APPENDIX B

Geotechnical Investigation Report

Preliminary Geotechnical Investigation

Bassett Townhome Project
San Jose, California

Report No. 247567 has been prepared for:

LEGACY PARTNERS BASSETT, LLC

4000 East Third Avenue, Suite 600, Foster City, California 94404

January 13, 2016

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PRELIMINARY GEOTECHNICAL INVESTIGATION
BASSETT TOWNHOME PROJECT
SAN JOSE, CALIFORNIA

1.0 INTRODUCTION

This report presents the results of our preliminary geotechnical investigation for the Bassett Townhome project to be constructed in San Jose, California. The site location is shown on the Vicinity Map, Figure 1. The purpose of our investigation was to evaluate the geologic and subsurface conditions and to provide preliminary geotechnical recommendations for design of the proposed project.

As you know, we prepared a geotechnical report for a similar project on an adjacent site in 2011. The information contained in this report is based on the subsurface conditions on the adjacent site and assume similar subsurface conditions exist at the Bassett Townhome project site.

For our use, we received preliminary architectural plans titled, "Bassett Townhomes," prepared by BDE Architecture dated April 17, 2015.

1.1 Project Description

Based on the preliminary plans provided, we understand the project consists of developing the site with 30, three-story townhome units with slab-on-grade floors. The townhomes will likely be of wood-frame construction. The project will also include paved driveways, pedestrian walkways, landscaping and underground utilities.

The site currently consists of three parcels encompassing approximately ¾ of an acre. The project area in bounded by train tracks to the north, Terraine Street to the west, Bassett Street to the south and North San Pedro Street to the east. The street-level layout of the proposed development is shown on the Site Plan, Figure 2. Structural loads have not been provided to us; therefore we assumed that structural loads will be representative for this type of construction.

1.2 Scope of Services

Our scope of services was presented in our agreement with you dated November 30, 2015. To accomplish this work, we provided the following services:

- Review of the previous exploration of subsurface conditions and laboratory testing at the adjacent project site.
- Engineering analysis to evaluate structure foundations, and site earthwork.
- Preparation of this report to summarize our findings and to present our preliminary conclusions and recommendations.

2.0 SITE CONDITIONS

2.1 Site Geology

The Bassett Townhome project area is shown on geologic maps by Wesling and Helley (1989) Wagner (1991) and Graymer (2006) as alluvium. Graymer describes the alluvium as Holocene age. Wesling and Helley (1989) describe the soil as Holocene age medium to dark gray, dense, sandy to silty clay floodplain deposits with lenses of coarser silts, sands and pebbles found between levee deposits of Coyote Creek and Guadalupe River.

2.2 Previous Exploration Program

Subsurface exploration for the adjacent project was performed on May 26, 2011 and June 1, 2011. The exploration included drilling two borings with truck-mounted hollow stem auger drilling equipment to depths of 30 and 45 feet. The exploration also included advancing two cone penetration tests (CPTs) to depths of 75 feet. The logs of the borings and CPTs are included in Appendix A; laboratory tests are discussed in Appendix B.

2.3 Subsurface Conditions

Soils encountered in the CPTs were generally interpreted to include interbedded layers of silty clay, clayey silt and clay with a few layers of silty sand, sand and gravelly sand. Undrained shear strengths were interpreted to range from 200 to more than 3,200 pounds per square foot (psf), indicating soft to very stiff silts and clays. The silty sand, sand and gravelly sand layers were interpreted to be loose to very dense.

The borings encountered 2½ to 4½ feet of medium dense clayey gravel fill at the ground surface. Beneath the 2½ feet of soil fill in EB-1, the exploration encountered 1 foot of asphalt. Below the asphalt or fill, the borings encountered medium stiff to very stiff lean to silty clay, soft to stiff silt and medium stiff fat clay to depths of approximately 20 feet. From 20 to 30 feet below the ground surface (the maximum depth for boring EB-1), the borings encountered loose to medium dense silty sand and medium dense to dense poorly graded sand. From 30 to 40 feet below the ground surface, boring EB-2 encountered medium dense silty sand and soft to very stiff fat clay. From 40 to 45 feet below the ground surface, boring EB-2 encountered very stiff silt and dense poorly graded sand.

Two plasticity index (PI) tests were performed on a silt and lean clay sample collected from EB-1 at a depth of 14½ feet and EB-2 at a depth of 5½ feet, respectively. The tests results in PIs of 12 and 7 for the silt and lean clay, respectively, indicating low plasticity and expansion potential of soil below the fill.

2.4 Ground Water

Free ground water was encountered in the borings at depths of 17 and 20½ feet. Based on pore pressure dissipation measurements, CPTs encountered groundwater at a depth of 14 feet. Based on the depth to historically high ground water map prepared by the California Geological Survey for the San Jose West Quadrangle (CGS, 2002), the depth to historically high ground water levels in the site vicinity is estimated to be approximately 10 feet below the ground surface. Based on the above information, we judge a ground water depth of 10 feet to be appropriate for design. Our borings and CPTs were backfilled immediately after drilling. Fluctuations in the level of the ground water may occur due to variations in rainfall, underground drainage patterns, and other factors not evident at the time measurements were made.

3.0 GEOLOGIC HAZARDS

A brief qualitative evaluation of geologic hazards was made during this investigation. Our comments concerning these hazards are presented below.

3.1 Fault Rupture

The San Francisco Bay Area is one of the most seismically active regions in the United States. The significant earthquakes that occur in the Bay Area are generally associated with crustal movement along well-defined active fault zones of the San Andreas Fault system, which regionally trend in a northwesterly direction. The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone (known formerly as a Special Studies Zone), a Santa Clara County Fault Rupture Hazard Zone (SCC, 2002) or a City of San Jose Fault Hazard Zone (1983). As shown on Figure 3, no known surface expression of active faults is believed to cross the site. Fault rupture through the site, therefore, is not anticipated.

3.2 Maximum Estimated Ground Shaking

The peak ground acceleration was chosen based on data from Table 1, which summarizes different probabilistic and deterministically derived peak ground accelerations. Based on the available data, we judge a peak ground acceleration of 0.50g to be appropriate for geotechnical analyses.

Table 1. Summary of Peak Ground Acceleration Values

Data Source	Type	Peak Ground Acceleration (g)	Notes
USGS Seismic Hazard Curves, Response Parameters and Design Parameters program v5.1.0	PGA_M	0.50	Equation 11.8-1 of ASCE 7-10
CGS Seismic Hazard Zone Report 048, Figure 3.5	Probabilistic 10% in 50 years	0.50	
USGS 2008 Interactive Deaggregation Web Tool	Probabilistic 10% in 50 years	0.50	
USGS 2008 Interactive Deaggregation Web Tool	Probabilistic 2% in 50 years	0.76	
Caltrans ARS Web Tool v2.3.06	Deterministic	0.49	Silvery Creek Fault, period of 0.01 second

3.3 Future Earthquake Probabilities

Although research on earthquake prediction has greatly increased in recent years, seismologists cannot predict when or where an earthquake will occur. The U.S. Geological Survey’s Working Group on California Earthquake Probabilities (WGCEP, 2014) estimates there is a 72 percent chance of at least one magnitude 6.7 earthquake occurring in the San Francisco Bay region between 2014 and 2044. This result is an important outcome of WGCEP’s work because any major earthquake can cause damage throughout the region. The 1989 Loma Prieta earthquake demonstrated this potential by causing severe damage in Oakland and San Francisco, more than 50 miles from the fault epicenter.

Although earthquakes can cause damage at a considerable distance, shaking will be very intense near the fault rupture. Therefore, earthquakes located in urbanized areas of the region have the potential to cause much more damage than the 1989 Loma Prieta earthquake.

3.4 Liquefaction

The site is located within an area mapped by the State of California and the Santa Clara County as having the potential for seismically induced liquefaction. During cyclic ground shaking, such as earthquakes, cyclically-induced stresses may cause increased pore water pressures within the soil matrix, which results in liquefaction. Liquefied soil may lose shear strength that may lead to large shear deformations and/or flow failure (Youd et al., 2001). Liquefied soil can also settle as pore pressures dissipate following an earthquake. Limited field data is available on this subject; however, settlement on the order of 2 to 3 percent of the thickness of the liquefied zone has been measured in some cases.

Soils most susceptible to liquefaction are loose to moderately dense, saturated, non-cohesive soils with poor drainage, such as sands and silts with interbedded or capping layers of relatively low permeability soil.

We performed liquefaction analyses based on both Standard Penetration Test (SPT blow counts from borings) and CPT methods. Both liquefaction analysis followed the methods presented by the 1998 NCEER Workshops (Youd et al., 2001) in accordance with guidelines set forth in the CGS Special Publication 117A (2008). The NCEER methods for SPT and CPT analysis update simplified procedures presented by Seed and Idriss (1971). In broad terms, these methods are used to calculate a factor of safety against liquefaction triggering by comparing the resistance of the soil to cyclic shaking to the seismic demand that can be caused during seismic events.

The resistance to cyclic shaking is quantified by the Cyclic Resistance Ratio (CRR), which is a function of soil density, layer depth, ground water depth, earthquake magnitude, and soil behavior. The Cyclic Stress Ratio (CSR) is used to quantify the stresses that are anticipated to develop during cyclic shaking. The formula for CSR is shown below:

$$CSR = 0.65\,(a_{max}/g)(\sigma_{vo}/\sigma'_{vo})r_d$$

Where a_{max} is the peak horizontal acceleration at the ground surface generated by an earthquake, g is the acceleration of gravity, σ_{vo} and σ'_{vo} are total and effective overburden stresses, respectively, and r_d is a stress reduction coefficient.

The factor of safety (FS) against liquefaction can be expressed as the ratio of the CRR to CSR. If the FS for a soil layer is less than 1.3 (per DMP SP117A), the soil layer is considered liquefiable during a moderate to large seismic event.

$$FS = CRR/CSR$$

We evaluated the liquefaction potential of the potentially liquefiable layers using a peak ground acceleration of 0.50g (based on data from Table 1 above) and moment magnitude of 6.64 (USGS 2008). Based on our explorations and the depth to historic high ground water map prepared by the CGS, a design ground water level at 10 feet below the existing ground surface was used for our liquefaction analysis.

3.4.1 CPT Based Liquefaction Analysis

Our CPT tip pressures were corrected for the overburden and fines content. The CPT method utilizes the soil behavior type index (I_C) and the exponential factor “n” applied to the Normalized Cone Resistance “Q” to evaluate how plastic the soil behaves.

Soils that have CPT tip resistance greater than 160 tons per square foot (tsf) are considered too dense to liquefy and have been screened out of our analyses. Soils that have significant amounts of plastic fines (I_c greater than 2.6) are considered too plastic to liquefy but are included below per DMG SP117A (SCEC 1999). No liquefaction analyses were performed on layers above the design ground water depth. Our analysis was performed on the CPTs using computer software for liquefaction analysis.

Our CPT based analysis suggests that approximately 1 inch of total liquefaction induced settlement may occur in the sands and silts with differential settlements on the order of 2/3 inch in 50 horizontal feet.

3.4.2 SPT Based Liquefaction Analysis

For SPT based liquefaction analyses, CRR calculations are based on SPT blow counts. To account for effective overburden stresses and soil behavior, we corrected the field measured SPT blow counts for the overburden, stress reduction versus depth, fine-grained soil content, hammer energy ratio, boring diameter, rod length and sampling method (SPT sampler without liners).

Generally, soils that have greater than 35 percent of plastic fines or corrected SPT blow counts greater than 30 blows per foot are considered either too plastic or too dense to liquefy, respectively. Soil layers with corrected blow counts greater than 30 have been screened out of analysis and are not presented below. However, based on the liquid limit and water content of the silt layer encountered in borings EB-1 and EB-2 (from approximately 11 to 20 feet), the silt is susceptible to cyclic softening based on criteria by Bray and Sancio (2006). As methods to evaluate settlements of silts during ground shaking have not been clearly established, we have assumed the settlements will be half of the settlement calculated with traditional liquefaction evaluation methods.

Our SPT based analysis suggests that between 3 and 4 inch of total liquefaction induced settlement may occur in the sands and silts with differential settlements on the order of 2-2/3 inch in 50 horizontal feet.

3.4.3 Potential for Ground Rupture/Sand Boils

The methods of analysis used to determine estimated total settlement assume that there is no possibility of surface ground rupture. In order for liquefaction induced sand boils or fissures to occur, the pore water pressure induced within the liquefied strata must exert a large enough force to break through the surface layer.

The silt layer encountered from approximately 11 to 20 feet will likely not behave as a loose sand during a seismic event; however, there is not enough research in this area to completely discount the possibility. Assuming the silt layer could contribute to ground rupture, it is our opinion that there may not be sufficient thickness of non-liquefiable soil over the potentially liquefiable silt to prevent ground rupture. However, it is our opinion that not enough soil would be ejected at the ground surface to cause significant additional settlement. Therefore, we conclude that the above settlement estimates are reasonable. Please refer to the Foundation section of this report for additional recommendations.

3.5 Dry Seismic Settlement

If near-surface soils vary in composition both vertically and laterally, strong earthquake shaking can cause non-uniform densification of loose to medium dense cohesionless soil strata. This results in movement of the near-surface soils. The previous explorations encountered a loose sand layer above the design ground water depth that may densify during a strong earthquake.

Based on the method developed by Tokimatsu and Seed (1987) and a peak ground acceleration of 0.5g, we estimate that the loose sand layer encountered in boring EB-2 during the previous investigation would settle on the order of less than ¼ inch.

3.6 Lateral Spreading

Lateral spreading typically occurs as a form of horizontal displacement of relatively flat-lying alluvial material toward an open or “free” face such as an open body of water, channel, or excavation. In soils this movement is generally due to failure along a weak plane, and may often be associated with liquefaction. As cracks develop within the weakened material, blocks of soil displace laterally towards the open face. Cracking and lateral movement may gradually propagate away from the face as blocks continue to break free.

Guadalupe River is located approximately 1,000 feet west of the site boundary. Because of the distance to Guadalupe River, we judge the risk of lateral spreading at the site to be low.

3.7 Seismically Induced Waves

The site is at approximate elevation 80 feet msl. It is situated about 7 miles south of the San Francisco Bay mud flats which are essentially at sea level; beyond the mud flats to the north are a series of salt evaporators. These evaporators consist of dikes and levees that extend northward into the shallows/mud flats for approximately one mile. The site is also not located near any major drainage areas or reservoir that would be affected by or generate a seismically induced wave. Therefore, this potential hazard is not anticipated at the site.

3.8 Flooding and Reservoir Inundation

The nearest stream or river shown on the USGS Topographic Map (2015) of the area is Guadalupe River, which is located approximately 1,000 feet west of the site boundary. The Guadalupe River flows to the northwest.

The Flood Insurance Rate Map (FEMA, 2009) shows that the proposed project area is located in an area depicted as Flood Areas- Zone X, which is defined as “areas of 0.2 percent annual chance flood, areas of 1 percent annual chance flood with average depths of less than 1 foot or drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.”

The project area is located in a gently sloping urban area therefore most of the surface waters at the site are the result of rainfall or import water for irrigation. While either of these sources is capable of producing minor local flooding caused by plugged drains, adequate grading and drainage system maintenance should reduce this hazard to a minor problem.

3.9 Soil Erosion

Due to the proposed development (buildings, landscaping and pavements) covering the majority of the site and the relatively flat site topography, soil erosion is not anticipated to be an issue for the site.

3.10 Subsidence

Ground-water removal from the aquifers beneath Santa Clara Valley has caused subsidence of the ground surface over broad areas by compaction of the dewatered sediments. The rate of subsidence was greatest in the first half of the 20th century when pumping for agriculture was at its peak. Poland (1971) shows the area of Bassett Townhome Project subsided about 6 to 8 feet in the period from

1934 to 1967. Subsidence has stopped or greatly slowed now because of improved ground-water management. In our judgment regional subsidence will not pose a hazard at the project site.

3.11 Soil Expansion

Plasticity Index (PI) tests of near surface soils collected during our previous investigations resulted in PIs of 12 and 7, indicating low to moderate expansion potential due to changes in soil moisture content. Therefore, we judge that, if typical recommendations for this condition are followed during design and construction, soil expansion will not pose a threat to the proposed improvements.

4.0 CONCLUSIONS AND PRELIMINARY DEVELOPMENT RECOMMENDATIONS

From a geotechnical engineering viewpoint, it is our opinion that the site is suitable for the proposed development. The preliminary recommendations that follow are intended to be used for conceptual planning and preliminary design of the project. A design-level geotechnical investigation should be performed once a conceptual design has been finalized. Results from a design-level investigation would be used to supplement the preliminary findings and develop specific geotechnical recommendations for the project.

4.1 Primary Geotechnical Concerns

The primary geotechnical and geologic concerns at the site are as follows:

- Strong seismic shaking
- Demolition of the existing buildings prior to site development
- Potential for liquefaction induced settlement
- Potential for ground surface rupture
- Undocumented fill

We have prepared a brief description of the issues and present typical approaches to manage potential concerns associated with the long-term performance of the development.

4.1.1 Strong Seismic Shaking

We recommend that, at a minimum, the proposed project be designed in accordance with the seismic design criteria as discussed in the Maximum Estimated Ground Shaking section above, and the site seismic coefficients presented in Table 1.

4.1.2 Demolition Debris

Construction debris both above and below grade is anticipated as a result of the site demolition required prior to site grading. The debris should be either: 1) collected and off-hauled to an appropriate facility prior to beginning the earthwork for the project, or 2) the concrete crushed and re-used as fill at the site. If generated, recycled materials containing asphalt concrete (AC) should not be used below interior floor slabs, therefore if recycled materials are proposed to be re-used beneath interior floor slabs, AC pavements should be segregated from the debris. It has been our experience that some debris will remain in the soil on-site after the demolition contractor has completed their work. Therefore, it should be anticipated that some debris would be encountered in excavations for underground utilities and foundations. Some coordination between the demolition contractor, grading contractor and geotechnical engineer is needed to identify the scope of the excavation backfill and other similar work items. Recommendations for re-use of recycled materials are presented in the Earthwork section of this report.

4.1.3 Potential Liquefaction Induced Settlement

Based on the engineering analysis performed there is a likely potential for localized liquefaction during a major earthquake in sand and silt strata located at various depths between approximately 10 to 50 feet. Our analysis indicates that several layers theoretically can liquefy, ranging from 1 to 4 inches of total settlement in the upper 50 feet (2/3 to 2-2/3 inches differential in 50 horizontal feet). However, it is our opinion that the CPT based settlement value of approximately 1 inch are more representative of the site conditions. The proposed structures should be designed to accommodate the resulting seismic and static settlements; detailed recommendations are presented in the Foundation section of this report.

4.1.4 Potential Ground Surface Rupture

Assuming the silt layer encountered from approximately 11 to 20 feet could contribute to ground rupture, it is our opinion that there may not be sufficient thickness of non-liquefiable soil over the potentially liquefiable silt to fully prevent ground rupture. However, the settlement estimates are judged to accurately reflect the potential for ground rupture and the structures can be designed to resist the adverse effects of potential ground rupture.

