APPENDIX A

GEOTECHNICAL INVESTIGATION (TRC)

TRC

Geotechnical Investigation

North San Pedro Apartments

San Jose, California

Report No. 184153 has been prepared for:

First Community Housing

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GEOTECHNICAL INVESTIGATION
NORTH SAN PEDRO APARTMENTS
SAN JOSE, CALIFORNIA

1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the North San Pedro Apartments development located 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 geotechnical recommendations for design of the proposed project.

For our review, we received:

- Architectural plan titled, "N San Pedro Apartments, A Development for First Community Housing, Bassett St, San Jose, CA, Site Plan, Sheet 2" prepared by The Office of Jerome King Architecture and Planning, dated November 14, 2010.

- Civil plan titled, "Site Development Permit, N San Pedro Apartments, A Development for First Community Housing, Bassett St, San Jose, CA, Grading and Drainage Plan, Sheet 3.1" prepared by The Office of Jerome King Architecture and Planning, dated November 15, 2010.

- Structural plan titled, "N San Pedro Apartments, A Development for First Community Housing, Bassett St, San Jose, CA, Framing Section and Typical Details, Sheet S4" prepared by The Office of Jerome King Architecture and Planning, dated October 22, 2010.

- Architectural plan titled, "N San Pedro Apartments, A Development for First Community Housing, Bassett St, San Jose, CA, Exterior Elevations, Sheet 4.0," prepared by The Office of Jerome King Architecture and Planning, dated May 25, 2010.

1.1 Project Description

The 0.73-acre site is located at 201 Bassett Street in San Jose and is currently vacant. The site is bordered by Bassett Street to the southeast, Terraine Street to the northeast, Union Pacific Railroad tracks to the northwest and Highway 87 (elevated) to the southwest. As presently planned, the site is to be developed with a five-story, steel-framed structure on a podium over an at-grade parking garage. Additional improvements will include pavements, underground utilities, and landscaping.

Based on the plans provided, only minor grading will be required. Preliminary loads given by the structural engineer indicate column loads between 200 and 400 kips. The layout of the proposed development is shown on the Site Plan, Figure 2.

1.2 Scope of Services

Our scope of services was presented in our agreement with you dated April 26, 2011. To accomplish this work, we have provided the following services:

- Exploration of subsurface conditions by drilling two borings in the area of the proposed development and retrieving soil samples for observation and laboratory testing. We also advanced two Cone Penetration Tests (CPT), including one with seismic shear wave velocity testing.

- Evaluation of the physical and engineering properties of the subsurface soils by visually classifying the samples and performing various laboratory tests on selected samples.
- Engineering analysis to evaluate building foundations, site earthwork, slabs-on-grade, and pavements.
- Preparation of this report to summarize our findings and to present our conclusions and recommendations.

2.0 SITE CONDITIONS

2.1 Site Reconnaissance

Our Staff Engineer performed a reconnaissance of the site on June 1, 2011. At the time of the reconnaissance, the site consisted of a vacant lot including trees, shrubs, and asphalt-paved areas. The site is relatively flat, except for the east corner which is approximately 3 to 4 feet above the existing adjacent grade.

2.2 Exploration Program

Subsurface exploration was performed on May 26, 2011 and June 1, 2011 using conventional, truck-mounted drilling equipment and CPT equipment to investigate, sample, and log subsurface soils. Two hollow-stem auger exploratory boring were drilled to depths of 30 and 45 feet. Two CPTs were advanced on May 26, 2011 to depths of 75 feet. Shear wave velocity measurements were taken in one CPT at an interval of 5 feet. Our borings and CPTs were permitted and backfilled in accordance with the Santa Clara Valley Water District guidelines. The approximate locations of the borings and CPTs are shown on the Site Plan, Figure 2. A representative bulk sample of the surface soil was obtained for pavement design purposes. The logs of these borings and details regarding our field investigation are included in Appendix A; laboratory tests are discussed in Appendix B.

2.3 Subsurface Conditions

Soils encountered in CPT-1 and CPT-2 were generally interpreted to include interbedded layers of silty clay, clayey silt, and clay, with a few layers consisting of silty sand, sand, and gravelly sand. The CPT interpretations were generraly consistent with our experience in the area, and the conditions encountered in the borings. Undrained shear strengths were interpreted to be between approximately 200 psf to more than 3,200 psf, indicating soft to very stiff silty and clayey soils. Granular layers encountered were interpreted as loose to dense. Fine grain soil content in the granular soils ranged from 7 to 45 percent of the samples tested. Shear wave velocity measurements ranged between 500 and 1,200 feet per second in the upper 75 feet.

Boring EB-1 encountered about 3½ feet of fill consisting of 2½ feet of medium dense clayey gravels over one foot of asphalt. Boring EB-2 encountered about 4½ feet of fill consisting medium dense clayey gravels. Below the fill, our borings generally encountered medium stiff to very stiff lean clay to silty clay, soft to stiff silt, and medium stiff fat clay to depths of approximately 20 feet. An interbedded layer of loose silty sand was encountered in EB-2 from approximately 9½ to 11½ feet. Below the depth of 20 feet, loose to medium dense silty sand and medium dense to dense poorly graded sand was encountered to depths of approximately 30 feet. In general, below the depth of 30 feet, soft to very stiff fat clay was encountered to a depth of approximately 40 feet. Between 40 and 50 feet thicker layers of sands with interbedded layers of clays were encountered

and below 50 feet finer grained clays and silts were generally encountered to 75 feet, the maximum depth explored.

Two Plasticity Index (PI) tests were performed on a silt and a lean clay sample at depths of approximately 15 and 5½ feet, for borings EB-1 and EB-2, respectively. The tests resulted in PI's of 12 and 7 for borings EB-1 and EB-2, respectively, indicating low plasticity and expansion potential of the soil below the fill.

2.4 Ground Water

Free ground water was encountered during subsurface exploration in both exploratory borings at depths ranging from approximately 17 to 20½ feet. Based on pore pressure dissipation measurements, our CPTs encountered groundwater at a depth of 14 feet below grade. According to the depth to historically high ground water map, prepared by the California Geological Survey for the San Jose West Quadrangle (CGS, 2002), ground water levels in the site vicinity are reportedly on the order of 10 feet below ground surface (bgs). Based on the above information, we judge a design ground water depth of 10 feet below the ground surface to be appropriate for geotechnical design purposes. All 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. A Regional Fault Map illustrating known active faults relative to the site is presented in Figure 3. The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone (known formerly as a Special Studies Zone). The site is also not located within a Santa Clara County Earthquake Hazard Zone (2002) for fault rupture 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

We performed a ground motion hazard analysis in general accordance with Chapter 21 of the ASCE document 7-05 titled "Minimum Design Loads for Buildings and Other Structures." Because we are using Next Generation Attenuation relationships, we are interpreting the application of ASCE 7-05 in accordance with the California Office of Statewide Health Planning and Development (OSHPD) Code Application Notice (CAN) for 2-1802A.6.2 and DSA Bullitin 09-01, which went into effect March 1, 2009. In general the CAN requires the use of the 84th percentile of the deterministic Maximum Credible Earthquake (MCE) response spectra instead of 150 percent of the median deterministic response. Additionally, the CAN requires that the maximum directional spectral acceleration shaking be used, rather than the geometric mean shaking for both the probabilistic and deterministic analyses.

NGA response spectra calculations were performed using the computer software EZ-FRISK version 7.60 published by Risk Engineering, Inc. The recommended site response was developed by performing ground motion hazard analysis using the mean response of the four New Generation

Attenuation (NGA) relationships published by Abrahamson and Silva (2008), Boore and Atkinson (2008), Campbell and Bozorgnia (2008), and Chiou and Youngs (2007). For both deterministic and probabilistic response spectra the geometric mean results have been rotated to represent the estimated maximum directional shaking using modification factors presented by Watson-Lamprey and Boore (2007). Details regarding our seismicity model, site soil profile, NGA calculation parameters, and rotation modification factors are discussed below.

Seismicity Model

2008 USGS fault models were used to evaluate both the probabilistic and deterministic seismic hazards.

Soil Profile

The site shear-wave velocity profile was estimated based on shear wave velocities measured from CPT-1 using Table 1613.5.5 from the 2010 CBC. The site subsurface profile is judged to be consistent with Site Class D classification. Based on published geologic maps and our local experience, we estimate that the depth to bedrock with a shear wave velocity of 1,000 meters per second is approximately 150 meters below the ground surface.

Calculation Parameters

For the NGA analysis, input parameters V_s30, Depth to V_s of 1,000 meters per second, and Z25, were taken as about 228 meters per second, 150 meters, and 1.07 kilometers, respectively. For both probabilistic and deterministic analysis the mean result of the four attenuation relationships was used to determine the recommended MCE response spectrum.

Rotation Factors

As required by OSHPD CAN 2-1802A.6.2, both deterministic and probabilistic geometric mean response spectra (Sa_{GMRotI}) calculated using NGA equations have been modified by the factors shown in Table 1.

Because distance and magnitude effects on the rotational factors have been shown to be slight for the earthquake magnitudes, distances and structural periods of interest for the subject project, the factors presented by Watson-Lamprey and Boore (2007, Table 2) were not modified for distance or magnitude effects. Table 1, below, summarizes the factors used to estimate the maximum directional shaking spectral acceleration response $(Sa_{MaxRotI})$, as presented by Watson-Lamprey and Boore (2007).

Table 1. Factors Used to Estimate Maximum Directional Shaking

Period (second)	Rotation Factor $\ln(Sa_{MaxRotI}/Sa_{GMRotI})$
0.00	0.184
0.05	0.176
0.10	0.178
0.20	0.196
0.30	0.212
0.40	0.219
0.50	0.225
0.75	0.231
1.00	0.237
2.00	0.240

As discussed in Chapter 21.2.2 of ASCE 7-05, both a deterministic and probabilistic seismic hazard analysis were performed, with the lower of either the deterministic spectral acceleration or the 2 percent in 50-year exceedance spectral acceleration from the probabilistic analysis used to determine the MCE spectral acceleration for the site for 5 percent structural damping. The site-specific MCE spectral response was then checked so that the spectral response acceleration at any period does not fall below 80 percent of the general mapped spectrum.

Table 2 below summarizes the details of our calculations including the calculated deterministic response spectrum, the lower bound of the deterministic response spectrum, the resulting deterministic response spectrum, the calculated probabilistic response spectrum having a 2 percent chance of exceedance in 50 years, and the recommended site-specific response spectrum.

Table 2. Summary of MCE Response Spectrum Calculations for 5 Percent Structural Damping

Period (sec)	Calculated Deterministic MCE	Deterministic Lower Bound MCE	Resulting Deterministic MCE	Probabilistic MCE	Recommended Site Specific MCE
0.00	0.48	1.50	1.50	0.78	0.78
0.05	0.55	1.50	1.50	0.93	0.93
0.10	0.76	1.50	1.50	1.31	1.31
0.20	1.05	1.50	1.50	1.72	1.50
0.30	1.10	1.50	1.50	1.80	1.50
0.40	1.08	1.50	1.50	1.74	1.50
0.50	1.07	1.50	1.50	1.68	1.50
0.75	0.99	1.20	1.20	1.47	1.20
1.00	0.88	0.90	0.90	1.29	0.90
2.00	0.59	0.45	0.59	0.80	0.59

Table 3, below, presents recommended S_{MS}, S_{M1}, S_{DS} and S_{D1} seismic coefficients resulting from the site-specific ground motion analysis.

Table 3. Recommended Seismic Coefficients

Seismic Design Coefficient	Value
S_{MS}	1.5
S_{M1}	1.18
S_{DS}	1.0
S_{D1}	0.79

A peak ground acceleration of 0.40g, which is equal to $S_{DS}/2.5$ as discussed in Item 20 of the CGS Note 48, can be expected to occur at the site.

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, 2007) estimates there is a 63 percent chance of at least one magnitude 6.7 or greater earthquake occurring in the San Francisco Bay region between 2007 and 2037. 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

3.4.1 General Background

The site is located within an area zoned by the State of California as having potential for seismically induced liquefaction hazards (CGS, 2002 – San Jose West Quadrangle). The site is also located within a Santa Clara County Geologic Hazard Zone (2002) for 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.

3.4.2 Analysis and Results

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 site grade was used for our liquefaction analysis. As discussed in the subsurface description above, several sand and silt layers were encountered below the design ground water depth. These layers were evaluated to assess liquefaction potential and the effects liquefaction may have on the proposed development. No liquefaction analyses were performed on layers above the design ground water depth.

Our 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 Standard Penetration Tests (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. Our SPT blow counts were corrected 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). 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. 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. We evaluated the liquefaction potential of the medium dense sand and silt strata encountered below the design ground water depth using a peak ground acceleration of 0.4g, which is equal to $S_{DS}/2.5$ as discussed in item 20 of the CGS – Note 48.

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 1.0, the soil layer is considered liquefiable during a moderate to large seismic event.

$$FS = CRR/CSR$$

Soils that have corrected SPT blow counts greater than 30 blows per foot or corrected CPT tip resistance greater than 160 tons per square foot (tsf) are considered either too plastic or too dense to liquefy. Such soil layers have been screened out of analysis and are not presented below.

In the past soils that have greater than 35 percent of plastic fines or an I_C greater than 2.6 have been considered too plastic to liquefy; 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 according to criteria presented by Bray et al. (2004). As methods to evaluate settlements of silts during ground shaking have not been clearly established, we have assumed the settlement will be half of the settlement calculated with traditional liquefaction evaluation methods. The settlement values in parenthesis below indicate half of the liquefaction settlement.

In addition we have limited our analysis to the upper 50 feet because the likelihood of an at-grade structure supported on shallow foundations being affected by liquefaction deeper than 50 feet is low. A summary of our SPT and CPT analysis is presented in the table below.

Table 4. Results of Liquefaction Analyses – SPT Method

Boring Number	Depth to Top of Sand/Silt Layer (feet)	Thickness of Layer (feet)	*SPT (N_{160CS})	Factor of Safety	Potential for Liquefaction	Estimated Strain (%)	Estimated Total Settlement (in.)
EB-1	11.0	6.5	18	0.5	Likely	2.3	1.8 (0.9)
	20.5	7.0	15	0.4	Likely	2.6	2.2
	27.5	1.5	21	0.7	Likely	1.5	0.3
						Total =	**3.4**
EB-2	10.0	1.8	15	0.6	Likely	2.8	0.6
	11.7	8.8	21	0.6	Likely	2.1	2.2 (1.1)
	27.5	2.5	19	0.5	Likely	2.4	0.7
	30.0	2.5	27	0.2	Likely	1.6	0.5
						Total =	**2.9**

* SPT blow counts corrected for overburden, sampling methods, and fines content

Table 5. Results of Liquefaction Analyses – CPT Method

CPT Number	Depth to Top of Sand/Silt Layer (feet)	Layer Thickness (feet)	I_c	$*q_{c1N}$ (tsf)	Factor of Safety	Potential for Liquefaction	Estimated Total Settlement (in.)
CPT-1	21.7	1.0	1.7	134.4	0.64	Likely	0.3
	24.0	0.5	2.1	115.2	0.45	Likely	0.1
	26.6	2.0	2.2	107.3	0.39	Likely	0.5
						Total =	**0.9**
CPT-2	24.3	2.1	2.4	100.8	0.36	Likely	0.4
	28.2	0.7	2.2	119.1	0.47	Likely	0.2
						Total =	**0.6**

* CPT tip pressure corrected for overburden and fines content

Our analyses indicate that several sand and silt layers below the design ground water depth may theoretically liquefy resulting in about 3½-inches of total settlement. Volumetric change and settlement were estimated using the Ishihara and Yoshimine (1990) method. The liquefiable silt layer encountered from approximately 11 to 20 feet in our borings would not liquefy according to the liquefaction results for the CPTs; however, we feel the silt layer likely extends across the site. Therefore, we have assumed about an additional one inch of settlement from the silt layer in the CPT liquefaction results and we estimate differential settlement from liquefaction on the order of 1½ inches in 60 feet.

3.4.3 Potential for Ground Rupture/Sand Boils

The methods of analysis used to estimate the 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 not likely behave as 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 soils to prevent ground rupture. However, we do not feel that enough soil could be ejected at the ground surface to cause significant additional settlement, and conclude that the above settlement estimates are reasonable. Please refer to the Foundation section of this report for 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. Based on the corrected SPT blow counts and laboratory testing data, it appears that there is one loose sand layer above the design groundwater depth that may densify during a strong earthquake.

We performed dry sand settlement calculations following the Tokimatsu and Seed method (1987) and using a peak ground acceleration of 0.4g. Our calculations indicated that during a seismic event the settlement from the loose sand layer encountered from approximately 9 to 10 feet in boring EB-2 is 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. Generally, failure in this mode is analytically unpredictable since it is difficult to evaluate where the first tension crack will occur. Guadalupe River is approximately 1,000 feet west of the site. Based on our review, there are no creeks or open bodies of water within an appropriate distance from the site for lateral spreading to occur on the site. For this reason, the probability of lateral spreading occurring at the site during a seismic event is low.

3.7 Landsliding

The site is in a flat area and is not located within an area zoned by the State of California as having potential for seismically-induced landslide hazards (CGS, 2002). For these reasons, we judge landsliding during a seismic event is not a hazard for the site.

4.0 CORROSION EVALUATION

To evaluate the corrosion potential of the subsurface soils at the site, we submitted three samples collected during our subsurface investigation to an analytical laboratory for pH, resistivity, soluble sulfate and chloride content testing. The results of these tests are summarized in Table 6 below.

Table 6. Results of Corrosivity Testing

Sample	Depth (feet)	Chloride (mg/kg)	Sulfate (mg/kg)	pH	Resistivity (ohm-cm)	Estimated Corrosivity Based on Resistivity	Estimated Corrosivity Based on Sulfates
EB-1, 2A	3.5	16	<5	7.2	2,125	Severe	Negligible
EB-1, 3A	5.5	9	<5	7.2	2,234	Severe	Negligible
EB-2, 4A	9.0	3	<5	8.2	4,341	Moderate	Negligible

Notes: 1. mg/kg = milligrams per kilogram.

Many factors can affect the corrosion potential of soil including soil moisture content, resistivity, permeability and pH, as well as chloride and sulfate concentration. In general, soil resistivity, which is a measure of how easily electrical current flows through soils, is the most influential factor. Based on the findings of studies presented in ASTM STP 1013 titled "Effects of Soil Characteristics on Corrosion" (February, 1989), the approximate relationship between soil resistivity and soil corrosiveness was developed as shown in Table 7 below.

Table 7. Relationship Between Soil Resistivity and Soil Corrosivity

Soil Resistivity (ohm-cm)	Classification of Soil Corrosiveness
0 to 900	Very Severely Corrosive
900 to 2,300	Severely Corrosive
2,300 to 5,000	Moderately Corrosive
5,000 to 10,000	Mildly Corrosive
10,000 to >100,000	Very Mildly Corrosive

Chloride and sulfate ion concentrations, and pH appear to play secondary roles in affecting corrosion potential. High chloride levels tend to reduce soil resistivity and break down otherwise protective surface deposits, which can result in corrosion of buried metallic improvements or reinforced concrete structures. Sulfate ions in the soil can lower the soil resistivity and can be highly aggressive to Portland cement concrete by combining chemically with certain constituents of the concrete, principally tricalcium aluminate. This reaction is accompanied by expansion and eventual disruption of the concrete matrix. Potentially high sulfate content could also cause corrosion of the reinforcing steel in concrete. The 1997 Uniform Building Code (UBC) Table No. 19-A-4 provides requirements for concrete exposed to sulfate-containing solutions as summarized in Table 8.

Table 8. Relationship Between Sulfate Concentration and Sulfate Exposure (UBC Table No. 19-A-4)

Water-Soluble Sulfate (SO_4) in soil, ppm	Sulfate Exposure
0 to 1,000	Negligible
1,000 to 2,000	Moderate[1]
2,000 to 20,000	Severe
over 20,000	Very Severe

[1]= seawater

Acidity is an important factor of soil corrosivity. The lower the pH (the more acidic the environment), the higher will the soil corrosivity be with respect to buried metallic structures. As soil pH increases above 7 (the neutral value), the soil is increasingly more alkaline and less corrosive to buried steel structures due to protective surface films which form on steel in high pH environments. A pH between 5 and 8.5 is generally considered relatively passive from a corrosion standpoint.

As shown in Table 6, soil resistivity results ranged from 2,125 to 4,341 ohm-centimeters. Based on these results and the resistivity correlations presented in Table 7, the corrosion potential to buried metallic improvements may be characterized as moderately to severely corrosive. We recommend that a corrosion protection engineer be consulted about appropriate corrosion protection methods for buried metallic materials.

Based on our previous experience and Table 8, (No. 19-A-4 of the UBC), in our opinion, sulfate exposure to Portland cement concrete (PCC) may be considered as negligible for the native subsurface materials sampled.

5.0 CONCLUSIONS AND RECOMMENDATIONS

From a geotechnical engineering viewpoint, the proposed apartments development may be constructed as planned, in our opinion, provided the design and construction are performed in accordance with the recommendations presented in this report.

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

- Strong seismic shaking
- The potential for liquefaction-induced total and differential settlement
- Undocumented fill
- Corrosion potential of the near-surface soils

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.

5.1.1 Strong Seismic Shaking

We recommend that, at a minimum, the proposed residential structure be designed in accordance with the seismic design criteria as discussed in the Maximum Estimated Ground Shaking section above. The site seismic coefficients are presented in Table 3.

5.1.2 Liquefaction-Induced Total and Differential Settlement

Our analyses indicate that several layers theoretically can liquefy, resulting in up to 3½ inches of total settlement. We estimate differential settlements from liquefaction will be on the order of 1½-inches in 60 feet. 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 soils to fully prevent ground rupture. However, the settlement estimates are judged to accurately reflect the potential for ground rupture and the structure can be designed to resist the adverse effects of potential ground rupture. The proposed apartment structure should be designed to accommodate the resulting seismic and static settlements. Detailed recommendations are presented in the Foundation section of this report.

5.1.3 Undocumented Fill

Borings EB-1 and EB-2 encountered approximately 3½ to 4½ feet of fill, respectively, below the ground surface. The fill consists of medium dense clayey gravel and asphalt. Based on the dry densities and blow counts, the fill does 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 recommend that the total depth of undocumented fills be removed and replaced with clean engineered fill. Detailed recommendations are presented in following sections of this report.

5.1.4 Corrosion Potential of the Soils

As discussed above, the corrosion potential to buried metallic improvements constructed within the native clayey soils may be characterized as moderate to severely corrosive. A qualified corrosion engineer should be contacted to provide specific recommendations regarding corrosion protection for buried metal pipe or buried metal pipe-fittings.

5.2 Plans, Specifications, and Construction Review

We recommend that our firm perform a plan review of the geotechnical aspects of the project design for general conformance with our recommendations. In addition, subsurface materials encountered in the relatively small diameter, widely spaced borings may vary significantly from other subsurface materials on the site. Therefore, we also recommend that a representative of our firm observe and confirm the geotechnical specifications of the project construction. This will allow us to form an opinion about the general conformance of the project plans and construction with our recommendations. In addition, our observations during construction will enable us to note subsurface conditions that may vary from the conditions encountered during our investigation and, if needed, provide supplemental recommendations. For the above reasons, our geotechnical recommendations are contingent upon our firm providing geotechnical observation and testing services during construction.

6.0 EARTHWORK

6.1 Clearing and Site Preparation

The proposed project area should be cleared of all surface and subsurface improvements to be removed and deleterious materials including existing irrigation lines, utilities, fills, pavements, debris, designated trees, shrubs, and associated roots. Abandonment of existing buried utilities is discussed below. Excavations extending below the planned finished site grades should be cleaned and backfilled with suitable material compacted as recommended in the "Compaction" section of this report. We recommend that backfilling of holes or pits results from demolition and removal of existing building foundations, buried structures or other improvements be carried out under our observation and that the backfill be observed and tested during placement.

After clearing, any vegetated areas within the proposed improvements should be stripped to sufficient depth to remove all surface vegetation and topsoil containing greater than 3 percent organic matter by weight. The actual stripping depth required depends on site usage prior to construction and should be established in the field by us at the time of construction. The stripped materials should be removed from the site or may be stockpiled for use in landscaped areas, if desired.

6.2 Removal of Existing Fill

As mentioned above, fill was encountered in borings EB-1 and EB-2 to a depth of about 3½ to 4½feet. We recommend the total depth of fill be removed and replaced and engineered fill.

If some of the existing fill material is free of asphalt material and meets the requirements in the "Material for Fill" section below, it may be reused as engineered fill. All fill should be compacted in accordance with the recommendations for fill presented in the "Compaction" section of this report.

6.3 Abandoned Utilities

Abandoned utilities within the proposed building areas should be removed in their entirety. Utilities within the proposed building area would only be considered for in-place abandonment provided they do not conflict with new improvements, that the ends and all laterals are located and completely grouted, and the previous fills associated with the utility do not pose a risk to the structure.

Utilities outside the building areas should be removed or abandoned in-place by grouting or plugging the ends with concrete. Fills associated with utilities abandoned in-place could pose

some risk of settlement; utilities that are plugged could also pose some risk of future collapse or erosion should they leak or become damaged.

6.4 Subgrade Preparation

After the site has been properly cleared, stripped and necessary excavations have been made, exposed surface soils in those areas to receive fill, slabs on grade, footings, or pavements should be scarified to a depth of 6 inches, moisture conditioned, and compacted in accordance with the recommendations for fill presented in the "Compaction" section. The finished compacted subgrade should be firm and relatively non-yielding under the weight of compaction equipment.

6.5 Material for Fill

All on-site soils below the stripped layer having an organic content of less than 3 percent by weight are suitable for use as fill at the site. In general, fill material should not contain rocks or lumps larger than 6 inches in greatest dimension, with 15 percent or less larger than 2½ inches in the greatest dimension. Excavated existing fill that does not contain asphalt material may be used as new fill provided it meets the above requirements for new fill.

Import fill material should be inorganic, have a PI of 15 or less and should have sufficient binder to reduce the potential for sidewall caving of foundation and utility trenches. Samples of the proposed import fill should be submitted to us at least 10 working days prior to delivery to the site to allow for visual review and laboratory testing. This will allow us to evaluate the general conformance of the import fill with our recommendations.

Consideration should also be given to the environmental characteristics and corrosion potential of any imported fill. Suitable documentation should be provided for import material. In addition, it may be appropriate to perform laboratory testing of the environmental characteristics and corrosion potential of imported materials. Import soils should not be more corrosive than the on-site native materials, including pH, soluble sulfates, chlorides and resistivity.

6.6 Compaction

All fill, as well as scarified surface soils in those areas to receive fill, should be uniformly compacted to at least 90 percent relative compaction as determined by ASTM Test Designation D1557, latest edition. Fill should be placed in lifts no greater than 8 inches in uncompacted thickness. Each successive lift should be firm and relatively non-yielding under the weight of construction equipment.

In pavement areas, the upper 6 inches of subgrade and full depth of aggregate base should be compacted to at least 95 percent relative compaction (ASTM D1557, latest edition). Aggregate base and all import soils should be compacted at a moisture content near the laboratory optimum moisture content.

6.7 Wet Soils and Wet Weather Conditions

Earthwork such as subgrade preparation, fill placement and trench backfill may be difficult for soil containing high moisture content or during wet weather. If the soil is significantly above its optimum moisture content, it will become soft, yielding, and difficult to compact. Based on the results of our laboratory tests, the in-situ moisture contents of the near surface soils are generally near to above optimum moisture contents. If saturated soils are encountered, aerating or blending with drier soils to achieve a workable moisture content may be required. We recommend that

earthwork be performed during periods of suitable weather conditions, such as the "summer" construction season.

There are several alternatives to facilitate subgrade preparation, fill placement and trench backfill if the soil is wet or earthwork is performed during the wet winter season.

- Scarify and air dry until the fill materials have a suitable moisture content for compaction,
- Over-excavate the fill and replace with suitable on-site or import materials with an appropriate moisture content,
- Install a layer of geo-synthetic (geotextile or geogrid) to reduce surface yielding and bridge over soft fill,
- Chemically treat the higher moisture content soils with quicklime (CaO), kiln-dust, or cement to reduce the moisture content and increase the strength of the fill.

The implementation of these methods should be reviewed on a case-by-case basis so that a cost effective approach may be used for the specific conditions at the time of construction.

6.8 Trench Backfill

Bedding and pipe embedment materials to be used around underground utility pipes should be well graded sand or gravel conforming to the pipe manufacturer's recommendations and should be placed and compacted in accordance with project specifications, local requirements or governing jurisdiction. General fill to be used above pipe embedment materials should be placed and compacted in accordance with local requirements or the recommendations contained in this section, whichever is more stringent.

On-site soils may be used as general fill above pipe embedment materials provided they meet the requirements of the "Material for Fill" section of this report. General fill should be placed in lifts not exceeding 8 inches in uncompacted thickness and should be compacted to at least 90 percent relative compaction (ASTM D1557, latest edition) by mechanical means only. Water jetting of trench backfill should not be allowed. The upper 6 inches of general fill in all pavement areas subject to wheel loads should be compacted to at least 95 percent relative compaction.

Utility trenches located adjacent to footings should not extend below an imaginary 1:1 (horizontal:vertical) plane projected downward from the footing bearing surface to the bottom edge of the trench. Where utility trenches will cross beneath footing bearing planes, the footing concrete should be deepened to encase the pipe or the utility trench should be backfilled with sand/cement slurry or lean concrete within the foundation-bearing plane.

Where relatively higher permeability sand or gravel backfill is used in trenches through lower permeability soils, we recommend that a cut-off plug of compacted clayey soil or a 2-sack cement/sand slurry be placed where such trenches enter the building and pavement areas. This would reduce the likelihood of water entering the trenches from the landscaped areas and seeping through the trench backfill into the building and pavement areas, and coming into contact with expansive subgrade soils.

6.9 Temporary Excavations

The contractor should be responsible for all temporary slopes and trenches excavated at the site and design of any required temporary shoring. Shoring, bracing, and benching should be performed by the contractor in accordance with the strictest governing safety standards.

6.10 Surface Drainage

Positive surface water drainage gradients, at least 2 percent in landscaping and 0.5 percent in pavement areas, should be provided to direct surface water away from foundations and slabs towards suitable discharge facilities. Ponding of surface water should not be allowed on or adjacent to structures, slabs-on-grade, or pavements. Roof runoff should be directed away from foundation and slabs-on-grade. Downspouts may discharge onto splash-blocks provided the area is covered with concrete slabs or asphalt concrete pavements.

6.11 Landscaping Considerations

We recommend restricting the amount of surface water infiltrating these soils near structures and slabs-on-grade. This may be accomplished by:

- Selecting landscaping that requires little or no watering, especially within 3 feet of structures, slabs-on-grade, or pavements,
- Using low flow rate sprinkler heads, or drip irrigation systems
- Regulating the amount of water distributed to lawn or planter areas by installing timers on the sprinkler system,
- Providing surface grades to drain rainfall or landscape watering to appropriate collection systems and away from structures, slabs-on-grade, or pavements,
- Preventing water from draining toward or ponding near building foundations, slabs-on-grade, or pavements, and
- Avoiding open planting areas within 3 feet of the building perimeter.

We recommend that the landscape architect consider these items when developing the landscaping plans.

6.12 Construction Observation

A representative from our company should observe the geotechnical aspects of the grading and earthwork for general conformance with our recommendations including site preparation, selection of fill materials, and the placement and compaction of fill. To facilitate your construction schedule we request sufficient notification (48 hours) for site visits. The project plans and specifications should incorporate all recommendations contained in the text of this report.

7.0 FOUNDATIONS

As discussed in the Conclusions and Recommendations section there is a potential for liquefaction settlement and ground rupture; therefore conventional footings are not recommended. Provided that the site is prepared in accordance with the "Earthwork" section of this report and the proposed apartment structure can be designed to accommodate the following estimated amounts of settlement, the structures may be supported on footings with connecting grade beams or a reinforced mat foundation as discussed in the sections below.

7.1 2010 CBC Site Coefficients and Site Seismic Coefficients

Chapter 16 of the 2010 California Building Code (CBC) outlines the procedure for seismic design of structures. Based on our explorations, and review of available geologic maps (Rogers and

Williams, 1974), the site is underlain by medium stiff to hard soils extending to depths on the order of up to greater than 500 feet, 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 9 below.

Table 9. 2010 CBC Site Class and Site Seismic Coefficients

Latitude: 37.3400 N **Longitude: 121.8985 W**	**CBC Table/ Figure**	**Factor/ Coefficient**	**Value**
Soil Profile Type	Table 1613.5.2	Site Class	D
Mapped Spectral Response Acceleration for MCE at 0.2 second Period	Figure 1613.5(3)	S_s	1.5
Mapped Spectral Response Acceleration for MCE at 1 Second Period	Figure 1613.5(4)	S_1	0.6
Site Coefficient	Table 1613.5.3(1)	F_a	1.0
Site Coefficient	Table 1613.5.3(2)	F_v	1.5
Adjusted MCE Spectral Response Parameter	Equation 16A-37	S_{MS}	1.5
Adjusted MCE Spectral Response Parameter	Equation 16A-38	S_{M1}	0.9
Design Spectral Response Acceleration Parameter	Equation 16A-39	S_{DS}	1.0
Design Spectral Response Acceleration Parameter	Equation 16A-40	S_{D1}	0.6

The mapped values are shown for reference only. We performed a site-specific ground motion hazard analysis for the site, which resulted in slightly higher seismic coefficients. We recommend that the proposed residential structure be designed in accordance with the seismic design criteria as discussed in the Maximum Estimated Ground Shaking section above. The site seismic coefficients are presented in Table 3.

7.2 Footings with Grade Beams

The proposed apartment structure 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 high liquefaction 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.

Footings constructed on native soil or engineered fill 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.

These maximum allowable bearing pressures are net values; the weight of the footing may be neglected for design purposes. All footings located adjacent to utility trenches should have their

bearing surfaces below an imaginary 1:1 (horizontal:vertical) plane projected upward from the bottom edge of the trench to the footing.

All continuous footings should be reinforced with top and bottom steel to provide structural continuity and to help span local irregularities. We should observe all footing excavations before reinforcing steel is placed. All footings should be connected to all adjacent footings with grade beams. Vertical and horizontal bearing of the grade beams should be the same as for the footings.

7.3 Foundation Settlement

Preliminary loads given by the structural engineer indicate column loads between 200 and 400 kips. Based on the assumed load and the maximum allowable bearing pressures recommended above, we estimate that total static settlement for footings will be approximately 1-inch, with differential settlements of ½-inch over a horizontal distance of 50 feet. We estimate differential settlements from liquefaction will be on the order of 1½-inch in 60 feet. We should be retained to review the final foundation plans and structural loads to verify the above settlement estimates.

7.4 Reinforced Mat Foundations

The proposed structure may alternatively be supported on a conventionally reinforced mat foundation. 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.

7.5 Mat Foundation Settlement

Based on the estimated areal mat pressure, we estimate that total settlement will be on the order of 1¼-inch for mats bearing at 2 feet below grade. If the mat foundation, designed in accordance with the above recommendations, is not capable of resisting up to 1¼-inch and ½-inch of differential movement from the center to the corner of the interior mat, respectively, additional reinforcement may be required. If deeper foundations are desired, we should be retained to further evaluate the foundations.

As discussed in the "Liquefaction" section, differential settlement of mat foundations due to liquefaction may occur during strong ground shaking. To reduce the potential impact of liquefaction-induced settlement, the mats should also be designed to tolerate 1½ inches of differential settlement over a horizontal distance of 60 feet.

7.6 Lateral Loads

Lateral loads may be resisted by friction between the bottom of footings or mats and the supporting subgrade. A maximum allowable frictional resistance of 0.3 may be used for design. In addition, lateral resistance may be provided by passive pressures acting against footings or deepened mat edges poured neat against competent soil. We recommend that an allowable passive pressure based on an equivalent fluid pressure of 300 pounds per cubic foot (pcf) be used in design. The upper 12 inches of soil should be neglected when determining lateral passive resistance.

8.0 CONCRETE SLABS-ON-GRADE

8.1 Garage Slabs (with Footings)

The at-grade parking garage slab should be at least 5 inches thick, have a compressive strength of at least 3,000 pounds per square inch (psi), and supported on at least 6 inches of Class 2 aggregate base compacted to at least 95 percent relative compaction. Adequate slab reinforcement should be provided to satisfy the anticipated use and loading requirements.

As an alternative, the garage could be asphalt concrete pavement and recommendations are presented in the Pavements section below.

8.2 Slabs-on-Grade for Lobby

We recommend that slabs-on-grade be supported on at least 6 inches of non-expansive fill (NEF) to reduce the likelihood of slab damage from heave. Before slab construction, the subgrade surface should be proof-rolled to provide a smooth, firm surface for slab support. Slab reinforcing should be provided in accordance with the anticipated use and loading of the slab.

If desired to limit moisture rise through slab-on-grade floors, the guidelines presented in the "Moisture Protection Considerations" section below should be considered.

8.3 Moisture Protection Considerations

Since the long-term performance of concrete slab-on-grade depends to a large degree on good design, workmanship, and materials, the following general guidelines are presented for consideration by the developer, design team, and contractor. The purpose of these guidelines is to aid in producing a concrete slab of sufficient quality to allow successful installation of floor coverings and reduce the potential for floor covering failures due to moisture-related problems associated with the slab-on-grade construction. These guidelines may be supplemented, as necessary, based on the specific project requirements.

- A minimum 15-mil thick vapor barrier meeting minimum ASTM E 1745, Class A requirements should be placed directly below the slab. The vapor barrier should extend to the edge of the mat. At least 4 inches of free-draining gravel, such as ½-inch or ¾-inch crushed rock with no more than 5 percent passing the ASTM No. 200 sieve, should be placed below the vapor barrier to serve as a capillary break (no sand). The crushed rock should be consolidated in place with vibratory equipment. The vapor barrier should be sealed at all seams and penetrations.

- The concrete water/cement ratio should not exceed 0.45. Midrange plasticizers could be used to facilitate concrete placement and workability.

- Water should not be added after initial batching, unless the slump of the concrete is less than specified, and the resulting water/cement ratio will not exceed 0.45.

- Polishing the concrete surface with metal trowels should not be permitted.

- All concrete surfaces to receive any type of floor covering should be moist-cured for a minimum of 7 days. Moist curing methods may include frequent sprinkling, or using coverings such as burlap, cotton mats, or carpet. The covering should be placed as soon

as the concrete surface is firm enough to resist surface damage. The covering should be kept continuously wet and not allowed to dry out during the required curing period.

- Water vapor emission levels and pH should be determined before floor installation as required by the manufacturer of the floor covering materials. Measurements and calculations should be made according to ASTM F1869-98 and F710-98 protocol.

The guidelines presented above are based on information obtained from various technical sources, including the American Concrete Institute (ACI), and are intended to present information that can be used to reduce potential long-term impacts from slab moisture infiltration. It should be noted that the application of these guidelines does not affect the geotechnical aspects of the foundation performance.

9.0 RETAINING WALLS

9.1 Lateral Earth Pressures

Any proposed retaining or sound walls should be designed to resist lateral earth pressures from adjoining natural materials, backfill, and surcharge loads. Provided that adequate drainage is provided as recommended below, we recommend that walls restrained from movement at the top be designed to resist an equivalent fluid pressure of 45 pounds per cubic foot (pcf) plus a uniform pressure of 8H pounds per square foot, where H is the distance in feet between the bottom of the footing and the top of the wall. Restrained walls should also be designed to resist an additional uniform pressure equivalent to one-half of any surcharge loads applied at the surface. Any unrestrained retaining walls with adequate drainage should be designed to resist an equivalent fluid pressure of 45 pcf plus one-third of any surcharge loads.

The above lateral earth pressures assume level backfill conditions and sufficient drainage behind the walls to prevent build-up of hydrostatic pressure from surface water infiltration and/or a rise in the ground water level. If adequate drainage is not provided, we recommend an equivalent fluid pressure of 40 pcf be added to the values recommended above for both restrained and unrestrained walls. Damp-proofing of the walls should be included in areas where wall moisture and efflorescence would be undesirable.

9.2 Drainage

Adequate drainage may be provided by a subdrain system behind the walls. This system should consist of a 4-inch minimum diameter perforated pipe placed near the base of the wall (perforations placed downward). The pipe should be bedded and backfilled with Class 2 Permeable Material per Caltrans Standard Specifications, latest edition. The permeable backfill should extend at least 2 feet out from the wall and to within 2 feet of outside finished grade. Alternatively, ½-inch to ¾-inch crushed rock may be used in place of the Class 2 Permeable Material provided the crushed rock and pipe are enclosed in filter fabric, such as Mirafi 140N or equivalent. The upper 2 feet of wall backfill should consist of relatively low permeable compacted on-site clayey soil. The subdrain outlet should be connected to a free-draining outlet or sump.

Miradrain, Geotech Drainage Panels, or Enkadrain drainage matting may be used for wall drainage as an alternative to the Class 2 Permeable Material or drain rock backfill. The drainage panel should be connected to the perforated pipe at the base of the wall, or to some other closed or through-wall system. Miradrain panels should terminate 18 to 24 inches from final exterior grade. The Miradrain panel filter fabric should be extended over the top of and behind the panel to protect it from intrusion of the adjacent soil.

9.3 Backfill

Backfill placed behind the walls should be compacted to at least 90 percent relative compaction using light compaction equipment. If heavy compaction equipment is used, the walls should be temporarily braced.

9.4 Foundation

Retaining walls may be supported on a continuous spread footing designed in accordance with the recommendations presented in the "Footings" section of this report. Lateral load resistance for the walls may be developed in accordance with the recommendations presented in the "Lateral Loads" section.

10.0 PAVEMENTS

10.1 Asphalt Concrete

We obtained a representative bulk sample of the surface soil from the parking area and performed an R-value test to provide data for pavement design. The results of the test are included in Appendix B and indicate an R-value of 28. We judged an R-value of 15 to be applicable for design. Using estimated traffic indices for various pavement-loading requirements, we developed the following recommended pavement sections based on Procedure 608 of the Caltrans Highway Design Manual, presented in Table 10.

Table 10. Recommended Asphalt Concrete Pavement Design Alternatives
Pavement Components
Design R–Value = 15

General Traffic Condition	Design Traffic Index	Asphalt Concrete (Inches)	Aggregate Baserock* (Inches)	Total Thickness (Inches)
Automobile	4.0	2.5	6.5	9.0
Parking	4.5	2.5	8.0	10.5
Automobile	5.0	3.0	8.0	11.0
Parking Channel	5.5	3.0	10.0	13.0
Truck Access &	6.0	3.5	10.5	14.0
Parking Areas	6.5	4.0	11.5	15.5

*Caltrans Class 2 aggregate base; minimum R-value equal to 78.

The traffic indices used in our pavement design are considered reasonable values for the proposed development and should provide a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance. The traffic parameters used for design were selected based on engineering judgment and not on information furnished to us such as an equivalent wheel load analysis or a traffic study. Because of the presence of moderately expansive clay at the site, some increased amount of maintenance should be expected.

10.2 Exterior Portland Cement Concrete (PCC) Pavements

Recommendations for exterior PCC pavements are presented below in Table 11. Since the expected Average Daily Truck Traffic (ADTT) is not known at this time, we have provided

alternatives for minimum pavement thickness. An allowable ADTT should be chosen that is greater than expected for the development.

Table 11. Recommended Minimum PCC Pavement Thickness

Allowable ADTT	Minimum PCC Pavement Thickness (inches)
0.8	5
13	5½
130	6

Our design is based on an R-value of 15 and a 28-day unconfined compressive strength for concrete of at least 3,500 pounds per square inch. In addition, our design assumes that pavements are restrained laterally by a concrete shoulder or curb and that all PCC pavements are underlain by at least 6 inches of Class 2 aggregate base. We recommend that adequate construction and control joints be used in design of the PCC pavements to control the cracking inherent in this construction.

10.3 Pavement Cutoff

Surface water infiltration beneath pavements could significantly reduce the pavement design life. While the amount of reduction in pavement life is difficult to quantify, in our opinion, the normal design life of 20 years may be reduced to less than 10 years. Therefore, long-term maintenance greater than normal may be required.

To limit the need for additional long-term maintenance, it would be beneficial to protect at-grade pavements from landscape water infiltration by means of a concrete cut-off wall, deepened curbs, redwood header, "Deep-Root Moisture Barrier," or equivalent. However, if reduced pavement life and greater than normal pavement maintenance are acceptable, the cutoff barrier may be eliminated. If desired to install pavement cutoff barriers, they should be considered where pavement areas lay downslope of any landscape areas that are to be sprinkled or irrigated, and should extend to a depth of at least 4 inches below the base rock layer.

10.4 Asphalt Concrete, Aggregate Base and Subgrade

Asphalt concrete and aggregate base should conform to and be placed in accordance with the requirements of Caltrans Standard Specifications, latest edition, except that ASTM Test Designation D1557 should be used to determine the relative compaction of the aggregate base. Pavement subgrade should be prepared and compacted as described in the "Earthwork" section of this report.

10.5 Flatwork and Sidewalks

We recommend that exterior slabs-on-grade, such as flatwork and sidewalks be at least 4 inches thick and be underlain by at least 4 inches of Class 2 aggregate base compacted to a minimum of 90 percent relative compaction in accordance with ASTM Test Method D1557, latest edition. If sidewalks are subject to wheel loads, they should be designed in accordance with the "Exterior Portland Cement Concrete Pavements" section of this report.

11.0 LIMITATIONS

This report has been prepared for the sole use of First Community Housing, specifically for design of the proposed North San Pedro Apartments Development 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 investigation, which includes 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 or between borings 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.

11.0 REFERENCES

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

SOURCE AERIAL PHOTO: Google Earth, May 2011.

N

SCALE (FEET)

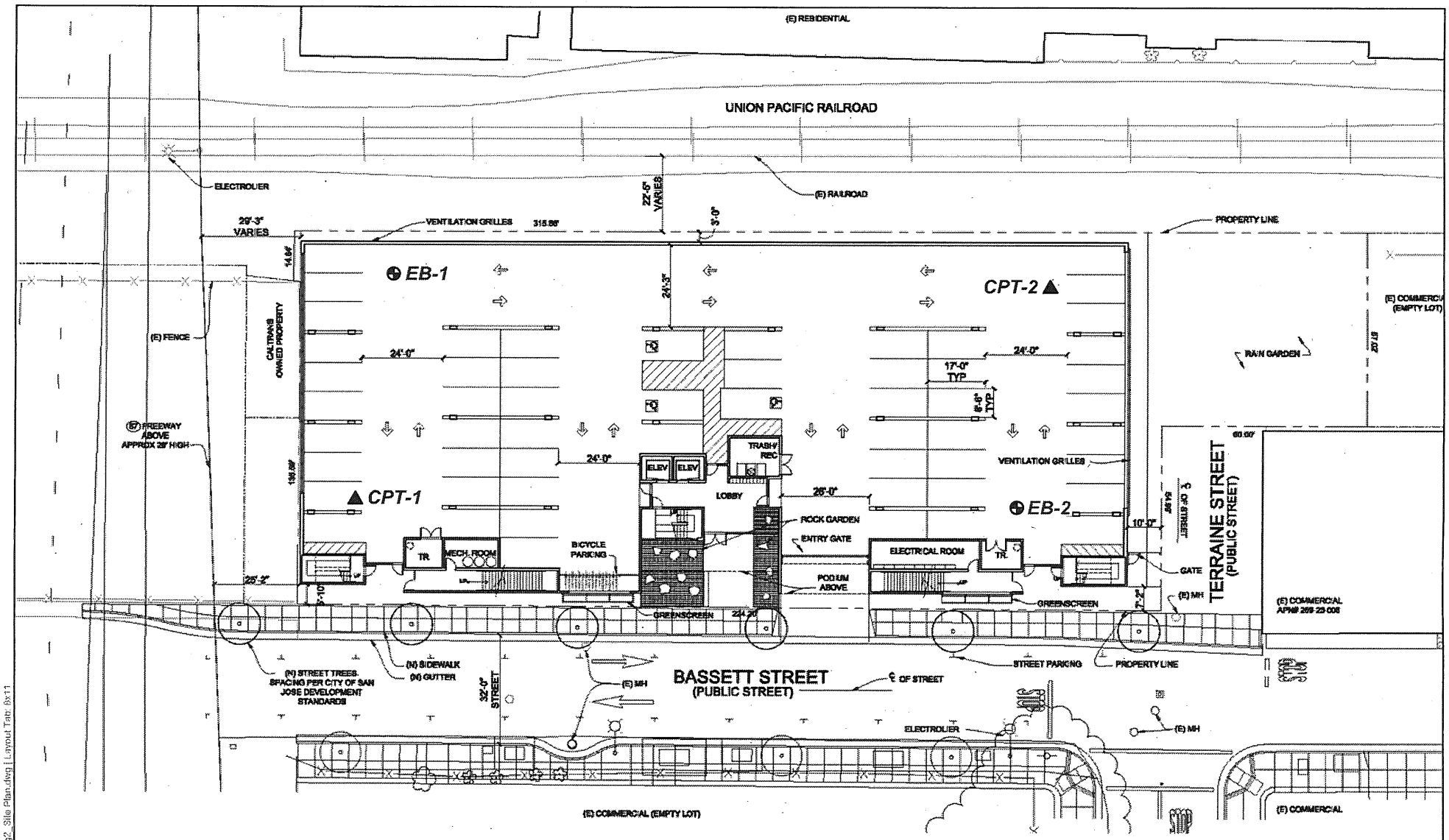
0 500 1,000

VICINITY MAP

North San Pedro Apartments
201 Bassett Street
San Jose, California

TRC | 184153 | **FIGURE 1**

FILE NAME: N:\CAD\North San Pedro Apts_San Jose\Fig1_Vicinity Map.dwg | Layout Tab: 8x11



SOURCE: Site plan by The Office of Jerome King, FAIA, November 2010.

LEGEND

EB-2 ⊕	Approximate location of exploratory boring
CPT-2 ▲	Approximate location of cone penetration test

N

SCALE (FEET)

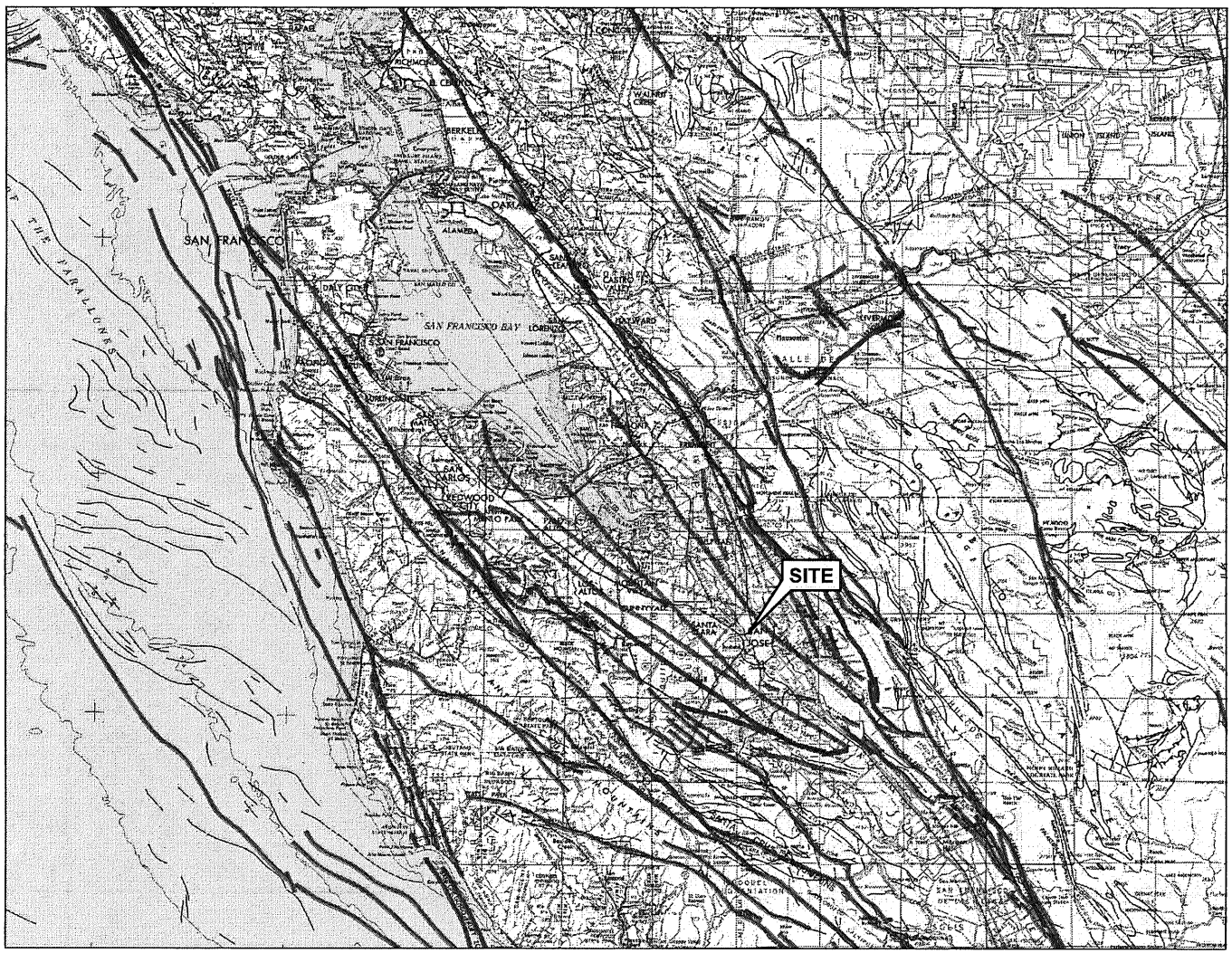
0 40 80

SITE PLAN

North San Pedro Apartments
201 Bassett Street
San Jose, California

TRC | 184153 | **FIGURE 2**

FILE NAME: N:\CAD\North San Pedro Apts_San Jose\Fig2_Site Plan.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	Holocene/Historic				Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.
			200			
			10,000			Displacement during Holocene time[2]
						Faults showing evidence of displacement during late Quaternary time[2,3]
			700,000			
	Early Quaternary	Pleistocene				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.
			2,000,000			
Pre-Quaternary		Pliocene				
			5,000,000			Fault showing evidence of no displacement during Quaternary time or faults without recognized Quaternary displacement.
		Miocene				

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

North San Pedro Apartments
201 Bassett Street
San Jose, California

TRC 184153 **FIGURE 3**

FILE NAME: H:\CAD\North San Pedro Apts_San Jose\Fig3_Regional Fault Map.dwg | Layout Tab: 11x17

APPENDIX A

FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using conventional, truck-mounted, hollow-stem auger drilling equipment and cone penetration test (CPT) equipment. Two 8-inch diameter exploratory borings were drilled on June 1, 2011 to a maximum depth of 45 feet. Two CPTs were advanced on May 26, 2011 to a maximum depth of 75 feet. The approximate locations of the exploratory borings and CPTs are shown on Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). The logs of the borings, as well as a key to the classification of the soil, are included as part of this appendix.

The locations of borings and CPTs were approximately determined by pacing from existing site boundaries. Elevations of the borings were not determined. The locations of the borings should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. Penetration resistance blow counts were obtained by dropping a 140-pound hammer 30 inches. Modified California 3.0-inch outside diameter (O.D.) samples and Standard Penetration Test (SPT) 2-inch O.D. samples were obtained by driving the samplers 18 inches and recording the number of hammer blows for each 6 inches of penetration. Unless otherwise indicated, the blows per foot recorded on the boring logs represent the accumulated number of blows required to drive the samplers the last two 6-inch increments. When using the SPT sampler, the sum of the last two 6-inch increments is the uncorrected SPT measured blow count. Relatively undisturbed samples were also obtained with 2.875-inch inside diameter Shelby Tube sampler, which were hydraulically pushed. The various samplers are denoted at the appropriate depth on the boring logs and symbolized as shown on Figure A-1.

The attached boring and related information depict subsurface conditions at the locations indicated and on the date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these boring locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.

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

EXPLORATORY BORING: EB-1

DRILL RIG: MOBILE B-90
BORING TYPE: 8-INCH HOLLOW STEM AUGER
LOGGED BY: AC
START DATE: 6-1-11 FINISH DATE: 6-1-11

PROJECT NO: 184153
PROJECT: NORTH SAN PEDRO APARTMENTS
LOCATION: SAN JOSE, CA
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

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS (SURFACE ELEVATION:)	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	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 soft trace sand		11		38	82	
		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
		POORLY GRADED SAND WITH SILT medium dense, moist, gray, fine sand	SP-SM	31				
	30	**FAT CLAY (CH)**	CH	21		31		

Continued Next Page

GROUND WATER OBSERVATIONS:
▽: FREE GROUND WATER MEASURED DURING DRILLING AT 17.0 FEET

LA_CORP.GDT 6/13/11 MV*

EXPLORATORY BORING: EB-1 Cont'd

DRILL RIG: MOBILE B-90
BORING TYPE: 8-INCH HOLLOW STEM AUGER
LOGGED BY: AC
START DATE: 6-1-11 FINISH DATE: 6-1-11

PROJECT NO: 184153
PROJECT: NORTH SAN PEDRO APARTMENTS
LOCATION: SAN JOSE, CA
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)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf) ○ Pocket Penetrometer △ Torvane ● Unconfined Compression ▲ U-U Triaxial Compression 1.0 2.0 3.0 4.0
	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

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: 184153
PROJECT: NORTH SAN PEDRO APARTMENTS
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.

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)	MATERIAL DESCRIPTION AND REMARKS (SURFACE ELEVATION:)	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
	0	**CLAYEY GRAVELS WITH SAND (GC) [FILL]** medium dense, moist, brown, fine to coarse gravel (sub-angular), fine to coarse sand, trace red brick fragments	GC, FILL	49		5	117	
				21		14	90	
	5	**LEAN CLAY WITH SILT (CL)** very stiff, moist, dark brown, low plasticity, trace fine sand. Plasticity Index = 7, Liquid Limit = 25	CL	10		16	105	
				11		12	103	43
	10	**SILTY SAND (SM)** loose, moist, brown, fine sand	SP-SM					
		SILT (ML) stiff, moist, brown, low to moderate plasticity	ML	14		37	81	
	15							
		soft						
	20	Groundwater encountered at 20.5 feet	CL					
		LEAN CLAY (CH) medium stiff, moist, dark gray, moderate to high plasticity, trace fine sand	CL	14		15	115	
		SANDY LEAN CLAY (CL) medium stiff, moist to wet, gray-brown, low plasticity, fine to coarse sand, fine to coarse gravel (sub-angular/rounded)	SM	32				7
	25	**SILTY SAND (SM)** medium dense, wet, brown, fine to coarse sand, fine-coarse gravel (sub-angular/rounded)	SP	31				
		POORLY GRADED SAND (SP) dense, wet, brown, fine to coarse sand, fine to coarse gravel (sub-angular/rounded)	SM					
	30		ML	14				68 28

Continued Next Page

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

LA CORP.GDT 6/13/11 MV*

EXPLORATORY BORING: EB-2 Cont'd

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: 184153
PROJECT: NORTH SAN PEDRO APARTMENTS
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.)	SAMPLER	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	16				
			SM					
		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							

Undrained Shear Strength (ksf)
○ Pocket Penetrometer
△ Torvane
● Unconfined Compression
▲ U-U Triaxial Compression
1.0 2.0 3.0 4.0

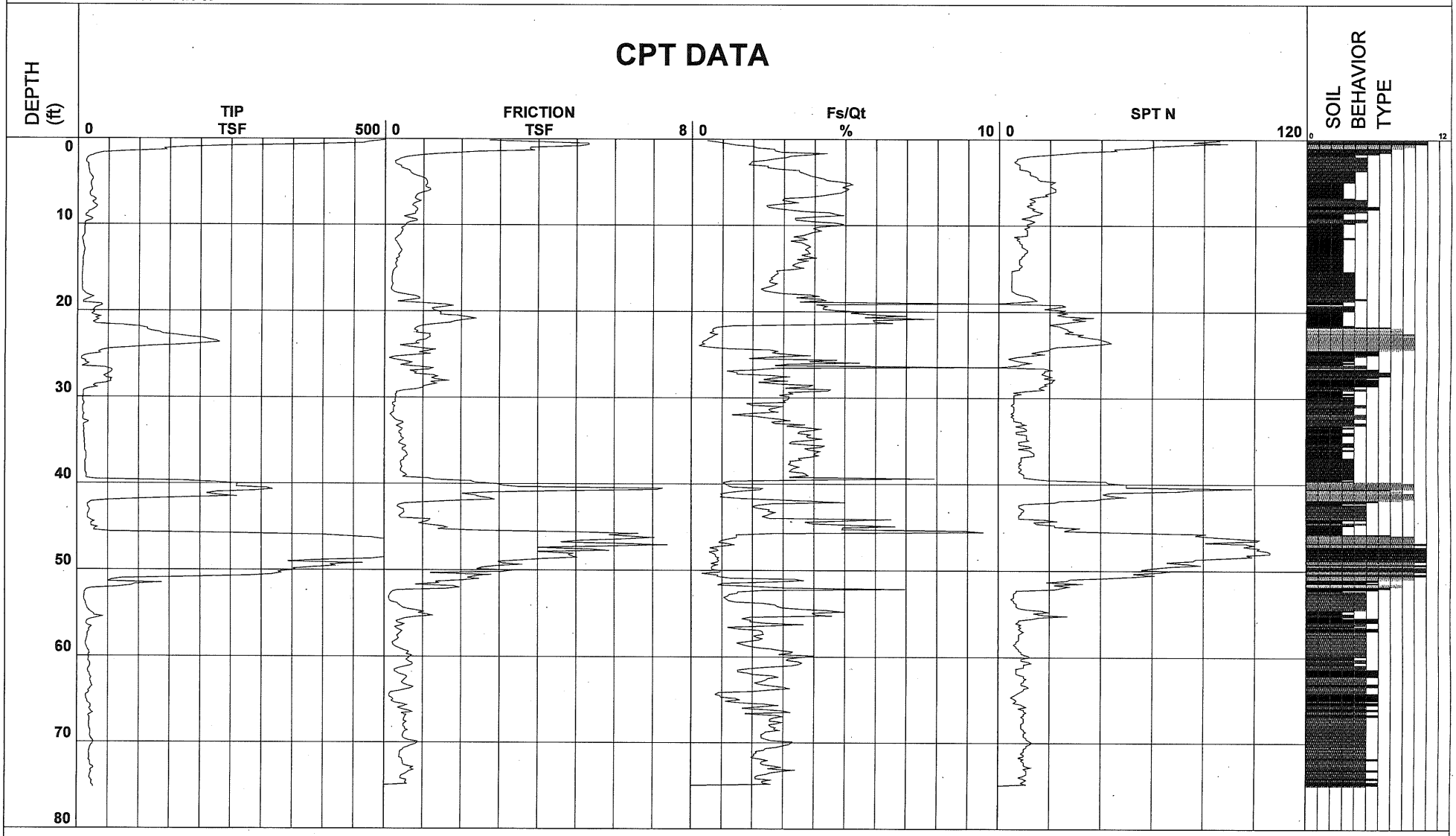
GROUND WATER OBSERVATIONS:
▽: FREE GROUND WATER MEASURED DURING DRILLING AT 20.5 FEET

LA CORP.GDT 6/13/11 MV*

TRC Solutions

Project	North San Pedro Apartments	Operator	BH-JB	Filename	SDF(313).cpt
Job Number	33512	Cone Number	DSG1023	GPS	
Hole Number	CPT-01	Date and Time	5/26/2011 8:24:11 AM	Maximum Depth	75.13 ft
Water Table Depth	14.00 ft				

Net Area Ratio .8



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

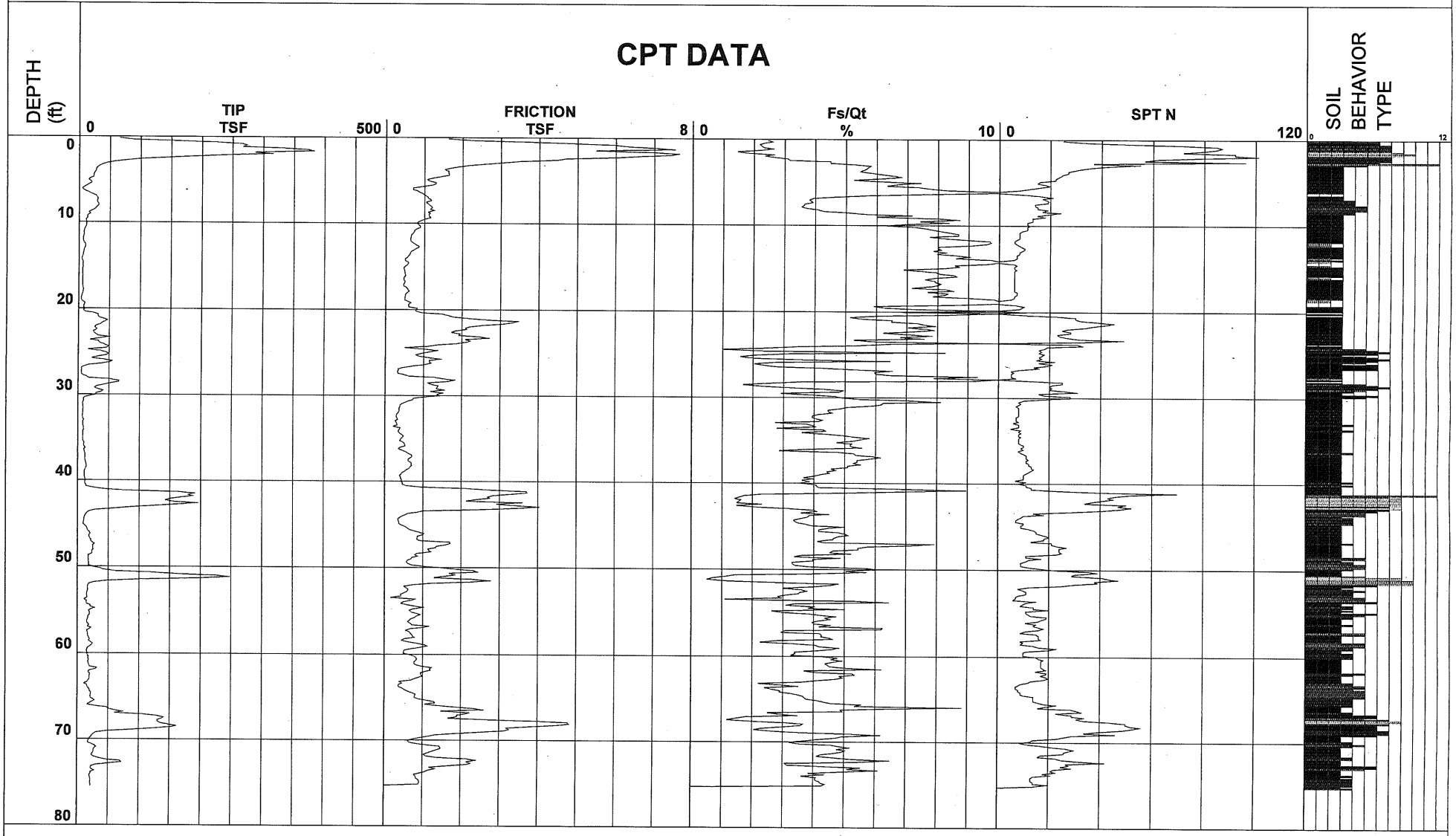
Cone Size 10cm squared

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

TRC Solutions

Project	North San Pedro Apartments	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



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 PROGRAM

The laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site and to aid in verifying soil classification.

Moisture Content: The natural water content was measured (ASTM D2216) on 15 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

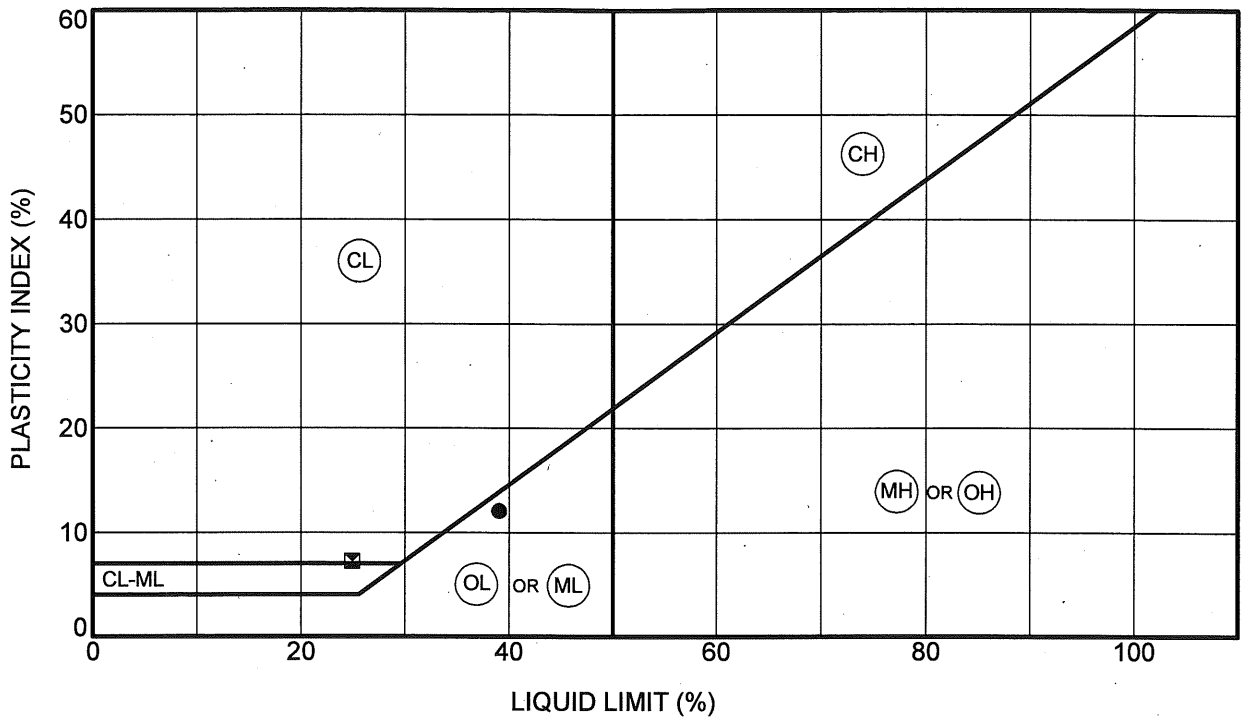
Dry Densities: In place dry density tests (ASTM D2937) were performed on 14 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Plasticity Index: Plasticity Index determinations (ASTM D4318) were performed on two samples of the subsurface soils to measure the range of water contents over which these materials exhibit plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential. Results of these tests are presented on the Plasticity Chart of this appendix and on the log of the boring at the appropriate sample depth.

Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was performed on five samples of the subsurface soils to aid in the classification of these soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Consolidation: One consolidation test (ASTM D2435) was performed on a relatively undisturbed sample of the subsurface clayey soils to assist in evaluating the compressibility property of this soil. Results of the consolidation test are presented graphically on Figure B-1.

* * * * * * * * * * *



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**

TRC

PLASTICITY CHART AND DATA

Project: NORTH SAN PEDRO APARTMENTS

Location: SAN JOSE, CA

Project No.: 184153

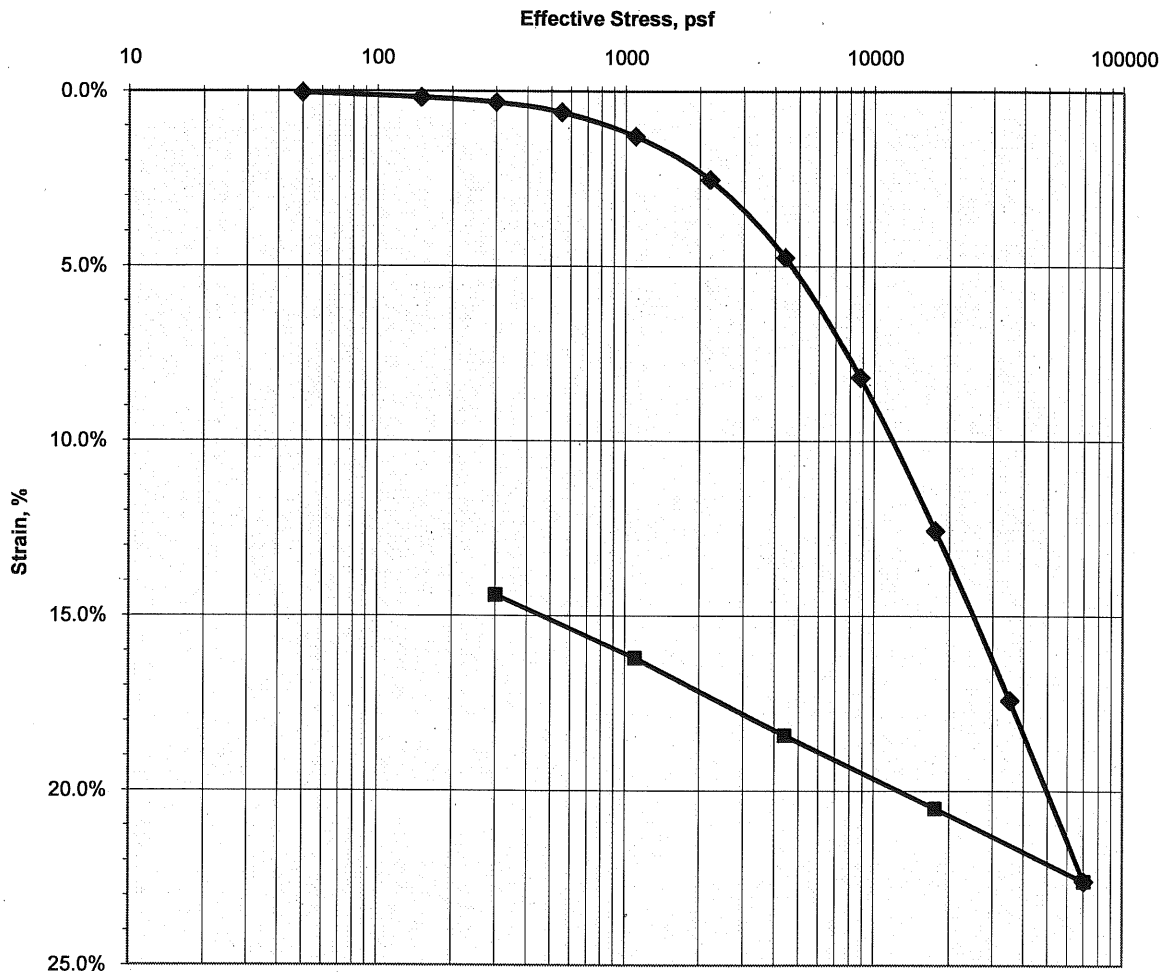
FIGURE B-1

LA_CORP.GDT 6/12/11 MV*

Consolidation Test
ASTM D2435

Job No.:	028-2248	**Boring:**	EB-2	**Run By:**	MD
Client:	TRC	**Sample:**	6A	**Reduced:**	PJ
Project:	N. San Pedro Apts. - 184153	**Depth, ft.:**	19.0(Tip)	**Checked:**	PJ/DC
Soil Type:	Dark Gray Sandy CLAY, trace Gravel			**Date:**	6/21/2011

Strain-Log-P Curve

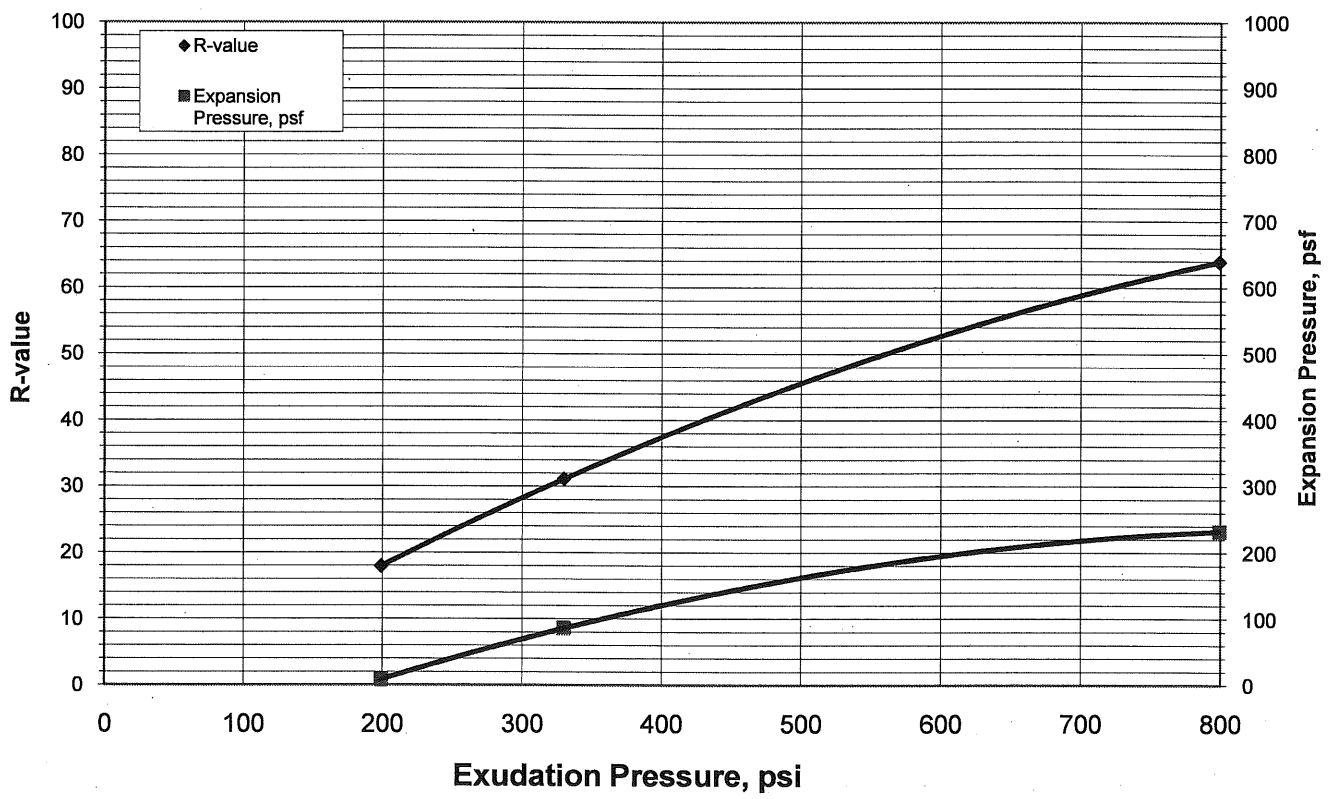


Ass. Gs = 2.7	Initial	Final
Moisture %:	31.8	26.4
Dry Density, pcf:	89.1	98.5
Void Ratio:	0.891	0.711
% Saturation:	96.3	100

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.

R-value Test Report (Caltrans 301)

Job No.:	028-2248	**Date:**	06/08/11	**Initial Moisture,**	10.8%
Client:	TRC	**Tested**	MD	**R-value by Stabilometer**	**28**
Project:	N. San Pedro Apts - 184153	**Reduced**	RU		
Sample	EB-1;Bulk @ Top 5'	**Checked**	DC	**Expansion Pressure**	**70** psf
Soil Type:	Very Dark Brown Clayey SAND, trace Gravel				

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: