Appendix G:
Geology and Soils Supporting Information

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GEOTECHNICAL INVESTIGATION

for

2600 CAMINO RAMON
San Ramon, California

Prepared For:

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San Ramon, California

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1.0 INTRODUCTION AND BACKGROUND

This report presents the results our geotechnical investigation for the parking garage proposed for 2600 Camino Ramon in San Ramon, California. This investigation was performed in accordance with our proposal dated 15 November 2013.

The site consists of a portion of an asphalt-paved parking lot in the southeast corner of the property at 2600 Camino Ramon. The location of the site is shown on Figure 1. The site is approximately rectangular with plan dimensions of about 300 feet by 400 feet and is bound by Bishop Drive on the south, parking lots and driveways on the east and north, and a driveway and a four-story building on the west. The site is currently occupied by an asphalt-paved parking lot and driveways, landscaped areas, and below-grade utilities. The majority of the site is relatively level, with ground surface elevations between approximately 450 to 453 feet[1], as shown on Figure 2. A narrow berm slopes up about three to four feet higher than surrounding grades between the existing parking lot and Bishop Drive.

We understand plans are to demolish the existing improvements within the footprint of the new development and construct a 5-story parking garage at grade. Maximum plan dimensions for the parking garage are about 375 by 190 feet. We anticipate additional improvements will include new asphalt and concrete pavement, concrete flatwork, and landscaping adjacent to the parking garage. We understand the finished floor elevation will range from 446 to 454 feet[2], with pad subgrade elevations about one foot lower. Therefore, cuts to about seven feet and fills up to about two feet will be needed. Dead plus live loads were estimated by the project structural engineer to be about 300 to 700 kips for frame columns, 300 to 625 kips for non-frame columns, and 875 to 1200 kips for girder columns.

2.0 SCOPE OF SERVICES

Our scope of services, outlined in our proposal dated 15 November 2013, consisted of reviewing available subsurface information for the site vicinity, exploring the subsurface conditions at the site, and performing laboratory tests and engineering analyses to develop conclusions and recommendations regarding:

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[1] Elevations are from a topographic survey provided by IDG Parkitects, Inc. on 15 January 2014 and are based on the National Geodetic Vertical Datum of 1929 (NGVD 1929).

[2] From Conceptual Grading and Drainage Site Plan by RJA dated 19 March 2014

- soil and groundwater conditions at the site
- appropriate foundation type(s) for the proposed parking garage
- design criteria for the most appropriate foundation type(s), including values for vertical and lateral capacities
- estimated foundation settlement
- below-grade walls
- excavation
- temporary shoring
- seismic hazards, including ground rupture, liquefaction, and differential compaction
- seismic design criteria in accordance with the 2013 California Building Code (CBC)
- floor slabs
- concrete flatwork
- flexible (asphalt concrete) and rigid (Portland cement concrete) pavement design
- utility trenches
- fill quality and compaction criteria
- site grading, including criteria for fill quality and compaction
- subgrade preparation and moisture protection for floor slabs
- corrosion potential of near-surface soil
- construction considerations.

Note that because the parking garage will not have a basement, we have not included recommendations for temporary shoring.

3.0 FIELD INVESTIGATION

Subsurface conditions were explored at the site by drilling three borings, designated B-1 through B-3, and performing five cone penetration tests (CPTs), designated CPT-1 through CPT-5. The approximate locations of the borings and CPTs are presented on Figure 2. Prior to performing our field investigation, we obtained drilling permits from Contra Costa County Environmental Health Division (CCCEHD), notified Underground Service Alert, and retained a private underground utility locating service to check that locations of exploratory points were clear of existing utilities.

The borings were drilled on 21 December 2013 to depths between about 50 and 51½ feet below the existing ground surface (bgs) using a truck-mounted drill rig equipped with hollow stem augers and operated by Exploration Geoservices, Inc. of San Jose, California. During drilling, our field engineer logged the borings and obtained representative samples of the soil encountered for classification and laboratory testing. The boring logs are presented in Appendix A on Figures A-1 through A-3. The soil encountered in the borings was classified in accordance with the soil classification system presented on Figure A-4. Soil samples were obtained using three different types of samplers: two driven split-barrel samplers and one pushed thin-walled sampler. The sampler types are as follows:

- Sprague & Henwood (S&H) split-barrel sampler with a 3.0-inch outside diameter and 2.5-inch inside diameter, lined with steel tubes with an inside diameter of 2.43 inches

- Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and 1.5-inch inside diameter, without liners.

- Shelby Tube (ST) sampler with a 3.0-inch outside diameter and a 2.875-inch inside diameter.

The sampler types were chosen on the basis of soil type being sampled and desired sample quality for laboratory testing. In general, the S&H sampler was used to obtain samples in medium stiff to very stiff cohesive soil and the SPT sampler was used to evaluate the relative density of sandy soil. The Shelby Tube sampler was used to obtain relatively undisturbed samples of soft to medium stiff cohesive soil.

The SPT and S&H samplers were driven with 140-pound, hydraulic trip wireline safety hammer falling 30 inches. The samplers were driven up to 18 inches and the hammer blows required to drive the samplers every six inches of penetration were recorded and are presented on the boring logs. A "blow count" is defined as the number of hammer blows per six inches of penetration or 50 blows for six inches or less of penetration. The driving of samplers was discontinued if the observed (recorded) blow count was 50 for six inches or less of penetration. The blow counts required to drive the S&H

sampler were converted to approximate SPT N-values using a factor of 0.6 to account for sampler type and hammer energy and are shown on the boring logs. The blow counts used for this conversion were the last two blow counts of the 18-inch sampler drive.

The Shelby Tube sampler is pushed hydraulically into the soil; the pressure required to advance the sampler is shown on the logs, measured in pounds per square inch (psi).

The CPTs were advanced to depths between about 38½ and 80 feet bgs on 21 December 2013 by John Sarmiento & Associates of Orinda, California. The CPT logs presents tip resistance and friction ratio by depth, as well as interpreted standard penetration test blow counts, soil shear strength parameters, and soil classifications. The logs of the CPTs are presented in Appendix B on Figures B-1 through B-5. The classification chart for the CPT logs is presented on Figure B-6.

The CPTs were performed by hydraulically pushing a 1.75-inch-diameter, cone-tipped probe with a projected area of 15 square centimeters into the ground. The cone-tipped probe measures tip resistance, and the friction sleeve behind the cone tip measures frictional resistance. Electrical strain gauges within the cone continuously measure soil parameters for the entire depth advanced. Soil data, including tip resistance and frictional resistance, were recorded by a computer while the test was conducted. Accumulated data were processed by computer to provide engineering information such as the types and approximate strength characteristics of the soil encountered.

Upon completion of the field investigation, the borings and CPTs were backfilled with cement grout in accordance with CCCEHD requirements and under the observation of a CCCEHD inspector, and pavement surfaces were patched. The soil cuttings from the borings were placed into 55-gallon drums which were stored temporarily at the site, tested, and transported off-site for disposal.

4.0 LABORATORY TESTING

Soil samples obtained from the borings were re-examined in the office for classification and representative samples were selected for laboratory testing. The laboratory testing program was designed to correlate and evaluate engineering properties of the soil at the site. Soil samples were tested to measure moisture content, dry density, fines content, Atterberg limits, strength, compressibility, and resistance value (R-value). The geotechnical laboratory test results are presented on the boring logs and in Appendix C. Testing was also performed on a near-surface sample from boring B-1 to evaluate the corrosion potential of the soil. The results of the corrosivity testing are presented on Figure C-6 in Appendix C.

5.0 SUBSURFACE CONDITIONS

Subsurface information from our field investigation indicates portions of the site are underlain by a thin layer of fill consisting of stiff to very stiff clay. The fill was encountered in borings B-1 and B-2 and is between about 1 and 1½ feet thick. The pavement section as measured in the borings consists of 3 inches of asphalt concrete over 21 inches of aggregate base.

Beneath the pavement section and fill (where present), the soil at the site generally consists of medium stiff to very stiff clay to the maximum depth explored of about 80 feet bgs. Results of Atterberg limits tests performed on the near-surface clay indicate it has a moderate expansion potential,[3] with a plasticity index of 23. Thin layers of granular soil consisting of sand with silt, silty sand with variable gravel content, clayey sand, and clayey silty sand were encountered within the clay in the borings and CPTs. The granular layers are loose to medium dense and range in thickness from about 1 to 4 feet.

Groundwater was measured in the borings and CPTs between depths of about 14.7 and 29 feet bgs, which correspond to approximate minimum and maximum Elevations 421 and 437.3 feet. The groundwater levels observed during drilling do not represent stable groundwater conditions, and the groundwater level at the site is expected to vary seasonally.

6.0 REGIONAL SEISMICITY

The major active faults in the area are the Calaveras, Mount Diablo Thrust, and Hayward faults. These and other faults in the region are shown on Figure 3. For each of the active faults within 50 kilometers (km) of the site, the distance from the site and estimated maximum Moment magnitude,[4] M_w, [Working

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[3] Expansive soil undergoes large volume changes with changes in moisture content (i.e. it shrinks when dried and swells when wetted.)

[4] Moment magnitude is an energy-based scale and provides a physically meaningful measure of the size of a faulting event. Moment magnitude is directly related to average slip and fault rupture area.

Group on California Earthquake Probabilities (WGCEP) (2008) and Cao et al. (2003)] are summarized in Table 1.

TABLE 1
Regional Faults and Seismicity

Fault Segment	**Approximate Distance from Site (km)**	**Direction from Site**	**Mean Characteristic Moment Magnitude**
Total Calaveras	0.9	West	7.03
Mount Diablo Thrust	5.0	Northeast	6.70
Total Hayward	14	West	7.00
Total Hayward-Rodgers Creek	14	West	7.33
Green Valley Connected	15	North	6.80
Greenville Connected	17	East	7.00
Great Valley 5, Pittsburg Kirby Hills	30	Northeast	6.70
Great Valley 7	39	East	6.90
N. San Andreas - Peninsula	44	West	7.23
N. San Andreas (1906 event)	44	West	8.05
Monte Vista-Shannon	44	Southwest	6.50

Figure 3 also shows the earthquake epicenters for events with magnitude greater than 5.0 from January 1800 through January 1996. In 1868, an earthquake with an estimated maximum intensity of X on the MM scale occurred on the southern segment (between San Leandro and Fremont) of the Hayward Fault. The estimated M_w for the earthquake is 7.0. In 1861, an earthquake of unknown magnitude (probably a M_w of about 6.5) was reported on the Calaveras Fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill earthquake (M_w = 6.2).

Since 1800, four major earthquakes have been recorded on the San Andreas Fault. In 1836 an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale (Figure 4) occurred east of Monterey Bay (Toppozada and Borchardt 1998). The estimated M_w for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to a M_w of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas Fault from Shelter Cove to San Juan Bautista, approximately 470 kilometers in length. It had a maximum intensity of XI (MM), a M_w of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The most recent earthquake to affect the Bay Area was the Loma Prieta Earthquake of 17 October 1989 with a M_w of 6.9 and an epicenter in the Santa Cruz Mountains, approximately 81 km from the site.

The 2008 WGCEP at the U.S. Geologic Survey (USGS) predicted a 63 percent chance of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Area in 30 years. More specific estimates of the probabilities for different faults in the Bay Area are presented in Table 2.

TABLE 2

WGCEP (2008) Estimates of 30-Year Probability of a Magnitude 6.7 or Greater Earthquake

Fault	Probability (percent)
Hayward-Rodgers Creek	31
N. San Andreas	21
Calaveras	7
San Gregorio	6
Concord-Green Valley	3
Greenville	3
Mount Diablo Thrust	1

7.0 DISCUSSION AND CONCLUSIONS

We conclude that from a geotechnical engineering standpoint, the site can be developed as planned, provided the recommendations presented in this report are incorporated into the project plans and specifications, and are implemented during construction. The primary geotechnical concerns for the project are the presence of fill and moderately expansive near-surface soil, liquefaction potential, and moderately compressible soil below the garage. Our conclusions regarding seismic hazards, expansive soil, foundations, settlement, and other geotechnical issues are presented in this section.

7.1 Seismic Hazards

During a major earthquake on one of the nearby faults, strong to very strong shaking is expected to occur at the site. Strong shaking during an earthquake can result in ground failure such as that associated with soil liquefaction,[5] lateral spreading,[6] and cyclic densification.[7] We used the results of the borings and CPTs to evaluate the potential for these phenomena to occur at the site. The results of our evaluation are presented below.

7.1.1 Fault Rupture

Historically, ground surface displacements closely follow the trace of geologically young faults. The site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act, and no active or potentially active faults exist on the site. We therefore conclude the risk of fault offset rupture at the site from a known fault is low. In a seismically active area, the remote possibility exists for future faulting in areas where no active faults previously existed; however, based on the available fault studies, we conclude the risk of surface faulting and consequent secondary ground failure from previously unknown faults is low.

7.1.2 Soil Liquefaction and Associated Hazards

Liquefaction is a phenomenon in which saturated soil temporarily loses strength from the build-up of excess pore water pressure, especially during earthquake-induced cyclic loading. Flow failure, lateral spreading, differential settlement, loss of bearing strength, ground fissures, and sand boils are evidence of excess pore pressure generation and liquefaction. We evaluated the potential for liquefaction to occur at the site in accordance with Special Publication 117A, *Guidelines for Evaluating and Mitigating Seismic Hazards Zones in California*, dated 11 September 2008, as described below.

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[5] Liquefaction is a transformation of soil from a solid to a liquefied state during which saturated soil temporarily loses strength resulting from the buildup of excess pore water pressure, especially during earthquake-induced cyclic loading. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits.

[6] Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported downslope or in the direction of a free face by earthquake and gravitational forces.

[7] Cyclic densification is a phenomenon in which non-saturated, cohesionless soil is compacted by earthquake vibrations, causing differential settlement.

The level of ground shaking that may occur at the site during future earthquakes is uncertain because the location, recurrence interval, and magnitude of future earthquakes are not known. A peak ground acceleration (PGA) of 0.903 times gravity was used in our liquefaction analysis. This PGA was calculated using the procedures specified in Section 11.8 of ASCE 7-10 for the Maximum Considered Earthquake, using site class D. We assumed an earthquake magnitude of 7.3, which is the maximum Moment magnitude for the Hayward Fault, located about 14 km from the site as shown in Table 1. Note that the Calaveras fault is significantly closer, but has a lower maximum Moment magnitude. A high groundwater level at Elevation 439 feet was used in our liquefaction analyses.

We used the results of the CPTs to evaluate liquefaction potential at the site. The liquefaction analyses using CPT data were performed in accordance with the methodology presented in the publication titled Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, prepared by the National Center for Earthquake Engineering Research (NCEER), dated 31 December 1997. The susceptibility of sand to liquefaction under seismic loading was evaluated in general accordance with the procedure presented by Seed and Idriss (1982). Our liquefaction analysis using the CPT data indicates that thin layers of loose to medium dense granular soil below the groundwater table in the CPTs are susceptible to liquefaction (FSliq<1.3) during the design-level earthquake, as defined by ASCE 7-10.

We estimated liquefaction-induced settlement using the procedure outlined in the NCEER report. The strain potential of any potentially liquefiable layers was estimated in accordance with the method developed by Tokimatsu and Seed (1984), which relates $(N_1)_{60,CS}$ values to strain potential. The CPT tip resistance $(q_{C1N})_{CS}$ was converted to an $(N_1)_{60,CS}$ value assuming the ratio $(q_{C1N})_{CS}/(N_1)_{60,CS}$ (blows/foot) is equal to five. This value is consistent with published values for clean sand. In each of the CPTs, two to five layers of potentially liquefiable soil were encountered, each less than two feet thick, with calculated total liquefaction-induced settlements of about ¼ to ½ inch.

In addition, we evaluated the potential for liquefaction using the results of our borings. In each of the borings, two to four layers of potentially liquefiable soil were encountered, each about 1 to 2½ feet thick, with calculated total liquefaction-induced settlements of about ¾ to 1¼ inches. However, liquefaction analyses using SPT data from hollow stem auger borings generally produce conservatively large settlements because the soil below the groundwater level tends to heave in borings drilled using hollow stem augers and as a result, the SPT blow counts are conservative; we judge the actual settlements may be on the order of half of that calculated using the boring results. In addition, several of the layers identified in the borings as potentially liquefiable contained substantial amounts of clay and gravel, and will likely settle less than calculated. Therefore, while some liquefiable soil may be present, we judge liquefaction-induced settlement would be less than that calculated using the boring data.

The results of our liquefaction analyses indicate there are thin layers of loose to medium dense sand with variable clay, silt, and gravel content below the groundwater table that are susceptible to liquefaction during a major earthquake on a nearby fault. Based on our liquefaction analyses using the borings and CPTs, we conclude that up to about ½ inch of liquefaction-induced total settlement may occur at the site as a result of a major earthquake. The liquefaction may occur in isolated areas and differential settlement may be abrupt; therefore, differential settlements equivalent to the total settlement of ½ inch should be anticipated over short distances.

The potential for liquefaction-induced ground rupture and sand boils to occur at the site depends on the thickness of the liquefiable soil layer relative to the thickness of the overlying non-liquefiable material. Ishihara (1985) presented an empirical relationship that provides criteria that can be used to evaluate whether liquefaction-induced surface ruptures and sand boils would be expected to occur under a given level of shaking for a liquefiable layer overlain by a non-liquefiable layer. For the design-level earthquake defined by ASCE 7-10, we conclude that the potential for surface manifestations of liquefaction to occur at the site is moderate. Where surface manifestations occur, additional settlement may occur.

7.1.3 Lateral Spreading

Lateral spreading occurs when a continuous layer of soil liquefies at depth and the soil layers above move toward an unsupported face, such as an open slope cut, or in the direction of a regional slope or gradient. The potential for lateral spreading to occur at a site is typically evaluated using an empirical relationship developed by Youd, Hansen, and Bartlett (2002). This relationship incorporates the thickness of the liquefiable layer, the fines content and mean grain-size diameter of the liquefiable soil, the relative density of the liquefiable soil, the magnitude and distance of the earthquake from the site, the slope of the ground surface, and boundary conditions (such as a free face or edge of shoreline), to estimate the horizontal ground movement. The interpreted $(N_1)_{60}$ values for the potentially-liquefiable soil layers in the borings and CPTs are generally greater than 15, with the exception of one to three thin layers (each 2.5 feet thick or less) encountered in the borings. Typically layers with interpreted $(N_1)_{60}$ values greater than 15 are not considered to have the potential for lateral spreading. The layers with $(N_1)_{60}$ values less than 15 encountered in the borings were not encountered in the CPTs. For reasons discussed in Section 7.1.2, we consider the $(N_1)_{60}$ values from the borings to be conservatively low. Therefore, we conclude the potential for lateral spreading at the site is low.

7.1.4 Cyclic Densification

Cyclic densification of non-saturated sand (sand above the groundwater table) caused by earthquake vibrations may result in settlement. Several feet of medium dense clayey sand and silty sand with gravel and loose to medium dense silty sand were encountered above the groundwater level in the borings. We compute that settlement up to about ⅓ inch may occur due to strong shaking from a large earthquake, with a possibility of abrupt differential settlements on the order of ¼ inch.

7.2 Foundation Support

The site is underlain by moderately expansive soil. Expansive soil is subject to high volume changes during fluctuations in moisture content, which can cause cracking of foundations, floor slabs, and flatwork. The detrimental effects of near-surface expansive soil can be mitigated by moisture conditioning the expansive soil below slabs and flatwork, placing non-expansive fill below slabs and flatwork, supporting foundations below the zone of severe moisture change, and/or designing foundations to resist the movements associated with the volume changes.

Based on our field investigation, we anticipate the soil exposed at the foundation level of the parking garage will be the native stiff to very stiff clay, existing undocumented fill or engineered fill placed to raise the building pad. To provide uniform support, the existing fill will need to be overexcavated and recompacted prior to placement of the new fill. The foundations can bear on native stiff to very stiff clay or engineered fill; therefore, we conclude a shallow foundation may be used to support the parking garage. Because of the presence of moderately expansive clay, the footings should be deepened, and a deepened continuous footing or grade beam should be used around the perimeter.

There are moderately compressible soils below the garage footprint; in addition, as presented in Section 7.1, we anticipate cyclic densification and liquefaction-induced settlement will occur. Therefore, we recommend the foundation consist of strip footings where feasible, and isolated spread footings where the column spacing is too great to spread the loads between columns. In addition, there are moderately and lightly loaded columns adjacent to heavily loaded columns, and our analysis indicates significant differential settlement will occur. In order to spread the loads to reduce the amount of settlement of the heavily loaded columns, the footings should be underlain by compacted aggregate base that extends beyond the limits of the footings. Our settlement analyses indicate total static settlement under the anticipated foundation pressures for the moderately loaded foundations will be about ⅓ to ⅔ inch, while the more heavily loaded footings will settle about ¾ to 1 inch. In general, differential settlement will be less than ½ inch between columns, except where a heavily loaded column is near a moderately or lightly loaded column. In this case, we expect differential settlement up to about ¾ inch may occur. Some footings will bottom at Elevation 445 feet or lower; these footing will be closer to the compressible layer and will settle about 1½ inches, with about 1 inch of differential settlement. To reduce total and differential settlement, footings bottomed at Elevation 445 feet and below should have a reduced bearing pressure. In addition to the foregoing static settlements, seismically-induced settlement may occur during a major earthquake, as discussed in Section 7.1.

7.3 Garage Ground Floor

The ground level floor may be designed as a floor slab or pavement. If designed as a floor slab, the subgrade should be prepared as recommended in Section 8.1 to mitigate the effects of expansive soil. The floor slab will bear on engineered fill and can be supported on grade. If designed as a pavement, it may be flexible (asphalt concrete) or rigid (Portland cement concrete). Recommendations for both pavement types are presented in Section 8.6. If the surface is designed as pavement, it should be separate from the structure, as the performance of pavement differs from a floor slab. Pavements experience some movement under vehicle loads and with changes in moisture of the expansive soil.

7.4 Corrosion Potential

We performed a corrosivity test on a soil sample collected from boring B-1 at 3½ feet bgs. The soil sample was tested in accordance with Caltrans and ASTM protocols by Environmental Technical Services (ETS) of Petaluma, California. The corrosivity test results are presented in Appendix C on Figure C-6.

7.5 Construction Considerations

As discussed previously, the site is underlain by moderately expansive near-surface soil. If the clayey soil subgrade is exposed and allowed to dry during excavation for the foundation and is not properly moisture-conditioned prior to placement of concrete, significant heave may occur as soil moisture levels increase after construction. Therefore, it is essential to maintain moisture during construction. Typically, it is necessary to spray the exposed bottom and sides of excavations on a daily basis to prevent drying.

If construction activities are performed during the winter/rainy season, the near-surface soils will be saturated, soft, and easily remolded. Methods of stabilizing saturated subgrade are discussed in Section 8.1. Wet soil will require significant drying before it can be used as fill or backfill.

8.0 RECOMMENDATIONS

Our recommendations regarding design of foundations, below-grade walls, pavement, and other geotechnical aspects of this project are presented in this section.

8.1 Earthwork

8.1.1 Site Preparation

Site preparation should include removal of all existing structures, foundations, slabs, pavements, and underground utilities within the footprint of the planned development. Any subsurface structures and debris should be removed. The existing undocumented fill should be overexcavated, moisture conditioned, and recompacted where below footings and at least five feet beyond the footings. All areas to receive improvements should be stripped of vegetation and organic topsoil. Stripped materials should be removed from the site or stockpiled for later use in landscaped areas, if approved by the landscape architect. Underground utilities should be removed to the service connections and properly capped or plugged with concrete. Where existing utility lines will not interfere with the planned construction, they may be abandoned in-place, provided the lines are filled with lean concrete or cement grout to the limits of the project. Voids resulting from demolition activities should be properly backfilled with engineered fill as described later in this section.

From a geotechnical standpoint, concrete and asphalt generated by demolition may be crushed and reused as fill providing it is free of organic material and rocks or lumps greater than three inches in greatest dimension. The acceptability of using crushed asphalt at the site should be verified by the architect. Where crushed asphalt pavement materials are used as fill, particles between 1½ and 3 inches in greatest dimension should comprise no more than 30 percent of the fill by weight.

In areas where wet and/or weak subgrade soils are encountered, an alternative to mitigate this problem is scarifying and aerating the upper 12 inches of soil to reduce its moisture content so that it can be compacted to the required compaction. For this alternative, several weeks of dry, warm weather may be required. Other alternatives to mitigate weak subgrade areas are: 1) mixing and compacting the upper 12 to 18 inches of the weak soil with lime or kiln dust, 2) excavating the upper 12 to 18 inches of the weak soil, and backfilling with a lean concrete backfill, and 3) excavating the upper 12 to 18 inches of the weak soil, placing a geotextile (Mirafi 500X or equivalent), and placing and compacting select fill over the fabric.

8.1.2 Fill Placement

We anticipate fill placement will consist fill placement to raise site grades up to about 3-1/2 feet, utility trench backfill, placement of backfill around below-grade walls, and placement of select fill to mitigate the effects of the moderately expansive soil beneath the floor slab and exterior concrete flatwork. Wall backfill should meet the criteria for select fill.

Select fill should consist of imported or on-site soil that is free of organic matter and hazardous material, contain no rocks or lumps larger than three inches in greatest dimension, have a liquid limit less than 40 and plasticity index less than 12, and be approved by the geotechnical engineer. In addition, select fill placed outside of the garage footprint should contain at least 20 percent fines (particles passing the No. 200 sieve) to reduce the potential for surface water to infiltrate near the foundations or behind below-grade walls; select fill placed below the floor slab may consist of Class 2 Aggregate Base. Select fill should be placed in lifts not exceeding eight inches in loose thickness, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent relative compaction for fill thickness equal to or less than five feet and 95 percent compaction for fill thickness greater than five feet.

If lime treatment will be used beneath exterior concrete flatwork in lieu of select fill or Class 2 AB, the upper 12 inches of the expansive soil should be treated in place with about four to eight percent (to be determined by the contractor) dolomitic quicklime by dry weight of soil. A specialty subcontractor typically performs lime treatment, and we recommend this work be performed only by an experienced contractor. Prior to lime treatment, we recommend the area be graded to a level pad in accordance with our previous recommendations and all below-grade obstructions removed. The soil treated with lime should be mixed and compacted in one lift. The lime should be thoroughly blended with the soil and allowed to set for 24 hours prior to compaction. The lime-treated soil should be moisture-conditioned to above optimum moisture content and compacted to at least 90 percent relative compaction. Lime-treated soil should be removed from landscaping areas as it will inhibit growth of vegetation. It should be noted that disposal of lime-treated soil is typically expensive because of the high pH of the treated soil.

If native expansive clay is to be used as general site fill, it should be moisture-conditioned to at least three percent above optimum moisture content, placed in horizontal lifts not exceeding eight inches in loose thickness, and compacted to between 88 and 92 percent relative compaction.

We should approve all sources of engineered fill at least three days before use at the site. The grading subcontractor should provide analytical test results or other suitable environmental documentation indicating the imported fill is free of hazardous materials at least three days before use at the site. If this data is not available, up to two weeks should be allowed to perform analytical testing on the proposed import material.

8.1.3 Subgrade Preparation

We recommend at least 12 inches of non-expansive fill, consisting of either lime-treated soil or select fill be placed in the proposed garage area if a floor slab will be constructed; the non-expansive fill should extend at least five feet beyond garage footprint. Where lime-treatment will be performed, it will need to be removed and replaced with non-expansive soil in landscaped areas. Prior to placement of select fill in building areas, the subgrade soil exposed by stripping and grading should be scarified to a depth of at least 12 inches, moisture-conditioned to at least three percent above optimum moisture content, and compacted to between 88 and 93 percent relative compaction. The soil subgrade should be kept moist until it is covered by select fill.

In asphalt and concrete pavement areas, where non-expansive engineered fill is exposed at soil subgrade, the upper six inches should be moisture-conditioned to above optimum moisture content and compacted to at least 95 percent relative compaction[8] to provide a smooth non-yielding surface. If expansive on-site clay and/or undocumented fill is at subgrade in pavement areas, the upper 12 inches should be moisture-conditioned to at least two percent over optimum moisture content and compacted to at least 90 percent relative compaction.

As a minimum preparation for exterior concrete flatwork (including patio slabs and sidewalks), and where fill is to be placed outside the garage footprint, the native expansive soil at subgrade should be scarified to a depth of at least 12 inches, moisture-conditioned to at least three percent above optimum moisture content, and compacted to between 88 and 92 percent relative compaction. If non-expansive soil is exposed at subgrade, it should be scarified to a depth of at least six inches, moisture-conditioned to at least two percent above optimum moisture content, and compacted to at least 90 percent relative compaction.

If it is desirable to reduce the potential for differential movement and cracking where expansive soil is present, exterior concrete flatwork should be underlain by at least 12 inches of select fill, lime-treated soil, or Caltrans Class 2 aggregate base (AB), as recommended in Section 8.5. Select fill at subgrade in concrete flatwork areas should be moisture-conditioned to above optimum moisture content and compacted to at least 95 percent relative compaction.

8.1.4 Temporary Cut Slopes

We anticipate there may be minor excavations for site improvements. The soil to be excavated consists predominantly of clay and sand, which may be excavated using conventional earth-moving equipment such as loaders and backhoes. Excavations that will be deeper than five feet and will be entered by workers should be shored or sloped in accordance with the Occupational Safety and Health Administration (OSHA) standards (29 CFR Part 1926). We judge that temporary cuts in native soil which are less than 12 feet high and inclined no steeper than 1.5:1 (horizontal: vertical) will be stable provided that they are not surcharged by equipment or building material.

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[8] Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557-09 laboratory compaction procedure.

8.1.5 Utility Trenches

Excavations for utility trenches can be made with a backhoe. All trenches should conform to the current OSHA requirements for slopes, shoring, and other safety concerns.

To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of sand or fine gravel. After the pipes and conduits are tested, inspected (if required), and approved, they should be covered to a depth of six inches with sand or fine gravel, which should be mechanically tamped. If groundwater is encountered during trench excavation, the gravel used as bedding and cover should be replaced with Caltrans Class 2 permeable material below the water level, or the open-graded gravel used as bedding and cover should be wrapped in filter fabric (Mirafi 140N or equivalent) to reduce the potential for infiltration of fines.

Backfill for utility trenches and other excavations is also considered fill and should be placed and compacted according to the recommendations previously presented. If imported clean sand or gravel (defined as soil with less than 10 percent fines) is used as backfill, it should be compacted to at least 95 percent relative compaction. Jetting of trench backfill should not be permitted. Special care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to the pavement section.

Where utility trenches backfilled with sand or gravel enter the building pad, an impermeable plug consisting of native clay or lean concrete, at least five feet in length, should be installed at the building line. Further, where sand- or gravel-backfilled trenches cross planter areas and pass below asphalt or concrete pavements, a similar plug should be placed at the edge of the pavement. The plug should extend from the bottom of the trench to the subgrade elevation. The purpose of these recommendations is to reduce the potential for water to become trapped in trenches beneath the parking garage or pavements. This trapped water can cause heaving of soils beneath slabs and softening of subgrade soil beneath pavements.

8.2 Foundation Support

The parking garage should be supported on continuous footings bearing on native stiff to very stiff clay or engineered fill founded at least 24 inches below lowest adjacent soil subgrade. Where columns spacing is too great to sufficiently spread the load, isolated footings may be used, provided the differential settlement is acceptable. To reduce the potential for movement of the footings due to shrink and swell of the expansive clay, we recommend that a continuous footing or grade beam be placed around the perimeter and bottomed at least 24 inches below the lowest adjacent soil subgrade. Continuous footing should be at least 18 inches wide and isolated footings should be at least 24 inches wide.

Footings bottomed above Elevation 445 feet should be designed for an allowable bearing pressure of 4,500 pounds per square foot (psf) for dead plus live load conditions, with a one-third increase for total design loads, including wind or seismic loads. A modulus of subgrade reaction of 80 kips per cubic foot (kcf) may be used for footings where column loads are less than 800 kips and 54 kcf for footings where column loads are greater than 800 kips. Footings bottomed at or below Elevation 445 feet should be designed for an allowable bearing pressure of 3,200 psf for dead plus live load conditions, with a one-third increase for total design loads. A modulus of subgrade reaction of 38 kcf should be used for these deeper footings.

To limit the differential settlement between moderately or lightly loaded columns and heavily loaded columns, we recommended the heavily loaded footings (800 kips and higher for dead plus live loads) be underlain by at least 36 inches of Class 2 aggregate base extending at least four feet beyond the limits of the footings and compacted to at least 95 percent relative compaction. At the time, we understand no heavily loaded footings will bear at Elevation 445 feet or below. We should be informed if this condition occurs as additional recommendations will be needed.

Resistance to lateral loads can be mobilized by a combination of passive pressure acting against the vertical faces of the footings and friction along the base of the footings. Passive resistance may be calculated using an equivalent fluid pressure of 270 pounds per cubic foot (pcf). The upper foot of soil should be ignored unless confined by a concrete slab or pavement. Friction along the bottom of the foundation may be calculated using an allowable friction coefficient of 0.25. These values include a factor of safety of about 1.5.

The bottom and sides of the footing excavations should be wetted following excavation and maintained in a moist condition until concrete is placed. If the foundation soil dries during construction, the foundation will heave when exposed to moisture, which may result in cracking and distress. We should observe footing excavations prior to placement of reinforcing steel, and we should recheck the excavation just prior to concrete placement to confirm the excavation is sufficiently moist. The excavation for the footings should be free of standing water, debris, and disturbed materials prior to placing concrete.

8.3 Floor Slabs

The floor slabs will bear on at least 12 inches of lime-treated soil or select fill over native stiff to very stiff clay or engineered fill. The slab may be designed to bear on grade. Where moisture is not a concern, the floor should be underlain by at least 6 inches of Class 2 aggregate base compacted to at least 95 percent relative compaction.

Where moisture on the slab would be detrimental, a capillary moisture break consists of at least four inches of clean, free-draining gravel or crushed rock. The vapor retarder should meet the requirements for Class C vapor retarders stated in ASTM E1745-97. The vapor retarder should be placed in accordance with the requirements of ASTM E1643-98. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder. The vapor retarder should be covered with two inches of sand to aid in curing the concrete and to protect the vapor retarder during slab construction. The particle size of the gravel/crushed rock and sand should meet the gradation requirements presented in Table 3.

TABLE 3

Gradation Requirements for Capillary Moisture Break

Sieve Size	Percentage Passing Sieve
Gravel or Crushed Rock	
1 inch	90 – 100
3/4 inch	30 – 100
1/2 inch	5 – 25
3/8 inch	0 – 6
Sand	
No. 4	100
No. 200	0 – 5

The sand overlying the membrane should be moist at the time concrete is placed; however, there should be no free water present in the sand. Excess water trapped in the sand could eventually be transmitted as vapor through the slab. If rain is forecast prior to pouring the slab, the sand should be covered with plastic sheeting to avoid wetting. If the sand becomes wet, concrete should not be placed until the sand has been dried or replaced.

Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab. Therefore, concrete for the floor slab should have a low w/c ratio – less than 0.50. If approved by the project structural engineer, the sand can be eliminated and the concrete can be placed directly over the vapor retarder, provided the w/c ratio of the concrete does not exceed 0.45 and water is not added in the field. If necessary, workability should be increased by adding plasticizers. In addition, the slab should be properly cured. Before the floor covering is placed, the contractor should check that the concrete surface and the moisture emission levels (if emission testing is required) meet the manufacturer's requirements.

8.4 Below-Grade Walls

Below-grade walls should be designed to resist both static lateral earth pressures and, where the wall is greater than six feet high, lateral pressures caused by earthquakes. Restrained walls less than six feet high should be designed for the at-rest earth pressures and, where applicable, a traffic increment, as presented below. For walls that are greater than six feet high, we recommend they be designed for the more critical of the following criteria:

- at-rest equivalent fluid weight of 65 pcf for walls that are fully backdrained and 95 pcf for walls without a backdrain, plus a traffic increment where the wall is adjacent to streets. The traffic increment consists of a uniform (rectangular distribution) lateral pressure of 100 psf, applied over the top 10 feet of the wall.

- active equivalent fluid weight of 45 pcf plus a seismic increment of 35 pcf for walls that are fully backdrained, and 85 pcf plus a seismic increment of 20 pcf for walls without a backdrain.

If surcharge loads are present above an imaginary 30-degree line (from the horizontal) projected up from the bottom of a below-grade wall, a surcharge pressure should be included in the wall design.

A backdrain can be provided behind below-grade walls to prevent the buildup of hydrostatic pressure. One acceptable method for backdraining below-grade walls is to place a prefabricated drainage panel against the backside of the newly cast wall and collect water in a drain at the base of the wall. The drain may consist of a flat drain or a 4-inch-diamter perforated pipe surrounded by at least 4 inches of uniformly graded crushed rock wrapped in filter fabric.

To protect against moisture migration, below-grade walls should be waterproofed and water stops placed at all construction joints. The waterproofing should be placed directly against the backside of the walls. During placement of backfill behind below-grade walls, the walls should be braced or hand compaction equipment should be used to prevent unwanted surcharges on walls or foundations (as determined by the structural engineer). As recommended in Section 8.1.3, retaining wall backfill should meet the criteria for select fill.

8.5 Concrete Flatwork

If it is desirable to reduce the potential for differential movement and cracking, exterior concrete flatwork should be underlain by at least 12 inches of select fill, lime-treated soil, or Caltrans Class 2 AB, which should extend at least two feet beyond the slab edges. Even with 12 inches of select fill, lime-treated soil, or AB, exterior slabs may experience some cracking due to shrinking and swelling of the underlying expansive soil. Thickening the slabs and adding reinforcement will control this cracking to some degree, if desired. In addition, where slabs provide access to the structure, it would be prudent to dowel the slab to the foundation at the entrance to permit rotation of the slab as the exterior ground shrinks and swells and to prevent a vertical offset at the entries. Recommendations for subgrade preparation beneath concrete flatwork are provided in Section 8.1.

8.6 Pavement Design

8.6.1 Asphalt Concrete Pavement

The State of California resistance value (R-value) method for flexible pavement design was used to develop recommendations for asphalt concrete pavement sections. We anticipate the final soil subgrade in areas to receive asphalt concrete pavement will generally consist of clay. The R-value test performed on a sample of near-surface clay collected during our field investigation indicates the material has an R-value of 8. We used an R-value of 8 in our calculations.

For our calculations, we used traffic indices (TIs) of 4.5, 5.5, and 6.5. We can provide recommended pavement sections for other TIs upon request. Our pavement section recommendations are presented in Table 4.

TABLE 4
Asphaltic Concrete Pavement Section Design (R-Value of 8)

TI	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
4.5	2.5	9.0
5.5	3.0	11.5
6.5	4.0	13.0

Recommendations for subgrade preparation beneath pavement sections are provided in Section 8.1.2. AB should be compacted to at least 95 percent relative compaction.

8.6.2 Portland Cement Concrete Pavement

Concrete pavement design is based on a maximum single-axle load of 18,000 pounds and a maximum tandem axle of 32,000 pounds (corresponds to a garbage truck). The recommended rigid pavement section for these axle loads is seven inches of Portland cement concrete over six inches of Caltrans Class 2 AB. If only passenger cars or light trucks will use the pavement, the recommended minimum pavement section is five inches of Portland cement concrete over six inches of Class 2 AB. AB should conform to the current State of California Department of Transportation (Caltrans) Standard Specifications. Recommendations for subgrade preparation and AB compaction for Portland cement concrete pavement are the same as those for asphalt concrete pavement.

The modulus of rupture of the concrete should be at least 500 psi at 28 days. Contraction joints should be constructed at 15-foot spacing. Where the outer edge of a concrete pavement meets asphalt pavement, the concrete slab should be thickened by 50 percent at a taper not to exceed a slope of 1 in 10.

8.7 Drainage

Positive surface drainage should be provided around the structure to direct surface water away from the foundation. To reduce the potential for water ponding adjacent to the structure, we recommend the ground surface within a horizontal distance of ten feet from the building slope down away from the building with a surface gradient of at least five percent in unpaved areas and two percent in paved areas. In addition, roof downspouts should be discharged into controlled drainage facilities to keep the water away from the foundation. Because the subgrade soil consists predominantly of clay, it will have a relatively low permeability. If infiltration basins, bioswales, or permeable pavement are planned, they should be lined with an impermeable membrane and drains should be provided that direct the water to an appropriate outlet; additional recommendations will be needed if any of these types of improvements are planned. Unlined infiltration basins or bioswales should not be placed within five feet of the foundations.

8.8 Irrigation and Landscaping Limitations

The use of water-intensive landscaping around the perimeter of the structure should be avoided to reduce the amount of water introduced to the expansive clay subgrade. In addition, irrigation of landscaping around the structure should be limited to drip or bubbler-type systems. The purpose of these recommendations is to avoid large differential moisture changes adjacent to the foundations, which has been known to cause large differential settlement over short horizontal distances in expansive soil, resulting in cracking of slabs and architectural damage.

Moderately expansive native clay is expected to be present at or near the subgrade level. For this condition, prior experience and industry literature indicate some species of high water-demand[9] trees can induce ground surface settlement by drawing water from the expansive soil and causing it to shrink. Where these types of trees are planted adjacent to structures, the ground-surface settlement may result in damage to the structures. This problem usually occurs ten or more years after project completion as the trees reach mature height. To reduce the risk of tree-induced, ground-surface settlement, we recommend trees of the following genera not be planted within a horizontal distance from the structure

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[9] "Water-demand" refers to the ability of the tree to withdraw large amounts of water from the soil subgrade, rather than soil suction exerted by the root system.

equal to the mature height of the tree: *Eucalyptus, Populus, Quercus, Crataegus, Salix, Sorbus (simple-leafed), Ulmus, Cupressus, Chamaecyparis, and Cupressocyparis*. Because this is a limited list and does not include all genera than may induce ground-surface settlement, the project landscape architect should use judgment in limiting other types or trees with similar properties in the vicinity of the structure.

8.9 Seismic Design

Although potentially-liquefiable soil was encountered in the borings and CPTs at the site, we conclude the potentially liquefiable soil occurs in relatively thin layers. Therefore, on the basis of our evaluation of the average shear wave velocity in the upper 100 feet of the site and accounting for the softening of the potentially liquefiable material, we conclude that site class D may be used for design. For seismic design in accordance with the provisions of 2013 California Building Code (CBC) we recommend the following:

- risk targeted Maximum Considered Earthquake (MCE_R) S_S and S_1 of 2.335g and 0.888g, respectively
- site class D
- mapped MCE_R spectral response acceleration parameters, F_a and F_v of 1.0 and 1.5, respectively
- Risk Targeted MCE_R spectral acceleration parameters at short period, S_{MS}, and at one-second period, S_{M1}, of 2.335g and 1.332g, respectively
- Design Earthquake (DE) spectral acceleration parameters at short period, S_{DS}, and at one-second period, S_{D1}, of 1.556g and 0.888g, respectively.

9.0 ADDITIONAL RECOMMENDATIONS – SERVICES DURING DESIGN AND CONSTRUCTION

During final design we should be retained to consult with the design team as geotechnical questions arise. Prior to construction, we should review the geotechnical aspects of the project plans and specifications to check their conformance with the intent of our recommendations. During construction, it is imperative that we observe footings excavations, footing subgrade preparation, slab subgrade preparation, compaction of backfill, as the geotechnical engineer of record. These observations will allow us to compare the actual with the anticipated soil conditions and to check that the contractors' work conforms with the geotechnical aspects of the plans and specifications. The recommendations contained in this report assume that we will be on-site during construction to make modification to them as needed.

10.0 LIMITATIONS

The conclusions and recommendations presented in this report result from limited engineering studies based on our interpretation of the geotechnical conditions existing at the site at the time of this investigation. Actual subsurface conditions may vary. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that described in this report, Langan Treadwell Rollo should be notified to make supplemental recommendations, as necessary.

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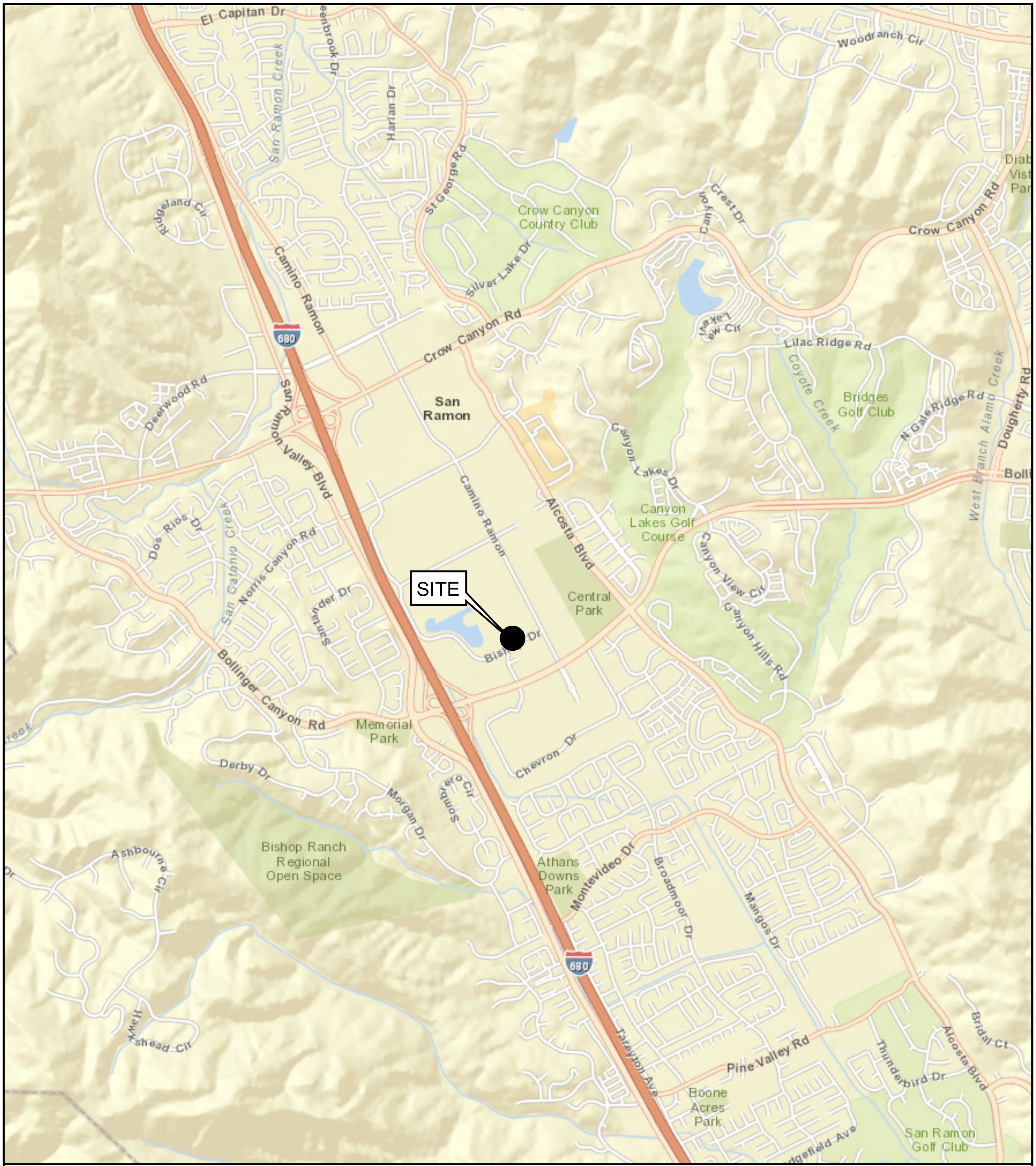
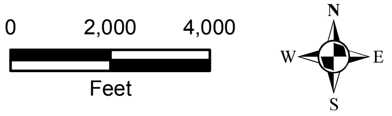
FIGURES



NOTES:

World street basemap is provided through Langan's Esri ArcGIS software licensing and ArcGIS online.
Credits: Sources: Esri, DeLorme, NAVTEQ, USGS, Intermap, iPC, NRCAN.



2600 CAMINO RAMON
San Ramon, California

LANGAN TREADWELL ROLLO

SITE LOCATION MAP

Date 1/16/2014	Project 731628101	Figure 1

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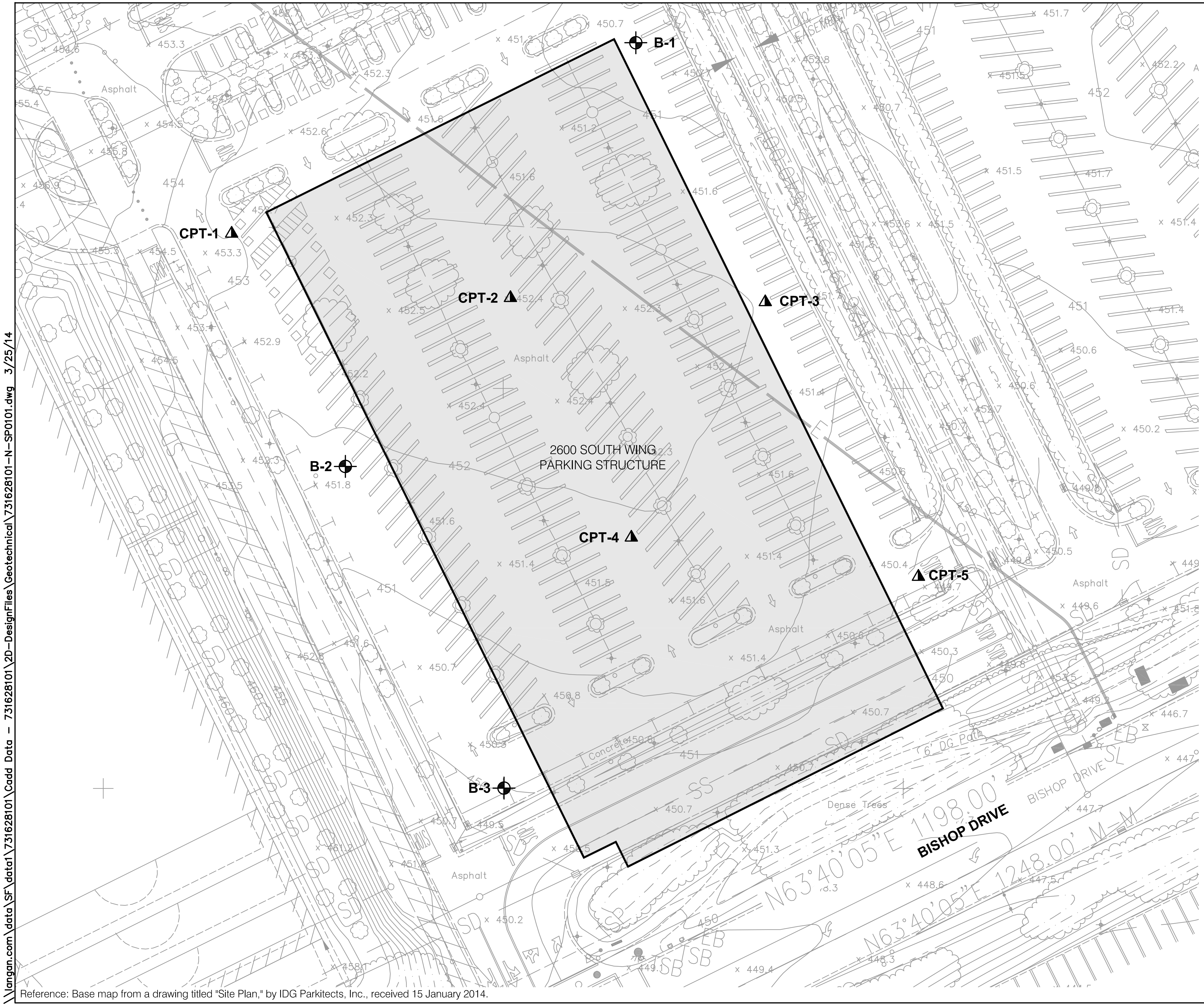

Reference: Base map from a drawing titled "Site Plan," by IDG Parkitects, Inc., received 15 January 2014.

EXPLANATION

B-1 ⊕	Approximate location of boring by Treadwell & Rollo, December 2013
CPT-1 ▲	Approximate location of cone penetration test by Treadwell & Rollo, December 2013

0 50 Feet

Approximate scale

2600 CAMINO RAMON
San Ramon, California

SITE PLAN

Date 01/07/14	Project No. 731628101	Figure 2

LANGAN TREADWELL ROLLO

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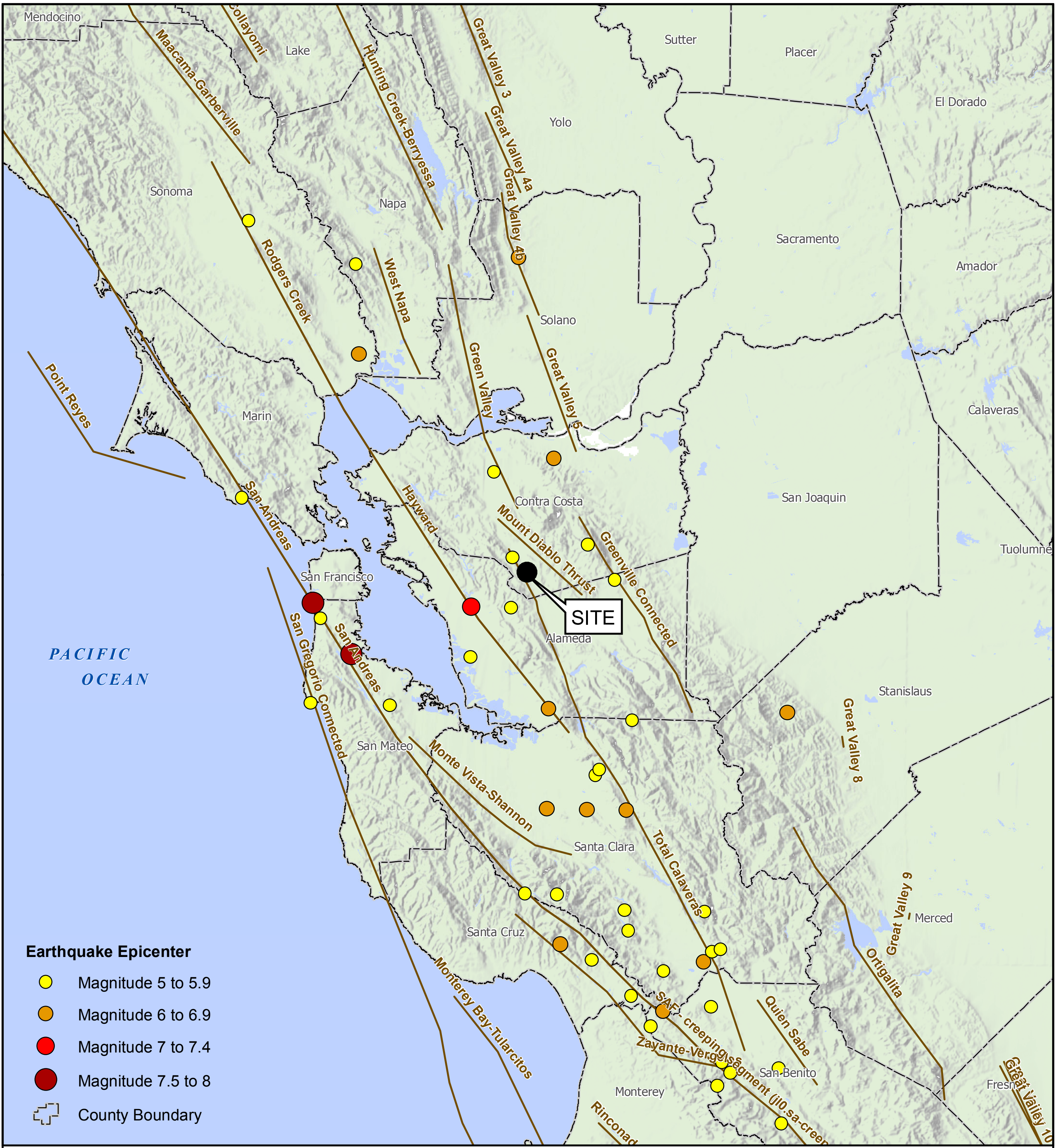

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Notes:

1. The earthquake source data is provided by the California Department of Conservation's (DOC) California Geological Survey (CGS).
2. Basemap hillshade and County boundaries provided by USGS and California Department of Transportation.
3. Map displayed in California State Coordinate System, California (Teale) Albers, North American Datum of 1983 (NAD83), Meters.

0 5 10 20
Miles

N

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LANGAN TREADWELL ROLLO

MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS IN THE SAN FRANCISCO BAY AREA

Date 1/7/2014	Project 731628101	Figure 3

I **Not felt by people, except under especially favorable circumstances. However, dizziness or nausea may be experienced.**

Sometimes birds and animals are uneasy or disturbed. Trees, structures, liquids, bodies of water may sway gently, and doors may swing very slowly.

II **Felt indoors by a few people, especially on upper floors of multi-story buildings, and by sensitive or nervous persons.**

As in Grade I, birds and animals are disturbed, and trees, structures, liquids and bodies of water may sway. Hanging objects swing, especially if they are delicately suspended.

III **Felt indoors by several people, usually as a rapid vibration that may not be recognized as an earthquake at first. Vibration is similar to that of a light, or lightly loaded trucks, or heavy trucks some distance away. Duration may be estimated in some cases.**

Movements may be appreciable on upper levels of tall structures. Standing motor cars may rock slightly.

IV **Felt indoors by many, outdoors by a few. Awakens a few individuals, particularly light sleepers, but frightens no one except those apprehensive from previous experience. Vibration like that due to passing of heavy, or heavily loaded trucks. Sensation like a heavy body striking building, or the falling of heavy objects inside.**

Dishes, windows and doors rattle; glassware and crockery clink and clash. Walls and house frames creak, especially if intensity is in the upper range of this grade. Hanging objects often swing. Liquids in open vessels are disturbed slightly. Stationary automobiles rock noticeably.

V **Felt indoors by practically everyone, outdoors by most people. Direction can often be estimated by those outdoors. Awakens many, or most sleepers. Frightens a few people, with slight excitement; some persons run outdoors.**

Buildings tremble throughout. Dishes and glassware break to some extent. Windows crack in some cases, but not generally. Vases and small or unstable objects overturn in many instances, and a few fall. Hanging objects and doors swing generally or considerably. Pictures knock against walls, or swing out of place. Doors and shutters open or close abruptly. Pendulum clocks stop, or run fast or slow. Small objects move, and furnishings may shift to a slight extent. Small amounts of liquids spill from well-filled open containers. Trees and bushes shake slightly.

VI **Felt by everyone, indoors and outdoors. Awakens all sleepers. Frightens many people; general excitement, and some persons run outdoors.**

Persons move unsteadily. Trees and bushes shake slightly to moderately. Liquids are set in strong motion. Small bells in churches and schools ring. Poorly built buildings may be damaged. Plaster falls in small amounts. Other plaster cracks somewhat. Many dishes and glasses, and a few windows break. Knickknacks, books and pictures fall. Furniture overturns in many instances. Heavy furnishings move.

VII **Frightens everyone. General alarm, and everyone runs outdoors.**

People find it difficult to stand. Persons driving cars notice shaking. Trees and bushes shake moderately to strongly. Waves form on ponds, lakes and streams. Water is muddied. Gravel or sand stream banks cave in. Large church bells ring. Suspended objects quiver. Damage is negligible in buildings of good design and construction; slight to moderate in well-built ordinary buildings; considerable in poorly built or badly designed buildings, adobe houses, old walls (especially where laid up without mortar), spires, etc. Plaster and some stucco fall. Many windows and some furniture break. Loosened brickwork and tiles shake down. Weak chimneys break at the roofline. Cornices fall from towers and high buildings. Bricks and stones are dislodged. Heavy furniture overturns. Concrete irrigation ditches are considerably damaged.

VIII **General fright, and alarm approaches panic.**

Persons driving cars are disturbed. Trees shake strongly, and branches and trunks break off (especially palm trees). Sand and mud erupts in small amounts. Flow of springs and wells is temporarily and sometimes permanently changed. Dry wells renew flow. Temperatures of spring and well waters varies. Damage slight in brick structures built especially to withstand earthquakes; considerable in ordinary substantial buildings, with some partial collapse; heavy in some wooden houses, with some tumbling down. Panel walls break away in frame structures. Decayed pilings break off. Walls fall. Solid stone walls crack and break seriously. Wet grounds and steep slopes crack to some extent. Chimneys, columns, monuments and factory stacks and towers twist and fall. Very heavy furniture moves conspicuously or overturns.

IX **Panic is general.**

Ground cracks conspicuously. Damage is considerable in masonry structures built especially to withstand earthquakes; great in other masonry buildings - some collapse in large part. Some wood frame houses built especially to withstand earthquakes are thrown out of plumb, others are shifted wholly off foundations. Reservoirs are seriously damaged and underground pipes sometimes break.

X **Panic is general.**

Ground, especially when loose and wet, cracks up to widths of several inches; fissures up to a yard in width run parallel to canal and stream banks. Landsliding is considerable from river banks and steep coasts. Sand and mud shifts horizontally on beaches and flat land. Water level changes in wells. Water is thrown on banks of canals, lakes, rivers, etc. Dams, dikes, embankments are seriously damaged. Well-built wooden structures and bridges are severely damaged, and some collapse. Dangerous cracks develop in excellent brick walls. Most masonry and frame structures, and their foundations are destroyed. Railroad rails bend slightly. Pipe lines buried in earth tear apart or are crushed endwise. Open cracks and broad wavy folds open in cement pavements and asphalt road surfaces.

XI **Panic is general.**

Disturbances in ground are many and widespread, varying with the ground material. Broad fissures, earth slumps, and land slips develop in soft, wet ground. Water charged with sand and mud is ejected in large amounts. Sea waves of significant magnitude may develop. Damage is severe to wood frame structures, especially near shock centers, great to dams, dikes and embankments, even at long distances. Few if any masonry structures remain standing. Supporting piers or pillars of large, well-built bridges are wrecked. Wooden bridges that "give" are less affected. Railroad rails bend greatly and some thrust endwise. Pipe lines buried in earth are put completely out of service.

XII **Panic is general.**

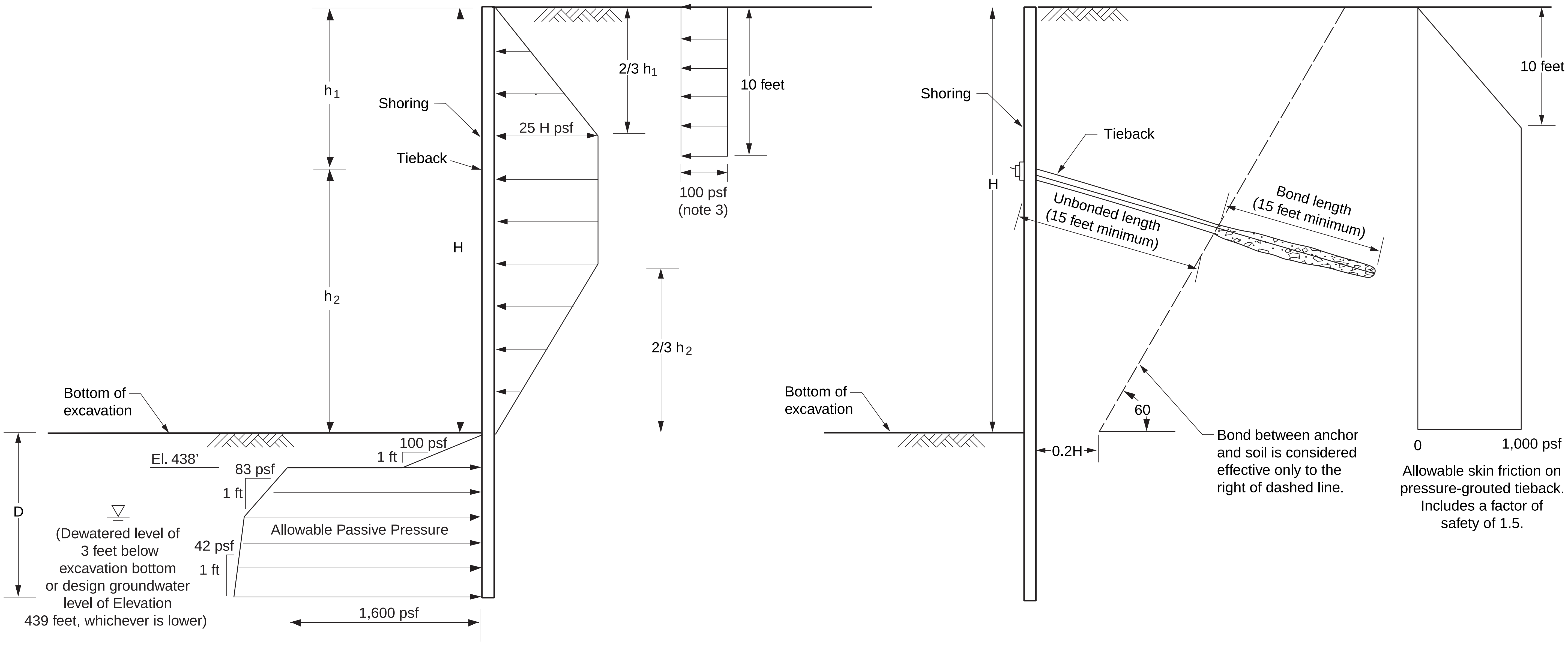
Damage is total, and practically all works of construction are damaged greatly or destroyed. Disturbances in the ground are great and varied, and numerous shearing cracks develop. Landslides, rock falls, and slumps in river banks are numerous and extensive. Large rock masses are wrenched loose and torn off. Fault slips develop in firm rock, and horizontal and vertical offset displacements are notable. Water channels, both surface and underground, are disturbed and modified greatly. Lakes are dammed, new waterfalls are produced, rivers are deflected, etc. Surface waves are seen on ground surfaces. Lines of sight and level are distorted. Objects are thrown upward into the air.

2600 CAMINO RAMON
San Ramon, California

MODIFIED MERCALLI INTENSITY SCALE

LANGAN TREADWELL ROLLO

Date 01/07/14 | Project No. 731628101 | Figure 4



Notes:
1. The above pressure diagram assumes that the shoring walls consist of pervious soldier-pile-and-lagging system.
2. Passive pressure values include a factor of safety of about 1.5 and can be applied over a width of three soldier pile diameters or pile spacing, whichever is smaller.
3. Pressure due to vehicle surcharge along streets (heavy equipment should come no closer than 5 feet to face of excavation).
4. D and H in feet.

2600 CAMINO RAMON
San Ramon, California

TYPICAL LATERAL EARTH PRESSURES AND TIEBACK CRITERIA FOR TEMPORARY SHORING SYSTEM

Date 01/04/14	Project No. 731628101	Figure 5

LANGAN TREADWELL ROLLO

APPENDIX A

LOGS OF BORINGS

PROJECT: **2600 CAMINO RAMON**
San Ramon, California

Log of Boring B-1

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2
Date started: 12/21/13
Date finished: 12/21/13
Drilling method: Hollow Stem Auger with Hydraulic Trip
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Downhole Safety
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by: E. Toth

	SAMPLES					MATERIAL DESCRIPTION	LABORATORY TEST DATA					
DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	Ground Surface Elevation: 450.5 feet[2]	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						3 inches asphalt concrete (AC)						
1						21 inches aggregate base (AB)						
2						FILL						
3					CL	CLAY (CL) dark brown, black, stiff, moist, with organics, trace sand, wood, debris						
4	S&H	■	5 9 12	13	CL	CLAY (CL) dark brown, stiff, moist, trace sand Corrosion Test, see Figure C-6						
5												
6	S&H	■	5 9 10	11		olive-brown Triaxial Test, see Figure C-2	PP TxUU	 500	3,250 3,120		 22.4	 103
7												
8						grades silty						
9	S&H	■	6 8 11	11	SM	SILTY SAND with GRAVEL (SM) brown, medium dense, moist						
10												
11	SPT	◪	5 4 3	7		loose, decreased gravel, with clay						
12												
13					CL	CLAY (CL) brown with dark brown mottling, medium stiff to stiff, moist, variable sand content						
14	S&H	■	4 6 7	8			PP		1,000			
15												
16					SC	▽ CLAYEY SAND (SC) olive-brown, loose, wet, fine-grained (12/21/13, measured groundwater elevation)						
17												
18						LL = 26, PI = 8, see Figure C-1				42.9	19.0	
19	S&H	■	7 6 7	8	CL	CLAY (CL) olive-brown, medium stiff to stiff, wet	PP		1,000			
20												
21												
22					SP-SM	SAND with SILT (SP-SM) olive-brown, loose, wet, fine- to medium-grained						
23												
24	SPT	◪	4 5 3	8	CL	CLAY (CL) dark brown, medium stiff to stiff, wet, trace sand						
25												
26												
27												
28												
29	S&H	■	8 14 16	18		olive-brown with orange and black mottling, very stiff, with carbonite nodules	PP		2,500			
30												

TEST GEOTECH LOG 731628101.GPJ TR.GDT 2/6/14

LANGAN TREADWELL ROLLO

Project No.: 731628101
Figure: A-1a

PROJECT: **2600 CAMINO RAMON**
San Ramon, California

Log of Boring B-1

PAGE 2 OF 2

DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]			Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						CLAY (CL) (continued)						
31												
32												
33												
34	S&H	■	8 9 12	13		olive-gray with orange mottling, stiff, increased sand content	PP		1,750			
35												
36												
37												
38												
39	S&H	■ □	7 12 14	16		very stiff						
40					CL							
41												
42												
43												
44	S&H	■	8 12 17	17		gray to olive-gray with orange and black mottling	PP		3,200			
45												
46												
47												
48												
49	S&H	■	8 9 12	13		olive-gray and olive with orange and gray mottling, stiff, increased sand content						
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 50 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 16.5 feet below ground surface during drilling.
PP = pocket penetrometer.
LL = liquid limit, PI = plasticity index.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively, to account for sampler type and hammer energy.

[2] Elevations based on NGVD 1929 from topographic survey provided by IDG Parkitects, Inc., on 01/15/14.

LANGAN TREADWELL ROLLO

Project No.:	Figure:
731628101	A-1b

PROJECT: **2600 CAMINO RAMON**
San Ramon, California

Log of Boring B-2

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2
Logged by: E. Toth
Date started: 12/21/13
Date finished: 12/21/13
Drilling method: Hollow Stem Auger with Hydraulic Trip
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Downhole Safety
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)

Ground Surface Elevation: 452 feet[2]

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0						3 inches asphalt concrete (AC) FILL						
1						21 inches aggregate base (AB)						
2–3	S&H		7 10 19	17	CL	CLAY (CL) dark brown to black, very stiff, moist, trace fine-grained sand and gravel	PP		2,750			
4					CL	CLAY (CL) brown, very stiff, moist, trace sand						
5–6	S&H		8 10 12	13	SC	CLAYEY SAND (SC) brown, medium dense, moist LL = 29, PI = 10, see Figure C-1						
9	S&H		8 9 12	13		grades silty						
9.5–10					SM	SILTY SAND (SM) brown, medium dense, moist, fine-grained						
10–11	SPT		3 4 5	9		loose				29.5	11.1	
14	S&H		7 9 15	14	CL	CLAY (CL) olive-brown with orange and black mottling, stiff, moist	PP		2,250			
19	S&H		8 10 11	13	SC	CLAYEY SAND (SC) brown, medium dense, moist to wet ▽ (12/21/13, measured groundwater elevation)						
20–21	SPT		5 7 7	14	CL	CLAY (CL) olive-brown with black mottling, stiff, wet, trace calcium carbonate nodules						
23–24	S&H		8 8 9	10	SC-SM	CLAYEY SILTY SAND (SC-SM) olive-brown with black mottling, loose to medium dense, wet, fine-grained, trace angular gravel LL = 23, PI = 5, see Figure C-1				26.5	16.7	
25–26	ST			500 psi	CH	CLAY (CH) olive-brown with orange and black mottling, stiff, wet, trace rootlets dark brown at 25 feet	PP		1,500			
27–28					CL	CLAY (CL) olive-brown, stiff, wet						
29–30	S&H		9 15 15	18		very stiff, trace calcium carbonate nodules, with sand						

LANGAN TREADWELL ROLLO

Project No.: 731628101
Figure: A-2a

PROJECT: **2600 CAMINO RAMON**
San Ramon, California

Log of Boring B-2

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						CLAY (CL) (continued)						
31												
32												
33												
34	S&H		10 14 16	18	CL	dark gray to olive-gray						
35												
36												
37												
38					SM	SILTY SAND (SM) olive-brown with orange mottling, medium dense, wet, fine-grained						
39	S&H		9 13 14	16		increased silt content						
40						CLAY (CL) olive-gray, very stiff, moist	PP		1,700			
41												
42												
43												
44	S&H		9 19 17	22	CL	olive-gray with orange and black mottling, calcium carbonate nodules						
45												
46												
47												
48												
49	S&H		9 13 16	17								
50					SC	CLAYEY SAND (SC) olive-brown with orange mottling, medium dense, wet						
51	SPT		7 9 10	19	CL	CLAY (CL) olive-gray with orange mottling, very stiff, wet, calcium carbonate nodules						
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 51.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 20 feet below ground surface during drilling.
PP = pocket penetrometer.
LL = liquid limit, PI = plasticity index.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively, to account for sampler type and hammer energy.
[2] Elevations based on NGVD 1929 from topographic survey provided by IDG Parkitects, Inc., on 01/15/14.

LANGAN TREADWELL ROLLO

Project No.: 731628101

Figure: A-2b

PROJECT: **2600 CAMINO RAMON**
San Ramon, California

Log of Boring B-3

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2
Date started: 12/21/13
Date finished: 12/21/13
Drilling method: Hollow Stem Auger with Hydraulic Trip
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Downhole Safety
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)
Logged by: E. Toth

	SAMPLES					MATERIAL DESCRIPTION	LABORATORY TEST DATA					
DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	Ground Surface Elevation: 450 feet[2]	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						3 inches asphalt concrete (AC)						
1						21 inches aggregate base (AB)						
3	S&H		9 18 27	27	CL	CLAY with SAND (CL) brown with gray mottling, very stiff, moist LL = 45, PI = 23, see Figure C-1 Resistance Value Test, see Figure C-5	PP		3,500			
6	S&H		5 8 10	11		brown, stiff, trace fine-grained sand	PP		3,000			
8					SM	SILTY SAND (SM) brown, loose, moist, fine-grained						
9	S&H		3 4 7	7	CL	CLAY (CL) brown, medium stiff, moist						
11	SPT		4 5 6	11	SM	SILTY SAND (SM) brown, medium dense, wet, high silt content						
12						CLAY (CL) olive-brown, stiff, moist, trace sand						
14	S&H		6 9 13	13			PP		1,500			
17					CL							
19	S&H		6 6 7	8		medium stiff to stiff, wet	PP		1,200			
21	ST			325 psi		medium stiff Triaxial Test, see Figure C-3 Consolidation Test, see Figure C-4	TxUU	3,500	650		23.9 23.1	102 100
23	SPT		6 11 14	25	SM	SILTY SAND (SM) olive-brown, medium dense, wet, fine-grained, high silt content						
24	SPT		4 3 3	6		CLAY (CH) dark gray with brown mottling, medium stiff, wet, trace sand						
26	ST			175 psi	CH							
29	S&H		7 12 17	17	CL	▽ CLAY (CL) dark brown, very stiff, wet	PP		1,600			
30												

LANGAN TREADWELL ROLLO

Project No.: 731628101
Figure: A-3a

PROJECT: **2600 CAMINO RAMON**
San Ramon, California

Log of Boring B-3

PAGE 2 OF 2

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31						CLAY (CL) (continued) trace calcium carbonate nodules (12/21/13, measured groundwater elevation)						
32												
33												
34	S&H	■ ☐	6 9 12	13		gray, stiff						
35												
36												
37												
38												
39	S&H	■ ☐	9 13 16	17		olive-gray with orange mottling, very stiff						
40					CL							
41												
42												
43												
44	S&H	■	11 13 16	17		gray and olive-gray with orange mottling, trace fine-grained sand						
45												
46												
47												
48												
49	S&H	■	6 8 11	11		olive-gray with orange and black mottling, stiff						
50	DIST	☒			SC	CLAYEY SAND (SC) olive with orange-brown and black mottling, medium dense, wet, fine-grained						
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 50.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 29 feet below ground surface during drilling.
PP = pocket penetrometer.
LL = liquid limit, PI = plasticity index.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively, to account for sampler type and hammer energy.

[2] Elevations based on NGVD 1929 from topographic survey provided by IDG Parkitects, Inc., on 01/15/14.

LANGAN TREADWELL ROLLO

Project No.: 731628101

Figure: A-3b

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions		Symbols	Typical Names
Coarse-Grained Soils (more than half of soil > no. 200 sieve size	**Gravels** (More than half of coarse fraction > no. 4 sieve size)	**GW**	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	Sands (More than half of coarse fraction < no. 4 sieve size)	**SW**	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
Fine -Grained Soils (more than half of soil < no. 200 sieve size)	**Silts and Clays** LL = < 50	**ML**	Inorganic silts and clayey silts of low plasticity, sandy silts, gravelly silts
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	Silts and Clays LL = > 50	**MH**	Inorganic silts of high plasticity
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic silts and clays of high plasticity
Highly Organic Soils		**PT**	Peat and other highly organic soils

GRAIN SIZE CHART

Classification	Range of Grain Sizes	
	U.S. Standard Sieve Size	**Grain Size in Millimeters**
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel coarse fine	3" to No. 4 3" to 3/4" 3/4" to No. 4	76.2 to 4.76 76.2 to 19.1 19.1 to 4.76
Sand coarse medium fine	No. 4 to No. 200 No. 4 to No. 10 No. 10 to No. 40 No. 40 to No. 200	4.76 to 0.075 4.76 to 2.00 2.00 to 0.420 0.420 to 0.075
Silt and Clay	Below No. 200	Below 0.075

▽ Unstabilized groundwater level

▼ Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

Sample taken with Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter. Darkened area indicates soil recovered

Classification sample taken with Standard Penetration Test sampler

Undisturbed sample taken with thin-walled tube

⊠ Disturbed sample

⊙ Sampling attempted with no recovery

Core sample

Analytical laboratory sample

Sample taken with Direct Push or Drive sampler

SAMPLER TYPE

C	Core barrel
CA	California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter
D&M	Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube
O	Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube
PT	Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube
S&H	Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter
SPT	Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.5-inch inside diameter
ST	Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

2600 CAMINO RAMON
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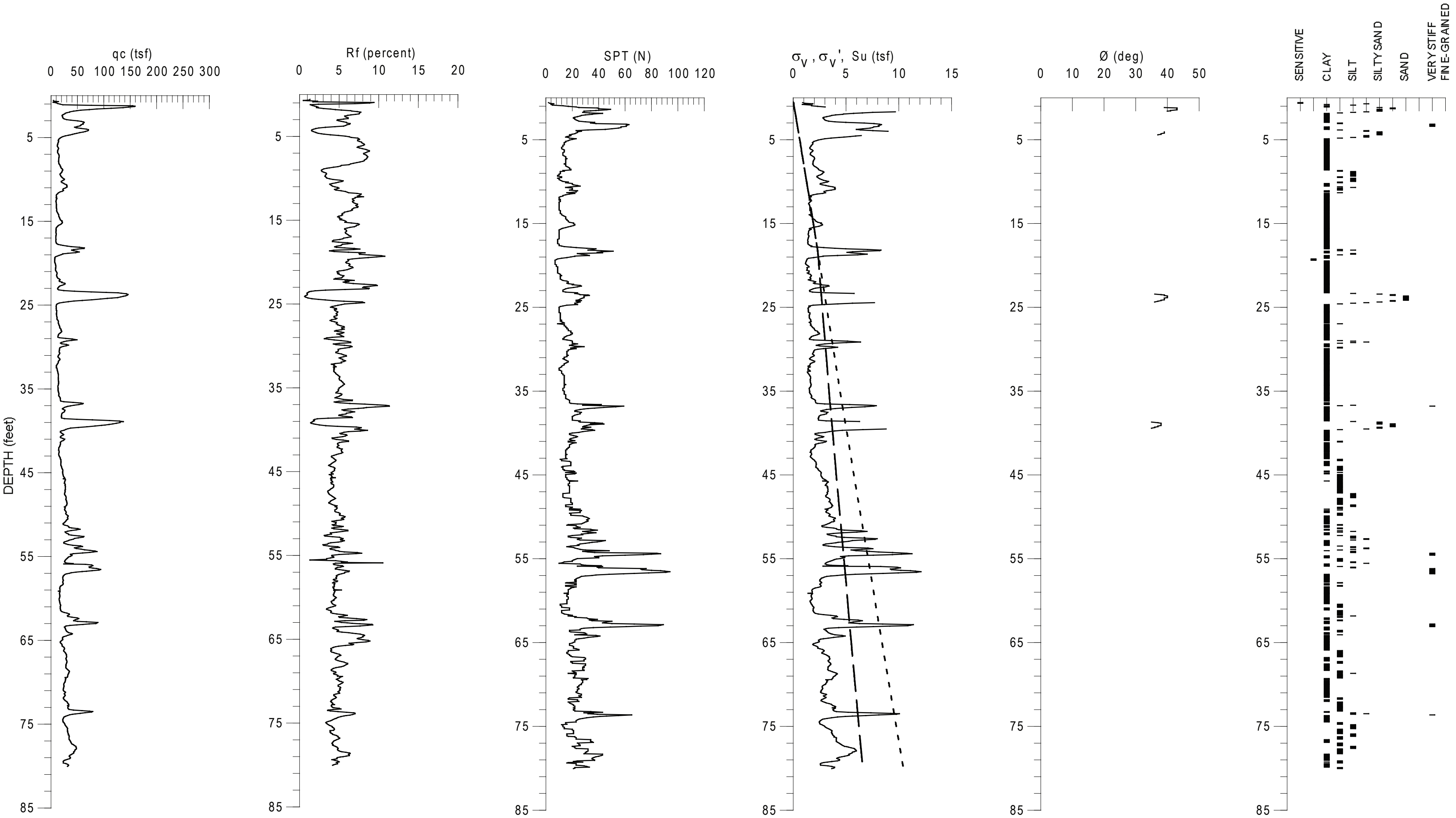
LANGAN TREADWELL ROLLO

CLASSIFICATION CHART

Date 01/07/14 | Project No. 731628101 | Figure A-4

APPENDIX B

LOGS OF CONE PENETRATION TESTS



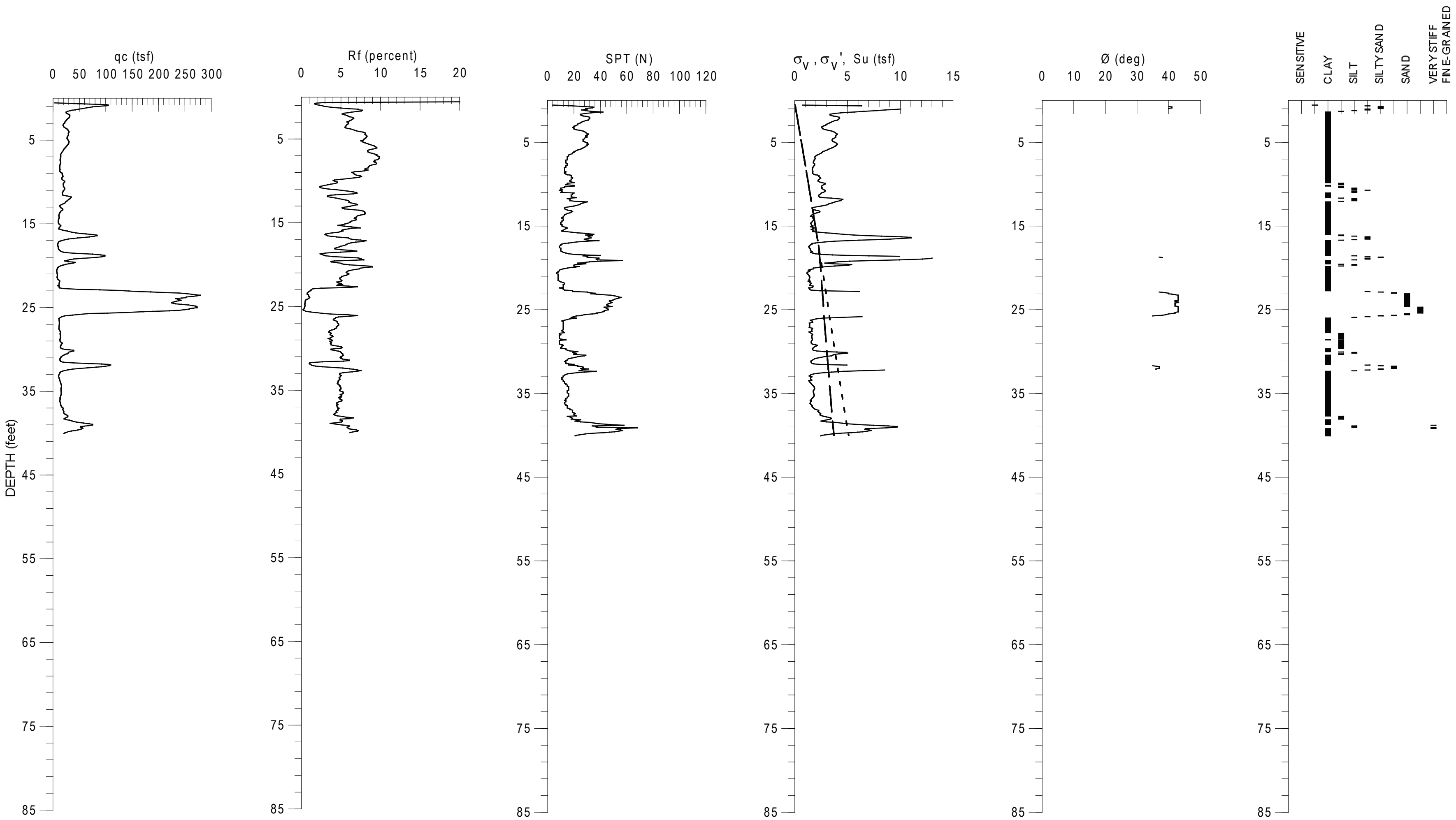
Terminated at 80 feet.
Groundwater encountered at 18.4 feet.
Date performed 12/21/13.
Ground surface elevation: 453 feet, NGVD 1929.

2600 CAMINO RAMON
San Ramon, California

CONE PENETRATION TEST RESULTS
CPT-1

Date 01/07/14	Project No. 731628101	Figure B-1

LANGAN TREADWELL ROLLO



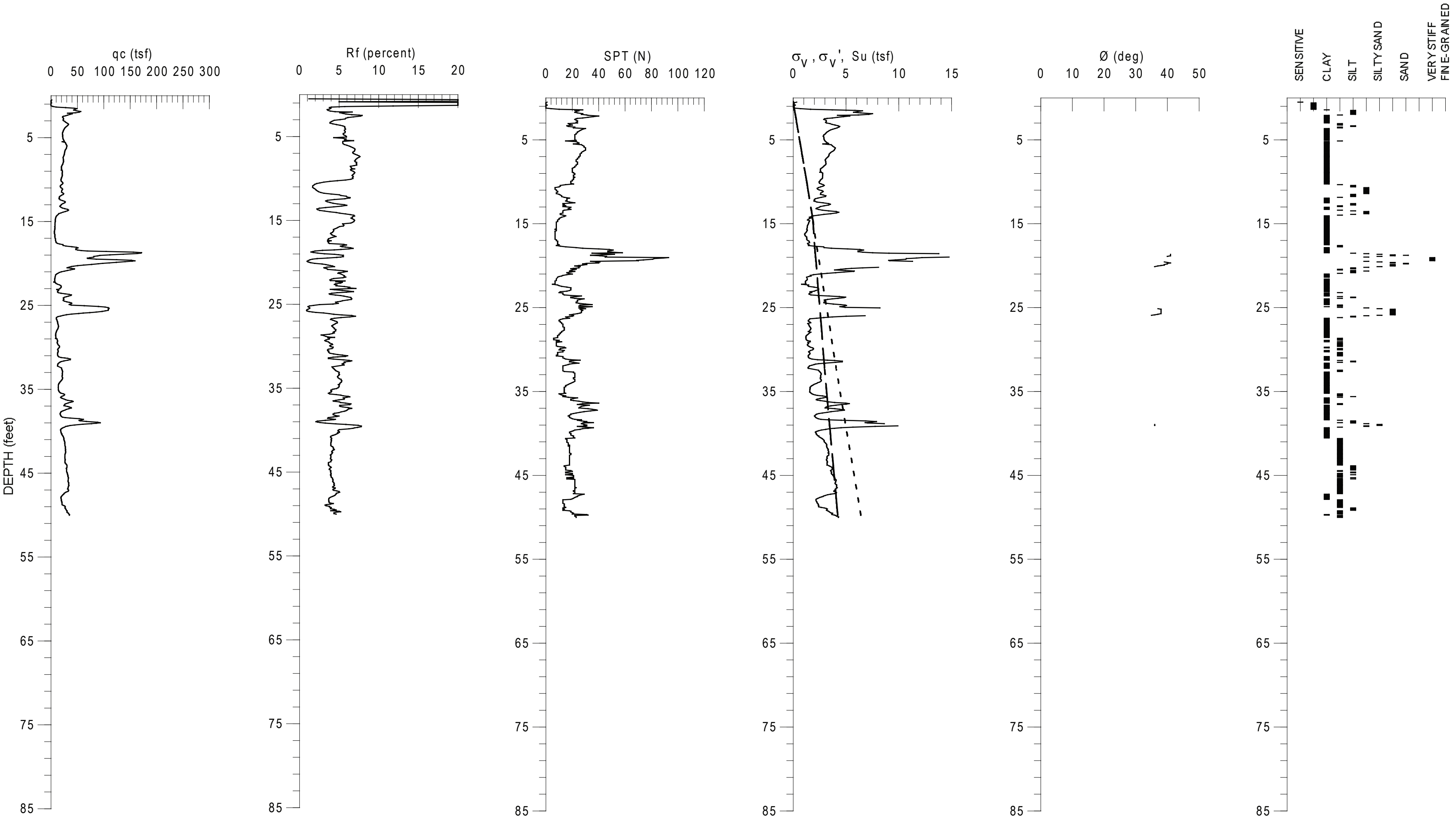
Terminated at 40 feet.
Groundwater encountered at 17.7 feet.
Date performed 12/21/13.
Ground surface elevation: 452 feet, NGVD 1929.

2600 CAMINO RAMON
San Ramon, California

CONE PENETRATION TEST RESULTS
CPT-2

Date 01/07/14	Project No. 731628101	Figure B-2

LANGAN TREADWELL ROLLO



Effective vertical stress, σ_V'

Total vertical stress, σ_V

Undrained Shear Strength, Su

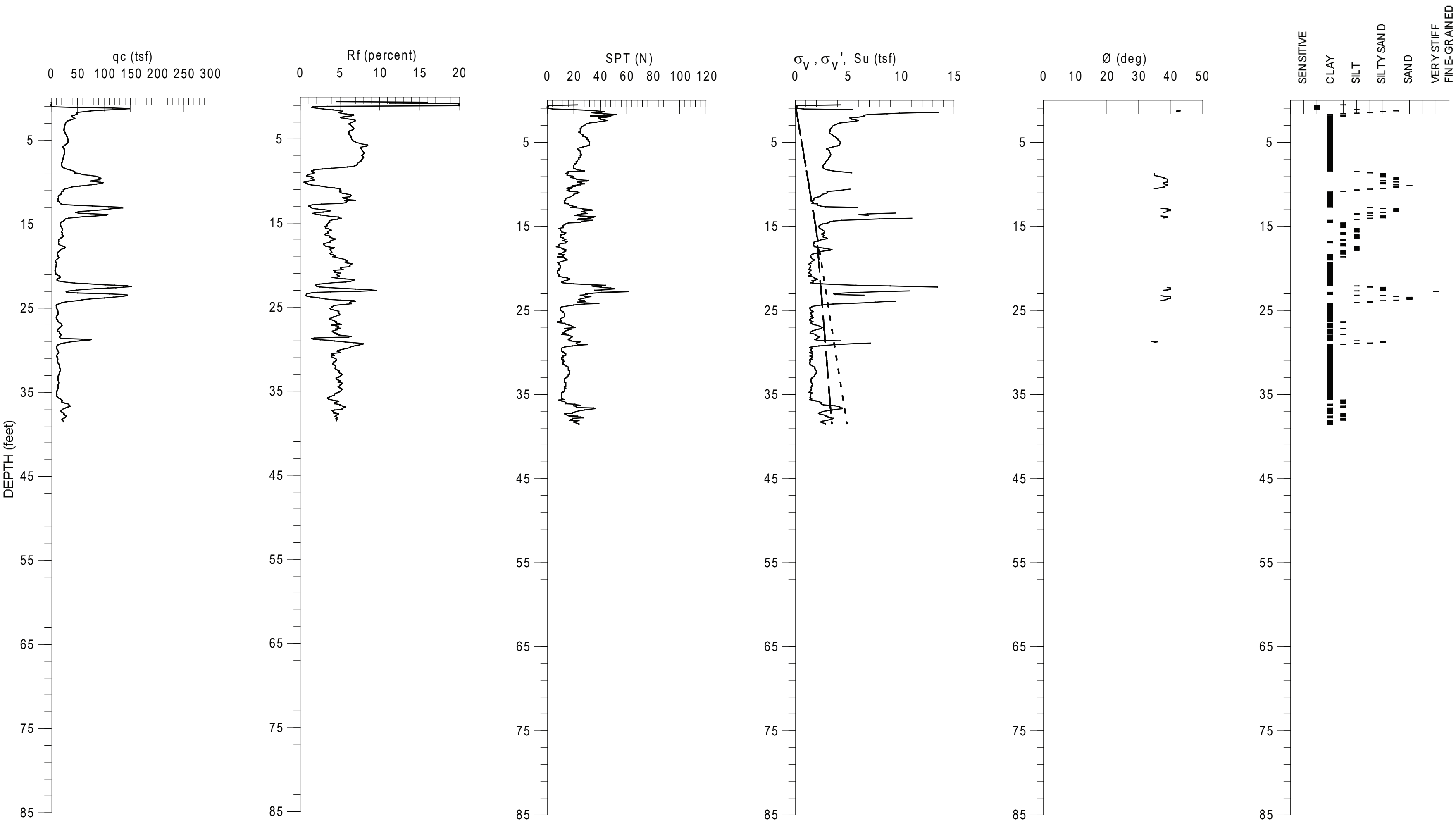
Terminated at 50 feet.
Groundwater encountered at 14.7 feet.
Date performed 12/21/13.
Ground surface elevation: 452 feet, NGVD 1929.

2600 CAMINO RAMON
San Ramon, California

CONE PENETRATION TEST RESULTS
CPT-3

Date 01/07/14	Project No. 731628101	Figure B-3

LANGAN TREADWELL ROLLO



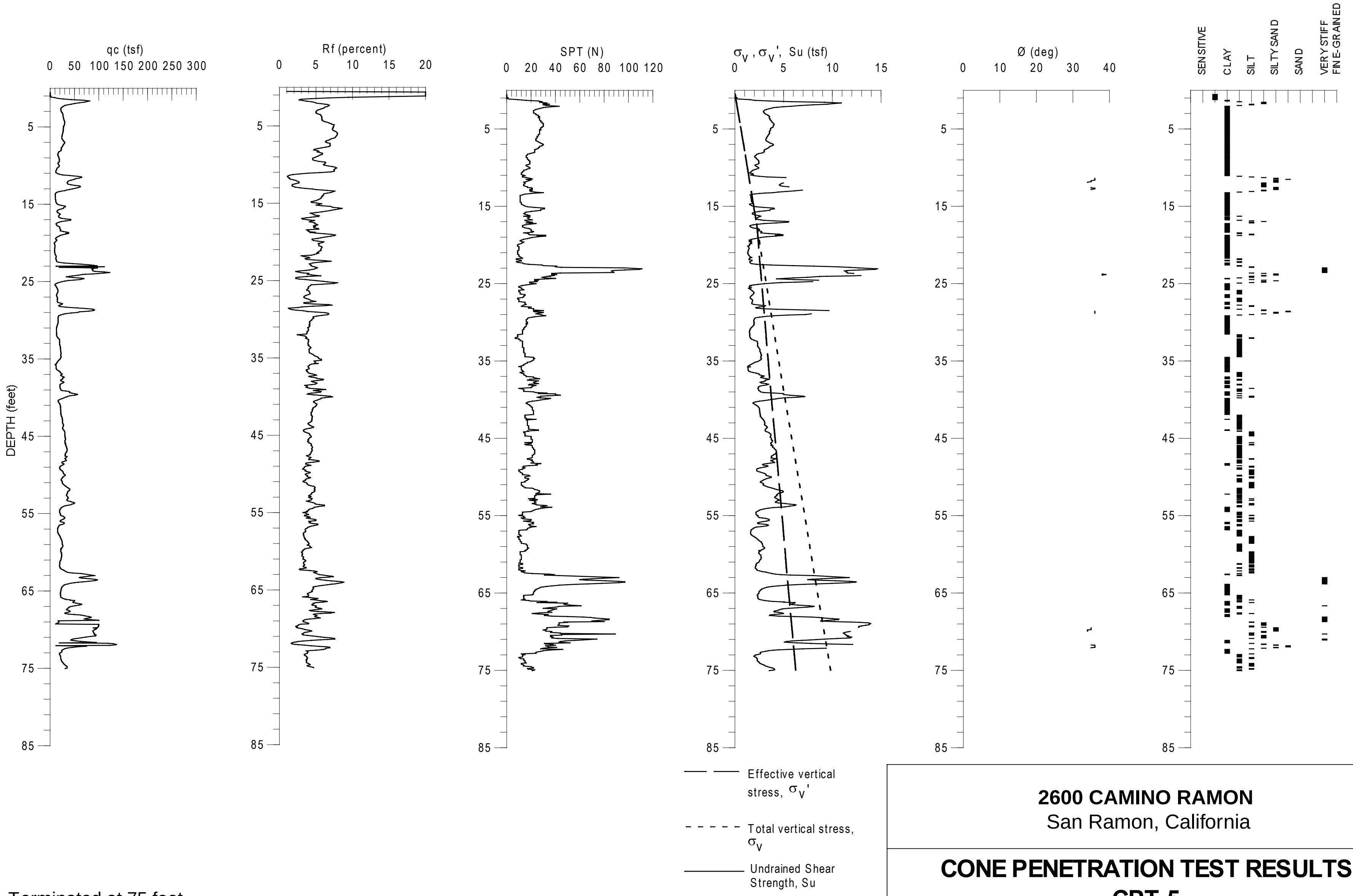
Terminated at 38.5 feet.
Groundwater encountered at 15.6 feet.
Date performed 12/21/13.
Ground surface elevation: 451.5 feet, NGVD 1929.

2600 CAMINO RAMON
San Ramon, California

CONE PENETRATION TEST RESULTS
CPT-4

Date 01/07/14	Project No. 731628101	Figure B-4

LANGAN TREADWELL ROLLO



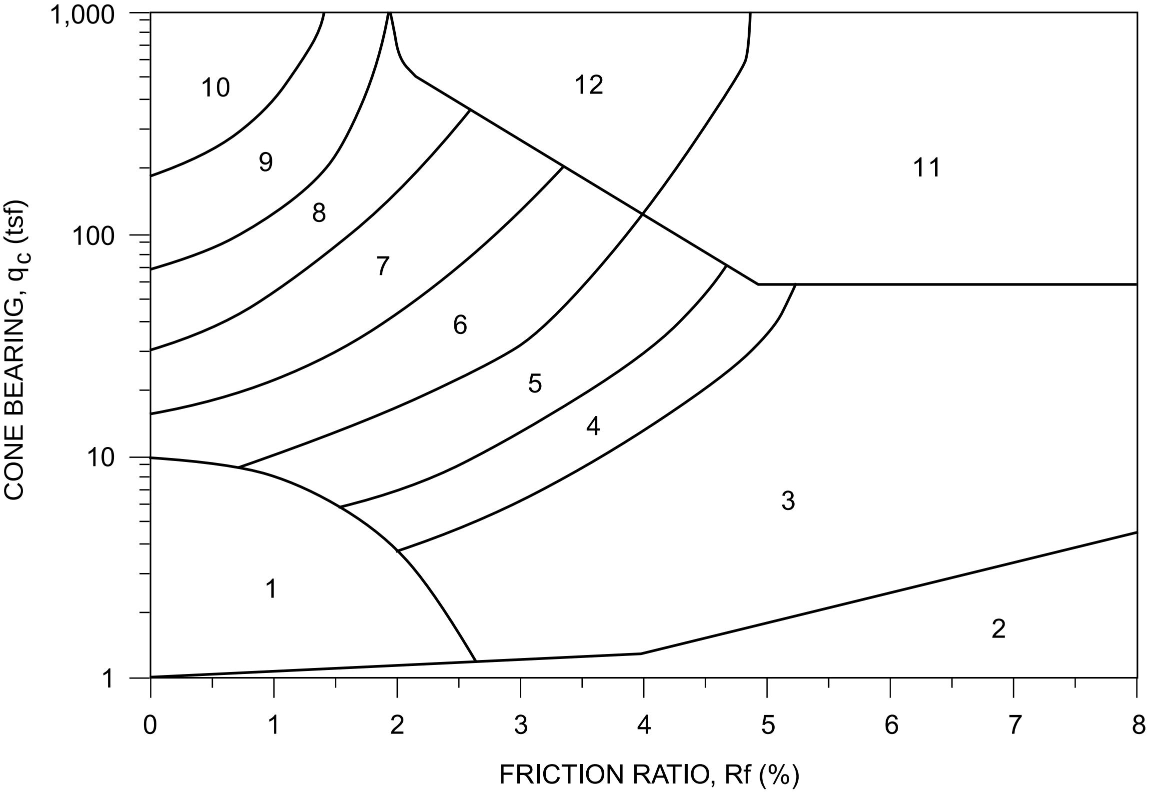
Terminated at 75 feet.
Groundwater encountered at 17.7 feet.
Date performed 12/21/13.
Ground surface elevation: 450 feet, NGVD 1929.

2600 CAMINO RAMON
San Ramon, California

CONE PENETRATION TEST RESULTS
CPT-5

Date 01/07/14	Project No. 731628101	Figure B-5

LANGAN TREADWELL ROLLO



ZONE	q_c/N^1	Su Factor $(Nk)^2$	SOIL BEHAVIOR TYPE^1
1	2	15 (10 for $q_c \leq 9$ tsf)	Sensitive Fine-Grained
2	1	15 (10 for $q_c \leq 9$ tsf)	Organic Material
3	1	15 (10 for $q_c \leq 9$ tsf)	CLAY
4	1.5	15	SILTY CLAY to CLAY
5	2	15	CLAYEY SILT to SILTY CLAY
6	2.5	15	SANDY SILT to CLAYEY SILT
7	3	---	SILTY SAND to SANDY SILT
8	4	---	SAND to SILTY SAND
9	5	---	SAND
10	6	---	GRAVELLY SAND to SAND
11	1	15	Very Stiff Fine-Grained (*)
12	2	---	SAND to CLAYEY SAND (*)

(*) Overconsolidated or Cemented
q_c = Tip Bearing
f_s = Sleeve Friction
Rf = f_s/q_c x 100 = Friction Ratio

Note: Testing performed in accordance with ASTM D3441.

References: 1. Robertson, 1986, Olsen, 1988.
2. Bonaparte & Mitchell, 1979 (young Bay Mud $q_c \leq 9$).
Estimated from local experience (fine-grained soils $q_c > 9$).

2600 CAMINO RAMON
San Ramon, California

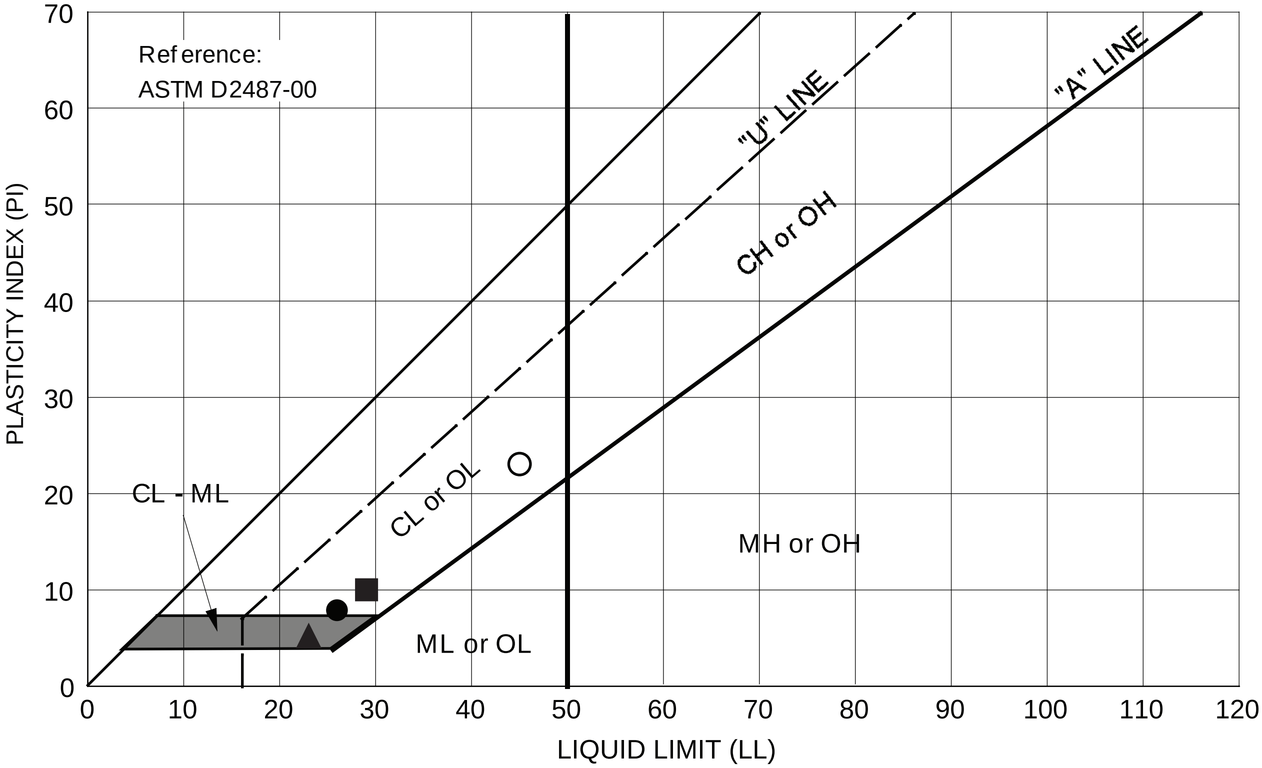
LANGAN TREADWELL ROLLO

CLASSIFICATION CHART FOR CONE PENETRATION TESTS

Date 01/07/14 | Project No. 731628101 | Figure B-6

APPENDIX C

LABORATORY TEST RESULTS



Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-1 at 18.5 feet	CLAYEY SAND (SC), olive-brown	19.0	26	8	42.9
■	B-2 at 6 feet	CLAYEY SAND (SC), brown	--	29	10	--
▲	B-2 at 23.5 feet	CLAYEY SILTY SAND (SC-SM), olive-brown with black mottling	16.7	23	5	26.5
○	B-3 at 3 feet	CLAY with SAND (CL), brown with gray mottling	--	45	23	--

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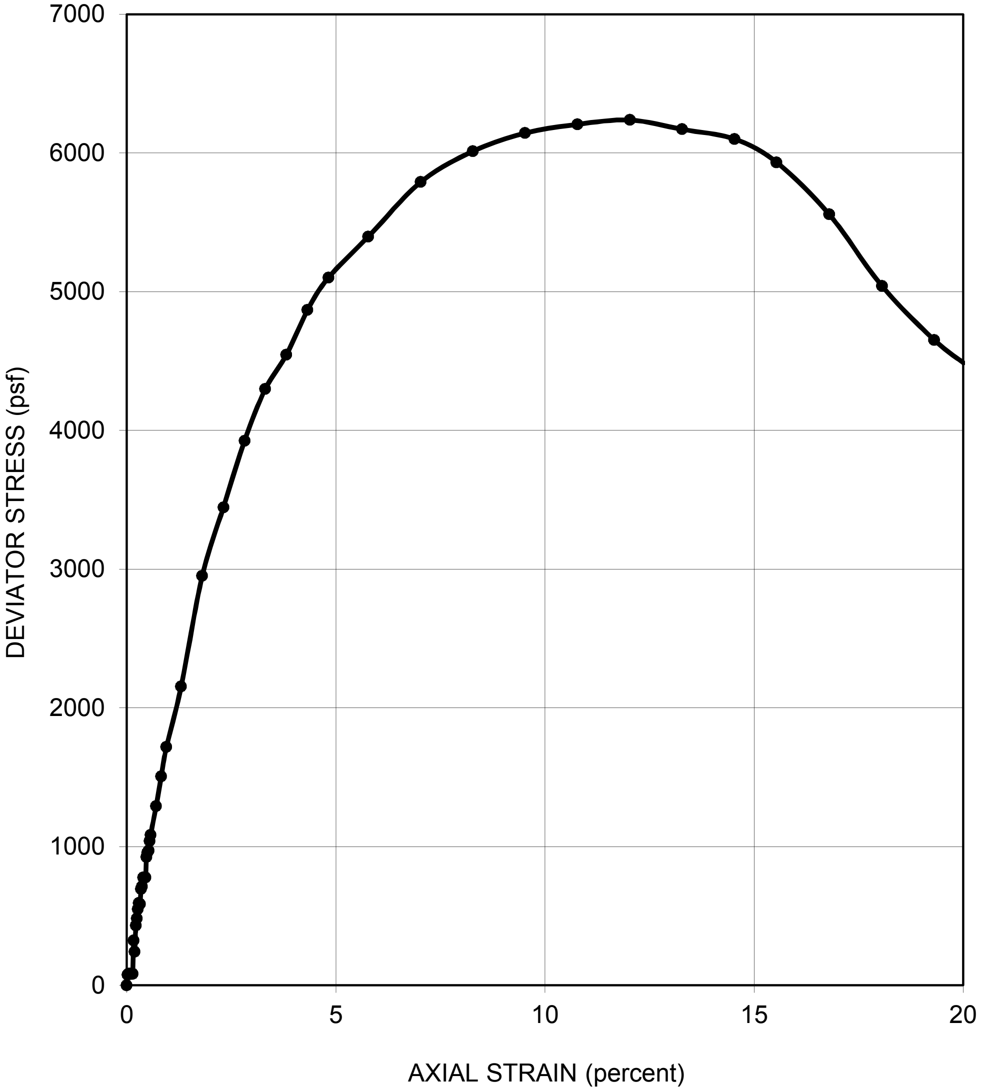
San Ramon, California

LANGAN TREADWELL ROLLO

PLASTICITY CHART

Date 01/16/14 | Project No. 731628101 | Figure C-1



SAMPLER TYPE	Sprague & Henwood			SHEAR STRENGTH	3,120	psf
DIAMETER (in.)	2.40	HEIGHT (in.)	5.70	STRAIN AT FAILURE	12.0	%
MOISTURE CONTENT		22.4	%	CONFINING PRESSURE	500	psf
DRY DENSITY		103	pcf	STRAIN RATE	0.75	% / min
DESCRIPTION	CLAY (CL), olive-brown				SOURCE	B-1 @ 6 feet

2600 CAMINO RAMON
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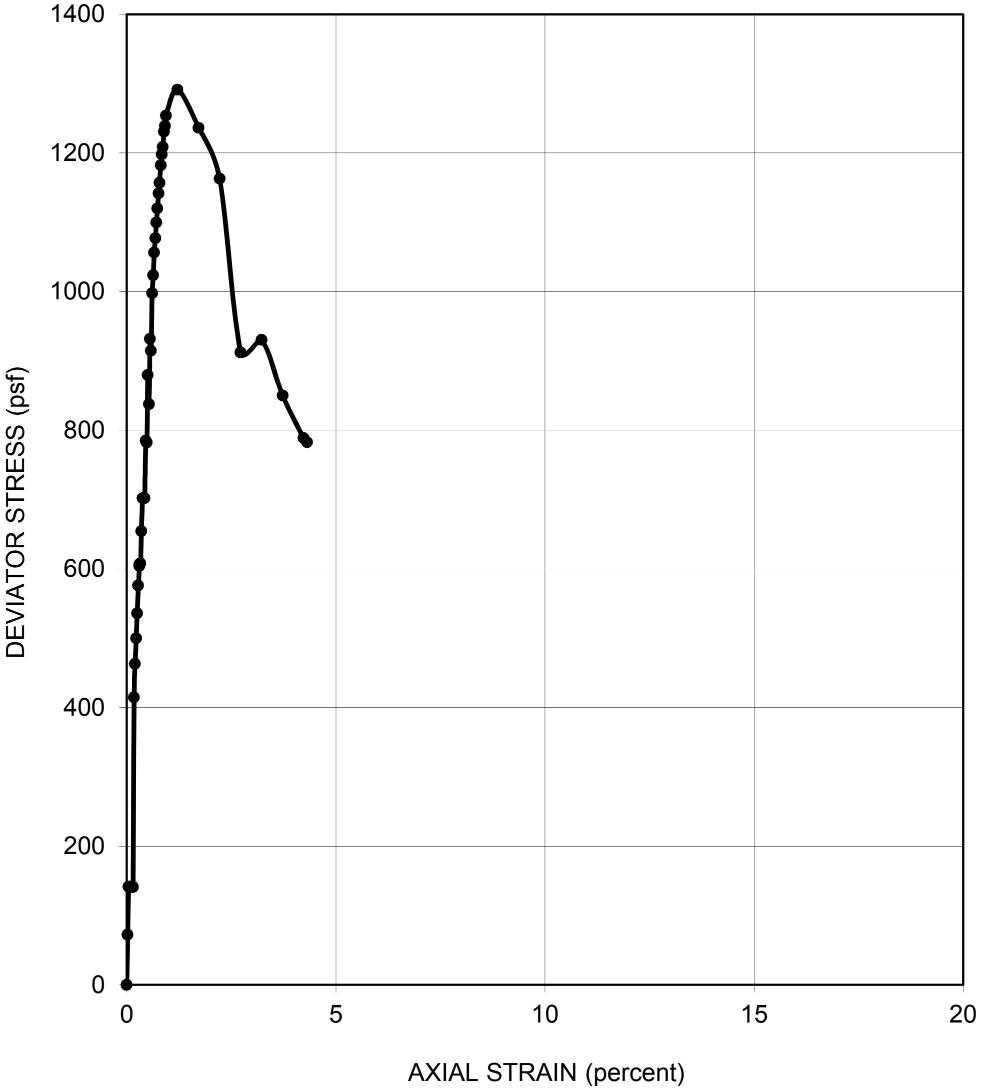
LANGAN TREADWELL ROLLO

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 01/16/14 | Project No. 731628101 | Figure C-2



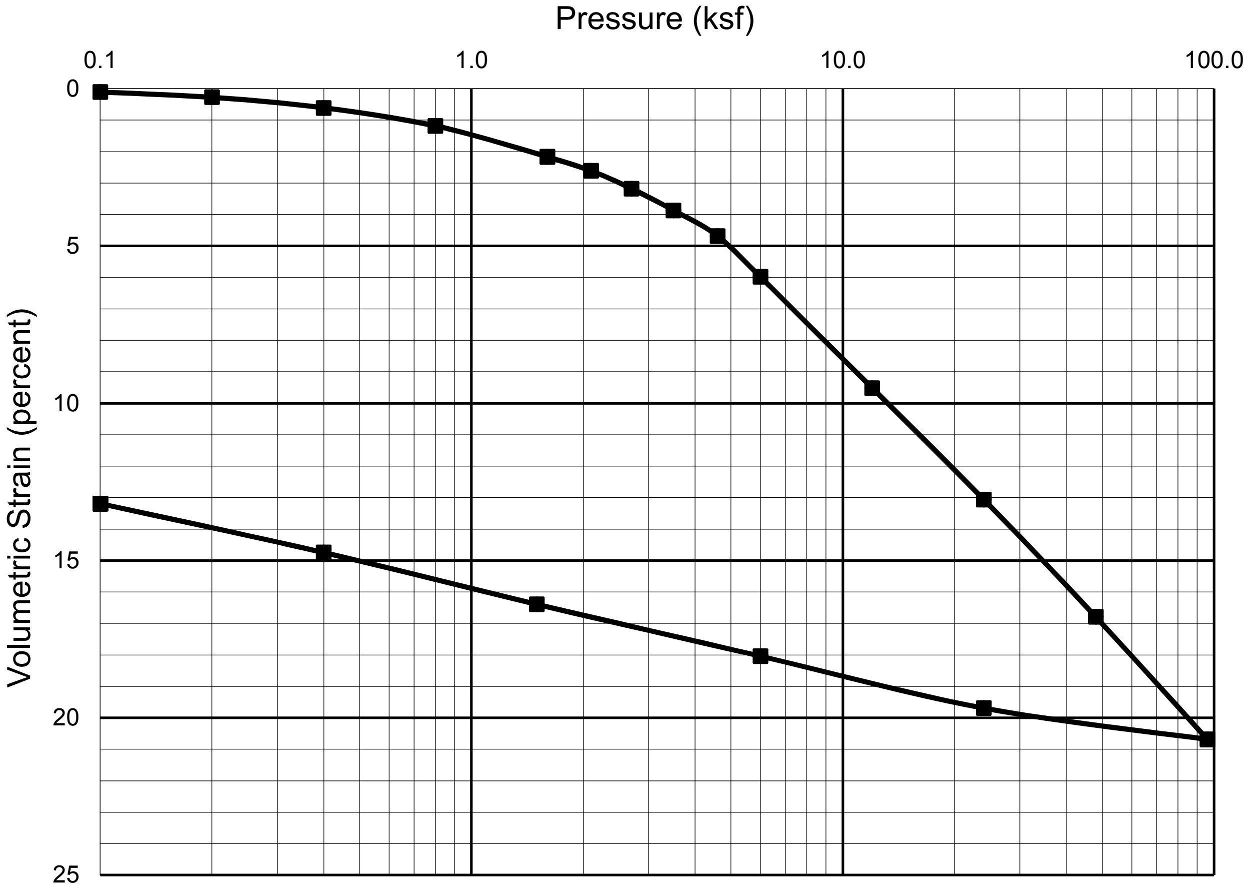
SAMPLER TYPE	Shelby Tube			SHEAR STRENGTH	650	psf
DIAMETER (in.)	2.86	HEIGHT (in.)	6.15	STRAIN AT FAILURE	1.2	%
MOISTURE CONTENT		23.9	%	CONFINING PRESSURE	3,500	psf
DRY DENSITY		102	pcf	STRAIN RATE	0.50	% / min
DESCRIPTION	CLAY (CL), olive-brown				SOURCE	B-3 @ 20 feet

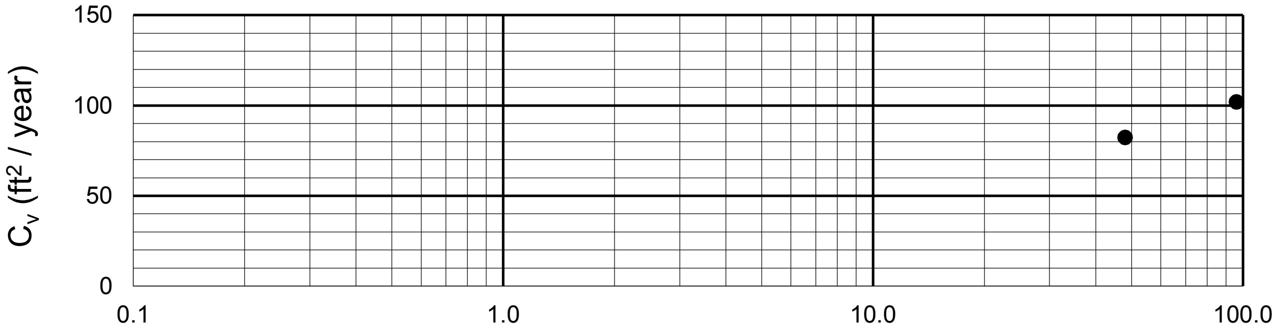
2600 CAMINO RAMON
San Ramon, California

LANGAN TREADWELL ROLLO

UNCONSOLIDATED-UNDRAINED
TRIAXIAL COMPRESSION TEST

Date 01/16/14 | Project No. 731628101 | Figure C-3





Sampler Type: Shelby Tube			Condition		Before Test			After Test	
Diameter (in) 2.42	Height (in) 1.00		Water Content	w_o	23.1	%	w_f	17.1	%
Overburden Pressure, p_o	1,900	psf	Void Ratio	e_o	0.68		e_f	0.46	
Preconsol. Pressure, p_c	4,000	psf	Saturation	S_o	91	%	S_f	100	%
Compression Ratio, $C_{\varepsilon c}$	0.14		Dry Density	γ_d	100	pcf	γ_d	116	pcf

LL -- PL -- PI -- G_s 2.70 (assumed)

Classification CLAY (CL), olive-brown Source B-3 @ 20 feet

2600 CAMINO RAMON
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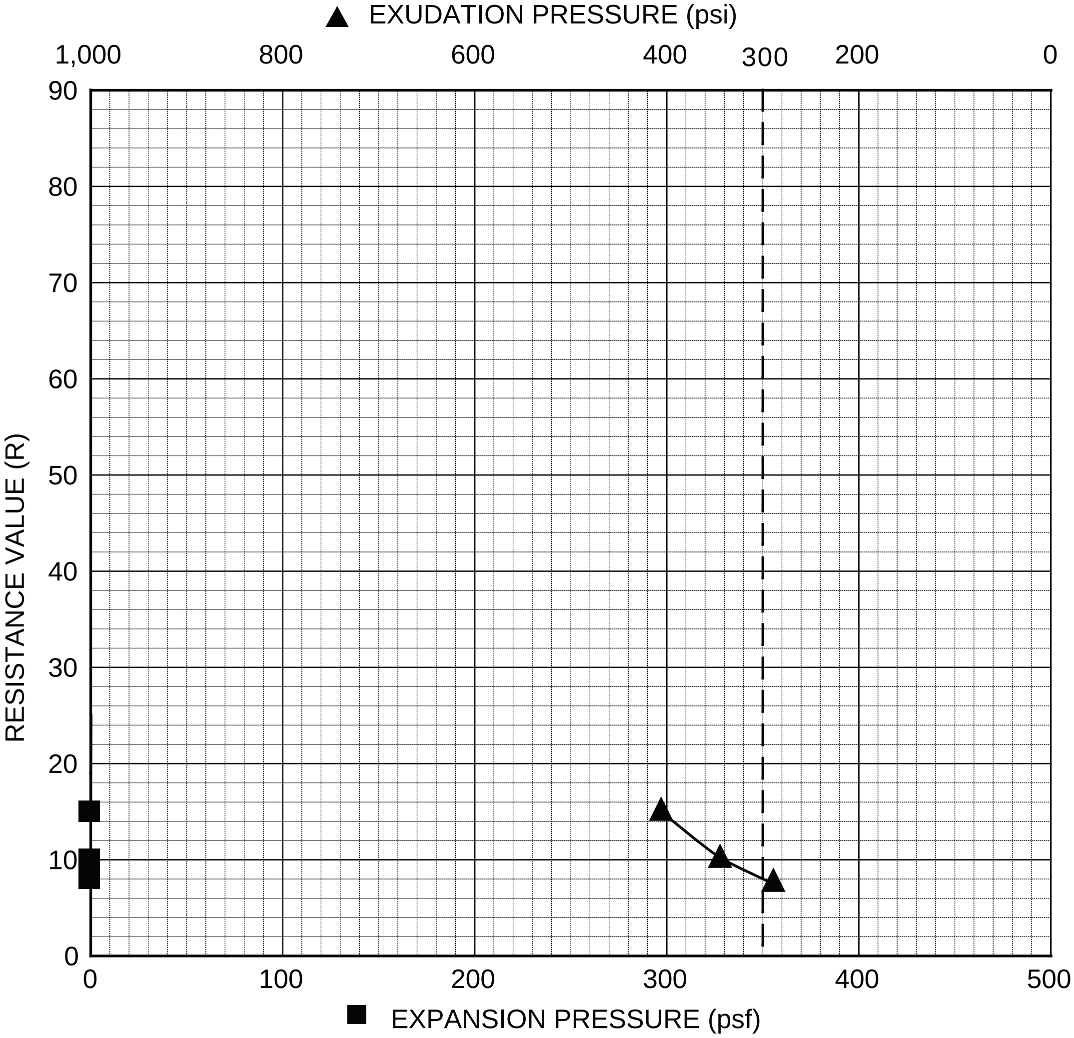
CONSOLIDATION TEST REPORT

Date 01/16/14 Project No. 731628101 Figure C-4



Specimen ID:	A	B	C	D
Water Content (%)	22.1	20.9	20.0	--
Dry Density (pcf)	103.4	104.9	107.2	--
Exudation Pressure (psi)	290	348	406	--
Expansion Pressure (psf)	0	0	0	--
Resistance Value (R)	8	10	15	--

Sample Source	Sample Description	Sand Equivalent	Expansion Pressure	R value
B-3 at 2-5 feet	CLAY with SAND (CL), brown with gray mottling	--	--	8

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RESISTANCE VALUE TEST DATA

Date 01/16/14 | Project No. 731628101 | Figure C-5

E T S

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JOB SITE: 2600 Camino Ramon, San Ramon, California
JOB #: 731628101

DATE RECEIVED 1/7/2014
DATE of COMPLETION 1/14/2014
ANALYST(S) D. Salinas S. Santos
SUPERVISOR D. Jacobson
LAB DIRECTOR G.S. Conrad PhD

LAB SAMPLE NUMBER	SAMPLE ID	DESCRIPTION of SOIL and/or SEDIMENT	SOIL pH -log[H+]	NOMINAL RESISTIVITY ohm-cm	ELECTRICAL CONDUCTIVITY µmhos/cm	SULFATE SO4 ppm	CHLORIDE Cl ppm
05667-1	CR1/SR	B-1-2 @ 3.5'	**5.98**	**1,433**	**[698]**	**83**	**96**
Method	Detection	Limits —>	---	1	0.1	1	1

LAB SAMPLE NUMBER	SAMPLE ID	DESCRIPTION of SOIL and/or SEDIMENT	SALINITY ECe mmhos/cm	SOLUBLE SULFIDES (S=) ppm	SOLUBLE CYANIDES (CN=) ppm	REDOX mV	PERCENT MOISTURE %
05667-1	CR1/SR	B-1-2 @ 3.5'				**+305.3**	
Method	Detection	Limits —>	---	0.1	0.1	-400 -> +800	0.1

** COMMENTS **

Resistivity is below 1,500 ohm-cm, i.e., low (assign 2-10 pts, depending on specs); soil reaction (i.e., pH) is mildly acidic (assign Ø pts); sulfate level is very low (SO4 @ <200 ppm, assign Ø pts); and chloride level is low enough (Cl @ >100 ppm, assig Ø pts); soil is very mildly reduced (assign Ø-3.5 pts, depending on specs). Standard CalTrans times to perforation for this soil are as follows: for 18 ga steel the time is ~11 yrs, and for 12 ga it goes up to ≈24 yrs. For gray/ductile/mild steels and cast iron the calculated average pitting rate for this soil (according to Uhlig) is 0.26 mm/yr putting the 2 mm depth time at <8 yrs, and the 4 mm depth time would be <16 yrs. Sulfate level is low enough that there would be no measurable adverse impact on concrete cement, mortar and grout; chloride level should be low enough that there should be no adverse impact on steel reinforcement. ment. Soil redox is such that there could be some very mild adverse impact on construction materials (i.e., concrete or steel). This soil, in prnciple, could benefit from alkaline (i.e., lime or cement) treatment in that raising its pH to the 7.5-8.5 range would increase the CalTrans 18 ga time to perforation to ~29 yrs; and the pitting rate would decline to 0.094 mm/yr putting the 2 mm depth time at >21 yrs. But lime treatment only persists in protected locations (i.e., underneath slabs, bldgs, etc.); and while cement treatment is more permanent, there can be practical limitations. Otherwise, metal longevity can be improved by upgrading (e.g. increased gauge or more resistant steels, etc.). In fact, many times strength considerations will require use of heavier steel than used in the presented examples such that perforation or pitting times can be well beyond specified life span. Where this is not the case, cathodic protection of coated steel assets could be done as a potential solution for buried assets. Other alternatives include increased or specialized engineering fill, and/or use of plastic, fiberglass or concrete assets. Based on these these results, standard concrete mixes should be fine in this soil. The total points for buried steel is in the range of 2-13.5 pts, depending on the specifications: generally if total ≥10 pts, remediation is required (as this soil could exceed 10 pts, depending on specifications); in addition, specific results could cause outright rejection (e.g. resistivity @ <1,500 ohm-cm, etc.).

\\\NOTES: Methods are from following sources: extractions by Cal Trans protocols as per Cal Test 417 (SO4), 422 (Cl), and 532/643 (pH & resistivity); &/or by ASTM Vol. 4.08 & ASTM Vol. 11.01 (=EPA Methods of Chemical Analysis, or Standard Methods); pH - ASTM G 51; Spec. Cond. - ASTM D 1125; resistivity - ASTM G 57; redox - Pt probe/ISE; sulfate - extraction Title 22, detection ASTM D 516 (=EPA 375.4); chloride - extraction Title 22, detection ASTM D 512 (=EPA 325.3); sulfides - extraction by Title 22, and detection EPA 376.2 (= SMEWW 4500-S D); cyanides - extraction by Title 22, and detection by ASTM D 4374 (=EPA 335.2).

Figure C-6

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QUALITY CONTROL REVIEWER

Richard D. Rodgers

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Senior Principal

GEOTECHNICAL INVESTIGATION
Bishop Ranch – BR3A
San Ramon, California

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LANGAN TREADWELL ROLLO

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LIST OF FIGURES

1.0 INTRODUCTION AND BACKGROUND

This report presents the results of our geotechnical investigation for the proposed Bishop Ranch 3A (BR3A) development to be constructed at 2600 Bollinger Canyon Road in San Ramon, California. This investigation was performed in accordance with our proposal dated 12 October 2015.

The BR3A site is at the northeast corner of the intersection of Bollinger Canyon Road and Camino Ramon, as shown on Figure 1. The 11-acre site is trapezoidal, with maximum plan dimensions of about 730 by 610 feet, and is bound by Bollinger Canyon Road on the south, Camino Ramon on the west, the Iron Horse Regional Trail on the east, and an office building on the north. The site is undeveloped and covered with scattered grass and weeds and areas of soil stockpiles. One fenced contractor equipment storage yard is present at the site. The ground surface at the site slopes gradually up to the northwest, from Elevation 437 to 446 feet.[1]

We understand the proposed development includes construction of a hotel, a parking garage, one residential building, and one mixed-use building, as shown on Figure 2. The hotel is planned to be four to five stories at grade, with maximum plan dimensions of approximately 50 by 240 feet, and will be in the northwest portion of the site. The parking garage will be east of the hotel and will be three to five stories at grade with approximate plan dimensions of 130 by 280 feet. The residential building will be four stories with one basement level and will have maximum plan dimensions of approximately 280 by 610 feet; this building will be in the eastern portion of the site. A mixed-use building with four to five levels of housing over a one-level fitness center and two basement levels is planned for the southwestern portion of the site; this building will have maximum planned dimensions of approximately 220 by 250 feet. Building loads were not available at the time this report was prepared. Additional improvements will include new pavement, concrete flatwork, and landscaping adjacent to the buildings.

---

[1] Elevations are based on the National Geodetic Vertical Datum of 1929 (NGVD 1929) and are based on the plan titled "City of San Ramon City Center, BR3 Site, Existing Utility and Easements, City of San Ramon, Contra Costa County, California" by RJA dated 9 January 2014.

2.0 SCOPE OF SERVICES

Our scope of services, outlined in our proposal dated 12 October 2015, consisted of reviewing available existing subsurface information, exploring the subsurface conditions at the site, and performing laboratory tests and engineering analyses to develop conclusions and recommendations regarding:

- soil and groundwater conditions
- seismic hazards, including ground rupture, liquefaction, lateral spreading, and differential compaction
- mitigation of seismic hazards, if needed
- the most appropriate foundation type(s) for the proposed hotel, garage, residential building, and mixed-use building
- design criteria for the most appropriate foundation type(s), including values for vertical and lateral capacities
- estimated foundation settlement
- floor slabs
- soil subgrade preparation
- fill quality and compaction criteria
- site grading and excavation
- below-grade walls
- temporary shoring
- seismic design criteria in accordance with 2013 California Building Code (CBC)
- construction considerations.

3.0 FIELD EXPLORATION AND LABORATORY TESTING

Subsurface conditions were explored at the site by drilling fourteen borings and advancing nine cone penetration tests (CPTs). The approximate locations of the borings and CPTs are presented on Figure 2. Prior to performing our field investigation we obtained a soil boring permit from the Contra Costa County Environmental Health Division (CCCEHD) and an encroachment permit from the City of San Ramon, notified Underground Service Alert (USA), and retained a private underground utility locating service to check that locations of exploratory points were clear of existing utilities.

3.1 Borings

The borings, designated B-1 through B-14, were drilled using a truck-mounted drill rig equipped with hollow-stem augers on 19 January 2016 and 25 through 28 April 2016 by Exploration Geoservices, Inc. of San Jose, California. The borings were drilled to depths from 35 to 50 feet beneath the existing ground surface (bgs), during which time our engineer logged the soil encountered in the borings and obtained samples of the material encountered for visual classification and laboratory testing. The logs of the borings are presented in Appendix A on Figures A-1 through A-14. The soil encountered in the borings was classified in accordance with the Classification Chart shown on Figure A-15. Soil samples were obtained using two types of driven samplers and one pushed thin-walled sampler:

- Sprague & Henwood (S&H) split-barrel sampler with a 3.0-inch outside diameter and 2.5-inch inside diameter, lined with steel or brass tubes with an inside diameter of 2.43 inches
- Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and 1.43-inch inside diameter
- Shelby Tube (ST) sampler with a 3.0 inch outside diameter and a 2.875-inch inside diameter.

The SPT and S&H samplers were driven with a 140-pound, hydraulic trip wireline safety hammer falling 30 inches. The samplers were driven up to 18 inches and the hammer blows required to advance the samplers every six inches of penetration were recorded and are presented on the boring logs. A “blow count” is defined as the number of hammer blows per six inches of penetration. The blow counts required to drive the S&H and SPT samplers were converted to approximate SPT N-values using factors of 0.6 and 1.0, respectively, to account for sampler type and hammer energy and are shown on the boring logs. The blow counts used for this conversion were the last two blow counts since the sampler was driven more than 12 inches.

The Shelby Tube was pushed hydraulically into the soil; the pressure required to advance the sampler is shown on the logs, measured in pounds per square inch (psi).

Upon completion, the boreholes were backfilled with cement grout in accordance with CCCEHD requirements and under the observation of a CCCEHD inspector. Soil cuttings generated during drilling of the borings were scattered on-site adjacent to the boreholes.

3.2 Cone Penetration Tests

On 25 and 26 April 2016, nine CPTs designated CPT-1 through CPT-9 were advanced by Middle Earth Geo Testing of Fremont, California. The CPTs were advanced to 50 feet bgs.

The CPTs were performed by hydraulically pushing a 1.4-inch-diameter, cone-tipped probe with a projected area of 10 square centimeters into the ground. The cone-tipped probe measures tip resistance, and the friction sleeve behind the cone tip measures frictional resistance. Electrical strain gauges within the cone continuously measure soil parameters for the entire depth advanced. Soil data, including tip resistance and frictional resistance, were recorded by a computer while the test was conducted. Accumulated data were processed by computer to provide engineering information such as the types and approximate strength characteristics of the soil encountered. The CPT logs showing tip resistance, side friction, friction ratio by depth, as well as interpreted standard penetration blow counts and soil behavior type are presented in Appendix B on Figures B-1 through B-9. Soil types were estimated using the classification chart shown on Figure B-10.

3.3 Laboratory Testing

Soil samples obtained from the borings were re-examined in the office for classification and representative samples were selected for laboratory testing. The laboratory testing program was designed to correlate and evaluate engineering properties of the soil at the site. Samples were tested to measure dry density, moisture content, plasticity (Atterberg limits), strength, consolidation, resistance value (R-value), and corrosion potential. Results of the laboratory tests are included on the boring logs and on Figures C-1 through C-10 in Appendix C. The results of the corrosivity testing are presented in Appendix D.

4.0 SUBSURFACE CONDITIONS

Subsurface information from our field investigation indicates localized portions of the site are covered by a thin layer of fill consisting of gravel, clayey sand with gravel, and sandy clay with gravel. Because the fill was only observed in localized areas and because soil stockpiles were observed on the site during our investigation, we judge the fill likely originated from previous soil stockpiles and is therefore uncompacted. The fill is approximately 1 to 2.5 feet thick and was encountered in borings B-2, B-5, and B-9.

In general, the site is underlain by medium stiff to hard clay with varying sand content sand to the maximum depth explored of 50 feet bgs. Laboratory testing performed on near-surface samples of the clay indicates that it has a moderate to very high expansion potential[2] with plasticity indices between 23 and 49. Thin layers of granular soil consisting of sand, silty sand, sand with clay, and clayey sand with variable gravel content were interlayered with the clay in the borings and CPTs. The granular layers are loose to dense and range in thickness from about ½ to 2½ feet.

Where groundwater was measured during our investigation, it was measured between depths of 12 and 23 feet bgs, corresponding to approximate Elevations 432 and 423 feet, respectively. The groundwater levels observed during drilling do not represent stable groundwater conditions, and the groundwater level at the site is expected to vary seasonally.

5.0 REGIONAL SEISMICITY

The major active faults in the area are the Calaveras, Mount Diablo Thrust, and Hayward faults. These and other faults in the region are shown on Figure 3. For each of the active faults within 50 kilometers (km) of the site, the distance from the site and estimated maximum Moment

---

[2] Expansive soil undergoes volume changes with changes in moisture content.

magnitude,[3] M_w, [Working Group on California Earthquake Probabilities (WGCEP) (2008) and Cao et al. (2003)] are summarized in Table 1.

TABLE 1
Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Mean Moment Magnitude
Total Calaveras	1.4	West	7.03
Mount Diablo Thrust	4.6	Northeast	6.70
Total Hayward	15	West	7.00
Total Hayward-Rodgers Creek	15	West	7.33
Green Valley Connected	15	North	6.80
Greenville Connected	16	East	7.00
Great Valley 5, Pittsburg Kirby Hills	30	Northeast	6.70
Great Valley 7	38	East	6.90
N. San Andreas - Peninsula	44	West	7.23
N. San Andreas (1906 event)	44	West	8.05
Monte Vista-Shannon	45	Southwest	6.50

Figure 3 also shows the earthquake epicenters for events with magnitude greater than 5.0 from January 1800 through January 1996. In 1868, an earthquake with an estimated maximum intensity of X on the MM scale occurred on the southern segment (between San Leandro and Fremont) of the Hayward fault. The estimated M_w for the earthquake is 7.0. In 1861, an earthquake of unknown magnitude (probably a M_w of about 6.5) was reported on the Calaveras fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill earthquake ($M_w = 6.2$).

Since 1800, four major earthquakes have been recorded on the San Andreas fault. In 1836 an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale (Figure 4) occurred east of Monterey Bay (Toppozada and Borchardt 1998). The estimated M_w

---

[3] Moment magnitude is an energy-based scale and provides a physically meaningful measure of the size of a faulting event. Moment magnitude is directly related to average slip and fault rupture area.

for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to a M_w of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas fault from Shelter Cove to San Juan Bautista, approximately 470 kilometers in length. It had a maximum intensity of XI (MM), a M_w of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The Loma Prieta Earthquake of 17 October 1989 with a M_w of 6.9 and an epicenter in the Santa Cruz Mountains, approximately 81 km from the site. The most recent earthquake to affect the Bay Area occurred on 24 August 2014 and was located on the West Napa fault, approximately 60 kilometers north of the site, with a M_w of 6.0.

The 2008 WGCEP at the U.S. Geologic Survey (USGS) predicted a 63 percent chance of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Area in 30 years. More specific estimates of the probabilities for different faults in the Bay Area are presented in Table 2.

TABLE 2
WGCEP (2008) Estimates of 30-Year Probability of a Magnitude 6.7 or Greater Earthquake

Fault	Probability (percent)
Hayward-Rodgers Creek	31
N. San Andreas	21
Calaveras	7
San Gregorio	6
Concord-Green Valley	3
Greenville	3
Mount Diablo Thrust	1

6.0 SEISMIC HAZARDS

The site is in a seismically active area and will be subject to strong shaking during a major earthquake on a nearby fault. Strong shaking during an earthquake can result in ground failure such as that associated with soil liquefaction[4], lateral spreading[5], seismic densification[6], and fault rupture. Each of these conditions has been evaluated based on our literature review, field investigation, and studies, and is discussed in this section.

6.1 Fault Rupture

Historically, ground surface displacements closely follow the trace of geologically young faults. The site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act and no known active or potentially active faults exist on the site. Therefore, we conclude the risk of surface faulting and consequent secondary ground failure is low.

6.2 Liquefaction and Associated Hazards

When a saturated, cohesionless soil liquefies during an earthquake, it experiences a temporary loss of shear strength due to a transient rise in excess pore water pressure generated by strong ground motion. Flow failure, lateral spreading, differential settlement, loss of bearing strength, ground fissures, and sand boils are evidence of excess pore pressure generation and liquefaction.

The level of ground shaking used in our liquefaction evaluation was based on the Risk-Target Maximum Considered Earthquake (MCE_R) mapped values. A peak geometric mean ground acceleration (PGA_M) of 0.882g was used in our analyses. This PGA_M was obtained from mapped

---

[4] Liquefaction is a transformation of soil from a solid to a liquefied state during which saturated soil temporarily loses strength resulting from the buildup of excess pore water pressure, especially during earthquake-induced cyclic loading. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits.

[5] Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported downslope or in the direction of a free face by earthquake and gravitational forces.

[6] Seismically-induced densification, also known as differential compaction, is a phenomenon in which non-saturated, cohesionless soil is compacted by earthquake vibrations, causing differential settlement.

values specified in the provisions of the 2013 California Building Code (CBC)/ASCE 7-10 for the MCE_R, using site class D. We assumed an earthquake magnitude of 7.33, which is the maximum Moment magnitude for the Hayward fault, located 15 km from the site, as shown on Table 1. The Calaveras fault is significantly closer to the site, but was not selected for our evaluation since it has a lower maximum Moment magnitude. A groundwater level of 12 feet bgs was used in the analyses.

We used the results of the CPTs to evaluate liquefaction potential at the site. The liquefaction analyses using CPT data were performed in accordance with the methodology presented in the publication titled *Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils*, prepared by the Youd et al., dated October 2001. Our liquefaction analysis using the CPT data indicates that thin layers of loose to medium dense granular soil below the groundwater table in the CPTs are susceptible to liquefaction (FS_{liq}<1.3) following a major earthquake on a nearby fault.

We estimated liquefaction-induced settlement using the procedure outlined in the NCEER report. In each of the CPTs, one to three layers of potentially liquefiable soil were encountered, each less than 1½ feet thick, with calculated total liquefaction-induced settlements of about ¼ to ½ inch.

In addition, we evaluated the potential for liquefaction using the results of our borings. In nine of the borings, one to two layers of potentially liquefiable soil were encountered, with thicknesses of about ½ to 2½ feet, and calculated total liquefaction-induced settlements between about ¼ and 1¼ inches. However, liquefaction analyses using SPT data from hollow stem auger borings generally produce conservatively large settlements because the soil below the groundwater level tends to heave in the borings during drilling and as a result, the SPT blow counts are conservative; we judge the actual settlements may be on the order of half of that calculated using the boring results. In addition, several of the layers identified in the borings as potentially liquefiable contained substantial amounts of clay and gravel, and will likely settle less than calculated. Therefore, while some liquefiable soil may be present, we judge liquefaction-induced settlement would be less than that calculated using the boring data.

The results of our liquefaction analyses indicate there are thin layers of loose to medium dense sand with variable clay, silt, and gravel content below the groundwater table that are susceptible to liquefaction during a major earthquake on a nearby fault. Based on our liquefaction analyses using the borings and CPTs, we conclude that up to about ½ inch of liquefaction-induced total settlement may occur at the site as a result of a major earthquake on a nearby fault. The liquefaction may occur in isolated areas and differential settlement may be abrupt; therefore, differential settlements equivalent to the total settlement of ½ inch should be anticipated over short distances. These total and differential settlements are expected to occur beneath the foundation levels of the planned at-grade hotel, at-grade parking garage, and the residential building with one basement level. The planned mixed-use building with two basement levels will likely be founded below the majority of the liquefiable soil layers; for this building, we calculate less than ¼ inch of total and differential liquefaction-induced settlement could occur during a major earthquake.

The potential for liquefaction-induced ground rupture and sand boils to occur at the site depends on the thickness of the liquefiable soil layer relative to the thickness of the overlying non-liquefiable material. Ishihara (1985) presented an empirical relationship that provides criteria that can be used to evaluate whether liquefaction-induced surface ruptures and sand boils would be expected to occur under a given level of shaking for a liquefiable layer overlain by a non-liquefiable layer. The potentially-liquefiable layers encountered at the site are relatively thin and deep (the layers closest to the ground surface are 3 feet thick or less and are at least 12 feet deep); therefore, we conclude that the potential for surface manifestations of liquefaction to occur at the site is low.

6.3 Lateral Spreading

Lateral spreading occurs when a continuous layer of soil liquefies at depth and the soil layers above move toward an unsupported face, such as an open slope cut, or in the direction of a regional slope or gradient. The potential for lateral spreading to occur at a site is typically evaluated using an empirical relationship developed by Youd, Hansen, and Bartlett (2002). This relationship incorporates the thickness of the liquefiable layer, the fines content and mean grain-size diameter of the liquefiable soil, the relative density of the liquefiable soil, the magnitude and distance of the earthquake from the site, the slope of the ground surface, and boundary conditions (such as a free face or edge of shoreline), to estimate the horizontal ground movement. The potentially liquefiable layers encountered in the CPTs and borings are thin, isolated, and discontinuous. In addition, the $(N_1)_{60\text{-}CS}$ values for the potentially liquefiable soil layers in the borings are generally greater than 15, with the exception of one thin layer encountered in boring B-4; soil with $(N_1)_{60\text{-}CS}$ values greater than 15 are considered too dense to laterally spread. For reasons discusses in Section 6.2, we consider the blowcounts from the hollow stem auger borings to be conservatively low. Therefore, we conclude the potential for lateral spreading beneath the site is low.

6.4 Seismic Densification

Seismic densification (also referred to as differential compaction) can occur during strong ground shaking in loose, clean granular deposits above the water table, resulting in ground surface settlement. In borings B-2 and B-5, 1 to 2½ feet of gravel and clayey sand with gravel fill were encountered below the ground surface. We evaluated the potential for differential compaction to occur in the granular fill at the site using methodology presented in Tokimatsu and Seed (1984). Based on this method, we estimate ground surface settlements associated with seismic densification on the order of 1/3 inch or less as a result of strong shaking during a major earthquake on a nearby fault.

7.0 DISCUSSION AND CONCLUSIONS

We conclude that from a geotechnical engineering standpoint, the site can be developed as planned, provided the recommendations presented in this report are incorporated into the project plans and specifications and are implemented during construction. The primary geotechnical concerns for the project are:

1) the presence of moderately to very highly expansive near-surface soil
2) excavation of the site below the groundwater to construct below-grade levels
3) selection of an appropriate foundation system to support anticipated building loads and reduce total and differential static settlements to within tolerable limits.

Our conclusions regarding expansive soil and other geotechnical issues are presented in this section.

7.1 Groundwater and Dewatering

Groundwater levels were measured during our field investigation between about 12 and 23 feet bgs, corresponding to approximate Elevations 434.5 and 423 feet, respectively. We conclude a design water level of 10 feet bgs, corresponding to approximate Elevations of 437 to 427 should be used at the site.

We understand the planned hotel and parking garage will be constructed at grade. We do not anticipate encountering groundwater in foundation excavations for the at-grade buildings; however, if localized water is present during excavation, dewatering should be performed. Where excavations encounter groundwater, wet, disturbed subgrade soil will require stabilization prior to placement of improvements. One method of stabilizing subgrade consists of overexcavating the disturbed material and replacing it with a lean concrete rat slab. Groundwater encountered in foundation excavations will need to be pumped out prior to placing concrete.

We understand the proposed residential building will be constructed with one basement level and the proposed mixed-use building will be constructed with two basement levels. We anticipate excavation for one to two basement levels will extend to about 12 and 24 feet bgs, respectively. Groundwater will likely be encountered in the basement excavations and we anticipate temporary dewatering will be required during construction. The dewatering system should be designed and installed by an experienced contractor. An active dewatering system is recommended if shoring is to consist of soldier piles and lagging. The active dewatering system should be designed to maintain the groundwater at least three feet below the lowest level of excavation. Even with active dewatering, the soil at the base of the excavation will likely be near saturation, and may require stabilization using one of the methods previously described for the at-grade buildings.

7.2 Foundation Support

The site is generally underlain by moderately to highly expansive near-surface soil. Very highly expansive soil was encountered in one boring, B-14, within the footprint of the residential building. Expansive soil is subject to high volume changes during seasonal fluctuations in moisture content, which can cause cracking of foundations and floor slabs. The detrimental effects of near-surface expansive soil can be mitigated by moisture conditioning the expansive soil below slabs, placing non-expansive fill below slabs, supporting foundations below the zone of severe moisture change, and/or designing foundations to resist the movements associated with the volume changes. Because moderately to highly expansive soil is present within the footprints of the at-grade hotel and parking garage, we conclude the building foundations of these structures will need to be designed to reduce the potential for movement due to moisture change. If very highly expansive material is encountered within the footprint of the at-grade buildings, the building foundations will need to be supported below the zone of moisture change. Because the residential and mixed-use buildings will have below-grade levels, we concluded the foundations will be supported below the zone of severe moisture change and will not be susceptible to the effects of volume changes. Foundation options for the planned buildings are discussed in the following sections.

7.2.1 At-Grade Structures

The native soil at the foundation level of the planned hotel and parking garage has moderate strength and relatively low compressibility, and we conclude these buildings can be supported on spread footings bearing on native soil. If very highly expansive soil is encountered within the footprints of the at-grade buildings during construction, the structures will need to be supported on drilled piers or the very highly expansive soil beneath the footings will need to be overexcavated to below the zone of moisture change and replaced with lean concrete to the planned footing bottom. Prior to construction, we can perform additional exploration and laboratory testing within the footprint of the at-grade structures to check for the presence of very highly expansive soil.

Because moderately to highly expansive soil has been encountered within the footprints of the at-grade buildings, the spread footings should be continuous and deepened around the perimeter of the buildings and deepened at interior column locations. The continuous perimeter footings will act as a barrier to reduce the potential for moisture change beneath the floor slabs. We estimate the total static settlement of properly constructed spread footings, designed using the recommendations presented in Section 8.2, will be about 1 inch, with differential settlement across a horizontal distance of 30 feet on the order of ½ inch. In addition, seismically-induced settlement may occur during a major earthquake, as discussed in Section 6.1.

7.2.2 Residential Building

We anticipate the soil exposed at the foundation level of the planned residential building with one basement level will be medium stiff to stiff clay with variable sand content and medium dense silty or clayey sand. The soil at the foundation level has moderate strength and moderate compressibility and is below the zone of moisture change. We conclude the residential building should be supported on a mat to limit static settlements to anticipated project tolerances, and because the foundation will most likely be constructed close to or below the groundwater table and near thin layers of potentially-liquefiable sand. If liquefiable sand layers are exposed at the foundation level during excavation, the sand will need to be either overexcavated and recompacted, overexcavated and replaced with lean concrete, or lime treated in accordance with the recommendations in Section 8.1.2. We should further evaluate the potential for additional subgrade preparation due to the presence of liquefiable soil after the final foundation depth is known. We estimate the total static settlement of a properly constructed mat for the residential building, designed using the recommendations presented in Section 8.2, will between 1 and 1¼ inches. In addition, seismically-induced settlement may occur during a major earthquake, as discussed in Section 6.1. Differential settlement will depend on the rigidity of the mat.

7.2.3 Mixed-Use Building

We anticipate the soil exposed at the foundation level of the planned mixed-use building with two basement levels will be stiff to very stiff clay with variable sand content. The soil at the foundation has moderate to high strength and moderate compressibility and is below the zone of moisture change. Because the foundation will be constructed below groundwater and to limit total and differential static settlement to anticipated project tolerances, we recommend the residential building be supported on a mat. We estimate the total static settlement of a properly constructed mat for the mixed-use building, designed using the recommendations presented in Section 8.2, will be up to 1.5 inches. Differential settlement will depend on the rigidity of the mat.

7.3 Floor Slabs

The floor slab for the at-grade buildings will be underlain by medium stiff to very stiff clay; therefore, we conclude the slab may be supported on grade. Because the near-surface soil is moderately to highly expansive, the floor slab and capillary break/vapor retarder (recommended in Section 8.3) should be underlain by at least 18 inches of non-expansive soil to mitigate the potential for movement of the slabs. The non-expansive soil may consist of imported select fill or lime-treated native soil. If very highly expansive soil is encountered within the footprints of the at-grade buildings during construction, at least 24 inches of non-expansive soil will be required to mitigate the potential for movement of the slabs.

The residential and mixed-use buildings will include one and two below-grade levels, respectively. Because these buildings will be founded on mats below the expansive soil zone, we conclude the subgrade treatments discussed above will not be required for these buildings.

If the floor slab/mats extend below or are less than 30 inches above the design groundwater level, the slab/mats will need to be waterproofed (recommended in Section 8.3). If the floor slab or mat bottom will be at least 30 inches above the design water level but moisture on the finished floor is undesirable, a capillary moisture break and water vapor retarder (recommended in Section 8.3) can be installed beneath the slab/mat to reduce water vapor transmission through the slab. Mats extending below the design groundwater level will need to designed to resist hydrostatic uplift pressure.

7.4 Excavation and Temporary Cut Slopes

We anticipate excavations on the order of about 15 to 30 feet will be needed to construct the below grade levels of the residential and mixed-use buildings, respectively. In addition, excavations will be required for hotel and parking garage footings, elevator pits, and below-grade utilities. The soil to be excavated consists predominantly of clay and sand, which can be excavated using conventional earth-moving equipment such as loaders and backhoes. Excavations that will be deeper than five feet and will be entered by workers will need to be shored or sloped in accordance with the Occupational Safety and Health Administration (OSHA) standards (29 CFR Part 1926). Recommendations for temporary cut slopes and shoring are provided in Section 8.5.

7.5 Corrosion Potential

Corrosivity testing was performed on a sample from a depth of 3½ feet in boring B-2 and a sample from a depth of 3 feet in boring B-7. The soil samples were tested in accordance with Caltrans and ASTM protocols by CERCO Analytical of Concord, California. The corrosivity test results are presented on Figure D-1 in Appendix D. Below grade structures will need to be designed for the corrosive conditions encountered at the site.

7.6 Construction Considerations

As previously discussed, the site is underlain by moderately to very highly expansive near-surface soil. If the soil subgrade is exposed and allowed to dry during excavation for the foundations and is not properly moisture-conditioned prior to placement of concrete, significant heave may occur as soil moisture levels increase after construction, which could damage the structure. Therefore, it is essential to maintain moisture during construction. Typically, it is necessary to spray the exposed bottom and sides of foundation excavations on a daily basis to prevent drying.

In addition, if construction activities are performed during the winter/rainy season, the near-surface soils will be saturated, soft, and easily remolded. If weak or saturated soil is encountered during construction, it can be stabilized by overexcavating the upper 12 to 24 inches of the weak soil, placing a stabilization geotextile (Mirafi RS380i or equivalent) over the sides and bottoms of the overexcavated areas, and placing and compacting granular fill, such as ½- to ¾-inch crushed rock or Class 2 aggregate base (AB), over the geotextile fabric. Wet soil will require significant drying before it can be used as fill or backfill.

8.0 RECOMMENDATIONS

Our recommendations regarding design of foundations, below-grade walls, temporary shoring, floor slabs, pavement, and other geotechnical aspects of this project are presented in this section.

8.1 Earthwork

8.1.1 Site Preparation

Site preparation should include removal of all existing structures, foundations, slabs, pavements, and underground utilities, if any, within the footprint of the planned development. All areas to receive improvements should be stripped of vegetation, organic topsoil, and fill from soil stockpiles. Stripped materials should be removed from the site or stockpiled for later use in landscaped areas, if approved by the landscape architect. Any subsurface structures should be removed, and any fill uncovered should be overexcavated and recompacted. Underground utilities should be removed to the service connections and properly capped or plugged with concrete. Where existing utility lines will not interfere with the planned construction, they may be abandoned in-place, provided the lines are filled with lean concrete or cement grout to the limits of the project. Voids resulting from demolition activities should be properly backfilled with engineered fill as described in Section 8.1.3.

8.1.2 Subgrade Preparation

Because moderately to highly expansive soil was encountered within the footprints of the at-grade hotel and parking garage, we recommend the floor slabs and underlying capillary break/vapor retarders (recommended in Section 8.3) for these buildings be underlain by at least 18 inches of non-expansive soil consisting of either select fill or lime-treated native soil, as described in the following sections. As discussed in Section 7.3, if very highly expansive soil is encountered within the footprints of the at-grade buildings during construction, at least 24 inches of non-expansive soil should be used to mitigate the potential for movement of the slabs. Criteria for select fill are provided in Section 8.1.3. As previously discussed, the proposed residential and mixed use buildings will be supported on mats below the zone of moisture change and therefore, the subgrade treatments discussed above are not required for these buildings. The soil subgrade should be kept moist until it is covered by fill or improvements.

Slab-on-Grade

Placement of Select Fill (Alternative No. 1)

If Alternative No. 1 is selected, we recommend the building pad be overexcavated to allow placement of at least 18 inches of select fill beneath the slab-on-grade floor and underlying capillary moisture break. Where the site will be raised at least 18 inches above the existing ground surface following demolition, the fill used to raise the grade for the building pad should consist of select fill. The select fill should extend at least five feet beyond the building footprint. The native expansive soil at the base of the overexcavation or at current grade prior to raising grades should be scarified to a depth of at least 12 inches, moisture-conditioned to at least three percent above optimum moisture content, and compacted to between 88 and 92 percent relative compaction.[7] Any existing fill encountered at the base of the overexcavation or at current grade prior to raising grades should be overexcavated and recompacted as recommended in Section 8.1.1 prior to placement of select fill. Select fill should be moisture conditioned to above optimum moisture content and compacted to at least 90 percent relative compaction. The geotechnical engineer should observe subgrade preparation prior to placement of select fill.

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[7] Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557 laboratory compaction procedure.

Lime Treatment (Alternative No. 2)

If Alternative No. 2 is selected, the upper 18 inches of the building pad (measured below the capillary moisture break) should be treated in place with between four to eight percent (to be determined by the specialty contractor performing the lime treatment) dolomitic quicklime by dry weight of soil. The limit of lime treatment should extend at least five feet beyond the building footprint. A specialty subcontractor typically performs lime treatment, and we recommend this work be performed only by an experienced contractor. The contractor should determine the percent lime to be used and confirm there are no chemical constituents present that would adversely affect the lime treatment. Prior to lime treatment, we recommend the building pad be graded in accordance with our previous recommendations and all below-grade obstructions be removed. The soil treated with lime should be mixed and compacted in one lift. The lime should be thoroughly blended with the soil and allowed to cure for 24 hours prior to remixing and compaction. The lime-treated soil should be moisture-conditioned to above optimum moisture content and compacted to at least 90 percent relative compaction. Lime-treated soil should be removed from landscaping areas as it will inhibit growth of vegetation. It should be noted that disposal of lime-treated soil is typically expensive because of the high pH of the treated soil.

Below-Grade Mat Foundations

The residential and mixed-use buildings will have below-grade levels, and we conclude the mat foundations may bear on native soil. Although the excavations for the residential building and the mixed-use building site will be actively dewatered, the soil at basement subgrade level will likely be saturated or near saturation. To protect the subgrade, we recommend heavy construction equipment (such as scrapers) not be allowed within two feet of subgrade and that final excavation be made with an excavator equipped with a smooth bucket. The soil subgrade should be kept moist until the mat foundations or rat slabs are in place.

Pavement Areas

In asphalt and concrete pavement areas, where engineered or select fill is exposed at soil subgrade, the upper six inches should be moisture-conditioned to above optimum moisture content and compacted to at least 95 percent relative compaction to provide a smooth non-yielding surface. If expansive native clay is at subgrade in pavement areas, the upper 12 inches should be moisture-conditioned to at least two percent over optimum moisture content and compacted to at least 90 percent relative compaction.

Flatwork Areas

As a minimum preparation for exterior concrete flatwork, including concrete slabs and sidewalks, the upper 12 inches of expansive native soil at subgrade should be moisture-conditioned to at least three percent above optimum moisture content and compacted to between 88 and 92 percent relative compaction. If it is desirable to reduce the potential for differential movement and cracking, concrete flatwork should be underlain by at least 12 inches of select fill, lime-treated soil, or Caltrans Class 2 AB, as recommended in Section 8.4. Select fill and Class 2 AB at subgrade in concrete flatwork areas should be moisture-conditioned to above optimum moisture content and compacted to at least 95 percent relative compaction. Lime-treated soil at subgrade should be moisture-conditioned to above optimum moisture content and compacted to at least 90 percent relative compaction.

8.1.3 Fill Placement

We anticipate fill placement at the site will consist primarily of minor grading for the at-grade building pads, pavement, and flatwork areas and backfill for utility trenches and around elevator pit walls. Because the on-site soil is moderately to very highly expansive, the floor slabs of the planned at-grade buildings should be underlain by at least 18 inches of select fill or lime-treated soil, as recommended in Section 8.1.2. Prior to placement of fill, the subgrade soil should be scarified, moisture-conditioned, and recompacted as recommended in Section 8.1.2.

If native expansive clay is to be used as general site fill, it should be moisture-conditioned to at least three percent above optimum moisture content, placed in horizontal lifts not exceeding eight inches in loose thickness, and compacted to between 88 and 92 percent relative compaction.

Select fill should consist of imported or on-site soil that is free of organic matter and hazardous material, contain no rocks or lumps larger than three inches in greatest dimension, have a liquid limit less than 40 and plasticity index less than 12, and be approved by the geotechnical engineer. In addition, select fill used within the at-grade building footprints should contain at least 20 percent fines (particles passing the No. 200 sieve) to reduce the potential for surface water to infiltrate beneath slabs. Select fill should be placed in lifts not exceeding eight inches in loose thickness, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent relative compaction for fill thickness equal to or less than five feet and 95 percent relative compaction for fill thickness greater than five feet.

We should approve all sources of engineered fill at least three days before use at the site. The grading subcontractor should provide analytical test results or other suitable environmental documentation indicating the imported fill is free of hazardous materials at least three days before use at the site. If this data is not available, up to two weeks should be allowed to perform analytical testing on the proposed import material.

8.1.4 Utility Trenches

Excavations for utility trenches can be made with a backhoe. All trenches should conform to the current OSHA requirements for slopes, shoring, and other safety concerns.

To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of sand or fine gravel. After the pipes and conduits are tested, inspected (if required), and approved, they should be covered to a depth of six inches with sand or fine gravel, which should be mechanically tamped. If groundwater is encountered during trench excavation, the gravel used as bedding and cover should be replaced with Caltrans Class 2 permeable material below the water level, or the open-graded gravel used as bedding and cover should be wrapped in filter fabric (Mirafi 140N or equivalent) to reduce the potential for infiltration of fines.

Backfill for utility trenches and other excavations is also considered fill and should be placed and compacted according to the recommendations previously presented. If imported clean sand or gravel (defined as soil with less than 10 percent fines) is used as backfill, it should be compacted to at least 95 percent relative compaction. Jetting of trench backfill should not be permitted. Special care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to the pavement section.

Where utility trenches backfilled with sand or gravel enter the building pad, an impermeable plug consisting of native clay or lean concrete, at least five feet in length, should be installed at the building line. Further, where sand- or gravel-backfilled trenches cross planter areas and pass below asphalt or concrete pavements, a similar plug should be placed at the edge of the pavement. The plug should extend from the bottom of the trench to the subgrade elevation. The purpose of these recommendations is to reduce the potential for water to become trapped in trenches beneath the building or pavements. This trapped water can cause heaving of soils beneath slabs and softening of subgrade soil beneath pavements.

8.2 Foundation Support

The planned at-grade hotel and parking garage may be supported on a combination of deepened continuous perimeter footings and isolated interior spread footings bearing on native soil. We recommend mat foundations be used for support of the residential and mixed-use buildings because, with one and two basement levels, respectively, the foundation will be constructed close to or below the design groundwater table and it is desirable to limit static settlements to anticipated project tolerances. Also, some areas of potentially liquefiable soil are present at the foundation level of the residential building, and a mat will reduce the potential for differential settlement. If needed, uplift can be resisted using tiedown anchors. Recommendations for footings and mat foundations are presented in the following sections. We can provide recommendations for tiedown anchors upon request.

8.2.1 Spread Footings for At-Grade Buildings

Continuous footings should be at least 18 inches wide and isolated spread footings should be at least 24 inches wide. Because moderately to highly expansive soil was encountered within the footprints of the at-grade buildings, we recommend the continuous perimeter footings be bottomed at least 36 inches below the lowest adjacent exterior soil subgrade or the top of the select fill or lime-treated layer, whichever is deeper, to reduce the potential for movement of the footings due to shrink and swell of the expansive clay. The interior footings should extend at least 30 inches below the lowest adjacent soil subgrade (measured from the top of the select fill or lime-treated soil). Footings located adjacent to utility trenches or other foundations should bear below an imaginary 1.5:1 (horizontal to vertical) plane projected upward from the bottom edge of the adjacent trench or the bottom of the adjacent foundation. As discussed in Section 7.2.1, if very highly expansive soil is encountered beneath the footings during construction, it should be overexcavated to below the zone of moisture change and replaced with lean concrete to the planned footing bottom, or drilled piers should be used.

Footings should be designed using an allowable bearing pressure for dead plus live loads of 5,000 pounds per square foot (psf) assuming a maximum column load on the order of 600 to 700 kips. This allowable bearing pressure may be increased by one-third for total loads, including wind or seismic forces, and include factors of safety of at least 2.0 and 1.5 for dead plus live loads and total loads, respectively. To design footings using the modulus of subgrade reaction method, we recommend a modulus of subgrade reaction of 60 kips per cubic foot (kcf) be used.

Lateral loads can be resisted by a combination of passive pressure acting on the vertical faces of the footings and friction along the base of the footings. We recommend passive resistance be calculated using a uniform pressure (rectangular distribution) of 2,300 psf in native soil and lime-treated soil and an equivalent fluid weight (triangular distribution) of 350 pounds per cubic foot (pcf) in select fill. The upper foot of soil should be ignored unless it is confined by slabs or pavement. Frictional resistance at the base of the footings should be computed using a friction coefficient of 0.30. These values include a factor of safety of about 1.5.

The footing excavations should be free of standing water, debris, and disturbed materials prior to placing concrete. If loose or soft soil or non-engineered fill is encountered in a footing excavation, the weak soil or fill should be overexcavated to expose stiff to very stiff clay.

The excavated material should be replaced with either structural concrete or sand-cement slurry with a minimum 28-day compressive strength of at least 50 pounds per square inch (psi).

The bottoms and sides of excavations should be wetted following excavation and maintained in a moist condition until concrete is placed. If the foundation soil dries during construction, the foundation will eventually heave, which may result in cracking and distress. We should check foundation excavations prior to placement of reinforcing steel to confirm suitable bearing material is present. We should recheck the condition of the excavations just prior to concrete placement to confirm the excavations are sufficiently moist.

8.2.2 Mat Foundations

The residential and mixed-use buildings should be supported on a mat bearing on native, undisturbed soil. We expect the average mat bearing pressures for the residential and mixed-use building will be about 750 psf and 1,200 psf, respectively, based on our experience with buildings of similar size; however, concentrated stresses may occur at interior columns and at the edges of the mat. The mat foundations may be designed to impose a maximum dead plus live load pressure under columns and walls equivalent to allowable bearing capacities of 4,800 psf and 4,500 psf for the residential and mixed-use buildings, respectively. The allowable bearing pressure can be increased by one-third for total design loads, including wind and seismic loads. The allowable bearing pressures for dead plus live and total design loads include factors of safety of about 2.0 and 1.5, respectively. During a seismic event, these pressures may be exceeded under portions of the mat, and we should review the predicted stress distributions when available. Mats extending below the design groundwater level should be designed to resist hydrostatic uplift forces.

To design the mats using the modulus of subgrade reaction method, we recommend initial moduli of subgrade reaction of 7 kcf and 10 kcf for the residential and mixed-use buildings, respectively; these values are representative of the anticipated settlement under the estimated average mat bearing pressures. After the mat analyses are completed, we should review the computed settlement and bearing pressure profiles to check that the modulus value is appropriate.

Resistance to lateral loads can be mobilized by a combination of passive pressure acting against the vertical faces of the mat and friction along the base of the mat. Passive resistance for the mats for the residential and mixed-use buildings may be calculated using a uniform pressure of 1,900 psf. Frictional resistance should be computed using a base friction coefficient of 0.20, assuming a waterproofing membrane is placed below the mat. These values include a factor of safety of about 1.5.

We should observe mat subgrade prior to placement of reinforcing steel. If weak soil is encountered at the bottom of the mat excavation, it should be overexcavated and replaced with lean concrete or sand-cement slurry as described for spread footings in Section 8.2.1. Mat excavations should be free of standing water, debris, and disturbed materials prior to placing concrete. The bottom and sides of the mat excavations should be wetted following excavation and maintained in a moist condition until concrete is placed. If the foundation soil dries during construction, the foundation will heave when exposed to moisture, which may result in cracking and distress.

Where the bottom of the mat is at least 30 inches above the design groundwater level and moisture on the mat is undesirable, a capillary break and water vapor retarder should be provided beneath the mat as recommended in Section 8.3. Where the mat will extend below the design groundwater level, or is within 30 inches of the design groundwater level, permanent waterproofing will be required beneath the mat. We recommend a waterproofing consultant be retained to determine the most appropriate system for this project and to provide input regarding waterproofing details. Installation of waterproofing should be performed in accordance with the manufacturer's requirements.

8.3 Concrete Floor Slabs

Floor slabs may be supported on grade. The floor slabs and capillary moisture break should be underlain by at least 18 inches of properly compacted select fill or lime-treated native soil, as discussed in Section 8.1.2. If the previously-compacted soil subgrade is disturbed during foundation and utility excavation, the subgrade should be scarified, moisture-conditioned, and rerolled to provide a firm, unyielding surface prior to construction of the slab-on-grade floor. To further reduce the potential for cracking of slab-on-grade floors, we recommend the slab be reinforced with at least No. 4 bars spaced at 18 inches, each way.

Where moisture on the floor slab is undesirable, we recommend installing a capillary moisture break and water vapor retarder beneath the floor to reduce water vapor transmission through the floor slab. Where moisture is not a concern, such as at the parking garage, the floor should be underlain by at least 6 inches of Class 2 aggregate base compacted to at least 95 percent relative compaction. A capillary moisture break consists of at least four inches of clean, free-draining gravel or crushed rock. The vapor retarder should meet the requirements for Class C vapor retarders stated in ASTM E1745-97. The vapor retarder should be placed in accordance with the requirements of ASTM E1643-98. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder. The vapor retarder should be covered with two inches of sand to aid in curing the concrete and to protect the vapor retarder during slab construction. The particle size of the gravel/crushed rock and sand should meet the gradation requirements presented in Table 3.

TABLE 3
Gradation Requirements for Capillary Moisture Break

Sieve Size	Percentage Passing Sieve
Gravel or Crushed Rock	
1 inch	90 – 100
3/4 inch	30 – 100
1/2 inch	5 – 25
3/8 inch	0 – 6
Sand	
No. 4	100
No. 200	0 – 5

The sand overlying the membrane should be moist at the time concrete is placed; however, there should be no free water present in the sand. Excess water trapped in the sand could eventually be transmitted as vapor through the slab. If rain is forecast prior to pouring the slab, the sand should be covered with plastic sheeting to avoid wetting. If the sand becomes wet, concrete should not be placed until the sand has been dried or replaced.

Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab. Therefore, concrete for the floor slab should have a low w/c ratio – less than 0.50. If approved by the project structural engineer, the sand can be eliminated and the concrete can be placed directly over the vapor retarder, provided the w/c ratio of the concrete does not exceed 0.45 and water is not added in the field. If necessary, workability should be increased by adding plasticizers. In addition, the slab should be properly cured. Before the floor covering is placed, the contractor should check that the concrete surface and the moisture emission levels (if emission testing is required) meet the manufacturer's requirements.

If the elevator pits are within 30 inches of the design groundwater level and moisture migration is a concern, the elevator pits should be waterproofed. We recommend a waterproofing consultant be retained to determine the most appropriate system for this project and to provide input regarding waterproofing details. Installation of waterproofing should be performed in accordance with the manufacturer's requirements.

8.4 Below-Grade Walls

The walls of below-grade structures should be designed as restrained walls. The walls should be designed to resist both static lateral earth pressures and lateral pressures caused by earthquakes. We used the procedures outlined in Sitar et al. (2012) to compute the seismic increment using a Design Earthquake (DE) peak ground acceleration (PGA; 40 percent of S_{DS}) equal to 0.60g. The more critical condition of either at-rest pressure or active pressure plus a seismic increment (total pressure) should be checked. At-rest and total equivalent fluid pressure for the DE level of shaking at the site, both for level backfill, are presented in Table 4 for fully drained and undrained conditions.

TABLE 4

Lateral Earth Pressures
Below-Grade Walls
Level Ground Surface

Drainage Condition			
Drained		**Undrained**	
Static Pressure At-rest Pressure (pcf)	**Total Pressure Active plus Seismic Pressure Increment (pcf)**	**Static Pressure At-rest Pressure (pcf)**	**Total Pressure Active plus Seismic Pressure Increment (pcf)**
60	40 + 38	90	80 + 19

If surcharge loads are present above an imaginary 30-degree line (from the horizontal) projected up from the bottom of a retaining wall, a surcharge pressure should be included in the wall design. Where vehicular traffic will pass within 10 feet of retaining walls, traffic loads should be considered in the design of the walls. Traffic loads may be modeled by a uniform pressure of 100 psf applied in the upper 10 feet of the walls.

A backdrain can be provided behind below-grade walls to prevent the buildup of hydrostatic pressure. One acceptable method for backdraining walls is to place a prefabricated drainage panel against the backside of the newly cast wall. The panel should extend down to a perforated PVC collector pipe or an equivalent "flat" pipe (such as AdvanEdge) at the base of the wall or shoring. The PVC pipe should be bedded on and covered by at least 4 inches of Class 2 permeable material (per Caltrans Standard Specifications) or drain rock, and the aggregate material should be surrounded by filter fabric (Mirafi 140NC or equivalent). If a flat pipe surrounded by a filter fabric is used, it is not necessary to surround it with rock. A closed collector pipe should be sloped to drain to a suitable outlet. If water is collected in a sump, a pumping system may be required to carry the water to the storm drain system. We should review the manufacturer's specifications for proposed prefabricated drainage panel material and drain pipe to verify they are appropriate for the intended use.

As an alternative to using prefabricated drainage panel, the wall may be drained using Caltrans Class 2 permeable material (Caltrans Standard Specifications Section 68-1.025) or clean drain rock wrapped in a geotextile filter fabric (Mirafi 140N or equivalent). The gravel drain should be at least 12 inches wide and should extend up the back of the wall to about two feet below the ground surface; the upper two feet should be covered with a clay cap to reduce infiltration of surface water. A four-inch-diameter perforated PVC collector pipe should be placed within the gravel blanket near the base of the wall to drain the water to a suitable discharge.

Below-grade walls should be waterproofed, and water stops should be placed at all construction joints.

Wall backfill should be placed and compacted to the recommendations in Section 8.1.3. If heavy equipment is used, the wall should be appropriately designed to withstand loads exerted by the equipment and/or temporarily braced.

8.5 Temporary Cut Slopes and Shoring

8.5.1 Temporary Cut Slopes

Temporary cut slopes may be made during site grading, foundation installation, basement and elevator pit excavation, and utility installation. The safety of workers and equipment in or near excavations is the responsibility of the contractor. The contractor should be familiar with the most recent Occupational Safety and Health Administration (OSHA) Trench and Excavation Safety standards (29 CFR Part 1926). Excavations that will be deeper than five feet and will be entered by workers should be shored or sloped in accordance with OSHA standards. We judge that temporary cuts in native soil which are less than 10 feet high and inclined no steeper than 1.5:1 (horizontal: vertical) will be stable provided that they are above the groundwater level and not surcharged by equipment or building material. Temporary shoring will be required where temporary slopes are not possible because of space constraints. During construction, we should observe cut slopes to verify the inclinations are appropriate for the conditions encountered. Where excavations encounter groundwater, they should be dewatered.

8.5.2 Temporary Shoring

For design of a cantilevered shoring system, we recommend using an active earth pressure equivalent to a fluid weight of 40 pcf above the ground water table and 80 pcf below the groundwater table, assuming the ground behind the shoring is level. Where excavation depths exceed 12 feet, tiebacks or internal bracing will likely be required. Figure 5 presents the lateral pressures we recommend for design of a tied-backed or internally-braced soldier beam and lagging wall.

If traffic is within a distance equal to the shoring depth, a uniform surcharge load of 100 psf acting on the upper 10 feet should be used in the design. In addition, shoring should be designed for surcharge loads where there will be construction equipment, stockpiled soil, adjacent footings, or other surcharge loads above an imaginary 60-degree line (from the horizontal) projected from the bottom of the shoring. If these conditions exist, we should be consulted on a case-by-case basis to compute the additional pressure increment.

Lateral resistance can be gained by passive pressure acting on the face of the toe of the soldier piles. Passive resistance can be computed using a uniform pressure of 2,500 psf plus an equivalent fluid weight of 80 pcf. These values include a factor of safety of at least 1.5. Passive pressure can be assumed to act over an area of three soldier pile widths assuming the toe of the soldier pile is filled with structural concrete. The upper foot of soil should be ignored when computing passive resistance.

The shoring designer should evaluate the required penetration depth of the soldier piles. The soldier piles should have sufficient axial capacity to support the vertical load component of the tiebacks and the vertical load acting on the piles, if any. To compute the axial capacity of the piles, we recommend using an allowable friction of 550 psf on the perimeter of the piles below the excavation level, which includes a factor of safety of 1.5. Vertical support from end bearing is neglected.

Tiebacks may be used to restrain the shoring. The vertical load from the tiebacks should be accounted for in the design of the vertical elements. Design criteria for tiebacks are presented on Figure 5. As shown, tiebacks should derive their load-carrying capacity from the soil behind an imaginary line sloping upward from a point H/5 feet away from the bottom of the excavation at an angle 60 degrees from horizontal, where H is the wall height in feet. Tiebacks with bar and strand tendons should have a minimum unbonded length of 10 and 15 feet, respectively. The unbonded length should be created by placing an oversized rigid smooth plastic casing (i.e. PVC pipe) over the bars or strands; flexible plastic does not provide adequate bond-break for the unbonded zone. The tiebacks should have a minimum bond length of 15 feet each and be spaced at least six times the grouted diameter of the bonded zone or four feet, whichever is greater. The bottom of the excavation should not extend more than two feet below a row of unsecured tiebacks.

The shoring designer should be responsible for determining the actual length of tieback required. The determination should be based on the designer's familiarity with the installation method to be used. The computed bond length should be confirmed by a performance- and proof-testing program under the observation of an engineer experienced in this type of work.

Tiebacks will generally be installed in medium stiff to stiff clay with variable sand and gravel content and medium dense to dense sand with varying silt, clay, and gravel content. Allowable capacities of the tiebacks will depend upon the drilling method, shaft diameter, grout pressure, and workmanship. Because of the tendency of sand and gravel layers to cave, augers should not be used in these materials. We recommend a smooth-cased method (such as a Klemm rig) be used to install tiebacks in these materials. For estimating purposes, we recommend using the skin friction value for pressure-grouted tiebacks given on Figure 5; this value includes a factor of safety of at least 1.5.

The anticipated deflections of the shoring system should be estimated to check if they are acceptable. The shoring system should be sufficiently rigid to prevent detrimental movement of the temporary shoring and possible damage to existing improvements, including underground utilities and structures, adjacent to the site. In our experience, the deflection of a properly designed shoring system should generally be held to one inch or less. The shoring and tieback system should be designed so that it does not conflict with nor damage existing improvements outside the site boundaries.

The shoring system should be installed by an experienced shoring specialty contractor. The contractor should be familiar with applicable local, state, and federal regulations for temporary shoring, including the current OSHA Excavation and Trench Safety Standards. The shoring designer should be responsible for shoring design. We should review the final shoring plans to check that they are consistent with the recommendations presented in this report. In addition, we recommend a representative from our office observe the installation of the temporary shoring system.

The first two production tiebacks and two percent of the remaining tiebacks should be performance-tested to 1.25 times the design load. The remaining tiebacks should be confirmed by a proof-test to 1.25 times the design load. The performance tests will be used to determine the load carrying capacity of the tiebacks and the residual movement. The performance-tested tiebacks should be checked 24 hours after initial lock off to confirm stress relaxation has not occurred. If any tiebacks fail to meet the proof-testing requirements, additional tiebacks should be added to compensate for the deficiency, as determined by the shoring designer.

The movement of each tieback should be monitored with a free-standing, tripod-mounted dial gauge during performance and proof testing. The performance test is used to verify the capacity and the load-deformation behavior of the tiebacks. It is also used to separate and identify the causes of tieback movement, and to check that the designed unbonded length has been established. In the performance test, the load is applied to the tieback in several cycles of incremental loading and unloading. During the test, the tieback load and movement are measured. The maximum test load should be held for a minimum of 10 minutes, with readings taken at 0, 1, 3, 6, and 10 minutes. If the difference between the 1- and 10-minute readings is less than 0.04 inch during the loading, the test is discontinued. If the difference is more than 0.04 inch, the holding period is extended by 50 minutes to 60 minutes, and the movements should be recorded at 15, 20, 25, 30, 45, and 60 minutes.

A proof test is a simple test used to measure the total movement of the tieback during one cycle of incremental loading. The maximum test load should be held for a minimum of 10 minutes, with readings taken at 0, 1, 2, 3, 6, and 10 minutes. If the difference between the 1- and 10-minute readings is less than 0.04 inch, the test is discontinued. If the difference is more than 0.04 inch, the holding period is extended by 50 minutes to 60 minutes, and the movements should be recorded at 15, 20, 25, 30, 45, and 60 minutes.

We should evaluate the tieback test results to determine whether the tiebacks are acceptable. A performance- or proof-tested tieback with a ten-minute hold is acceptable if the tieback carries the maximum test load with less than 0.04 inch movement between one and 10 minutes, and total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length. A performance- or proof-tested tieback with a 60-minute hold is acceptable if the tieback carries the maximum test load with less than 0.08 inch movement between six and 60 minutes, and total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length. Tiebacks that fail to meet the first criterion will be assigned a reduced capacity.

If the total movement of the tiebacks at the maximum test load does not exceed 80 percent of the theoretical elastic elongation of the unbonded length or the tieback fails the load test, the contractor should replace the tiebacks.

8.6 Pavement Design

8.6.1 Asphalt Concrete Pavement

The State of California resistance value (R-value) method for flexible pavement design was used to develop recommendations for asphalt concrete pavement sections. We anticipate the final soil subgrade in areas to receive asphalt concrete pavement will generally consist of clay. The R-value test performed on clay collected from the boring B-13 indicates the material has an R-value of 1. We used an R-value of 5 in our calculations, which is the minimum R-value to use in design pavement using this method.

For our calculations, we used traffic indices (TIs) of 5.0, 6.0, and 7.0. Our pavement section recommendations are presented on Table 5. Recommendations for subgrade preparation beneath pavement sections are provided in Section 8.1.2. AB should be compacted to at least 95 percent relative compaction.

TABLE 5
Asphaltic Concrete Pavement Section Design

TI	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
5.0	3.0	10.0
6.0	3.5	13.0
7.0	4.0	15.5

8.6.2 Portland Cement Concrete Pavement

Concrete pavement design is based on a maximum single-axle load of 18,000 pounds and a maximum tandem axle of 32,000 pounds (corresponds to a garbage truck). The recommended rigid pavement section for these axle loads is seven inches of Portland cement concrete over six inches of Caltrans Class 2 AB. If only passenger cars or light trucks will use the pavement, the recommended minimum pavement section is five inches of Portland cement concrete over six inches of Class 2 AB. AB should conform to the current State of California Department of Transportation (Caltrans) Standard Specifications. Recommendations for subgrade preparation and AB compaction for Portland cement concrete pavement are the same as those for asphalt concrete pavement.

The modulus of rupture of the concrete should be at least 500 psi at 28 days. Contraction joints should be constructed at 15-foot spacing. Where the outer edge of a concrete pavement meets asphalt pavement, the concrete slab should be thickened by 50 percent at a taper not to exceed a slope of 1 in 10.

8.7 Concrete Flatwork

If it is desirable to reduce the potential for differential movement and cracking, exterior concrete flatwork should be underlain by at least 12 inches of select fill, lime-treated soil, or Caltrans Class 2 AB, which should extend at least two feet beyond the slab edges. Even with 12 inches of select fill, lime-treated soil, or AB, exterior slabs may experience some cracking due to shrinking and swelling of the underlying expansive soil. Thickening the slabs and adding additional reinforcement will control this cracking to some degree. In addition, where slabs provide access to the building, it would be prudent to dowel the slab to the foundation at the entrance to permit rotation of the slab as the exterior ground shrinks and swells and to prevent a vertical offset at the entries. Recommendations for subgrade preparation beneath concrete flatwork are provided in Section 8.1.2.

8.8 Drainage

Positive surface drainage should be provided around the building to direct surface water away from the foundations. To reduce the potential for water ponding adjacent to the structure, we recommend the ground surface within a horizontal distance of ten feet from the building slope down away from the building with a surface gradient of at least five percent in unpaved areas and two percent in paved areas. In addition, roof downspouts should be discharged into controlled drainage facilities to keep the water away from the foundations. Because the subgrade soil consists predominantly of clay, it will have a relatively low permeability. If infiltration basins, bioswales, or permeable pavement are planned, they should be lined with an impermeable membrane and drains should be provided that direct the water to an appropriate outlet. Unlined infiltration basins or bioswales should not be placed within five feet of the foundations.

8.9 Irrigation and Landscaping Limitations

The use of water-intensive landscaping around the perimeter of the buildings should be avoided to reduce the amount of water introduced to the expansive clay subgrade. In addition, irrigation of landscaping around the buildings should be limited to drip or bubbler-type systems. The purpose of these recommendations is to avoid large differential moisture changes adjacent to the foundations, which has been known to cause large differential settlement over short horizontal distances in expansive soil, resulting in cracking of slabs and architectural damage.

Moderately to very highly expansive native clay is expected to be present at or near the subgrade level. For this condition, prior experience and industry literature indicate some species of high water-demand[8] trees can induce ground surface settlement by drawing water from the expansive soil and causing it to shrink. Where these types of trees are planted adjacent to structures, the ground-surface settlement may result in damage to the structures. This problem usually occurs ten or more years after project completion as the trees reach mature height. To reduce the risk of tree-induced, ground-surface settlement, we recommend trees of the following genera not be planted within a horizontal distance from the building equal to the

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[8] "Water-demand" refers to the ability of the tree to withdraw large amounts of water from the soil subgrade, rather than soil suction exerted by the root system.

mature height of the tree: *Eucalyptus, Populus, Quercus, Crataegus, Salix, Sorbus (simple-leafed), Ulmus, Cupressus, Chamaecyparis, and Cupressocyparis*. Because this is a limited list and does not include all genera that may induce ground-surface settlement, the project landscape architect should use judgment in limiting other types or trees with similar properties in the vicinity of the buildings.

8.10 2013 California Building Code Mapped Values

Although some potentially liquefiable soil layers were encountered in the borings and CPTs at the site, we judge these layers are thin, isolated, and discontinuous. Therefore, we conclude that Site Class D as defined in 2013 CBC is appropriate for the site. For seismic design in accordance with the provisions of 2013 California Building Code (CBC) we recommend the following:

- Risk Targeted Maximum Considered Earthquake (MCE_R) S_S and S_1 of 2.288g and 0.872g, respectively.
- Site Class D
- Site Coefficients, F_a and F_v of 1.0 and 1.5.
- MCE_R spectral response acceleration parameters at short period, S_{MS}, and at one-second period, S_{M1}, of 2.288g and 1.308g, respectively.
- Design Earthquake (DE) spectral response acceleration parameters at short period, S_{DS}, and at one-second period, S_{D1}, of 1.525g and 0.872g, respectively.

9.0 ADDITIONAL RECOMMENDATIONS – SERVICES DURING DESIGN AND CONSTRUCTION

During final design we should be retained to consult with the design team as geotechnical questions arise. Prior to construction, we should review the geotechnical aspects of the project plans and specifications to check their conformance with the intent of our recommendations. During construction, it is imperative that we observe subgrade preparation, compaction of backfill, shoring installation and testing, foundation excavations, and mat subgrade as the geotechnical engineer of record. These observations will allow us to compare the actual with the anticipated soil conditions and to check that the contractors' work conforms with the geotechnical aspects of the plans and specifications. The recommendations contained in this report assume that we will be on-site during construction to make modification to them as needed.

10.0 LIMITATIONS

The conclusions and recommendations presented in this report result from limited engineering studies based on our interpretation of the geotechnical conditions existing at the site at the time of this investigation. Actual subsurface conditions may vary. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that described in this report, Langan Treadwell Rollo should be notified to make supplemental recommendations, as necessary.

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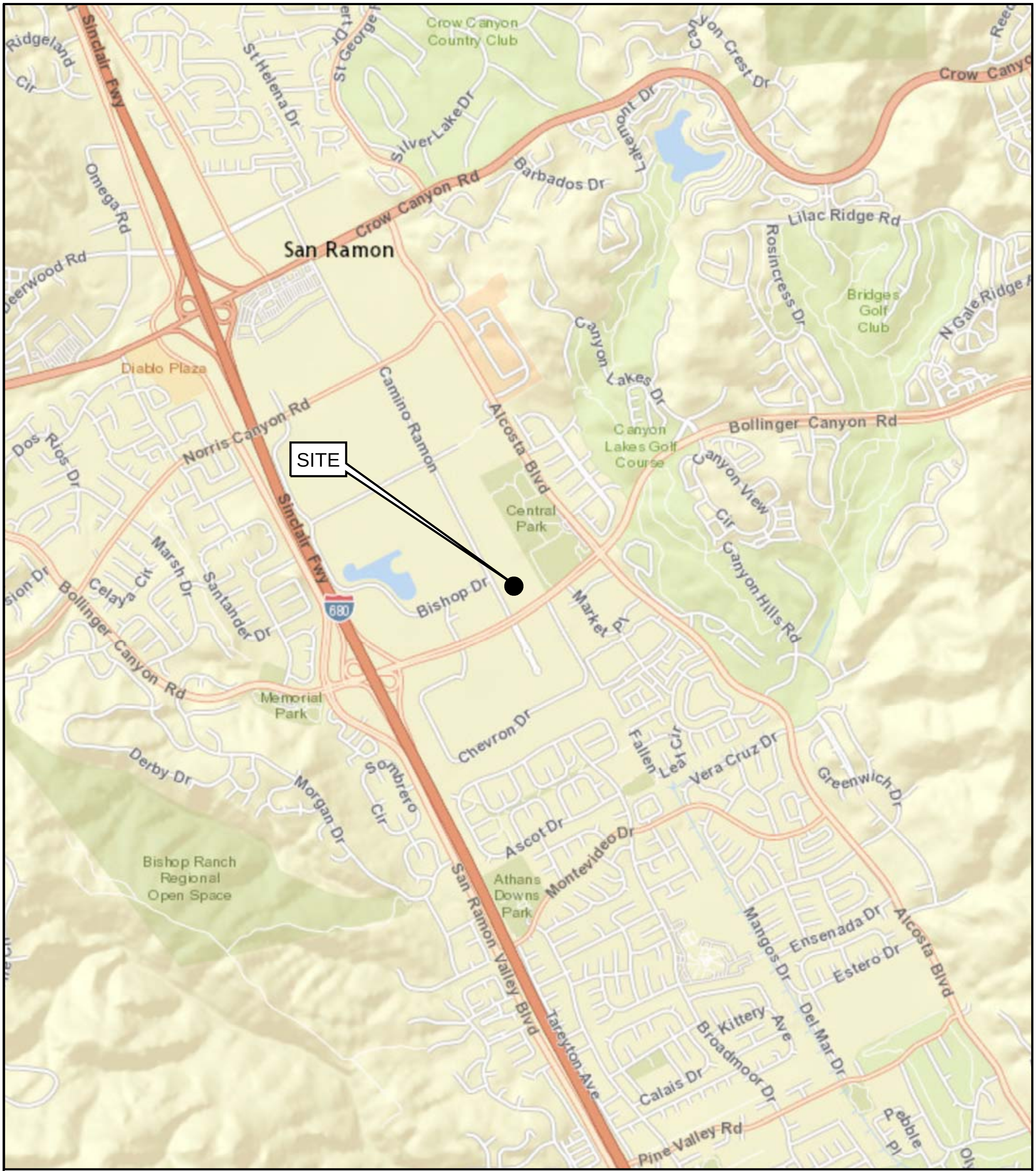
FIGURES



NOTES:

World street basemap is provided through Langan's Esri ArcGIS software licensing and ArcGIS online.
Credits: Sources: Esri, DeLorme, NAVTEQ, USGS, Intermap, iPC, NRCAN.

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BISHOP RANCH - BR3A
San Ramon, California

SITE LOCATION MAP

LANGAN TREADWELL ROLLO

Date 05/13/16	Project No. 750633001	Figure 1

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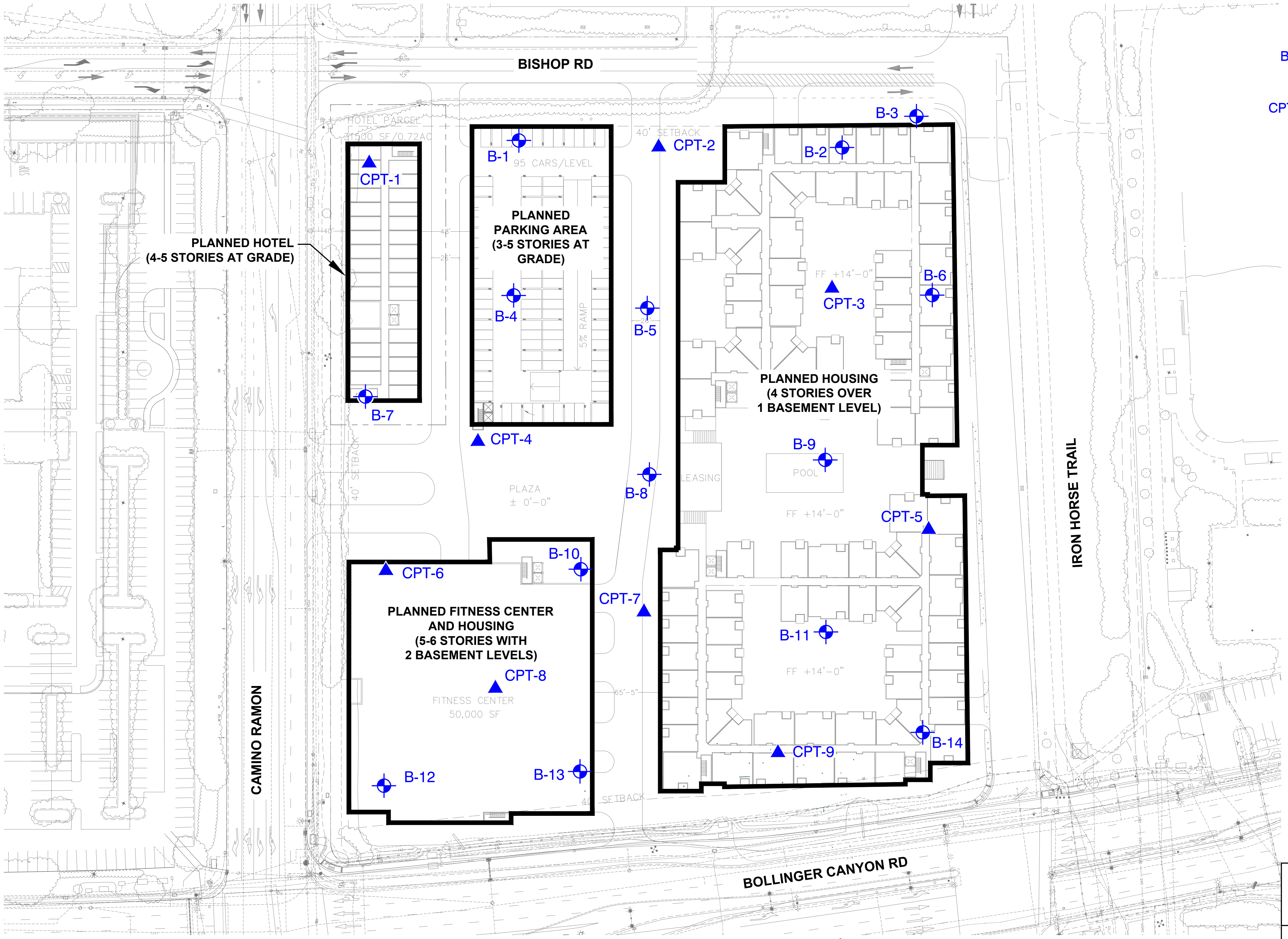

EXPLANATION

B-1 ⊕	Approximate location of boring by Langan Treadwell Rollo, January and April 2016
CPT-1 ▲	Approximate location of cone penetration test by Langan Treadwell Rollo, April 2016

0 100 Feet

Approximate scale

REFERENCE: Base Map by Studio T Square Architecture Planning Urban Design, Sheet Title Level 2-6 Plan, dated 09/18/2015.

BISHOP RANCH - BR3A
San Ramon,California

SITE PLAN

Date 07/12/16	Project No. 750633001	Figure 2

LANGAN TREADWELL ROLLO

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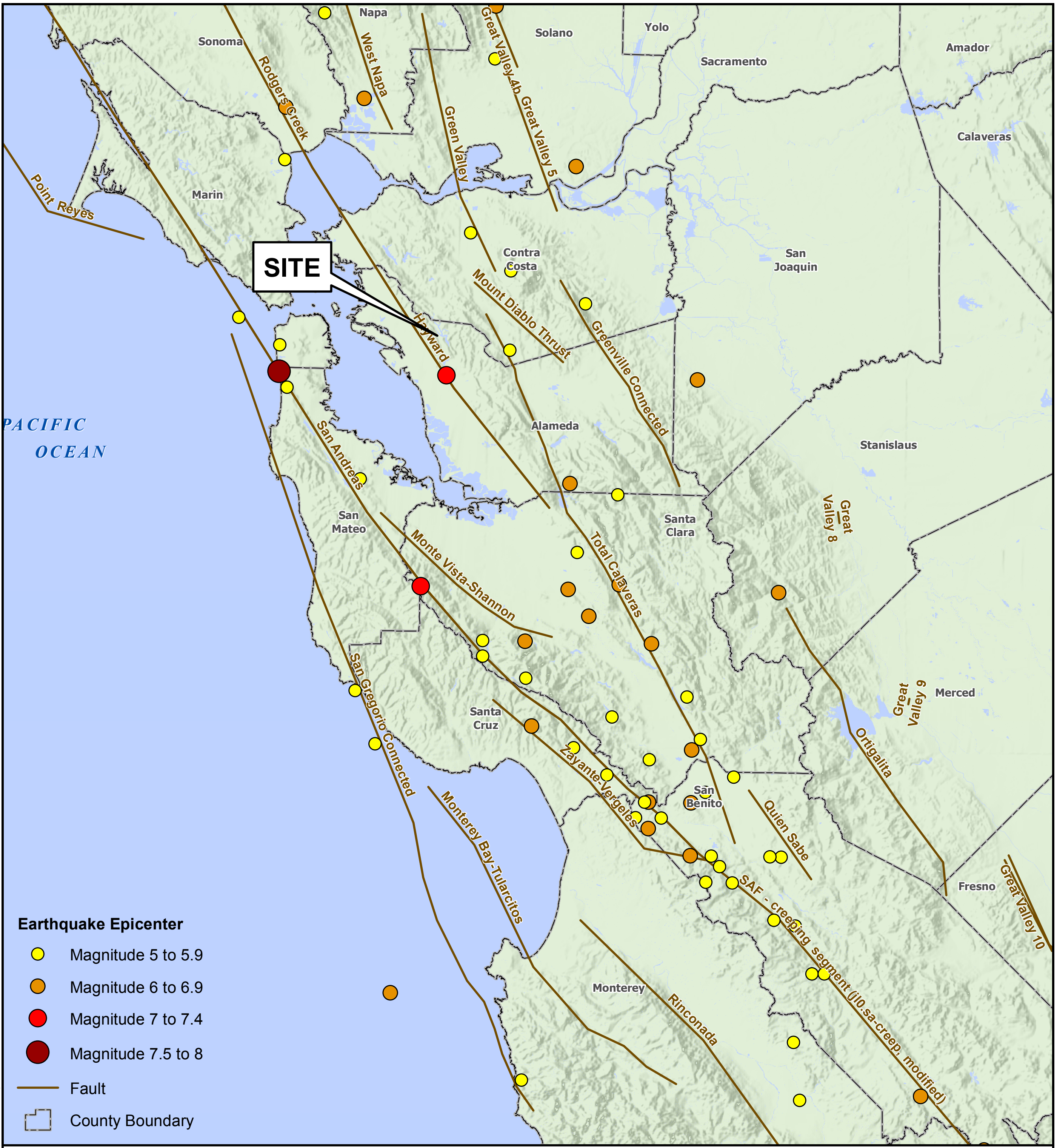
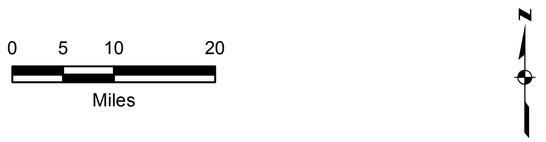


Notes:

1. Quaternary fault data displayed are based on a generalized version of U.S Geological Survey (USGS) Quaternary Fault and fold database, 2010. For cartographic purposes only.
2. The Earthquake Epicenter (Magnitude) data is provided by the USGS and is current through 08/26/2014.
3. Basemap hillshade and County boundaries provided by USGS and California Department of Transportation.
4. Map displayed in California State Coordinate System, California (Teale) Albers, North American Datum of 1983 (NAD83), Meters.



BISHOP RANCH - BR3A
San Ramon, California

LANGAN TREADWELL ROLLO

MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS IN THE SAN FRANCISCO BAY AREA

Date 5/13/2016 | Project No. 750633001 | Figure 3

\\langan.com\data\OAK\data0\750633001\ArcGIS\ArcMap_Documents\Fault Map.mxd User: agekas

I Not felt by people, except under especially favorable circumstances. However, dizziness or nausea may be experienced.

Sometimes birds and animals are uneasy or disturbed. Trees, structures, liquids, bodies of water may sway gently, and doors may swing very slowly.

II Felt indoors by a few people, especially on upper floors of multi-story buildings, and by sensitive or nervous persons.

As in Grade I, birds and animals are disturbed, and trees, structures, liquids and bodies of water may sway. Hanging objects swing, especially if they are delicately suspended.

III Felt indoors by several people, usually as a rapid vibration that may not be recognized as an earthquake at first. Vibration is similar to that of a light, or lightly loaded trucks, or heavy trucks some distance away. Duration may be estimated in some cases.

Movements may be appreciable on upper levels of tall structures. Standing motor cars may rock slightly.

IV Felt indoors by many, outdoors by a few. Awakens a few individuals, particularly light sleepers, but frightens no one except those apprehensive from previous experience. Vibration like that due to passing of heavy, or heavily loaded trucks. Sensation like a heavy body striking building, or the falling of heavy objects inside.

Dishes, windows and doors rattle; glassware and crockery clink and clash. Walls and house frames creak, especially if intensity is in the upper range of this grade. Hanging objects often swing. Liquids in open vessels are disturbed slightly. Stationary automobiles rock noticeably.

V Felt indoors by practically everyone, outdoors by most people. Direction can often be estimated by those outdoors. Awakens many, or most sleepers. Frightens a few people, with slight excitement; some persons run outdoors.

Buildings tremble throughout. Dishes and glassware break to some extent. Windows crack in some cases, but not generally. Vases and small or unstable objects overturn in many instances, and a few fall. Hanging objects and doors swing generally or considerably. Pictures knock against walls, or swing out of place. Doors and shutters open or close abruptly. Pendulum clocks stop, or run fast or slow. Small objects move, and furnishings may shift to a slight extent. Small amounts of liquids spill from well-filled open containers. Trees and bushes shake slightly.

VI Felt by everyone, indoors and outdoors. Awakens all sleepers. Frightens many people; general excitement, and some persons run outdoors.

Persons move unsteadily. Trees and bushes shake slightly to moderately. Liquids are set in strong motion. Small bells in churches and schools ring. Poorly built buildings may be damaged. Plaster falls in small amounts. Other plaster cracks somewhat. Many dishes and glasses, and a few windows break. Knickknacks, books and pictures fall. Furniture overturns in many instances. Heavy furnishings move.

VII Frightens everyone. General alarm, and everyone runs outdoors.

People find it difficult to stand. Persons driving cars notice shaking. Trees and bushes shake moderately to strongly. Waves form on ponds, lakes and streams. Water is muddied. Gravel or sand stream banks cave in. Large church bells ring. Suspended objects quiver. Damage is negligible in buildings of good design and construction; slight to moderate in well-built ordinary buildings; considerable in poorly built or badly designed buildings, adobe houses, old walls (especially where laid up without mortar), spires, etc. Plaster and some stucco fall. Many windows and some furniture break. Loosened brickwork and tiles shake down. Weak chimneys break at the roofline. Cornices fall from towers and high buildings. Bricks and stones are dislodged. Heavy furniture overturns. Concrete irrigation ditches are considerably damaged.

VIII General fright, and alarm approaches panic.

Persons driving cars are disturbed. Trees shake strongly, and branches and trunks break off (especially palm trees). Sand and mud erupts in small amounts. Flow of springs and wells is temporarily and sometimes permanently changed. Dry wells renew flow. Temperatures of spring and well waters varies. Damage slight in brick structures built especially to withstand earthquakes; considerable in ordinary substantial buildings, with some partial collapse; heavy in some wooden houses, with some tumbling down. Panel walls break away in frame structures. Decayed pilings break off. Walls fall. Solid stone walls crack and break seriously. Wet grounds and steep slopes crack to some extent. Chimneys, columns, monuments and factory stacks and towers twist and fall. Very heavy furniture moves conspicuously or overturns.

IX Panic is general.

Ground cracks conspicuously. Damage is considerable in masonry structures built especially to withstand earthquakes; great in other masonry buildings - some collapse in large part. Some wood frame houses built especially to withstand earthquakes are thrown out of plumb, others are shifted wholly off foundations. Reservoirs are seriously damaged and underground pipes sometimes break.

X Panic is general.

Ground, especially when loose and wet, cracks up to widths of several inches; fissures up to a yard in width run parallel to canal and stream banks. Landsliding is considerable from river banks and steep coasts. Sand and mud shifts horizontally on beaches and flat land. Water level changes in wells. Water is thrown on banks of canals, lakes, rivers, etc. Dams, dikes, embankments are seriously damaged. Well-built wooden structures and bridges are severely damaged, and some collapse. Dangerous cracks develop in excellent brick walls. Most masonry and frame structures, and their foundations are destroyed. Railroad rails bend slightly. Pipe lines buried in earth tear apart or are crushed endwise. Open cracks and broad wavy folds open in cement pavements and asphalt road surfaces.

XI Panic is general.

Disturbances in ground are many and widespread, varying with the ground material. Broad fissures, earth slumps, and land slips develop in soft, wet ground. Water charged with sand and mud is ejected in large amounts. Sea waves of significant magnitude may develop. Damage is severe to wood frame structures, especially near shock centers, great to dams, dikes and embankments, even at long distances. Few if any masonry structures remain standing. Supporting piers or pillars of large, well-built bridges are wrecked. Wooden bridges that "give" are less affected. Railroad rails bend greatly and some thrust endwise. Pipe lines buried in earth are put completely out of service.

XII Panic is general.

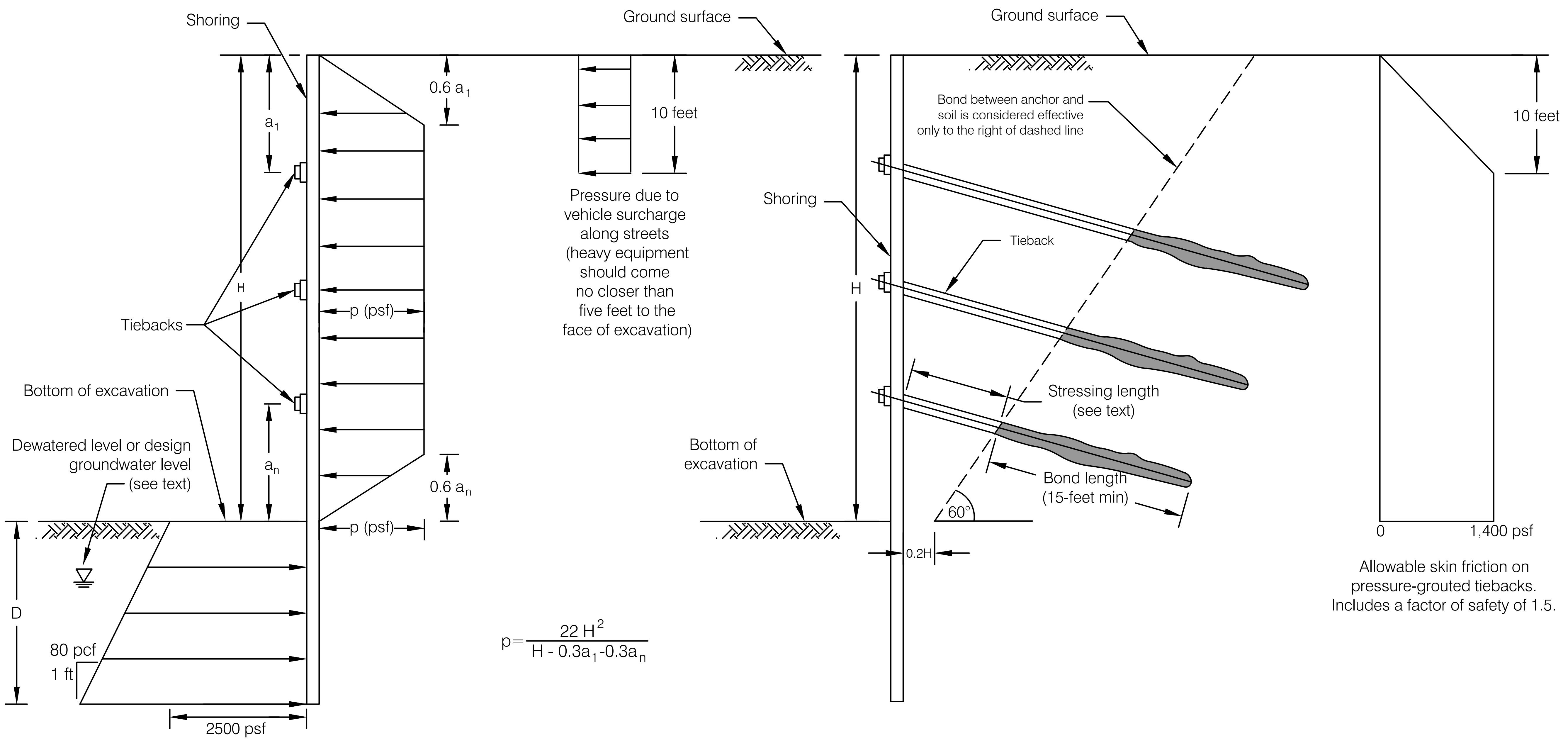
Damage is total, and practically all works of construction are damaged greatly or destroyed. Disturbances in the ground are great and varied, and numerous shearing cracks develop. Landslides, rock falls, and slumps in river banks are numerous and extensive. Large rock masses are wrenched loose and torn off. Fault slips develop in firm rock, and horizontal and vertical offset displacements are notable. Water channels, both surface and underground, are disturbed and modified greatly. Lakes are dammed, new waterfalls are produced, rivers are deflected, etc. Surface waves are seen on ground surfaces. Lines of sight and level are distorted. Objects are thrown upward into the air.

BISHOP RANCH - BR3A
San Ramon, California

MODIFIED MERCALLI INTENSITY SCALE

LANGAN TREADWELL ROLLO

Date 05/13/16 | Project No. 750633001 | Figure 4



$$p=\frac{22\ H^2}{H-0.3a_1-0.3a_n}$$

NOT TO SCALE

Notes:

1. The above pressure diagram assumes that the shoring consists of pervious-soldier-pile-and-lagging system.
2. Passive pressure includes a factor of safety of about 1.5.
3. For soldier piles spaced at more than three times the soldier pile diameter, the passive pressure should be assumed to act over three diameters.
4. psf = pounds per square foot; pcf = pounds per cubic foot.
5. Surcharge pressure, from construction equipment, if any, should be added to the above shoring pressure.
6. The recommended pressures do not include surcharges from adjacent buildings. Surcharge pressure from adjacent buildings should be added to the above shoring pressures.
7. D, H, and a in feet.

BISHOP RANCH - BR3A
San Ramon, California

**DESIGN PARAMETERS FOR
TIED BACK SOLDIER-PILE-AND-LAGGING
TEMPORARY SHORING SYSTEM**

Date 07/26/16	Project No. 750633001	Figure 5

LANGAN TREADWELL ROLLO

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APPENDIX A

LOGS OF BORINGS

PROJECT: **BISHOP RANCH - BR3A**
San Ramon, California

Log of Boring B-1

PAGE 1 OF 2

Boring location: See Figure 2
Date started: 4/27/16
Date finished: 4/27/16
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Downhole Wireline
Samplers: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)

Logged by: K. Watkins
Drilled By: Exploration Geo Services

Ground Surface Elevation: 446 feet^2

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ 6"	SPT N-Value^1	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
1						CLAY with SAND (CL) dark brown, stiff, moist, fine- to coarse-grained sand						
2												
3	S&H	■	4 11 11	13	CL						19.2	104
4						LL = 47, PI = 28, see Figure C-8						
5						very stiff						
6	S&H	■	10 14 21	21								
7						CLAY with SAND (CL) olive-gray, stiff, moist, fine- to coarse-grained sand						
8	S&H	■	8 9 11	12								
9												
10					CL							
11	S&H	■	7 10 13	14								
12						fine-grained sand						
13	S&H	■	5 6 10	10			PP		1,500			
14												
15					SM	SILTY SAND (SM) ∇ olive-gray, medium dense, wet, fine- to medium-grained (4/27/2016, 7:45 a.m.)						
16	SPT	◪	8 8 7	15		LL = 21, PI = 3, see Figure C-8				30.3	18.7	
17						CLAY (CL) olive-gray, stiff, wet, trace sand						
18	SPT	◪	4 6 6	12								
19												
20			100 psi									
21	ST	▌	300 psi			medium stiff TxUU Test, see Figure C-4 Consolidation Test, see Figure C-1	TxUU	2,000	910		18.9 30.4	112 92
22							PP		750			
23					CL							
24												
25						gray, stiff						
26	S&H	■	7 10 11	13								
27												
28												
29												
30												

LANGAN TREADWELL ROLLO

Project No.: 750633001
Figure: A-1a

PROJECT: **BISHOP RANCH - BR3A**
San Ramon, California

Log of Boring B-1

PAGE 2 OF 2

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	S&H	■	6 8 14	13		CLAY (CL) (continued) gray-brown	PP		1,750			
32												
33					CL							
34												
35												
36	S&H	■	4 8 10	11		olive-gray						
37												
38												
39												
40												
41												
42												
43												
44												
45												
46												
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 36.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 15 feet below ground surface at time of drilling.
PP = pocket penetrometer.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.

[2] Elevations based on NGVD29 datum and site plan titled "City of San Ramon City Center, BR3 Site, Existing Utility and Easements," by RJA Engineers, 9 January 2014.

LANGAN TREADWELL ROLLO

Project No.: 750633001

Figure: A-1b

PROJECT: **BISHOP RANCH - BR3A**
San Ramon, California

Log of Boring B-2

PAGE 1 OF 2

Boring location: See Figure 2
Date started: 1/19/16
Date finished: 1/19/16
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Downhole Wireline
Samplers: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)

Logged by: K. Watkins
Drilled By: Exploration Geo Services

LABORATORY TEST DATA

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 445.8 feet[2]	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
					GP	GRAVEL (GP) gray, moist, coarse, angular FILL						
1												
2						SANDY CLAY with GRAVEL (CL) dark brown and light brown, very stiff, moist, fine- to medium-grained sand, fine subangular gravel Corrosion Test, see Appendix D						
3	S&H		13 15 14	17	CL							
4												
5												
6	S&H		9 8 8	10		CLAY (CH) dark brown, stiff, moist						
7					CH							
8	S&H		8 11 18	17		SANDY CLAY (CL) gray-brown, very stiff, moist, fine-grained to medium-grained sand						
9					CL							
10												
11	S&H		7 14 13	16		CLAY with SAND (CH) olive-gray with light brown and dark brown mottling, very stiff, moist, fine-grained sand	PP		2,500			
12												
13												
14												
15						olive-gray, stiff, wet						
16	S&H		5 9 15	14								
17					CH							
18												
19												
20			300 psi									
21	ST											
22			400 psi			very stiff	PP		2,400			
23						∇ (1/19/2016, 11:16 a.m.)						
24						CLAY (CL) gray-brown to dark brown, very stiff, wet, trace fine- to coarse-grained sand						
25												
26	S&H		6 11 18	17								
27					CL		PP		2,100			
28												
29												
30												

LANGAN TREADWELL ROLLO

Project No.: 750633001
Figure: A-2a

PROJECT: **BISHOP RANCH - BR3A**
San Ramon, California

Log of Boring B-2

PAGE 2 OF 2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
31	S&H	■	8 11 17	17		CLAY (CL) (continued)	PP		2,400			
32					CL							
33												
34												
35						CLAY with SAND (CL) olive-gray, very stiff, wet, fine- to medium-grained sand						
36	S&H	■	6 10 16	16								
37					CL							
38												
39												
40						CLAY with SAND (CH) olive-gray with brown mottling, very stiff, wet, fine-grained sand						
41	S&H	■	12 16 28	26			PP		3,250			
42												
43												
44												
45					CH							
46	S&H	■	13 16 20	22			PP		3,000			
47												
48												
49	S&H	■	13 18 31	29								
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 50 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 23 feet below ground surface during drilling.
PP = pocket penetrometer.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.

[2] Elevations based on NGVD29 datum and site plan titled "City of San Ramon City Center, BR3 Site, Existing Utility and Easements," by RJA Engineers, 9 January 2014.

LANGAN TREADWELL ROLLO

Project No.: 750633001

Figure: A-2b

PROJECT: **BISHOP RANCH - BR3A**
San Ramon, California

Log of Boring B-3

PAGE 1 OF 2

Boring location: See Figure 2
Date started: 1/19/16
Date finished: 1/19/16
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Downhole Wireline
Samplers: Sprague & Henwood (S&H)

Logged by: K. Watkins
Drilled By: Exploration Geo Services

LABORATORY TEST DATA

Ground Surface Elevation: 445.7 feet[2]

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						CLAY with SAND and GRAVEL (CL) dark brown, stiff, moist						
1												
2	GRAB	☒			CL							
3												
4												
5	S&H	■	7 10 12	13	CH	CLAY with SAND (CH) dark brown, stiff, moist, fine-grained sand, fine subangular gravel						
6												
7						CLAY with SAND (CL) dark brown, stiff to very stiff, moist, fine to- medium-grained sand						
8	S&H	■	7 12 13	15	CL							
9												
10	S&H	■	8 12 13	15		SANDY CLAY (CL) light brown with dark brown mottling, stiff to very stiff, moist, fine- to medium-grained sand						
11												
12					CL							
13												
14						CLAY with SAND (CL)						
15	S&H	■	6 7 9	10		▽ light brown, stiff, wet, fine-grained sand (1/19/2016, 8:18 a.m.)						
16							PP		1,250			
17												
18												
19												
20	S&H	■	4 5 7	7	CL	medium stiff, trace coarse-grained sand						
21							PP		750			
22												
23												
24												
25	S&H	■	7 11 12	14		olive-gray, stiff						
26												
27						CLAY (CL) olive-gray to light brown, stiff, wet, trace fine- to medium-grained sand	PP		1,600			
28					CL							
29												
30												

LANGAN TREADWELL ROLLO

Project No.: 750633001
Figure: A-3a

PROJECT: **BISHOP RANCH - BR3A**
San Ramon, California

Log of Boring B-3

PAGE 2 OF 2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
	S&H	▨	8 9 14	14		CLAY (CL) (continued) olive-gray						
31							PP		1,800			
32												
33												
34					CL							
35	S&H	▨	8 11 22	20		very stiff						
36							PP		2,100			
37												
38												
39	S&H	▨	9 15 23	23	CL	SANDY CLAY (CL) yellow-brown with dark brown mottling, very stiff, wet, fine- to medium-grained sand	PP		2,500			
40												
41												
42												
43												
44												
45												
46												
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 40 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 15 feet below ground surface during drilling.
PP = pocket penetrometer.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.
[2] Elevations based on NGVD29 datum and site plan titled "City of San Ramon City Center, BR3 Site, Existing Utility and Easements," by RJA Engineers, 9 January 2014.

LANGAN TREADWELL ROLLO

Project No.: 750633001

Figure: A-3b

PROJECT: **BISHOP RANCH - BR3A**
San Ramon, California

Log of Boring B-4

PAGE 1 OF 2

Boring location: See Figure 2
Date started: 4/29/16
Date finished: 4/29/16
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Downhole Wireline
Samplers: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Logged by: K. Watkins
Drilled By: Exploration Geo Services

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value¹	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 446 feet²	LABORATORY TEST DATA Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
1						CLAY with SAND (CH) dark brown with olive-gray mottling, very stiff, moist, fine-grained sand, trace organic material						
2												
3	S&H	▨	9 11 16	16	CH	LL = 50, PI = 30, see Figure C-8						
4												
5												
6	S&H	▨	5 8 10	11		dark brown, stiff, no organics						
7						SANDY CLAY (CL) olive-gray, stiff, moist, fine-grained sand						
8	S&H	▨	6 9 10	11	CL	TxUU Test, see Figure C-5	TxUU	900	1,340		15.3	107
9												
10						CLAY with SAND (CL) olive-gray, very stiff, moist, fine-grained sand						
11	S&H	▨	6 11 16	16	CL							
12												
13	S&H	▨	5 8 11	11		stiff, fine- to medium-grained sand						
14					SC	CLAYEY SAND (SC) olive-gray, medium dense, moist, fine-grained						
15												
16	SPT	◩	3 4 4	8		CLAY with SAND (CL) olive-gray, medium stiff to stiff, moist, fine-grained sand						
17												
18												
19												
20						stiff						
21	S&H	▨	4 7 8	9			PP		1,000		22.9	101
22					CL							
23												
24												
25						gray						
26	S&H	▨	6 8 10	11								
27												
28												
29												
30					CL							

LANGAN TREADWELL ROLLO

Project No.: 750633001
Figure: A-4a

PROJECT: **BISHOP RANCH - BR3A**
San Ramon, California

Log of Boring B-4

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
31	S&H	▨	7 12 18	18		CLAY (CL) gray, very stiff, moist, trace fine-grained sand						
32												
33					CL							
34												
35												
36	S&H	▨	7 10 15	15		olive with gray-brown mottling, stiff to very stiff	PP		2,250			
37												
38												
39												
40												
41												
42												
43												
44												
45												
46												
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 36.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered at time of drilling.
PP = pocket penetrometer.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.

[2] Elevations based on NGVD29 datum and site plan titled "City of San Ramon City Center, BR3 Site, Existing Utility and Easements," by RJA Engineers, 9 January 2014.

LANGAN TREADWELL ROLLO

Project No.: 750633001
Figure: A-4b

PROJECT: **BISHOP RANCH - BR3A**
San Ramon, California

Log of Boring B-5

Boring location: See Figure 2
Date started: 4/26/16
Date finished: 4/26/16
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Downhole Wireline
Samplers: Sprague & Henwood (S&H)

Logged by: K. Watkins
Drilled By: Exploration Geo Services

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ 6"	SPT N-Value^1	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 445.5 feet^2	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
1 2	S&H	■	9 11 12	14	SC	CLAYEY SAND with GRAVEL (SC) yellow-brown, medium dense, moist, fine- to coarse-grained, fine to coarse gravel FILL						
3 4	S&H	■	8 11 15	16		CLAY with SAND (CH) dark brown, very stiff, moist, fine- to coarse-grained sand						
5 6 7	S&H	■	11 13 20	20	CH	fine-grained sand						
8 9	S&H	■	5 9 15	14	CH	CLAY (CH) gray, stiff, moist, organic inclusions						
10 11 12 13	S&H	■	13 20 25	27	CL	SANDY CLAY (CL) gray and olive-gray, very stiff, moist, fine-grained sand	PP		3,250			
14 15 16	S&H	■	6 8 14	13	SM ∇	SILTY SAND (SM) olive-gray, medium dense, wet, fine-grained, nonplastic (4/25/2016, 8:32 a.m.)				36.6	23.1	
17 18 19					CL	CLAY with SAND (CL) olive-gray, stiff, wet, fine-grained sand						
20 21 22 23	S&H	■	5 8 9	10	CL	CLAY (CL) olive-gray, stiff, wet, trace fine-grained sand	PP		1,500			
24 25 26	S&H	■	6 10 15	15	CL	CLAY with SAND (CL) olive-gray, stiff to very stiff, fine-grained sand	PP		1,750			
27 28					SP-SC	SAND with CLAY (SP-SC) olive-gray, medium dense, wet, fine-grained sand						
29 30					CL	CLAY with SAND (CL) olive-gray, very stiff, wet, fine-grained sand						

LANGAN TREADWELL ROLLO

Project No.: 750633001
Figure: A-5a

PROJECT: **BISHOP RANCH - BR3A**
San Ramon, California

Log of Boring B-5

PAGE 2 OF 2

DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]			Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	S&H	▨	8 11 17	17		CLAY with SAND (CL) (continued)						
32												
33												
34												
35												
36	S&H	▨	10 17 22	23	CL							
37												
38												
39												
40												
41	S&H	▨	10 17 21	23								
42												
43												
44												
45												
46												
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 41.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 15 feet below ground surface during drilling.
PP = pocket penetrometer.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.

[2] Elevations based on NGVD29 datum and site plan titled "City of San Ramon City Center, BR3 Site, Existing Utility and Easements," by RJA Engineers, 9 January 2014.

LANGAN TREADWELL ROLLO

Project No.: 750633001

Figure: A-5b

PROJECT: **BISHOP RANCH - BR3A**
San Ramon, California

Log of Boring B-6

Boring location: See Figure 2
Date started: 4/25/16 Date finished: 4/25/16
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches Hammer type: Downhole Wireline
Samplers: Sprague & Henwood (S&H)

Logged by: K. Watkins
Drilled By: Exploration Geo Services

LABORATORY TEST DATA

Ground Surface Elevation: 443.5 feet^2

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value^1	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
1			5			CLAY with SAND (CL) dark brown with olive-gray mottling, stiff, moist, fine- to coarse-grained sand, trace fine angular gravel						
2	S&H		8 9	10	CL	LL = 45, PI = 23, see Figure C-8						
3			8			stiff to very stiff, no gravel						
4	S&H		11 14	15								
5			7			CLAY (CH) gray, very stiff, moist, trace fine-grained sand						
6	S&H		11 17	17	CH							
7						CLAY with SAND (CL) olive-gray, very stiff, moist, fine-grained sand						
8	S&H		9 12 17	17	CL							
9												
10			7			SANDY CLAY (CL) olive-gray, stiff, moist, fine-grained sand						
11	S&H		8 9	10			PP		1,500			
12												
13					CL							
14												
15			4			∇ (4/25/2016, 2:00 p.m.)						
16	S&H		8 17	15	SC	CLAYEY SAND with GRAVEL (SC) olive-gray, medium dense, wet, fine- to coarse-grained sand, fine angular gravel						
17												
18						CLAY (CL) olive-gray, stiff to very stiff, wet						
19												
20			6			stiff, trace fine-grained sand						
21	S&H		8 12	12								
22												
23					CL							
24												
25			11			gray with olive-gray mottling, very stiff, decreased sand content						
26	S&H		12 15	16								
27												
28												
29												
30												

LANGAN TREADWELL ROLLO

Project No.: 750633001
Figure: A-6a

PROJECT:	**BISHOP RANCH - BR3A** San Ramon, California	**Log of Boring B-6** PAGE 2 OF 2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
31	S&H	▨	10 16 24	24		CLAY (CL) (continued)						
32												
33												
34												
35												
36	S&H	▨ ☐	12 17 21	23	CL		PP		2,750			
37												
38												
39												
40												
41	S&H	▨	10 18 22	24		olive-gray, trace fine-grained sand						
42												
43												
44												
45												
46												
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 41.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 15.5 feet below ground surface during drilling.
PP = pocket penetrometer.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.

[2] Elevations based on NGVD29 datum and site plan titled "City of San Ramon City Center, BR3 Site, Existing Utility and Easements," by RJA Engineers, 9 January 2014.

LANGAN TREADWELL ROLLO

Project No.:	Figure:
750633001	A-6b

PROJECT: **BISHOP RANCH - BR3A**
San Ramon, California

Log of Boring B-7

PAGE 1 OF 2

Boring location: See Figure 2
Date started: 4/27/16
Date finished: 4/27/16
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Downhole Wireline
Samplers: Sprague & Henwood (S&H)

Logged by: K. Watkins
Drilled By: Exploration Geo Services

LABORATORY TEST DATA

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 442.5 feet[2]	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
1						CLAY (CH) dark brown, medium stiff to stiff, moist, trace fine-grained sand						
2												
3	S&H	■	3 6 8	8	CH	LL = 60, PI = 40, see Figure C-8 Corrosion Test, see Appendix D					22.7	101
4												
5												
6	S&H	■	9 22 25	28	CL	CLAY with SAND (CL) brown with gray-brown mottling, very stiff, moist, fine-grained sand						
7					CL	SANDY CLAY (CL) gray-brown, very stiff, moist, fine-grained sand						
8	S&H	■ □	7 8 9	10		CLAY (CL) olive-gray, stiff, moist, trace fine-grained sand						
9												
10												
11	S&H	■	5 9 13	13								
12												
13	S&H	■	5 6 7	8		medium stiff to stiff						
14												
15												
16	S&H	■	5 7 9	10		stiff ▽ (4/27/2016, 11:45 a.m.)						
17												
18												
19					CL							
20												
21	S&H	■ □	9 11 13	14		stiff						
22												
23												
24												
25												
26	S&H	■ □	10 17 18	21		very stiff						
27												
28												
29												
30												

LANGAN TREADWELL ROLLO

Project No.: 750633001
Figure: A-7a

PROJECT: **BISHOP RANCH - BR3A**
San Ramon, California

Log of Boring B-7

PAGE 2 OF 2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
31	S&H	■	9 17 20	22		CLAY (CL) (continued) gray-brown						
32												
33					CL							
34												
35												
36	S&H	■	10 20 21	25								
37												
38												
39												
40												
41												
42												
43												
44												
45												
46												
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 36.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 16.5 feet below ground surface during drilling.
PP = pocket penetrometer.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.

[2] Elevations based on NGVD29 datum and site plan titled "City of San Ramon City Center, BR3 Site, Existing Utility and Easements," by RJA Engineers, 9 January 2014.

LANGAN TREADWELL ROLLO

Project No.: 750633001

Figure: A-7b

TEST GEOTECH LOG 750633001-GEOTECH BISHOPRANCH.GPJ TR.GDT 7/27/16

PROJECT: **BISHOP RANCH - BR3A**
San Ramon, California

Log of Boring B-8

PAGE 1 OF 2

Boring location: See Figure 2
Date started: 4/27/16
Date finished: 4/27/16
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Downhole Wireline
Samplers: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)

Logged by: K. Watkins
Drilled By: Exploration Geo Services

DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]		Ground Surface Elevation: 444.5 feet[2]	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
1						CLAY with SAND (CL) dark brown, very stiff, moist, fine- to medium-grained sand LL = 47, PI = 28, see Figure C-8						
2					CL							
3	S&H	■	5 12 15	16								
4												
5						CLAY (CL) dark brown, stiff, moist						
6	S&H	■	5 9 11	12	CL							
7												
8	S&H	■	7 10 13	14		SANDY CLAY (CL) olive-gray, stiff, moist, fine-grained sand						
9												
10					CL							
11												
12												
13	S&H	■	7 6 8	8	SM	SILTY SAND (SM) olive-gray, loose, moist, fine- to coarse-grained, nonplastic				28.6	17.6	
14					CL	SANDY CLAY (CL) olive-gray, medium stiff to stiff, moist, fine-grained sand						
15												
16	S&H	■	10 15 16	19		CLAY (CL) yellow-brown, very stiff, moist, trace fine-grained sand, some silt	PP		2,000			
17												
18					▽	(4/27/2016, 2:10 p.m.)						
19												
20						stiff, wet						
21	S&H	■	2 7 9	10	CL							
22												
23												
24												
25											31.2	92
26	ST	■	100 psi 500 psi			Consolidation Test, see Figure C-2					31.0	88
27						CLAYEY SAND (SC) olive-gray, wet, fine-grained						
28					SC							
29					CL	CLAY (CL) gray, very stiff, wet						
30												

LANGAN TREADWELL ROLLO

Project No.: 750633001
Figure: A-8a

PROJECT: **BISHOP RANCH - BR3A**
San Ramon, California

Log of Boring B-8

PAGE 2 OF 2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
31	S&H		13 19 21	24		CLAY (CL) (continued)						
32							PP		3,500			
33												
34					CL							
35						olive-gray, trace fine-grained sand						
36	S&H		12 16 21	22			PP		3,000			
37												
38												
39						CLAYEY SAND (SC) olive-gray with dark brown mottling, dense, wet, fine- to coarse-grained, trace fine subangular gravel						
40					SC							
41	S&H		12 20 34	32		LL = 30, PI = 16, see Figure C-9				35.9	16.5	
42	SPT		7 11 17	28	CL	CLAY (CL) olive-gray, very stiff, wet						
43												
44												
45												
46												
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 43 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 18.2 feet below ground surface during drilling.
PP = pocket penetrometer.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.

[2] Elevations based on NGVD29 datum and site plan titled "City of San Ramon City Center, BR3 Site, Existing Utility and Easements," by RJA Engineers, 9 January 2014.

LANGAN TREADWELL ROLLO

Project No.: 750633001

Figure: A-8b

PROJECT: **BISHOP RANCH - BR3A**
San Ramon, California

Log of Boring B-9

PAGE 1 OF 2

Boring location: See Figure 2
Date started: 4/25/16
Date finished: 4/25/16
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Downhole Wireline
Samplers: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Logged by: K. Watkins
Drilled By: Exploration Geo Services

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 444 feet[2]	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
					CL	SANDY CLAY with GRAVEL (CL) brown, moist, fine- to coarse-grained sand, fine subangular gravel FILL						
1			6									
2	S&H		10	13		CLAY (CH) dark brown with olive-gray mottling, stiff, moist, trace fine- to coarse-grained sand, trace fine angular gravel						
			12									
3			10									
4	S&H		13	17	CH	very stiff, decreased sand content						
			15									
5			10									
6	S&H		16	18		gray						
			14									
7						CLAY with SAND (CL) gray-brown, very stiff, moist, fine-grained sand						
8			12									
	S&H		18	23								
9			20									
					CL							
10			9			increased sand and silt content						
11	S&H		12	17								
			16									
12						SANDY CLAY (CL) olive-gray, medium stiff to stiff, moist, fine- to coarse-grained sand						
13			3		CL							
	S&H		5	8								
			8			∇ (4/25/2016, 11:45 a.m.)						
14			7			CLAYEY SAND (SC) olive-gray, medium dense, wet, fine- to medium-grained sand, trace fine angular gravel						
15	SPT		8	16	SC							
			8									
16												
17						CLAY (CL) olive-gray, stiff, wet, trace fine-grained sand						
18			4									
	SPT		5	10	CL							
19			5									
20			8									
21	S&H		12	16	CL	SANDY CLAY (CL) olive-gray, very stiff, wet, fine-grained sand						
			14									
22						CLAY (CL) gray, very stiff, wet, trace fine-grained sand						
23												
24												
25			7									
26	S&H		11	17	CL							
			17									
27												
28												
29												
30												

LANGAN TREADWELL ROLLO

Project No.: 750633001
Figure: A-9a

PROJECT: **BISHOP RANCH - BR3A**
San Ramon, California

Log of Boring B-9

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
	S&H	▨	8 14 20	20		CLAY (CL) (continued)						
31												
32					CL							
33												
34						CLAY with SAND (CL) olive-gray, very stiff, wet, fine-grained sand						
35												
36	S&H	▨	9 18 26	26								
37												
38					CL							
39												
40												
41	S&H	▨	5 9 13	13		stiff						
42												
43												
44												
45												
46												
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 41.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 14 feet below ground surface during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.

[2] Elevations based on NGVD29 datum and site plan titled "City of San Ramon City Center, BR3 Site, Existing Utility and Easements," by RJA Engineers, 9 January 2014.

LANGAN TREADWELL ROLLO

Project No.: 750633001

Figure: A-9b

PROJECT: **BISHOP RANCH - BR3A**
San Ramon, California

Log of Boring B-10

PAGE 1 OF 2

Boring location: See Figure 2

Date started: 4/26/16 Date finished: 4/26/16

Drilling method: Hollow Stem Auger

Hammer weight/drop: 140 lbs./30 inches Hammer type: Downhole Wireline

Samplers: Sprague & Henwood (S&H), Shelby Tube (ST)

Logged by: K. Watkins
Drilled By: Exploration Geo Services

LABORATORY TEST DATA

Ground Surface Elevation: 444.5 feet[2]

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
1						CLAY with SAND (CH) dark brown with yellow-brown mottling, stiff, moist, fine- to coarse-grained sand						
2												
3	S&H	■	5 7 10	10		decrease in sand content						
4												
5					CH	very stiff, trace fine angular gravel						
6	S&H	■	9 14 15	17								
7												
8	S&H	■	5 7 13	12		stiff						
9												
10						SANDY CLAY (CL) olive-gray, stiff, moist, fine- to medium-grained sand						
11	S&H	■	8 10 12	13	CL							
12												
13	S&H	■	6 8 8	10	SM	SILTY SAND (SM) olive-gray, loose to medium dense, moist, fine-grained, trace clay						
14												
15						SANDY CLAY (CL) olive-gray, stiff, moist, fine-grained sand						
16	S&H	■	7 8 14	13							17.2	113
17					CL							
18												
19												
20						▽ (4/26/2016, 12:30 p.m.)						
21	S&H	■	5 7 12	11		CLAY with SAND (CL) olive-gray, stiff, wet, fine-grained sand						
22							PP		1,000			
23												
24												
25		■	100 psi			stiff to very stiff	PP		2,500			
26	ST	□										
27			250 psi									
28												
29					CL	CLAY (CL) gray-brown, very stiff, wet, trace fine-grained sand						
30												

LANGAN TREADWELL ROLLO

Project No.: 750633001

Figure: A-10a

PROJECT: **BISHOP RANCH - BR3A**
San Ramon, California

Log of Boring B-10

PAGE 2 OF 2

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	S&H	■	8 13 16	17		CLAY (CL) (continued)						
32												
33												
34												
35												
36	S&H	■	6 11 14	15		gray-brown with olive-gray mottling, stiff to very stiff	PP		2,000			
37					CL							
38												
39												
40												
41	S&H	■	6 12 20	19		olive-gray, very stiff						
42												
43												
44						CLAY with SAND (CL) olive-gray, very stiff, wet, fine- to medium-grained sand						
45												
46	S&H	■	7 15 23	23							17.8	114
47												
48					CL							
49												
50												
51	S&H	■ □	12 19 25	26			PP		3,500			
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 51.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 20 feet below ground surface during drilling.
PP = pocket penetrometer.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.

[2] Elevations based on NGVD29 datum and site plan titled "City of San Ramon City Center, BR3 Site, Existing Utility and Easements," by RJA Engineers, 9 January 2014.

LANGAN TREADWELL ROLLO

Project No.: 750633001

Figure: A-10b

PROJECT: **BISHOP RANCH - BR3A**
San Ramon, California

Log of Boring B-11

PAGE 1 OF 2

Boring location: See Figure 2
Date started: 4/25/16
Date finished: 4/25/16
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Downhole Wireline
Samplers: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Logged by: K. Watkins
Drilled By: Exploration Geo Services

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value^1	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 443 feet^2	LABORATORY TEST DATA Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
1						CLAY with SAND (CH) dark brown, very stiff, moist, fine- to coarse-grained sand, trace fine subangular gravel						
2	S&H	■	10 12 16	17	CH							
3												
4	S&H	■	11 13 17	18		CLAY (CH) dark brown, very stiff, moist, trace fine-grained sand						
5												
6	S&H	■	12 15 17	19	CH							
7												
8	S&H	■	10 19 25	26	CL	SANDY CLAY (CL) gray-brown, very stiff, moist, fine-grained sand						
9												
10						CLAYEY SAND with GRAVEL (SC) olive-gray, medium dense, moist, fine- to coarse-grained sand						
11	S&H	■	6 9 14	14	SC							
12						SANDY CLAY (CL) olive-gray, stiff, moist, fine-grained sand						
13					CL							
14						▽ (4/26/2016, 10:11 a.m.)						
15						SILTY SAND (SM) olive-gray, medium dense, wet, fine-grained, nonplastic coarse-grained						
16	S&H	■	7 14 14	17	SM					34.3	21.9	
17						CLAY (CL) olive-gray, medium stiff, wet						
18												
19												
20												
21	SPT	◩	3 3 3	6								
22					CL							
23												
24												
25						gray-brown, very stiff, trace fine-grained sand						
26	S&H	■	6 14 17	19								
27												
28						CLAY (CH) gray, very stiff, wet, trace fine-grained sand						
29					CH							
30												

LANGAN TREADWELL ROLLO

Project No.: 750633001
Figure: A-11a

PROJECT: **BISHOP RANCH - BR3A**
San Ramon, California

Log of Boring B-11

PAGE 2 OF 2

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	S&H	■	7 14 19	20		CLAY (CH) (continued)						
32					CH							
33												
34						CLAY (CL) gray-brown, very stiff, wet, trace fine-grained sand						
35					CL							
36	S&H	■	7 10 17	16							23.0	92
37												
38												
39												
40												
41												
42												
43												
44												
45												
46												
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 36.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 14.3 feet below ground surface during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.
[2] Elevations based on NGVD29 datum and site plan titled "City of San Ramon City Center, BR3 Site, Existing Utility and Easements," by RJA Engineers, 9 January 2014.

LANGAN TREADWELL ROLLO

Project No.: 750633001

Figure: A-11b

PROJECT: **BISHOP RANCH - BR3A**
San Ramon, California

Log of Boring B-12

PAGE 1 OF 2

Boring location: See Figure 2
Date started: 4/28/16
Date finished: 4/28/16
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Downhole Wireline
Samplers: Sprague & Henwood (S&H), Shelby Tube (ST)

Logged by: K. Watkins
Drilled By: Exploration Geo Services

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: 441.5 feet[2]	LABORATORY TEST DATA Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						CLAY with SAND (CH) dark brown, very stiff, moist, fine- to medium-grained sand						
1												
2					CH							
3	S&H	■	10 12 18	18		LL = 54, PI = 35, see Figure C-9						
4												
5						SANDY CLAY (CL) olive-gray, stiff, moist, fine-grained sand						
6	S&H	■	8 9 10	11	CL	yellow-brown						
7												
8	S&H	■	5 8 9	10	CL	CLAY (CL) yellow-brown, stiff, moist, trace fine-grained sand LL = 34, PI = 14, see Figure C-9						
9												
10						SILTY SAND (SM) olive-gray to yellow-brown, loose to medium dense, moist, fine-grained						
11	S&H	■	5 7 9	10	SM					39.3	16.0	
						CLAY (CL) yellow-brown, medium stiff to stiff, moist, trace fine-grained sand, some silt						
12												
13	S&H	■	4 5 9	8								
14							PP		750			
15					CL	▽ (4/28/2016, 8:45 a.m.)						
16	S&H	■	4 5 8	8			PP		1,200			
17												
18	ST	□	100 psi 200 psi			TxUU Test, see Figure C-6 Consolidation Test, see Figure C-3	TxUU	1,800	680		29.5 24.0	94 99
19												
20					SC	CLAYEY SAND (SC) brown, wet, fine-grained						
21	S&H	■	5 8 9	10		CLAY with SAND (CL) olive-gray, stiff, wet, fine-grained sand						
22					CL							
23												
24						CLAY (CL) gray, very stiff, wet						
25												
26	S&H	■	10 16 20	22	CL		PP		2,250			
27												
28												
29												
30					CL							

TEST GEOTECH LOG 750633001-GEOTECH BISHOPRANCH.GPJ TR.GDT 7/27/16

LANGAN TREADWELL ROLLO

Project No.: 750633001
Figure: A-12a

PROJECT: **BISHOP RANCH - BR3A**
San Ramon, California

Log of Boring B-12

PAGE 2 OF 2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
31	S&H	▨	8 11 15	16		CLAY with SAND (CL) gray-brown, very stiff, wet, fine-grained sand	PP		1,500			
32					CL							
33												
34												
35						CLAY (CL) gray-brown, very stiff, wet, trace fine-grained sand						
36	S&H	▨	6 14 18	19			PP		1,500			
37												
38					CL							
39												
40						olive-gray, hard						
41	S&H	▨	13 26 34	36			PP		>4,000		19.0	112
42												
43												
44												
45												
46												
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 41.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 15 feet below ground surface during drilling.
PP = pocket penetrometer.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.

[2] Elevations based on NGVD29 datum and site plan titled "City of San Ramon City Center, BR3 Site, Existing Utility and Easements," by RJA Engineers, 9 January 2014.

LANGAN TREADWELL ROLLO

Project No.: 750633001

Figure: A-12b

PROJECT: **BISHOP RANCH - BR3A**
San Ramon, California

Log of Boring B-13

PAGE 1 OF 2

Boring location: See Figure 2
Date started: 4/26/16
Date finished: 4/26/16
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Downhole Wireline
Samplers: Sprague & Henwood (S&H), Grab (GRAB)

Logged by: K. Watkins
Drilled By: Exploration Geo Services

Ground Surface Elevation: 442.5 feet[2]

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
1–2	GRAB	☒			CH	CLAY with SAND (CH) dark brown, stiff, moist, fine- to coarse-grained sand Resistance Value Test, see Figure C-10						
3–4	S&H	▨	4 7 14	13	CH							
5–6	S&H	▨	11 13 14	16	CH	very stiff, trace fine subangular gravel						
7–9	S&H	▨	13 14 15	17	CL	SANDY CLAY (CL) olive-gray, very stiff, moist, fine-grained sand						
10–12	S&H	▨	10 12 15	16	CL	CLAY with SAND (CL) olive-gray, very stiff, moist, fine-grained sand						
12–14	S&H	▨	7 10 15	15	CL	SANDY CLAY (CL) olive-gray, stiff to very stiff, moist, fine- to medium-grained sand				39.2	15.4	
14–16	S&H	▨	6 7 10	10	SC ▽	CLAYEY SAND (SC) olive-gray, medium dense, moist, fine- to medium-grained LL = 28, PI = 11, see Figure C-9 (4/26/2016, 1:35 p.m.)						
17–19					CL	CLAY (CL) olive-gray, stiff, wet						
20–24	S&H	▨	6 9 10	11	CL	trace fine-grained sand						
25–30	S&H	▨	9 16 21	22	CL	olive-gray with dark brown mottling, very stiff						

LANGAN TREADWELL ROLLO

Project No.: 750633001
Figure: A-13a

PROJECT: **BISHOP RANCH - BR3A**
San Ramon, California

Log of Boring B-13

PAGE 2 OF 2

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	S&H	▨	8 13 19	19		CLAY (CL) olive-gray with gray-brown mottling, decreased sand content						
32												
33												
34												
35												
36	S&H	▨	9 11 15	16	CL	gray-brown, trace fine-grained sand						
37												
38												
39												
40												
41	S&H	▨	7 8 15	14		stiff	PP		2,250			
42												
43												
44												
45												
46												
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 41.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 15 feet below ground surface during drilling.
PP = pocket penetrometer.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.
[2] Elevations based on NGVD29 datum and site plan titled "City of San Ramon City Center, BR3 Site, Existing Utility and Easements," by RJA Engineers, 9 January 2014.

LANGAN TREADWELL ROLLO

Project No.: 750633001

Figure: A-13b

PROJECT: **BISHOP RANCH - BR3A**
San Ramon, California

Log of Boring B-14

Boring location: See Figure 2
Date started: 4/25/16 Date finished: 4/25/16
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches Hammer type: Downhole Wireline
Samplers: Sprague & Henwood (S&H)

Logged by: K. Watkins
Drilled By: Exploration Geo Services

	SAMPLES					MATERIAL DESCRIPTION	LABORATORY TEST DATA					
DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	Ground Surface Elevation: 440.5 feet[2]	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
1		■				CLAY with SAND (CH) dark brown with olive-gray mottling, very stiff, moist, fine- to coarse-grained sand, trace organic material						
2	S&H	■	12 16 23	23								
3		■			CH	dark brown, stiff to very stiff, decrease in gravel content LL = 69, PI = 49, see Figure C-9						
4	S&H	■	8 11 14	15								
5		■				dark gray, very stiff, decrease in sand content						
6	S&H	■	10 15 26	25								
7						SANDY CLAY (CL) olive-gray, stiff, moist, fine-grained sand						
8	S&H	■	7 7 10	10							19.1	105
9												
10					CL							
11	S&H	■	8 9 14	14								
12												
13												
14												
15		■			CL	⊻ CLAY (CL) olive-gray, stiff, wet (4/25/2016, 9:00 a.m.)						
16	S&H	■	8 11 11	13	SP	SAND (SP) red-brown, medium dense, wet, fine- to medium-grained	PP		1,000			
17						CLAY (CL) olive-gray, stiff, wet, trace fine-grained sand						
18												
19												
20		■				very stiff						
21	S&H	■	7 11 16	16								
22												
23					CL							
24												
25		■										
26	S&H	■	7 10 14	14		stiff TxUU Test, see Figure C-7	TxUU	2,600	1,710		25.4	101
27												
28												
29												
30												

PROJECT: **BISHOP RANCH - BR3A**
San Ramon, California

Log of Boring B-14

PAGE 2 OF 2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
31	S&H	▨	5 9 12	13		CLAY (CL) gray-brown						
32					CL							
33												
34						CLAY with SAND (CL) olive-gray with yellow-brown mottling, very stiff, wet, fine-grained sand						
35												
36	S&H	▨	7 11 25	22	CL							
37												
38												
39						CLAY (CL) olive-gray, very stiff, wet, trace fine-grained sand						
40					CL							
41	S&H	▨	13 16 23	23								
42												
43												
44												
45												
46												
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 41.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 15 feet below ground surface during drilling.
PP = pocket penetrometer.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.
[2] Elevations based on NGVD29 datum and site plan titled "City of San Ramon City Center, BR3 Site, Existing Utility and Easements," by RJA Engineers, 9 January 2014.

LANGAN TREADWELL ROLLO

Project No.: 750633001

Figure: A-14b

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions		Symbols	Typical Names
Coarse-Grained Soils (more than half of soil > no. 200 sieve size	**Gravels** (More than half of coarse fraction > no. 4 sieve size)	**GW**	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	Sands (More than half of coarse fraction < no. 4 sieve size)	**SW**	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
Fine -Grained Soils (more than half of soil < no. 200 sieve size)	**Silts and Clays** LL = < 50	**ML**	Inorganic silts and clayey silts of low plasticity, sandy silts, gravelly silts
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	Silts and Clays LL = > 50	**MH**	Inorganic silts of high plasticity
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic silts and clays of high plasticity
Highly Organic Soils		**PT**	Peat and other highly organic soils

GRAIN SIZE CHART

Classification	Range of Grain Sizes	
	U.S. Standard Sieve Size	Grain Size in Millimeters
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel	3" to No. 4	76.2 to 4.76
coarse	3" to 3/4"	76.2 to 19.1
fine	3/4" to No. 4	19.1 to 4.76
Sand	No. 4 to No. 200	4.76 to 0.075
coarse	No. 4 to No. 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	0.420 to 0.075
Silt and Clay	Below No. 200	Below 0.075

▽ Unstabilized groundwater level

▼ Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

⬒	Sample taken with Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter. Darkened area indicates soil recovered
◩	Classification sample taken with Standard Penetration Test sampler
◨	Undisturbed sample taken with thin-walled tube
☒	Disturbed sample
⊙	Sampling attempted with no recovery
◫	Core sample
◙	Analytical laboratory sample
▥	Sample taken with Direct Push or Drive sampler

SAMPLER TYPE

C	Core barrel
CA	California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter
D&M	Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube
O	Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube
PT	Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube
S&H	Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter
SPT	Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.5-inch inside diameter
ST	Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

BISHOP RANCH - BR3A
San Ramon, California

LANGAN TREADWELL ROLLO

CLASSIFICATION CHART

Date 05/18/16 | Project No. 750633001 | Figure A-15

APPENDIX B

LOGS OF CONE PENETRATION TESTS

Figure B-1

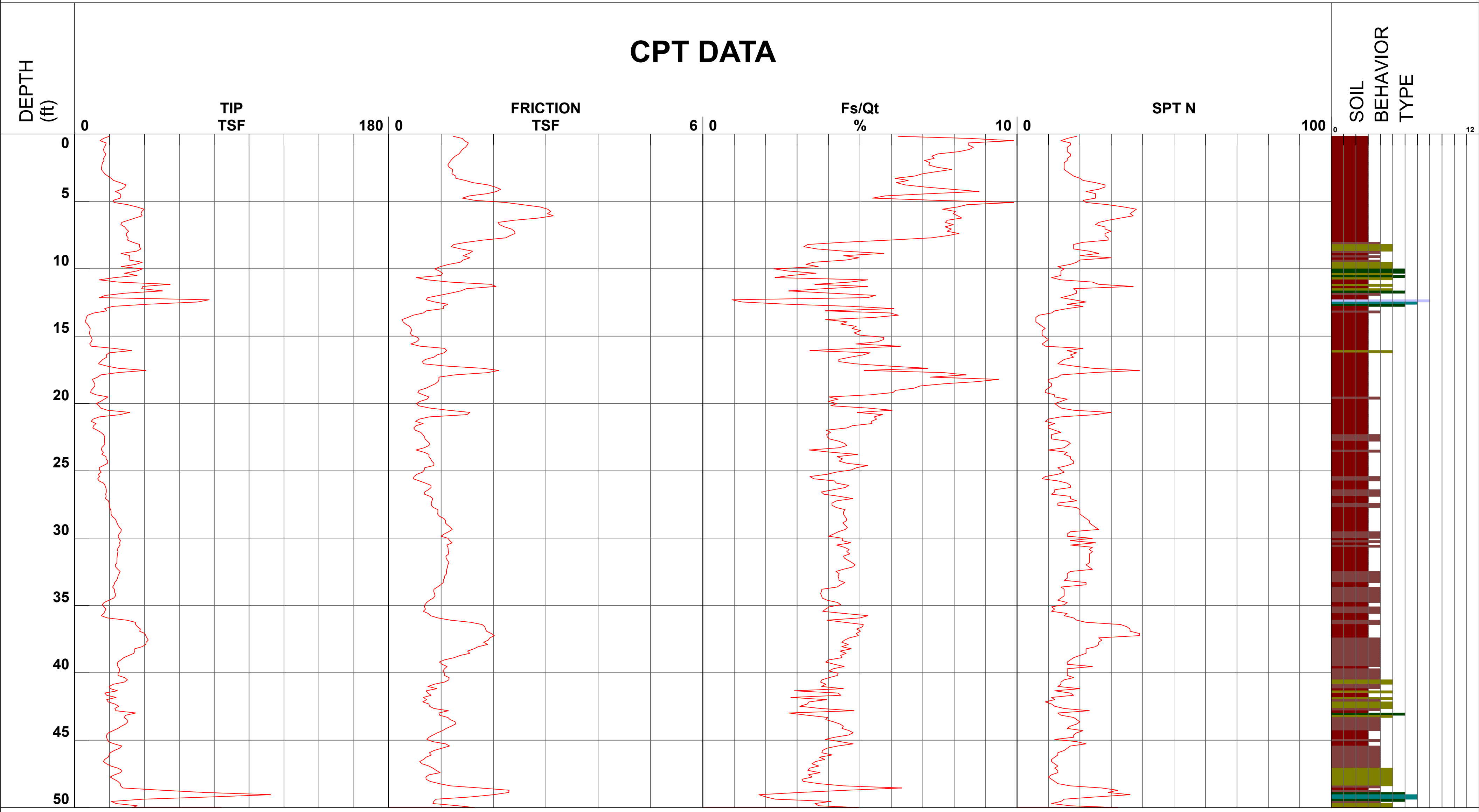
Treadwell & Rollo

Project Bishop Ranch BR3
Job Number G16-046-10L
Hole Number CPT-01
EST GW Depth During Test 12.00 ft

Operator BH-JH-SF
Cone Number DDG1350
Date and Time 4/26/2016 5:05:07 PM

Filename SDF(797).cpt
GPS
Maximum Depth 50.36 ft

Net Area Ratio .8



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

Cone Size 10cm squared

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

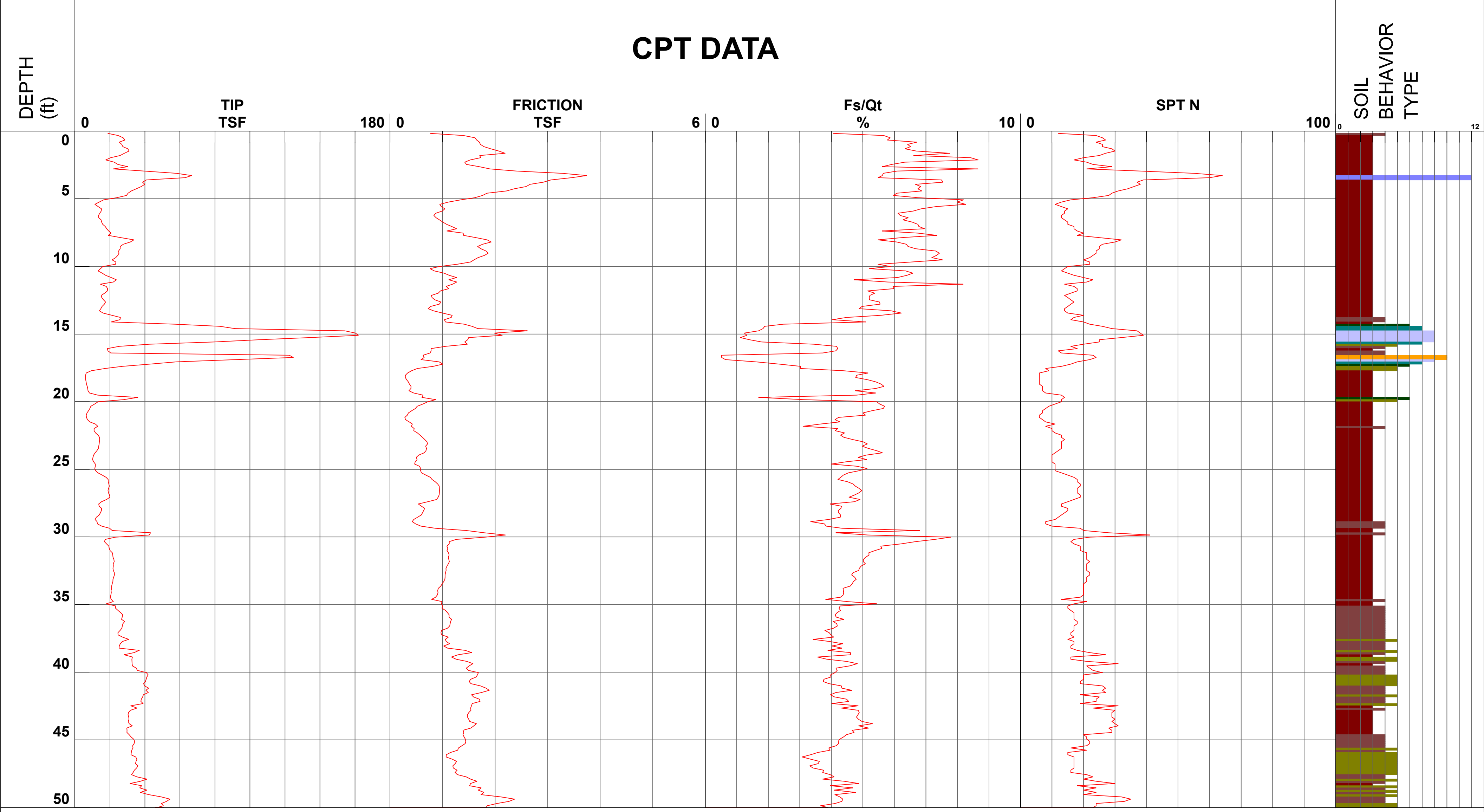
Figure B-2

Middle Earth
Geo Testing Inc.

Treadwell & Rollo

Project Bishop Ranch BR3
Job Number G16-046-10L
Hole Number CPT-02
EST GW Depth During Test 12.00 ft
Operator BH-JH-SF
Cone Number DDG1350
Date and Time 4/26/2016 4:03:58 PM
Filename SDF(796).cpt
GPS
Maximum Depth 50.36 ft

Net Area Ratio .8



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

Cone Size 10cm squared

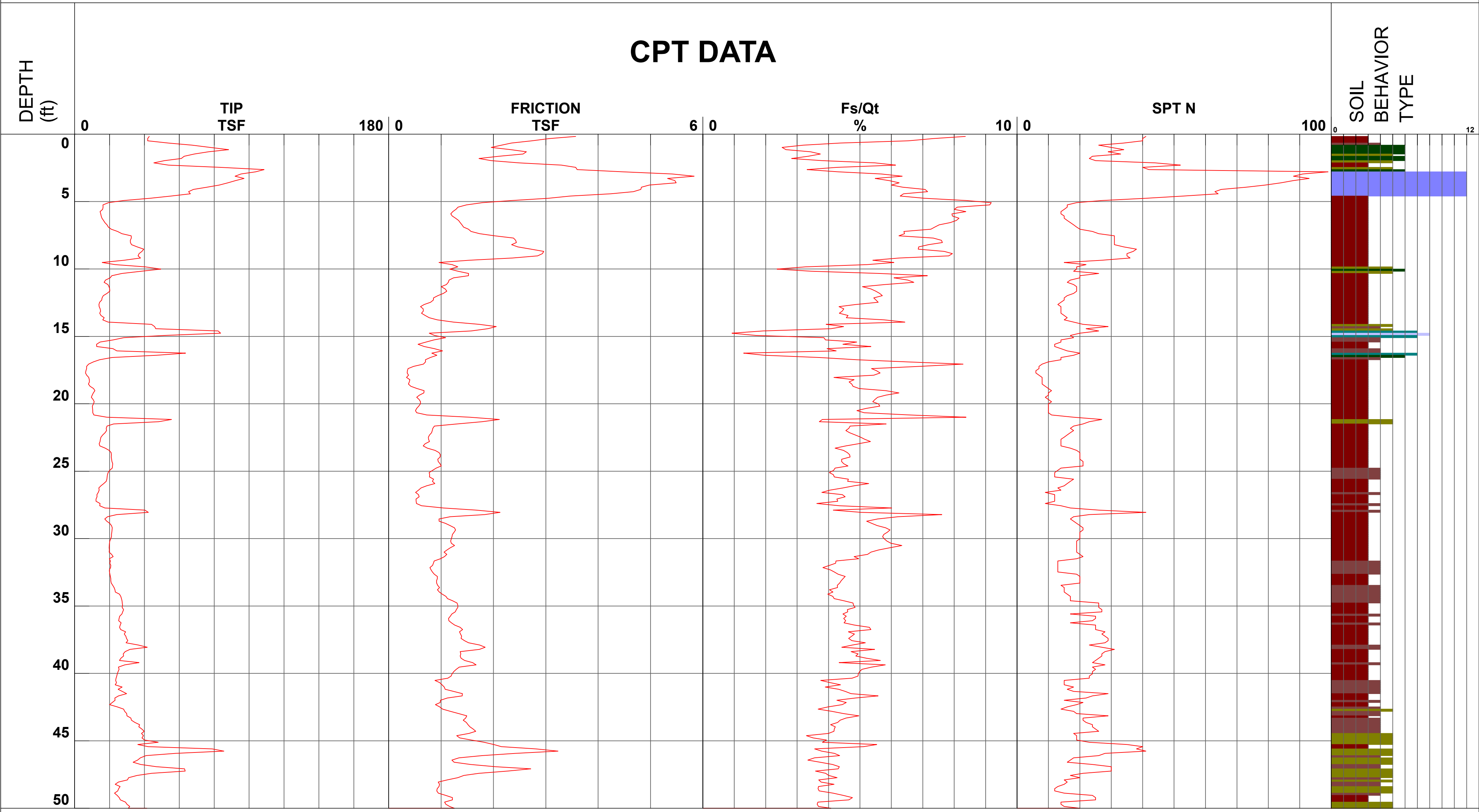
S*Soil behavior type and SPT based on data from UBC-1983

Figure B-3

Treadwell & Rollo

Project	Bishop Ranch BR3	**Operator**	BH-JH-SF	**Filename**	SDF(794).cpt
Job Number	G16-046-10L	**Cone Number**	DDG1350	**GPS**	
Hole Number	CPT-03	**Date and Time**	4/26/2016 1:36:08 PM	**Maximum Depth**	50.52 ft
EST GW Depth During Test		14.00 ft			

Net Area Ratio .8



■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay

■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt

■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand

■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

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

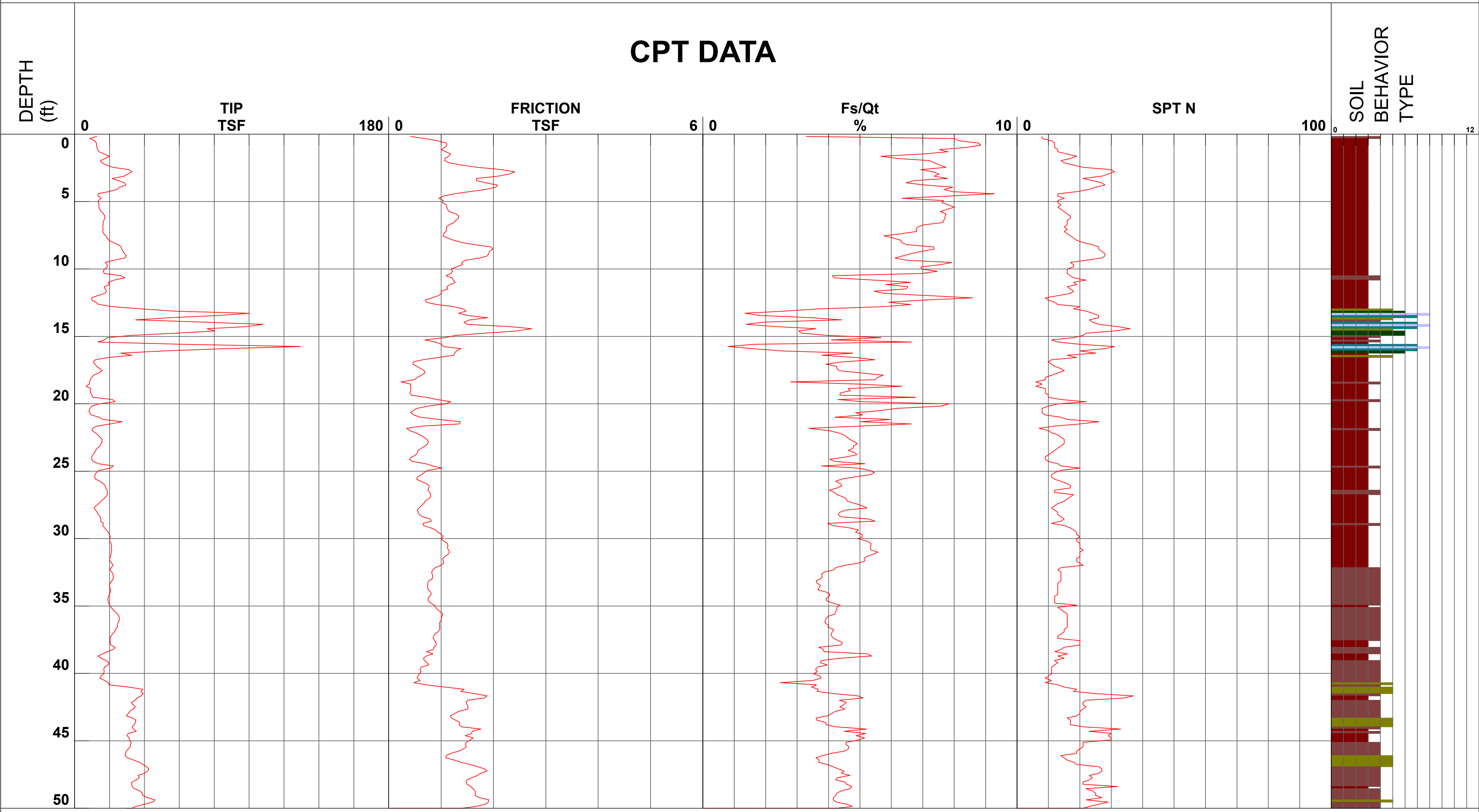
Treadwell & Rollo

Figure B-4

Middle Earth Geo Testing Inc.

Project	Bishop Ranch BR3	**Operator**	BH-JH-SF	**Filename**	SDF(793).cpt
Job Number	G16-046-10L	**Cone Number**	DDG1350	**GPS**	
Hole Number	CPT-04	**Date and Time**	4/26/2016 12:25:31 PM	**Maximum Depth**	50.36 ft
EST GW Depth During Test		16.00 ft			

Net Area Ratio .8



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

Cone Size 10cm squared

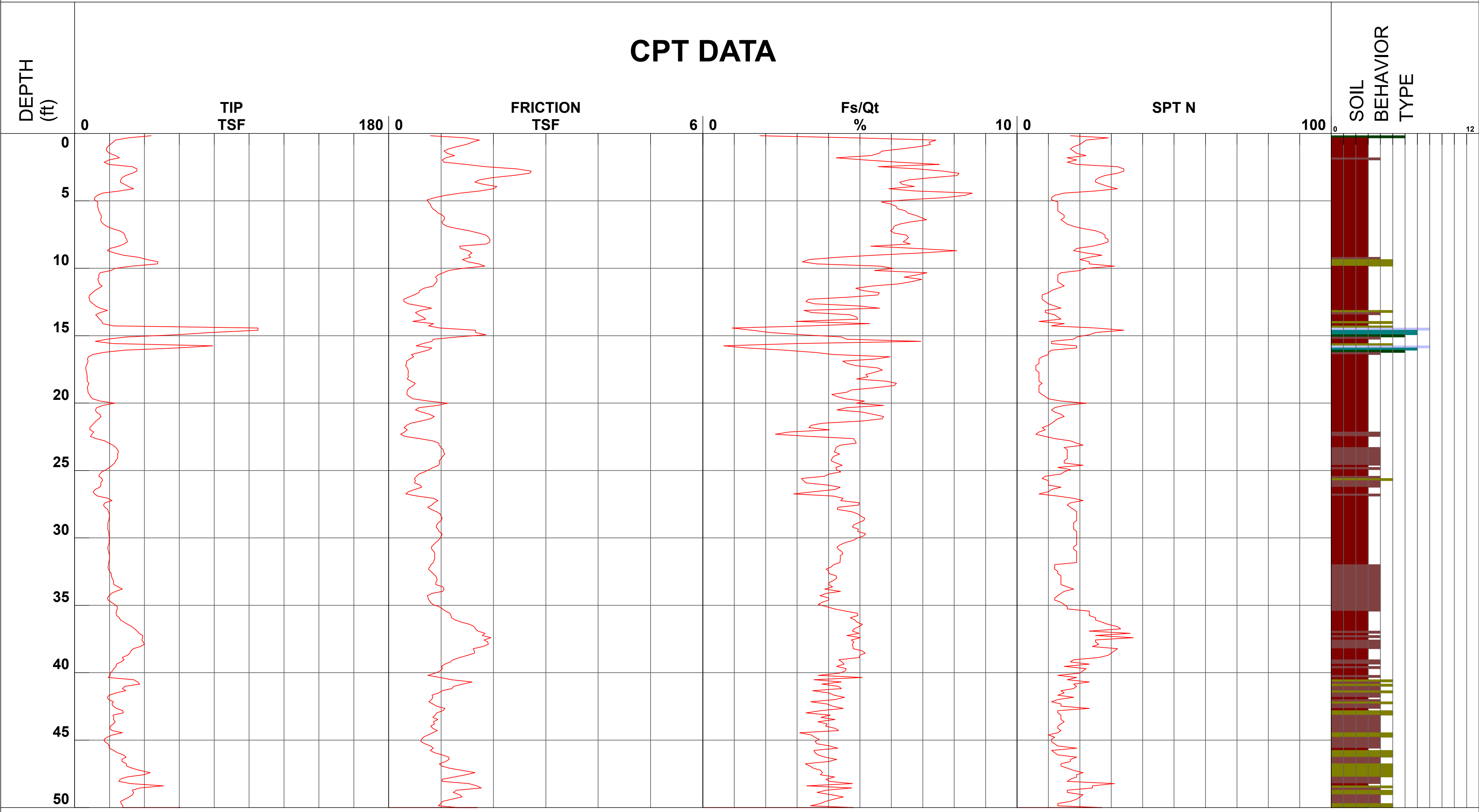
S*Soil behavior type and SPT based on data from UBC-1983

Figure B-5

Treadwell & Rollo

Project	Bishop Ranch BR3	**Operator**	BH-JH-SF	**Filename**	SDF(795).cpt
Job Number	G16-046-10L	**Cone Number**	DDG1350	**GPS**	
Hole Number	CPT-05	**Date and Time**	4/26/2016 2:53:04 PM	**Maximum Depth**	50.52 ft
EST GW Depth During Test		12.00 ft			

Net Area Ratio .8



■ 1 - sensitive fine grained

■ 2 - organic material

■ 3 - clay

■ 4 - silty clay to clay

■ 5 - clayey silt to silty clay

■ 6 - sandy silt to clayey silt

■ 7 - silty sand to sandy silt

■ 8 - sand to silty sand

■ 9 - sand

■ 10 - gravelly sand to sand

■ 11 - very stiff fine grained (*)

■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

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

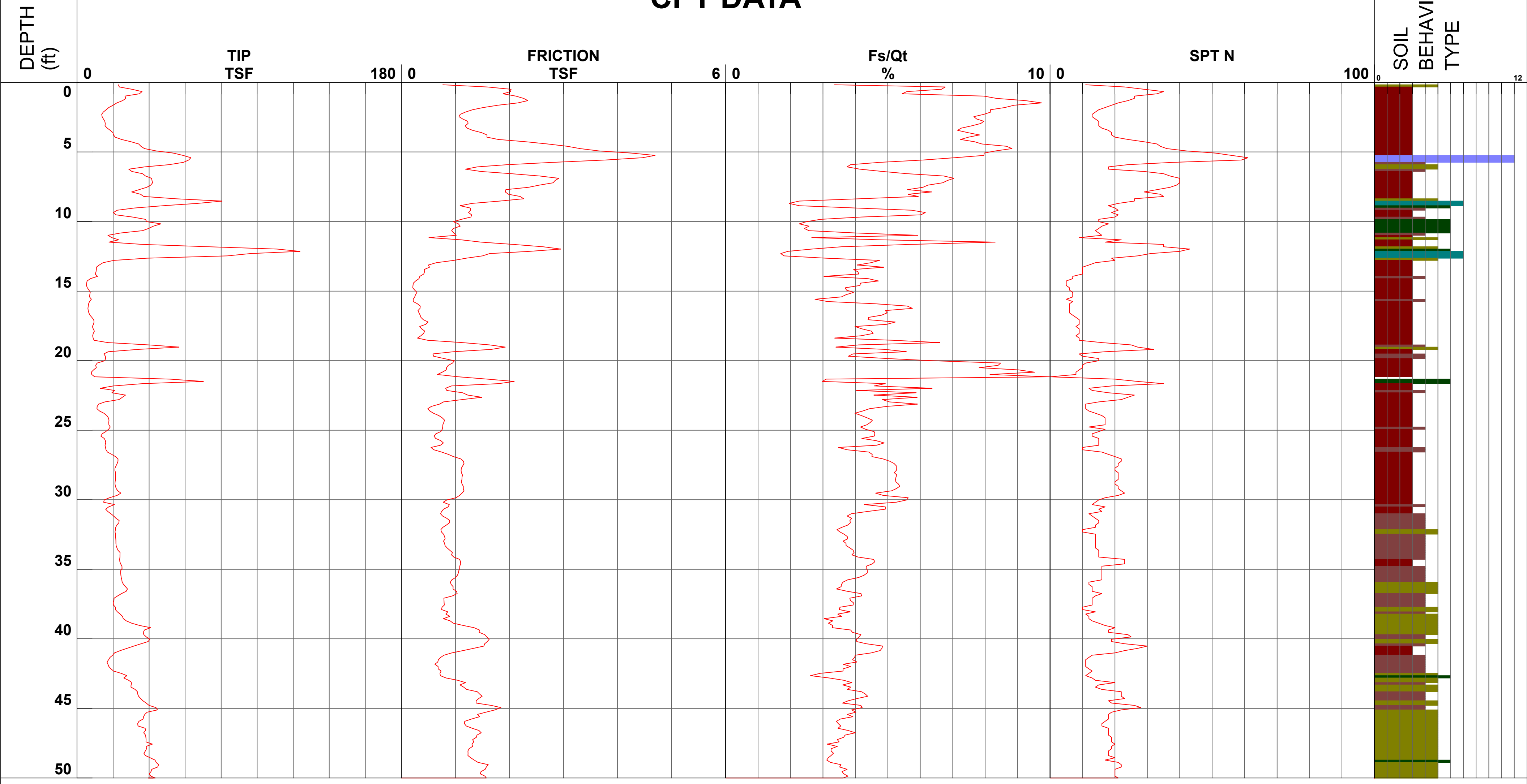
Figure B-6

Treadwell & Rollo

Project	Bishop Ranch BR3	**Operator**	BH-JH-SF	**Filename**	SDF(792).cpt
Job Number	G16-046-10L	**Cone Number**	DDG1350	**GPS**	
Hole Number	CPT-06	**Date and Time**	4/26/2016 10:41:19 AM	**Maximum Depth**	50.52 ft
EST GW Depth During Test		16.00 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

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

Figure B-7

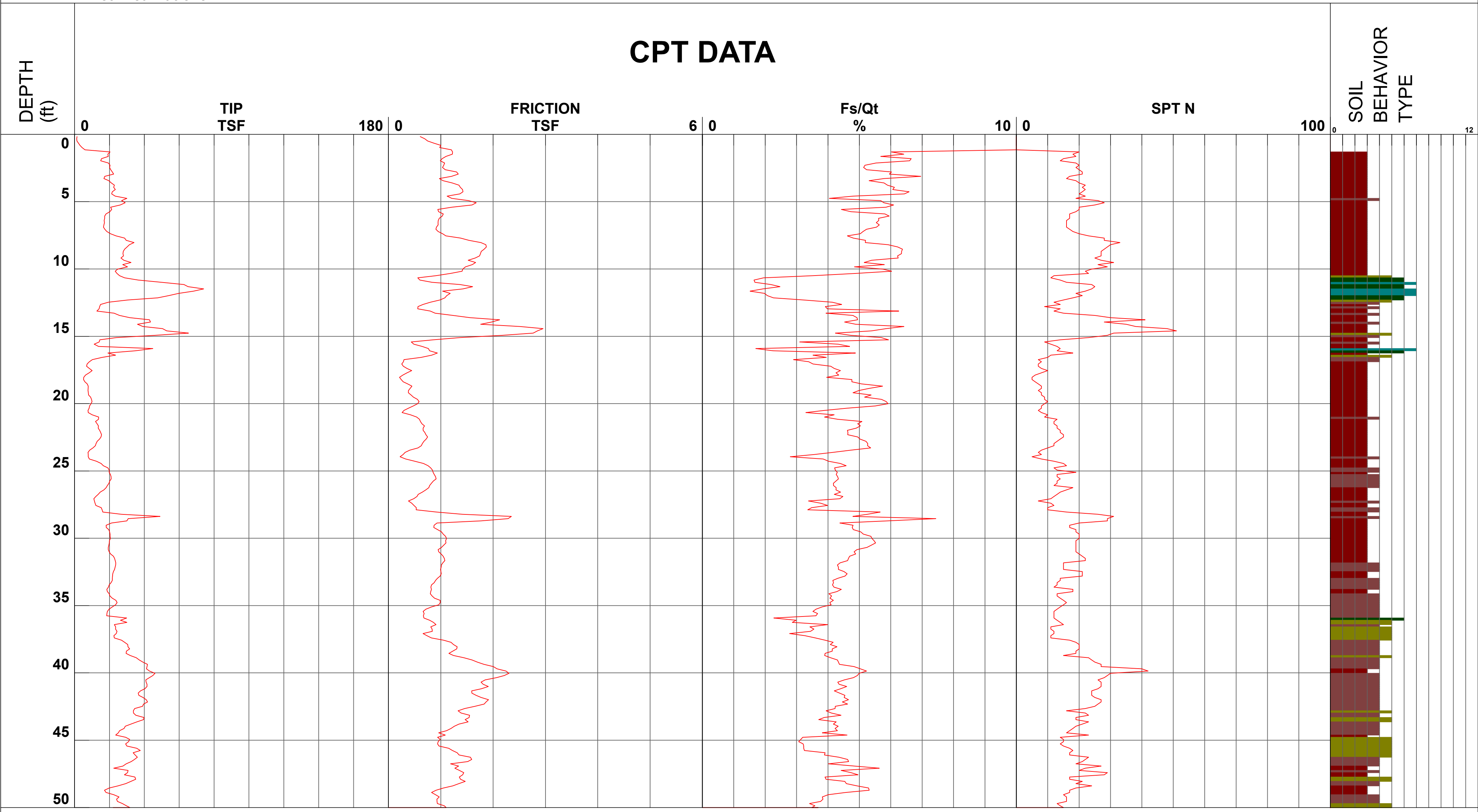
Treadwell & Rollo

Middle Earth
Geo Testing Inc.

Project	Bishop Ranch BR3	**Operator**	BH-JH-SF	**Filename**	SDF(791).cpt
Job Number	G16-046-10L	**Cone Number**	DDG1350	**GPS**	
Hole Number	CPT-07	**Date and Time**	4/26/2016 9:36:20 AM	**Maximum Depth**	50.52 ft
EST GW Depth During Test		18.00 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

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

Figure B-8

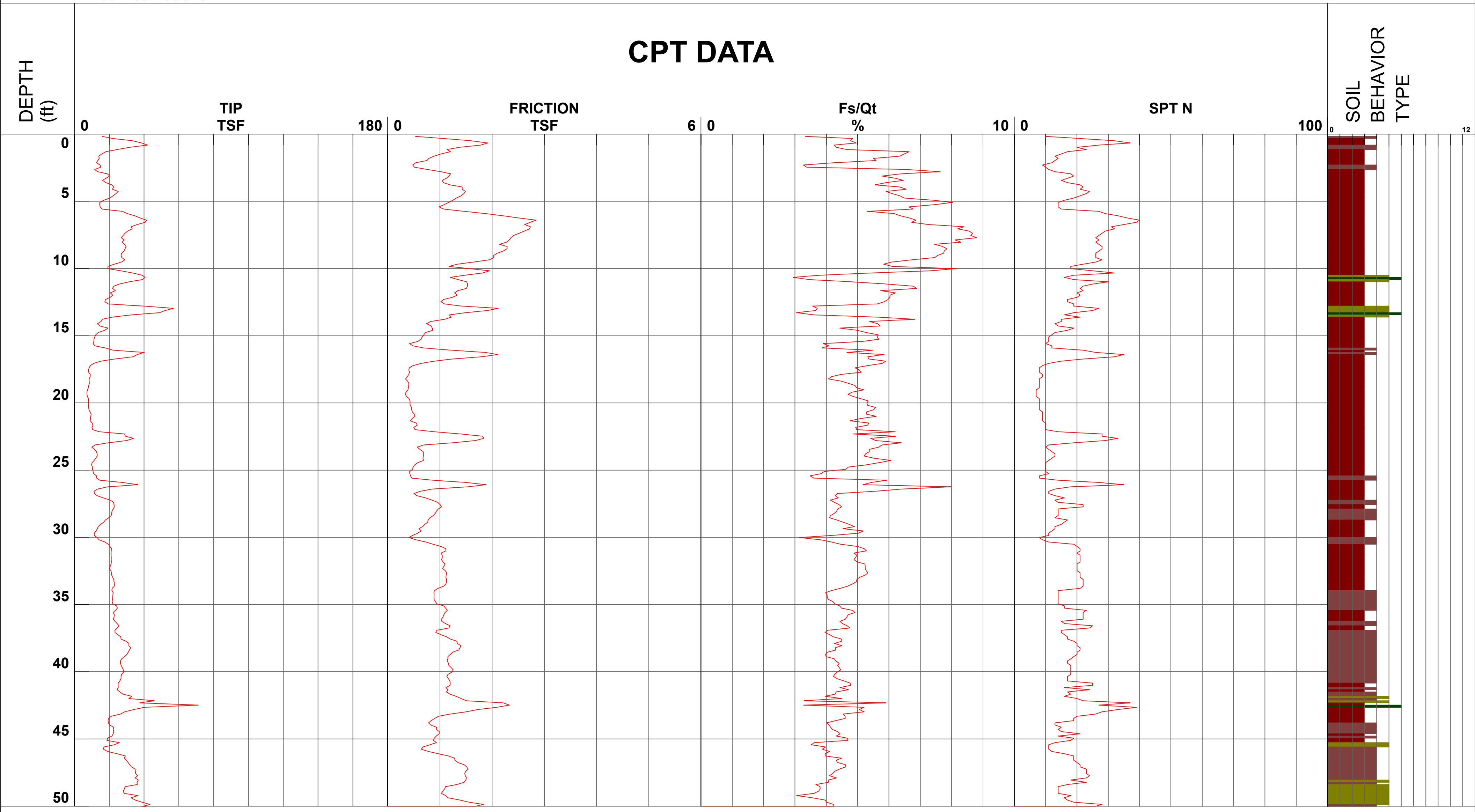
Treadwell & Rollo

Middle Earth Geo Testing Inc.

Project	Bishop Ranch BR3	Operator	BH-JH-SF	Filename	SDF(789).cpt
Job Number	G16-046-10L	Cone Number	DDG1350	GPS	
Hole Number	CPT-08	Date and Time	4/26/2016 7:26:21 AM	Maximum Depth	50.36 ft
EST GW Depth During Test		18.00 ft			

Net Area Ratio .8

CPT DATA



■ 1 - sensitive fine grained

■ 2 - organic material

■ 3 - clay

■ 4 - silty clay to clay

■ 5 - clayey silt to silty clay

■ 6 - sandy silt to clayey silt

■ 7 - silty sand to sandy silt

■ 8 - sand to silty sand

■ 9 - sand

■ 10 - gravelly sand to sand

■ 11 - very stiff fine grained (*)

■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

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

Middle Earth Geo Testing Inc.

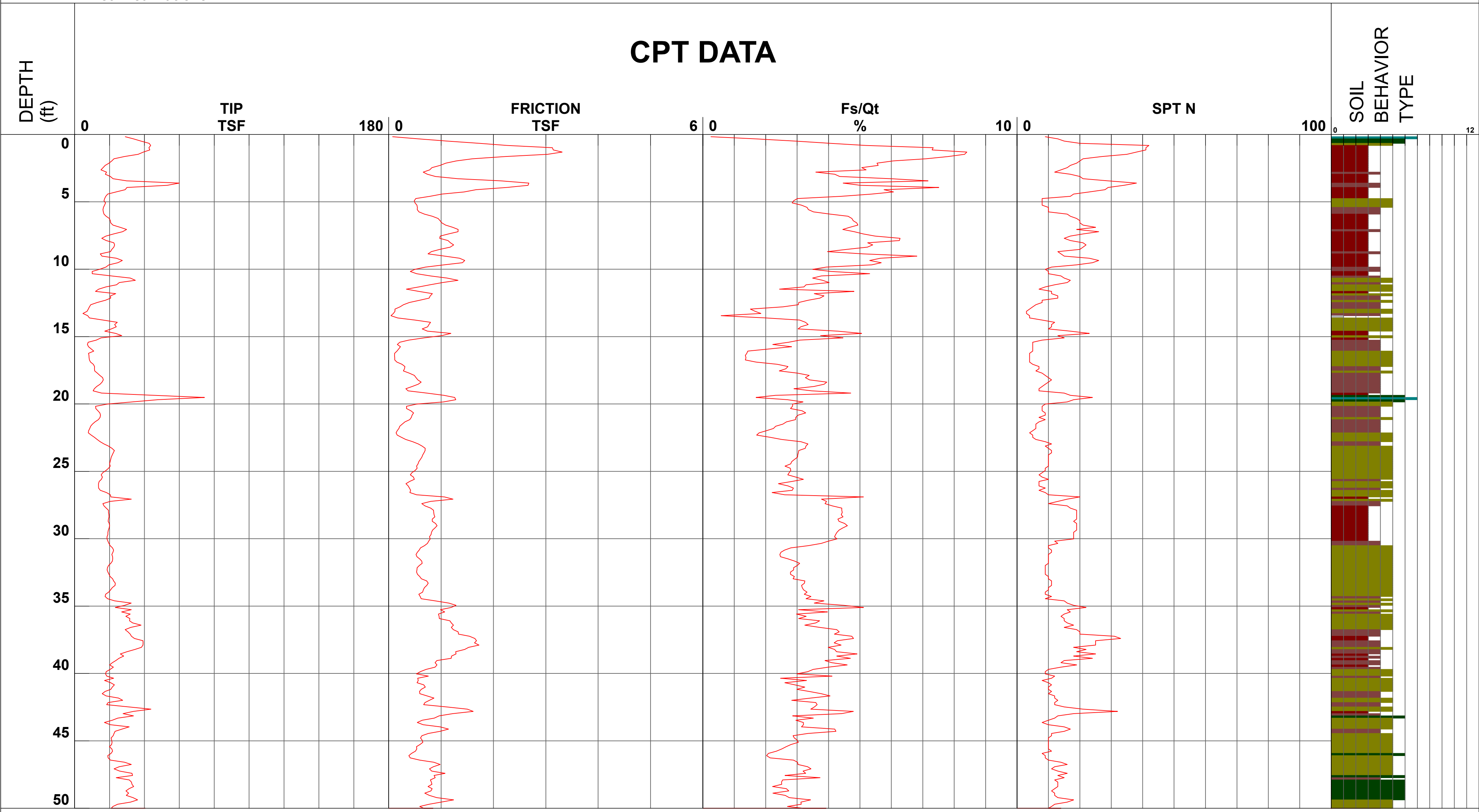
Treadwell & Rollo

Figure B-9

Project	Bishop Ranch BR3	Operator	BH-JH	Filename	SDF(864).cpt
Job Number	750633001	Cone Number	DDG1281	GPS	
Hole Number	CPT-09	Date and Time	5/5/2016 3:02:21 PM	Maximum Depth	50.52 ft
EST GW Depth During Test		15.00 ft			

Net Area Ratio .8

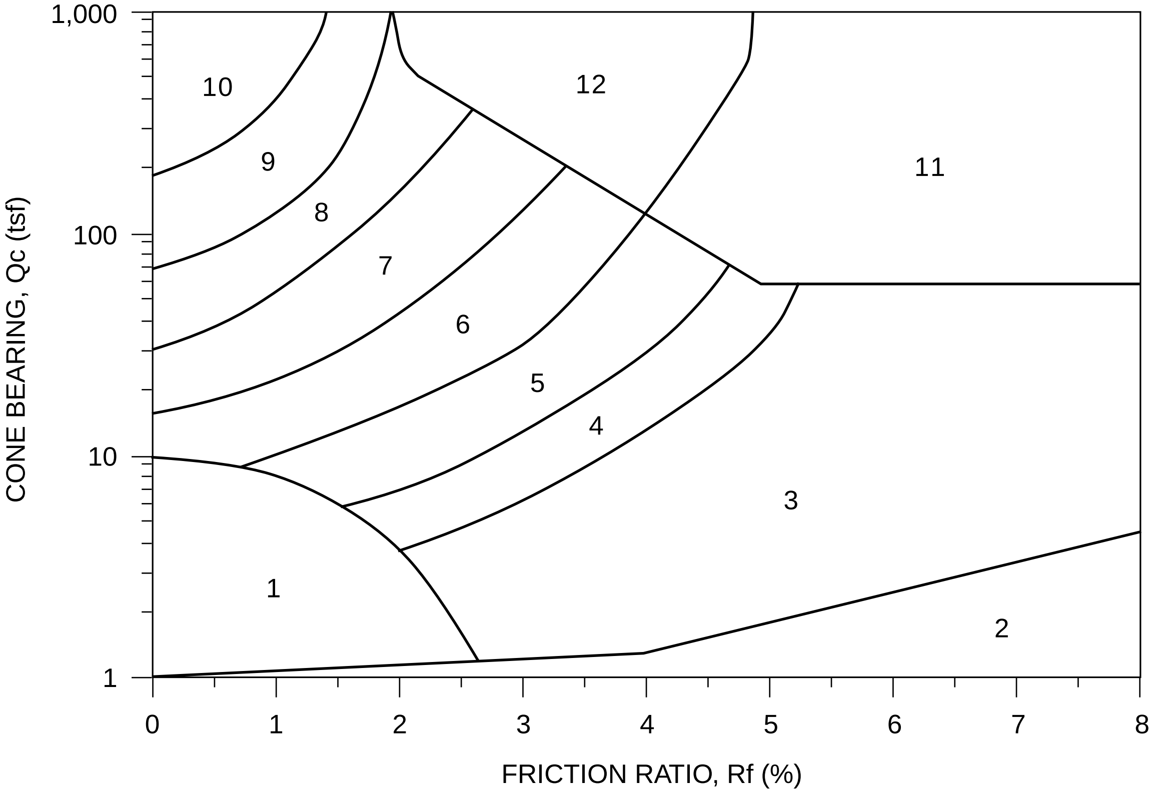
CPT DATA



- ⬜ 1 - sensitive fine grained
- ⬛ 2 - organic material
- 🟥 3 - clay
- 🟫 4 - silty clay to clay
- 🟨 5 - clayey silt to silty clay
- 🟩 6 - sandy silt to clayey silt
- 🟦 7 - silty sand to sandy silt
- 🟪 8 - sand to silty sand
- 🟧 9 - sand
- 🟥 10 - gravelly sand to sand
- 🟦 11 - very stiff fine grained (*)
- 🟦 12 - sand to clayey sand (*)

Cone Size 10cm squared

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



ZONE	Qc/N[1]	Su Factor (Nk)[2]	SOIL BEHAVIOR TYPE[1]
1	2	15 (10 for Qc ≤ 9 tsf)	Sensitive Fine-Grained
2	1	15 (10 for Qc ≤ 9 tsf)	Organic Material
3	1	15 (10 for Qc ≤ 9 tsf)	CLAY
4	1.5	15	SILTY CLAY to CLAY
5	2	15	CLAYEY SILT to SILTY CLAY
6	2.5	15	SANDY SILT to CLAYEY SILT
7	3	---	SILTY SAND to SANDY SILT
8	4	---	SAND to SILTY SAND
9	5	---	SAND
10	6	---	GRAVELLY SAND to SAND
11	1	15	Very Stiff Fine-Grained (*)
12	2	---	SAND to CLAYEY SAND (*)

(*) Overconsolidated or Cemented
Qc = Tip Bearing
Fs = Sleeve Friction
Rf = Fs/Qc x 100 = Friction Ratio

Note: Testing performed in accordance with ASTM D3441.

References: 1. Robertson, 1986, Olsen, 1988.
2. Bonaparte & Mitchell, 1979 (young Bay Mud Qc ≤9).
Estimated from local experience (fine-grained soils Qc > 9).

BISHOP RANCH - BR3A
San Ramon, California

LANGAN TREADWELL ROLLO

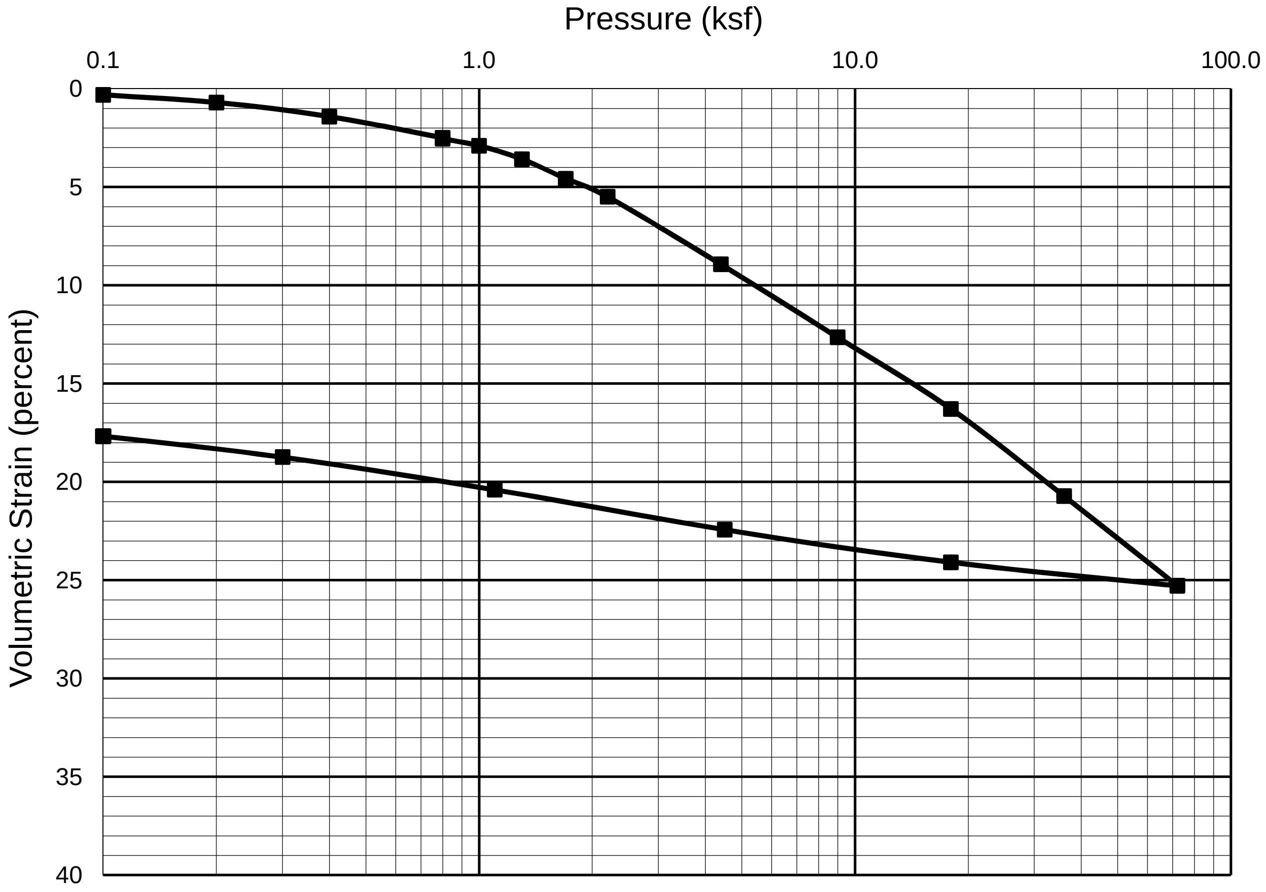
CLASSIFICATION CHART FOR CONE PENETRATION TESTS

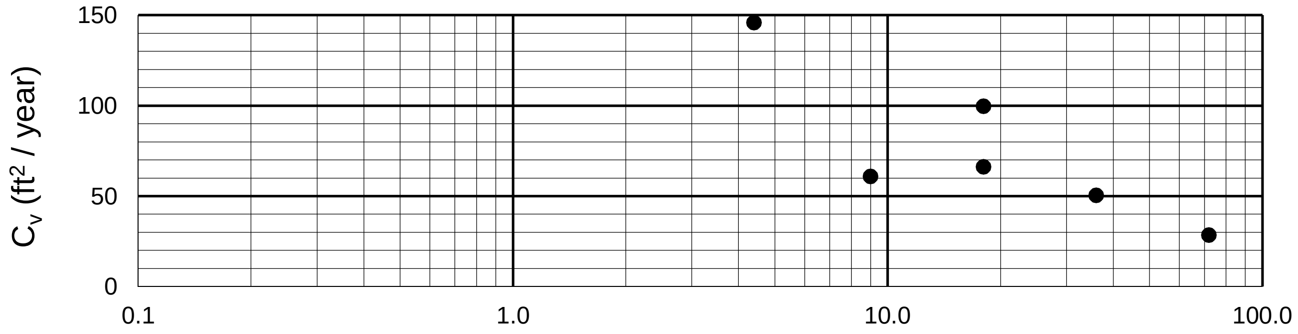
Date 07/14/16	Project No. 750633001	Figure B-10

SARIMIENTO

APPENDIX C

LABORATORY TEST RESULTS





Sampler Type: Shelby Tube			Condition	Before Test			After Test		
Diameter (in)	2.42	Height (in) 1.00	Water Content	w_o	30.4	%	w_f	18.9	%
Overburden Pressure, p_o	2,200	psf	Void Ratio	e_o	0.83		e_f	0.51	
Preconsol. Pressure, p_c	3,000	psf	Saturation	S_o	98	%	S_f	100	%
Compression Ratio, $C_{\varepsilon c}$	0.17		Dry Density	γ_d	92	pcf	γ_d	112	pcf

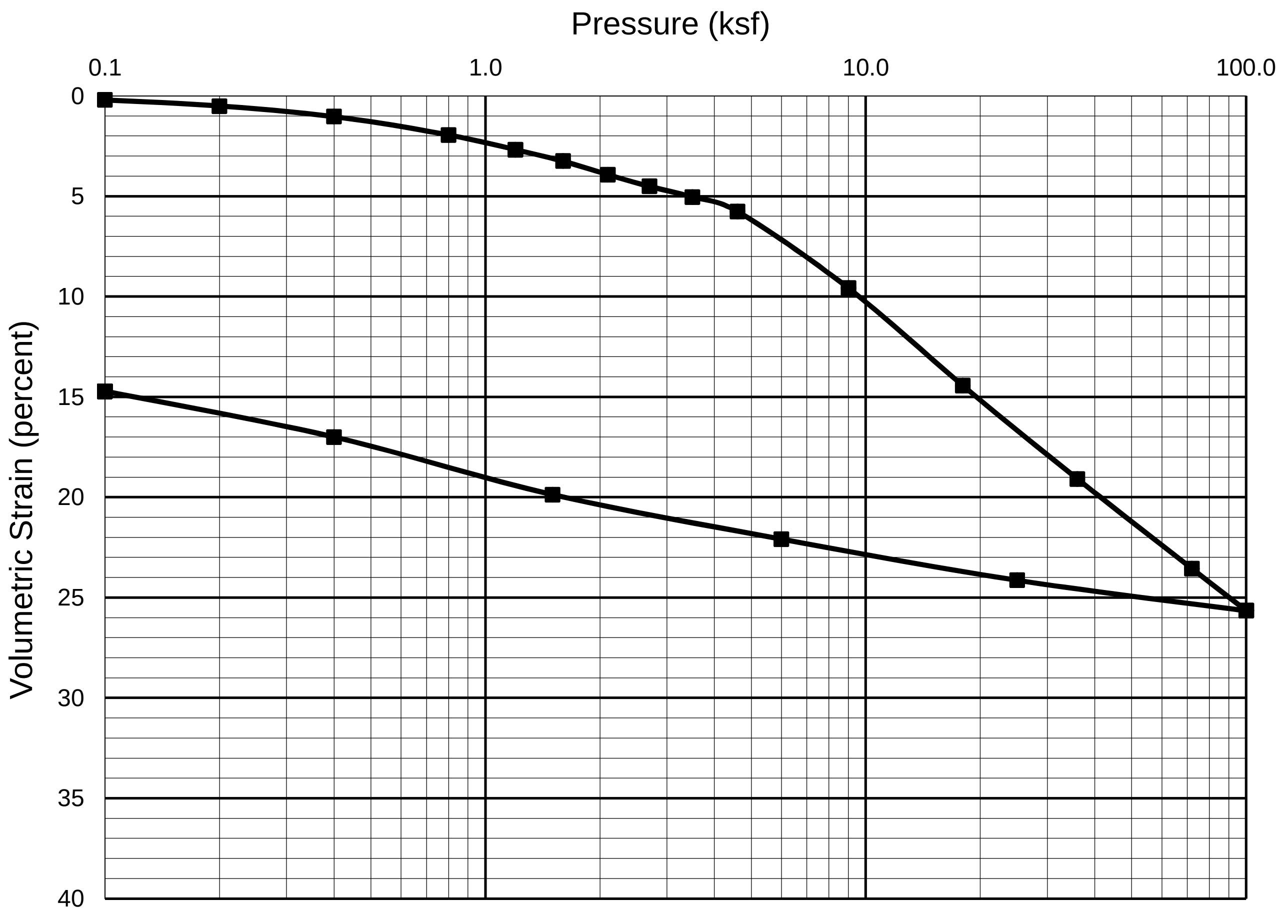
LL - -	PL - -	PI - -	G_s 2.70	(assumed)
Classification CLAY (CL), olive-gray		Source	B-1 at 20 feet	

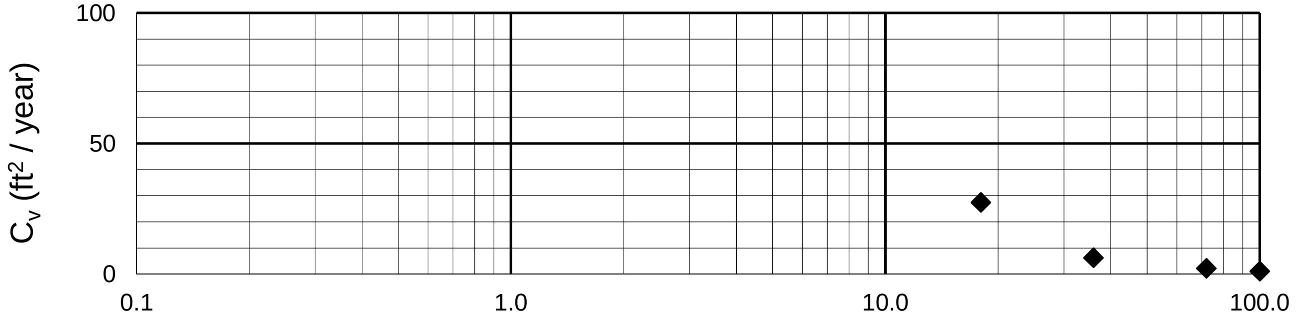
BISHOP RANCH - BR3A
San Ramon, California

LANGAN TREADWELL ROLLO

CONSOLIDATION TEST REPORT

Date	07/12/16	Project No.	750633001	Figure C-1





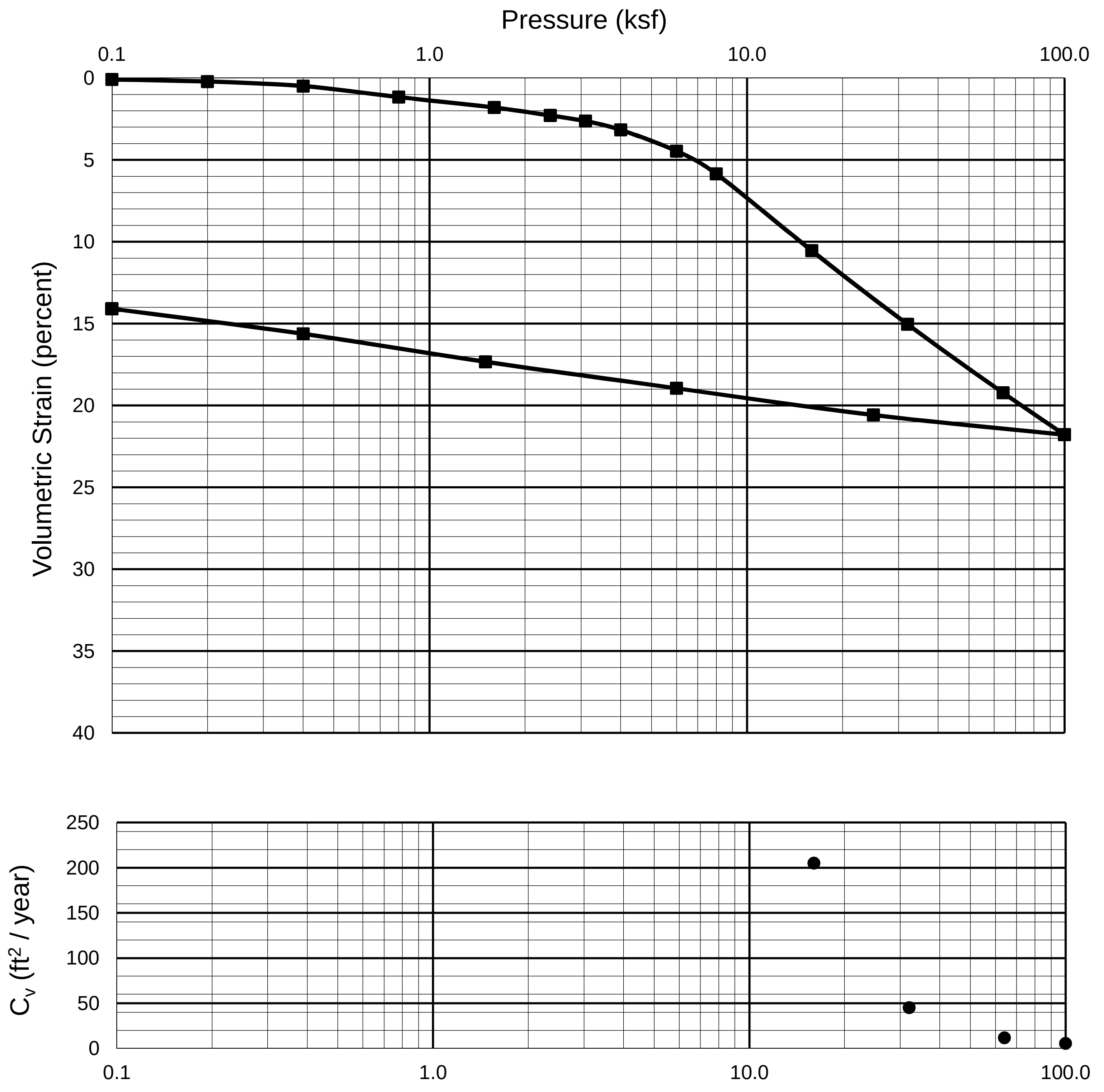
Sampler Type:	Shelby Tube		Condition		Before Test		After Test
Diameter (in)	2.42	Height (in) 1.00	Water Content	w_o	31.0 %	w_f	23.7 %
Overburden Pressure, p_o	2,400	psf	Void Ratio	e_o	0.92	e_f	0.64
Preconsol. Pressure, p_c	5,500	psf	Saturation	S_o	91 %	S_f	100 %
Compression Ratio, $C_{\varepsilon c}$	0.17		Dry Density	γ_d	88 pcf	γ_d	103 pcf
LL - -		PL - -	PI - -			G_s	2.70 (assumed)
Classification: CLAY (CL), yellow-brown			Source		B-8 at 25 feet		

BISHOP RANCH - BR3A
San Ramon, California

LANGAN TREADWELL ROLLO

CONSOLIDATION TEST REPORT

Date 07/12/16 Project No. 750633001 Figure C-2



Sampler Type: Shelby Tube				Condition	Before Test			After Test		
Diameter (in)	2.42	Height (in)	1.00	Water Content	w_o	24.0	%	w_f	17.4	%
Overburden Pressure, p_o	2,000	psf		Void Ratio	e_o	0.71		e_f	0.47	
Preconsol. Pressure, p_c	5,500	psf		Saturation	S_o	91	%	S_f	100	%
Compression Ratio, $C_{\varepsilon c}$	0.15			Dry Density	γ_d	99	pcf	γ_d	115	pcf

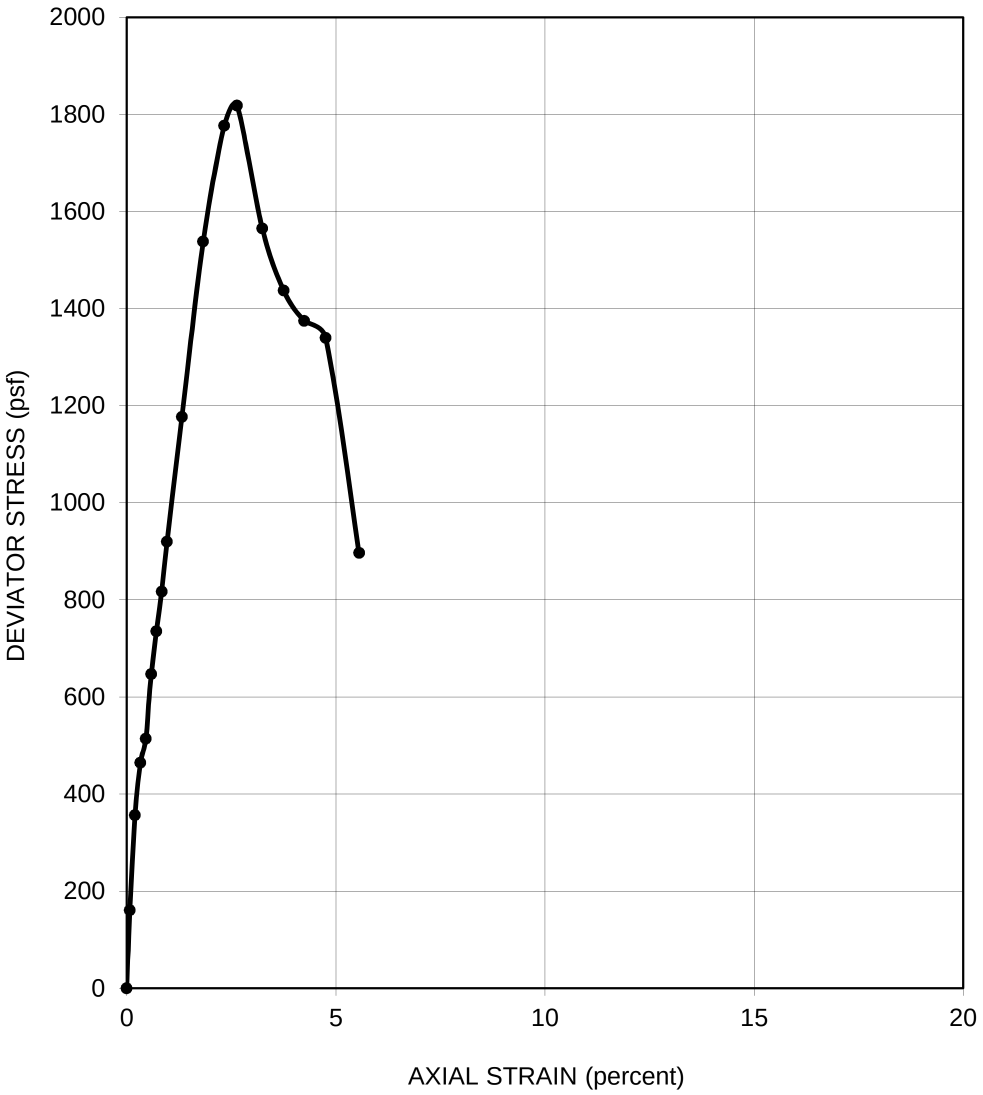
LL	- -	PL - -	PI	- -	G_s 2.70	(assumed)
Classification	CLAY (CL), yellow-brown		Source	B-12 at 17.5 feet		

BISHOP RANCH - BR3A
San Ramon, California

LANGAN TREADWELL ROLLO

CONSOLIDATION TEST REPORT

Date	07/12/16	Project No.	750633001	Figure C-3



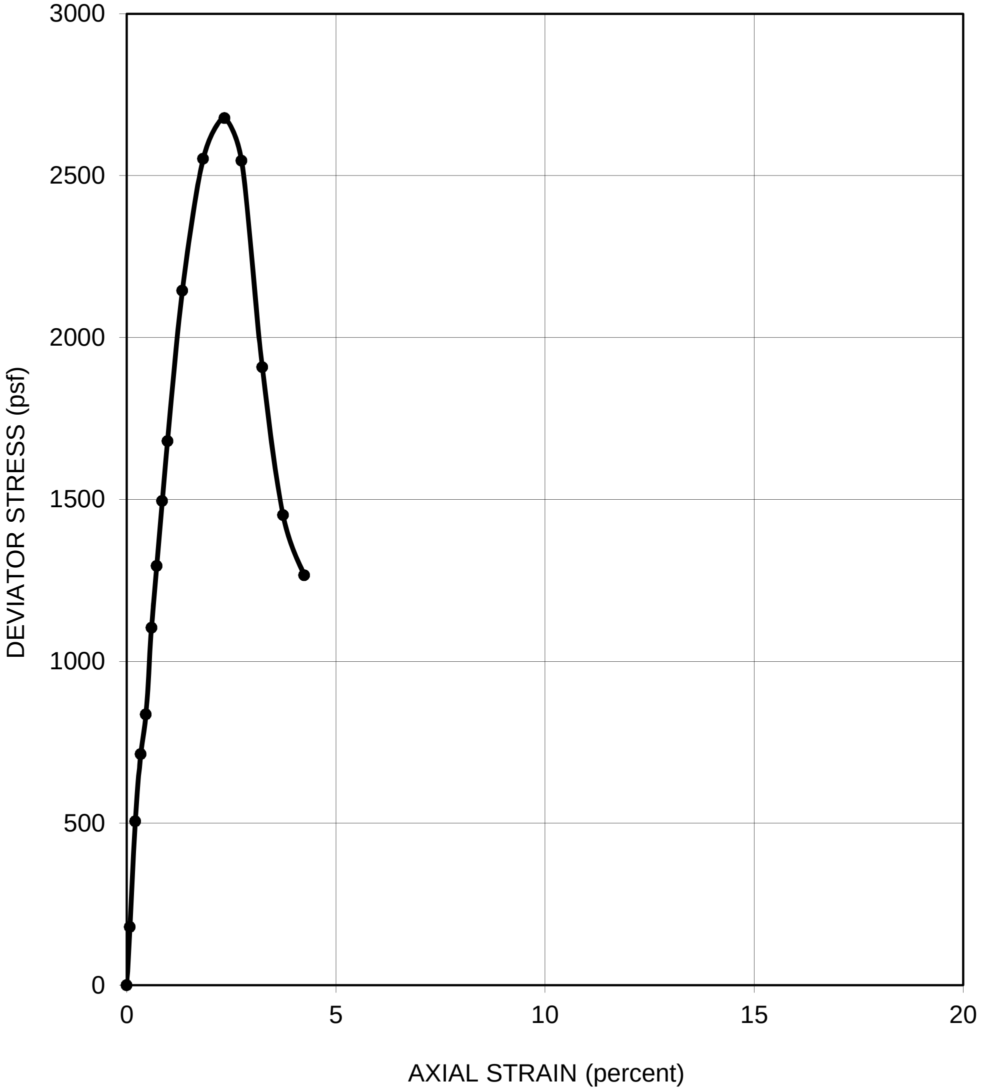
SAMPLER TYPE	Shelby Tube			SHEAR STRENGTH	910	psf
DIAMETER (in.)	2.85	HEIGHT (in.)	6.1	STRAIN AT FAILURE	2.6	%
MOISTURE CONTENT		18.9	%	CONFINING PRESSURE	2,000	psf
DRY DENSITY		112	pcf	STRAIN RATE	0.75	% / min
DESCRIPTION	CLAY (CL), olive-gray				SOURCE	B-1 @ 20 feet

BISHOP RANCH - BR3A
San Ramon, California

LANGAN TREADWELL ROLLO

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 07/12/16 Project No. 750633001 Figure C-4



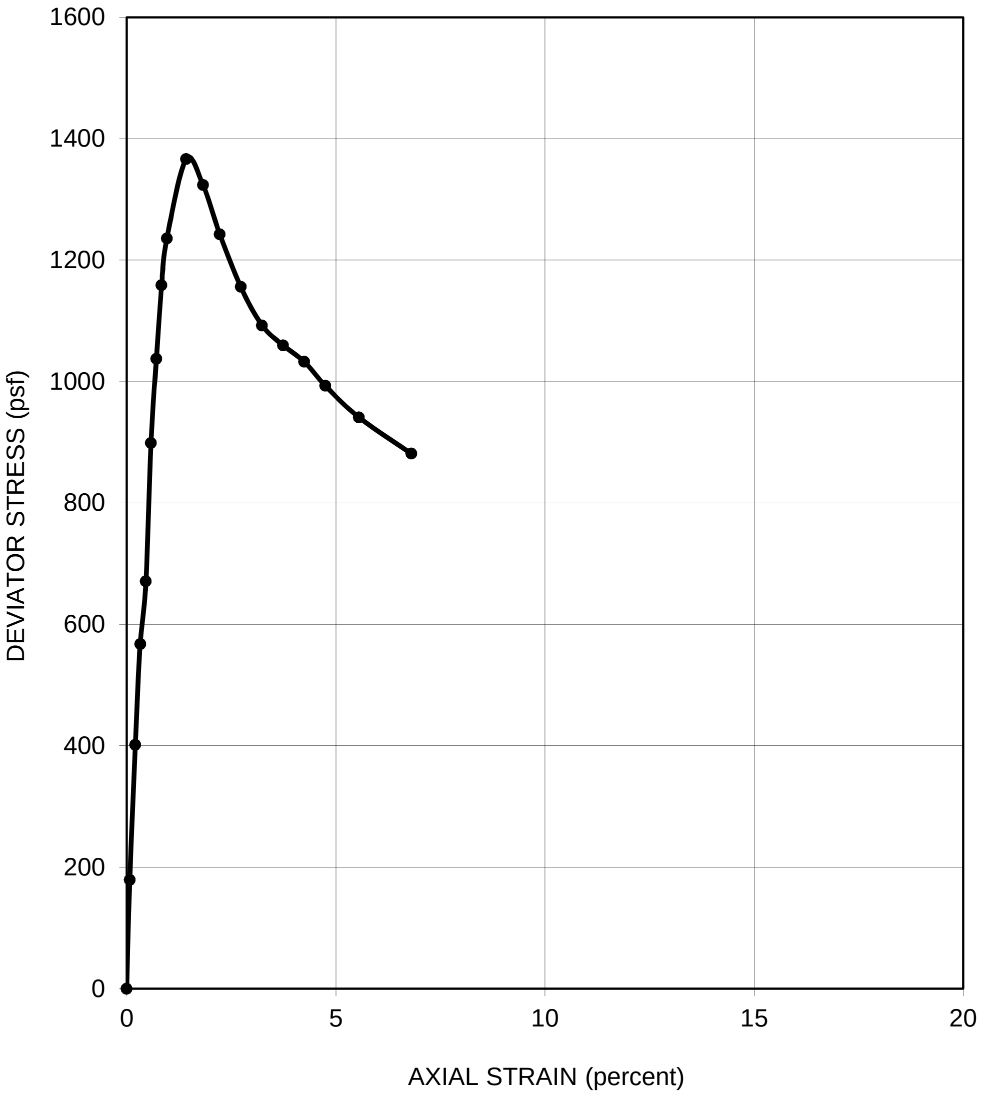
SAMPLER TYPE	Sprague and Henwood			SHEAR STRENGTH	1,340	psf
DIAMETER (in.)	2.40	HEIGHT (in.) 5.41		STRAIN AT FAILURE	2.3	%
MOISTURE CONTENT		15.3	%	CONFINING PRESSURE	900	psf
DRY DENSITY		107	pcf	STRAIN RATE	0.50	% / min
DESCRIPTION	SANDY CLAY (CL), olive-gray				SOURCE	B-4 @ 8.5 feet

BISHOP RANCH - BR3A
San Ramon, California

LANGAN TREADWELL ROLLO

UNCONSOLIDATED-UNDRAINED
TRIAXIAL COMPRESSION TEST

Date 07/12/16 | Project No. 750633001 | Figure C-5



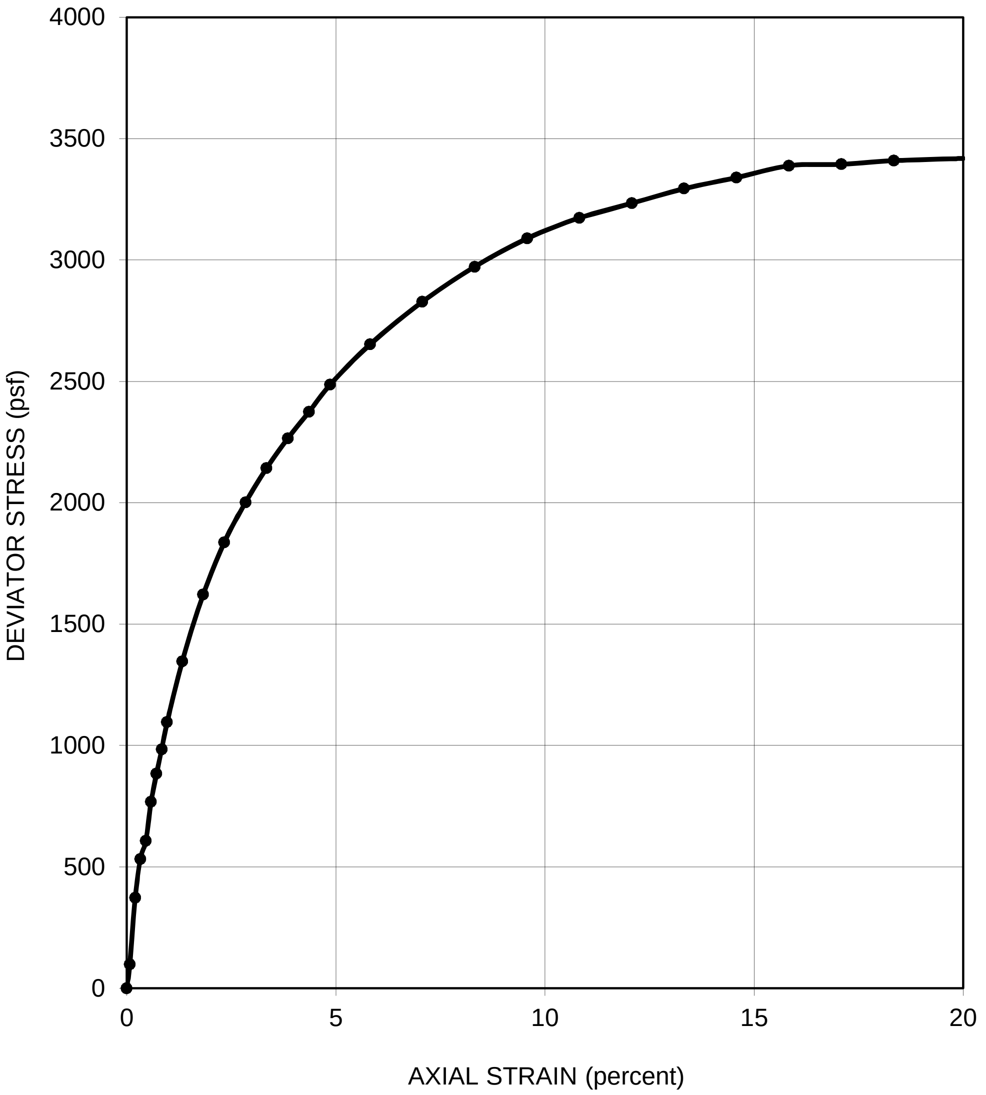
SAMPLER TYPE	Shelby Tube			SHEAR STRENGTH	680	psf
DIAMETER (in.) 2.85	HEIGHT (in.) 6.08			STRAIN AT FAILURE	1.4	%
MOISTURE CONTENT		29.5	%	CONFINING PRESSURE	1,800	psf
DRY DENSITY		94	pcf	STRAIN RATE	0.75	% / min
DESCRIPTION	CLAY (CL), yellow-brown				SOURCE	B-12 @ 17.5 feet

BISHOP RANCH - BR3A
San Ramon, California

LANGAN TREADWELL ROLLO

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 07/12/16 | Project No. 750633001 | Figure C-6



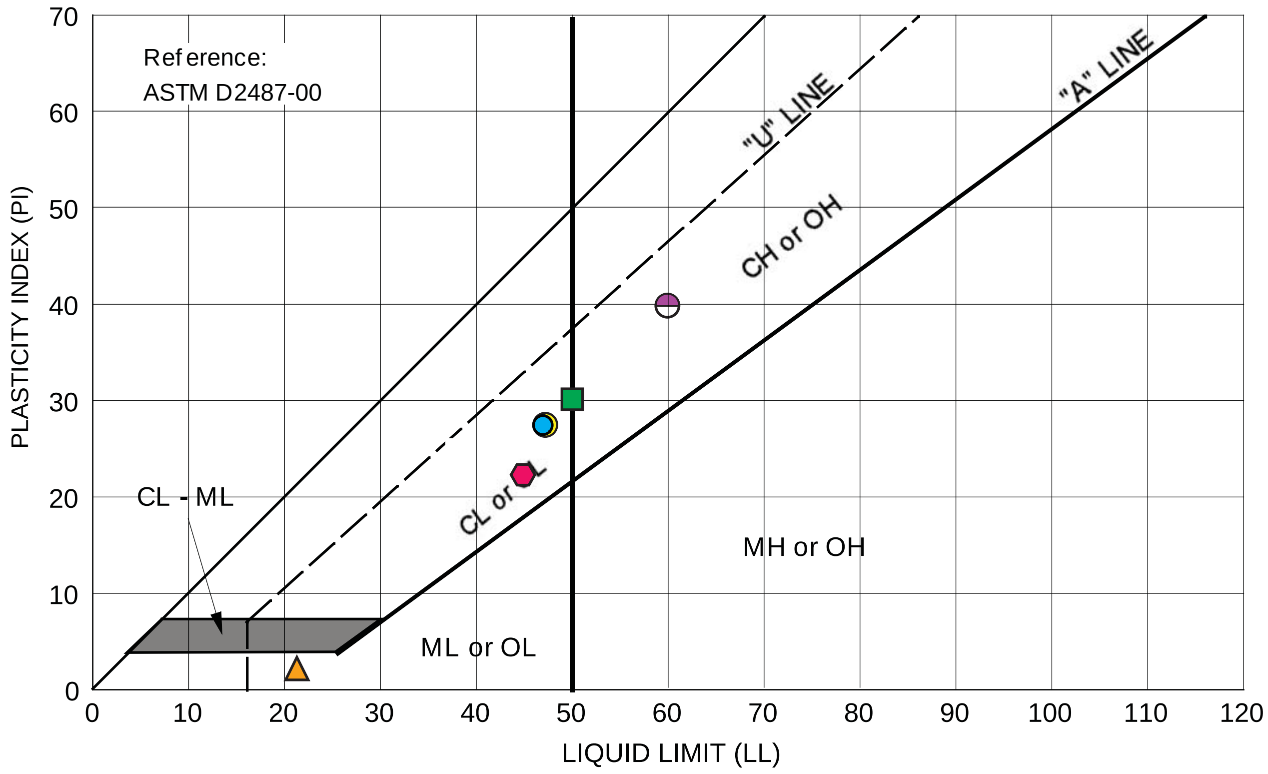
SAMPLER TYPE	Sprague and Henwood		SHEAR STRENGTH	1,710	psf
DIAMETER (in.) 2.40	HEIGHT (in.) 5.62		STRAIN AT FAILURE	20.0	%
MOISTURE CONTENT	25.4	%	CONFINING PRESSURE	2,600	psf
DRY DENSITY	101	pcf	STRAIN RATE	0.75	% / min
DESCRIPTION	CLAY (CL), olive-gray		SOURCE	B-14 @ 26 feet	

BISHOP RANCH - BR3A
San Ramon, California

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

LANGAN TREADWELL ROLLO

Date 07/12/16 | Project No. 750633001 | Figure C-7



NP = Non Plastic

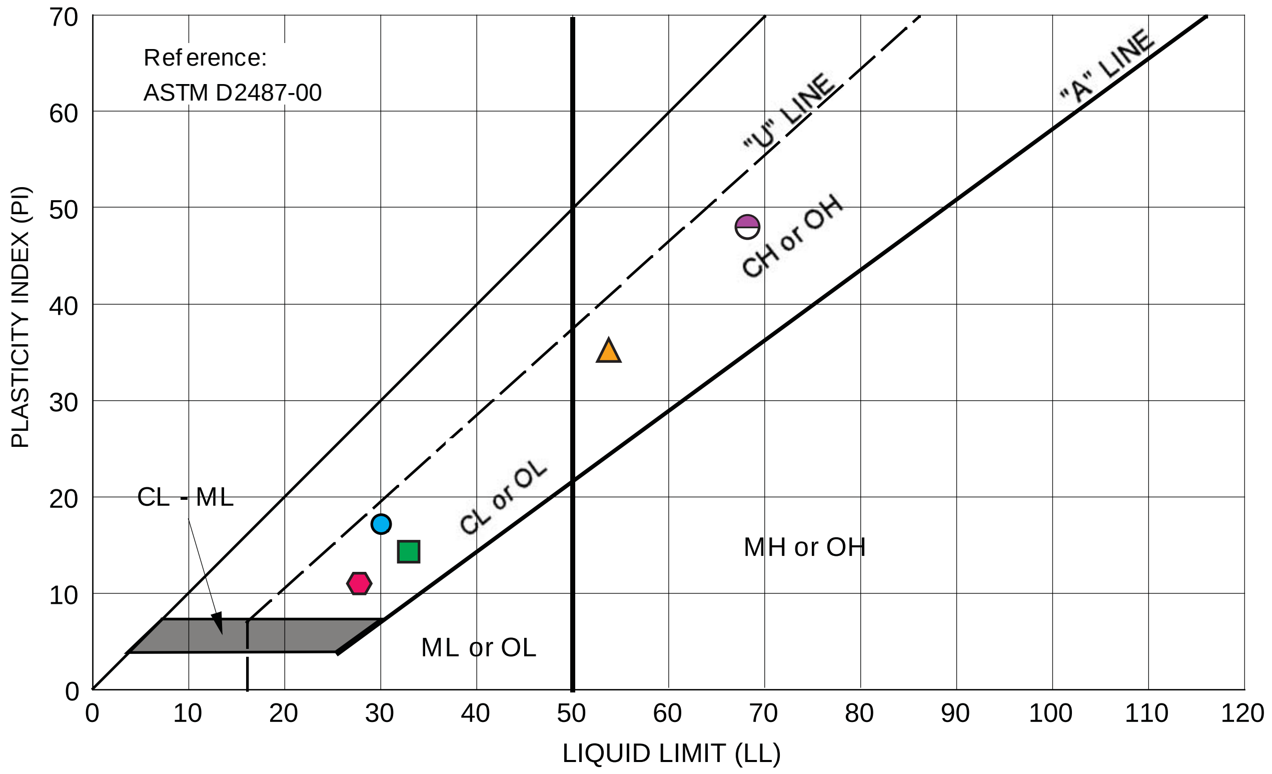
Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-1 to 3.5 feet	CLAY with SAND (CL), dark brown	19.2	47	28	--
▲	B-1 at 15 feet	SILTY SAND (SM), olive-gray	18.7	21	3	30.3
■	B-4 at 3 feet	CLAY with SAND (CH), dark brown with olive-gray mottling	--	50	30	--
⬢	B-6 at 1.5 feet	CLAY with SAND (CL), dark brown with olive-gray mottling	--	45	23	--
◓	B-7 at 3.5 feet	CLAY (CH), dark brown	22.7	60	40	--
◐	B-8 at 3.5 feet	CLAY with SAND (SC), dark brown	--	47	28	--

BISHOP RANCH - BR3A
San Ramon, California

LANGAN TREADWELL ROLLO

PLASTICITY CHART

Date 07/12/16 | Project No. 750633001 | Figure C-8



NP = Non Plastic

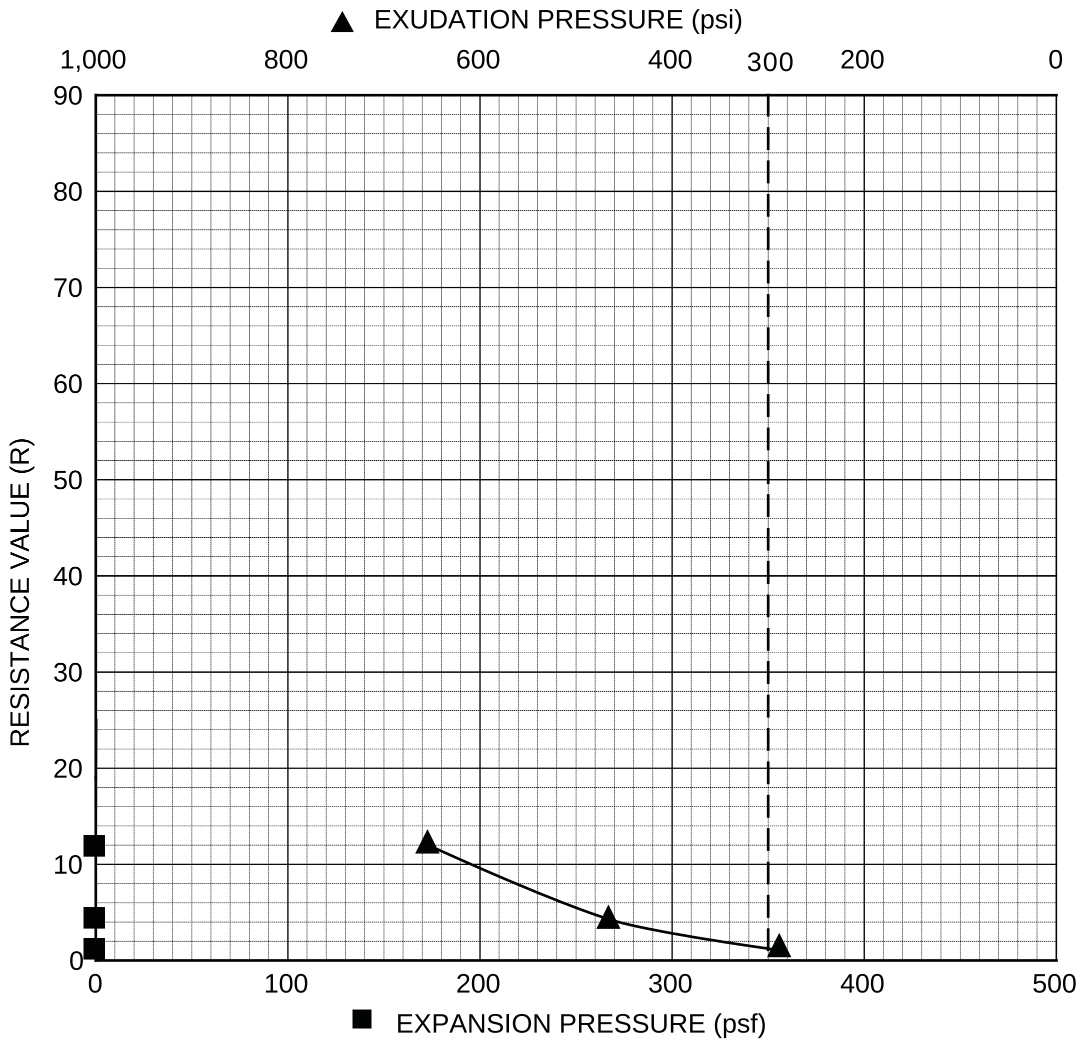
Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-8 to 40 feet	CLAYEY SAND (SC), olive-gray with dark brown mottling	16.5	30	16	35.9
▲	B-12 at 3.5 feet	CLAY with SAND (CH), dark brown	--	54	35	--
■	B-12 at 8.5 feet	CLAY (CL), yellow-brown	--	34	14	--
⬢	B-13 at 13.5 feet	CLAYEY SAND (SC), olive-gray	15.4	28	11	39.2
◓	B-14 at 3.5 feet	CLAY with SAND (CH), dark brown	--	69	49	--
						--

BISHOP RANCH - BR3A
San Ramon, California

LANGAN TREADWELL ROLLO

PLASTICITY CHART

Date 07/12/16 | Project No. 750633001 | Figure C-9



Specimen ID:	A	B	C	D
Water Content (%)	24.4	22.7	19.6	--
Dry Density (pcf)	93.8	99.7	104.7	--
Exudation Pressure (psi)	296	464	657	--
Expansion Pressure (psf)	0	0	0	--
Resistance Value (R)	1	4	12	--

Sample Source	Sample Description	Sand Equivalent	Exudation Pressure (psi)	R value
B-13 at 0-2 feet	CLAY with SAND (CH), dark brown	--	300	1

BISHOP RANCH - BR3A
San Ramon, California

LANGAN TREADWELL ROLLO

RESISTANCE VALUE TEST DATA

Date 07/12/16 | Project No. 750633001 | Figure C-10

APPENDIX D

CORROSION TEST RESULTS

California State Certified Laboratory No. 2153

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

23 May, 2016

Job No. 1605101
Cust. No.11308

Ms. Elena Ayers
Langan Treadwell Rollo
501 14th Street, 3rd Floor
Oakland, CA 94612

Subject: Project No.: 750633001.700.3
Project Name: Bishop Ranch B3A
Corrosivity Analysis – ASTM Test Methods

Dear Ms. Ayers:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on May 12, 2016. Based on the analytical results, a brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, sample 001 is classified as "corrosive" and 002 is classified as "moderately corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentrations are none detected to 15 mg/kg

The sulfate ion concentrations range from none detected to 25 mg/kg and are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The pH of the soils ranged from 7.18 to 7.30, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials are both 420-mV which is indicative of aerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call JDH Corrosion Consultants, Inc. at (925) 927-6630.

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

Cheryl McMillen for

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

California State Certified Laboratory No. 2153

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

Client: Langan Treadwell Rollo
Client's Project No.: 750633001.700.3
Client's Project Name: Bishop Ranch B3A
Date Sampled: Not Indicated
Date Received: 12-May-16
Matrix: Soil
Authorization: Chain of Custody

Date of Report: 20-May-2016

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1605101-001	B-2 2 @ 3.5'	420	7.30	-	1,300	-	N.D.	25
1605101-002	B-7 1 @ 3'	420	7.18	-	2,400	-	N.D.	N.D.

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	18-May-2016	18-May-2016	-	16-May-2016	-	18-May-2016	18-May-2016

* Results Reported on "As Received" Basis

N.D. - None Detected

Cheryl McMillen
Laboratory Director

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

DISTRIBUTION

Electronic copy:

Mr. Alexander Mehran
Sunset Development Company
2600 Camino Ramon, Suite 201
San Ramon, California 94583

QUALITY CONTROL REVIEWER

Lori A Simpson

Lori A Simpson, G.E.
Principal

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Final Geotechnical Investigation
Bishop Ranch City Center Project
Parcel 1 & 1A
San Ramon, California

Prepared for

Sunset Development Company
San Ramon, California

MACTEC Project No. 4096088527

Rambod Hadidi
Rambod Hadidi
Senior Engineer

Donald W Quigley
Donald W. Quigley, PE, GE
Senior Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
DONALD WALKER QUIGLEY
703
EXP 9-30-09
GEOTECHNICAL
STATE OF CALIFORNIA

October 9, 2008

MACTEC Engineering and Consulting, Inc.
28 Second Street, Suite 700
San Francisco, CA 94105 - (415) 543-8422

Final Geotechnical Investigation
Bishop Ranch City Center Project
Parcel 1& 1A
San Ramon, California

MACTEC Project No. 4096088527

This document was prepared by MACTEC Engineering and Consulting, Inc. (MACTEC) at the direction of Sunset Development Company for the sole use of Sunset Development Company and their consultants, the only intended beneficiaries of this work. No other party should rely on the information contained herein without the prior written consent of MACTEC. This report and the interpretations, conclusions, and recommendations contained within are based in part on information presented in other documents that are cited in the text. Therefore, this report is subject to the limitations and qualifications presented in the referenced documents.

Our professional services have been performed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. No other warranty, expressed or implied, is made as to the professional advice included in this report. If any of the project information provided to MACTEC has changed, we should be notified so that we may amend our recommendations as necessary.

CONTENTS

TABLES

PLATES

APPENDICES

1.0 INTRODUCTION

This report presents the results of MACTEC Engineering and Consulting, Inc.'s (MACTEC) final geotechnical investigation for the planned office buildings and parking garages on Parcel 1 & 1A at Bishop Ranch in San Ramon, California, as shown on Vicinity Map, Plate 1-1, and Site Plan, Plate 1-2. This investigation was performed for Sunset Development Company (Sunset).

MACTEC previously performed a preliminary geotechnical investigation for the project, as described in our report *Preliminary Geotechnical Investigation Report, San Ramon City Center Project, Bishop Ranch, San Ramon, California*, dated July 24, 2007. As described in our 2007 report, MACTEC evaluated subsurface soil and groundwater conditions at the project site by reviewing previous geotechnical reports by MACTEC and others at and near the site. From these documents, we developed geotechnical conclusions and preliminary recommendations for planning of the proposed development. We also recommended that additional subsurface investigations be performed to confirm and/or augment the site data available from previous investigations and/or to support the design requirements of the project teams. This report summarizes the results of the additional investigations and provides final recommendations for the design of the project.

1.1 Project Description

As shown on Plate 1-2, we understand that the planned development on Parcel 1 & 1A will consist of: 1) three, similar, 6-story office buildings, with approximate footprint areas of 33,000 square feet each; 2) a large 5-story parking garage (approximately 125,000 square feet in plan area) with a small café building attached; 3) a smaller 5-story parking structure (approximately 40,000 square feet in plan area); 4) an entry plaza and fountain; and 5) new roads and at-grade parking lot spaces. Based on our correspondence with project structural engineer, Middlebrook & Louie, the office building column loads (dead plus live) are estimated to be about 1,320 kips for interior columns and 325 kips for exterior columns. The parking garage column loads (dead plus live) range from 770 to 1,750 kips for interior columns and 1,000 to 1,600 kips for exterior columns. None of the buildings will have basements.

The planned building site includes vacant land and existing landscaping areas and surface parking lots. The property, in general, slopes very gently towards the southwest with elevations ranging from about 440 (Mean Sea Level) at the northeastern corner of Parcel 1A to 426 at the southwestern corner of

Parcel 1. The finished ground floor elevations of the three office buildings will range from 438 to 439. At the parking garages, the finished ground floor level will slope from 436 to 431 at the larger parking garage and from 429 to 427 at the smaller parking garage. The proposed locations of the office buildings are currently primarily vacant land. Surface parking lots with minimal landscaping features, cover the proposed location of the parking garages. Site grading for construction of the parking garage pads is anticipated to be minor. Up to about 4 feet of fill and cut will be needed for grading the office building pads. The fill and cut volumes are estimated to be approximately 42,000 and 287,000 cubic yards, respectively (*Sunset Development Company, Conceptual Grading Plan – Business Complex, San Ramon City Center, San Ramon, California, dated April 30, 2007*).

We understand that the project will be constructed in three phases, each one year apart. Phase I will consist of site grading, including building pads for all planned structures, and construction of the northwest office building and the larger parking garage concurrently. Phase II will consist of constriction of the northeast office building and smaller parking garage concurrently. Phase III will be construction of the remaining office building.

1.2 Scope of Services

The scope of our services, as stated in our proposal dated July 14, 2008, included supplementing the existing boring data by drilling test borings, performing Cone Penetration Tests (CPTs), conducting laboratory tests, performing geotechnical engineering analyses, and developing recommendations for final design of the project. As listed in our proposal, the site information was used to develop conclusions and recommendations for the following:

- Subsurface soil and groundwater conditions;
- Site preparation and minor grading, including fill and backfill compaction criteria;
- Subgrade preparation for concrete slab-on-grades and asphalt concrete pavements;
- Pavement section thicknesses;
- Geotechnical engineering design criteria for use in foundation design, including soil bearing values and resistance to lateral loads for shallow foundations;
- Design criteria for 14-inch by 14-inch precast, prestressed, driven concrete piles; including axial load capacities (downward and uplift), lateral capacities based on embedment depths and fixity, and factors of safety used and allowances for drag loads.

- Estimated amounts of ground and foundation settlements;
- Assessment of potential geohazard risks associated with seismicity, including liquefaction potential, seismically-induced settlement, and fault rupture; and
- Appropriate seismic factors for structural design input in accordance with the California Building Code (2007). No site-specific response analyses were to be provided.

Our services did not include an assessment for the presence of potentially toxic and hazardous material on or beneath the site.

2.0 METHODS OF INVESTIGATION

2.1 Review of Previous Data

A variety of published and unpublished sources were reviewed to evaluate geotechnical data and geologic hazards relevant to the site. Appropriate maps that were reviewed included topographic maps, geologic maps, and fault maps by the United States Geological Survey and the California Geological Survey (previously known as the California Division of Mines and Geology). We also reviewed the following geotechnical reports (or excepts thereof) for the project site and vicinity, several of which were prepared by MACTEC (then known as Harding Lawson Associates [HLA]):

- MACTEC, *Preliminary Geotechnical Investigation Report; San Ramon City Center; Bishop Ranch; San Ramon, California,* Prepared for Sunset Development Company, MACTEC Project 4096075707, dated July 24, 2007.
- Kleinfelder, *Geotechnical Investigation at Chevron/Texaco Campus Lots 16, 20 and 21 of the Bishop Ranch Business Park, San Ramon, California*; prepared for Watry Design; Kleinfelder Project 53512/Geo; dated June 9, 2005.
- ENGEO, *Preliminary Geotechnical Exploration, San Ramon City Center, San Ramon, California*, prepared for City of San Ramon, California, ENGEO Project 5172.001.01, dated March 29, 2001.
- HLA, *Geotechnical Investigation, Bishop Ranch 1 Development, San Ramon, California*, prepared for Sunset Development Company, HLA Project 50044.1, dated May 15, 2000.
- HLA, *Geotechnical Investigation, Bishop Ranch 1 Development, Bishop Ranch Business Park, San Ramon, California*, prepared for Sunset Development Company, HLA Project 8294,019.03, dated October 6, 1986.
- HLA, *Soil Investigation, Bollinger Business Center, Bishop Ranch, San Ramon, California*, prepared for Sunset Development Company, HLA Project 8294,009.03, dated April 6, 1982.
- Kleinfelder, *Geotechnical Investigation Report, Chevron Park, San Ramon, California,* dated June, 1981.

Copies of relevant boring log and CPT results from the previous reports are presented in Appendix A.

We have also reviewed the following project information provided by Sunset:

- Sunset Development Company, *Conceptual Grading Plan – Business Complex, San Ramon City Center, San Ramon, California*, dated April 30, 2007.

- David Evans and Associates, Inc., *Bishop Ranch 1 Grading Plan, San Ramon, Contra Costa County, California*, prepared for Sunset Development Company, dated June 22, 2000.

Information from these documents was used to plan our subsurface investigation for this project and to assist us in preparing this report.

2.2 Field Exploration

2.2.1 Borehole Clearance

On August 7, 2008, prior to the start of our field investigation, our subcontractor, Advanced Geological Services, Inc. (AGS), performed a geophysical survey to mark the boring and CPT locations and identify buried utilities nearby. We also notified Underground Service Alert (USA) and marked the approximate exploration locations with paint for utility clearance, as required by law. Additionally, we obtained a drilling permit from the Contra Costa County Environmental Health Division.

2.2.2 Test Borings

We explored the subsurface conditions by drilling test borings on August 18 - 20, 2008. The approximate locations of these borings are shown on the Site Plan, Plate 1-2. Soil and classification criteria (Plates B-1 and B-2) and the logs of the borings (Plates B-3 through B-11) are presented in Appendix B.

Our drilling subcontractor, Gregg Drilling, drilled borings B-1, B-8, and B-9 to depths of 70 feet with a truck-mounted drill rig equipped with 6-inch diameter hollow-stem augers. Borings B-2 through B-7 were drilled by a hand auger to a depth of 5 feet. Borings were drilled with Level D personal protective equipment and soil cuttings generated during drilling were placed in 55-gallon drums, transported to the empty lot adjacent to Bishops Ranch One East and Bollinger Canyon Road, and were spread on the ground.

Our field engineer observed the drilling of the borings and logged the soil strata encountered. Grab samples were obtained from auger cuttings in the upper five feet of each boring. Deeper samples were obtained at regular intervals in the hollow–stem auger borings using either a Sprague and Henwood (S&H) split barrel sampler (3.0-inch outside diameter, 2.43-inch inside diameter), lined with 6-inch long brass tubes, or a 2-inch outside diameter Standard Penetration Test (SPT) sampler. The S&H and SPT samplers were driven by a 140-pound downhole hammer falling 30 inches using the automatic trip method. The number of blows required to drive the samplers the final 12 inches of an 18-inch drive was recorded. The observed S&H blow counts were converted to approximate SPT N-values [1]. N-values are shown on the boring logs. Borings B-2 through B-7 were backfilled with soil cuttings and capped with asphalt. Borings B-1, B-8, and B-9 were tremie backfilled with bentonite cement grout upon completion. Approximate ground elevations at each boring were determined from survey plans provided to us by Sunset and are shown on boring logs.

2.2.3 Cone Penetration Tests

Our subcontractor, Gregg Drilling, pushed seven CPTs to 64 to 70 feet below ground surface on August 18 and 19, 2008. The approximate location of the CPTs is shown on Plate 1-2. Upon completion of the CPTs, the CPT holes were grouted with bentonite cement grout. Level D personal protective equipment was used during CPT testing. The results of the CPT investigation are presented in Appendix C. Approximate ground elevations at each CPT were determined from plans provided to us by Sunset and are summarized in Appendix C.

2.3 Laboratory Testing

We re-examined the soil samples from the borings in our office to check field classifications and to select samples for laboratory testing. Laboratory tests performed included moisture content and dry density, consolidation, sieve analysis (#-200 sieve fraction), Atterberg limits, and strength tests. Results of these tests are summarized on the boring logs in accordance with the key on Plate B-1. Complete test reports are presented in Appendix D.

---

[1] The SPT N-value is defined as the number of blows of a 140-pound hammer, falling freely through the height of 30 inches, required to drive a standard split-barrel sampler (2-inch outside diameter and 1-3/8-inch inside diameter) for the last 12 inches of an 18-inch drive. For SPT procedures, see ASTM D1586-84.

3.0 SITE AND SUBSURFACE CONDITIONS

3.1 Site Conditions

The project site includes vacant land, landscaping areas and surface parking lots. Parcel 1 includes at-grade, asphalt-paved parking area, with minimal landscape areas. The parcel is bounded on the north side by the by parking lots of Parcel 1A. The Bishop Ranch One East access roadway bounds the east and south sides of Parcel 1. The Bishop Ranch One access roadway bounds the west side. Multi-story office structures are located on the adjacent land to the west.

Parcel 1A includes a vacant, with elevations ranging from 435 to 445. Ground cover includes uncultivated, annual and perennial vegetation, with some shrubbery at the northern portion of the parcel and at-grade, asphalt-paved parking area, with minimal landscapes at the southern part of the parcel. Bollinger Canyon Road bounds the north side of Parcel 1A. The Bishop Ranch One East access roadway bounds the east edge. Parcel 1 is located to the south. The Bishop Ranch One access roadway bounds the west edge.

3.2 Subsurface Conditions

The subsurface conditions at the site are presented graphically on Cross-Sections A-A' to C-C' (Plates 3-1 through 3-3), which summarize boring and CPT data from the current and previous geotechnical investigations at and near the site.

The subsurface conditions in the project area are interpreted to be relatively uniform. Expansive clay soils blanket most of the site and extend to at least 5 feet below the ground surface, and to as much as 10 feet in some locations. The ENGEO (*2001*) report indicated that possibly as much as 10 feet of fill soil had been placed within areas of the current Parcel 1A at the vacant lots. We have also encountered as much as 5 feet of fill in the vacant lot that northern portion of the Parcel 1A. The fill soil was reported to have been excavated from nearby parcels during construction activities. At the boring locations, the fill soils were silty or clayey sands or low to moderately expansive clays.

The stiff-to-hard native clay and fill surface soils overlie moderately compressible clays and silts to depths extending to about 30 feet to 40 feet below grade. Deeper clays are generally stiff to very stiff with inclusions of relatively strong alluvial gravels, sands, with various fine contents to the depths explored (about 75 feet maximum). These gravel and sands are not continuous and their thickness, depth varies significantly within the site.

Groundwater has been encountered as shallow as 7 feet below the site grade during previous exploration, but has varied to as deep as 20 feet in some locations during drilling. Ground water was encountered at 13 to 20 feet below the site grade during our current borings. A pore water pressure dissipation test at CPT-06 indicated that the water table was 11 feet below the ground surface.

4.0 GEOLOGY AND SEISMICITY

4.1 Geologic Setting

The site is located within the San Ramon Valley, a portion of the California Coastal Ranges geomorphic province (*California Geomorphic Provinces, Note 36, California Geological Survey*, revised *December 2002*). In general, the geologic structure and topography are characteristic of the San Francisco Bay Area. This region is generally defined by northwest-trending ridges and valleys that generally parallel the geologic structures, including the major fault systems. The San Ramon Valley fill includes quaternary-aged alluvium up to approximately 300 feet in thickness. The valley is drained by both North and South San Ramon Creeks that are actively cutting into the alluvial surface soils. Tertiary-aged sedimentary rocks comprise surrounding slopes and underlying valley geology.

4.2 Faults and Seismicity

The numerous faults in Northern California include active, potentially active, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (previously the California Division of Mines and Geology) for the Alquist-Priolo Earthquake Fault Zoning Program (*Hart, 1999*). By definition, an active fault is one that has had surface displacement within Holocene time (about the last 11,000 years). A potentially active fault is a fault that has demonstrated surface displacement of Quaternary age deposits (last 1.6 million years). Inactive faults have not moved in the last 1.6 million years.

The site is not within a currently-established Alquist-Priolo Earthquake Fault Zone for surface rupture hazards. The nearest active faults are the Calalveras fault, located about 0.6 miles to the west, and the Concord-Green Valley fault, located about 8 miles northeast. Significant faults near the project site and their characteristics are presented in Table 4-1.

4.3 Site Classification and Code Seismic Criteria

We have determined the site seismic parameters in accordance with the Section 1613A of the 2007 edition of the California Building Code (CBC) using the United States Geological Survey (*USGS, 2007*) program, Earthquake Ground Motion Parameters, Version 5.0.8. The site location used was Latitude 37.7610° and Longitude -121.9540° and our opinion is that the Site Class can be considered "D" (stiff soil profile). The mapped parameters and the site-specific design parameters are presented in the following table:

Item	Designation	Value
Site Class (Table 1613.5.2)	-	D
Mapped Spectral Acceleration for Short Periods (Figure 1613.5(1))	S_S	1.919g
Mapped Spectral Acceleration for 1-sec Period (Figure 1613.5(2))	S_1	0.713g
Site Coefficient (Table 1613.5.3(1))	F_a	1.0
Site Coefficient (Table 1613.5.3(2))	F_v	1.5
MCE* Spectral Response Acceleration for Short Periods	S_{MS}	1.919g
MCE* Spectral Response Acceleration for 1-sec Periods	S_{M1}	1.069g
Design Spectral Response Acceleration for Short Periods	S_{DS}	1.279g
Design Spectral Response Acceleration for 1-sec Periods	S_{D1}	0.713g

MCE: Maximum Considered Earthquake

5.0 DISCUSSIONS AND CONCLUSIONS

We conclude that the development of the planned new buildings is feasible from a geotechnical standpoint provided our conclusions and recommendations are incorporated into the design and construction of the project. The most significant geologic hazard to the site is the seismic ground shaking. The main geotechnical engineering considerations for this project are: 1) the presence of the expansive surface soils, and 2) presence of undocumented fill at some areas, and 3) presence of relatively compressible clays beneath the site.

The significant geologic hazards and geotechnical considerations are discussed in the following sections.

5.1 Geologic Hazards

5.1.1 Surface Fault Rupture

The nearest active faults are the Calalveras fault, located about 0.6 miles to the west, and the Concord-Green Valley fault, located about 8 miles northeast. Therefore, potential for surface fault rupture at the site is considered to be low.

5.1.2 Seismic Shaking

The primary geologic hazard at the site is the potential for strong earthquake shaking resulting from major displacements on nearby faults or other active faults in the region. The proposed office buildings and garages should be expected to experience periodic smaller earthquakes and possibly a large earthquake during their design lives. Seismic design criteria, based on the 2007 California Building Code, are presented in Section 4.3.

5.1.3 Liquefaction

Liquefaction is a phenomenon in which saturated, cohesionless soil can temporarily lose strength due to buildup of excess pore water pressure, especially during cyclic loading such as earthquake shaking. Soils most susceptible to liquefaction are loose, clean, saturated, uniformly graded, fine-grained sands. In some cases, as pore pressures dissipate, sand is forced to the surface in "boils" due to upward water flow resulting in surface settlement.

The subsurface data for the site indicate some saturated sand layers and lenses within the predominantly clayey soils at the site. These sand units are relatively thin, discontinuous, and/or contain appreciable concentrations of fine-grained materials. It is our interpretation that liquefaction potential at the site is limited, and that settlement caused by liquefaction would be relatively small (less than one inch).

5.1.4 Seismic Densification

Densification of unsaturated sandy soils above the ground water level subjected to earthquake shaking can cause settlement at the ground surface. However, because most near surface sand layers below the site are relatively thin, contain appreciable fine content, and appear to be interbedded with fine grained layers, it is our interpretation that settlement caused by soil densification would be negligible.

5.2 Geotechnical Considerations

Expansive Soils: Based on the results of borehole logging and laboratory testing performed for this investigation, the surficial clay soils (both native and fill soils) exhibit moderate expansion potential. These soils are anticipated to shrink and swell with fluctuations in moisture content. The volume changes in expansive soils can be effectively reduced by initially moisture conditioning them to a high moisture content (to cause expansion to occur before construction), and then blanketing with imported select fill having a low expansion potential. Alternatively, the existing upper zone of expansive clays may be lime-treated to reduce their expansion potential to that of a low or non-expansive imported select fill and reduce the quantity of select fill needed to be imported for the development.

Undocumented Fill: Some project areas have received fill soil apparently imported from nearby parcels undergoing development. These stockpiled material should be categorized, geotechnically, as an undocumented fill. The vertical and lateral extent of these soils are not known with precision and should be confirmed during construction. It is our opinion that such fill soils, where identified, will need to be excavated and recompacted or disposed offsite.

Compressible Clays: Beneath the upper native and fill clay soils, the underlying alluvial clays could undergo significant consolidation if they were loaded by shallow spread foundations beneath the heavy column loads of the proposed buildings. As a result, pile foundations that extend into deeper alluvial soils, below depths of 30 to 40 feet beneath the ground surface, should be used to support the buildings. This is consistent with past development within Bishop Ranch, where existing structures above three stories in height have been founded on driven pre-cast, prestressed concrete piles.

We have also considered supporting the structures on conventional spread and/or continuous footings founded on improved ground using Rammed Aggregate Piers (RAP). RAPs are a method of ground improvement, which is known as Geopiers, a propriety name of the Geopier Foundation Company. RAPs are constructed by drilling approximately 30-inch-diameter shafts, and replacing the excavated soil with crushed rock aggregate, placed and compacted in thin layers. The "replacement area" of the RAP shafts is typically about 35 percent of the footing area.

Our estimates for a typical RAP installation indicate that footings built over RAP-improved ground to a depth of 20 feet will settle as much as 3 to 4 inches under expected column loads. Therefore, we conclude that this alternate foundation option is not likely to be appropriate for this project. However, we have requested the Geopier Company to further evaluate this option and to determine if there is a cost-effective way to use Geopiers and still keep foundation settlements to acceptable amounts.

6.0 GEOTECHNICAL RECOMMENDATIONS

6.1 Earthwork

6.1.1 Site Preparation

In areas to be graded, the ground surface should be stripped of vegetation, soils containing organic matter, and other deleterious material (i.e., demolition debris, etc.). Existing asphalt pavements and utilities should be removed from the planned building areas. Stripped vegetation and organic matter should not be used as engineered fill, but could be used as landscaping material with the approval of the landscape architect. Debris should be disposed offsite.

Where site stripping exposes undocumented fill soils, they should be removed completely within and 5 feet beyond planned building areas and in other areas where ground settlement could be a concern (such as the entry plaza or fountain). In exterior areas to be graded, the existing fill soils should only be excavated as needed to allow compaction of the subgrade soils to a firm and unyielding conditions. If isolated zones of weak, saturated soils are encountered during grading, they should be removed to expose firm soils; this should be determined in the field by the geotechnical engineer.

6.1.2 Excavation Considerations

Temporary excavations must comply with current requirements of Cal-OSHA. Additionally, all cuts deeper than five feet should be sloped or shored. It is our opinion that temporary excavations can be sloped at 1(H):1(V) or flatter; however, it is the responsibility of the contractor to maintain safe and stable slopes or design and provide shoring during construction.

Excavations deeper than seven feet below the ground surface could encounter groundwater. Although groundwater inflows will probably not be large, because of the generally fine-grained nature of the site soils, the groundwater should be removed from the excavations to prevent softening of the excavation base to and to enable proper compaction of the subgrade and subsequent fills and backfills.

6.1.3 Subgrade Preparation

The subgrade beneath the site fills should be scarified to at least 8 inches, moisture conditioned to at least 2 to 4 percent over optimum moisture content and compacted to at least 90 percent relative compaction[2]. The moisture content of the fill should be maintained until covered by subsequent fill material. The need and extent of moisture conditioning will vary depending on the time of year that grading commences. During late spring and early summer, the site will be saturated or close to saturation and little or no moisture conditioning will be necessary. Therefore, this would be the optimum time for site grading. Later, during the dry season, shrinkage cracks may be several feet deep. Therefore, prolonged watering and flooding, and the possible use of wetting agents, could be necessary to pre-swell the soils and close shrinkage cracks for their full depth. Alternatively, the grading contractor may elect to overexcavate the drier expansive soils to facilitate moisture conditioning. Because the exact depth of required moisture conditioning will not be known until site grading actually commences, we suggest that the contract documents contain provisions for either alternative at the contractor's option. Alternatively, onsite expansive soils can be treated with high calcium quicklime or dolomite (magnesium) quicklime and properly cured to reduce their expansion potential.

6.1.4 Material for Fill and Placement Criteria

Select Fill material should consist of relatively non-expansive granular soils with a Liquid Limit of less than 40 and a Plasticity Index of less than 15. Material for Select Fill should not contain any cobbles or rock fragments larger than 4 inches in diameter, organic matter, debris, or expansive clay soils. Granular (sand) portion of the existing fills could be suitable for Select Fill, but the fill and natural near-surface clay soils will not be suitable, as-is, due to their high expansion potential. They can be used for Select Fill if lime-treated, or for General Fill below the level of Select Fill.. The geotechnical engineer should approve all proposed fill and backfill materials prior to being placed at the site.

Select Fill should be placed in the following areas:

- Within building areas, to a distance of at least five feet beyond building perimeter, and extending to at least 18 inches below the ground floor subgrade.

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[2] Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same soil, as determined by the ASTM D1557 laboratory compaction test procedure. Optimum moisture is the water content (percentage by dry weight) which corresponds to the maximum dry density.

- Within areas of exterior concrete slabs, and walkways, for a distance of at least three feet beyond their edges, and extending 12 inches below slab subgrade level. Alternatively, the Select Fill can be omitted and replaced with 4 inches of rock base course placed over properly moisture conditioned and compacted soil subgrade. This option, though less costly to construct, could require some slab maintenance because of greater shrink/swell movements of the soil subgrade.

6.1.5 Compaction of Fill (Including Lime-Treated Clays)

Fills and backfills should be placed in 8-inch (maximum) loose lifts, moisture conditioned as required, and compacted to at least 90 percent relative compaction (95 percent in upper 6 inches under paved areas). Following completion of fill compaction, the ground surface should be kept moist to avoid excessive moisture loss. All surfaces should be finished to present a smooth, firm, unyielding subgrade. Finished fill (and cut) slopes should be no greater than 2 (horizontal):1(vertical).

6.1.6 Utility Trenches

We recommend that utility conduit and pipe bedding material consist of sand with less than 10 percent fines. The bedding should extend from the bottom of the trench to 1 foot above the top of the pipe. Sand bedding should be placed in a trench free of standing water and mechanically compacted to at least 90 percent relative compaction.

Trench backfill above the pipe bedding should meet the criteria for Select Fill, as described above, in areas where settlement of the trench backfill would be a concern. In landscape areas, onsite soils can used as backfill, but some long-term settlement should be anticipated. Trench backfill should be placed in uniform layers not exceeding 8 inches in loose thickness, moisture-conditioned to near-optimum moisture content (2 to 4 percent above Optimum for clay soils), and then compacted to at least 90 percent Relative Compaction. Jetting or water flooding should not be permitted for any backfill compaction.

6.1.7 Surface Drainage

The finish exterior grades should be designed to drain surface water away from the proposed building. Slopes of at least 2 percent are recommended within 5 feet of the building. Where such surface gradients are difficult to achieve, we recommend that area drains and/or surface drainage swales be installed to direct surface water to a suitable discharge location. Downspout drainage should preferably be collected in closed pipe systems and routed to a suitable discharge outlet. Rainwater should not be allowed to pond on pavements or around the buildings. Surface water ponding should also be prevented during construction to reduce disturbance and loosening of site soils.

6.1.8 Fill Settlements

As much as four feet of fill will be needed for grading the office building pads, primarily near the west end of the southernmost (Phase III) office building. Raising the grades will cause settlement of underlying clays and silts. We estimate that by raising the grades one to four feet, the ground could settle as much as 1 to 3 inches. The settlements will occur relatively quickly, within four to six months. We recommend that fill settlements be checked with survey monuments placed on top of the fill after site grading is complete. This data will be used to confirm that post-construction settlements of pavements, floor slabs, and shallow foundations will be small.

6.2 Foundations

6.2.1 Driven Piles

Considering the relatively compressible nature of the clays and silts at the site and the load demand from the office building and parking garages, we judge that deep foundations will be required to support the new structures. In the past, precast prestressed concrete piles have been used to support buildings with more than two to three stories. The piles have been typically 12-inch square, with length of 30 to 60 feet. Considering that 14-inch square piles are now more commonly used because of lateral load demands for seismic design, we recommend their use for the project. The piles will primarily derive their supporting capacity from skin friction in the clay soils beneath the site. Where sand and gravel layers are encountered, additional capacity can be achieved in end bearing at the pile tip. However, the sand gravel layers are not continuous across the site and therefore, cannot be relied upon for design of pile lengths and capacities.

Allowable axial downward and uplift capacities of 14-inch-square precast prestressed concrete piles are presented in Plate 6-1. The allowable compressive (downward) capacities given for service loading cases (dead load plus reduced live loads) include a Factor of Safety (FS) of 2.0 for skin friction and a FS of 3.0 for end bearing. The allowable compressive capacities under seismic loading conditions include a 1.5 FS for skin friction and a FS of 2.0 for end bearing. Uplift capacities for short-term loads are based on skin friction and a FS of 2.0.

The pile capacities are based on a 3-foot-thick pile cap with the pile cap top at approximately one foot below finished grade elevation. For pile groups with pile spacing of at least three pile widths center-to-center, a group efficiency of 1.0 may be assumed for estimating axial capacity for both static and seismic conditions.

The capacities presented are based on the strength of the soils. The structural capacities of the piles should also be sufficient to carry the design loads. In addition, we recommend that piles be checked for a long term loading condition of dead and sustained live loads, plus a drag load of 150 kips caused by potential small amounts of site settlement.

We estimate that post-construction settlement of the proposed buildings supported on the piles as recommended will be less than 1/2 inch. Differential settlements will be less than 1/4 of inch.

6.2.2 Lateral Resistance

Lateral loads can be resisted by passive soil pressure on the pile caps. The short-term passive resistance of soils can be assumed to be equal to 300 pounds per cubic foot (pcf).

Additional lateral resistance can be obtained from pile bending. We have computed lateral capacities of 14-inch square concrete piles using LPILE computer program by ENSOFT, Inc. A graph of pile head deflection vs. pile head lateral load for fixed head and free-head conditions are presented in Plate 6-2. Distribution of the lateral deflection, internal shear, and moment with depth for piles with a maximum deflection of one (1) inch is presented in Plates 6-3, 6-4, and 6-5. Pile response to specific lateral loads can be evaluated, if desired, during foundation design.

The lateral pile capacities are for single piles. For piles spaced 3 pile diameters on center, group reduction factors should be applied to the lateral load capacities (to reduce the lateral load for a given amount of deflection at the pile head). The reduction factors depend on the configuration of the pile group. Reduction factors for several configurations are presented in the following table:

No. of Piles in Group	Configuration	Reduction Factor
4	2x2	0.60
6	2x3	0.43
9	3x3	0.40

6.2.3 Indicator Piles

We recommend that an indicator pile program be conducted to confirm design pile capacities and lengths prior to start of production pile driving at each building. We recommend driving at least 5 indicator piles for the office building and 5 indicator piles for the parking garage planned to be constructed in Phase I. We will evaluate the need for additional indicator piles for other phases of the project after reviewing the Phase I indicator pile program. The piles should be driven with the same equipment that will be used for driving the production piles.

We recommend that indicator pile driving be monitored with a pile driving analyzer (PDA) to evaluate soil resistance, driving criteria, and the stresses in the pile during driving and during re-strike at least two-weeks after the indicator piles are installed. That will allow an evaluation of soil setup (increase in skin friction with time after driving) and adjustment of pile lengths if appropriate. The PDA monitoring of pile driving will also enable an evaluation of pile stresses and end bearing resistances when dense sand and gravel layers are encountered. This will support decisions of whether to drive through dense layers or stop piles short of design lengths.

6.2.4 Pile Installation

Pile driving for other projects at Bishop Ranch has experienced some ground surface heave and cracking adjacent to piles. If this occurs, the cracks adjacent to piles should be filled with a sand slurry or grout. Alternatively, if pre-drilling is used for initial pile placement and control of ground heave, the diameter of the pre-drill auger should be no greater than the pile width. The depth of pre-drilling should be limited to 15 feet or to the groundwater level.

During pile driving, our engineer or technician should be present to record blow counts and observe pile installation details so that we can respond to various conditions as they develop. In previous projects at Bishop Ranch, a small number of piles have been broken, probably as a result of trying to drive through dense sand and gravel layers, typically below depths of 30 to 40 feet. If similar conditions are encountered for this project, we should assist the design and construction team to develop appropriate remedial measures, such as replacing the broken piles or limiting driving resistance by stopping piles short of their design lengths.

We recommend that adequate measures be taken to reduce vibrations that could be felt at nearby buildings. We also recommended that a pre-construction survey be conducted before driving operations commence and establish a baseline to check for vibration impacts caused by pile driving.

6.2.5 Alternative Pile Types

Contractors could propose alternative pile types as cost saving items. In particular, auger-cast-in-place (ACIP) or auger pressure grouted (APG) piles are being used in Bay Area in competition with driven piles. We can assist in evaluating such alternative pile types if desired.

6.3 Concrete Slabs on Grade

We recommend that concrete floor slabs be supported on at least 18 inches of Select Fill over native soils as described in Section 6.1. If migration of moisture vapor through the slab would be objectionable, the upper 6 inch of Select Fill can be replaced with 6 inches of free-draining granular fill, such as ¼ to ¾-inch crushed rock, to provide capillary moisture break. The crushed rock should be vibrated with compaction equipment to a well-keyed state. The crushed rock should be covered with a synthetic membrane at least 10 mils thick beneath the floor slab. To help provide puncture protection and to aid in slab curing, the membrane can be covered with about 2 inches of clean-washed sand.

Slab reinforcement should be provided in accordance to with the anticipated use and loading on the floor slabs. Structural requirements and/or concentrated loads will require additional reinforcing. Minor movements of the concrete slab with resulting cracking should be expected. The recommendations above, if properly implemented, should help reducing the magnitude of the cracking.

6.4 Miscellaneous Footings Foundations

For small structures and miscellaneous building appurtenances, having relatively light loads, shallow foundations can be used. Shallow spread footings or mat foundations should be founded at least 30 inches below the lowest adjacent ground on properly prepared subgrade soils. Alternatively, they can be supported near the ground surface on 18 inches of Select Fill as described in Section 6.1.4. Footings/mats located adjacent to utility trenches should have their bearing surfaces situated below an imaginary 1½:1 (horizontal to vertical) plane projected upward from the bottom of the adjacent utility trench.

Footings/mats conforming to the above requirements could be designed using allowable bearing pressures of no greater than 2000 pounds per square foot (psf). This allowable pressure can be increases one-third for short term load conditions, including wind or seismic forces. These values are net allowable bearing capacities (the weight of the footing can be neglected. For the bearing pressures above, the settlement of the footings is expected to be less than one inch.

Resistance to lateral loads can be derived from passive resistance acting on the faces of foundation elements oriented perpendicular to the direction of loading and friction acting between the base of the foundations and the supporting subgrade. We recommend using an equivalent fluid pressure of 300 pcf to compute passive resistance. The upper 12 inches of embedment should be ignored for passive resistance calculations except where the ground is paved or covered by a slab or pavement. A friction coefficient of 0.3 applied to dead loads can be used to compute base friction. The above values include a factor of safety of 1.5.

Resistance to uplift loads can be provided by the dead load of the structure and weight of the footing plus any soil cover.

6.5 Flexible Asphalt Pavements

We suggest the following flexible pavement structural section thicknesses.

Traffic Index	Asphalt Cement Thickness (inches)	Class 2[(1)] Aggregate Base Thickness (inches)	Class 2[(1)] Aggregate Subbase Thickness (inches)
4	2	8.5	--
	2	4	6
5	2.5	11	--
	2.5	6	6
6	3	14	--
	3	7	8

(1) Caltrans designation

The above pavement thicknesses are based on an R-value of 5 for the clay subgrade soils (*HLA, Geotechnical Investigation, Bishop Ranch 1 Development, San Ramon, California*, prepared for Sunset Development Company, HLA Project 50044.1, dated May 15, 2000). We anticipate that a Traffic Index of 4 could be used for parking areas with lower traffic loads and frequencies, while Traffic Indexes of 5 and 6 would be applicable to occasional to regular heavier traffic loadings and frequencies associated with entry/access roads and truck loading areas, respectively.

Soil subgrades in asphalt-paved areas should be smooth and nonyielding. The upper six-inches should be moisture conditioned, as necessary, to greater than Optimum Moisture Content and compacted to at least 95 percent relative compaction. The subgrade should not be allowed to dry out prior to pavement construction. If soft, unstable, or saturated soils are encountered, the questionable soil should be excavated and replaced with subbase material or aggregate base material. The aggregate base and subbase should conform to the criteria specified for Class 2 Aggregate Base and Subbase in the current, adopted Caltrans Standard Specifications. The Subbase and Aggregate Base courses should be moisture conditioned to slightly above optimum moisture content and compacted to at least 95 percent relative compaction prior to placement of the Asphalt Concrete.

6.6 Corrosion Potential

On the basis of the results of resistivity, pH, and chloride and sulfate measurements on surface soils, it appears that, in general, surficial materials are very corrosive to reinforced concrete foundation elements and buried utilities. Data for onsite soils are described below. The corrosion potential for any imported select fill should also be checked.

6.6.1 Soil Resistivity

Soil resistivity is a measure of the ability of a soil to conduct electrical current. Resistivity is usually related to the amount of soluble salts in the soil. Low resistivities generally indicate more corrosive conditions. Seawater has a resistivity of about 70 ohm-cm.

A commonly used soil classification for interpretation of corrosive environments on metals is presented below.

Soil Resistivity (ohm-cm)	Degree of Corrosivity
0 – 1000	Very corrosive
1,000 - 2,000	Corrosive
2,000 - 5,000	Fairly corrosive
5,000 – 10,000	Mildly corrosive
10,000 and above	Negligible

Another factor influencing corrosion potential is pH. Values below pH 7 indicate acidic conditions, and hence, a corrosive environment for metals and concrete.

Resistivity and pH measurements were performed on soil samples from the borings, and analyzed at field moisture contents. The test results are summarized below:

Boring	Depth (feet)	pH	Resistivity (ohm-cm)
B-1	5.0	8.72	750
B-8	5.5	7.20	830

These test results indicate that the soil samples are considered "very corrosive."

6.6.2 Sulfates and Chlorides

The concentrations of sulfate and chloride in soils can also have a corrosive effect on buried utilities and foundation elements. General correlations between sulfate and chloride concentrations and corrosivity are presented below:

Chloride Concentration (mg/kg)	Degree of Corrosivity
Over 1,500	Severe
300 – 1,500	Positive
0 – 300	Negligible

Sulfate Concentration (mg/kg)	Degree of Corrosivity
Over 5,000	Severe
2,000 - 5,000	Considerable
1,000 - 2,000	Positive
0 – 1,000	Negligible

Sulfates are increasingly corrosive to ferrous metals at concentrations above 1,000 mg/kg, and to concrete above 2,000 mg/kg. In addition to a corrosive attack that is chemical, sulfates can exhibit a physical attack on concrete at higher concentrations. Chloride does not demonstrate a physical attack on concrete, but it is corrosive to metals. Sulfate and chloride test results are summarized below:

Boring	Depth (feet)	Chloride Concentration (mg/kg)	Sulfate Concentration (mg/kg)
B-1	5.0	29.2	138.6
B-8	5.5	9.4	49,2

On the basis of the results of measurements on surface soils, the degree of corrosively from sulfate and chloride concentration is considered "negligible".

7.0 ADDITIONAL GEOTECHNICAL SERVICES DURING CONSTRUCTION

MACTEC should perform additional geotechnical consultation during design on an as-needed basis. We should review the final plans and specifications during the design to check for conformance with the intent of our geotechnical recommendations. We should also review the bid documents and bids during the Construction Administration Phase to check for items that could result in unnecessary risk of change orders during construction.

If changes are made in the project, the conclusions and recommendations presented in this report may not be applicable; therefore, we should review any changes to verify that our conclusions and recommendations are valid and modify them if required.

During construction, we should perform site visits as needed to check geotechnical aspects of the work and perform quality control testing of the following work items:

- Observe the stripping and excavation operations for proper removal of all unsuitable materials;
- Evaluate the suitability of on-site and import soils for fill placement; collect and submit soil samples for required or recommended laboratory testing where necessary;
- Review the contractor's plans for lime treatment and lime materials, as well as mixing, curing, and compacting procedures (if lime treatment is selected). Observe and test lime treated soils;
- Observe and test the exposed subgrades prior to placement of compacted fills, slabs, pavements or foundations;
- Observe and test backfilling of utility trenches;
- Observe the indicator pile driving programs and adjust pile lengths as appropriate;
- Observe the driving of production piles;

At the completion of geotechnical-related construction, we should prepare a summary report of the work we observed for submittal to building officials.

TABLES

Table 4-1: Major Named Faults Near the Project Site

Fault	Distance from Site (kilometers)	Direction from Site	Slip Rate (mm/yr)	Maximum Moment Magnitude
Calaveras	1	WSW	6	6.8
Concord – Green Valley	14	N	6	6.9
Hayward	15	WSW	9	7.1
Greenville	16	NE	2	6.9
Great Valley	27	ENE	1.5	6.7
San Andreas	44	WSW	24	7.9
Monte Vista - Shannon	45	SW	0.4	6.5
Rodgers Creek	49	NW	9	7.0
San Gregorio	54	WSW	5	7.3
West Napa	67	NNW	1	6.5
Sargent	72	S	3	6.8
Ortigalita	80	SE	1	6.9
Point Reyes	95	WNW	0.3	6.8

Checked RIA
Approved

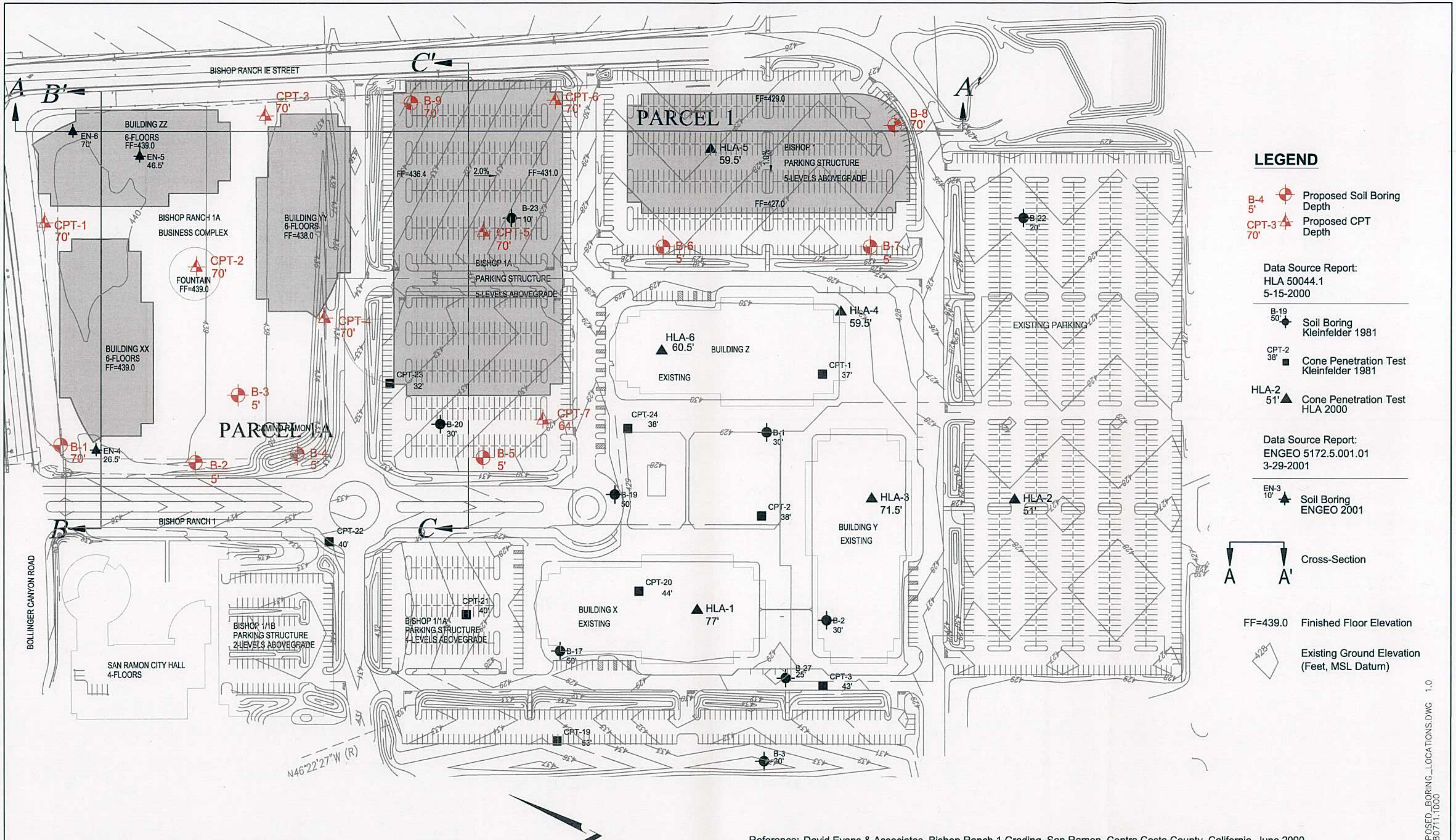
PLATES

MACTEC

Vicinity Map
Bishop Ranch City Center Project
Parcel 1 & 1A
San Ramon, California

PLATE:
1-1

DRAWN	JOB NUMBER	CHECKED	CHECKED DATE	APPROVED	APPROVED DATE
JLA	4096088527	RA	10/08		09/2008



Note: The existing ground elevation contours at the vacant lot north of Parcel 1A are approximate and do not reflect the latest survey results by David Evans & Associates (*Bishop Ranch 1, As Built Topography, Lot 4, Subdivision 8454 (435 M 28), San Ramon, Contra Costa County, California. Dated April 01, 2003)*

Reference: David Evans & Associates, Bishop Ranch 1 Grading, San Ramon, Contra Costa County, California, June 2000
Sunset Developement Co., Conceptual Grading Plan - Business Complex, San Ramon, California, April 2007

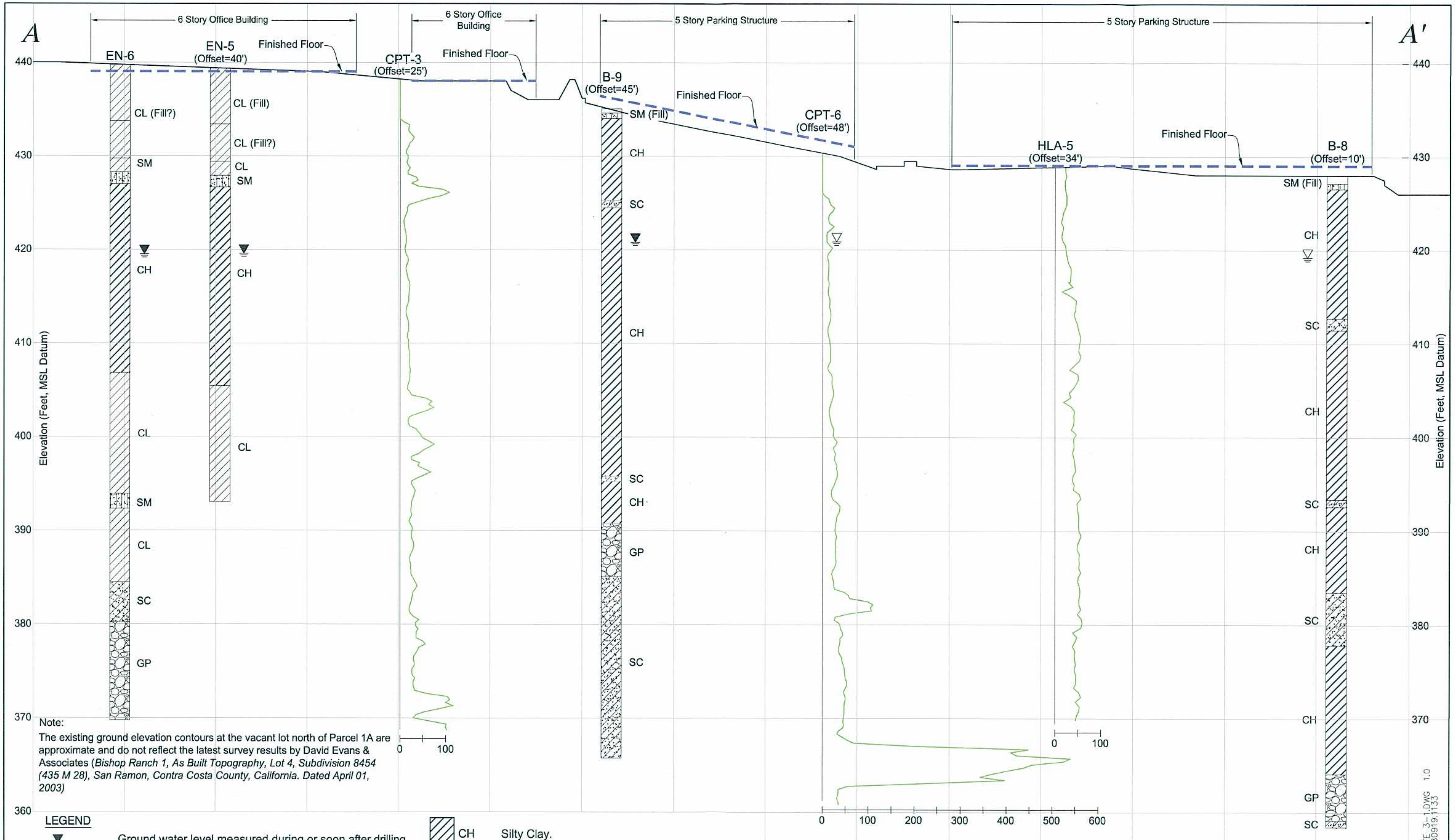
PROPOSED_BORING_LOCATIONS.DWG 1.0
20080711.1000

MACTEC

Site Plan
Bishop Ranch City Center Project
Parcel 1 & 1A
San Ramon, California

PLATE
1-2

DRAWN	JOB NUMBER	CHECKED	CHECKED DATE	APPROVED	APPROVED DATE
JLA	4096088527	RH	10/2008		10/08



Note:
The existing ground elevation contours at the vacant lot north of Parcel 1A are approximate and do not reflect the latest survey results by David Evans & Associates (*Bishop Ranch 1, As Built Topography, Lot 4, Subdivision 8454 (435 M 28), San Ramon, Contra Costa County, California. Dated April 01, 2003)*

LEGEND

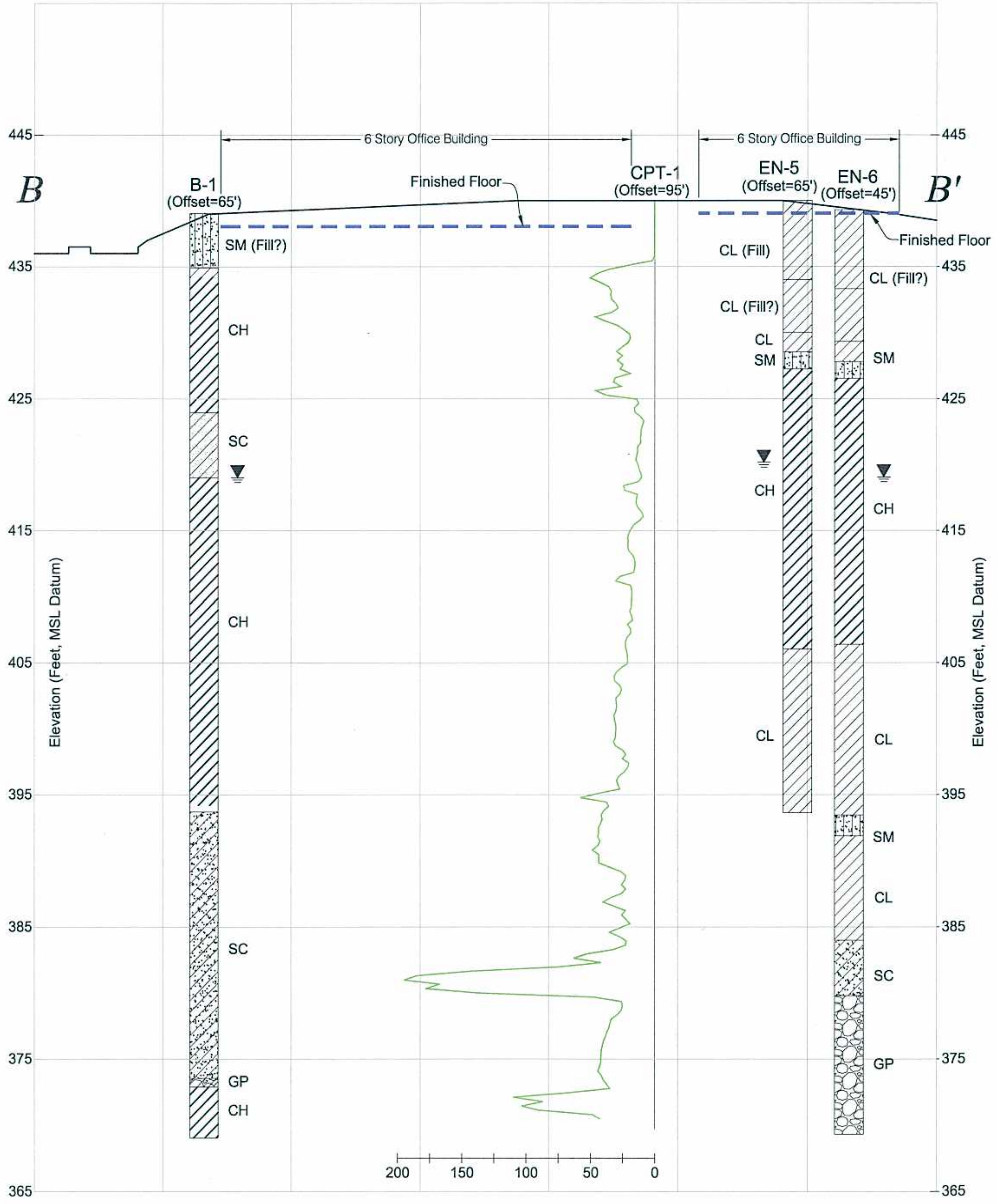
▼	Ground water level measured during or soon after drilling.
▽	Ground water level measured from CPT.
0 100	CPT tip resistance (tsf)
▨ CL	Sandy Clay
▨ CH	Silty Clay.
▨ GP	Poorly graded gravels or gravel-sand mixture
▨ SM	Silty sand, sand-silt mixture.
▨ SC	Clayey sands, sand-clay mixtures.

MACTEC

Cross Section A-A'
Bishop Ranch City Center Project
Parcel 1 & 1A
San Ramon, California

PLATE
3-1

DRAWN	JOB NUMBER	CHECKED	CHECKED DATE	APPROVED	APPROVED DATE
JLA	4096088527	RH	10/2008	TDWB	10/08



Note: The existing ground elevation contours at the vacant lot north of Parcel 1A are approximate and do not reflect the latest survey results by David Evans & Associates (*Bishop Ranch 1, As Built Topography, Lot 4, Subdivision 8454 (435 M 28), San Ramon, Contra Costa County, California. Dated April 01, 2003)*

LEGEND

▼		Ground water level measured during or soon after drilling.
▽		Ground water level measured from CPT.
0 100		CPT tip resistance (tsf)
▨	CL	Sandy Clay
▨	CH	Silty Clay.
▨	GP	Poorly graded gravels or gravel-sand mixture
▨	SM	Silty sand, sand-silt mixture.
▨	SC	Clayey sands, sand-clay mixtures.

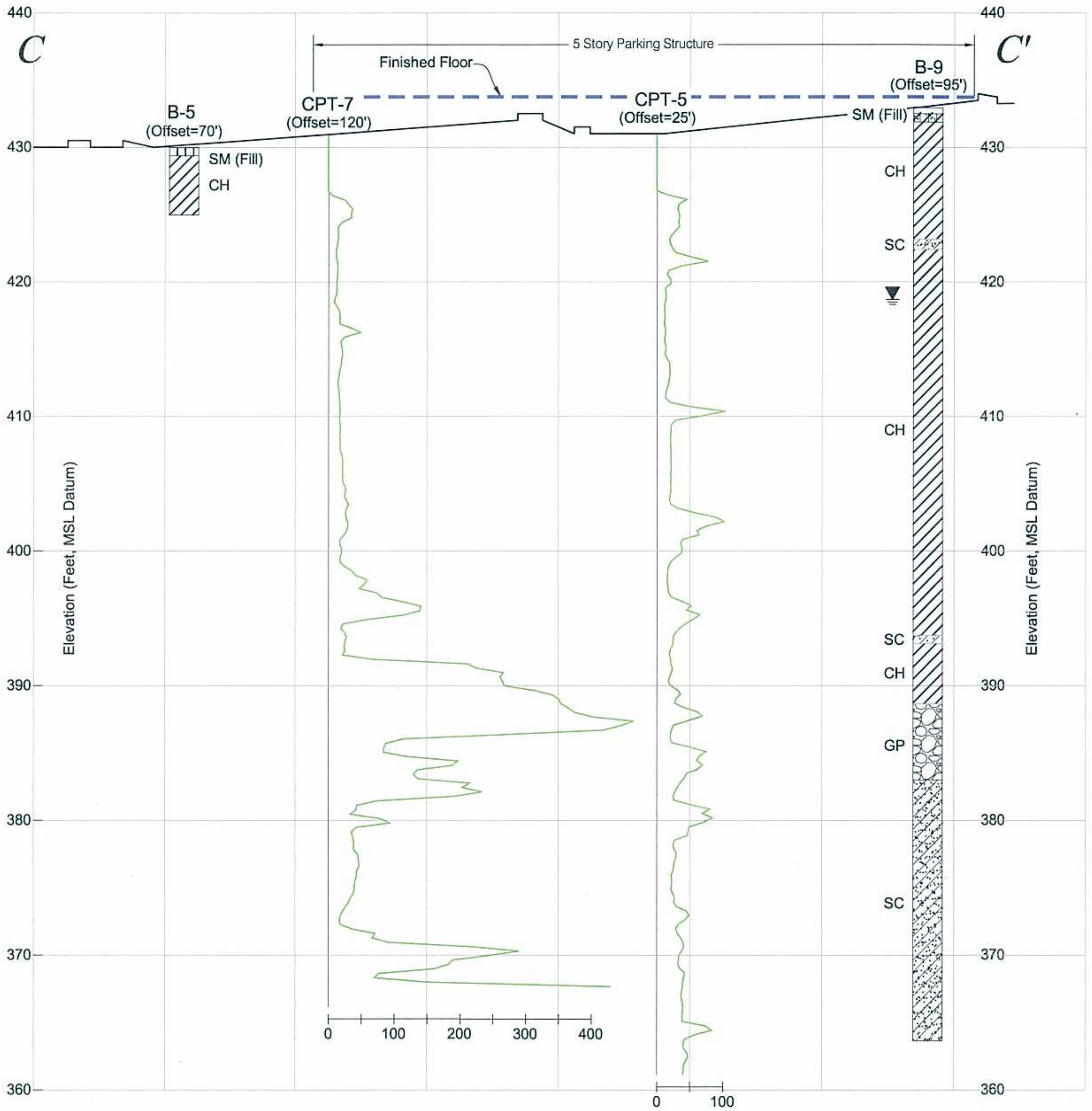
PLATE 3-2.DWG 1.0
20080919.1125

MACTEC

Cross Section B-B'
Bishop Ranch City Center Project
Parcel 1 & 1A
San Ramon, California

PLATE
3-2

DRAWN	JOB NUMBER	CHECKED	CHECKED DATE	APPROVED	APPROVED DATE
JLA	4096088527	RH	10/2008		10/08



LEGEND

▼		Ground water level measured during or soon after drilling.
▽		Ground water level measured from CPT.
0 100		CPT tip resistance (tsf)
▨	CL	Sandy Clay
▨	CH	Silty Clay.
▨	GP	Poorly graded gravels or gravel-sand mixture
▨	SM	Silty sand, sand-silt mixture.
▨	SC	Clayey sands, sand-clay mixtures.

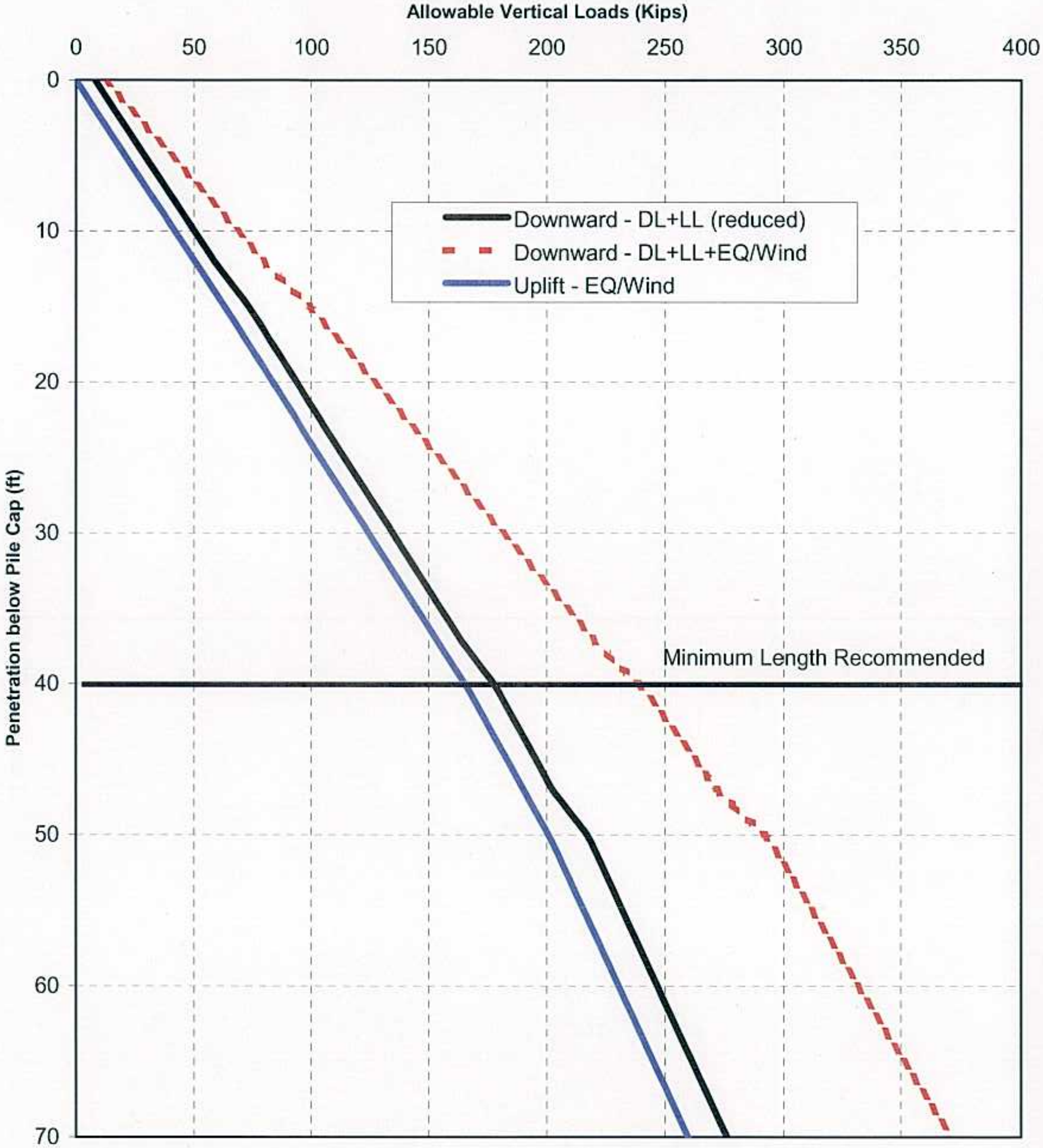
PLATE 3-3.DWG 1.0
20080919.1129

MACTEC

Cross Section C-C'
Bishop Ranch City Center Project
Parcel 1 & 1A
San Ramon, California

PLATE
3-3

DRAWN	JOB NUMBER	CHECKED	CHECKED DATE	APPROVED	APPROVED DATE
JLA	4096088527	RJH	10/2008		10/08

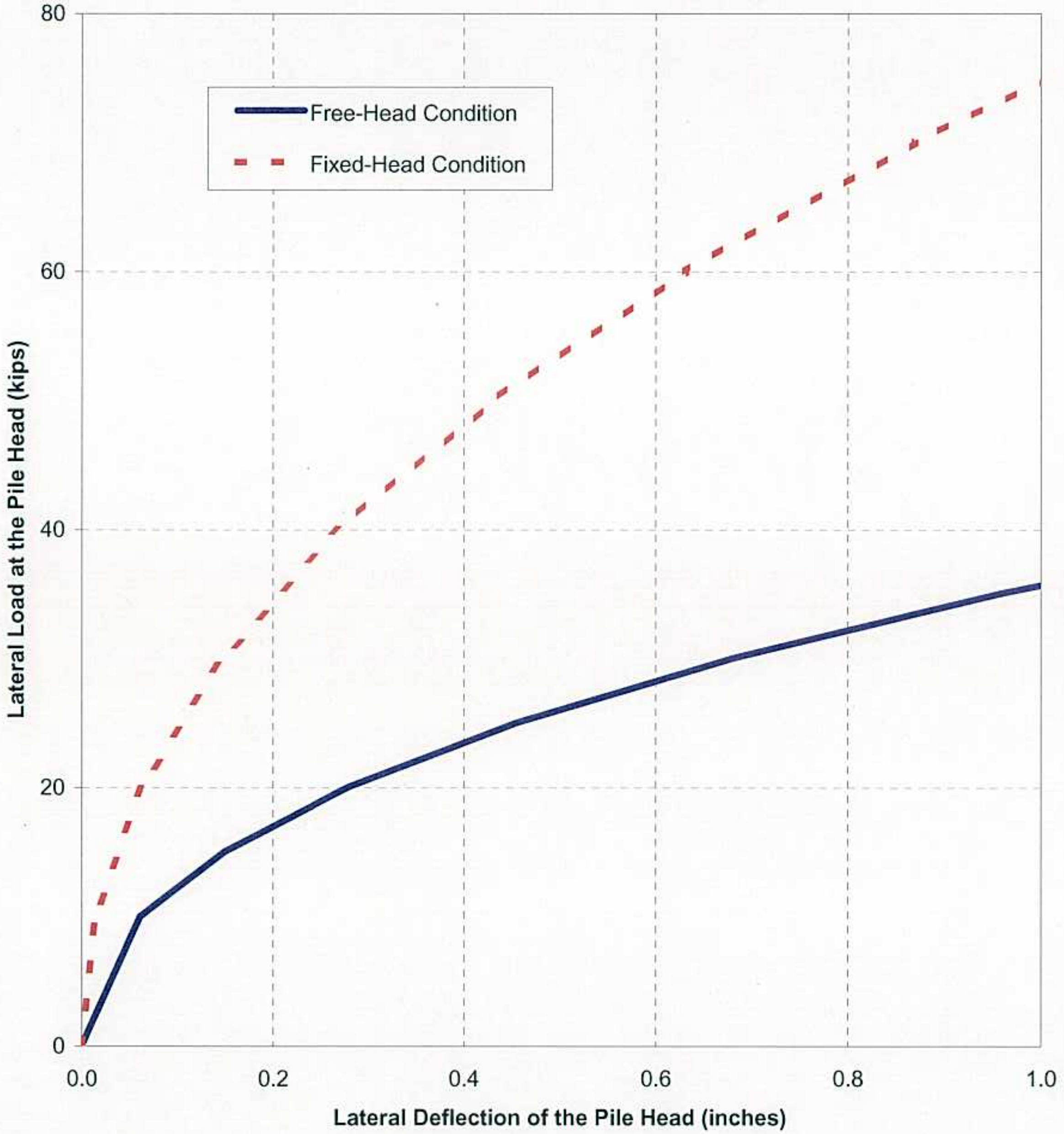


MACTEC

Allowable Pile Capacities
14-inch Square Precast Concrete Piles
Bishop Ranch City Center Project - Parcel 1&1A
San Ramon, California

Plate
6-1

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED DATE
RH	4096088527		9/08	



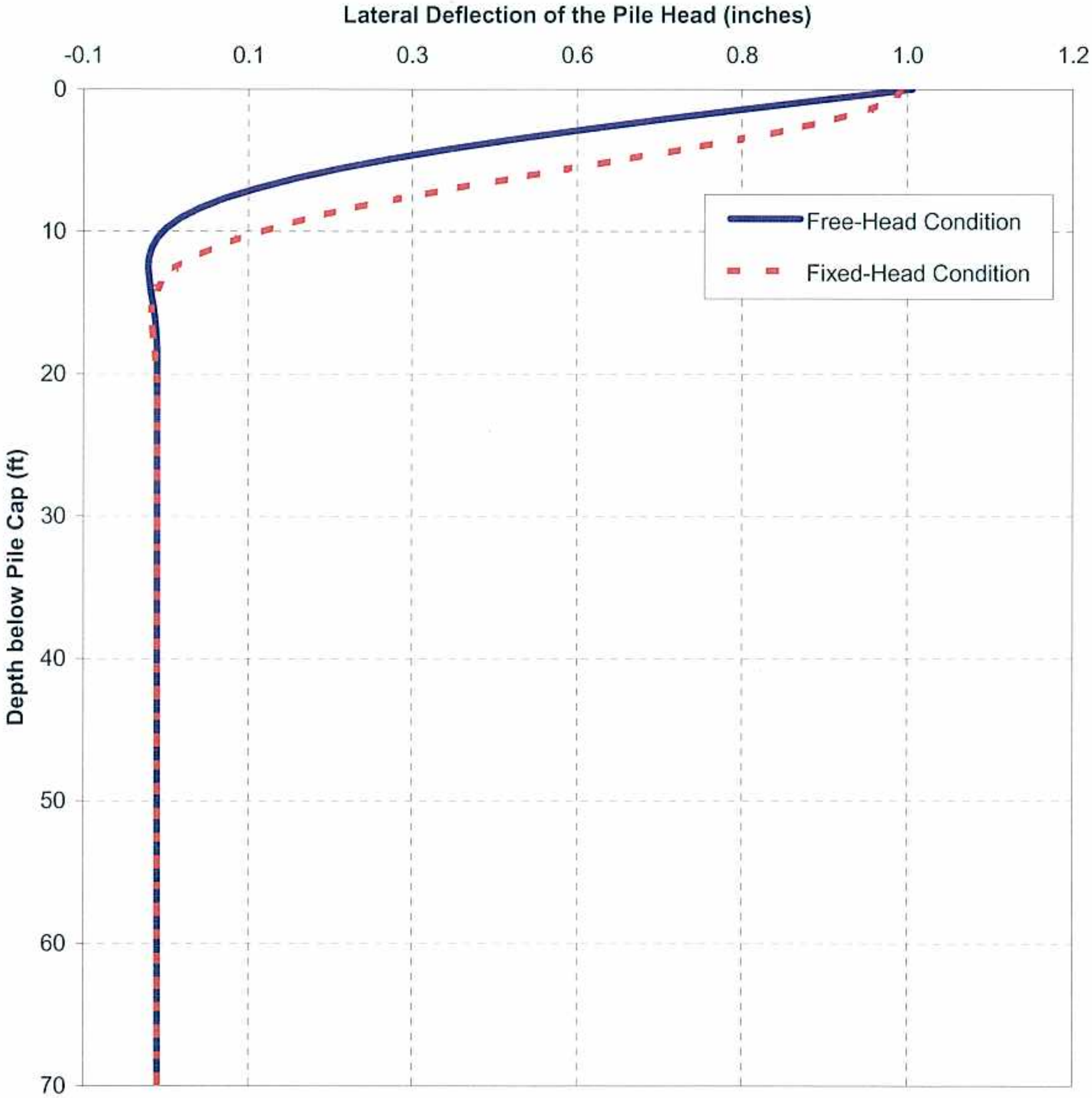
The area of steel in the pile cross section was assumed to be approxomately one percent of the total area.

MACTEC

Lateral Load vs Lateral Deflection
14-inch Square Precast Concrete Piles
Bishop Ranch City Center Project - Parcel 1&1A
San Ramon, California

Plate
6-2

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED DATE
RH	4096088527		9/08	



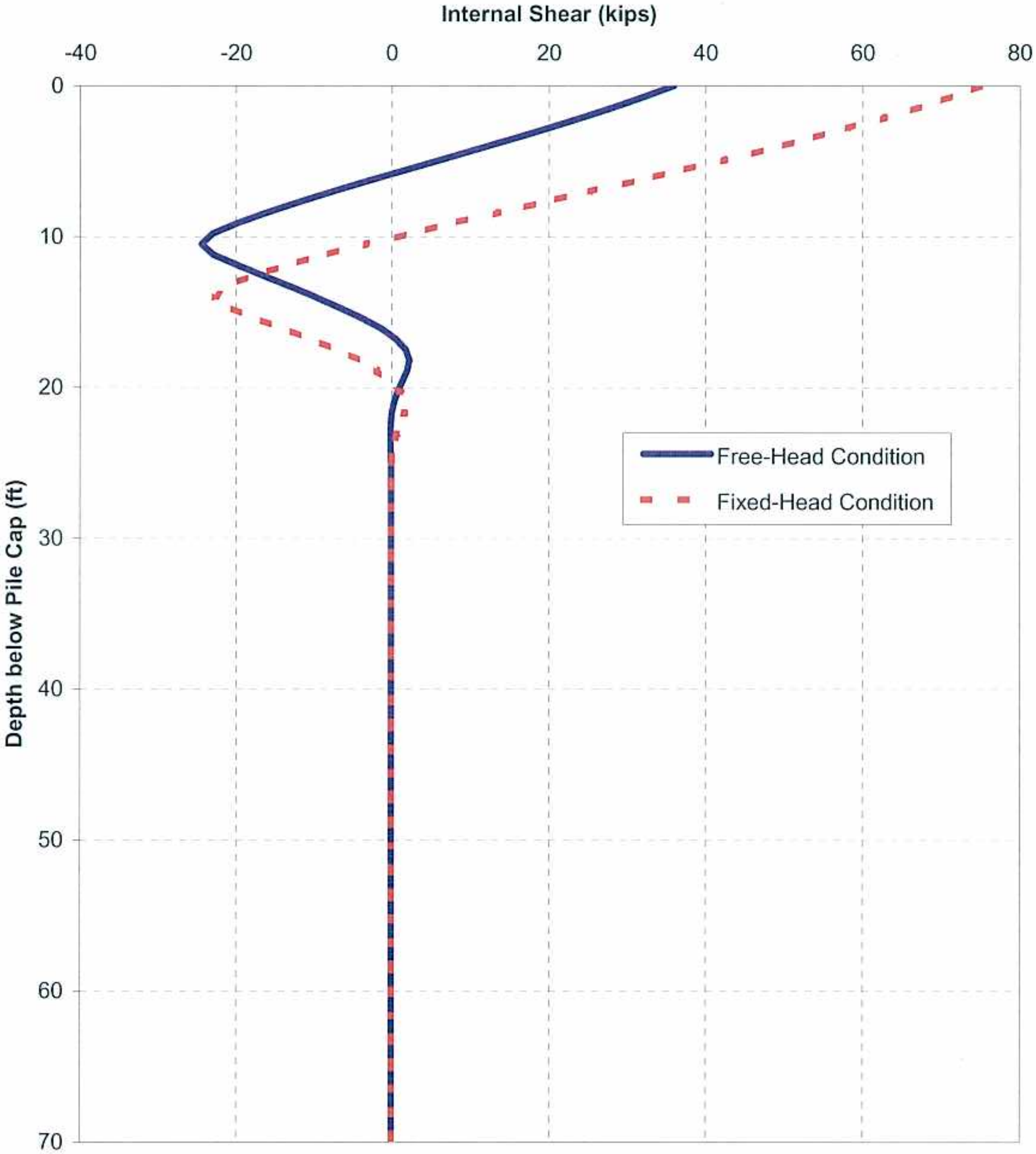
The area of steel in the pile cross section was assumed to be approxomately one percent of the total area.

MACTEC

Lateral Deflection vs Depth (1-inch Max Deflection)
14-inch Square Precast Concrete Pile
Bishop Ranch City Center Project - Parcel 1&1A
San Ramon, California

Plate
6-3

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED DATE
RH	4096088527		9/08	



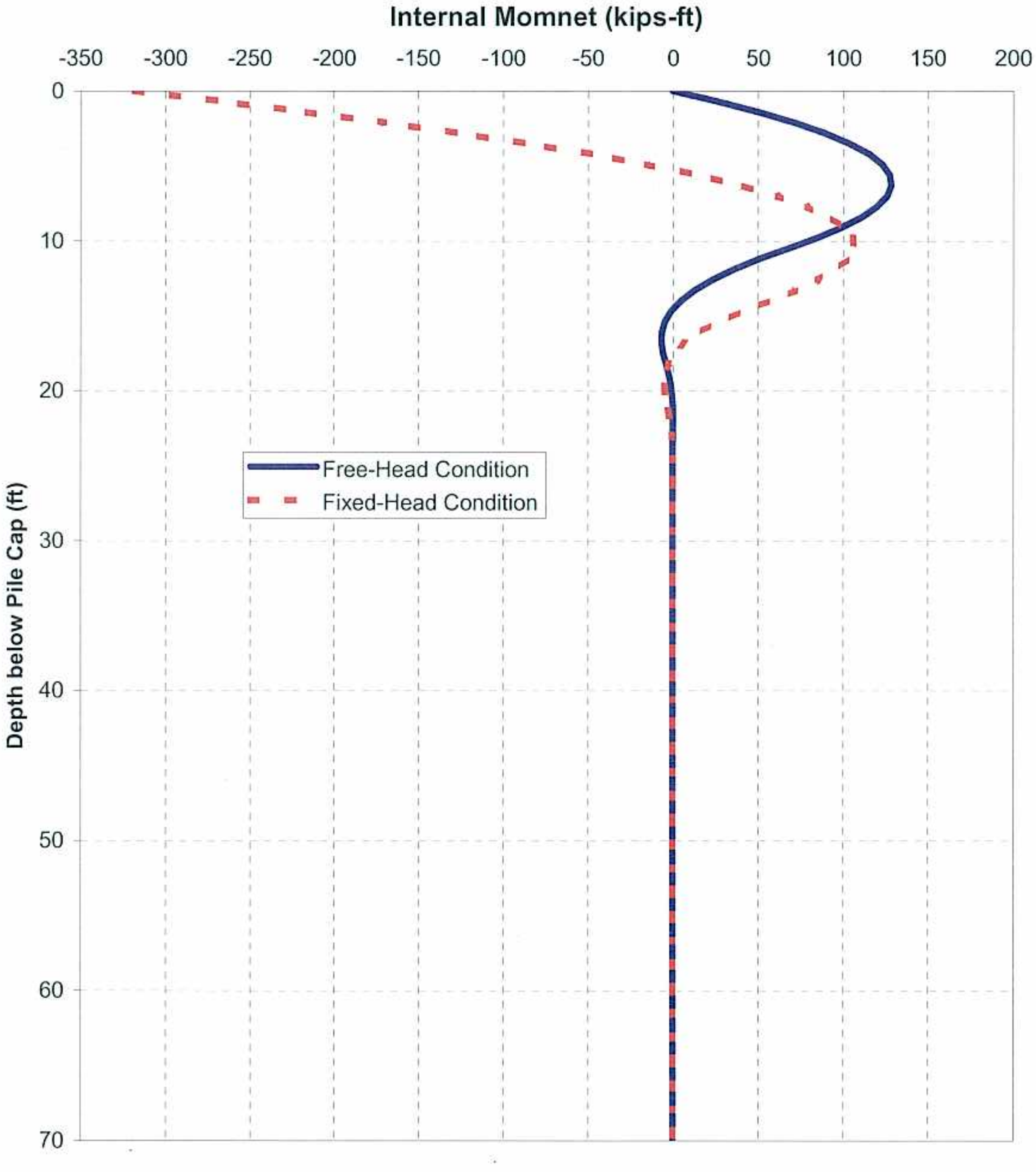
The area of steel in the pile cross section was assumed to be approxomately one percent of the total area.

MACTEC

Internal Shear vs Depth (1-inch Max Deflection)
14-inch Square Precast Concrete Pile
Bishop Ranch City Center Project - Parcel 1&1A
San Ramon, California

Plate
6-4

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED DATE
RH	4096088527		9/08	



The area of steel in the pile cross section was assumed to be approxomately one percent of the total area.

MACTEC

Internal Moment vs Depth (1-inch Max Deflection)
14-inch Square Precast Concrete Pile
Bishop Ranch City Center Project - Parcel 1&1A
San Ramon, California

Plate
6-5

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED DATE
RH	4096088527		9/08	

APPENDIX A

BORING AND CPT LOGS FROM PREVIOUS INVESTIGATIONS

Checked RA

Approved DNG

***Preliminary Geotechnical Exploration, San Ramon City Center, San Ramon, California*, prepared for City of San Ramon, California, Prepared by ENGEO, Project 5172.001.01, dated March 29, 2001.**

DATE OF BORING: January 22, 2001

SURFACE ELEVATION: Approx. feet (meters)

DEPTH (FEET)	DEPTH (METERS)	SAMPLE NUMBER	LOG, LOCATION AND TYPE OF SAMPLE	DESCRIPTION	BLOWS/FT.	qu UNCON STRENGTH (TSF) *FIELD PENET. APPROX.	IN PLACE DRY UNIT WEIGHT (PCF)	IN PLACE MOIST. CONTENT % DRY WEIGHT
0								
	1	4-1		SILTY CLAY (CH), black, damp, hard, with sand and some wood debris. (Fill)	39	+4.5*	106.8	15.8
				SILTY CLAY (CL), greyish brown, abundant carbonates, hard, damp.				
5								
	2	4-2			26	4.5*	99.0	11.4
				SILTY SAND (SM), light olive, medium dense, damp.				
				SANDY CLAY (CL), olive, very stiff, moist.				
10	3							
		4-3			27	3.5*		
	4							
15				∇				
	5	4-4		SILTY CLAY (CL), olive brown, stiff, very moist to wet, trace carbonates.	15	1.5*	90.4	31.3
				SILTY CLAY (CH), dark grey, very stiff, wet.				
20	6							
		4-5			18			
				SANDY CLAY (CL), olive brown, stiff, wet.				
	7							
				SILTY CLAY (CH), light olive brown, very stiff, wet, trace carbonates.				
25								
	8	4-6			12			28.8
				Bottom of boring at approximately 26$^{1}/_{2}$ feet. Ground water encountered at 15 feet during drilling.				
30	9							

ENGEO
INCORPORATED

SAN RAMON CITY CENTER
SAN RAMON, CALIFORNIA

BORING NO.: B-4
DATE: March 2001
PROJ. NO.: 5172.5.001.01
CHECKED BY mr

FIGURE NO. 9

DATE OF BORING: January 22, 2001

SURFACE ELEVATION: Approx. feet (meters)

DEPTH (FEET)	DEPTH (METERS)	SAMPLE NUMBER	DESCRIPTION	BLOWS/FT.	qu UNCON STRENGTH (TSF) *FIELD PENET. APPROX.	IN PLACE DRY UNIT WEIGHT (PCF)	IN PLACE MOIST. CONTENT % DRY WEIGHT
0			SILTY CLAY (CL), black, very stiff, dry to moist, with trace hay and other debris. (Fill)				
	1	5-1		28	+4.5*	101.2	16.2
5							
		5-2-1 5-2-2		30			
	2		SANDY CLAY with gravel (CL), light brown, moist, hard, poorly sorted. (Fill?)				
10	3		SILTY CLAY (CL), light brown, very stiff, moist.				
		5-3	SILTY SAND (SM), olive brown, medium dense, moist.	43	2.0*	112.0	15.5
	4						
15			SILTY CLAY (CH), greyish brown with black mottling, stiff, wet, trace charcoal.				
	5	5-4		16	2.0*	93.0	30.8
20	6		∇				
		5-5	Mottling changes to white.	23	3.0*		
	7						
			SILTY CLAY (CH), olive grey, very stiff, very moist.				
25							
	8	5-6		26	2.0	94.0	29.9
30	9						
		5-7		29	3.0*		
	10						

ENGEO
INCORPORATED

SAN RAMON CITY CENTER
SAN RAMON, CALIFORNIA

BORING NO.: B-5
DATE: March 2001
PROJ. NO.: 5172.5.001.01
CHECKED BY mr

FIGURE NO. 10

DATE OF BORING: January 22, 2001

SURFACE ELEVATION: Approx. feet (meters)

DEPTH (FEET)	DEPTH (METERS)	SAMPLE NUMBER	LOG, LOCATION AND TYPE OF SAMPLE	DESCRIPTION	BLOWS/FT.	qu UNCON STRENGTH (TSF) *FIELD PENET. APPROX.	IN PLACE DRY UNIT WEIGHT (PCF)	MOIST. CONTENT % DRY WEIGHT
35			▨	SANDY CLAY (CL), olive brown, hard, moist, some rust stains.				
	11	5-8	▨		56	4.0*	114.2	17.9
	12		▨					
40			▨	Trace charcoal.				
		5-9	▨		50	4.0*		
	13		▨					
45			▨	SILTY CLAY (CL), olive brown, very stiff, very moist, with trace carbonates, fine sand.				
	14	5-10	▨		26	2.5*	100.7	25.6
				Bottom of boring at approximately $46^1/_2$ feet. Ground water encountered at 20 feet during drilling.				
	15							
50								
	16							
55								
	17							
	18							
60								
	19							
65								
	20							

ENGEO
INCORPORATED

SAN RAMON CITY CENTER
SAN RAMON, CALIFORNIA

BORING NO.: B-5
DATE: March 2001
PROJ. NO.: 5172.5.001.01
CHECKED BY Mr

FIGURE NO. 10

MET 5172.GPJ 3/30/01

DATE OF BORING: January 29, 2001

SURFACE ELEVATION: Approx. feet (meters)

DEPTH (FEET)	DEPTH (METERS)	SAMPLE NUMBER	LOG, LOCATION AND TYPE OF SAMPLE	DESCRIPTION	BLOWS/FT.	qu UNCON STRENGTH (TSF) *FIELD PENET. APPROX.	IN PLACE DRY UNIT WEIGHT (PCF)	IN PLACE MOIST. CONTENT % DRY WEIGHT
0			▨	SILTY CLAY (CL), black, very stiff, dry to moist, with trace hay and other debris. (Fill)				
	1							
5								
	2		▨	SANDY CLAY with gravel (CL), light brown, hard, moist, poorly sorted. (Fill?)				
10	3		▨	SILTY CLAY (CL), light brown, stiff, moist.				
			▥	SILTY SAND (SM), olive brown, medium dense, moist.				
	4							
15			▨	SILTY CLAY (CH), greyish brown with black mottling, stiff, wet, trace charcoal.				
	5							
20	6			▽				
	7							
25				SILTY CLAY (CH), olive grey, very stiff, very moist.				
	8							
				Grades to very stiff.				
30	9							
	10							
35			▨	SANDY CLAY (CL), olive brown, hard, moist.				
	11							
40	12							
	13							
45				SILTY CLAY (CL), olive brown, very stiff, very moist, with trace carbonate.				
	14		▥	SILTY SAND (SM), olive brown, dense, wet, medium grained.				
			▨	SILTY CLAY (CL), light olive brown, very stiff, wet.				
	15							

ENGEO
INCORPORATED

SAN RAMON CITY CENTER
SAN RAMON, CALIFORNIA

BORING NO.: B-6
DATE: March 2001
PROJ. NO.: 5172.5.001.01
CHECKED BY Mr

FIGURE NO. 11

DATE OF BORING: January 29, 2001

SURFACE ELEVATION: Approx. feet (meters)

DEPTH (FEET)	DEPTH (METERS)	SAMPLE NUMBER	LOG, LOCATION AND TYPE OF SAMPLE	DESCRIPTION	BLOWS/FT.	qu UNCON STRENGTH (TSF) *FIELD PENET. APPROX.	IN PLACE DRY UNIT WEIGHT (PCF)	IN PLACE MOIST. CONTENT % DRY WEIGHT
50	16	6-1	▨	SILTY CLAY with sand (CL), light olive brown, very stiff, wet, some chunks of carbonates, minor rust stains.	27	3.0*	99.5	26.4
55	17	6-2	⁄	CLAYEY SAND with gravel (SC), brown, very dense, wet.	50/6"		126.1	12.6
60 65	18 19 20 21		░	GRAVELLY SAND (SP), greyish brown, very dense, wet.				
70 75 80 85 90 95	22 23 24 25 26 27 28 29 30			Bottom of boring at approximately 70 feet. Ground water encountered at 20 feet during drilling.				

MET 5172.GPJ 3/30/01

ENGEO
INCORPORATED

SAN RAMON CITY CENTER
SAN RAMON, CALIFORNIA

BORING NO.: B-6
DATE: March 2001
PROJ. NO.: 5172.5.001.01
CHECKED BY MJ

FIGURE NO. 11

***Geotechnical Investigation, Bishop Ranch 1 Development, San Ramon, California*, prepared for Sunset Development Company,**
Prepared by HLA, Project 50044.1, dated May 15, 2000

HOLGUIN, FAHAN & ASSOCIATES, INC.
ENVIRONMENTAL MANAGEMENT CONSULTANTS

RECEIVED
APR 2 4 2000
HARDING LAWSON

April 21, 2000

Mr. Ryan Shafer
Harding, Lawson and Associates, Inc.
383 Fourth St
Suite 300
Oakland, CA 94607

PROJECT NAME: CPT Testing at Bishop Ranch 1
PROJECT NO.: 50044.1

Dear Mr. Shafer:

Enclosed please find copies of the cone penetrometer testing (CPT) data for the above referenced project along with a copy of the corresponding invoice.

The cone penetrometer testing conducted for this project consisted of pushing an instrumented cone-tipped probe into the ground while simultaneously recording the resistance to penetration at the cone tip and along the friction sleeve.

The cone penetrometer testing described in this report was conducted in general accordance with the current ASTM specifications (ASTM D5778-95 and D3441-94) using an electronic cone penetrometer.

The CPT equipment operated by Holguin, Fahan & Associates, Inc. (HFA) consists of a cone assembly mounted at the end of a series of hollow sounding rods. A set of hydraulic rams is used to continuously push the cone and rods into the soil at a rate of 20-mm per second (approximately four feet per minute) while the cone tip resistance and sleeve friction resistance are recorded every 50-mm (approximately two inches) and stored in digital form. A specially designed all wheel drive 23-ton truck provides the required reaction weight for pushing the cone assembly and is also used to transport and house the test equipment.

The cone penetrometer assembly used for this project consists of a conical tip and a cylindrical friction sleeve. The conical tip has a 60° apex angle and a diameter of 35.6-mm (1.40-inch) resulting in a projected cross-sectional area of 10 cm^2 (1.5 square inches). The cylindrical friction sleeve is 133-mm (5.25-inch) in length and has an outside diameter of 35.8-mm (1.41-inch), resulting in a surface area of 150 cm^2 (23 square inches).

The interior of the cone penetrometer is instrumented with strain gauges that allow simultaneous measurement of cone tip and friction sleeve resistance during penetration. Continuous electric signals from the strain gauges are transmitted by a shielded cable in the sounding rods to the PC-based data acquisition hardware in the CPT truck. The sounding log is also displayed on a monitor.

ENVIRONMENTAL: SCIENTISTS • GEOLOGISTS • ENGINEERS
Contaminated Site Assessments • Phase I Audits • Site Remediation • Hazardous Waste Management

143 South Figueroa Street
Ventura, California 93001
(805) 652-0219
(805) 652-0793 FAX
Internet: first_last@hfa.com

5240 Lakeshore Drive
Cypress, California 90630
(714) 236-1741
(714) 236-1745 FAX
Internet: first_last@oc.hfa.com

2820 Pegasus Drive, Suite [illegible]
Bakersfield, California 93308
(661) 391-0517
(661) 391-0826 FAX
Internet: first_last@bak.hfa.com

3001 South 35th Street, Suite C-11
Phoenix, Arizona 85034
(800) 789-0249 • (602) 426-1000
(602) 426-0113 FAX
Internet: first_last@phx.hfa.com

The CPT data processing is performed using the truck mounted computer based data acquisition and presentation system. The computer generated graphical logs include cone resistance, friction resistance, friction ratio, and pore pressure ratio versus depth at a user selectable scale.

Soil behavior type interpretations are based on the following reference: Robertson, P.K. and Campanella, R.C., 1989, "Guidelines for Geotechnical Design using the Cone Penetrometer Test and CPT with Pore Pressure Measurement." Soil Mechanics series No. 120, Civil Engineering Department, University of British Columbia, Vancouver, B.C., V6T 1Z4, September 1989.

Interpretations and plotting has been done using HFA's proprietary data interpretation and presentation software. It is important to note that the data is not averaged. All interpretations are point interpretations at the corresponding depth listed.

It is also important to note that the soil behavior type correlations are based on a combination of theory, field research, research performed under laboratory conditions, and literature review. The information presented in the tabulated and/or graphical logs should,. therefore, be viewed as a guideline rather than as precise measurements.

Some care is recommended when using the soil behavior type interpretations. If a tabulation depth happens to fall on a soil layer interface, or a seam of soil differing from the rest of the layer, the tabulated data can be misleading. The solution to this problem is the proper use of the graphical CPT logs. The tip and sleeve penetration resistance logs are the primary source of profile description; the soil behavior type logs are supplemental. The graphical logs of tip and sleeve resistance should be examined and layer boundaries delineated in accordance with the project requirements. The soil behavior type interpretations are only representative of the response of the soil to the large shear deformations imposed during cone penetration. This is not necessarily a prediction of grain size distribution. However, it has been found that the interpreted soil behavior types generally agree well with the soil types defined in accordance with the grain size distribution methods such as used in the Unified Soil Classification System.

Limitations

Holguin, Fahan & Associates, Inc. (HFA) presents the attached data in accordance with ASTM Standards D5778-95 and D3441-94 and generally accepted cone penetrometer testing practices and standards. The attached data further relates only to the specific project and location discussed in the data. Judgement may be required to verify the CPT soil behavior interpretations.

The "Client" may distribute this data or excerpts therefrom provided the following statement is prominently displayed and included with the distribution:

> "Neither CLIENT nor HFA make any guarantee or warranty, express or implied, regarding this data. THE USE OF THIS INFORMATION SHALL BE AT THE USER'S SOLE RISK REGARDLESS OF ANY FAULT OR NEGLIGENCE OF THE CLIENT OR HFA."

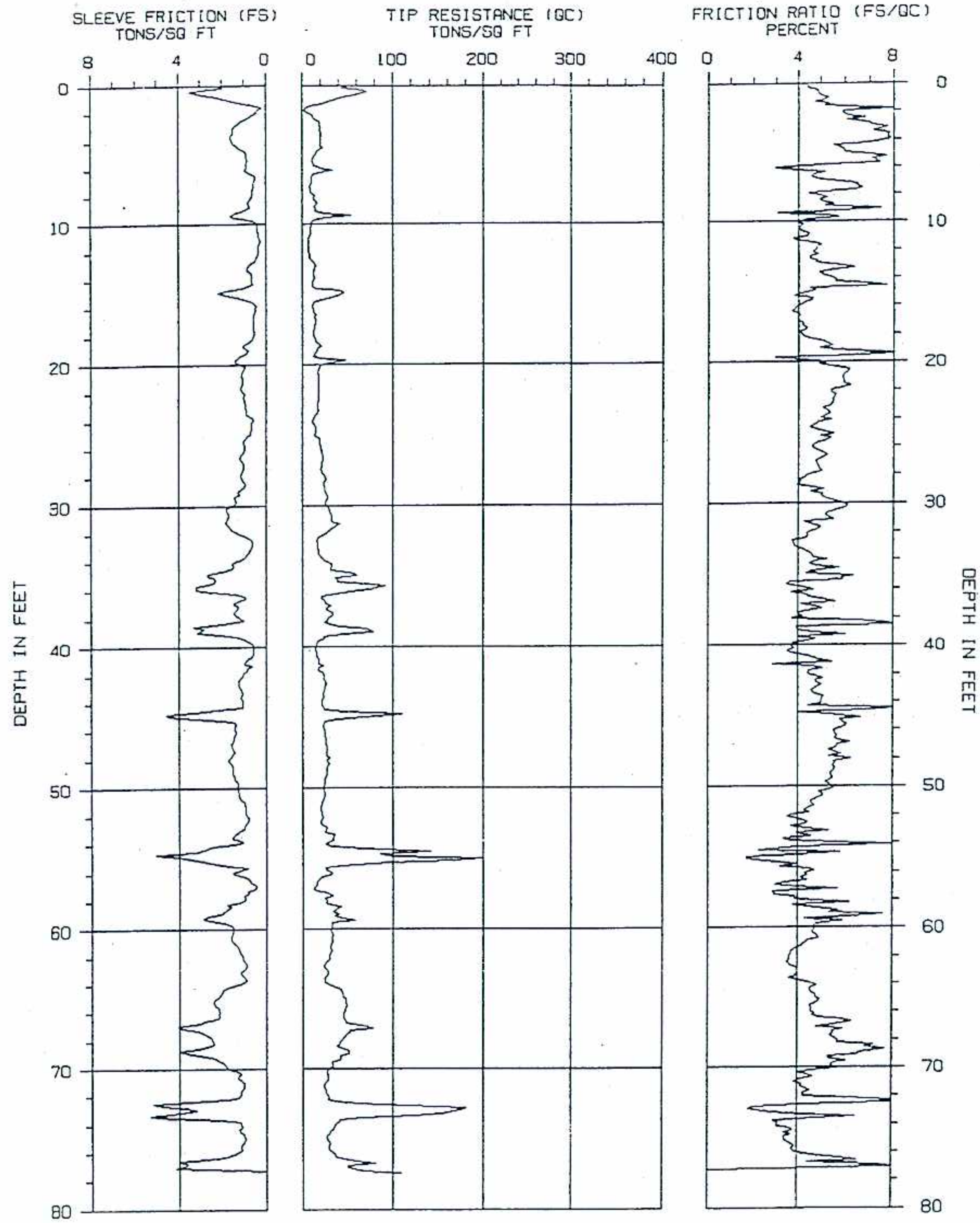
Please feel free to call if you have any questions.

Respectfully submitted,

Dick Carlton

Dick Carlton
CPT Operations Manager
Holguin, Fahan & Associates, Inc.

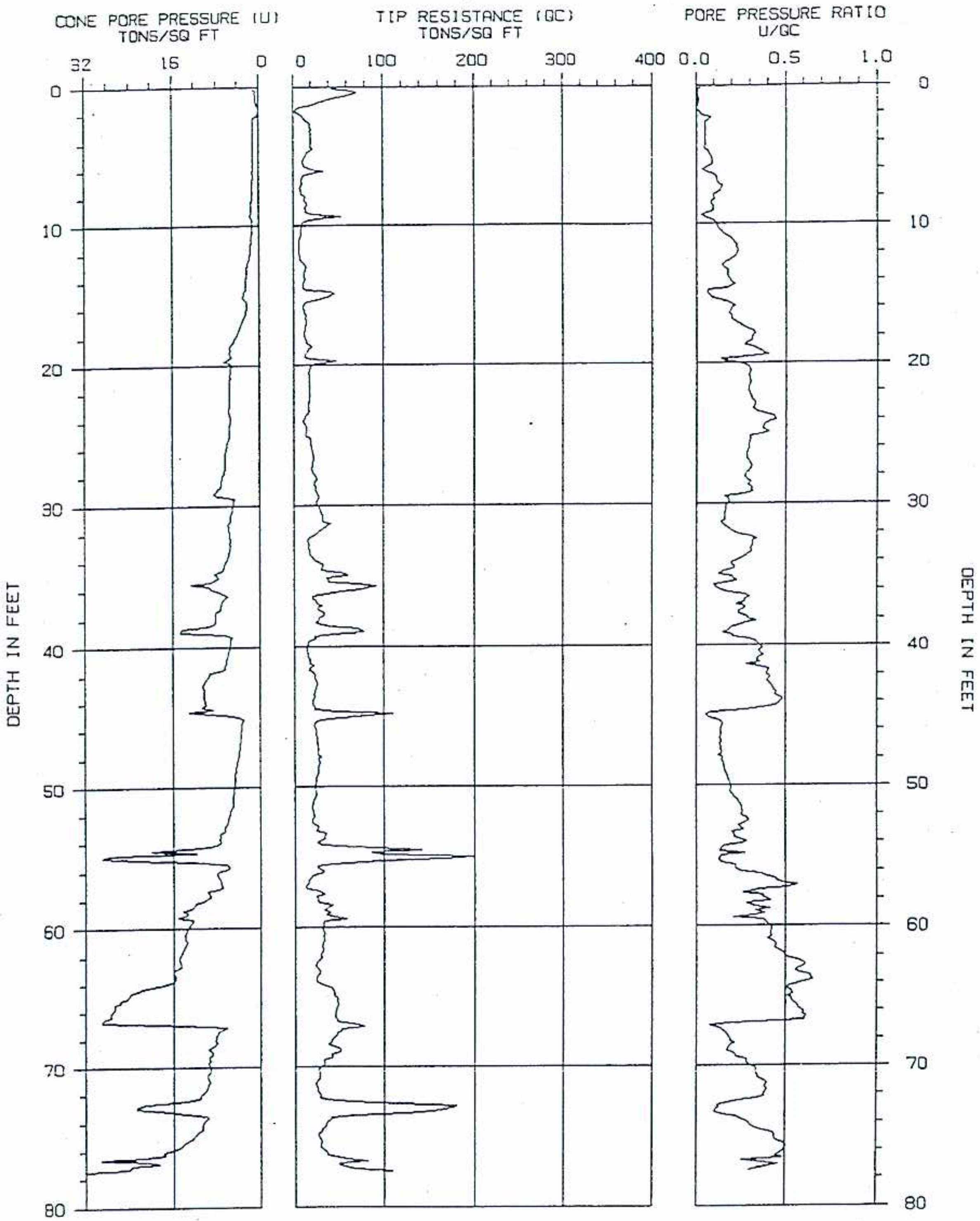
:DC\Enclosures



TIP RESISTANCE NOT CORRECTED FOR END AREA EFFECT

CONE PENETRATION TEST	SOUNDING NUMBER: HLA-1
PROJECT NAME : HLA/BISHOP RH 1	CONE/RIG : 491/BH.VO/R#4
PROJECT NUMBER : 50044.1	DATE/TIME: 04-13-00 07:51

HFA



TIP RESISTANCE NOT CORRECTED FOR END AREA EFFECT

CONE PENETRATION TEST

SOUNDING NUMBER: HLA-1

PROJECT NAME : HLA/BISHOP RH 1

PROJECT NUMBER : 50044.1

CONE/RIG : 491/BH.V0/R#4

DATE/TIME: 04-13-00 07:51

CPT INTERPRETATIONS

SOUNDING : HLA-1
PROJECT : HLA/BISHOP RH 1
DATE/TIME: 04-13-00 07:51

PROJECT No.: 50044.1
CONE/RIG : 491/BH,VO/R#4

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
.150	.49	71.51	4.83	*VERY STIFF FINE GRAINED	72	100			
.300	.98	35.48	5.29	CLAY	35	57		2.1	
.450	1.48	9.71	5.33	CLAY	10	16		.6	
.600	1.97	7.22	5.93	CLAY	7	12		.5	
.750	2.46	14.49	6.82	CLAY	14	23		1.0	
.900	2.95	20.46	7.10	CLAY	20	33		1.4	
1.050	3.44	20.78	7.18	CLAY	21	33		1.4	
1.200	3.94	19.89	7.86	CLAY	20	32		1.3	
1.350	4.43	22.88	5.53	CLAY	23	37		1.3	
1.500	4.92	14.36	6.02	CLAY	14	23		.9	
1.650	5.41	11.96	7.12	CLAY	12	19		.8	
1.800	5.91	21.54	4.12	CLAY to SILTY CLAY	14	23		1.4	
1.950	6.40	11.75	5.15	CLAY	12	19		.8	
2.100	6.89	10.37	5.03	CLAY	10	17		.7	
2.250	7.38	9.09	6.47	CLAY	9	14		.6	
2.400	7.87	14.04	4.45	CLAY	14	22		.9	
2.550	8.37	15.70	4.95	CLAY	16	24		1.0	
2.700	8.86	14.74	5.16	CLAY	15	22		1.0	
2.850	9.35	53.47	3.02	SANDY SILT to CLAYEY SILT	21	32		3.5	
3.000	9.84	10.75	4.39	CLAY	11	16		.7	
3.150	10.33	9.79	4.09	CLAY	10	14		.6	
3.300	10.83	8.37	4.44	CLAY	8	12		.5	
3.450	11.32	7.10	3.98	CLAY	7	10		.4	
3.600	11.81	6.99	4.86	CLAY	7	10		.4	
3.750	12.30	8.35	4.84	CLAY	8	12		.5	
3.900	12.80	13.43	4.86	CLAY	13	18		.8	
4.050	13.29	12.96	6.37	CLAY	13	17		.8	
4.200	13.78	13.24	5.04	CLAY	13	18		.8	
4.350	14.27	11.13	5.60	CLAY	11	15		.7	
4.500	14.76	40.94	4.51	CLAY to SILTY CLAY	27	36		2.4	
4.650	15.26	28.66	3.82	CLAY to SILTY CLAY	19	25		1.9	
4.800	15.75	11.60	4.28	CLAY	12	15		.7	
4.950	16.24	12.87	3.82	CLAY	13	16		.8	
5.100	16.73	14.68	4.03	CLAY	15	18		.9	
5.250	17.22	13.94	4.16	CLAY	14	17		.9	
5.400	17.72	12.30	4.33	CLAY	12	15		.8	
5.550	18.21	15.15	4.34	CLAY	15	19		.9	
5.700	18.70	20.14	5.27	CLAY	20	24		1.1	
5.850	19.19	15.11	5.65	CLAY	15	18		.9	
6.000	19.69	47.97	2.95	SANDY SILT to CLAYEY SILT	19	23		3.1	
6.150	20.18	19.25	4.88	CLAY	19	23		1.2	

*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 110 pcf
ASSUMED DEPTH OF WATER TABLE = 7.5 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

Interpretations based on: Robertson and Campanella, 1989.

SOUNDING : HLA-1

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
6.300	20.67	17.95	6.12	CLAY	18	21		1.1	
6.450	21.16	17.97	5.91	CLAY	18	21		1.1	
6.600	21.65	18.65	6.22	CLAY	19	22		1.0	
6.750	22.15	18.06	5.40	CLAY	18	21		1.1	
6.900	22.64	17.46	5.47	CLAY	17	20		1.1	
7.050	23.13	17.17	5.40	CLAY	17	19		1.1	
7.200	23.62	14.00	5.38	CLAY	14	16		.8	
7.350	24.11	11.92	5.43	CLAY	12	13		.7	
7.500	24.61	14.98	4.51	CLAY	15	17		.9	
7.650	25.10	15.02	5.50	CLAY	15	16		.9	
7.800	25.59	19.76	5.13	CLAY	20	22		1.2	
7.950	26.08	21.05	4.61	CLAY	21	23		1.3	
8.100	26.57	22.50	5.24	CLAY	23	24		1.2	
8.250	27.07	21.20	4.76	CLAY	21	23		1.3	
8.400	27.56	21.33	4.95	CLAY	21	23		1.3	
8.550	28.05	25.79	4.37	CLAY to SILTY CLAY	17	18		1.4	
8.700	28.54	23.24	4.09	CLAY to SILTY CLAY	15	16		1.4	
8.850	29.04	25.43	5.12	CLAY	25	26		1.4	
9.000	29.53	28.00	5.01	CLAY	28	29		1.6	
9.150	30.02	27.36	5.89	CLAY	27	28		1.5	
9.300	30.51	30.32	5.68	CLAY	30	31		1.7	
9.450	31.00	33.10	5.38	CLAY	33	34		1.8	
9.600	31.50	36.84	4.55	CLAY to SILTY CLAY	25	25		2.1	
9.750	31.99	26.30	4.48	CLAY to SILTY CLAY	18	18		1.4	
9.900	32.48	15.70	4.35	CLAY	16	16		.9	
10.050	32.97	17.38	3.84	CLAY to SILTY CLAY	12	11		1.0	
10.200	33.46	18.44	4.46	CLAY	18	18		1.1	
10.350	33.96	26.94	5.25	CLAY	27	26		1.5	
10.500	34.45	31.00	4.80	CLAY	31	30		1.7	
10.650	34.94	60.78	4.35	CLAYEY SILT to SILTY CLAY	30	29		3.5	
10.800	35.43	39.37	5.93	CLAY	39	38		2.2	
10.950	35.93	75.48	4.24	CLAYEY SILT to SILTY CLAY	38	36		4.3	
11.100	36.42	21.92	4.35	CLAY to SILTY CLAY	15	14		1.3	
11.250	36.91	26.02	5.62	CLAY	26	25		1.4	
11.400	37.40	27.62	5.02	CLAY	28	26		1.5	
11.550	37.89	27.83	4.18	CLAY to SILTY CLAY	19	17		1.5	
11.700	38.39	31.65	7.55	CLAY	32	30		1.7	
11.850	38.88	78.14	4.01	CLAYEY SILT to SILTY CLAY	39	36		4.5	
12.000	39.37	24.98	4.01	CLAY to SILTY CLAY	17	15		1.5	
12.150	39.86	15.81	3.78	CLAY to SILTY CLAY	11	10		.9	
12.300	40.35	16.49	3.52	CLAY to SILTY CLAY	11	10		1.0	
12.450	40.85	17.65	4.56	CLAY	18	16		1.0	
12.600	41.34	22.77	2.87	CLAYEY SILT to SILTY CLAY	11	10		1.4	
12.750	41.83	23.03	4.46	CLAY	23	21		1.4	
12.900	42.32	24.75	5.05	CLAY	25	22		1.3	
13.050	42.81	25.05	5.00	CLAY	25	22		1.3	
13.200	43.31	22.90	4.63	CLAY	23	20		1.4	
13.350	43.80	20.97	5.07	CLAY	21	19		1.1	
13.500	44.29	23.79	4.47	CLAY	24	21		1.4	
13.650	44.78	110.47	4.12	CLAYEY SILT to SILTY CLAY	55	48		6.4	

*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 110 pcf
ASSUMED DEPTH OF WATER TABLE = 7.5 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

SOUNDING : HLA-1

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
13.800	45.28	25.64	5.83	CLAY	26	22		1.4	
13.950	45.77	23.62	6.03	CLAY	24	21		1.2	
14.100	46.26	25.26	5.58	CLAY	25	22		1.3	
14.250	46.75	26.05	5.78	CLAY	26	22		1.4	
14.400	47.24	26.77	5.62	CLAY	27	23		1.4	
14.550	47.74	26.68	5.87	CLAY	27	23		1.4	
14.700	48.23	29.06	5.55	CLAY	29	25		1.6	
14.850	48.72	27.17	5.59	CLAY	27	23		1.4	
15.000	49.21	26.36	5.42	CLAY	26	22		1.4	
15.150	49.70	24.47	5.18	CLAY	24	21		1.3	
15.300	50.20	23.56	5.55	CLAY	24	20		1.2	
15.450	50.69	23.28	5.07	CLAY	23	19		1.2	
15.600	51.18	21.07	4.57	CLAY	21	17		1.2	
15.750	51.67	21.29	4.29	CLAY to SILTY CLAY	14	12		1.2	
15.900	52.17	22.56	3.50	CLAYEY SILT to SILTY CLAY	11	9		1.3	
16.050	52.66	22.73	4.33	CLAY to SILTY CLAY	15	12		1.3	
16.200	53.15	24.43	5.36	CLAY	24	20		1.3	
16.350	53.64	35.20	3.35	CLAYEY SILT to SILTY CLAY	18	14		2.1	
16.500	54.13	31.82	7.94	CLAY	32	26		1.7	
16.650	54.63	87.12	5.81	*VERY STIFF FINE GRAINED	87	70			
16.800	55.12	167.90	1.75	SAND to SILTY SAND	42	34	76		40.5
16.950	55.61	25.94	3.21	CLAYEY SILT to SILTY CLAY	13	10		1.5	
17.100	56.10	31.23	4.32	CLAY to SILTY CLAY	21	17		1.7	
17.250	56.59	16.02	4.41	CLAY	16	13		.9	
17.400	57.09	12.32	3.98	CLAY	12	10		.6	
17.550	57.58	34.16	2.93	CLAYEY SILT to SILTY CLAY	17	13		2.1	
17.700	58.07	25.62	4.77	CLAY	26	20		1.3	
17.850	58.56	37.43	4.86	CLAY	37	29		2.0	
18.000	59.06	35.56	7.57	CLAY	36	28		1.9	
18.150	59.55	32.67	5.98	CLAY	33	25		1.7	
18.300	60.04	31.85	4.75	CLAY	32	25		1.7	
18.450	60.53	33.08	4.86	CLAY	33	26		1.8	
18.600	61.02	32.04	4.38	CLAY to SILTY CLAY	21	16		1.7	
18.750	61.52	32.12	3.88	CLAYEY SILT to SILTY CLAY	16	12		1.7	
18.900	62.01	30.00	3.60	CLAYEY SILT to SILTY CLAY	15	11		1.8	
19.050	62.50	24.64	3.51	CLAYEY SILT to SILTY CLAY	12	9		1.4	
19.200	62.99	28.19	4.01	CLAY to SILTY CLAY	19	14		1.6	
19.350	63.48	24.54	3.91	CLAY to SILTY CLAY	16	12		1.4	
19.500	63.98	29.42	4.63	CLAY to SILTY CLAY	20	15		1.5	
19.650	64.47	43.57	4.55	CLAY to SILTY CLAY	29	22		2.4	
19.800	64.96	47.95	4.82	CLAY to SILTY CLAY	32	24		2.6	
19.950	65.45	49.10	4.80	CLAY to SILTY CLAY	33	24		2.7	
20.100	65.94	46.29	4.57	CLAY to SILTY CLAY	31	23		2.5	
20.250	66.44	45.83	4.95	CLAY to SILTY CLAY	31	23		2.5	
20.400	66.93	71.89	5.60	*VERY STIFF FINE GRAINED	72	53			
20.550	67.42	52.13	5.51	CLAY	52	38		2.8	
20.700	67.91	43.89	5.67	CLAY	44	32		2.4	
20.850	68.41	37.88	7.26	CLAY	38	28		2.0	
21.000	68.90	51.75	6.61	CLAY	52	38		2.8	
21.150	69.39	41.00	5.38	CLAY	41	30		2.2	

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*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL

ASSUMED TOTAL UNIT WT = 110 pcf

ASSUMED DEPTH OF WATER TABLE = 7.5 ft

N(60) = EQUIVALENT SPT VALUE (60% Energy)

N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)

Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY

Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH

PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

Interpretations based on: Robertson and Campanella, 1989.

SOUNDING : HLA-1

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
21.300	69.88	33.38	5.47	CLAY	33	24		1.7	
21.450	70.37	28.40	3.98	CLAY to SILTY CLAY	19	14		1.6	
21.600	70.87	27.64	4.20	CLAY to SILTY CLAY	18	13		1.4	
21.750	71.36	24.88	4.15	CLAY to SILTY CLAY	17	12		1.4	
21.900	71.85	26.79	4.26	CLAY to SILTY CLAY	18	13		1.3	
22.050	72.34	35.97	9.56	CLAY	36	26		1.9	
22.200	72.83	179.86	2.01	SILTY SAND to SANDY SILT	60	43	74		39.5
22.350	73.33	124.77	4.26	*VERY STIFF FINE GRAINED	100	89			
22.500	73.82	40.77	2.96	CLAYEY SILT to SILTY CLAY	20	14		2.4	
22.650	74.31	33.95	3.70	CLAYEY SILT to SILTY CLAY	17	12		2.0	
22.800	74.80	27.21	3.68	CLAYEY SILT to SILTY CLAY	14	10		1.5	
22.950	75.30	29.89	3.57	CLAYEY SILT to SILTY CLAY	15	10		1.7	
23.100	75.79	29.32	3.73	CLAYEY SILT to SILTY CLAY	15	10		1.7	
23.250	76.28	37.28	4.94	CLAY	37	26		1.9	
23.400	76.77	80.88	4.45	CLAYEY SILT to SILTY CLAY	40	28		4.5	
23.550	77.26	67.98	*****		0	0			.0

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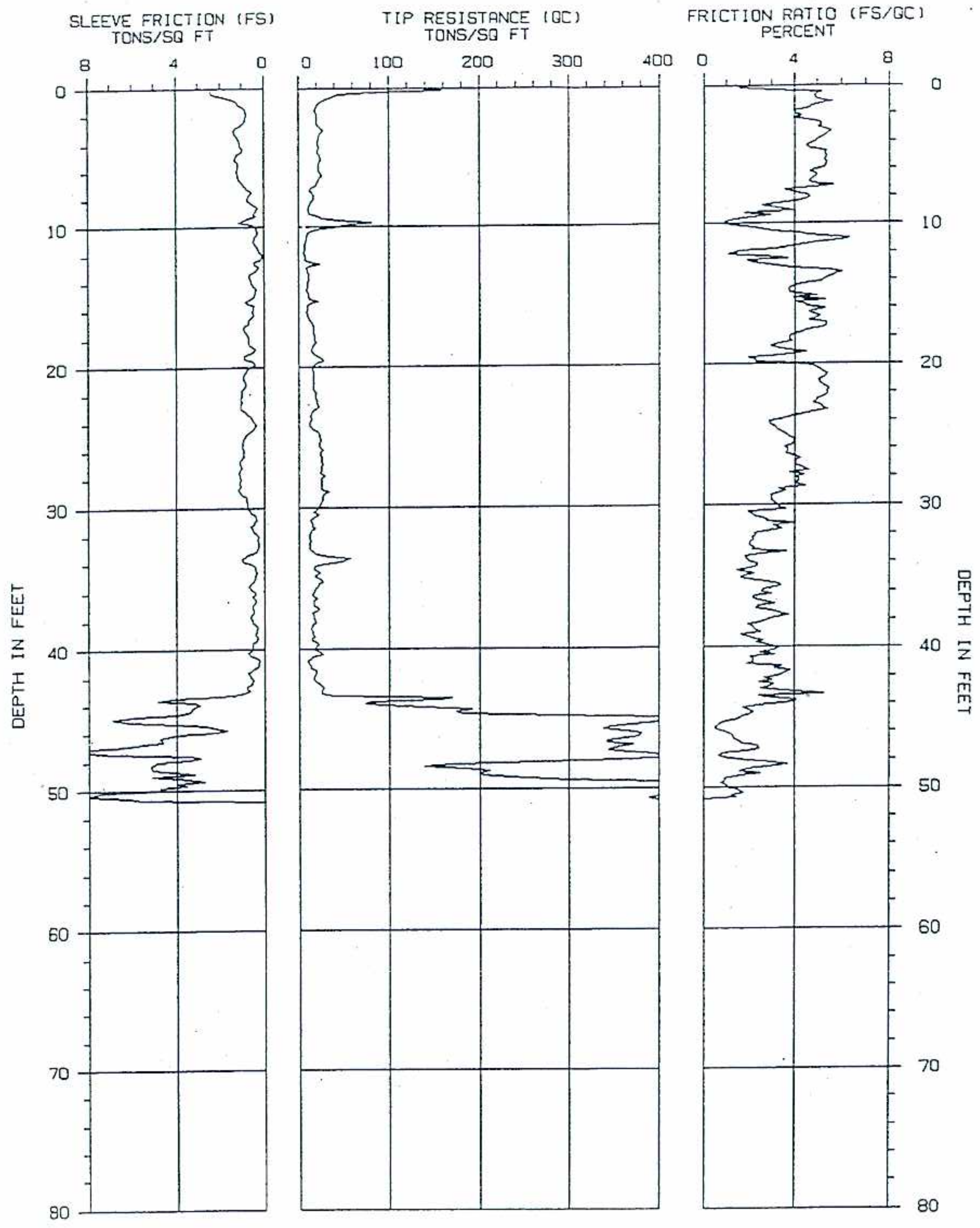
*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 110 pcf
ASSUMED DEPTH OF WATER TABLE = 7.5 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

HOLGUIN, FAHAN & ASSOCIATES, INC.

Interpretations based on: Robertson and Campanella, 1989.



TIP RESISTANCE NOT CORRECTED FOR END AREA EFFECT

CONE PENETRATION TEST

SOUNDING NUMBER: HLA-2

PROJECT NAME : HLA/BISHOP RH 1

PROJECT NUMBER : 50044.1

CONE/RIG : 491/BH.VQ/R#4

DATE/TIME: 04-12-00 07:36

HFA

CPT INTERPRETATIONS

SOUNDING : HLA-2
PROJECT : HLA/BISHOP RH 1
DATE/TIME: 04-12-00 07:36

PROJECT No.: 50044.1
CONE/RIG : 491/BH,VO/R#4

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
.150	.49	45.08	5.18	CLAY	45	72		2.7	
.300	.98	24.81	5.08	CLAY	25	40		1.5	
.450	1.48	18.84	4.76	CLAY	19	30		1.3	
.600	1.97	20.40	4.01	CLAY to SILTY CLAY	14	22		1.4	
.750	2.46	20.73	4.52	CLAY	21	33		1.4	
.900	2.95	27.43	5.02	CLAY	27	44		1.6	
1.050	3.44	22.90	5.41	CLAY	23	37		1.3	
1.200	3.94	22.05	4.94	CLAY	22	35		1.3	
1.350	4.43	21.52	4.53	CLAY	22	34		1.4	
1.500	4.92	24.79	5.27	CLAY	25	40		1.4	
1.650	5.41	22.99	5.30	CLAY	23	37		1.3	
1.800	5.91	23.01	5.30	CLAY	23	37		1.3	
1.950	6.40	24.28	4.95	CLAY	24	39		1.4	
2.100	6.89	20.10	4.61	CLAY	20	32		1.3	
2.250	7.38	12.83	4.84	CLAY	13	20		.8	
2.400	7.87	16.85	4.60	CLAY	17	26		1.1	
2.550	8.37	12.58	4.03	CLAY	13	19		.8	
2.700	8.86	12.00	3.06	CLAY to SILTY CLAY	8	12		.8	
2.850	9.35	25.85	2.93	CLAYEY SILT to SILTY CLAY	13	19		1.7	
3.000	9.84	58.27	.93	SILTY SAND to SANDY SILT	19	28	61		41.5
3.150	10.33	12.47	2.48	CLAYEY SILT to SILTY CLAY	6	9		1.0	
3.300	10.83	9.65	4.63	CLAY	10	14		.6	
3.450	11.32	6.95	4.99	CLAY	7	10		.4	
3.600	11.81	6.48	3.16	CLAY	6	9		.4	
3.750	12.30	7.24	2.68	CLAY to SILTY CLAY	5	7		.5	
3.900	12.80	13.00	2.17	CLAYEY SILT to SILTY CLAY	7	9		1.0	
4.050	13.29	10.96	5.47	CLAY	11	15		.7	
4.200	13.78	11.11	5.51	CLAY	11	15		.7	
4.350	14.27	10.16	4.33	CLAY	10	13		.6	
4.500	14.76	10.96	3.74	CLAY	11	14		.7	
4.650	15.26	21.16	3.97	CLAY to SILTY CLAY	14	18		1.4	
4.800	15.75	10.47	4.46	CLAY	10	13		.6	
4.950	16.24	10.92	4.60	CLAY	11	14		.7	
5.100	16.73	14.30	4.94	CLAY	14	18		.9	
5.250	17.22	17.51	5.33	CLAY	18	22		1.1	
5.400	17.72	17.80	4.33	CLAY	18	22		1.1	
5.550	18.21	18.76	3.80	CLAY to SILTY CLAY	13	15		1.2	
5.700	18.70	14.34	2.93	CLAY to SILTY CLAY	10	12		.9	
5.850	19.19	19.33	4.51	CLAY	19	23		1.2	
6.000	19.69	20.08	2.27	CLAYEY SILT to SILTY CLAY	10	12		1.5	
6.150	20.18	15.83	4.80	CLAY	16	19		1.0	

*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 110 pcf
ASSUMED DEPTH OF WATER TABLE = 7.5 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

Interpretations based on: Robertson and Campanella, 1989.

SOUNDING : HLA-2

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
6.300	20.67	17.06	5.36	CLAY	17	20		1.1	
6.450	21.16	17.02	5.02	CLAY	17	20		1.1	
6.600	21.65	18.50	5.40	CLAY	19	21		1.2	
6.750	22.15	19.31	5.36	CLAY	19	22		1.2	
6.900	22.64	21.27	4.93	CLAY	21	24		1.3	
7.050	23.13	17.14	5.15	CLAY	17	19		1.1	
7.200	23.62	14.34	4.16	CLAY	14	16		.9	
7.350	24.11	12.47	2.87	CLAY to SILTY CLAY	8	9		.7	
7.500	24.61	21.24	3.19	CLAYEY SILT to SILTY CLAY	11	12		1.3	
7.650	25.10	25.05	3.70	CLAY to SILTY CLAY	17	18		1.6	
7.800	25.59	24.22	3.97	CLAY to SILTY CLAY	16	18		1.5	
7.950	26.08	24.96	3.69	CLAY to SILTY CLAY	17	18		1.6	
8.100	26.57	25.71	4.10	CLAY to SILTY CLAY	17	18		1.6	
8.250	27.07	25.94	4.06	CLAY to SILTY CLAY	17	18		1.6	
8.400	27.56	24.88	4.60	CLAY	25	26		1.4	
8.550	28.05	26.15	4.05	CLAY to SILTY CLAY	17	18		1.6	
8.700	28.54	26.32	4.02	CLAY to SILTY CLAY	18	18		1.7	
8.850	29.04	28.66	3.61	CLAYEY SILT to SILTY CLAY	14	15		1.8	
9.000	29.53	26.51	3.00	CLAYEY SILT to SILTY CLAY	13	14		1.7	
9.150	30.02	22.50	3.29	CLAYEY SILT to SILTY CLAY	11	12		1.4	
9.300	30.51	22.26	1.96	SANDY SILT to CLAYEY SILT	9	9		1.6	
9.450	31.00	15.23	2.78	CLAYEY SILT to SILTY CLAY	8	8		.9	
9.600	31.50	17.91	3.07	CLAYEY SILT to SILTY CLAY	9	9		1.1	
9.750	31.99	12.11	2.25	CLAYEY SILT to SILTY CLAY	6	6		.8	
9.900	32.48	13.11	2.17	CLAYEY SILT to SILTY CLAY	7	7		.9	
10.050	32.97	14.11	2.21	CLAYEY SILT to SILTY CLAY	7	7		1.0	
10.200	33.46	33.27	2.51	SANDY SILT to CLAYEY SILT	13	13		2.1	
10.350	33.96	29.15	2.02	SANDY SILT to CLAYEY SILT	12	11		2.2	
10.500	34.45	18.74	2.03	CLAYEY SILT to SILTY CLAY	9	9		1.3	
10.650	34.94	21.92	1.80	SANDY SILT to CLAYEY SILT	9	8		1.6	
10.800	35.43	22.35	2.79	CLAYEY SILT to SILTY CLAY	11	11		1.4	
10.950	35.93	17.59	2.63	CLAYEY SILT to SILTY CLAY	9	8		1.0	
11.100	36.42	19.86	2.25	CLAYEY SILT to SILTY CLAY	10	9		1.4	
11.250	36.91	16.72	3.11	CLAYEY SILT to SILTY CLAY	8	8		1.0	
11.400	37.40	16.42	2.96	CLAYEY SILT to SILTY CLAY	8	8		1.0	
11.550	37.89	18.67	2.92	CLAYEY SILT to SILTY CLAY	9	9		1.1	
11.700	38.39	15.06	1.97	CLAYEY SILT to SILTY CLAY	8	7		1.0	
11.850	38.88	16.42	2.48	CLAYEY SILT to SILTY CLAY	8	8		1.0	
12.000	39.37	15.87	2.16	CLAYEY SILT to SILTY CLAY	8	7		1.1	
12.150	39.86	19.63	2.79	CLAYEY SILT to SILTY CLAY	10	9		1.2	
12.300	40.35	26.96	2.51	CLAYEY SILT to SILTY CLAY	13	12		1.6	
12.450	40.85	10.86	2.17	CLAYEY SILT to SILTY CLAY	5	5		.7	
12.600	41.34	13.53	3.41	CLAY to SILTY CLAY	9	8		.8	
12.750	41.83	17.91	3.60	CLAY to SILTY CLAY	12	11		1.0	
12.900	42.32	19.08	3.04	CLAYEY SILT to SILTY CLAY	10	9		1.1	
13.050	42.81	27.62	2.89	CLAYEY SILT to SILTY CLAY	14	12		1.7	
13.200	43.31	38.81	5.26	CLAY	39	35		2.1	
13.350	43.80	77.90	4.10	CLAYEY SILT to SILTY CLAY	39	34		4.4	
13.500	44.29	190.78	1.73	SAND to SILTY SAND	48	42	82		42.5
13.650	44.78	289.55	2.04	SAND to SILTY SAND	72	63	94		44.0

*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 110 pcf
ASSUMED DEPTH OF WATER TABLE = 7.5 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

SOUNDING : HLA-2

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
13.800	45.28	398.77	1.01	SAND	80	70	100		45.5
13.950	45.77	338.71	.51	GRAVELLY SAND to SAND	56	49	98		44.5
14.100	46.26	378.77	1.25	SAND	76	66	100		45.0
14.250	46.75	342.68	1.61	SAND to SILTY SAND	86	74	98		44.5
14.400	47.24	344.46	2.47	SILTY SAND to SANDY SILT	100	99	98		44.5
14.550	47.74	440.45	.66	GRAVELLY SAND to SAND	73	63	100		46.0
14.700	48.23	169.62	3.02	SANDY SILT to CLAYEY SILT	68	58		9.8	
14.850	48.72	211.45	1.93	SAND to SILTY SAND	53	45	84		42.5
15.000	49.21	233.10	1.67	SAND to SILTY SAND	58	49	86		43.0
15.150	49.70	467.87	.76	GRAVELLY SAND to SAND	78	66	100		46.0
15.300	50.20	494.86	1.51	SAND	99	83	100		46.0
15.450	50.69	389.23	1.43	SAND	78	65	100		45.0

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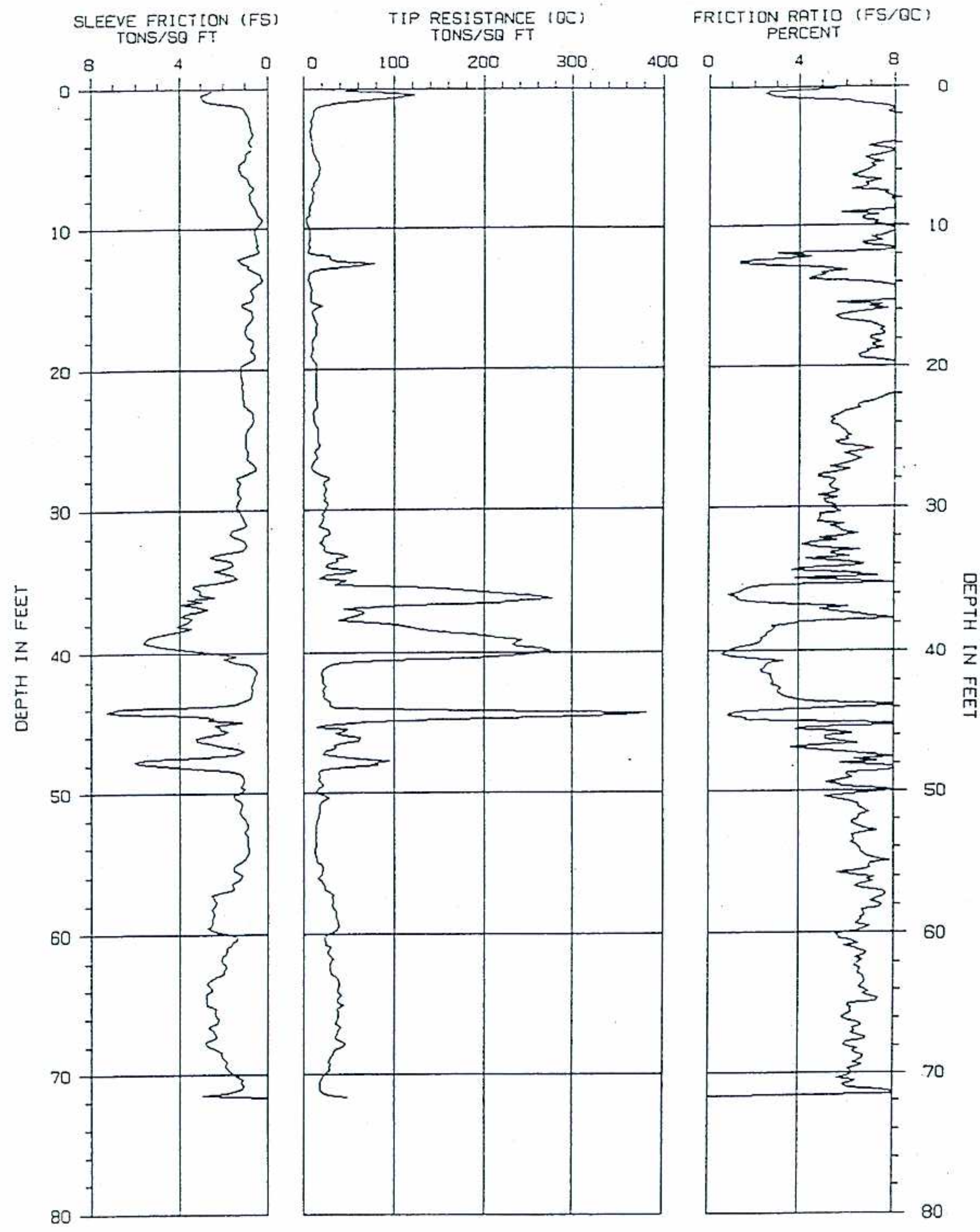
*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 110 pcf
ASSUMED DEPTH OF WATER TABLE = 7.5 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

HOLGUIN, FAHAN & ASSOCIATES, INC.

Interpretations based on: Robertson and Campanella, 1989.



TIP RESISTANCE NOT CORRECTED FOR END AREA EFFECT

CONE PENETRATION TEST

SOUNDING NUMBER: HLA-3

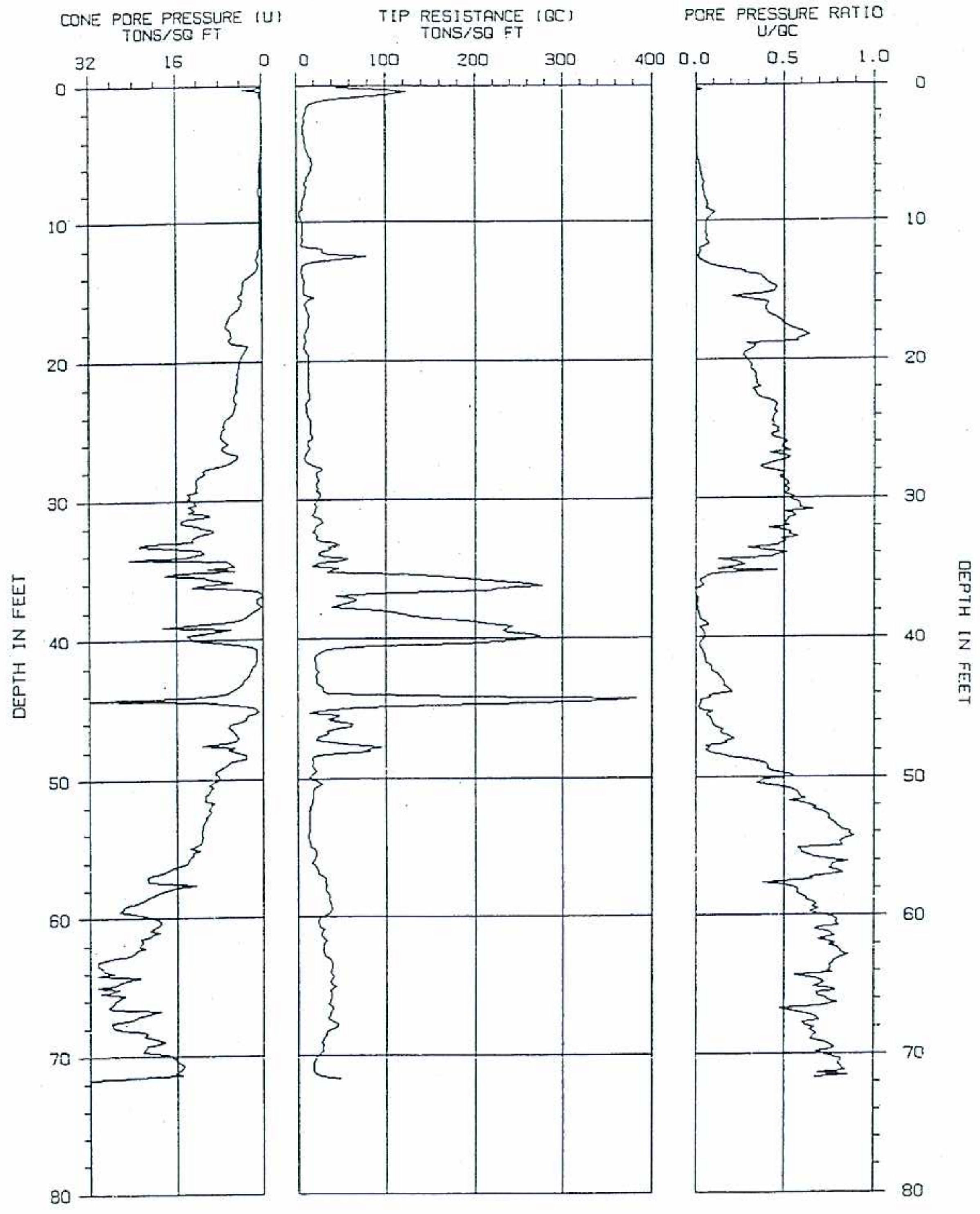
PROJECT NAME : HLA/BISHOP RH 1

PROJECT NUMBER : 50044.1

CONE/RIG : 491/BH.V0/R#4

DATE/TIME: 04-13-00 09:26

HFA



TIP RESISTANCE NOT CORRECTED FOR END AREA EFFECT

CONE PENETRATION TEST

SOUNDING NUMBER: HLA-3

PROJECT NAME : HLA/BISHOP RH 1

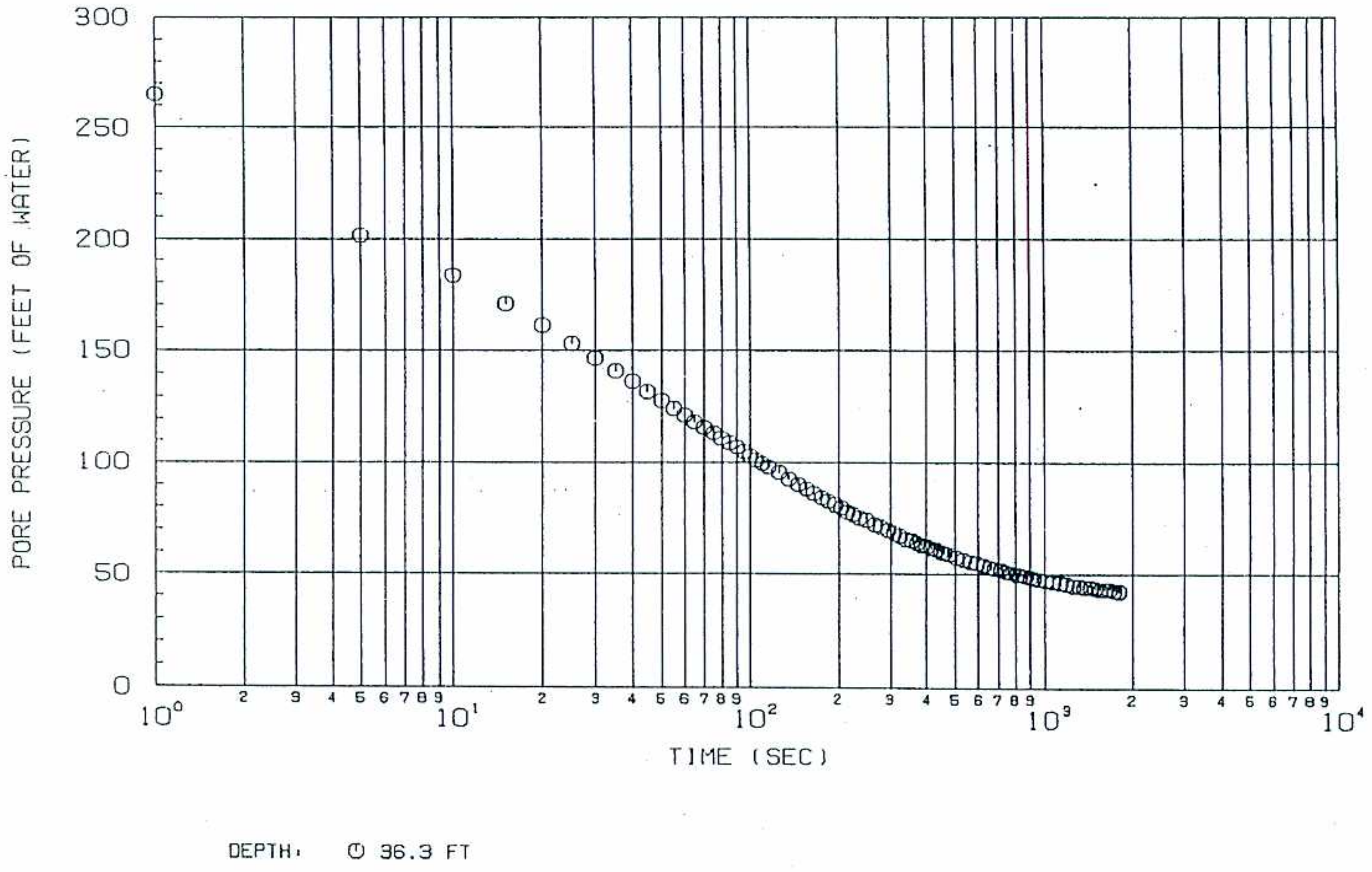
PROJECT NUMBER : 50044.1

CONE/RIG : 491/BH.VO/R#4

DATE/TIME: .04-13-00 09:26

HFA

PORE PRESSURE DISSIPATION CURVES



TIP-SENSING PIEZOMETRIC CPT

SOUNDING NUMBER: HLA-3

PROJECT NAME : HLA/BISHOP RH 1

PROJECT NUMBER : 50044.1

CONE/RIG : 491/BH.V0/R¤4

DATE/TIME: 04-13-00 09:26

HFA

CPT INTERPRETATIONS

SOUNDING : HLA-3
PROJECT : HLA/BISHOP RH 1
DATE/TIME: 04-13-00 09:26

PROJECT No.: 50044.1
CONE/RIG : 491/BH,VO/R#4

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
.150	.49	122.22	2.48	SILTY SAND to SANDY SILT	41	65	82		
.300	.98	40.17	6.15	CLAY	40	64		2.4	
.450	1.48	13.62	7.86	CLAY	14	22		.9	
.600	1.97	10.64	8.27	CLAY	11	17		.7	
.750	2.46	9.24	8.48	CLAY	9	15		.6	
.900	2.95	7.48	9.83	CLAY	7	12		.5	
1.050	3.44	8.52	8.29	CLAY	9	14		.6	
1.200	3.94	9.82	8.01	CLAY	10	16		.6	
1.350	4.43	10.56	7.54	CLAY	11	17		.7	
1.500	4.92	13.34	7.33	CLAY	13	21		.9	
1.650	5.41	17.36	7.52	CLAY	17	28		1.1	
1.800	5.91	17.59	6.94	CLAY	18	28		1.2	
1.950	6.40	14.89	6.24	CLAY	15	24		1.0	
2.100	6.89	11.17	6.79	CLAY	11	18		.7	
2.250	7.38	11.92	6.24	CLAY	12	19		.8	
2.400	7.87	10.13	7.97	CLAY	10	16		.6	
2.550	8.37	7.54	8.27	CLAY	8	11		.5	
2.700	8.86	5.99	8.31	CLAY	6	9		.4	
2.850	9.35	3.36	6.65	ORGANIC MATERIAL	3	5		.3	
3.000	9.84	6.08	7.13	CLAY	6	9		.4	
3.150	10.33	6.56	8.59	CLAY	7	9		.4	
3.300	10.83	7.03	7.08	CLAY	7	10		.4	
3.450	11.32	6.88	6.79	CLAY	7	10		.4	
3.600	11.81	7.84	7.52	CLAY	8	11		.5	
3.750	12.30	36.73	3.38	CLAYEY SILT to SILTY CLAY	18	25		2.4	
3.900	12.80	31.27	2.88	CLAYEY SILT to SILTY CLAY	16	21		2.0	
4.050	13.29	6.42	4.94	CLAY	6	9		.4	
4.200	13.78	6.84	4.40	CLAY	7	9		.4	
4.350	14.27	8.82	8.45	CLAY	9	12		.5	
4.500	14.76	7.73	8.51	CLAY	8	10		.5	
4.650	15.26	12.02	9.11	CLAY	12	16		.7	
4.800	15.75	11.43	6.93	CLAY	11	15		.7	
4.950	16.24	11.28	5.87	CLAY	11	14		.7	
5.100	16.73	14.36	6.21	CLAY	14	18		.9	
5.250	17.22	13.45	7.57	CLAY	13	17		.8	
5.400	17.72	11.54	7.56	CLAY	12	14		.7	
5.550	18.21	8.80	7.41	CLAY	9	11		.5	
5.700	18.70	9.88	7.49	CLAY	10	12		.6	
5.850	19.19	9.20	6.49	CLAY	9	11		.5	
6.000	19.69	14.21	8.09	CLAY	14	17		.9	
6.150	20.18	13.79	8.80	CLAY	14	16		.8	

*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 110 pcf
ASSUMED DEPTH OF WATER TABLE = 7.5 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

Interpretations based on: Robertson and Campanella, 1989.

SOUNDING : HLA-3

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
6.300	20.67	14.19	8.11	CLAY	14	17		.9	
6.450	21.16	13.45	8.42	CLAY	13	16		.8	
6.600	21.65	13.83	8.03	CLAY	14	16		.8	
6.750	22.15	14.30	7.59	CLAY	14	16		.9	
6.900	22.64	14.70	6.54	CLAY	15	17		.9	
7.050	23.13	10.83	6.18	CLAY	11	12		.6	
7.200	23.62	12.00	5.32	CLAY	12	13		.7	
7.350	24.11	14.83	5.39	CLAY	15	16		.9	
7.500	24.61	16.29	5.94	CLAY	16	18		1.0	
7.650	25.10	16.02	5.97	CLAY	16	18		1.0	
7.800	25.59	17.19	5.73	CLAY	17	19		1.1	
7.950	26.08	13.38	6.56	CLAY	13	14		.8	
8.100	26.57	11.83	6.60	CLAY	12	13		.7	
8.250	27.07	9.07	5.77	CLAY	9	10		.5	
8.400	27.56	20.16	5.53	CLAY	20	21		1.1	
8.550	28.05	22.88	5.49	CLAY	23	24		1.3	
8.700	28.54	25.37	5.28	CLAY	25	27		1.4	
8.850	29.04	23.07	5.46	CLAY	23	24		1.3	
9.000	29.53	26.28	5.08	CLAY	26	27		1.5	
9.150	30.02	24.47	5.40	CLAY	24	25		1.3	
9.300	30.51	22.97	4.91	CLAY	23	23		1.3	
9.450	31.00	19.33	4.79	CLAY	19	20		1.2	
9.600	31.50	28.47	5.68	CLAY	28	29		1.6	
9.750	31.99	21.29	5.86	CLAY	21	21		1.1	
9.900	32.48	20.97	4.53	CLAY	21	21		1.3	
10.050	32.97	26.34	6.58	CLAY	26	26		1.4	
10.200	33.46	36.35	6.14	CLAY	36	36		2.0	
10.350	33.96	24.58	6.76	CLAY	25	24		1.3	
10.500	34.45	53.75	3.70	CLAYEY SILT to SILTY CLAY	27	26		3.1	
10.650	34.94	47.12	3.81	CLAYEY SILT to SILTY CLAY	24	23		2.7	
10.800	35.43	117.19	2.88	SANDY SILT to CLAYEY SILT	47	45		6.8	
10.950	35.93	228.06	1.36	SAND to SILTY SAND	57	55	89		44.0
11.100	36.42	226.68	1.32	SAND to SILTY SAND	57	54	89		44.0
11.250	36.91	44.49	6.09	CLAY	44	42		2.5	
11.400	37.40	59.57	6.53	CLAY	60	56		3.4	
11.550	37.89	93.75	4.11	CLAYEY SILT to SILTY CLAY	47	44		5.4	
11.700	38.39	131.34	2.92	SANDY SILT to CLAYEY SILT	53	49		7.6	
11.850	38.88	216.78	2.52	SILTY SAND to SANDY SILT	72	67	87		43.5
12.000	39.37	231.91	2.26	SILTY SAND to SANDY SILT	77	71	89		43.5
12.150	39.86	273.82	1.14	SAND	55	50	93		44.5
12.300	40.35	187.83	1.05	SAND	38	34	82		42.5
12.450	40.85	30.04	2.68	CLAYEY SILT to SILTY CLAY	15	14		1.9	
12.600	41.34	20.76	2.32	CLAYEY SILT to SILTY CLAY	10	9		1.2	
12.750	41.83	21.31	2.75	CLAYEY SILT to SILTY CLAY	11	10		1.3	
12.900	42.32	25.03	2.75	CLAYEY SILT to SILTY CLAY	13	11		1.5	
13.050	42.81	22.43	3.09	CLAYEY SILT to SILTY CLAY	11	10		1.3	
13.200	43.31	25.83	3.64	CLAYEY SILT to SILTY CLAY	13	11		1.6	
13.350	43.80	30.66	8.39	CLAY	31	27		1.7	
13.500	44.29	383.11	1.71	SAND to SILTY SAND	96	84	100		45.5
13.650	44.78	157.57	1.69	SAND to SILTY SAND	39	35	76		41.5

*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 110 pcf
ASSUMED DEPTH OF WATER TABLE = 7.5 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

SOUNDING : HLA-3

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
13.800	45.28	15.28	13.92	CLAY	15	13		.9	
13.950	45.77	35.48	5.61	CLAY	35	31		1.9	
14.100	46.26	60.99	5.14	CLAY to SILTY CLAY	41	35		3.4	
14.250	46.75	35.39	4.67	CLAY to SILTY CLAY	24	20		1.9	
14.400	47.24	23.11	6.31	CLAY	23	20		1.2	
14.550	47.74	94.31	6.37	*VERY STIFF FINE GRAINED	94	81			
14.700	48.23	32.91	9.74	CLAY	33	28		1.8	
14.850	48.72	18.70	6.08	CLAY	19	16		.9	
15.000	49.21	18.23	5.93	CLAY	18	15		1.0	
15.150	49.70	17.06	5.86	CLAY	17	14		1.0	
15.300	50.20	22.97	6.65	CLAY	23	19		1.2	
15.450	50.69	19.01	6.02	CLAY	19	16		1.0	
15.600	51.18	18.21	6.54	CLAY	18	15		1.0	
15.750	51.67	17.00	6.55	CLAY	17	14		.9	
15.900	52.17	14.64	6.27	CLAY	15	12		.8	
16.050	52.66	14.17	6.78	CLAY	14	12		.8	
16.200	53.15	14.26	6.26	CLAY	14	12		.8	
16.350	53.64	13.94	6.23	CLAY	14	11		.7	
16.500	54.13	13.04	6.61	CLAY	13	11		.7	
16.650	54.63	14.34	6.93	CLAY	14	12		.8	
16.800	55.12	20.16	7.07	CLAY	20	16		1.1	
16.950	55.61	20.88	6.60	CLAY	21	17		1.0	
17.100	56.10	16.51	7.22	CLAY	17	13		.9	
17.250	56.59	23.65	6.53	CLAY	24	19		1.2	
17.400	57.09	28.70	7.55	CLAY	29	23		1.5	
17.550	57.58	32.53	7.25	CLAY	33	26		1.7	
17.700	58.07	32.48	7.52	CLAY	32	26		1.7	
17.850	58.56	35.54	6.74	CLAY	36	28		1.9	
18.000	59.06	37.60	6.65	CLAY	38	29		2.0	
18.150	59.55	38.16	7.03	CLAY	38	30		2.1	
18.300	60.04	25.92	5.58	CLAY	26	20		1.3	
18.450	60.53	24.39	6.28	CLAY	24	19		1.2	
18.600	61.02	28.45	6.33	CLAY	28	22		1.5	
18.750	61.52	28.98	6.86	CLAY	29	22		1.5	
18.900	62.01	29.36	6.58	CLAY	29	22		1.5	
19.050	62.50	29.23	6.62	CLAY	29	22		1.5	
19.200	62.99	35.37	6.75	CLAY	35	27		1.9	
19.350	63.48	39.52	6.81	CLAY	40	30		2.1	
19.500	63.98	37.11	6.85	CLAY	37	28		2.0	
19.650	64.47	39.30	6.99	CLAY	39	30		2.1	
19.800	64.96	43.57	6.17	CLAY	44	33		2.4	
19.950	65.45	37.94	6.19	CLAY	38	28		2.0	
20.100	65.94	37.09	5.88	CLAY	37	28		2.0	
20.250	66.44	38.03	6.65	CLAY	38	28		2.0	
20.400	66.93	37.56	6.40	CLAY	38	28		2.0	
20.550	67.42	36.22	6.87	CLAY	36	27		1.9	
20.700	67.91	44.64	5.94	CLAY	45	33		2.4	
20.850	68.41	32.48	6.31	CLAY	32	24		1.7	
21.000	68.90	28.21	6.74	CLAY	28	21		1.4	
21.150	69.39	28.36	6.72	CLAY	28	21		1.4	

*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 110 pcf
ASSUMED DEPTH OF WATER TABLE = 7.5 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

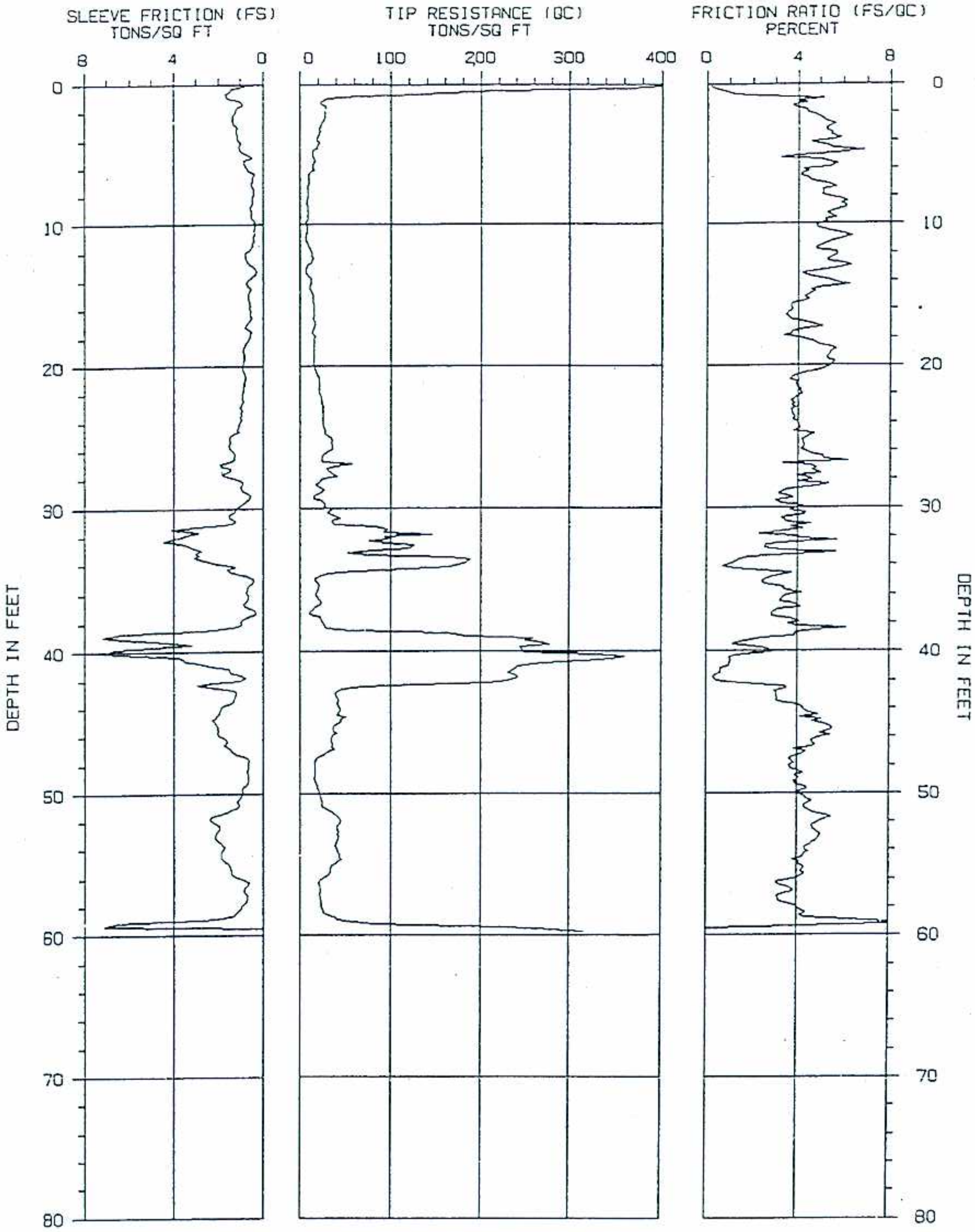
Interpretations based on: Robertson and Campanella, 1989.

SOUNDING : HLA-3

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
21.300	69.88	26.26	6.27	CLAY	26	19		1.3	
21.450	70.37	19.38	5.69	CLAY	19	14		.9	
21.600	70.87	18.31	5.90	CLAY	18	13		1.0	
21.750	71.36	21.63	8.53	CLAY	22	16		1.2	

*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 110 pcf
ASSUMED DEPTH OF WATER TABLE = 7.5 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

Interpretations based on: Robertson and Campanella, 1989.



TIP RESISTANCE NOT CORRECTED FOR END AREA EFFECT

CONE PENETRATION TEST

SOUNDING NUMBER: HLA-4

PROJECT NAME : HLA/BISHOP RH 1

PROJECT NUMBER : 50044.1

CONE/RIG : 491/BH.VD/R#4

DATE/TIME: 04-12-00 17:19

HFA

CPT INTERPRETATIONS

SOUNDING : HLA-4
PROJECT : HLA/BISHOP RH 1
DATE/TIME: 04-12-00 17:19

PROJECT No.: 50044.1
CONE/RIG : 491/BH,VO/R#4

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
.150	.49	218.69	.68	SAND	44	70	99		
.300	.98	30.72	5.12	CLAY	31	49		1.8	
.450	1.48	25.15	3.78	CLAY to SILTY CLAY	17	27		1.7	
.600	1.97	28.47	4.40	CLAY to SILTY CLAY	19	30		1.7	
.750	2.46	27.34	5.09	CLAY	27	44		1.6	
.900	2.95	22.43	5.41	CLAY	22	36		1.3	
1.050	3.44	21.75	5.51	CLAY	22	35		1.3	
1.200	3.94	19.74	5.56	CLAY	20	32		1.1	
1.350	4.43	19.82	5.38	CLAY	20	32		1.2	
1.500	4.92	14.34	5.63	CLAY	14	23		.9	
1.650	5.41	16.76	5.01	CLAY	17	27		1.1	
1.800	5.91	14.32	5.39	CLAY	14	23		.9	
1.950	6.40	9.92	4.13	CLAY	10	16		.6	
2.100	6.89	10.62	4.59	CLAY	11	17		.7	
2.250	7.38	8.77	5.64	CLAY	9	14		.6	
2.400	7.87	9.09	5.07	CLAY	9	14		.6	
2.550	8.37	7.90	6.13	CLAY	8	12		.5	
2.700	8.86	9.16	6.11	CLAY	9	13		.6	
2.850	9.35	9.56	5.46	CLAY	10	14		.6	
3.000	9.84	7.58	5.30	CLAY	8	11		.5	
3.150	10.33	7.99	5.14	CLAY	8	11		.5	
3.300	10.83	6.95	6.34	CLAY	7	10		.4	
3.450	11.32	9.07	5.31	CLAY	9	12		.6	
3.600	11.81	12.79	4.79	CLAY	13	17		.8	
3.750	12.30	14.55	5.49	CLAY	15	20		.9	
3.900	12.80	10.35	6.03	CLAY	10	14		.6	
4.050	13.29	6.69	4.80	CLAY	7	9		.4	
4.200	13.78	12.47	4.79	CLAY	12	16		.8	
4.350	14.27	11.11	6.22	CLAY	11	14		.7	
4.500	14.76	13.21	4.76	CLAY	13	17		.8	
4.650	15.26	15.00	4.49	CLAY	15	19		.9	
4.800	15.75	16.10	3.69	CLAY to SILTY CLAY	11	13		1.0	
4.950	16.24	16.10	3.55	CLAY to SILTY CLAY	11	13		1.0	
5.100	16.73	15.72	3.65	CLAY to SILTY CLAY	10	13		1.0	
5.250	17.22	15.66	5.07	CLAY	16	19		1.0	
5.400	17.72	15.08	3.76	CLAY to SILTY CLAY	10	12		.9	
5.550	18.21	15.93	4.56	CLAY	16	19		1.0	
5.700	18.70	16.15	5.49	CLAY	16	19		1.0	
5.850	19.19	15.78	5.44	CLAY	16	19		1.0	
6.000	19.69	15.70	5.60	CLAY	16	18		1.0	
6.150	20.18	17.38	5.32	CLAY	17	20		1.1	

*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 110 pcf
ASSUMED DEPTH OF WATER TABLE = 8.3 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

SOUNDING : HLA-4

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
6.300	20.67	20.88	3.87	CLAY to SILTY CLAY	14	16		1.3	
6.450	21.16	21.27	3.95	CLAY to SILTY CLAY	14	16		1.3	
6.600	21.65	21.37	4.16	CLAY to SILTY CLAY	14	16		1.3	
6.750	22.15	23.39	4.01	CLAY to SILTY CLAY	16	18		1.5	
6.900	22.64	25.15	3.88	CLAY to SILTY CLAY	17	19		1.6	
7.050	23.13	25.75	3.68	CLAY to SILTY CLAY	17	19		1.6	
7.200	23.62	25.85	3.87	CLAY to SILTY CLAY	17	19		1.6	
7.350	24.11	26.07	4.00	CLAY to SILTY CLAY	17	19		1.6	
7.500	24.61	27.96	3.83	CLAY to SILTY CLAY	19	20		1.8	
7.650	25.10	34.74	4.36	CLAY to SILTY CLAY	23	25		2.0	
7.800	25.59	36.16	4.26	CLAY to SILTY CLAY	24	26		2.0	
7.950	26.08	29.87	4.45	CLAY to SILTY CLAY	20	21		1.7	
8.100	26.57	24.88	5.15	CLAY	25	26		1.4	
8.250	27.07	40.24	4.61	CLAY to SILTY CLAY	27	28		2.3	
8.400	27.56	35.76	5.03	CLAY	36	37		2.0	
8.550	28.05	22.54	4.61	CLAY	23	23		1.4	
8.700	28.54	20.69	4.84	CLAY	21	21		1.3	
8.850	29.04	18.59	3.17	CLAYEY SILT to SILTY CLAY	9	10		1.1	
9.000	29.53	24.69	3.05	CLAYEY SILT to SILTY CLAY	12	13		1.5	
9.150	30.02	30.74	3.92	CLAY to SILTY CLAY	20	21		1.9	
9.300	30.51	35.73	4.30	CLAY to SILTY CLAY	24	24		2.0	
9.450	31.00	36.03	3.46	CLAYEY SILT to SILTY CLAY	18	18		2.3	
9.600	31.50	96.56	4.21	CLAYEY SILT to SILTY CLAY	48	48		5.6	
9.750	31.99	93.12	3.89	CLAYEY SILT to SILTY CLAY	47	46		5.4	
9.900	32.48	96.15	3.84	CLAYEY SILT to SILTY CLAY	48	47		5.6	
10.050	32.97	82.30	3.35	SANDY SILT to CLAYEY SILT	33	32		4.7	
10.200	33.46	166.60	1.85	SILTY SAND to SANDY SILT	56	54	81		42.5
10.350	33.96	175.93	1.00	SAND	35	34	82		43.0
10.500	34.45	60.00	2.50	SANDY SILT to CLAYEY SILT	24	23		3.9	
10.650	34.94	17.00	2.67	CLAYEY SILT to SILTY CLAY	9	8		1.0	
10.800	35.43	21.48	2.80	CLAYEY SILT to SILTY CLAY	11	10		1.3	
10.950	35.93	18.25	4.19	CLAY	18	17		1.1	
11.100	36.42	21.41	3.30	CLAYEY SILT to SILTY CLAY	11	10		1.3	
11.250	36.91	16.93	4.14	CLAY	17	16		1.0	
11.400	37.40	11.81	2.84	CLAY to SILTY CLAY	8	7		.7	
11.550	37.89	24.56	4.08	CLAY to SILTY CLAY	16	15		1.5	
11.700	38.39	29.70	6.13	CLAY	30	27		1.6	
11.850	38.88	184.24	3.89	*SAND to CLAYEY SAND	92	85			
12.000	39.37	269.75	1.47	SAND to SILTY SAND	67	62	93		44.0
12.150	39.86	246.55	2.72	SILTY SAND to SANDY SILT	82	75	90		44.0
12.300	40.35	360.61	1.04	SAND	72	65	100		45.5
12.450	40.85	270.11	1.06	SAND	54	49	92		44.0
12.600	41.34	234.99	.63	SAND	47	42	88		43.5
12.750	41.83	241.98	.33	SAND	48	43	89		43.5
12.900	42.32	128.25	2.27	SILTY SAND to SANDY SILT	43	38	71		40.0
13.050	42.81	41.07	2.97	CLAYEY SILT to SILTY CLAY	21	18		2.6	
13.200	43.31	42.13	3.10	CLAYEY SILT to SILTY CLAY	21	19		2.6	
13.350	43.80	42.98	4.07	CLAYEY SILT to SILTY CLAY	21	19		2.4	
13.500	44.29	42.85	4.54	CLAY to SILTY CLAY	29	25		2.4	
13.650	44.78	44.83	5.05	CLAY	45	39		2.5	

*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 110 pcf
ASSUMED DEPTH OF WATER TABLE = 8.3 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

Interpretations based on: Robertson and Campanella, 1989.

SOUNDING : HLA-4

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
13.800	45.28	37.48	5.47	CLAY	37	32		2.1	
13.950	45.77	40.51	5.03	CLAY	41	35		2.2	
14.100	46.26	34.67	4.61	CLAY to SILTY CLAY	23	20		1.9	
14.250	46.75	34.31	4.67	CLAY to SILTY CLAY	23	20		1.9	
14.400	47.24	26.62	4.12	CLAY to SILTY CLAY	18	15		1.6	
14.550	47.74	16.70	3.80	CLAY to SILTY CLAY	11	9		.9	
14.700	48.23	17.46	3.73	CLAY to SILTY CLAY	12	10		1.0	
14.850	48.72	17.48	3.92	CLAY to SILTY CLAY	12	10		1.0	
15.000	49.21	17.23	3.85	CLAY to SILTY CLAY	11	10		1.0	
15.150	49.70	20.88	4.45	CLAY	21	17		1.2	
15.300	50.20	22.56	4.32	CLAY to SILTY CLAY	15	12		1.3	
15.450	50.69	24.28	4.53	CLAY	24	20		1.3	
15.600	51.18	31.76	4.45	CLAY to SILTY CLAY	21	17		1.7	
15.750	51.67	42.45	5.49	CLAY	42	35		2.3	
15.900	52.17	44.00	4.78	CLAY to SILTY CLAY	29	24		2.4	
16.050	52.66	42.09	4.87	CLAY to SILTY CLAY	28	23		2.3	
16.200	53.15	42.81	4.99	CLAY	43	35		2.3	
16.350	53.64	39.05	4.59	CLAY to SILTY CLAY	26	21		2.1	
16.500	54.13	41.47	4.53	CLAY to SILTY CLAY	28	22		2.3	
16.650	54.63	45.93	3.85	CLAYEY SILT to SILTY CLAY	23	18		2.5	
16.800	55.12	35.10	4.23	CLAY to SILTY CLAY	23	19		1.9	
16.950	55.61	32.38	4.36	CLAY to SILTY CLAY	22	17		1.7	
17.100	56.10	22.73	3.57	CLAY to SILTY CLAY	15	12		1.3	
17.250	56.59	22.39	3.65	CLAY to SILTY CLAY	15	12		1.3	
17.400	57.09	21.69	3.40	CLAYEY SILT to SILTY CLAY	11	9		1.2	
17.550	57.58	23.67	3.27	CLAYEY SILT to SILTY CLAY	12	9		1.4	
17.700	58.07	24.96	3.99	CLAY to SILTY CLAY	17	13		1.5	
17.850	58.56	31.14	4.15	CLAY to SILTY CLAY	21	16		1.6	
18.000	59.06	59.17	9.80	CLAY	59	46		3.3	
18.150	59.55	262.82	*****		0	0			.0

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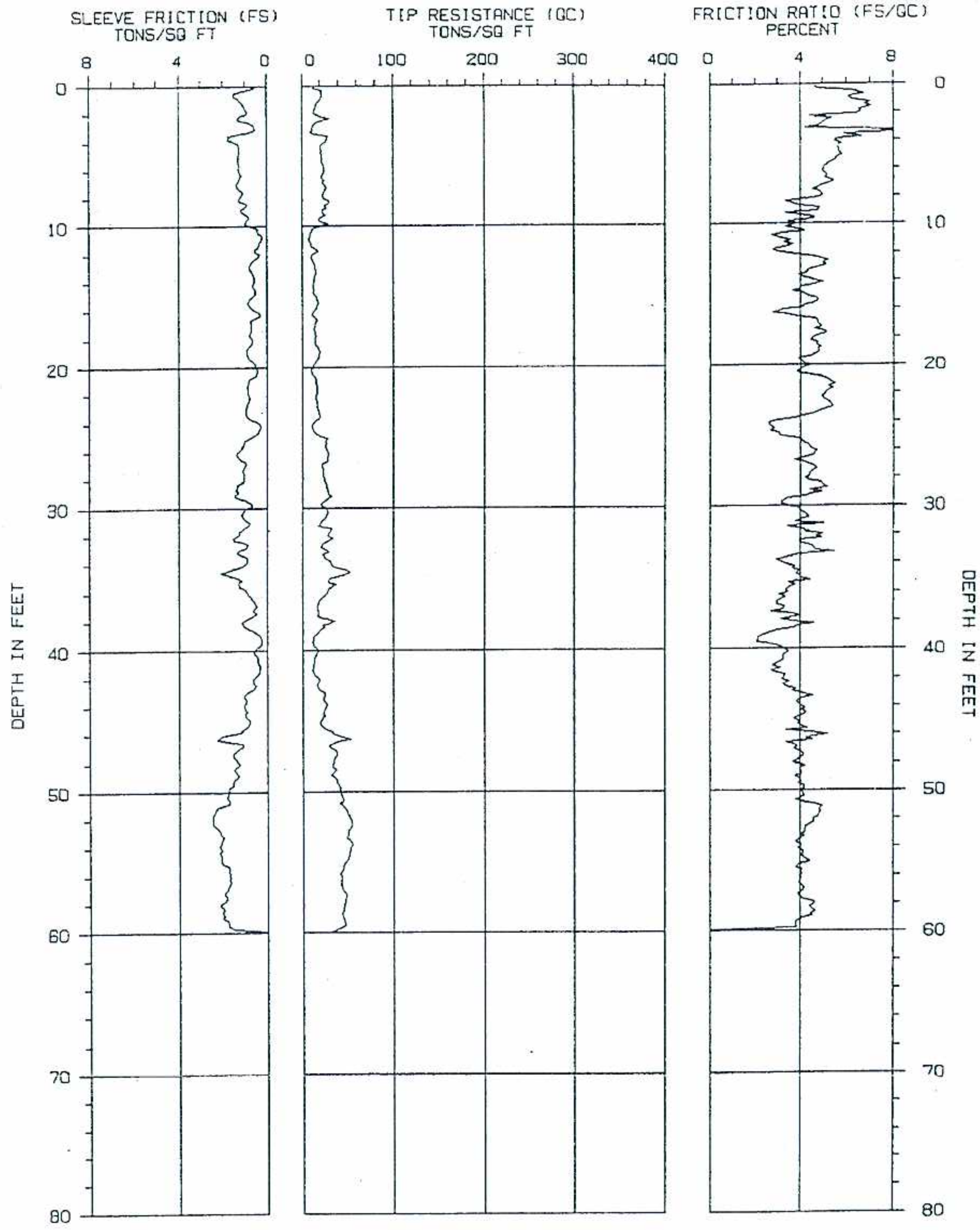
*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 110 pcf
ASSUMED DEPTH OF WATER TABLE = 8.3 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

Interpretations based on: Robertson and Campanella, 1989.



TIP RESISTANCE NOT CORRECTED FOR END AREA EFFECT

CONE PENETRATION TEST

SOUNDING NUMBER: HLA-5

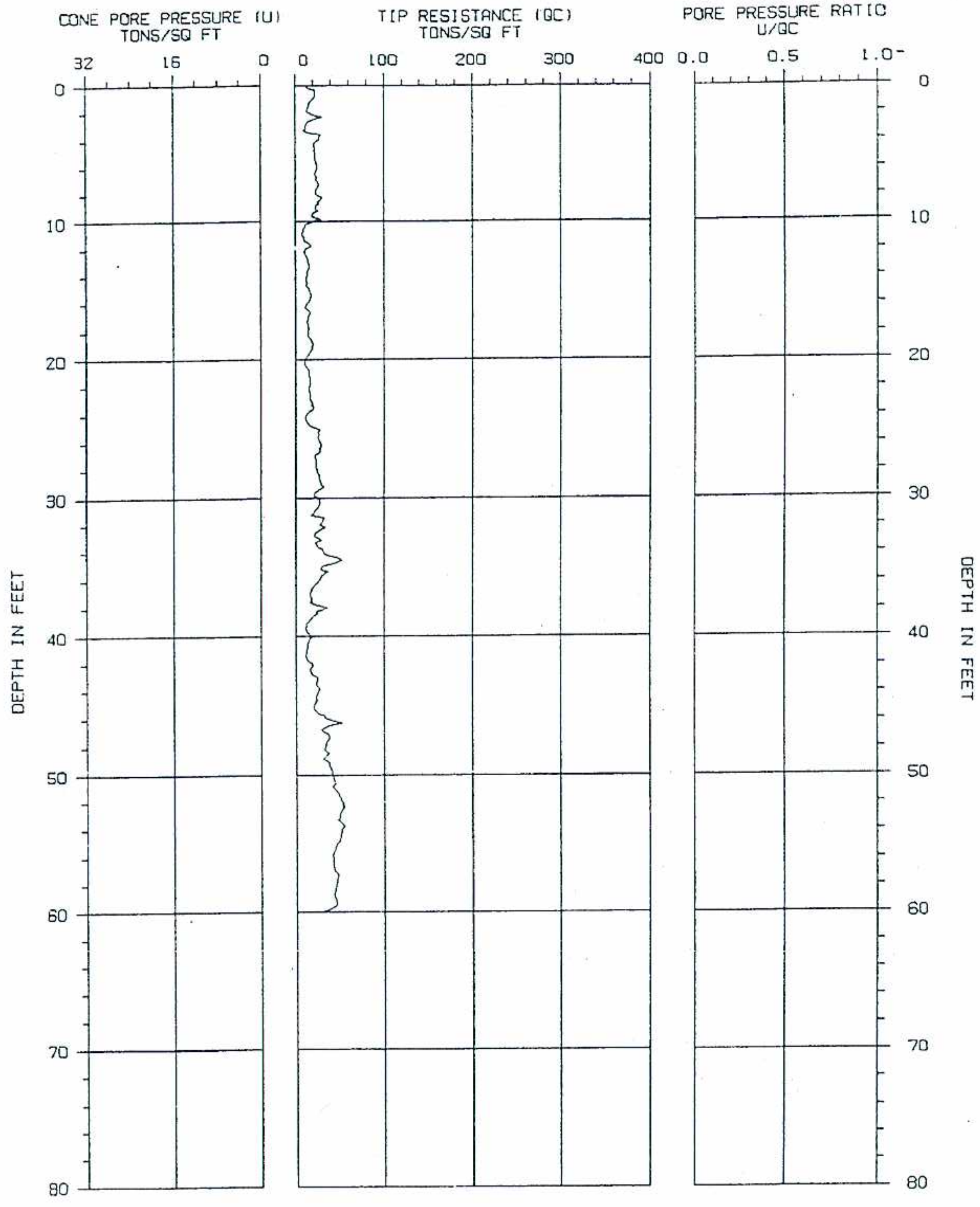
PROJECT NAME : HLA/BISHOP RH 1

PROJECT NUMBER : 50044.1

CONE/RIG : 491/BH.VO/R#4

DATE/TIME: 04-13-90 13:56

HFA



TIP RESISTANCE NOT CORRECTED FOR END AREA EFFECT

CONE PENETRATION TEST

SOUNDING NUMBER: HLA-5

PROJECT NAME : HLA/BISHOP RH 1

PROJECT NUMBER : 50044.1

CONE/RIG : 491/BH.V0/R#4

DATE/TIME: 04-13-90 13:56

HFA

CPT INTERPRETATIONS

SOUNDING : HLA-5
PROJECT : HLA/BISHOP RH 1
DATE/TIME: 04-13-90 13:56

PROJECT No.: 50044.1
CONE/RIG : 491/BH,VO/R#4

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
.150	.49	22.99	6.27	CLAY	23	37		1.4	
.300	.98	20.48	6.13	CLAY	20	33		1.2	
.450	1.48	15.47	6.76	CLAY	15	25		1.0	
.600	1.97	14.43	6.60	CLAY	14	23		1.0	
.750	2.46	23.11	5.40	CLAY	23	37		1.4	
.900	2.95	11.66	4.76	CLAY	12	19		.8	
1.050	3.44	15.30	8.44	CLAY	15	24		1.0	
1.200	3.94	26.94	5.55	CLAY	27	43		1.6	
1.350	4.43	22.03	5.60	CLAY	22	35		1.3	
1.500	4.92	22.20	5.74	CLAY	22	36		1.3	
1.650	5.41	23.11	5.56	CLAY	23	37		1.3	
1.800	5.91	25.30	5.05	CLAY	25	40		1.5	
1.950	6.40	22.99	5.10	CLAY	23	37		1.3	
2.100	6.89	23.92	5.48	CLAY	24	38		1.4	
2.250	7.38	25.39	4.80	CLAY	25	40		1.5	
2.400	7.87	23.71	5.00	CLAY	24	36		1.4	
2.550	8.37	29.40	3.34	CLAYEY SILT to SILTY CLAY	15	22		1.9	
2.700	8.86	24.03	4.83	CLAY	24	35		1.4	
2.850	9.35	20.01	4.03	CLAY to SILTY CLAY	13	19		1.3	
3.000	9.84	28.83	3.48	CLAYEY SILT to SILTY CLAY	14	20		1.9	
3.150	10.33	11.24	3.87	CLAY	11	16		.7	
3.300	10.83	8.54	2.75	CLAY to SILTY CLAY	6	8		.6	
3.450	11.32	10.37	3.25	CLAY to SILTY CLAY	7	9		.6	
3.600	11.81	18.57	2.82	CLAYEY SILT to SILTY CLAY	9	12		1.2	
3.750	12.30	11.51	4.60	CLAY	12	15		.7	
3.900	12.80	15.00	4.98	CLAY	15	20		1.0	
4.050	13.29	15.68	4.30	CLAY	16	20		1.0	
4.200	13.78	13.60	4.12	CLAY	14	18		.9	
4.350	14.27	13.13	4.60	CLAY	13	17		.8	
4.500	14.76	14.28	3.67	CLAY to SILTY CLAY	10	12		.9	
4.650	15.26	17.97	4.72	CLAY	18	23		1.1	
4.800	15.75	16.87	4.17	CLAY	17	21		1.1	
4.950	16.24	11.51	2.82	CLAY to SILTY CLAY	8	9		.7	
5.100	16.73	15.61	4.68	CLAY	16	19		1.0	
5.250	17.22	14.28	4.92	CLAY	14	17		.9	
5.400	17.72	15.49	5.10	CLAY	15	19		1.0	
5.550	18.21	14.47	4.45	CLAY	14	17		.9	
5.700	18.70	19.25	4.88	CLAY	19	23		1.2	
5.850	19.19	18.74	4.70	CLAY	19	22		1.2	
6.000	19.69	14.32	4.06	CLAY	14	17		.9	
6.150	20.18	10.88	4.23	CLAY	11	13		.7	

*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 110 pcf
ASSUMED DEPTH OF WATER TABLE = 8.8 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

SOUNDING : HLA-5

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
6.300	20.67	15.11	4.46	CLAY	15	17		.9	
6.450	21.16	16.29	5.30	CLAY	16	18		1.0	
6.600	21.65	16.55	5.41	CLAY	17	19		1.0	
6.750	22.15	15.83	5.07	CLAY	16	18		1.0	
6.900	22.64	17.29	5.29	CLAY	17	19		1.1	
7.050	23.13	18.89	5.10	CLAY	19	21		1.2	
7.200	23.62	19.67	4.11	CLAY to SILTY CLAY	13	14		1.2	
7.350	24.11	11.71	2.64	CLAY to SILTY CLAY	8	8		.8	
7.500	24.61	14.74	2.65	CLAYEY SILT to SILTY CLAY	7	8		.9	
7.650	25.10	27.75	3.23	CLAYEY SILT to SILTY CLAY	14	15		1.8	
7.800	25.59	25.83	4.32	CLAY to SILTY CLAY	17	18		1.4	
7.950	26.08	29.64	4.71	CLAY	30	31		1.7	
8.100	26.57	28.53	4.11	CLAY to SILTY CLAY	19	20		1.6	
8.250	27.07	22.84	4.40	CLAY	23	24		1.4	
8.400	27.56	24.11	4.48	CLAY	24	25		1.5	
8.550	28.05	24.96	4.23	CLAY to SILTY CLAY	17	17		1.6	
8.700	28.54	27.13	4.97	CLAY	27	28		1.5	
8.850	29.04	29.66	4.92	CLAY	30	30		1.7	
9.000	29.53	26.49	3.43	CLAYEY SILT to SILTY CLAY	13	13		1.7	
9.150	30.02	26.64	3.47	CLAYEY SILT to SILTY CLAY	13	13		1.7	
9.300	30.51	27.26	4.17	CLAY to SILTY CLAY	18	18		1.5	
9.450	31.00	21.54	3.96	CLAY to SILTY CLAY	14	14		1.3	
9.600	31.50	32.14	3.44	CLAYEY SILT to SILTY CLAY	16	16		2.0	
9.750	31.99	27.62	4.99	CLAY	28	27		1.5	
9.900	32.48	26.24	3.98	CLAY to SILTY CLAY	17	17		1.6	
10.050	32.97	27.07	4.57	CLAY	27	26		1.5	
10.200	33.46	25.09	3.97	CLAY to SILTY CLAY	17	16		1.5	
10.350	33.96	32.57	3.15	CLAYEY SILT to SILTY CLAY	16	16		2.0	
10.500	34.45	48.97	3.64	CLAYEY SILT to SILTY CLAY	24	23		2.8	
10.650	34.94	33.57	3.96	CLAY to SILTY CLAY	22	21		1.9	
10.800	35.43	36.92	3.48	CLAYEY SILT to SILTY CLAY	18	17		2.3	
10.950	35.93	27.62	3.33	CLAYEY SILT to SILTY CLAY	14	13		1.7	
11.100	36.42	21.20	3.33	CLAYEY SILT to SILTY CLAY	11	10		1.3	
11.250	36.91	17.00	2.97	CLAYEY SILT to SILTY CLAY	9	8		1.0	
11.400	37.40	18.08	2.74	CLAYEY SILT to SILTY CLAY	9	8		1.1	
11.550	37.89	35.59	3.16	CLAYEY SILT to SILTY CLAY	18	16		2.2	
11.700	38.39	25.13	3.88	CLAY to SILTY CLAY	17	15		1.5	
11.850	38.88	14.64	2.55	CLAYEY SILT to SILTY CLAY	7	7		.8	
12.000	39.37	11.96	2.13	CLAYEY SILT to SILTY CLAY	6	5		.8	
12.150	39.86	16.36	3.10	CLAY to SILTY CLAY	11	10		.9	
12.300	40.35	14.89	3.29	CLAY to SILTY CLAY	10	9		.8	
12.450	40.85	13.28	3.23	CLAY to SILTY CLAY	9	8		.7	
12.600	41.34	11.96	3.11	CLAY to SILTY CLAY	8	7		.6	
12.750	41.83	14.74	3.33	CLAY to SILTY CLAY	10	9		.8	
12.900	42.32	17.53	3.51	CLAY to SILTY CLAY	12	10		1.0	
13.050	42.81	19.35	3.83	CLAY to SILTY CLAY	13	11		1.1	
13.200	43.31	23.65	4.52	CLAY	24	21		1.4	
13.350	43.80	26.51	3.88	CLAY to SILTY CLAY	18	15		1.6	
13.500	44.29	23.39	4.03	CLAY to SILTY CLAY	16	14		1.4	
13.650	44.78	22.94	3.96	CLAY to SILTY CLAY	15	13		1.4	

*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 110 pcf
ASSUMED DEPTH OF WATER TABLE = 8.8 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH .
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

Interpretations based on: Robertson and Campanella, 1989.

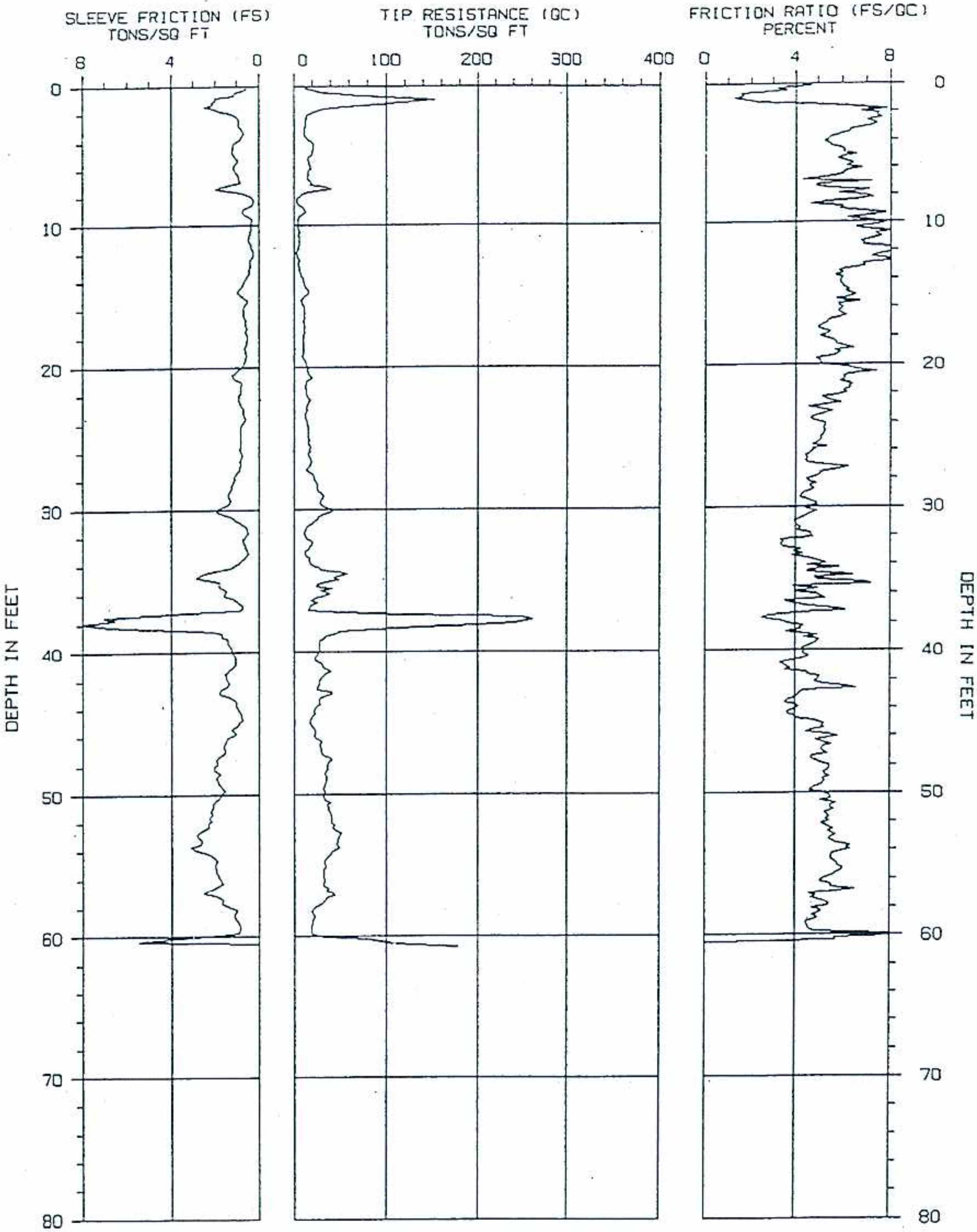
SOUNDING : HLA-5

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
13.800	45.28	21.54	3.91	CLAY to SILTY CLAY	14	12		1.3	
13.950	45.77	33.93	3.37	CLAYEY SILT to SILTY CLAY	17	15		2.1	
14.100	46.26	52.77	4.27	CLAYEY SILT to SILTY CLAY	26	23		3.0	
14.250	46.75	29.57	3.76	CLAYEY SILT to SILTY CLAY	15	13		1.8	
14.400	47.24	38.64	4.03	CLAYEY SILT to SILTY CLAY	19	16		2.1	
14.550	47.74	35.18	3.98	CLAYEY SILT to SILTY CLAY	18	15		1.9	
14.700	48.23	33.29	4.22	CLAY to SILTY CLAY	22	19		1.8	
14.850	48.72	33.46	4.04	CLAY to SILTY CLAY	22	19		1.8	
15.000	49.21	38.24	3.92	CLAYEY SILT to SILTY CLAY	19	16		2.1	
15.150	49.70	41.17	4.20	CLAY to SILTY CLAY	27	23		2.3	
15.300	50.20	43.11	4.08	CLAYEY SILT to SILTY CLAY	22	18		2.4	
15.450	50.69	44.36	3.85	CLAYEY SILT to SILTY CLAY	22	18		2.4	
15.600	51.18	46.06	4.98	CLAY	46	38		2.5	
15.750	51.67	51.58	4.83	CLAY to SILTY CLAY	34	28		2.9	
15.900	52.17	53.94	4.58	CLAY to SILTY CLAY	36	29		3.0	
16.050	52.66	51.67	4.29	CLAYEY SILT to SILTY CLAY	26	21		2.9	
16.200	53.15	49.82	4.07	CLAYEY SILT to SILTY CLAY	25	20		2.8	
16.350	53.64	54.98	3.81	CLAYEY SILT to SILTY CLAY	27	22		3.1	
16.500	54.13	52.20	3.92	CLAYEY SILT to SILTY CLAY	26	21		2.9	
16.650	54.63	50.41	4.10	CLAYEY SILT to SILTY CLAY	25	20		2.8	
16.800	55.12	46.16	4.41	CLAY to SILTY CLAY	31	24		2.5	
16.950	55.61	42.60	4.11	CLAYEY SILT to SILTY CLAY	21	17		2.3	
17.100	56.10	42.87	3.95	CLAYEY SILT to SILTY CLAY	21	17		2.3	
17.250	56.59	43.11	3.97	CLAYEY SILT to SILTY CLAY	22	17		2.4	
17.400	57.09	45.63	4.14	CLAYEY SILT to SILTY CLAY	23	18		2.5	
17.550	57.58	46.61	3.99	CLAYEY SILT to SILTY CLAY	23	18		2.6	
17.700	58.07	46.40	4.61	CLAY to SILTY CLAY	31	24		2.5	
17.850	58.56	43.42	4.65	CLAY to SILTY CLAY	29	22		2.4	
18.000	59.06	45.40	4.12	CLAYEY SILT to SILTY CLAY	23	18		2.5	
18.150	59.55	45.87	3.84	CLAYEY SILT to SILTY CLAY	23	18		2.5	

*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 110 pcf
ASSUMED DEPTH OF WATER TABLE = 8.8 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

HOLGUIN, FAHAN & ASSOCIATES, INC.

Interpretations based on: Robertson and Campanella, 1989.



TIP RESISTANCE NOT CORRECTED FOR END AREA EFFECT

CONE PENETRATION TEST

SOUNDING NUMBER: HLA-6

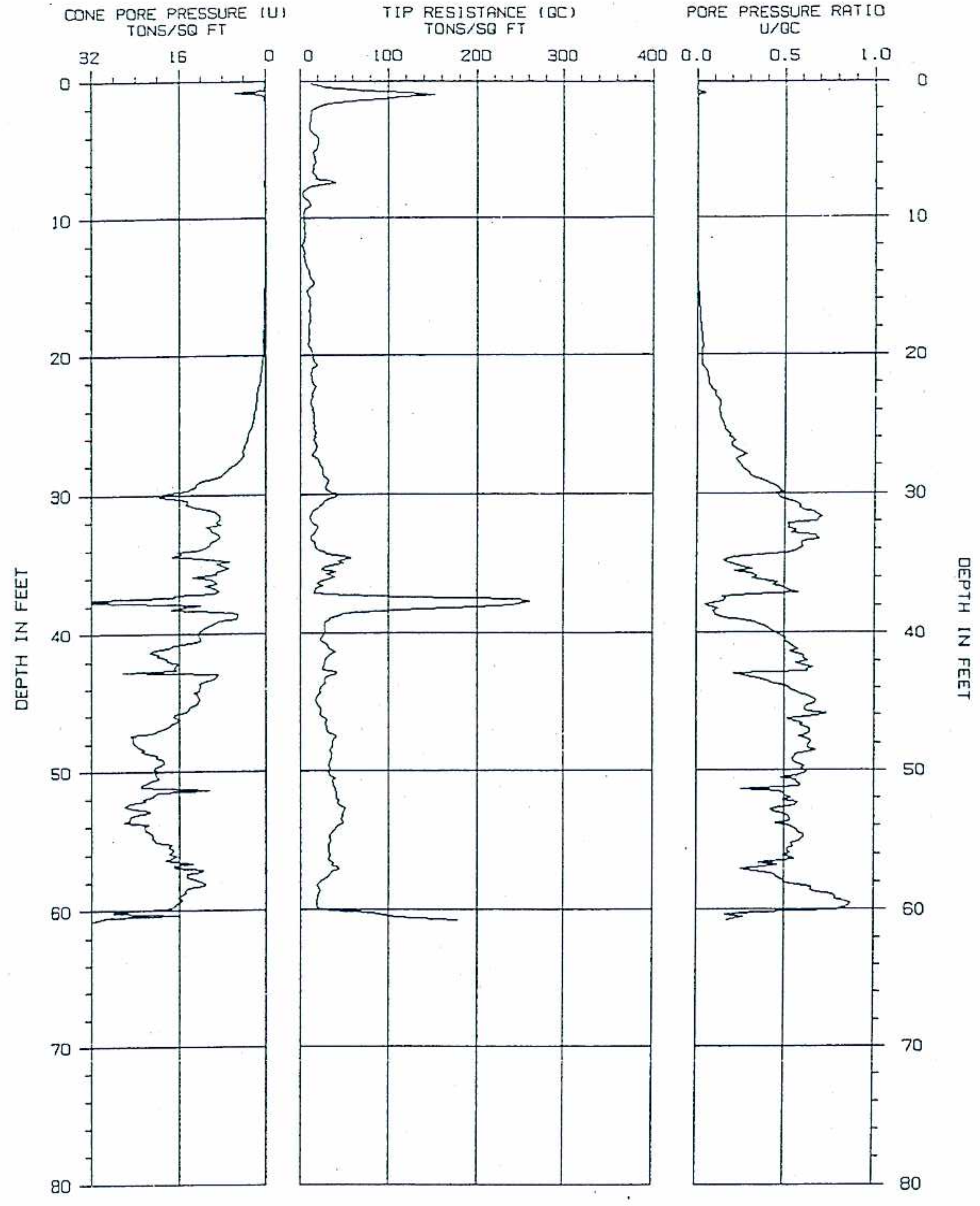
PROJECT NAME : HLA/BISHOP RH 1

CONE/RIG : 491/BH.V0/R#4

PROJECT NUMBER : 50044.1

DATE/TIME: 04-13-00 11:55

HFA



TIP RESISTANCE NOT CORRECTED FOR END AREA EFFECT

CONE PENETRATION TEST	SOUNDING NUMBER: HLA-6

PROJECT NAME : HLA/BISHOP RH 1

PROJECT NUMBER : 50044.1

CONE/RIG : 491/BH.V0/R#4

DATE/TIME: 04-13-00 11:55

HFA

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*                                                                    *
*                     CPT INTERPRETATIONS                            *
*                                                                    *
*  SOUNDING : HLA-6                    PROJECT No.: 50044.1          *
*  PROJECT  : HLA/BISHOP RH 1          CONE/RIG : 491/BH,VO/R#4      *
*  DATE/TIME: 04-13-00 11:55                                         *
*                                                                    *
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PAGE 1 of 3

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
.150	.49	30.61	3.59	CLAYEY SILT to SILTY CLAY	15	24		2.0	
.300	.98	151.96	1.32	SAND to SILTY SAND	38	61	88		
.450	1.48	54.64	4.58	CLAY to SILTY CLAY	36	58		3.2	
.600	1.97	18.63	6.82	CLAY	19	30		1.2	
.750	2.46	12.51	7.59	CLAY	13	20		.8	
.900	2.95	11.83	7.27	CLAY	12	19		.8	
1.050	3.44	11.41	6.31	CLAY	11	18		.7	
1.200	3.94	19.31	5.44	CLAY	19	31		1.3	
1.350	4.43	21.67	5.49	CLAY	22	35		1.3	
1.500	4.92	19.33	6.05	CLAY	19	31		1.1	
1.650	5.41	16.89	5.80	CLAY	17	27		1.1	
1.800	5.91	18.23	6.20	CLAY	18	29		1.2	
1.950	6.40	16.17	6.06	CLAY	16	26		1.1	
2.100	6.89	19.46	4.32	CLAY	19	31		1.3	
2.250	7.38	40.39	4.88	CLAY to SILTY CLAY	27	42		2.4	
2.400	7.87	6.84	5.85	CLAY	7	10		.4	
2.550	8.37	4.02	6.47	CLAY	4	6		.2	
2.700	8.86	11.98	6.01	CLAY	12	17		.8	
2.850	9.35	6.42	7.79	CLAY	6	9		.4	
3.000	9.84	4.91	6.92	CLAY	5	7		.3	
3.150	10.33	5.48	6.57	CLAY	5	7		.3	
3.300	10.83	5.97	7.37	CLAY	6	8		.4	
3.450	11.32	6.05	6.78	CLAY	6	8		.4	
3.600	11.81	3.87	8.01	ORGANIC MATERIAL	4	5		.3	
3.750	12.30	4.42	7.47	CLAY	4	5		.2	
3.900	12.80	4.78	8.37	ORGANIC MATERIAL	5	6		.4	
4.050	13.29	7.35	6.12	CLAY	7	9		.4	
4.200	13.78	10.20	5.69	CLAY	10	12		.6	
4.350	14.27	13.00	5.92	CLAY	13	15		.8	
4.500	14.76	15.66	6.26	CLAY	16	18		1.0	
4.650	15.26	8.63	6.26	CLAY	9	10		.5	
4.800	15.75	11.47	5.84	CLAY	11	13		.7	
4.950	16.24	12.02	5.82	CLAY	12	14		.7	
5.100	16.73	10.92	5.86	CLAY	11	12		.7	
5.250	17.22	11.49	5.13	CLAY	11	13		.7	
5.400	17.72	10.81	5.46	CLAY	11	12		.7	
5.550	18.21	10.37	5.59	CLAY	10	11		.6	
5.700	18.70	10.73	6.06	CLAY	11	12		.6	
5.850	19.19	9.79	5.92	CLAY	10	11		.6	
6.000	19.69	13.41	5.07	CLAY	13	15		.8	
6.150	20.18	15.38	6.18	CLAY	15	17		1.0	

*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 110 pcf
ASSUMED DEPTH OF WATER TABLE = 12.3 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

HOLGUIN, FAHAN & ASSOCIATES, INC.

Interpretations based on: Robertson and Campanella, 1989.

SOUNDING : HLA-6

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
6.300	20.67	19.10	6.23	CLAY	19	20		1.1	
6.450	21.16	13.49	5.93	CLAY	13	14		.8	
6.600	21.65	13.32	6.31	CLAY	13	14		.8	
6.750	22.15	16.46	5.77	CLAY	16	17		1.0	
6.900	22.64	14.51	5.93	CLAY	15	15		.9	
7.050	23.13	14.17	5.08	CLAY	14	15		.9	
7.200	23.62	13.19	4.78	CLAY	13	14		.8	
7.350	24.11	15.44	5.31	CLAY	15	16		.9	
7.500	24.61	16.10	5.28	CLAY	16	16		1.0	
7.650	25.10	16.57	5.07	CLAY	17	17		1.0	
7.800	25.59	17.72	4.74	CLAY	18	18		1.1	
7.950	26.08	16.32	4.78	CLAY	16	16		1.0	
8.100	26.57	19.18	4.54	CLAY	19	19		1.2	
8.250	27.07	16.15	5.39	CLAY	16	16		1.0	
8.400	27.56	21.12	5.07	CLAY	21	21		1.2	
8.550	28.05	25.13	4.50	CLAY to SILTY CLAY	17	16		1.4	
8.700	28.54	26.47	4.65	CLAY	26	26		1.5	
8.850	29.04	32.27	4.37	CLAY to SILTY CLAY	22	21		1.8	
9.000	29.53	29.08	4.54	CLAY to SILTY CLAY	19	19		1.6	
9.150	30.02	44.15	4.35	CLAY to SILTY CLAY	29	28		2.5	
9.300	30.51	27.89	4.66	CLAY	28	26		1.5	
9.450	31.00	19.89	3.97	CLAY to SILTY CLAY	13	13		1.2	
9.600	31.50	12.56	4.22	CLAY	13	12		.7	
9.750	31.99	13.47	4.68	CLAY	13	13		.8	
9.900	32.48	19.59	3.47	CLAY to SILTY CLAY	13	12		1.2	
10.050	32.97	12.58	4.21	CLAY	13	12		.7	
10.200	33.46	17.29	3.88	CLAY to SILTY CLAY	12	11		1.0	
10.350	33.96	21.75	5.33	CLAY	22	20		1.2	
10.500	34.45	49.14	4.62	CLAY to SILTY CLAY	33	30		2.8	
10.650	34.94	49.46	4.85	CLAY to SILTY CLAY	33	30		2.8	
10.800	35.43	25.07	7.18	CLAY	25	23		1.4	
10.950	35.93	38.98	4.03	CLAYEY SILT to SILTY CLAY	19	18		2.2	
11.100	36.42	19.67	5.29	CLAY	20	18		1.2	
11.250	36.91	16.66	4.62	CLAY	17	15		1.0	
11.400	37.40	117.55	4.16	*VERY STIFF FINE GRAINED	100	100			
11.550	37.89	240.66	3.51	*SAND to CLAYEY SAND	100	100			
11.700	38.39	89.40	4.16	CLAYEY SILT to SILTY CLAY	45	39		5.1	
11.850	38.88	33.89	4.93	CLAY	34	30		1.9	
12.000	39.37	28.28	4.95	CLAY	28	25		1.5	
12.150	39.86	28.72	4.32	CLAY to SILTY CLAY	19	17		1.6	
12.300	40.35	23.69	4.56	CLAY	24	20		1.4	
12.450	40.85	31.59	3.32	CLAYEY SILT to SILTY CLAY	16	14		2.0	
12.600	41.34	40.03	3.55	CLAYEY SILT to SILTY CLAY	20	17		2.5	
12.750	41.83	30.06	5.06	CLAY	30	26		1.6	
12.900	42.32	27.55	5.23	CLAY	28	23		1.5	
13.050	42.81	41.38	4.35	CLAY to SILTY CLAY	28	23		2.3	
13.200	43.31	28.13	3.95	CLAY to SILTY CLAY	19	16		1.7	
13.350	43.80	25.45	4.05	CLAY to SILTY CLAY	17	14		1.5	
13.500	44.29	23.33	3.64	CLAY to SILTY CLAY	16	13		1.4	
13.650	44.78	18.63	4.03	CLAY to SILTY CLAY	12	10		1.1	

*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 110 pcf
ASSUMED DEPTH OF WATER TABLE = 12.3 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

SOUNDING : HLA-6

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
13.800	45.28	21.95	5.15	CLAY	22	18		1.1	
13.950	45.77	23.60	4.49	CLAY	24	19		1.4	
14.100	46.26	30.74	4.91	CLAY	31	25		1.7	
14.250	46.75	30.06	5.26	CLAY	30	25		1.6	
14.400	47.24	32.25	5.40	CLAY	32	26		1.7	
14.550	47.74	38.50	4.91	CLAY	39	31		2.1	
14.700	48.23	37.07	5.48	CLAY	37	30		2.0	
14.850	48.72	35.42	5.28	CLAY	35	29		1.9	
15.000	49.21	34.91	5.30	CLAY	35	28		1.9	
15.150	49.70	33.16	4.73	CLAY	33	27		1.8	
15.300	50.20	32.40	5.46	CLAY	32	26		1.7	
15.450	50.69	37.48	5.74	CLAY	37	30		2.0	
15.600	51.18	39.37	5.66	CLAY	39	31		2.2	
15.750	51.67	41.60	5.48	CLAY	42	33		2.3	
15.900	52.17	44.74	5.16	CLAY	45	35		2.5	
16.050	52.66	51.03	5.41	CLAY	51	40		2.8	
16.200	53.15	48.23	5.52	CLAY	48	38		2.7	
16.350	53.64	49.12	6.35	CLAY	49	38		2.7	
16.500	54.13	40.54	5.92	CLAY	41	31		2.2	
16.650	54.63	34.08	5.58	CLAY	34	26		1.8	
16.800	55.12	34.40	5.81	CLAY	34	26		1.8	
16.950	55.61	32.82	5.73	CLAY	33	25		1.8	
17.100	56.10	33.18	5.15	CLAY	33	25		1.8	
17.250	56.59	37.82	5.55	CLAY	38	29		2.0	
17.400	57.09	44.49	4.61	CLAY to SILTY CLAY	30	22		2.4	
17.550	57.58	31.78	5.29	CLAY	32	24		1.7	
17.700	58.07	21.86	4.76	CLAY	22	16		1.2	
17.850	58.56	22.82	4.78	CLAY	23	17		1.2	
18.000	59.06	20.44	4.50	CLAY	20	15		1.1	
18.150	59.55	18.91	4.60	CLAY	19	14		1.0	
18.300	60.04	34.29	8.78	CLAY	34	25		1.8	
18.450	60.53	109.39	*****		0	0			.0

*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 110 pcf
ASSUMED DEPTH OF WATER TABLE = 12.3 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

HOLGUIN, FAHAN & ASSOCIATES, INC.

Interpretations based on: Robertson and Campanella, 1989.

Kleinfelder, *Geotechnical Investigation Report, Chevron Park, San Ramon, California,* dated June 1981.

DEPTH IN FEET	DRY DENSITY lb/ft³	MOISTURE CONTENT % DRY WEIGHT	BLOW COUNT	SAMPLE	USCS	DESCRIPTION
0					CH	Black, silty clay, roots, medium stiff, dry, desiccated.
	113.2	15.2	26	1-2		
						Brown, clayey sandy silt, medium dense, dry, caliche, small seep holes.
5			18	1-5		
10			20	1-10	ML	
15			20	1-15		▼ Brown to dark brown, silty clay, stiff, damp.
20			23	1-20	CL	
25			20	1-25	SM	Brown to light brown, silty sand, caliche, medium dense, damp. Silty clay between 28 - 31 ft.
30			21	1-30		Bottom of boring at 30 ft.

J.H. KLEINFELDER & ASSOCIATES
GEOTECHNICAL CONSULTANTS • MATERIALS TESTING

CHEVRON PARK
SAN RAMON, CALIFORNIA

LOG OF BORING NO. B-1

PLATE A-2

PREPARED BY: PLC DATE: 8/81
CHECKED BY: DCM DATE: 8/81

PROJECT NO. B-1109-1

DEPTH IN FEET	DRY DENSITY lb/ft³	MOISTURE CONTENT % DRY WEIGHT	BLOW COUNT	SAMPLE	USCS	DESCRIPTION
0			35	2-2	CH	Black, silty clay, desiccated, roots to 3 ft, dry, stiff.
5	105.1	16.8	16	2-5	ML	Brown, sandy clayey silt, dense, dry, caliche.
10	111.1	5.0	23	2-10		
					SM	Brown, silty sand, fine to medium grained, medium dense, dry, with fine gravel.
15	98.5	23.5	18	2-15	CL/CH	▼ Dark brown, mottled gray, silty clay, medium stiff, damp.
20			18	2-20		
25			21	2-25		Color change to greenish gray.
30			22	2-30		Bottom of boring at 30 ft.

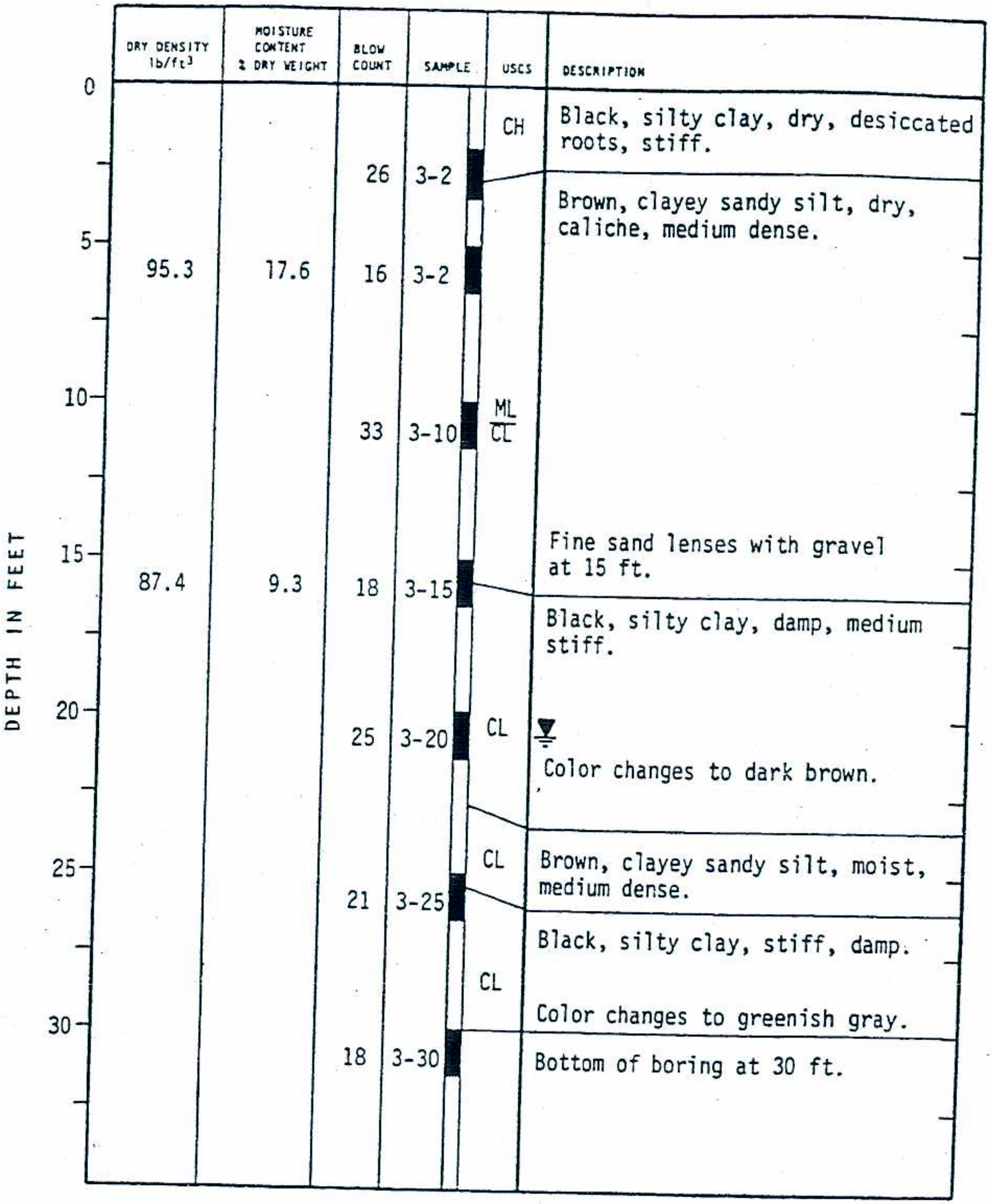
J.H. KLEINFELDER & ASSOCIATES
GEOTECHNICAL CONSULTANTS • MATERIALS TESTING

PREPARED BY: PLC DATE: 8/81
CHECKED BY: DCM DATE: 8/81

CHEVRON PARK
SAN RAMON, CALIFORNIA
LOG OF BORING NO. B-2

PROJECT NO. B-1109-1

PLATE
A-3



DRY DENSITY lb/ft^3	MOISTURE CONTENT % DRY WEIGHT	BLOW COUNT	SAMPLE	USCS	DESCRIPTION
				CH	Black, silty clay, dry, desiccated roots, stiff.
		26	3-2		Brown, clayey sandy silt, dry, caliche, medium dense.
95.3	17.6	16	3-2		
		33	3-10	ML CL	
87.4	9.3	18	3-15		Fine sand lenses with gravel at 15 ft.
					Black, silty clay, damp, medium stiff.
		25	3-20	CL	▼ Color changes to dark brown.
				CL	Brown, clayey sandy silt, moist, medium dense.
		21	3-25		Black, silty clay, stiff, damp.
				CL	Color changes to greenish gray.
		18	3-30		Bottom of boring at 30 ft.

J.H. KLEINFELDER & ASSOCIATES
GEOTECHNICAL CONSULTANTS • MATERIALS TESTING

PREPARED BY: PLC DATE: 8/81
CHECKED BY: DCM DATE: 8/81

CHEVRON PARK
SAN RAMON, CALIFORNIA

LOG OF BORING NO. B-3

PROJECT NO. B-1109-1

PLATE
A-4

DEPTH IN FEET	DRY DENSITY lb/ft^3	MOISTURE CONTENT % DRY WEIGHT	BLOW COUNT	SAMPLE	USCS	DESCRIPTION
0					CL	Black, silty clay, desiccated, dry, stiff.
						Dark brown to brown, sandy clayey silt, caliche, stiff,dry.
5			19	17-5		
10	101.4	18.3	25	17-10	ML	
15			36	17-15		
						With pebbles.
20	91.3	30.4	16	17-20	SC	▼ Brown, clayey sand, medium dense, wet.
					CH/CL	Dark grey, silty clay, medium stiff to stiff.
25	99.5	26.4	17	17-25		Color change to greyish brown.
30	110.3	19.0	40	17-30		
35						

J.H. KLEINFELDER & ASSOCIATES
GEOTECHNICAL CONSULTANTS • MATERIALS TESTING

PREPARED BY: PLC DATE: 8/81
CHECKED BY: DCM DATE: 8/81

CHEVRON PARK
SAN RAMON, CALIFORNIA

LOG OF BORING NO. B-17

PROJECT NO. B-1109-1

PLATE A-18

DEPTH IN FEET

	DRY DENSITY lb/ft³	MOISTURE CONTENT % DRY WEIGHT	BLOW COUNT	SAMPLE	USCS	DESCRIPTION
35			27	17-35		Brown, fine sand with trace of clay.
40			32	17-40	SM	Sand and gravel at 41-43 ft.
45			34	17-45		
50			38	17-50		Bottom of boring at 50 ft.

J.H. KLEINFELDER & ASSOCIATES

GEOTECHNICAL CONSULTANTS • MATERIALS TESTING

CHEVRON PARK

SAN RAMON, CALIFORNIA

LOG OF BORING NO. B-17 (con't)

PLATE

A-18.1

PREPARED BY: PLC DATE: 8/81

CHECKED BY: DCM DATE: 8/81

PROJECT NO. B-1109-1

DEPTH IN FEET	DRY DENSITY lb/ft³	MOISTURE CONTENT % DRY WEIGHT	BLOW COUNT	SAMPLE	USCS	DESCRIPTION
0					CH	Black, silty clay, stiff, dry, dessiccated.
						Brown, sandy clayey silt with pebbles, medium dense, dry.
5			11	19-5		
10			10	19-10		Sand at 10½-11½ ft. moist.
					ML	
15			18	19-15		▼
20			21	19-20		
						Dark brown, sity clay, medium stiff dry to damp.
25	108.0	20.5	19	19-25	CL	Color change to grey brown.
30			25	19-30		
					SM	Brown, clayey fine sand medium dense.
35						

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PREPARED BY: PLC DATE: 8/81
CHECKED BY: DCM DATE: 8/81

CHEVRON PARK
SAN RAMON, CALIFORNIA
LOG OF BORING NO. B-19

PROJECT NO. B-1109-1

PLATE
A-20

DEPTH IN FEET

	DRY DENSITY lb/ft³	MOISTURE CONTENT % DRY WEIGHT	BLOW COUNT	SAMPLE	USCS	DESCRIPTION
35			30	19-35	SM	Brown, fine sand and gravel, wet.
40						
45						
					GP	Brown, sand and gravel with clay.
50			37	19-50	CL	Grey-brown, sandy clay with gravel, stiff.
						Bottom of boring at 50 ft.

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CHEVRON PARK
SAN RAMON, CALIFORNIA
LOG OF BORING NO. B-19 (con't)

PROJECT NO. B-1109-1

PLATE
A-20.1

DEPTH IN FEET	DRY DENSITY lb/ft^3	MOISTURE CONTENT % DRY WEIGHT	BLOW COUNT	SAMPLE	USCS	DESCRIPTION
0					CH	Black, silty clay, stiff, roots, dry, desiccated.
			22	20-2		
5			21	20-5	ML	Brown, sandy clayey silt, dry dense, grading change to sandy silt/silty sand with fine gravel, caliche.
10			13	20-10		
15	97.1	24.4	15	20-15		
					CL	▽ Brown to dark brown, silty clay, medium stiff, damp.
20			20	20-20		
25			20	20-25	ML	Brown, sandy clay silt, damp to moist, medium dense.
30			17	20-30		Bottom of boring at 30 ft.

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CHEVRON PARK
SAN RAMON, CALIFORNIA

LOG OF BORING NO. B-20

PROJECT NO. B-1109-1

PLATE
A-21

DEPTH IN FEET	DRY DENSITY lb/ft³	MOISTURE CONTENT % DRY WEIGHT	BLOW COUNT	SAMPLE	USCS	DESCRIPTION
0					CH	Black, silty clay, desiccated, dry, stiff.
				22-2		Brown, silty clay, caliche, medium stiff, dry.
5				22-5		
10	108.0	33.0		22-10	CH	Trace of sand.
						▼
15				22-15		Color change to dark brown with sand.
20				22-20		Bottom of boring at 20 ft.

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CHEVRON PARK
SAN RAMON, CALIFORNIA
LOG OF BORING NO. B-22

PROJECT NO. B-1109-1

PLATE
A-23

DEPTH IN FEET	DRY DENSITY lb/ft^3	MOISTURE CONTENT % DRY WEIGHT	BLOW COUNT	SAMPLE	USCS	DESCRIPTION
0				Bulk	CH	Black, silty clay, desiccated dry stiff.
5					CL	Dark brown to brown, silty clay, stiff, trace of sand and pebbles below 8 ft.
10						Bottom of boring at 10 ft. NFWE

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CHEVRON PARK
SAN RAMON, CALIFORNIA
LOG OF BORING NO. B-23

PROJECT NO. B-1109-1

PLATE
A-24

DEPTH IN FEET	DRY DENSITY lb/ft³	MOISTURE CONTENT % DRY WEIGHT	BLOW COUNT	SAMPLE	USCS	DESCRIPTION
0					CH	Black to dark brown, silty clay, dry, stiff caliche, disiccated.
			14	27-2		
5			19	27-5	ML	Light brown, sandy clayey silt, trace of gravel at 6½,grading to med. sand at 9-10'.
10			13	27-10	CL ML	Mottled brown-grey, silty clay, medium stiff.
15			16	27-15		Grading to clayey silt.
20			15	27-20	SM	▼ Brown, sand, wet.
					CL	Black, silty clay, stiff, damp.
25			16	27-25		Color change to greenish grey.
						Bottom of boring at 25 ft.
30						

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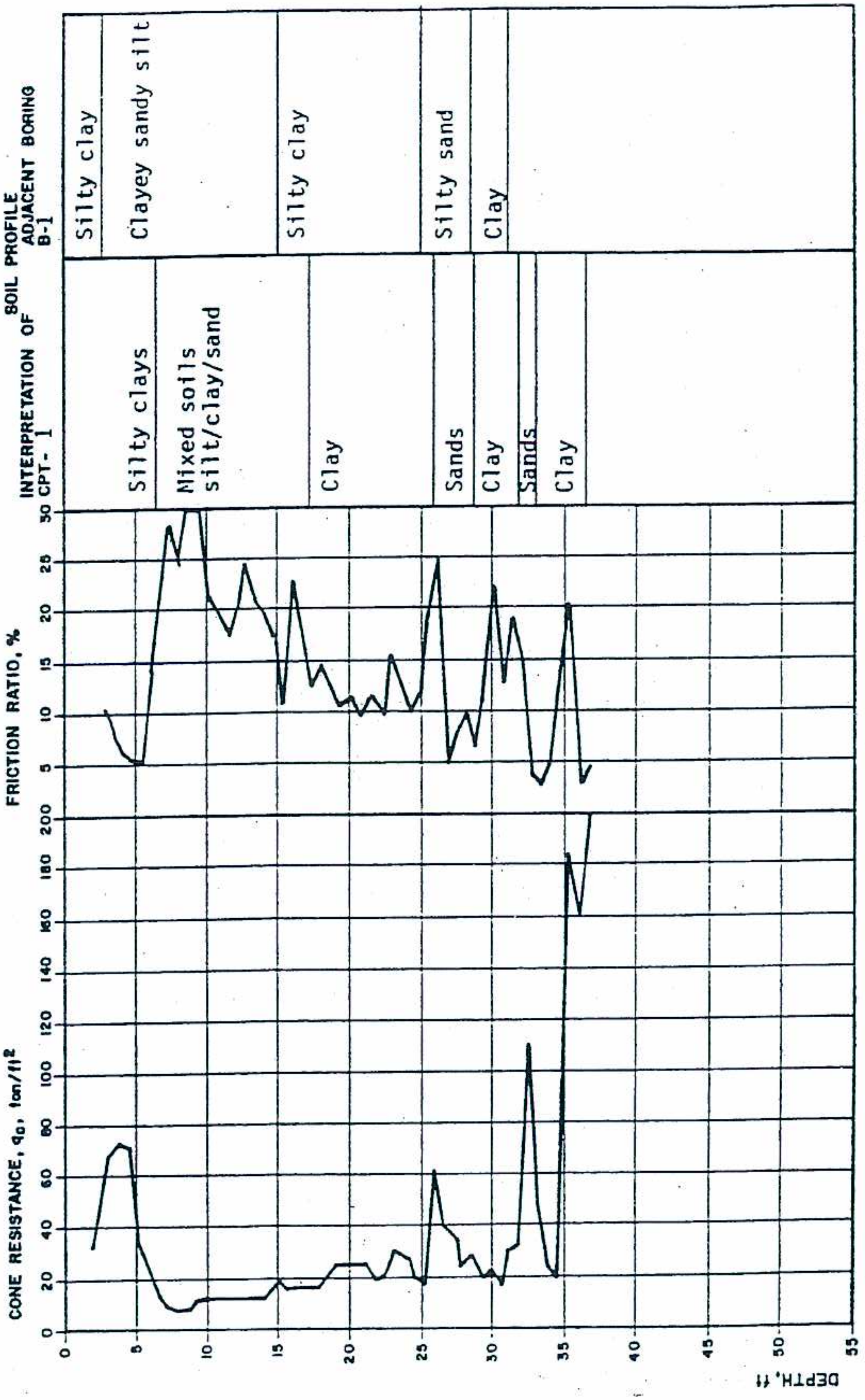
CHECKED BY: DCM DATE: 8/81

CHEVRON PARK
SAN RAMON, CALIFORNIA

LOG OF BORING NO. B-27

PROJECT NO. B-1109-1

PLATE A-28



J.H. KLEINFELDER & ASSOCIATES

GEOTECHNICAL CONSULTANTS • MATERIALS TESTING

CHEVRON PARK
SAN RAMON, CALIFORNIA

LOG OF CPT - 1

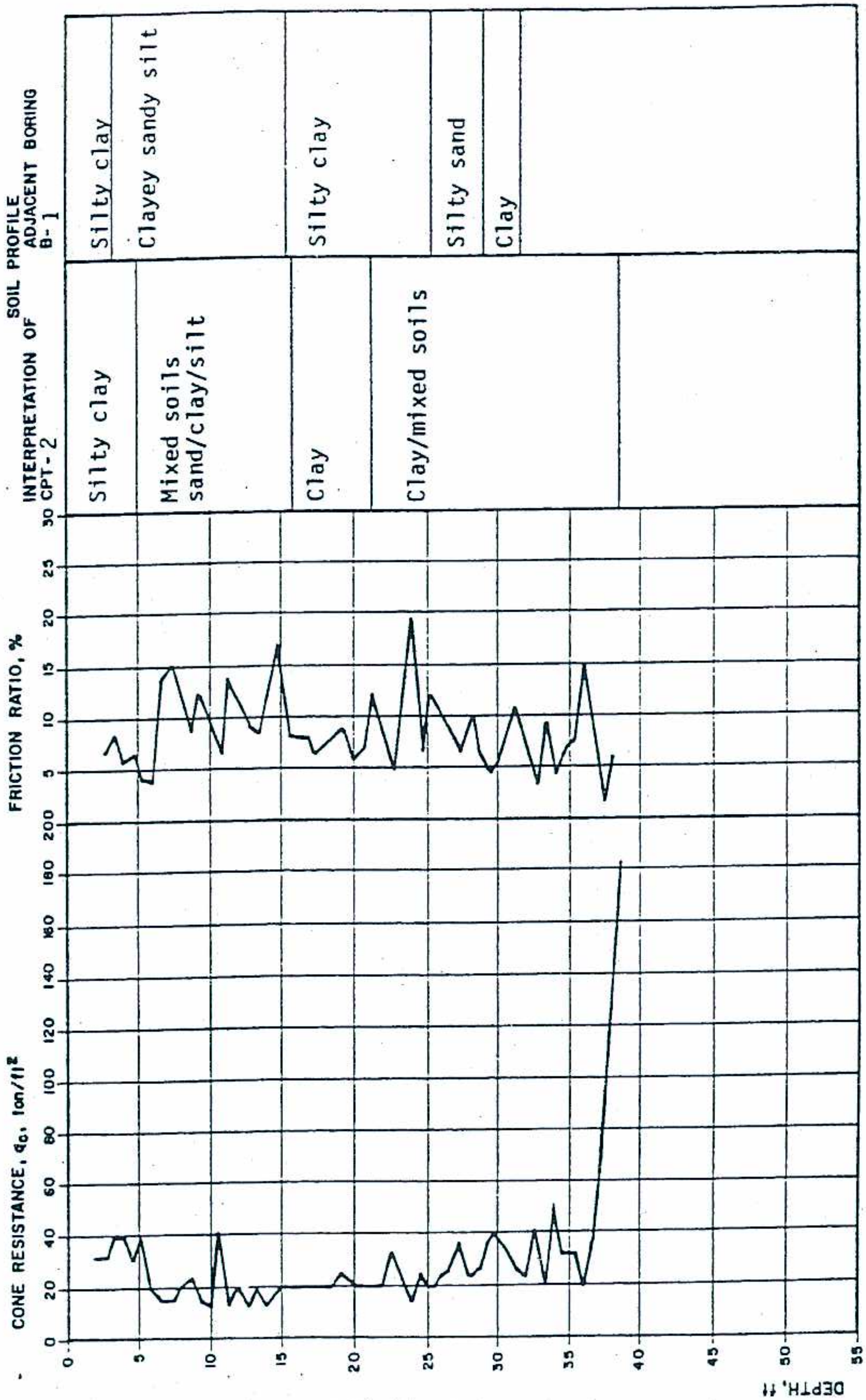
PLATE

B-3

PREPARED BY: PLC DATE: 8/81

CHECKED BY: DCM DATE: 8/81

PROJECT NO. B-1109-1



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PREPARED BY: PLC DATE: 8/81

CHECKED BY: DCM DATE: 8/81

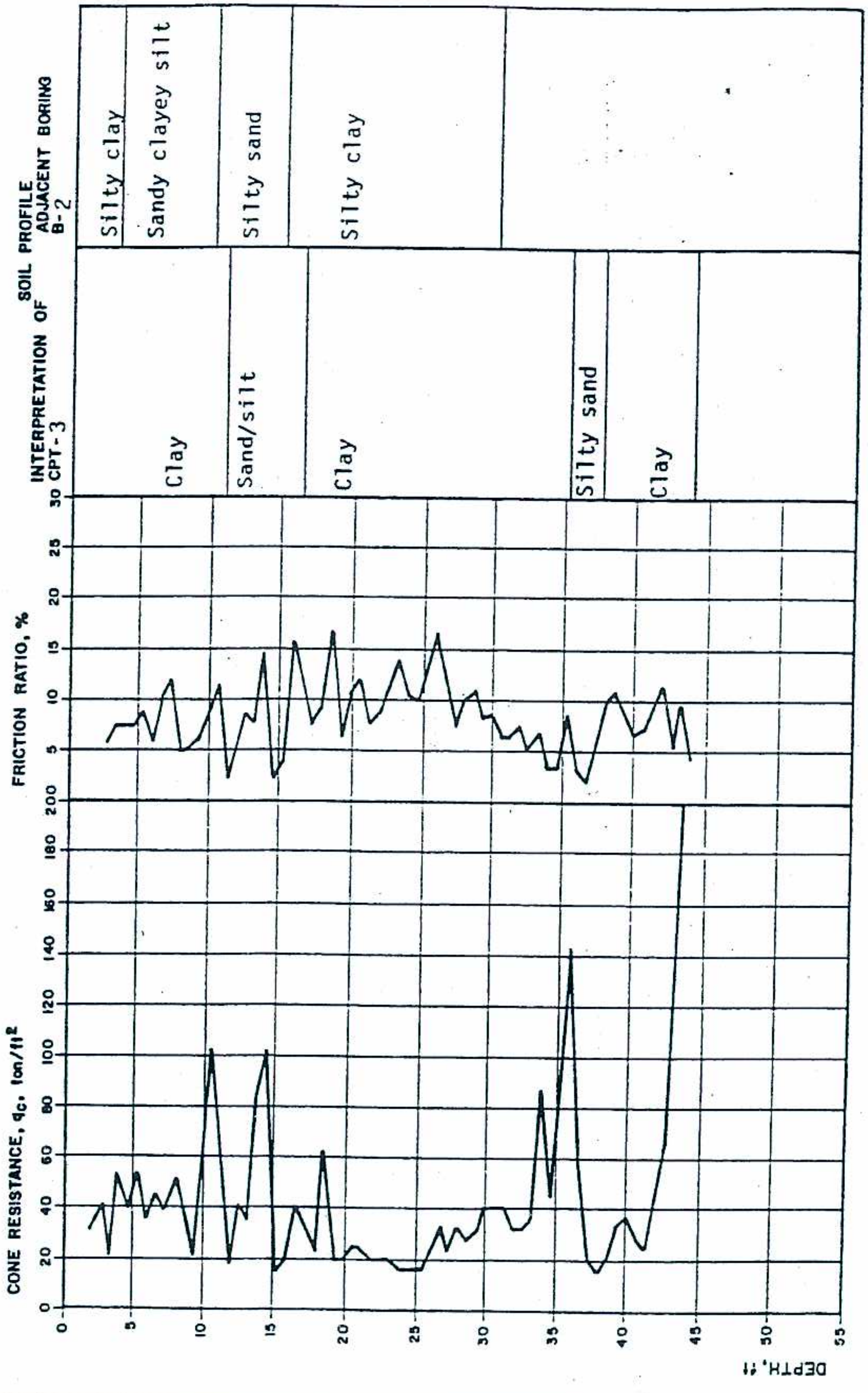
CHEVRON PARK
SAN RAMON, CALIFORNIA

LOG OF CPT - 2

PROJECT NO. B-1109-1

PLATE

B-4



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PREPARED BY: PLC DATE: 8/81
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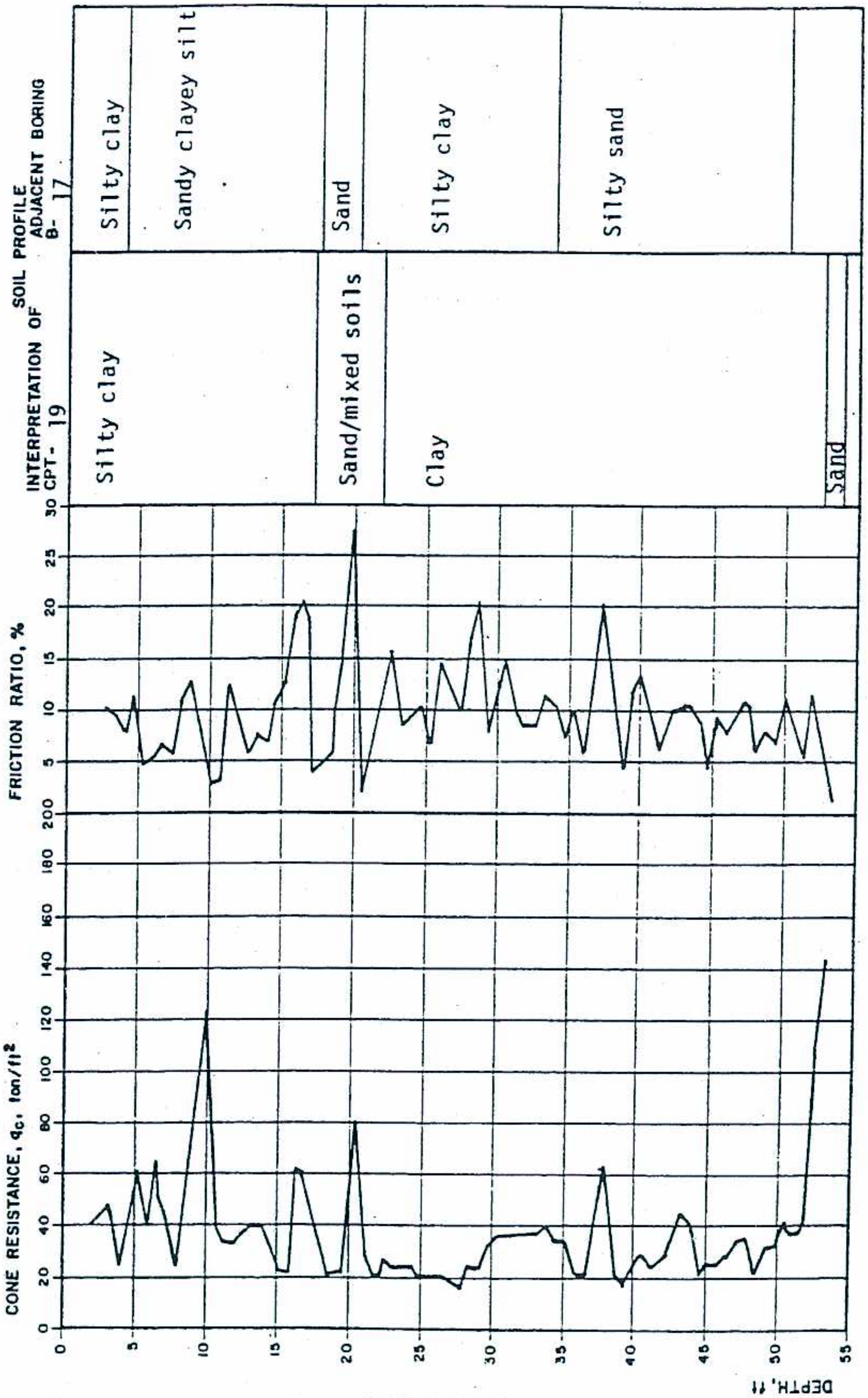
CHEVRON PARK
SAN RAMON, CALIFORNIA

LOG OF CPT - 3

PROJECT NO. B-1109-1

PLATE

B-5



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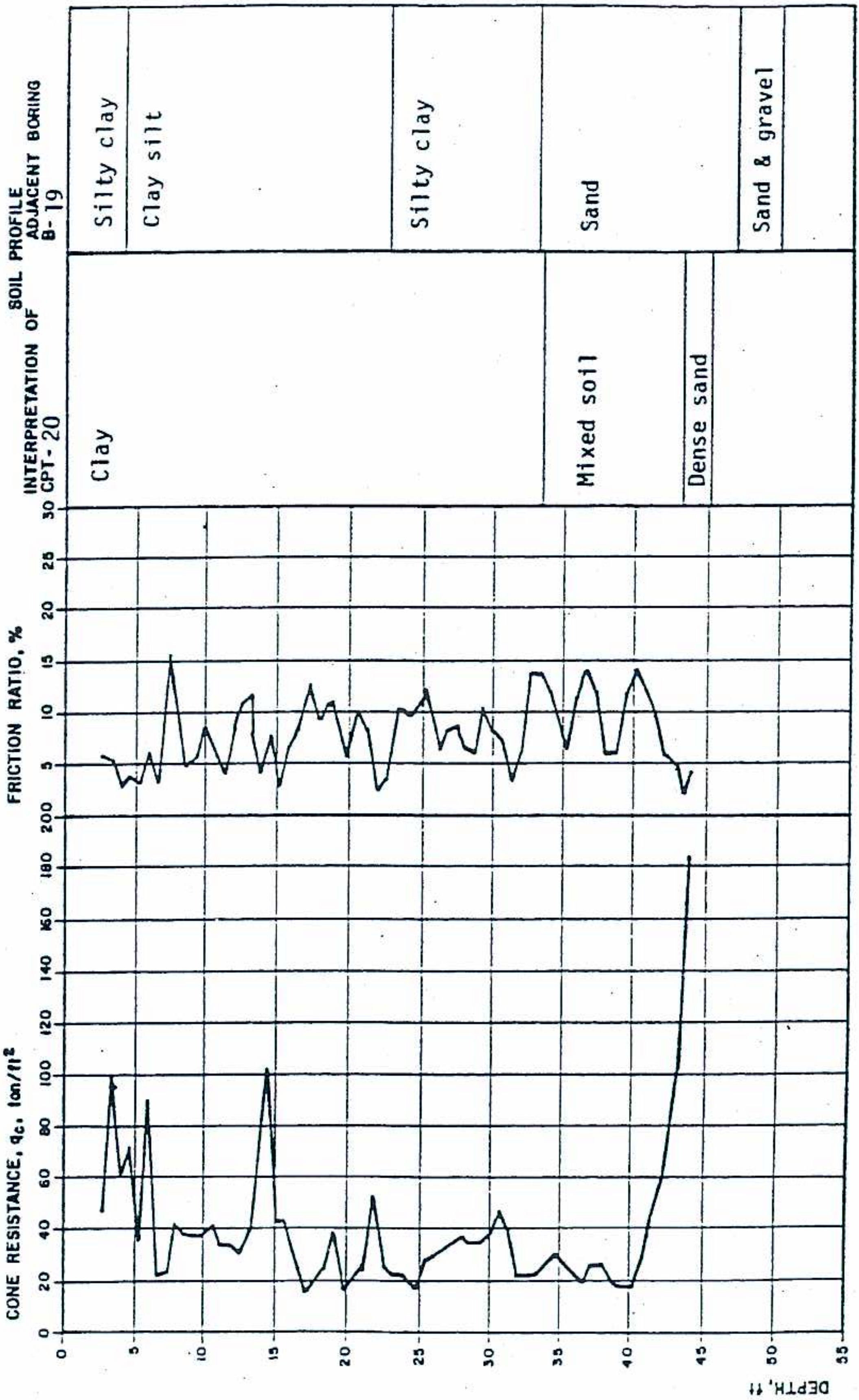
CHEVRON PARK
SAN RAMON, CALIFORNIA

LOG OF CPT - 19

PROJECT NO. B-1109-1

PLATE
B-21



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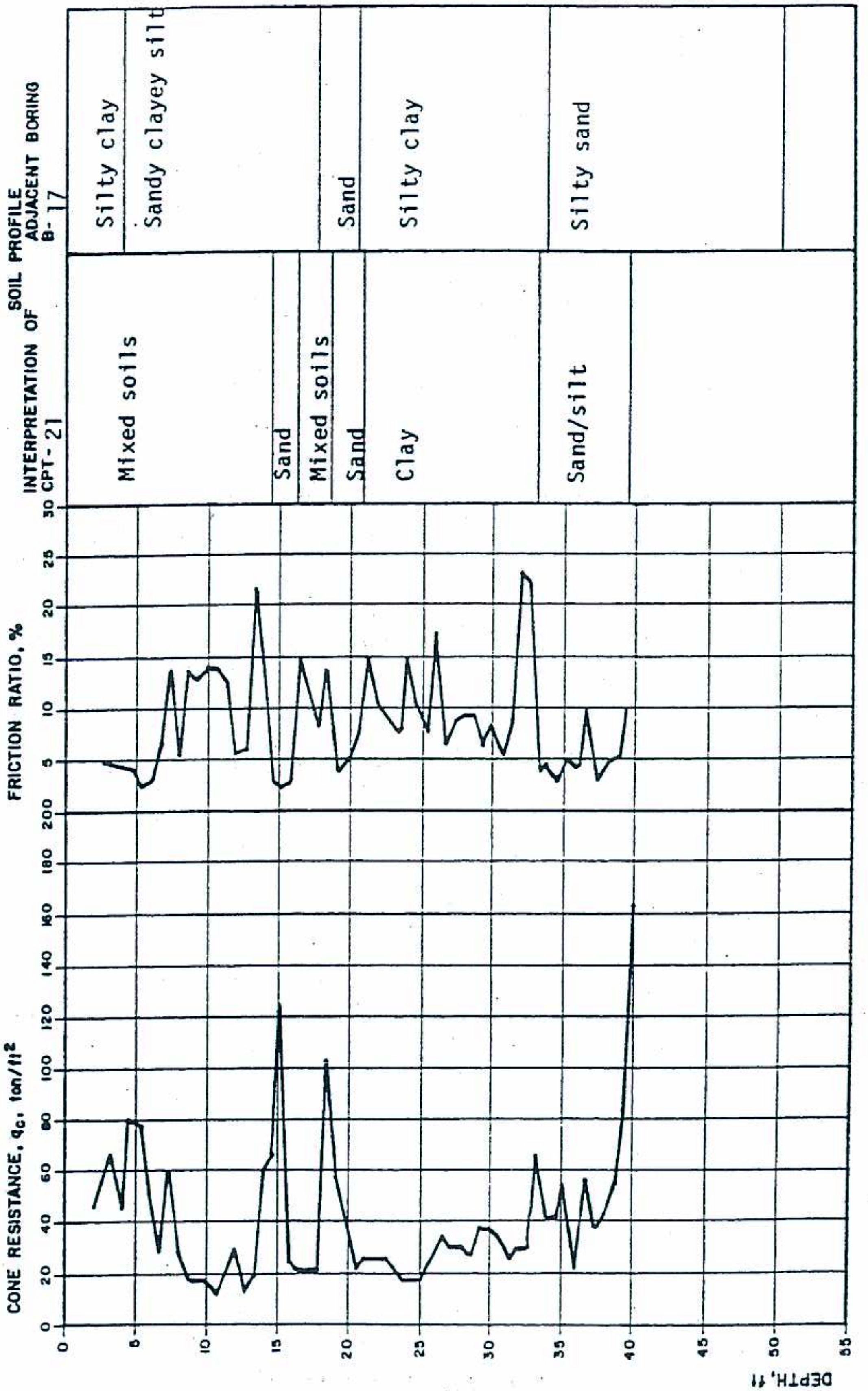
CHEVRON PARK
SAN RAMON, CALIFORNIA

LOG OF CPT-20

PROJECT NO. B-1109-1

PLATE

B-22



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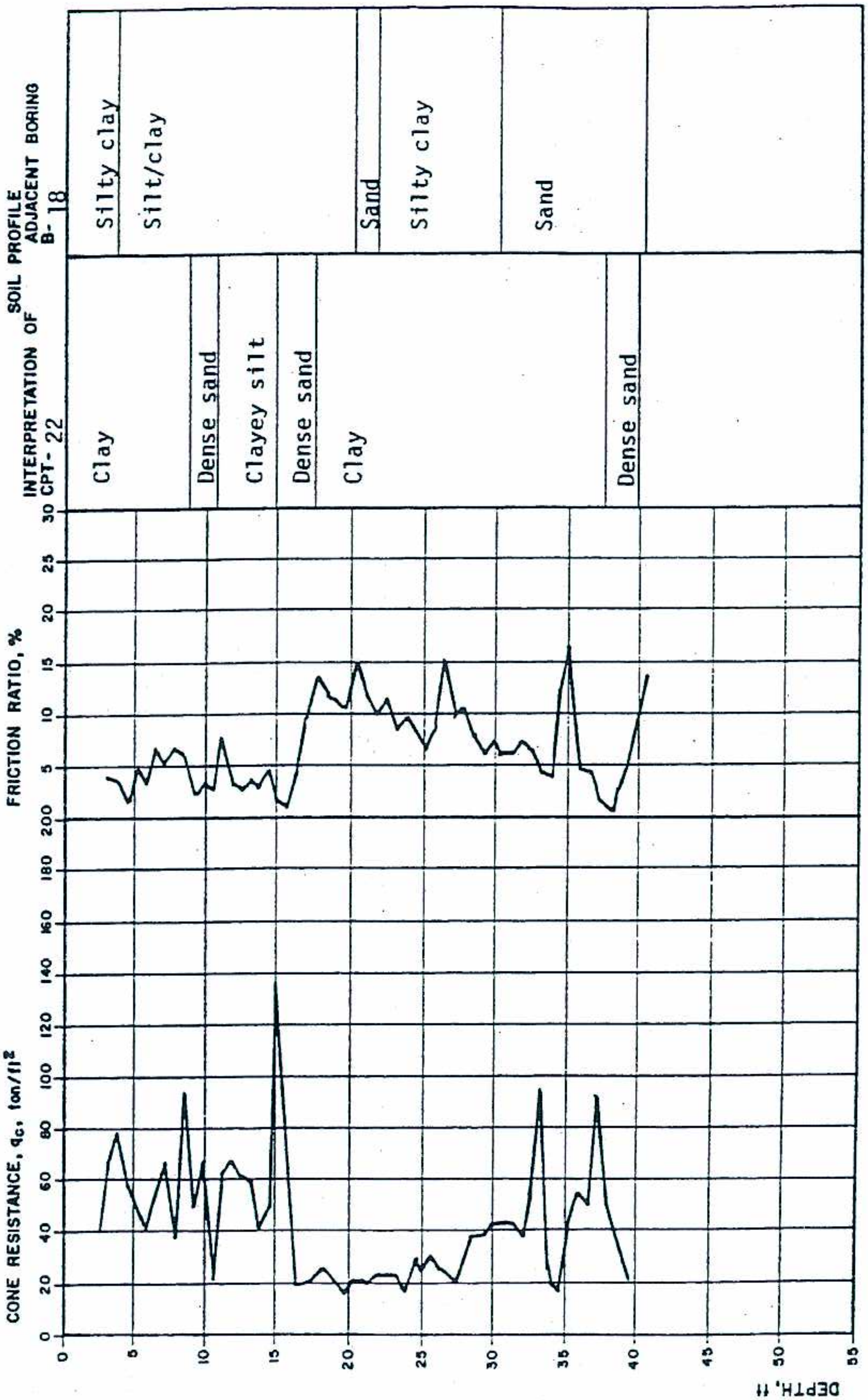
CHEVRON PARK
SAN RAMON, CALIFORNIA

LOG OF CPT-21

PROJECT NO. B-1109-1

PLATE

B-23



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PREPARED BY: PLC DATE: 8/81

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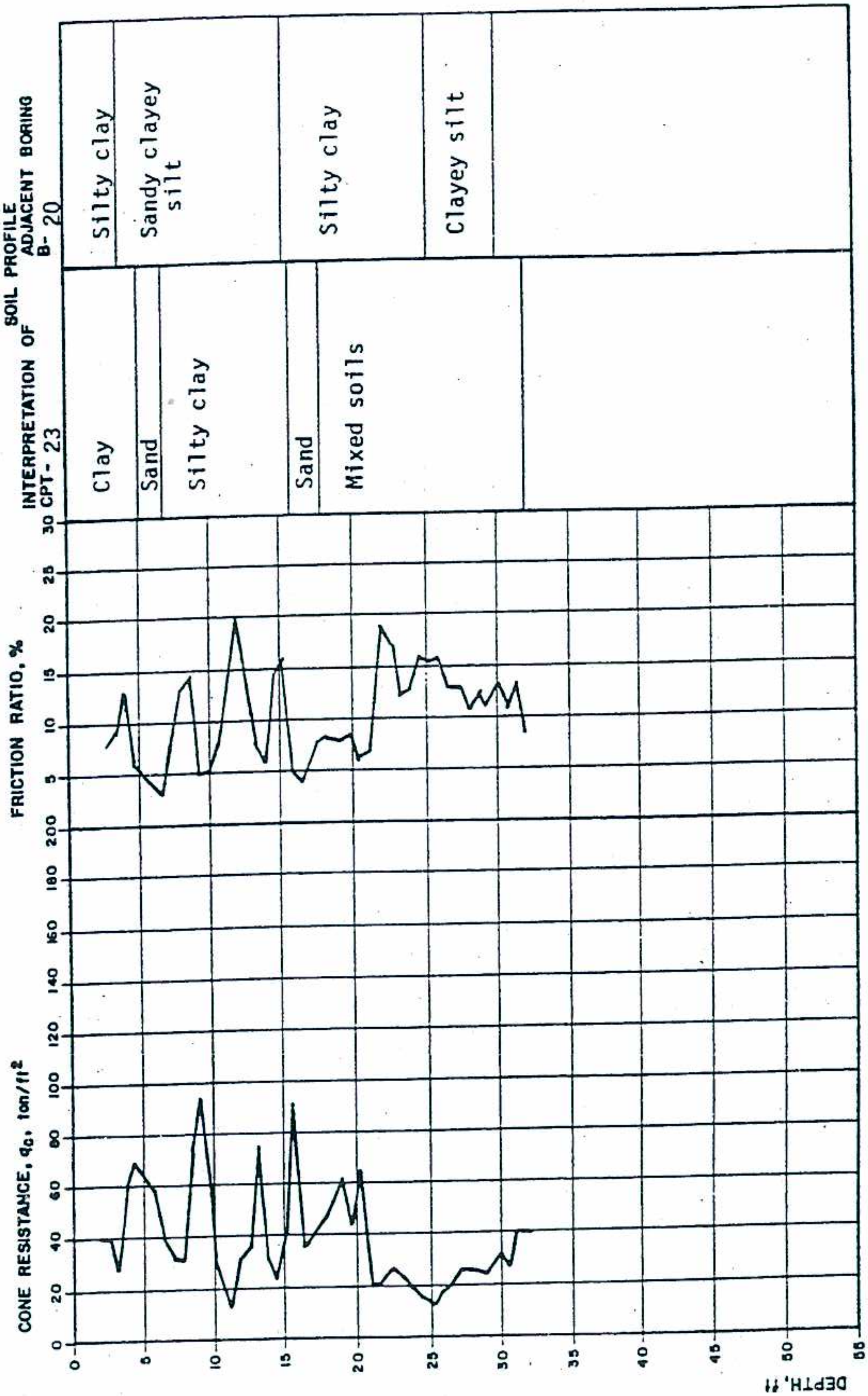
CHEVRON PARK
SAN RAMON, CALIFORNIA

LOG OF CPT - 22

PROJECT NO. B-1109-1

PLATE

B-24



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PREPARED BY: PLC DATE: 8/81
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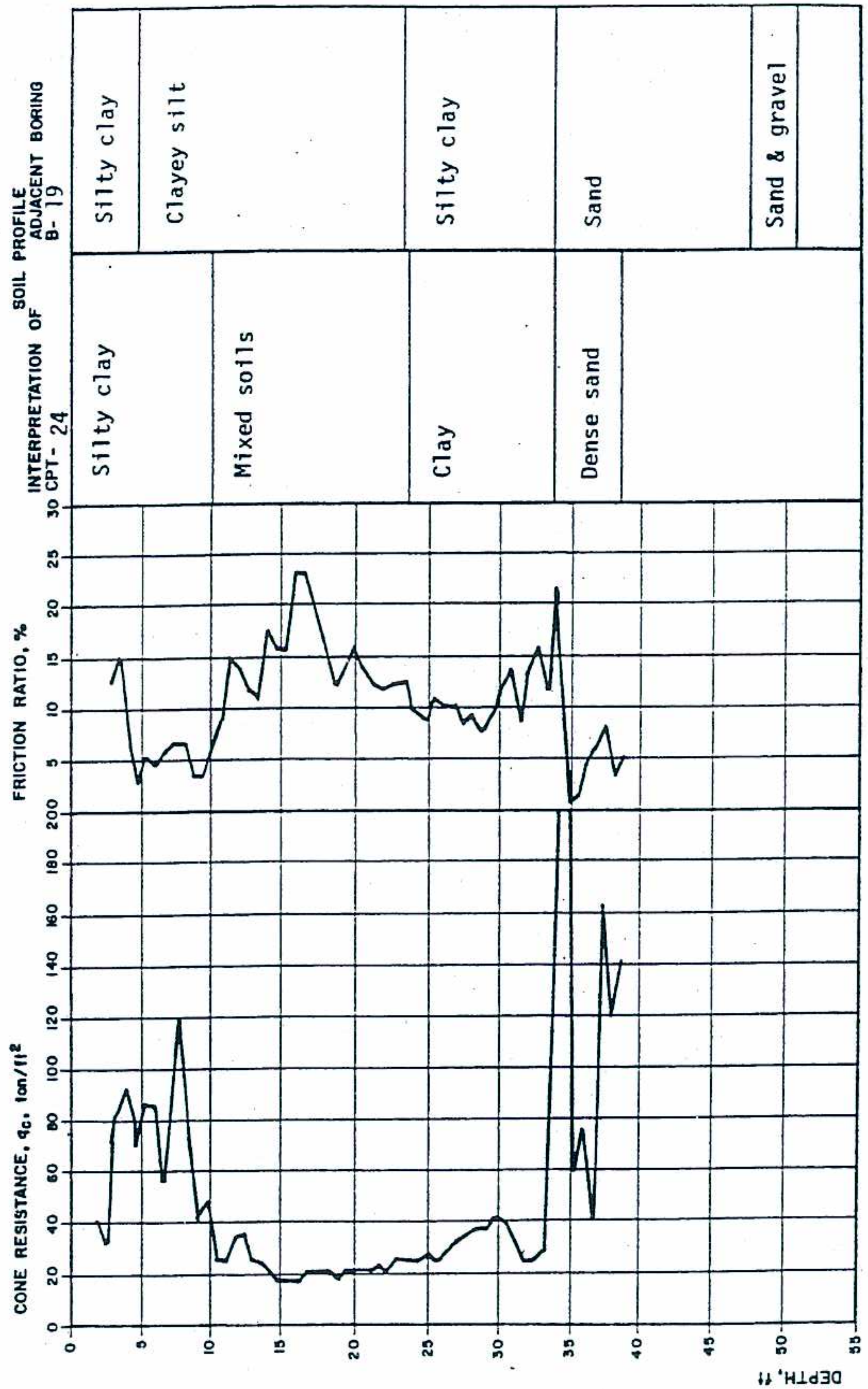
CHEVRON PARK
SAN RAMON, CALIFORNIA

LOG OF CPT - 23

PROJECT NO. B-1109-1

PLATE

B-25



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GEOTECHNICAL CONSULTANTS • MATERIALS TESTING

PREPARED BY: PLC DATE: 8/81
CHECKED BY: DCM DATE: 8/81

CHEVRON PARK
SAN RAMON, CALIFORNIA

LOG OF CPT - 24

PROJECT NO. B-1109-1

PLATE

B-26

APPENDIX B

BORING LOGS FROM THIS INVESTIGATION

Checked RH

Approved

UNIFIED SOIL CLASSIFICATION SYSTEM - ASTM D2488-93

MAJOR DIVISIONS			SYMBOLS		TYPICAL NAMES
COARSE-GRAINED SOILS OVER 50% RETAINED ON No.200 SIEVE SIZE	GRAVELS MORE THAN 1/2 OF COARSE FRACTION RETAINED ON No.4 SIEVE SIZE	CLEAN GRAVELS WITH LESS THAN 5% FINES	GW	⁘	Well-graded gravels or gravel-sand mixtures, little or no fines
			GP	○	Poorly graded gravels or gravel-sand mixtures, little or no fines
		GRAVELS WITH OVER 15% FINES	GM	◫	Silty gravels, gravel-sand-silt mixtures
			GC	▨	Clayey gravels, gravel-sand-clay mixtures
	SANDS MORE THAN 1/2 OF COARSE FRACTION PASSING No.4 SIEVE SIZE	CLEAN SANDS WITH LESS THAN 5% FINES	SW	⁙	Well-graded sand or gravelly sands, little or no fines
			SP	⁘	Poorly graded sands or gravelly sands, little or no fines
		SANDS WITH OVER 15% FINES	SM	◫	Silty sand, sand-silt mixtures
			SC	▨	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS OVER 50% PASSING No.200 SIEVE SIZE	SILTS & CLAYS LIQUID LIMIT 50% OR LESS		ML	▥	Inorganic silts and sandy or gravelly silts, rock flour
			CL	▨	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	☰	Organic silts and organic silty clays of low plasticity
	SILTS & CLAYS LIQUID LIMIT GREATER THAN 50%		MH	▥	Inorganic silts, micaceous or diatomaceous fine sandy soils, elastic silts
			CH	▨	Inorganic clays of high plasticity, fat clays
			OH	≈	Organic clays and silty clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			PT	⁂	Peat and other highly organic soils

◨	NX Core Sampler
◩	SPT Sampler
◧	Sprague & Henwood Sampler
■	Direct Push
◫	Pitcher Barrel
☒	Grab or Bulk Sample
▼	G.W. measured after water level stabilizes
▽	G.W. measured during or soon after drilling
Perm	Permeability
Consol	Consolidation
LL	Liquid Limit (%)
PI	Plasticity Index (%)
EI	Expansion Index (%)
Gs	Specific Gravity
MA	Particle Size Analysis
-200=55%	Percent Passing No. 200 Sieve

Shear Strength (psf) ↓ ↓ Confining Pressure

TxUU 3200 (2600) (FM) or (S)	-Unconsolidated Undrained Triaxial Shear (field moisture or saturated)
TxCU 3200 (2600) (P)	-Consolidated Undrained Triaxial Shear (with or without pore pressure measurement.)
TxCD 3200 (2600)	-Consolidated Drained Triaxial Shear
SSCU 3200 (2600) (P)	-Simple Shear Consolidated Undrained (with or without pore pressure measurement.)
SSCD 3200 (2600)	-Simple Shear Consolidated Drained
DSCD 2700 (2000)	-Consolidated Drained Direct Shear
UC 470	-Unconfined Compression
LVS 700	-Laboratory Vane Shear

KEY TO TEST DATA

Source: ASTM D 2488-93, based on Unified Soil Classification system

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Soil Classification Chart and Key to Test Data BORING

Bishop Ranch City Center Project
Parcel 1&1A
San Ramon, California

B-1

DRAWN	JOB NUMBER	CHECKED	CHCK'D DATE	APPROVED	APPRV'D DATE
RH	4096088527	RH	10/08		10/08

RELATIVE DENSITY OF COARSE-GRAINED SOILS

Relative Density	Standard Penetration Test Blow Count (blows per foot)
very loose	<4
loose	4-10
medium dense	10-30
dense	30-50
very dense	>50

CONSISTENCY OF FINE-GRAINED SOILS

Consistency	Approximate Blows/foot (SPT)	Undrained Shear Strength (psf)
very soft	<2	0 - 250
soft	2-4	250 - 500
medium stiff	4-8	500 - 1,000
stiff	8-15	1,000 - 2,000
very stiff	15-30	2,000 - 4,000
hard	>30	>4,000

NATURAL MOISTURE CONTENT

Dry	- Requires considerable moisture to obtain optimum moisture content for compaction
Moist	- Near the optimum moisture content for compaction
Wet	- Requires drying to obtain optimum moisture content for compaction

Note: Where laboratory data are not available, the field classifications given above provide a general indication of material properties; the classifications may require modification based on judgment or laboratory testing.

PHYSPROPS-SOIL.DWG 40.0
20070208.1259

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Physical Properties Criteria for Soil Classification
Bishop Ranch City Center Project
Parcel 1&1A
San Ramon, California

PLATE:

B-2

DRAWN	JOB NUMBER	CHECKED	CHECKED DATE	APPROVED	APPROVED DATE
RL	4096-08-8527	RM	9/08		12/08

Date 8/19/08
Equipment Gregg Drilling
Drilling Method HSA
Sampler SPT/ S&H
Hammer Weight 140 lbs **Drop** 30
Logged by RL **Datum** MSL
Surface Elevation 439

Other Tests/Drilling Notes	Moisture Content (%)	Dry Density (pcf)	Shear Strength*** (ksf)	Shear Strength ** (ksf)	Blows per Foot	Depth (ft.)	
						0	Dark gray coarse SAND with silt (SM), moist (Fill?)
							Dark gray LEAN CLAY (CL),very Stiff, moist
Corrosion			4.5	2	20*	5	
LL=38, PL = 24, PI = 14, Consol	18.2	101.0	3.0	0.8	21*	10	Becomes light gray
							Light brown clayey SAND (SC), medium dense, wet
TXUU 986 (1800)			0.75	0.7	8*	15	Light brown FAT CLAY (CH), medium stiff, wet
			2.0	1.1	14*	20	∇ Becomes light gray with brown spots, stiff
LL=55, PL = 27, PI = 28	24.8	101.2	2.0	1.5	11*	25	Becomes light brown
TXUU 2182 (2995)			1.5	1.3	13*	30	Becomes light gray with white streaks
LL= 44, PL = 22, PI = 22	19.3	109.4	3.0	1.1	19*	35	Light gray LEAN CLAY (CL), stiff, wet
						40	

* Blowcounts have been converted to approximate SPT-N values using a conversion factor of 0.8
** From field Torvane measurements
*** From field Pocket Penetrometer measurements

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Log of Boring B-1
Bishop Ranch City Center Project
Parcel 1&1A
San Ramon, California

BORING
B-3

DRAWN	JOB NUMBER	CHECKED	CHCK'D DATE	APPROVED	CHCK'D DATE
DB	4096-08-8527	RA	10/08		10/08

Date 8/19/08
Equipment Gregg Drilling
Drilling Method HSA
Sampler SPT/ S&H
Hammer Weight 140 lbs **Drop** 30
Logged by RL **Datum** MSL
Surface Elevation 439

Other Tests/Drilling Notes	Moisture Content (%)	Dry Density (pcf)	Shear Strength*** (ksf)	Shear Strength ** (ksf)	Blows per Foot	Depth (ft.)	Sampler Type	Graphic Log	
			2.0	0.9	20*	40			Light brown clayey SAND (SC), medium dense, wet
-200 = 48.2%,			2.0	0.7	21*	45			
MA, -200 = 85.7%			3.5	1.2	27*	50			Light brown LEAN CLAY (CL), stiff, wet
						55			
TXUU 4543 (4795)			3.25	1.1	28*	60			Light brown FAT CLAY (CH), very stiff, wet with greenish spots
						65			
									Dark brown GRAVEL with fines (GP), very dense, wet
			3.0	1.1	62*				Light brown FAT CLAY (CH), hard, wet
						70			Bottom of boring at 70 feet below ground surface. Groundwater encountered at 20 feet below ground level. Boring was grouted with cement grout after completion.

GEOTECH_BORING_NEW_MACTEC BISHOP RANCH.GPJ GEOTECH.GDT 10/8/08

* Blowcounts have been converted to approximate SPT-N values using a conversion factor of 0.8
** From field Torvane measurements
*** From field Pocket Penetrometer measurements

MACTEC

Log of Boring B-1
Bishop Ranch City Center Project
Parcel 1&1A
San Ramon, California

BORING
B-3

DRAWN	JOB NUMBER	CHECKED	CHCK'D DATE	APPROVED	CHCK'D DATE
DB	4096-08-8527	RH	10/08		10/08

Date 8/19/08
Equipment Gregg Drilling
Drilling Method Hand Auger
Sampler Grab
Hammer Weight N/A **Drop** N/A
Logged by RL **Datum** MSL
Surface Elevation 439

Other Tests/Drilling Notes	Moisture Content (%)	Dry Density (pcf)	Shear Strength*** (ksf)	Shear Strength ** (ksf)	Blows per Foot	Depth (ft.)	Sampler Type	Graphic Log	
						0		⁙	Dark gray coarse SAND with silt (SM) Moist (Fill?)
							☒	⁙	
							☒	▨	Dark gray FAT CLAY (CH), Moist
						5	☒	▨	
									Bottom of boring 5 foot below ground surface. No groundwater encountered Backfilled with soil and tamped down.

* Blowcounts have been converted to approximate SPT-N values using a conversion factor of 0.8
** From field Torvane measurements
*** From field Pocket Penetrometer measurements

MACTEC

Log of Boring B-2
Bishop Ranch City Center Project
Parcel 1&1A
San Ramon, California

BORING
B-4

DRAWN	JOB NUMBER	CHECKED	CHCK'D DATE	APPROVED	CHCK'D DATE
DB	4096-08-8527	RH	10/08		10/08

Page 1 of 1

GEOTECH_BORING_NEW_MACTEC BISHOP RANCH.GPJ GEOTECH.GDT 10/8/08

Date 8/19/08
Equipment Gregg Drilling
Drilling Method Hand Auger
Sampler Grab
Hammer Weight N/A **Drop** N/A
Logged by RL **Datum** MSL
Surface Elevation 438

Other Tests/Drilling Notes	Moisture Content (%)	Dry Density (pcf)	Shear Strength*** (ksf)	Shear Strength ** (ksf)	Blows per Foot	Depth (ft.)	Sampler Type	Graphic Log	
						0	☒	░	Dark gray coarse SAND with silt (SM) Moist (Fill?)
							☒	▨	Dark gray FAT CLAY (CH), Moist
						5			Bottom of boring 5 foot below ground surface. No groundwater encountered Backfilled with soil and tamped down.

GEOTECH_BORING_NEW_MACTEC BISHOP RANCH.GPJ GEOTECH.GDT 10/8/08

* Blowcounts have been converted to approximate SPT-N values using a conversion factor of 0.8
** From field Torvane measurements
*** From field Pocket Penetrometer measurements

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Log of Boring B-3
Bishop Ranch City Center Project
Parcel 1&1A
San Ramon, California

BORING
B-5

DRAWN	JOB NUMBER	CHECKED	CHCK'D DATE	APPROVED	CHCK'D DATE
DB	4096-08-8527	RH	10/08		10/08

Date 8/19/08
Equipment Gregg Drilling
Drilling Method Hand Auger
Sampler Grab
Hammer Weight N/A **Drop** N/A
Logged by RL **Datum** MSL
Surface Elevation 433

Other Tests/Drilling Notes	Moisture Content (%)	Dry Density (pcf)	Shear Strength*** (ksf)	Shear Strength ** (ksf)	Blows per Foot	Depth (ft.)	Sampler Type	Graphic Log	
						0	☒	▨	Dark gray coarse SAND with silt (SM) Moist (Fill?)
							☒	▨	Dark gray FAT CLAY (CH), Moist
						5			Bottom of boring 5 foot below ground surface. No groundwater encountered Backfilled with soil and tamped down.

* Blowcounts have been converted to approximate SPT-N values using a conversion factor of 0.8
** From field Torvane measurements
*** From field Pocket Penetrometer measurements

MACTEC

Log of Boring B-4

Bishop Ranch City Center Project
Parcel 1&1A
San Ramon, California

BORING
B-6

DRAWN	JOB NUMBER	CHECKED	CHCK'D DATE	APPROVED	CHCK'D DATE
DB	4096-08-8527	RA	10/08	DNA	10/08

Date 8/19/08
Equipment Gregg Drilling
Drilling Method Hand Auger
Sampler Grab
Hammer Weight N/A **Drop** N/A
Logged by RL **Datum** MSL
Surface Elevation 430

Other Tests/Drilling Notes	Moisture Content (%)	Dry Density (pcf)	Shear Strength*** (ksf)	Shear Strength ** (ksf)	Blows per Foot	Depth (ft.)	Sampler Type	Graphic Log	
						0			Asphalt Concrete 2"
								▒	Dark gray coarse SAND with silt (SM) Moist (Fill)
							☒ ☒	▨	Dark gray FAT CLAY (CH), Moist
						5	☒	▨	
									Bottom of boring 5 foot below ground surface. No groundwater encountered Backfilled with soil and tamped down.

GEOTECH_BORING_NEW_MACTEC BISHOP RANCH.GPJ GEOTECH.GDT 10/8/08

* Blowcounts have been converted to approximate SPT-N values using a conversion factor of 0.8
** From field Torvane measurements
*** From field Pocket Penetrometer measurements

MACTEC

Log of Boring B-5
Bishop Ranch City Center Project
Parcel 1&1A
San Ramon, California

BORING
B-7

DRAWN	JOB NUMBER	CHECKED	CHCK'D DATE	APPROVED	CHCK'D DATE
DB	4096-08-8527	RH	10/08		10/08

Date 8/19/08
Equipment Gregg Drilling
Drilling Method Hand Auger
Sampler Grab
Hammer Weight N/A **Drop** N/A
Logged by RL **Datum** MSL
Surface Elevation 429

Other Tests/Drilling Notes	Moisture Content (%)	Dry Density (pcf)	Shear Strength*** (ksf)	Shear Strength ** (ksf)	Blows per Foot	Depth (ft.)	Sampler Type	Graphic Log	
						0		■	Asphalt Concrete 2"
								░	Dark gray coarse SAND with silt (SM) Moist (Fill)
							☒	▨	Dark gray FAT CLAY (CH), Moist
							☒	▨	
						5			Bottom of boring 5 foot below ground surface. No groundwater encountered Backfilled with soil and tamped down.

* Blowcounts have been converted to approximate SPT-N values using a conversion factor of 0.8
** From field Torvane measurements
*** From field Pocket Penetrometer measurements

MACTEC

Log of Boring B-6
Bishop Ranch City Center Project
Parcel 1&1A
San Ramon, California

BORING
B-8

DRAWN	JOB NUMBER	CHECKED	CHCK'D DATE	APPROVED	CHCK'D DATE
DB	4096-08-8527	RA	10/08		10/08

Date 8/19/08
Equipment Gregg Drilling
Drilling Method Hand Auger
Sampler Grab
Hammer Weight N/A **Drop** N/A
Logged by RL **Datum** MSL
Surface Elevation 427

Other Tests/Drilling Notes	Moisture Content (%)	Dry Density (pcf)	Shear Strength*** (ksf)	Shear Strength ** (ksf)	Blows per Foot	Depth (ft.)	Sampler Type	Graphic Log	
						0		▨	Asphalt Concrete 2"
							☒	▨	Dark gray coarse SAND with silt (SM) Moist (Fill?)
								▨	Dark gray FAT CLAY (CH), Moist
						5	☒		Bottom of boring 5 foot below ground surface. No groundwater encountered Backfilled with soil and tamped down.

GEOTECH_BORING_NEW_MACTEC BISHOP RANCH.GPJ GEOTECH.GDT 10/8/08

* Blowcounts have been converted to approximate SPT-N values using a conversion factor of 0.8
** From field Torvane measurements
*** From field Pocket Penetrometer measurements

MACTEC

Log of Boring B-7
Bishop Ranch City Center Project
Parcel 1&1A
San Ramon, California

BORING **B-9**

DRAWN	JOB NUMBER	CHECKED	CHCK'D DATE	APPROVED	CHCK'D DATE
DB	4096-08-8527	DA	10/08	DCR	10/08

Date 8/18/08
Equipment Gregg Drilling
Drilling Method HSA
Sampler SPT/ S&H
Hammer Weight 140 lbs **Drop** 30
Logged by RL **Datum** MSL
Surface Elevation 428

Other Tests/Drilling Notes	Moisture Content (%)	Dry Density (pcf)	Shear Strength*** (ksf)	Shear Strength ** (ksf)	Blows per Foot	Depth (ft.)	Sampler Type	Graphic Log	
						0			Asphalt Concrete
									Dark gray coarse SAND with silt (SM) moist (Fill)
									Dark gray FATCLAY (CH), very stiff, moist
Corrosion			3.5	0.8	18*	5			Becomes light gray
No Sample Recovery					10*	10			
LL= 60, PL = 32, PI = 28, Consol	32.4	88.3	1	0.8	12*				▽
					13*	15			Light brown clayey SAND (SC), medium dense, wet
									Light gray FAT CLAY (CH), very stiff, moist with some brown spots
TXUU 1987 (1915)			2.5	0.8	16*	20			
LL=45, PL = 19, PI = 26	20.6	110.6	2.5	0.9	16*	25			Light gray LEAN CLAY (CL), very stiff, wet
TXUU 1512 (2578)			4.5	0.9	14*	30			
			2.25	0.7	14*	35			Light brown FAT CLAY (CH), stiff, wet
									Light brown clayey SAND (SC), medium dense, wet
						40			

* Blowcounts have been converted to approximate SPT-N values using a conversion factor of 0.8
** From field Torvane measurements
*** From field Pocket Penetrometer measurements

GEOTECH_BORING_NEW_MACTEC BISHOP RANCH.GPJ GEOTECH.GDT 10/8/08

MACTEC

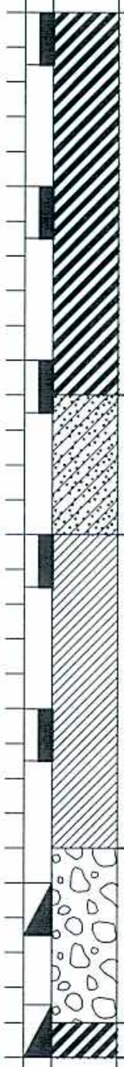
Log of Boring B-8
Bishop Ranch City Center Project
Parcel 1&1A
San Ramon, California

BORING
B-10

DRAWN	JOB NUMBER	CHECKED	CHCK'D DATE	APPROVED	CHCK'D DATE
DB	4096-08-8527	RA	10/08		

Date 8/18/08
Equipment Gregg Drilling
Drilling Method HSA
Sampler SPT/ S&H
Hammer Weight 140 lbs **Drop** 30
Logged by RL **Datum** MSL
Surface Elevation 428

Other Tests/Drilling Notes	Moisture Content (%)	Dry Density (pcf)	Shear Strength*** (ksf)	Shear Strength ** (ksf)	Blows per Foot	Depth (ft.)	
			3.5	0.8	18*	40	Light brown FAT CLAY (CH), very stiff to hard, wet with white streaks
TXUU 4025 (3485)			2.5	1.2	27*	45	
			4.5	0.8	36*	50	
							Light brown clayey SAND (SC), dense, wet with greenish spots
			4.0	0.8	30*	55	Light brown LEAN CLAY (CL), very Stiff, wet with greenish spots
LL=36, PL = 16, PI = 20	22.6	106.5			19*	60	
							Dark brown GRAVEL(GP) with fines, very dense, wet
MA, -200 = 3.4%					54*	65	
					16*		Light brown FAT CLAY (CH), very stiff, wet
						70	Bottom of boring at 70 feet below ground surface. Groundwater encountered at 13 feet below ground level. Boring was grouted with cement grout and topped off with asphalt after completion.



* Blowcounts have been converted to approximate SPT-N values using a conversion factor of 0.8
** From field Torvane measurements
*** From field Pocket Penetrometer measurements

MACTEC

Log of Boring B-8

Bishop Ranch City Center Project
Parcel 1&1A
San Ramon, California

BORING
B-10

DRAWN	JOB NUMBER	CHECKED	CHCK'D DATE	APPROVED	CHCK'D DATE
DB	4096-08-8527	RH	10/08		

Date 8/20/08
Equipment Gregg Drilling
Drilling Method HSA
Sampler SPT/ S&H
Hammer Weight 140 lbs **Drop** 30
Logged by RL **Datum** MSL
Surface Elevation 435

Other Tests/Drilling Notes	Moisture Content (%)	Dry Density (pcf)	Shear Strength*** (ksf)	Shear Strength ** (ksf)	Blows per Foot	Depth (ft.)	Sampler Type	Graphic Log	
						0			Asphalt Concrete
									Dark Gray coarse SAND with silt (SM), moist (Fill)
									Dark gray LEAN CLAY (CL), very stiff, moist with some green spots.
						5			
LL=45, PL = 22, PI = 23	18.4	82.7	3.0	13	15*				Becomes light Brown, stiff, moist
						10			
			2.0	6	15*				Light brown clayey SAND (SC), medium dense with some black spots.
									Light brown FAT CLAY (CH), Stiff, Wet, with black spots.
						15			
TXUU 1944 (1800)			1.0	10	10*				∇ with some gravel
						20			
LL=69, PL = 29, PI = 40	26.6	98	2.0	13	16*				Becomes dark brown with black ad green spots, very stiff.
						25			
TXUU 2030 (2405)			2.25	1.7	13*				Becomes dark gray with green spots, stiff.
						30			
LL=49, PL = 23, PI = 26	20.7	109.8	3.0	0.9	17*				Light brown LEAN CLAY(CL), very stiff, wet
						35			
TXUU 2052 (2995)			2.25	0.9	21*				
						40			

* Blowcounts have been converted to approximate SPT-N values using a conversion factor of 0.8
** From field Torvane measurements
*** From field Pocket Penetrometer measurements

MACTEC

Log of Boring B-9
Bishop Ranch City Center Project
Parcel 1&1A
San Ramon, California

BORING
B-11

DRAWN	JOB NUMBER	CHECKED	CHCK'D DATE	APPROVED	CHCK'D DATE
DB	4096-08-8527	RH	10/08		10/08

Date 8/20/08
Equipment Gregg Drilling
Drilling Method HSA
Sampler SPT/ S&H
Hammer Weight 140 lbs **Drop** 30
Logged by RL **Datum** MSL
Surface Elevation 435

Other Tests/Drilling Notes	Moisture Content (%)	Dry Density (pcf)	Shear Strength*** (ksf)	Shear Strength ** (ksf)	Blows per Foot	Depth (ft.)	Sampler Type	Graphic Log	
			2.25	0.7	14*	40	■	░	Light brown clayey SAND (SC), medium dense, wet
								▨	Light gray FAT CLAY (CH), stiff, moist
					50/3"	45	■	○	Dark brown GRAVEL with fines (GP), very dense, wet
-200 = 77.7%			3.0	1.6	20*	50	■	▨	Light brown FAT CLAY(CH) very stiff, wet with some green spots
						55		▨	
MA, -200 = 43.4%	18.3	114.9			19*	60	■	░	Light brown clayey SAND (SC), medium dense, wet
						65		░	
-200 = 75.7%					42*		■	▨	Light brown FAT CLAY (CH), very stiff, wet
						70			Bottom of boring at 70 feet below ground surface. Groundwater encountered at 15 feet below ground level. Boring was grouted with cement grout and topped off with asphalt after completion.

* Blowcounts have been converted to approximate SPT-N values using a conversion factor of 0.8
** From field Torvane measurements
*** From field Pocket Penetrometer measurements

MACTEC

Log of Boring B-9
Bishop Ranch City Center Project
Parcel 1&1A
San Ramon, California

BORING
B-11

DRAWN	JOB NUMBER	CHECKED	CHCK'D DATE	APPROVED	CHCK'D DATE
DB	4096-08-8527		10/08		

APPENDIX C

CPT INVESTIGATION RESULTS

Checked

Approved

GREGG DRILLING & TESTING, INC.
GEOTECHNICAL AND ENVIRONMENTAL INVESTIGATION SERVICES

August 22, 2008

Mactec
Attn: Rambod Hadidi
28 Second St., Suite 700
San Francisco, California 94105

Subject: CPT Site Investigation
Bishop Ranch
San Ramon, California
GREGG Project Number: 08-220MA

Dear Mr. Hadidi:

The following report presents the results of GREGG Drilling & Testing's Cone Penetration Test investigation for the above referenced site. The following testing services were performed:

1	Cone Penetration Tests	(CPTU)	☒
2	Pore Pressure Dissipation Tests	(PPD)	☒
3	Seismic Cone Penetration Tests	(SCPTU)	☐
4	Resistivity Cone Penetration Tests	(RCPTU)	☐
5	UVOST Laser Induced Fluorescence	(UVOST)	☐
6	Groundwater Sampling	(GWS)	☐
7	Soil Sampling	(SS)	☐
8	Vapor Sampling	(VS)	☐
9	Vane Shear Testing	(VST)	☐
10	SPT Energy Calibration	(SPTE)	☐

A list of reference papers providing additional background on the specific tests conducted is provided in the bibliography following the text of the report. If you would like a copy of any of these publications or should you have any questions or comments regarding the contents of this report, please do not hesitate to contact our office at (925) 313-5800.

Sincerely,
GREGG Drilling & Testing, Inc.

Mary Walden
Operations Manager

GREGG DRILLING & TESTING, INC.
GEOTECHNICAL AND ENVIRONMENTAL INVESTIGATION SERVICES

Cone Penetration Test Sounding Summary

-Table 1-

CPT Sounding Identification	Date	Termination Depth (Feet)	Depth of Groundwater Samples (Feet)	Depth of Soil Samples (Feet)	Depth of Pore Pressure Dissipation Tests (Feet)	App. Ground Surface Elev (Feet)
CPT-01	8/19/08	70	-	-	-	439
CPT-02	8/19/08	70	-	-	-	439
CPT-03	8/19/08	70	-	-	-	438
CPT-04	8/19/08	70	-	-	-	434
CPT-05	8/18/08	70	-	-	-	430
CPT-06	8/18/08	70	-	-	66.1	430
CPT-07	8/18/08	64	-	-	-	429

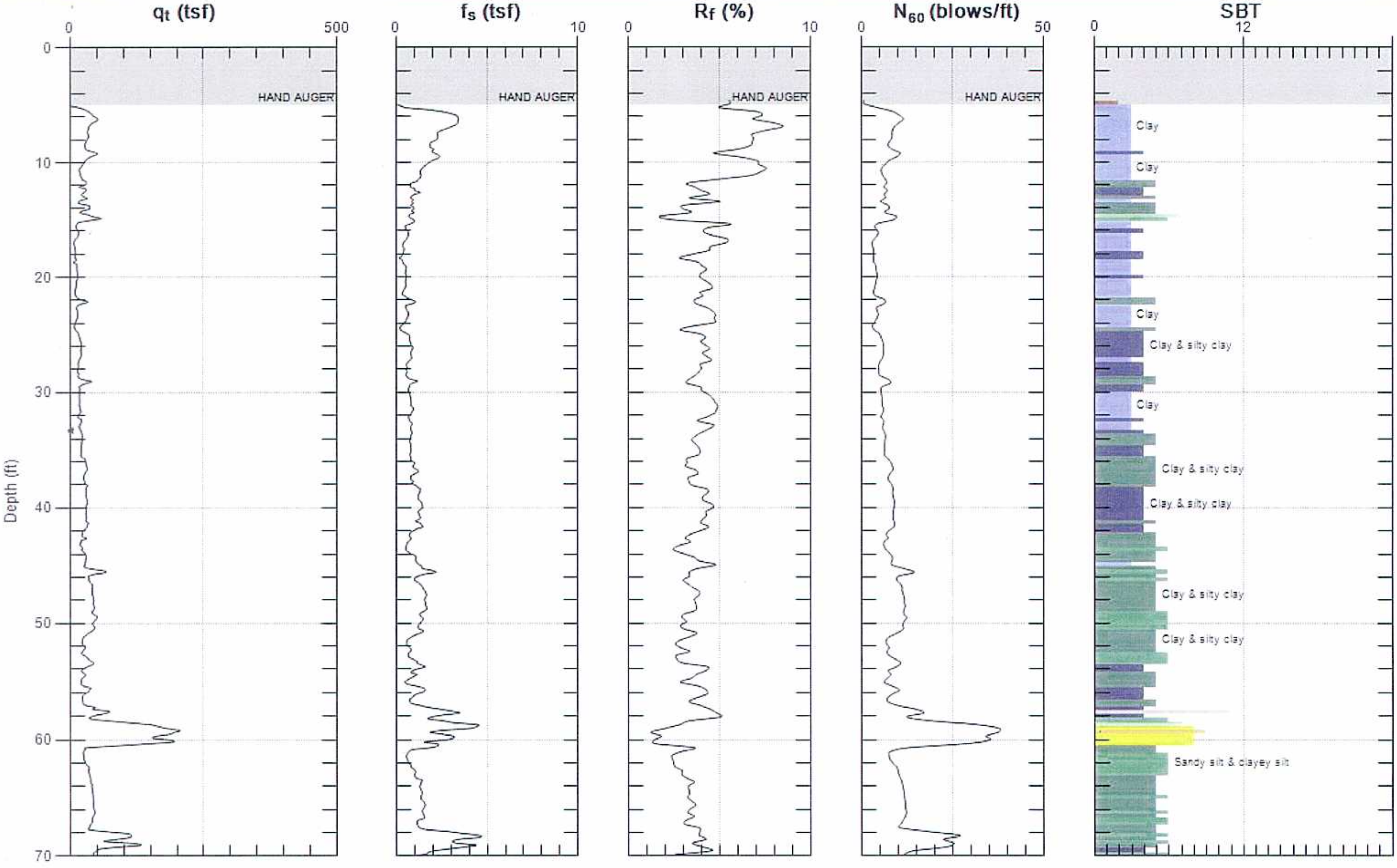
GREGG **MACTEC**

Site: BISHOP RANCH

Sounding: CPT-1

Engineer: R.HADIDI

Date: 8/19/2008 11:49



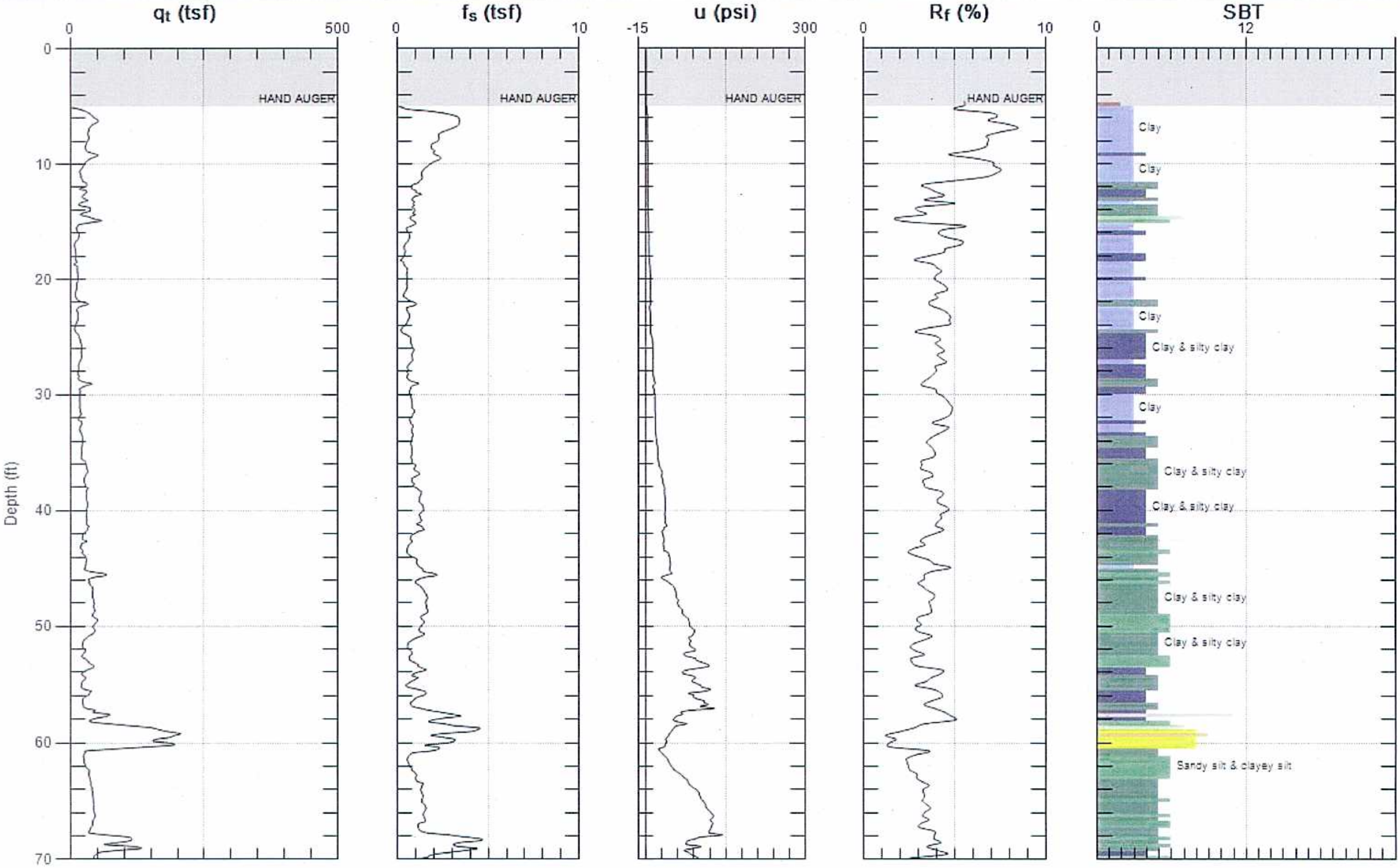
Max. Depth: 70.210 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **MACTEC**

Site: BISHOP RANCH
Sounding: CPT-1

Engineer: R.HADIDI
Date: 8/19/2008 11:49



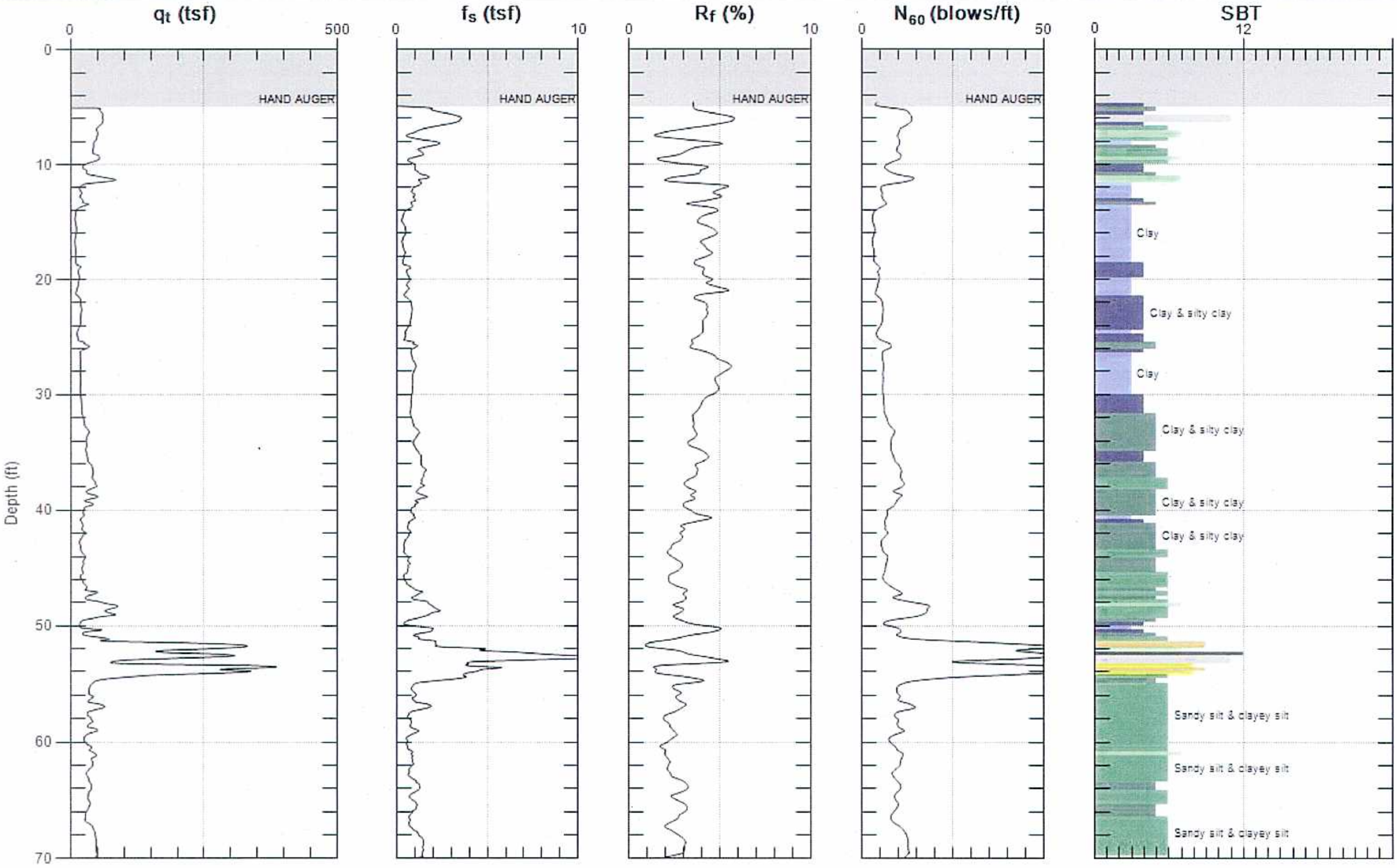
Max. Depth: 70.210 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **MACTEC**

Site: BISHOP RANCH
Sounding: CPT-2

Engineer: R.HADIDI
Date: 8/19/2008 09:25



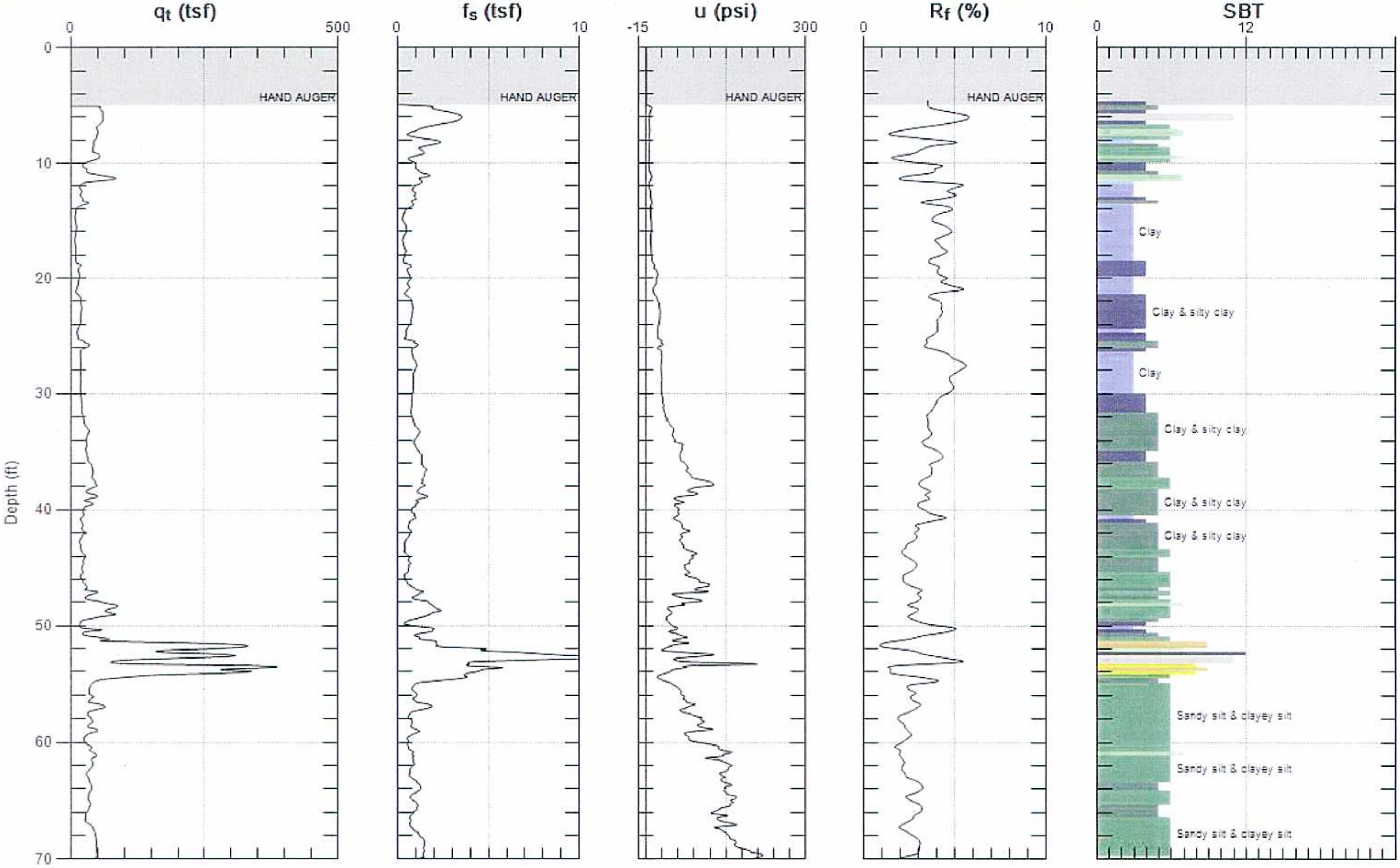
Max. Depth: 70.210 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **MACTEC**

Site: BISHOP RANCH
Sounding: CPT-2

Engineer: R.HADIDI
Date: 8/19/2008 09:25



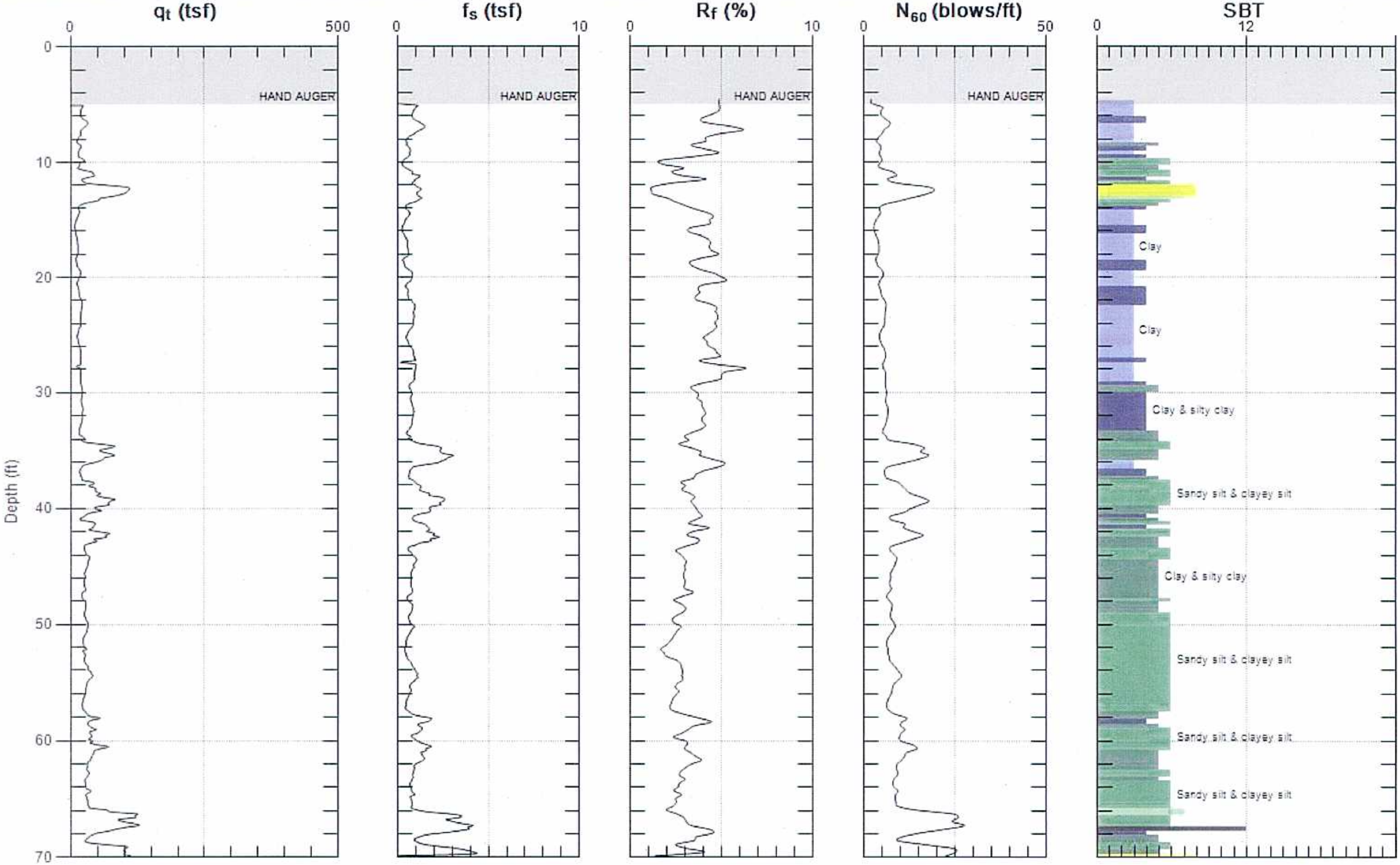
Max. Depth: 70.210 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG MACTEC

Site: BISHOP RANCH
Sounding: CPT-3

Engineer: R.HADIDI
Date: 8/19/2008 10:31



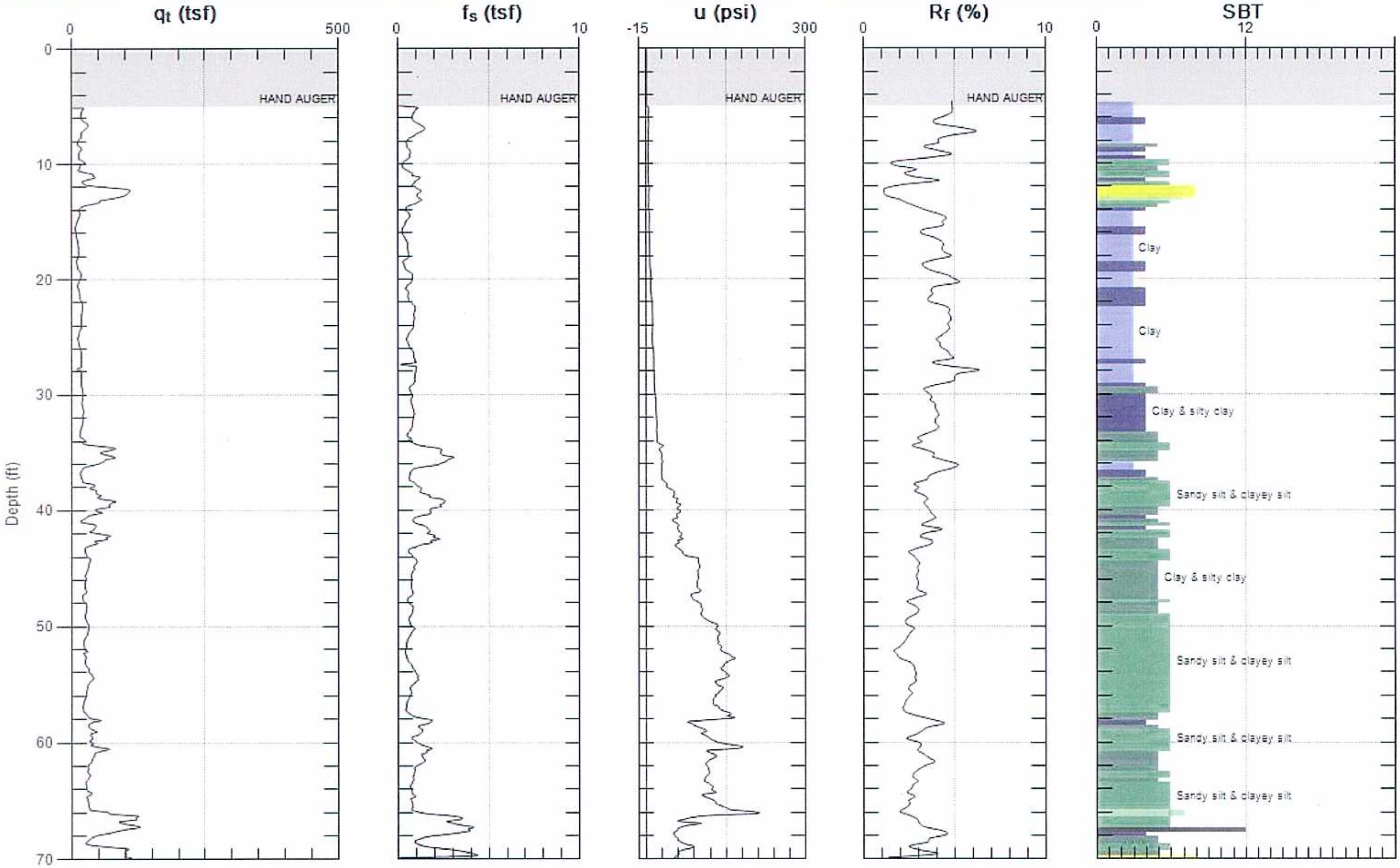
Max. Depth: 70.046 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG MACTEC

Site: BISHOP RANCH
Sounding: CPT-3

Engineer: R.HADIDI
Date: 8/19/2008 10:31



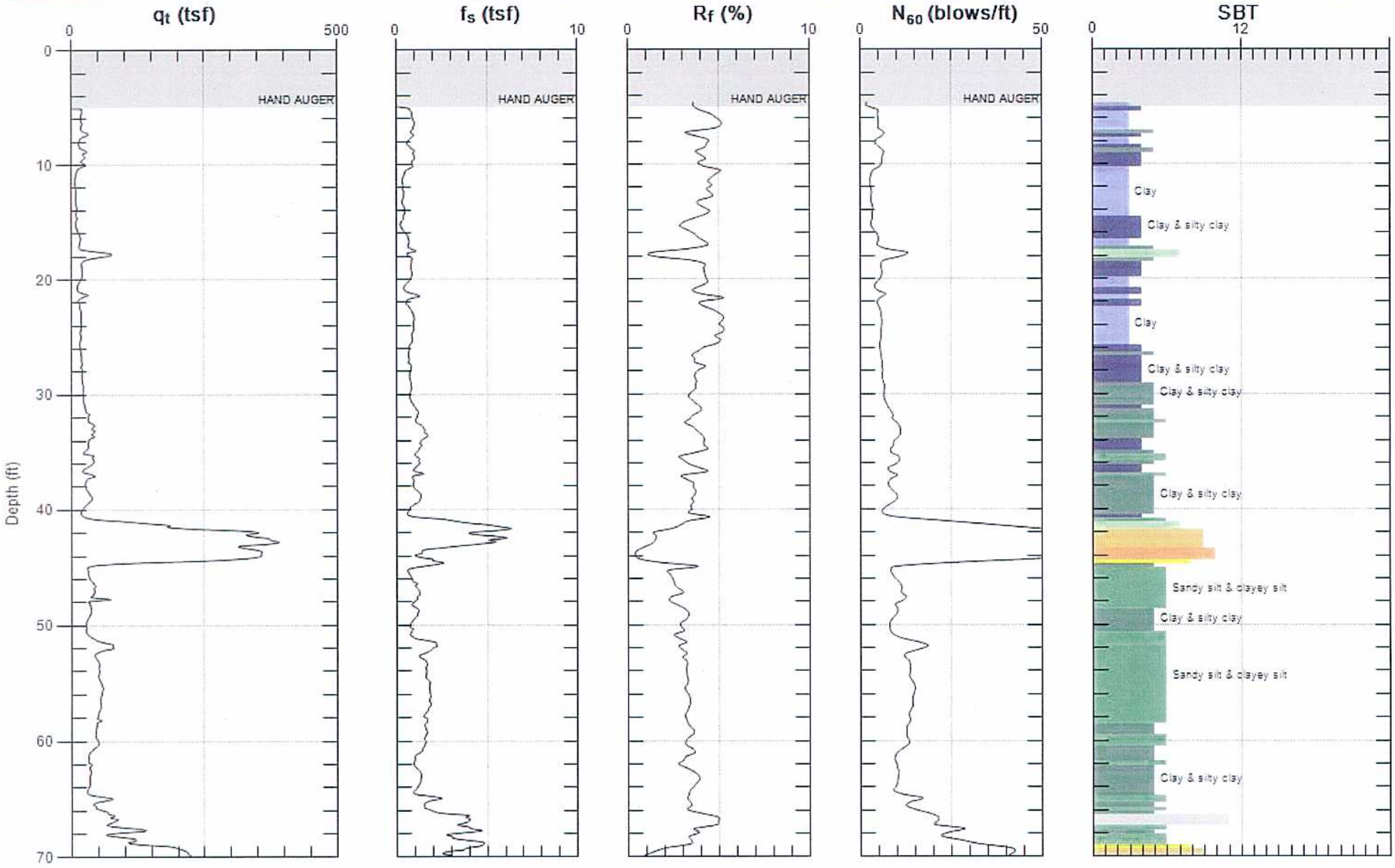
Max. Depth: 70.046 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **MACTEC**

Site: BISHOP RANCH
Sounding: CPT-4

Engineer: R.HADIDI
Date: 8/19/2008 08:05



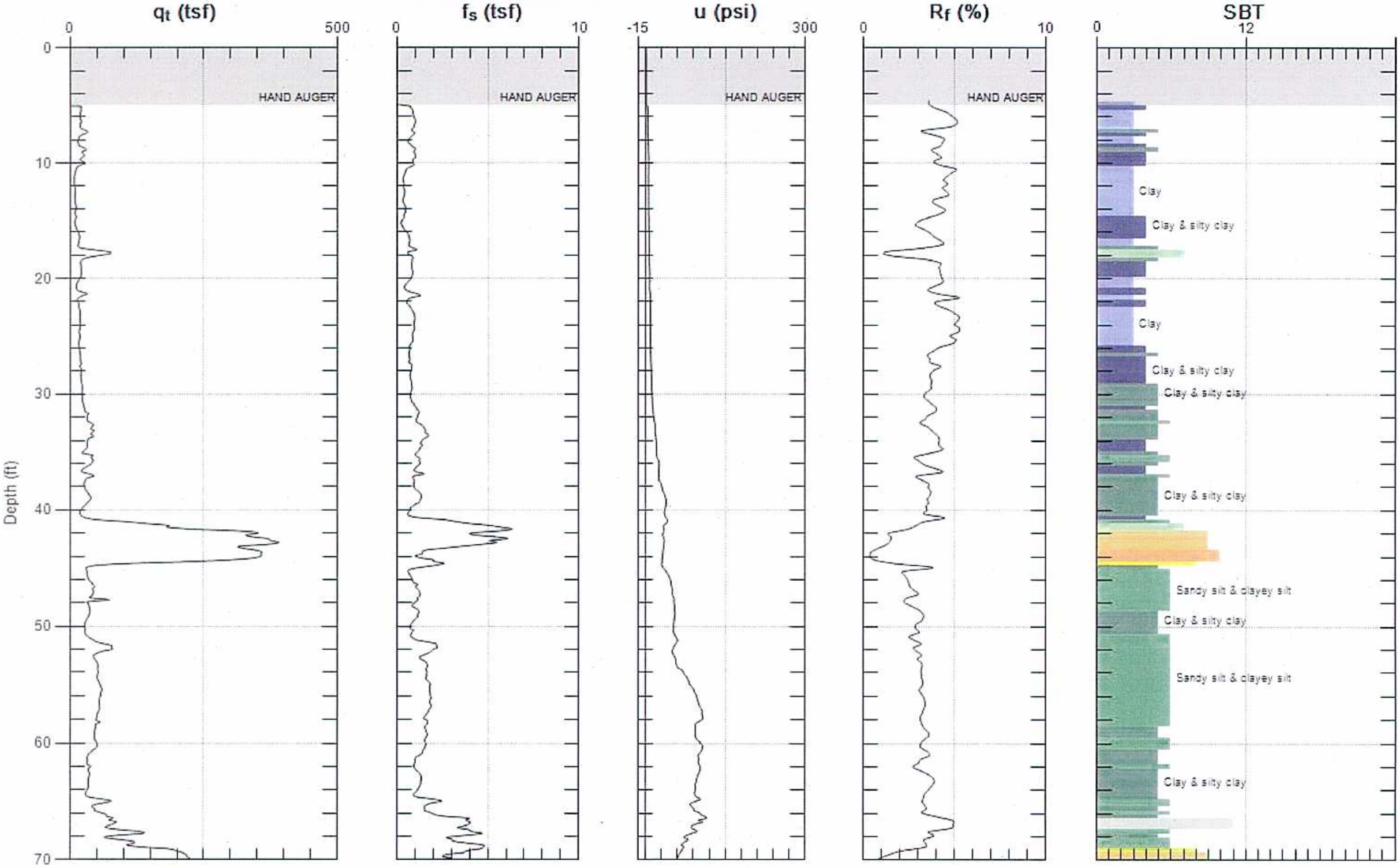
Max. Depth: 70.210 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **MACTEC**

Site: BISHOP RANCH
Sounding: CPT-4

Engineer: R.HADIDI
Date: 8/19/2008 08:05



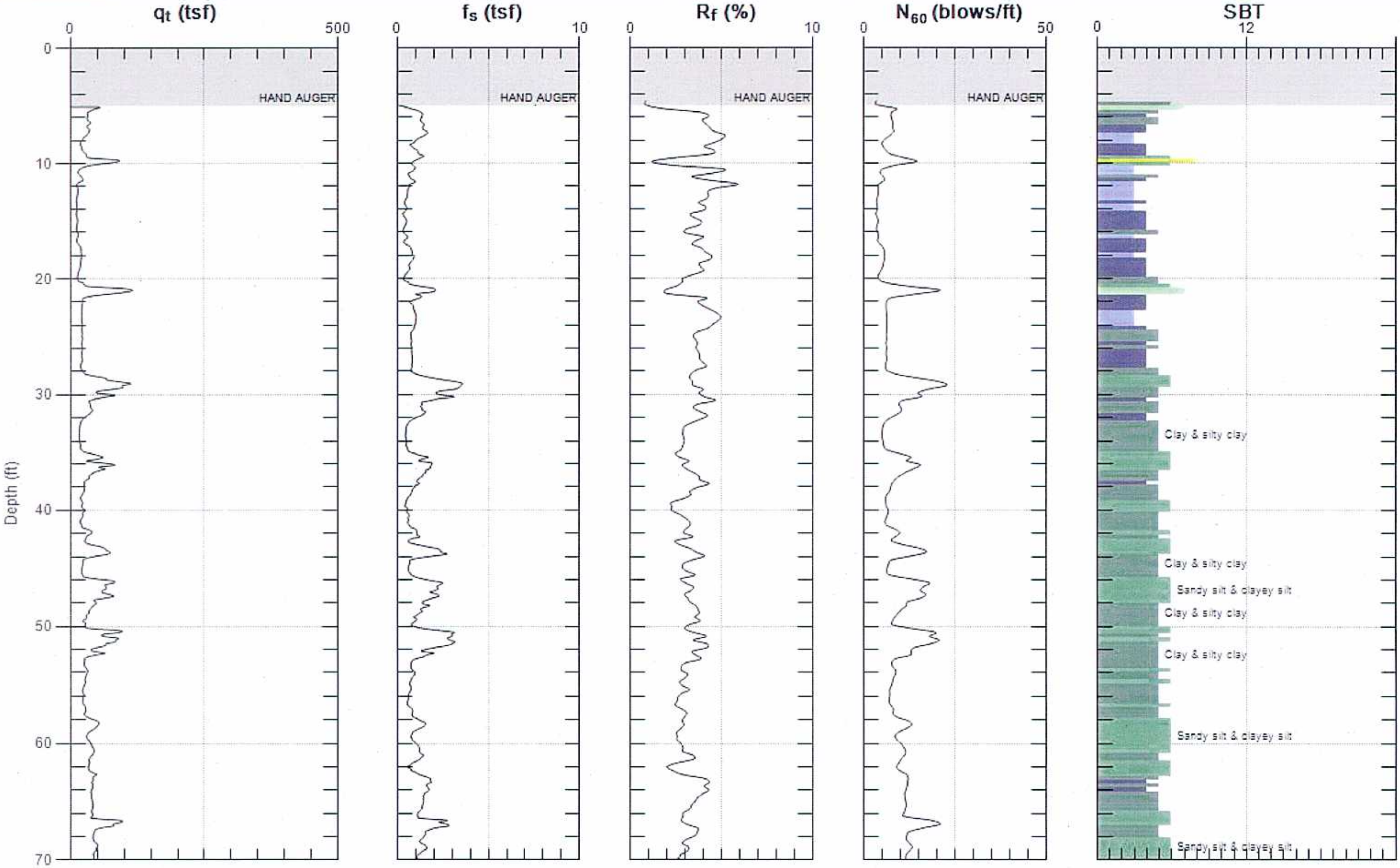
Max. Depth: 70.210 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG MACTEC

Site: BISHOP RANCH
Sounding: CPT-5

Engineer: R.HADIDI
Date: 8/18/2008 01:00



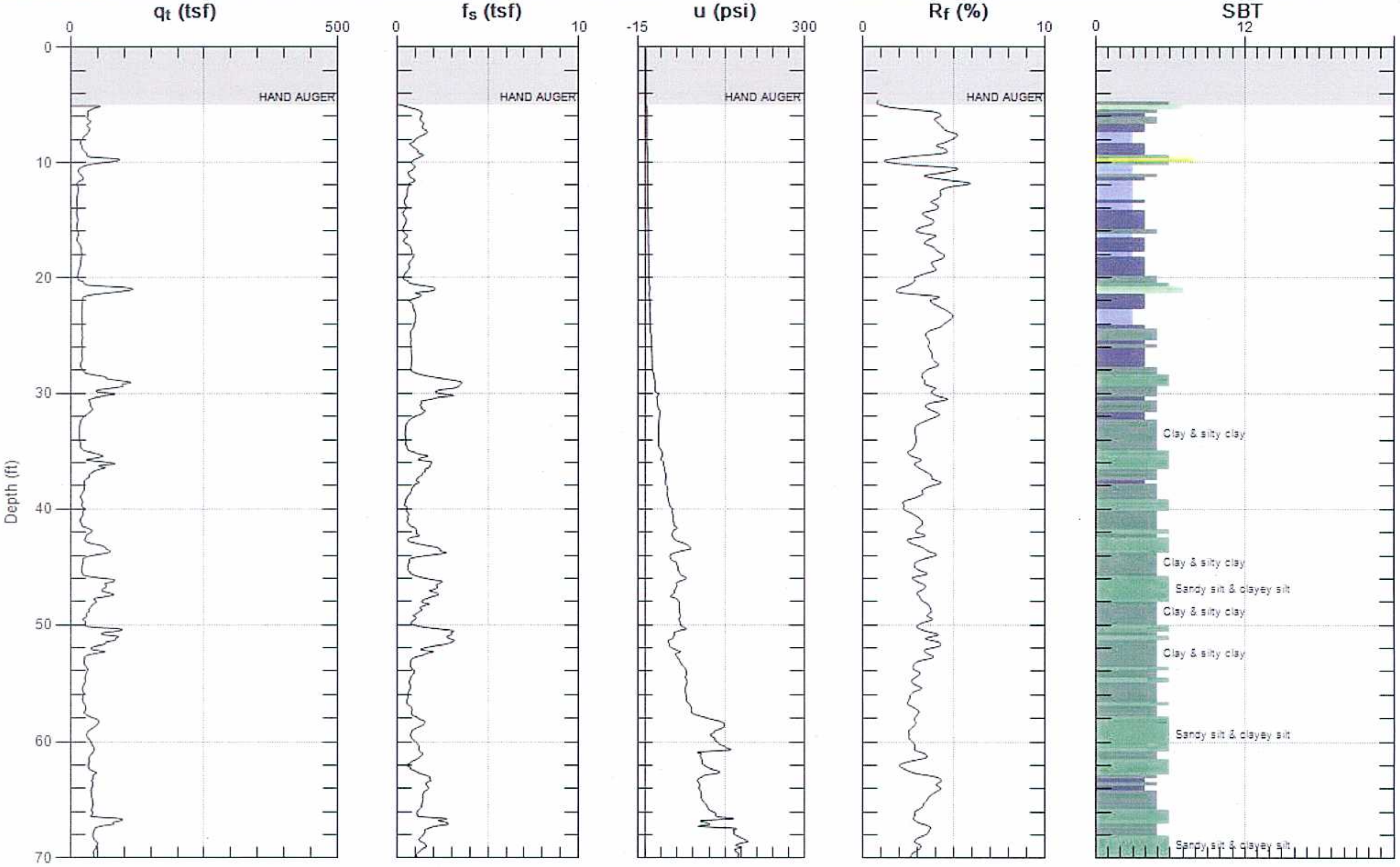
Max. Depth: 70.538 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG MACTEC

Site: BISHOP RANCH
Sounding: CPT-5

Engineer: R.HADIDI
Date: 8/18/2008 01:00



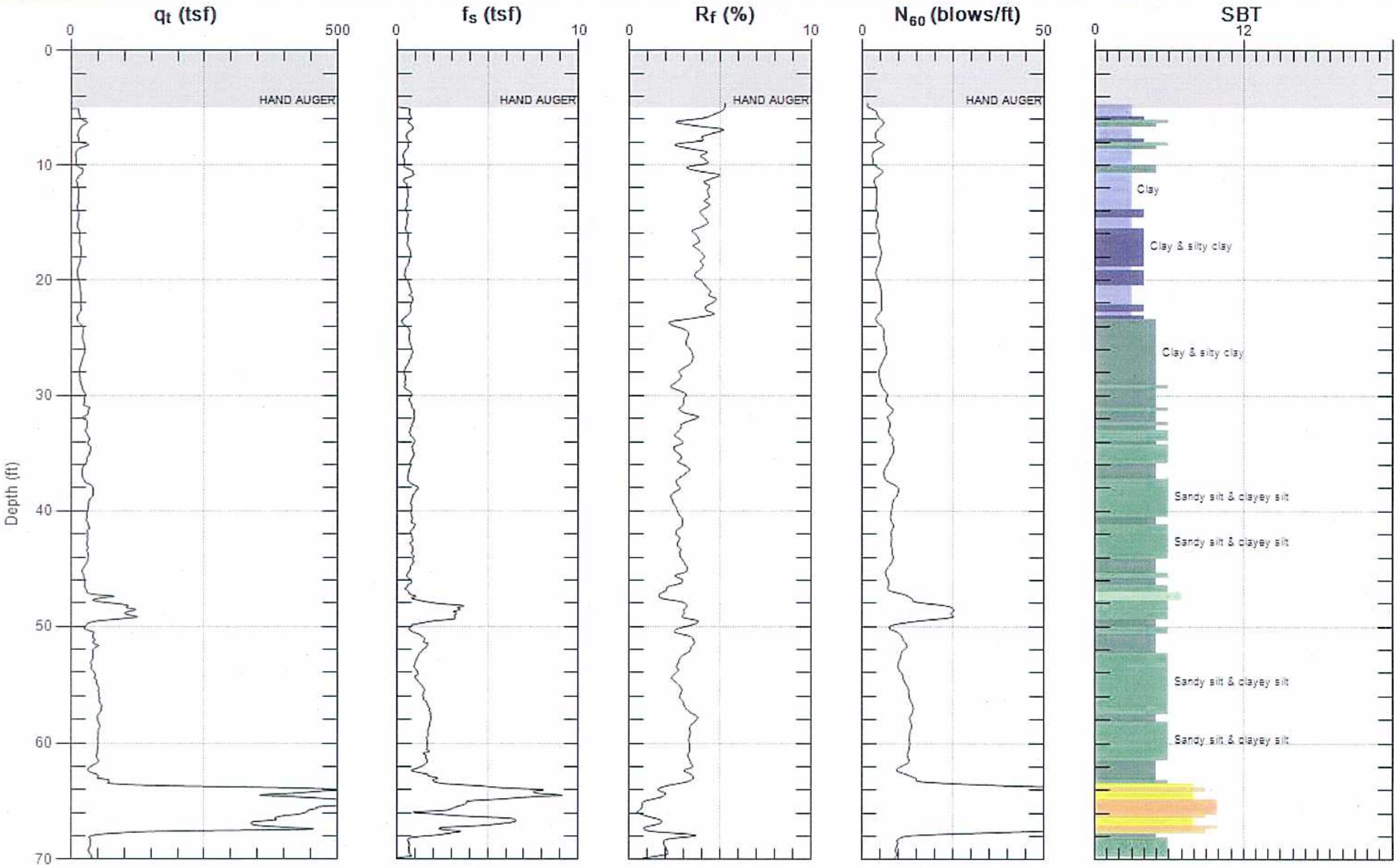
Max. Depth: 70.538 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **MACTEC**

Site: BISHOP RANCH
Sounding: CPT-6

Engineer: R.HADIDI
Date: 8/18/2008 10:28



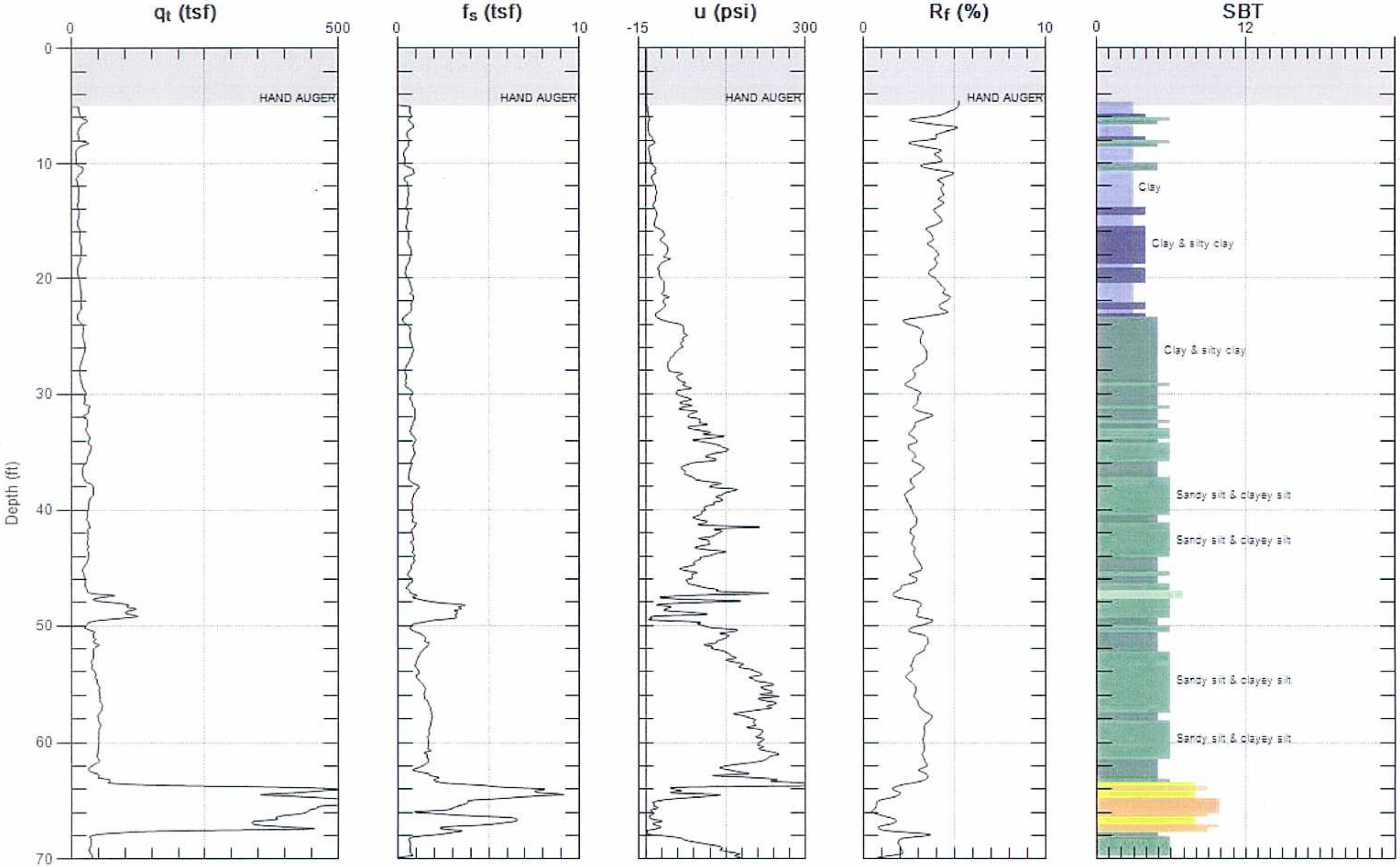
Max. Depth: 70.046 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

MACTEC

Site: BISHOP RANCH
Sounding: CPT-6

Engineer: R.HADIDI
Date: 8/18/2008 10:28

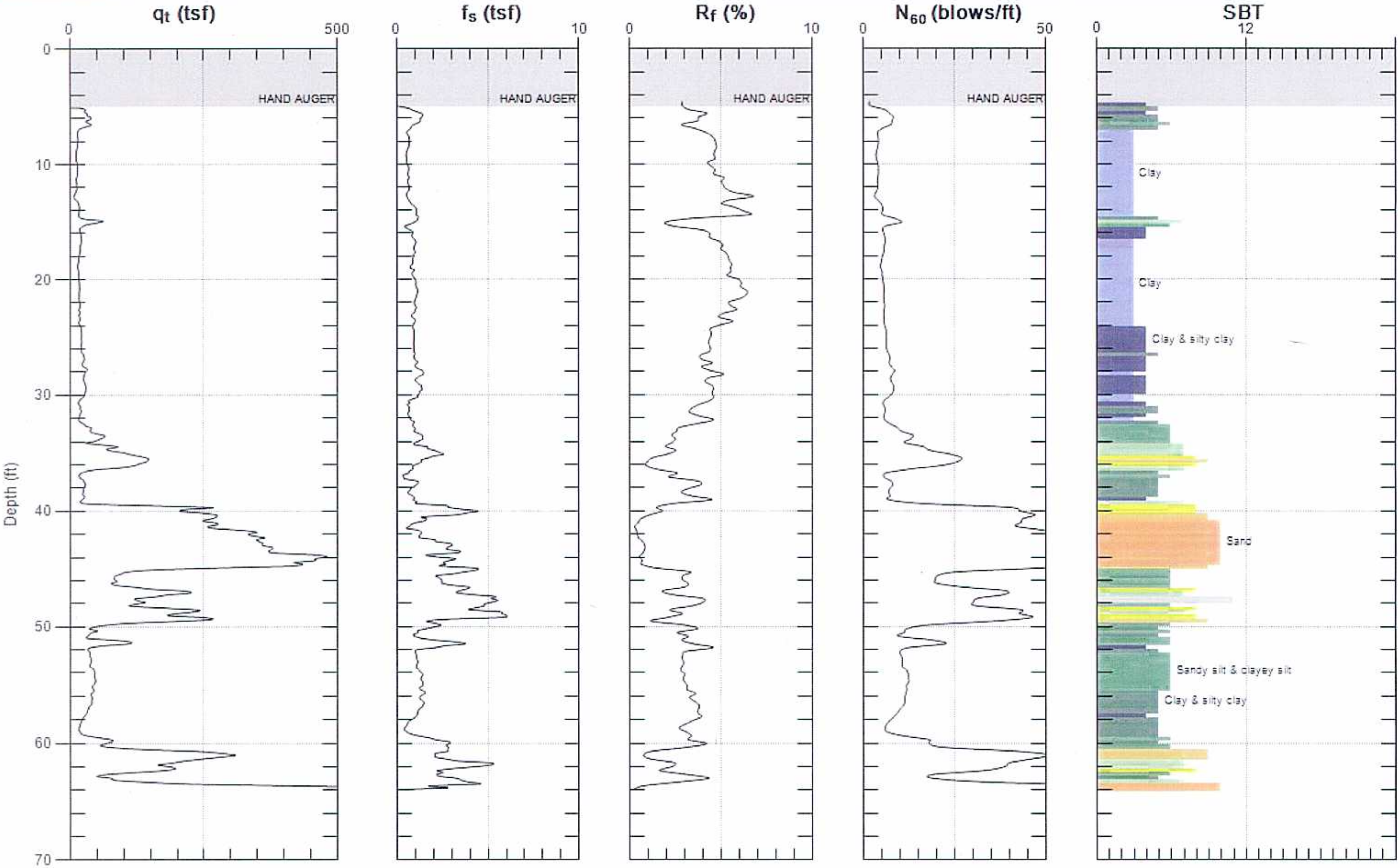


Max. Depth: 70.046 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

Site: BISHOP RANCH
Sounding: CPT-7

Engineer: R.HADIDI
Date: 8/18/2008 08:47



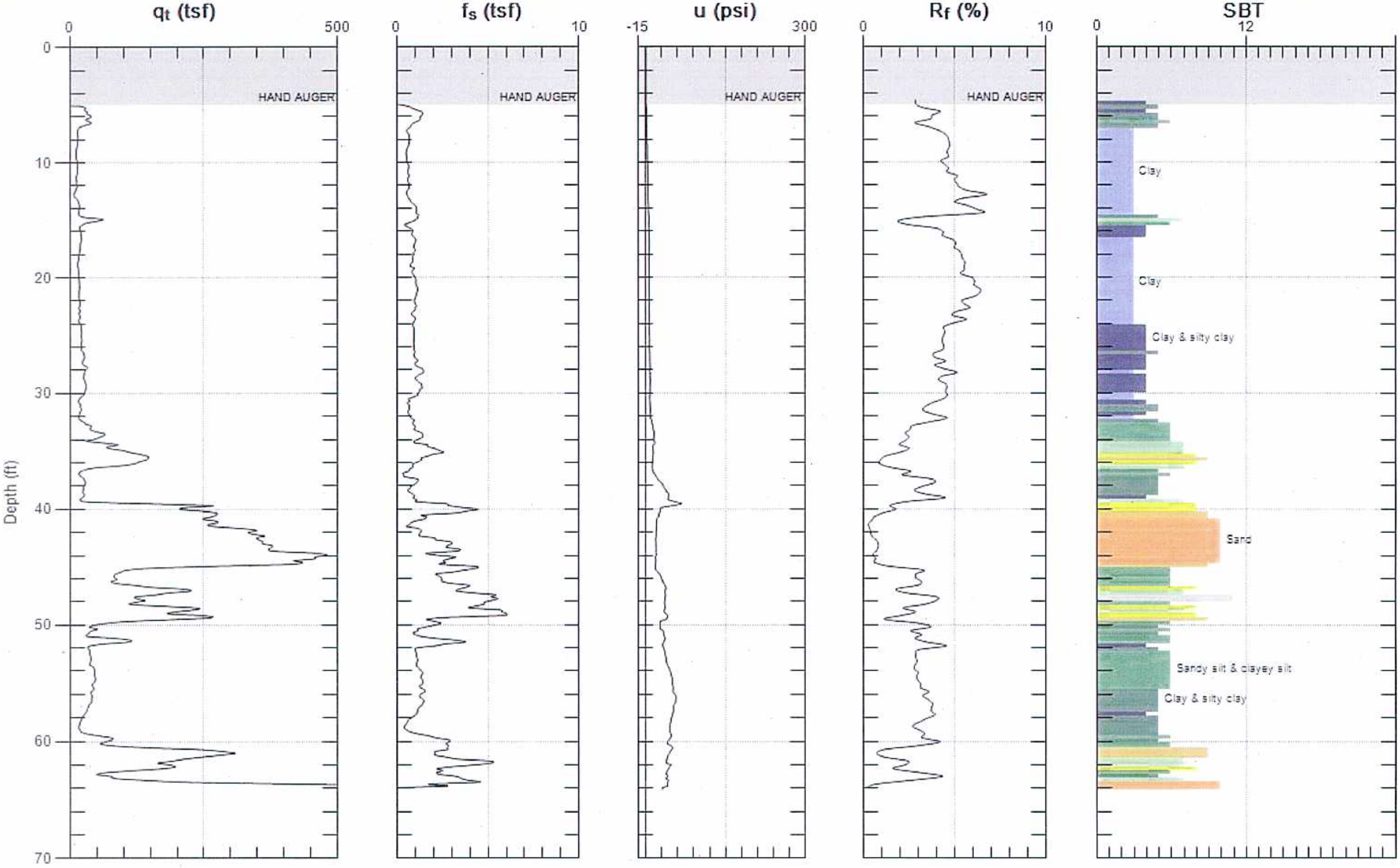
Max. Depth: 64.140 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **MACTEC**

Site: BISHOP RANCH
Sounding: CPT-7

Engineer: R.HADIDI
Date: 8/18/2008 08:47



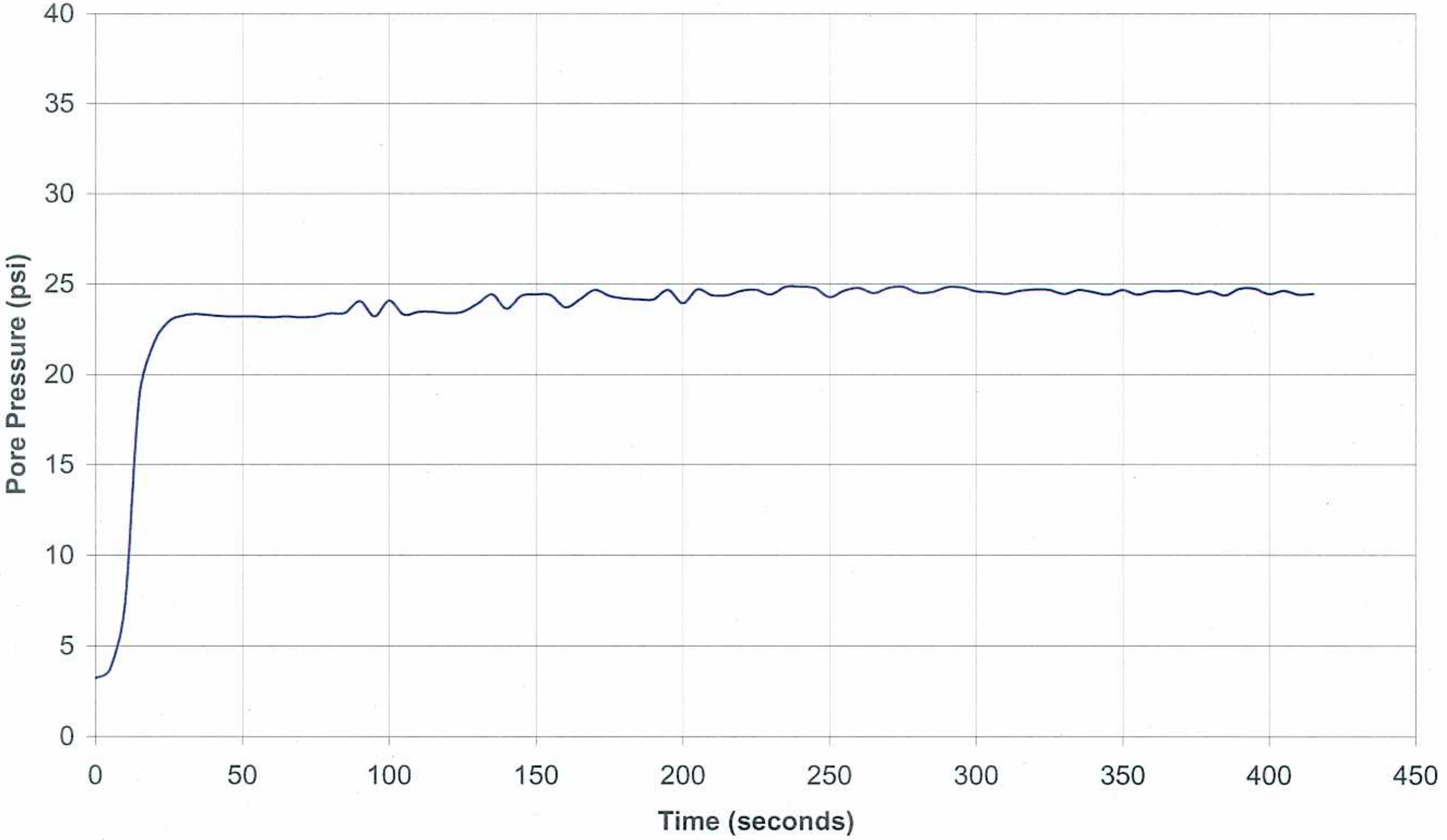
Max. Depth: 64.140 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

Sounding: CPT-6
Depth: 66.109
Site: BISHOP RANCH
Engineer: R.HADIDI



APPENDIX PPDT

Cone Penetration Testing Procedure (CPT)

Gregg Drilling carries out all Cone Penetration Tests (CPT) using an integrated electronic cone system, *Figure CPT*. The soundings were conducted using a 20 ton capacity cone with a tip area of 10 cm^2 and a friction sleeve area of 150 cm^2. The cone is designed with an equal end area friction sleeve and a tip end area ratio of 0.80.

The cone takes measurements of cone bearing (q_c), sleeve friction (f_s) and penetration pore water pressure (u_2) at 5-cm intervals during penetration to provide a nearly continuous hydrogeologic log. CPT data reduction and interpretation is performed in real time facilitating on-site decision making. The above mentioned parameters are stored on disk for further analysis and reference. All CPT soundings are performed in accordance with revised (2002) ASTM standards (D 5778-95).

The cone also contains a porous filter element located directly behind the cone tip (u_2), *Figure CPT*. It consists of porous plastic and is 5.0mm thick. The filter element is used to obtain penetration pore pressure as the cone is advanced as well as Pore Pressure Dissipation Tests (PPDT's) during appropriate pauses in penetration. It should be noted that prior to penetration, the element is fully saturated with silicon oil under vacuum pressure to ensure accurate and fast dissipation.

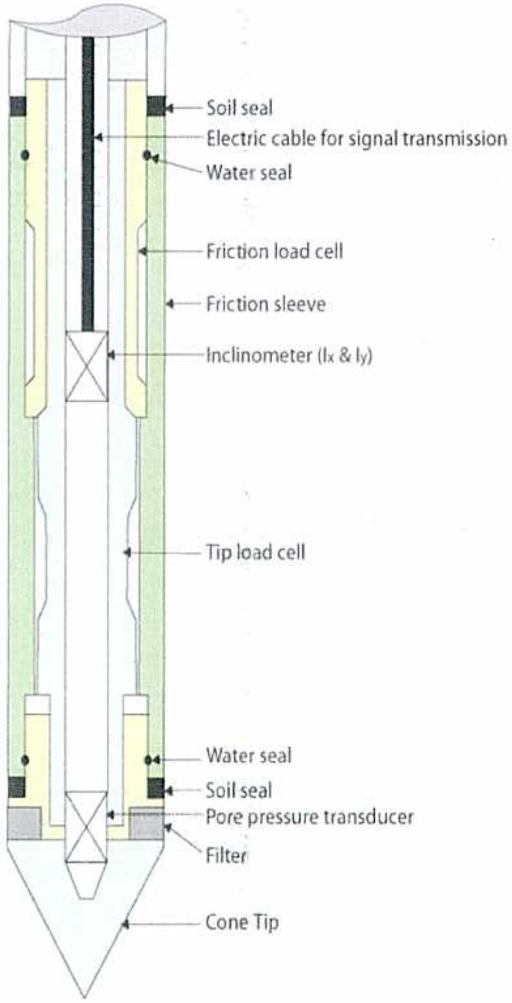


Figure CPT

When the soundings are complete, the test holes are grouted using a Gregg In Situ support rig. The grouting procedures generally consist of pushing a hollow CPT rod with a "knock out" plug to the termination depth of the test hole. Grout is then pumped under pressure as the tremie pipe is pulled from the hole. Disruption or further contamination to the site is therefore minimized.

Cone Penetration Test (CPT) Interpretation

Gregg have recently updated their CPT interpretation and plotting software (2007). The software takes the CPT data and performs basic interpretation in terms of soil behavior type (SBT) and various geotechnical parameters using current published empirical correlations based on the comprehensive review by Lunne, Robertson and Powell (1997). The interpretation is presented in tabular format using MS Excel. The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg does not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and does not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software.

The following provides a summary of the methods used for the interpretation. Many of the empirical correlations to estimate geotechnical parameters have constants that have a range of values depending on soil type, geologic origin and other factors. The software uses 'default' values that have been selected to provide, in general, conservatively low estimates of the various geotechnical parameters.

Input:

1 Units for display (Imperial or metric) (atm. pressure, pa = 0.96 tsf or 0.1 MPa)
2 Depth interval to average results,(ft or m). Data are collected at either 0.02 or 0.05m and can be averaged every 1, 3 or 5 intervals.
3 Elevation of ground surface (ft or m)
4 Depth to water table, z_w (ft or m) – input required
5 Net area ratio for cone, a (default to 0.80)
6 Relative Density constant, C_{Dr} (default to 350)
7 Young's modulus number for sands, α (default to 5)
8 Small strain shear modulus number
    a. for sands, S_G (default to 180 for SBT_n 5, 6, 7)
    b. for clays, C_G (default to 50 for SBT_n 1, 2, 3 & 4)
9 Undrained shear strength cone factor for clays, N_{kt} (default to 15)
10 Over Consolidation ratio number, k_{ocr} (default to 0.3)
11 Unit weight of water, (default to $\gamma_w = 62.4$ lb/ft^3 or 9.81 kN/m^3)

Column

1 Depth, z, (m) – CPT data is collected in meters
2 Depth (ft)
3 Cone resistance, q_c (tsf or MPa)
4 Sleeve friction, f_s (tsf or MPa)
5 Penetration pore pressure, u (psi or MPa), measured behind the cone (i.e. u_2)
6 Other – any additional data, if collected, e.g. electrical resistivity or UVIF
7 Total cone resistance, q_t (tsf or MPa) $q_t = q_c + u\,(1\text{-}a)$

8	Friction Ratio, R_f (%)	$R_f = (f_s/q_t) \times 100\%$
9	Soil Behavior Type (non-normalized), SBT	see note
10	Unit weight, γ (pcf or kN/m^3)	based on SBT, see note
11	Total overburden stress, σ_v (tsf)	$\sigma_{vo} = \gamma\ z$
12	Insitu pore pressure, u_o (tsf)	$u_o = \gamma_w\ (z - z_w)$
13	Effective overburden stress, σ'_{vo} (tsf)	$\sigma'_{vo} = \sigma_{vo} - u_o$
14	Normalized cone resistance, Q_{t1}	$Q_{t1} = (q_t - \sigma_{vo}) / \sigma'_{vo}$
15	Normalized friction ratio, F_r (%)	$F_r = f_s / (q_t - \sigma_{vo}) \times 100\%$
16	Normalized Pore Pressure ratio, B_q	$B_q = u - u_o / (q_t - \sigma_{vo})$
17	Soil Behavior Type (normalized), SBT_n	see note
18	SBT_n Index, I_c	see note
19	Normalized Cone resistance, Q_{tn} (n varies with Ic)	see note
20	Estimated permeability, k_{SBT} (cm/sec or ft/sec)	see note
21	Equivalent SPT N_{60}, blows/ft	see note
22	Equivalent SPT $(N_1)_{60}$ blows/ft	see note
23	Estimated Relative Density, D_r, (%)	see note
24	Estimated Friction Angle, ϕ', (degrees)	see note
25	Estimated Young's modulus, E_s (tsf)	see note
26	Estimated small strain Shear modulus, Go (tsf)	see note
27	Estimated Undrained shear strength, s_u (tsf)	see note
28	Estimated Undrained strength ratio	s_u/σ_v'
29	Estimated Over Consolidation ratio, OCR	see note

Notes:

1 Soil Behavior Type (non-normalized), SBT Lunne et al. (1997) listed below

2 Unit weight, γ either constant at 119 pcf or based on Non-normalized SBT (Lunne et al., 1997 and table below)

3 Soil Behavior Type (Normalized), SBT_n Lunne et al. (1997)

4 SBT_n Index, I_c $I_c = ((3.47 - \log Q_{t1})^2 + (\log F_r + 1.22)^2)^{0.5}$

5 Normalized Cone resistance, Q_{tn} (n varies with Ic)

$Q_{tn} = ((q_t - \sigma_{vo})/pa)\ (pa/(\sigma'_{vo})^n$ and recalculate I_c, then iterate:

When $I_c < 1.64$, $n = 0.5$ (clean sand)
When $I_c > 3.30$, $n = 1.0$ (clays)
When $1.64 < I_c < 3.30$, $n = (I_c - 1.64)0.3 + 0.5$
Iterate until the change in n, $\Delta n < 0.01$

6 Estimated permeability, k_{SBT} (based on Normalized SBT_n) (Lunne et al., 1997 and table below)

7 Equivalent SPT N_{60}, blows/ft Lunne et al. (1997)

$$\frac{(q_t/p_a)}{N_{60}} = 8.5\left(1-\frac{I_c}{4.6}\right)$$

8 Equivalent SPT $(N_1)_{60}$ blows/ft $(N_1)_{60} = N_{60}\,C_N$,
where $C_N = (pa/\sigma'_{vo})^{0.5}$

9 Relative Density, D_r, (%) $D_r^2 = Q_{tn} / C_{Dr}$
Only SBT$_n$ 5, 6, 7 & 8 *Show 'N/A' in zones 1, 2, 3, 4 & 9*

10 Friction Angle, ϕ', (degrees) $\tan\phi' = \frac{1}{2.68}\left[\log\left(\frac{q_c}{\sigma'_{vo}}\right) + 0.29\right]$
Only SBT$_n$ 5, 6, 7 & 8 *Show 'N/A' in zones 1, 2, 3, 4 & 9*

11 Young's modulus, E_s $E_s = \alpha\, q_t$
Only SBT$_n$ 5, 6, 7 & 8 *Show 'N/A' in zones 1, 2, 3, 4 & 9*

12 Small strain shear modulus, Go
a. $G_o = S_G\,(q_t\ \sigma'_{vo}\,pa)^{1/3}$ *For SBT$_n$ 5, 6, 7*
b. $G_o = C_G\,q_t$ *For SBT$_n$ 1, 2, 3& 4*
Show 'N/A' in zones 8 & 9

13 Undrained shear strength, s_u $s_u = (q_t - \sigma_{vo}) / N_{kt}$
Only SBT$_n$ 1, 2, 3, 4 & 9 *Show 'N/A' in zones 5, 6, 7 & 8*

14 Over Consolidation ratio, OCR $OCR = k_{ocr}\,Q_{t1}$
Only SBT$_n$ 1, 2, 3, 4 & 9 *Show 'N/A' in zones 5, 6, 7 & 8*

SBT Zones **SBT$_n$ Zones**

The following updated and simplified SBT descriptions have been used in the software:

1	sensitive fine grained	1	sensitive fine grained
2	organic soil	2	organic soil
3	clay	3	clay
4	clay & silty clay	4	clay & silty clay
5	clay & silty clay		
6	sandy silt & clayey silt		
7	silty sand & sandy silt	5	silty sand & sandy silt
8	sand & silty sand	6	sand & silty sand
9	sand		
10	sand	7	sand
11	very dense/stiff soil*	8	very dense/stiff soil*
12	very dense/stiff soil*	9	very dense/stiff soil*

* heavily overconsolidated and/or cemented

Track when soils fall with zones of same description and print that description (i.e. if soils fall only within SBT zones 4 & 5, print 'clays & silty clays')

Estimated Permeability (see Lunne et al., 1997)

SBT_n	Permeability (ft/sec)	(m/sec)
1	$3x\ 10^{-8}$	$1x\ 10^{-8}$
2	$3x\ 10^{-7}$	$1x\ 10^{-7}$
3	$1x\ 10^{-9}$	$3x\ 10^{-10}$
4	$3x\ 10^{-8}$	$1x\ 10^{-8}$
5	$3x\ 10^{-6}$	$1x\ 10^{-6}$
6	$3x\ 10^{-4}$	$1x\ 10^{-4}$
7	$3x\ 10^{-2}$	$1x\ 10^{-2}$
8	$3x\ 10^{-6}$	$1x\ 10^{-6}$
9	$1x\ 10^{-8}$	$3x\ 10^{-9}$

Estimated Unit Weight (see Lunne et al., 1997)

SBT	Approximate Unit Weight (lb/ft^3)	(kN/m^3)
1	111.4	17.5
2	79.6	12.5
3	111.4	17.5
4	114.6	18.0
5	114.6	18.0
6	114.6	18.0
7	117.8	18.5
8	120.9	19.0
9	124.1	19.5
10	127.3	20.0
11	130.5	20.5
12	120.9	19.0

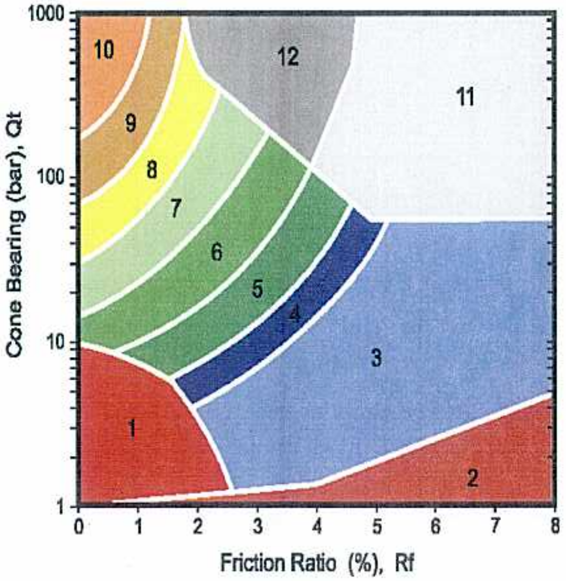
Cone Penetration Test Data & Interpretation

The Cone Penetration Test (CPT) data collected from your site are presented in graphical form in the attached report. The plots include interpreted Soil Behavior Type (SBT) based on the charts described by Robertson (1990). Typical plots display SBT based on the non-normalized charts of Robertson et al (1986). For CPT soundings extending greater than 50 feet, we recommend the use of the normalized charts of Robertson (1990) which can be displayed as SBTn, upon request. The report also includes spreadsheet output of computer calculations of basic interpretation in terms of SBT and SBTn and various geotechnical parameters using current published correlations based on the comprehensive review by Lunne, Robertson and Powell (1997), as well as recent updates by Professor Robertson. The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg InSitu and Gregg Drilling & Testing Inc. do not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and do not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software.

Some interpretation methods require input of the groundwater level to calculate vertical effective stress. An estimate of the in-situ groundwater level has been made based on field observations and/or CPT results, but should be verified by the user.

A summary of locations and depths is available in Table 1. Note that all penetration depths referenced in the data are with respect to the existing ground surface.

Note that it is not always possible to clearly identify a soil type based solely on q_t, f_s, and u_2. In these situations, experience, judgment, and an assessment of the pore pressure dissipation data should be used to infer the correct soil behavior type.



(After Robertson, et al., 1986)

ZONE	SBT	
1	🟥	Sensitive, fine grained
2	🟫	Organic materials
3	🟦	Clay
4	🟦	Silty clay to clay
5	🟩	Clayey silt to silty clay
6	🟩	Sandy silt to clayey silt
7	🟩	Silty sand to sandy silt
8	🟨	Sand to silty sand
9	🟫	Sand
10	🟧	Gravely sand to sand
11	⬜	Very stiff fine grained*
12	⬛	Sand to clayey sand*

*over consolidated or cemented

Figure SBT

APPENDIX CPT

Pore Pressure Dissipation Tests (PPDT)

Pore Pressure Dissipation Tests (PPDT's) conducted at various intervals measured hydrostatic water pressures and determined the approximate depth of the ground water table. A PPDT is conducted when the cone is halted at specific intervals determined by the field representative. The variation of the penetration pore pressure (u) with time is measured behind the tip of the cone and recorded by a computer system.

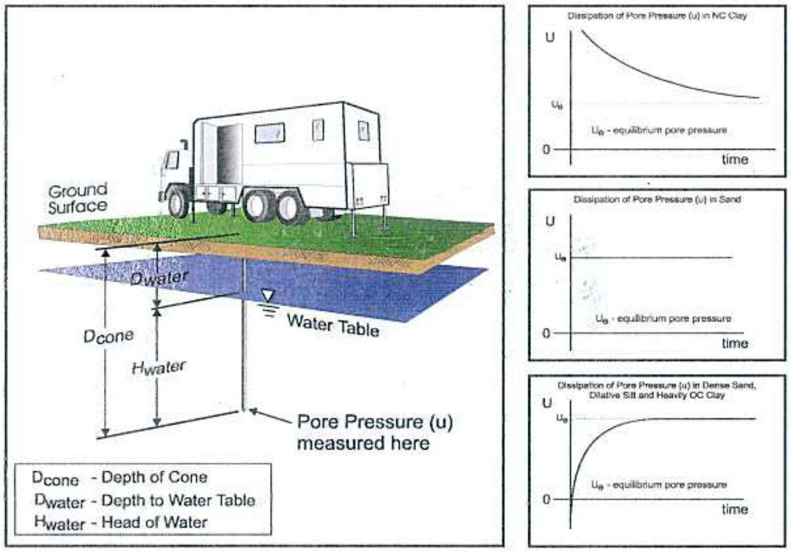
Pore pressure dissipation data can be interpreted to provide estimates of:

- Equilibrium piezometric pressure
- Phreatic Surface
- In situ horizontal coefficient of consolidation (c_h)
- In situ horizontal coefficient of permeability (k_h)

In order to correctly interpret the equilibrium piezometric pressure and/or the phreatic surface, the pore pressure must be monitored until such time as there is no variation in pore pressure with time, *Figure PPDT*. This time is commonly referred to as t_{100}, the point at which 100% of the excess pore pressure has dissipated.

A complete reference on pore pressure dissipation tests is presented by Robertson et al. 1992.

A summary of the pore pressure dissipation tests is summarized in Table 1.



Water Table Calculation

$$D_{water} = D_{cone} - H_{water}$$

where $H_{water} = U_e$ (depth units)

Useful Conversion Factors:
1psi = 0.704m = 2.31 feet (water)
1tsf = 0.958 bar = 13.9 psi
1m = 3.28 feet

Figure PPDT

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Copies of ASTM Standards are available through www.astm.org

APPENDIX D

GEOTECHNICAL LABORATORY TEST RESULTS

Checked

Approved

MOISTURE CONTENT & UNIT WEIGHT TEST RESULTS

Sample Identification	Depth, ft.	Wet Unit Weight, lb/ft.^{3}	Dry Unit Weight, lb/ft.^{3}	Moisture Content, %
B-1	10-10.5	119.3	101.0	18.2
B-1	25.0	126.2	101.2	24.8
B-1	35.0	130.5	109.4	19.3
B-8	12.5-13	117.0	88.3	32.4
B-8	25.5-26	133.4	110.6	20.6
B-8	60.5-61	130.6	106.5	22.6
B-9	6-6.5	98.0	82.7	18.4
B-9	20.0	124.1	98.0	26.6
B-9	30.0	132.5	109.8	20.7
B-9	60.0	136.0	114.9	18.3

Test Method: ASTM D2216, ASTM D2937

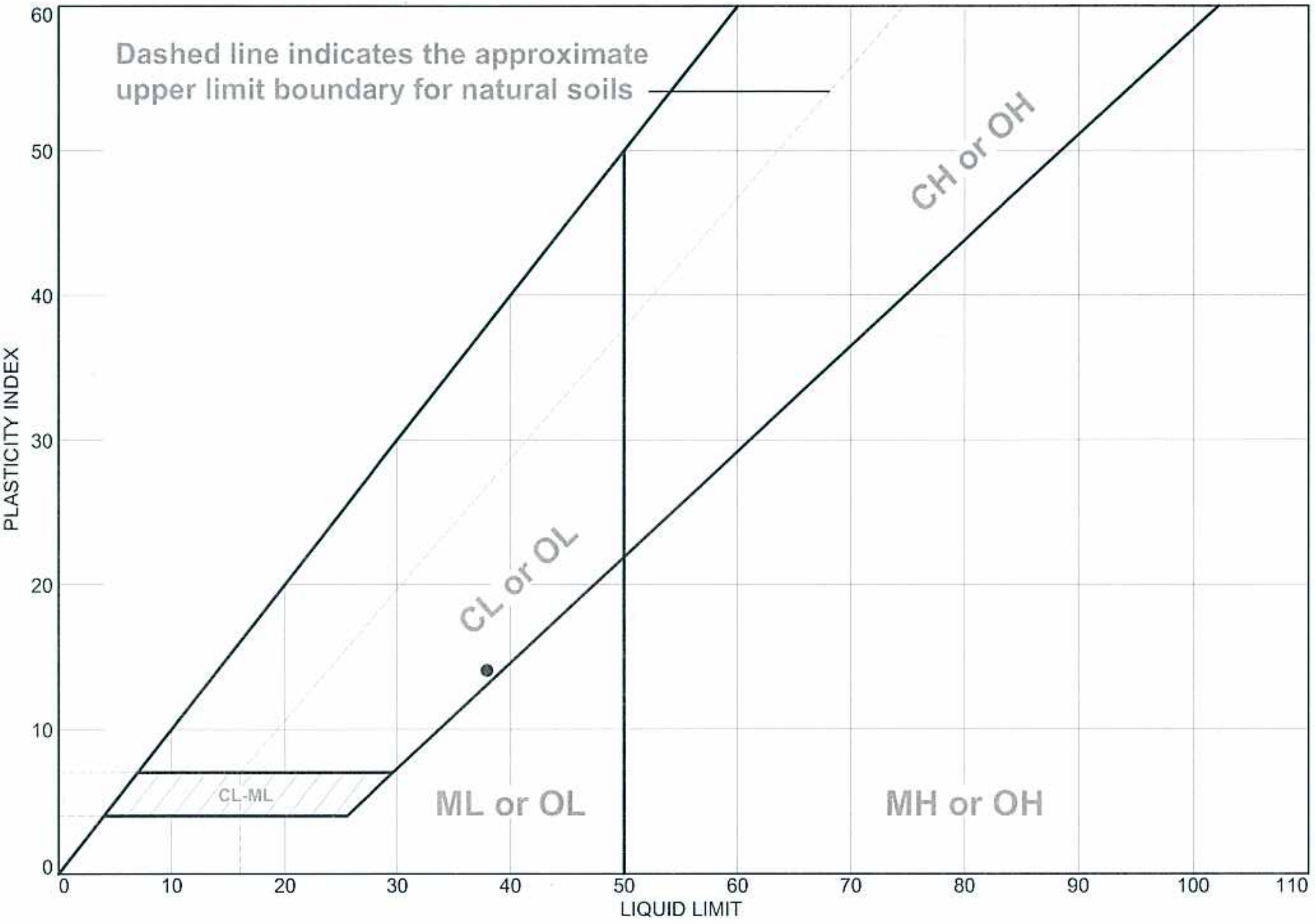
PROJECT NUMBER: 08-288 August 27, 2008

Bishop Ranch Parcel 1A
Job #4096-08-8527

SIERRA TESTING LABORATORIES, INC.
GEOTECHNICAL AND MATERIALS TESTING SERVICES

5040 Robert J. Mathews Blvd., El Dorado Hills, CA 95762
Phone: (916) 939-3460 FAX: (916) 939-3507

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		38	24	14			

Project No. 08-288 **Client:** MACTEC

Project: Bishop Ranch Parcel 1A

#4096-08-8527

●**Location:** B-1 **Depth:** 10-10.5 **Sample Number:** S6480

Remarks:

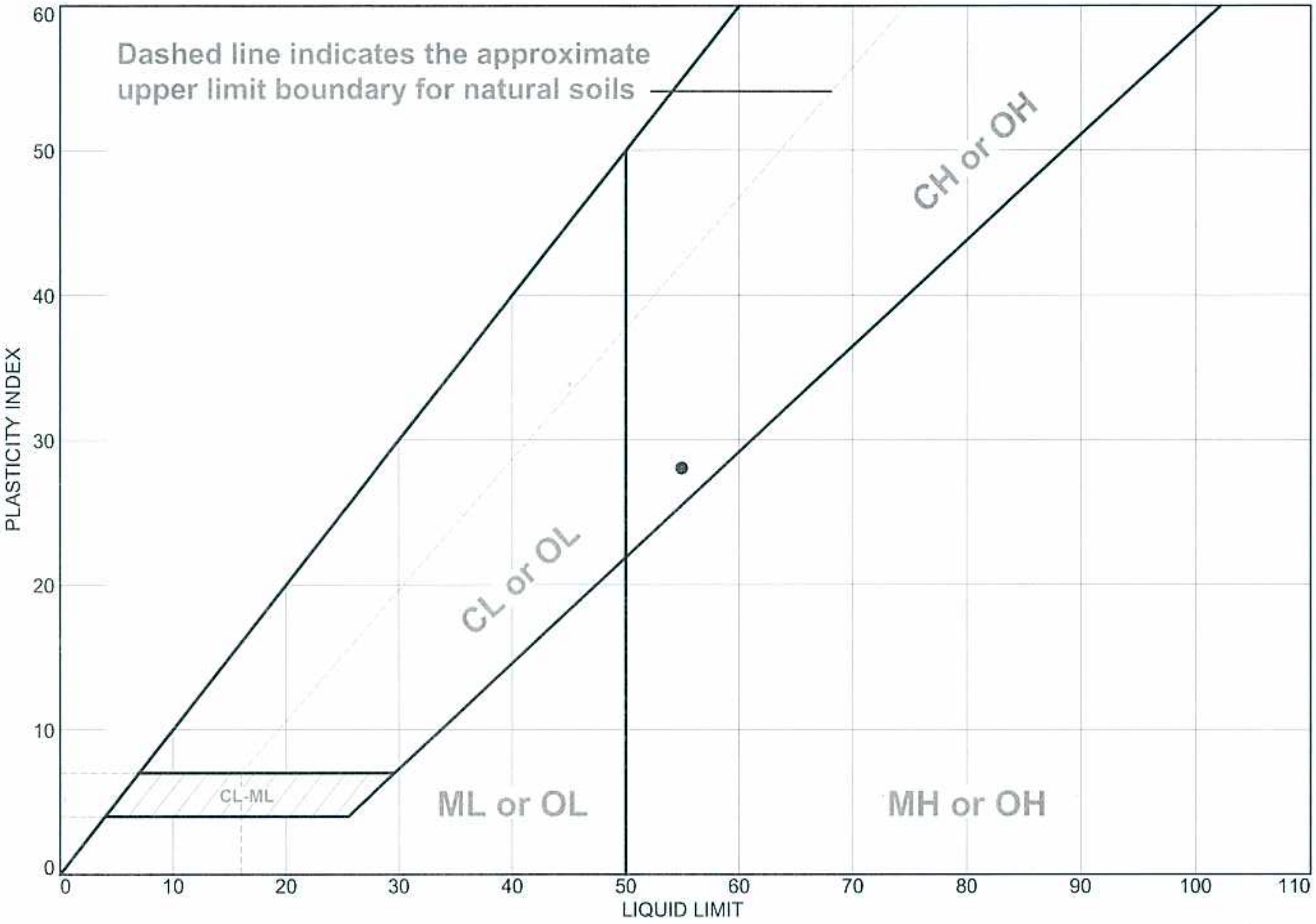
SIERRA TESTING LABS, INC.

El Dorado Hills, CA

Figure

Tested By: KS **Checked By:** CMW

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		55	27	28			

Project No. 08-288 **Client:** MACTEC

Project: Bishop Ranch Parcel 1A

#4096-08-8527

●**Location:** B-1 **Depth:** 25.0 **Sample Number:** S6482

Remarks:

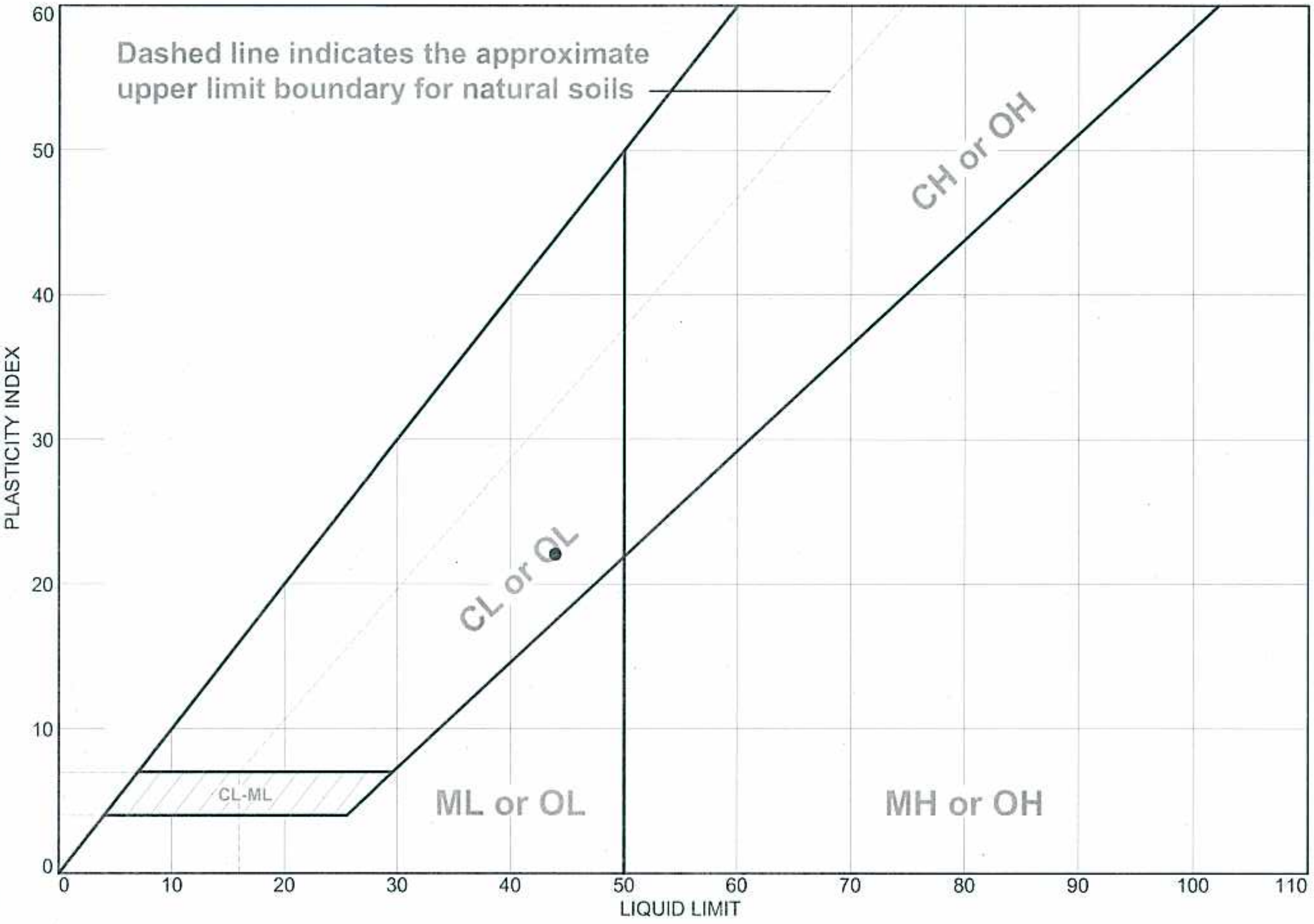
SIERRA TESTING LABS, INC.

El Dorado Hills, CA

Figure

Tested By: KS **Checked By:** CMW

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		44	22	22			

Project No. 08-288 **Client:** MACTEC

Project: Bishop Ranch Parcel 1A

#4096-08-8527

● **Location:** B-1 **Depth:** 35.0 **Sample Number:** S6484

Remarks:

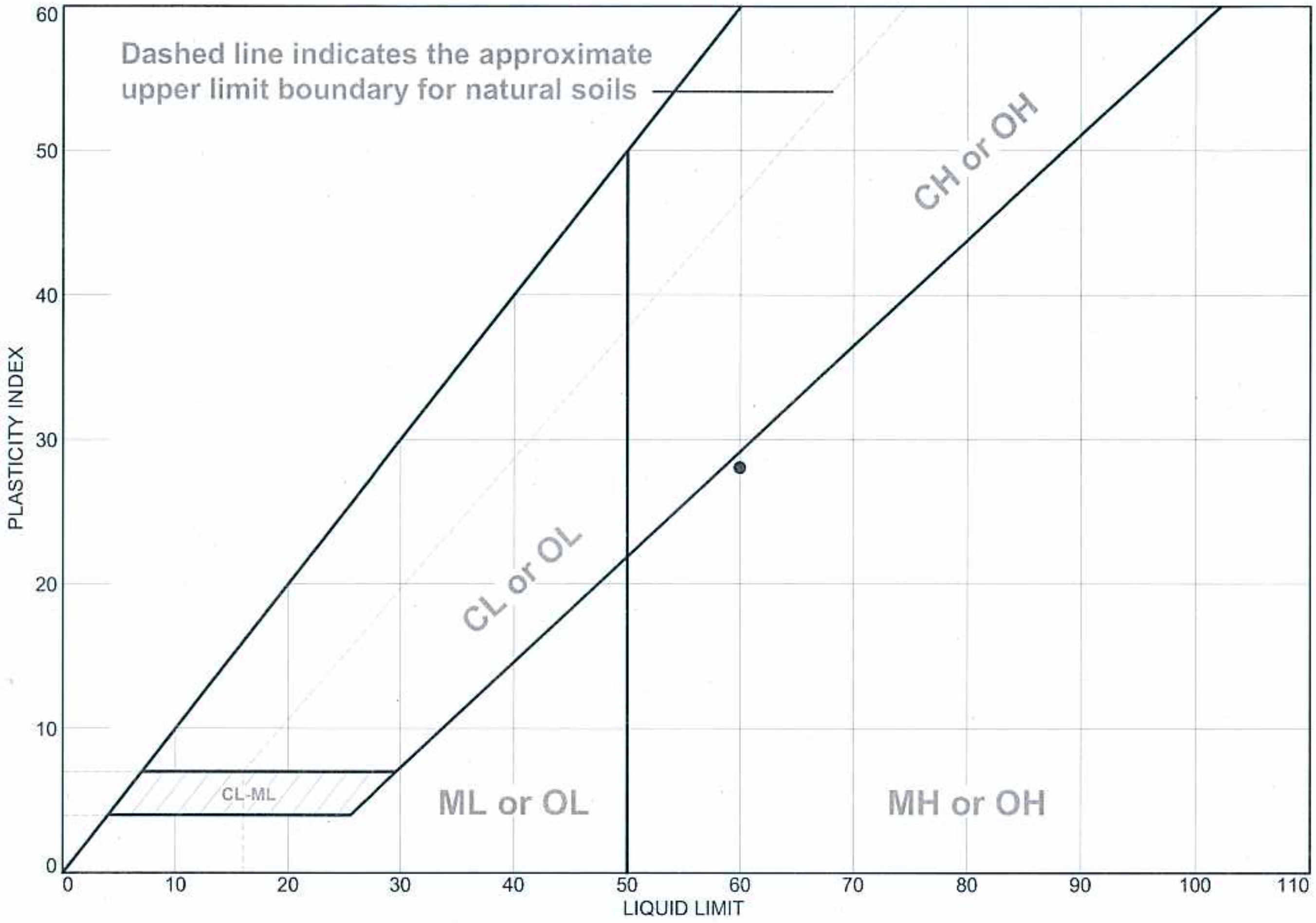
SIERRA TESTING LABS, INC.

El Dorado Hills, CA

Figure

Tested By: KS **Checked By:** CMW

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		60	32	28			

Project No. 08-288 **Client:** MACTEC

Project: Bishop Ranch Parcel 1A

#4096-08-8527

●**Location:** B-8 **Depth:** 12.5-13.0 **Sample Number:** S6491

Remarks:

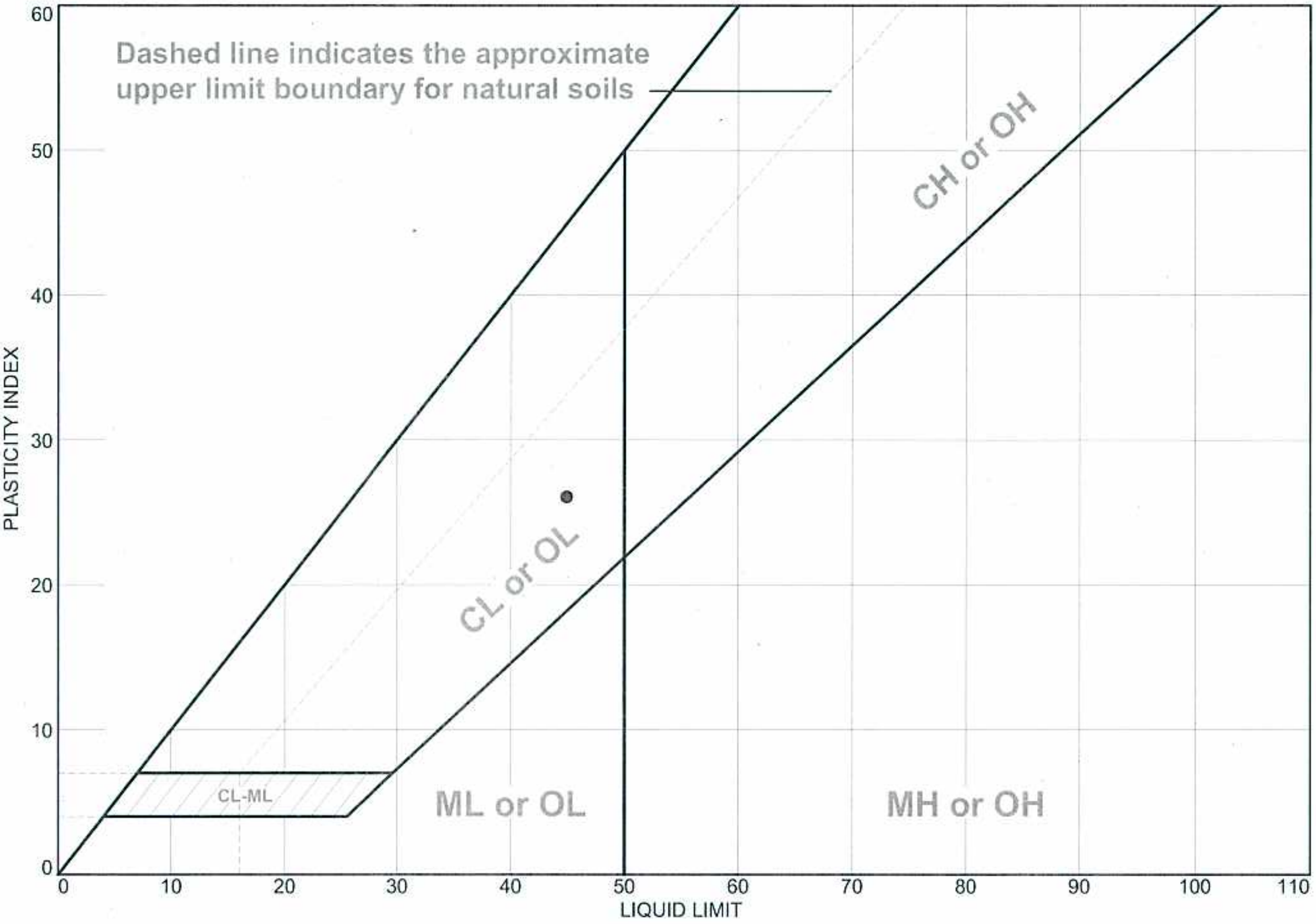
SIERRA TESTING LABS, INC.

El Dorado Hills, CA

Figure

Tested By: KS **Checked By:** CMW

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		45	19	26			

Project No. 08-288 **Client:** MACTEC

Project: Bishop Ranch Parcel 1A

#4096-08-8527

●**Location:** B-8 **Depth:** 25.5-26.0 **Sample Number:** S6493

Remarks:

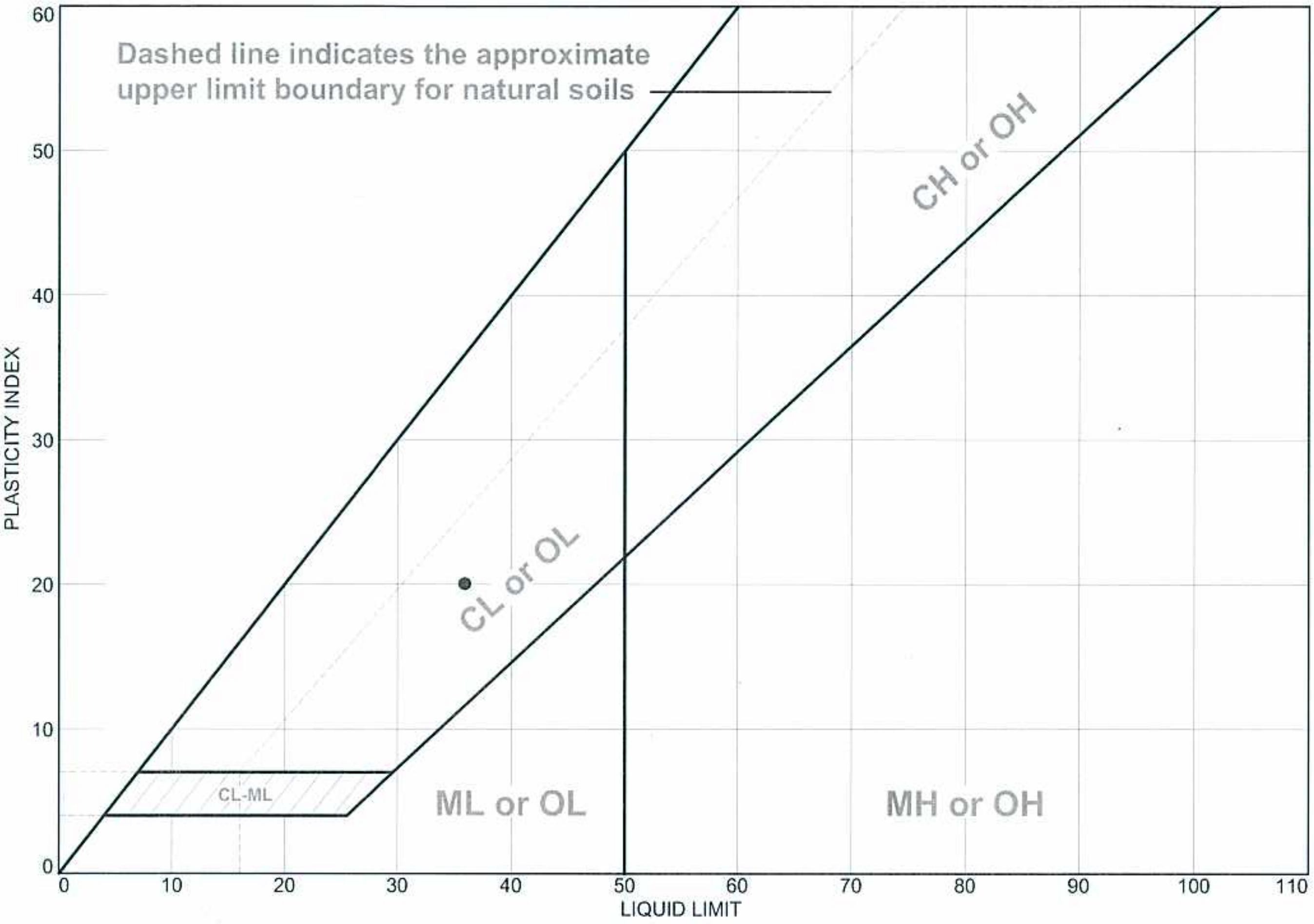
SIERRA TESTING LABS, INC.

El Dorado Hills, CA

Figure

Tested By: KS **Checked By:** CMW

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		36	16	20			

Project No. 08-288 **Client:** MACTEC

Project: Bishop Ranch Parcel 1A
#4096-08-8527

●**Location:** B-8 **Depth:** 60.5-61.0 **Sample Number:** S6496

Remarks:

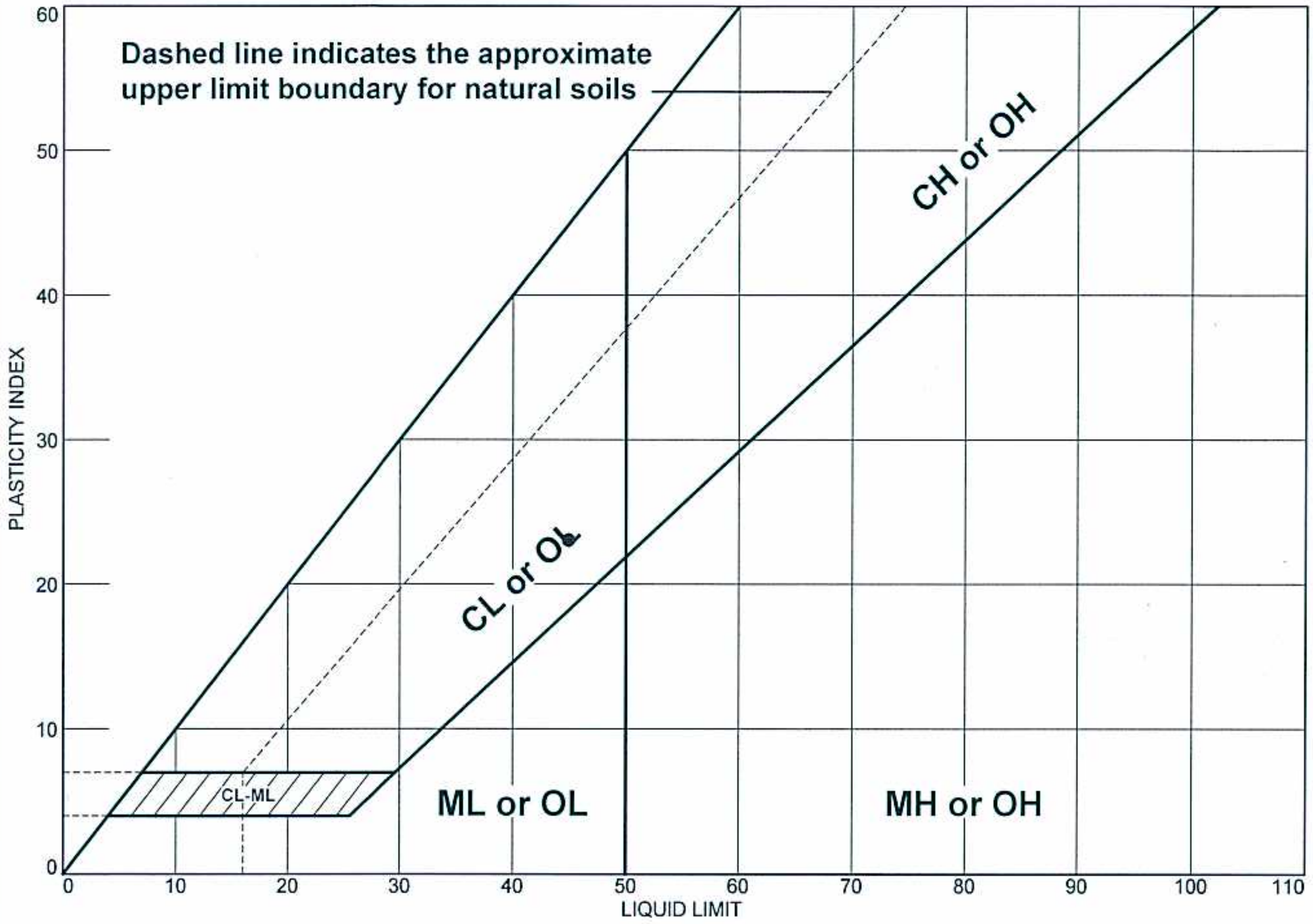
SIERRA TESTING LABS, INC.

El Dorado Hills, CA

Figure

Tested By: JS **Checked By:** CMW

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		45	22	23			

Project No. 08-288 **Client:** MACTEC

Project: Bishop Ranch Parcel 1A

#4096-08-8527

●**Location:** B-9 **Depth:** 6-6.5 **Sample Number:** S6498

Remarks:

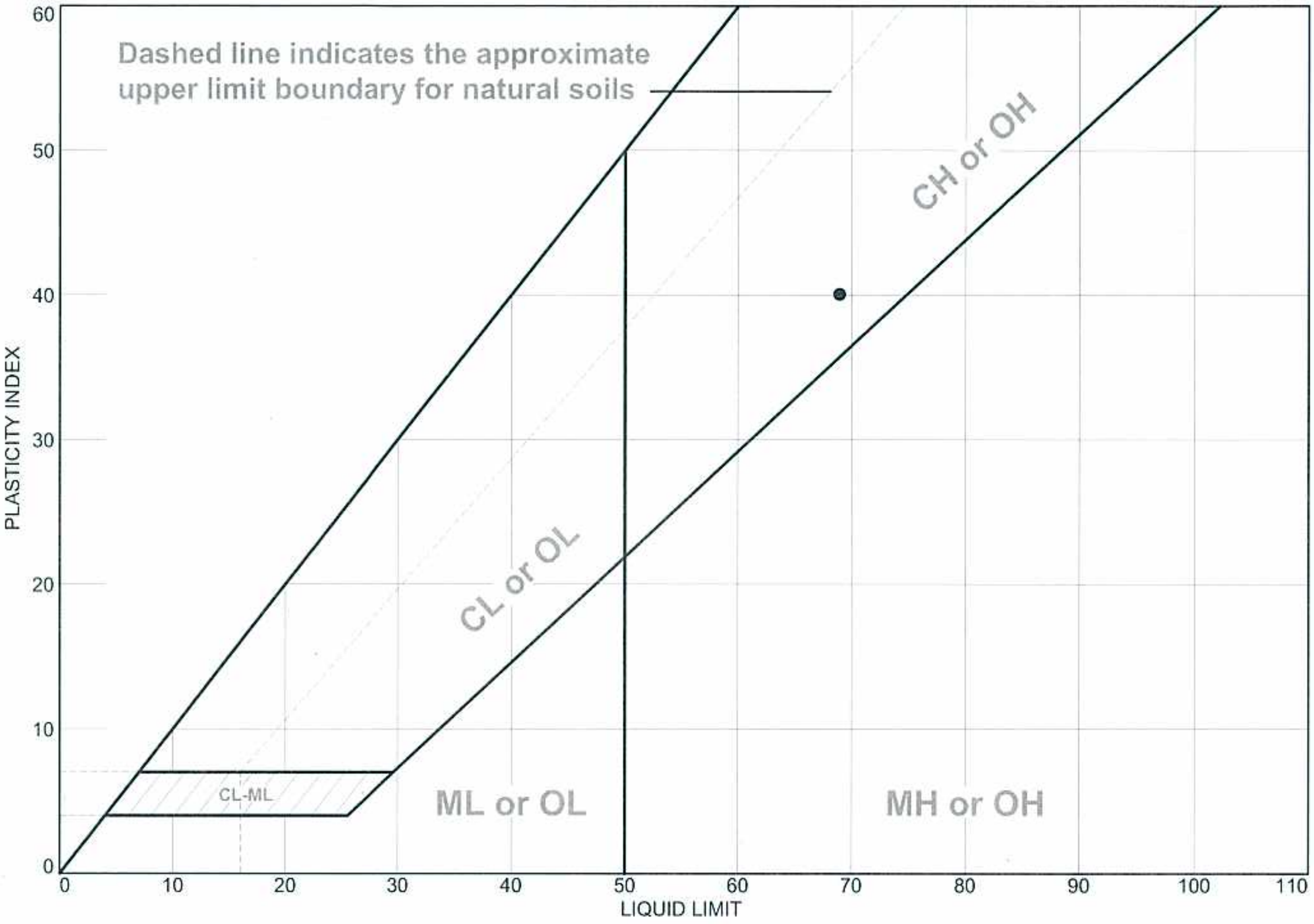
SIERRA TESTING LABS, INC.

El Dorado Hills, CA

Figure

Tested By: KS **Checked By:** CMW

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		69	29	40			

Project No. 08-288 **Client:** MACTEC

Project: Bishop Ranch Parcel 1A

#4096-08-8527

●**Location:** B-9 **Depth:** 20.0 **Sample Number:** S6500

Remarks:

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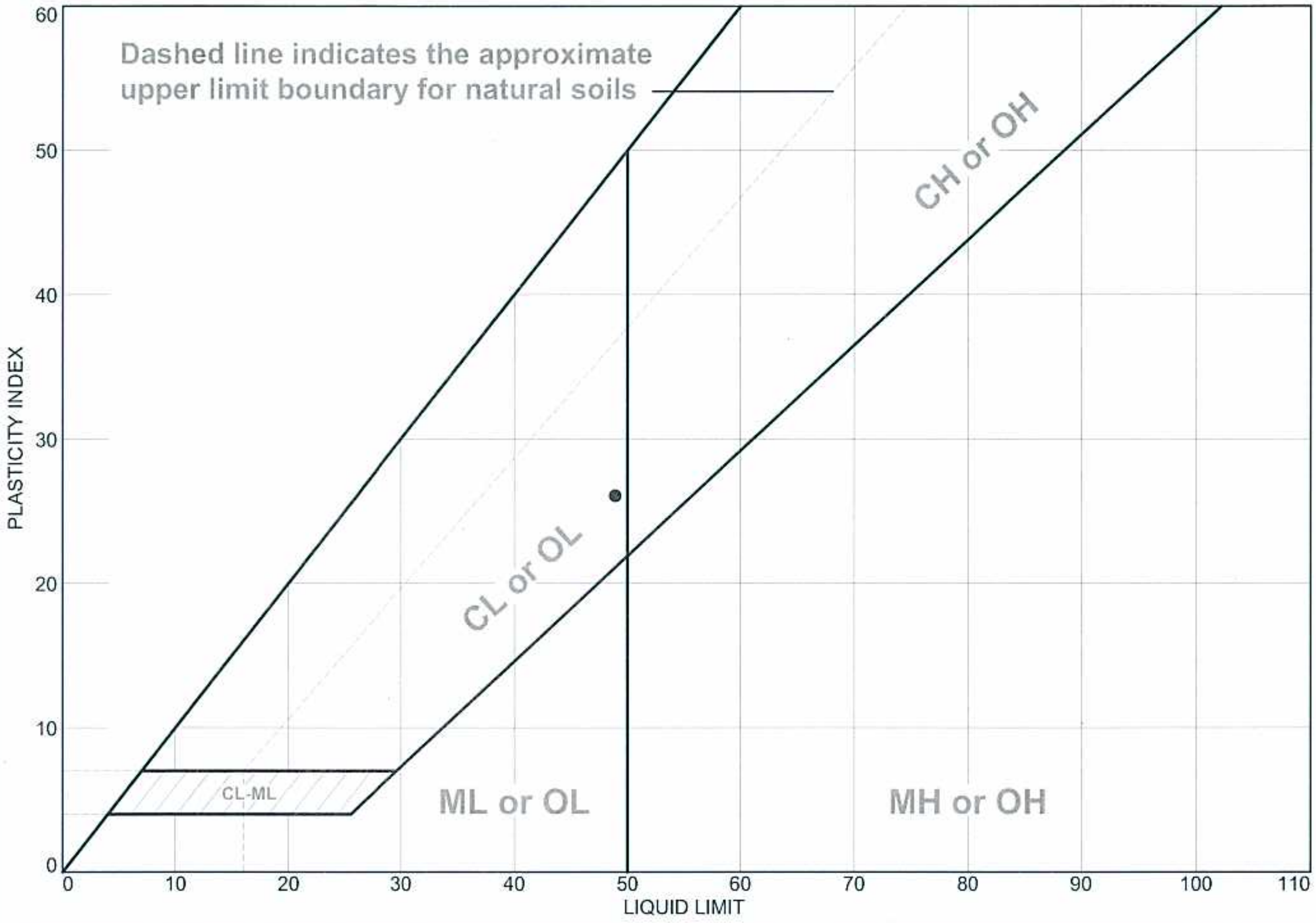
Figure

Tested By: KS **Checked By:** CMW

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●		49	23	26			

Project No. 08-288 **Client:** MACTEC

Project: Bishop Ranch Parcel 1A

#4096-08-8527

●**Location:** B-9 **Depth:** 30.0 **Sample Number:** S6502

Remarks:

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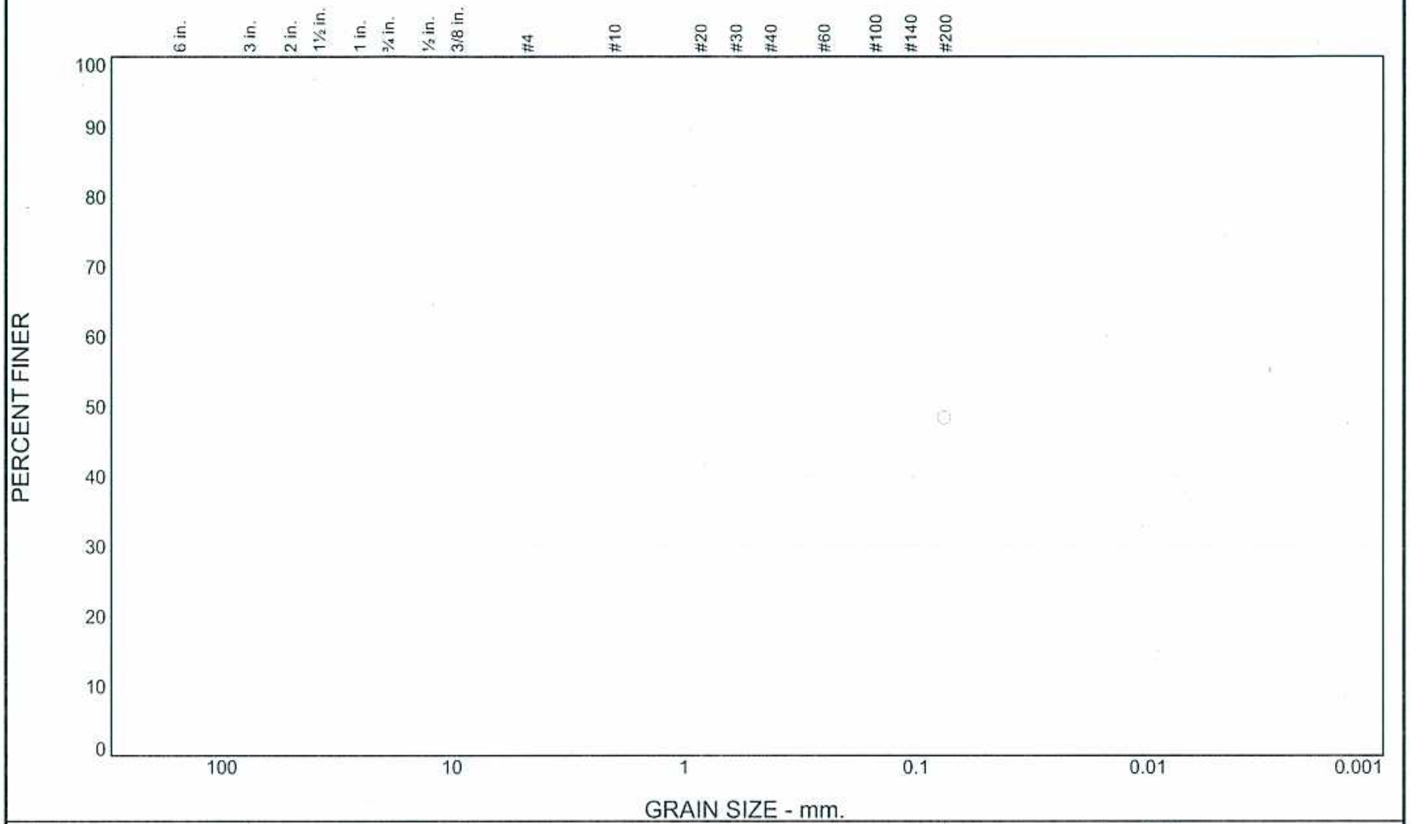
El Dorado Hills, CA

Figure

Tested By: KS **Checked By:** CMW

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						48.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	48.2		

* (no specification provided)

Material Description

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

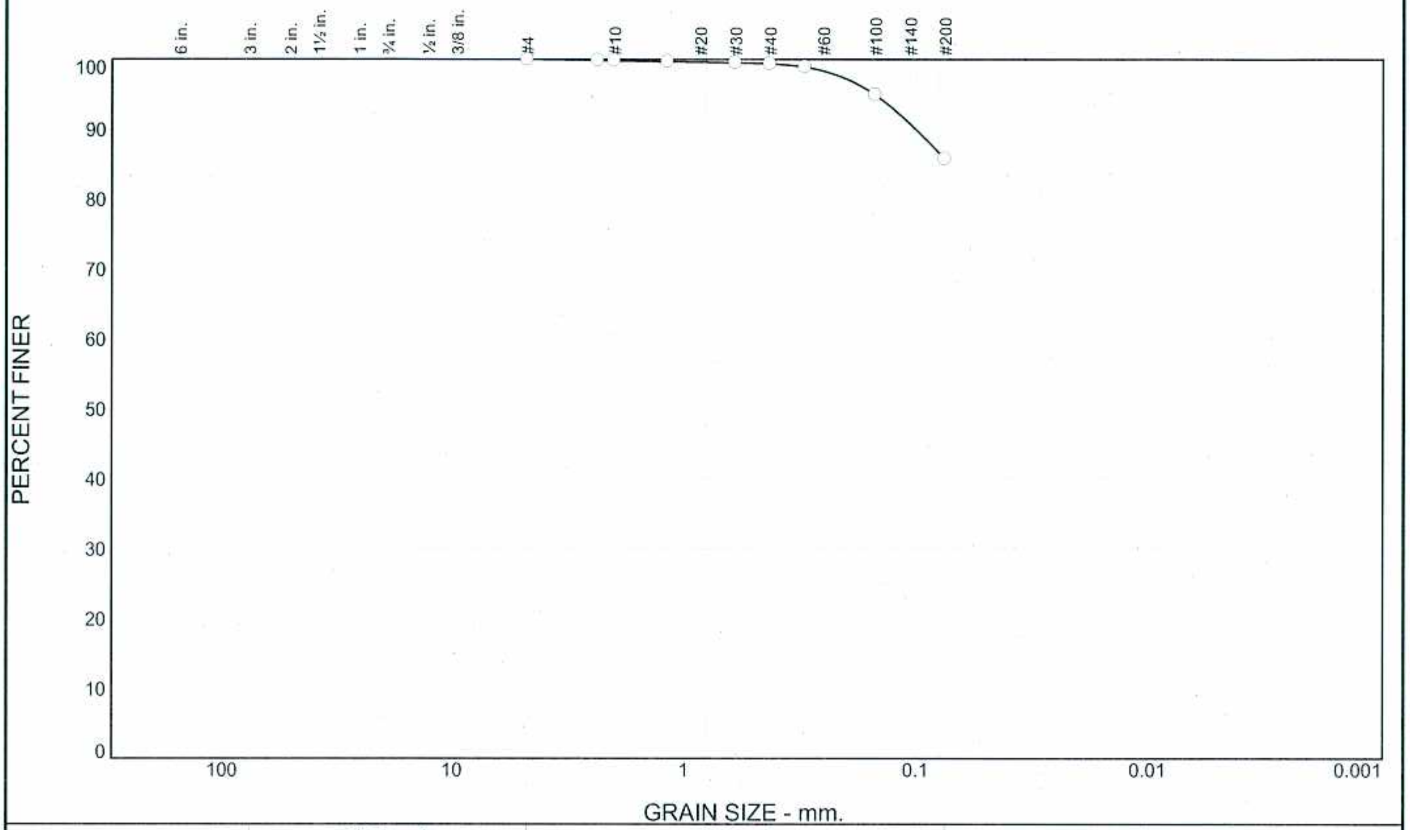
Location: B-1
Sample Number: S6485 **Depth:** 45.0 **Date:** 8/27/08

SIERRA
TESTING LABS, INC.
El Dorado Hills, CA

Client: MACTEC
Project: Bishop Ranch Parcel 1A
#4096-08-8527
Project No: 08-288 **Figure**

Tested By: MPW **Checked By:** CMW

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.2	0.4	13.7	85.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	99.8		
#10	99.8		
#16	99.7		
#30	99.5		
#40	99.4		
#50	98.9		
#100	94.9		
#200	85.7		

* (no specification provided)

Material Description

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.1005 D_{85}= D_{60}=

D_{50}= D_{30}= D_{15}=

D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Location: B-1

Sample Number: S6486 **Depth:** 50.0 **Date:** 8/27/08

SIERRA
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El Dorado Hills, CA

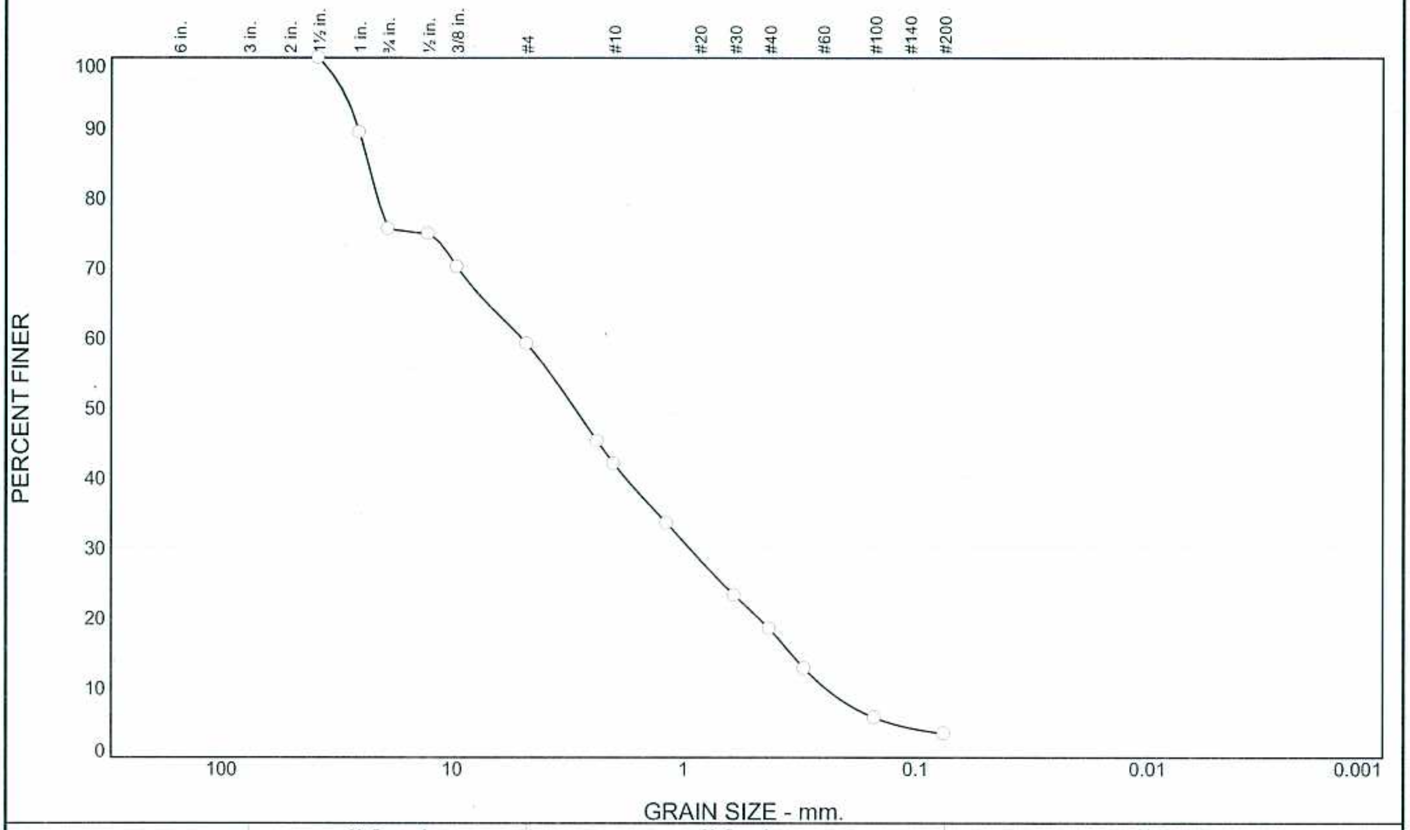
Client: MACTEC

Project: Bishop Ranch Parcel 1A
#4096-08-8527

Project No: 08-288 **Figure**

Tested By: CM **Checked By:** CMW

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	24.5	16.4	17.2	23.5	15.0	3.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/2 Inch	100.0		
1 Inch	89.3		
3/4 Inch	75.5		
1/2 Inch	74.8		
3/8 Inch	70.1		
#4	59.1		
#8	45.2		
#10	41.9		
#16	33.4		
#30	23.1		
#40	18.4		
#50	12.7		
#100	5.7		
#200	3.4		

* (no specification provided)

Material Description

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 25.7766 D_{85}= 23.3519 D_{60}= 5.0361

D_{50}= 2.9748 D_{30}= 0.9474 D_{15}= 0.3459

D_{10}= 0.2464 C_u= 20.44 C_c= 0.72

Classification

USCS= SP AASHTO=

Remarks

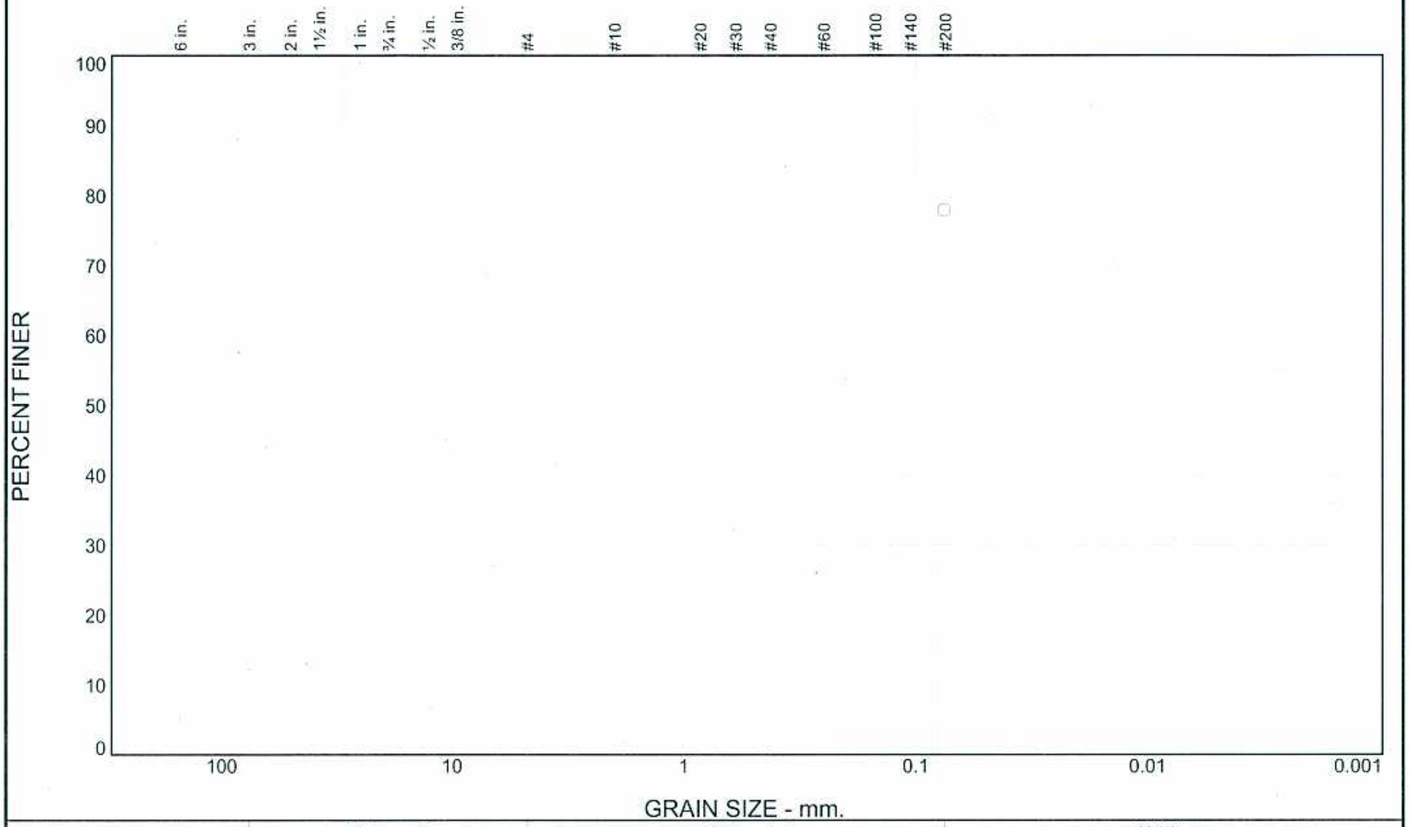
Location: B-8
Sample Number: S6497 **Depth:** 65-66.5 **Date:** 8/27/08

SIERRA
TESTING LABS, INC.
El Dorado Hills, CA

Client: MACTEC
Project: Bishop Ranch Parcel 1A
#4096-08-8527
Project No: 08-288 **Figure**

Tested By: JO **Checked By:** CMW

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						77.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	77.7		

* (no specification provided)

Material Description

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

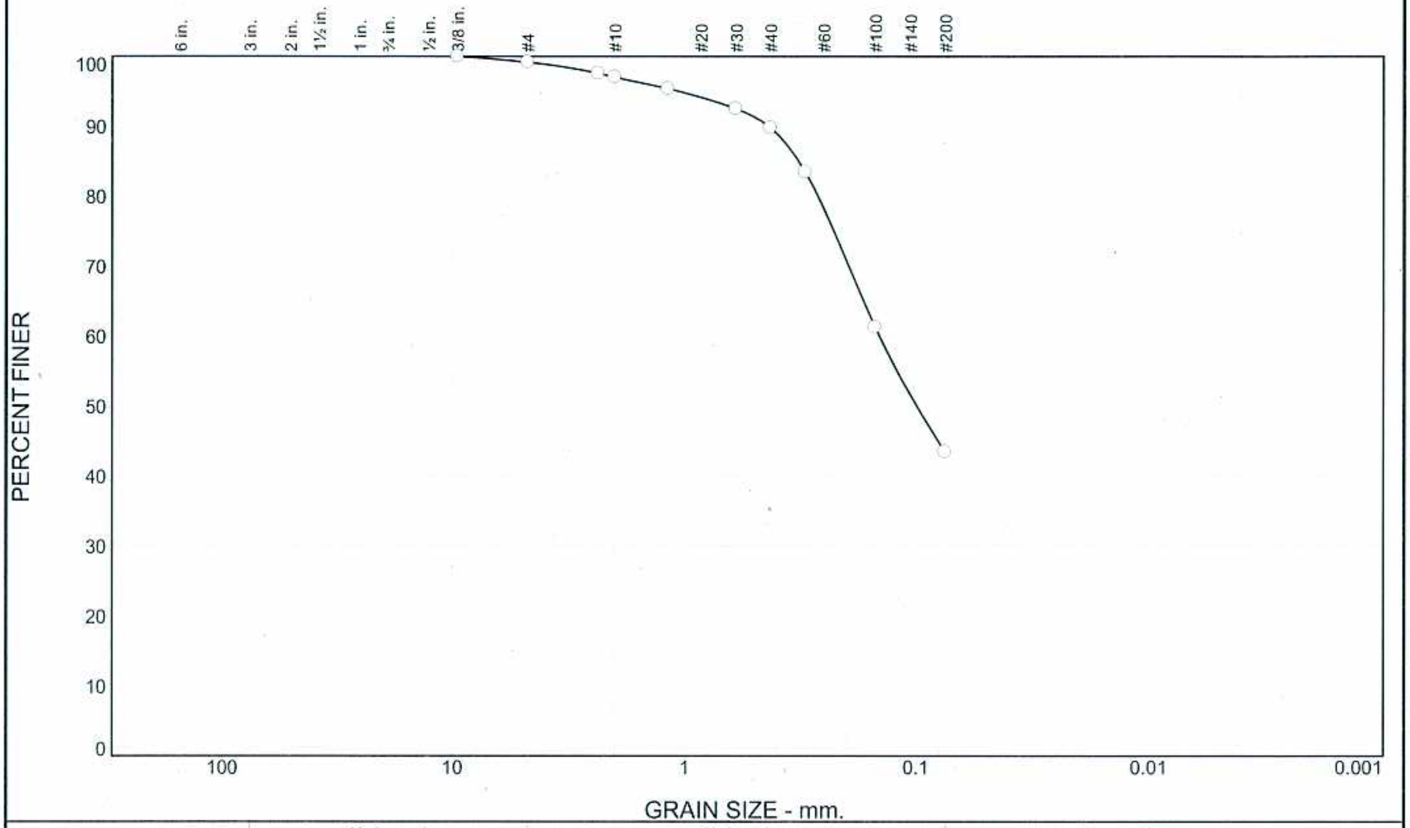
Location: B-9
Sample Number: S6504 **Depth:** 50.5-51.0 **Date:** 8/27/08

SIERRA
TESTING LABS, INC.
El Dorado Hills, CA

Client: MACTEC
Project: Bishop Ranch Parcel 1A
#4096-08-8527
Project No: 08-288 **Figure**

Tested By: MPW **Checked By:** CMW

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.8	2.2	7.2	46.4	43.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 Inch	100.0		
#4	99.2		
#8	97.6		
#10	97.0		
#16	95.4		
#30	92.5		
#40	89.8		
#50	83.4		
#100	61.3		
#200	43.4		

* (no specification provided)

Material Description

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.4331 D_{85}= 0.3209 D_{60}= 0.1439
D_{50}= 0.0993 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

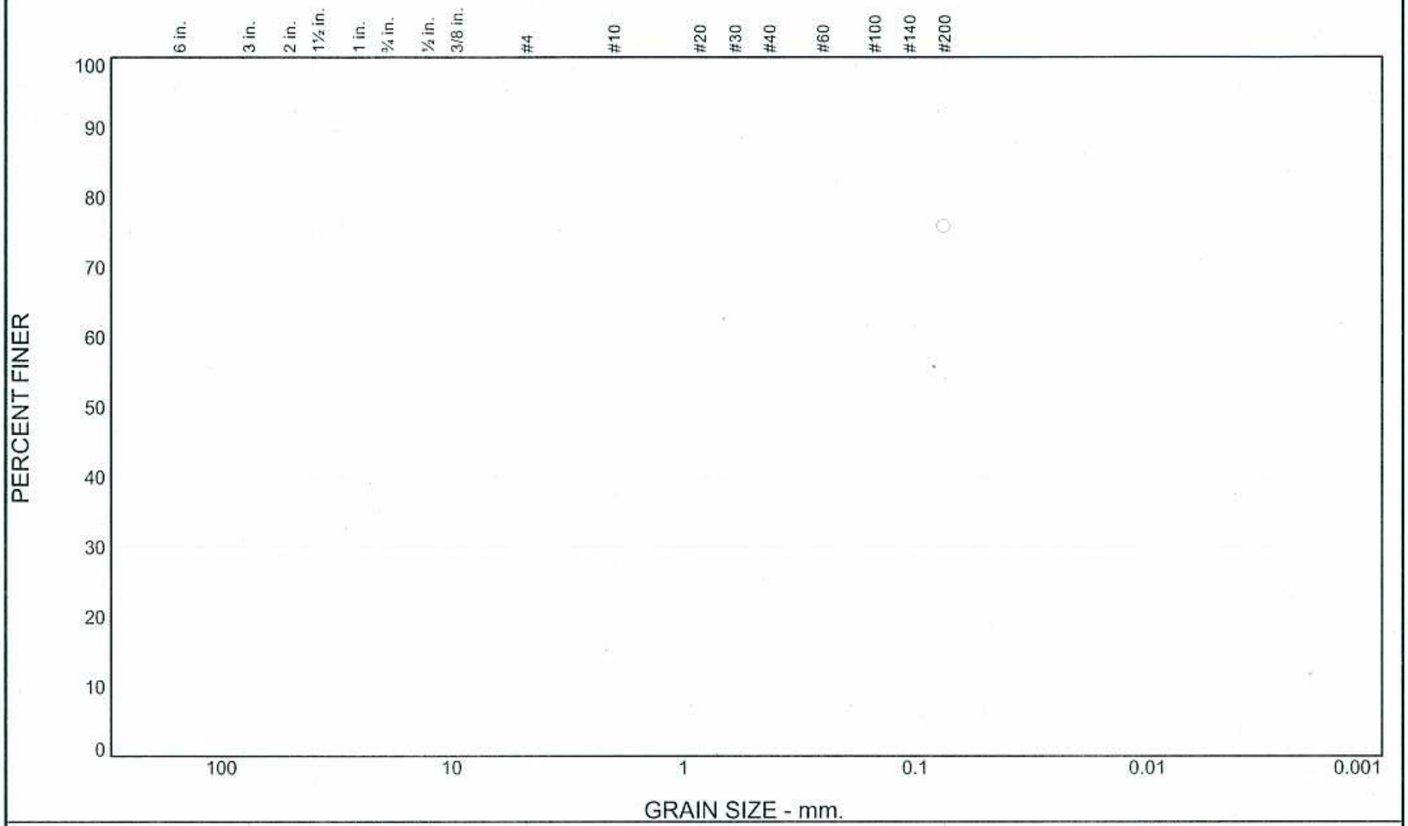
Location: B-9
Sample Number: S6505 **Depth:** 60.0 **Date:** 8/27/08

SIERRA
TESTING LABS, INC.
El Dorado Hills, CA

Client: MACTEC
Project: Bishop Ranch Parcel 1A
#4096-08-8527
Project No: 08-288 **Figure**

Tested By: CM **Checked By:** JF

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						75.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	75.7		

* (no specification provided)

Material Description

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

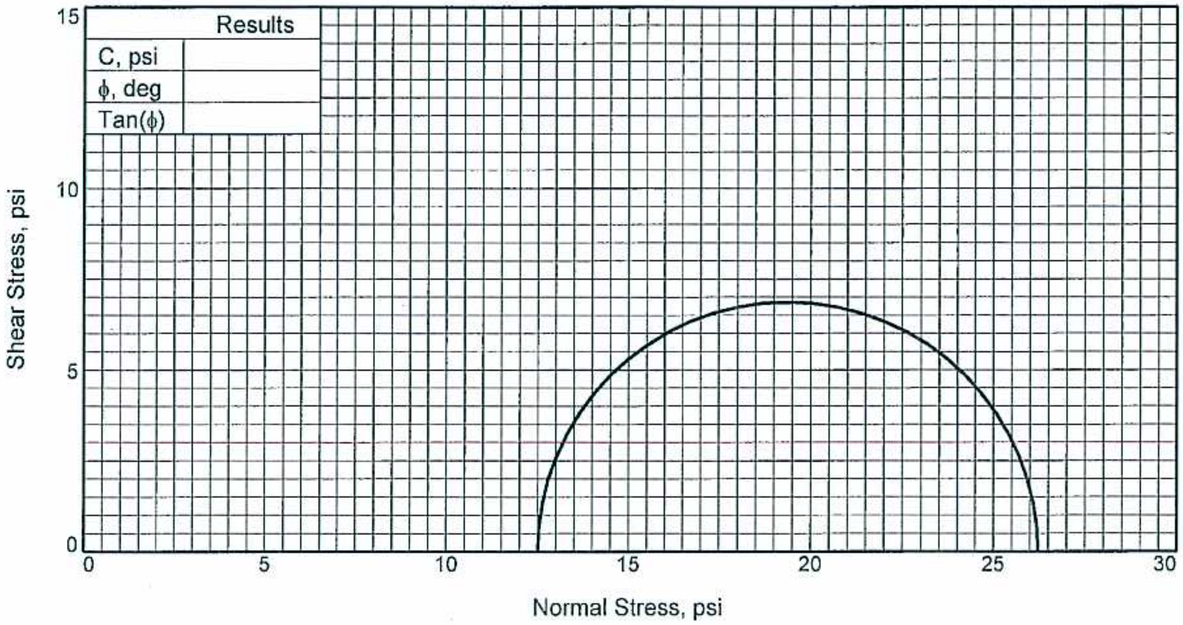
Remarks

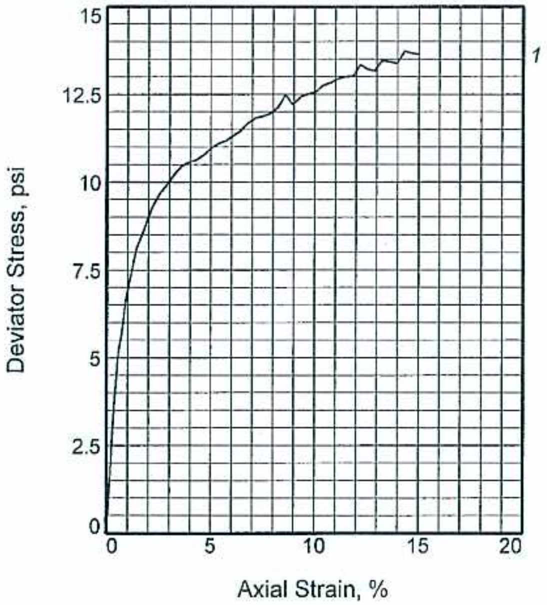
Location: B-9
Sample Number: S6506 **Depth:** 68.5 **Date:** 8/27/08

SIERRA
TESTING LABS, INC.
El Dorado Hills, CA

Client: MACTEC
Project: Bishop Ranch Parcel 1A
#4096-08-8527
Project No: 08-288 **Figure**

Tested By: MPW **Checked By:** CMW





Sample No.		1
Initial	Water Content, %	35.3
	Dry Density, pcf	87.0
	Saturation, %	99.8
	Void Ratio	0.9740
	Diameter, in.	2.41
	Height, in.	5.58
At Test	Water Content, %	35.2
	Dry Density, pcf	87.0
	Saturation, %	99.4
	Void Ratio	0.9740
	Diameter, in.	2.41
	Height, in.	5.58
Strain rate, in./min.		0.03
Back Pressure, psi		0.0
Cell Pressure, psi		12.5
Fail. Stress, psi		13.7
Ult. Stress, psi		
σ_1 Failure, psi		26.2
σ_3 Failure, psi		12.5

Type of Test:
Unconsolidated Undrained

Sample Type: Undisturbed

Description:

Specific Gravity= 2.75

Remarks:

Figure ________

Client: MACTEC

Project: Bishop Ranch Parcel 1A
#4096-08-8527

Location: B-1

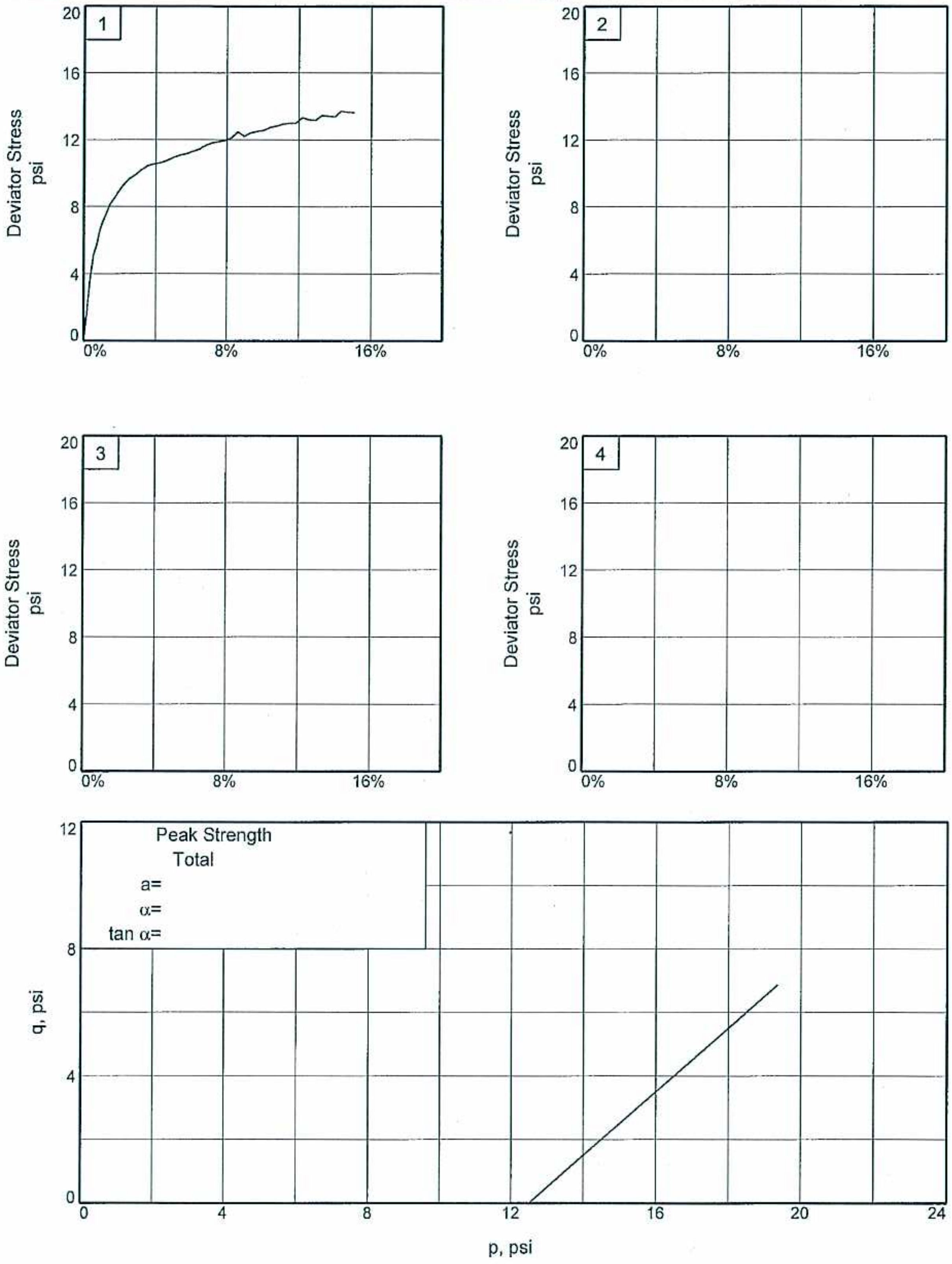
Sample Number: S6481 **Depth:** 15.0

Proj. No.: 08-288 **Date Sampled:** 8/27/08

TRIAXIAL SHEAR TEST REPORT

SIERRA TESTING LABS, INC.

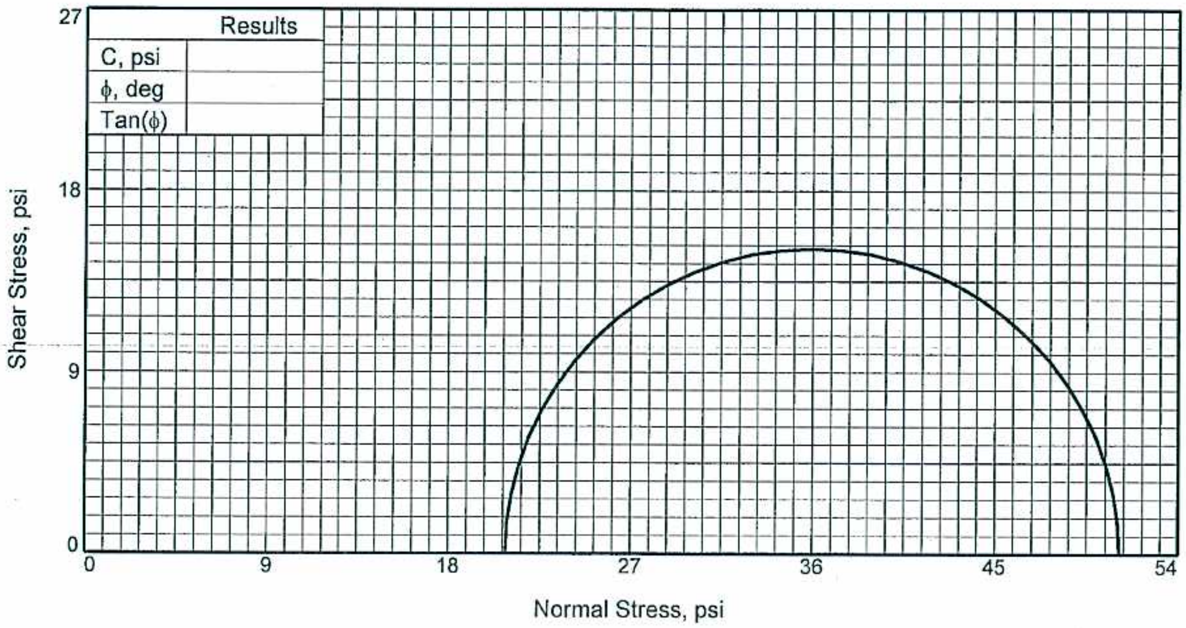
Tested By: MW **Checked By:** MPW



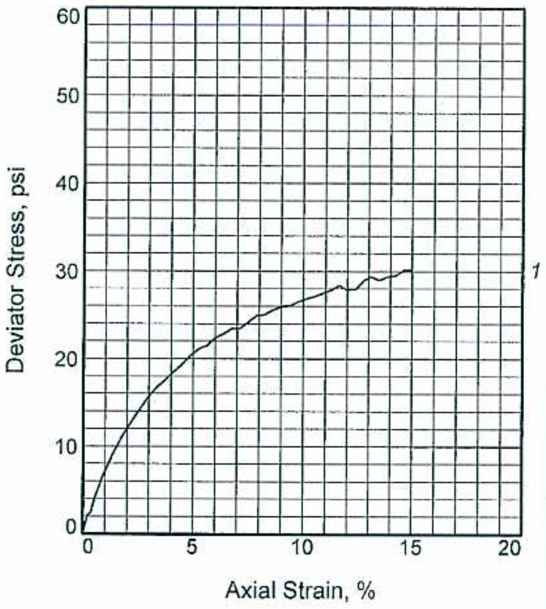
Client: MACTEC
Project: Bishop Ranch Parcel 1A
Location: B-1 **Depth:** 15.0 **Sample Number:** S6481
Project No.: 08-288 **Figure** ______

Sierra Testing Labs, Inc.

Tested By: MW **Checked By:** MPW



Results	
C, psi	
ϕ, deg	
Tan(ϕ)	



	Sample No.	1
Initial	Water Content, %	25.2
	Dry Density, pcf	100.9
	Saturation, %	98.8
	Void Ratio	0.7019
	Diameter, in.	2.41
	Height, in.	5.33
At Test	Water Content, %	25.0
	Dry Density, pcf	100.9
	Saturation, %	97.9
	Void Ratio	0.7019
	Diameter, in.	2.41
	Height, in.	5.33
	Strain rate, in./min.	0.03
	Back Pressure, psi	0.0
	Cell Pressure, psi	20.8
	Fail. Stress, psi	30.3
	Ult. Stress, psi	
	σ_1 Failure, psi	51.1
	σ_3 Failure, psi	20.8

Type of Test:
Unconsolidated Undrained

Sample Type: Undisturbed

Description:

Specific Gravity= 2.75

Remarks:

Figure ________

Client: MACTEC

Project: Bishop Ranch Parcel 1A
#4096-08-8527

Location: B-1

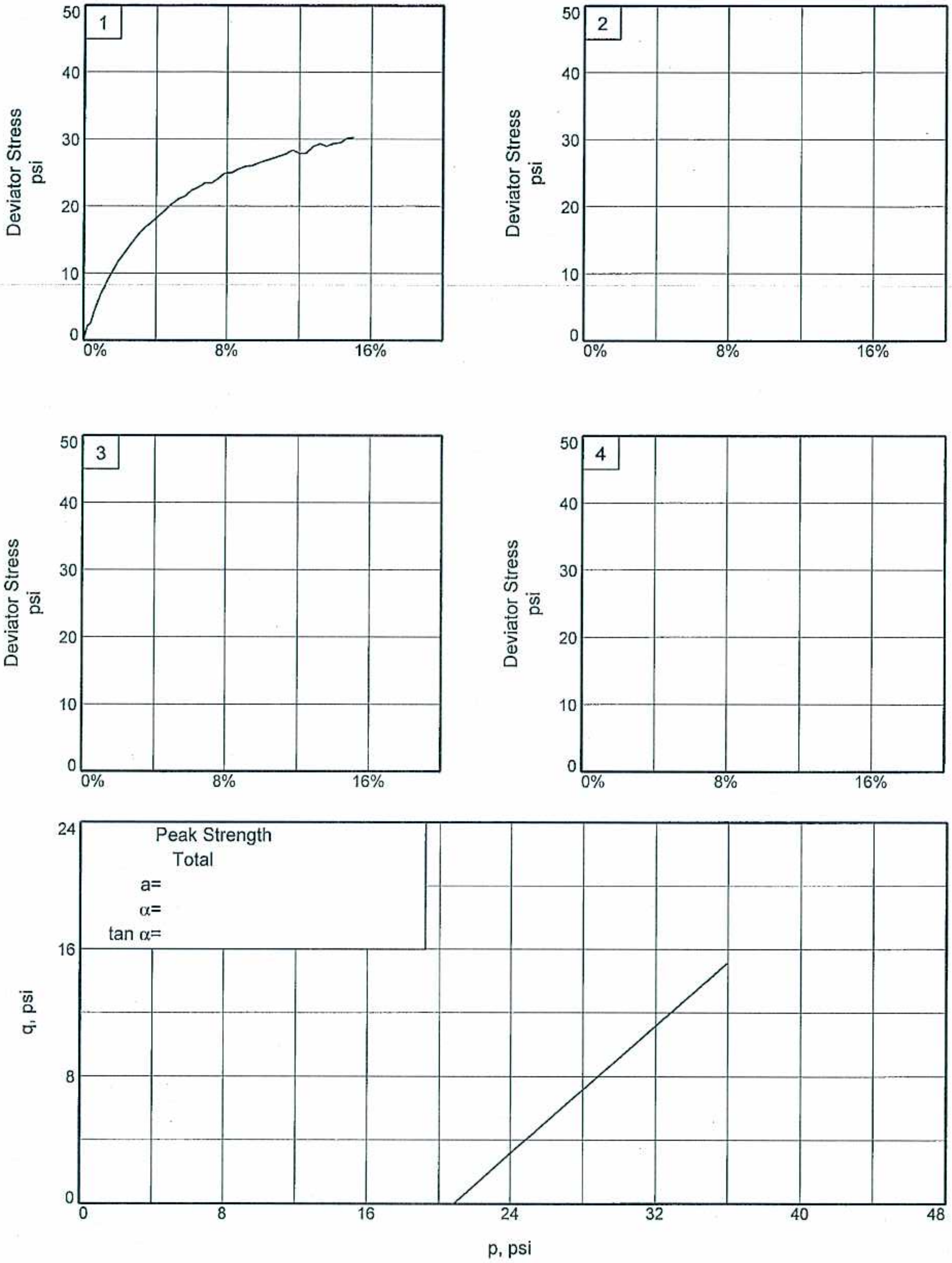
Sample Number: S6483 **Depth:** 30.0

Proj. No.: 08-288 **Date Sampled:** 8/26/08

TRIAXIAL SHEAR TEST REPORT

SIERRA TESTING LABS, INC.

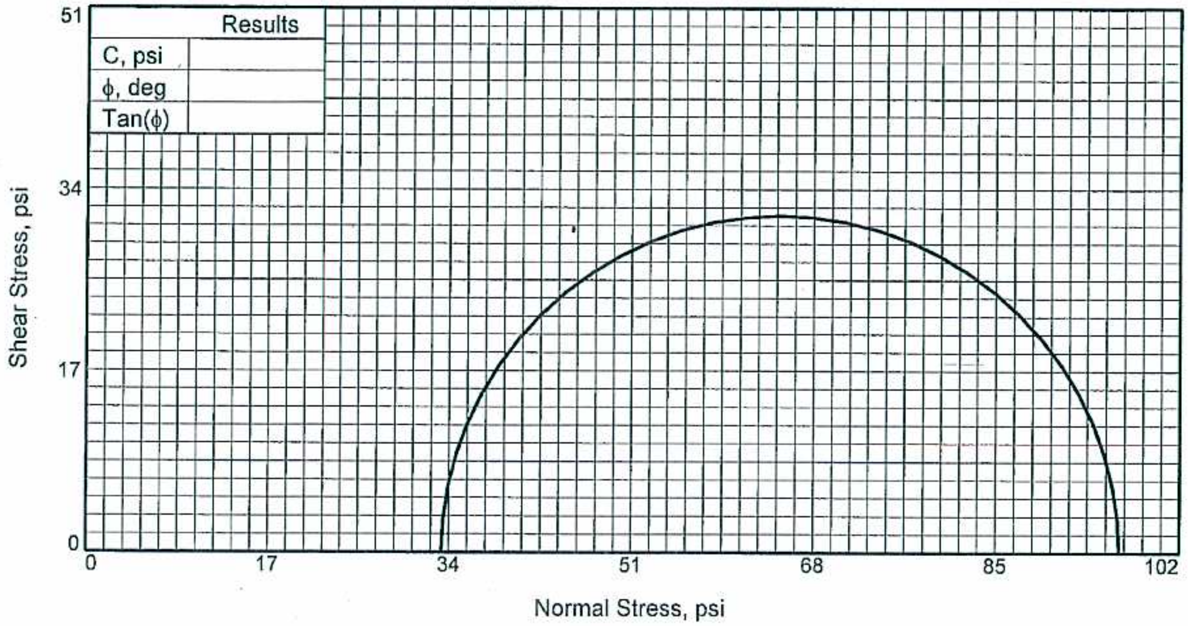
Tested By: MW **Checked By:** MPW

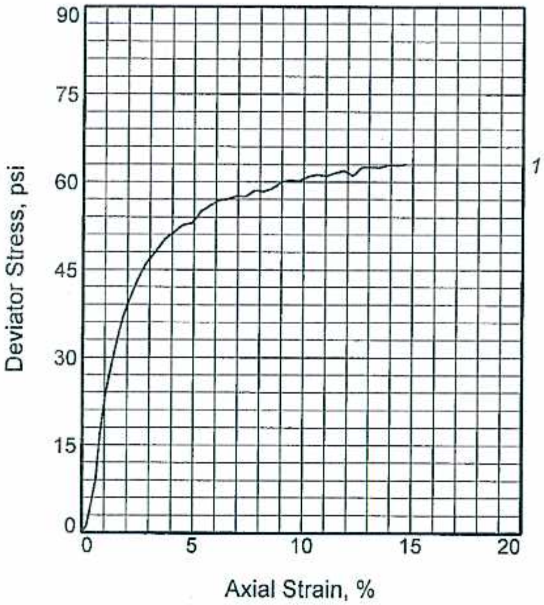


Client: MACTEC
Project: Bishop Ranch Parcel 1A
Location: B-1 **Depth:** 30.0 **Sample Number:** S6483
Project No.: 08-288 **Figure** ______

Sierra Testing Labs, Inc.

Tested By: MW **Checked By:** MPW





Sample No.		1
Initial	Water Content, %	20.1
	Dry Density, pcf	110.2
	Saturation, %	99.2
	Void Ratio	0.5585
	Diameter, in.	2.41
	Height, in.	4.90
At Test	Water Content, %	20.0
	Dry Density, pcf	110.2
	Saturation, %	98.4
	Void Ratio	0.5585
	Diameter, in.	2.41
	Height, in.	4.90
Strain rate, in./min.		0.03
Back Pressure, psi		0.0
Cell Pressure, psi		33.3
Fail. Stress, psi		63.1
Ult. Stress, psi		
σ_1 Failure, psi		96.4
σ_3 Failure, psi		33.3

Type of Test:
Unconsolidated Undrained
Sample Type: Undisturbed
Description:

Specific Gravity= 2.75
Remarks:

Figure ________

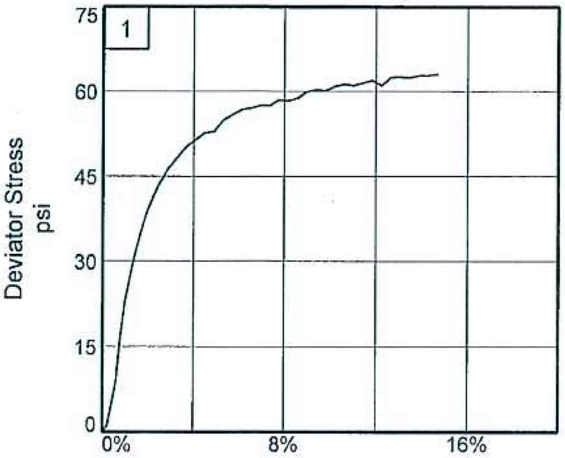
Client: MACTEC

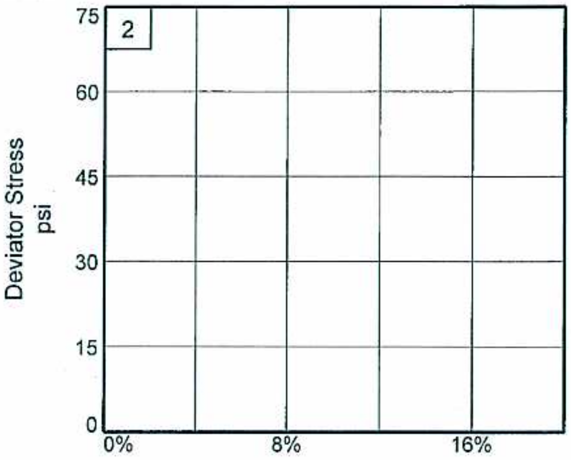
Project: Bishop Ranch Parcel 1A
#4096-08-8527
Location: B-1
Sample Number: S6487 **Depth:** 60.0
Proj. No.: 08-288 **Date Sampled:** 8/27/08

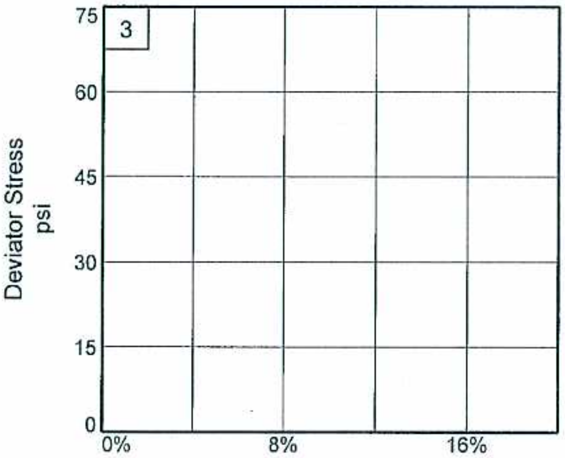
TRIAXIAL SHEAR TEST REPORT

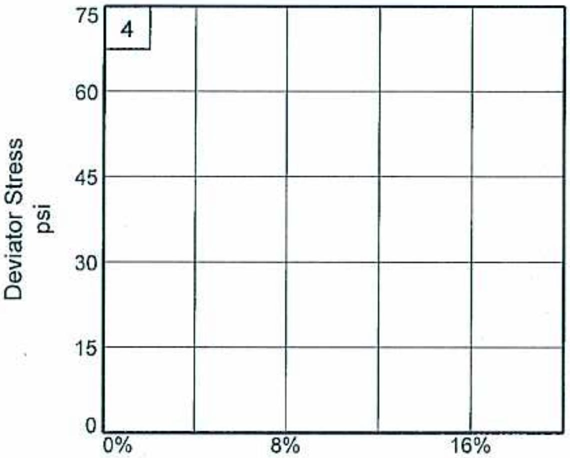
SIERRA TESTING LABS, INC.

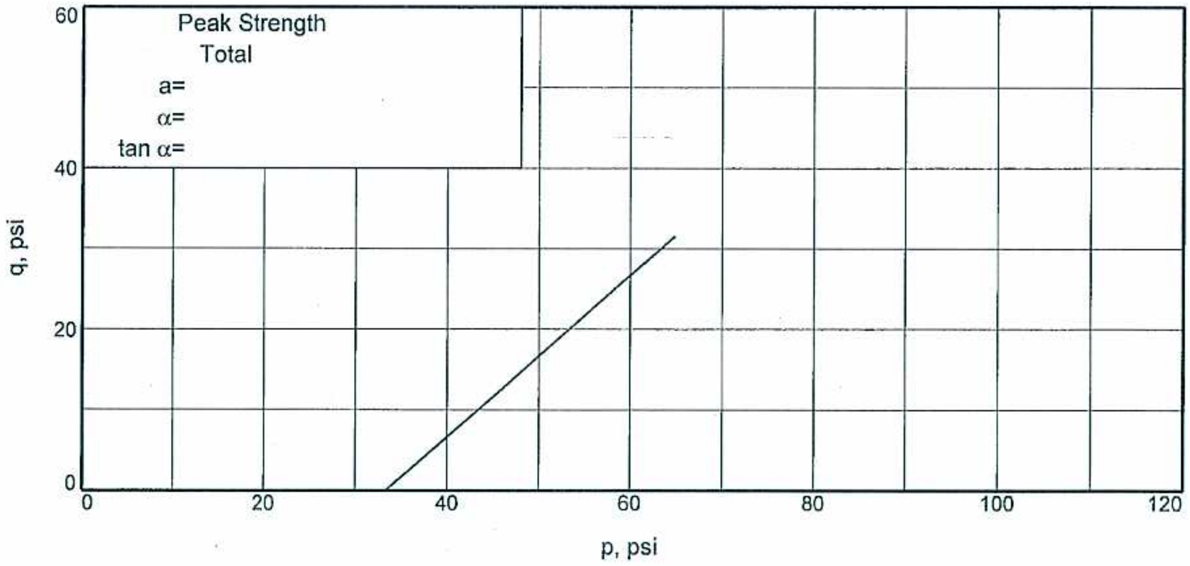
Tested By: MW **Checked By:** MPW











Client: MACTEC

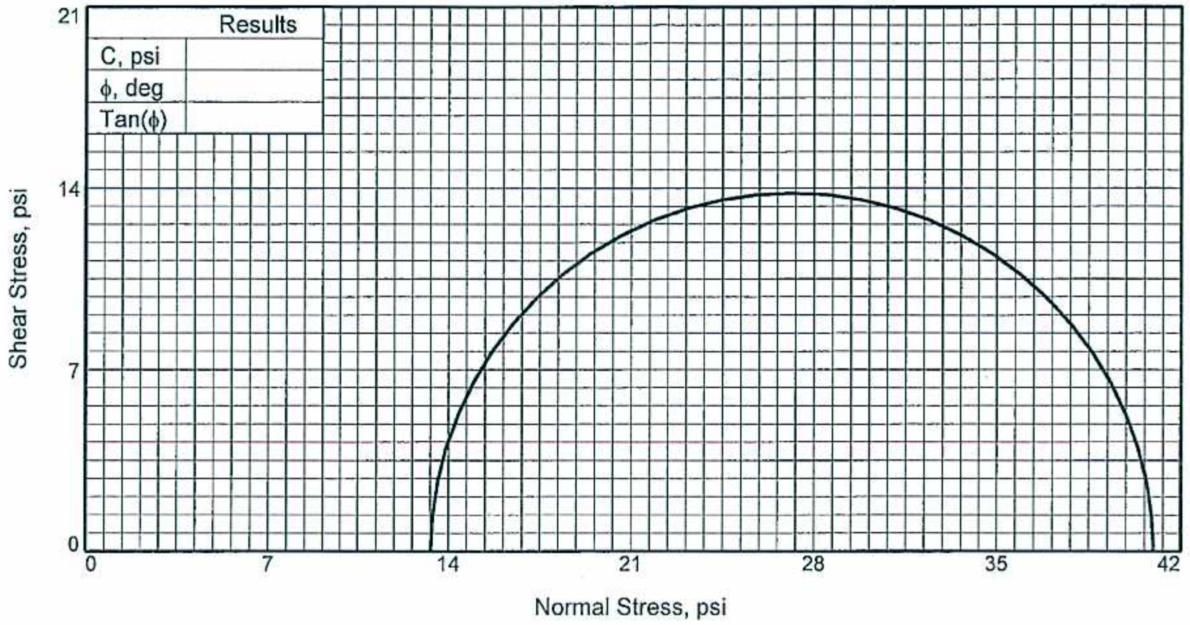
Project: Bishop Ranch Parcel 1A

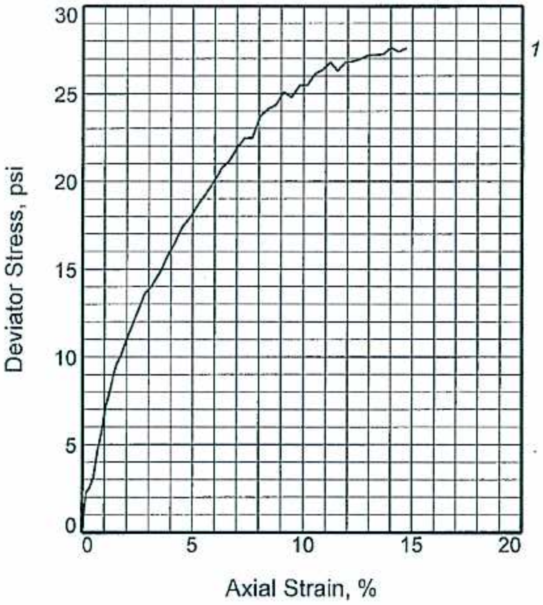
Location: B-1 **Depth:** 60.0 **Sample Number:** S6487

Project No.: 08-288 **Figure** _______

Sierra Testing Labs, Inc.

Tested By: MW **Checked By:** MPW





	Sample No.	1
Initial	Water Content, %	27.1
	Dry Density, pcf	98.2
	Saturation, %	99.7
	Void Ratio	0.7475
	Diameter, in.	2.41
	Height, in.	5.71
At Test	Water Content, %	26.8
	Dry Density, pcf	98.2
	Saturation, %	98.6
	Void Ratio	0.7475
	Diameter, in.	2.41
	Height, in.	5.71
	Strain rate, in./min.	0.03
	Back Pressure, psi	0.0
	Cell Pressure, psi	13.3
	Fail. Stress, psi	27.6
	Ult. Stress, psi	
	σ_1 Failure, psi	40.9
	σ_3 Failure, psi	13.3

Type of Test:
Unconsolidated Undrained

Sample Type: Undisturbed

Description:

Specific Gravity= 2.75

Remarks:

Figure _______

Client: MACTEC

Project: Bishop Ranch Parcel 1A
#4096-08-8527

Location: B-8

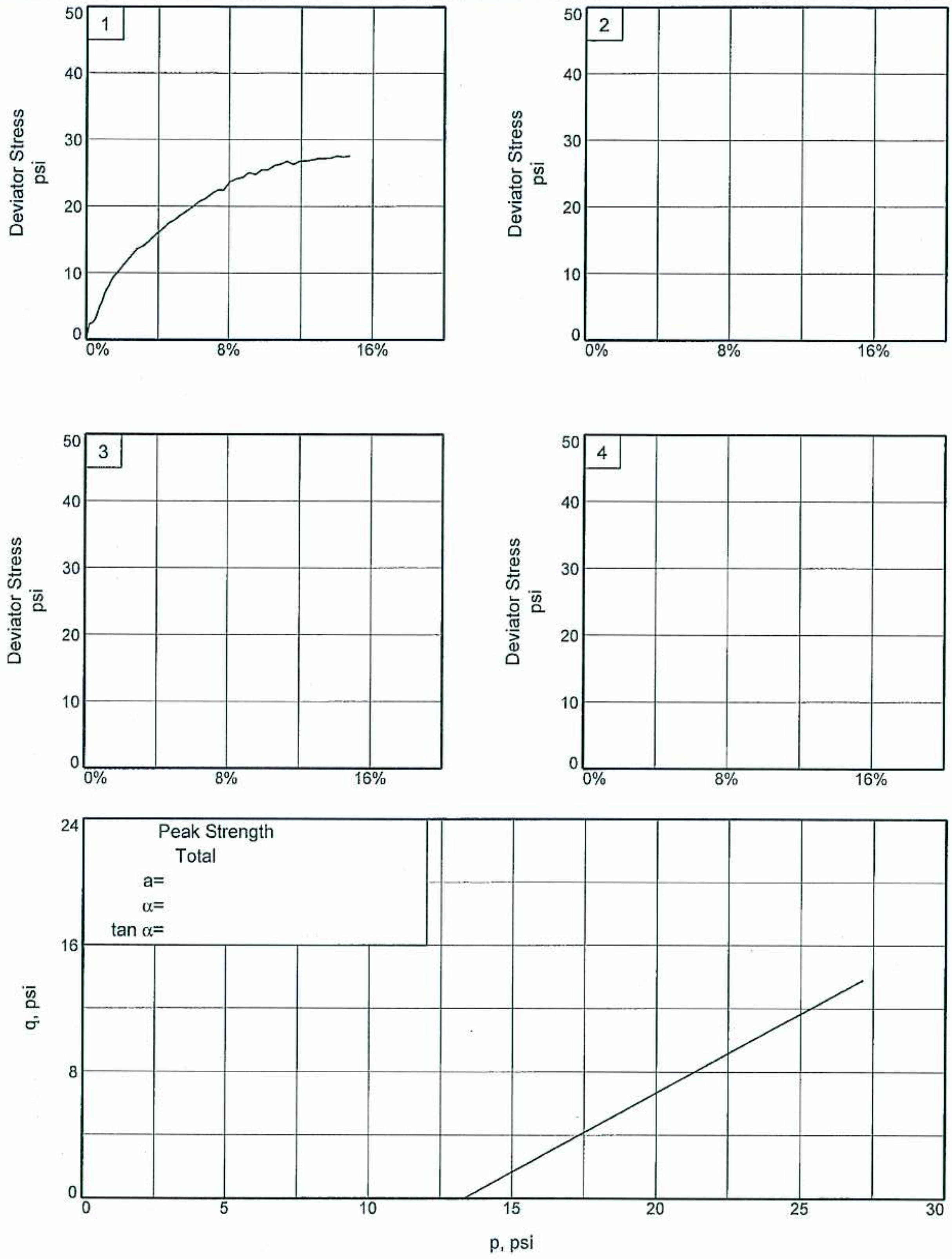
Sample Number: S6492 **Depth:** 20.5-21.0

Proj. No.: 08-288 **Date Sampled:** 8/27/08

TRIAXIAL SHEAR TEST REPORT

SIERRA TESTING LABS, INC.

Tested By: MW **Checked By:** MPW



Client: MACTEC

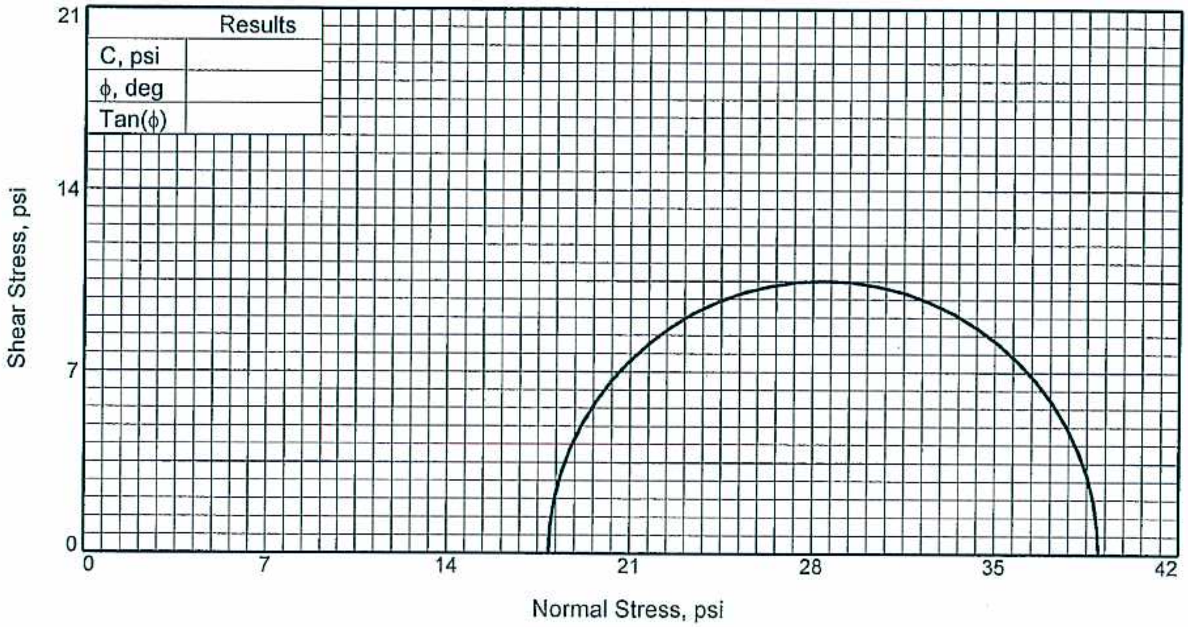
Project: Bishop Ranch Parcel 1A

Location: B-8 **Depth:** 20.5-21.0 **Sample Number:** S6492

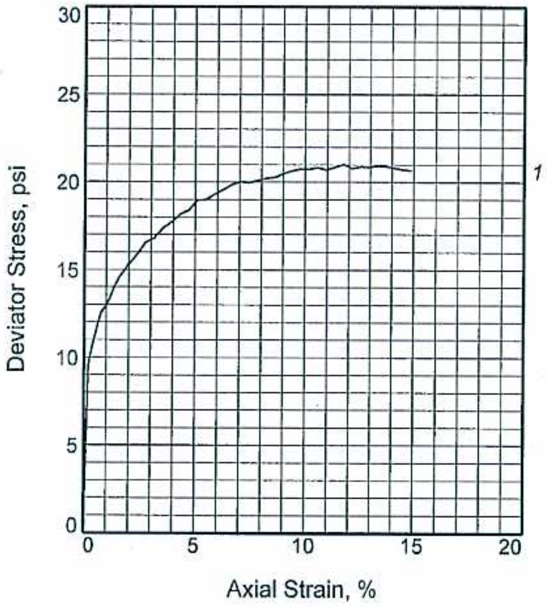
Project No.: 08-288 **Figure** ______

Sierra Testing Labs, Inc.

Tested By: MW **Checked By:** MPW



Results	
C, psi	
ϕ, deg	
Tan(ϕ)	



	Sample No.	1
Initial	Water Content, %	20.7
	Dry Density, pcf	108.8
	Saturation, %	98.7
	Void Ratio	0.5780
	Diameter, in.	2.41
	Height, in.	5.10
At Test	Water Content, %	20.6
	Dry Density, pcf	108.8
	Saturation, %	98.0
	Void Ratio	0.5780
	Diameter, in.	2.41
	Height, in.	5.10
	Strain rate, in./min.	0.03
	Back Pressure, psi	0.0
	Cell Pressure, psi	17.9
	Fail. Stress, psi	21.0
	Ult. Stress, psi	
	σ_1 Failure, psi	38.9
	σ_3 Failure, psi	17.9

Type of Test:
Unconsolidated Undrained

Sample Type: Undisturbed

Description:

Specific Gravity= 2.75

Remarks:

Figure ________

Client: MACTEC

Project: Bishop Ranch Parcel 1A
#4096-08-8527

Location: B-8

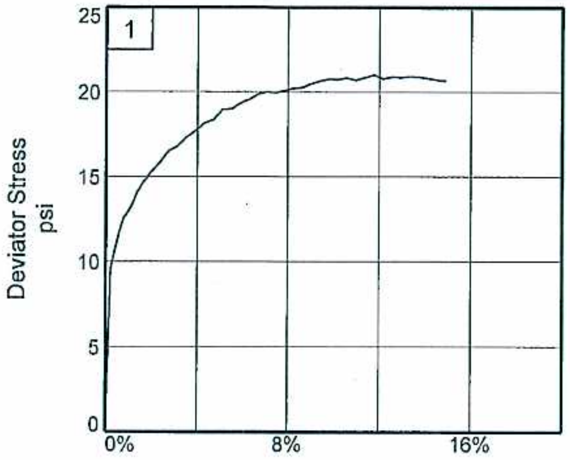
Sample Number: S6494 **Depth:** 30.5-31.0

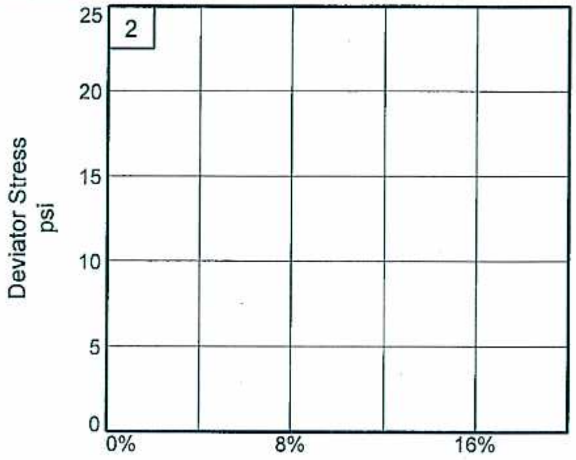
Proj. No.: 08-288 **Date Sampled:** 8/26/08

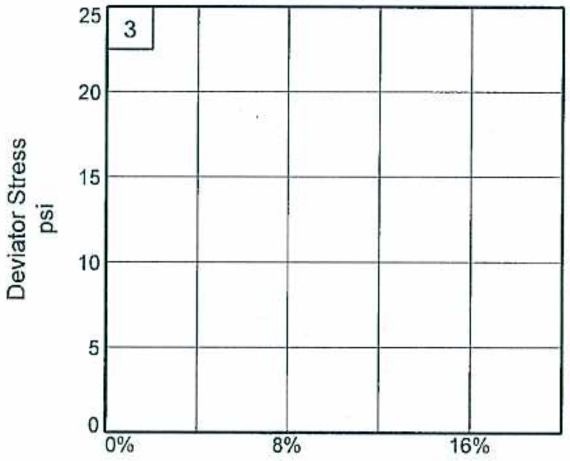
TRIAXIAL SHEAR TEST REPORT

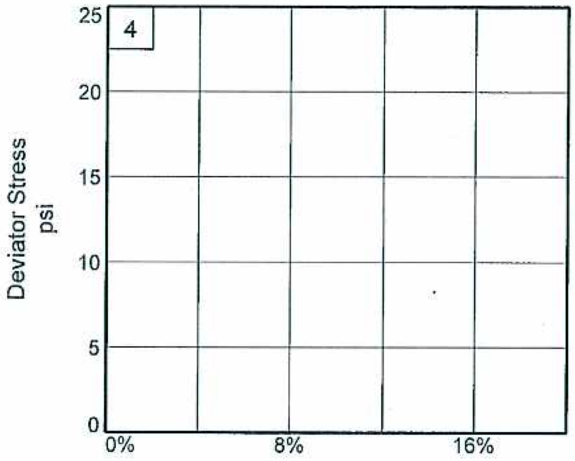
SIERRA TESTING LABS, INC.

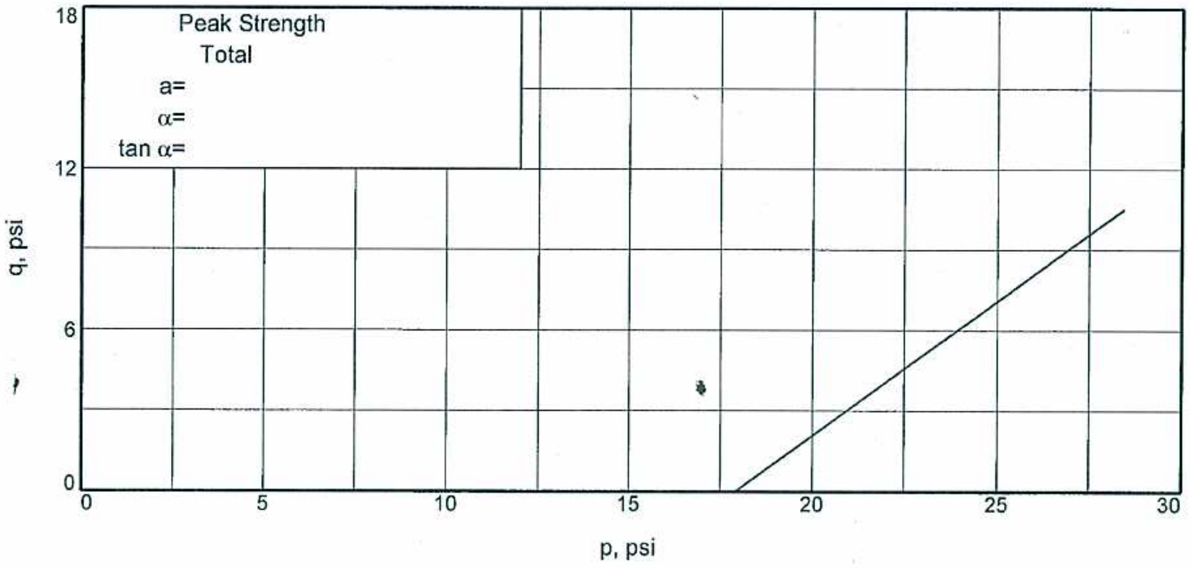
Tested By: MW **Checked By:** MPW







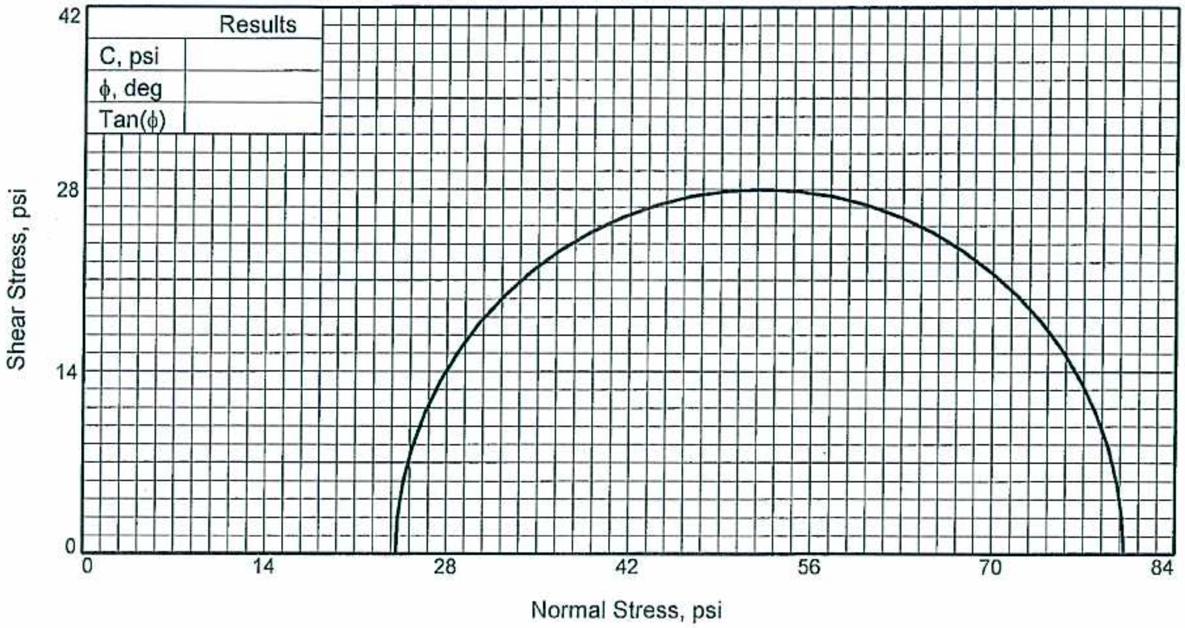


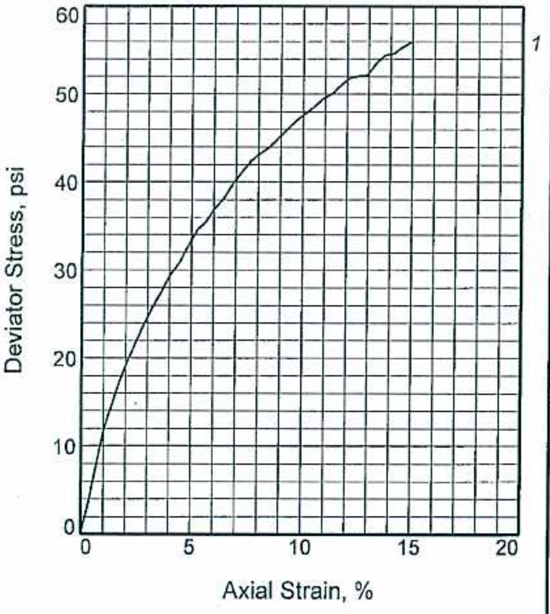


Client: MACTEC
Project: Bishop Ranch Parcel 1A
Location: B-8 **Depth:** 30.5-31.0 **Sample Number:** S6494
Project No.: 08-288 **Figure** _______

Sierra Testing Labs, Inc.

Tested By: MW **Checked By:** MPW





Sample No.		1
Initial	Water Content, %	18.7
	Dry Density, pcf	112.9
	Saturation, %	98.5
	Void Ratio	0.5210
	Diameter, in.	2.41
	Height, in.	4.95
At Test	Water Content, %	18.5
	Dry Density, pcf	112.9
	Saturation, %	97.7
	Void Ratio	0.5210
	Diameter, in.	2.41
	Height, in.	4.95
Strain rate, in./min.		0.03
Back Pressure, psi		0.0
Cell Pressure, psi		24.2
Fail. Stress, psi		55.9
Ult. Stress, psi		
σ_1 Failure, psi		80.1
σ_3 Failure, psi		24.2

Type of Test:
Unconsolidated Undrained

Sample Type: Undisturbed

Description:

Specific Gravity= 2.75

Remarks:

Figure _______

Client: MACTEC

Project: Bishop Ranch Parcel 1A
#4096-08-8527

Location: B-8

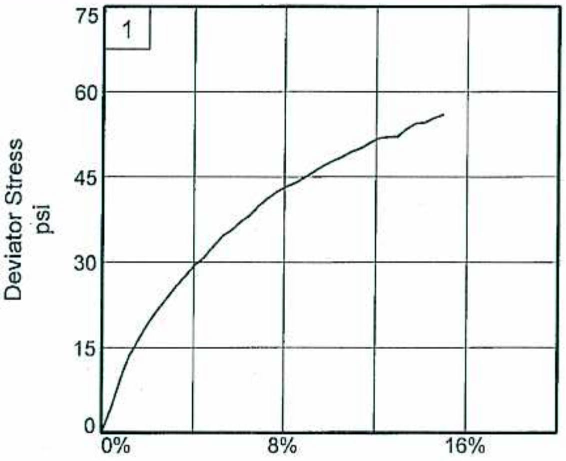
Sample Number: S6495 **Depth:** 45.2-46.0

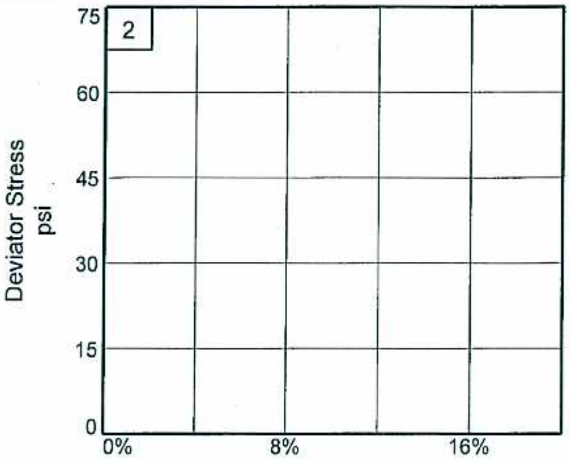
Proj. No.: 08-288 **Date Sampled:** 8/27/08

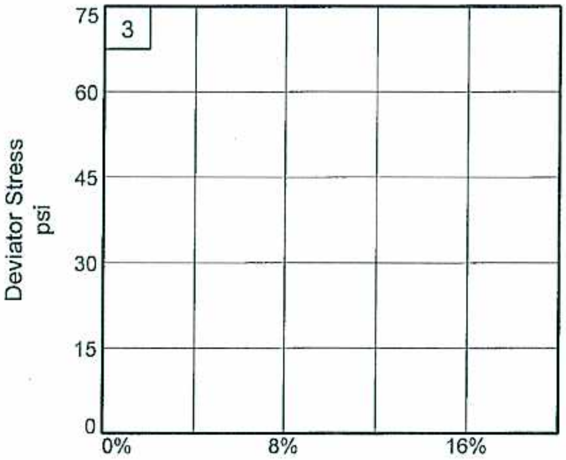
TRIAXIAL SHEAR TEST REPORT

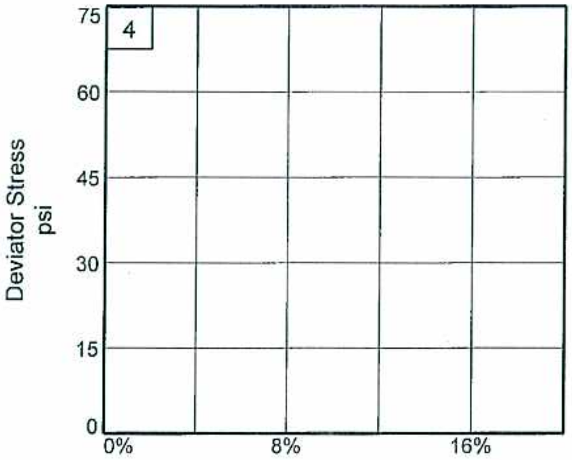
SIERRA TESTING LABS, INC.

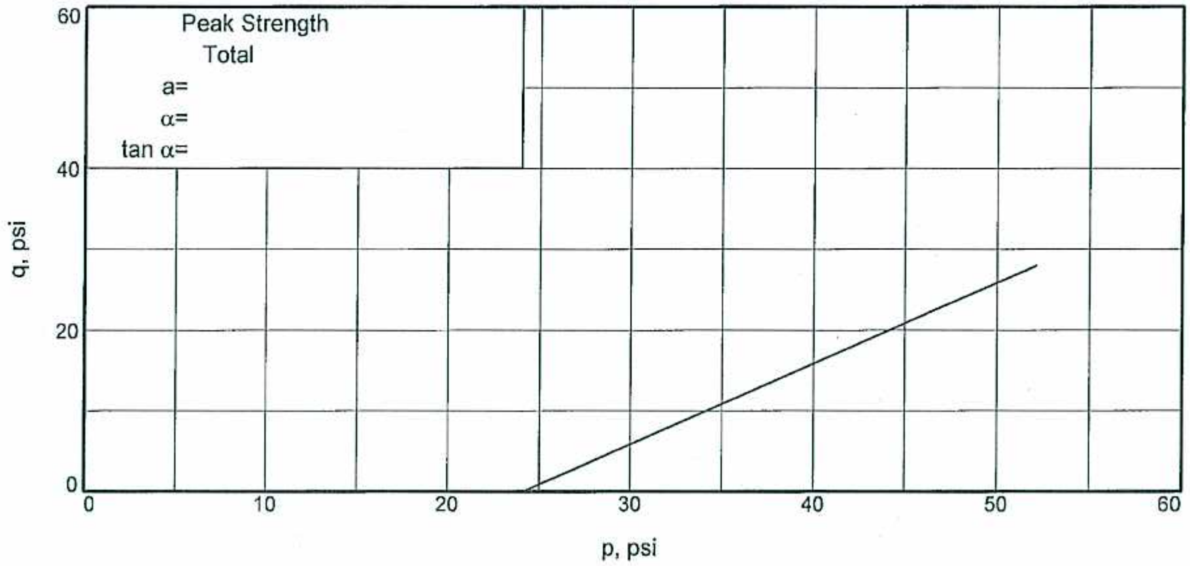
Tested By: MW **Checked By:** MPW











Client: MACTEC

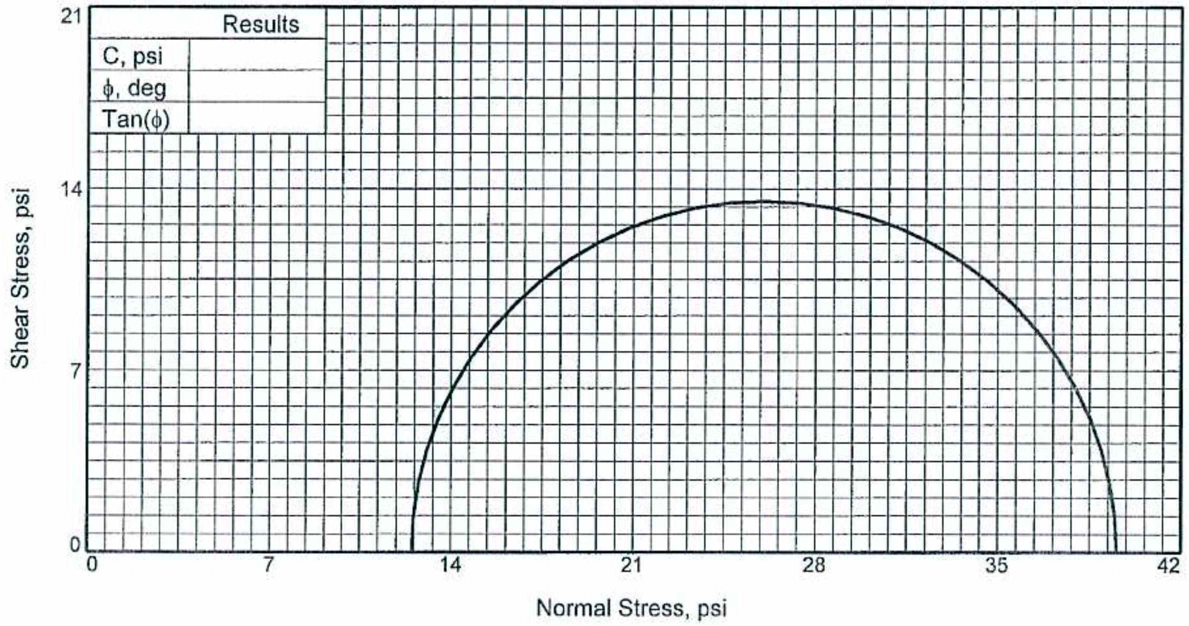
Project: Bishop Ranch Parcel 1A

Location: B-8 **Depth:** 45.2-46.0 **Sample Number:** S6495

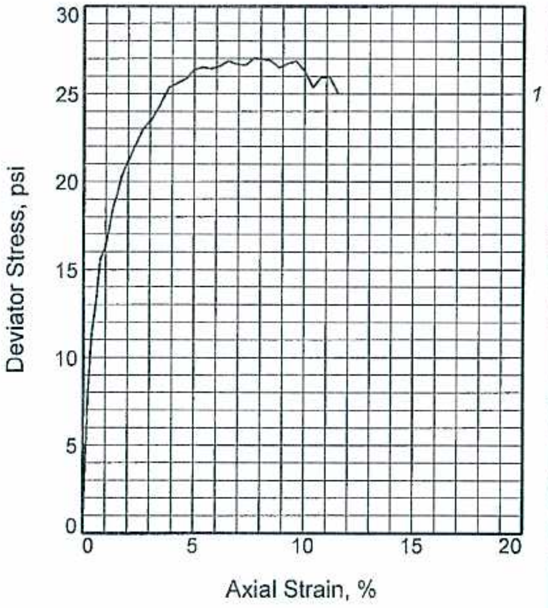
Project No.: 08-288 **Figure** ______

Sierra Testing Labs, Inc.

Tested By: MW **Checked By:** MPW



Results	
C, psi	
ϕ, deg	
Tan(ϕ)	



Sample No.		1
Initial	Water Content, %	32.6
	Dry Density, pcf	89.2
	Saturation, %	98.8
	Void Ratio	0.8904
	Diameter, in.	2.41
	Height, in.	5.20
At Test	Water Content, %	32.4
	Dry Density, pcf	89.2
	Saturation, %	98.3
	Void Ratio	0.8904
	Diameter, in.	2.41
	Height, in.	5.20
Strain rate, in./min.		0.03
Back Pressure, psi		0.0
Cell Pressure, psi		12.5
Fail. Stress, psi		27.0
Ult. Stress, psi		
σ_1 Failure, psi		39.5
σ_3 Failure, psi		12.5

Type of Test:
Unconsolidated Undrained

Sample Type: Undisturbed

Description:

Specific Gravity= 2.70

Remarks:

Figure ________

Client: MACTEC

Project: Bishop Ranch Parcel 1A
#4096-08-8527

Location: B-9

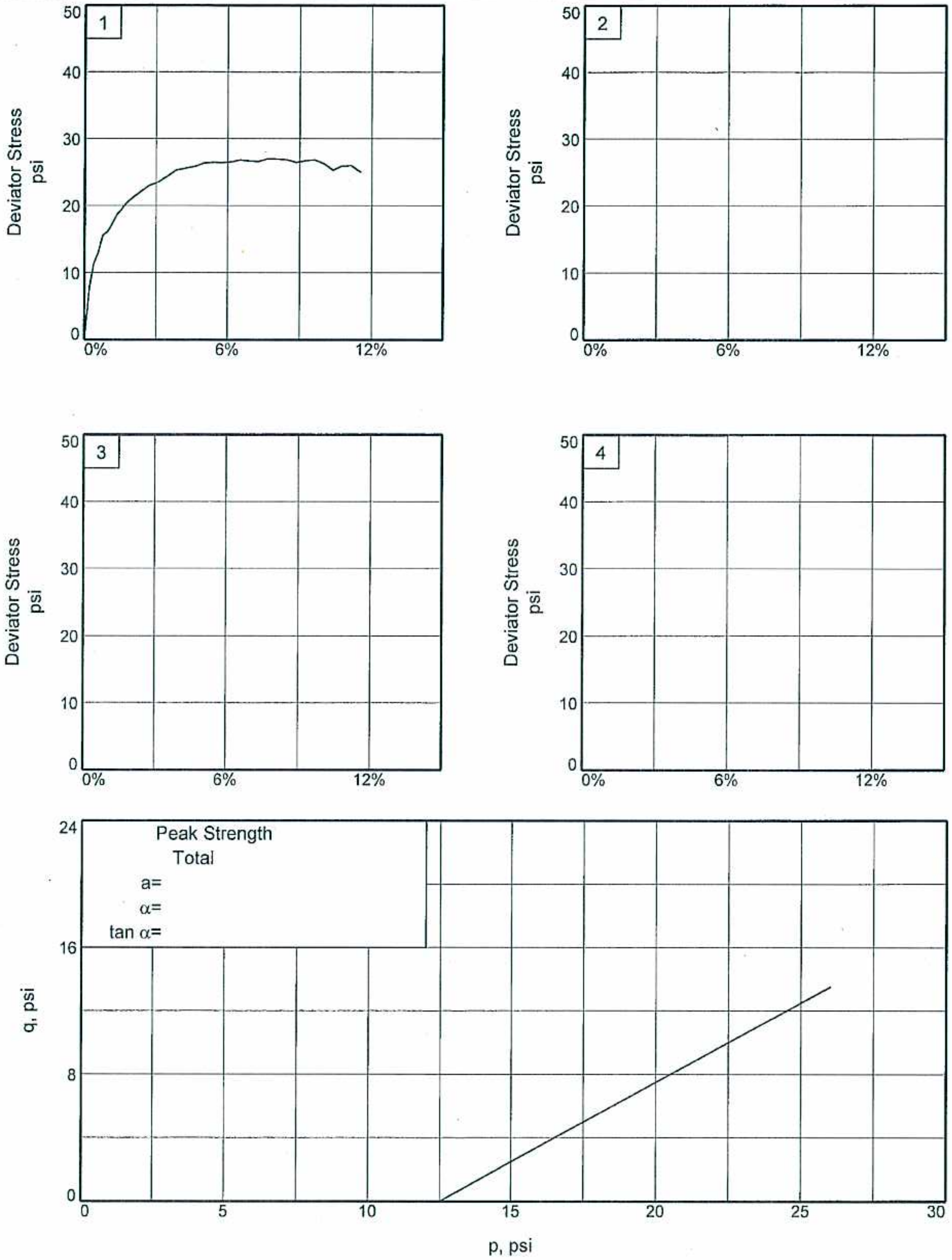
Sample Number: S6499 **Depth:** 15.0

Proj. No.: 08-288 **Date Sampled:** 8/27/08

TRIAXIAL SHEAR TEST REPORT

SIERRA TESTING LABS, INC.

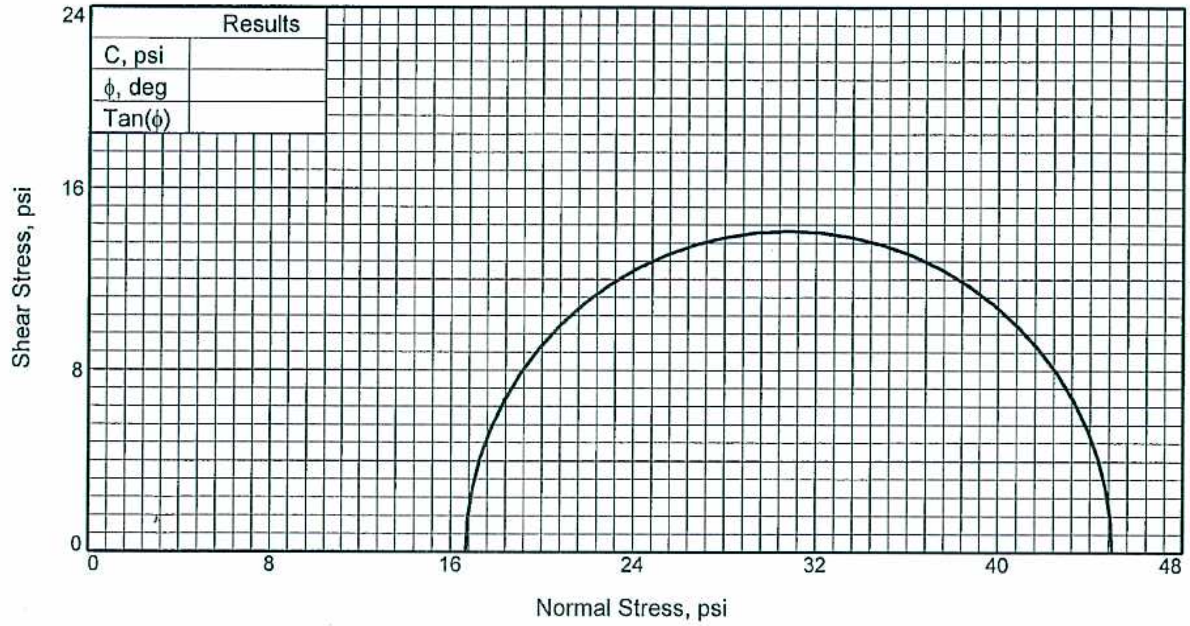
Tested By: MW **Checked By:** MPW

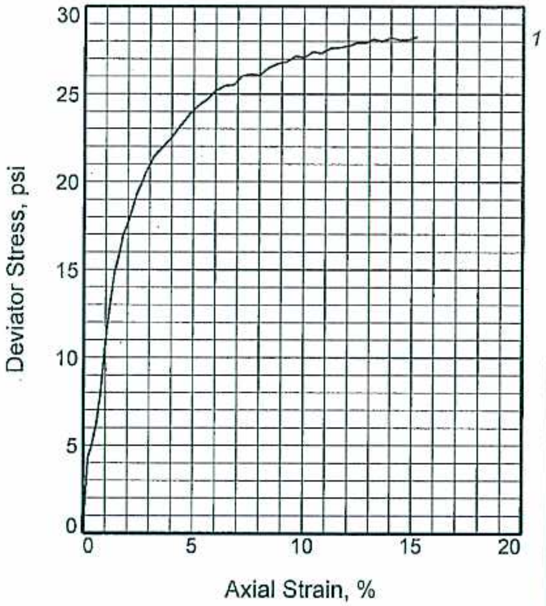


Client: MACTEC
Project: Bishop Ranch Parcel 1A
Location: B-9 **Depth:** 15.0 **Sample Number:** S6499
Project No.: 08-288 **Figure** _______

Sierra Testing Labs, Inc.

Tested By: MW **Checked By:** MPW





	Sample No.	1
Initial	Water Content, %	27.9
	Dry Density, pcf	95.6
	Saturation, %	98.9
	Void Ratio	0.7624
	Diameter, in.	2.41
	Height, in.	5.00
At Test	Water Content, %	27.8
	Dry Density, pcf	95.6
	Saturation, %	98.5
	Void Ratio	0.7624
	Diameter, in.	2.41
	Height, in.	5.00
	Strain rate, in./min.	0.03
	Back Pressure, psi	0.0
	Cell Pressure, psi	16.7
	Fail. Stress, psi	28.3
	Ult. Stress, psi	
	σ_1 Failure, psi	44.9
	σ_3 Failure, psi	16.7

Type of Test:
Unconsolidated Undrained

Sample Type: Undisturbed

Description:

Specific Gravity= 2.70

Remarks:

Figure _______

Client: MACTEC

Project: Bishop Ranch Parcel 1A
#4096-08-8527

Location: B-9

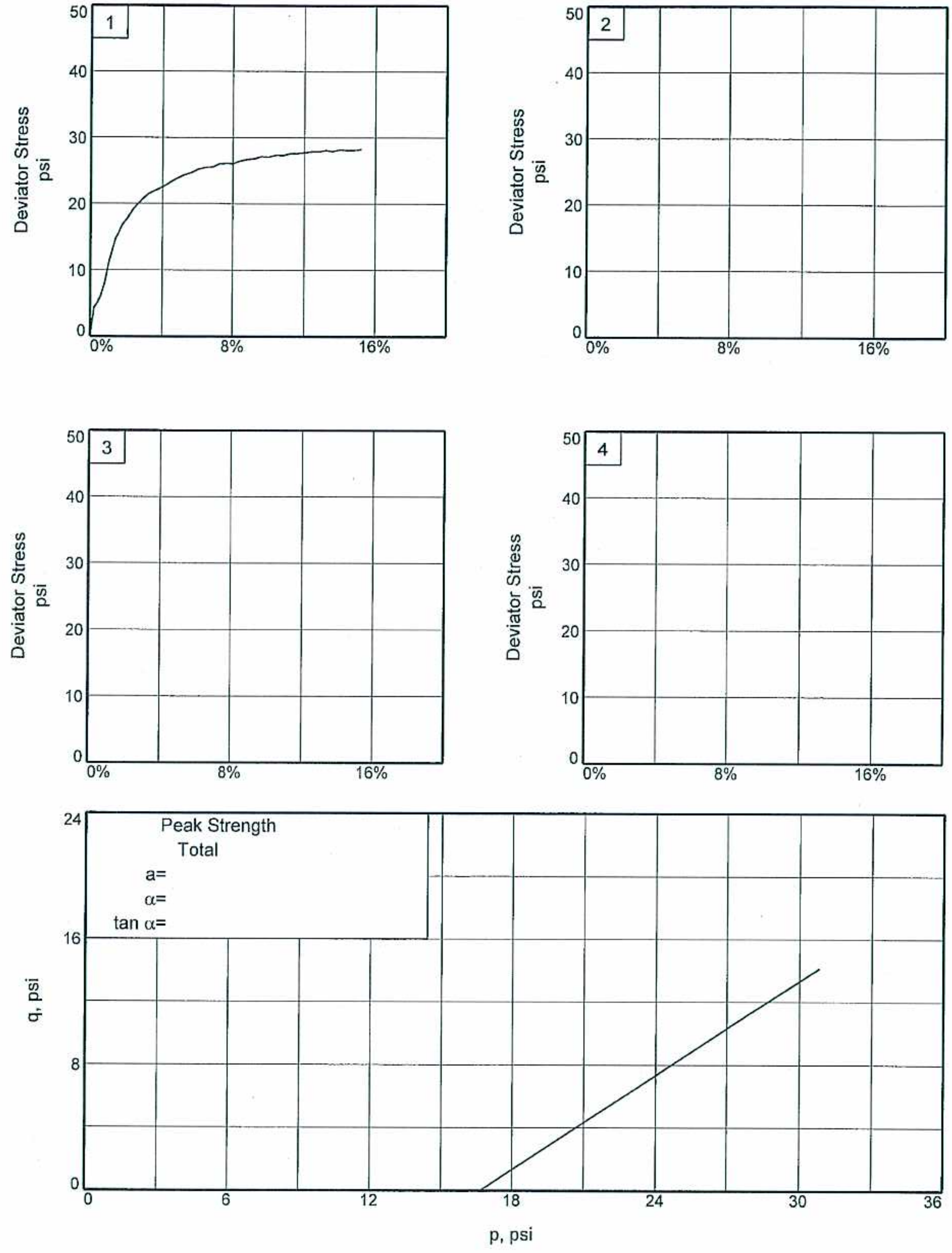
Sample Number: S6501 **Depth:** 25.0

Proj. No.: 08-288 **Date Sampled:** 8/27/08

TRIAXIAL SHEAR TEST REPORT

SIERRA TESTING LABS, INC.

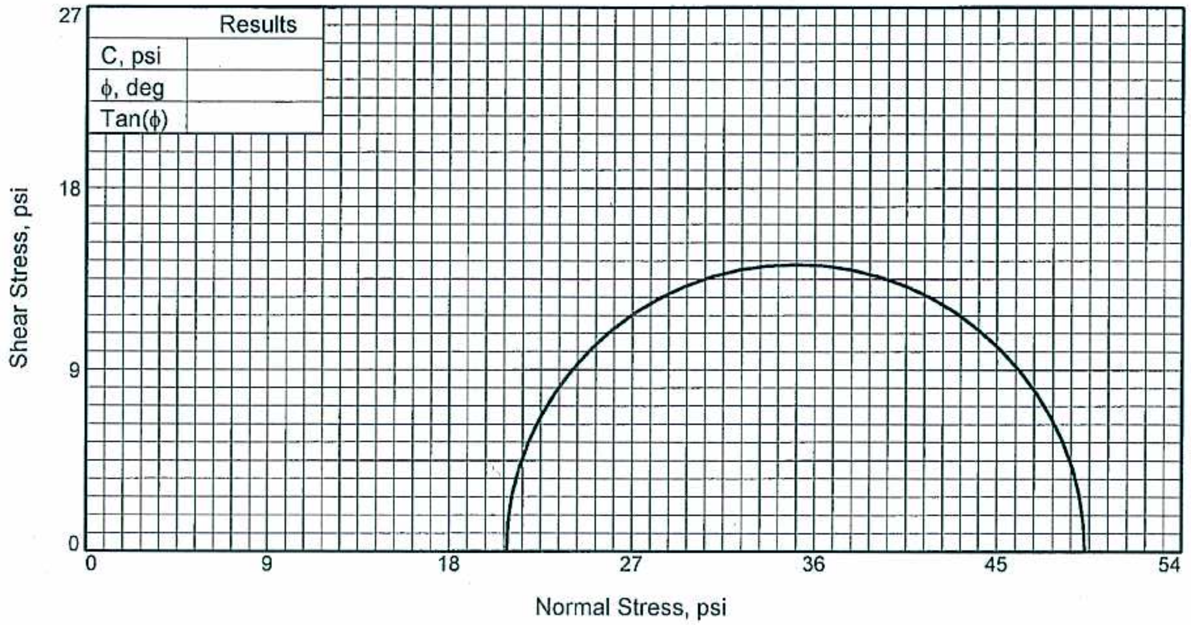
Tested By: MW **Checked By:** MPW

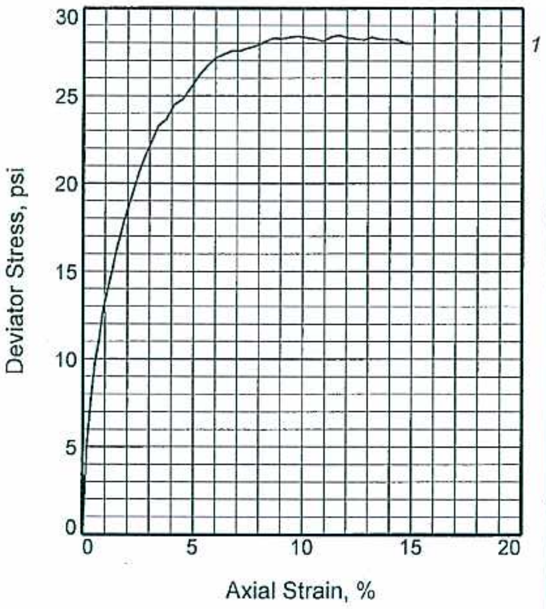


Client: MACTEC
Project: Bishop Ranch Parcel 1A
Location: B-9 **Depth:** 25.0 **Sample Number:** S6501
Project No.: 08-288 **Figure** _______

Sierra Testing Labs, Inc.

Tested By: MW **Checked By:** MPW





Sample No.		1
Initial	Water Content, %	20.2
	Dry Density, pcf	110.1
	Saturation, %	99.2
	Void Ratio	0.5590
	Diameter, in.	2.41
	Height, in.	5.32
At Test	Water Content, %	19.9
	Dry Density, pcf	110.1
	Saturation, %	97.9
	Void Ratio	0.5590
	Diameter, in.	2.41
	Height, in.	5.32
Strain rate, in./min.		0.03
Back Pressure, psi		0.0
Cell Pressure, psi		20.8
Fail. Stress, psi		28.5
Ult. Stress, psi		
σ_1 Failure, psi		49.3
σ_3 Failure, psi		20.8

Type of Test:
Unconsolidated Undrained

Sample Type: Undisturbed

Description:

Specific Gravity= 2.75

Remarks:

Figure ________

Client: MACTEC

Project: Bishop Ranch Parcel 1A
#4096-08-8527

Location: B-9

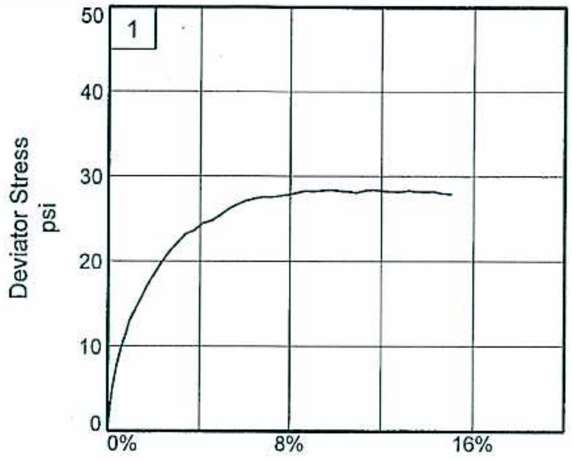
Sample Number: S6503 **Depth:** 35.0

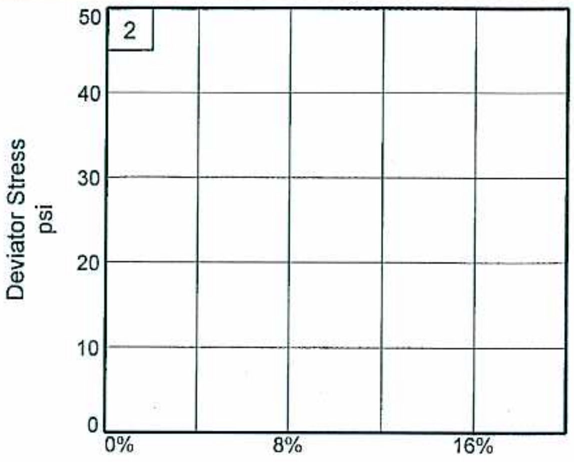
Proj. No.: 08-288 **Date Sampled:** 8/27/08

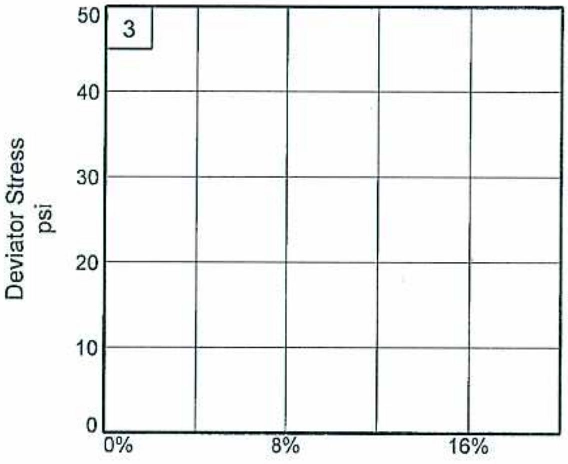
TRIAXIAL SHEAR TEST REPORT

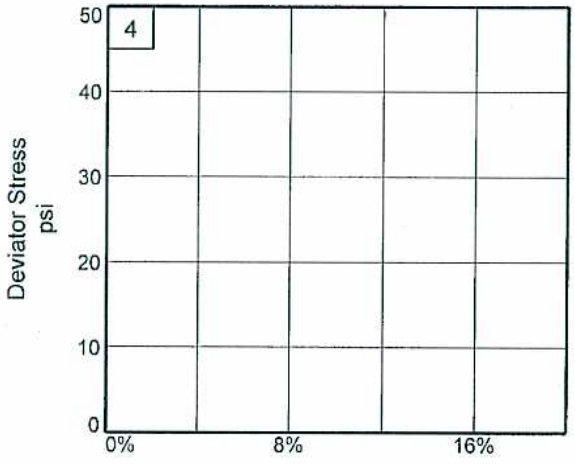
SIERRA TESTING LABS, INC.

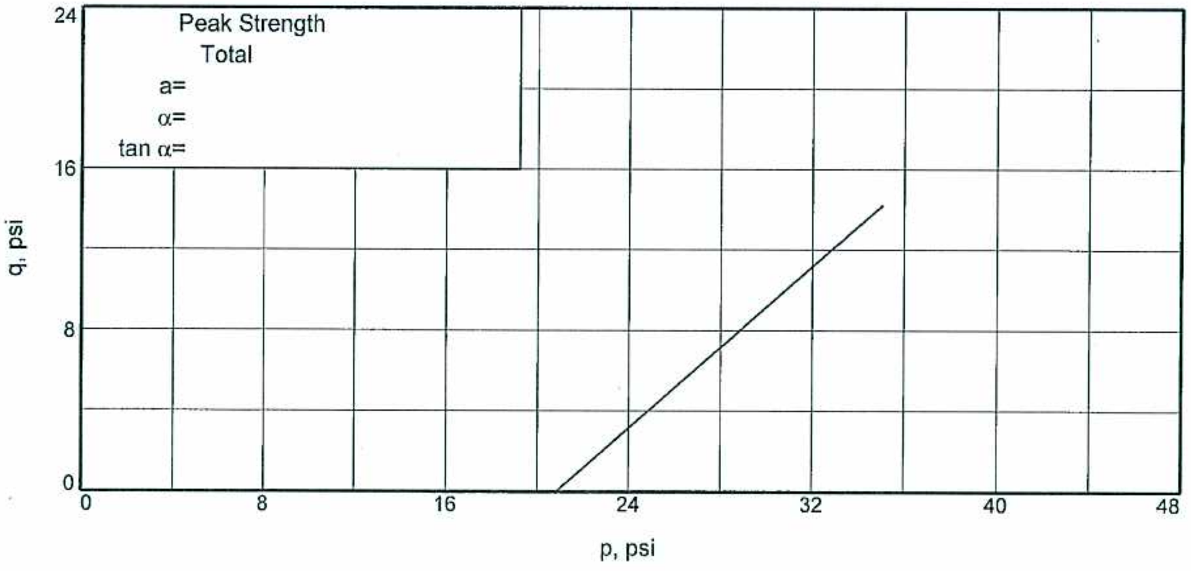
Tested By: MW **Checked By:** MPW









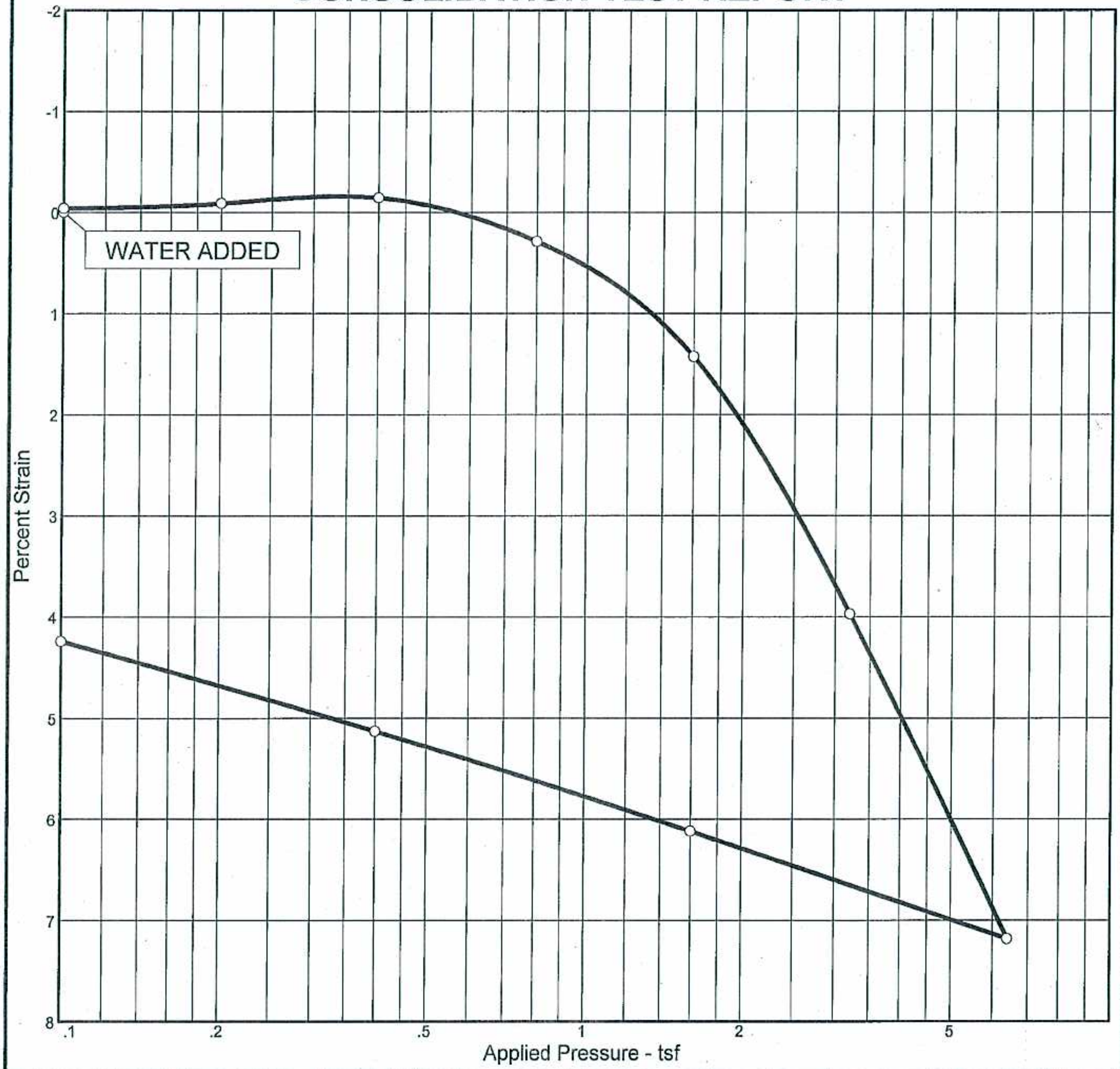


Client: MACTEC
Project: Bishop Ranch Parcel 1A
Location: B-9 **Depth:** 35.0 **Sample Number:** S6503
Project No.: 08-288 **Figure** ______

Sierra Testing Labs, Inc.

Tested By: MW **Checked By:** MPW

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_s	Swell Press. (tsf)	Swell %	e_o
Sat.	Moist.											
71.5 %	18.0 %	100.3	38	14	2.70		1.84	0.18	0.03	0.56		0.681

MATERIAL DESCRIPTION	USCS	AASHTO

Project No. 08-288 **Client:** MACTEC

Project: Bishop Ranch Parcel 1A
#4096-08-8527

Location: B-1

SIERRA TESTING LABS, INC.

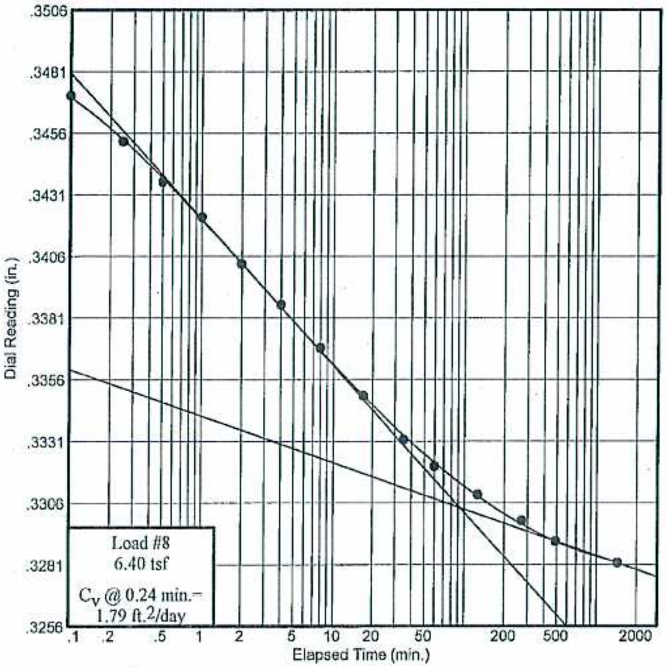
El Dorado Hills, CA

Remarks:

Figure

Dial Reading vs. Time

Project No.: 08-288
Project: Bishop Ranch Parcel 1A
#4096-08-8527
Location: B-1



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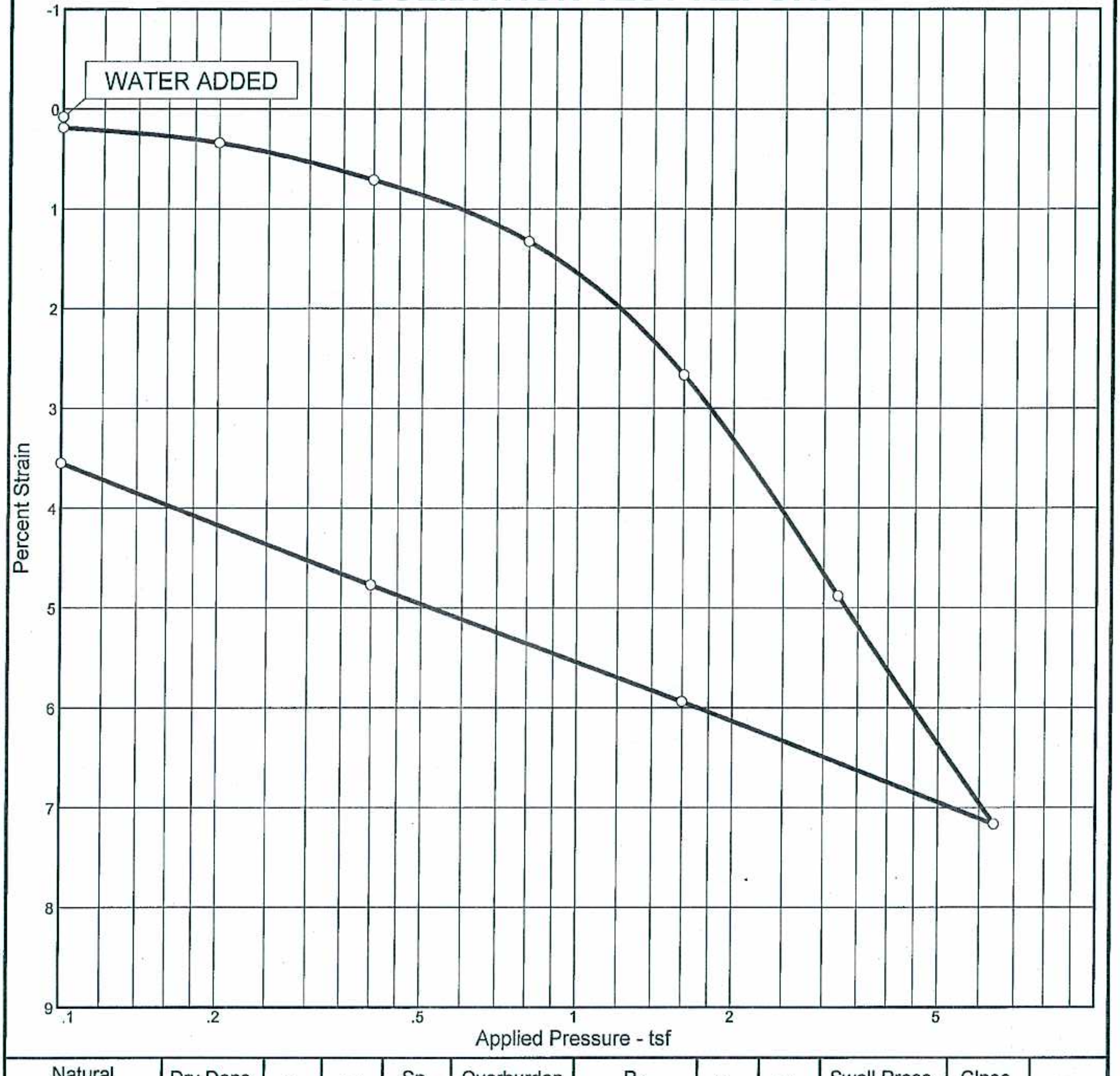
El Dorado Hills, CA

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_s	Swell Press. (tsf)	Clpse. %	e_o
Sat.	Moist.											
94.4 %	33.3 %	86.3	60	28	2.70		1.23	0.15	0.04		0.1	0.952

MATERIAL DESCRIPTION	USCS	AASHTO

Project No. 08-288 **Client:** MACTEC

Project: Bishop Ranch Parcel 1A
#4096-08-8527

Location: B-8

Remarks:

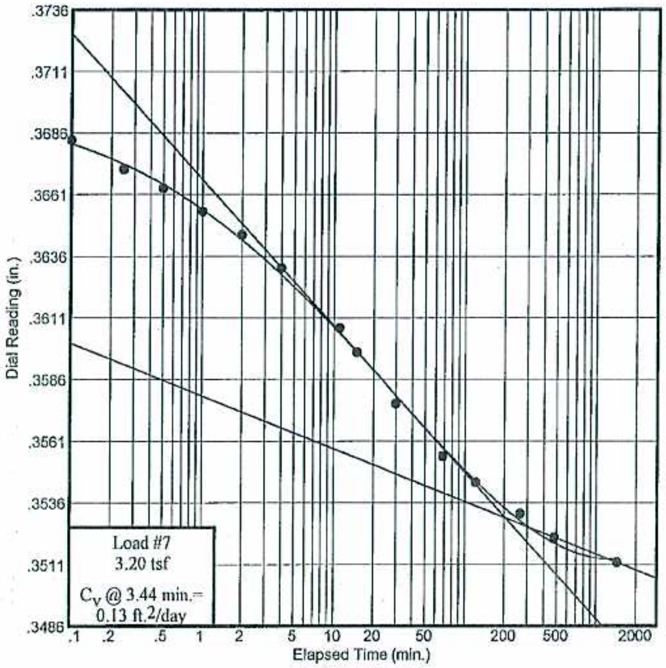
SIERRA TESTING LABS, INC.

El Dorado Hills, CA

Figure

Dial Reading vs. Time

Project No.: 08-288
Project: Bishop Ranch Parcel 1A
#4096-08-8527
Location: B-8



SIERRA TESTING LABS, INC.

El Dorado Hills, CA

Figure

RESISTIVITY & CORROSION PACKAGE
LABORATORY TEST RESULTS

Sample ID.	pH	pH Specifications	Minimum Resistivity, ohm-cm (x1,000)	Specifications Minimum Resistivity, ohm-cm (x1,000)	Chloride, ppm	Sulfate, ppm
B-1 @ 5.0'	8.72	-	0.75	-	29.2	138.6
B-8 @ 5.5'-6.0'	7.20	-	0.83	-	9.4	49.2

Note: Testing performed by Sunland Analytical, Rancho Cordova, CA

Test Method: Cal test #643, #417 &#422

REMARKS:

LAB NUMBER: S6479 & S6490

PROJECT NUMBER:	**08-152**	March 14, 2008

Bishop Ranch Parcel 1A
Job #4096-08-8527

SIERRA TESTING LABORATORIES, INC.
GEOTECHNICAL AND MATERIALS TESTING SERVICES

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