4.1.5 Undocumented Fill

The previous borings encountered approximately 3½ to 4½ feet of fill, respectively. The fill consisted of medium dense clayey gravel and AC. Based on the dry densities and blow counts, the fill did not appear to be compacted engineered fill. To reduce the risk of settlement of foundations bearing on undocumented fills, and because fill containing asphalt should not be used below residential structures, we recommended the removal of the total depth of undocumented fill and replacement with clean, engineered fill. Detailed Earthwork recommendations would be provided in a design-level report.

4.2 Design-Level Geotechnical Investigation

Our preliminary geotechnical investigation was based on historical information from an adjacent site. In addition, because subsurface conditions may vary considerably from those predicted by the widely-spaced borings, and in order to confirm that our report recommendations have been properly implemented, we recommend that we be retained to 1) perform a design-level geotechnical investigation once site development plans are completed, 2) review the final construction plans and specifications, and 3) observe the earthwork and foundation installation.

5.0 PRELIMINARY FOUNDATIONS RECOMMENDATIONS

Based on our investigation, we anticipate the proposed structures may be supported on shallow foundations consisting of footings with grade beams or conventionally reinforced concrete mats as discussed below.

5.1 2013 CBC Site Coefficients and Site Seismic Coefficients

Chapter 16 of the 2013 California Building Code (CBC) outlines the procedure for seismic design of structures. Based on the previous explorations, the site is generally underlain by soft to very stiff clays and silts and loose to dense sands and gravels, which corresponds to a soil profile type D. Based on the above information and local seismic sources, the site may be characterized for design using the information in Table 2 below.

Table 2. 2013 CBC Site Class and Site Seismic Coefficients

Latitude: 37.3404 N **Longitude: 121.8976 W**	**CBC Reference**	**Factor/ Coefficient**	**Value**
Soil Profile Type	Section 1613.3.2	Site Class	D
Mapped Spectral Response Acceleration for MCE at 0.2 second Period	Figure 1613.3.1(1)	S_s	1.50
Mapped Spectral Response Acceleration for MCE at 1 Second Period	Figure 1613.3.1(2)	S_1	0.60
Site Coefficient	Table 1613.3.3(1)	F_a	1.0
Site Coefficient	Table 1613.3.3(2)	F_v	1.5
Adjusted MCE Spectral Response Parameter	Equation 16-37	S_{MS}	1.50
Adjusted MCE Spectral Response Parameter	Equation 16-38	S_{M1}	0.90
Design Spectral Response Acceleration Parameter	Equation 16-39	S_{DS}	1.00
Design Spectral Response Acceleration Parameter	Equation 16-40	S_{D1}	0.60

5.2 Footings

On a preliminary basis, the proposed townhomes may be supported on spread footings and grade beams bearing on natural, undisturbed soil or compacted engineered fill. Conventional footings are not recommended because of the relatively high magnitude of potential liquefaction induced settlement and potential for ground rupture. All footings should have a minimum width of at least 24 inches and footing bottoms should extend at least 24 inches below lowest adjacent finished grade. Lowest adjacent finished grade may be taken as the bottom of interior slab-on-grade or the finished exterior grade, excluding landscape topsoil, whichever is lower.

On a preliminary basis, we anticipate that footings constructed in accordance with the above recommendations would be capable of supporting maximum allowable bearing pressures of 2,000 pounds per square foot (psf) for dead loads, 3,000 psf for combined dead and live loads, and 4,000 psf for all loads including wind or seismic. These allowable bearing pressures are based upon factors of safety of 3.0, 2.0, and 1.5 for dead, dead plus live, and seismic loads, respectively.

7.1 Reinforced Mat Foundations

The proposed improvements may be supported on conventionally reinforced mat foundations at least 10 inches thick, bearing at least 12 inches below the lowest adjacent finished grade. Based on the subsurface conditions, the mat may be designed for an average allowable bearing pressure of 1,000 pounds per square foot (psf) for dead plus live loads with maximum localized allowable bearing pressures of 3,000 psf at column or wall loads. Allowable bearing pressures may be increased by one-third for all loads including wind or seismic. These allowable bearing pressures are net values; the weight of the mat can be neglected for design purposes.

The mat should be reinforced with top and bottom steel, as appropriate, to provide structural continuity and to permit spanning of local irregularities. These recommendations may be revised depending on the particular design method selected by the structural engineer. It is essential that we observe the subgrade of the mat foundation prior to placement of reinforcing steel.

6.0 LIMITATIONS

This report has been prepared for the sole use of Legacy Partners Bassett, LLC, specifically for preliminary design of the proposed Bassett Townhomes project in San Jose, California. The opinions, conclusions, and recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in the San Francisco Bay Area at the time this report was written. No other warranty, expressed or implied, is made or should be inferred.

The opinions, conclusions and recommendations contained in this report are based upon the information obtained from our prior investigation, which included data from widely separated discrete locations, visual observations from our site reconnaissance, and review of other geotechnical data provided to us, along with local experience and engineering judgment. The recommendations presented in this report are based on the assumption that soil and geologic conditions at the adjacent site do not deviate substantially from those encountered or extrapolated from the information collected during our investigation. We are not responsible for the data presented by others.

We should be retained to review the geotechnical aspects of the final plans and specifications for conformance with our recommendations. The recommendations provided in this report are based on the assumption that we will be retained to provide observation and testing services during construction to confirm that conditions are similar to that assumed for design and to form an opinion as to whether the work has been performed in accordance with the project plans and specifications. If we are not retained for these services, TRC cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of TRC’s report by others. Furthermore, TRC will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services and/or at the time another consultant is retained for follow up service to this report.

The opinions presented in this report are valid as of the present date for the property evaluated. Changes in the condition of the property will likely occur with the passage of time due to natural processes and/or the works of man. In addition, changes in applicable standards of practice can occur as a result of legislation and/or the broadening of knowledge. Furthermore, geotechnical issues may arise that were not apparent at the time of our investigation. Accordingly, the opinions presented in this report may be invalidated, wholly or partially, by changes outside of our control. Therefore, this report is subject to review and should not be relied upon after a period of three years, nor should it be used, or is it applicable, for any other properties.

7.0 REFERENCES

Bray, J.D. and Sancio, R.B., 2006, *Assessment of the Liquefaction Susceptibility of Fine-Grained Soils*, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 132 (9).

California Building Code, 2013, *Structural Engineering Design Provisions*, Vol. 2.

California Geological Survey, 2002, *State of California Seismic Hazard Report, San Jose West Quadrangle*, Seismic Hazard Zone Report 058.

City of San Jose, 1983, *City of San Jose Fault Hazard Map, San Jose West Quadrangle,* scale 1:24,000.

County of Santa Clara Office of Emergency Services, 2011, *Annex to 2010 Association of Bay Area Local Hazard Mitigation Plan – Taming Natural Disasters*, Figure 4-18: Potential Dam Failure Inundation Areas, December.

County of Santa Clara Planning Office, 2013, County Geologic Hazard Zone Maps, http://www.sccgov.org/sites/PLANNING/GIS/GEOHAZARDZONES/Pages/SCCGeoHazardZoneMaps.aspx.

Federal Emergency Management Agency, 2009, Flood Insurance Rate Map, Santa Clara County, California and Incorporated Areas, Panel 234 of 830, Map 06085C0234H, effective date May 18, 2009.

Graymer, R.W. et al., 2006, *Geologic Map of the San Francisco Bay Region,* U.S. Geological Survey, California Geological Survey, Scientific Investigation Map 2918

Ishihara, K. and Yoshimine, M., 1990, *Evaluation of Settlements in Sand Deposits Following Liquefaction During Earthquakes*, Soils and Foundations, 32 (1): 173-188.

Poland, J.F., 1971, *Land subsidence in the Santa Clara Valley, Alameda, San Mateo, and Santa Clara Counties, California*: U.S. Geological Survey Technical Report 2, scale 1:125,000.

Southern California Earthquake Center (SCEC), 1999, *Recommended Procedures for Implementation of DMG Special Publication 117A, Guidelines for Analyzing and Mitigating Liquefaction Hazards in California*, University of Southern California, March.

Tokimatsu, K., and H.B. Seed. (1987). “Evaluation of settlements in sands due to earthquake shaking.” J. Geotech. Eng. Div., ASCE, 113(8), 861-78.

United States Geological Survey, 2008, *Geologic Hazards Science Center – 2008 Interactive Deaggregations*, http://geohazards.usgs.gov/deaggint/2008/

U.S. Geological Survey, 2013, *US Seismic Design Maps*, Earthquake Hazards Program, http://earthquake.usgs.gov/designmaps/us/application.php.

Wagner, D.L. et al, 1991, *Geologic Mpa of the San Francisco- San Jose Quadrangle, Regional Geologic Map Series, Map No. 5A, Sheet 1,* State of California Division of Mines and Geology, 1:250,000 scale.

Wesling, J.R. and Helley, E.J., 1989, *Quaternary Geologic Map of the San Jose West Quadrangle, Santa Clara County, California*, Department of the Interior, U.S. Geological Survey, Open-File Report 89-672.

WGCEP [Working Group on California Earthquake Probabilities], 2014, The Uniform California Earthquake Rupture Forecast, Version 2: U.S Geological Survey, Open File Report 2014-2044.

Youd, T.L. and C.T. Garris, 1995, *Liquefaction-Induced Ground-Surface Disruption:* Journal of Geotechnical Engineering, Vol. 121, No. 11, pp. 805 - 809.

Youd, T.L. and Idriss, I.M., et al., 2001, *Liquefaction Resistance of Soils: Summary Report From the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils,* ASCE Geotechnical and Geoenviromental Journal, October 2001.

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SOURCE AERIAL PHOTO: Google Earth, March 2015.

