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ENGINEERING GROUP

GEOTECHNICAL INVESTIGATION
STATION AVENUE MIXED-USE DEVELOPMENT
6400 STATE FARM DRIVE
ROHNERT PARK, CALIFORNIA

September 28, 2018

Job No. 2647.001

Prepared for:
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CERTIFICATION

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EXECUTIVE SUMMARY

Miller Pacific Engineering Group was retained by Laulima Rohnert Station, LLC to perform a geotechnical investigation for the planned Station Avenue mixed-use development located at 6400 State Farm Drive in Rohnert Park, California. We understand that project details are not yet fully-developed; however, the project generally would consist of developing 30 acres with 270 apartments, 445 residential units, approximately 130,000 square feet of office and 120,000 square feet of retail. No significant below-grade structures are currently anticipated. Ancillary improvements are expected to include extensive exterior hardscape/flatwork and asphalt paving, new underground utilities, new site drainage, landscaping, lighting, and other improvements "typical" of such developments. No structural information is available at this time; however, we understand the structures will likely be a maximum of five stories and, as such, are anticipated to impose moderate to heavy foundation loads.

The purpose of our investigation was to explore subsurface conditions within the proposed development area, and to develop geotechnical criteria for planning, design and construction of the proposed improvements. Based upon the site reconnaissance, subsurface exploration and other information detailed within this report, we conclude the site conditions are suitable for the proposed improvements from a geotechnical standpoint. Primary geotechnical considerations for the project include the following:

- The geotechnical investigation included six cone penetration tests and eight borings to evaluate subsurface conditions. Based on the exploration and mapped geology, the project site is located within Holocene age alluvial fan deposits and is underlain with variable soils to depths in excess of 100 feet. In general, subsurface conditions are primarily medium stiff to very stiff clays with interbedded lenses of medium dense to dense clayey and silty sands. Groundwater is relatively shallow and is estimated to be between 6 and 14 feet below the ground surface.
- The site is near several active faults and will likely experience strong seismic shaking associated with future earthquakes. The improvements will need to be designed to resist relatively high seismic loads in accordance with provisions of the 2016 California Building Code or other codes in effect when final design occurs.
- Near surface soils exhibit high plasticity and high expansion potential. Lime treatment is recommended to a depth of 36 inches below buildings and 18 inches below pavement and flatwork areas to reduce the expansion potential of the surficial soils.
- Building settlements could occur due to the consolidation of underlying medium stiff clay layers and liquefaction of sandy soils. More detailed settlement analyses (based on laboratory testing of site soils and planned building loads) should be performed once building layouts and structural loads are available.
- Based on the subsurface conditions and available building information, foundation options include either shallow foundations provided potential total and differential settlement are acceptable. Alternatively, a deep foundation system that extends into stiff clays and dense sands is appropriate for the site conditions and expected building loads. With deep foundations, static and seismic settlements would be minimized.

These and other considerations are discussed in greater detail in the remainder of this report. This report should be reviewed in its entirety when making decisions concerning the findings of this investigation.

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1.0 INTRODUCTION

This report presents the results of our design-level Geotechnical Investigation for the planned Station Avenue mixed-use development. As shown on Figure 1, the project site is located within an approximately 30-acre parcel (APN 143-051-072) at 6400 State Farm Drive in in Rohnert Park, California.

Our work was performed in accordance with Phase 2 of our Agreement for Professional Services dated April 13, 2018. We previously prepared a Preliminary Geotechnical Investigation for the site which provided preliminary geotechnical criteria for planning and design (Miller Pacific, 2018). The purpose of our design-level investigation was to explore subsurface conditions and to develop geotechnical criteria for design and construction of the proposed improvements. The scope of our services includes:

- Review of available, published geologic mapping and geotechnical background information from our files, the City of Rohnert Park, and any geologic/geotechnical background information supplied by you.
- Review of aerial photographs to evaluate previous site development.
- Subsurface exploration consisting of six cone penetration tests and eight test borings located within the general vicinity of the planned improvements.
- Evaluating relevant geologic hazards including seismic shaking, expansive soils, liquefaction, settlement, and other hazards.
- Engineering analyses to develop design-level geotechnical recommendations and design criteria related to foundations, site grading, seismic design and other geotechnical-related items.
- Preparing a Geotechnical Investigation report which summarizes the subsurface exploration, evaluation of relevant geologic hazards, and geotechnical recommendations and design criteria.

This report completes our Phase 2 services for the project. Subsequent phases of work should include geotechnical plan review and observation and testing of geotechnical-related work items during construction.

2.0 PROJECT DESCRIPTION

While planning and preliminary design are still underway, we understand the project is expected to consist of a mixed-use, residential/commercial development consisting of 28 new buildings and related improvements. Based on our review of preliminary drawings, the southern portion of the site will include several buildings for multi-family residential use while new structures on the northern portion of the site will provide office, retail and hotel space. While detailed structural information is not yet available, the new structures will be up to five stories in height and will likely induce moderate to heavy foundation loads. A new, four-story parking structure is also planned near the southeast portion of the site and will provide parking for the surrounding residences. No significant below-grade structures are currently anticipated.

Ancillary improvements will include a central courtyard area with associated landscaping and exterior hardscape, new underground utilities, pavements for new roads and parking areas, site drainage and other improvements typical of such developments. A new park and community area are also planned along the south side of the development. Site grading is expected to include cuts and fills of a few feet as required to achieve the design grades for the new roadways, parking areas, building pads and other improvements. The proposed improvements are shown on the Site Plan, Figure 2.

3.0 SITE CONDITIONS

3.1 Regional Geology

The project site is located within the Coast Ranges geomorphic province of California. It is typified by generally northwest-trending ridges and intervening valleys that formed as a result of movement along a group of northwest-trending fault systems, including the San Andreas Fault. Bedrock geology within the San Francisco bay area is dominated by sedimentary, igneous, and metamorphic rocks of the Jurassic-Cretaceous age Franciscan Complex. Most of Franciscan rock types are composed of sandstone and pervasively sheared shale. It also includes less common rocks such as chert, serpentinite, basalt, greenstone, and exotic low- to high-grade metamorphic rocks, including phyllite, schist, and eclogite.

The project site is located within level terrain in central Rohnert Park, just east of Highway 101 and north of Copeland Creek. Regional geologic mapping by the California Geological Survey (CGS, 2003) indicates the site is underlain by Holocene-age alluvial fan deposits (map symbol Qhff). Alluvial deposits generally are described as unconsolidated soils deposited by fluvial (water) processes. Soils within this unit are predominantly clayey with interbedded lenses of silt, sand and gravel. A Regional Geologic Map and descriptions of the mapped geologic units are presented on Figure 3.

3.2 Seismicity

The project site is located within the seismically active San Francisco Bay Area and will therefore experience the effects of future earthquakes. Earthquakes are the product of the build-up and sudden release of strain along a “fault” or zone of weakness in the earth's crust. Stored energy may be released as soon as it is generated, or it may be accumulated and stored for long periods of time. Individual releases may be so small that they are detected only by sensitive instruments, or they may be violent enough to cause destruction over vast areas.

Faults are seldom single cracks in the earth's crust but are typically composed of localized shear zones which link together to form larger fault zones. Within the Bay Area, faults are concentrated along the San Andreas Fault zone. The movement between rock formations along either side of a fault may be horizontal, vertical, or a combination, and is radiated outward in the form of energy waves. The amplitude and frequency of earthquake ground motions partially depends on the material through which it is moving. The earthquake force is transmitted through hard rock in short, rapid vibrations, while this energy becomes a long, high-amplitude motion when moving through soft ground materials, such as Bay Mud.

3.2.1 Regional Active Faults

The California Geological Survey (previously known as the California Division of Mines and Geology) defines a "Holocene-active fault" as one that had surface displacement within Holocene time (the last 11,700 years). CGS further defines a "pre-Holocene fault" as a fault whose recency of past movement is older than 11,700 years. Similarly, an "age-undetermined fault" is defined as a fault whose age of most recent movement is not known or is unconstrained by dating methods or limitations in stratigraphic resolution. CGS has mapped various faults in the region as part of their Fault Activity Map of California (CGS, 2010). Many of these faults are shown in relation to the project site on the attached Active Fault Map, Figure 4.

The nearest known Holocene-active fault is the Rodgers Creek Fault located approximately 5.6 kilometers (3.5 miles) to the northeast. Mapping by the California Geological Survey also shows the Tolay Fault located about 3.2 kilometers (2.0 miles) south of the site. The Tolay Fault is characterized as a Quaternary fault; however, the age of displacements along the fault zone are undifferentiated.

3.2.2 Historic Fault Activity

Numerous earthquakes have occurred in the region within historic times. The results of our USGS earthquake search catalogue indicates that at least five earthquakes with a Richter Magnitude of 5.0 or larger have occurred within 100 kilometers (62 miles) of the site between 1900 and 2018. The approximate locations of many of these earthquakes are shown on the Historic Earthquake Map, Figure 5.

3.2.3 Probability of Future Earthquakes

The site will likely experience moderate to strong ground shaking from future earthquakes originating on any of several active faults in the San Francisco Bay region. The historical records do not directly indicate either the maximum credible earthquake or the probability of such a future event. To evaluate earthquake probabilities in California, the USGS has assembled a group of researchers into the "Working Group on California Earthquake Probabilities" (USGS 2003, 2008, 2013) to estimate the probabilities of earthquakes on active faults. These studies have been published cooperatively by the USGS, CGS, and Southern California Earthquake Center (SCEC) as the Uniform California Earthquake Rupture Forecast, Versions 1, 2, and 3. In these studies, potential seismic sources were analyzed considering fault geometry, geologic slip rates, geodetic strain rates, historic activity, micro-seismicity, and other factors to arrive at estimates of earthquakes of various magnitudes on a variety of faults in California.

Conclusions from the most recent UCERF3 and USGS indicate the highest probability of an earthquake with a magnitude greater than 6.7 originating on any of the active faults in the San Francisco Bay region by 2043 is assigned to the Hayward/Rodgers Creek Fault system. The Rodgers Creek Fault is located approximately 5.6 kilometers (3.5 miles) northeast of the site and is assigned a probability of 33 percent. The San Andreas Fault, located approximately 26.6 kilometers (16.5 miles) west of the site, is assigned a 22 percent probability of an earthquake with a magnitude greater than 6.7 by 2043. Additional studies by the USGS regarding the probability of large earthquakes in the Bay Area are ongoing. These current evaluations include data from additional active faults and updated geological data.

3.3 Site History

In addition to available geotechnical and geologic reference information, we reviewed several historic aerial photographs and topographic maps to assess site history. Aerial photographs were taken from Google Earth aerial imagery. The photographs were taken between 1993 and 2018 and show that the site has experienced little to no significant changes during that time. Based on reference data, the site was a hayfield in 1976 and was developed for State Farm in the late 1970's.

3.4 Surface Conditions

The project site encompasses an approximately 30-acre parcel located about a quarter-mile east of Highway 101. The site is bounded to the east by the Rohnert Park SMART station and associated parking lot, to the south by Enterprise Drive, to the west by State Farm Drive and to the north by Rohnert Park Expressway. The former, approximately 300,000-square foot State Farm Insurance office building exists toward the center of the property and the areas surrounding the existing structure were developed as asphalt-paved parking areas and driveways. The southeast corner of the site is currently occupied by the City of Rohnert Park's Corporation Yard which consists of two single-story buildings surrounded by paved driveways and parking areas.

The ground surface is level to gently sloping with surface elevations ranging from about 100 to 105 feet[1]. The northern portion of the site is largely undeveloped and is landscaped with a few mature trees and low grasses. Much of the site is secured by fencing and vehicle access is provided by driveways located on the west and south sides of the property. Existing surface conditions throughout the property and surrounding areas are shown on Figure 6.

3.5 Reference Geotechnical Data

Several subsurface explorations have been conducted by Miller Pacific and other Consultants as part of the original and current site development or other nearby projects. Prior to completing our subsurface exploration, we reviewed the following reports:

> Harding-Lawson Associates, *Geologic/Seismic Hazards Study and Soil Investigation, State Farm Insurance Company, Northern California Office, Rohnert Park, California, Commission Number 5521-622D*, March 8, 1976.
>
> Miller Pacific Engineering Group, *Preliminary Geotechnical Investigation, Rohnert Station Mixed-Use Development, 6400 State Farm Drive, Rohnert Park, California*, June 19, 2018.

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[1] Surface elevations based on Google Earth Aerial imagery accessed on September 7, 2018.

Moore & Taber, *Preliminary Soils Investigation, Northbay Savings and Loan Site, Rohnert Park, California*, September 20, 1977.

Winzler & Kelly, *Groundwater Monitoring Well Installation and Fourth Quarter 2007 Groundwater Monitoring Report, Rohnert Park Corporation Yard, 600 Enterprise Drive, Rohnert Park, California, CSDHS Case #0001092, NCRWQCB*, January 31, 2008.

3.6 Field Exploration and Laboratory Testing

As part of our preliminary investigation, we previously explored subsurface conditions near the proposed improvements with six CPTs at the approximate locations shown on Figure 6. The CPTs were advanced to depths ranging from about 65 to 115 feet below ground surface on March 29, 2018. Cone penetration testing is an exploration technique that provides a continuous profile of data throughout the depth of exploration. CPTs are particularly useful in defining stratigraphy, relative soil strength and in assessing liquefaction potential. This test method has been standardized and is described in detail by the ASTM D3441. The method generally consists of pushing an instrumented cone-tipped probe through the ground at a controlled rate and measuring tip resistance (at the front of the cone) and frictional resistance (along the sides of the cone). The data is processed by a computer to estimate engineering properties such as soil type, relative density, and strength. A CPT Soil Interpretation Chart and CPT plots of interpreted subsurface conditions from our previous exploration are shown in Appendix A, Figures A-1 to A-7.

We explored subsurface conditions near the proposed improvements as part of this design-level investigation on August 8, 9 and 10, 2018 with eight borings at the approximate locations shown on Figure 6. The borings were excavated using truck-mounted drilling equipment to approximate depths ranging from 3.5 to 51.5 feet below ground surface. The borings were logged by our Field Engineer and samples were obtained for classification and laboratory testing. We prepared boring logs based on soil descriptions in the field, as well as visual examination and testing of the soil samples in our laboratory. The boring logs from our design-level field investigation are presented in Appendix A, Figures A-8 to A-20.

Laboratory testing of soil samples from the exploratory borings included determination of moisture content, dry density, Atterberg limits, expansion index, unconfined compressive strength, soil-lime proportioning, gradation, and the amount of material passing a No. 200 sieve. The results of our laboratory tests are presented on the boring logs with the exception of the sieve analyses, Atterberg limits, soil-lime proportioning and expansion index and which are presented on Figures A-21 through A-25. Our laboratory testing program is discussed in greater detail in Appendix A.

The approximate locations of the nearby borings and monitoring wells from the previous geotechnical investigations noted in Section 3.5 are shown on the Exploration Plan, Figure 6. The boring logs and monitoring well data from the previous investigations are included in Appendix B.

3.7 Subsurface Conditions

The interpreted subsurface conditions encountered are generally consistent with the mapped geologic conditions at the site (CGS, 2003). Based on our subsurface exploration and reference data, the site is generally underlain by deep alluvial deposits composed of primarily medium stiff to very stiff clayey soils containing variable amounts of silt, sand and gravel. Several layers of medium dense to dense sandy soils were also encountered in the CPTs and borings generally between 10 to 15 feet below ground surface. The thickness of these sandy soil layers varied from about three to four feet.

The near-surface soils generally consist of dark gray to black, medium stiff to stiff clay. The clay is locally known as adobe and exhibits high plasticity and very high expansion potential. Borings 3, 5 and 6, located close to the existing building, encountered three to six feet of sandy fill soils which were presumably placed beneath the new buildings during the original site development to mitigate expansive soil conditions beneath the existing structures.

3.8 Groundwater

Groundwater was encountered at Borings 1 and 8 at approximately 13 feet below ground surface. Because the borings were not left open for an extended period of time, a stabilized depth to groundwater may not have been observed. Groundwater elevations fluctuate seasonally, and higher groundwater levels may be present during and after periods of intense and/or sustained rainfall.

A cursory search of the State Water Resources Control Board’s Geotracker website indicates that groundwater monitoring wells were installed as part of previous environmental studies near the southeast corner of the site (Winzler and Kelly, 2008). The monitoring data from these studies was collected from 1987 to 2004 and indicates the groundwater level varied from about 6 to 24 feet below ground surface. Therefore, for analysis purposes, we assumed that groundwater is located at five feet below the ground surface.

4.0 GEOLOGIC HAZARDS

This section summarizes our review of commonly considered geologic hazards and discusses their potential impacts on the planned improvements. The primary geologic hazards which could affect the proposed development include strong seismic ground shaking, expansive soils, liquefaction, and seismic densification. Other geologic hazards are judged less than significant regarding the proposed project. Geologic hazards, potential impacts and mitigation measures are discussed in further detail in the following sections.

4.1 Fault Surface Rupture

Under the Alquist-Priolo Earthquake Fault Zoning Act, the California Division of Mines and Geology (now known as the California Geological Survey) produced 1:24,000 scale maps showing known active and potentially active faults and defining zones within which special fault studies are required. The nearest known active fault to the site is the Rodgers Creek Fault located approximately 5.6 kilometers (3.5 miles) to the northeast. The site is not located within an Alquist-Priolo Special Studies Zone. We therefore judge the potential for fault surface rupture in the development area to be low.

Evaluation: *Less than significant.*
Mitigation: *No mitigation measures are required.*

4.2 Seismic Shaking

The site will likely experience seismic ground shaking similar to other areas in the seismically active Bay Area. The intensity of ground shaking will depend on the characteristics of the causative fault, distance from the fault, the earthquake magnitude and duration, and site-specific geologic conditions. Estimates of peak ground accelerations are based on either deterministic or probabilistic methods.

Deterministic methods use empirical attenuation relations that provide approximate estimates of median peak ground accelerations. A summary of the active faults that could most significantly affect the planning area, their maximum credible magnitude, closest distance to the center of the planning area, and probable peak ground accelerations are summarized in Table 1. The calculated accelerations should only be considered as reasonable estimates. Many factors (e.g., soil conditions, orientation to the fault, etc.) can influence the actual ground surface accelerations.

Table 1 – Deterministic Peak Ground Accelerations for Active Faults

Fault	Moment Magnitude for Characteristic Earthquake	Closest Estimated Distance (km)	Median Peak Ground Acceleration (g)	Median PGA +1 Std Dev (g)
Rodgers Creek	7.3	5.6	0.37	0.61
San Andreas	8.0	26.6	0.22	0.36
Maacama	7.4	20.7	0.21	0.34
Hayward	7.3	44.1	0.12	0.20
West Napa	6.6	30.4	0.11	0.19

Reference: Abrahamson & Silva, Boore & Atkinson, Campbell & Bozorgnia, and Chiou & Youngs 2008 NGA models using V_{s30} = 270 m/s.

Probabilistic Seismic Hazard Analysis analyzes all possible earthquake scenarios while incorporating the probability of each individual event to occur. The probability is determined in the form of the recurrence interval, which is the average time for a specific earthquake acceleration to be exceeded. The design earthquake is not solely dependent on the fault with the closest distance to the site and/or the largest magnitude, but rather the probability of given seismic events occurring on both known and unknown faults.

We calculated the peak ground acceleration for two separate probabilistic conditions, including the two percent chance of exceedance in 50 years (2,475-year statistical return period) and the ten percent chance of exceedance in 50 years (475-year statistical return period). The peak ground acceleration values were calculated utilizing the USGS Unified Hazard Tool. The results of the probabilistic analyses are presented below in Table 2.

Table 2 – Probabilistic Peak Ground Accelerations for Active Faults

Probability of Exceedance	Statistical Return Period	Magnitude	Peak Ground Acceleration (g)
2% in 50 years	2,475 years	7.0	0.77
10% in 50 years	475 years	7.0	0.47

Reference: USGS Unified Hazard Tool accessed on June 14, 2018.

Ground shaking can result in structural failure and collapse of structures or cause non-structural building elements (such as light fixtures, shelves, cornices, etc.) to fall, presenting a hazard to building occupants and contents. Compliance with provisions of the most recent version of the California Building Code (2016 CBC) should result in structures that do not collapse in an earthquake. Damage may still occur, and hazards associated with falling objects or non-structural building elements will remain.

The potential for strong seismic shaking at the project site is high. Due to their proximity and historic rates of activity, the Rodgers Creek, San Andreas and Maacama Faults present the highest potential for severe ground shaking. The significant adverse impact associated with strong seismic shaking is potential damage to structures and improvements.

Evaluation: *Less than significant with mitigation.*

Mitigation: *Minimum mitigation includes design of new structures in accordance with the provisions of the 2016 California Building Code or subsequent codes in effect when final design occurs. Recommended seismic design coefficients and spectral accelerations are presented in Section 5.1 of this report.*

4.3 Liquefaction and Related Effects

Liquefaction refers to the sudden, temporary loss of soil shear strength during strong ground shaking. Liquefaction-related phenomena include liquefaction-induced settlement, flow failure, and lateral spreading. These phenomena can occur where there are saturated, loose, granular deposits. Recent advances in liquefaction studies (Idriss & Boulanger, 2008 & 2010) indicate that liquefaction can occur in granular materials with a relatively high fines content provided the fines exhibit a plasticity less than seven. As shown on Figure 7, the project site is mapped within an area susceptible to liquefaction (Association of Bay Area Governments, 2018). Additionally, several layers of medium dense to dense sandy soils were also encountered at variable depths in Borings 1 and 8. The thickness of these sandy soils varied from about three to four feet.

Lateral spreading refers to a specific type of liquefaction-induced ground failure characterized primarily by horizontal displacement of surficial soil layers as a consequence of liquefaction of a subsurface granular layer (Youd, 1995). Lateral spreads generally move down gentle slopes or slip toward a free face such as an incised river channel. From the available subsurface data, it does not appear that potentially liquefiable deposits are continuous across the site. The site is relatively level and is located more than 500 feet north of Copeland Creek and 800 feet south of Hinebaugh Creek. Therefore, the potential for lateral spreading is considered to be low.

4.3.1 Liquefaction Evaluation

To evaluate soil liquefaction, the seismic energy from an earthquake is compared with the ability of the soil to resist pore pressure generation, known as the Cyclic Resistance Ratio (CRR). The earthquake energy is termed the cyclic stress ratio (CSR) and is a function of the maximum considered earthquake peak ground acceleration and depth. Soil resistance to liquefaction is based on its relative density, and the amount and plasticity of the fines (silts and clays). The relative density of cohesionless soil is correlated with the Standard Penetration Test (SPT) blow count data measured in the field and corrected for hammer efficiency, overburden and percent fines to determine the $(N1)_{60,CS}$ value.

We analyzed the potential for liquefaction utilizing the data from our CPTs and test borings and the procedures outlined by Idriss and Boulanger (2008 & 2010). We also evaluated liquefaction potential from the CPT soundings utilizing the software program Cliq 2.0, developed by Geologismiki (2006) in conjunction with methods prescribed by Robertson (2009). For our evaluation, we considered a magnitude 7.3 earthquake producing a PGA of 0.64 g, which corresponds to the PGA_M value as defined in the ASCE 7-13 Section 18.1 and discussed further in Section 5.1 in this report. We also assumed a groundwater depth of five feet below ground surface based on the historic groundwater data. The results of our liquefaction analyses indicate the sandy alluvial soil layers encountered at about 10 to 15 feet below ground surface are susceptible to liquefaction under the estimated peak ground acceleration.

4.3.2 Estimated Post-Liquefaction Settlement

We estimated the amount of post-liquefaction settlement utilizing the procedures outlined by Idriss and Boulanger (2008 & 2010). The results of our post-liquefaction settlement analyses indicate total settlements up to two inches, and one inch of differential settlement over 50 feet, may occur during a strong seismic event.

Additionally, we utilized procedures outlined by Ozocak and Sert (2010) to calculate the Liquefaction Potential Index (LPI), which is a gauge to determine if liquefiable layers will impact the ground surface. LPI is a function of the thickness, depth, and factor of safety of liquefiable layers within a soil column. The resulting LPI value corresponds to a relative potential for surface deformation impacting the ground surface. Typically, an LPI value less than 5 indicates a low risk of surface manifestation while values between 5 and 15 indicate a moderate risk. LPI values greater than 15 indicate a major or high probability of liquefaction impacting the ground surface. The results of our analyses indicate the LPI for the site ranges from about 1 to 10, correlative with a “minor” to “moderate" probability of surface manifestation.

Based on our analyses, as described above, it is our opinion that liquefaction presents a low to moderate risk of damage to the planned improvements. The proposed improvements should be designed to accommodate the estimated liquefaction-induced settlement.

Evaluation:	*Less than significant with mitigation.*
Mitigation:	*Foundation systems should be designed to withstand up to two inches of total and one inch of differential settlement over 50 feet. Further discussion of foundation*

systems and design criteria to mitigate the effects of liquefaction are provided in Section 5.3. Additionally, flexible utility connections should be required to allow for movement without rupturing if liquefaction does occur.

4.4 Settlement

Significant settlements can occur when new loads are applied to soft, compressible soils or loose, granular soils. The rate and magnitude of potential settlements are dependent on the new loads that are applied, the stress history of subsurface soils, the presence of drainage layers, the thickness and compressibility of subsurface materials, and other factors.

Based on our subsurface exploration, the site is underlain by deep alluvial soils which consist primarily of medium stiff to very stiff clay containing variable amounts of silt, sand and gravel. Consolidation testing completed as part of the previous investigation by Harding-Lawson indicates the shallow clayey soils are over-consolidated. Additionally, the stress history of the alluvial deposits was evaluated using correlations with CPT data (Mayne and Kemper, 1988). The CPT correlations provide estimated values of over consolidation ratio (OCR) for the alluvial soils. These correlations indicate the clayey soils within the upper 40 feet are generally overconsolidated with an estimated OCR of two or greater, while soils at greater depths are slightly overconsolidated to normally consolidated.

While detailed structural information is not yet available, the new structures will be up to five stories in height and will likely induce moderate to heavy foundation loads. We anticipate the highest new building loads and risk of settlement will be associated with the four-story parking structure planned near the southeast portion of the site. We performed a preliminary evaluation of building settlements using assumed values for new building loads and compressibility properties for the clayey alluvial soils. Our preliminary settlement evaluation is discussed further under Section 5.3. Based upon the results of our preliminary analyses, we judge there is a moderate risk of damage to the planned improvements due to settlement.

Evaluation: *Less than significant with mitigation.*

Mitigation: *Alternatives to reduce the potential for settlement could include minimizing new fill and building loads, using load balancing to reduce the net loading imposed on the clayey soils, or using deep foundations to transfer loads to the deeper soils. Additional subsurface exploration and laboratory testing should be performed to further evaluate the compressibility properties of subsurface soils. A detailed settlement analysis should also be performed once grading plans, building layouts and structural loads are available. Additional discussion regarding anticipated settlements, mitigation measures, and optional foundation systems is presented in subsequent sections of this report.*

4.5 Seismic Densification

Seismic ground shaking can induce settlement in unsaturated, loose, granular soils. Settlement occurs as the loose soil particles rearrange into a denser configuration when subjected to seismic ground shaking. Varying degrees of settlement can occur throughout a deposit, resulting in differential settlement of structures founded on such deposits. Based on our exploration, subsurface soils primarily consist of medium stiff to very stiff clays. Therefore, the risk of seismic densification impacting the new structures is generally low.

Evaluation: *Less than significant.*
Mitigation: *No mitigation measures are required.*

4.6 Expansive Soils

Soil expansion occurs when clay particles interact with water causing seasonal volume changes in the soil matrix. The clay soil swells when saturated and then contracts when dried. This phenomenon generally decreases in magnitude with increasing confinement pressures at increasing depths. These volume changes may damage lightly loaded foundations, concrete slabs, pavements, retaining walls and other improvements. Expansive soils also cause soil creep on sloping ground.

Laboratory testing indicates the clayey surface soils exhibit a high plasticity and a very high expansion potential. Therefore, the risk of damage to the planned improvements due to expansive soils is high.

Evaluation: *Less than significant with mitigation.*
Mitigation: *Expansive soil mitigation measures can include removal and replacement with select (imported) soils or lime treatment to reduce expansion potential. Lime treatment is frequently performed in Sonoma County and is likely a cost-effective option for improved site performance when compared to removing and replacing with imported soil. Recommendations for lime treatment are provided in Section 5.2 of this report. Additionally, building foundations should be designed to account for some expansive soil movement as discussed under Section 5.4.*

4.7 Erosion

Sandy soils on moderately steep slopes or clayey soils on steep slopes are susceptible to erosion when exposed to concentrated surface water flow. The potential for erosion is increased when established vegetation is disturbed or removed during normal construction activity.

The work area is relatively level and the proposed improvements indicate that much of the site will be covered with new buildings, pavements, or concrete flatwork. Therefore, erosion is not considered to be a significant long-term geologic hazard. However, care should be taken during construction to prevent excess erosion when the soils are exposed.

Evaluation: *Less than significant with mitigation.*
Mitigation: *Mitigation measures include designing a site drainage system to collect surface water and discharging it into an established storm drainage system. The project Civil Engineer of Architect is responsible for designing the site drainage system and, an erosion control plan could be developed prior to construction per the current guidelines of the California Stormwater Quality Association's Best Management Practice Handbook.*

4.8 Flooding

The project site is located at elevations ranging from about 100 to 105 feet above sea level and is not mapped within a FEMA-designated flood hazard area (Federal Emergency Management Agency, 2008). Therefore, large scale flooding is not considered a significant hazard at the project site. The project Civil Engineer or Architect is responsible for site drainage and should evaluate localized flooding potential due to maximum credible rainfall events.

Evaluation: *Less than significant with mitigation.*
Mitigation: *The project Civil Engineer is responsible for site drainage and should evaluate localized flooding potential and provide appropriate storm drainage design.*

5.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our investigation, we conclude the site conditions are suitable for the proposed improvements. The primary geotechnical considerations for design and construction will include designing new structures to resist strong seismic ground shaking, designing foundations to accommodate settlements due to new static loads and potential liquefaction, and providing mitigation for expansive soils. Additional discussion and recommendations addressing these and other considerations are presented in the following sections.

5.1 Seismic Design

Minimum mitigation of ground shaking includes seismic design of new structures in conformance with the provisions of the most recent edition (2016) of the California Building Code. The magnitude and character of these ground motions will depend on the earthquake and the site response characteristics. Based on the interpreted subsurface conditions and close proximity of the Rodgers Creek Fault, we recommend the CBC coefficients and site values shown in Table 3 be used to calculate the design base shear of new improvements as applicable.

Table 3 – 2016 California Building Code Seismic Design Criteria

Parameter	Design Value
Site Class	D
Site Latitude	38.3467°N
Site Longitude	-122.7033°W
Spectral Response (short), S_S	1.659 g
Spectral Response (1-sec), S_1	0.654 g
Site Coefficient, F_a	1.0
Site Coefficient, F_V	1.5
Spectral Response (Short), S_{MS}	1.659 g
Spectral Response (1 sec), S_{M1}	0.980 g
Design Spectral Response (short), S_{DS}	1.106 g
Design Spectral Response (1 sec), S_{D1}	0.654 g
MCE_G PGA Adjusted, PGA_M	0.64 g
Seismic Design Category	D

Reference: USGS US Seismic Design Maps, accessed on September 17, 2018.

Due to the presence of relatively thin layers of loose sandy soils that are potentially liquefiable, we judge the classifies as “Site Class F” per the 2016 California Building Code. However, per the CBC and section 20.3.1 of ASCE 7-10 standard, a non-linear site-specific response analysis (i.e., SHAKE) is not required since it is anticipated that the proposed buildings will have a fundamental period of less than 0.5 seconds. Therefore, we recommend classifying the site as “Site Class D” for design purposes. We should be consulted if it is determined that the fundamental period of any of the proposed structures is greater than 0.5 seconds.

5.2 Site Grading

We anticipate that site grading will be limited to cuts and fills of a few feet in height as required to achieve the finished grades for the new buildings, roadways and other improvements. Site grading and earthwork should be performed in accordance with the recommendations and criteria outlined in the following sections.

5.2.1 Site Preparation

Clear pavements, old foundations, over-sized debris, grass, brush, roots, and organic material from areas to be graded. Debris and rocks larger than six inches and vegetation are not suitable for structural fill and should be removed from the site. Existing foundations and utilities which are to be abandoned as part of the work should be removed from structural areas (e.g., below building pads or roadways). In non-structural areas, utilities could be abandoned in place provided cement grout completely fills all voids in the utility.

Based on our subsurface exploration and previous investigations within the site vicinity, we anticipate that near-surface soils are expansive throughout the site. One exception may include beneath the former building where several feet of fill appears to have been placed during the original site development. Site preparation should include removing any expansive soils or loose fill material from within new building areas. The removal should extend a minimum of five feet laterally beyond the new building footprints and to a minimum depth of 36 inches. The soils that are removed should be replaced with compacted fill that conforms to the requirements outlined in Section 5.2.2. Lime treatment or other soil modification techniques may be used as an alternative to removal and replacement of expansive soils. The use of lime treatment is discussed further in Section 5.2.4.

We recommend subgrade soils beneath new pavements and exterior slabs areas be lime treated or removed and replaced with select fill to a minimum depth of 18 inches. Lime treatment will reduce the expansion potential and improve the strength and stiffness of the subgrade soils, thereby reducing the required thickness of new pavements and concrete slab. If lime treatment or removal and replacement are not used, the subgrade surface should be scarified to a depth of 12 inches, moisture conditioned to above the optimum moisture content, and compacted as described in Section 5.2.2. Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density, as determined by ASTM D1557. The subgrade should be firm and unyielding when proof-rolled with heavy, rubber-tired construction equipment. If soft, wet or otherwise unsuitable materials are encountered at subgrade elevation during construction, we can provide supplemental recommendations to address the specific condition.

5.2.2 Fill Materials, Placement and Compaction

Fill should consist of non-expansive materials that are free of organic matter, have a Liquid Limit of less than 40 (ASTM D 4318), a Plasticity Index of less than 15 (ASTM D 4318), and a minimum R-value of 20 (California Test 301). The fill material should contain no more than 50 percent of particles passing a No. 200 sieve and should have a maximum particle size of four inches. The onsite clayey soils are likely not suitable for reuse as fill due to their high plasticity and expansion potential. Any imported fill material needs to be tested to determine its suitability.

Fill materials should be moisture conditioned to above the optimum moisture content prior to compaction. Properly moisture conditioned fill materials should be placed in loose, horizontal lifts of eight inches or less and uniformly compacted to the relative compaction shown in Table 4.

Table 4 – Compaction Requirements

Soil Type	Location	Relative Compaction
On-site, untreated	Landscaped areas	85%
On-site, untreated	Concrete slabs and walkways (no vehicle traffic)	88 to 92%
On-site, untreated	Parking lots, driveways and pavements	95%
Lime treated soil	Upper 18 inches below new pavements	95%
Lime treated soil	Below buildings and all other lime treated soil	92%
Imported Fill	Upper 18 inches below new pavements	95%
Imported Fill	Below buildings and all other imported fill	92%

5.2.3 Excavations

Based on our subsurface exploration, excavations for underground utilities and other improvements will encounter predominantly medium stiff to very stiff clayey soils. Layers of medium dense sandy and gravelly soils may also be encountered within the alluvium (generally below a depth of ten feet) as well as within the fill beneath the former building area. We anticipate the clayey soils will exhibit firm behavior in unsupported excavations, while sandy and gravelly soils will be susceptible to flowing below groundwater and running to fast raveling above groundwater. Definitions of the various ground behaviors are presented in the Tunnelman's Ground Classification for Soils, Figure 8. The clayey soils generally classify as OSHA Type B soils while the sandy and gravelly soils classify as Type C.

Temporary support of excavations will be required to ensure the safety of workers and to reduce the potential for failure of the excavation sidewalls and damage to surrounding improvements. Excavation stability and the structural design of temporary shoring should be made the sole responsibility of the Contractor. The selected support system should be designed to resist later pressures from earth, hydrostatic and construction surcharge loads. As a minimum, shoring systems should be designed based on the criteria provided in Table 5. Excavations deeper than five feet may encounter groundwater and could require temporary dewatering. If required, the design of temporary dewatering systems should be made the sole responsibility of the Contractor.

Table 5 – Shoring Design Criteria

Parameter	Design Value
Active Earth Pressure, Unrestrained[1]	50 pcf
Active Earth Pressure, Restrained[2]	35 x H psf
Lateral Passive Resistance[1]	300 pcf
Minimum Surcharge Pressure[3,4]	125 psf

Notes:

(1) Equivalent fluid pressure.
(2) Rectangular distribution, H is wall height in feet
(3) Apply surcharge load to upper 5 feet of shoring.
(4) Surcharge load to be adjusted at the discretion of the Contractor’s shoring designer.

5.2.4 Lime Treatment

As an alternative to removal and replacement, lime treatment can be used to improve the near-surface expansive soils. Treating the soils with lime will reduce the expansion potential and increase the strength, thereby reducing the required thickness of new asphalt pavement sections. Lime treated soils also perform relatively well during wet weather conditions which helps facilitate site grading and construction throughout the rainy season. A disadvantage of lime treatment is that it will increase the soil pH which may adversely affect future landscaping at the site. If lime treatment is used, treated soils may need to be removed from planned landscaping areas to allow for future planting.

We performed one soil-lime proportioning test to estimate the optimum lime content for treatment. The test was performed using on-site soils and high-calcium quicklime provided by Griffin Soil Stabilization. The test results are shown in Appendix A and indicate that a minimum of five percent high-calcium lime by weight should be mixed with the on-site soils to the treatment depth. As a basis for determining the amount of applied lime, a dry unit weight of 90 pounds per cubic foot should be assumed. Because different lime contractors use different types of lime and quality of the lime may also vary from one contractor to another, the lime percentage may need to be adjusted if Griffin’s high-calcium lime is not used at the project site. Also, the addition of lime will tend to “fluff up” the on-site soils both because of the volume of lime added and also because lime will tend to lower maximum unit weights, increasing the volume.

If used, lime treatment should be performed in accordance with Section 24 of the 2015 Caltrans Standard Specifications. The recommended minimum treatment depth is 36 inches under buildings and 18 inches in pavement and concrete flatwork areas. The lime-treatment should extend at least five feet beyond building pads, and three feet beyond flatwork and pavement areas.

Within planned treatment areas, soil preparation should consist of thoroughly moisture conditioning the expansive soil to at least three percent above the optimum moisture content to close shrinkage cracks for the full depth of the treatment zone. Treated soil materials should be moisture-conditioned to near optimum moisture content, spread in loose lifts of up to 18-inches thick (based on approval of compaction equipment by the Geotechnical Engineer) and compacted as described above.

5.3 Preliminary Settlement Evaluation

We completed a preliminary analysis to estimate potential settlements that may occur due to new building loads. The preliminary building settlements were estimated using the following assumptions:

- The clayey alluvial soils within the upper 40 feet are overconsolidated with an OCR of 2.0 while deeper soils are normally consolidated. These values were estimated using the CPT correlations and reference consolidation tests discussed previously.
- The upper 40 feet of clayey alluvial soils have a compression index (Cc, strain-based) of 0.10 and recompression index (Cr, strain-based) of 0.008. These values were estimated from the consolidation test results from Harding-Lawson's previous site investigation. The Cc and Cr values for deeper soils were estimated to be 0.15 and 0.05, respectively.
- Columns are supported on footings sized at three feet by three feet with bearing pressures of 2,000 pounds per square foot and are spaced at 20 feet.
- Wall loads are supported on two-foot-wide continuous footings with bearing pressures of 2,000 pounds per square feet.
- "Immediate" settlements were estimated using elastic theory with a constrained modulus (Es) of 150 kips per square foot.
- Settlement due to secondary compression was neglected in our preliminary analysis.

Using these assumptions, we estimate that up to about two inches of total settlement and one inch of differential settlement over a distance of 50 feet may occur due to the new loads. A detailed settlement analysis for the various structures should be performed once building layouts and loads are better characterized. This should include an evaluation of settlements for the parking garage where the most significant new loads are anticipated. Additional subsurface exploration and laboratory testing should also be performed to further evaluate the compressibility properties of subsurface soils and further calibrate the model used in our analysis.

5.4 Foundation Design

Based on our investigation, we judge a relatively rigid shallow foundation system consisting of interconnected continuous footings, post-tensioned slabs or mat slabs would be appropriate for the new residential and commercial structures. As discussed under Section 5.2, building pads should either be lime treated or capped with non-expansive fill to reduce expansion potential. Geotechnical design criteria for spread footings are presented below in Table 6 and criteria for post-tensioned or mat slabs are presented in Table 7.

Table 6 – Spread Footing Design Criteria

Parameter	Design Value
Minimum Embedment	18 inches
Minimum Width	18 inches
Allowable Bearing Pressure[1,2]	3,000 psf
Allowable Base Friction Coefficient	0.35
Allowable Lateral Passive Resistance[3]	300 pcf
Maximum Unsupported Interior Span	6 feet
Maximum Unsupported Edge Span	3 feet

(1) Value represents a net pressure (i.e. weight of foundation and backfill over the foundation can be neglected). Increase design values by 1/3 for total design loads including seismic.

(2) Controlled by settlement. Additional analyses will be required to check settlements using actual building layouts and structural loads.

(3) Equivalent fluid pressure, not to exceed 3,000 psf. Neglect upper six inches unless confined by concrete.

Table 7 – Rigid Mat Slab or Post-Tensioned Slab Design Criteria

Parameter	Design Value
Modulus of Subgrade Reaction – Non-Treated	75 lb/in^3
Modulus of Subgrade Reaction – Lime Treated	225 lb/in^3
Minimum Thickness at Edge of Slab[1]	12 inches
Maximum Unsupported Interior Span[2]	6 feet
Maximum Unsupported Edge Cantilever[2]	3 feet
Edge Moisture Variation (e_m), Center Lift	6 feet
Edge Moisture Variation (e_m), Edge Lift	3 feet
Differential Soil Movement (y_m), Center Lift	0.5 inch
Differential Soil Movement (y_m), Edge Lift	0.5 inch

(1) Actual thickness, load distribution, and unsupported spans must be determined by Structural Engineer to reduce deformations to acceptable levels.

(2) Assumes rigid slab behavior with idealized fixed end conditions.

As discussed in the previous section, foundations should be designed to account for up to two inches of total settlement and one inch of differential settlement over 50 feet. Load balancing may be considered as a means of reducing the potential settlements for the new buildings. This approach would include overexcavating beneath the structure and replacing a portion of the soil that is removed with lightweight material consisting of lava rock, cellular concrete or geofoam. Alternatively, load balancing could be achieved by creating below-grade areas (e.g. basements or underground parking) in which removal of soil would offset new building loads and result in lower net pressures.

We anticipate that building loads for the planned parking garage will be larger than the other structures and a deep foundation will likely be required to reduce potential building settlements. Various deep foundation alternatives are judged to be appropriate, including torque-down piles, auger-cast piles, drilled piers or driven piles. Alternatively, ground improvement with rammed aggregate piers or similar systems could be used to increase the bearing capacity and stiffness of the subsurface soils. A brief overview of these foundation systems is provided below.

> ***Torque-Down Piles* -** Torque down piles are full displacement, steel pipe piles that consist of a large-diameter steel shaft with a closed-ended, conical tip and helix. The pile is “screwed” into the ground to depth and the steel shaft displaces the surrounding soil as it advances. The piles are typically filled with grout after installation. Torque-down piles achieve vertical capacity through friction between the soil and the steel pipe and end bearing of the closed-ended tip.
>
> ***Augercast Piles*** – Augercast piles are installed by rotating a continuous flight, hollow shaft auger to displace the soil to a specified depth. High strength cement grout is pumped under pressure through the hollow shaft as the auger is slowly withdrawn. Reinforcing is typically installed while the cement grout is still fluid or through the hollow shaft of the auger prior to the withdrawal and grouting process.
>
> ***Drilled Piers* –** Drilled piers are a larger-diameter system (typically 18 to 24 inches in diameter) which consist of excavating a hole to a desired depth, installing a steel reinforcing cage, and filling the hole with concrete. Drilled piers can be constructed in a variety of soil conditions; however, the drilled hole may need to be supported with drilling fluid or casing as required by ground conditions.
>
> ***Driven Piles* -** Driven piles are steel or precast concrete piles driven with a large pile hammer until a suitable driving resistance and bearing capacity is achieved. Drilled piles often generate significant noise and vibrations which may preclude their use at the site. If driven piles are selected, pre-drilling would likely be required to reduce vibrations and minimize impacts to nearby utilities and other improvements.
>
> ***Compacted Aggregate Piers*** – Compacted aggregate piers are a ground improvement method which consists of drilling out and replacing compressible soil with compacted aggregate. The compacted aggregate piers help to create a stronger “composite” aggregate pier/soil matrix which increases bearing capacity and soil stiffness and decreases compressibility.

For the purposes of evaluating the various foundation systems, key factors associated with each alternative are summarized in Table 8. We can consult with the design team to provide supplemental criteria for foundation design once a system has been selected.

Table 8 – Comparison of Key Factors for Deep Foundation Alternatives

Assumed Foundation Type	Estimated Vertical Capacity(1,2) (kips)	Noise and Vibrations	Spoils
Torque-Down Piles (12.75-in-diam)	45 to 75	Low	None to Minor
Augercast Piles (18-in-diam)	75 to 110	Low	Minor to Moderate
Drilled Piers (24-in-diam)	55 to 90	Low	Significant
Driven Piles (12-in-diam)	45 to 75	Significant	Minor to Moderate
Compacted Aggregate Pier	4,000 psf (bearing capacity)	Moderate to Significant	Significant

(1) Estimated allowable capacity with factor of safety of 2.0.

(2) Assumes foundation will extend to estimated depths of 40 to 50 feet. Higher capacities could be achieved if foundations are extended deeper.

5.5 Retaining Walls

Retaining walls are not currently anticipated for the development. If needed, basement walls and site retaining walls should be preliminarily designed to resist lateral pressures from earth and surcharge loads, as shown in Table 9. Retaining walls that can slightly deflect at the top can be designed using the unrestrained criteria shown below. Walls that are structurally connected and not allowed to deflect (e.g., tied-back walls) are restrained and are commonly designed using a uniform active earth pressure distribution rather than an equivalent fluid pressure.

Table 9 – Active Earth Pressure for Retaining Wall Design

Backfill Inclination[1]	Unrestrained[2,3]		Restrained[3,4]	
	Lime Treated or Select Fill	Untreated Soils	Lime Treated or Select Fill	Untreated Soils
Level	30 pcf	50 pcf	30 x H psf	35 x H psf
3:1	40 pcf	60 pcf	35 x H psf	40 x H psf
2:1	50 pcf	70 pcf	40 x H psf	45 x H psf

Notes:

(1) Interpolate earth pressures for intermediate slopes

(2) Equivalent fluid pressure

(3) Wall design should account for a seismic surcharge of 12 x H (in psf) in addition to active pressure

(4) Rectangular distribution, H is wall height in feet

Wall drainage is required for all retaining walls taller than 3 feet. Wall drainage should consist of Caltrans Class 1B permeable material within filter fabric or Caltrans Class 2 permeable material. A composite drainage panel such as Miradrain 6000 (or approved equivalent) could also be used. The drainage should be collected in a 4-inch perforated PVC drain line at the base of the wall and discharged to an appropriate discharge location. The permeable material should extend at least

12 inches from the back of the wall and be continuous from the bottom of the wall to within 12 inches of the ground surface. A schematic retaining wall backdrain detail is shown on Figure 9.

5.6 Interior Concrete Slabs-On-Grade

Reinforced concrete slab-on-grade floors are judged to be appropriate for the site conditions. The concrete slabs-on-grade may be poured monolithically or separated with a cold joint. We recommend that interior concrete slabs have a minimum thickness of five inches and be reinforced with steel reinforcing bars (not mesh) with rebar extending through crack control joints. Slabs should be placed on a moist subgrade to reduce potential for future expansive behavior. The project Structural Engineer should specifically design the concrete slabs, including locations of crack control joints.

To reduce the potential for moisture to move upward through the slab, a four-inch layer of clean, free draining, ¾-inch angular gravel should be placed beneath interior concrete slabs to form a capillary moisture break. The gravel must be placed on a properly moisture conditioned and compacted subgrade that has been approved by the Geotechnical Engineer. A plastic membrane vapor barrier, 15 mils or thicker, should be placed over the compacted base rock. The vapor barrier shall meet the ASTM E1745 Class A requirements and be installed per ASTM E1643. Eliminating the capillary moisture break and/or plastic vapor barrier may result in excess moisture intrusion through the floor slabs resulting in poor performance of floor coverings, mold growth, or other adverse conditions.

We note that over time, placing sand between the vapor barrier and concrete is becoming less common because of elevated interior moisture contents. If sand is used, it should be dry, and if it is not used, the slab should be carefully designed with a lower water-cement ratio (generally less than 0.45) since eliminating the sand can cause cracking or “curling” of the new concrete. For slabs that are not sensitive to moisture vapor, we recommend at least four inches of Class 2 aggregate base (Caltrans, 2015) compacted to 95 percent relative compaction.

5.7 Exterior Concrete Slabs

Exterior concrete walkway slabs and other concrete slabs that are not subjected to vehicle loads should be a minimum of four-inches-thick and underlain with four inches or more of Class 2 aggregate base. The aggregate base should be moisture conditioned to near optimum and compacted to at least 95 percent relative compaction. The upper eight inches of subgrade on which aggregate base is placed should be prepared as previously discussed under Section 5.2.

Where improved performance is desired (i.e., reduced risks of cracking or small movements), exterior slabs can be thickened to five inches and reinforced with steel reinforcing bars (not welded wire mesh). We recommend crack control joints no farther than six feet apart in both directions and that the reinforcing bars extend through the control joints. Some movement or offset at sidewalk joints should be expected as the underlying soils expand and shrink from seasonal moisture changes.

5.8 Site and Foundation Drainage

New grading could result in adverse drainage patterns causing water to pond around the development. Careful consideration should be given to design of finished grades at the site. We recommend that the building areas be raised slightly and that the adjoining landscaped areas be sloped downward at least 0.25 feet for five feet (five percent) from the perimeter of building foundations. Where hard surfaces, such as concrete or asphalt adjoin foundations, slope these surfaces at least 0.10 feet in the first five feet (two percent).

Roof gutter downspouts may discharge onto the pavements, but should not discharge onto landscaped areas immediately adjacent to the buildings. Provide area drains for landscape planters adjacent to buildings and collect downspout discharges into a tight pipe collection system that discharges well away from the building foundations. Site drainage should be discharged away from the building area and outlets should be designed to reduce erosion. Site drainage improvements should be connected into an established storm drainage system.

5.9 Underground Utilities

Excavations for utilities will generally encounter a combination of medium dense to dense sand and medium stiff to very stiff clayey soils containing variable amounts of sand and gravel. Groundwater may be encountered at shallow depths. Trench excavations having a depth of five feet or more must be excavated and shored in accordance with OSHA regulations, as discussed in Section 5.2.3.

Unless otherwise recommended by the pipe manufacturer, pipe bedding and embedment materials should consist of well-graded sand with 90 to 100 percent of particles passing the No. 4 sieve and no more than five percent finer than the No. 200 sieve. Crushed rock or pea gravel may also be considered for pipe bedding. Provide the minimum bedding thickness beneath the pipe in accordance with the manufacturer’s recommendations (typically three to six inches). Trench backfill may consist of on-site soils, provided that the soil meets the fill criteria outlined in Section 5.2.2 or imported aggregate baserock. Trench backfill should be moisture conditioned, placed in thin lifts and compacted to at least 90 percent. Use equipment and methods that are suitable for work in confined areas without damaging utility conduits.

5.10 Pavements

We have calculated pavement sections in accordance with Caltrans procedures for flexible pavement design (Caltrans, 2015). Our calculations assume an R-value of five for untreated soils and 60 for lime treated soils. If lime treatment is used, additional laboratory testing should be performed prior to construction to confirm the assumed R-value for the treated soils is appropriate.

We have provided a range of Traffic Indices from 4.0 to 8.0 depending on the expected traffic loads for a twenty-year design life. In general, areas expected to experience loading from heavy vehicles should be designed using the higher Traffic Index, while parking areas and other lightly-loaded areas can utilize a thinner pavement section based on the lower Traffic Index. The recommended pavement sections are presented in Table 9.

Table 9 – Preliminary Asphalt-Concrete Pavement Sections

Traffic Index	Untreated Subgrade (R-value = 5)		Lime Treated Subgrade[1] (R-value = 60)	
	Asphalt Concrete (inches)	Aggregate Base (inches)	Asphalt Concrete (inches)	Aggregate Base (inches)
4.0	2.5	8.0	2.0	4.0
5.0	3.0	10.0	3.0	5.0
6.0	4.0	12.0	3.5	6.0
7.0	5.0	14.0	4.0	7.0
8.0	6.0	16.0	5.0	8.0

(1) Calculated using a minimum lime treatment depth of 18 inches.

The aggregate base and asphalt concrete should conform to the most recent version of Caltrans Standard Specifications and should be compacted to at least 95 percent relative compaction. Additionally, the aggregate base should be firm and unyielding under heavy, rubber-tired construction equipment. If heavier truck traffic or "superior" performance is desired, the thickness of the aggregate base and asphalt thickness may be increased.

6.0 SUPPLEMENTAL GEOTECHNICAL SERVICES

Following review and consideration of this report, we should consult with the project team regarding the "preferred" foundation type for the new structures. Supplemental exploration and lab testing should be performed to evaluate the compressibility of subsurface soils. When building layouts and structural loads are available, a detailed settlement analysis should also be performed to refine the estimated building settlements presented in this report.

As project plans near completion, we should review them to confirm that the intent of our recommendations has been sufficiently incorporated. During construction, we should be present intermittently to observe and test geotechnical portions of the work. We anticipate this will include observation and/or testing of foundation excavations, moisture conditioning and compaction of subgrade soils, fill placement and compaction, lime treatment (if used), and other geotechnical-related work items. The purpose of our observation and testing is to confirm that site conditions are as anticipated, to adjust our recommendations and design criteria if needed, and to confirm that the Contractor's work is performed in accordance with the project plans and specifications.

7.0 LIMITATIONS

We believe this report has been prepared in accordance with generally accepted geotechnical engineering practices in the northern San Francisco Bay Area at the time the report was prepared. This report has been prepared for the exclusive use of the Laulima Rohnert Station, LLC and/or their assignees specifically for this project. No other warranty, expressed or implied, is made. Our evaluations and recommendations are based on the data obtained during our subsurface exploration program and our experience with soils in this geographic area.

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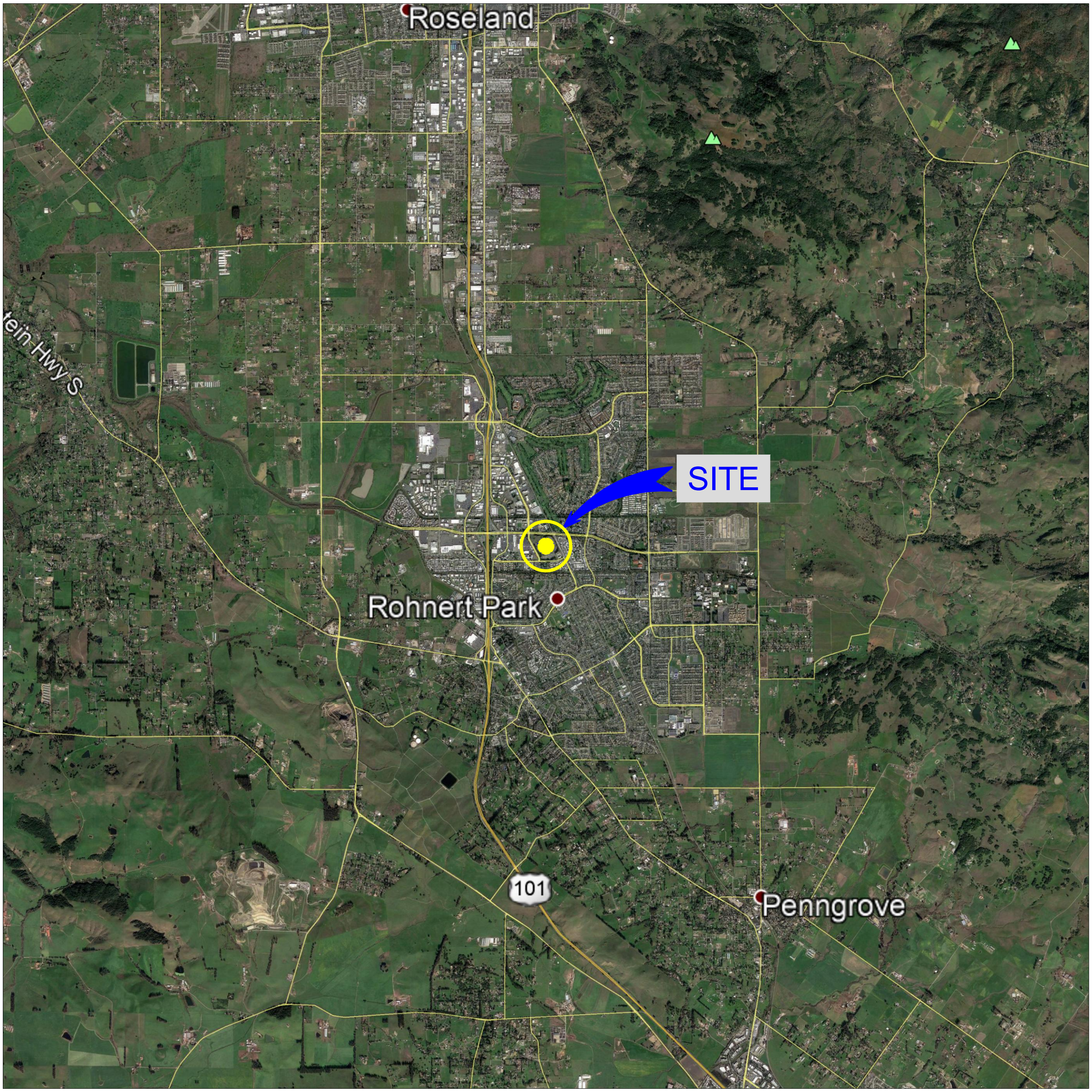

SITE: LATITUDE, 38.3467°
LONGITUDE, -122.7033°

SITE LOCATION

N.T.S.

NORTH

REFERENCE: Google Earth, 2018

MPEG
MILLER PACIFIC ENGINEERING GROUP

FILENAME: 2647.001 Figures.dwg

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SITE LOCATION MAP

Rohnert Station
6400 State Farm Drive
Rohnert Park, California

Project No. 2647.001 Date: 9/12/2018

Drawn MMT
Checked

1
FIGURE

DEVELOPMENT PLAN

Buildings B1, C1, D1 & F1
Ground Level: Retail
Second Level: Office

Building G1
Ground Level: Retail
2nd & 3rd Levels: Office

Buildings A1, A2, D2, D3, E1 & E2
Ground Level: Retail

Building D4
Ground Level: Retail & Maintenance

Building B2
Ground Level: Retail
Second Level: Retail

Building A3
Ground Level: Parking
2nd & 3rd: Parking

Building A4
Ground Level: Hotel
2nd, 3rd, 4th & 5th: Hotel

Buildings C2–C3
Ground Level: Parking & Residential
2nd & 3rd: Residential

Buildings H1–H5
Ground Level: Parking & Residential
2nd & 3rd: Residential

Buildings J1 & J3,
Ground Level: Residential
2nd, 3rd & 4th: Residential

Building J2
Ground Level: Parking
2nd, 3rd & 4th: Parking

Buildings J4, J5 & C4
Ground Level: Club House

Reference: Laulima Development, "Development Site Plan", 9/11/18

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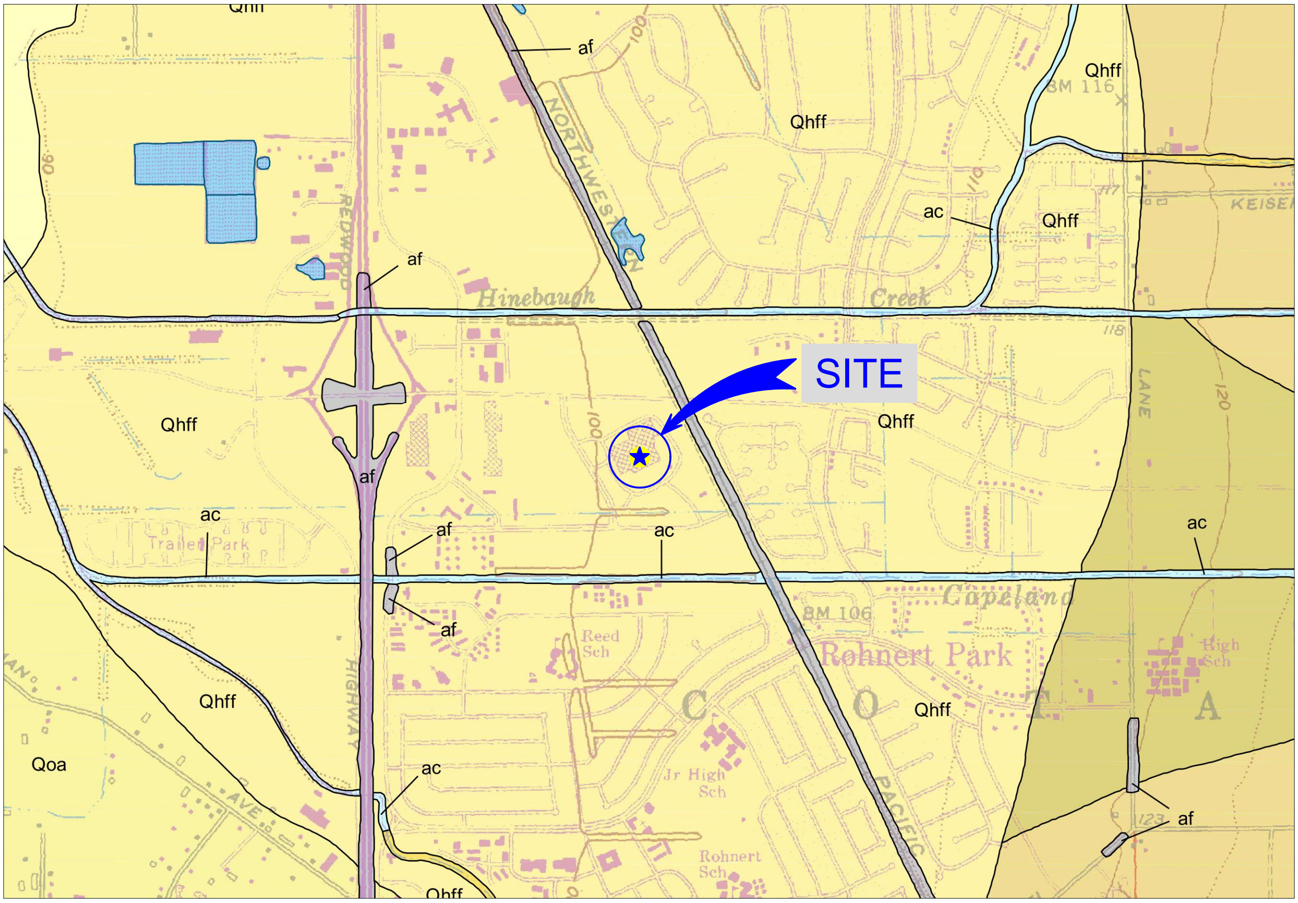
SITE PLAN

Rohnert Station
6400 State Farm Drive
Rohnert Park, California
Project No. 2647.001 Date: 9/24/2018

Designed
Drawn RCA
Checked

2
FIGURE



REGIONAL GEOLOGIC MAP

(NOT TO SCALE)

NORTH

LEGEND

ac	**Artificial Stream Channel**
af	**Artificial Fill -** May be engineered and/or non-engineered.
Qhff	**Holocene Alluvial Fan Deposits, Fine Facies** - Fine grained alluvial fan and floodplain overbank deposits on very gently sloping portions of the valley floor; composed of predominantly clay with interbedded lenses of coarser alluvium.
Qoa	**Early to late Pleistocene Alluvial Deposits, Undivided** - Alluvial fan, stream terrace, basin, and channel deposits.

Reference: Clahan, Kevin, B., Bezore, Stephen P., Koehler, Richard D., Witter, Robert C., Geologic Map of the Cotati 7.5' Quadrangle, Sonoma County, California: A Digital Database, 2003.

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REGIONAL GEOLOGIC MAP

Rohnert Station
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Project No. 2647.001 Date: 9/12/2018

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Checked

3
FIGURE

SITE COORDINATES
LAT. 38.3467°
LON. -122.7033°

DATA SOURCE:
1) U.S. Geological Survey, U.S. Department of the Interior, "Earthquake Outlook for the San Francisco Bay Region 2014-2043", Map of Known Active Faults in the San Francisco Bay Region, Fact Sheet 2016-3020, Revised August 2016 (ver. 1.1).

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ACTIVE FAULT MAP

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Project No. 2647.001
Date: 9/12/2018

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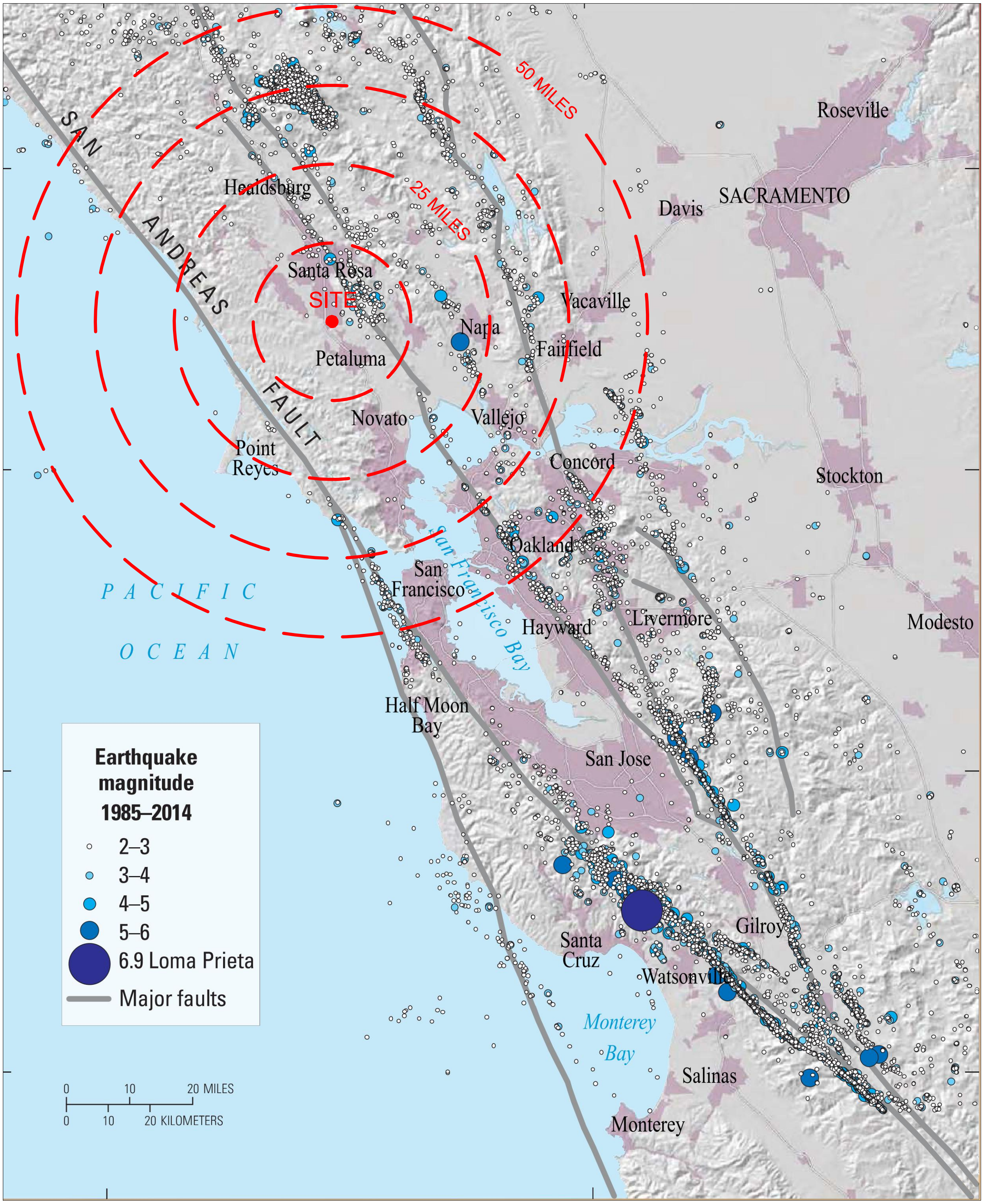
4
FIGURE



SITE COORDINATES
LAT. 38.3467°
LON. -122.7033°

SCALE
0 12.5 25 50 MILES

NORTH

DATA SOURCE:
1) U.S. Geological Survey, U.S. Department of the Interior, "Earthquake Outlook for the San Francisco Bay Region 2014-2043", Map of Earthquakes Greater Than Magnitude 2.0 in the San Francisco Bay Region from 1985-2014, Fact Sheet 2016-3020, Revised August 2016 (ver. 1.1).

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HISTORIC EARTHQUAKE ACTIVITY

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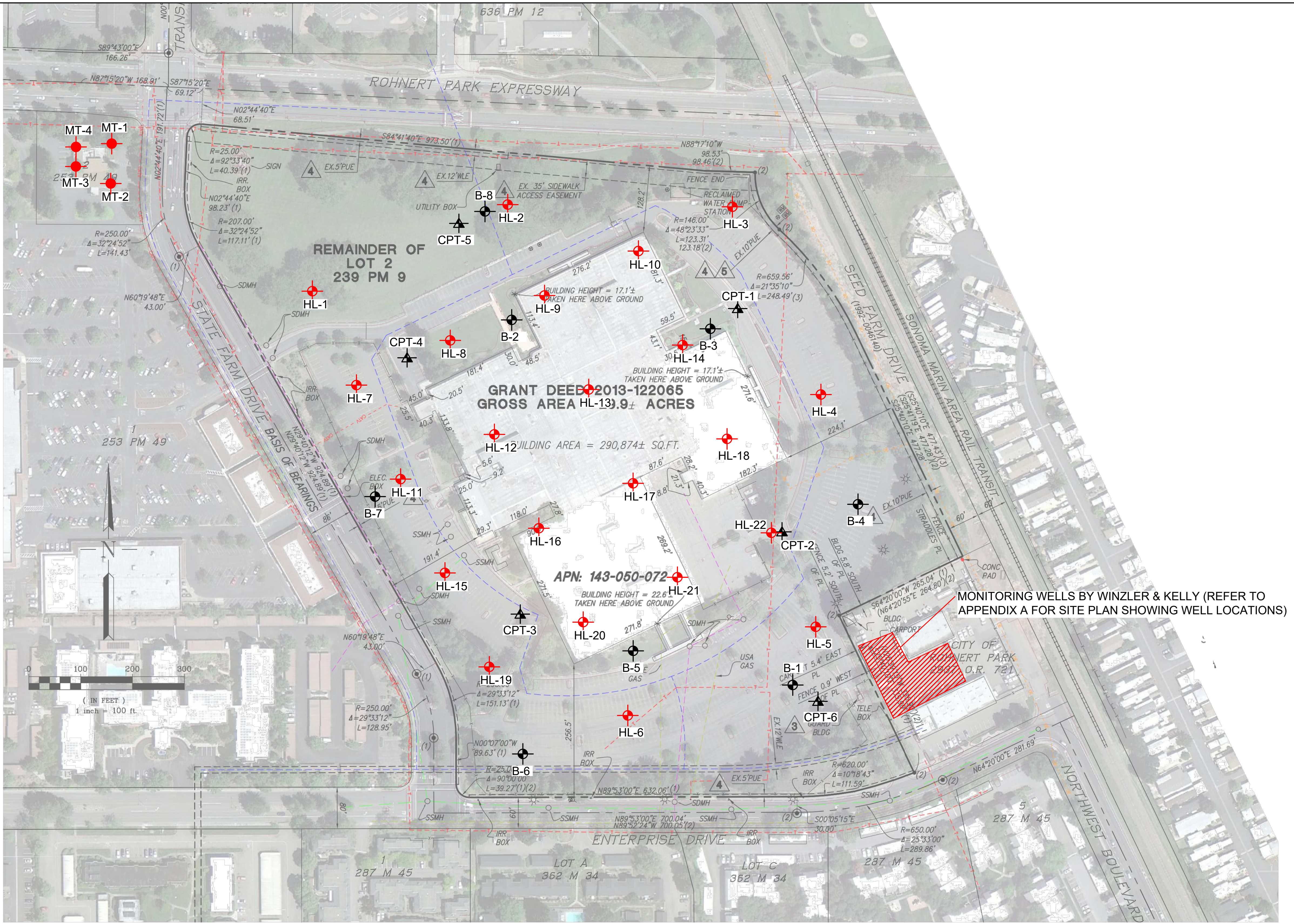
5
FIGURE



Legend:

▲ Approximate CPT location by Miller Pacific, 2018

⊕ Approximate boring location by Miller Pacific, 2018

⊕ Approximate boring location by Harding Lawson, 1976

● Approximate boring location by Moore & Taber, 1977

Reference: Ruggeri-Jensen-Azar, "ALTA Survey", 8/22/17

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EXPLORATION PLAN

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Project No. 2647.001 Date: 9/24/2018

Designed
Drawn RCA
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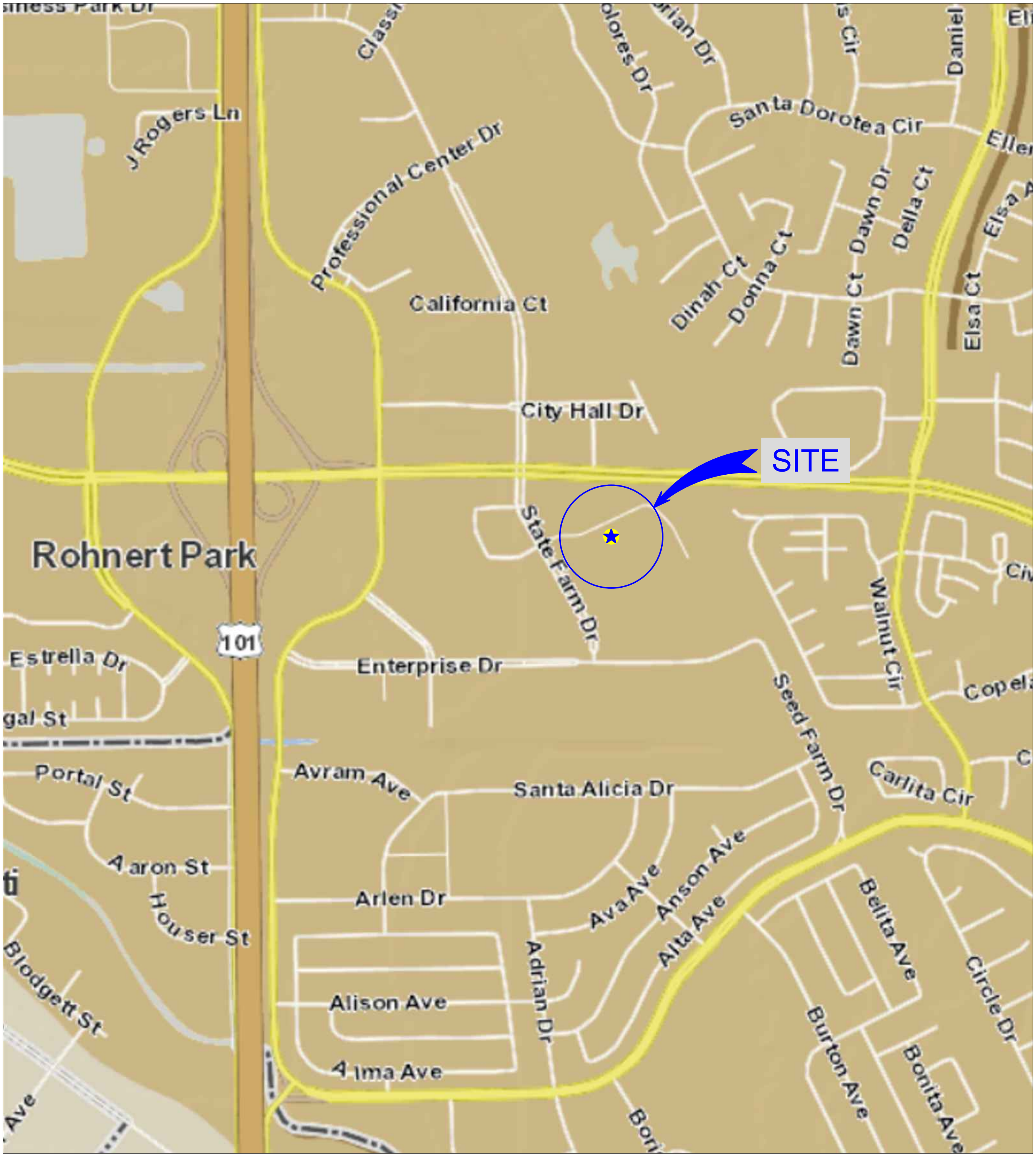
6
FIGURE



Susceptibility Level:

No Scale

■	Very High	■	Moderate	■	Very Low	—	Local Road
■	High	■	Low	▬	Major Road		

Map Reference: ABAG Geographic Information System.

NORTH

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LIQUEFACTION SUSCEPTIBILITY MAP

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Project No. 2647.001 Date: 9/12/2018

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7
FIGURE

Tunnelman's Ground Classification for Soils[1]

Classification		Behavior	Typical Soil Types
Firm		Heading can be advanced without initial support, and final lining can be constructed before ground starts to move.	Loess above water table; hard clay, marl, cemented sand and gravel when not highly overstressed.
Raveling	Slow raveling ----- Fast raveling	Chunks or flakes of material begin to drop out of the arch or walls sometime after the ground has been exposed, due to loosening or to over-stress and "brittle" fracture (ground separates or breaks along distinct surfaces, opposed to squeezing ground). In fast raveling ground, the process starts within a few minutes, otherwise the ground is slow raveling.	Residual soils or sand with small amounts of binder may be fast raveling below the water tale, slow raveling above. Stiff fissured clays may be slow or fast raveling depending upon degree of overstress.
Squeezing		Ground squeezes or extrudes plastically into tunnel, without visible fracturing or loss of continuity, and without perceptible increase in water content. Ductile, plastic yield and flow due to overstress.	Ground with low frictional strength. Rate of squeeze depends on degree of overstress. Occurs at shallow to medium depth in clay of very soft to medium consistency. Stiff to hard clay under high cover may move in combination of raveling at excavation surface and squeezing at depth behind surface.
Running	Cohesive - running ----- Running	Granular materials without cohesion are unstable at a slope greater than their angle of repose (+/- 30° – 35°). When exposed at steeper slopes they run like granulated sugar or dune sand until the slope flattens to the angle of repose.	Clean, dry granular materials. Apparent cohesion in moist sand, or weak cementation in any granular soil, may allow the material to stand for a brief period of raveling before it breaks down and runs. Such behavior is cohesive-running.
Flowing		A mixture of soil and water flows into the tunnel like a viscous fluid. The material can enter the tunnel from the invert as well as from the face, crown, and walls, and can flow for great distances, completely filling the tunnel in some cases.	Below the water table in silt, sand, or gravel without enough clay content to give significant cohesion and plasticity. May also occur in highly sensitive clay when such material is disturbed.
Swelling		Ground absorbs water, increases in volume, and expands slowly into the tunnel.	Highly preconsolidated clay with plasticity index in excess of about 30, generally containing significant percentages of montmorillonite.

[1] Modified by Heuer (1974) from Terzaghi (1950)

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TUNNELMANS GROUND CLASSIFICATION FOR SOILS

Rohnert Station
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Rohnert Park, California

Project No. 2647.001 Date: 9/12/2018

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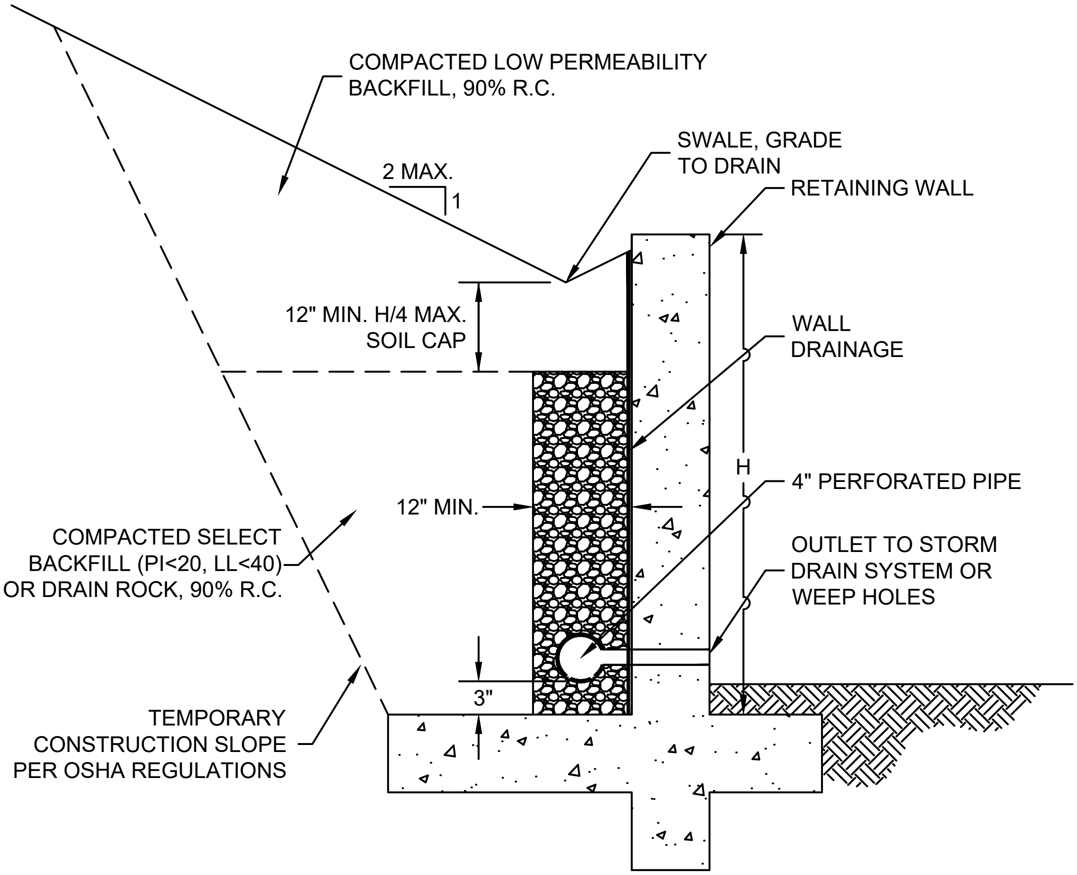
8
FIGURE



NOTES:

1. Wall drainage should consist of clean, free draining 3/4 inch crushed rock (Class 1B Permeable Material) wrapped in filter fabric (Mirafi 140N or equivalent) or Class 2 Permeable Material. Alternatively, pre-fabricated drainage panels (Miradrain G100N or equivalent), installed per the manufacturers recommendations, may be used in lieu of drain rock and fabric.

2. All retaining walls adjacent to interior living spaces shall be water/vapor proofed as specified by the project architect or structural engineer.

3. Perforated pipe shall be SCH 40 or SDR 35 for depths less than 20 feet. Use SCH 80 or SDR 23.5 perforated pipe for depths greater than 20 feet. Place pipe perforations down and slope at 1% to a gravity outlet. Alternatively, drainage can be outlet through 3" diameter weep holes spaced approximately 20' apart.

4. Clean outs should be installed at the upslope end and at significant direction changes of the perforated pipe. Additionally, all angled connectors shall be long bend sweep connections.

5. During compaction, the contractor should use appropriate methods (such as temporary bracing and/or light compaction equipment) to avoid over-stressing the walls. Walls shall be completely backfilled prior to construction in front of or above the retaining wall.

6. Refer to the geotechnical report for lateral soil pressures.

7. All work and materials shall conform with Section 68, of the latest edition of the Caltrans Standard Specifications.

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SCHEMATIC RETAINING WALL BACKDRAIN

Rohnert Station
6400 State Farm Drive
Rohnert Park, California

Project No. 2647.001
Date: 9/12/2018

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9
FIGURE

APPENDIX A
SUBSURFACE EXPLORATION AND LABORATORY TESTING

A. SUBSURFACE EXPLORATION

We explored subsurface conditions near the proposed improvements with six CPTs and eight soil borings at the approximate locations shown on Figure 6. The CPTs were pushed to depths ranging from about 55 to 116 feet below ground surface while the borings were excavated to depths ranging from about 3.5 to 51.5 feet below ground surface using truck-mounted drilling equipment. The subsurface conditions encountered during our exploration are summarized and presented on the CPT and boring logs, Figures A-1 through A-20. The depth to groundwater was noted during drilling and measured before backfilling the borings.

“Undisturbed” samples were obtained from the soil boring using a 3-inch diameter, split-barrel Modified California Sampler with 2.5 by 6-inch tube liners or a Standard Penetration Test (SPT) Sampler. The samplers were driven by a 140-pound hammer at a 30-inch drop. The number of blows required to drive the samplers 18 inches was recorded and is reported on the boring logs as blows per foot for the last 12 inches of driving. The samples obtained were examined in the field, sealed to prevent moisture loss, and transported to our laboratory.

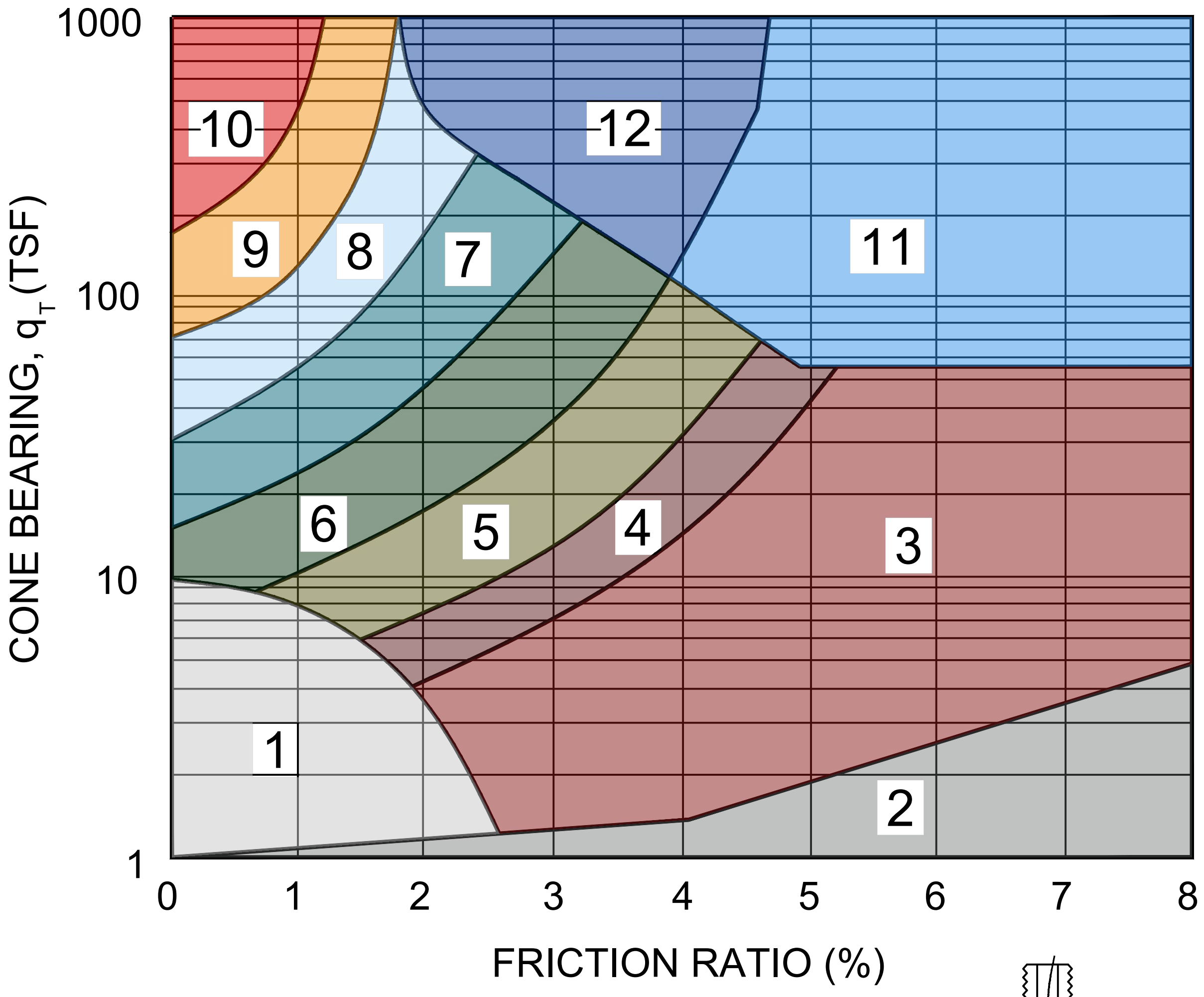
B. LABORATORY TESTING

We conducted laboratory tests on selected intact samples to classify soils and to estimate engineering properties. The following laboratory tests were conducted in general accordance with the test method cited:

- Laboratory Determination of Water (Moisture Content) of Soil, Rock, and Soil-Aggregate Mixtures, ASTM D 2216
- Density of Soil in Place by the Drive-Cylinder Method, ASTM D2937
- Unconfined Compressive Strength of Cohesive Soil, ASTM D2166
- Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index, ASTM D4318
- Expansion Index of Soil, ASTM D4829
- Amount of Material in Soils Finer than No. 200 (75-μm) Sieve, ASTM D1140
- Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis, ASTM D6913
- Standard Test Method for Using pH to Estimate the Soil-Lime Proportion Requirements for Soil Stabilization, ASTM D6276

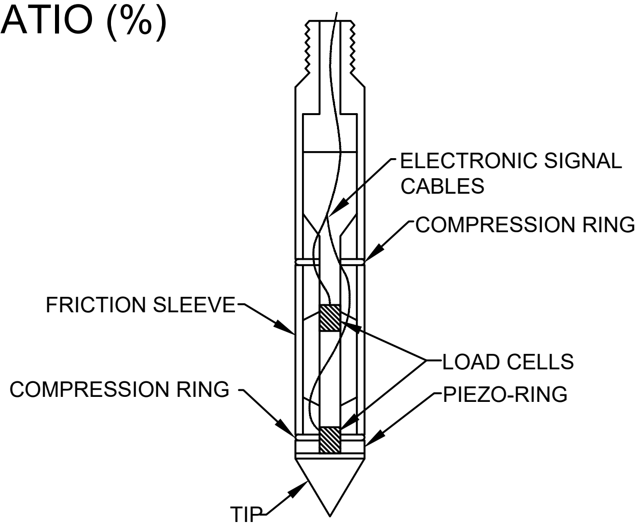
The moisture content, dry density, amount of material passing a No. 200 sieve and unconfined compression test results are shown on the exploratory boring logs, whereas the Atterberg Limits, Expansion Index, gradation and Soil-Lime Proportion test results are shown on Figures A-21 through A-25.

The exploratory boring logs, description of soils encountered, and the laboratory test data reflect conditions only at the location of the boring at the time they were excavated or retrieved. Conditions may differ at other locations and may change with the passage of time due to a variety of causes including natural weathering, climate and changes in surface and subsurface drainage.



Zone:	Qc/N	Soil Behavior Type:
1)	2	Sensitive Fine Grained
2)	1	Organic Material
3)	1	Clay
4)	1.5	Silty Clay to Clay
5)	2	Clayey Silt to Silty Clay
6)	2.5	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	Very Stiff Fine Grained (*)
12)	2	Sand to Clayey Sand (*)

(*) Overconsolidated or Cemented



CONE PENETROMETER

(NO SCALE)

Reference: Robertson, P.K. (1986), "In-Situ Testing and Its Application to Geotechnical Engineering," Canadian Geotechnical Journal, Vol. 23; No. 23; No. 4, pp. 573-594

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CPT SOIL INTERPRETATION CHART

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Project No. 2647.001 Date: 6/14/2018

Drawn MMT
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A-1
FIGURE

Miller Pacific Engineering

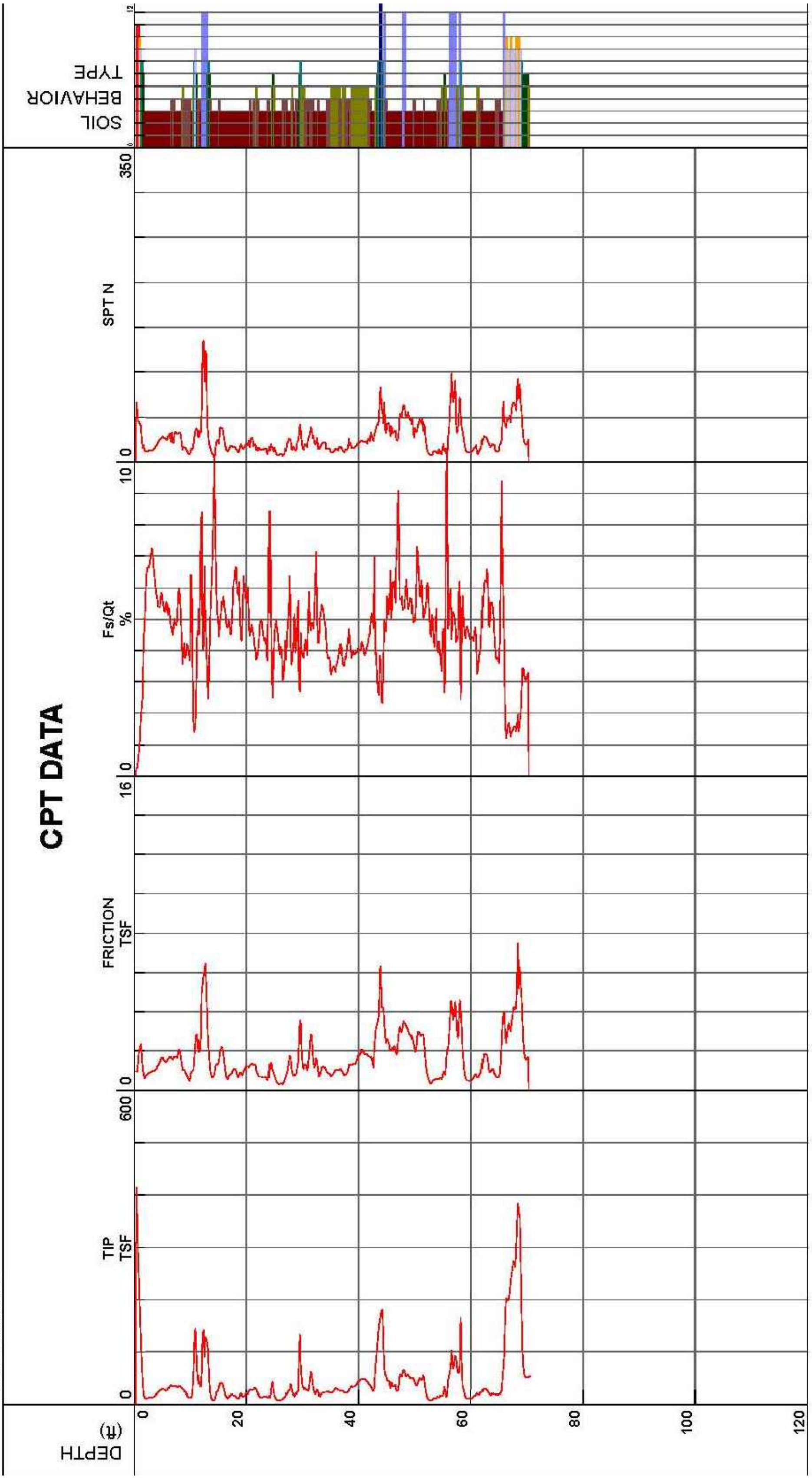
Project Rohnert Station Mixed Use Developmen
Job Number 2647.001
Hole Number CPT-01
EST GW Depth During Test 9.00 ft

Operator RB-JM
Cone Number DDG1418
Date and Time 5/29/2018 11:58:35 AM

Filename SDF(096).cpt
GPS
Maximum Depth 70.70 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

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CPT-1 DATA

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Rohnert Park, California

Project No. 2647.001 Date: 6/14/2018

Drawn MMT
Checked

A-2
FIGURE

Miller Pacific Engineering

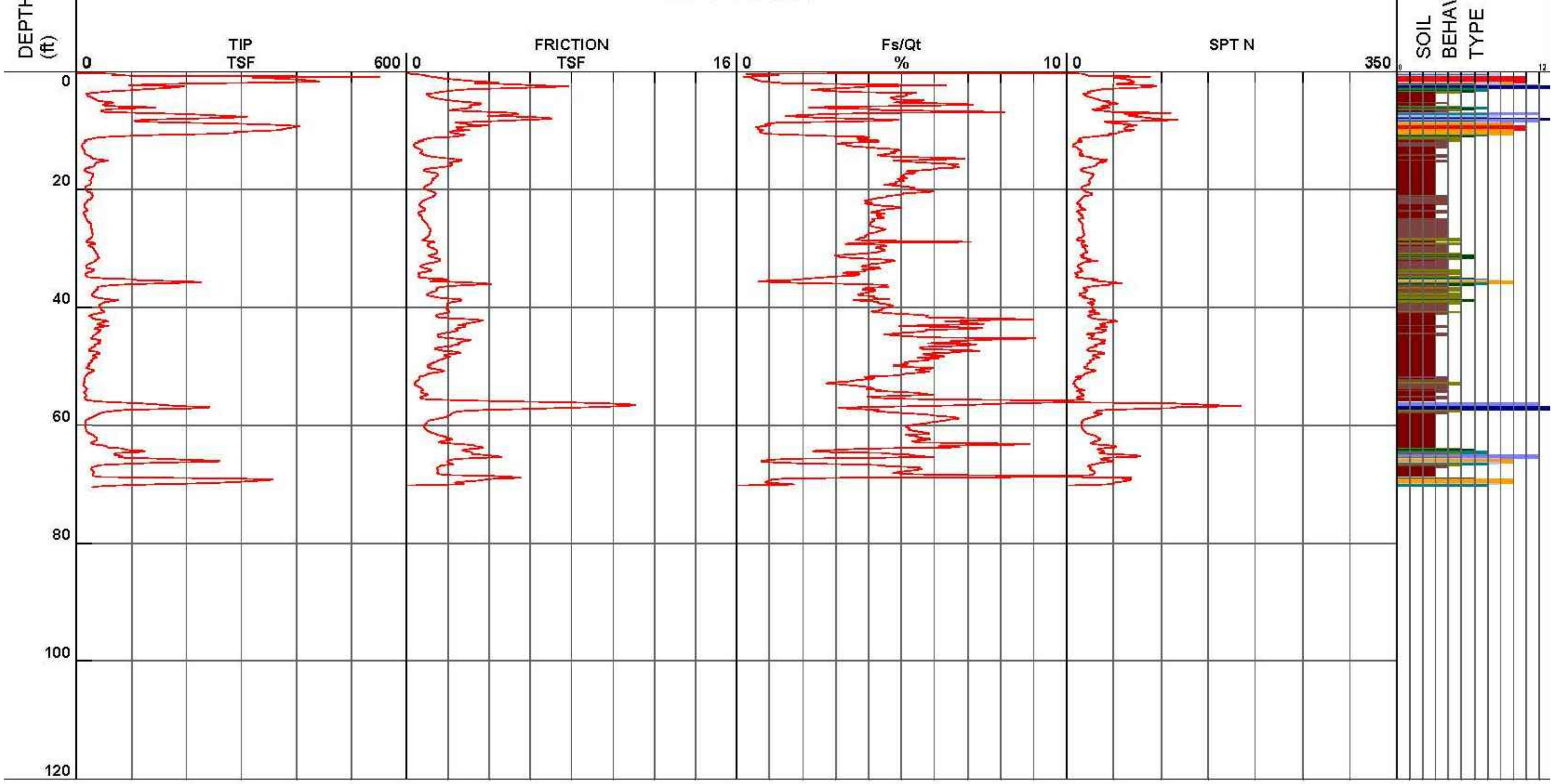
Project: Rohnert Station Mixed Use Developmen
Job Number: 2647.001
Hole Number: CPT-02
EST GW Depth During Test: 9.00 ft

Operator: RB-JM
Cone Number: DDG1418
Date and Time: 5/29/2018 11:01:34 AM

Filename: SDF(095).cpt
GPS:
Maximum Depth: 70.54 ft

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

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

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CPT-2 DATA

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Project No. 2647.001
Date: 6/14/2018

Drawn MMT
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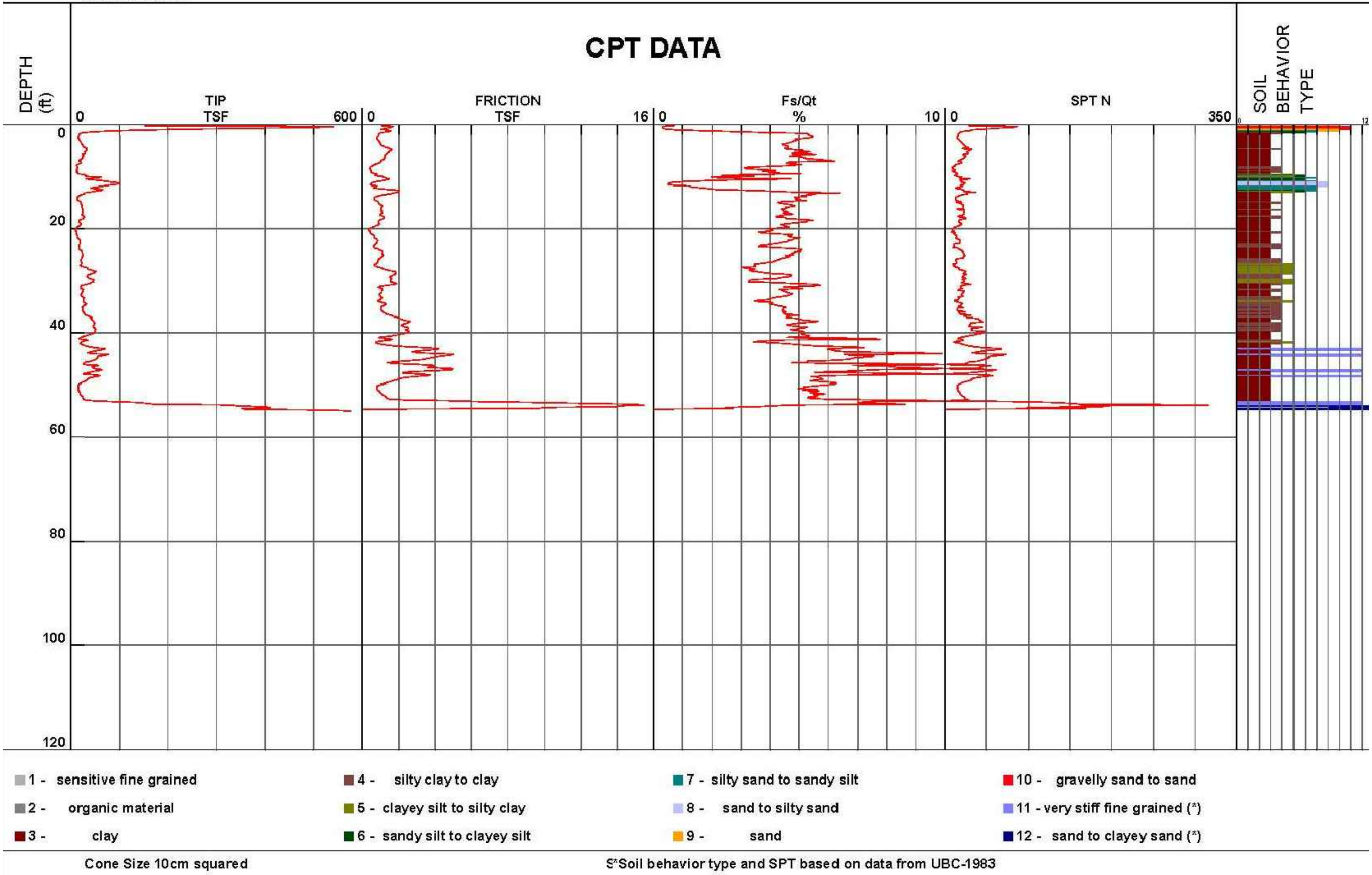
A-3
FIGURE

Miller Pacific Engineering

Project	Rohnert Station Mixed Use Developmen	Operator	RB-JM	Filename	SDF(098).cpt
Job Number	2647.001	Cone Number	DDG1418	GPS	
Hole Number	CPT-03	Date and Time	5/29/2018 2:00:26 PM	Maximum Depth	54.95 ft
EST GW Depth During Test			7.00 ft		

Net Area Ratio .8

CPT DATA



Cone Size 10cm squared

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

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CPT-3 DATA

Rohnert Station
6400 State Farm Drive
Rohnert Park, California

Project No. 2647.001
Date: 6/14/2018

Drawn MMT
Checked

A-4
FIGURE

Miller Pacific Engineering

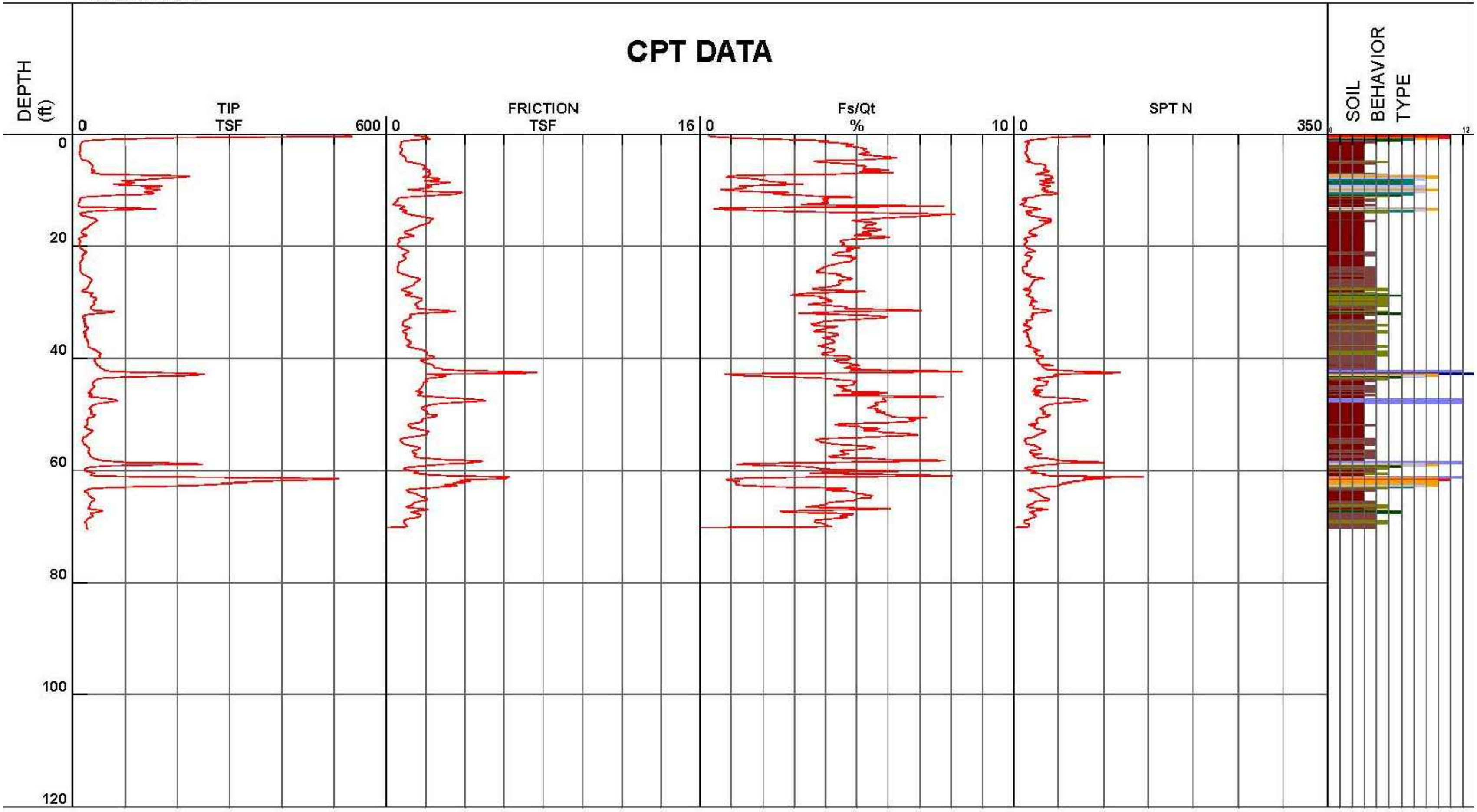
Project Rohnert Station Mixed Use Developmen
Job Number 2647.001
Hole Number CPT-04
EST GW Depth During Test 11.70 ft

Operator RB-JM
Cone Number DDG1418
Date and Time 5/29/2018 10:01:42 AM

Filename SDF(094).cpt
GPS
Maximum Depth 70.54 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

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CPT-4 DATA

Rohnert Station
6400 State Farm Drive
Rohnert Park, California

Project No. 2647.001 Date: 6/14/2018

Drawn MMT
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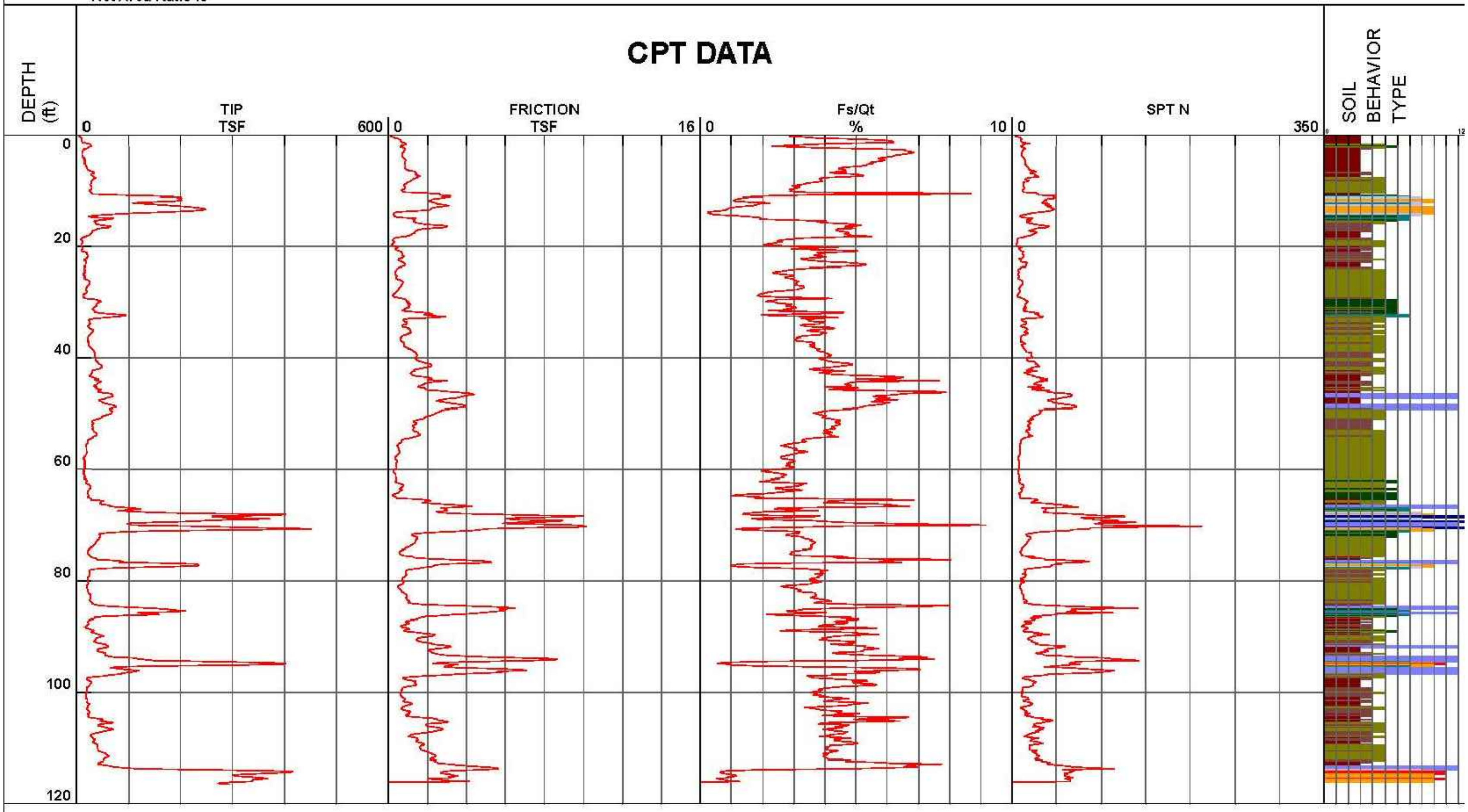
A-5
FIGURE

Miller Pacific Engineering

Project	Rohnert Station Mixed Use Development	Operator	RB-JM	Filename	SDF(093).cpt
Job Number	2647.001	Cone Number	DDG1418	GPS	
Hole Number	CPT-05	Date and Time	5/29/2018 8:26:15 AM	Maximum Depth	116.47 ft
EST GW Depth During Test			9.70 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

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CPT-5 DATA

Rohnert Station
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Rohnert Park, California

Project No. 2647.001 Date: 6/14/2018

Drawn MMT
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A-6
FIGURE

Miller Pacific Engineering

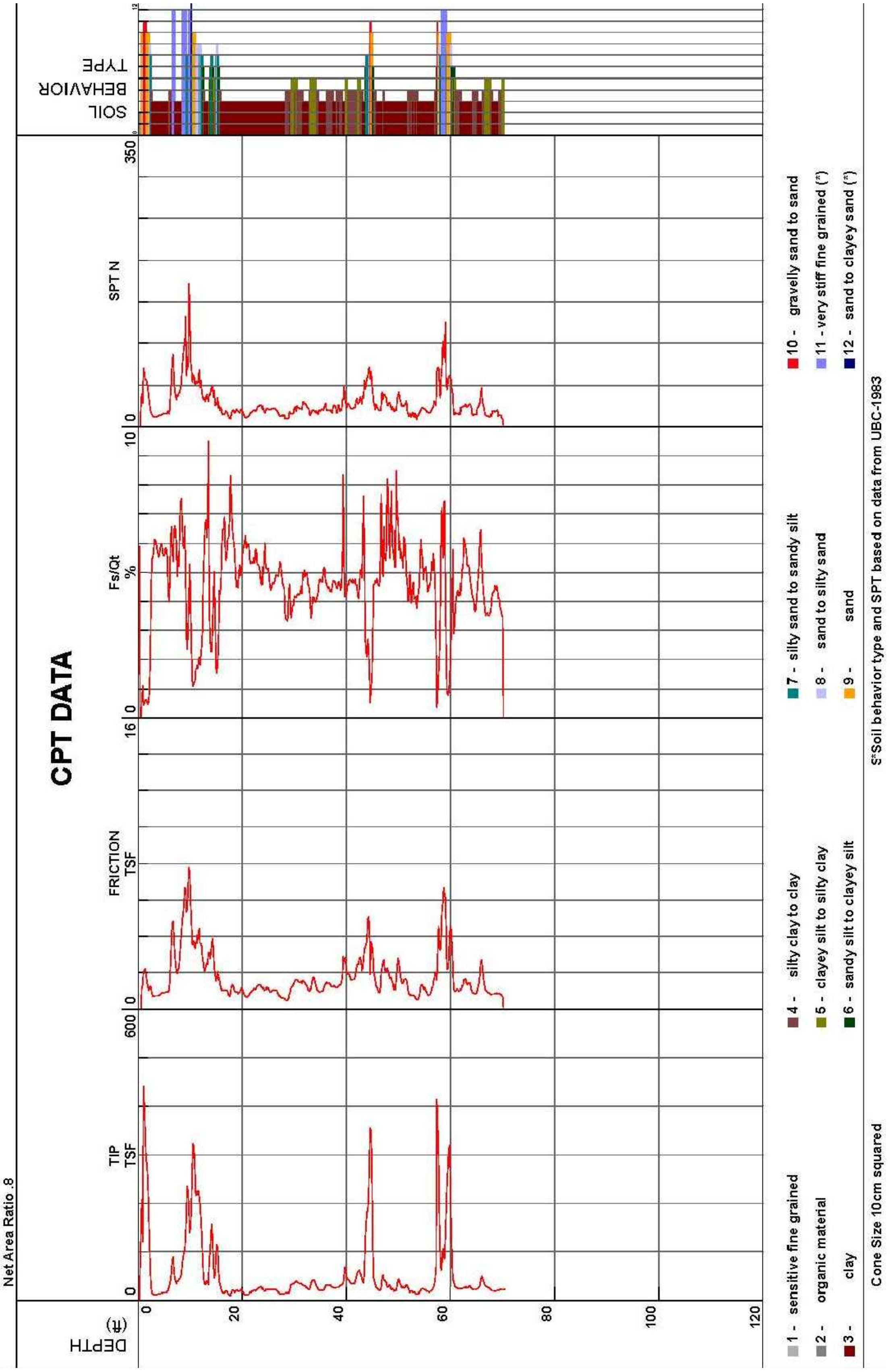
Project Rohnert Station Mixed Use Development
Job Number 2647.001
Hole Number CPT-06
EST GW Depth During Test 14.00 ft

Operator RB-JM
Cone Number DDG1418
Date and Time 5/29/2018 12:55:38 PM

Filename SDF(097).cpt
GPS
Maximum Depth 70.54 ft

Net Area Ratio .8

CPT DATA



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CPT-6 DATA

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Project No. 2647.001 Date: 6/14/2018

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A-7
FIGURE

MAJOR DIVISIONS		SYMBOL		DESCRIPTION
COARSE GRAINED SOILS over 50% sand and gravel	CLEAN GRAVEL	GW	▨	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	▨	Poorly-graded gravels or gravel-sand mixtures, little or no fines
	GRAVEL with fines	GM	▨	Silty gravels, gravel-sand-silt mixtures
		GC	▨	Clayey gravels, gravel-sand-clay mixtures
	CLEAN SAND	SW	▨	Well-graded sands or gravelly sands, little or no fines
		SP	▨	Poorly-graded sands or gravelly sands, little or no fines
	SAND with fines	SM	▨	Silty sands, sand-silt mixtures
		SC	▨	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS over 50% silt and clay	SILT AND CLAY liquid limit <50%	ML	▨	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
		CL	▨	Inorganic clays of low to medium plasticity, gravely clays, sandy clays, silty clays, lean clays
		OL	▨	Organic silts and organic silt-clays of low plasticity
	SILT AND CLAY liquid limit >50%	MH	▨	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
		CH	▨	Inorganic clays of high plasticity, fat clays
		OH	▨	Organic clays of medium to high plasticity
HIGHLY ORGANIC SOILS		PT	▨	Peat, muck, and other highly organic soils
ROCK			▨	Undifferentiated as to type or composition

KEY TO BORING AND TEST PIT SYMBOLS

CLASSIFICATION TESTS

PI	PLASTICITY INDEX
LL	LIQUID LIMIT
SA	SIEVE ANALYSIS
HYD	HYDROMETER ANALYSIS
P200	PERCENT PASSING NO. 200 SIEVE
P4	PERCENT PASSING NO. 4 SIEVE

SAMPLER TYPE

■	MODIFIED CALIFORNIA	⊠	HAND SAMPLER
▯	STANDARD PENETRATION TEST	⊠	ROCK CORE
▨	THIN-WALLED / FIXED PISTON	X	DISTURBED OR BULK SAMPLE

NOTE: Test boring and test pit logs are an interpretation of conditions encountered at the excavation location during the time of exploration. Subsurface rock, soil or water conditions may vary in different locations within the project site and with the passage of time. Boundaries between differing soil or rock descriptions are approximate and may indicate a gradual transition.

STRENGTH TESTS

TV	FIELD TORVANE (UNDRAINED SHEAR)
UC	LABORATORY UNCONFINED COMPRESSION
TXCU	CONSOLIDATED UNDRAINED TRIAXIAL
TXUU	UNCONSOLIDATED UNDRAINED TRIAXIAL
	UC, CU, UU = 1/2 Deviator Stress

SAMPLER DRIVING RESISTANCE

Modified California and Standard Penetration Test samplers are driven 18 inches with a 140-pound hammer falling 30 inches per blow. Blows for the initial 6-inch drive seat the sampler. Blows for the final 12-inch drive are recorded onto the logs. Sampler refusal is defined as 50 blows during a 6-inch drive. Examples of blow records are as follows:

25	sampler driven 12 inches with 25 blows after initial 6-inch drive
85/7"	sampler driven 7 inches with 85 blows after initial 6-inch drive
50/3"	sampler driven 3 inches with 50 blows during initial 6-inch drive or beginning of final 12-inch drive

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SOIL CLASSIFICATION CHART

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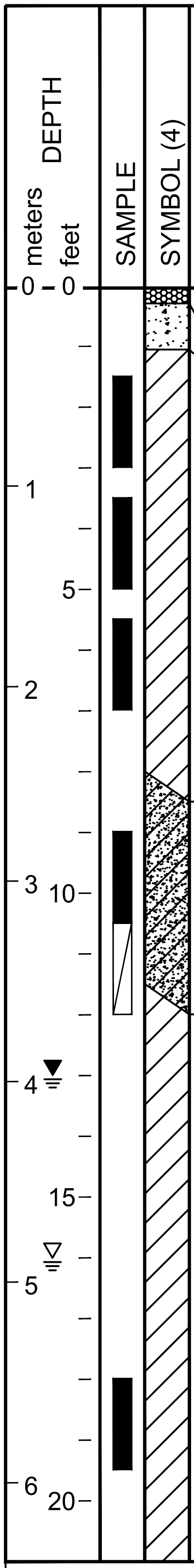
Project No. 2647.001
Date: 9/12/2018

Drawn NGK
Checked

A-8
FIGURE

BORING 1

EQUIPMENT: Truck-Mounted Mobile B53 with 6-inch Hollow Stem Augers
DATE: 8/8/18
ELEVATION: 110 feet*
*REFERENCE: Google Earth, 2018



	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
3.5" Asphalt Concrete						
9.0" Aggregate Baserock						
CLAY (CH) dark gray, black, moist, stiff, high plasticity clay [Alluvium]	16	77	37.8	UC 1700		
	15	84	36.9			
	29	81	38.2	UC 3000		
Clayey SAND with Gravel (SC) brown, moist, medium dense, fine sand [Alluvium]	35	101	16.2		P200 20.8%	
	15		21.0			
Sandy CLAY (CH) brown, moist to wet, stiff, high plasticity [Alluvium]						
grades soft to medium stiff	5	81	38.6	UC 550		

▽ Water level encountered during drilling
▼ Water level measured after drilling

NOTES:
(1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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BORING LOG

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A-9
FIGURE

BORING 1
(CONTINUED)

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)		BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
	20			Sandy CLAY (CH) brown, moist to wet, stiff, high plasticity [Alluvium]						
				CLAY with Sand (CH) light brown, wet, stiff, medium to high plasticity [Alluvium]						
7										
	25	■								
8					17	90	31.1	UC 1350		
9	30									
10										
	35	▯								
				grades dark gray, very stiff						
11					20		35.9			
12	40									

▽ Water level encountered during drilling

▼ Water level measured after drilling

NOTES:
(1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3= 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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A-10
FIGURE

BORING 1
(CONTINUED)

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)		BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
	40		▨	CLAY with Sand (CH) dark gray, wet, very stiff, medium to high plasticity [Alluvium]						
13		■	▨		46	103	22.3			
	45		▨							
14			▨							
15	50	▯	▨		25		30.3			
				Bottom of boring at 50.0 feet. Groundwater encountered at 16.0 feet during drilling and measured at 13.0 feet immediately after drilling						
16										
	55									
17										
18										
	60									

▽ Water level encountered during drilling

▼ Water level measured after drilling

NOTES:
(1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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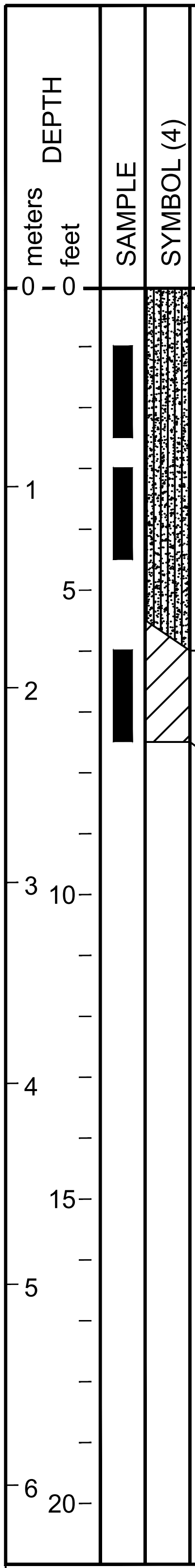
A-11
FIGURE

BORING 2

EQUIPMENT: Truck-Mounted Mobile B53 with 4-inch Solid Flight Augers
DATE: 8/9/18
ELEVATION: 108 feet*
*REFERENCE: Google Earth, 2018



	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
Silty SAND with Gravel (SM) light brown, dry, medium dense, fine sand [Fill]	31	75	22.0		P200 15.6%	SA
	48	78	29.4			
CLAY (CH) light gray, moist, stiff, high plasticity [Alluvium]	17	99	25.1	UC 2850		
Bottom of boring at 7.5 feet. No groundwater observed during drilling.						

▽ Water level encountered during drilling
▼ Water level measured after drilling

NOTES:
(1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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A-12
FIGURE

BORING 3

EQUIPMENT: Truck-Mounted Mobile B53 with 4-inch Solid Flight Augers
DATE: 8/9/18
ELEVATION: 106 feet*
*REFERENCE: Google Earth, 2018

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)		BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
0	0		▩	Clayey SAND with Gravel (SC) light brown, dry, medium dense, fine sand [Fill]						
		■	▩			93	15.2			
1		■	▨	CLAY (CH) dark gray to black, moist, very stiff, high plasticity [Alluvium]	10	79	39.1	UC 1150		
	5	■	▨		27	87	34.1			
2				Bottom of boring at 5.5 feet. No groundwater observed during drilling.						
3	10									
4										
	15									
5										
6	20									

▽ Water level encountered during drilling
▼ Water level measured after drilling

NOTES:
(1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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A-13
FIGURE

BORING 4

EQUIPMENT: Truck-Mounted Mobile B53 with 4-inch Solid Flight Augers
DATE: 8/9/18
ELEVATION: 108 feet*
*REFERENCE: Google Earth, 2018

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)		BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
0	0			2" Asphalt Concrete						
				10.5" Aggregate Baserock						
1		■	▨	CLAY (CH) Black, moist, medium stiff, high plasticity clay. [Alluvium]	12	79	40.0			
				Bottom of boring at 3.5 feet. No groundwater observed during drilling.						
	5									
2										
3	10									
4										
	15									
5										
6	20									

▽ Water level encountered during drilling
▼ Water level measured after drilling

NOTES:
(1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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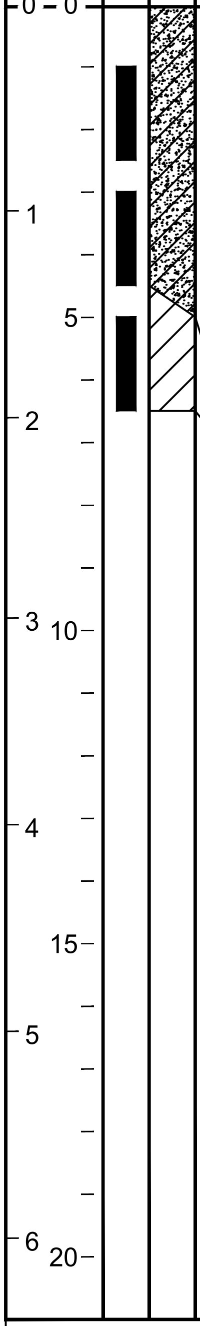
FIGURE

BORING 5

EQUIPMENT: Truck-Mounted Mobile B53 with 4-inch Solid Flight Augers
DATE: 8/9/18
ELEVATION: 107 feet*
*REFERENCE: Google Earth, 2018



	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
Clayey SAND with Gravel (SC) brown and gray, moist, medium dense, fine to coarse sand [Fill]						
	43	111	12.6	UC 1200		
	46	104	14.2		P200 19.3%	
CLAY (CH) black, moist, stiff, high plasticity [Alluvium]	17	78	41.6			
Bottom of boring at 6.5 feet No groundwater observed during drilling						

∇ Water level encountered during drilling
▼ Water level measured after drilling

NOTES:
(1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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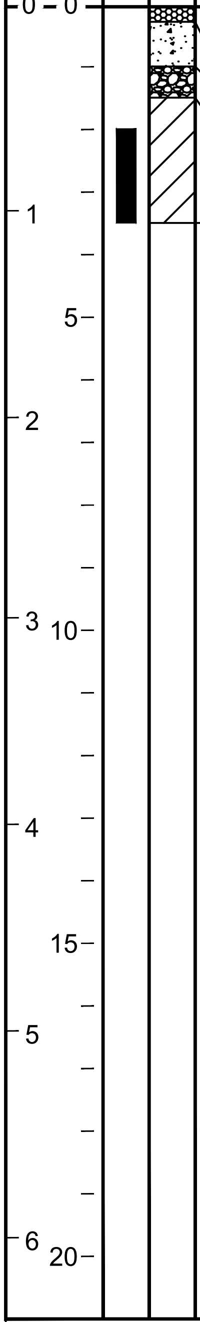
FIGURE

BORING 6

EQUIPMENT: Truck-Mounted Mobile B53 with 4-inch Solid Flight Augers
DATE: 8/9/18
ELEVATION: 105 feet*
*REFERENCE: Google Earth, 2018



	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
3" Asphalt Concrete						
9" Aggregate Baserock						
6" Gravelly Fill						
CLAY (CH) black, moist, stiff, high plasticity [Alluvium]	18	72	45.2			
Bottom of boring at 3.5 feet No groundwater observed during drilling						

▽ Water level encountered during drilling
▼ Water level measured after drilling

NOTES:
(1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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A-16
FIGURE

BORING 7

EQUIPMENT: Truck-Mounted Mobile B53 with 4-inch Solid Flight Augers
DATE: 8/9/18
ELEVATION: 103 feet*
*REFERENCE: Google Earth, 2018

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)		BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
0	0			3.5" Asphalt Concrete						
				9.5" Aggregate Baserock						
				3" Gravelly Fill						
		■	▨	CLAY (CH) black, moist, stiff, high plasticity [Alluvium]	14	72	28.2			
1		■	▨		32	76	36.9			
	5			Bottom of boring at 4.5 feet No groundwater observed during drilling						
2										
3	10									
4										
	15									
5										
6	20									

▽ Water level encountered during drilling
▼ Water level measured after drilling

NOTES:
(1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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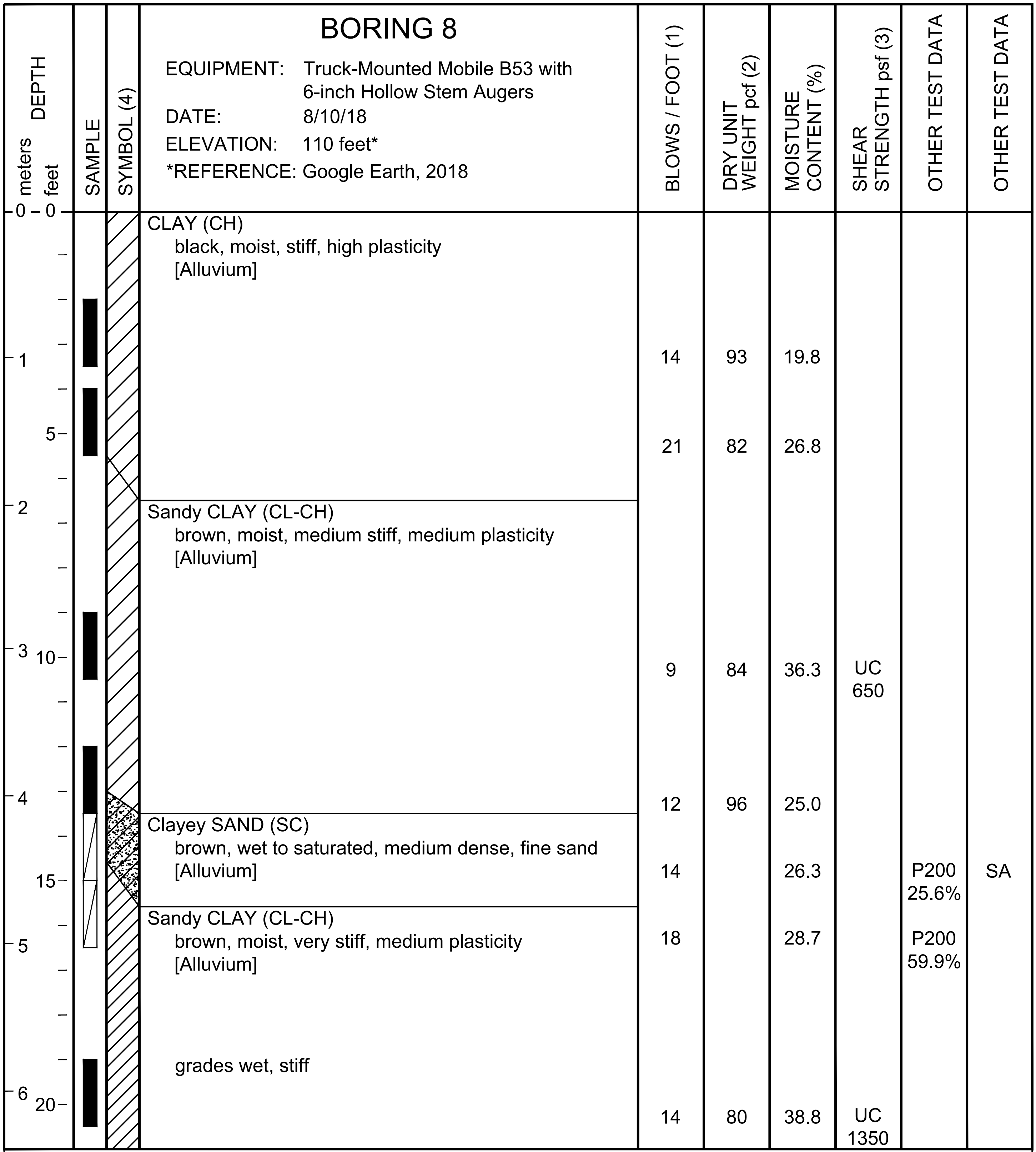
A-17
FIGURE

BORING 8

EQUIPMENT: Truck-Mounted Mobile B53 with 6-inch Hollow Stem Augers
DATE: 8/10/18
ELEVATION: 110 feet*
*REFERENCE: Google Earth, 2018



CLAY (CH)
black, moist, stiff, high plasticity
[Alluvium]

Sandy CLAY (CL-CH)
brown, moist, medium stiff, medium plasticity
[Alluvium]

Clayey SAND (SC)
brown, wet to saturated, medium dense, fine sand
[Alluvium]

Sandy CLAY (CL-CH)
brown, moist, very stiff, medium plasticity
[Alluvium]

grades wet, stiff

BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
14	93	19.8			
21	82	26.8			
9	84	36.3	UC 650		
12	96	25.0			
14		26.3		P200 25.6%	SA
18		28.7		P200 59.9%	
14	80	38.8	UC 1350		

▽ Water level encountered during drilling
▼ Water level measured after drilling

NOTES:
(1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3= 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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A-18
FIGURE

meters feet DEPTH	SAMPLE	SYMBOL (4)	BORING 8 (CONTINUED)	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
20		▨	Sandy CLAY (CL-CH) brown, wet, stiff, medium plasticity [Alluvium]						
7		▨							
25	■	▨	Silty CLAY (CL-CH) brown, wet, stiff, medium plasticity [Alluvium]	15	90	32.0	UC 1500		
8		▨							
9	■	▨	grades blue-gray						
30	■	▨		17	92	31.4	UC 1450		
10		▨							
35	■	▨		26	107	22.4	UC 1150		
11		▨							
12		▨							
40		▨							

▽ Water level encountered during drilling

▼ Water level measured after drilling

NOTES:
(1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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FIGURE

BORING 8
(CONTINUED)

DEPTH meters / feet	SAMPLE	SYMBOL (4)		BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
40 13		▨	Silty CLAY (CL-CH) blue-gray, wet, stiff, medium plasticity [Alluvium]						
45 14	■	▩	Clayey SAND (SC) blue-gray, wet, dense, fine to coarse sand [Alluvium]	73	102	23.5	UC 1850	P200 43.1%	SA
15 50	▯	▨	CLAY (CH) Blue-gray, moist, stiff, high plasticity clay, trace sand. [Alluvium]	24		32.1			
16 55 17 18 60			Bottom of boring at 51.5 feet. Groundwater observed during drilling at 13.0 feet. Groundwater measured at 13.0 feet immediately after drilling.						

∇ Water level encountered during drilling

▼ Water level measured after drilling

NOTES:
(1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3= 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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BORING LOG

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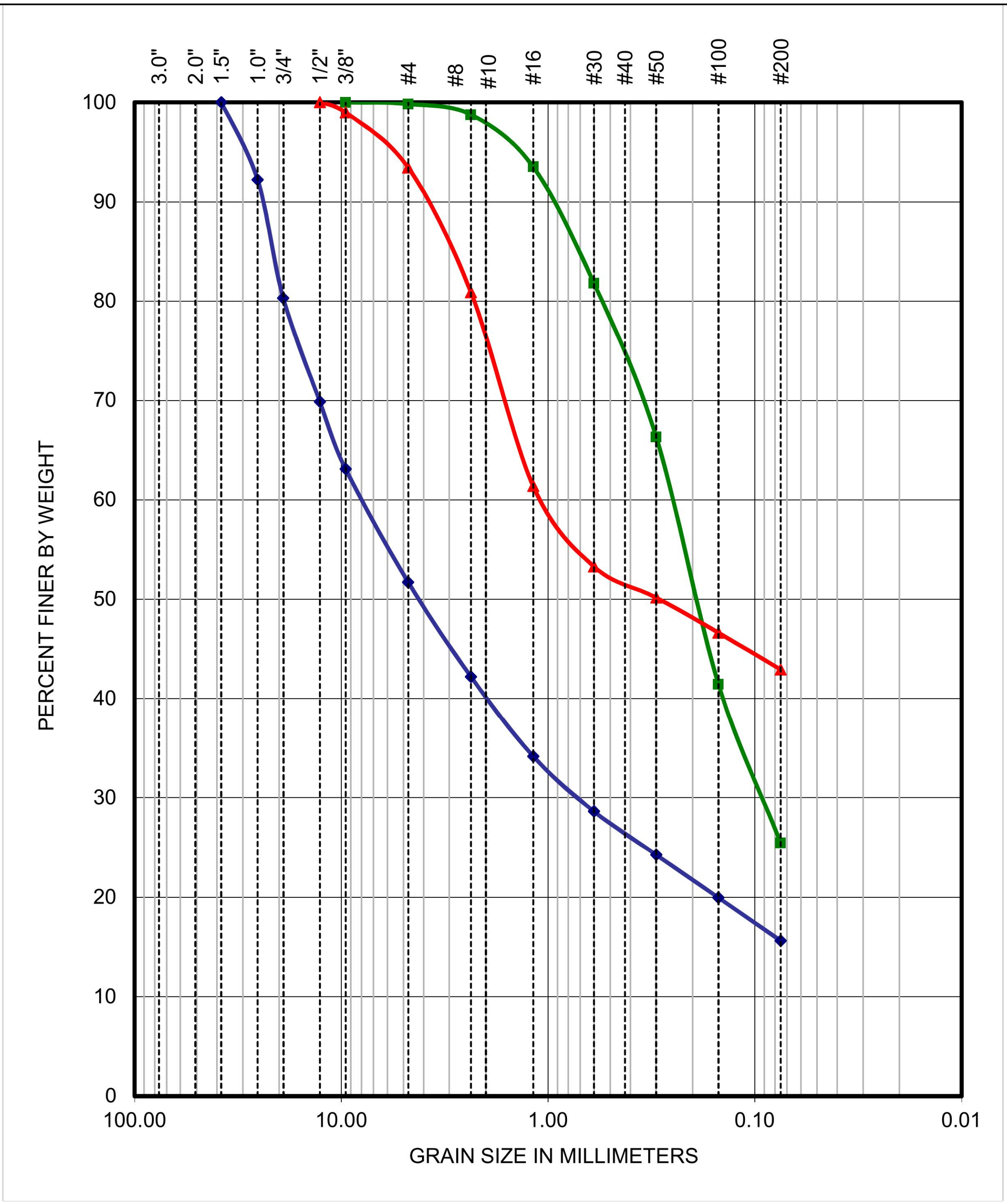
FIGURE



SYMBOL	SAMPLE SOURCE	CLASSIFICATION
—◆—	B2 at 2.0 ft	Clayey GRAVEL with Sand (GC)
—■—	B8 at 13.5 ft	Clayey SAND (SC)
—▲—	B8 at 45.0 ft	Clayey SAND (SC)

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SIEVE ANALYSIS RESULTS

Rohnert Station
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Project No. 2647.001
Date: 9/12/2018

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A-21
FIGURE

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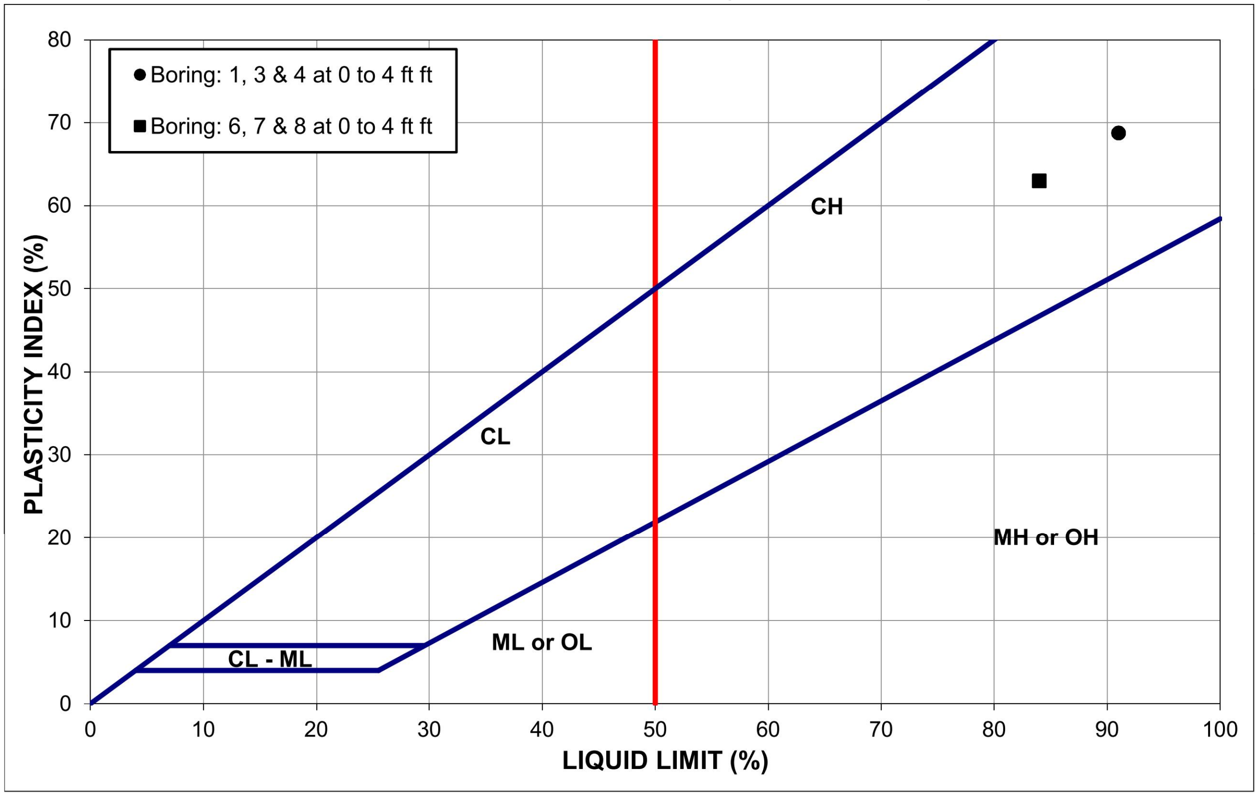
ATTERBERG LIMITS TEST (ASTM D 4318)



Sample	Classification	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
Boring: 1, 3 & 4 at 0 to 4 ft ft	CLAY (CH) dark gray to black	91	22	69
Boring: 6, 7 & 8 at 0 to 4 ft ft	CLAY (CH) dark gray to black	84	21	63

PI = 0-3: Non-Plastic
PI = 3-15: Slightly Plastic
PI = 15-30: Medium Plasticity
PI = >30: High Plasticity

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ATTERBERG LIMITS TEST RESULTS

Rohnert Station
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Project No. 2647.001 Date: 9/12/2018

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A-22
FIGURE

EXPANSION INDEX (ASTM D4829)

Project Name: ROHNERT STATION
Tested By: NAR
Date: 9/11/2018

Project Number: 2647.001
Sample Source: B1, B3, & B4 (Composite) at 2.0 to 4.0 ft

Sample Description: CLAY (CH)

Sample Height before Saturation (in.): 1.000

Sample/Ring Diameter: (in.): 4.000

Sample Volume before Saturation (cu.ft.): 0.007272

Ring Number: EI-4

Ring Tare (gm): 367

Weight Ring + Moist Soil (gm): 693.2

Approx. Moisture Content (%): 18.0%
Estimated Specific Gravity (2.60-2.70): 2.68
Approx. EI Dry Density (pcf): 90.0

Approximate Saturation: 50.0%

Dial Readings with 1000 psf Load:

Start Time:	3:39	Dial Reading T0:	0.0000
Time 1:	3:48	Dial Reading T1:	0.1000
Time 2:	4:30	Dial Reading T2:	0.2034
Time 3:	6:45	Dial Reading T3:	0.2129
Time 4:	7:34	Dial Reading T4:	0.2131
Time 5:	12:00	Dial Reading T5:	0.2137

Final Height of Sample (in.): 1.214

Pan Identification: 19-T

Weight of Ring + Wet Soil + Pan (gm): 1226.0

Pan Tare (gm): 444.8

Weight of Ring + Dry Soil + Pan (gm): 1089.3

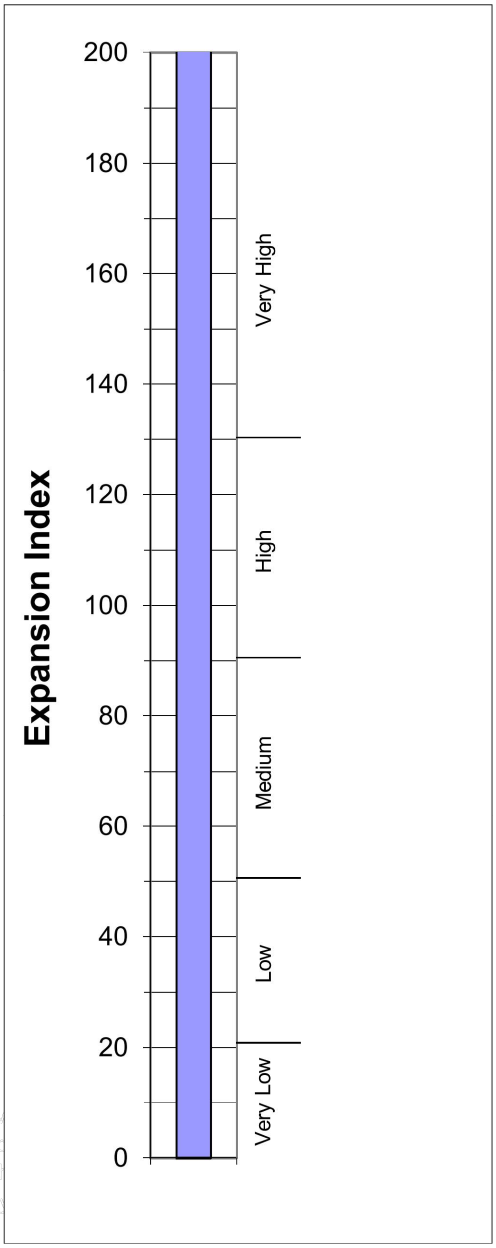
Initial Moisture Content: 17.5%
Initial Dry Density (pcf): 84.1
Prepared Sample Saturation: 47.6%

100
1000
0.0%

Percent Expansion: 21.4%

Final Moisture Content: 49.3%
Final Dry Density (pcf): 69.3
EI $_{50}$**:** 210
Potential Expansion Very High

20.5 Low
50.5 Med
90.5 Hig
130.5 Ver



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FILENAME: 2647.001 BL and Lab Results.dwg

EXPANSION INDEX TEST RESULTS

Rohnert Station
6400 State Farm Drive
Rohnert Park, California
Project No. 2647.001
Date: 9/12/2018

Drawn NGK
Checked

A-23
FIGURE

EXPANSION INDEX (ASTM D4829)

Project Name: ROHNERT STATION
Tested By: NAR
Date: 9/11/2018

Project Number: 2647.001
Sample Source: B6, B7, & B8 (Composite) at 2.0 to 4.0 ft

Sample Description: CLAY (CH)

Sample Height before Saturation (in.): 1.000

Sample/Ring Diameter: (in.): 4.000

Sample Volume before Saturation (cu.ft.): 0.007272

Ring Number: EI-3

Ring Tare (gm): 366.2

Weight Ring + Moist Soil (gm): 707.8

Approx. Moisture Content (%): 17.0%
Estimated Specific Gravity (2.60-2.70): 2.68
Approx. EI Dry Density (pcf): 90.0

Approximate Saturation: 50.0%

Dial Readings with 1000 psf Load:

Start Time:	2;10	Dial Reading T0:	0.0000
Time 1:	2:17	Dial Reading T1:	0.0679
Time 2:	7:20	Dial Reading T2:	0.2340
Time 3:	9:15	Dial Reading T3:	0.2343
Time 4:	11:00	Dial Reading T4:	0.2345
Time 5:	11:15	Dial Reading T5:	0.2345

Final Height of Sample (in.): 1.235

Pan Identification: 18T

Weight of Ring + Wet Soil + Pan (gm): 1242.0

Pan Tare (gm): 443.2

Weight of Ring + Dry Soil + Pan (gm): 1100.6

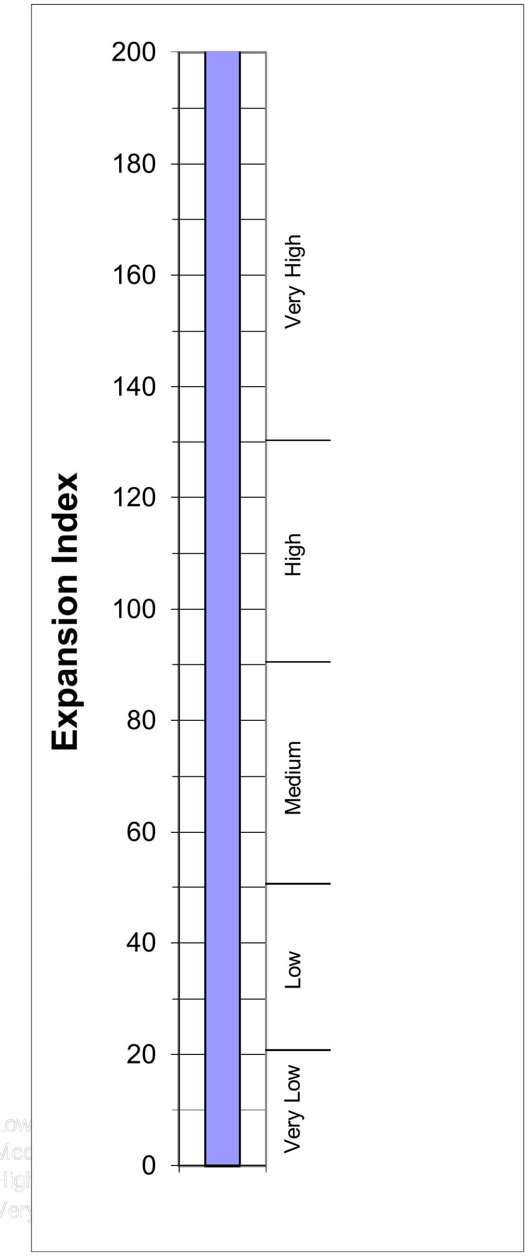
Initial Moisture Content: 17.3%
Initial Dry Density (pcf): 88.3
Prepared Sample Saturation: 51.9%

100
1000
0.0%

Percent Expansion: 23.5%

Final Moisture Content: 48.6%
Final Dry Density (pcf): 71.5
***EI*$_{50}$:** 238
Potential Expansion Very High

20.5 Low
50.5 Med
90.5 High
130.5 Very



MPEG
MILLER PACIFIC
ENGINEERING GROUP

FILENAME: 2647.001 BL and Lab Results.dwg

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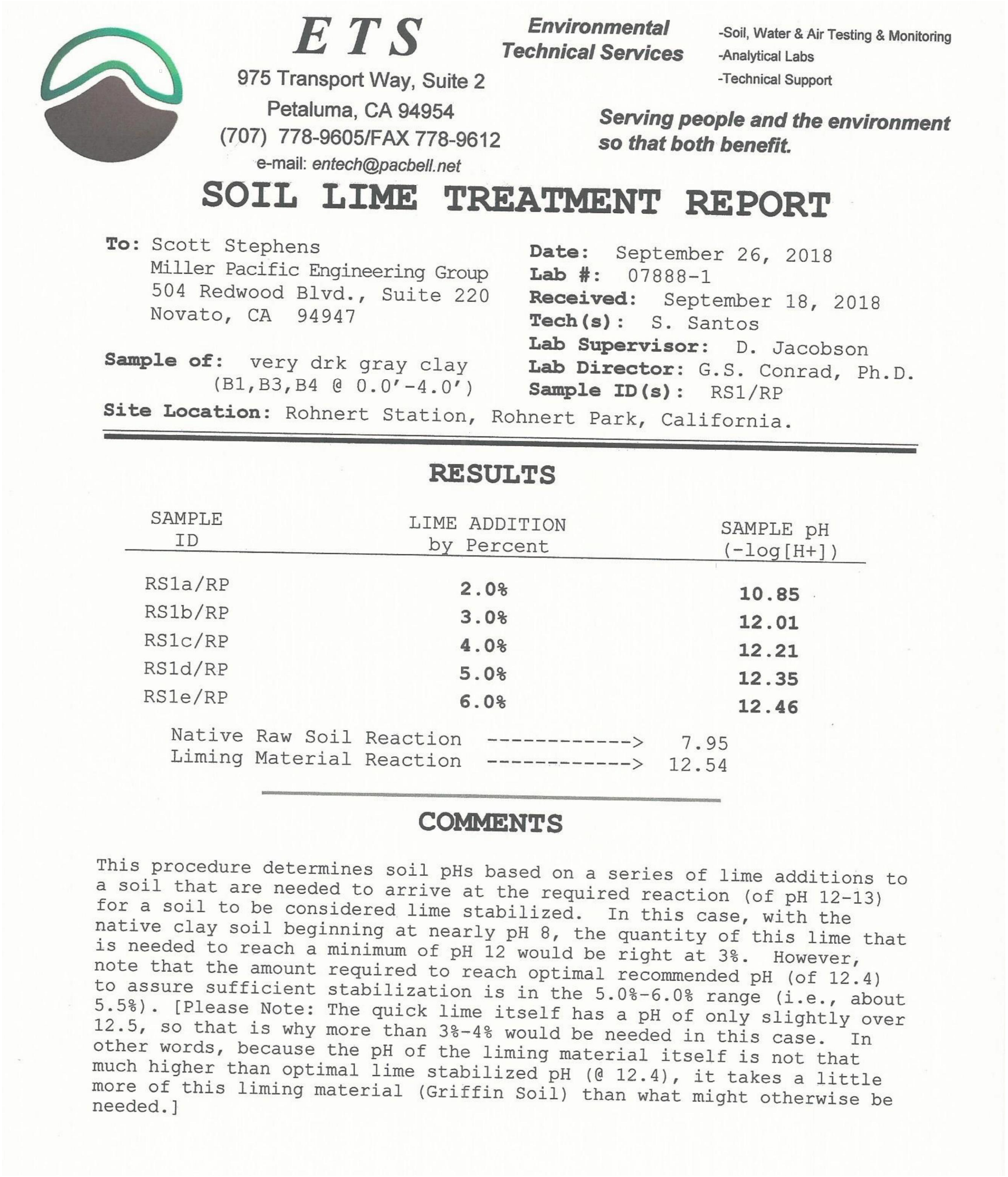
EXPANSION INDEX TEST RESULTS

Rohnert Station
6400 State Farm Drive
Rohnert Park, California
Project No. 2647.001
Date: 9/12/2018

Drawn NGK
Checked

A-24
FIGURE



ETS

975 Transport Way, Suite 2
Petaluma, CA 94954
(707) 778-9605/FAX 778-9612
e-mail: *entech@pacbell.net*

Environmental Technical Services

-Soil, Water & Air Testing & Monitoring
-Analytical Labs
-Technical Support

Serving people and the environment so that both benefit.

SOIL LIME TREATMENT REPORT

To: Scott Stephens
Miller Pacific Engineering Group
504 Redwood Blvd., Suite 220
Novato, CA 94947

Date: September 26, 2018
Lab #: 07888-1
Received: September 18, 2018
Tech(s): S. Santos
Lab Supervisor: D. Jacobson
Lab Director: G.S. Conrad, Ph.D.
Sample ID(s): RS1/RP

Sample of: very drk gray clay (B1,B3,B4 @ 0.0’-4.0’)

Site Location: Rohnert Station, Rohnert Park, California.

RESULTS

SAMPLE ID	LIME ADDITION by Percent	SAMPLE pH (-log[H+])
RS1a/RP	**2.0%**	**10.85**
RS1b/RP	**3.0%**	**12.01**
RS1c/RP	**4.0%**	**12.21**
RS1d/RP	**5.0%**	**12.35**
RS1e/RP	**6.0%**	**12.46**

Native Raw Soil Reaction ------------> 7.95
Liming Material Reaction ------------> 12.54

COMMENTS

This procedure determines soil pHs based on a series of lime additions to a soil that are needed to arrive at the required reaction (of pH 12-13) for a soil to be considered lime stabilized. In this case, with the native clay soil beginning at nearly pH 8, the quantity of this lime that is needed to reach a minimum of pH 12 would be right at 3%. However, note that the amount required to reach optimal recommended pH (of 12.4) to assure sufficient stabilization is in the 5.0%-6.0% range (i.e., about 5.5%). [Please Note: The quick lime itself has a pH of only slightly over 12.5, so that is why more than 3%-4% would be needed in this case. In other words, because the pH of the liming material itself is not that much higher than optimal lime stabilized pH (@ 12.4), it takes a little more of this liming material (Griffin Soil) than what might otherwise be needed.]

MPEG
MILLER PACIFIC ENGINEERING GROUP

FILENAME: 2647.001 BL and Lab Results.dwg

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SOIL LIME PROPORTION TEST RESULTS

Rohnert Station
6400 State Farm Drive
Rohnert Park, California

Project No. 2647.001 Date: 9/12/2018

Drawn NGK
Checked

A-25
FIGURE

APPENDIX B

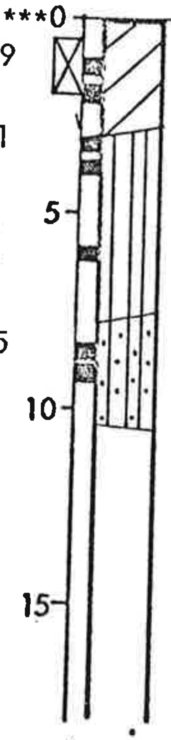
DATA FROM PREVIOUS GEOTECHNICAL INVESTIGATIONS

HARDING & LAWSON ASSOCIATES, 1976

LOG OF BORING 1

Equipment 6" Flight Auger
Elevation 100.5* Date 1/8/76

Shear Strength (lbs/sq ft) 4000 3000 2000 1000 0	Blows/foot**	Moisture Content (%)	Dry Density (pcf)
CU 580 (43.1)	18	17.9	89
***Remolded triaxial compression, classification, compaction and resistance value tests (see Plates 31, 27, 28, 34 and 35)	41	27.8	81
	24		
	11	26.7	95



BLACK CLAY (CH)
stiff, moist

GRAY-BROWN SANDY SILT (MH)
very stiff, moist

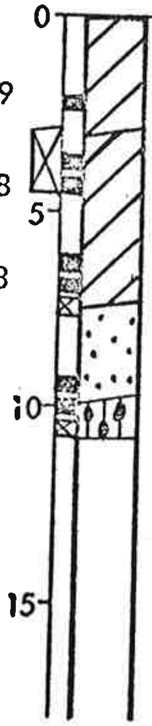
BROWN SILTY SAND (SM)
medium dense, wet

(no free water encountered)

LOG OF BORING 2

Equipment 6" Flight Auger
Elevation 101.2 Date 1/8/76

Shear Strength (lbs/sq ft)	Blows/foot**	Moisture Content (%)	Dry Density (pcf)
	17	25.1	89
Classification test (see Plate 27)	37	28.8	88
	18	10.7	118
	38		



BLACK CLAY (CH)
stiff, moist, with shrinkage cracks to 2'

GRAY-BROWN SANDY CLAY (CH)
stiff, moist

GRAY-BROWN GRAVELLY SAND (SP) - medium dense, wet

GRAY-BROWN SANDY SILTY GRAVEL (GM) - dense, wet

(no free water encountered)

*Elevation Datum: Elevation shown on Topographic Map by Geo-Graphic Aerial Survey, undated

**Field blow count converted to standard penetration blows/foot

HARDING - LAWSON ASSOCIATES
Consulting Engineers and Geologists

Job No. 1013,005.02 Appr: GE Date 1/14/76

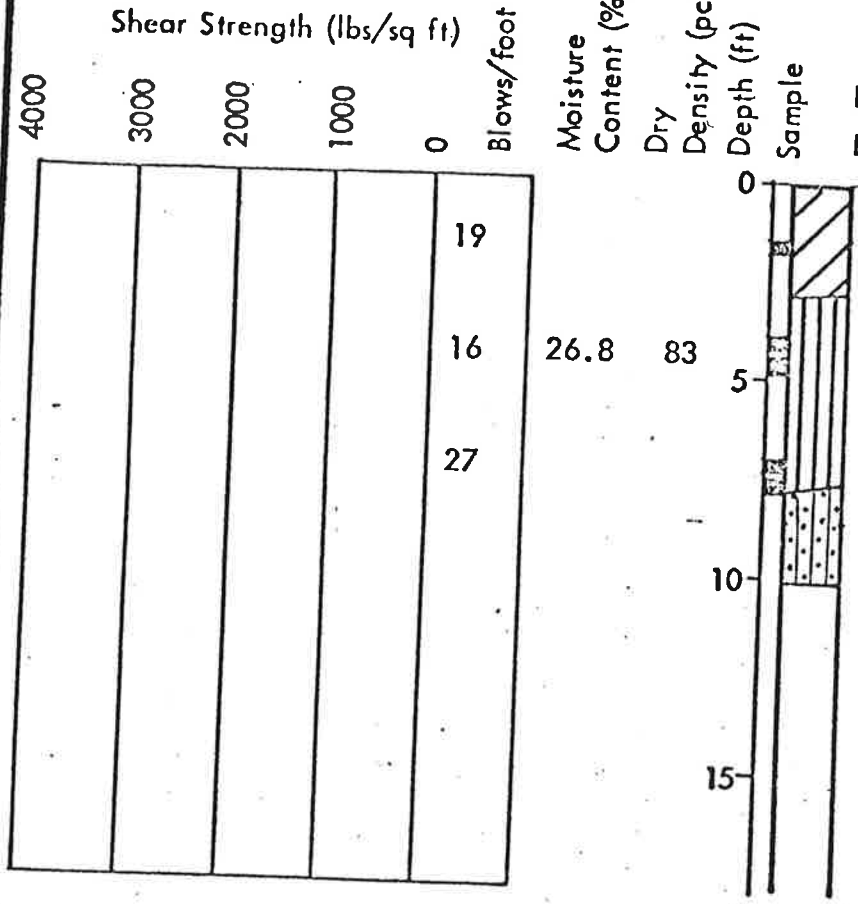
LOG OF BORINGS 1 & 2
State Farm Insurance Company
Northern California Office
Rohnert Park, California

PLATE 12

LOG OF BORING 3

Equipment 6" Flight Auger

Elevation 101.8 Date 1/8/76



BLACK CLAY (CH)
stiff, moist

GRAY-BROWN SANDY SILT (MH)
stiff, moist

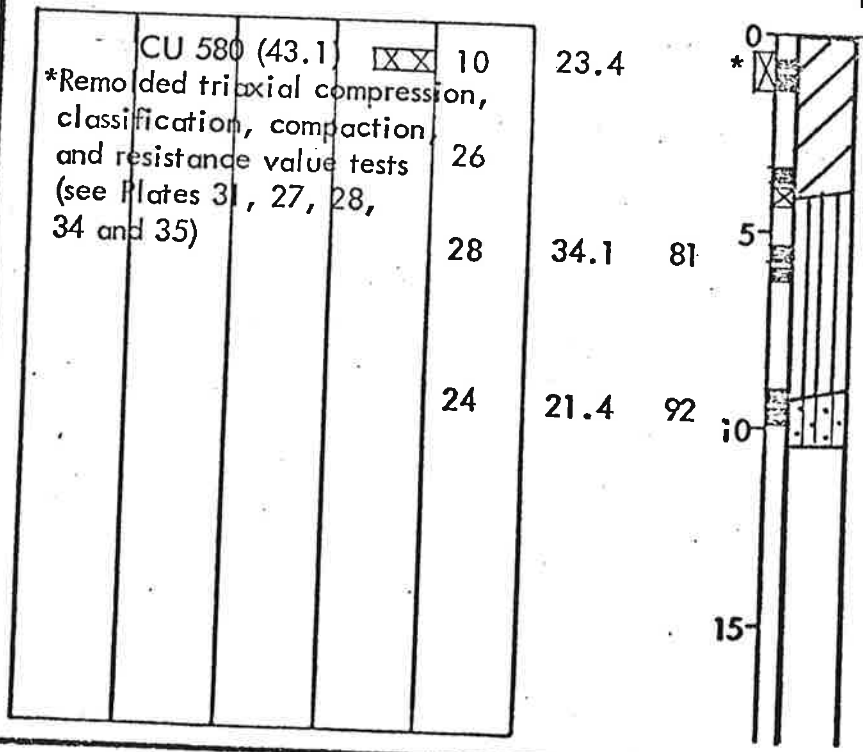
GRAY-BROWN GRAVELLY SILTY SAND (SM) - medium dense, wet

(no free water encountered)

LOG OF BORING 4

Equipment 6" Flight Auger

Elevation 103.8 Date 1/8/76



*Remolded triaxial compression, classification, compaction, and resistance value tests (see Plates 31, 27, 28, 34 and 35)

BLACK CLAY (CH)
stiff, moist, with shrinkage cracks to 2'

GRAY SANDY SILT (MH)
very stiff, moist

BROWN SILTY GRAVELLY SAND (SM) - medium dense, wet

(no free water encountered)

HARDING-LAWSON ASSOCIATES

Consulting Engineers and Geologists

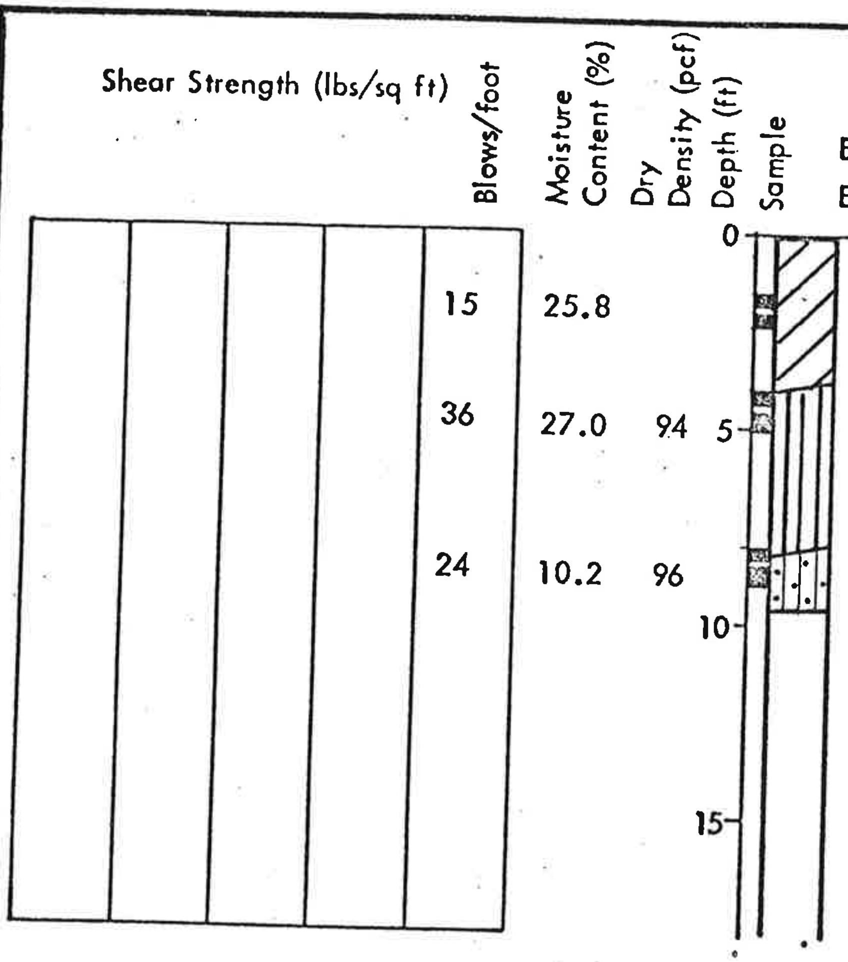
Job No. 1013,005.02 Appr: ES Date 1/14/76

LOG OF BORINGS 3&4
State Farm Insurance Company
Northern California Office
Rohnert Park, California

PLATE
13

LOG OF BORING 5

Equipment 6" Flight Auger
Elevation 102.5 Date 1/8/76



Shear Strength (lbs/sq ft)	Blows/foot	Moisture Content (%)	Dry Density (pcf)
	15	25.8	
	36	27.0	94
	24	10.2	96

BLACK CLAY (CH)
stiff, moist, with shrinkage cracks to 3'

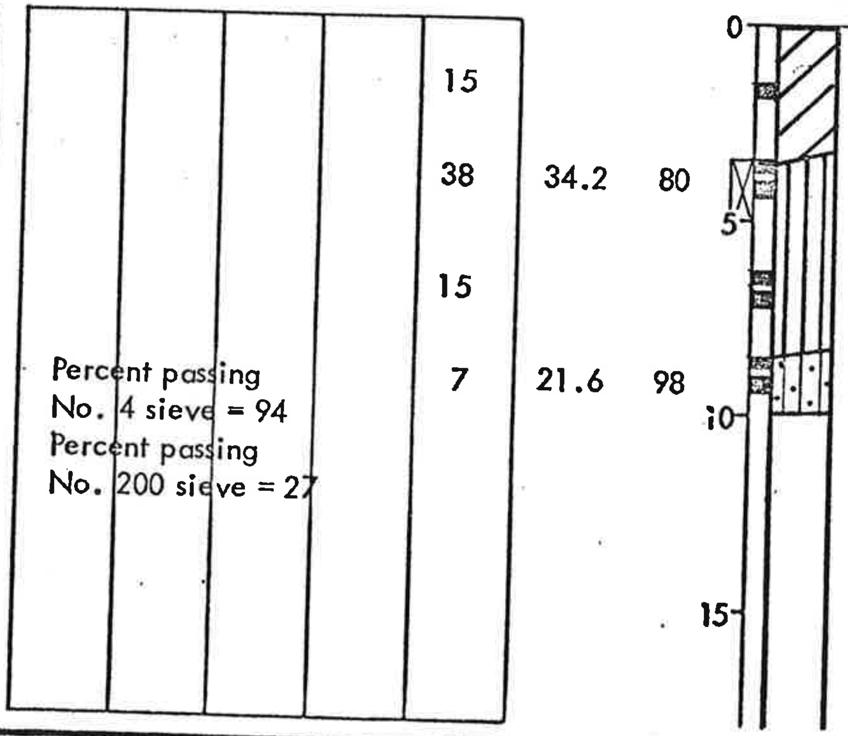
GRAY-BROWN SANDY SILT (MH)
very stiff, moist

BROWN SILTY GRAVELLY SAND (SM) - medium dense, moist

(no free water encountered)

LOG OF BORING 6

Equipment 6" Flight Auger
Elevation 102.5 Date 1/8/76



Shear Strength (lbs/sq ft)	Blows/foot	Moisture Content (%)	Dry Density (pcf)
	15		
	38	34.2	80
	15		
Percent passing No. 4 sieve = 94 Percent passing No. 200 sieve = 27	7	21.6	98

BLACK CLAY (CH)
stiff, moist

GRAY-BROWN SANDY SILT (MH)
very stiff, moist

BROWN SILTY GRAVELLY SAND (SM) - loose, wet

(no free water encountered)

HARDING - LAWSON ASSOCIATES
Consulting Engineers and Geologists

Job No. 1013,005.02 Appr: Date 1/14/76

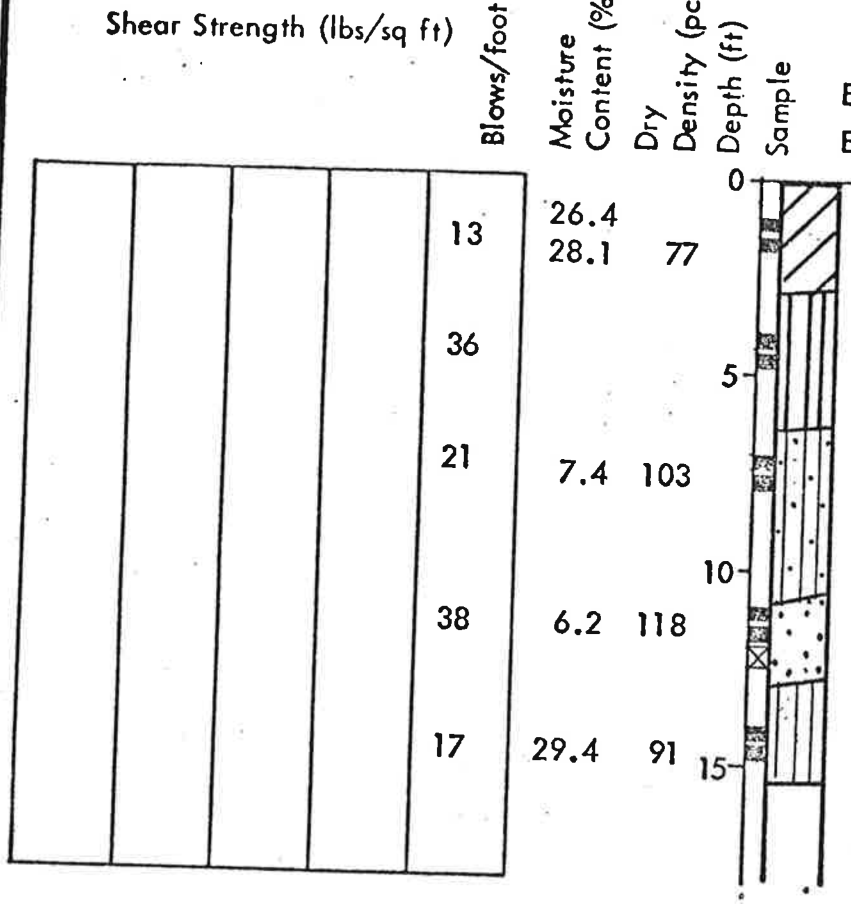
LOG OF BORINGS 5&6

State Farm Insurance Company
Northern California Office
Rohnert Park, California

PLATE 14

LOG OF BORING 7

Equipment 6" Flight Auger
Elevation 100.6 Date 1/6/76



Blows/foot	Moisture Content (%)	Dry Density (pcf)
13	26.4 28.1	77
36		
21	7.4	103
38	6.2	118
17	29.4	91

BLACK CLAY (CH)
stiff, moist, with shrinkage cracks to 1.5'

LIGHT BROWN SANDY SILT (MH)
very stiff, moist

GRAY-BROWN SILTY GRAVELLY SAND (SM) - medium dense, moist

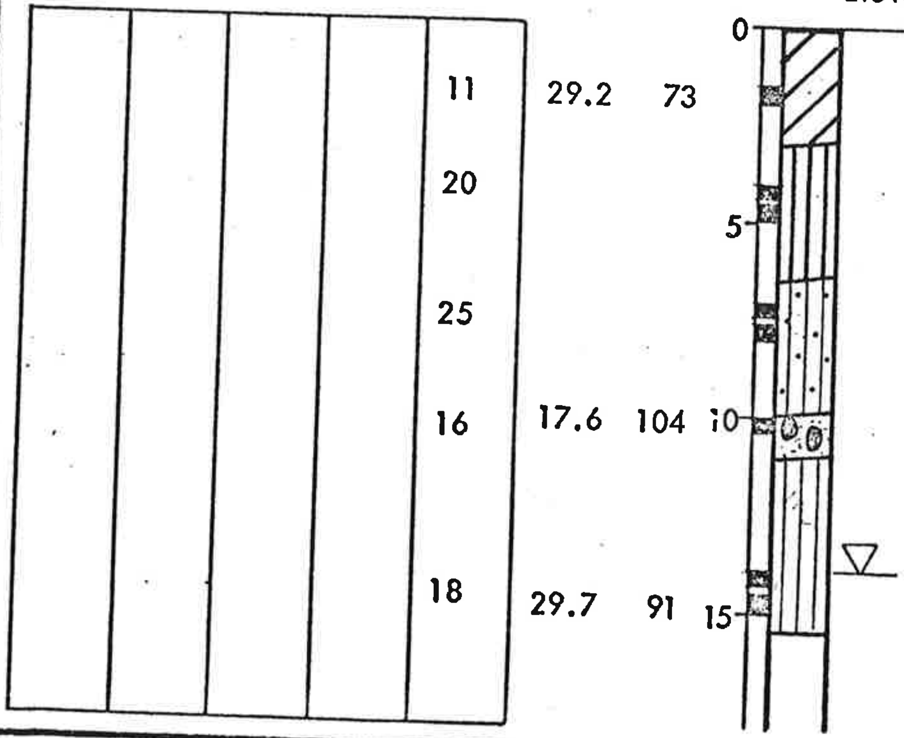
GRAY GRAVELLY SAND (SP)
dense, moist

BROWN SANDY SILT (ML)
medium stiff, wet

(no free water encountered

LOG OF BORING 8

Equipment 6" Flight Auger
Elevation 100.6 Date 1/7/76



Blows/foot	Moisture Content (%)	Dry Density (pcf)
11	29.2	73
20		
25		
16	17.6	104
18	29.7	91

BLACK CLAY (CH)
stiff, moist, with shrinkage cracks to 2'

LIGHT BROWN SANDY SILT (MH)
stiff, moist

LIGHT BROWN SILTY SAND (SM)
medium dense, moist

GRAY-BROWN SANDY GRAVEL (GP)
medium dense, moist

LIGHT GRAY-BROWN SANDY SILT (ML) - stiff, wet
water level 1/7/76

HARDING - LAWSON ASSOCIATES
Consulting Engineers and Geologists

Job No. 1013,005.02 Appr: Date 1/14/76

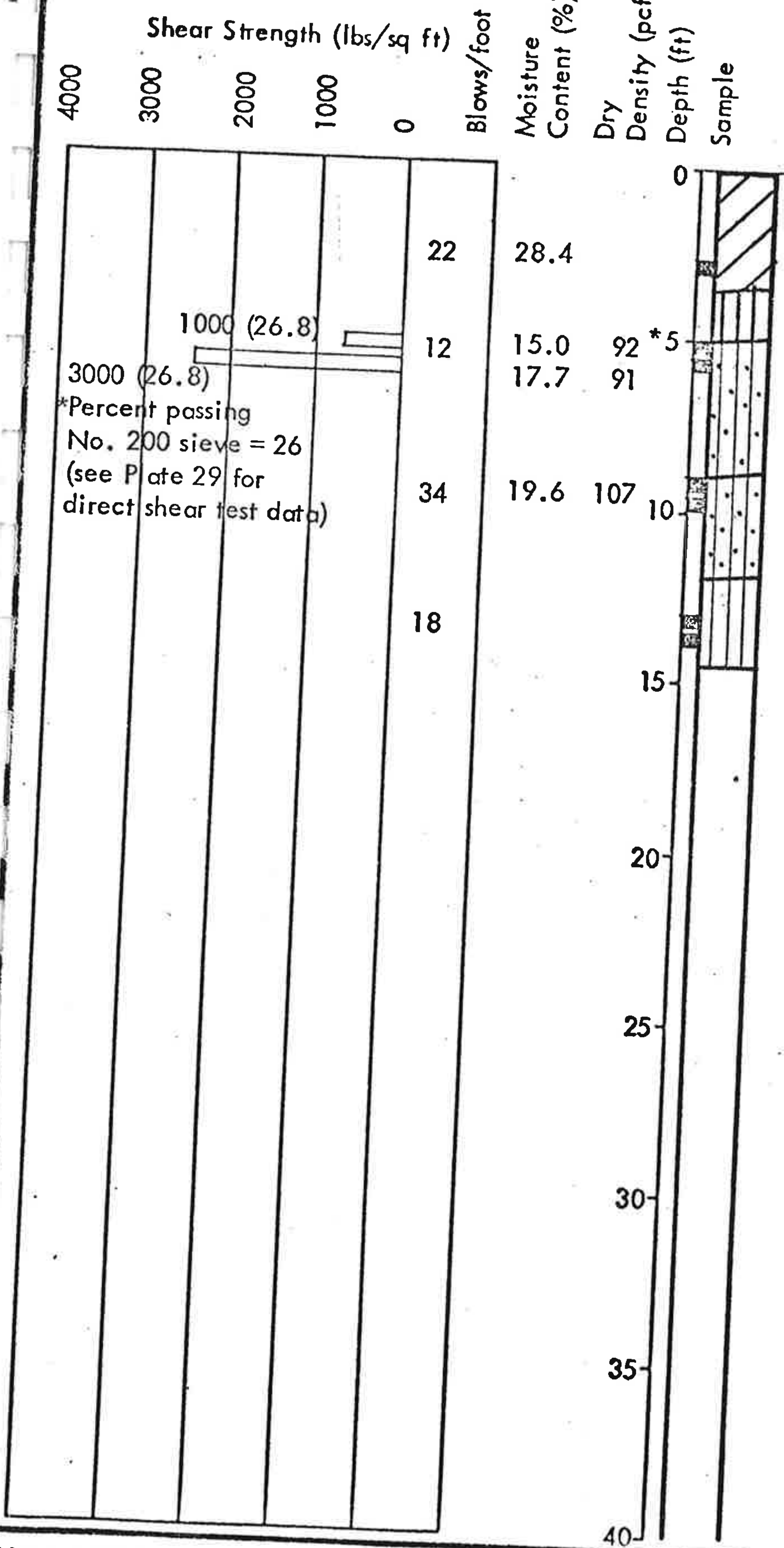
LOG OF BORINGS 7&8

State Farm Insurance Company
Northern California Office
Rohnert Park, California

PLATE
15

LOG OF BORING 9

Equipment 6" Flight Auger
Elevation 101.3 Date 1/7/76



BLACK CLAY (CH)
stiff, moist

GRAY-BROWN SANDY SILT (MH)
very stiff, moist

LIGHT BROWN SILTY SAND (SM)
medium dense, moist
wet below 7'

BLUE SILTY SAND (SM)
dense, wet

BROWN SANDY SILT (ML)
stiff, wet

(no free water encountered)

HARDING - LAWSON ASSOCIATES

Consulting Engineers and Geologists

Job No. 1013,005.02 Appr: GBL Date 1/14/76

LOG OF BORING 9

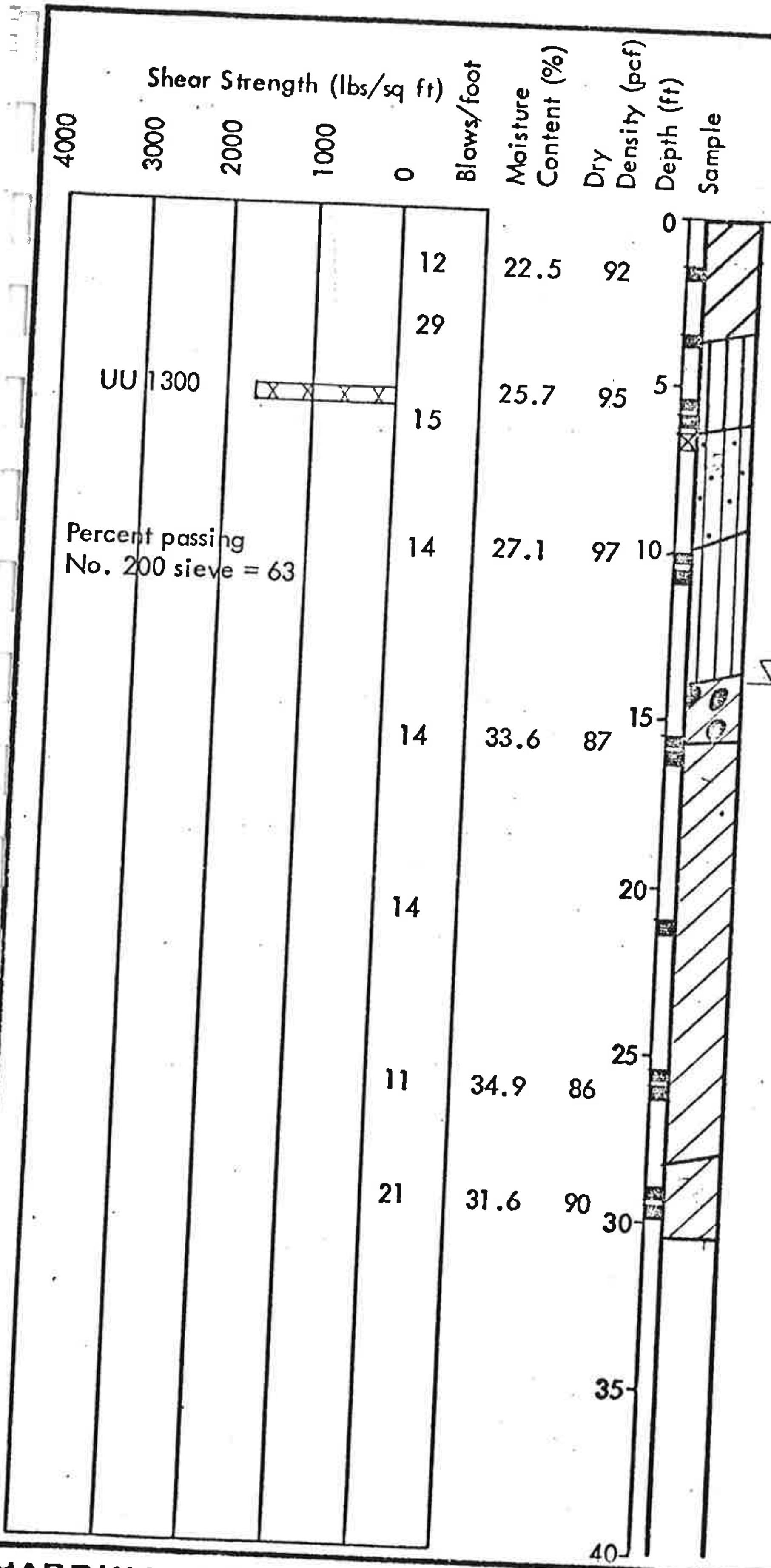
State Farm Insurance Company
Northern California Office
Rohnert Park, California

PLATE
16

LOG OF BORING 10

Equipment 7" Hollow-Stem Auger

Elevation 101.2 Date 1/5/76



BLACK CLAY (CH)
stiff, moist

BROWN SANDY SILT (MH)
stiff, moist

BROWN SILTY SAND (SM)
medium dense, wet

BROWN SANDY SILT (ML)
stiff, wet

∇ water level 1/6/76

GRAY-BROWN CLAYEY SANDY GRAVEL (GC) - medium dense, saturated

BROWN SANDY CLAY (CL)
stiff, saturated
with some gravel to 27'

BLUE SANDY CLAY (CL)
very stiff, saturated

HARDING - LAWSON ASSOCIATES

Consulting Engineers and Geologists

Job No. 1013,005.02 Appr: GB Date 1/14/76

LOG OF BORING 10
State Farm Insurance Company
Northern California Office
Rohnert Park, California

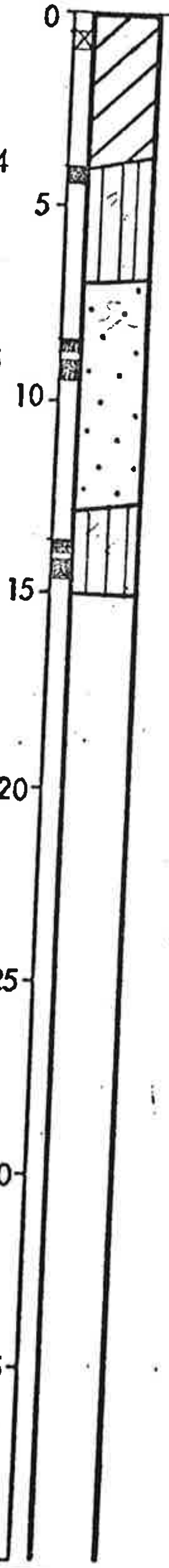
PLATE 17

LOG OF BORING 11

Equipment 6" Flight Auger

Elevation 100.4 Date 1/6/76

Shear Strength (lbs/sq ft)	Blows/foot	Moisture Content (%)	Dry Density (pcf)
	12		
	32	24.9	84
Percent passing No. 4 sieve = 66 , Percent passing No. 200 sieve = 5	23	15.8	105
	22	29.8	90



BLACK CLAY (CH)
stiff, moist

LIGHT GRAY-BROWN SANDY SILT
(ML) - very stiff, moist

GRAY-BROWN GRAVELLY SAND
(SP) - medium dense, wet

LIGHT BROWN SANDY SILT (ML)
stiff, wet

(no free water encountered)

HARDING - LAWSON ASSOCIATES
Consulting Engineers and Geologists

Job No. 1013,005.02 Appr: EB Date 1/14/76

LOG OF BORING 11
State Farm Insurance Company
Northern California Office
Rohnert Park, California

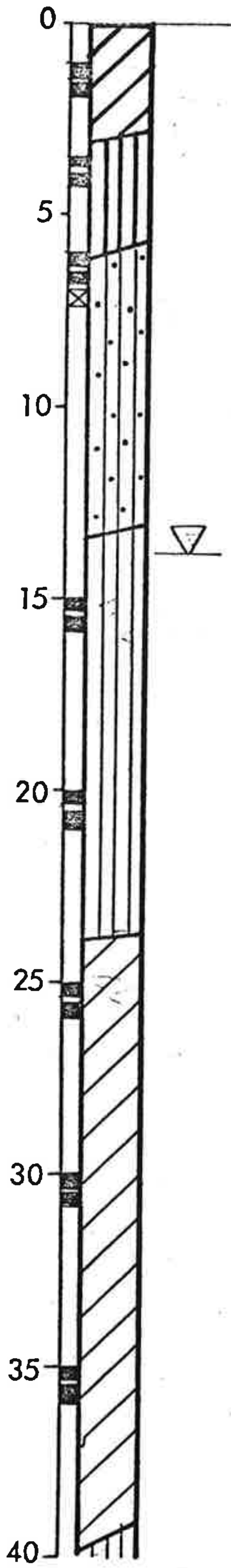
PLATE
18

LOG OF BORING 12

Equipment 7" Hollow-Stem Auger

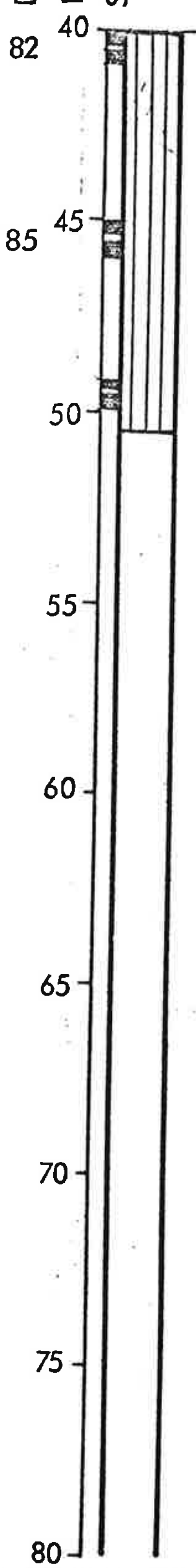
Elevation 100.7 Date 1/5/76

Shear Strength (lbs/sq ft)	Blows/foot	Moisture Content (%)	Dry Density (pcf)	
	16	24.5	92	BLACK CLAY (CH) stiff, moist
Consolidation test (see Plate 33)	25	31.5	90	LIGHT BROWN SANDY SILT (MH) very stiff, moist
	27	18.8	104	LIGHT BROWN SILTY SAND (SM) dense, wet
	19	34.0	85	water level 1/6/76 BROWN SANDY SILT (ML) stiff, saturated
	14	36.7	83	
	12			GRAY-BROWN SANDY CLAY (CL) stiff, saturated
	23	32.7	84	very stiff below 30'
	22			



(Continuation of Log)

Shear Strength (lbs/sq ft)	Blows/foot	Moisture Content (%)	Dry Density (pcf)	
	14	37.6	82	LIGHT BROWN SANDY SILT (ML) stiff, saturated
	24	35.6	85	
	30			



HARDING - LAWSON ASSOCIATES

Consulting Engineers and Geologists

Job No. 1013,005.02 Appr: GB Date 1/14/76

LOG OF BORING 12

State Farm Insurance Company

Northern California Office

Rohnert Park, California

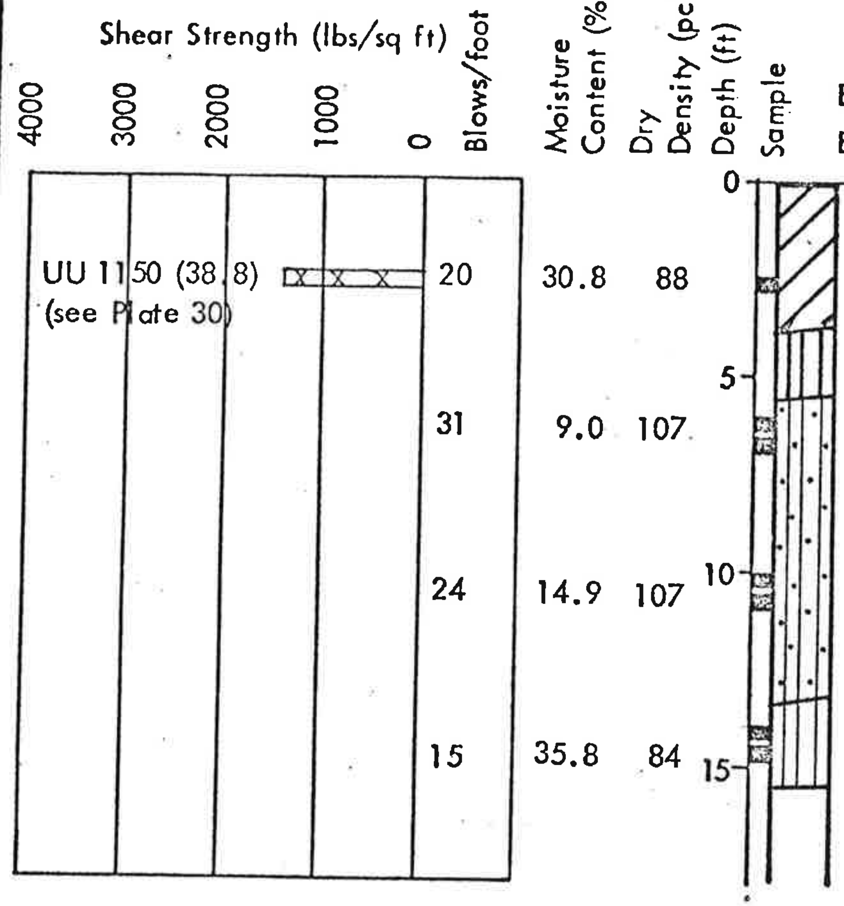
PLATE

19

LOG OF BORING 13

Equipment 6" Flight Auger

Elevation 101.2 Date 1/6/76



Blows/foot	Moisture Content (%)	Dry Density (pcf)
20	30.8	88
31	9.0	107
24	14.9	107
15	35.8	84

BLACK CLAY (CH)
stiff, moist

LIGHT GRAY-BROWN SANDY SILT (MH) - very stiff, moist

GRAY-BROWN SILTY GRAVELLY SAND (SM) - dense, moist

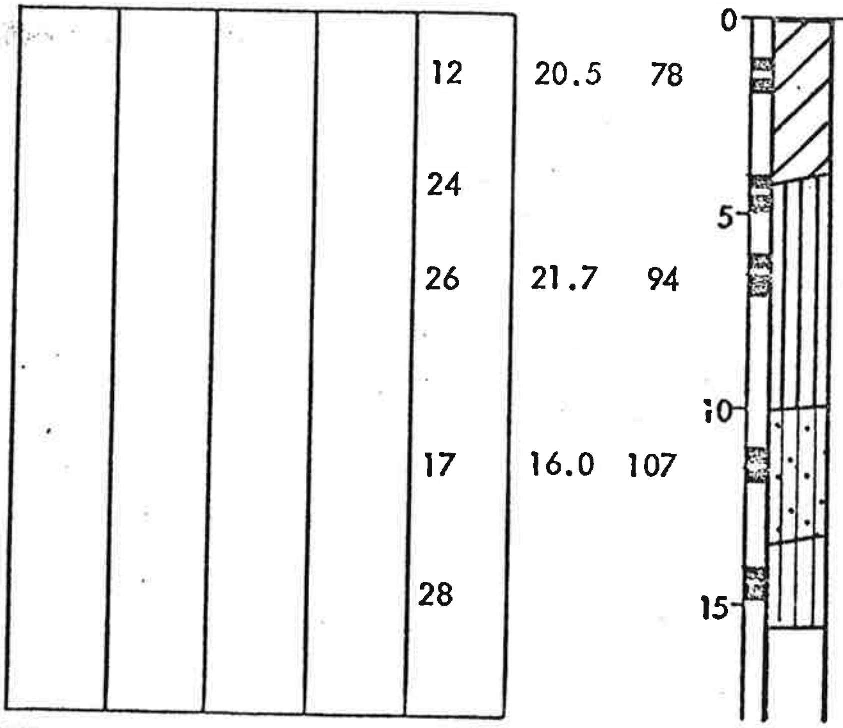
GRAY-BROWN SANDY SILT (ML)
stiff, wet

(no free water encountered)

LOG OF BORING 14

Equipment 6" Flight Auger

Elevation 102.1 Date 1/6/76



Blows/foot	Moisture Content (%)	Dry Density (pcf)
12	20.5	78
24		
26	21.7	94
17	16.0	107
28		

BLACK CLAY (CH)
stiff, moist, with shrinkage cracks to 1.5'

BROWN SANDY SILT (MH)
very stiff, moist

GRAY-BROWN SILTY GRAVELLY SAND (SM) - medium dense, wet

LIGHT BROWN SANDY SILT (ML)
stiff, wet

(no free water encountered)

HARDING - LAWSON ASSOCIATES
Consulting Engineers and Geologists

Job No. 1013,005.02 Appr: EB Date 1/14/76

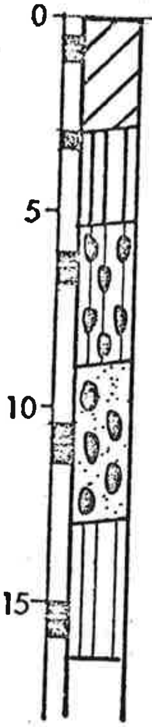
LOG OF BORINGS 13&14
State Farm Insurance Company
Northern California Office
Rohnert Park, California

PLATE 20

LOG OF BORING 15

Equipment 6" Flight Auger
Elevation 100.6 Date 1/6/76

Shear Strength (lbs/sq ft)	Blows/foot	Moisture Content (%)	Dry Density (pcf)
	14	24.1	72
	42		
	33	11.5	111
	32	11.4	113
	16	29.1	94



BLACK CLAY (CH)
stiff, moist to wet

LIGHT BROWN SANDY SILT (MH)
very stiff, moist

LIGHT BROWN SILTY SANDY GRAVEL (GM) - dense, moist

GRAY-BROWN SILTY SANDY GRAVEL (GP-GM) - dense, moist

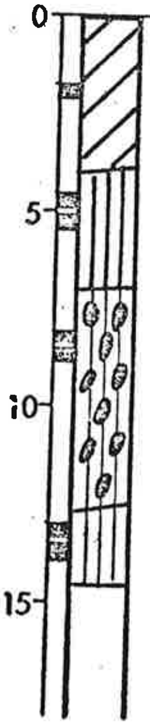
LIGHT BROWN SANDY SILT (ML)
stiff, wet

(no free water encountered)

LOG OF BORING 16

Equipment 6" Flight Auger
Elevation 101.3 Date 1/6/76

Shear Strength (lbs/sq ft)	Blows/foot	Moisture Content (%)	Dry Density (pcf)
Consolidation test (see Plate 32)	14	23.2	91
	27		
	22	17.6	115
	18		



BLACK CLAY (CH)
stiff, moist, with shrinkage cracks to 1.5'

LIGHT BROWN SANDY SILT (MH)
stiff, moist

LIGHT BROWN SILTY SANDY GRAVEL (GM) - medium dense, moist

LIGHT BROWN SANDY SILT (ML)
stiff, wet

(no free water encountered)

HARDING - LAWSON ASSOCIATES
Consulting Engineers and Geologists

Job No. 1013,005.02 Appr. [illegible] Date 1/14/76

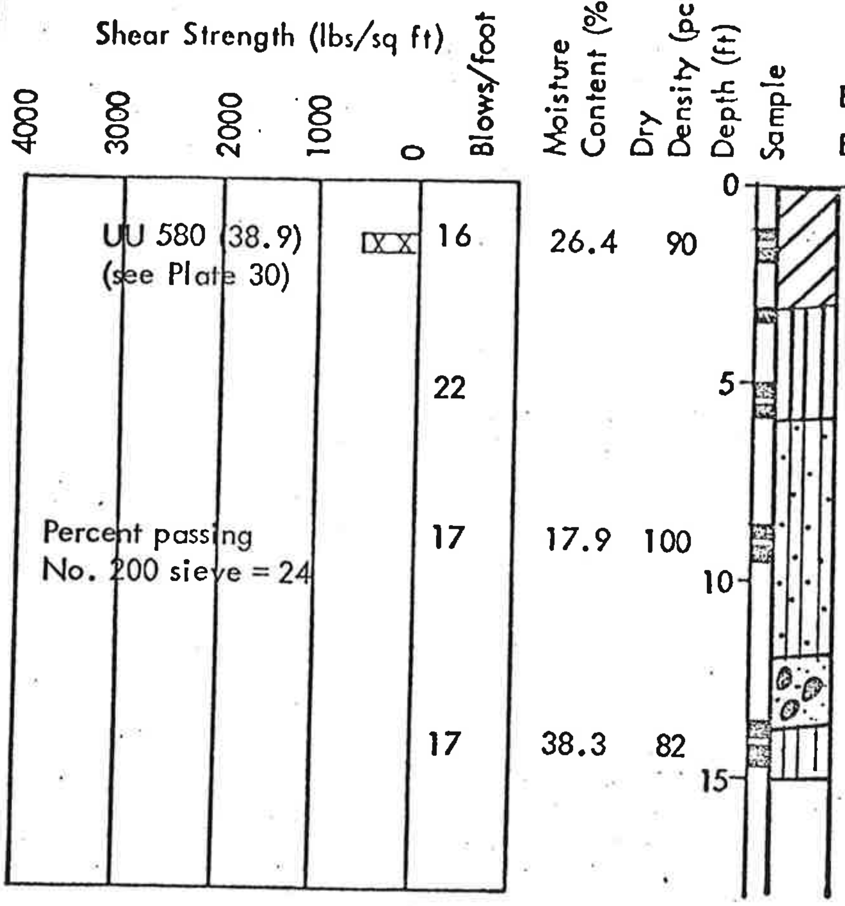
LOG OF BORINGS 15&16
State Farm Insurance Company
Northern California Office
Rohnert Park, California

PLATE 21

LOG OF BORING 17

Equipment 6" Flight Auger

Elevation 101.7 Date 1/6/76



UU 580 (38.9)
(see Plate 30)

Percent passing
No. 200 sieve = 24

Blows/foot	Moisture Content (%)	Dry Density (pcf)
16	26.4	90
22		
17	17.9	100
17	38.3	82

BLACK CLAY (CH)
stiff, wet

LIGHT BROWN SANDY SILT (MH)
very stiff, moist

LIGHT BROWN SILTY SAND (SM)
medium dense, moist

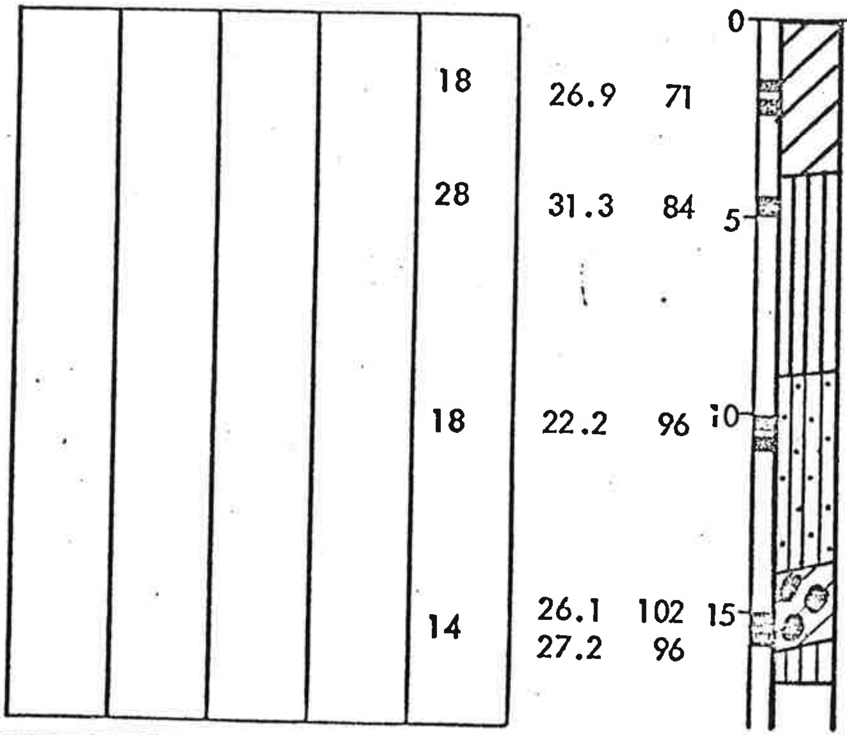
GRAY-BROWN SANDY GRAVEL (GP)
medium dense, wet
LIGHT BROWN SANDY SILT (ML)
stiff, wet

(no free water encountered)

LOG OF BORING 18

Equipment 6" Flight Auger

Elevation 102.5 Date 1/6/76



Blows/foot	Moisture Content (%)	Dry Density (pcf)
18	26.9	71
28	31.3	84
18	22.2	96
14	26.1 27.2	102 96

BLACK CLAY (CH)
stiff, moist, with shrinkage
cracks to 2.0'

LIGHT GRAY-BROWN SANDY SILT
(MH) - very stiff, moist

BROWN SILTY SAND (SM)
medium dense, moist

GRAY-BROWN CLAYEY SANDY
GRAVEL (GC) - medium dense,
saturated
BROWN SANDY SILT (ML)
stiff, wet
(no free water encountered)

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Job No. 1013,005.02 Appr: GF5 Date 1/14/76

LOG OF BORINGS 17&18

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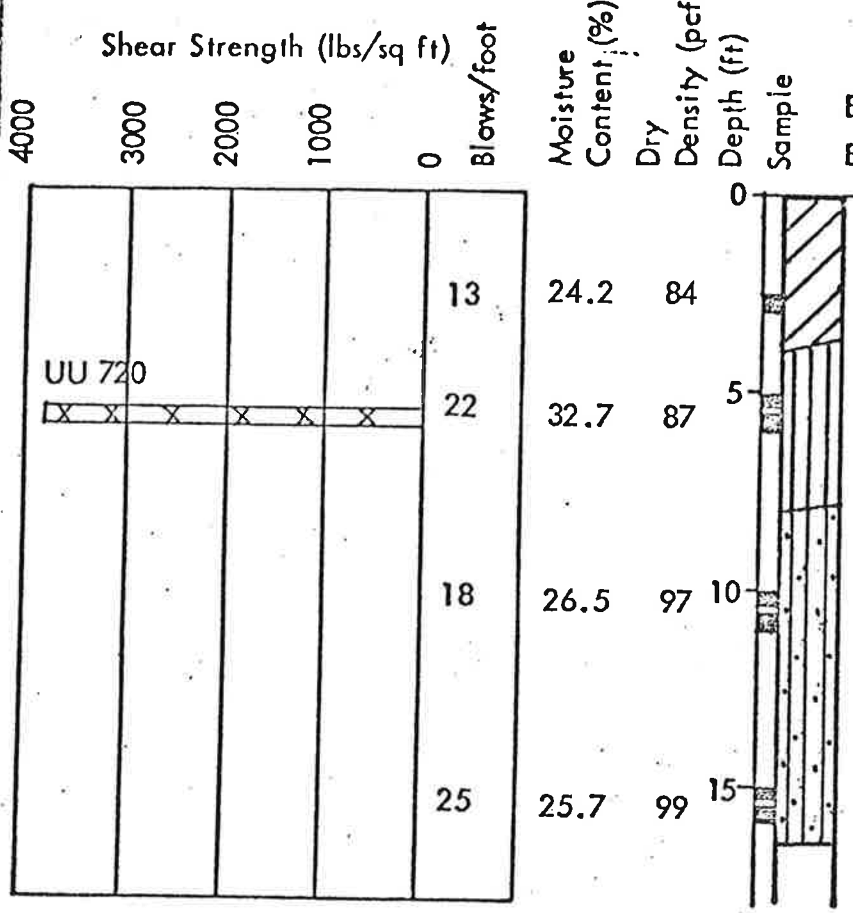
Rohnert Park, California

PLATE
22

LOG OF BORING 19

Equipment 6" Flight Auger

Elevation 100.0 Date 1/6/76



Shear Strength (lbs/sq ft)	Blows/foot	Moisture Content (%)	Dry Density (pcf)
	13	24.2	84
UU 720	22	32.7	87
	18	26.5	97
	25	25.7	99

BLACK CLAY (CH)
stiff, moist, with shrinkage cracks to 2'

