



Geotechnical Investigation

West Julian Street Residential Development
San Jose, California

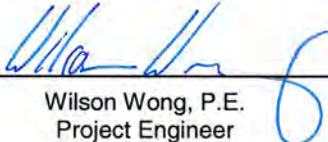
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TABLE OF CONTENTS

1.0	INTRODUCTION	1
1.1	Project Description.....	1
1.2	Scope of Services.....	1
2.0	SITE CONDITIONS	2
2.1	Site Reconnaissance.....	2
2.2	Exploration Program	2
2.3	Subsurface Conditions	2
2.4	Ground Water	3
3.0	GEOLOGIC HAZARDS	3
3.1	Fault Rupture	3
3.2	Maximum Estimated Ground Shaking.....	3
	Table 1. Summary of Peak Ground Acceleration Values	4
3.3	Future Earthquake Probabilities	4
3.4	Liquefaction	4
	3.4.1 General Background	4
	3.4.2 Analysis and Results	5
	Table 2. Results of Liquefaction Analyses – CPT Method	6
	3.4.3 Liquefaction Screening of Fine-Grained Soils	6
	Table 3. Results of Liquefaction Screening – Bray and Sancio Method	6
	3.4.4 Potential for Ground Rupture/Sand Boils	7
3.5	Dry Seismic Settlement	7
3.6	Lateral Spreading	7
4.0	CORROSION EVALUATION.....	7
	Table 4. Results of Corrosivity Testing.....	8
	Table 5. Relationship Between Soil Resistivity and Soil Corrosivity	8
	Table 6. Relationship Between Sulfate Concentration and Sulfate Exposure	8
5.0	CONCLUSIONS AND RECOMMENDATIONS	9
5.1	Primary Geotechnical Concerns	9
	5.1.1 Compressible Soils	9
	5.1.2 Strong Seismic Shaking.....	10
	5.1.3 Shallow Groundwater	10
	5.1.4 Liquefaction-Induced Total and Differential Settlement.....	10
	5.1.5 Demolition Debris.....	10
	5.1.6 Corrosion Potential of Near-Surface Soils	10
	5.1.7 Differential Settlement for Utilities Tie-ins.....	11
5.2	Plans, Specifications, and Construction Review	11
6.0	EARTHWORK	11
6.1	Clearing and Site Preparation	11
6.2	Removal of Undocumented Fill	11
6.3	Abandoned Utilities.....	11
6.4	Subgrade Preparation.....	12
6.5	Material for Fill	12
6.6	Reuse of On-site Recycled Materials	12
6.7	Compaction.....	13
6.8	Wet Soils and Wet Weather Conditions	13
6.9	Trench Backfill	13
6.10	Temporary Slopes and Trench Excavations	14
6.11	Temporary Shoring Support System	14
	Table 7. Temporary Shoring System Design Parameter.....	15
6.12	Temporary Dewatering	16

6.13	Surface Drainage	16
6.14	Landscaping Considerations	16
6.15	Construction Observation	17
7.0	FOUNDATIONS	17
7.1	2013 CBC Site Coefficients and Site Seismic Coefficients	17
	Table 8. 2013 CBC Site Class and Site Seismic Coefficients	18
7.2	Reinforced Mat Foundations	18
7.3	Mat Foundation Settlement	18
7.4	Modulus of Subgrade Reaction	19
7.5	Lateral Loads	19
7.6	Differential Settlement for Utility Tie-ins	19
7.7	Hydrostatic Uplift and Waterproofing	19
7.8	Driven Piles for Structure Foundation	19
	7.8.1 Vertical Loads	20
	Table 9. Estimated Allowable Capacities for 14-and 16-inch Driven, Precast, Prestressed Concrete Piles	20
	7.8.2 Lateral Loads on Piles	20
	Table 10. Estimated Lateral Pile Response – 14- and 16-inch Square Piles	21
	7.8.3 Passive Resistance Against Pile Caps and Grade Beams	21
	7.8.4 WEAP Analysis	21
	7.8.5 Indicator Piles	21
	7.8.6 PDA Monitoring	22
	7.8.7 Production Pile Installation	22
7.9	Augercast Piles	22
	7.9.1 Vertical Capacities	23
	Table 11. Estimated Allowable Capacities for 18-and 24-inch Augercast Piles	23
	7.9.2 Lateral Loads on Augercast Piles	23
	Table 12. Estimated Lateral Pile Response – 18- and 24-inch Round Piles	24
	7.9.3 Passive Resistance Against Pile Caps and Grade Beams	24
	7.9.4 Pile Load Tests	24
7.10	Garage Floor Slab	24
7.11	Differential Settlement for Utility Tie-ins	25
8.0	BASEMENT WALLS	25
8.1	Lateral Earth Pressures	25
8.2	Seismic Lateral Earth Pressures	25
8.3	Drainage	25
8.4	Backfill	26
8.5	Foundation	26
9.0	PAVEMENTS	26
9.1	Asphalt Concrete	26
	Table 13. Recommended Asphalt Concrete Pavement Design Alternatives	27
9.2	Exterior Portland Cement Concrete (PCC) Pavements	27
	Table 14. Recommended Minimum PCC Pavement Thickness	28
9.3	Pavement Cutoff	28
9.4	Asphalt Concrete, Aggregate Base and Subgrade	28
9.5	Flatwork and Sidewalks	28
10.0	LIMITATIONS	29
11.0	REFERENCES	29

FIGURE 1 — VICINITY MAP
FIGURE 2 — SITE PLAN
FIGURE 3 — REGIONAL FAULT MAP
FIGURE 4 — VERTICAL PILE CAPACITIES

APPENDIX A — FIELD INVESTIGATION
APPENDIX B — LABORATORY PROGRAM

**GEOTECHNICAL INVESTIGATION
WEST JULIAN STREET RESIDENTIAL DEVELOPMENT
SAN JOSE, CALIFORNIA**

1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed West Julian Street Residential Development 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 geotechnical recommendations for design of the proposed project.

For our use, we received a set of architectural drawings titled, "West Julian Street Apartments, 715 West Julian Street, San Jose, CA 95126," prepared by Anderson Architects, Inc., dated July 25, 2014.

1.1 Project Description

The approximately 1¼-acre irregularly shaped site is located between West Julian Street and Stockton Avenue and is currently occupied by an existing commercial and residential buildings, asphalt concrete parking lots, and landscaping. The layout of the proposed development is shown on the Site Plan, Figure 2. The site is bordered by West Julian Street to the south, Stockton Avenue to the northeast and existing residential structures to the west and north. As presently planned, the site consists of the construction of a 4-story concrete-framed residential structure with a one-level below-grade parking garage. Additional improvements will include pavements, underground utilities, and landscaping.

Based on the planned improvements, an excavation on the order of 15 feet is anticipated for the one-level below-grade parking garage. 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 October 7, 2014. To accomplish this work, we provided the following services:

- Exploration of subsurface conditions by drilling four borings in the area of the proposed development and retrieving soil samples for observation and laboratory testing. We also advanced three Cone Penetration Tests (CPTs).
- 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 structure foundations, site earthwork, slabs-on-grade, basement walls 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 Senior Staff Engineer performed a reconnaissance of the site on November 10, 2014. At the time of the reconnaissance, the site was occupied by existing commercial and residential buildings, asphalt concrete-paved parking stalls, concrete walkways, trees, and landscaping. The site appeared relatively flat with minor grade variation for drainage purposes.

2.2 Exploration Program

Subsurface exploration was performed on November 10 and 11, 2014 using conventional, truck-mounted hollow-stem auger drilling equipment to investigate, sample, and log subsurface soils. One hollow-stem auger exploratory boring was drilled to a depth of 50 feet. Subsurface exploration was also performed on November 10 and 11, 2014 using CPT equipment to investigate subsurface soils. Three CPTs were advanced to depths ranging from approximately 45 to 61 feet.

Our boring and CPTs were permitted and backfilled in accordance with Santa Clara Valley Water District guidelines. The approximate locations of the borings and CPTs are shown on the Site Plan, Figure 2. The logs of the borings and CPTs and details regarding our field investigation are included in Appendix A; laboratory tests are discussed in Appendix B.

2.3 Subsurface Conditions

All of our CPTs were performed within the footprint of the proposed residential structure. In general, soils encountered in the CPTs were interpreted to include interbedded layers of clay, silty clay, clayey silt, sandy silt, silty sand, sand and gravelly sand to depths ranging from approximately 41 to 58 feet. Below these depths, our CPTs were interpreted to include interbedded layers of sand and gravelly sand to a maximum depth of 61 feet, which was considered refusal with tip pressures greater than 300 tons per square foot. The medium dense sand layers across the site appear to be generally in the upper 15 feet and discontinuous below the depth of 15 feet and vary up to approximately 4 feet in thickness.

All of our borings were also performed within the footprint of the proposed residential structure. All of our borings encountered a pavement section consisting of 2 inches of asphalt concrete underlain by 8½ to 12 inches of aggregate base, except for EB-3 which encountered approximately 8 inches of crushed rock. Below the pavement section or crushed rock, our borings generally encountered medium stiff to hard lean clay and stiff to very stiff sandy lean clay to a depth of about 50 feet, the maximum depth explored. Boring EB-3 encountered medium dense poorly graded gravel underlain by medium dense silty sand to a depth of approximately 11½ feet. Additionally, several relatively thin interbedded layers consisting of medium dense clayey sand, dense silty sand, very dense poorly graded gravel, medium dense poorly graded sand, and very stiff fat clay were encountered at depths from about 7½ to 42 feet and ranging in thickness from approximately 1 to 4 feet. The lean clays in the 15 to 30 foot depth range were medium stiff to stiff and appeared relatively more compressible than clays encountered at other depths.

Three Plasticity Index (PI) tests were performed to determine the Plasticity Index (PI) of the representative clay soil samples from borings EB-1 and 2 at depths of approximately 2, 14½, and 19½ feet, respectively. The tests resulted in PI's of 17, 16, and 4, indicating moderate and low plasticity and expansion potential of the soils.

2.4 Ground Water

Free ground water was encountered during subsurface exploration in borings EB-1, EB-2, and EB-4 at depths of approximately 41, 31½, and 35 feet below grade, respectively. Based on pore pressure dissipation measurements, our CPTs encountered groundwater at depths ranging from approximately 33 to 39 feet below grade. 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 are on the order of 12 feet below the existing ground surface (bgs). Based on the above information, we judged a ground water depth of 12 feet to be appropriate for liquefaction analysis. Our boring 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), or a Santa Clara County Fault Rupture Hazard Zone (SCC, 2002). A Regional Fault Map illustrating known active faults relative to the site is presented in Figure 3. 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	PGAM	0.50	Equation 11.8-1 of ASCE 7-10
CGS Seismic Hazard Zone Report 058, 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	0.75	
Caltrans ARS Web Tool v2.2.06	Deterministic	0.46	Silver 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, 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). The site is also located within an area zoned in the Santa Clara County Geologic Hazard Zone maps as a Liquefaction Hazard Zone (2002). During cyclic ground shaking, such as earthquakes, cyclically-induced stresses may cause increased pore water pressures within the soil matrix, which results in liquefaction. The project site is located at approximately 37.3339 degrees north and 121.9070 degrees west. 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 12 feet below the existing site grade was used for our liquefaction analysis. As discussed in the subsurface description above, several gravel, 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 structures. 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 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 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 0.50g (based on Equation 11.8-1 of ASCE 7-10) and moment magnitude of 6.80 (USGS 2008).

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 I_c greater than 2.6 or CPT tip resistance greater than 160 tons per square foot (tsf) are considered either too plastic or too dense to liquefy, respectively. Such soil layers have been screened out of the analysis and are not presented below. A summary of our CPT analysis is presented in Table 2 below.

Table 2. Results of Liquefaction Analyses – CPT Method

CPT Number	Depth to Top of Sand/Silt Layer (feet)	Layer Thickness (feet)	lc	*qc _{1N} (tsf)	Factor of Safety	Potential for Liquefaction	Estimated Total Settlement (in.)
CPT-1	18.4	0.7	2.28	126.9	0.9	Likely	0.1
	27.4	1.0	2.37	124.8	0.8	Likely	0.2
	28.9	0.3	2.58	116.4	0.7	Likely	0.1
	41.1	0.3	2.26	108.0	0.6	Likely	0.1
Total =							0.5
CPT-2	12.1	1.8	2.23	92.3	0.6	Likely	0.5
	30.7	0.3	2.57	72.7	0.3	Likely	0.1
	35.4	1.0	2.17	111.4	0.6	Likely	0.2
Total =							0.8
CPT-3	12.1	0.5	2.22	109.5	0.8	Likely	0.1
Total =							0.1

* CPT tip pressure corrected for overburden and fines content

The current methods for estimating liquefaction settlement are generally applicable for the upper 50 feet and the effects of liquefaction settlement below 50 feet on the proposed structures should be minimal. Settlements below a depth of 50 feet are not included as part of total or differential liquefaction induced settlements at the ground surface for the foundation recommendations.

Our analyses indicate that several sand and silt layers below the design ground water depth may theoretically liquefy, resulting in approximately ¼-inch to ¾-inch of total settlement. Volumetric change and settlement were estimated using the Zhang, Robertson, and Brachman (2002) method. We estimate differential settlements from liquefaction will be on the order of ½-inch in 50 horizontal feet. A detailed discussion of estimated settlements is presented in the “Foundations” section of this report.

3.4.3 Liquefaction Screening of Fine-Grained Soils

We also performed a liquefaction screening for the lean clays following the conclusions presented in the paper titled “Assessment of the Liquefaction Susceptibility of Fine-Grained Soils,” prepared by Johnathan D. Bray and Rodolfo C. Sancio in 2006. The conclusions of the paper were that fine-grained soils with PI values less than 18 and moisture contents (WC) above 80 percent of the Liquid Limit (LL) are potentially susceptible to liquefaction.

We performed Atterberg Limits tests on two representative soil samples below the design ground water depth of 12 feet collected from Borings EB-1 and EB-2 at depths of approximately 14½ and 19½ feet, respectively, below the existing ground surface. Results of the Atterberg Limits tests and our liquefaction screening are summarized in Table 3 below.

Table 3. Results of Liquefaction Screening – Bray and Sancio Method

Boring	Sample Depth	Sample Description	LL	PI	WC	WC/LL (percent)
EB-2	14½	Lean Clay	31	16	27	87
EB-2	19½	Lean Clay	23	4	25	109

Based on the results of the screening, it appears that the lean clay represented in the samples from borings EB-1 and EB-2 at depths of 14½ and 19½ feet, respectively, is susceptible to liquefaction

based on the Bray and Sancio criteria as the PI is less than 18 and the WC is greater than 80 percent of the LL. We estimate that liquefaction of the fine grained soils would result in approximately ½-inch of total settlement with differential settlement on the order of ¼- inch.

3.4.4 Potential for Ground Rupture/Sand Boils

The methods of analysis used to estimate the total liquefaction induced settlement assume that there is no possibility of surface ground rupture. For liquefaction induced sand boils or fissures to occur, the pore water pressure induced within the liquefied strata must be large enough to break through the surface layer.

The bottom of the proposed structure with one-level below-grade parking garage will be approximately 15 feet below the ground surface. There is approximately 3½ feet of non-liquefiable material overlying a thin (less than 1 foot) potentially liquefiable stratum at the site below the estimated bottom of mat foundation. Based on the work by Youd and Garris (1995), there is not an adequate non-liquefiable material capping the shallow liquefiable layer at the site. However, because the potentially liquefiable stratum is less than 1 foot thick and does not appear to be continuous across the site, the potential for ground rupture is judged to be low.

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, our explorations encountered several medium dense granular soils ranging from approximately 1 to 4 feet in thickness above the design ground water depth that may densify during a strong earthquake resulting in approximately ¼-inch of seismically induced dry sand settlement. However, based on the proposed structure with one-level below-grade the excavation will extend to a depth of approximately 15 feet below-grade, removing this material. Therefore, we judge the probability of significant differential settlement of non-saturated granular layers at the site to be low.

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.

The Guadalupe River is located approximately ¼-mile northeast of the site boundary. The potentially liquefiable layers at the site below the garage level are thin, discontinuous, relatively deep and below the level of the riverbed. For these reasons, the probability of lateral spreading occurring at the site during a seismic event is judged to be low.

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 4 below.

Table 4. 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	11	1,013	7.7	1,240	Severe	Moderate
EB-2, 1B	2.0	6	96	7.7	1,915	Severe	Negligible
EB-3, 3A	5.5	5	54	7.8	7,065	Mild	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 classification developed by William J. Ellis (1978), the approximate relationship between soil corrosiveness was developed as shown in Table 5 below.

Table 5. 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 (PCC) 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. Soils containing high sulfate content could also cause corrosion of the reinforcing steel in concrete. Table 4.2.1 of the American Concrete Institute (ACI, 2008) provides requirements for concrete exposed to sulfate-containing solutions as summarized in Table 6.

Table 6. Relationship Between Sulfate Concentration and Sulfate Exposure

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 4, the soil resistivity results ranged from 1,240 to 7,065 ohm-centimeters. Based on these results and the resistivity correlations presented in Table 5, the corrosion potential to buried metallic improvements may be characterized as mildly 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 4.2.1 of the ACI, it is our opinion that sulfate exposure to PCC may be considered negligible to moderate for the native subsurface materials sampled.

5.0 CONCLUSIONS AND RECOMMENDATIONS

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

5.1 Primary Geotechnical Concerns

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

- Compressible soils
- Strong seismic shaking
- Shallow groundwater
- The potential for liquefaction-induced total and differential settlement
- Demolition of the existing buildings and pavements prior to site development
- Corrosion potential of the near-surface soils
- Differential Settlement for Utility Tie-ins

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 Compressible Soils

As discussed in the "Subsurface" section, we encountered layers of medium stiff clays between depths of 15 to 30 feet below grade. For a mat with an average allowable bearing pressure of 2,000 psf for dead plus live loads we estimated static settlement of about 1-inch at the center, ½-inch at the edge, and ¼-inch at the corner.

If the anticipated settlements are too high for the structure to be supported on a mat foundation, the structure may alternatively be supported on deep foundations. Detailed recommendations and a discussion of estimated settlements are presented in the "Foundations" section of this report.

5.1.2 Strong Seismic Shaking

We recommend that, at a minimum, the proposed parking structure 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 8.

5.1.3 Shallow Groundwater

As discussed in Section 2.4, ground water was encountered in our CPTs and exploratory borings at depths ranging from approximately 31 to 41 feet below the existing ground surface. Published information by CGS indicates historical high ground water levels are approximately 12 feet below existing grades. Due to the below normal precipitation over the past few years, and the time of the year the explorations were done, which is normally the lowest ground water level during the year, we judge that the normal depth to ground water would be in the range of 20 to 30 feet below the ground surface.

The proposed structure should be designed to resist hydrostatic uplift pressures and basement walls will have to resist hydrostatic pressure up to the design groundwater of 12 feet below the ground surface. The contractor should be aware that excavations/trenches extending near the ground water level may need to be stabilized and/or dewatered to facilitate placement of structures and/or placement and compaction of fill.

5.1.4 Liquefaction-Induced Total and Differential Settlement

Our analyses indicate that several layers theoretically can liquefy, ranging from $\frac{3}{4}$ -inch to $1\frac{1}{4}$ inches of total settlement in the upper 50 feet, with differential settlements from liquefaction on the order of $\frac{3}{4}$ -inch in a horizontal distance of 50 feet. The proposed structure should be designed to accommodate the potential seismic and as well as static settlements as discussed in the "Foundations" section.

5.1.5 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. 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. It has been our experience that 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.

5.1.6 Corrosion Potential of Near-Surface Soils

As discussed above, the corrosion potential to buried metallic improvements constructed within the native soils may be characterized as mildly 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.

5.1.7 Differential Settlement for Utilities Tie-ins

The utilities entering the structure could experience differential settlement at the tie-in locations. We recommend emergency shut-off valves and flexible utility and piping connections that can accommodate at least two inches of movement if the structure is supported on piles.

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 and CPTs 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 building foundations, slabs, 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 resulting 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 Undocumented Fill

If undocumented fill is encountered, it should be removed down to the native soil. If the fill material meets the requirements in the "Material for Fill" section below, it may be reused as engineered fill. Side slopes of fill removal excavations in building and pavement areas should be sloped at inclinations no steeper than 3:1 (horizontal:vertical) to minimize abrupt variations in fill thickness. 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 area 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, if 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 area 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

The subgrade for the below-grade structure will likely be saturated and difficult to compact. The contractor should minimize the use of rubber-tired equipment on the subgrade. A rat slab could be poured over the subgrade to facilitate a working surface.

For at-grade pavements and flatwork, after the site has been properly cleared, stripped and necessary excavations have been made, exposed surface soils in those areas to receive fill 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 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. The highly expansive clays, if excavated during grading, should be segregated and should not be re-used below the proposed structure.

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. Non-expansive fill (NEF) should have a PI of 10 or less. 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 Reuse of On-site Recycled Materials

Significant amounts of asphalt concrete/aggregate base grindings may be generated during removal of the existing pavements. If it is desired to reuse the grindings for new site pavement structural support, we recommend the asphalt concrete be pulverized and mixed with the underlying aggregate base to meet Caltrans Class 2 Aggregate Base requirements. If laboratory testing of the recycled material indicates that it meets Caltrans Class 2 specifications, it may be used as Class 2 Aggregate Base beneath pavements and sidewalks. Recycled material containing asphalt concrete grindings should not be used below building areas. Laboratory testing may be performed on initial grindings generated to evaluate the material further and refine the pavement recommendations.

6.7 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, at a moisture content near the laboratory optimum. 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), except for the native clays, which should be compacted as noted above. Aggregate base and all import soils should be compacted at a moisture content near the laboratory optimum moisture content.

6.8 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.

6.9 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 of the 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. If native expansive

soil is used for trench backfill, it should be compacted to between 87 to 92 percent at a moisture content between optimum and 2 percent over optimum. 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 very highly expansive subgrade soils.

6.10 Temporary Slopes and Trench 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. On a preliminary basis, site soils can be classified as Type C based on soil classification by OSHA. Therefore a maximum slope 1.5:1 (horizontal:vertical) should be anticipated. A TRC representative should be retained to verify soil conditions in the field at the time of the excavation.

6.11 Temporary Shoring Support System

As previously discussed, an excavation on the order of approximately 15 feet is planned to construct the below-grade parking garage. Saturated clay and ground water may be encountered. The excavations could potentially be temporarily supported by several methods including tiebacks, soil nailing, braced shoring, temporary slopes if space is adequate, or potentially other methods. Where shoring is required, restrained shoring will most likely be necessary to limit deflections and disruption to nearby improvements. It has been our experience that cantilever shoring might be feasible for temporary shoring to a height of about 10 to 13 feet where allowable deflections are limited. The choice of shoring method should be left to the contractor's judgment since economic considerations and/or the individual contractor's construction experience may determine which method is more economical and/or appropriate. However, other factors such as the location of nearby utilities and encroachment on adjacent properties may influence the choice of support.

The temporary shoring should be designed for additional surcharges due to adjacent loads such as from construction vehicles and street traffic. To prevent excessive surcharging of the walls, we recommend that heavy loads such as construction equipment and stockpiles of materials be kept at least 15 feet from the top of the excavations. If this is not possible, the shoring must be designed to resist the additional anticipated lateral loads. Shoring systems should be designed with sufficient rigidity to prevent detrimental lateral displacements. Minimum geotechnical parameters for design of a temporary shoring system are given in Table 7.

Table 7. Temporary Shoring System Design Parameter

Design Parameter	Design Value (psf)
Minimum Lateral Wall Surcharge ¹	120 psf
Earth Pressure – Cantilever Wall	40 pcf
Earth Pressure – Restrained Wall ² From ground surface to H/4 (ft)	Increase from 0 to 25H psf
Earth Pressure – Restrained Wall Below H/4 (ft)	Uniform pressure of 25H psf
Passive Pressure ³	300 pcf up to 1,500 psf max

Note: 1 For the upper 5 feet (minimum for incidental loading)

2 Where H equals height of excavation

3 Can assume to act over 2 times the diameter of soldier piles, neglecting the upper foot

To limit potential movements of the shoring system, the shoring designer and contractor should consider several design and construction issues. For the movements of shoring to be reduced, the designer will have to provide for a uniform and timely mobilization of the soil pressures. Tiebacks or internal bracing should be loaded to the design loads prior to excavation of the adjacent soil so that load induced strains in the retaining system will not result in the system moving toward the excavation. In addition, a relatively stiff shoring system should be designed to limit deflections under loading. In general, we recommend designing a shoring system to deflect less than 1-inch.

In addition, ground subsidence and deflections can be caused by other factors such as voids created behind the shoring system by over-excavation, soil sloughing, erosion of sand or silt layers due to perched water, etc. All voids behind the shoring system should be filled as soon as feasible by grouting to minimize potential problems during installation of the shoring system.

Since we drilled our borings with hollow-stem auger drilling equipment, we are not able to evaluate the potential for caving of on-site soils, which may become a factor during soldier pile and/or tieback installation. The contractor is responsible for evaluating excavation difficulties prior to construction. Pilot holes using proposed production drilling equipment may be prudent, to evaluate possible excavation difficulties such as caving soils, cobbles, boulders and/or other excavation difficulties.

In conjunction with the shoring installation, a monitoring program should be set up and carried out by the contractor to determine the effects of the construction on the adjacent buildings, street and other improvements such as sidewalks and utilities. As a minimum, we recommend horizontal and vertical surveying of reference points on the shoring and on the adjacent street, buildings and other improvements in addition to an initial crack survey. We also recommend that all supported and/or sensitive utilities be located and monitored by the contractor. Reference points should be set up and read prior to the start of construction activities. Points should also be set on the shoring as soon as initial installations are made. Alternatively, inclinometers could be installed by the contractor at critical locations for a more detailed monitoring of shoring deflections. Surveys should be made at least once a week and more frequently during critical construction activities, or if significant deflections are noted. TRC can provide inclinometer materials and we have the equipment and software to read and analyze the data quickly.

This report is intended for use by the design team. The contractor should perform additional subsurface exploration and/or geotechnical studies as they deem necessary for the chosen shoring system. The contractor is also responsible for site safety and the means and methods of construction, including temporary shoring. Temporary shoring must be designed by a licensed California Civil or Structural Engineer. Prior to construction, we recommend that the contractor forward his plan for the support system to the structural engineer and geotechnical engineer for preconstruction review.

6.12 Temporary Dewatering

As previously discussed, measured ground water elevations are approximately 15 feet below the planned excavation depth. The historic high ground water level is above the planned excavation depth; therefore, temporary dewatering may be necessary during construction at the building location. Temporary dewatering for construction should be the responsibility of the contractor. The selection of equipment and methods of dewatering should be left up to the contractor and, due to the variable nature of the subsurface conditions in the area, the contractor should be aware that modifications to the dewatering system, such as adding well points or wells, may be required during construction depending on the conditions encountered.

During excavations, we recommend that the ground water level be maintained 5 feet below the bottom of the excavation to help reduce earthwork difficulties. Ground water should not be drawn down deeper than necessary; however, as lowering the ground water table can cause subsidence and settlement of adjacent parcels. Additionally, it may be desired to allow dewatering to take place for some time (1 to 2 weeks) before the excavation begins to allow time for the soil to drain. We should review the dewatering and excavation plans prior to construction.

Should dewatering be temporarily shut down while the excavation is open, it could have considerable affects on the excavations, including flooding, destabilization of the bottom of the excavation, shoring failures, etc. Therefore, we recommend that consideration be given to having the dewatering contractor provide backup power in case of loss of power or other redundancies, as deemed necessary.

Special considerations may be required prior to discharge of ground water from dewatering activities depending on the environmental impacts at the site or at nearby locations. These requirements may include storage and testing under permit prior to discharge. Impacted ground water may require discharge to an offsite facility.

6.13 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.

6.14 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 perimeters.

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

6.15 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 settlement of soils and liquefaction to occur. In addition, the building will have one-level below-grade, which will be below the historic high ground water level. Provided that the site is prepared in accordance with the "Earthwork" section of this report and the proposed structure can be designed to accommodate the following estimated amounts of settlement, the structure may be supported on a reinforced mat foundation as discussed in the sections below.

If the estimated settlements are too high, as an alternative to a mat foundation, the structure may be supported on deep foundations consisting of driven, precast, prestressed concrete friction piles or augered cast-in-place piles. It is our opinion that driven or augercast pile foundations will be able to support the structure with only minor settlements and will provide adequate support during liquefaction and seismic events. Recommendations for friction and augercast-in-place piles, are presented in Sections 7.8 and 7.9, respectively.

7.1 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 our explorations, the site is generally underlain by medium stiff to hard clays, 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 8 below.

Table 8. 2013 CBC Site Class and Site Seismic Coefficients

Latitude: 37.3339 N Longitude: 121.9070 W	CBC Reference*	Factor/ Coefficient	2013 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

7.2 Reinforced Mat Foundations

The proposed residential structure may be supported on conventionally reinforced mat foundation. Based on the subsurface conditions, the mat may be designed for an average allowable bearing pressure of 2,000 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.

7.3 Mat Foundation Settlement

Our calculations for the residential building with a reinforced mat foundation designed for an average allowable bearing pressure of 2,000 pounds per square foot (psf) for dead plus sustained live loads indicate static settlement of about 1-inch at the center, ½-inch at the edge and ½-inch at the corner for a 2-foot-thick mat bearing at 15 feet below the existing site grades.

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 ¾-inch of differential settlement over a horizontal distance of 50 feet. We should be retained to review the final foundation plans and structural loads to verify the above settlement estimates.

We assume that the ramps into the below-grade garage will be within the footprint of the reinforced mat foundation. In addition we assume that the handicap walkways will be structurally supported. However, there may be differential settlement between the structurally supported ramps and walkways and adjacent flatwork. We recommend structurally supporting flatwork adjacent to the building for a span of at least 5 feet laterally from the building or a hinge slab or other method

should be used to accommodate portions of the structures that will be supported on different materials.

7.4 Modulus of Subgrade Reaction

For structural design of the mat, we recommend using a subgrade modulus that models the soil response under building loads. In developing the appropriate modulus of subgrade reaction (referred to as the “subgrade modulus”), we considered the varying soil conditions and stress distribution for the planned building layout. Based on the bearing pressure and settlements given above, for the proposed residential structure we recommend a modulus of subgrade reaction of 30 pounds per cubic inch (pci).

We would be pleased to provide supplemental consultation in refining the soil subgrade modulus value, if desired. In order to proceed with further analysis, we would need the output from the first iteration of the SAFE analysis or other finite element analysis of the mat soil structure interaction.

7.5 Lateral Loads

Lateral loads may be resisted by friction between the bottom of 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 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, with a maximum of 2,000 psf at depth. The upper 12 inches of soil should be neglected when determining lateral passive resistance.

7.6 Differential Settlement for Utility Tie-ins

The utilities entering the building could experience differential settlement specifically at the tie-in locations. We recommend emergency shut-off valves and flexible utility and piping connections that could accommodate at least two inches of vertical and horizontal movement.

7.7 Hydrostatic Uplift and Waterproofing

We recommend that the mat be designed to withstand hydrostatic uplift pressure from a design ground water level of 12 feet below the existing site grades. We also recommend that the mat foundation, all construction joints, and basement walls be waterproofed to limit moisture infiltration. We recommend that a waterproofing specialist design the waterproofing system, including the under-slab waterproofing design and all below-grade walls. A rat slab could be poured over the subgrade to protect the water-proofing as reinforcing steel is placed. Drainage systems above historic high ground water levels and design for hydrostatic pressures for the basement walls are discussed in subsequent sections.

7.8 Driven Piles for Structure Foundation

As discussed above, as an alternative to a mat, pile foundations could support the proposed residential structure with only minor settlements. The proposed residential may be supported on driven, precast, prestressed concrete friction piles. Conventional slabs-on-grade may be used in conjunction with a pile foundation provided that the subgrade soils consist of properly compacted, engineered fill and NEF.

Also, as discussed in Section 2.3, beneath the proposed structure, a continuous layer of sand and gravel that resulted in refusal was encountered in CPT-2 and CPT-3 from depths of approximately 57 to 61 feet. This layer may result in higher driving stresses. As discussed below, a wave

equation analysis of piles (WEAP) analysis should be performed to evaluate likely driving stresses through this dense sand layers. If the estimated driving stresses are not acceptable, it may be necessary to predrill the piles in some areas. Predrilled holes should have a diameter no greater than the smallest dimension of the precast piles. Variations in subsurface conditions are typical in this area and changes in driving conditions during construction should be expected.

7.8.1 Vertical Loads

Our explorations indicate that there is a continuous sand and gravel layer that may be able to provide end bearing support; however, deeper subsurface exploration would need to be performed in order to confirm the presence of such a layer. Therefore, pile support is expected to come predominantly from frictional support in the stiff clays and silts. We computed allowable downward vertical capacities for 14- and 16-inch-square concrete piles. A summary of the allowable pile capacities are presented in Table 9 below. In addition, Figure 4 shows the increase in pile capacity with length. The indicated capacities in Table 9 and Figure 4 are for dead plus live loads. Dead loads should not exceed two-thirds of the computed capacities. Uplift loads should also not exceed two-thirds of the computed downward capacities on Figure 4. The pile capacities may be increased by one-third under transient loading, including wind and seismic.

Gross capacity of the piles should not exceed the pile structural capacity. We have assumed a base of pile cap 5 below the proposed subgrades for our analysis. To effectively minimize pile group effects and reduction in individual pile capacity, piles should be located with a minimum center-to-center spacing of three times the pile width.

Table 9. Estimated Allowable Capacities for 14-and 16-inch Driven, Precast, Prestressed Concrete Piles

Pile Size	Length ¹ (feet)	Estimated Allowable Capacity (dead plus live loads) (kips)
14-inch square	23	50
14-inch square	45	100
14-inch square	66	150
16-inch square	21	50
16-inch square	40	100
16-inch square	59	150

Note: 1 – Length from bottom of pile cap to bottom of pile

Based on the maximum allowable loads for a single pile, we estimate total settlements of less than ¼-inch to mobilize allowable static capacities. Therefore, post-construction pile foundation settlements of about ½-inch should be anticipated.

7.8.2 Lateral Loads on Piles

Lateral load resistance for pile-supported structures may be developed through pile bending/soil interaction. The magnitude of the lateral load resistance is dependent upon many factors, including pile stiffness and embedment length, conditions of fixity at the pile cap, the physical properties of the surrounding soils, the tolerable top deflection and the yield moment capacity of the pile.

To estimate lateral capacities of piles, we used a computer program that models the soil response in the form of load-deflection (p-y) curves to estimate the capacity of the piles to resist the expected lateral loads. The lateral load characteristics for 14- and 16-inch-square, driven concrete piles with free head and fixed head conditions at 5 feet below subgrade are presented in Table 10. A 150 kip axial load was used.

Table 10. Estimated Lateral Pile Response – 14- and 16-inch Square Piles

Pile Size	Head Condition	Deflection (inches)	Maximum Shear Force (kips)	Maximum Moment (ft-kips)	Depth to Maximum Moment (ft)
14-inch	Free	¼	17	44	5.5
		½	22	68	6.0
	Fixed	¼	31	109	Top of Pile
		½	41	168	
16-inch	Free	¼	21	60	6.0
		½	28	92	6.5
	Fixed	¼	38	149	Top of Pile
		½	51	231	

The values presented in Table 10 represent the anticipated maximum shear and moment under short-term loading conditions and include no factor-of-safety. Suitable factors-of-safety should be selected on the basis of the type of loading. Pile stiffnesses (EI) of 1.4×10^{10} lb-in² and 2.4×10^{10} lb-in² have been assumed in our calculations of load deflection for the 14- and 16-inch piles, respectively. A minimum compressive strength of 6,000 pounds per square inch was assumed for concrete modulus calculations. If pile stiffness varies by no more than 20 percent than that reported above, load deflection characteristics may be approximated by multiplying the deflection values by the ratio of the pile stiffness (EI). We should evaluate the response of piles with significantly different stiffness.

The above lateral load characteristics are for single piles and may not be characteristic of the lateral load capacity of piles in a group. Group effects may reduce the allowable lateral load for a given deflection. We recommend that a pile group efficiency of 0.75 be used for pile groups 3-by-3 or smaller. A group reduction would not be necessary for groups of 1 or 2 piles. For pile groups larger than 3-by-3, we recommend that we review the final pile group layout and structural loads to further evaluate the pile group efficiency under lateral loading.

7.8.3 Passive Resistance Against Pile Caps and Grade Beams

If desired, the passive resistance of soil against pile caps and grade beams poured neat against well-compacted engineered fill or native soil may be used for lateral resistance. We recommend that an allowable passive pressure based on an equivalent fluid pressure of 300 pounds per cubic foot be used in design.

7.8.4 WEAP Analysis

At a minimum, we recommend that the pile contractor have a WEAP analysis performed to confirm compatibility and driveability of the pile driving system with the pile type and soil conditions at the site. We should review the WEAP results prior to mobilization of pile driving equipment to the site.

7.8.5 Indicator Piles

It has been our experience that uncertainties associated with production pile driving can be reduced considerably by implementing an indicator pile program. An indicator pile program will also provide a better means of confirming the limits of layers where high driving resistance may be encountered, and to more accurately estimate final pile lengths.

We recommend that two to three indicator piles be installed for the proposed residential structure before the final pile casting lengths have been selected. The indicator piles should be driven with

the same equipment that will be used to drive the production piles. We should review or select the indicator pile locations when structural drawings are made available. The indicator pile cast lengths should be based on the design lengths required to meet the desired capacity, plus 10 feet. It is expected that some indicator piles may not be driven to their entire length and will require cut off to provide the desired butt elevation; sufficient moment steel should be provided in the pile. Indicator piles can be used for support of the structure and, therefore, should be located appropriately. We also suggest that one or more spare piles be delivered to the site during the indicator program.

7.8.6 PDA Monitoring

We recommend that a Pile Driving Analyzer (PDA) be used during the indicator pile program to determine approximate pile capacities and driving stresses through dynamic testing. PDA monitoring may allow a reduction in production pile lengths and thus cost savings to the project. PDA monitoring should be performed during indicator driving and for restrikes; preferably restrikes should be performed no sooner than seven days after initial driving. Subsequent restrikes may be necessary based on initial restrike data. *Please note that restrike testing more than one day after installation may significantly alter the contractor's sequencing. Therefore, if restrike testing is selected for this project, it should be clearly identified on the plans and specifications to avoid unexpected costly change-orders for out of sequence moves.* PDA monitoring would be especially beneficial for checking stresses in the piles and for evaluating pile integrity on any piles suspected of being damaged during indicator or production driving. Piles designated for PDA monitoring during indicator pile installation should be at least 10 feet longer than design length so that the gauges are not driven into the ground.

7.8.7 Production Pile Installation

We recommend that a pile hammer capable of delivering a minimum rated driving energy of 60,000 foot-pounds be used. If indicator piles are installed, the same hammer should be used for both the indicator piles and the production piles. The pile contractor should perform WEAP analysis to confirm the compatibility and drivability of the pile driving system with the pile type and soil conditions at the site. We should be given sufficient time to review the wave equation results prior to mobilization of pile driving equipment to the site.

Since the piles are designed for skin friction support, they should be driven to the desired tip elevation. If difficult driving conditions are encountered, we should review the driving record and evaluate potential tip capacity to allow reduction in pile length. We may also recommend that a PDA be used during production driving to determine approximate pile capacities through dynamic analyses. PDA monitoring would be especially beneficial for checking restrike capacities of any piles short of required tip elevation or for evaluating pile integrity on any piles suspected of being damaged during driving. We should observe all indicator and production pile installation on a full-time basis.

7.9 Augercast Piles

While less common in the Bay Area than driven piles, augercast piles have been successfully used for projects throughout the Bay Area in similar soil conditions. Augercast piles are cast-in-place concrete piles that are drilled using a hollow-stem auger and pumping sand-cement grout through the bottom of the auger as the auger is retracted. Three types of augercast piles are available: augercast, pressure-grouted (APG) piles, which like piers, remove the soil column and replace it with grout; augercast, pressure-grouted displacement (APGD) piles, which displace the soil prior to grout placement and augercast, pressure-grouted partial-displacement (APGPD) piles, which partially displace the soil prior to grout placement. Augercast piles are a low noise and vibration installation compared to driven piles and would not require pre-drilling through the very dense sand

layers. Various types of steel reinforcing including rebar cages or H-piles may be installed into the still wet grout after drilling.

7.9.1 Vertical Capacities

As with driven piles, augercast piles will develop their vertical capacity predominately from frictional support in the stiff clays and silts and dense sands. We computed allowable downward vertical capacities for 18- and 24-inch diameter APG piles. A summary of the allowable pile capacities is presented in Table 11 below. In addition, Figure 4 shows the increase in pile capacity with length. The indicated capacities in Table 11 and Figure 4 are for dead plus live loads. Dead loads should not exceed two-thirds of the computed capacities. Uplift loads should also not exceed two-thirds of the computed downward capacities. The pile capacities may be increased by one-third under transient loading, including wind and seismic. Please note that displacement augercast piles are limited to a total drill depth of 75 to 80 feet below grade or less due to the equipment limitations.

We have assumed a base of pile cap at approximately 5 feet below the proposed subgrade for our analysis. To effectively minimize pile group effects and reduction in individual pile capacity, piles should be located with a minimum center-to-center spacing of three times the pile diameter.

Table 11. Estimated Allowable Capacities for 18-and 24-inch Augercast Piles

Pile Size	Length ¹ (feet)	Estimated Allowable Capacity (dead plus live loads) (kips)
18-inch round	23	50
18-inch round	45	100
18-inch round	66	150
24-inch round	18	50
24-inch round	34	100
24-inch round	50	150

Note: 1 – Length from bottom of pile cap to bottom of pile

Based on the maximum allowable loads for a single pile, we estimate total settlements of less than ¾-inch to mobilize allowable static capacities. Therefore, post-construction pile foundation settlements of less than ½-inch should be anticipated.

7.9.2 Lateral Loads on Augercast Piles

To estimate lateral capacities of piles, we used a computer program that models the soil response in the form of load-deflection (p-y) curves to estimate the capacity of the piles to resist the expected lateral loads. The lateral load characteristics for 18- and 24-inch diameter, augercast piles with fixed- and free-head conditions are presented in Tables 12 below for 5 feet below grade. A 150 kip axial load was used.

Table 12. Estimated Lateral Pile Response – 18- and 24-inch Round Piles

Pile Size	Head Condition	Deflection (inches)	Maximum Shear Force (kips)	Maximum Moment (ft-kips)	Depth to Maximum Moment (ft)
18-inch	Free	1/4	21	55	5.5
		1/2	28	85	6.5
	Fixed	1/4	38	137	Top of Pile
		1/2	51	211	
24-inch	Free	1/4	33	108	7.5
		1/2	44	167	8.0
	Fixed	1/4	59	272	Top of Pile
		1/2	81	424	

The analysis results represent the probable response of the piles under short-term loading conditions and include no factor-of-safety. Suitable factors-of-safety should be selected on the basis of the type of loading. Pile stiffnesses (EI) of 1.9×10^{10} lb-in² and 5.9×10^{10} lb-in² have been assumed in our calculations of load deflection for the 18- and 24-inch piles, respectively. We assumed a minimum compressive strength of 4,000 pounds per square inch for concrete modulus calculations. If pile stiffness varies by no more than 20 percent than that reported above, load deflection characteristics can be approximated by multiplying the deflection values by the ratio of the pile stiffness (EI). We should evaluate the response of piles with significantly different stiffness.

The above lateral load characteristics are for single piles and may not be characteristic of the lateral load capacity of piles in a group. Group effects may reduce the allowable lateral load for a given deflection. We recommend that a pile group efficiency of 0.75 be used for pile groups 3-by-3 or smaller. A group reduction would not be necessary for groups of 1 or 2 piles. For pile groups larger than 3-by-3, we recommend that we review the final pile group layout and structural loads to further evaluate the pile group efficiency under lateral loading.

7.9.3 Passive Resistance Against Pile Caps and Grade Beams

If desired, the passive resistance of soil against pile caps and grade beams poured neat against well-compacted engineered fill may be used for lateral resistance. We recommend that an allowable passive pressure based on an equivalent fluid pressure of 300 pounds per cubic foot be used in design.

7.9.4 Pile Load Tests

Load testing for augercast pile foundations typically consists of performing one or more full scale static load tests. Static load tests include installing a test pile, with four surrounding anchor piles supporting a load frame to resist jacking against the test pile. The test pile may or may not be installed in a production pile location. During installation of the test piles, the contractor should allow for monitoring pile displacement at the top of pile, 10 feet below top, middle, and pile tip. Monitoring can be by strain gauges or capped conduits placed in the pile, allowing telltales to be placed during testing. This will allow for observation of the loads at which the skin friction is mobilized. A more detailed description of static load tests is presented in ASTM D1143. A member of our staff should be present during installation of the test piles and load testing and have the opportunity to review the test results.

7.10 Garage Floor Slab

The floor slab for the garage should be designed to structurally span between pile caps and grade beams to minimize the impact of ground settlement and hydrostatic uplift if piles are used.

7.11 Differential Settlement for Utility Tie-ins

The utilities entering the structure could experience differential settlement specifically at the tie-in locations during a seismic event. We recommend emergency shut-off valves and flexible utility and piping connections that could accommodate at least two inches of vertical and horizontal movement.

8.0 BASEMENT WALLS

8.1 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.

8.2 Seismic Lateral Earth Pressures

We understand the basement walls may be designed for seismic lateral loading. For our analysis, we have assumed that the walls will have flat, non-sloping backfill. We used the Mononobe-Okabe approach to approximate the increased earth pressures induced by earthquakes. As discussed in Section 3.2 of our report, a peak ground acceleration of 0.5g is expected at the site. We performed calculations using this ground acceleration, and estimated an additional seismic increment of $6H^2$ for fixed walls. This seismic increment is a resultant applied to the wall in addition to the static lateral earth pressures given in Section 8.1. For fixed walls the additional seismic load would be applied as a uniform pressure with the resultant applied at mid-height.

8.3 Drainage

The basement walls should be designed to withstand hydrostatic pressures below a depth of 12 feet below existing site grades. Passive wall drainage should be provided above the design ground water of 12 feet below existing site grades. The passive wall drainage system should consist of a 4-inch minimum diameter perforated pipe placed at 12 feet below the existing site grades (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 12 inches out from the wall and to within 2 feet of outside finished grade. Alternatively, $\frac{1}{2}$ -inch to $\frac{3}{4}$ -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.

We recommend that design details for draining the basement walls above the design ground water level be determined prior to completion of construction documents as this is often a critical feature. A sump will likely be needed for drainage at this elevation unless storm drains are at an elevation that would accept the water by gravity. A suitable prefabricated drainage system designed for this specific use, such as 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 12 feet below existing site grades, or to some other closed or through-wall system. Miradrain panels should terminate 18 to 24 inches from final exterior grade. The prefabricated drainage system should be installed against the wall (if excavation is laid back) or shoring system and should be installed in at least 4-foot-wide vertical strips at 8 feet on-center around the basement walls. 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.

We recommend that design details for draining the basement walls above the design ground water level be determined prior to completion of construction documents as this is often a critical feature. A sump will likely be needed for drainage at this elevation unless storm drains are at an elevation that would accept the water by gravity. A horizontal collection system external to the basement should drain to a sump system. Waterproofing should be installed between the drainage system and the basement walls. The project structural engineer should review and approve any notch or penetrations planned in basement walls.

8.4 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.

8.5 Foundation

Basement walls may be supported on the mat or deep foundation designed in accordance with the recommendations presented in the appropriate sections of this report. Lateral load resistance for the walls may be developed in accordance with the recommendations in the corresponding "Lateral Loads" sections.

9.0 PAVEMENTS

9.1 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 less than 11. We judge an R-value of 10 to be applicable for design based on a subgrade consisting of untreated native soils. Using estimated traffic indices for various pavement-loading requirements and untreated native soils, we developed the following recommended pavement sections based on Procedure 608 of the Caltrans Highway Design Manual, presented in Table 13.

**Table 13. Recommended Asphalt Concrete Pavement Design Alternatives
Pavement Components
Design R-Value = 10**

General Traffic Condition	Design Traffic Index	Asphalt Concrete (Inches)	Aggregate Baserock* (Inches)	Total Thickness (Inches)
Automobile Parking	4.0	2.5	7.0	9.5
	4.5	2.5	8.5	11.0
Automobile Parking Channel	5.0	3.0	9.0	12.0
	5.5	3.0	11.0	14.0
Truck Access & Parking Areas	6.0	3.5	11.5	15.0
	6.5	4.0	13.0	17.0

*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. Because the native soils at the site are very highly expansive, some increased maintenance and reduction in pavement life should be expected. 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.

Because the full thickness of asphalt concrete is frequently not placed prior to construction traffic being allowed to use the streets (or parking lots), rutting and pavement failures can occur prior to project completion. To reduce this occurrence, we recommend that either the full design pavement section be placed prior to use by construction traffic, or a higher Traffic Index (TI) be specified where construction traffic will use the pavement.

In addition, it has been our experience that asphalt concrete pavements constructed over expansive soils and adjacent to non-irrigated open space areas may experience cracking parallel to the edge of the pavement. This is typically caused by seasonal shrinkage and swelling adjacent to non-irrigated edges of the pavement. The cracks typically occur within the first few years of construction, and are typically located within a few to several feet of the edge of the pavement. The cracks, if they occur, can be filled with a bituminous sealant. Otherwise, a moisture barrier would need to be installed to a depth of at least 24 inches to reduce the potential for shrinkage of the pavement subgrade soils.

9.2 Exterior Portland Cement Concrete (PCC) Pavements

Recommendations for exterior PCC pavements are presented below in Table 14. 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 14. 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 10 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.

9.3 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.

9.4 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.

9.5 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 6 inches of Class 2 aggregate base compacted to a minimum of 90 percent relative compaction in accordance with ASTM Test Method D1557, latest edition except for the native moderately expansive clays, which should be compacted according to the recommendations in the "Compaction" section. If sidewalks are subject to wheel loads, they should be designed in accordance with the "Exterior Portland Cement Concrete Pavements" section of this report.

We recommend that exterior slabs be isolated from adjacent foundations and that adequate construction and control joints be used in design of the concrete slabs to control cracking inherent in concrete construction.

10.0 LIMITATIONS

This report has been prepared for the sole use of Cypress Group, specifically for design of the proposed West Julian Street Residential 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 the boring and CPTs 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, November 2014.



APPROXIMATE SCALE (FEET)



VICINITY MAP

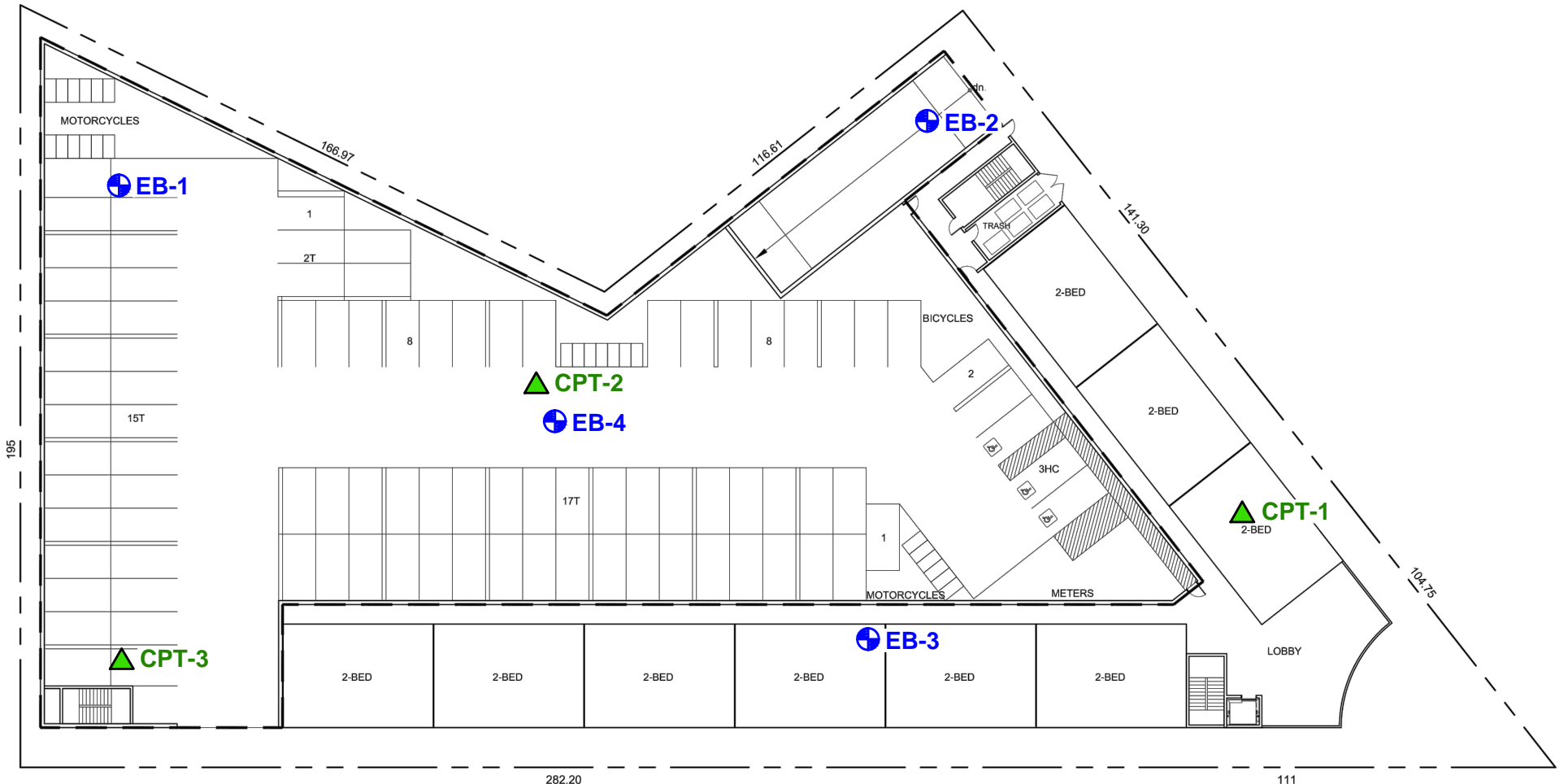
West Julian Street Residential Development
 715 West Julian Street
 San Jose, California



225871

FIGURE 1



FILE NAME: Z:\CAD_DRA\W\Current\West Julian St Res Dev_San Jose\Fig2_Site Plan.dwg | Layout.Tab: 8x11

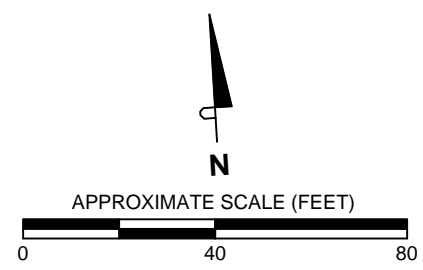


GROUND LEVEL

SOURCE: Option C Site Plan by Anderson Architects Inc., July 2014.

LEGEND

- Approximate locations of:
-  Exploratory boring
 -  Cone penetration test

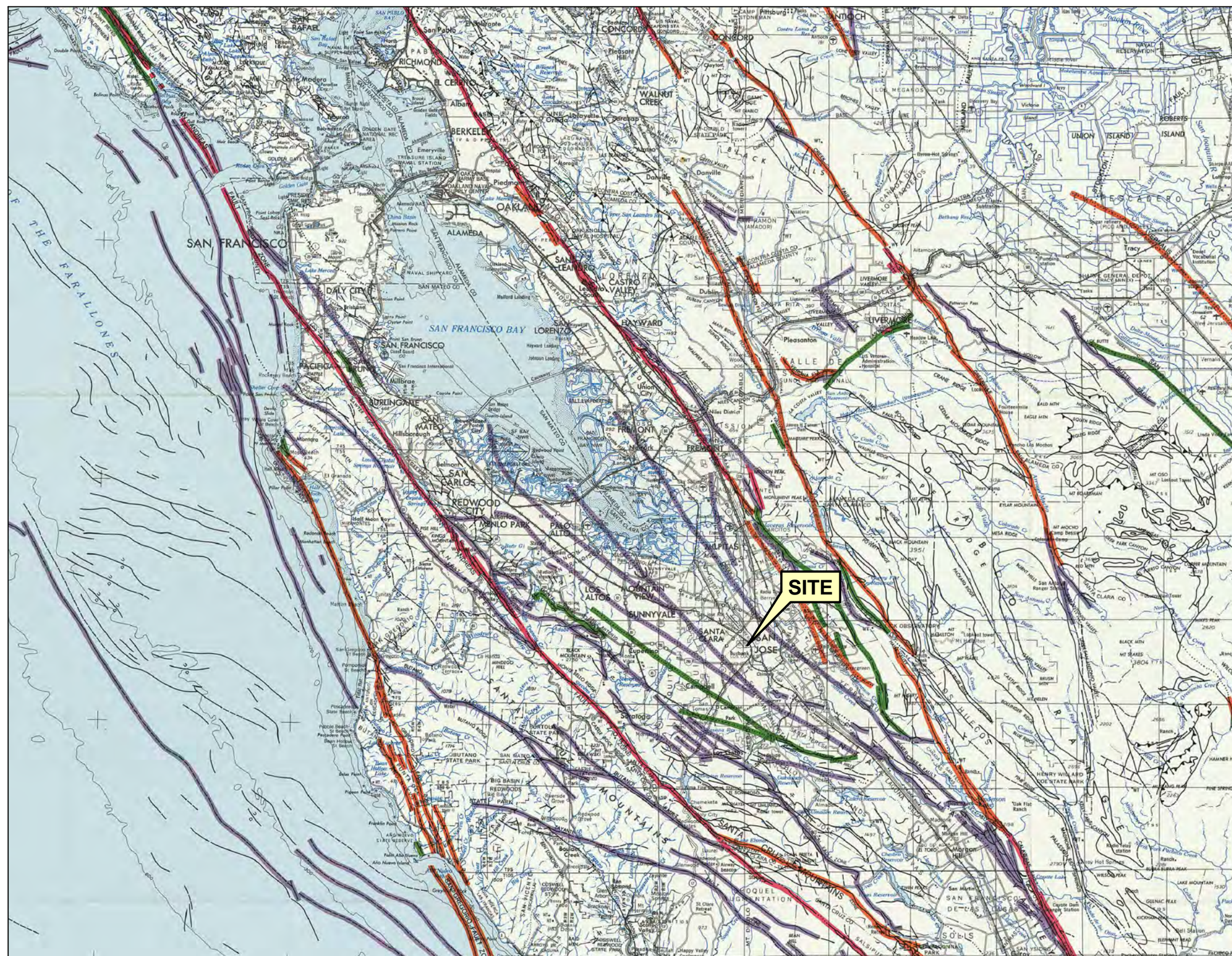


SITE PLAN

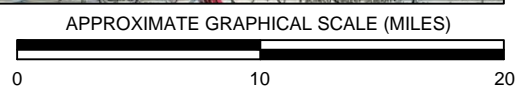
West Julian Street Residential Development
715 West Julian Street
San Jose, California

	225871	FIGURE 2
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FILE NAME: Z:\CAD_DRAW\Current\West Julian St. Res. Dev. San Jose\Fig3_Regional Fault Map.dwg | Layout: Tab: 11x17



SCALE: 1:500,000



Geologic Time Scale	Years Before Present (Approx.)	Fault Symbol	Recency of Movement on Land Offshore ¹	DESCRIPTION
Quaternary	Holocene/Historic			Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.
				Displacement during Holocene time ²
	Late Quaternary			Faults showing evidence of displacement during late Quaternary time ^{2,4}
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 Pliocene age.
Pre-Quaternary	Pliocene			
	Miocene			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)

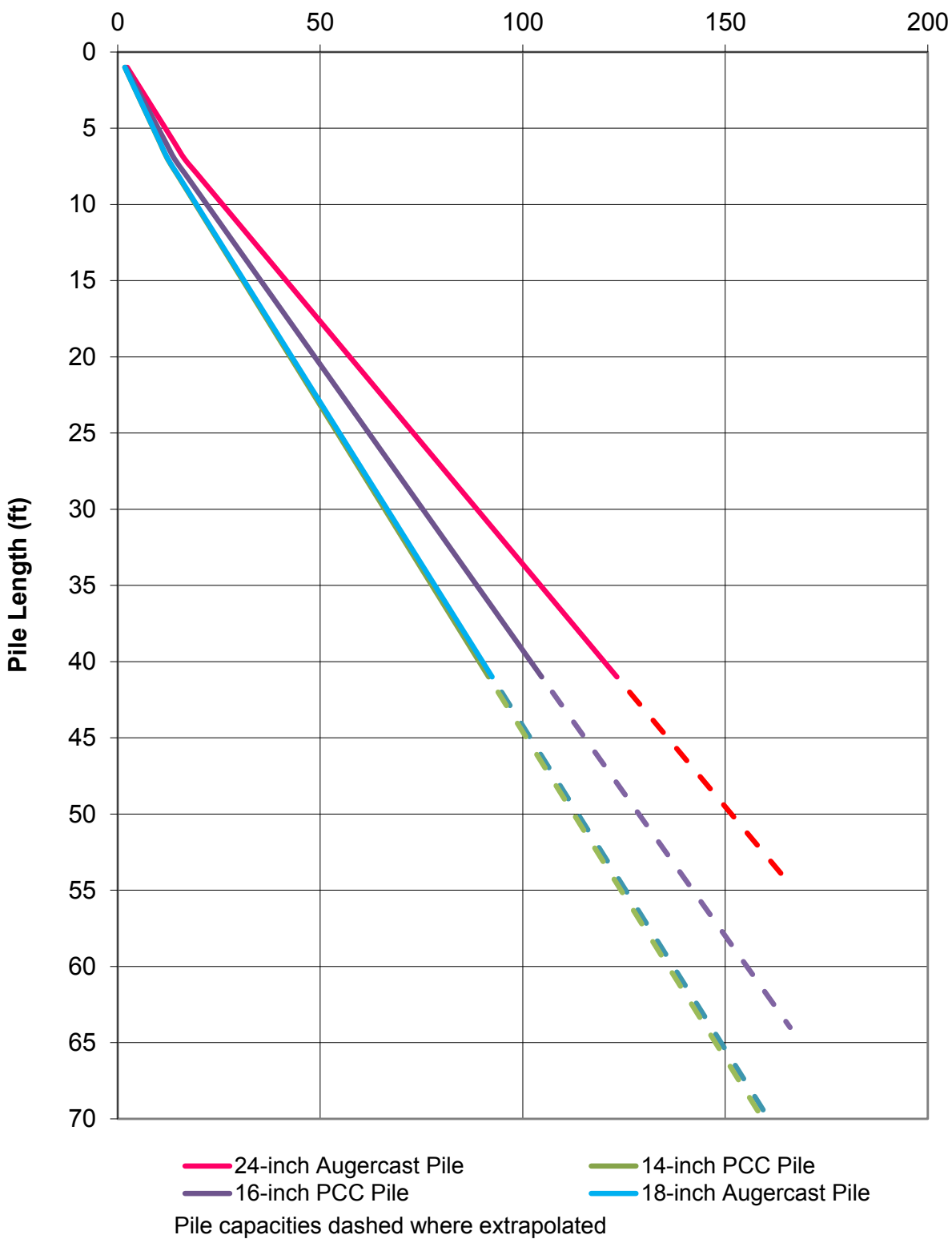
Some faults highlighted in purple are not considered active (Holocene Movement) by the State of California.

REGIONAL FAULT MAP

West Julian Street Residential Development
715 West Julian Street
San Jose, California

	225871	FIGURE 3
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Allowable Downward Vertical Capacity (Dead plus Live Load) (Kips)



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. Four 8-inch diameter exploratory borings were drilled on November 10 and 11, 2014 to a maximum depth of 50 feet. Three CPTs were advanced on November 11 and 12, 2014 to a maximum depth of 61 feet. The approximate locations of the exploratory borings and CPTs are shown on Figure 2. The soils encountered in the borings 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 and CPTs, as well as a key to the classification of the soil and CPTs, 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 boring were not determined. The locations of the boring should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the boring 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. 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 CPT logs 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 and CPT 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.

* * * * *

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)



EXPLORATORY BORING: EB-1

Sheet 1 of 2

DRILL RIG: MOBILE B-53
 BORING TYPE: 8 INCH HOLLOW STEM AUGER
 LOGGED BY: AC
 START DATE: 11-10-14 FINISH DATE: 11-10-14

PROJECT NO: 225871
 PROJECT: W. JULIAN RESIDENTIAL DEVELOPMENT
 LOCATION: SAN JOSE, CA
 COMPLETION DEPTH: 50.0 FT.

This log is a part of a report by TRC, 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)
	0		2" of AC over 12" of AB	AB/AC						
	0		CLAYEY SAND WITH GRAVEL (SC) medium dense, moist, gray brown, fine to coarse sand, fine sub-angular to sub-rounded gravel	SC	15	▲	21	100		
	0		LEAN CLAY (CL) hard moist, dark gray, moderate plasticity, Liquid Limit = 35, Plasticity Index = 17	CL	33	▲	17	109		
	0		LEAN CLAY WITH SAND (CL) hard, moist, gray, low to moderate plasticity, fine sand, trace fine sub-angular to sub-rounded gravel	CL	33	▲	15	115		
	0		brownish gray	CL	21	▲	21		77	
	0		LEAN CLAY (CL) medium stiff, moist, brown to dark brown, moderate plasticity	CL	22	▲	27	91		
	0		Liquid Limit = 31, Plasticity Index = 16							
	0		stiff, brownish gray	CL	25	▲	24	94		
	0		brown to dark brown	CL	29	▲	28	92		
	0			CL	23	▲	32	92		

Continued Next Page

GROUND WATER OBSERVATIONS:

▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 41.0 FEET

LA CORP.GDT 12/9/14 MV, CA*



EXPLORATORY BORING: EB-1 Cont'd

Sheet 2 of 2

DRILL RIG: MOBILE B-53

PROJECT NO: 225871

BORING TYPE: 8 INCH HOLLOW STEM AUGER

PROJECT: W. JULIAN RESIDENTIAL DEVELOPMENT

LOGGED BY: AC

LOCATION: SAN JOSE, CA

START DATE: 11-10-14

FINISH DATE: 11-10-14

COMPLETION DEPTH: 50.0 FT.

This log is a part of a report by TRC, 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		SANDY LEAN CLAY (CL) stiff, moist, light brown to light gray, low plasticity, fine sand	CL						
			LEAN CLAY (CL) stiff, moist, brown to bluish gray, moderate plasticity	CL						
			SANDY LEAN CLAY (CL) stiff, moist, brown to light gray, low to moderate plasticity, fine sand	SM	21	✕				
	35		SILTY SAND (SM) dense, moist, light reddish brown, fine sand	GP	50/6					
			POORLY GRADED GRAVEL (GP) very dense, moist, brown, fine to coarse sand, fine to coarse sub-rounded gravel	CL						
			LEAN CLAY (CL) soft to medium stiff, moist, dark brown, low plasticity		26	✕	22	104	31	
	40		CLAYEY SAND (SC) medium dense, moist, brown, fine to medium sand	SC						
			SANDY SILT (ML) medium stiff to very stiff, moist, light gray to brown, fine sand		24	✕	23	102		
	45			ML						
					29	✕	27	104	55	
	50		LEAN CLAY (CL) very stiff, moist, brown, low plasticity, trace fine sand Bottom of boring at 50 feet	CL						
	55									
	60									

GROUND WATER OBSERVATIONS:

▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 41.0 FEET

LA CORP.GDT 12/9/14 MV, CA*



EXPLORATORY BORING: EB-2

Sheet 1 of 2

DRILL RIG: MOBILE B-53
 BORING TYPE: 8 INCH HOLLOW STEM AUGER
 LOGGED BY: AC
 START DATE: 11-10-14 FINISH DATE: 11-10-14

PROJECT NO: 225871
 PROJECT: W. JULIAN RESIDENTIAL DEVELOPMENT
 LOCATION: SAN JOSE, CA
 COMPLETION DEPTH: 35.0 FT.

This log is a part of a report by TRC, 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)
	0		SURFACE ELEVATION:							
	0		2" of AC over 8.5" of AB	AC/AB						
	0		LEAN CLAY (CL) hard, moist, dark gray, moderate plasticity	CL	38	▲	16	88		○
	4		light grayish brown, low to moderate plasticity		29	▲	7	120		
	5		CLAYEY SAND (SC) medium dense, moist, light brown to gray, fine to coarse sand, trace fine sub-angular to sub-rounded gravel	SC	34	▲	5	112		
	6		POORLY GRADED SAND WITH GRAVEL (SP) medium dense, moist, brown, fine to coarse sand, fine to medium sub-angular to sub-rounded gravel	SP		▲				
	7		CLAYEY SAND (SC) medium dense, moist, brown, fine sand	SC	22	▲	31	89		○
	10		LEAN CLAY (CL) stiff, moist, brown, low plasticity, trace fine sand	CL	18	▲	34	85		○
	15		medium stiff, mottled dark brown and gray							
	18		SANDY LEAN CLAY (CL) medium stiff to stiff, moist, brown, low plasticity, fine sand, Liquid Limit =23 , Plasticity Index = 4	CL	23	▲	25	98	73	○
	20									
	25		dark brown to gray, moderate plasticity	CL	28	▲	30	93		○
	30			SP-SC	24	▲	35	90		○

Continued Next Page

GROUND WATER OBSERVATIONS:

▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 31.5 FEET

LA CORP.GDT 12/9/14 MV, CA*



EXPLORATORY BORING: EB-2 Cont'd

DRILL RIG: MOBILE B-53
 BORING TYPE: 8 INCH HOLLOW STEM AUGER
 LOGGED BY: AC
 START DATE: 11-10-14 FINISH DATE: 11-10-14

PROJECT NO: 225871
 PROJECT: W. JULIAN RESIDENTIAL DEVELOPMENT
 LOCATION: SAN JOSE, CA
 COMPLETION DEPTH: 35.0 FT.

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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)
	30		POORLY GRADED SAND WITH CLAY (SP-SC) medium dense, moist, brown, fine sand	CH						
			FAT CLAY (CH) very stiff, moist, dark gray, moderate to high plasticity							
			SANDY LEAN CLAY (CL) stiff, moist, brown to light gray, low plasticity, fine sand	CL						
	35		CLAYEY SAND (CH) medium dense, wet, brown, fine to coarse sand, fine to coarse sand	SC CL	16		23			
			SANDY LEAN CLAY (CL) stiff, moist, brown to light gray, low plasticity, fine sand							
			Bottom of boring at 35 feet							
	40									
	45									
	50									
	55									
	60									

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

GROUND WATER OBSERVATIONS:

▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 31.5 FEET

LA CORP.GDT 12/9/14 MV, CA*



EXPLORATORY BORING: EB-3

DRILL RIG: MOBILE B-53
 BORING TYPE: 8 INCH HOLLOW STEM AUGER
 LOGGED BY: AC
 START DATE: 11-11-14 FINISH DATE: 11-11-14

PROJECT NO: 225871
 PROJECT: W. JULIAN RESIDENTIAL DEVELOPMENT
 LOCATION: SAN JOSE, CA
 COMPLETION DEPTH: 35.0 FT.

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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)
	0		SURFACE ELEVATION:							
	0	8" of Crushed Rock								
	0	SANDY LEAN CLAY (CL)	hard, moist, dark gray, low plasticity, fine to coarse sand, trace rootlets	CL	33 50/6"	X	11	106		○
	0	POORLY GRADED GRAVEL WITH CLAY (GP-GC)	medium dense, moist, light brown, fine to coarse sub-angular to sub-rounded gravel	GP-GC	37	X	7	121		
	5	POORLY GRADED SAND WITH GRAVEL (SP)	medium dense, moist, brown, fine to coarse sand, fine sub-angular to sub-rounded gravel	SP	27	X	3	111		
	5	SILTY SAND (SM)	medium dense, moist, light reddish brown, fine sand	SM	23	X	14	98		
	10	LEAN CLAY (CL)	stiff, moist, brown, moderate placticity	CL	38	X	29	90		○
	15									
	20									
	25									
	30									

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

Continued Next Page

GROUND WATER OBSERVATIONS:

▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 29.0 FEET

LA CORP.GDT 12/9/14 MV, CA*



EXPLORATORY BORING: EB-3 Cont'd

Sheet 2 of 2

DRILL RIG: MOBILE B-53
 BORING TYPE: 8 INCH HOLLOW STEM AUGER
 LOGGED BY: AC
 START DATE: 11-11-14 FINISH DATE: 11-11-14

PROJECT NO: 225871
 PROJECT: W. JULIAN RESIDENTIAL DEVELOPMENT
 LOCATION: SAN JOSE, CA
 COMPLETION DEPTH: 35.0 FT.

This log is a part of a report by TRC, 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)
	30		LEAN CLAY (CL) stiff, moist, brown, moderate placticity	CL						○ Pocket Penetrometer △ Torvane ● Unconfined Compression ▲ U-U Triaxial Compression 1.0 2.0 3.0 4.0
	35		Bottom of boring at 35 feet							
	40									
	45									
	50									
	55									
	60									

GROUND WATER OBSERVATIONS:

▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 29.0 FEET

LA CORP.GDT 12/9/14 MV, CA*



EXPLORATORY BORING: EB-4

Sheet 1 of 2

DRILL RIG: MOBILE B-53
 BORING TYPE: 8 INCH HOLLOW STEM AUGER
 LOGGED BY: AC
 START DATE: 11-11-14 FINISH DATE: 11-11-14

PROJECT NO: 225871
 PROJECT: W. JULIAN RESIDENTIAL DEVELOPMENT
 LOCATION: SAN JOSE, CA
 COMPLETION DEPTH: 35.0 FT.

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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)				
			SURFACE ELEVATION:							○ Pocket Penetrometer	△ Torvane	● Unconfined Compression	▲ U-U Triaxial Compression	1.0 2.0 3.0 4.0
	0		2" of AC over 9.5" of AB	AC/AB										
	0		CLAYEY GRAVEL (GC) medium dense, moist, gray to brown, fine to coarse sand, fine sub-angular to sub-rounded gravel	GC	42		11	111						
	0		LEAN CLAY (CH) hard, moist, dark gray, moderate plasticity, trace fine sand	CL	50/6"		16	93						
	5		LEAN CLAY WITH SAND (CL) hard, moist, gray to brown, moderate plasticity, fine sand	CL	43		9	117						
	5		SANDY LEAN CLAY (CL) hard, moist, light gray to light brown, low plasticity, fine sand	CL										
	10		LEAN CLAY (CL) hard, moist, light brown, low plasticity	CL	45		8	107						
	10		trace fine sand	CL										
	15		LEAN CLAY WITH SAND (CL) hard, moist, light brown, low plasticity, fine sand	CL	43		17	105						
	20			CL										
	25			CL										
	30			CL										

Continued Next Page

GROUND WATER OBSERVATIONS:
 NO FREE GROUND WATER ENCOUNTERED

LA CORP.GDT 12/9/14 MV, CA*



EXPLORATORY BORING: EB-4 Cont'd

Sheet 2 of 2

DRILL RIG: MOBILE B-53
 BORING TYPE: 8 INCH HOLLOW STEM AUGER
 LOGGED BY: AC
 START DATE: 11-11-14 FINISH DATE: 11-11-14

PROJECT NO: 225871
 PROJECT: W. JULIAN RESIDENTIAL DEVELOPMENT
 LOCATION: SAN JOSE, CA
 COMPLETION DEPTH: 35.0 FT.

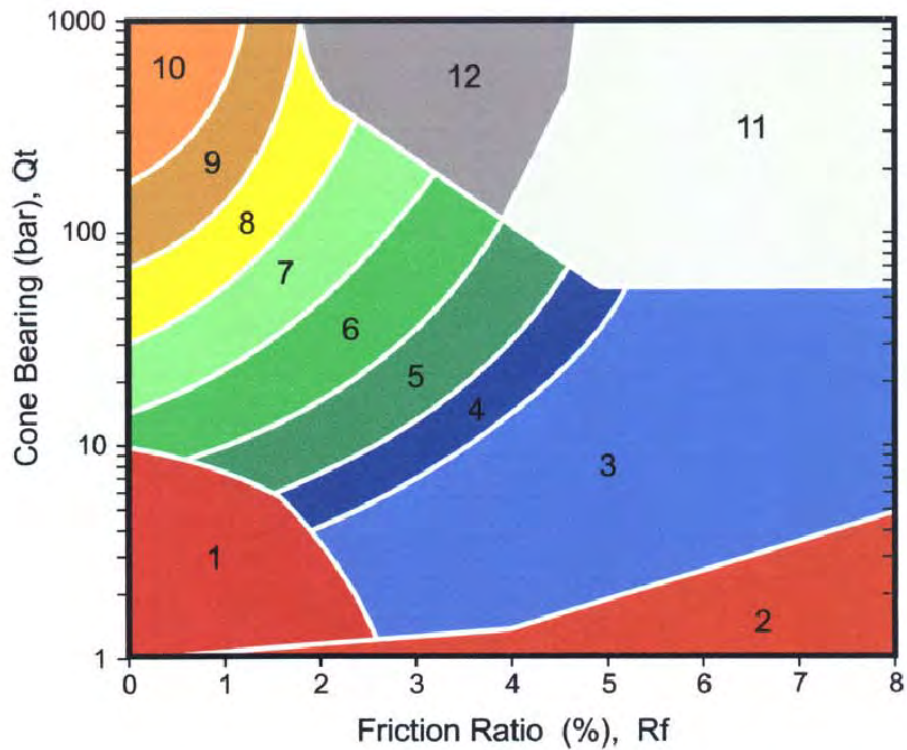
This log is a part of a report by TRC, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is 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
	30	[Hatched Box]	LEAN CLAY WITH SAND (CL) hard, moist, light brown, low plasticity, fine sand	CL						1.0 2.0 3.0 4.0
	35		Bottom of boring at 35 feet							
	40									
	45									
	50									
	55									
	60									

GROUND WATER OBSERVATIONS:
 NO FREE GROUND WATER ENCOUNTERED

LA CORP.GDT 12/9/14 MV, CA*



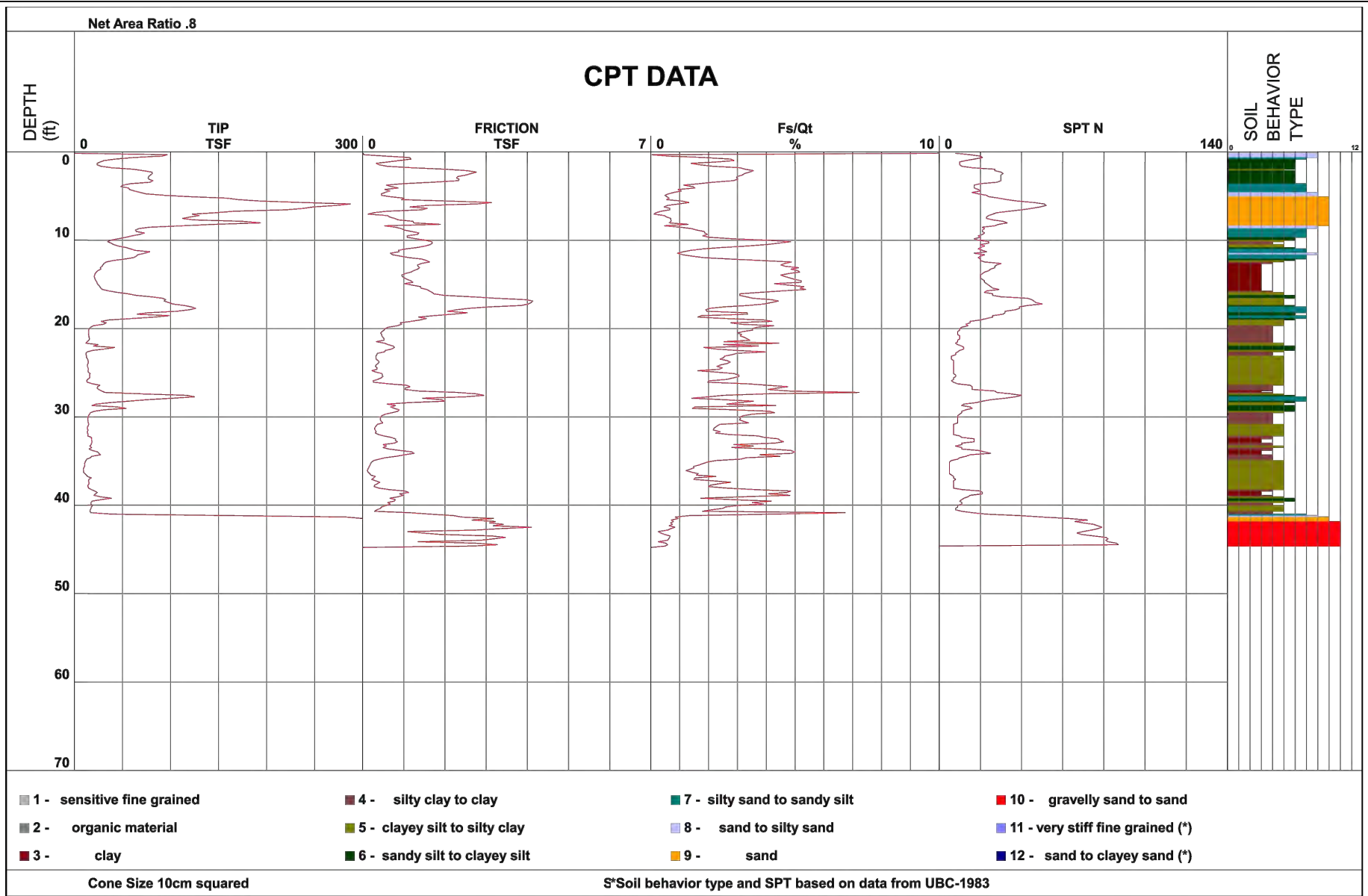


Zone	Q_t / N	Soil Behaviour Type
1	2	sensitive fine grained
2	1	organic material
3	1	clay
4	1.5	silty clay to clay
5	2	clayey silt to silty clay
6	2.5	sandy silt to clayey silt
7	3	silty sand to sandy silt
8	4	sand to silty sand
9	5	sand
10	6	gravelly sand to sand
11	1	very stiff fine grained *
12	2	sand to clayey sand *

* overconsolidated or cemented

Robertson (1990)

KEY TO CONE PENETROMETER TEST

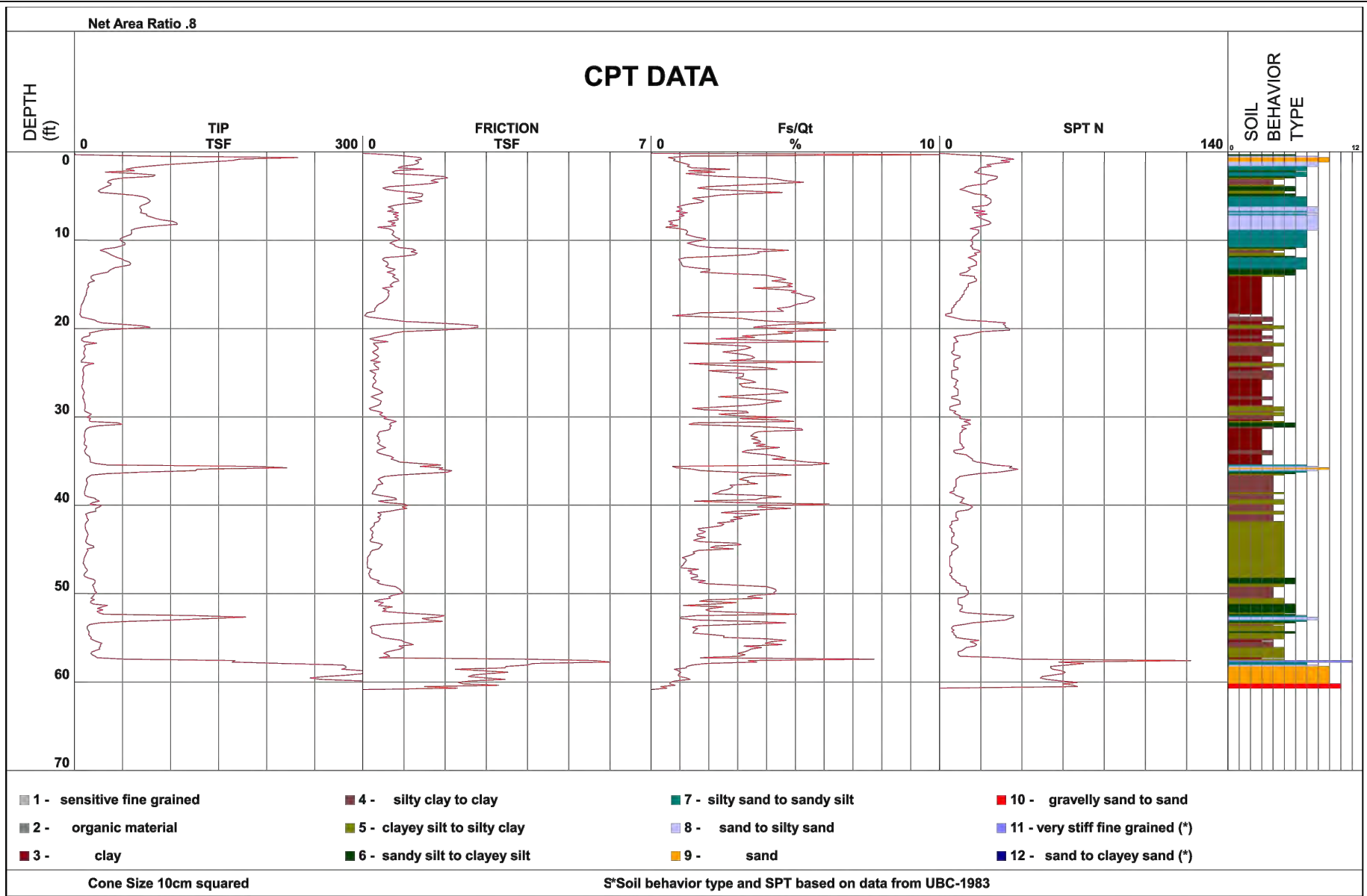


CPT-1

West Julian Street Residential Development
715 West Julian Street
San Jose, California

	225871	CPT-1
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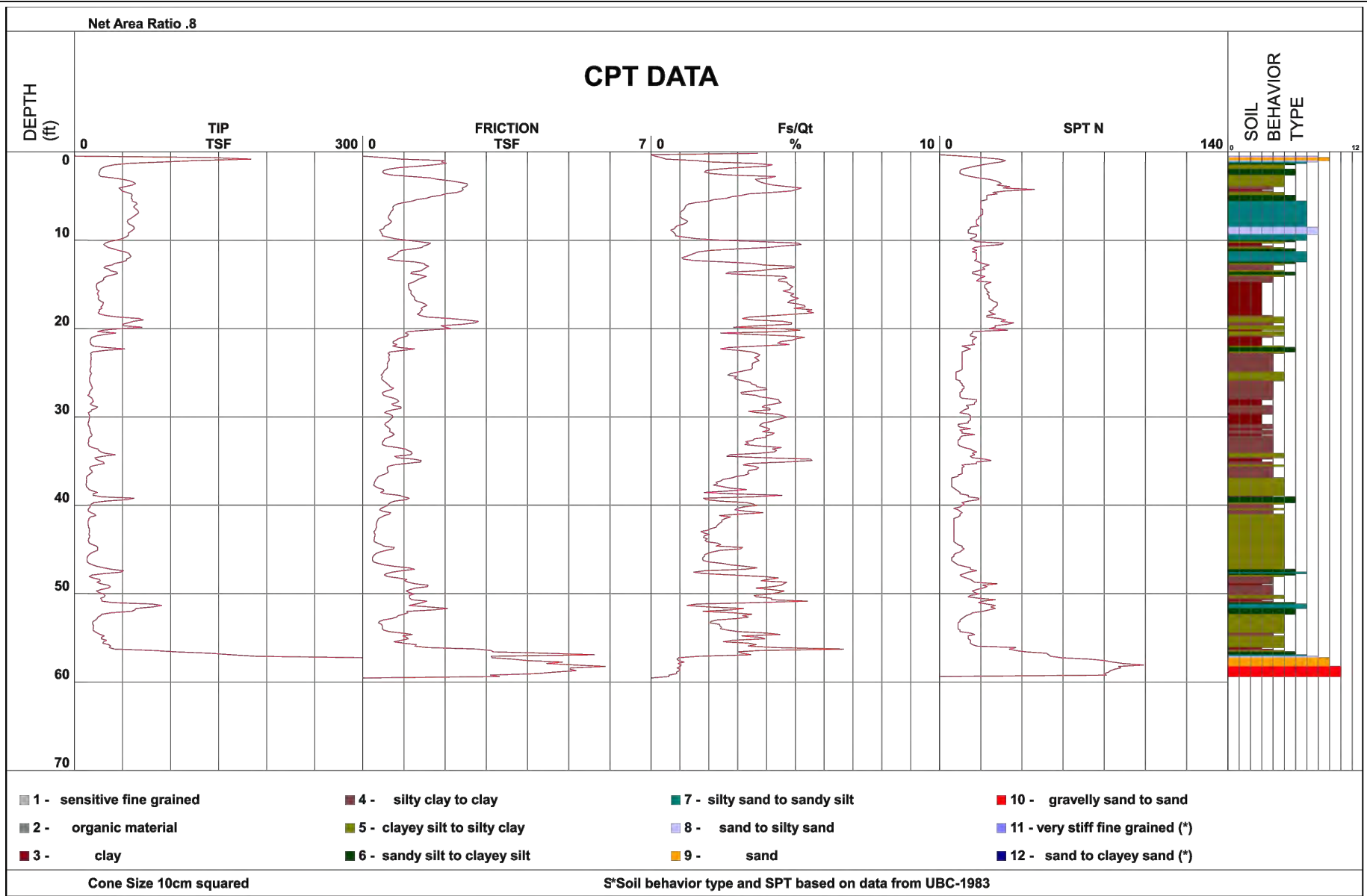


CPT-2

West Julian Street Residential Development
715 West Julian Street
San Jose, California

	225871	CPT-2
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FILE NAME: Z:\CAD_DRAWING\CURRENT\West Julian St Res Dev_San Jose\CPT-3.dwg | Layout Tab: 8x11



CPT-3

West Julian Street Residential Development
715 West Julian Street
San Jose, California

	225871	CPT-3
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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 30 samples of the materials recovered from the boring. These water contents are recorded on the boring log at the appropriate sample depths.

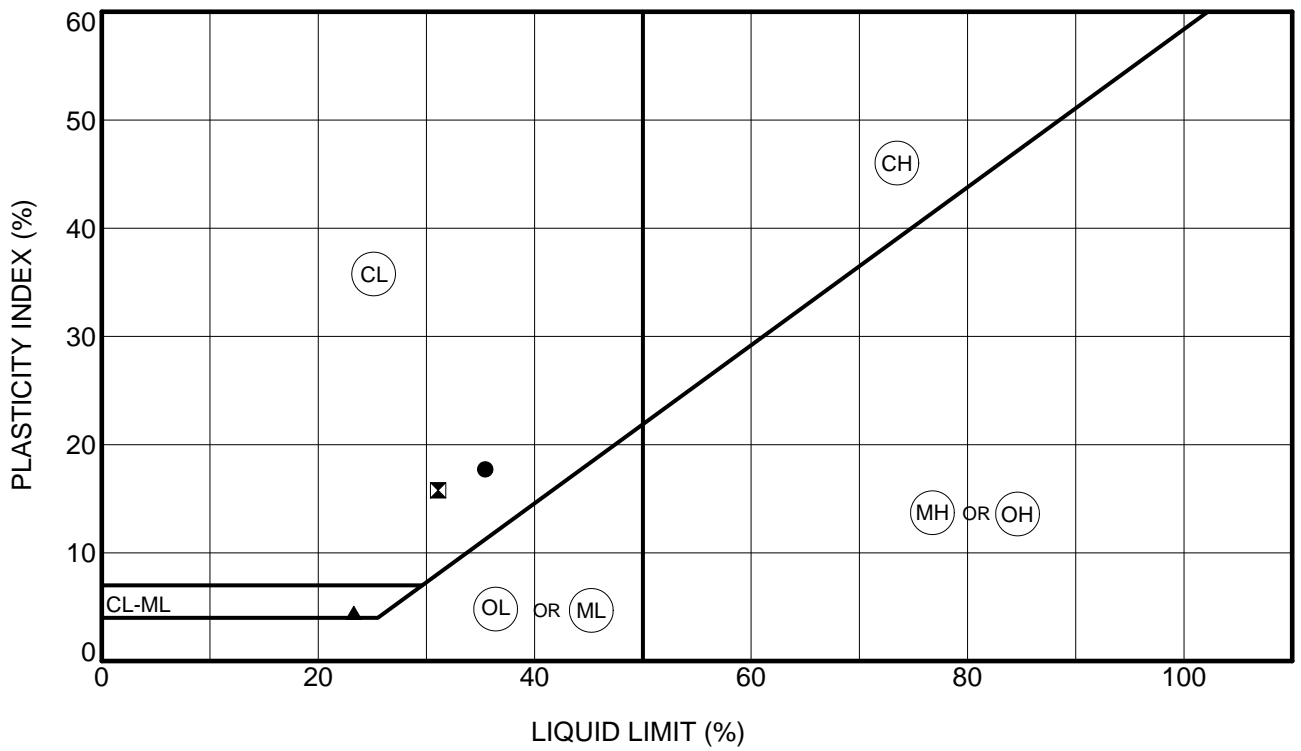
Dry Densities: In place dry density tests (ASTM D2937) were performed on 28 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring log at the appropriate sample depths.

Plasticity Index: Three Plasticity Index (PI) test determinations (ASTM D4318) were performed on samples of the subsurface soils to measure the range of water contents over which these material exhibits 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 logs of the boring at the appropriate sample depths.

Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was performed on four 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.

R-Value: An R-value resistance test (California Test Method No. 301) was performed on a representative sample of the surface soils at the site to provide data for the pavement design. The test indicated an R-value of 10 at an exudation pressure of 300 pounds per square inch. The results of the test are presented in this appendix.

* * * * *



Symbol	Boring No.	Depth (ft.)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Passing No. 200 Sieve	Unified Soil Classification Description
●	EB-1	2.0		35	18	17		DARK BROWN LEAN CLAY
☒	EB-1	14.5		31	15	16		BROWN LEAN CLAY
▲	EB-2	19.5		23	19	4		BROWN SANDY LEAN CLAY

LA CORP.GDT_12/3/14.MV.CA*



PLASTICITY CHART AND DATA

Project: W. JULIAN RESIDENTIAL DEVELOPMENT
 Location: SAN JOSE, CA
 Project No.: 225871

FIGURE B-1



R-value Test Report (Caltrans 301)

Job No.: 028-2394	Date: 11/20/14	Initial Moisture, <u>12.9%</u>
Client: TRC	Tested MD	R-value by Stabilometer 11
Project: W. Julian Street - 225871	Reduced RU	Expansion Pressure 60 psf
Sample EB-2;Bucket @ 1-5'	Checked DC	
Soil Type: Dark Olive Gray Clayey SAND w/ Gravel		

Specimen Number	A	B	C	D	Remarks:
Exudation Pressure, psi	214	658	340		
Prepared Weight, grams	1200	1200	1200		
Final Water Added, grams/cc	35	0	14		
Weight of Soil & Mold, grams	3196	3166	3141		
Weight of Mold, grams	2114	2102	2099		
Height After Compaction, in.	2.54	2.34	2.39		
Moisture Content, %	16.2	12.9	14.2		
Dry Density, pcf	111.0	122.0	115.6		
Expansion Pressure, psf	0.0	206.4	77.4		
Stabilometer @ 1000					
Stabilometer @ 2000	144	66	130		
Turns Displacement	3.63	3.45	3.5		
R-value	7	46	13		