N

APPROXIMATE SCALE (FEET)

0 500 1,000

VICINITY MAP

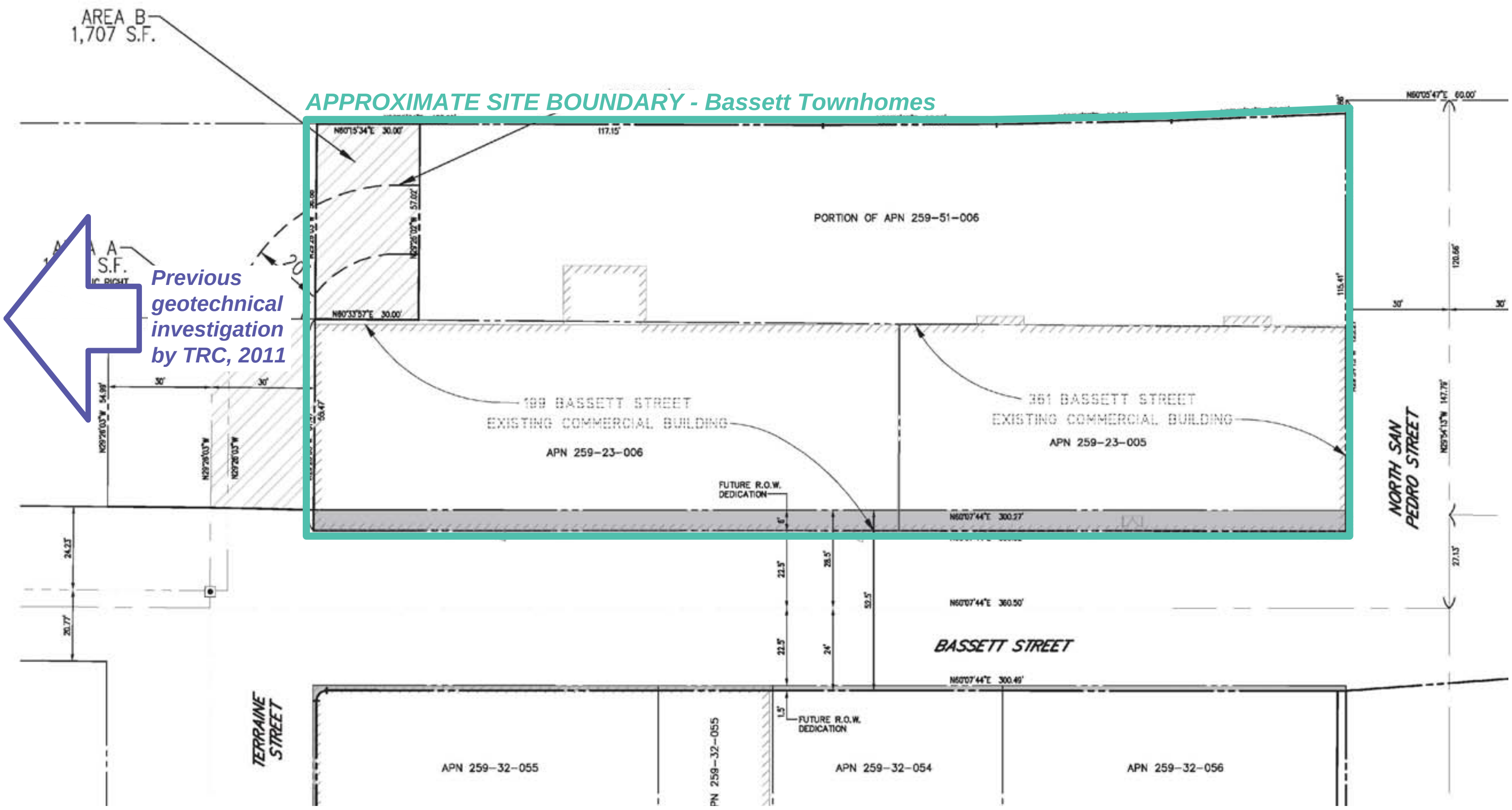
Bassett Townhome Project
Bassett and North San Pedro Streets
San Jose, California

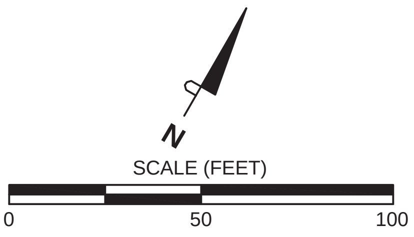
TRC

247567

FIGURE 1

FILE NAME: Z:\CAD_DRAW\Current\Bassett Townhomes_San Jose\Fig1_Vicinity Map.dwg | Layout Tab: 8x11





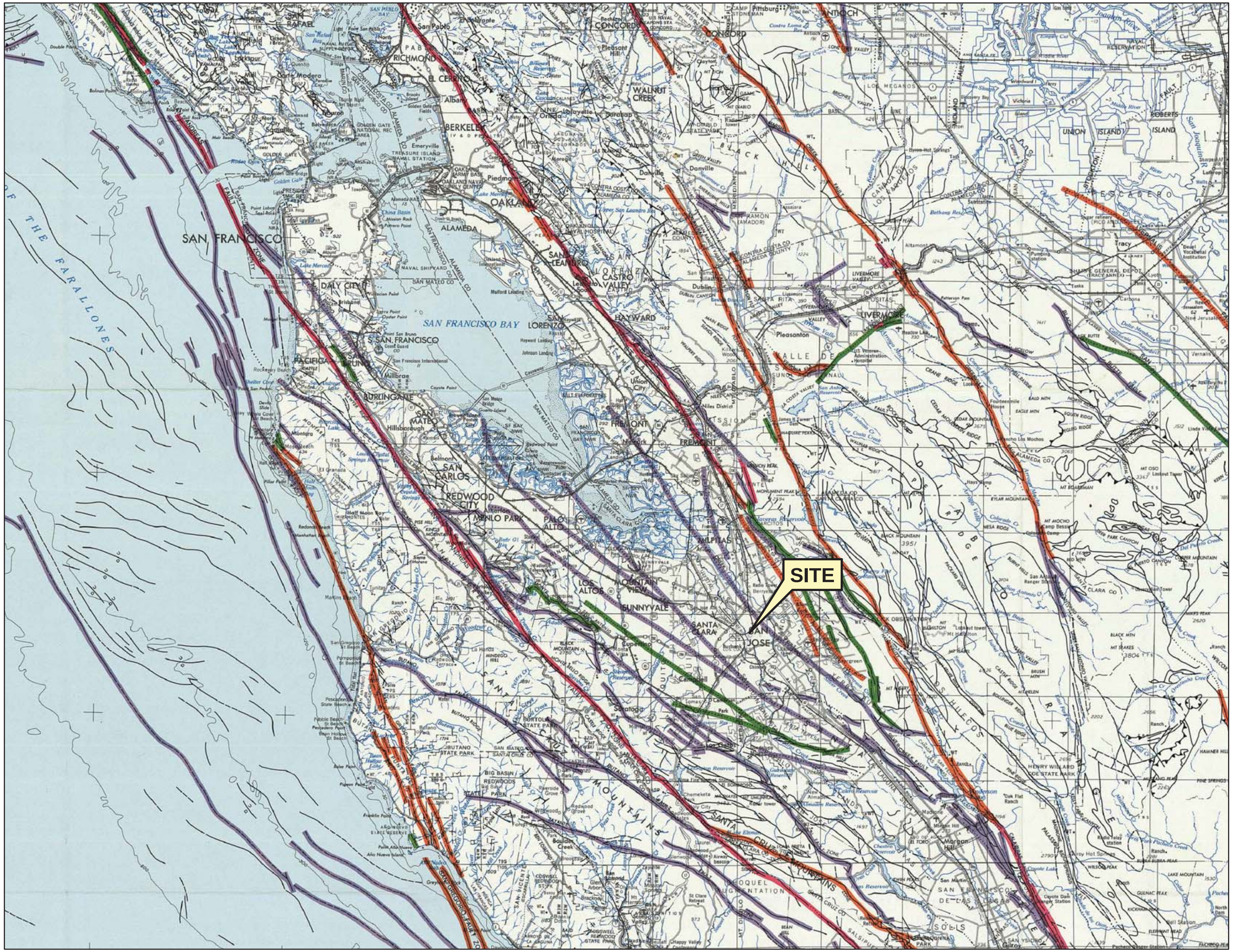
SOURCE: Bassett Townhome Project site plan by Civil Engineering Associates, December 2014.

SITE PLAN

Bassett Townhome Project
Bassett and North San Pedro Streets
San Jose, California

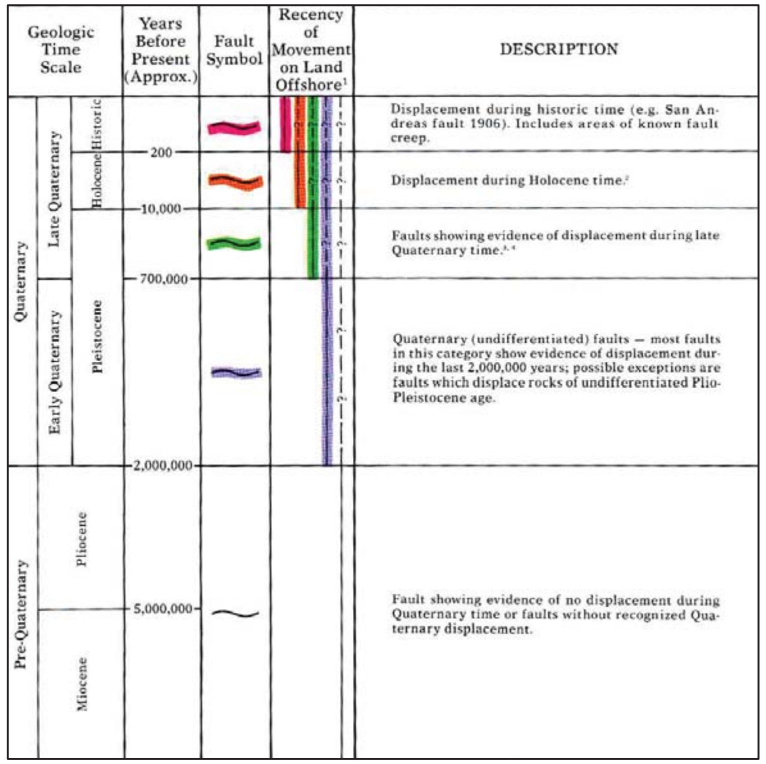
TRC 247567 **FIGURE 2**

FILE NAME: Z:\CAD_DRAW\Current\Bassett Townhomes_San Jose\Fig2_Site Plan_REV.dwg | Layout Tab: 8x11



SCALE: 1:500,000

APPROXIMATE GRAPHICAL SCALE (MILES)
0 10 20



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement on Land Offshore[1]	DESCRIPTION
Quaternary	Late Quaternary	Historic		🟥		Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.
		Holocene	200	🟧		Displacement during Holocene time.[2]
		Pleistocene	10,000	🟩		Faults showing evidence of displacement during late Quaternary time.[3,4]
	Early Quaternary		700,000	🟪		Quaternary (undifferentiated) faults — most faults in this category show evidence of displacement during the last 2,000,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.
Pre-Quaternary	Pliocene		2,000,000			
	Miocene		5,000,000	~		Fault showing evidence of no displacement during Quaternary time or faults without recognized Quaternary displacement.

NOTES:

Base map is a composite of part the San Francisco 1:250,000 scale map (reference code 37 122-A1-TF-250-00, 1980) and the San Jose 1:250,000 scale map (reference code 37 120-A1-TF-250-00, 1969). For cartographic details, refer to these maps. Bathymetric information is not intended for navigational purposes.

Transverse Mercator Projection 10,000-meter Universal Transverse Mercator grid, zone 10.

Minor corrections and additions to culture by California Division of Mines and Geology 1987.

From: Bortugno & others (1991)

Some faults highlighted in purple are not considered active (Holocene Movement) by the State of California.

REGIONAL FAULT MAP

Bassett Townhome Project
Bassett and North San Pedro Streets
San Jose, California

TRC
247567
FIGURE 3

FILE NAME: Z:\CAD_DRAW\Current\Bassett Townhomes_San Jose\Fig3_Regional Fault Map.dwg | Layout Tab: 11x17

APPENDIX A

BORING AND CPT LOGS FROM PREVIOUS INVESTIGATION

PRIMARY DIVISIONS			SOIL TYPE		SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (Less than 5% Fines)	GW	●	Well graded gravels, 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, plastic fines
			GC	▨	Clayey gravels, gravel—sand—clay mixtures, plastic fines
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (Less than 5% Fines)	SW	⁘	Well graded sands, gravelly sands, little or no fines
			SP	⁘	Poorly graded sands or gravelly sands, little or no fines
		SANDS WITH FINES	SM	▥	Silty sands, sand—silt—mixtures, non—plastic fines
			SC	▨	Clayey sands, sand—clay mixtures, plastic fines
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT IS 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 silts and organic silty clays of low plasticity
	SILTS AND CLAYS LIQUID LIMIT IS GREATER THAN 50 %		MH	▥	Inorganic silts, micaceous or diatomaceous 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

DEFINITION OF TERMS

U.S. STANDARD SIEVE SIZE — CLEAR SQUARE SIEVE OPENINGS

200 40 10 4 3/4" 3" 12"

SILTS AND CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

0.08 0.4 2 5 19 76mm

GRAIN SIZES

☒	TERZAGHI SPLIT SPOON STANDARD PENETRATION
⧗	MODIFIED CALIFORNIA
▯	ROCK CORE
◩	PITCHER TUBE
○	NO RECOVERY

SAMPLERS

SAND AND GRAVEL	BLOWS/FOOT*
VERY LOOSE	0—4
LOOSE	4—10
MEDIUM DENSE	10—30
DENSE	30—50
VERY DENSE	OVER 50

RELATIVE DENSITY

SILTS AND CLAYS	STRENGTH+	BLOWS/FOOT*
VERY SOFT	0—1/4	0—2
SOFT	1/4—1/2	2—4
MEDIUM STIFF	1/2—1	4—8
STIFF	1—2	8—16
VERY STIFF	2—4	16—32
HARD	OVER 4	OVER 32

CONSISTENCY

*Number of blows of 140 pound hammer falling 30 inches to drive a 2—inch O.D. (1—3/8 inch I.D.) split spoon (ASTM D—1586).
+Unconfined compressive strength in tons/sq.ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D—1586), pocket penetrometer, torvane, or visual observation.

KEY TO EXPLORATORY BORING LOGS

Unified Soil Classification System (ASTM D—2487)

TRC

FIGURE A—1

EXPLORATORY BORING: EB-1

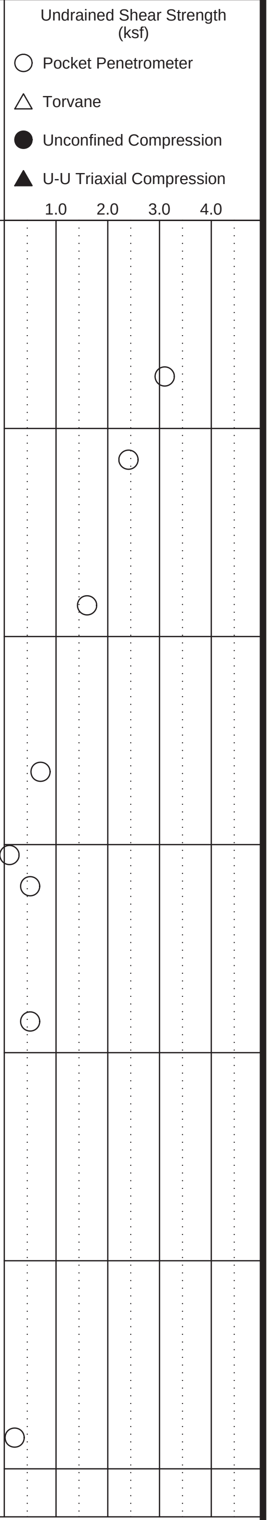
Sheet 1 of 2

DRILL RIG: MOBILE B-90	PROJECT NO:
BORING TYPE: 8-INCH HOLLOW STEM AUGER	PROJECT:
LOGGED BY: AC	LOCATION: SAN JOSE, CA
START DATE: 6-1-11 FINISH DATE: 6-1-11	COMPLETION DEPTH: 30.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS SURFACE ELEVATION:	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
	0	**CLAYEY GRAVELS (GC) [FILL]** medium dense, moist, brown fine to coarse sand, fine to coarse gravel (sub-angular/rounded)	GC, FILL	19	33	64	
		Asphalt (AC) [FILL]	AC, FILL				
		LEAN CLAY WITH SILT (CL) very stiff, moist, brown, low plasticity	CL	12	18	106	
	5	**LEAN CLAY (CL)** very stiff, moist, brown, low plasticity, trace fine sand	CL	16	19	108	
		SILTY CLAY (CL) stiff, moist, brown, low plasticity, shells present	CL	12	27	97	
	10						
		SILT (ML) medium stiff, moist, brown, low plasticity, trace sand and shells	ML				
	15	Plasticity Index = 12, Liquid Limit = 39		11	38	82	
		soft trace sand					
		FAT CLAY (CH) medium stiff, moist, dark gray, moderate to high plasticity, trace fine to coarse sand	CH	13	25	100	
	20	**LEAN CLAY WITH SAND (CL)** medium stiff, moist, light gray-brown, low plasticity, fine to coarse sand, trace fine gravel (sub-angular/rounded)	CL				
		SILTY SAND (SM) loose, wet, brown to greenish gray, fine sand	SM	11			
	25	medium dense		13			45
				31			
		POORLY GRADED SAND WITH SILT medium dense, moist, gray, fine sand	SP-SM				
	30	**FAT CLAY (CH)**	CH	21	31		

Continued Next Page



GROUND WATER OBSERVATIONS:

▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 17.0 FEET

EXPLORATORY BORING: EB-1 Cont'd

Sheet 2 of 2

DRILL RIG: MOBILE B-90	PROJECT NO:
BORING TYPE: 8-INCH HOLLOW STEM AUGER	PROJECT:
LOGGED BY: AC	LOCATION: SAN JOSE, CA
START DATE: 6-1-11 FINISH DATE: 6-1-11	COMPLETION DEPTH: 30.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

Undrained Shear Strength (ksf)

○ Pocket Penetrometer

△ Torvane

● Unconfined Compression

▲ U-U Triaxial Compression

1.0 2.0 3.0 4.0

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
	30		soft, moist, dark gray, high plasticity Bottom of boring at 30 feet						
	35								
	40								
	45								
	50								
	55								
	60								

GROUND WATER OBSERVATIONS:

∇ : FREE GROUND WATER MEASURED DURING DRILLING AT 17.0 FEET

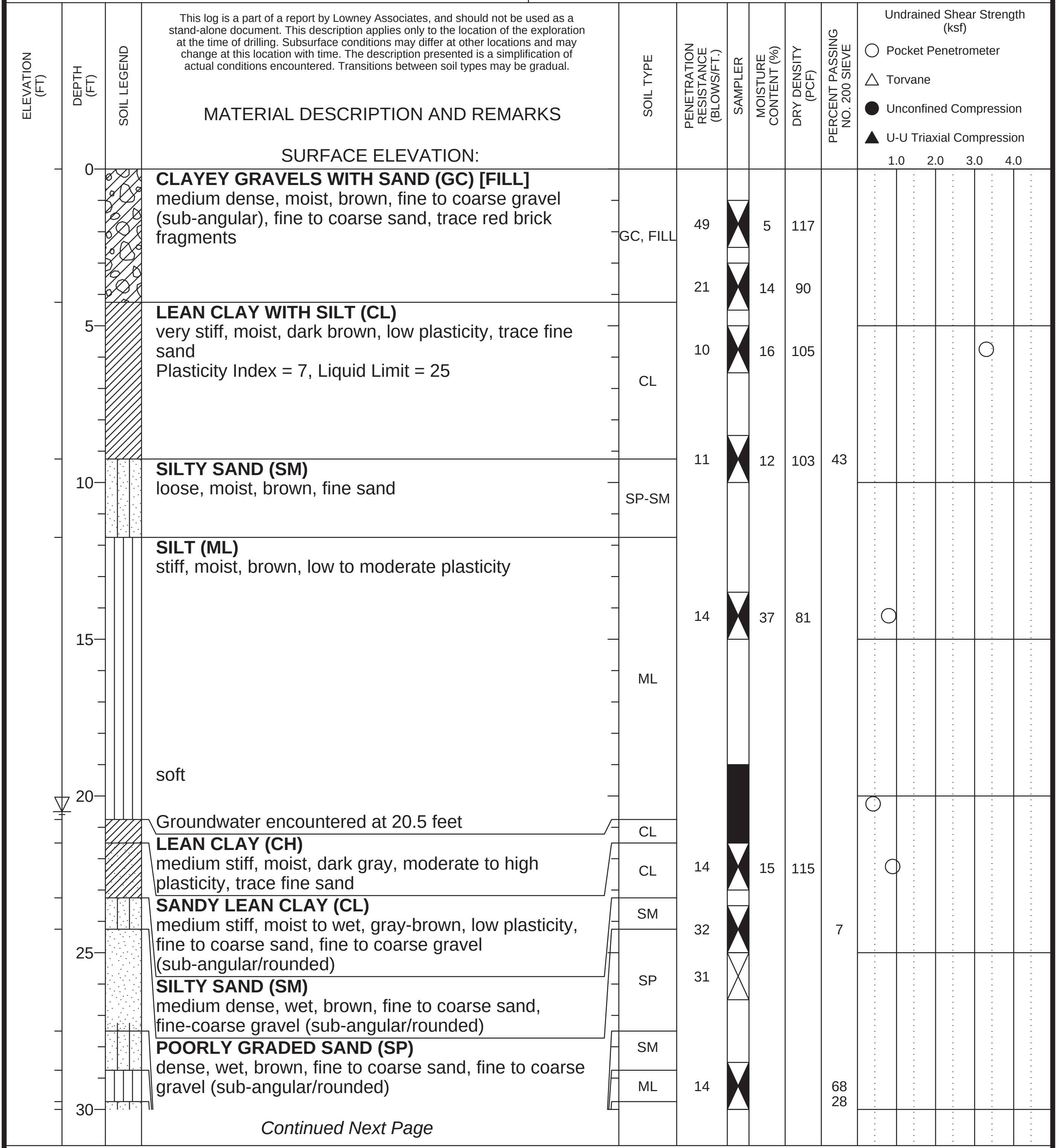
LA_CORP.GDT 6/13/11 MV*

EXPLORATORY BORING: EB-2

Sheet 1 of 2

DRILL RIG: MOBILE B-40
BORING TYPE: 8-INCH HOLLOW STEM AUGER
LOGGED BY: AC
START DATE: 6-1-11 FINISH DATE: 6-1-11
PROJECT NO:
PROJECT:
LOCATION: SAN JOSE, CA
COMPLETION DEPTH: 45.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.



MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION:

CLAYEY GRAVELS WITH SAND (GC) [FILL]
medium dense, moist, brown, fine to coarse gravel (sub-angular), fine to coarse sand, trace red brick fragments

LEAN CLAY WITH SILT (CL)
very stiff, moist, dark brown, low plasticity, trace fine sand
Plasticity Index = 7, Liquid Limit = 25

SILTY SAND (SM)
loose, moist, brown, fine sand

SILT (ML)
stiff, moist, brown, low to moderate plasticity

soft

Groundwater encountered at 20.5 feet

LEAN CLAY (CH)
medium stiff, moist, dark gray, moderate to high plasticity, trace fine sand

SANDY LEAN CLAY (CL)
medium stiff, moist to wet, gray-brown, low plasticity, fine to coarse sand, fine to coarse gravel (sub-angular/rounded)

SILTY SAND (SM)
medium dense, wet, brown, fine to coarse sand, fine-coarse gravel (sub-angular/rounded)

POORLY GRADED SAND (SP)
dense, wet, brown, fine to coarse sand, fine to coarse gravel (sub-angular/rounded)

Continued Next Page

SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
GC, FILL	49	5	117	
	21	14	90	
CL	10	16	105	
	11	12	103	43
SP-SM				
ML	14	37	81	
CL				
CL	14	15	115	
SM	32			7
SP	31			
SM				
ML	14			68 28

GROUND WATER OBSERVATIONS:
▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 20.5 FEET

LA_CORP.GDT 6/13/11 MV*

TRC

EXPLORATORY BORING: EB-2 Cont'd

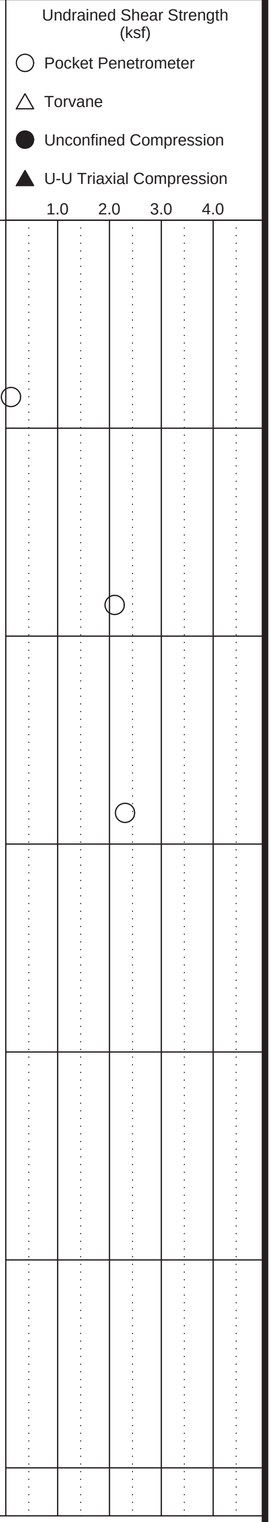
Sheet 2 of 2

DRILL RIG: MOBILE B-40
BORING TYPE: 8-INCH HOLLOW STEM AUGER
LOGGED BY: AC
START DATE: 6-1-11 FINISH DATE: 6-1-11

PROJECT NO:
PROJECT:
LOCATION: SAN JOSE, CA
COMPLETION DEPTH: 45.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

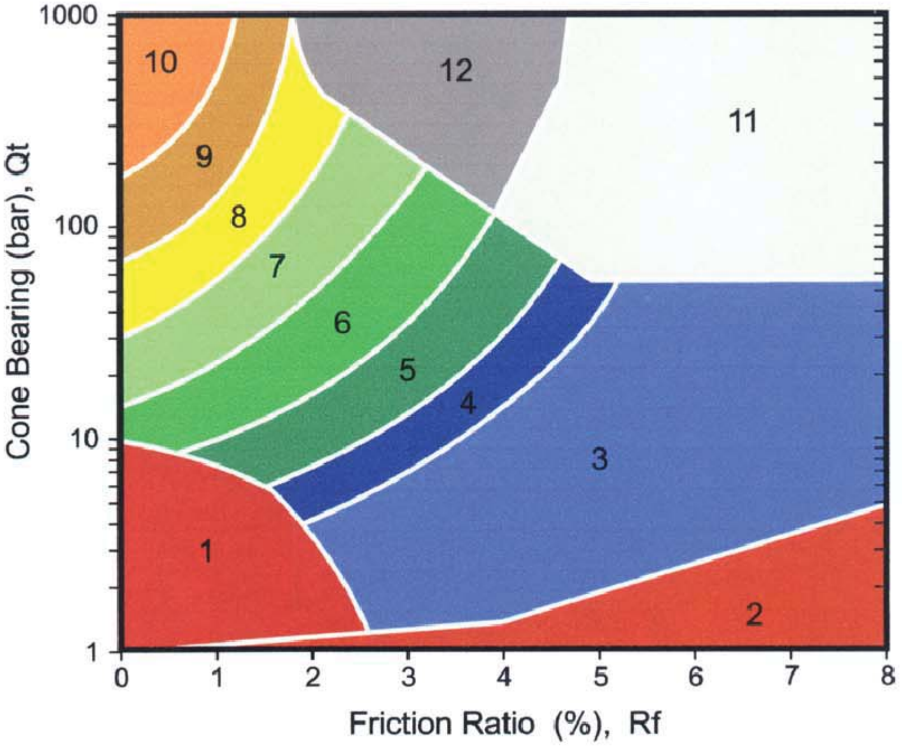
ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
	30	**SILTY SAND (SM)** medium dense, wet, brown-gray, fine to medium sand, trace gravel	SM SM	16			
		SANDY SILT (ML) medium stiff, moist, brown, low to moderate plasticity, fine sand					
		SILTY SAND (SM) medium dense, moist, gray, fine sand		12	37	85	
	35	**FAT CLAY (CH)** soft, moist, greenish-dark gray, high plasticity	CH				
		very stiff		18	40	82	
	40						
		SILT (ML) very stiff, moist, brown, low plasticity	ML				
		POORLY GRADED SAND (SP) dense, moist, brown, fine to coarse sand, fine-coarse gravel (sub-angular/rounded)	SP	51			
	45	Bottom of boring at 45 feet					
	50						
	55						
	60						



GROUND WATER OBSERVATIONS:
⊻ : FREE GROUND WATER MEASURED DURING DRILLING AT 20.5 FEET

LA_CORP.GDT 6/13/11 MV*

TRC



Zone		Q$_t$ / N	Soil Behaviour 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

Robertson (1990)

KEY TO CONE PENETROMETER TEST

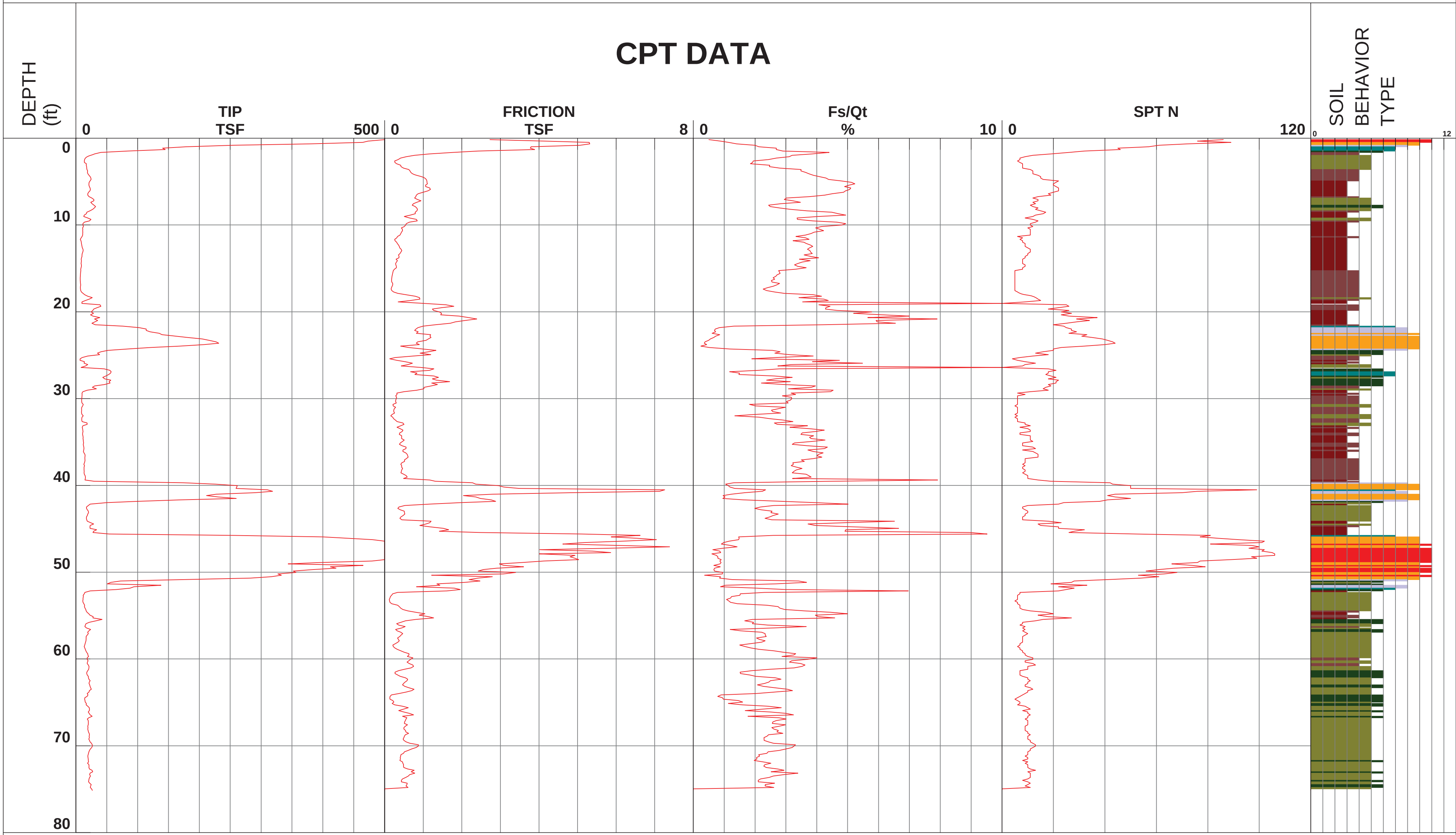
TRC Solutions

Project
Job Number 33512
Hole Number CPT-01
Water Table Depth 14.00 ft

Operator BH-JB
Cone Number DSG1023
Date and Time 5/26/2011 8:24:11 AM

Filename SDF(313).cpt
GPS
Maximum Depth 75.13 ft

Net Area Ratio .8



⬜ 1 - sensitive fine grained
⬛ 2 - organic material
🟥 3 - clay

🟫 4 - silty clay to clay
🟩 5 - clayey silt to silty clay
🟩 6 - sandy silt to clayey silt

🟦 7 - silty sand to sandy silt
🟪 8 - sand to silty sand
🟧 9 - sand

🟥 10 - gravelly sand to sand
🟪 11 - very stiff fine grained (*)
🟦 12 - sand to clayey sand (*)

Cone Size 10cm squared

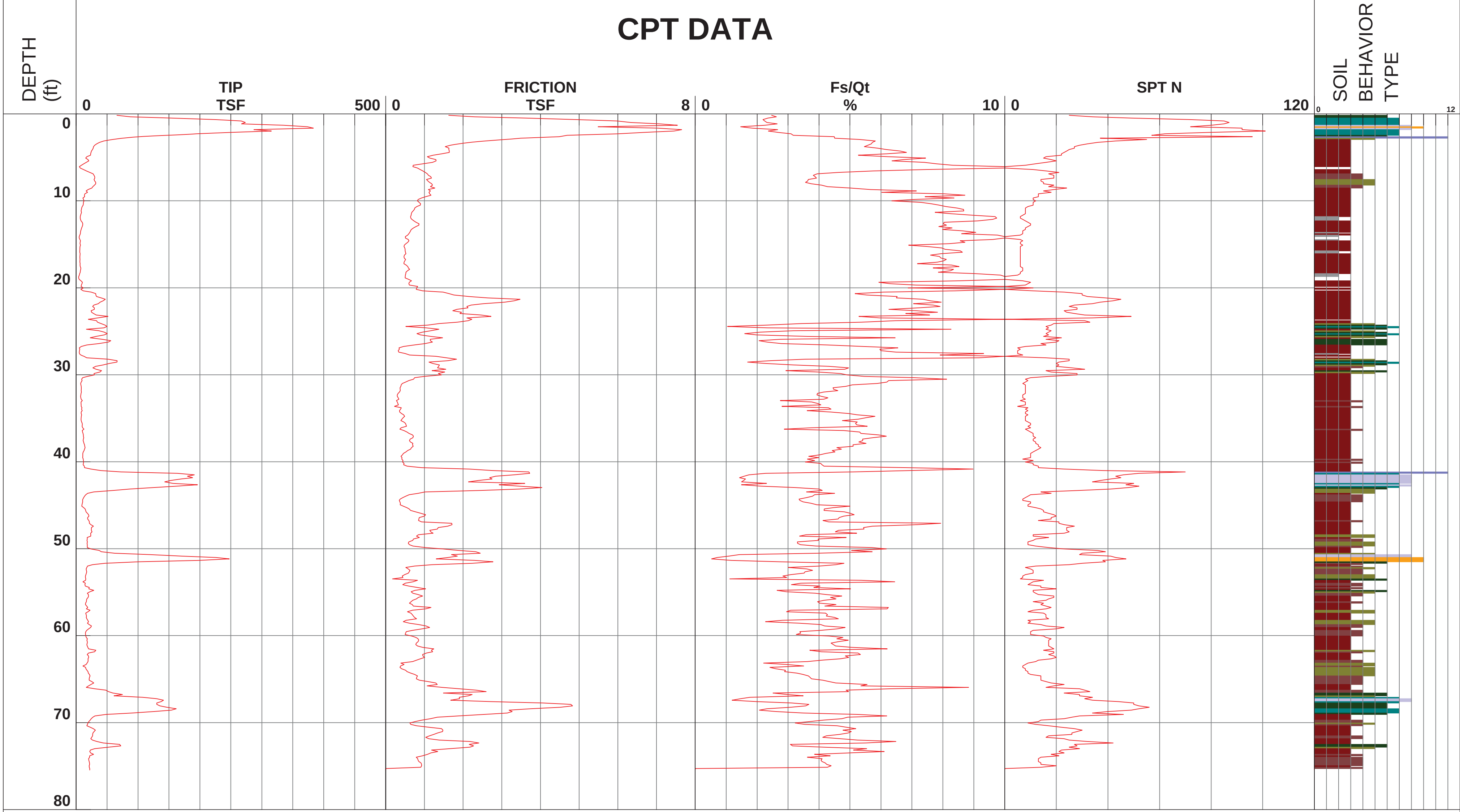
*Soil behavior type and SPT based on data from UBC-1983

TRC Solutions

Project		Operator	BH-JB	Filename	SDF(314).cpt
Job Number	33512	Cone Number	DSG1023	GPS	
Hole Number	CPT-02	Date and Time	5/26/2011 10:10:05 AM	Maximum Depth	75.46 ft
Water Table Depth	14.00 ft				

Net Area Ratio .8

CPT DATA



■ 1 - sensitive fine grained

■ 2 - organic material

■ 3 - clay

■ 4 - silty clay to clay

■ 5 - clayey silt to silty clay

■ 6 - sandy silt to clayey silt

■ 7 - silty sand to sandy silt

■ 8 - sand to silty sand

■ 9 - sand

■ 10 - gravelly sand to sand

■ 11 - very stiff fine grained (*)

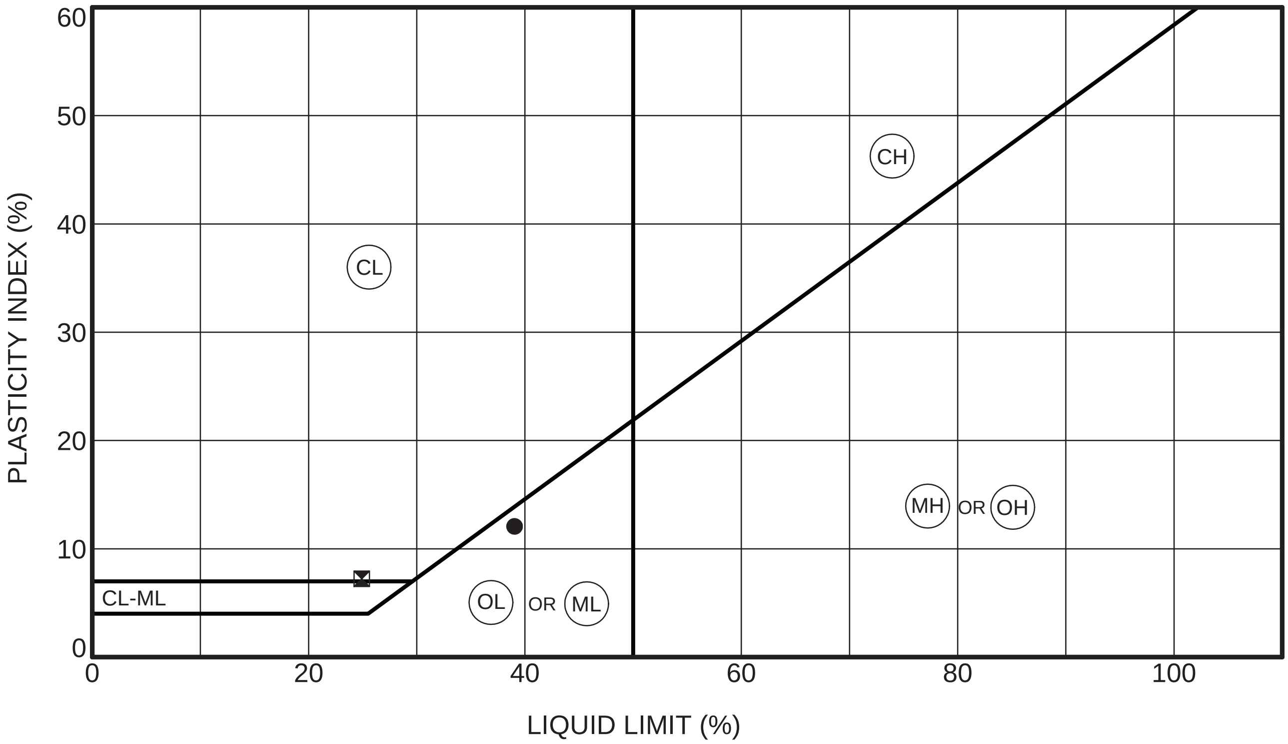
■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

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

APPENDIX B

LABORATORY TEST DATA FROM PREVIOUS INVESTIGATION



Symbol	Boring No.	Depth (ft.)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Passing No. 200 Sieve	Unified Soil Classification Description
●	**EB-1**	**14.5**	**38**	**39**	**27**	**12**		**SILT**
☒	**EB-2**	**5.8**	**16**	**25**	**18**	**7**		**LEAN CLAY WITH SILT**

LA CORP.GDT 6/12/11 MV*

PLASTICITY CHART AND DATA

Project:

Location: SAN JOSE, CA

Project No.:

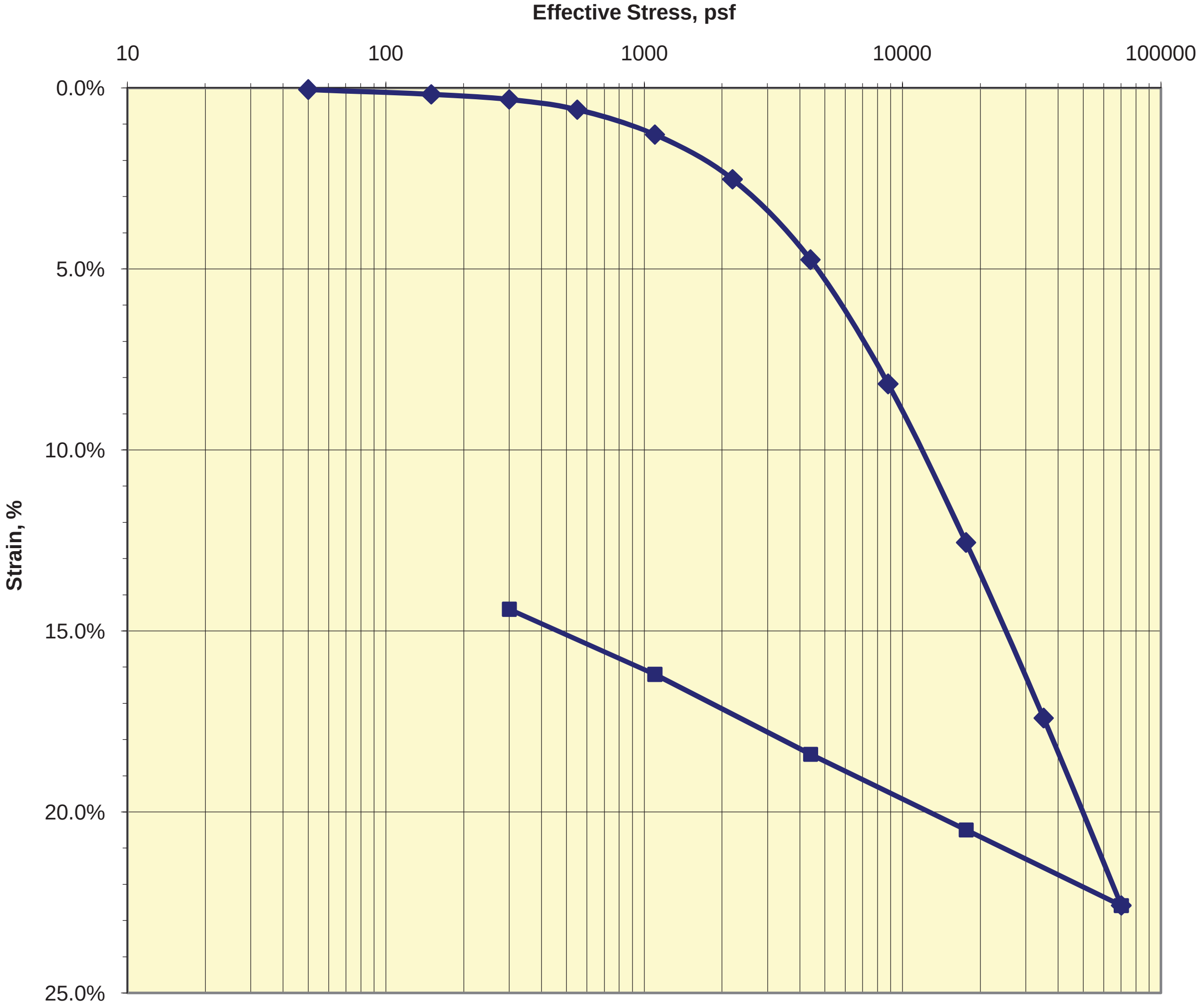
FIGURE B-1

Consolidation Test

ASTM D2435

Job No.:	028-2248	**Boring:**	EB-2	**Run By:**	MD
Client:	TRC	**Sample:**	6A	**Reduced:**	PJ
Project:		**Depth, ft.:**	19.0(Tip)	**Checked:**	PJ/DC
Soil Type:	Dark Gray Sandy CLAY, trace Gravel			**Date:**	6/21/2011

Strain-Log-P Curve



Ass. Gs = 2.7	**Initial**	**Final**
Moisture %:	31.8	26.4
Dry Density, pcf:	89.1	98.5
Void Ratio:	0.891	0.711
% Saturation:	96.3	100

Remarks: There is no data for the final rebound point at 50 psf because the crossbar lost contact with the specimen. The theoretical final height of the specimen was 0.905" based on the total volume of solids plus the final volume of water, assuming 100% saturation.

COOPER TESTING LABORATORY

R-value Test Report (Caltrans 301)

Job No.: 028-2248

Date: 06/08/11

Client: TRC

Tested MD

Project:

Reduced RU

Sample EB-1;Bulk @ Top 5'

Checked DC

Soil Type: Very Dark Brown Clayey SAND, trace Gravel

Initial Moisture, 10.8%

R-value by Stabilometer 28

Expansion Pressure 70 psf

Specimen Number	A	B	C	D
Exudation Pressure, psi	199	799	330	
Prepared Weight, grams	1200	1200	1200	
Final Water Added, grams/cc	38	9	21	
Weight of Soil & Mold, grams	3180	3169	3216	
Weight of Mold, grams	2066	2086	2105	
Height After Compaction, in.	2.55	2.41	2.53	
Moisture Content, %	14.3	11.7	12.8	
Dry Density, pcf	115.7	121.9	117.9	
Expansion Pressure, psf	8.6	232.2	86.0	
Stabilometer @ 1000				
Stabilometer @ 2000	116	41	97	
Turns Displacement	4.5	3.75	3.7	
R-value	18	64	31	

Remarks: