Appendix D

Geotechnical Report

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September 16, 2016
261765

Mr. Dean Hanson
MONTGOMERY 7, LLC
120 West Campbell Avenue, Suite D
Campbell, California 95008

RE: GEOTECHNICAL INVESTIGATION
MONTGOMERY 7 MIXED USE DEVELOPMENT
565 LORRAINE AVENUE
SAN JOSE, CALIFORNIA

Dear Mr. Hanson:

We are pleased to present the results of our geotechnical investigation for the above referenced project. Our report includes a description of the geotechnical and seismic aspects of the site along with our conclusions and geotechnical recommendations for design of Montgomery 7 Mixed Use Development.

We refer you to the text of this report for detailed recommendations. If you have any questions concerning our findings, please call us and we will be glad to discuss them with you.

Very truly yours,

TRC

Scott M. Leck, P.E., G.E.
Principal Geotechnical Engineer

SML:AC

Copies: Addressee (2 and email)

Geotechnical Investigation

Montgomery 7 Mixed Use Development
San Jose, California

Report No. 261765 has been prepared for:

MONTGOMERY 7, LLC
120 West Campbell Avenue, Suite D, Campbell, California 95008

September 19, 2016

Alberto Cortez
Senior Staff Engineer

Scott M. Leck, P.E., G.E.
Principal Geotechnical Engineer
Quality Assurance Reviewer

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GEOTECHNICAL INVESTIGATION
MONTGOMERY 7 MIXED USE DEVELOPMENT
SAN JOSE, CALIFORNIA

1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed Montgomery 7 Mixed Use 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 site plans titled, "Montgomery 7, Proposed Residential and Retail Development, 565 Lorraine Avenue, San Jose, California," prepared by Anderson Architects, Inc. dated May 3, 2016.

1.1 Project Description

As currently planned, the project is proposed to consist of the construction of an approximately 3,750 square foot, 10 story podium structure built over a below-grade community and fitness area. The ground level of the structure will consist of retail and the lobby for the studio residences above. The approximately 0.1-acre site is bounded by South Montgomery Street to the northwest, Lorraine Avenue to the south and an existing residence to the east. The site is currently occupied by a single family residence. Additional improvements will include pavements, underground utilities, and landscaping. The layout of the proposed development is shown on the Site Plan, Figure 2.

Based on information provided by the structural engineer, the mat foundation dead load pressure will be approximately 1,400 psf, and the live load pressure will be approximately 400 psf.

The project site is in a zone of required investigation for liquefaction hazard as mapped by the California Geological Survey (CGS). 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 July 21, 2016. To accomplish this work, we provided the following services:

- Exploration of subsurface conditions by drilling one boring in the area of the proposed development and retrieving soil samples for observation and laboratory testing. We also advanced two 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 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 September 2, 2016. At the time of the reconnaissance, the site was occupied by an existing single family residence, concrete driveway, and landscaping. The site appeared relatively flat with minor grade variation for drainage purposes.

2.2 Exploration Program

Subsurface exploration was performed on September 2, 2016 using CPT equipment to investigate subsurface soils. Two CPts were advanced and encountered refusal at depths of approximately 62½ to 66½ feet. Subsurface exploration was also performed on September 6, 2016 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 approximately 124 feet below the existing ground surface.

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

2.3 Subsurface Conditions

Our two CPTs were performed within the footprint of the proposed structure. In general, soils encountered in CPT-1 and CPT-2 were interpreted to include interbedded layers of clay, silty clay, clayey silt, sandy silt, silty sand, sand, and gravelly sand to the maximum explored refusal depths of 62½ and 66½ feet, respectively. The sand layers ranged in thickness from approximately ½-foot to 7 feet. Refusal was encountered in the dense sand with approximate tip pressures greater than 450 tons per square foot.

Our boring was performed within the footprint of the proposed structure. Fill consisting of hard sandy silty clay was encountered to a depth of approximately 4½ feet. Below the fill, our boring encountered interbedded layers consisting of loose silty sand, and loose to medium dense poorly graded sand to a depth of about 17 feet. Below the depth of 17 feet, dense poorly graded gravel was encountered to a depth of about 22 feet, followed by medium stiff to stiff sandy lean clay to a depth of approximately 32 feet. Below the depth of 32 feet, interbedded layers consisting of dense sitly sand, dense poorly graded sand, and dense to very dense poorly graded gravel were encountered to a depth of about 47 feet, followed by stiff to very stiff lean clay and stiff sandy silt to a depth of 59 feet. Below the depth of 59 feet, our boring generally encountered interbedded layers consisting of stiff to very stiff lean clay, stiff sandy lean clay, dense to very dense clayey gravel, dense to very dense clayey sand, and very dense poorly graded gravel to the maximum explored depth of 124 feet.

Four Plasticity Index (PI) tests were performed to determine the PI of the representative clay soil samples collected from boring EB-1 at depths of approximately 2, 24½, 29½, and 49½ feet, respectively. The tests resulted in PI's ranging from 7 to 17 indicating low to moderate plasticity and expansion potential of the near surface soils.

2.4 Ground Water

Free ground water was encountered during subsurface exploration in borings EB-1 at a depth of approximately 36½ feet below grade. Based on pore pressure dissipation measurements, CPT-1 encountered groundwater at a depth of approximately 28 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 is on the order of 20 feet below the existing ground surface (bgs). Based on the above information, we judged a ground water depth of 20 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, 2012). 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	PGA_M	0.50	Equation 11.8-1 of ASCE 7-10
CGS Seismic Hazard Zone Report 058, Figure 3.5	Probabilistic 10% in 50 years	0.51	
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	0.45	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, 2014) estimates there is a 72 percent chance of at least one magnitude 6.7 earthquake occurring in the San Francisco Bay region between 2014 and 2044. This result is an important outcome of WGCEP's work because any major earthquake can cause damage throughout the region. The 1989 Loma Prieta earthquake demonstrated this potential by causing severe damage in Oakland and San Francisco, more than 50 miles from the fault epicenter.

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

3.4 Liquefaction

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 (2012). During cyclic ground shaking, such as earthquakes, cyclically-induced stresses may cause increased pore water pressures within the soil matrix, which results in liquefaction. Liquefied soil may lose shear strength that may lead to large shear deformations and/or flow failure (Youd et al., 2001). Liquefied soil can also settle as pore pressures dissipate following an earthquake. Limited field data is available on this subject; however, settlement on the order of 2 to 3 percent of the thickness of the liquefied zone has been measured in some cases.

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

3.4.2 Analysis and Results

Based on our explorations and the depth to historic high ground water map prepared by the CGS, a design ground water level at 20 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 6.59 (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)	I_c	*q_{c1N} (tsf)	Factor of Safety	Potential for Liquefaction	Estimated Total Settlement (in.)
CPT-1	20.3	0.2	2.5	89.9	0.4	Likely	0.1
	30.7	0.5	2.5	92.1	0.3	Likely	0.2
	31.8	0.7	2.2	119.9	0.5	Likely	0.2
	36.4	0.3	2.3	133.7	0.6	Likely	0.1
	38.9	0.8	2.3	98.5	0.3	Likely	0.2
	42.8	0.3	2.3	143.9	0.7	Likely	0.1
	43.6	0.5	2.0	146.7	0.8	Likely	0.1
	47.6	0.5	2.0	133.1	0.6	Likely	0.1
	54.6**	0.8	2.4	99.8	0.4	Likely	0.2
	56.4**	1.6	2.4	110.3	0.4	Likely	0.4
	58.7**	1.0	2.4	123.1	0.6	Likely	0.2
	60.0**	1.5	2.4	132.6	0.7	Likely	0.3
						Total =	2.2
CPT-2	20.0	1.0	2.4	90.2	0.4	Likely	0.3
	27.1	0.8	2.5	121.9	0.6	Likely	0.2
	32.3	0.8	2.1	129.8	0.7	Likely	0.2
	33.6	0.5	1.7	147.3	0.8	Likely	0.1
	37.4	1.5	2.3	108.3	0.4	Likely	0.4
	47.1	0.3	2.0	123.8	0.5	Likely	0.1
	51.4**	0.3	2.1	137.5	0.7	Likely	0.1
	55.1**	0.8	2.1	129.2	0.6	Likely	0.2
	59.4**	0.7	2.1	142.3	0.9	Likely	0.1
						Total =	1.7

* CPT tip pressure corrected for overburden and fines content

** Settlements below depths of 50 feet included for deep foundation evaluation, not included as part of total or differential settlements at ground surface for shallow foundations

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 mat foundation recommendations.

Our analyses indicate that several sand and silt layers below the design ground water depth may theoretically liquefy, resulting in approximately 1 to 1¼ inches 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 ¼-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 one representative soil sample collected from boring EB-1 at depths of approximately 24½, 29½, and 49½ feet. 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-1	24½	Sandy Lean Clay	34	15	25	73
	29½	Sandy Lean Clay	40	17	30	75
	49½	Lean Clay	35	17	26	74

Based on the results of the screening, it appears that the lean clay represented in the sample from boring EB-1 at depth of 24½, 29½, and 49½ feet are not susceptible to liquefaction based on the Bray and Sancio criteria as the WC is less than 80 percent of the LL.

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 would be approximately 12 feet below the ground surface. There would be approximately 8 feet of non-liquefiable material overlying the relatively thin (approximately 1 foot) potentially liquefiable strata at the site below the bottom of the foundation. Based on the work by Youd and Garris (1995), there is an adequate non-liquefiable material capping the shallow liquefiable layer at the site. Therefore, we judge the potential for ground rupture 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.

We performed dry sand settlement calculations following Tokimatsu and Seed method (1987). Our calculations indicated that the medium dense sand encountered in the boring may densify and settle in the order of less than ¼-inch. 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 Los Gatos Creek runs under the intersection of Montgomery Street and Park Avenue and is located along the northwest side adjacent to the site. The potentially liquefiable layers at the site are thin, relatively deep and likely 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.

3.7 Flooding

The area of the proposed development has been mapped by the Federal Emergency Management Agency (2009) as an area designated as Zone A for the northwest corner of the site, and as a Zone D for the rest of the site. Zone A is defined as a “special flood hazard areas subject to inundation by the 1% annual chance flood; no base flood elevations determined,” and Zone D as a “areas in which flood hazards are undetermined, but possible.”

4.0 CORROSION EVALUATION

To evaluate the corrosion potential of the subsurface soils at the site, we submitted one sample collected during our subsurface investigation to an analytical laboratory for pH, resistivity, soluble sulfate and chloride content testing. The result of this test is 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	9.0	21	87	7.6	3,050	Moderate	Negligible

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

Many factors can affect the corrosion potential of soil including soil moisture content, resistivity, permeability and pH, as well as chloride and sulfate concentration. In general, soil resistivity, which is a measure of how easily electrical current flows through soils, is the most influential factor. Based on 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 result was 3,050 ohm-centimeters. Based on this result and the resistivity correlations presented in Table 5, the corrosion potential to buried metallic improvements may be characterized as moderately 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 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:

- Strong seismic shaking
- The potential for liquefaction-induced total and differential settlement
- Demolition of the existing buildings prior to site development
- Corrosion potential of the near-surface soils
- Basement excavation support
- 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 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.

5.1.2 Liquefaction-Induced Total and Differential Settlement

Our analyses indicate that several layers theoretically can liquefy, ranging from 1 to 1¼ inches of total settlement in the upper 50 feet, with differential settlements from liquefaction on the order of ¼-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.

5.1.3 Demolition Debris

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

5.1.4 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 moderately corrosive. A qualified corrosion engineer should be contacted to provide specific recommendations regarding corrosion protection for buried metal pipe or buried metal pipe-fittings.

5.1.5 Basement Excavation Support

The walls of the basement excavation may be supported by several methods including tiebacks, soldier beams and wood lagging or temporary slopes if space is adequate. The choice 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. Support of any adjacent existing structures without distress should also be the contractor's responsibility. We recommend that the contractor forward his plan for the support system to the structural engineer and geotechnical engineer for pre-construction review. In addition, it should be the contractor's responsibility to undertake a pre-construction survey with benchmarks and photographs of the adjacent properties.

5.1.6 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 movement.

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

As discussed above, fill was encountered in our boring to a depth of approximately 4½ feet below the existing grade. We understand the proposed structure consists of one-level below grade, therefore, we anticipate the entire fill will be removed. In general, 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

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.

Import fill material should be inorganic, have a PI of 20 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 15 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

Some asphalt concrete/aggregate base grindings may be generated during removal of any 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 and City of San Jose 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 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). Aggregate base and all import soils should be compacted at a moisture content near the laboratory optimum.

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 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 or governing jurisdiction. General fill to be used above pipe embedment materials should be placed and compacted in accordance with local requirements or the recommendations contained in this section, whichever is more stringent.

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

Utility trenches located adjacent to foundations 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 pavement areas. This would reduce the likelihood of water entering the trenches from the landscaped areas and seeping through the trench backfill into the improvement and pavement areas.

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 12 to 15 feet is planned to construct the one-level below-grade. 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 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 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.

6.13 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 preferably, 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.

6.14 Construction Observation

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

7.0 FOUNDATIONS

As discussed in the Conclusions and Recommendations section there is a potential for liquefaction induced settlement to occur. 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.

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 stiff to hard clays and loose to very dense sands and gravels, which corresponds to a soil profile type D. Based on the above information and local seismic sources, the site may be characterized for design using the information in Table 8 below.

Table 8. 2013 CBC Site Class and Site Seismic Coefficients

Latitude: 37.3259 N Longitude: 121.9004 W	CBC Reference	Factor/ Coefficient	Value
Soil Profile Type	Section 1613.3.2	Site Class	D
Mapped Spectral Response Acceleration for MCE at 0.2 second Period	Figure 1613.3.1(1)	S_s	1.50
Mapped Spectral Response Acceleration for MCE at 1 Second Period	Figure 1613.3.1(2)	S_1	0.60
Site Coefficient	Table 1613.3.3(1)	F_a	1.0
Site Coefficient	Table 1613.3.3(2)	F_v	1.5
Adjusted MCE Spectral Response Parameter	Equation 16-37	S_{MS}	1.50
Adjusted MCE Spectral Response Parameter	Equation 16-38	S_{M1}	0.90
Design Spectral Response Acceleration Parameter	Equation 16-39	S_{DS}	1.00
Design Spectral Response Acceleration Parameter	Equation 16-40	S_{D1}	0.60

7.2 Reinforced Mat Foundations

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

7.2.1 Mat Foundation Settlement

Our calculations for the structure with a reinforced mat foundation designed for an average allowable bearing pressure of 2,000 psf for dead plus sustained live loads indicate static settlement of about ½--inch (differential settlement of approximately ¼-inch in 50 horizontal feet) for a mat bearing 12 feet below the existing grade.

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 ¼-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 handicap ramp access 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.

7.2.2 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 structure we recommend a modulus of subgrade reaction of 50 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.

7.2.3 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.

7.3 Moisture Protection Considerations for Mat Foundations

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

- A minimum 15-mil thick vapor barrier meeting minimum ASTM E 1745, Class A requirements should be placed directly below the mat slab foundation. The vapor barrier should extend to the edge of the slab and should be sealed at all seams and penetrations.

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

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

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

- All concrete surfaces to receive any type of floor covering should be moist-cured for a minimum of 7 days. Moist curing methods may include frequent sprinkling, or using coverings such as burlap, cotton mats, or carpet. The covering should be placed as soon as the concrete surface is firm enough to resist surface damage. The covering should be kept continuously wet and not allowed to dry out during the required curing period.

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

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

7.4 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 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 unrestrained walls be designed to resist an equivalent fluid pressure of 45 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 retained soil. Unrestrained walls should also be designed to resist an additional uniform pressure equivalent to one-third of any surcharge loads applied at the surface.

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 0.50g is expected at the site. We performed calculations using this ground acceleration, and estimated an additional seismic increment of $11H^2$ 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

All walls over 2 feet in exposed height should be designed with adequate drainage. Adequate drainage may be provided by a subdrain system behind the walls. This system should consist of a 4-inch minimum diameter perforated pipe placed near the base of the wall (perforations placed downward). The pipe should be bedded and backfilled with Class 2 Permeable Material per Caltrans Standard Specifications, latest edition. The permeable backfill should extend at least 12 inches out from the wall and to within 2 feet of outside finished grade. Alternatively, ½- to ¾-inch crushed rock may be used in place of the Class 2 Permeable Material provided the crushed rock and pipe are enclosed in filter fabric, such as Mirafi 140N or equivalent. The upper 2 feet of wall backfill should consist of relatively low permeable compacted on-site clayey soil. The subdrain outlet should be connected to a free-draining outlet or sump.

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

8.4 Backfill

Where surface improvements will be located over the retaining wall backfill, backfill placed behind the walls should be compacted to at least 95 percent relative compaction using light compaction equipment. Where no surface improvements are planned, backfill should be compacted to at least 90 percent. If heavy compaction equipment is used, the walls should be temporarily braced.

8.5 Foundation

Basement walls may be supported on the mat foundation designed in accordance with the recommendations presented in the “Reinforced Mat Foundation” section of this report. Lateral load resistance for the walls may be developed in accordance with the recommendations in the “Lateral Loads” section.

9.0 PAVEMENTS

9.1 Asphalt Concrete

The proposed development does not include parking. Therefore, based on the subsurface soils encountered, we estimated an R-value to provide data for pavement design. We judge an R-value of 25 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 9.

Table 9. Recommended Asphalt Concrete Pavement Design Alternatives
Pavement Components
Design R–Value = 25

General Traffic Condition	Design Traffic Index	Asphalt Concrete (Inches)	Aggregate Baserock* (Inches)	Total Thickness (Inches)
Automobile Parking	4.0	2.5	5.0	7.5
	4.5	2.5	6.0	8.5
Automobile Parking Channel	5.0	3.0	6.5	9.5
	5.5	3.0	8.5	11.5
Truck Access & Parking Areas	6.0	3.5	8.5	12.0
	6.5	4.0	9.5	13.5

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

The traffic indices used in our pavement design are considered reasonable values for the proposed development and should provide a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance. Because the native soils at the site are low to moderately 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 10. 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 10. 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 4 inches of Class 2 aggregate base compacted to a minimum of 90 percent relative compaction in accordance with ASTM Test Method D1557, latest edition. If sidewalks are subject to wheel loads, they should be designed in accordance with the “Exterior Portland Cement Concrete Pavements” section of this report.

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 Montgomery 7, LLC, specifically for design of the proposed Montgomery 7 Mixed Use 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 borings 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.

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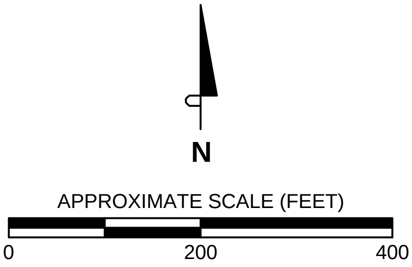
SOURCE AERIAL PHOTO: Google Earth, April 2016.

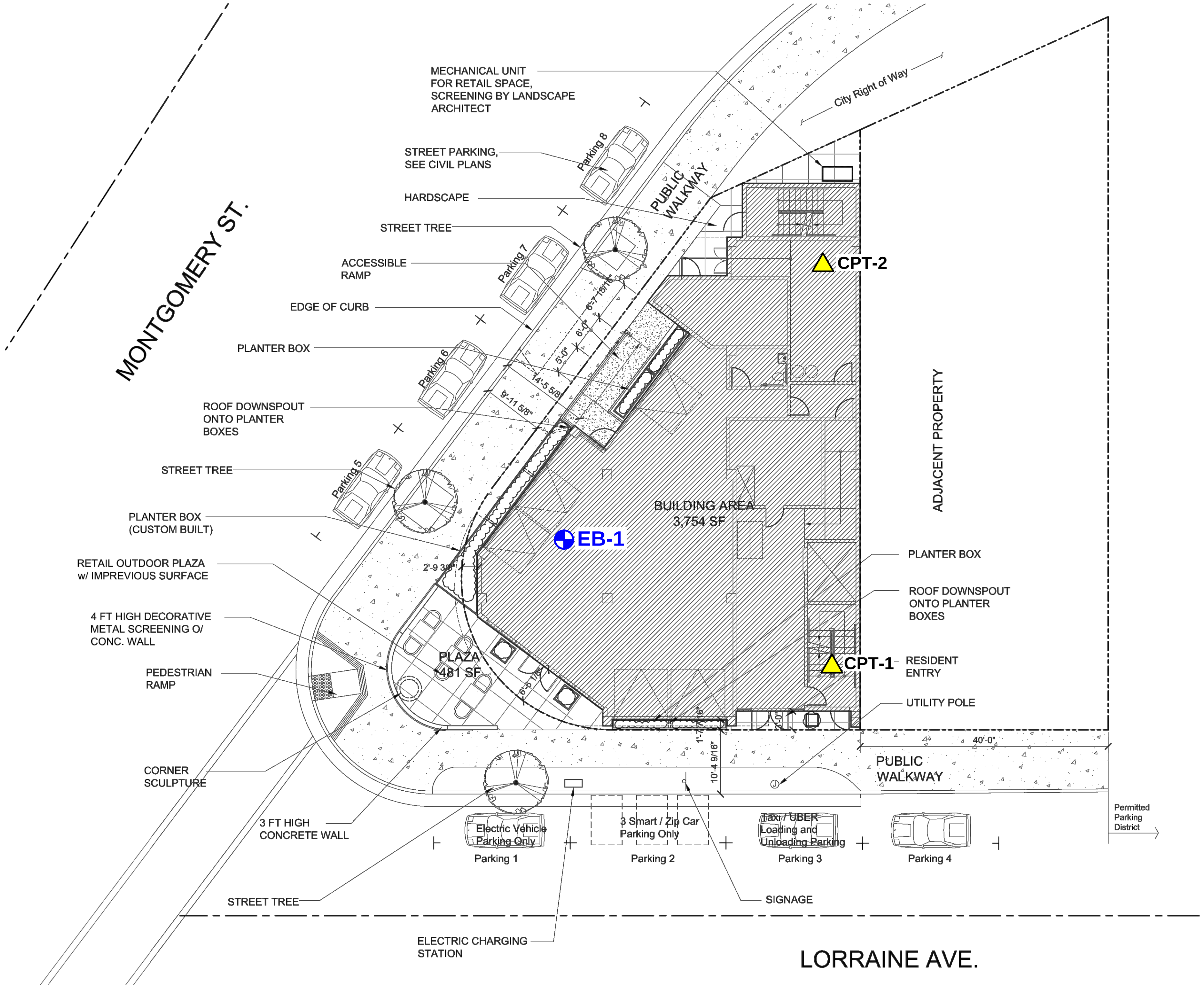


VICINITY MAP

Montgomery 7 Mixed Use Development
565 Lorraine Avenue
San Jose, California

TRC | 261765 | **FIGURE 1**

FILE NAME: Z:\CAD_DRAW\Current\Montgomery 7 Dev_San Jose\Fig1_Vicinity Map.dwg | Layout Tab: 8x11



LEGEND

Approximate locations of:

△ Cone penetration test (CPT)

⊕ Exploratory boring

SCALE (FEET)

0 20 40

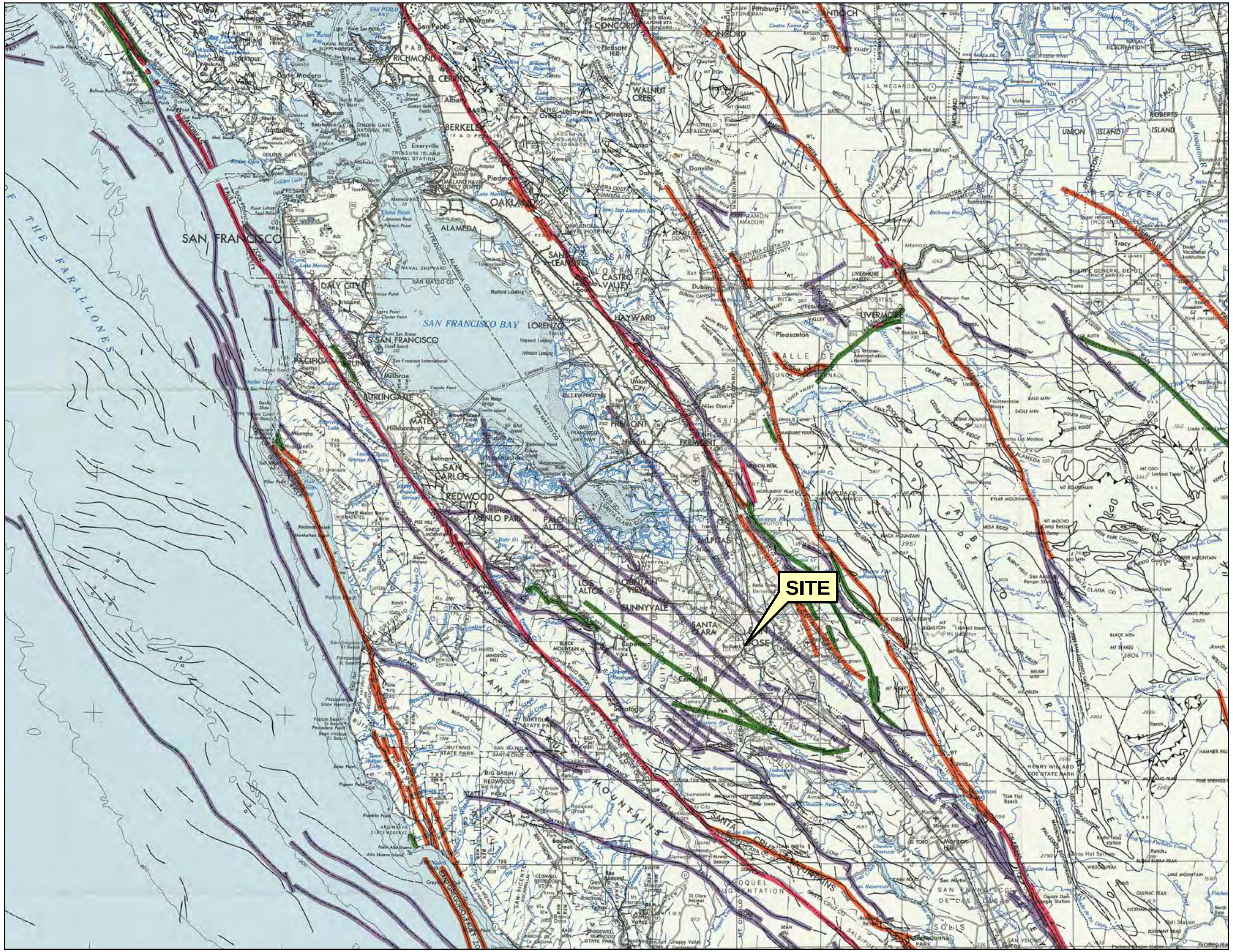
SOURCE: Proposed Site Plan by Anderson Architects Inc., May 2016.

SITE PLAN

Montgomery 7 Mixed Use Development
565 Lorraine Avenue
San Jose, California

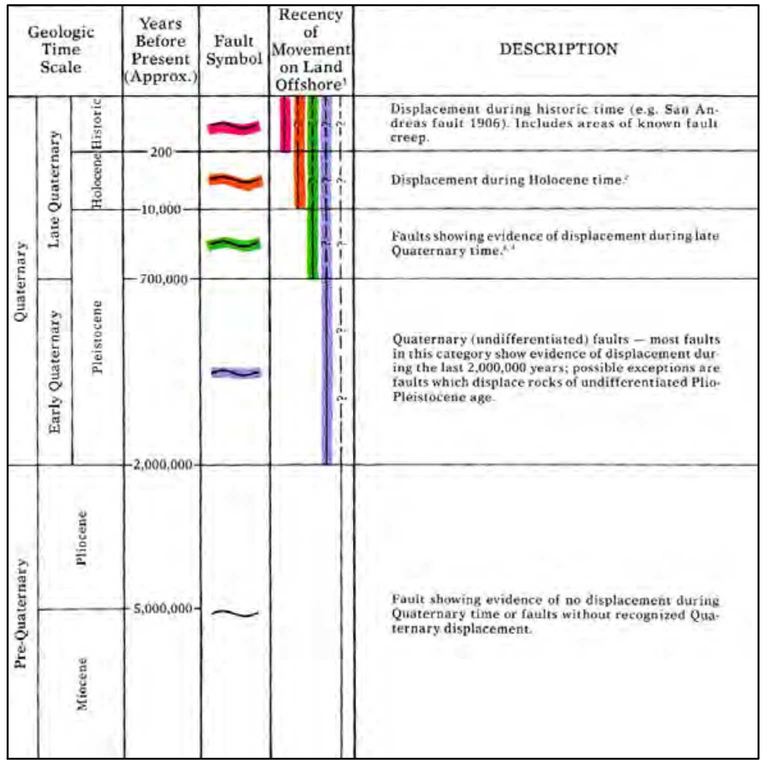
TRC 261765 **FIGURE 2**

FILE NAME: Z:\CAD_DRAW\Current\Montgomery 7 Dev_San Jose\Fig2_Site Plan.dwg | Layout Tab: 11x17



SCALE: 1:500,000

APPROXIMATE GRAPHICAL SCALE (MILES)



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

NOTES:

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

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

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

From: Bortugno & others (1991)

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

REGIONAL FAULT MAP

Montgomery 7 Mixed Use Development
565 Lorraine Avenue
San Jose, California

TRC

261765

FIGURE 3

FILE NAME: Z:\CAD_DRAW\Current\Montgomery 7 Dev_San Jose\Fig3_Regional Fault Map.dwg | Layout Tab: 11x17

APPENDIX A

FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using conventional, truck-mounted, hollow-stem auger drilling equipment and cone penetration test (CPT) equipment. One 8-inch diameter exploratory boring was drilled on September 6, 2016 to a maximum depth of 124 feet. Two CPTs were advanced on September 2, 2016 to maximum depths of 62½ and 66½ feet. The approximate locations of the exploratory boring and CPTs are shown on Figure 2. The soils encountered in the boring 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 boring and CPTs, as well as a key to the classification of the soil and CPTs, are included as part of this appendix.

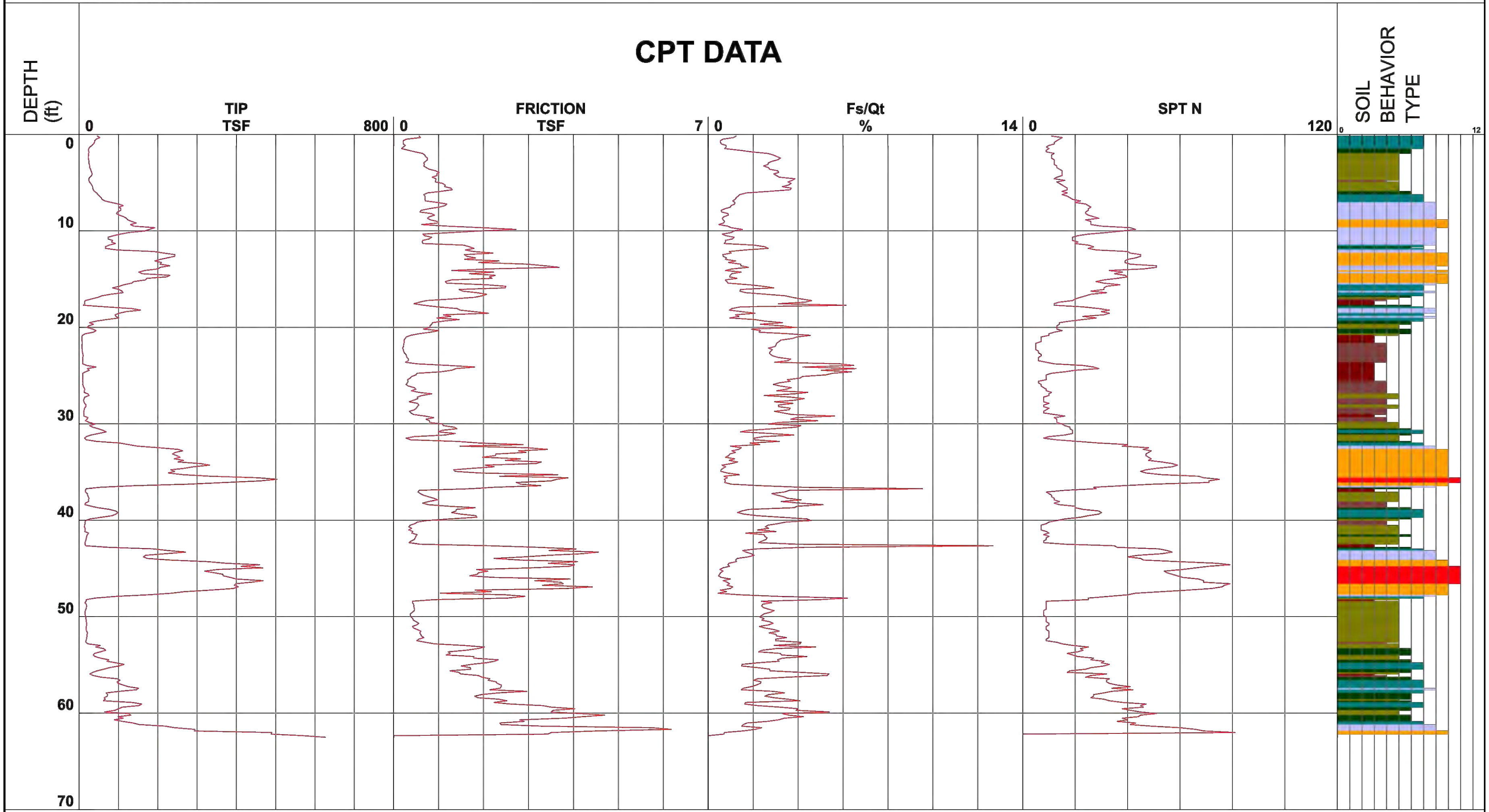
The locations of boring and CPTs were approximately determined by pacing from existing site boundaries. Elevations of the boring and CPTs were not determined. The locations of the boring and CPTs should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. Penetration resistance blow counts were obtained by dropping a 140-pound hammer 30 inches. Modified California 3.0-inch outside diameter (O.D.) samples and Standard Penetration Test (SPT) 2-inch O.D. samples were obtained by driving the samplers 18 inches and recording the number of hammer blows for each 6 inches of penetration. Unless otherwise indicated, the blows per foot recorded on the boring logs represent the accumulated number of blows required to drive the samplers the last two 6-inch increments. When using the SPT sampler, the sum of the last two 6-inch increments is the uncorrected SPT measured blow count. 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.

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

Net Area Ratio .8



🟫 **1 - sensitive fine grained**

⬛ **2 - organic material**

🟥 **3 - clay**

🟫 **4 - silty clay to clay**

🟩 **5 - clayey silt to silty clay**

🟩 **6 - sandy silt to clayey silt**

🟦 **7 - silty sand to sandy silt**

🟪 **8 - sand to silty sand**

🟧 **9 - sand**

🟥 **10 - gravelly sand to sand**

🟪 **11 - very stiff fine grained (*)**

🟦 **12 - sand to clayey sand (*)**

Cone Size 10cm squared

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

CONE PENETRATION TEST - CPT-1

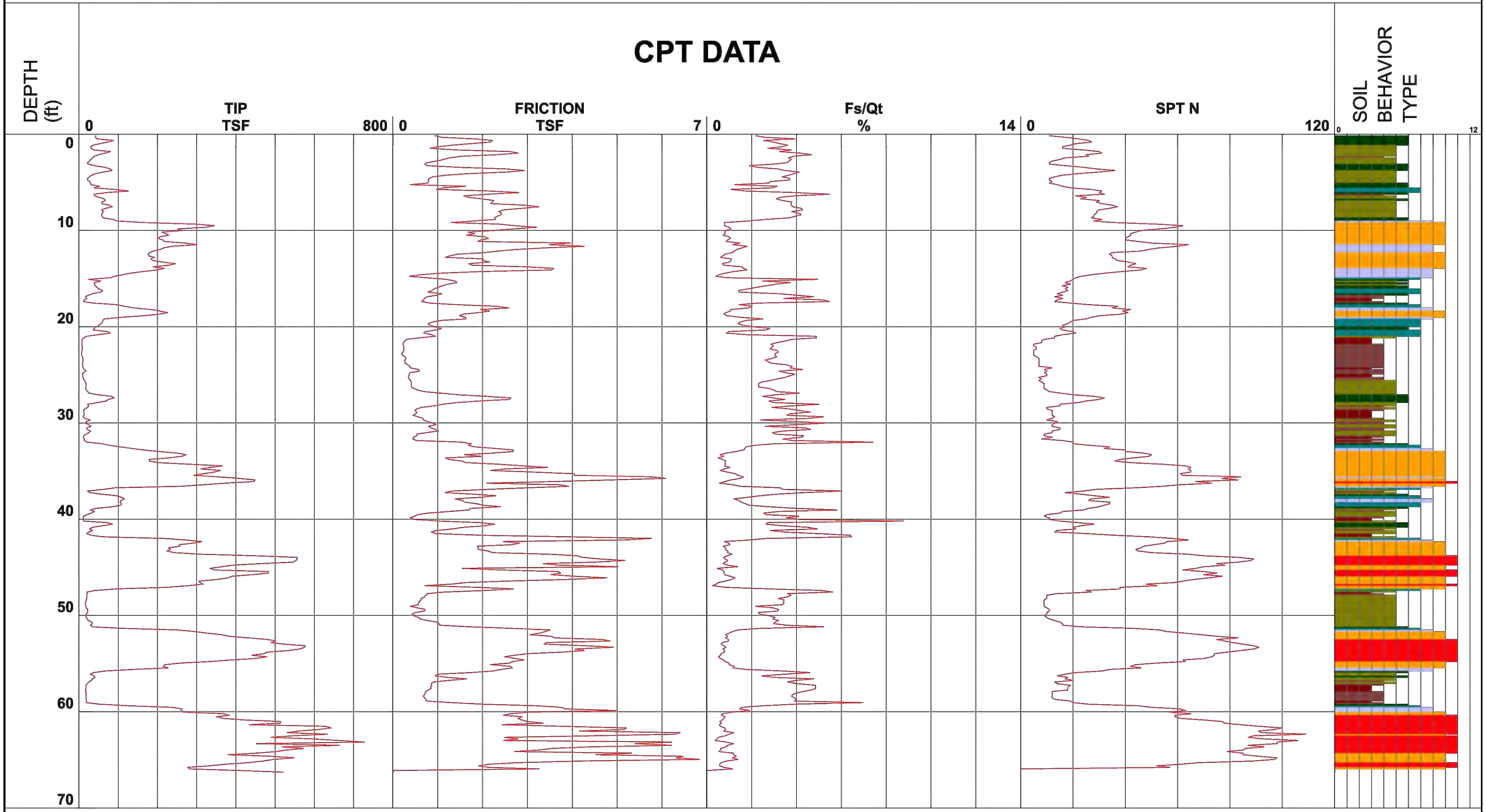
Montgomery 7 Mixed Use Development
565 Lorraine Avenue
San Jose, California

TRC
261765

CPT-1

FILE NAME: Z:\CAD_DRAW\Current\Montgomery 7 Dev_San Jose\CPT-1.dwg | Layout Tab: 8x11

Net Area Ratio .8



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

Cone Size 10cm squared

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

CONE PENETRATION TEST - CPT-2

Montgomery 7 Mixed Use Development
565 Lorraine Avenue
San Jose, California

TRC

261765

CPT-2

FILE NAME: Z:\CAD_DRAW\Current\Montgomery 7 Dev_San Jose\CPT-2.dwg | Layout Tab: 8x11

EXPLORATORY BORING: EB-1

Sheet 1 of 5

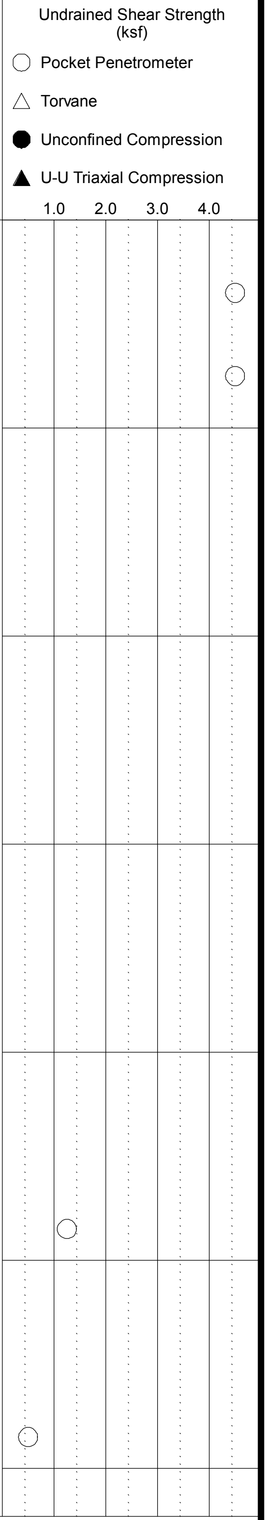
DRILL RIG: MOBILE B-61
BORING TYPE: 8-INCH HOLLOW STEM AUGER
LOGGED BY: AC
START DATE: 9-6-16 FINISH DATE: 9-6-16

PROJECT NO: 261765
PROJECT: MONTGOMERY 7 MIXED USE DEVELOPMENT
LOCATION: SAN JOSE, CA
COMPLETION DEPTH: 124.0 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.

SURFACE ELEVATION:

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
	0	**SANDY SILTY CLAY (CL) [FILL]** hard, moist, brown, low plasticity, fine sand, trace medium to coarse sand and fine gravel (sub-angular/rounded) and brick fragments Liquid Limit = 24, Plasticity Index = 7	CL, FILL	37 25	5 7	102 84	
	5	**SILTY SAND (SM)** loose, moist, brown, fine sand	SM	10	8		41
		POORLY GRADED SAND WITH SILT (SP-SM) loose, moist, brown, fine sand	SP-SM				
		SILTY SAND (SM) loose, moist, brown, fine sand	SM				
	10	**POORLY GRADED SAND WITH SILT (SP-SM)** loose, moist, brown, fine sand	SP-SM	11	4		
	15	medium dense, fine to coarse sand, fine gravel (sub-angular/rounded)		27	4		
	20	**POORLY GRADED GRAVEL WITH SILT (GP-GM)** dense, moist, brown, fine to coarse sand, fine to coarse gravel (sub-angular/rounded)	GP-GM	37	5		
	25	**SANDY LEAN CLAY (CL)** stiff, moist, dark brown, moderate plasticity, fine sand Liquid Limit = 34, Plasticity Index = 15	CL	21	25	96	
	30	medium stiff Liquid Limit = 40, Plasticity Index = 17		21	30	91	



Continued Next Page

GROUND WATER OBSERVATIONS:
▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 36.5 FEET

LA_CORP.GDT 9/15/16 MV, CA*

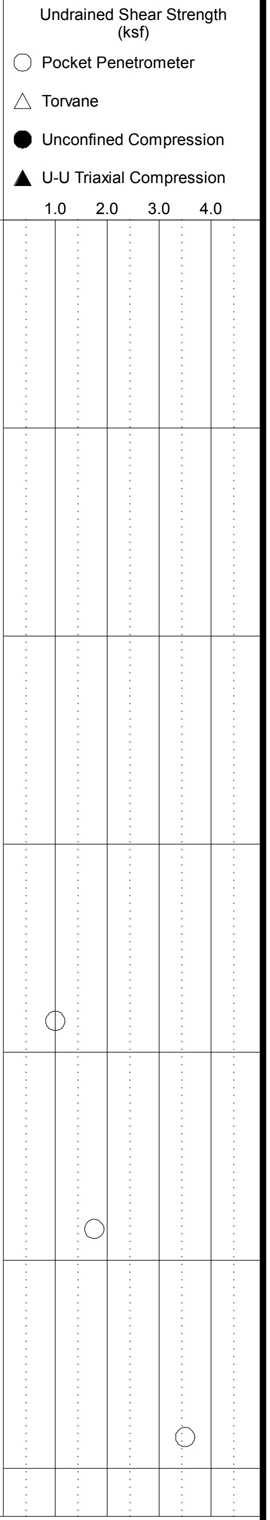
EXPLORATORY BORING: EB-1 Cont'd

Sheet 2 of 5

DRILL RIG: MOBILE B-61	PROJECT NO: 261765
BORING TYPE: 8-INCH HOLLOW STEM AUGER	PROJECT: MONTGOMERY 7 MIXED USE DEVELOPMENT
LOGGED BY: AC	LOCATION: SAN JOSE, CA
START DATE: 9-6-16 FINISH DATE: 9-6-16	COMPLETION DEPTH: 124.0 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
	30			CL					
			SILTY SAND (SM) dense, wet, brown, fine sand	SM	55	⧗			
	35		**POORLY GRADED SAND WITH SILT (SP-SM)** dense, wet, brown, fine to coarse sand, fine gravel (sub-angular/rounded)	SP-SM					
▽									
			POORLY GRADED GRAVEL (GP) very dense, wet, brown, fine to coarse sand, fine gravel (sub-angular/rounded)	GP	86	⧖			
	40								
					48	⧖			
	45		dense						
			LEAN CLAY (CL) stiff, moist, greenish gray, moderate plasticity, trace fine sand Liquid Limit = 35, Plasticity Index = 17	CL	26	⧗	26		
	50								
			SANDY SILT (ML) stiff, moist, dark gray, moderate plasticity, fine sand	ML	36	⧗	27		
	55		**SILTY SAND (SM)** medium dense, wet, dark gray, fine sand	SM					
			LEAN CLAY (CL) very stiff, moist, greenish gray, moderate plasticity, trace fine sand	CL	59	⧗			
				GC					
	60		*Continued Next Page*						



GROUND WATER OBSERVATIONS:

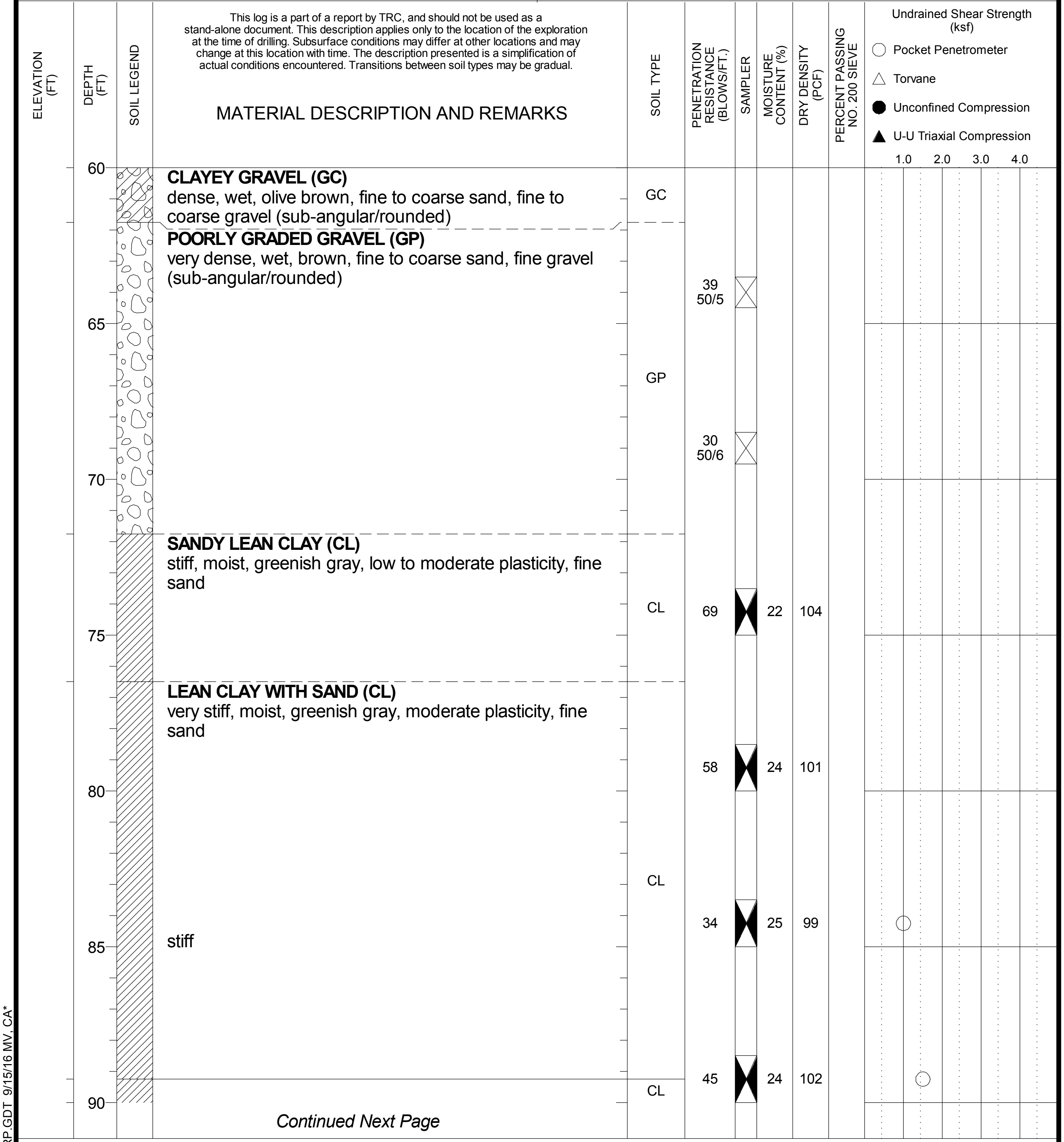
▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 36.5 FEET

EXPLORATORY BORING: EB-1 Cont'd

Sheet 3 of 5

DRILL RIG: MOBILE B-61	PROJECT NO: 261765
BORING TYPE: 8-INCH HOLLOW STEM AUGER	PROJECT: MONTGOMERY 7 MIXED USE DEVELOPMENT
LOGGED BY: AC	LOCATION: SAN JOSE, CA
START DATE: 9-6-16 FINISH DATE: 9-6-16	COMPLETION DEPTH: 124.0 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)	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)
	60	**CLAYEY GRAVEL (GC)** dense, wet, olive brown, fine to coarse sand, fine to coarse gravel (sub-angular/rounded)	GC						
		POORLY GRADED GRAVEL (GP) very dense, wet, brown, fine to coarse sand, fine gravel (sub-angular/rounded)							
				39 50/5	☒				
	65		GP						
				30 50/6	☒				
	70								
		SANDY LEAN CLAY (CL) stiff, moist, greenish gray, low to moderate plasticity, fine sand	CL	69	⧗	22	104		
	75								
		LEAN CLAY WITH SAND (CL) very stiff, moist, greenish gray, moderate plasticity, fine sand		58	⧗	24	101		
	80		CL						
				34	⧗	25	99		○
	85	stiff							
				45	⧗	24	102		○
	90		CL						

Continued Next Page

GROUND WATER OBSERVATIONS:

▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 36.5 FEET

LA_CORP.GDT 9/15/16 MV, CA*

TRC

EXPLORATORY BORING: EB-1 Cont'd

Sheet 4 of 5

DRILL RIG: MOBILE B-61
BORING TYPE: 8-INCH HOLLOW STEM AUGER
LOGGED BY: AC
START DATE: 9-6-16 FINISH DATE: 9-6-16

PROJECT NO: 261765
PROJECT: MONTGOMERY 7 MIXED USE DEVELOPMENT
LOCATION: SAN JOSE, CA
COMPLETION DEPTH: 124.0 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.

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

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf)
	90		**SANDY LEAN CLAY (CL)** stiff, moist, light olive brown, moderate plasticity, fine sand	CL						
	95		**CLAYEY GRAVEL (GC)** very dense, wet, brown, fine to coarse sand, fine gravel (sub-angular/rounded)	GC	50/4					
			CLAYEY SAND (SC) dense, wet, light brown, fine sand	SC	35 50/6					
	100		**POORLY GRADED GRAVEL (GP)** very dense, wet, brown, fine to coarse sand, fine to coarse gravel (sub-angular/rounded)	GP						
			CLAYEY SAND (SC) very dense, wet, brown, fine sand	SC	36 50/4					
	105 110		**POORLY GRADED GRAVEL (GP)** very dense, wet, brown, fine to coarse sand, fine to coarse gravel (sub-angular/rounded)	GP	50/6					
	115		**LEAN CLAY (CL)** stiff, moist, olive brown, moderate plasticity, trace fine sand	CL	57		25			
	120		**CLAYEY SAND (SC)** very dense, wet, light brown, fine sand	SC	83					

Continued Next Page

GROUND WATER OBSERVATIONS:
▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 36.5 FEET

EXPLORATORY BORING: EB-1 Cont'd

Sheet 5 of 5

DRILL RIG: MOBILE B-61
BORING TYPE: 8-INCH HOLLOW STEM AUGER
LOGGED BY: AC
START DATE: 9-6-16 FINISH DATE: 9-6-16

PROJECT NO: 261765
PROJECT: MONTGOMERY 7 MIXED USE DEVELOPMENT
LOCATION: SAN JOSE, CA
COMPLETION DEPTH: 124.0 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.

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

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
	120		**POORLY GRADED GRAVEL (GP)** very dense, wet, brown, fine to coarse sand, fine to coarse gravel (sub-angular/rounded)	GP					
					50/4	☒			
	125		Bottom of boring at 124 feet						
	130								
	135								
	140								
	145								
	150								

GROUND WATER OBSERVATIONS:
∇ : FREE GROUND WATER MEASURED DURING DRILLING AT 36.5 FEET

LA_CORP.GDT 9/15/16 MV, CA*

APPENDIX B

LABORATORY PROGRAM

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

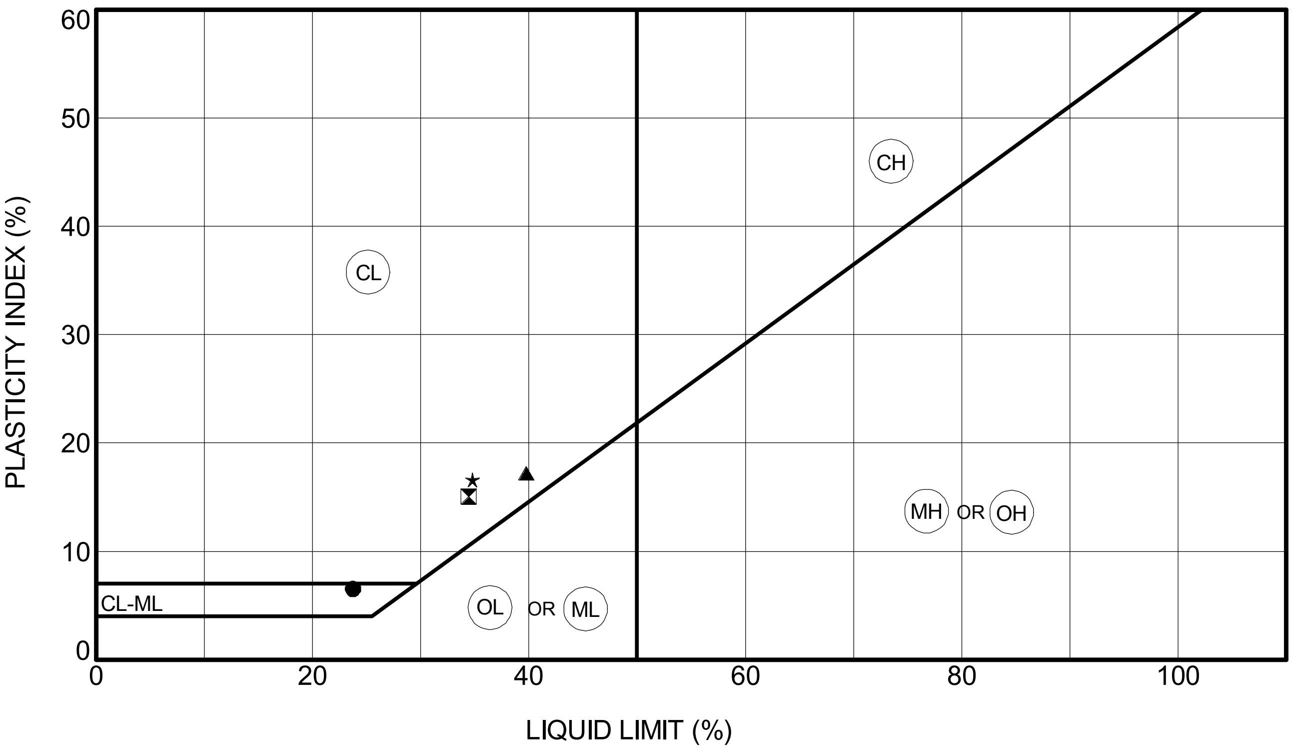
Moisture Content: The natural water content was measured (ASTM D2216) on 15 samples of the materials recovered from the boring. These water contents are recorded on the boring log at the appropriate sample depths.

Dry Densities: In place dry density tests (ASTM D2937) were performed on 8 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: Four 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 one sample of the subsurface soil to aid in the classification of these soils. Result of this test is shown on the boring logs at the appropriate sample depth.

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



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**		**24**	**17**	**7**		
⧗	**EB-1**	**24.5**		**34**	**19**	**15**		
▲	**EB-1**	**29.5**		**40**	**23**	**17**		
★	**EB-1**	**49.5**		**35**	**18**	**17**		

LA_CORP.GDT 9/15/16 MV, CA*

PLASTICITY CHART AND DATA

Project: MONTGOMERY 7 MIXED USE DEVELOPMENT

Location: SAN JOSE, CA

Project No.: 261765

FIGURE B-1

Corrosivity Tests Summary

CTL # 028-2589 **Date:** 9/9/2016 **Tested By:** PJ **Checked:** PJ

Client: TRC **Project:** Montgomery 7 **Proj. No:** 261765

Remarks:

Sample Location or ID			**Resistivity @ 15.5 °C (Ohm-cm)**			**Chloride**	**Sulfate**		**pH**	**ORP**		**Sulfide**	**Moisture**	
			As Rec.	**Min**	**Sat.**	**mg/kg**	**mg/kg**	**%**		(Redox)		Qualitative	At Test	**Soil Visual Description**
						Dry Wt.	Dry Wt.	Dry Wt.		E_H (mv)	At Test	by Lead	**%**	
Boring	**Sample, No.**	**Depth, ft.**	ASTM G57	Cal 643	ASTM G57	ASTM D4327	ASTM D4327	ASTM D4327	ASTM G51	ASTM G200	Temp °C	Acetate Paper	ASTM D2216	
EB-1	4A	9.0	-	-	3,050	21	87	0.0087	7.6	-	-	-	9.5	Olive Brown Silty SAND

APPENDIX C

RESULTS OF LIQUEFACTION ANALYSIS

The liquefaction analysis was performed as specified in Section 3.4 of the report in accordance with the 1998 NCEER Workshops (Youd et al., 2001) and in accordance with guidelines set forth in the CGS Special Publication 117A (2008). The NCEER methods for Cone Penetration Tests (CPT) analysis update simplified procedures presented by Seed and Idriss (1971).

* * * * * * * * * * *

GeoLogismiki
Geotechnical Engineers
Merarhias 56
http://www.geologismiki.gr

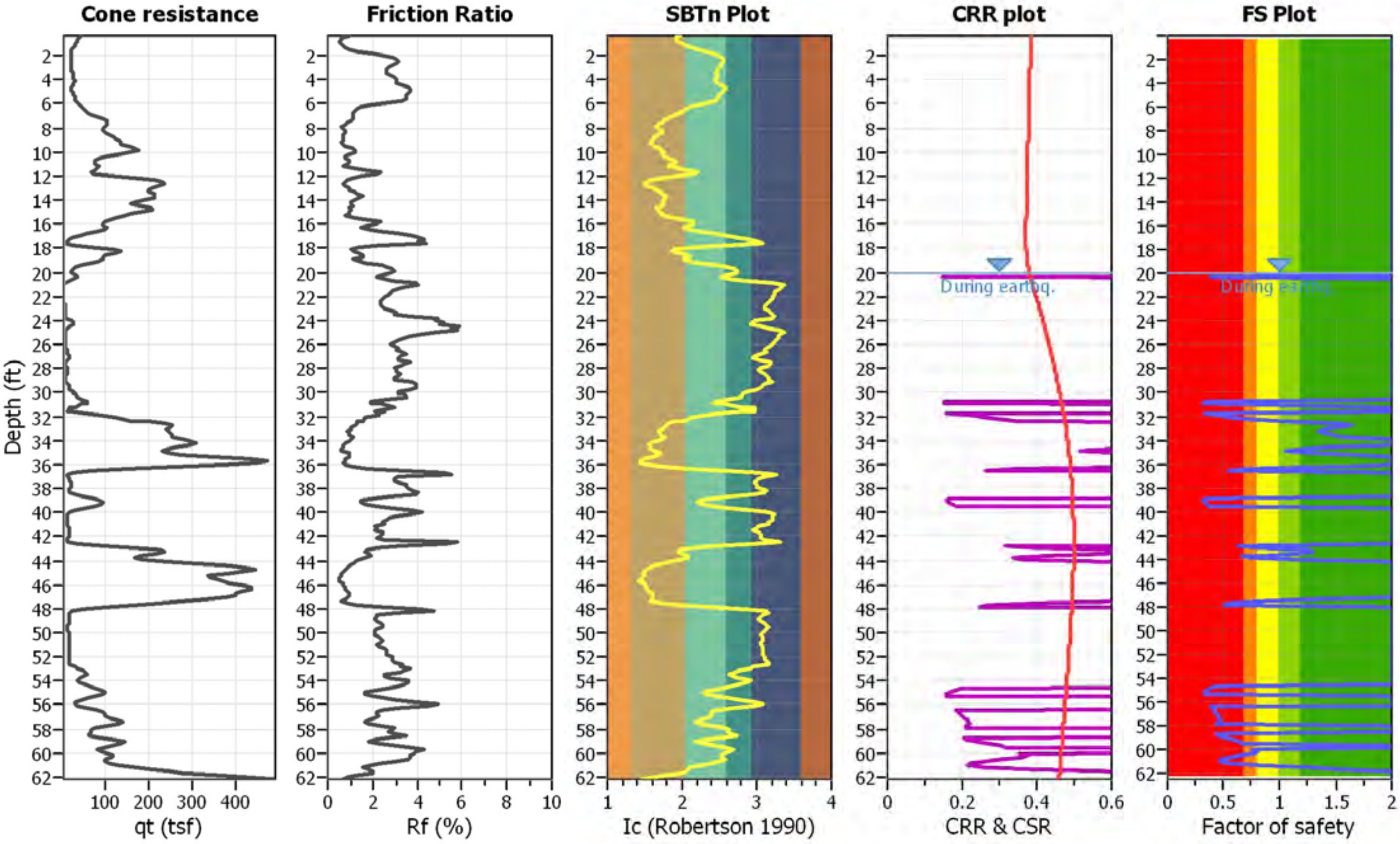
LIQUEFACTION ANALYSIS REPORT

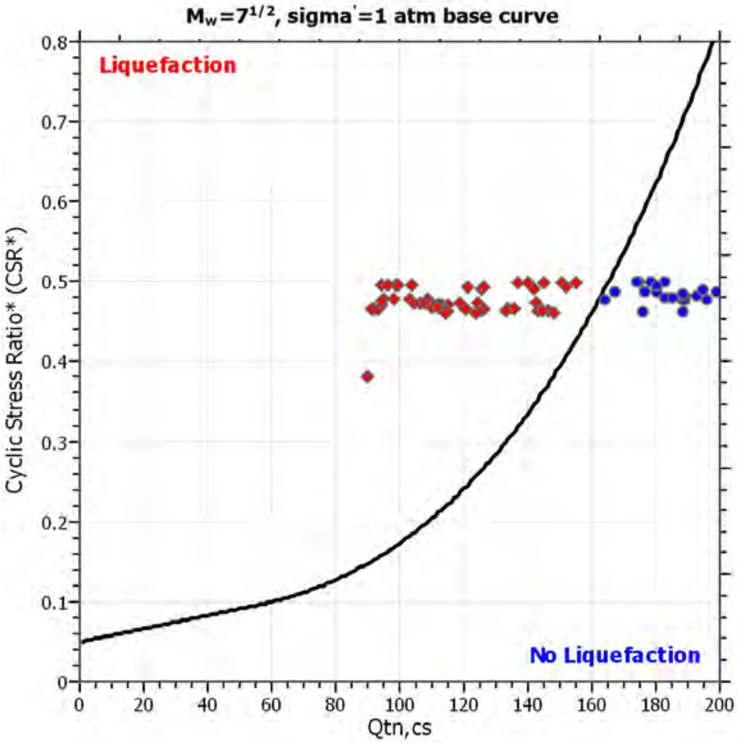
Project title : Montgomery 7 **Location : 565 Lorraine Avenue**

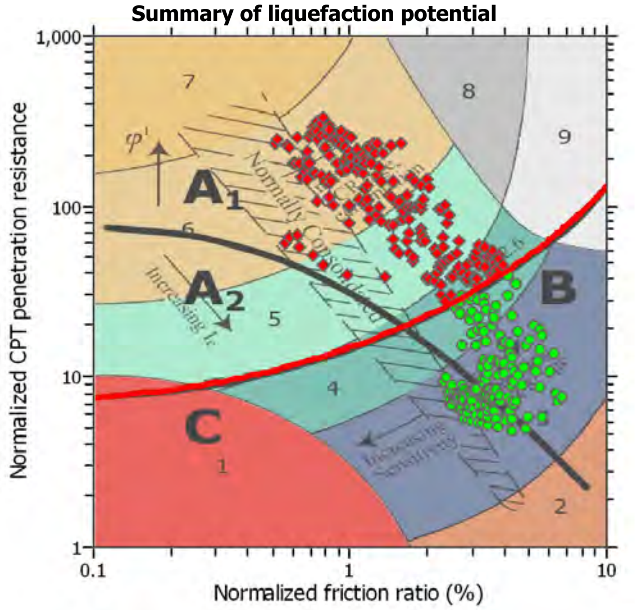
CPT file : CPT-1

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	27.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	20.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	8.00	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

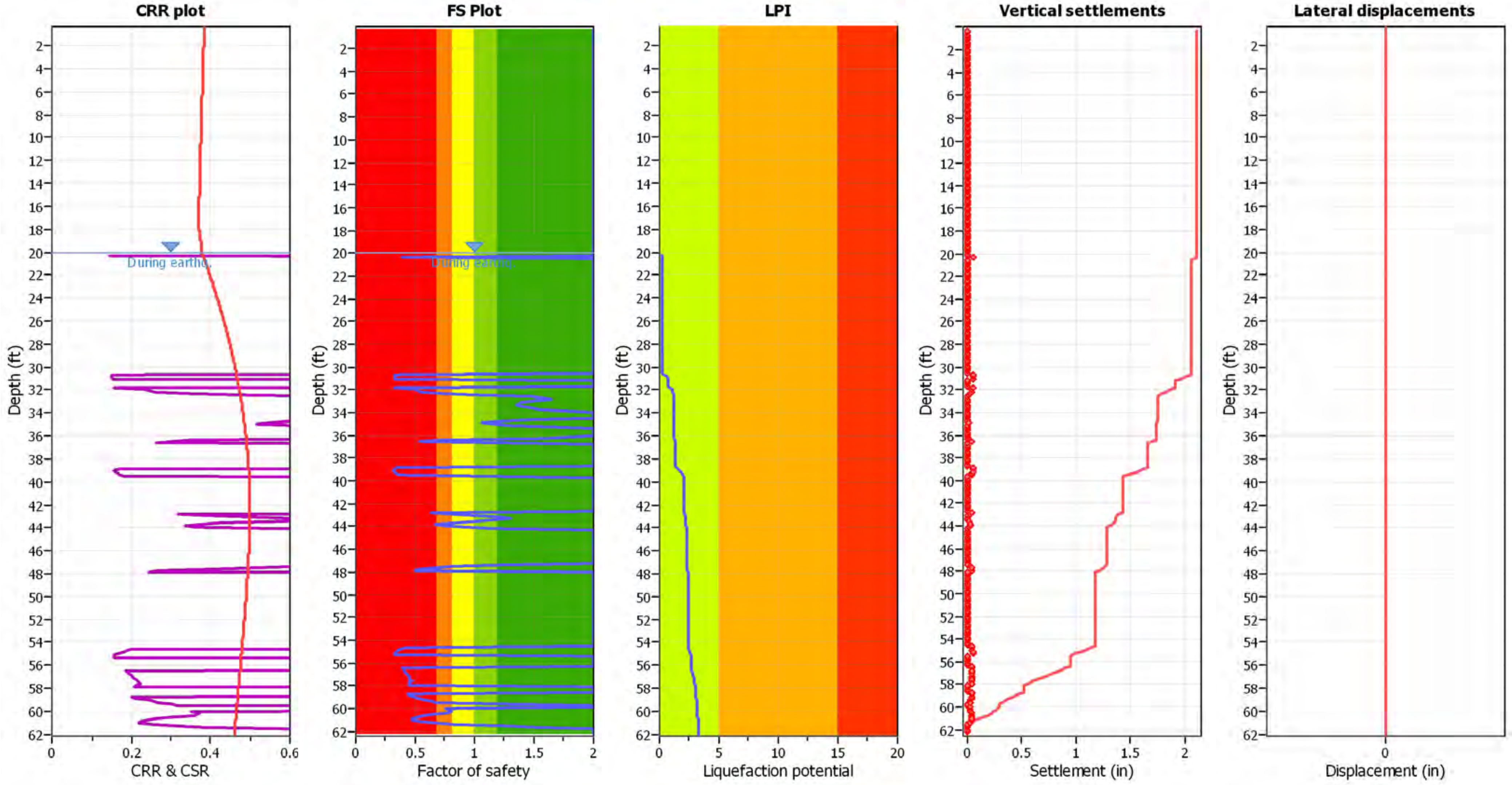






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	20.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	27.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

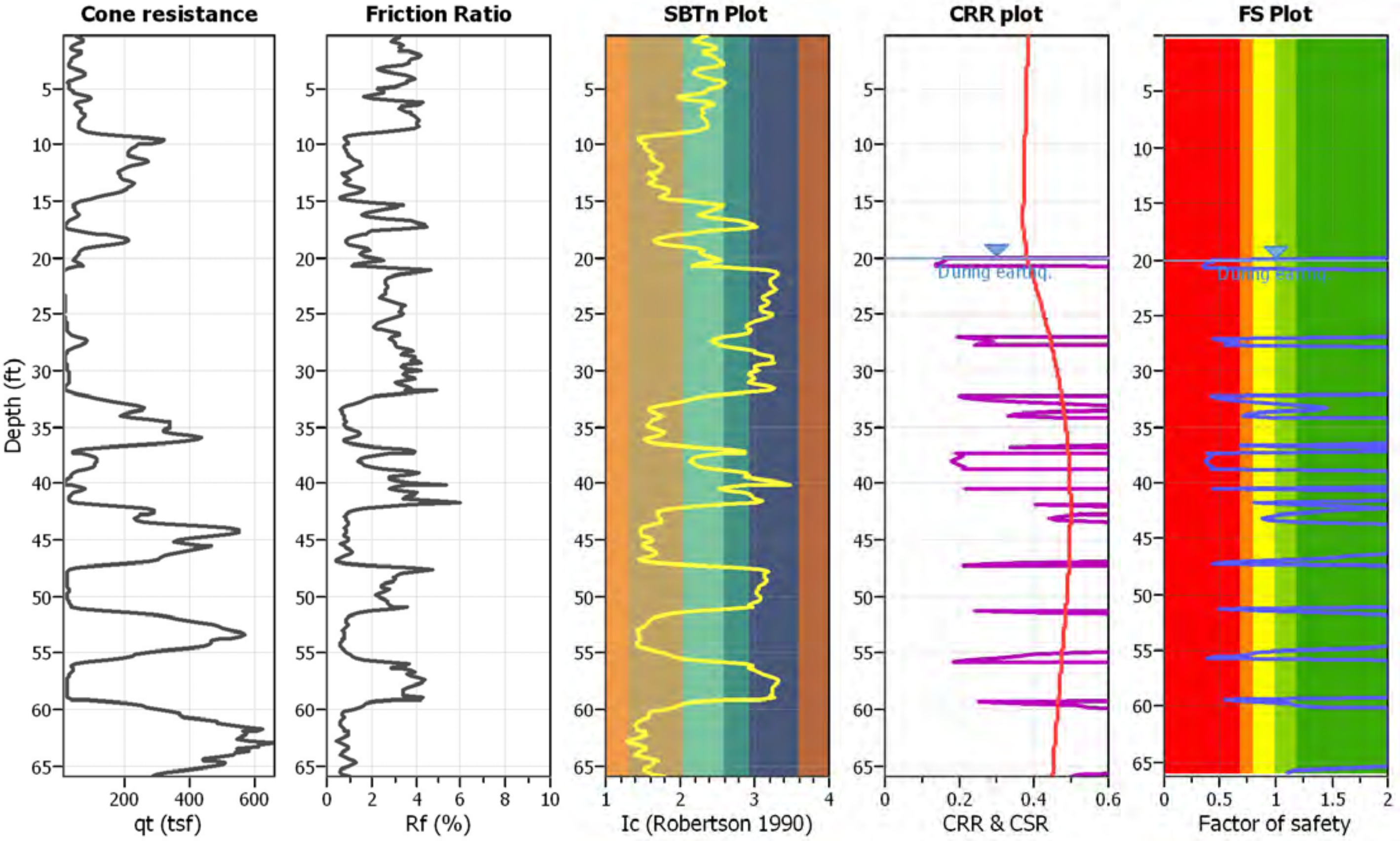
LIQUEFACTION ANALYSIS REPORT

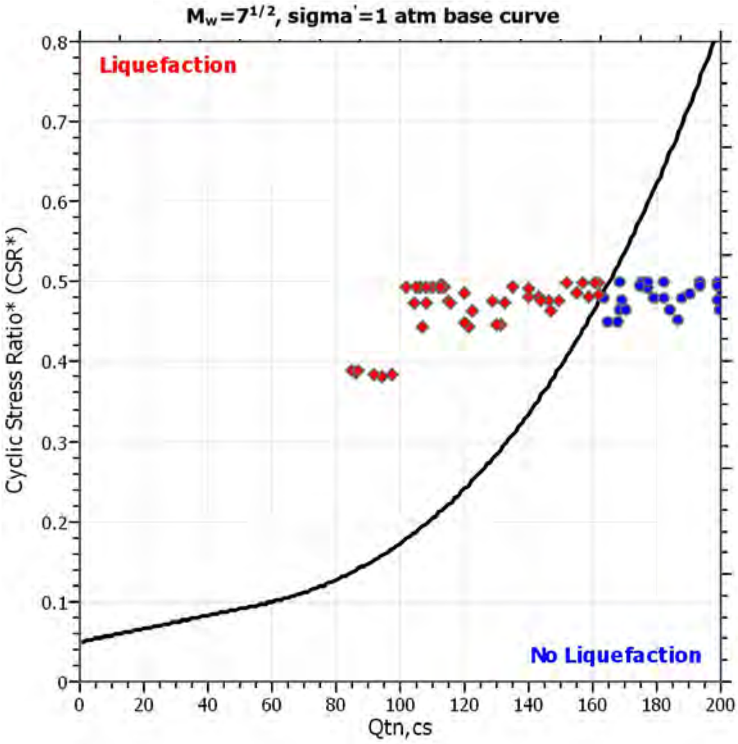
Project title : Montgomery 7 **Location : 565 Lorraine Avenue**

CPT file : CPT-2

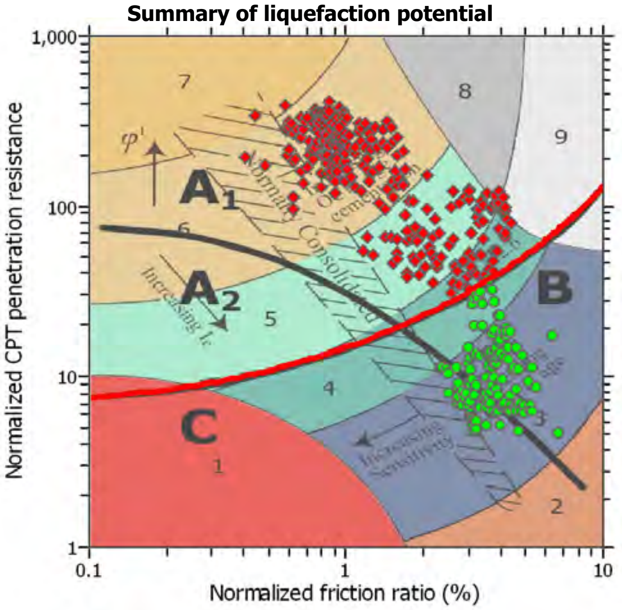
Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	27.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	20.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	8.00	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		


Cone resistance **Friction Ratio** **SBTn Plot** **CRR plot** **FS Plot**

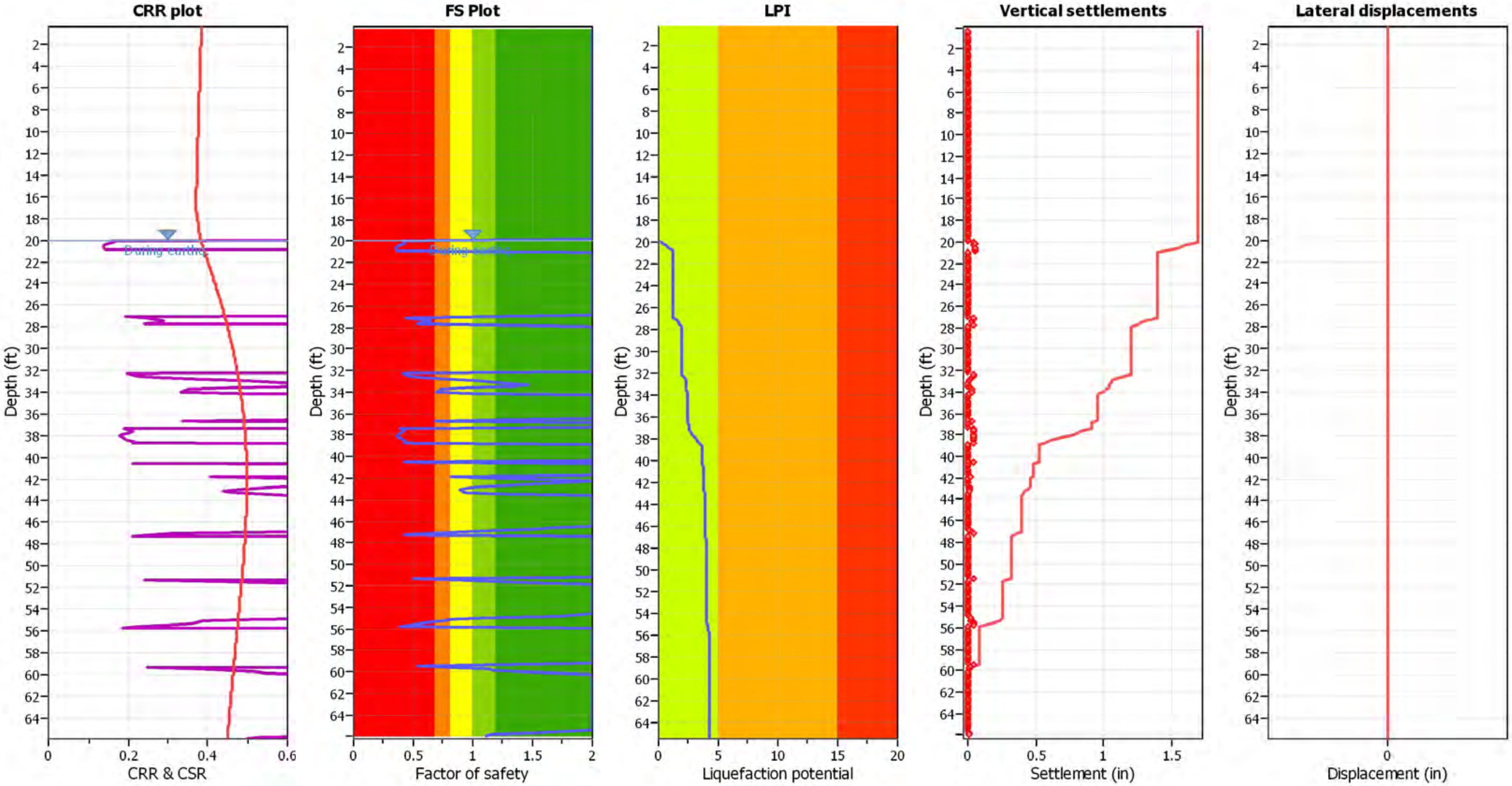

$M_w=7^{1/2}$, sigma'=1 atm base curve

Summary of liquefaction potential



Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	20.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	27.00 ft	Fill height:	N/A	Limit depth:	N/A

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