIONS

B = Benzene
T = Toluene
E = Ethylbenzene
X = Total Xylenes
BTEX = Total BTEX
TPH = Total Petroleum Hydrocarbons
ND = Not Detected
NA = Not Analyzed
NR = Not Recorded/No Recovery
OVA = Organic Vapor Analyzer
ppm = Parts Per Million

Qam, Qas, Qal = Quaternary Alluvium
Qat = Low Terrace Deposits
Qbc = Beaumont Formation
Qt = Fluviatile Terrace Deposits
Qao = Seymour Formation
Qle = Leona Formation
Q-Tu = Uvalde Gravel
Ewi = Wilcox Formation
Emi = Midway Group
Mc = Catahoula Formation
El = Laredo Formation
Kknm = Navarro Group and Marlbrook Marl
Kpg = Pecan Gap Chalk
Kau = Austin Chalk

Kef = Eagle Ford Shale
Kbu = Buda Limestone
Kdr = Del Rio Clay
Kft = Fort Terrett Member
Kgt = Georgetown Formation
Kep = Person Formation
Kek = Kainer Formation
Kes = Escondido Formation
Kew = Walnut Formation
Kgr = Glen Rose Formation
Kgru = Upper Glen Rose Formation
Kgrl = Lower Glen Rose Formation
Kh = Hensell Sand

KEY TO TERMS AND SYMBOLS (CONT'D)

TERMINOLOGY

SOIL STRUCTURE

Slickensided	Having planes of weakness that appear slick and glossy.
Fissured	Containing shrinkage or relief cracks, often filled with fine sand or silt; usually more or less vertical.
Pocket	Inclusion of material of different texture that is smaller than the diameter of the sample.
Parting	Inclusion less than 1/8 inch thick extending through the sample.
Seam	Inclusion 1/8 inch to 3 inches thick extending through the sample.
Layer	Inclusion greater than 3 inches thick extending through the sample.
Laminated	Soil sample composed of alternating partings or seams of different soil type.
Interlayered	Soil sample composed of alternating layers of different soil type.
Intermixed	Soil sample composed of pockets of different soil type and layered or laminated structure is not evident.
Calcareous	Having appreciable quantities of carbonate.
Carbonate	Having more than 50% carbonate content.

SAMPLING METHODS

RELATIVELY UNDISTURBED SAMPLING

Cohesive soil samples are to be collected using three-inch thin-walled tubes in general accordance with the Standard Practice for Thin-Walled Tube Sampling of Soils (ASTM D1587) and granular soil samples are to be collected using two-inch split-barrel samplers in general accordance with the Standard Method for Penetration Test and Split-Barrel Sampling of Soils (ASTM D1586). Cohesive soil samples may be extruded on-site when appropriate handling and storage techniques maintain sample integrity and moisture content.

STANDARD PENETRATION TEST (SPT)

A 2-in.-OD, 1-3/8-in.-ID split spoon sampler is driven 1.5 ft into undisturbed soil with a 140-pound hammer free falling 30 in. After the sampler is seated 6 in. into undisturbed soil, the number of blows required to drive the sampler the last 12 in. is the Standard Penetration Resistance or "N" value, which is recorded as blows per foot as described below.

SPLIT-BARREL SAMPLER DRIVING RECORD

Blows Per Foot	Description
25	25 blows drove sampler 12 inches, after initial 6 inches of seating.
50/7"	50 blows drove sampler 7 inches, after initial 6 inches of seating.
Ref/3"	50 blows drove sampler 3 inches during initial 6-inch seating interval.

NOTE: To avoid damage to sampling tools, driving is limited to 50 blows during or after seating interval.

PROJECT NO. ASA20-044-00

RABAKISTNER

REVISED 04/2012

RESULTS OF SOIL SAMPLE ANALYSES

PROJECT NAME: Calaveras Geotechnical Survey
J.K. Spruce Power Plant
San Antonio, Texas

FILE NAME: ASA20-044-00.GPJ 9/3/2020

Boring No.	Sample Depth (ft)	Blows per ft	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	USCS	Dry Unit Weight (pcf)	% -200 Sieve	Shear Strength (tsf)	Strength Test
B-1	0.0 to 1.5	30	6	21	19	2	SM		33		
	2.5 to 4.0	13	17								
	4.5 to 6.0	20	12	48	15	33	CL				
	6.5 to 8.0	14	10	28	21	7	SC-SM		31		
	8.5 to 10.0	23	22								
	13.5 to 14.8	50/10"	11								
	18.5 to 20.0	24	11						32		
B-2	0.0 to 1.5	30	7	NP	NP	NP	ML				
	2.5 to 4.0	7	21								
	4.5 to 6.0	12	14								
	6.5 to 8.0	16	12	31	23	8	ML		52		
	8.5 to 10.0	19	11								
	13.5 to 14.8	50/9"	9						27		
	18.5 to 20.0	37									
B-3	0.0 to 1.5	24	5								
	2.5 to 4.0	9	13						49		
	4.5 to 6.0	11	16	45	16	29	CL				
	6.5 to 8.0	10	15								
	8.5 to 10.0	14	17								
	13.5 to 15.0	31	10			NP	SM		28		
	18.5 to 20.0	40	23						47		
B-4	0.0 to 1.5	13	4						21		
	2.5 to 4.0	8	7						23		
	4.5 to 6.0	9	15								
	6.5 to 8.0		17	32	25	7	CL-ML	110		0.45	UC
	8.5 to 10.0	24	17						34		
	13.5 to 14.8	50/10"	13								
	18.5 to 20.0	50									
	23.5 to 25.0	45	24						57		
	28.5 to 29.6	50/7"									
	33.5 to 34.7	50/8"									
	38.5 to 39.9	50/11"									
	43.5 to 45.0	33									
	48.5 to 49.4	50/5"									
B-5	0.0 to 1.5	26	6								
	2.5 to 4.0	14	7								
	4.5 to 6.0	9	18								
	6.5 to 8.0		17	32	20	12	SC		31	1.75	PP
	8.5 to 10.0		16							1.13	PP

PP = Pocket Penetrometer TV = Torvane UC = Unconfined Compression FV = Field Vane UU = Unconsolidated Undrained Triaxial

CU = Consolidated Undrained Triaxial

RESULTS OF SOIL SAMPLE ANALYSES

PROJECT NAME: Calaveras Geotechnical Survey
J.K. Spruce Power Plant
San Antonio, Texas

FILE NAME: ASA20-044-00.GPJ 9/3/2020

Boring No.	Sample Depth (ft)	Blows per ft	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	USCS	Dry Unit Weight (pcf)	% -200 Sieve	Shear Strength (tsf)	Strength Test
B-5	13.5 to 14.9	50/11"	12								
	18.5 to 20.0	31									
	23.5 to 24.8	50/10"	22						41		
	28.5 to 29.8	50/9"									
	33.5 to 34.6	50/7"									
	38.5 to 40.0	44									
	43.5 to 45.0	21	21						59		
	48.5 to 49.3	50/4"	30								
B-6	0.0 to 1.5	21	4								
	2.5 to 4.0	17	6						21		
	4.5 to 6.0	12	16								
	6.5 to 8.0	14	13	30	14	16	CL				
	8.5 to 10.0	10	14								
	13.5 to 15.0	43	15						29		
	18.5 to 20.0	48	13						31		
B-7	0.0 to 1.5	32	13	NP	NP	NP	ML				
	2.5 to 4.0	23	5						21		
	4.5 to 6.0	9	21								
	6.5 to 8.0	12	12								
	8.5 to 10.0	14	14	32	13	19	CL				
	13.5 to 15.0	49	13								
	18.5 to 20.0	42									
B-8	0.0 to 1.5	14	7								
	2.5 to 4.0	21	1			NP	SM		20		
	4.5 to 6.0	9	17								
	6.5 to 8.0	16	13	41	14	27	CL				
	8.5 to 10.0	25	12								
	13.5 to 15.0	41	12						35		
	18.5 to 20.0	22									
B-9	0.0 to 1.5	22	6								
	2.5 to 4.0	19	2								
	4.5 to 6.0	12	12								
	6.5 to 8.0		11	33	13	20	SC		41	2.25	PP
	8.5 to 10.0	12	22	44	19	25	CL				
	13.5 to 15.0		13	35	14	21	CL		60	2.25	PP
	18.5 to 20.0	29	13								
	23.5 to 25.0	47	22								
	28.5 to 29.9	50/11"	20								
	33.5 to 34.9	50/11"									

PP = Pocket Penetrometer TV = Torvane UC = Unconfined Compression FV = Field Vane UU = Unconsolidated Undrained Triaxial

CU = Consolidated Undrained Triaxial

RESULTS OF SOIL SAMPLE ANALYSES

PROJECT NAME: Calaveras Geotechnical Survey
J.K. Spruce Power Plant
San Antonio, Texas

FILE NAME: ASA20-044-00.GPJ 9/3/2020

Boring No.	Sample Depth (ft)	Blows per ft	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	USCS	Dry Unit Weight (pcf)	% -200 Sieve	Shear Strength (tsf)	Strength Test
B-9	38.5 to 40.0	40									
	43.5 to 45.0	34	24								
	48.5 to 50.0	24									
B-10	0.0 to 1.5	26	1								
	2.5 to 4.0	8	5						54		
	4.5 to 6.0		16							2.25	PP
	6.5 to 8.0	16	12	35	15	20	SC		36		
	8.5 to 10.0		24							2.25	PP
	13.5 to 15.0	40	11								
	18.5 to 20.0	48									
	23.5 to 24.8	50/9"	22	39	29	10	CL				
	28.5 to 28.9	ref/5"	27						47		
	33.5 to 34.8	50/10"									
	38.5 to 40.0	32	19								
	43.5 to 45.0	40									
	48.5 to 50.0	46	23						70		
B-11	1.0 to 2.5	9	16	30	21	9	SC		44		
	2.5 to 4.0	6	19						60		
	4.5 to 6.0	4	25								
	6.5 to 8.0	13	17								
	8.5 to 10.0	28	15	26	20	6	CL-ML				
	13.5 to 15.0	23	11								
	18.5 to 20.0	44	22	35	23	12	CL				
B-12	0.0 to 1.5	22	2								
	2.5 to 4.0	8	16	69	20	49	CH				
	4.5 to 6.0	17	11								
	6.5 to 8.0	18	11								
	8.5 to 10.0	49	13						32		
B-13	1.0 to 2.5	12	13								
	2.5 to 4.0	7	16	44	15	29	CL				
	4.5 to 6.0	8	16								
	6.5 to 7.9	50/11"	14								
	8.5 to 10.0	44	11	NP	NP	NP	SM		31		

PP = Pocket Penetrometer TV = Torvane UC = Unconfined Compression FV = Field Vane UU = Unconsolidated Undrained Triaxial

CU = Consolidated Undrained Triaxial

Project Number: ASA20-044-00
Test Date: July 30, 2020

RABA KISTNER

DCP TEST DATA

B-12

Calaveras Geotechnical Survey
J.K. Spruce Power Plant
San Antonio, Texas

Type of Ham.	No. of Blows	Penetration		CBR (%)	M_R (ksi)	q_{ult} (ksf)
		Incre. (mm)	Cumm. (in)			
1	1	35	1.4	5	7.5	1.59
1	1	15	2	14	21	3.15
1	3	21	2.8	33	49.5	5.57
1	3	17	3.5	42	63	6.54
1	4	14	4	72	108	9.35
1	4	16	4.6	62	93	8.46
1	4	12	5.1	85	127.5	10.44
1	5	11	5.6	121	181.5	13.20
1	5	11	6	121	181.5	13.20
1	5	8	6.3	172	258	16.67
1	1	1	6.3	292	438	23.69
-	-	-	-	-	-	-
-	-	-	-	-	-	-
-	-	-	-	-	-	-
-	-	-	-	-	-	-
-	-	-	-	-	-	-
-	-	-	-	-	-	-
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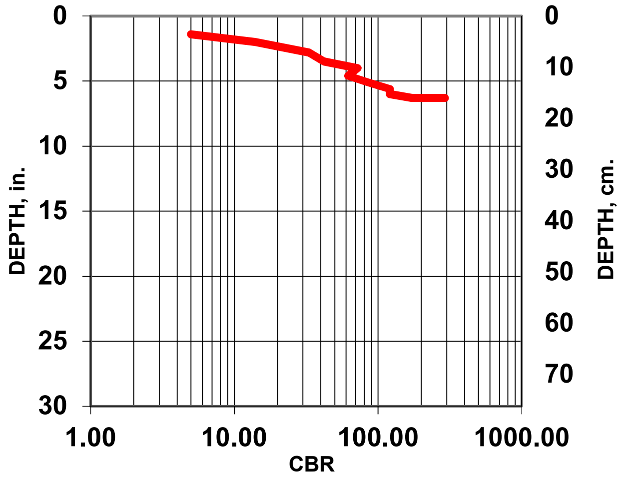
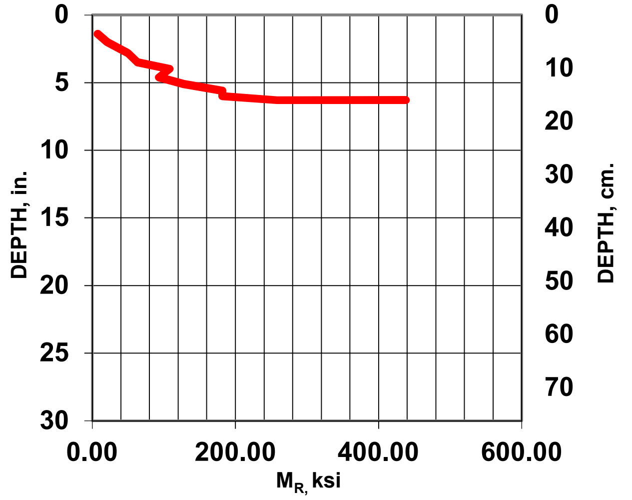
NOTES: Hammer 17.6 lbs = 1 Hammer 10.1 lbs = 2





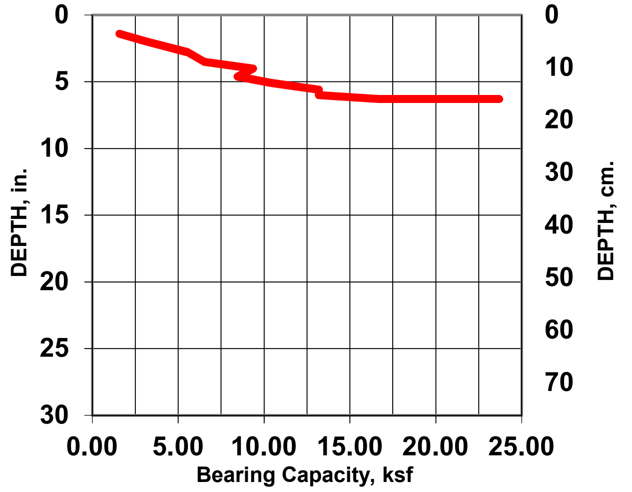


Figure 17a

Project Number: ASA20-044-00
Test Date: July 30, 2020

RABA KISTNER

DCP TEST DATA

B-13
Calaveras Geotechnical Survey
J.K. Spruce Power Plant
San Antonio, Texas

Type of Ham.	No. of Blows	Penetration		CBR (%)	M_R (ksi)	q_{ult} (ksf)
		Incre. (mm)	Cumm. (in)			
1	1	24	0.9	8	12	2.17
1	4	19	1.7	51	76.5	7.43
1	5	12	2.2	110	165	12.39
1	5	29	3.3	41	61.5	6.43
1	5	24	4.3	50	75	7.34
1	4	24	5.2	39	58.5	6.22
1	4	16	5.8	62	93	8.46
1	3	7	6.1	113	169.5	12.61
1	1	1	6.1	292	438	23.69
-	-	-	-	-	-	-
-	-	-	-	-	-	-
-	-	-	-	-	-	-
-	-	-	-	-	-	-
-	-	-	-	-	-	-
-	-	-	-	-	-	-
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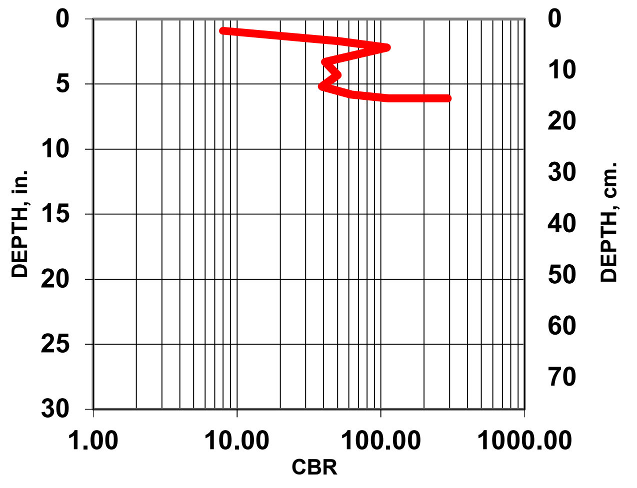
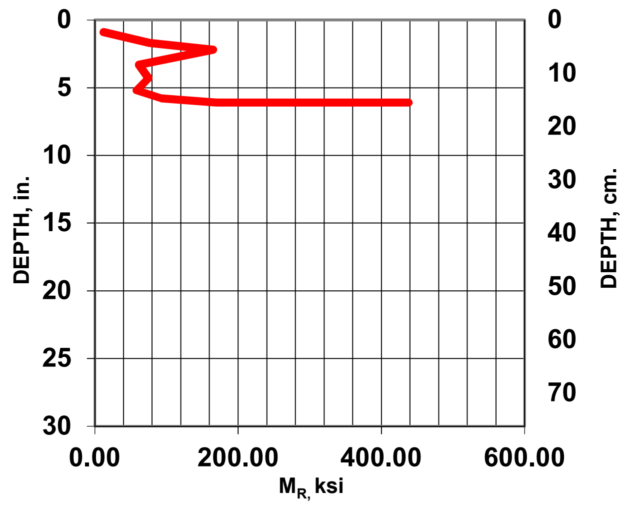
-	-	-	-	-	-	-
-	-	-	-	-	-	-
-	-	-	-	-	-	-
-	-	-	-	-	-	-
-	-	-	-	-	-	-

NOTES: Hammer 17.6 lbs = 1 Hammer 10.1 lbs = 2





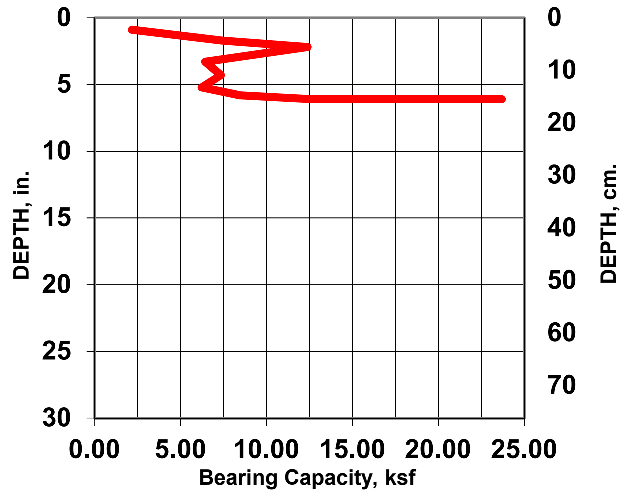
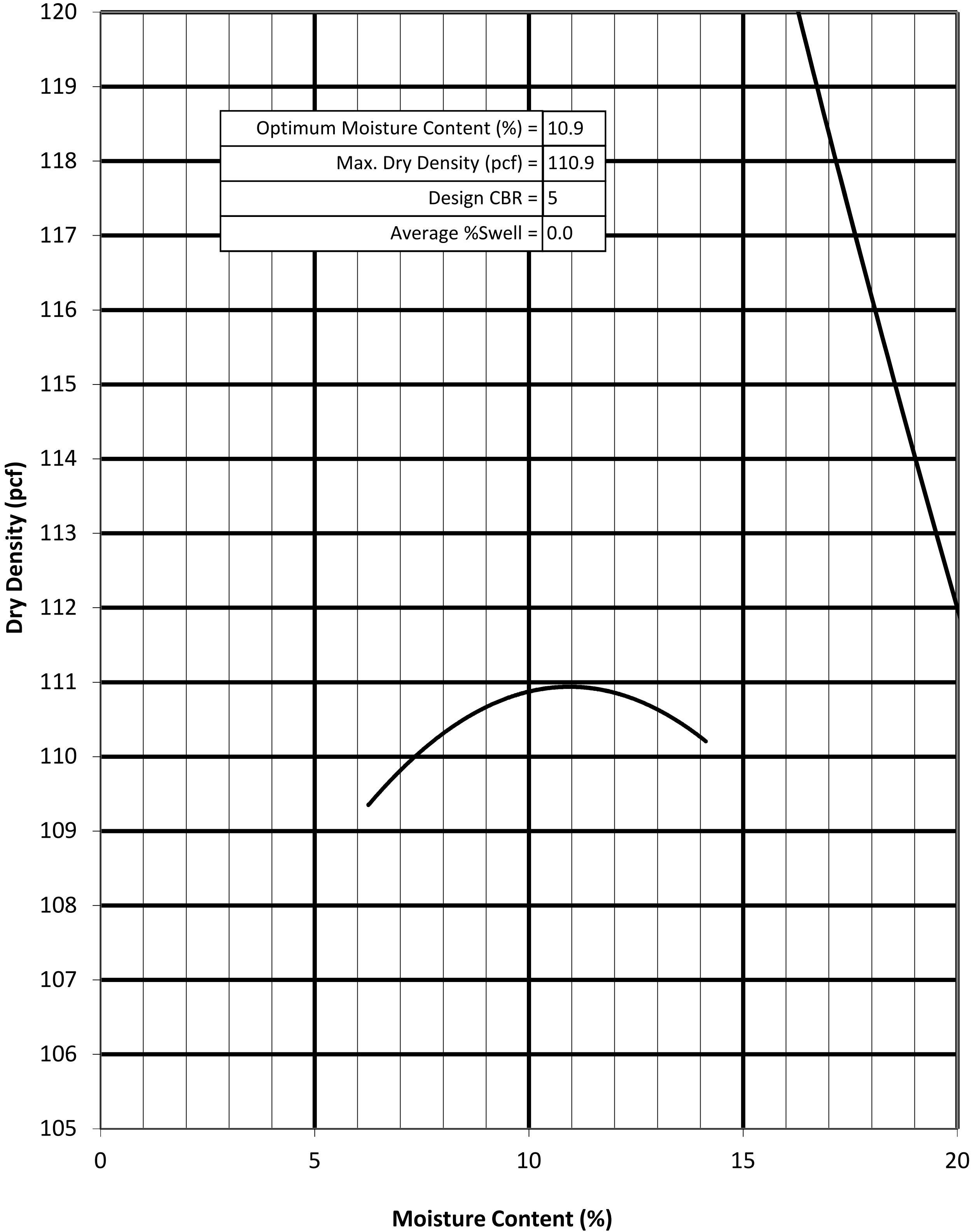


Figure 17b

MOISTURE-DENSITY RELATIONSHIP CURVE (ASTM D698)

Calaveras Geotechnical Survey
J. K. Spruce Power Plant
San Antonio, Texas

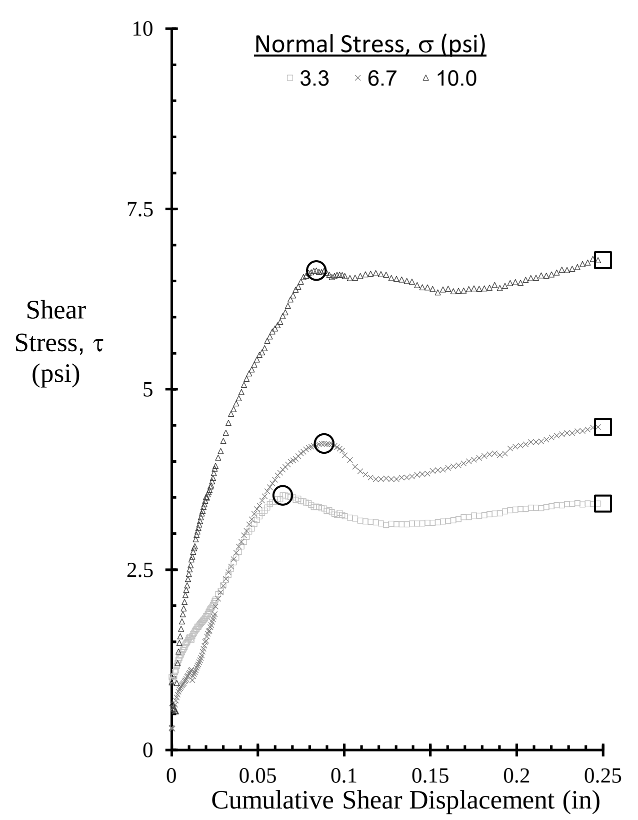


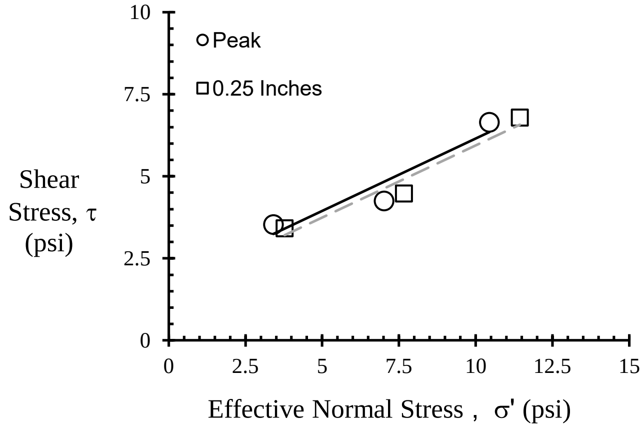
Optimum Moisture Content (%) =	10.9
Max. Dry Density (pcf) =	110.9
Design CBR =	5
Average %Swell =	0.0

Direct Shear of Soil Under Consolidated-Drained Conditions

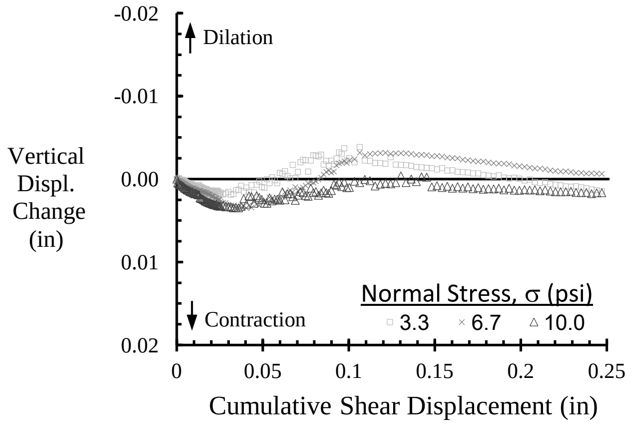
Client: Raba Kistner Consultants
Project: Calaveras Geotechnical Survey
Sample: B-5, S-5, (6.5-8)

TRI Log#: 58149.1
Test Method: ASTM D3080





Note: Area Correction Has Been Applied



Specimen Number		1	2	3
Initial Condition	Diameter, in	2.50	2.50	2.50
	Height, in (before consol)	1.00	1.00	1.00
	Water Content, %	16.8	16.2	16.4
	Saturation, %	74.8	69.9	70.0
	Dry Density, pcf	104.9	103.7	103.1
	Void Ratio	0.61	0.62	0.63
Consolidation Stress, σ' (psi)		3.3	6.7	10
Post-Consol	Height, in (prior to shear)	0.99	0.97	0.96
	Dry Density, pcf	106.0	106.4	107.5
	Void Ratio	0.61	0.60	0.58
Displacement rate (in/min)		1E-04		
Final Water Content, %		21.6	19.9	20.7
Peak	Normal Stress, σ' (psi)	3.41	7.02	10.45
	Shear Stress, τ (psi)	3.53	4.25	6.65
	Secant Friction Angle, Degrees	46.0	31.2	32.5
	Displacement (in)	0.06	0.09	0.08
	ϕ'_d, degrees	23.8		
	c'_d, psi	1.7		
0.25 Inches	Normal Stress, σ' (psi)	3.77	7.66	11.44
	Shear Stress, τ (psi)	3.42	4.48	6.79
	Secant Friction Angle, Degrees	42.1	30.3	30.7
	ϕ'_d, degrees	23.7		
	c'_d, psi	1.5		

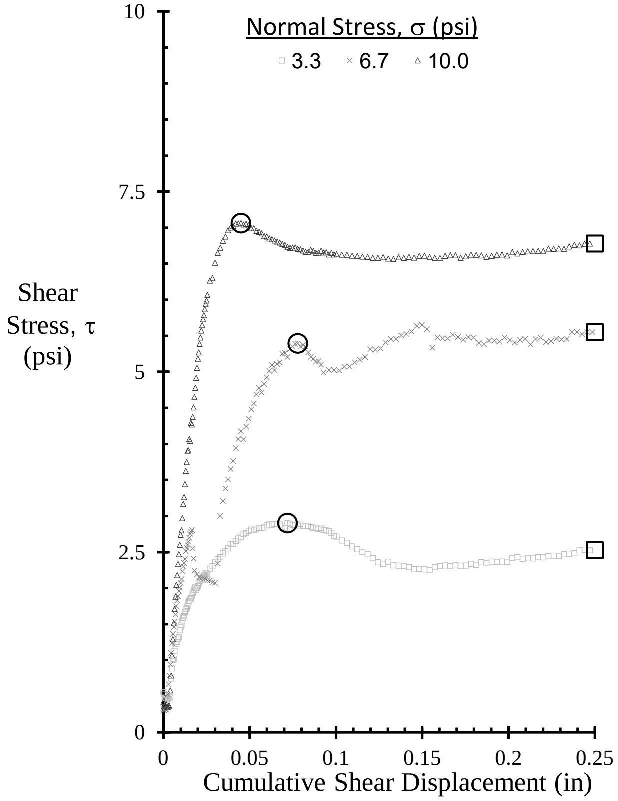
Note: The intact soil sample was extruded and specimens were prepared using a trimming turntable. A specific gravity of 2.70 was assumed for weight-volume calculations.

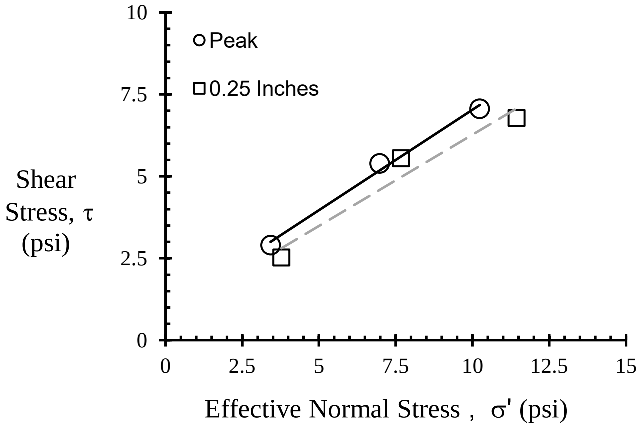
Jeffrey A. Kuhn, Ph.D., P.E., 8/21/20
Analysis & Quality Review/Date

Direct Shear of Soil Under Consolidated-Drained Conditions

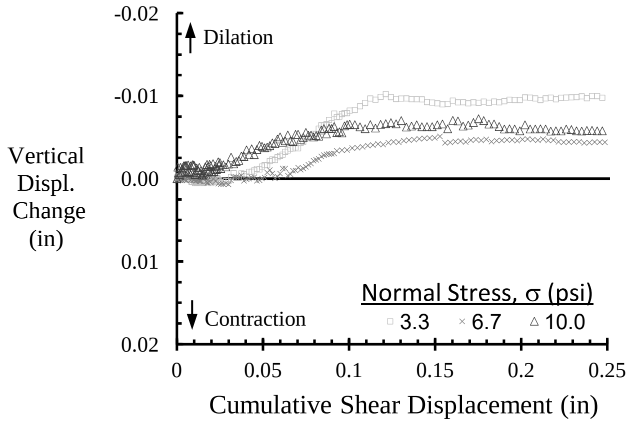
Client: Raba Kistner Consultants
Project: Calaveras Geotechnical Survey
Sample: B-9, S-5 (6.5-8)

TRI Log#: 58149.2
Test Method: ASTM D3080





Note: Area Correction Has Been Applied



	Specimen Number	1	2	3
Initial Condition	Diameter, in	2.50	2.50	2.50
	Height, in (before consol)	1.00	1.00	1.00
	Water Content, %	10.1	9.1	12.1
	Saturation, %	46.3	47.6	65.5
	Dry Density, pcf	106.2	111.2	112.3
	Void Ratio	0.59	0.51	0.50
Consolidation Stress, σ' (psi)		3.3	6.7	10
Post-Consol	Height, in (prior to shear)	0.99	0.97	0.95
	Dry Density, pcf	107.6	114.2	118.0
	Void Ratio	0.58	0.49	0.44
Displacement rate (in/min)		1E-04		
Final Water Content, %		19.0	20.1	19.1
Peak	Normal Stress, σ' (psi)	3.43	6.98	10.23
	Shear Stress, τ (psi)	2.90	5.39	7.06
	Secant Friction Angle, Degrees	40.3	37.7	34.6
	Displacement (in)	0.07	0.08	0.04
	ϕ'_d, degrees	31.5		
	c'_d, psi	0.9		
0.25 Inches	Normal Stress, σ' (psi)	3.77	7.67	11.44
	Shear Stress, τ (psi)	2.53	5.55	6.78
	Secant Friction Angle, Degrees	33.8	35.9	30.7
	ϕ'_d, degrees	29.1		
	c'_d, psi	0.7		

Note: The intact soil sample was extruded and specimens were prepared using a trimming turntable. A specific gravity of 2.70 was assumed for weight-volume calculations.

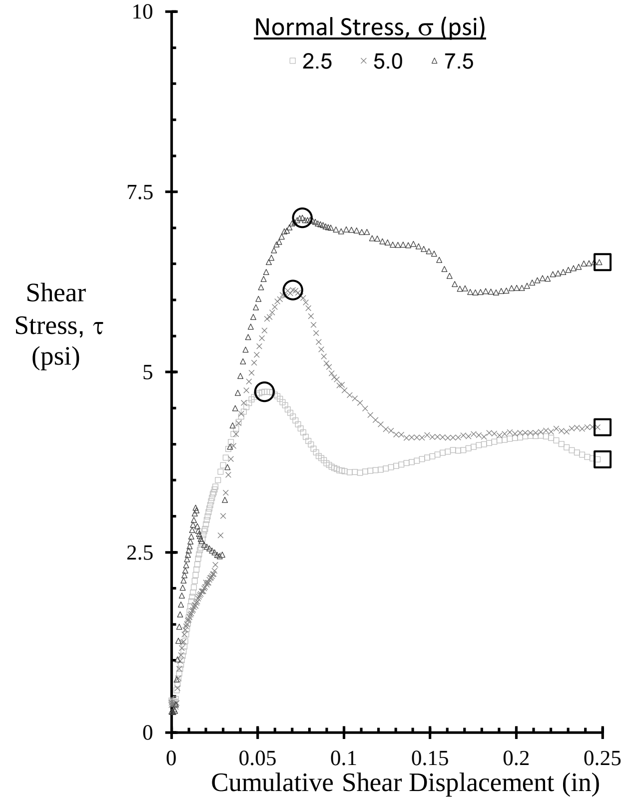
Jeffrey A. Kuhn, Ph.D., P.E., 9/3/20

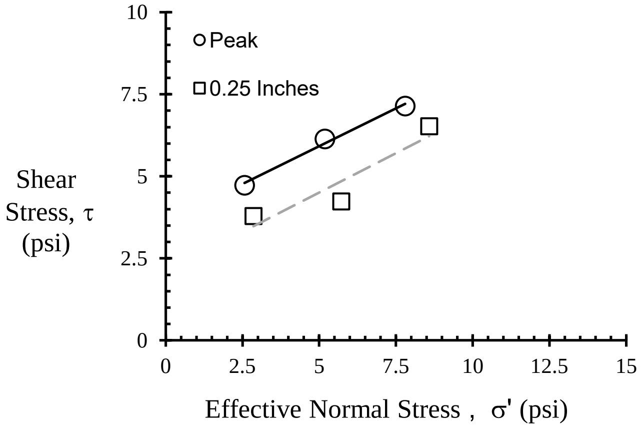
Analysis & Quality Review/Date

Direct Shear of Soil Under Consolidated-Drained Conditions

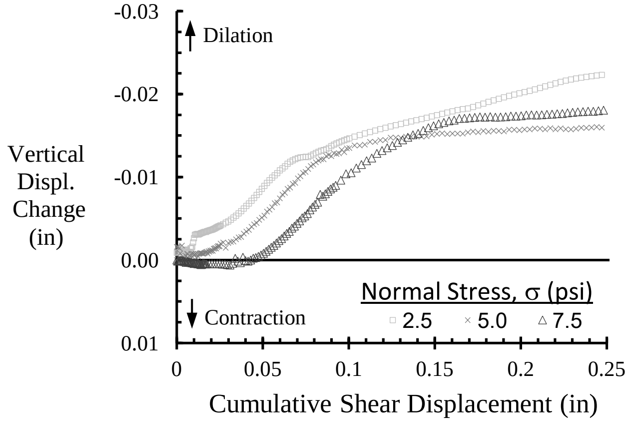
Client: Raba Kistner Consultants
Project: Calaveras Geotechnical Survey
Sample: B-10, S-4 (4.5-6)

TRI Log#: 58149.3
Test Method: ASTM D3080





Note: Area Correction Has Been Applied



	Specimen Number	1	2	3
Initial Condition	Diameter, in	2.50	2.50	2.50
	Height, in (before consol)	1.00	1.00	1.00
	Water Content, %	15.4	15.4	13.5
	Saturation, %	78.4	77.2	68.2
	Dry Density, pcf	110.2	109.4	109.9
	Void Ratio	0.53	0.54	0.53
	Consolidation Stress, σ' (psi)	2.5	5	7.5
Post-Consol	Height, in (prior to shear)	1.00	1.00	0.99
	Dry Density, pcf	110.2	109.7	111.1
	Void Ratio	0.55	0.55	0.53
	Displacement rate (in/min)	1E-04		
	Final Water Content, %	20.3	20.3	17.1
Peak	Normal Stress, σ' (psi)	2.57	5.19	7.80
	Shear Stress, τ (psi)	4.73	6.14	7.14
	Secant Friction Angle, Degrees	61.5	49.8	42.5
	Displacement (in)	0.05	0.07	0.08
	ϕ'_d, degrees	24.8		
	c'_d, psi	3.6		
0.25 Inches	Normal Stress, σ' (psi)	2.86	5.72	8.58
	Shear Stress, τ (psi)	3.79	4.23	6.52
	Secant Friction Angle, Degrees	53.0	36.5	37.2
	ϕ'_d, degrees	25.6		
	c'_d, psi	2.1		

Note: The intact soil sample was extruded and specimens were prepared using a trimming turntable. A specific gravity of 2.70 was assumed for weight-volume calculations.

Jeffrey A. Kuhn, Ph.D., P.E., 8/14/20

Analysis & Quality Review/Date

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.

GEOTECHNICAL
BUSINESS COUNCIL
of the Geoprofessional Business Association

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@geoprofessional.org www.geoprofessional.org

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RABA KISTNER

ATTACHMENT 21 METES AND BOUNDS

FIELD NOTE DESCRIPTION OF CALAVERAS LAKE

All that certain tract of land lying and situate about twelve miles in a Southeasterly direction from San Antonio, in Bexar County, Texas, said tract of land containing 7459.256 acres, more or less, and being out of the following surveys:

SURVEY	ORIGINAL GRANTEE	ABSTRACT	COUNTY BLOCK
121	Fernando Ruiz	619	5165
4	Jose De La Garza	4	4008
6	Juan Montez	11	4007
57	Jose Ma.Flores Perez	583	5149
5	Miguel Gortaris	252	5192
142	Pablo Villapando	772	5148
141	Pablo Villapando	773	5147
7	Miguel Gortaris	256	5140
6	Manuel Manjaros	463	5146
120	Edward Brown	58	5164

being more particularly described as follows, TO-WIT:

Beginning at a point in the Northwest right of way line of F.M. Road 1518, in the Fernando Ruiz Survey No. 121, Abstract No. 619, County Block 5165 in Bexar County, Texas; said point of beginning being 71.63 feet South 07° 39' 32" West along the Northwest right of way line of F.M. Road 1518 from its intersection with the Southwest right of way line of Stuart Road;

Thence with the Northwest right of way line of F.M. Road 1518 as follows:

South 07° 39' 32" West, 42.44 feet, to an angle point, which point is left 50.00 feet from and at a right angle to the centerline of said Road at Survey Station 173+53.61;

South 41° 34' 37" West, 3712.74 feet, parallel to the centerline of said Road, to a point in the centerline of Calaveras Creek;

Thence leaving said right of way line and with the centerline of Calaveras Creek upstream as follows:

North 61° 33' 30" West, 48.20 feet;
North 48° 20' 40" West, 117.10 feet;
North 67° 50' 25" West, 34.70 feet;
North 49° 42' 40" West, 60.40 feet;
North 56° 12' 02" West, 189.00 feet;
North 51° 24' 30" West, 64.62 feet;
North 46° 30' 00" West, 115.00 feet;
North 68° 07' 52" West, 67.00 feet;

Thence, leaving the centerline of Calaveras Creek, South 66° 28' 00" West, 2355.00 feet, to the Northeast right of way line of the Laguna Road;

Thence with the Northeast right of way line of the Laguna Road as follows:

North 53° 04' 00" West, 1161.40 feet;
North 53° 29' 00" West, 235.60 feet;
North 53° 25' 00" West, 235.60 feet, to a corner of Laguna Road;

South 60° 38' 27" West, 45.32 feet, with the edge of Laguna Road, to the Southeast right of way line of Laguna Road;

Thence leaving said right of way line, South 65° 15' 18" West, 262.34 feet;

Thence South 66° 18' 09" West, 597.66 feet to a 3/4 inch iron pin for corner of this tract;

Thence North 52° 43' 39" West, 349.50 feet to a 3/4 inch iron pin in fence line;

Thence North 55° 47' 39" West, 300.34 feet to a 3/4 inch iron pin for corner;

Thence North 34° 40'00" East, 399.68 feet to a 3/4 inch iron pin;

Thence North 34° 33' 00" East, 199.80 feet to a 1/2 inch iron pin;

Thence North 56° 35' 24" West, 63.00 feet to a 1/2 inch iron pin in the Southeast right of way line of Adkins-Elmendorf Road;

Thence North 56° 35' 09" West, 36.30 feet with edge of Adkins-Elmendorf Road to a 3/4 inch iron pin in the Northwest right of way line of said Road;

Thence, leaving said Road, North 09° 03' 51" East, 549.30 feet to a 3/4 inch pin in the abandoned Southwest right of way line of Laguna Road;

Thence North 54° 35' 35" West, 89.66 feet to a point on the East right of way line of Kilowatt Road (formerly F.M. Road 1518);

Thence North 63° 39' 21" West, 151.91 feet across the end of said Road, to a point on the West right of way line of said Road;

Thence, leaving said Road, South 59° 00' 02" West, 302.40 feet to a 3/4 inch iron pin for an interior corner;

Thence South 19° 19' 30" West, 607.50 feet to a 3/4 inch iron pin in the Northwest right of way line of Kilowatt Road ;

Thence with said right of way line as follows:

South 34° 27' 49" West, 59.78 feet;

South 33° 40' 27" West, 621.86 feet to the East flare corner of the intersection of Kilowatt Road and U.S. Highway No. 181;

South 73° 34' 16" West, 149.07 feet to the West flare corner of the intersection of Kilowatt Road and U.S. Highway No. 181;

Thence South 28° 30' 00" West, 250.00 feet across U.S. Highway No. 181 to the West flare corner of the intersection of Kilowatt Road and U.S. Highway No. 181;

Thence South 13° 50' 33" East, 103.19 feet to the East flare corner of the intersection of Kilowatt Road and U.S. Highway No. 181;

Thence with the Northwest right of way line of Kilowatt Road South 33° 45' 55" West, 634.99 feet to a corner;

Thence North 12° 33' 29" West, 878.90 feet to an iron pin in the Southwest right of way line of U.S. Highway No. 181;

Thence North 43° 15' 00" East, 259.00 feet across U.S. Highway No. 181 to a corner in the Northeast right of way line of said Highway;

Thence, leaving said Highway, North 02° 55' 23" East, 218.25 feet to an interior corner;

Thence North 62° 04' 23" West, 150.00 feet to a point;

Thence North 62° 10' 00" West, 201.25 feet to a point;

Thence North 62° 04' 23" West, 206.43 feet to a corner;

Thence North 12° 44' 02" West, 318.40 feet to a corner;

Thence North 58° 53' 58" East, 165.00 feet to a point;

Thence North 59° 22' 00" East, 266.33 feet to an interior corner;

Thence North 33° 08' 15" West, 640.28 feet to a corner;

Thence North 58° 32' 41" East, 64.01 feet to an iron pin;

Thence North 56° 57' 00" East, 10.99 feet to an interior corner;

Thence North 31° 23' 22" West, 140.74 feet to an interior corner;

Thence South 58° 35' 43" West, 172.99 feet to an iron pipe for corner;

Thence North 58° 57' 45" West, 471.00 feet to an interior corner;

Thence South 58° 48' 36" West, 538.21 feet to an interior corner;

Thence South 31° 58' 29" East, 97.39 feet to a corner;

Thence South 58° 42' 18" West, 360.00 feet to a corner;

Thence North 31° 58' 29" West, 98.05 feet to a 3/4 inch iron pin;

Thence North 34° 06' 57" West, 231.76 feet to a 3/4 inch iron pin for an interior corner;

Thence South 58° 14' 23" West, 670.90 feet to a 3/4 inch iron pin on the Northeast right of way line of U.S. Highway No. 181;

Thence South 04° 11' 00" West, 265.00 feet across U.S. Highway No. 181 to a 3/4 inch iron pin on the Southwest right of way line of said Highway;

Thence, leaving said Highway, South 58° 45' 59" West, 1339.11 feet to a 3/4 inch iron pin for corner;

Thence North 31° 12' 40" West, 542.91 feet to a 3/4 inch iron pin for corner;

Thence North 57° 48' 05" East, 335.25 feet;

Thence North 58° 20' 43" East, 248.12 feet;

Thence North 59° 30' 47" East, 430.17 feet to a 1/2 inch iron pin on the Southwest right of way of U.S. Highway No. 181;

Thence continuing North 59° 30' 47" East, 280.00 feet across U.S. Highway No. 181 to a 1/2 inch iron pin on the Northeast right of way line of said Highway;

Thence with the Northeast right of way line of U.S. Highway No. 181 as follows:

North 62° 16' 10" West, 243.62 feet to a point;

North 62° 24' 07" West, 386.71 feet to a 4 inch iron pin for corner;

Thence leaving said Highway, North 57° 53' 53" East, 241.12 feet to a 3/4 inch iron pin for an interior corner;

Thence North 51° 31' 04" West, 84.59 feet to a 3/4 inch iron pin;

Thence North 69° 49' 04" West, 127.68 feet to a 3/4 inch iron pin;

Thence North 61° 50' 00" West, 489.00 feet to an interior corner;

Thence South 60° 00' 00" West, 248.39 feet to corner in the Northeast right of way line of U.S. Highway No. 181;

Thence with the Northeast right of way line of said Highway North 61° 50' 00" West, 380.12 feet to a corner;

Thence leaving said Highway North 60° 00' 00" East, 1188.00 feet to an iron pin for an interior corner;

Thence North 29° 42' 41" West, 444.94 feet to an interior corner;

Thence South 58° 33' 50" West, 274.13 feet to a 3/4 inch iron pin for corner;

Thence North 63° 57' 34" West, 655.00 feet to a 3/4 inch iron pin for an interior corner;

Thence South 52° 33' 50" West, 258.22 feet to a corner;

Thence North 38° 23' 29" West, 495.00 feet to a 3/4 inch iron pin for an interior corner;

Thence South 52° 28' 14" West, 1056.34 feet to a 3/4 inch iron pin in the Northeast right of way line of U.S. Highway No. 181 for corner;

Thence with said right of way line North 62° 28' 05" West, 350.57 feet to a 1/2 inch iron pin for corner;

Thence, leaving said Highway, North 51° 29' 19" East, 341.06 feet to a 3/4 inch iron pin for an interior corner;

Thence North 38° 24' 10" West, 414.89 feet to a 3/4 inch iron pin for corner;

Thence North 51° 35' 50" East, 1006.50 feet to a 3/4 inch iron pin for corner;

Thence South 50° 26' 04" East, 94.90 feet to a point for an interior corner;

Thence North 50° 29' 30" East, 799.36 feet to a 3/4 inch iron pin for an interior corner;

Thence North 13° 30' 01" East, 600.08 feet to a 3/4 inch iron pin for corner;

Thence South 82° 29' 17" East, 884.46 feet to a 3/4 inch iron pin in the Southwest right of way line of Laguna Road;

Thence North 82° 40' 00" East, 80.00 feet with the edge of Laguna Road to the Northeast right of way line of said Road;

Thence, leaving said Road, North 70° 28' 20" East, 542.00 feet to an iron pin for corner;

Thence South 60° 28' 35" East, 128.02 feet to an iron pin for an interior corner;

Thence North 10° 35' 58" East, 486.02 feet to an iron pin for an interior corner;

Thence North 81° 58' 48" West, 162.00 feet to an iron pin for corner;

Thence North 28° 55' 16" West, 290.01 feet to an iron pin for an interior corner;

Thence South 44° 40' 39" West, 551.20 feet to a point;

Thence South 44° 55' 12" West, 384.99 feet for corner in the Northeast right of way line of Laguna Road;

Thence North 48° 51' 06" West, 136.95 feet with the Northeast right of way line of Laguna Road to a corner;

Thence, leaving said Road, North 44° 24' 16" East, 1000.22 feet for an interior corner;

Thence North 24° 04' 51" West, 74.58 feet for an interior corner;

Thence South 44° 20' 50" West, 84.42 feet to a corner;

Thence North 45° 42' 39" West, 69.34 feet to a corner;

Thence North 44° 17' 21" East, 89.84 feet for an interior corner;
Thence North 41° 14' 44" West, 127.35 feet for an interior corner;
Thence South 44° 19' 18" West, 130.09 feet to a point;
Thence South 46° 34' 07" West, 74.39 feet to a point;
Thence South 36° 44' 59" West, 18.13 feet to a point;
Thence South 45° 05' 28" West, 445.12 feet to a point;
Thence South 44° 42' 46" West, 390.56 feet to a corner on the Northeast right of way line of Laguna Road;
Thence North 47° 06' 32" West, 774.56 feet to a point;
Thence North 46° 54' 00" West, 289.70 feet to an iron pipe found at the intersection of the Southeast right of way line of Cassiano Road with the Northeast right of way line of Cassiano Road where said Road changes direction from Northeast to Northwest;
Thence continuing North 46° 54' 00" West, 13.08 feet with the Northeast right of way line of Cassiano Road to a corner;
Thence, leaving said Road, North 45° 21' 17" East, 853.02 feet to an iron pin for an interior corner;
Thence North 77° 41' 22" West, 337.91 feet to an iron pin for an interior corner;
Thence South 45° 25' 49" West, 679.94 feet to an iron pin found on the Northeast right of way line of Cassiano Road;
Thence with the edge of Cassiano Road South 18° 20' 02" West, 30.52 feet to the Southwest right of way line of said Road;
Thence with the Southwest right of way line of Cassiano Road, South 48° 03' 40" East, 238.43 feet to the intersection of the Southwest right of way line of Cassiano Road with the Northwest right of way line of Cassiano Road where said road changes direction from Southeast to Southwest;
Thence with the Northwest right of way line of said road, South 48° 05' 58" West, 481.52 feet to a 3/4 inch iron pin for corner;
Thence, leaving said Road, North 36° 07' 22" West, 106.90 feet to a 3/4 inch iron pin for an interior corner;
Thence North 83° 01' 32" West, 1,302.94 feet to a 3/4 inch iron pin for corner;
Thence North 57° 01' 22" East, 896.99 feet to a 3/4 inch iron pin for an interior corner;
Thence North 15° 35' 17" West, 199.99 feet to a 3/4 inch iron pin for an interior corner;
Thence North 26° 21' 02" West, 675.66 feet to a 3/4 inch iron pin for an interior corner;
Thence North 48° 10' 51" West, 137.31 feet to a point;
Thence North 48° 30' 10" West, 604.00 feet to a 3/4 inch iron pin for an interior corner;
Thence South 44° 07' 28" West, 2,638.52 feet to a 3/4 inch iron pin for corner;
Thence South 49° 16' 24" West, 1,685.93 feet to a 3/4 inch iron pin in the Northeast right of way line of U.S. Highway No. 181;
Thence South 74° 00' 00" West, 330.00 feet across U.S. Highway No. 181, to a 3/4 inch iron pin in the Southeast right of way line of said Highway;
Thence leaving said Highway, South 54° 20' 01" West, 1,000.26 feet to a 3/4 inch iron pin for an interior corner;
Thence South 43° 30' 45" West, 1,810.03 feet to a 3/4 inch iron pin for an interior corner;
Thence South 03° 28' 51" East, 890.06 feet to a 3/4 inch iron pin for an interior corner;
Thence South 53° 06' 52" East, 627.60 feet to a 1/2 inch iron pin in the Northeast right of way line of the Southern Pacific Railroad for corner;
Thence with said right of way line of the Southern Pacific Railroad as follows:
North 55° 18' 12" West, 51.95 feet to a point;
North 56° 31' 48" West, 102.56 feet to a point;
North 59° 29' 43" West, 102.72 feet to a point;

North 62° 43' 58" West, 102.54 feet to a point;
North 65° 34' 35" West, 137.03 feet to a point;
North 69° 01' 12" West, 967.51 feet to a 3/4 inch iron pin for corner;

Thence leaving said right of way line of the Southern Pacific Railroad, North 43° 27' 00" East, 888.23 feet to a 1-1/4 inch iron pipe;

Thence North 43° 22' 57" East, 1,199.65 feet to a 1-1/4 inch iron pipe;

Thence North 43° 38' 10" East, 191.89 feet to a 1-1/4 inch iron pipe;

Thence North 43° 36' 48" East, 1,457.84 feet to a 1/2 inch iron pin in the Southeast right of way line of U.S. Highway No. 181;

Thence North 42° 21' 00" East, 245.00 feet across U.S. Highway No. 181, to a 3/4 inch iron pin in the Northeast right of way line of U.S. Highway No. 181;

Thence leaving said Highway, North 42° 30' 44" East, 1595.46 feet to a 3/4 inch iron pin for an interior corner;

Thence North 47° 23' 22" West, 327.92 feet to a 3/4 inch iron pin for corner;

Thence North 27° 10' 09" East, 708.64 feet to a 3/4 inch iron pin for corner;

Thence North 59° 43' 13" East, 979.81 feet to a 3/4 inch iron pin for an interior corner;

Thence North 24° 09' 00" East, 383.65 feet to a 3/4 inch iron pin for an interior corner;

Thence North 45° 47' 32" West, 395.89 feet to a 3/4 inch iron pin;

Thence North 45° 47' 28" West, 804.09 feet to a 3/4 inch iron pin for corner;

Thence North 45° 24' 13" East, 800.08 feet to a 3/4 inch iron pin for corner;

Thence North 81° 29' 13" East, 999.76 feet to a 3/4 inch iron pin for an interior corner;

Thence North 00° 30' 32" West, 1,449.44 feet to a 3/4 inch iron pin for an interior corner;

Thence North 83° 59' 54" West, 1,200.28 feet to a 3/4 inch iron pin for corner;

Thence North 42° 30' 42" West, 1,199.88 feet to a 3/4 inch iron pin for an interior corner;

Thence South 83° 30' 22" West, 755.19 feet to a 3/4 inch iron pin for corner;

Thence North 03° 00' 14" East, 599.77 feet to a 3/4 inch iron pin for corner;

Thence South 85° 59' 32" East, 899.79 feet to a 3/4 inch iron pin for corner;

Thence South 63° 28' 35" East, 1,499.83 feet to a 3/4 inch pin for an interior corner;

Thence North 52° 31' 01" East, 399.88 feet to a 3/4 inch iron pin for an interior corner;

Thence North 28° 29' 55" West, 399.91 feet to a 3/4 inch iron pin for an interior corner;

Thence North 56° 29' 42" West, 1,500.06 feet to a 3/4 inch iron pin for an interior corner;

Thence North 65° 45' 09" West, 992.73 feet to a 3/4 inch iron pin for corner;

Thence North 48° 56' 08" West, 1,105.27 feet to a 1/2 inch iron pin in the Southeast right of way line of Foster Road for corner;

Thence with said right of way line of Foster Road, North 35° 44' 53" East, 468.66 feet to a 3/4 inch iron pin for corner;

Thence leaving said road, South 54° 17' 35" East, 200.02 feet to a 3/4 inch iron pin for an interior corner;

Thence North 35° 43' 05" East, 100.00 feet to a 3/4 inch iron pin for corner;

Thence South 54° 24' 43" East, 899.47 feet to a 3/4 inch iron pin for an interior corner;

Thence South 71° 24' 26" East, 1,844.02 feet to a 1/2 inch iron pin for an interior corner;

Thence North 35° 23' 20" East, 520.30 feet to the Southwest right of way of Hildebrandt Road;

Thence with the edge of Hildebrandt Road, North 43° 26' 22" East, 49.07 feet to the Northeast right of way line of said road for an interior corner;

Thence with the said Northeast right of way line of Hildebrandt Road, North 54° 05' 00" West, 1816.77 feet to a corner;

Thence leaving said road, North 36° 25' 04" East, 808.78 feet to a 3/4 inch iron pin for an interior corner;

Thence North 41° 37' 21" West, 1,055.46 feet to a 3/4 inch iron pin for corner;

Thence North 00° 47' 34" West, 699.50 feet to a 3/4 inch iron pin for an interior corner;

Thence South 80° 06' 48" West, 400.18 feet to a corner in the East right of way line of Foster Road;

Thence with the east right of way line of Foster Road North 00° 04' 54" West, 565.00 feet to a point;

Thence West, 95.00 feet across Foster Road to a 3/4 inch iron pin in the West right of way line of said road;

Thence, leaving said Road, North 65° 24' 13" West, 720.67 feet to a corner;

Thence North 51° 14' 31" West, 1526.83 feet to a 3/4 inch iron pin for corner;

Thence North 41° 14' 41" West, 1,580.04 feet to a 3/4 inch iron pin for corner;

Thence North 03° 09' 19" East, 1,615.74 feet to a 1/2 inch iron pin for an interior corner;

Thence North 06° 24' 17" West, 1,531.85 feet to a 3/4 inch iron pin for corner;

Thence North 46° 51' 34" East, 499.93 feet to a 3/4 inch iron pin for corner;

Thence South 66° 50' 12" East, 299.96 feet to a 3/4 inch iron pin for corner;

Thence South 19° 07' 58" West, 478.46 feet to a 3/4 inch iron pin for an interior corner;

Thence South 08° 16' 11" East, 189.18 feet to a 3/4 inch iron pin for an interior corner;

Thence South 57° 27' 03" East, 1,139.91 feet to a 3/4 inch iron pin for corner;

Thence South 10° 27' 28" East, 2098.58 feet to a 3/4 inch iron pin for an interior corner;

Thence South 51° 27' 20" East, 1,424.90 feet to a 3/4 inch iron pin in the West right of way line of Foster Road;

Thence East 95.00 feet across Foster Road to the East right of way line of said Road;

Thence with the East right of way line of Foster Road North 00° 04' 54" West, 580.00 feet to a corner;

Thence, leaving said Road, South 84° 11' 00" East, 544.55 feet for an interior corner;

Thence North 00° 11' 00" East, 183.80 feet to a corner;

Thence East, 3,444.80 feet to a corner;

Thence South 00° 15' 00" East, 3,755.00 feet to a 3/4 inch iron pin for an interior corner;

Thence South 50° 26' 39" East, 399.94 feet to a 3/4 inch iron pin for corner;

Thence South 01° 27' 42" East, 575.11 feet to a 3/4 inch iron pin for an interior corner;

Thence South 71° 56' 57" East, 898.54 feet to a 3/4 inch iron pin for an interior corner;

Thence North 00° 27' 34" West, 1,379.65 feet to a 3/4 inch iron pin for corner;

Thence North 84° 32' 20" East, 469.57 feet to a 3/4 inch iron pin for an interior corner;

Thence North 01° 32' 37" East, 939.96 feet to a 3/4 inch iron pin for corner;

Thence North 89° 32' 47" East, 500.35 feet to a 3/4 inch iron pin for corner;

Thence South 00° 57' 00" East, 724.91 feet to a 3/4 inch iron pin for an interior corner;

Thence North 52° 31' 53" East, 789.54 feet to a 3/4 inch iron pin for an interior corner;

Thence North 08° 27' 14" West, 1,149.71 feet to a 3/4 inch iron pin for corner;

Thence North 21° 32' 26" East, 749.61 feet to a 3/4 inch iron pin for corner;

Thence South 61° 17' 54" East, 203.44 feet to a 3/4 inch iron pin for corner;

Thence South 43° 24' 19" West, 149.92 feet to a 3/4 inch iron pin for an interior corner;

Thence South 15° 52' 20" West, 100.00 feet to a 3/4 inch iron pin for corner;

Thence South 29° 37' 49" West, 159.91 feet to a 3/4 inch iron pin for an interior corner;

Thence South 72° 24' 16" East, 209.37 feet to a 3/4 inch iron pin for corner;

Thence South 49° 25' 58" East, 805.93 feet to a 3/4 inch iron pin in the West right of way line of Gardner Road;

Thence with the edge of Gardner Road, North 89° 16' 42" East, 50.00 feet to the East right of way line of said Road;

Thence with the East right of way line of Gardner Road, North 00° 33' 00" West, 4,165.00 feet to a corner;

Thence leaving said Road, North 89° 09' 00" East, 2,277.00 feet to a corner;

Thence South 37° 45' 00" East, 2,905.00 feet to a 3/4 inch iron pin for an interior corner;

Thence North 51° 37' 03" East, 1,050.60 feet to a 3/4 inch iron pin to a point;

Thence North 51° 36' 50" East, 1,463.65 feet to a point;

Thence North 51° 30' 21" East, 52.71 feet to a corner;

Thence North 88° 26' 49" East, 406.59 feet to a 1/2 inch iron pin for an interior corner;

Thence North 00° 32' 25" West, 656.35 feet to a corner;

Thence North 89° 27' 39" East, 1,250.00 feet to a 1/2 inch iron pin for corner;

Thence South 01° 33' 16" East, 634.12 feet to a 1/2 inch iron pin for an interior corner;

Thence North 88° 26' 49" East, 973.77 feet to a 1/2 inch iron pin in the Northwest right of way line of Knowlton Road (formerly Stuart Road);

Thence with the Northwest right of way line of Knowlton Road, South 12° 24' 28" West, 201.35 feet for an interior corner of this tract and a corner of said Road;

Thence with the edge of Knowlton Road, South 77° 24' 17" East, 50.95 feet to a 1/2 inch iron pin in the Southeast right of way line of said Road;

Thence leaving said Road, North 73° 14' 36" East, 1,147.75 feet to a 1/2 inch iron pin for an interior corner;

Thence North 16° 16' 08" West, 800.00 feet to an interior corner;

Thence South 73° 14' 36" West, 708.79 feet to a corner in the Southeast right of way line of Knowlton Road;

Thence with said Southeast right of way line, North 12° 53' 35" East, 46.02 feet to an iron pin set in a 4"x4" concrete monument for corner;

Thence leaving said Road, North 73° 14' 34" East, 686.36 feet to an interior corner;

Thence North 36° 17' 24" East, 672.25 feet to a 3/4 inch iron pin for corner;

Thence North 74° 32' 58" East, 474.95 feet to a 3/4 inch iron pin for corner;

Thence South 11° 26' 54" East, 400.03 feet to a 3/4 inch iron pin;

Thence South 11° 27' 01" East, 1,600.15 feet to a 3/4 inch iron pin for an interior corner;

Thence North 28° 49' 40" East, 1,667.04 feet to a 3/4 inch iron pin for corner;

Thence South 87° 52' 27" East, 780.94 feet to a 3/4 inch iron pin for an interior corner;

Thence North 00° 12' 29" East, 669.91 feet to a 1/2 inch iron pin for corner;

Thence South 89° 47' 29" East, 30.00 feet to a 1/2 inch iron pin for an interior corner;

Thence North 00° 12' 31" East, 268.70 feet to a 1/2 inch iron pin for corner;

Thence North 41° 26' 15" East, 937.91 feet to a 1/2 inch iron pin in the Southwest right of way line of Sulphur Springs Road;

Thence with said right of way line of Sulphur Springs Road as follows:

South 58° 12' 30" East, 110.47 feet to a point;

South 56° 22' 59" East, 128.08 feet to a point;

South 52° 05' 51" East, 200.01 feet to a point;

Thence North 26° 31' 00" East, 73.00 feet across Sulphur Springs Road to a 3/4 inch iron pin in the Northeast right of way line of said Road;

Thence, leaving said Road, North 14° 05' 32" East, 300.02 feet to a 3/4 inch iron pin for an interior corner;

Thence North 23° 17' 58" West, 615.82 feet to a 3/4 inch iron pin for an interior corner;

Thence North 77° 13' 35" West, 624.11 feet to a 3/4 inch iron pin for corner;

Thence North 14° 06' 44" East, 859.23 feet to a 3/4 inch iron pin for corner;

Thence South 65° 40' 34" East, 253.20 feet to a 3/4 inch iron pin for an interior corner;

Thence North 41° 59' 18" East, 628.54 feet to a 3/4 inch iron pin;

Thence North 41° 59' 14" East, 1,445.47 feet to a 3/4 inch iron pin for an interior corner;

Thence North 14° 12' 14" East, 535.03 feet to a 3/4 inch iron pin for an interior corner;

Thence North 34° 10' 08" West, 1424.05 feet to a 3/4 inch iron pin for corner;

Thence North 15° 25' 57" West, 1,016.42 feet to a 3/4 inch iron pin for corner;

Thence South 77° 30' 43" East, 299.85 feet to a 3/4 inch iron pin for corner;

Thence South 26° 00' 41" East, 676.48 feet to a 3/4 inch iron pin;

Thence South 26° 00' 37" East, 1,751.39 feet to a 3/4 inch iron pin for an interior corner;

Thence South 76° 30' 50" East, 389.05 feet to a 3/4 inch iron pin for an interior corner;

Thence North 09° 58' 59" East, 585.21 feet to a 3/4 inch iron pin for corner;

Thence South 78° 13' 09" East, 199.98 feet to a 3/4 inch iron pin for corner;

Thence South 07° 47' 36" West, 669.99 feet to a 3/4 inch iron pin for corner;

Thence South 17° 40' 45" West, 1,343.20 feet to a 3/4 inch iron pin for corner;

Thence South 77° 08' 02" West, 432.03 feet to a 3/4 inch iron pin for an interior corner;

Thence South 36° 05' 50" West, 799.99 feet to a 3/4 inch iron pin for an interior corner;

Thence South 79° 38' 40" East, 749.85 feet to a 3/4 inch iron pin for corner;

Thence South 20° 51' 23" West, 699.89 feet to a 3/4 inch iron pin for an interior corner;

Thence South 04° 24' 49" East, 26.77 feet to a point;

Thence South 04° 24' 06" East, 743.33 feet to a 3/4 inch iron pin for corner;

Thence South 45° 16' 17" West, 388.14 feet to a 3/4 inch iron pin for an interior corner;

Thence South 20° 35' 46" West, 275.15 feet to a 3/4 inch iron pin for an interior corner;

Thence South 01° 15' 58" East, 120.84 feet to a 3/4 inch iron pin in the Northeast right of way line of Sulphur Springs Road;

Thence South 11° 05' 00" West, 120.00 feet across Sulphur Springs Road to a 3/4 inch iron pin in the Southwest right of way line of said Road;

Thence, leaving said Road, South 14° 34' 33" West, 530.08 feet to a 3/4 inch iron pin for corner;

Thence South 60° 01' 54" West, 290.04 feet to a 3/4 inch iron pin for an interior corner;

Thence South 16° 28' 56" West, 410.83 feet to a 3/4 inch iron pin for corner;

Thence South 52° 12' 30" West, 202.03 feet to a 3/4 inch iron pin for an interior corner;

Thence South 16° 18' 17" East, 393.74 feet to a 3/4 inch iron pin for corner;

Thence South 73° 28' 12" West, 580.70 feet to a 3/4 inch iron pin for an interior corner;

Thence South 19° 42' 29" East, 709.79 feet to a 3/4 inch iron pin for corner;

Thence South 83° 03' 09" West, 1,000.32 feet to a 3/4 inch iron pin for an interior corner;

Thence South 45° 23' 28" West, 689.06 feet to a 3/4 inch iron pin for an interior corner;

Thence South 08° 57' 13" West, 581.74 feet to a 3/4 inch iron pin for corner;

Thence South 80° 26' 58" West, 350.05 feet to a 3/4 inch iron pin for an interior corner;

Thence South 11° 56' 42" West, 1,000.07 feet to a 3/4 inch iron pin for an interior corner;

Thence South 29° 33' 33" East, 282.01 feet to a 3/4 inch iron pin in the Northwest right of way line of the relocated Stuart Road;

Thence continuing South 29° 33' 33" East, 101.00 feet across Stuart Road to a point in the Southwest right of way line of said Road;

Thence leaving said Road and continuing South 29° 33' 33" East, 315.39 feet to an iron pin for corner;

Thence South 60° 41' 31" West, 555.07 feet to an iron pin for corner;

Thence North 33° 19' 41" West, 430.78 feet to a point in the Southeast right of way line of the relocated Stuart Road;

Thence continuing North 33° 19' 41" West, 101.00 feet across Stuart Road to a point in the Northwest right of way line of said Road;

Thence leaving said Road and continuing North 33° 19' 41" West, 19.16 feet to an iron pin for an interior corner;

Thence South 73° 09' 18" West, 1781.31 feet to a 3/4 inch iron pin for an interior corner;

Thence South 31° 39' 08" East, 250.02 feet to a 3/4 inch iron pin for corner;

Thence South 06° 16' 23" West, 1480.49 feet to a 3/4 inch iron pin for an interior corner;

Thence South 39° 43' 45" East, 620.55 feet to a 3/4 inch iron pin for an interior corner;

Thence North 74° 01' 48" East, 564.92 feet to a 3/4 inch iron pin in the Northwest right of way line of Stuart Road;

Thence with the Northwest right of way line of Stuart Road as follows:

South 03° 59' 34" West, 842.14 feet to a point;

South 50° 25' 23" West, 14.44 feet to a point;

Thence South 31° 15' 00" East, 99.00 feet across Stuart Road to a 3/4 inch iron pin in the Southwest right of way line of said Road;

Thence leaving said Road, South 70° 17' 11" East, 784.46 feet to a 3/4 inch iron pin for corner;

Thence South 06° 47' 03" West, 386.65 feet to a corner;

Thence South 73° 39' 42" West, 892.22 feet to a corner in the Southeast right of way line of Stuart Road;

Sold - See Attached Notes

Thence with the Southeast right of way line of Stuart Road North 06° 44' 57" East, 198.69 feet to the most westerly corner of the San Lorenzo Cemetery;

Thence South 82° 59' 00" East, with the Southwest line of said Cemetery, 208.40 feet to an interior corner of herein described tract;

Thence North 09° 44' 31" East, 310.79 feet to an interior corner;

Thence North 83° 38' 04" West, 203.70 feet to a 3/4 inch iron pin in the Southeast right of way line of Stuart Road;

Thence North 29° 37' 00" West, 98.00 feet across Stuart Road to a 3/4 inch iron pin in the Northwest right of way line of said Road;

Thence, leaving said Road, North 60° 10' 32" West, 496.41 feet to a 3/4 inch iron pin for an interior corner;

Thence South 73° 43' 26" West, 1100.50 feet to a 3/4 inch iron pin;

Thence South 73° 43' 14" West, 200.02 feet to a 3/4 inch iron pin for an interior corner;

Thence South 33° 59' 04" West, 1,284.12 feet to a 3/4 inch iron pin for an interior corner;

Thence South 85° 28' 15" East, 499.95 feet to a 3/4 inch iron pin for corner;
Thence South 75° 26' 46" East, 470.25 feet to a 3/4 inch iron pin for corner;
Thence South 16° 27' 24" East, 199.98 feet to a 3/4 inch iron pin for corner;
Thence South 73° 36' 13" West, 329.18 feet to a 3/4 inch iron pin for an interior corner;
Thence South 36° 28' 22" West, 390.09 feet to a 3/4 inch iron pin for corner;
Thence South 52° 05' 07" West, 710.51 feet to a 3/4 inch iron pin for an interior corner;
Thence South 14° 38' 41" West, 253.97 feet to a 3/4 inch iron pin for an interior corner;
Thence North 72° 13' 53" East, 207.98 feet to a 3/4 inch iron pin for an interior corner;
Thence North 49° 15' 01" East, 216.10 feet to a 3/4 inch iron pin for corner;
Thence South 59° 24' 32" East, 233.38 feet to a 3/4 inch iron pin for corner;
Thence South 35° 55' 04" East, 639.90 feet to a 3/4 inch iron pin for corner;
Thence South 17° 12' 18" East, 336.66 feet to a 3/4 inch iron pin for corner;
Thence South 49° 07' 15" West, 1,646.47 feet to a 3/4 inch iron pin for an interior corner;
Thence South 00° 49' 14" West, 599.92 feet to a 3/4 inch iron pin for corner;
Thence South 37° 50' 21" West, 599.97 feet to a 3/4 inch iron pin for an interior corner;
Thence South 51° 10' 32" East, 703.38 feet to a 3/4 inch iron pin for an interior corner;
Thence North 76° 53' 51" East, 595.12 feet to a 1/2 inch iron pin for an interior corner;
Thence North 16° 59' 08" West, 37.76 feet to a 1/2 inch iron pin for corner;
Thence North 73° 36' 03" East, 187.70 feet to a 3/4 inch iron pin for corner;
Thence South 16° 51' 43" East, 723.49 feet to a 3/4 inch iron pin for corner;
Thence South 15° 42' 06" East, 695.28 feet to a 3/4 inch iron pin for an interior corner;
Thence North 74° 18' 06" East, 379.66 feet to a point;
Thence North 74° 03' 50" East, 412.33 feet to a 3/4 inch iron pin for corner;
Thence South 01° 46' 16" East, 1,509.97 feet to a 3/4 inch iron pin for an interior corner;
Thence South 17° 06' 48" East, 973.42 feet to a 3/4 inch iron pin in the Northwest right of way line of Bernhardt Road (formerly F.M. Road 1518);
Thence with the edge of Bernhardt Road South 12° 47' 15" East, 80.12 feet to a 3/4 inch iron pin in the Southeast right of way line of said Road;
Thence, leaving said Road, South 13° 11' 27" East, 674.93 feet to a 3/4 inch iron pin for corner;
Thence South 50° 35' 24" West, 416.09 feet to a 3/4 inch iron pin for an interior corner;
Thence South 14° 55' 13" East, 1,730.73 feet to a 3/4 inch iron pin for an interior corner;
Thence North 71° 45' 42" East, 60.77 feet to a corner;
Thence South 16° 20' 18" East, 800.00 feet to a 3/4 inch iron pin for an interior corner;
Thence North 73° 48' 51" East, 996.15 feet to the Point of Beginning.
LESS AND EXCEPT 10.08 acres, more or less, for U.S. Highway No. 181; 3.46 acres, more or less, for Foster Road; 2.29 acres, more or less, for Sulphur Springs Road; and 1.91 acres, more or less, for Stuart Road.

Containing 7,459.256 acres of land, more or less.

MERRITT W. KEEL
11601

I certify that this description as represented by survey notes was prepared under my direction.

Merritt W. Keel, P.E.