BROWN SANDY SILT (MH)
very stiff, moist

BROWN SILTY GRAVELLY SAND (SM) - medium dense, wet

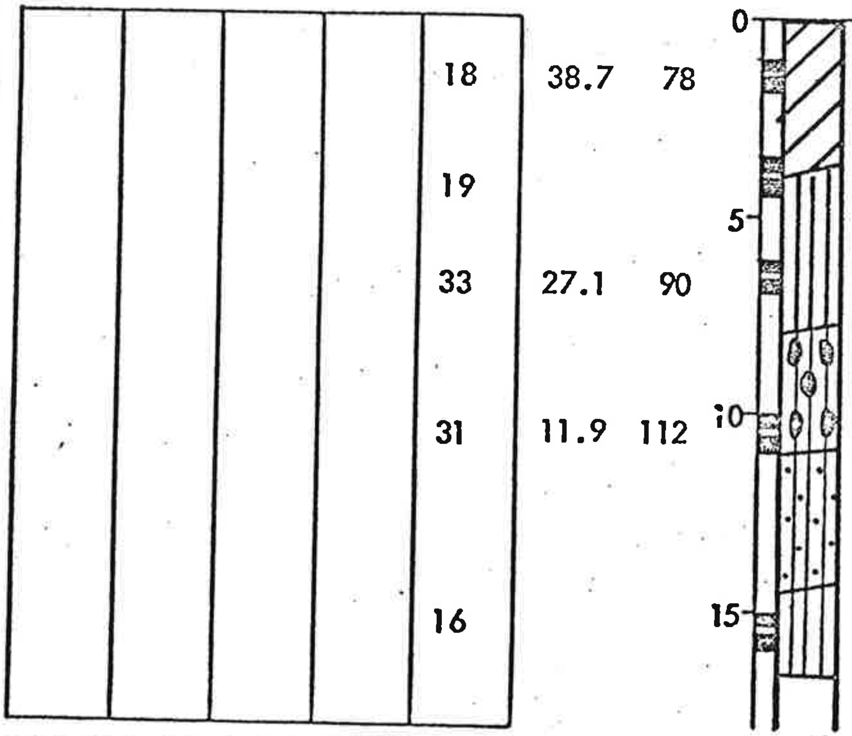
with pockets of brown sandy silt (ML), stiff below 14'

(no free water encountered)

LOG OF BORING 20

Equipment 6" Flight Auger

Elevation 101.4 Date 1/6/76



Blows/foot	Moisture Content (%)	Dry Density (pcf)
18	38.7	78
19		
33	27.1	90
31	11.9	112
16		

BLACK CLAY (CH)
stiff, moist to wet

BROWN SANDY SILT (MH)
stiff, moist

GRAY-BROWN SILTY SANDY GRAVEL (GM) - dense, moist

BROWN SILTY SAND (SM)
medium dense, wet

GRAY-BROWN SANDY SILT (ML)
stiff, wet

(no free water encountered)

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LOG OF BORINGS 19&20
State Farm Insurance Company
Northern California Office
Rohnert Park, California

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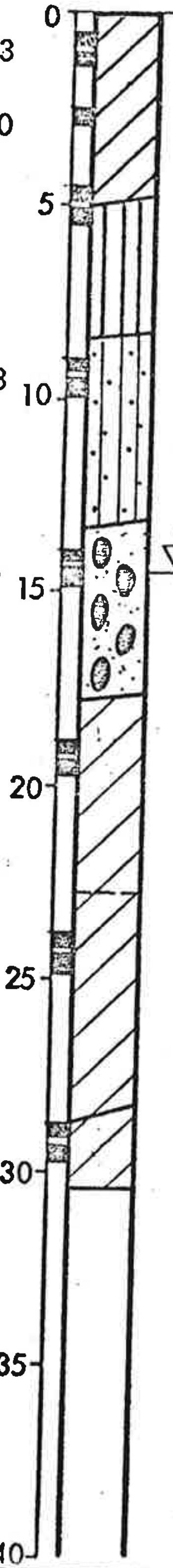
LOG OF BORING 21

Equipment 7" Hollow-Stem Auger
Elevation 102.7 Date 1/5/76

Shear Strength (lbs/sq ft)	Blows/foot	Moisture Content (%)	Dry Density (pcf)
	12	33.1	83
	14	27.4	90
	33		
Percent passing No. 200 sieve = 34	19	21.2	103
	41	17.0	112
	10	28.1	97
	19		
	22	37.1	83



BLACK CLAY (CH)
stiff, moist

BROWN SANDY CLAYEY SILT (MH)
very stiff, moist

BROWN SILTY SAND (SM)
medium dense, wet

GRAY-BROWN SANDY GRAVEL (GP) - dense, saturated
water level 1/6/76

BROWN SANDY CLAY (CL)
medium stiff, saturated

stiff, with some pockets
of clayey sand (SC) below 23'

BLUE-GRAY SANDY CLAY (CL)
very stiff, saturated

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LOG OF BORING 21
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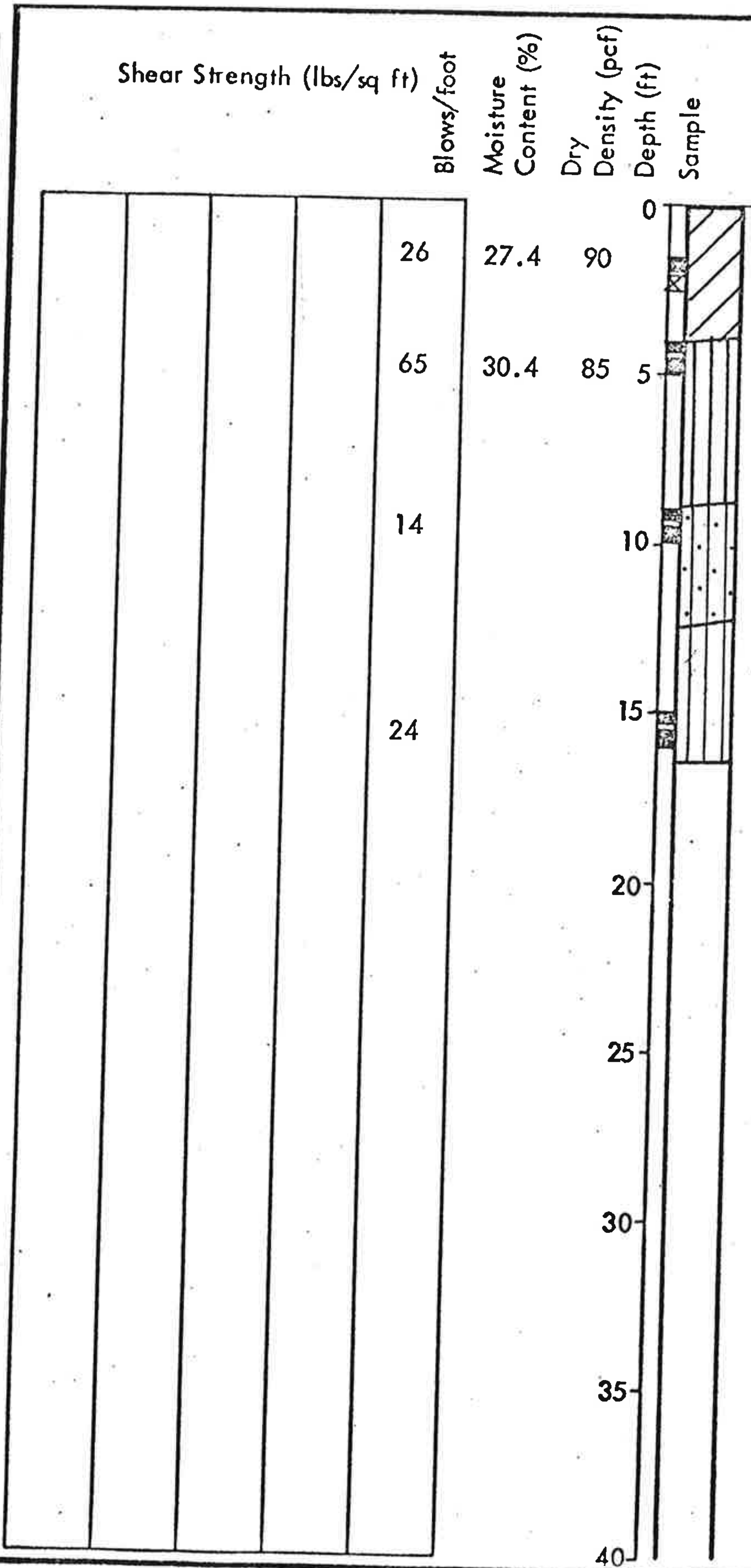
LOG OF BORING 22

Equipment 6" Flight Auger

Elevation 103.2 Date 1/6/76



BLACK CLAY (CH)
stiff, moist

BROWN SANDY SILT (MH)
very stiff, moist

BROWN SILTY SAND (SM)
medium dense, moist

BROWN SANDY SILT (ML)
stiff, moist

(no free water encountered)

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Job No. 1013,005.02 Appr: EB Date 1/14/76

LOG OF BORING 22

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Northern California Office
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PLATE
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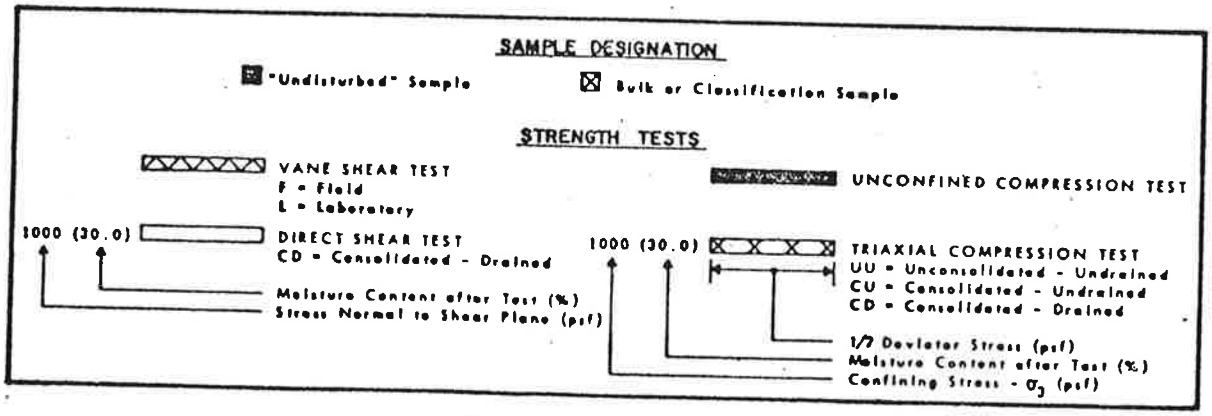
MAJOR DIVISIONS				TYPICAL NAMES
COARSE GRAINED SOILS MORE THAN HALF IS LARGER THAN #200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL GRADED GRAVELS, GRAVEL - SAND MIXTURES
			GP	POORLY GRADED GRAVELS, GRAVEL - SAND MIXTURES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, POORLY GRADED GRAVEL - SAND - SILT MIXTURES
			GC	CLAYEY GRAVELS, POORLY GRADED GRAVEL - SAND - CLAY MIXTURES
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS
			SP	POORLY GRADED SANDS, GRAVELLY SANDS
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS, POORLY GRADED SAND - SILT MIXTURES
			SC	CLAYEY SANDS, POORLY GRADED SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN HALF IS SMALLER THAN #200 SIEVE	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACIOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS

UNIFIED SOIL CLASSIFICATION SYSTEM



KEY TO TEST DATA

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Job No. 1013,005.02 Appr GB Date 1/14/76

SOIL CLASSIFICATION CHART
AND
KEY TO TEST DATA
State Farm Insurance Company

PLATE
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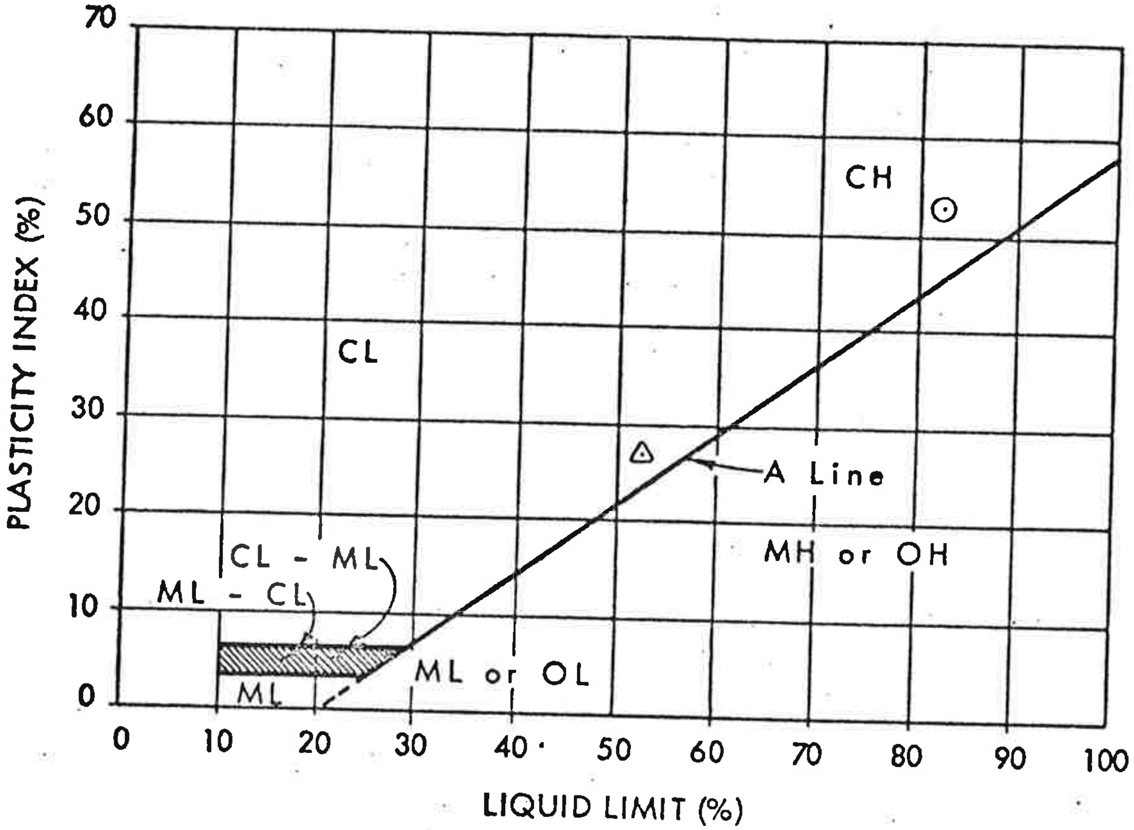
N02470



Symbol	Classification and Source	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
⊙	BLACK CLAY (CH) Boring 1 at 0.5' to 2.0' combined with Boring 4 at 0.5' to 1.5'	82	29	53	92
△	GRAY-BROWN SANDY CLAY (CH) Boring 2 at 3.0' to 4.5'	52	24	28	69

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PLASTICITY CHART

State Farm Insurance Company
Northern California Office
Rohnert Park, California

PLATE

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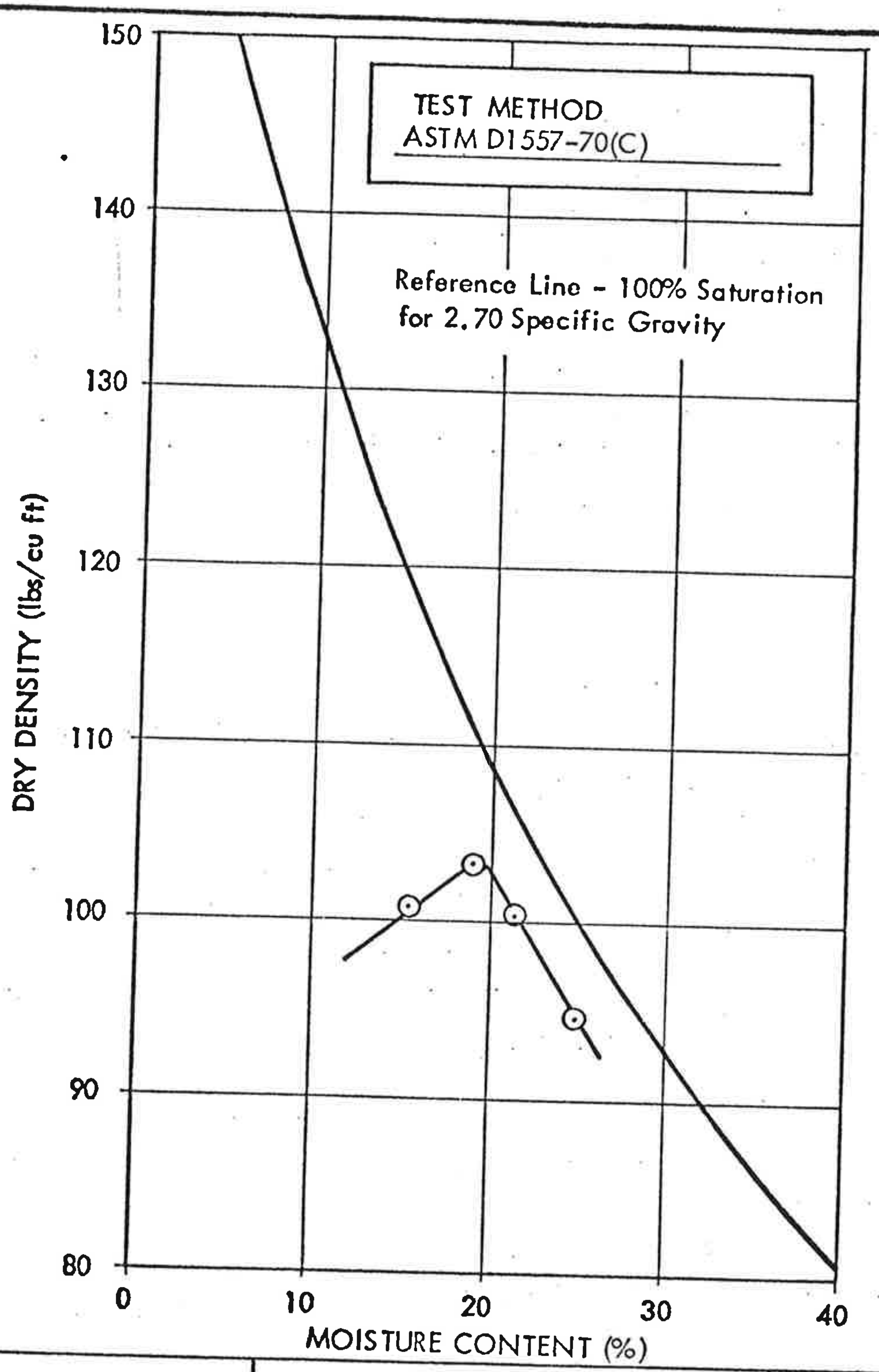
N02470



Symbol	Sample Source	Classification	Optimum Moisture (%)	Maximum Dry Density (pcf)
⊙	Boring 1 at 0.5 to 2.0' combined with Boring 4 at 0.5' to 1.5'	BLACK CLAY (CH)	19.5	104

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Job No. 1013,005.02 Appr: GB Date 2/4/76

COMPACTION TEST DATA

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Northern California Office
Rohnert Park, California

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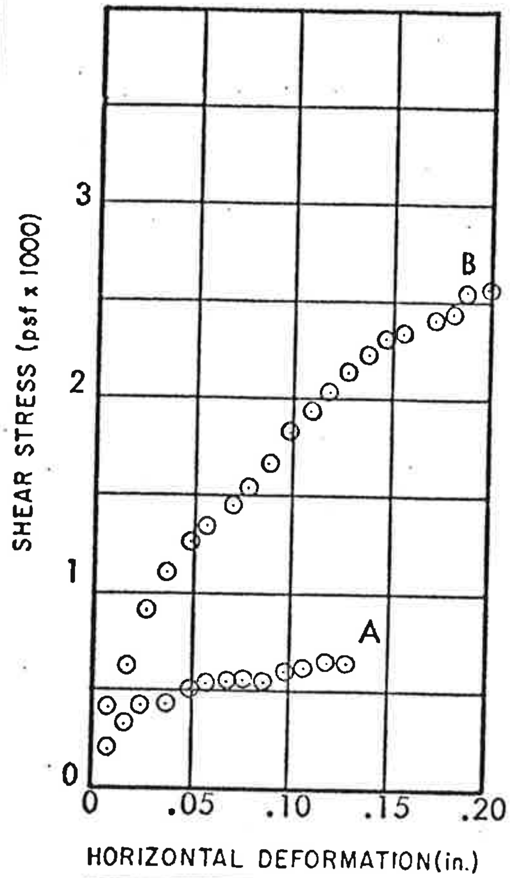
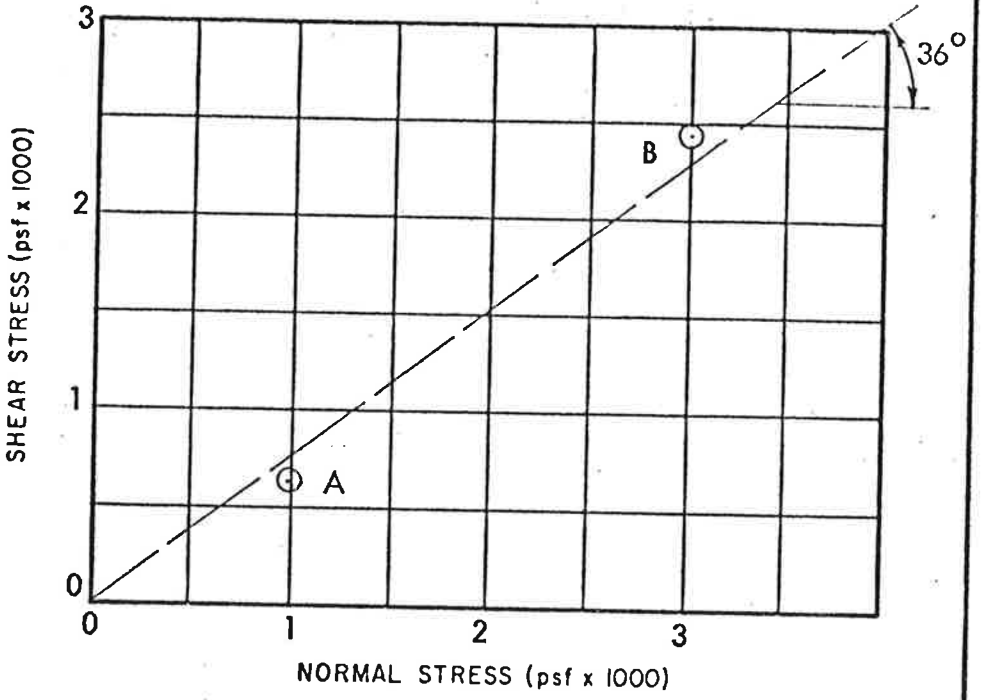




Test Type: Consolidated Drained

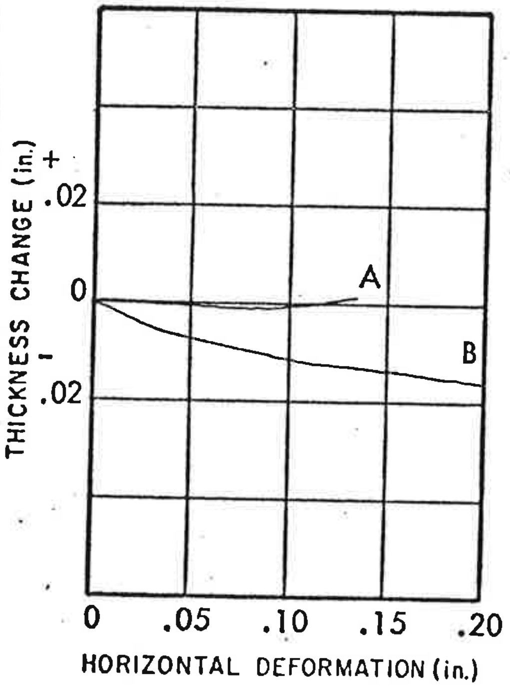
Controlled Deflection

Gs 2.60 (assumed)

	Test No.	A	B	C
Initial	Height (in.)	1.00	1.00	
	Moisture Content	15.0 %	17.7 %	%
	Void Ratio	0.767	0.780	
	Saturation	51 %	59 %	%
	Dry Density (pcf)	92	91	
Before Test	Time for 50% Consolidation (min.)	less than	1	
	Time for 95% Consolidation (min.)	---	---	
	Void Ratio after Consolidation	0.742	0.717	
Final	Moisture Content	26.8 %	26.8 %	%
	Void Ratio	0.742	0.686	
	Saturation	100 %	100 %	%
Normal Stress (psf)		1000	3000	
Maximum Shear (psf)		640	2450	
Time to Failure (min)		45	20	

Sample Source Boring 9 at 5.5'

Classification Light Brown Silty Sand (SM)



Ø' = 36°
C' =

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Job No. 1013,005.02 Appr: GB Date 2/17/76

DIRECT SHEAR TEST REPORT

State Farm Insurance Company
Northern California Office
Rohnert Park, California

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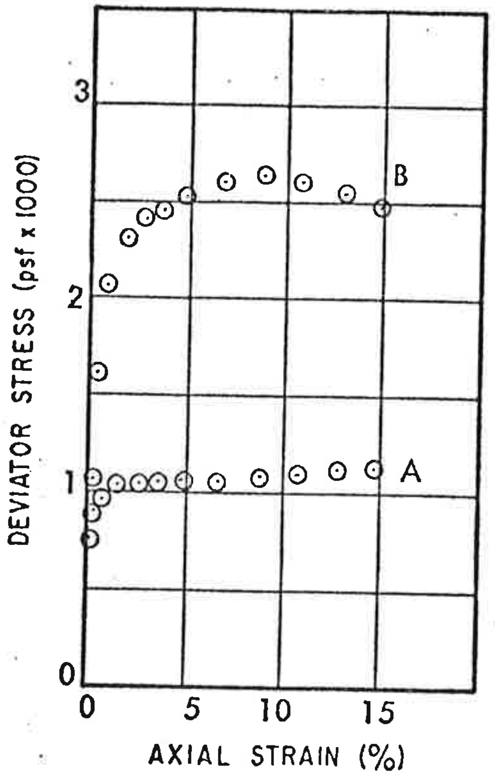

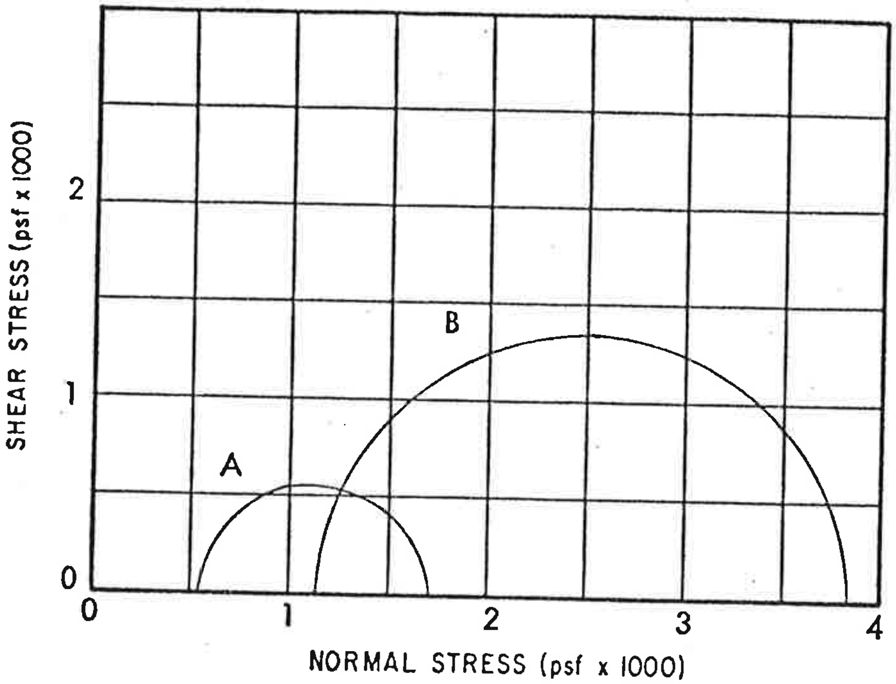

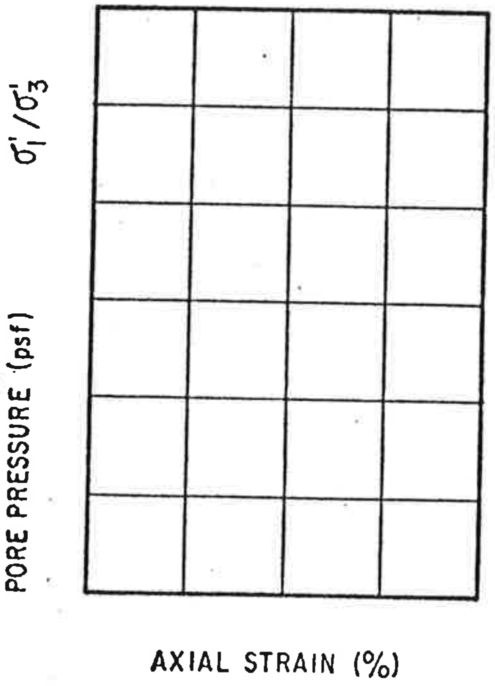

Ø =
C =

Test Type: Unconsolidated-undrained Controlled: Strain
Saturation Method: Backpressure G_s 2.70 (assumed)

Test No.		A	B	C
Initial	Diameter (in.)	2.43	2.43	
	Height (in.)	5.85	5.80	
	Moisture Content	26.4 %	30.8 %	%
	Void Ratio	0.867	0.926	
	Saturation	82 %	90 %	%
	Dry Density (pcf)	90	88	
Before Test	Moisture Content	38.9 %	38.8 %	%
	Void Ratio	1.052	1.029	
	Saturation	100 %	100 %	%
	Pressure (psf)	580	1150	
Final	Moisture Content	38.9 %	38.8 %	%
	Void Ratio	1.052	1.029	
σ_1 Major Prin. Stress (psf)		1680	3800	
σ_3 Minor Prin. Stress (psf)		580	1150	
Time to Failure (min.)		16	16	

Sample Source: A-Bor. 17 at 1.3', B-Bor. 13 at 2.5'

Classification: Black Clay (CH)

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Job No. 1013,005.02 Appr: GFB Date 2/17/76

TRIAXIAL COMPRESSION TEST REPORT

State Farm Insurance Company
Northern California Office
Rohnert Park, California

PLATE
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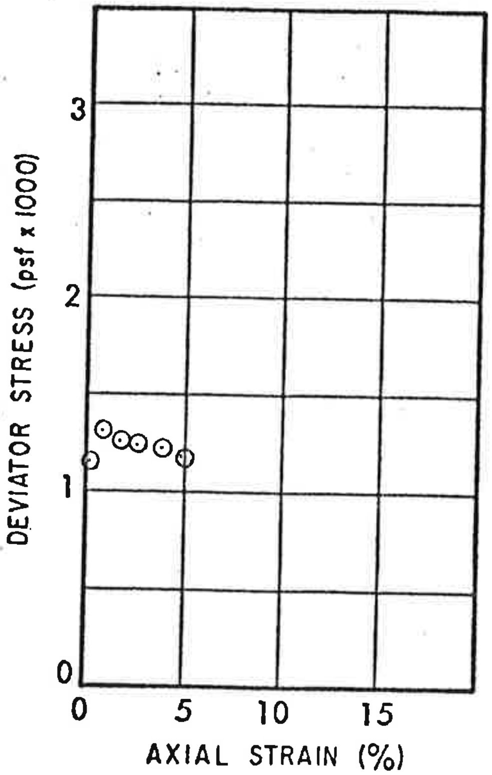

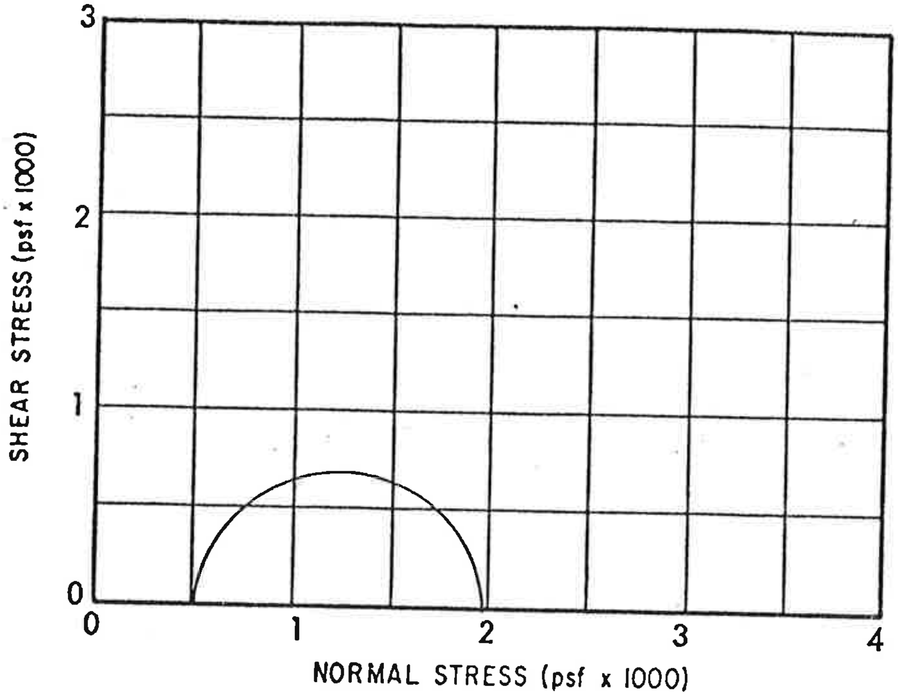

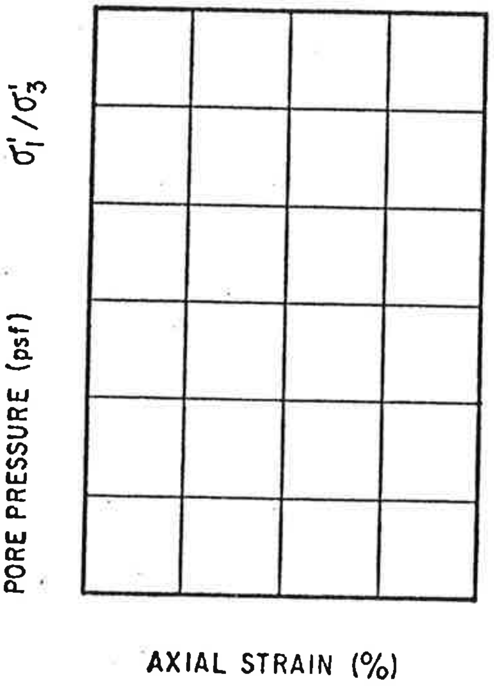

Ø =
C =

Test Type: Consolidated-undrained Controlled: Strain
Saturation Method: Backpressure G_s 2.75 (assumed)

	Test No.	A *	B	C
Initial	Diameter (in.)	2.43		
	Height (in.)	5.80		
	Moisture Content	23.2 %	%	%
	Void Ratio	0.886		
	Saturation	72 %	%	%
	Dry Density (pcf)	91		
Before Test	Moisture Content	43.1 %	%	%
	Void Ratio	1.175		
	Saturation	100 %	%	%
	Pressure (psf)	580		
Final	Moisture Content	43.1 %	%	%
	Void Ratio	1.175		
	σ_1 Major Prin. Stress (psf)	1920		
	σ_3 Minor Prin. Stress (psf)	580		
	Time to Failure (min.)	2		

Sample Source: Boring 1 at 0.5' to 2.0' combined with Boring 4 at 0.5' to 1.5'

Classification: Black Clay (CH)

*Remolded to 88% relative compaction and approximately 4% over optimum moisture content, based on compaction test data presented on Plate 28.

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Job No. 1013,005.02 Appr: GES Date 2/17/76

TRIAXIAL COMPRESSION TEST REPORT

State Farm Insurance Company
Northern California Office
Rohnert Park, California

PLATE
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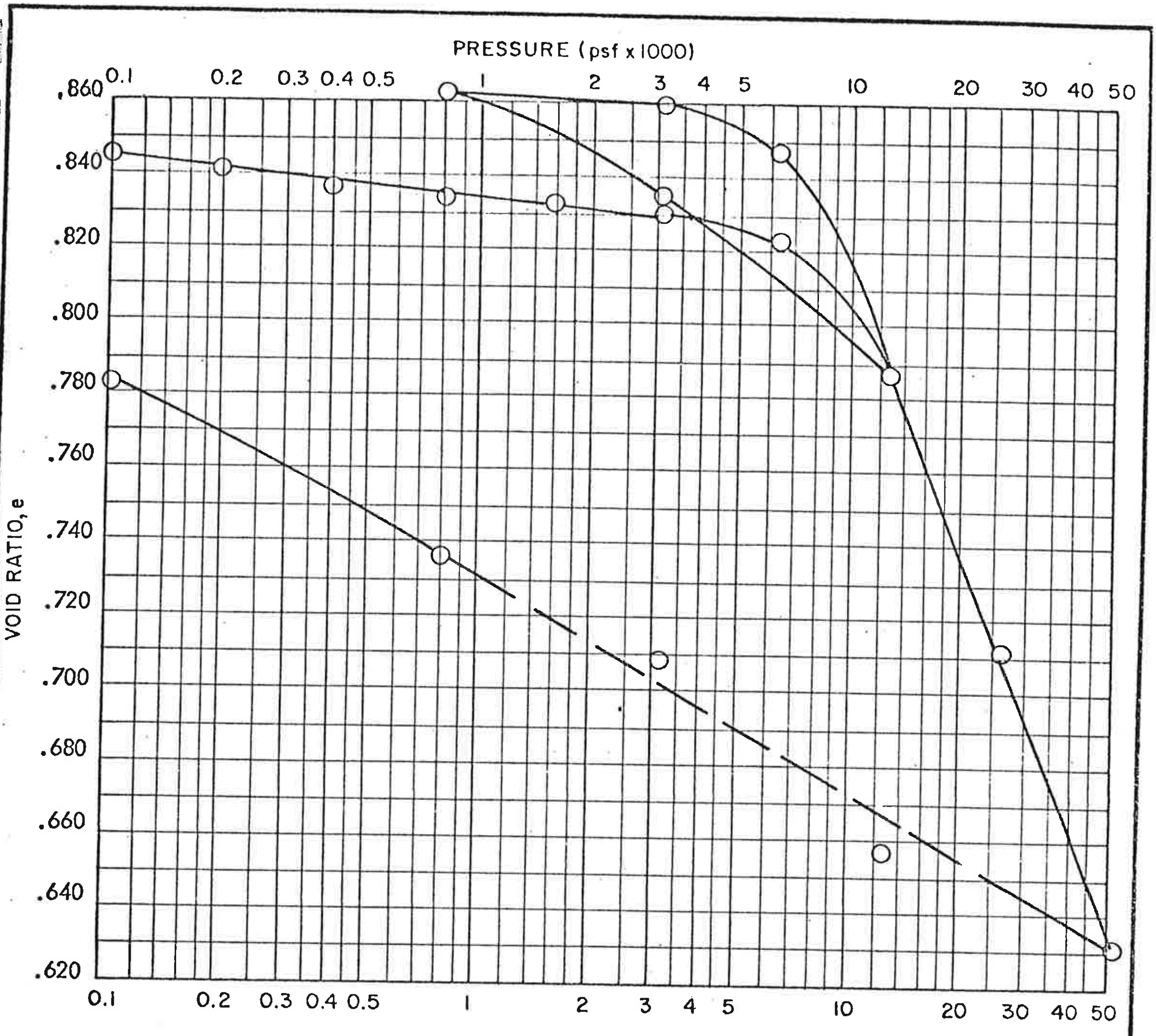

TYPE OF SPECIMEN Undisturbed		BEFORE TEST			AFTER TEST	
DIAMETER (in.) 2.43	HEIGHT (in.) 0.80	MOISTURE CONTENT	w_o	23.2 %	w_f	29.1 %
OVERBURDEN PRESS., P_o	210 psf	VOID RATIO	e_o	0.846	e_f	0.782
PRECONSOL. PRESS., P_c	8000 psf	SATURATION	S_o	74 %	S_f	100 %
COMPRESSION INDEX, C_c	0.26	DRY DENSITY	γ_d	91 pcf	γ_d	95 pcf

LL --

PL --

PI --

G_s 2.70 (assumed)

CLASSIFICATION Black Clay (CH)

SOURCE Boring 16 at 1.8'

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Consulting Engineers and Geologists

Job No. 1013,005.02 Appr. Date 2/4/76

CONSOLIDATION TEST REPORT
State Farm Insurance Company
Northern California Office
Rohnert Park, California

PLATE
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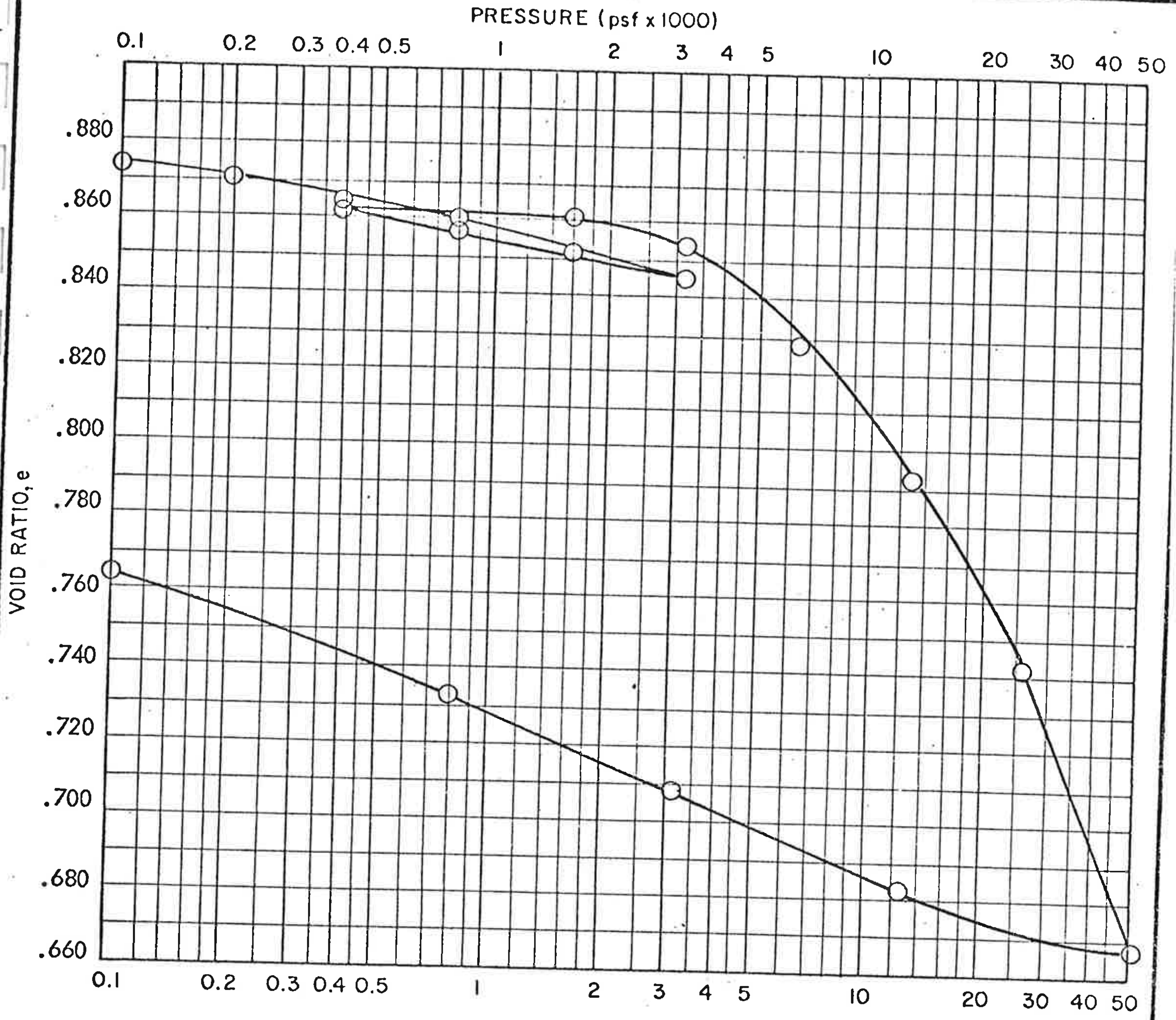

TYPE OF SPECIMEN Undisturbed		BEFORE TEST			AFTER TEST	
DIAMETER (in.) 2.43	HEIGHT(in.) 0.80	MOISTURE CONTENT	w_o	31.5 %	w_f	28.4 %
OVERBURDEN PRESS., P_o	420 psf	VOID RATIO	e_o	0.878	e_f	0.764
PRECONSOL. PRESS., P_c	5000 psf	SATURATION	S_o	97 %	S_f	100 %
COMPRESSION INDEX, C_c	0.13	DRY DENSITY	γ_d	90 pcf	γ_d	96 pcf
LL --	PL --	PI --		G_s 2.70 (assumed)		
CLASSIFICATION Light Brown Sandy Silt (MH)		SOURCE Boring 12 at 4.0'				

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Consulting Engineers and Geologists

Job No. 1013,005.02 Appr: GFJ Date 2/4/76

CONSOLIDATION TEST REPORT
State Farm Insurance Company
Northern California Office
Rohnert Park, California

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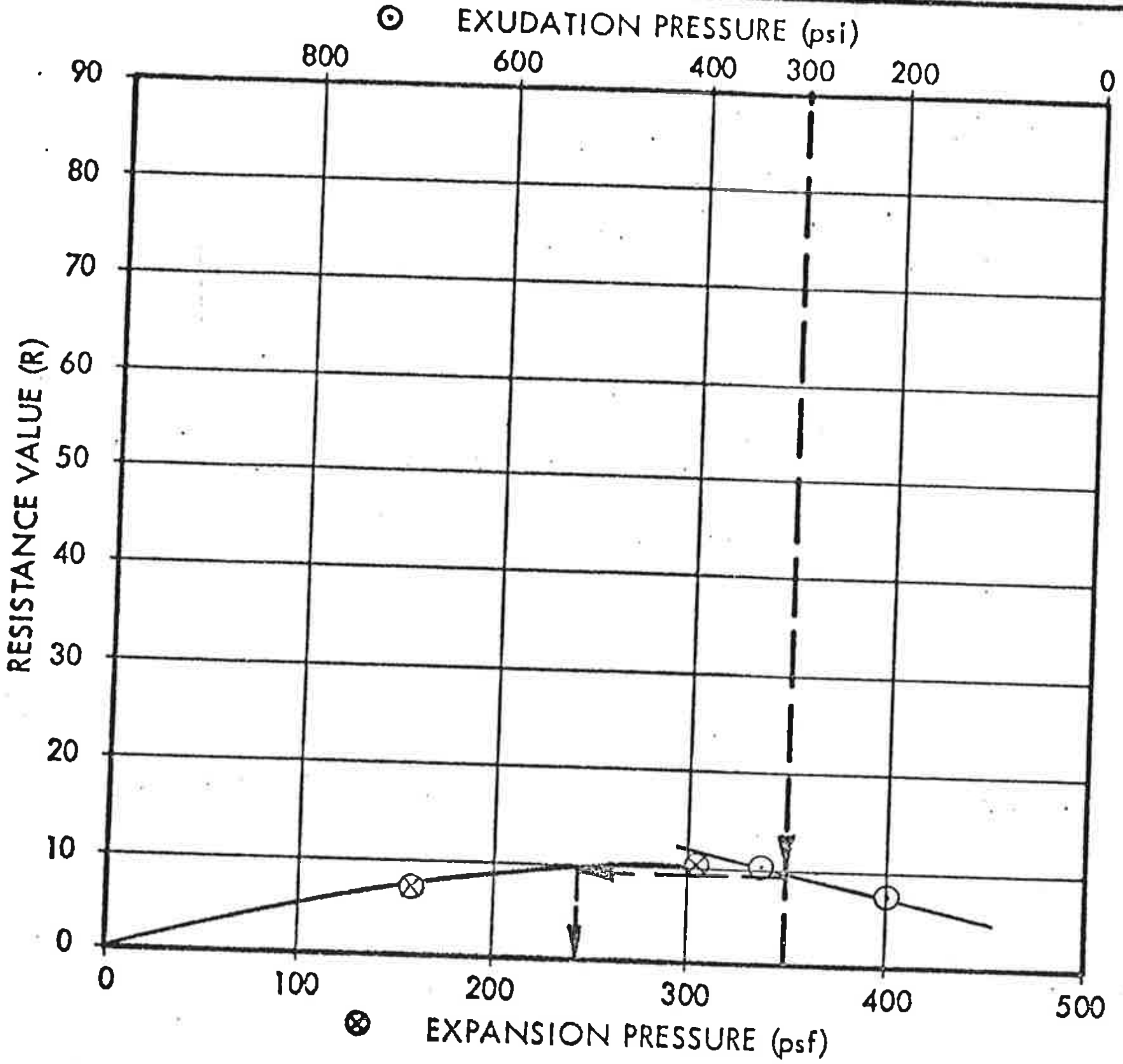

Specimen No.	1	2
Moisture Content (%)	33.6	34.3
Dry Density (psf)	87	84
Exudation Pressure (psi) *	330	200
Expansion Pressure (psf) *	305	160
Resistance Value (R) *	10	7

TEST DATA

*Water extruded under mold at pressures shown; therefore, R-value is less than 5

Sample Source	Classification	Sand Equivalent	Expansion Pressure	R value
Boring 1 at 0.5' to 2.0' combined with Boring 4 at 0.5' to 1.5'	BLACK CLAY (CH)	7	245	less than 5

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Job No. 1013,005.02 Appr GFS Date 2/4/76

RESISTANCE VALUE TEST DATA

State Farm Insurance Company
Northern California Office
Rohnert Park, California

PLATE

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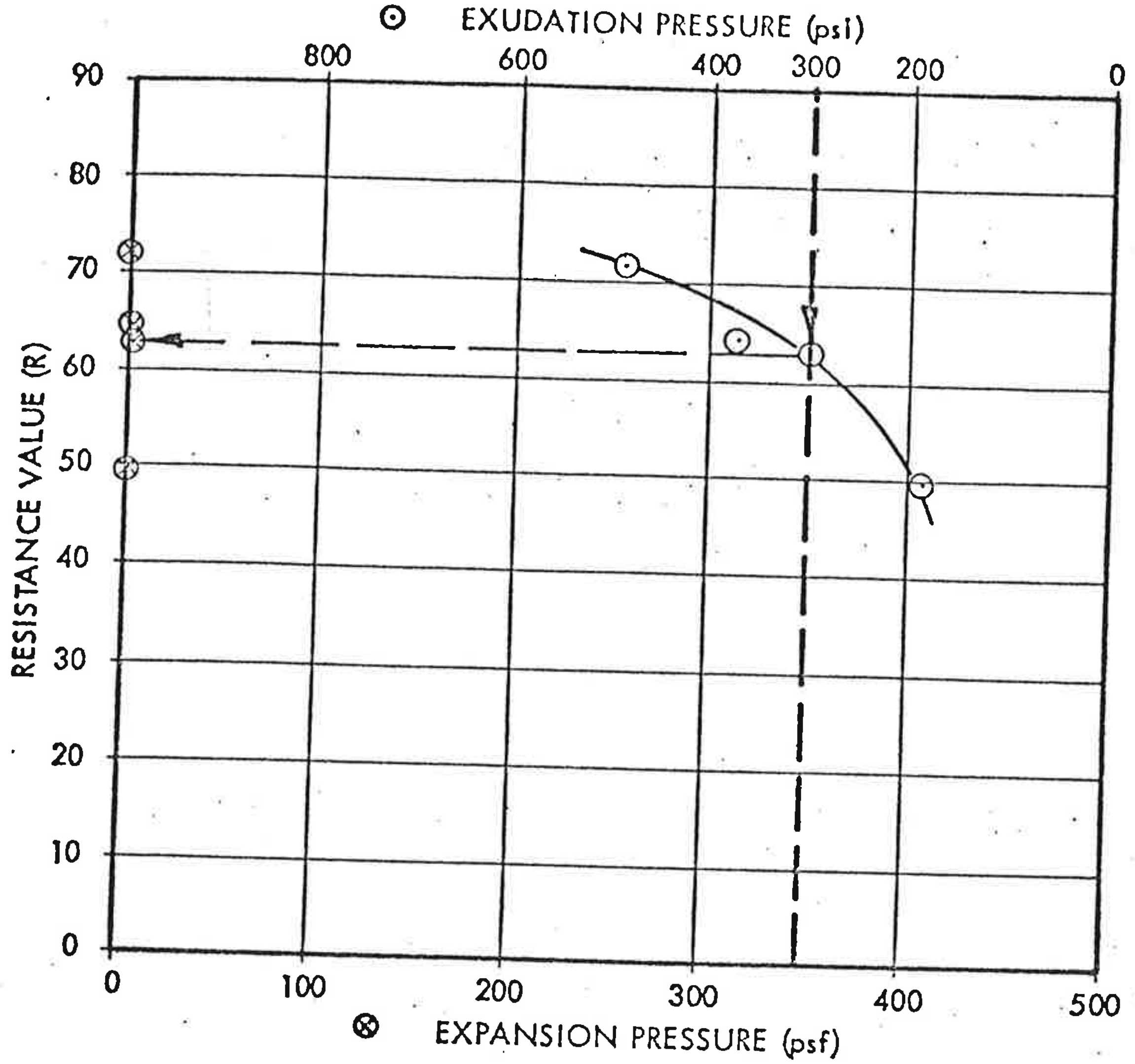

Specimen No.	1	2	3	4
Moisture Content (%)	37.5	36.1	35.0	33.9
Dry Density (psf)	83	84	85	86
Exudation Pressure (psi)	190	300	385	490
Expansion Pressure (psf)	0	0	0	0
Resistance Value (R)	50	63	64	72

TEST DATA

Sample Source	Classification	Sand Equivalent	Expansion Pressure	R value
Boring 1 at 0.5' to 2.0' combined with Boring 4 at 0.5' to 1.5'	BLACK CLAY (CH) with 4%, by dry weight, lime added in laboratory (cured 3 days)	13	0	63

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Consulting Engineers and Geologists

Job No. 1013,005.02 Appr. Date 2/4/76

RESISTANCE VALUE TEST DATA

State Farm Insurance Company
Northern California Office
Rohnert Park, California

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MOORE & TABER, 1977

MOORE & TABER NORTHERN CALIFORNIA
CONSULTING ENGINEERS AND GEOLOGISTS

577/76

TEST BORING LOG

TYPE 3½" Auger **ELEVATION** 97.8 **BORING Nº** 1

Unconfined Compressive Strength (TSF)	Other Tests	DRY DENSITY (lbs/cu.ft.)	MOISTURE (%)	BLOWS/FOOT 350 ft-lb	SAMPLE SIZE (INCHES)	SAMPLE Nº	DEPTH IN FEET	MATERIAL SYMBOL	UNIFIED SOIL CLASSIFICATION	
	Ⓢ	89	19	28	2.5	1			CL	Very stiff black silty clay with rootlets.
		105	22	26	1.4	2				
							5			
	Ⓢ	100	26	41	2.5	3			CL	Hard gray very fine to fine sandy clayey silt.
							10			
1.6		108	19	31	1.4	4			SC/	Very stiff gray and blue-gray clayey fine and fine to coarse sand and very fine sandy clay.
							15		CL	
4.0		94	30	22	1.4	5				
							20		CL	Stiff gray very fine sandy clayey silt.
		88	34	14	1.4	6				

No free groundwater surface encountered.

The boring logs show subsurface conditions at the dates and locations indicated, and it is not warranted that they are representative of subsurface conditions at other locations and times.

Logged By FPT **Date** 8/24/77

Figure-1
Page 1 of 4

MOORE & TABER NORTHERN CALIFORNIA
CONSULTING ENGINEERS AND GEOLOGISTS

577/76

TEST BORING LOG

TYPE 3½" Auger **ELEVATION** 97.1 **BORING Nº** 2

Unconfined Compressive Strength (TSF)	Other Tests	DRY DENSITY (lbs/cu.ft.)	MOISTURE (%)	BLOWS/FOOT 350 ft-lb	SAMPLE SIZE (INCHES)	SAMPLE Nº	DEPTH IN FEET	UNIFIED SOIL CLASSIFICATION	
2.0		109	19	17	1.4	1		CL	Stiff black silty clay with rootlets.
	Ⓢ	97	25	63	2.5	2			
							5	CL / SC	Hard gray very fine to fine sandy clay with some cemented nodules; grading to semicompact/dense gray clayey fine to coarse sand with fine gravel.
		105	21	20	1.4	3			
							10		
		104	18	40	1.4	4			
							15	CL	Very stiff gray very fine sandy clayey silt.
		94	29	23	1.4	5			

No free groundwater surface encountered.

Logged By FPT **Date** 8/24/77

Figure-1
Page 2 of 4

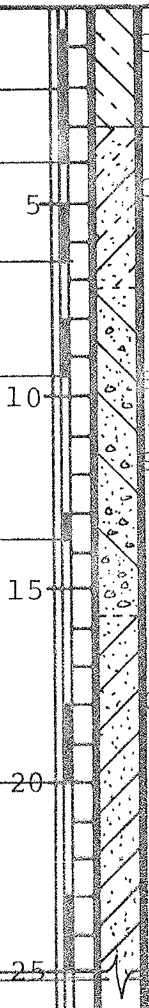
MOORE & TABER NORTHERN CALIFORNIA
CONSULTING ENGINEERS AND GEOLOGISTS

577/76

TEST BORING LOG

TYPE 3½" Auger **ELEVATION** 97.5 **BORING Nº** 3

Unconfined Compressive Strength (TSF)	Other Tests	DRY DENSITY (lbs/cu.ft.)	MOISTURE (%)	BLOWS/FOOT 350 ft-lb	SAMPLE SIZE (INCHES)	SAMPLE Nº
	Ⓢ	105	20	32	2.5	1
9.9		92	26	20	1.4	2
	Ⓢ	86	33	29	2.5	3
		122	11	63	1.4	4
		120	10	250+	1.4	5
		95	29	18	1.4	6
		90	33	31	1.4	7



UNIFIED SOIL CLASSIFICATION	
CL	Very stiff/stiff black silty clay with rootlets.
CL	Stiff to very stiff gray very fine sandy clayey silt.
SC SW	Dense to very dense gray slightly clayey fine gravelly fine to coarse sand grading to fine to coarse sandy fine gravel.
CL	Stiff to very stiff gray to blue-gray very fine to fine sandy clay.

No free groundwater surface encountered.

Logged By FPT **Date** 8/24/77

Figure-1
Page 3 of 4

MOORE & TABER NORTHERN CALIFORNIA
CONSULTING ENGINEERS AND GEOLOGISTS

577/76

TEST BORING LOG

TYPE 3½" Auger **ELEVATION** 97.1 **BORING Nº** 4

Unconfined Compressive Strength (TSF)		DRY DENSITY (lbs/cu.ft.)	MOISTURE (%)	BLOWS/FOOT 350 ft-lb	SAMPLE SIZE (INCHES)	SAMPLE Nº	DEPTH IN FEET	MATERIAL SYMBOL	UNIFIED SOIL CLASSIFICATION	
6.2		96	21	18	1.4	1			CL	Stiff black silty clay.
6.8		98	24	24	1.4	2	5		CL	Very stiff gray very fine sandy clayey silt to semicompact clayey silty very fine-fine sand with scattered gravel.
							10		SC	
		95	27	19	1.4	3				

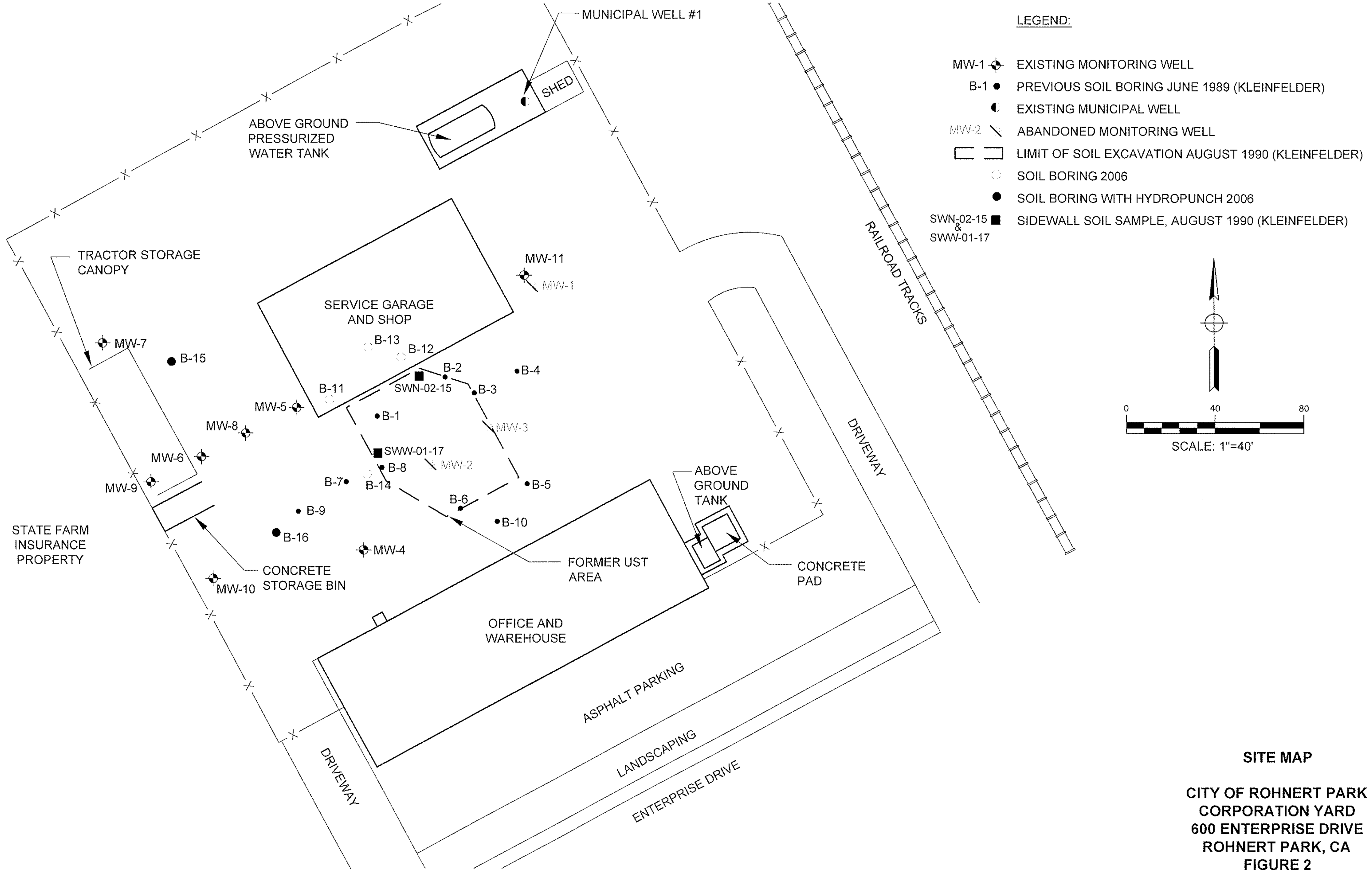
No free groundwater surface encountered.

Logged By FPT **Date** 8/24/77

Figure-1
Page 4 of 4

WINZLER & KELLY, 2008

J:\02056 - Rohnert Park\02056-06-009 Corp Yard 600 Enterprise Drive\CAD 02056-06-009\SITE MAP.dwg 12-10-07 12:42:28 PM NCox



SITE MAP

CITY OF ROHNERT PARK
CORPORATION YARD
600 ENTERPRISE DRIVE
ROHNERT PARK, CA
FIGURE 2

WINZLER & KELLY
CONSULTING ENGINEERS

Table 1. Water Level Data and Well Construction Details

Rohnert Park Corporate Yard
600 Enterprise Drive, Rohnert Park, California

Well ID	Date	Groundwater Elevation (Mean Sea Level)	Depth-to-Water	Top of Casing	Screen Interval	Sand Pack Interval	Bentonite/ Grout Interval
					feet		
MW-1	5/1/87	92.91	11.35	104.26	8.5-33.5	6.5-33.5	0-4
	6/27/89	89.76	14.50				
	11/9/90	83.98	20.28				
	1/30/91	81.63	22.63				
	3/24/92	92.58	11.68				
	5/2/03	95.08	9.18				
	2/19/04	96.65	7.61				
	5/19/04	94.27	9.99				
	8/25/04	92.65	11.61				
	Well Abandoned (October 27, 2004)						
MW-2	5/1/87	92.21	11.88	104.09	8.5-33.5	6.5-33.5	0-4
	6/27/89	90.21	13.88				
	Well Abandoned (July 1990)						
MW-3	5/1/87	92.71	10.04	102.75	8-33.5	6.5-33.5	0-4.5
	6/27/89	89.62	13.13				
	11/9/90	83.77	18.98				
	1/30/91	81.33	21.42				
	3/24/92	92.12	10.63				
	5/2/03	94.79	7.96				
	2/19/04	96.24	6.51				
	5/19/04	94.15	8.60				
	8/25/04	92.54	10.21				
	Well Abandoned (October 27, 2004)						
MW-4	11/9/90	83.43	19.73	103.16	23-28	22-28.5	0-22
	1/30/91	80.98	22.18				
	3/24/92	91.18	11.98				
	5/2/03	94.08	9.08				
	2/19/04	95.54	7.62				
	5/19/04	93.94	9.22				
	8/25/04	92.24	10.92				
	11/22/04	92.18	10.98				
	2/22/05	95.85	7.31				
	6/3/05	95.29	7.87				
	3/30/06	97.09	6.07				
	6/28/06	94.72	8.44				
	9/19/06	93.46	9.70				
	10/19/06	93.33	9.83				
	3/23/07	95.31	7.85				
	6/14/07	94.07	9.09				
	8/28/07	92.86	10.30				
	12/5/07	93.38	9.78				

Table 1. Water Level Data and Well Construction Details

Rohnert Park Corporate Yard
600 Enterprise Drive, Rohnert Park, California

Well ID	Date	Groundwater Elevation (Mean Sea Level)	Depth-to-Water	Top of Casing	Screen Interval	Sand Pack Interval	Bentonite/ Grout Interval
					feet		
MW-5	11/9/90	82.77	20.98	103.75	26-31	23.5-32	0-23.5
	1/30/91	80.27	23.48				
	3/24/92	90.22	13.53				
	5/2/03	94.02	9.73				
	2/19/04	95.33	8.42				
	5/19/04	93.79	9.96				
	8/25/04	92.52	11.23				
	11/22/04	92.01	11.74				
	2/22/05	95.93	7.82				
	6/3/05	95.43	8.32				
	3/30/06	97.30	6.45				
	6/28/06	94.75	9.00				
	9/19/06	93.47	10.28				
	10/19/06	93.31	10.44				
	3/23/07	95.35	8.40				
	6/14/07	94.10	9.65				
	8/28/07	92.87	10.88				
	12/5/07	93.39	10.36				
MW-6	11/9/90	82.6	20.78	103.38	23-31	25-33	0-25
	1/30/91	80.12	23.26				
	3/24/92	89.61	13.77				
	5/2/03	93.68	9.70				
	2/19/04	94.84	8.54				
	5/19/04	93.68	9.70				
	8/25/04	92.06	11.32				
	11/22/04	91.86	11.52				
	2/22/05	95.51	7.87				
	6/3/05	95.17	8.21				
	3/30/06	96.97	6.41				
	6/28/06	94.53	8.85				
	9/19/06	93.30	10.08				
	10/19/06	93.10	10.28				
	3/23/07	95.12	8.26				
	6/14/07	93.84	9.54				
	8/28/07	92.66	10.72				
	12/5/07	93.08	10.30				

Table 1. Water Level Data and Well Construction Details

Rohnert Park Corporate Yard
600 Enterprise Drive, Rohnert Park, California

Well ID	Date	Groundwater Elevation (Mean Sea Level)	Depth-to-Water	Top of Casing	Screen Interval	Sand Pack Interval	Bentonite/ Grout Interval
						feet	
MW-7	11/9/90	82.53	21.82	104.35	24.5-29.5	23.5-30	0-23.5
	1/30/91	79.95	24.40				
	3/24/92	89.53	14.82				
	5/2/03	93.59	10.76				
	2/19/04	94.75	9.60				
	5/19/04	93.56	10.79				
	8/25/04	91.96	12.39				
	11/22/04	91.74	12.61				
	2/22/05	95.56	8.79				
	6/3/05	95.25	9.10				
	3/30/06	97.08	7.27				
	6/28/06	94.58	9.77				
	9/19/06	93.27	11.08				
	10/19/06	93.07	11.28				
	3/23/07	95.21	9.14				
	6/14/07	93.87	10.48				
	8/28/07	92.66	11.69				
	12/5/07	93.04	11.31				
MW-8	2/19/04	94.78	8.64	103.42	8-18	7.5-18.5	0-7.5
	5/19/04	93.70	9.72				
	8/25/04	92.13	11.29				
	11/22/04	91.92	11.50				
	2/22/05	95.55	7.87				
	6/3/05	95.20	8.22				
	3/30/06	97.02	6.40				
	6/28/06	94.54	8.88				
	9/19/06	93.33	10.09				
	10/19/06	93.15	10.27				
	3/23/07	95.15	8.27				
	6/14/07	93.88	9.54				
	8/25/07	92.73	10.69				
	12/5/07	93.15	10.27				
MW-9	2/19/04	94.56	9.41	103.97	8-18	7.5-18	0-7.5
	5/19/04	93.61	10.36				
	8/25/04	92.06	11.91				
	11/22/04	91.81	12.16				
	2/22/05	95.34	8.63				
	6/3/05	95.06	8.91				
	3/30/06	96.85	7.12				
	6/28/06	94.44	9.53				
	9/19/06	93.26	10.71				
	10/19/06	93.06	10.91				
	3/23/07	95.02	8.95				
	6/14/07	93.77	10.20				
	8/28/07	92.63	11.34				
	12/5/07	92.98	10.99				

Table 1. Water Level Data and Well Construction Details

Rohnert Park Corporate Yard
600 Enterprise Drive, Rohnert Park, California

Well ID	Date	Groundwater Elevation (Mean Sea Level)	Depth-to-Water	Top of Casing	Screen Interval	Sand Pack Interval	Bentonite/ Grout Interval
					feet		
MW-10	12/5/2007	93.02	10.80	103.82	9.5-29.5	8.5-29.5	8.5-6.0
MW-11	12/5/2007	94.11	10.17	104.28	10.0-30.0	9.0-30.0	9.0-6.5

Notes: Monitoring wells MW-1 through MW-7 were surveyed on July 18, 2003 by Winzler & Kelly using the SCWA SA-15 benchmark. Monitoring wells MW-8 and MW-9 were surveyed on March 1, 2004. Elevations were measured relative to mean sea level. MW10 and MW-11 were installed on November 27, 2007 and surveyed November 29, 2007.