



Geotechnical Investigation

Mixed Use Development
21 North 21st Street, San Jose, California

Report No. 264960 has been prepared for:

FIRST COMMUNITY HOUSING

75 East Santa Clara Street, Suite 1300,

San Jose, California 95113

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Alberto Cortez, E.I.T.
Senior Staff Engineer

Mustafa B. Dogan, P.E., G.E.
Senior Geotechnical Engineer

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

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**GEOTECHNICAL INVESTIGATION
MIXED USE DEVELOPMENT
21 NORTH 21ST STREET, SAN JOSE, CALIFORNIA**

1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed Mixed Use Development at 21 North 21st Street project to be constructed in San Jose, California. The site location is shown on the Vicinity Map, Figure 1. The purpose of our investigation was to evaluate the geologic and subsurface conditions and to provide geotechnical recommendations for design and construction of the proposed project.

1.1 Project Description

As currently planned, the project will consist of the construction of seven to eight story building consisting of five to six stories of residential units over two stories of concrete podium, commercial space and parking at grade and on the second floor. The proposed building footprint will occupy most of the rectangular approximately ½-acre site. The site is currently vacant and is bordered by Roosevelt Community Center to the northwest, South Bay Sports Training and Batting Cages to the southeast, a parking lot to the southwest, and North 21st Street to the northeast. Additional improvements will include pavements, underground utilities, and landscaping. The approximate site boundary is shown on the Site Plan, Figure 2.

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 October 6, 2016. To accomplish this work, we provided the following services:

- Exploration of subsurface conditions by drilling four borings in the area of the proposed development and retrieving soil samples for observation and laboratory testing. We also advanced 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 October 18, 2016. At the time of the reconnaissance, the site was a vacant site with trees and shrubs. The site appeared relatively flat with minor grade variation for drainage purposes.

2.2 Exploration Program

Subsurface exploration was performed on October 18, 2016 using conventional, truck-mounted hollow-stem auger drilling equipment to investigate, sample, and log subsurface soils. Four hollow-stem auger exploratory borings were drilled to depths of between 35 to 75 feet. In addition, two CPTs were advanced to depths ranging from approximately 103 to 120 feet on October 26, 2016 using CPT equipment to investigate subsurface soils.

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

2.3 Subsurface Conditions

The CPTs were performed within the footprint of the proposed mixed-use structure. In general, soils encountered in CPT-1 and CPT-2 were interpreted to include interbedded layers of clay, sandy silt, and silty sand to depths of 69 and 67 feet, respectively. Below these depths, a thick dense sand to gravelly sand layer was encountered in CPT-1 and CPT-2 to approximate depths of 81 and 83 feet, respectively. Below these depths, interbedded layers of clay, sandy silt, silty sand, sand, and gravelly sand were encountered to the maximum explored depth of 120 feet. CPT-2 encountered refusal at a depth of 103 feet in the dense sand to gravelly sand layer with approximate tip pressures greater than 550 tons per square foot.

All of our borings were performed within the footprint of the proposed mixed-use structure. Our borings generally encountered medium stiff to hard lean clay, and medium stiff to hard sandy lean clay to a depth of 50 feet. Compressible clays were encountered in the upper 24 to 48 feet below grade. Several thin interbedded layers of clayey sand ranging from approximately $\frac{1}{2}$ to 2 feet in thickness were encountered in the borings. Below the depth of 50 feet, boring EB-1 encountered interbedded layers consisting of stiff to very stiff lean clay, and stiff to very stiff sandy lean clay to a depth of about 69½ feet. Below this depth, medium dense clayey sand was encountered to a depth of 71½ feet, underlain by dense poorly graded sand to the maximum explored depth of 75 feet. Our borings and CPTs were backfilled immediately after drilling.

Three Plasticity Index (PI) tests were performed to determine the PI of the representative clay soil samples collected from borings EB-1, EB-2, and EB-4 at depths of approximately 2, 19½, and 31 feet, respectively. The tests resulted in PI's ranging from 10 to 14 indicating low to moderate plasticity and expansion potential of the soils.

2.4 Ground Water

Free ground water was encountered during subsurface exploration in all of the borings at depths ranging from approximately 22 to 28 feet below grade. Based on pore pressure dissipation measurements performed in the CPTs, ground water was inferred to be at a depth of approximately 17½ feet below grade. Based on the depth to historically high ground water map prepared by the California Geological Survey for the San Jose East Quadrangle (CGS, 2000), the depth to historically high ground water levels in the site vicinity is on the order of 10 feet below the existing ground surface (bgs). Based on the above information, we judged a ground water depth of 10 feet to be appropriate for liquefaction analysis. 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). 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 |
|--|-------------------------------|------------------------------|--|
| United States Geological Survey (USGS) Seismic Hazard Curves, Response Parameters and Design Parameters program v5.1.0 | PGA _M | 0.50 | Equation 11.8-1 of American Society of Civil Engineers (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 | 0.52 | |
| USGS 2008 Interactive Deaggregation Web Tool | Probabilistic 2% in 50 years | 0.79 | |
| Caltrans ARS Web Tool v2.2.06 | Deterministic | 0.58 | 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 (SCC, 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 10 feet below the existing site grade was used for our liquefaction analysis. As discussed in the subsurface description above, several sand and silt layers were encountered below the design ground water depth. These layers were evaluated to assess liquefaction potential and the effects liquefaction may have on the proposed 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 6.63 (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 | 11.5 | 0.8 | 2.5 | 120.8 | 1.0 | Likely | 0.1 |
| | 90.2** | 5.9 | 2.1 | 96.4 | 0.6 | Likely | 1.7 |
| | 96.5** | 0.8 | 2.0 | 113.6 | 0.8 | Likely | 0.2 |
| | 97.8** | 2.5 | 1.9 | 111.9 | 0.7 | Likely | 0.5 |
| | 104.2** | 4.9 | 2.3 | 93.0 | 0.5 | Likely | 1.5 |
| | 114.0** | 1.8 | 2.3 | 76.0 | 0.4 | Likely | 0.6 |
| | 118.0** | 0.2 | 2.6 | 90.4 | 0.5 | Likely | 0.1 |
| | 118.8** | 0.3 | 2.5 | 89.8 | 0.5 | Likely | 0.1 |
| | 119.6** | 0.5 | 2.5 | 87.8 | 0.5 | Likely | 0.2 |
| Total = | | | | | | | 5.0 |
| CPT-2 | 12.1 | 1.2 | 2.6 | 124.9 | 1.0 | Likely | 0.1 |
| | 52.5** | 0.2 | 2.6 | 84.6 | 0.4 | Likely | 0.1 |
| | 67.3** | 0.8 | 2.3 | 116.5 | 0.8 | Likely | 0.2 |
| | 82.0** | 1.0 | 2.1 | 131.7 | 1.0 | Likely | 0.1 |
| | 93.5** | 0.3 | 2.5 | 104.3 | 0.6 | Likely | 0.1 |
| | 96.0** | 7.2 | 2.0 | 117.0 | 0.8 | Likely | 1.4 |
| Total = | | | | | | | 2.0 |

* 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 shallow foundation recommendations.

Our analyses indicate that one silt layer below the design ground water depth may theoretically liquefy, resulting in approximately less than 1/4-inch of total settlement. Volumetric change and settlement were estimated using the Zhang, Robertson, and Brachman (2002) method. We estimate differential settlements from liquefaction will be on the order of less than 1/2-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

Jonathan D. Bray and Rodolfo C. Sancio in 2006. The conclusions of the paper were that fine-grained soils with PI values less than 12 and moisture contents (WC) above 85 percent of the Liquid Limit (LL) are potentially susceptible to liquefaction.

We performed Atterberg Limits tests on two representative soil samples collected from boring EB-2 and EB-4 at depths of approximately 19½ and 31 feet, respectively. Results of the Atterberg Limits tests and our liquefaction screening are summarized in Table 3 below.

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

| Boring | Sample Depth | Sample Description | LL | PI | WC* | WC/LL (percent) |
|--------|--------------|--------------------|----|----|-----|-----------------|
| EB-2 | 19½ | Lean Clay | 32 | 14 | 21 | 66 |
| EB-4 | 31 | Lean Clay | 31 | 10 | 23 | 74 |

Based on the results of the screening, it appears that the lean clay represented in the samples from boring EB-2 at a depth of 19½ feet and the sample from boring EB-4 at a depth of 31 feet is not susceptible to liquefaction based on the Bray and Sancio criteria as the WC is less than 85 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.

There is approximately 11½ feet of non-liquefiable material overlying the relatively thin potentially liquefiable strata at the site. 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. The soils above the design groundwater depth of 10 feet are cohesive clays. 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 Coyote Creek is located approximately 500 feet west of the site boundary. The potentially liquefiable layers at the site are thin. 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 site is located in a Federal Emergency Management Agency (FEMA) Zone X (FEMA 2014), which is defined as “areas determined to be outside the 0.2 percent annual chance floodplain.

4.0 CORROSION EVALUATION

To evaluate the corrosion potential of the subsurface soils at the site, we submitted two samples collected during our subsurface investigation to an analytical laboratory for pH, resistivity, soluble sulfate and chloride content testing. The results of these tests are summarized in Table 4 below.

Table 4. Results of Corrosivity Testing

| Sample | Depth (feet) | Chloride (mg/kg) | Sulfate (mg/kg) | pH | Resistivity (ohm-cm) | Estimated Corrosivity Based on Resistivity | Estimated Corrosivity Based on Sulfates |
|--------|--------------|------------------|-----------------|-----|----------------------|--|---|
| EB-1 | 5.5 | 168 | 317 | 7.4 | 937 | Severely | Negligible |
| EB-2 | 2.0 | 38 | 53 | 7.6 | 2,285 | Severely | Negligible |

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

Many factors can affect the corrosion potential of soil including soil moisture content, resistivity, permeability and pH, as well as chloride and sulfate concentration. In general, soil resistivity, which is a measure of how easily electrical current flows through soils, is the most influential factor. Based on classification developed by William J. Ellis (1978), the approximate relationship between soil corrosiveness was developed as shown in Table 5 below.

Table 5. Relationship Between Soil Resistivity and Soil Corrosivity

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

Chloride and sulfate ion concentrations and pH appear to play secondary roles in affecting corrosion potential. High chloride levels tend to reduce soil resistivity and break down otherwise protective surface deposits, which can result in corrosion of buried metallic improvements or reinforced concrete structures. Sulfate ions in the soil can lower the soil resistivity and can be highly aggressive to Portland cement concrete (PCC) by combining chemically with certain constituents of the concrete, principally tricalcium aluminate. This reaction is accompanied by expansion and eventual disruption of the concrete matrix. Soils containing high sulfate content could also cause corrosion of the reinforcing steel in concrete. Table 4.2.1 of the American Concrete Institute (ACI, 2008) provides requirements for concrete exposed to sulfate-containing solutions as summarized in Table 6.

Table 6. Relationship Between Sulfate Concentration and Sulfate Exposure

| Water-Soluble Sulfate (SO ₄) in soil, ppm | Sulfate Exposure |
|---|-----------------------|
| 0 to 1,000 | Negligible |
| 1,000 to 2,000 | Moderate ¹ |
| 2,000 to 20,000 | Severe |
| over 20,000 | Very Severe |

¹ = seawater

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

As shown in Table 4, the soil resistivity results ranged from 937 to 2,285 ohm-centimeters. Based on these results and the resistivity correlations presented in Table 5, the corrosion potential to buried metallic improvements may be characterized as severely corrosive. We recommend that a corrosion protection engineer be consulted about appropriate corrosion protection methods for buried metallic materials.

Based on our previous experience and Table 4.2.1 of the ACI, it is our opinion that sulfate exposure to PCC may be considered negligible to moderate for the native subsurface materials sampled.

5.0 CONCLUSIONS AND RECOMMENDATIONS

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

5.1 Primary Geotechnical Concerns

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

- Compressible soils
- Strong seismic shaking
- The potential for liquefaction-induced total and differential settlement
- Moderately expansive near surface soils
- Corrosion potential of the near-surface soils
- Differential settlement for utility tie-ins

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

5.1.1 Compressible Soils

As discussed in the “Subsurface” section, we encountered layers of compressible clays in the upper 24 to 48 feet below grade. Provided the estimated settlements are acceptable from a structural and architectural perspective, the proposed 7 to 8 story mixed use building structure may be supported on footings or a mat foundation. If the anticipated settlements are too high for the structure, as an alternative it may be supported on deep foundations. Detailed recommendations and a discussion of estimated settlements are presented in the “Foundations” section of this report.

5.1.2 Strong Seismic Shaking

We recommend that, at a minimum, the proposed mixed use 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 7.

5.1.3 Liquefaction-Induced Total and Differential Settlement

Our analyses indicate that one layer theoretically can liquefy, resulting in less than 1/4-inch of total settlement in the upper 50 feet, with differential settlements from liquefaction on the order of less than 1/4-inch in a horizontal distance of 50 feet. The proposed structure should be designed to accommodate the potential seismic as well as static settlements as discussed in the “Foundations” section.

5.1.4 Moderately Expansive Near Surface Soils

To reduce the potential for damage to the planned structures due to the presence of moderately expansive surficial soils, we recommend slabs-on-grade have sufficient reinforcement and be supported on a layer of non-expansive fill and that any shallow footings extend below the zone of seasonal moisture fluctuation. Detailed recommendations are presented in the following sections of this report.

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

5.1.6 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 three 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

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 an 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 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.

6.7 Wet Soils and Wet Weather Conditions

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

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

- Scarify and air dry until the fill materials have a suitable moisture content for compaction,
- Over-excavate the fill and replace with suitable on-site or import materials with an appropriate moisture content,
- Install a layer of geo-synthetic (geotextile or geogrid) to reduce surface yielding and bridge over soft fill,

- Chemically treat the higher moisture content soils with quicklime (CaO), kiln-dust, or cement to reduce the moisture content and increase the strength of the fill.

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

6.8 Trench Backfill

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

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

Utility trenches located adjacent to 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.

6.9 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 Occupational Safety and Health Administration (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.

6.10 Surface Drainage

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

6.11 Landscaping Considerations

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

- Selecting landscaping that requires little or no watering, especially within 3 feet of structures, slabs-on-grade, or pavements,
- Using low flow rate sprinkler heads, or 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.

6.12 Construction Observation

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

7.0 FOUNDATIONS

As discussed in the Conclusions and Recommendations section there is a potential for settlement of compressible soils and liquefaction 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 either on a shallow footing foundation or a reinforced mat foundation as discussed in the sections below.

If the estimated settlements are too high, as an alternative to shallow foundations, the structure may be supported on deep foundations consisting of driven, precast, prestressed concrete friction piles or augered cast-in-place piles. It is our opinion that driven or augercast pile foundations will be able to support the structure with only minor settlements and will provide adequate support during liquefaction and seismic events. Recommendations for friction and augercast-in-place piles, are presented in Sections 7.4 and 7.5, respectively.

7.1 2013 CBC Site Coefficients and Site Seismic Coefficients

Chapter 16 of the 2013 California Building Code (CBC) outlines the procedure for seismic design of structures. Based on our explorations, the site is generally underlain by stiff to hard clays and medium dense 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 7 below.

Table 7. 2013 CBC Site Class and Site Seismic Coefficients

| Latitude: 37.3462 N Longitude: 121.8715 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 Footings

The proposed mixed use structure may be supported on conventional spread footing foundations bearing on natural, undisturbed soil or compacted engineered fill. All footings should have a minimum width of at least 18 inches and footing bottoms should extend at least 24 inches below lowest adjacent finished grade. Lowest adjacent finished grade may be taken as the bottom of interior slab-on-grade or the finished exterior grade, excluding landscape topsoil, whichever is lower.

Footings constructed on native soil or engineered fill in accordance with the above recommendations would be capable of supporting maximum allowable bearing pressures of 2,000 pounds per square foot (psf) for dead loads, 3,000 psf for combined dead and live loads, and 4,000 psf for all loads including wind or seismic. These allowable bearing pressures are based upon factors of safety of 3.0, 2.0, and 1.5 for dead, dead plus live, and seismic loads, respectively.

These maximum allowable bearing pressures are net values; the weight of the footing may be neglected for design purposes. All footings located adjacent to utility trenches should have their bearing surfaces below an imaginary 1:1 (horizontal:vertical) plane projected upward from the bottom edge of the trench to the footing.

All continuous footings should be reinforced with top and bottom steel to provide structural continuity and to help span local irregularities. Footing excavations should be kept moist by regular sprinkling with water to prevent desiccation. It is essential that we observe the all footing excavations before reinforcing steel is placed. If loose soils are encountered, they should be removed from the bottom of the footings.

7.2.1 Footing Foundation Settlement

Based on the estimated loads of 600 kips and the maximum allowable bearing pressures recommended above, we estimate that total static settlement for footings will be up to approximately 2 inches, with differential settlements of $\frac{1}{2}$ -inch 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. Total potential liquefaction induced settlements of $\frac{1}{4}$ -inch differential should also be considered by the structural designer.

7.2.2 Lateral Loads on Footings

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

7.3 Reinforced Mat Foundations

Alternatively, the proposed mixed use 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 1,000 pounds per square foot (psf) for dead plus live loads with maximum localized allowable bearing pressures of 3,000 psf at column or wall loads. Allowable bearing pressures may be increased by one-third for all loads including wind or seismic. These allowable bearing pressures are net values; the weight of the mat can be neglected for design purposes. Mat slabs greater than 8 inches in thickness may be constructed directly on subgrade soils prepared in accordance with the recommendations provided in the Earthwork section.

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

7.3.1 Mat Foundation Settlement

Our calculations for the proposed structure with a reinforced mat foundation with a footprint of 130 feet by 70 feet designed for an average allowable bearing pressure of 1,000 psf for dead plus sustained live loads indicate static settlement of about 3 inches on the center (with differential settlement of 2 inches between center and corner) for a 2-foot-thick mat bearing at the existing site grades.

As discussed in the "Liquefaction" section, differential settlement of mat foundations due to liquefaction may occur during strong ground shaking. To reduce the potential impact of liquefaction-induced settlement, the mats should also be designed to tolerate $\frac{1}{4}$ -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.

7.3.2 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 15 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.

7.3.3 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 1,500 psf at depth. The upper 12 inches of soil should be neglected when determining lateral passive resistance.

7.4 Driven Piles for Structure Foundation

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

Also, as discussed in Section 2.3, beneath the proposed structure, a continuous layer of sand and gravel was encountered in CPT-1 and CPT-2 at an approximate depth of 69 and 67 feet, respectively. This layer may result in higher driving stresses. As discussed below, a wave equation analysis of piles (WEAP) analysis should be performed to evaluate likely driving stresses through this dense sand layer. If the estimated driving stresses are not acceptable, it may be necessary to predrill the piles in some areas. Predrilled holes should have a diameter no greater than the smallest dimension of the precast piles. Variations in subsurface conditions are typical in this area and changes in driving conditions during construction should be expected.

7.4.1 Vertical Loads

Pile support is expected to come predominantly from frictional support in the stiff clays and silts. We computed allowable downward vertical capacities for 14- and 16-inch-square concrete piles. A summary of the allowable pile capacities are presented in Table 8 below. In addition, Figure 4 shows the increase in pile capacity with length. The indicated capacities in Table 8 and Figure 4 are for dead plus live loads. Dead loads should not exceed two-thirds of the computed capacities. Uplift loads should also not exceed two-thirds of the computed downward capacities on Figure 4. The pile capacities may be increased by one-third under transient loading, including wind and seismic.

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

Table 8. Estimated Allowable Capacities for 14-and 16-inch Driven, Precast, Prestressed Concrete Piles

| Pile Size | Length ¹ (feet) | Estimated Allowable Capacity (dead plus live loads) (kips) |
|----------------|----------------------------|--|
| 14-inch square | 38 | 100 |
| 14-inch square | 53 | 150 |
| 14-inch square | 65 | 200 |
| 16-inch square | 33 | 100 |
| 16-inch square | 48 | 150 |
| 16-inch square | 60 | 200 |

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

Based on the maximum allowable loads for a single pile, we estimate total settlements of less than $\frac{3}{4}$ -inch to mobilize allowable static capacities. Therefore, post-construction pile foundation settlements of about $\frac{1}{2}$ -inch should be anticipated.

7.4.2 Lateral Loads on Piles

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

To estimate lateral capacities of piles, we used a computer program that models the soil response in the form of load-deflection (p-y) curves to estimate the capacity of the piles to resist the expected lateral loads. The lateral load characteristics for 14- and 16-inch-square, driven concrete piles with free head and fixed head conditions at 5 feet below subgrade are presented in Table 9. An estimated 150 kip axial load was used.

Table 9. Estimated Lateral Pile Response – 14- and 16-inch Square Piles

| Pile Size | Head Condition | Deflection (inches) | Maximum Shear Force (kips) | Maximum Moment (ft-kips) | Depth to Maximum Moment (ft) |
|-----------|----------------|---------------------|----------------------------|--------------------------|------------------------------|
| 14-inch | Free | $\frac{1}{4}$ | 22 | 57 | 4.8 |
| | | $\frac{1}{2}$ | 30 | 90 | 5.6 |
| | Fixed | $\frac{1}{4}$ | 44 | 146 | Top of Pile |
| | | $\frac{1}{2}$ | 61 | 229 | |
| 16-inch | Free | $\frac{1}{4}$ | 27 | 78 | 5.6 |
| | | $\frac{1}{2}$ | 37 | 123 | 6.4 |
| | Fixed | $\frac{1}{4}$ | 54 | 199 | Top of Pile |
| | | $\frac{1}{2}$ | 74 | 311 | |

The values presented in Table 9 represent the anticipated maximum shear and moment under short-term loading conditions and include no factor-of-safety. Suitable factors-of-safety should be selected on the basis of the type of loading. Pile stiffnesses (EI) of 1.4×10^{10} lb-in² and 2.4×10^{10} lb-in² have been assumed in our calculations of load deflection for the 14- and 16-inch piles, respectively. A minimum compressive strength of 6,000 pounds per square inch was assumed for concrete modulus calculations. If pile stiffness varies by no more than 20 percent than that reported above, load

deflection characteristics may be approximated by multiplying the deflection values by the ratio of the pile stiffness (EI). We should evaluate the response of piles with significantly different stiffness.

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

7.4.3 Passive Resistance Against Pile Caps and Grade Beams

If desired, the passive resistance of soil against pile caps and grade beams poured neat against well-compacted engineered fill or native soil may be used for lateral resistance. We recommend that an allowable passive pressure based on an equivalent fluid pressure of 300 pounds per cubic foot be used in design, with a maximum of 1,500 psf at depth.

7.4.4 WEAP Analysis

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

7.4.5 Indicator Piles

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

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

7.4.6 PDA Monitoring

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

7.4.7 Production Pile Installation

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

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

7.5 Augercast Piles

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

7.5.1 Vertical Capacities

As with driven piles, augercast piles will develop their vertical capacity predominately from frictional support in the stiff clays and silts and dense sands. We computed allowable downward vertical capacities for 18- and 24-inch diameter APG piles. A summary of the allowable pile capacities is presented in Table 10 below. In addition, Figure 4 shows the increase in pile capacity with length. The indicated capacities in Table 10 and Figure 4 are for dead plus live loads. Dead loads should not exceed two-thirds of the computed capacities. Uplift loads should also not exceed two-thirds of the computed downward capacities. The pile capacities may be increased by one-third under transient loading, including wind and seismic.

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

Table 10. Estimated Allowable Capacities for 18-and 24-inch Augercast Piles

| Pile Size | Length ¹ (feet) | Estimated Allowable Capacity (dead plus live loads) (kips) |
|---------------|----------------------------|--|
| 18-inch round | 37 | 100 |
| 18-inch round | 53 | 150 |
| 18-inch round | 64 | 200 |
| 24-inch round | 27 | 100 |
| 24-inch round | 42 | 150 |
| 24-inch round | 53 | 200 |

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

Based on the maximum allowable loads for a single pile, we estimate total settlements of less than $\frac{3}{4}$ -inch to mobilize allowable static capacities. Therefore, post-construction pile foundation settlements of less than $\frac{1}{2}$ -inch should be anticipated.

7.5.2 Lateral Loads on Augercast Piles

To estimate lateral capacities of piles, we used a computer program that models the soil response in the form of load-deflection (p-y) curves to estimate the capacity of the piles to resist the expected lateral loads. The lateral load characteristics for 18- and 24-inch diameter, augercast piles with fixed- and free-head conditions are presented in Table 11 below for 5 feet below grade. A 150 kip axial load was used.

Table 11. Estimated Lateral Pile Response – 18- and 24-inch Round Piles

| Pile Size | Head Condition | Deflection (inches) | Maximum Shear Force (kips) | Maximum Moment (ft-kips) | Depth to Maximum Moment (ft) |
|-----------|----------------|---------------------|----------------------------|--------------------------|------------------------------|
| 18-inch | Free | $\frac{1}{4}$ | 26 | 69 | 4.8 |
| | | $\frac{1}{2}$ | 36 | 110 | 5.6 |
| | Fixed | $\frac{1}{4}$ | 52 | 177 | Top of Pile |
| | | $\frac{1}{2}$ | 71 | 277 | |
| 24-inch | Free | $\frac{1}{4}$ | 41 | 137 | 6.4 |
| | | $\frac{1}{2}$ | 56 | 215 | 7.2 |
| | Fixed | $\frac{1}{4}$ | 80 | 343 | Top of Pile |
| | | $\frac{1}{2}$ | 109 | 532 | |

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

The above lateral load characteristics are for single piles and may not be characteristic of the lateral load capacity of piles in a group. Group effects may reduce the allowable lateral load for a given deflection. We recommend that a pile group efficiency of 0.75 be used for pile groups 3-by-3 or

smaller. For pile groups larger than 3-by-3, we recommend that we review the final pile group layout and structural loads to further evaluate the pile group efficiency under lateral loading.

7.5.3 Passive Resistance Against Pile Caps and Grade Beams

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

7.5.4 Pile Load Tests

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

7.6 Slabs-On-Grade

Due to the moderate expansion potential of the near surface soils, we recommend that slabs-on-grade used in conjunction with shallow footings or deep foundations be supported on at least 6 inches of non-expansive fill (NEF) to reduce the likelihood of slab damage from heave. Structural slabs designed to span between foundation elements do not need to be supported on non-expansive fill. If desired to limit moisture rise through slab-on-grade floors, the guidelines presented in the “Moisture Protection Considerations” section of this report should be considered.

Post-construction cracking of concrete slabs-on-grade is inherent in any project, especially where soil expansion potential is high. In our opinion, consideration should be given toward a maximum control joint spacing of 10 to 15 feet in both directions for the interior slab-on-grade construction. Adequate slab reinforcement should be provided to satisfy the anticipated use and loading requirements.

Due to the moderate expansion potential of the clayey native soil, we recommend that the contractor take special measures to protect the subgrade from any inflow of water during construction, especially after the floor slab has been cast. Areas to receive special attention include slab joints and areas where building columns pass through the floor slab.

7.7 Garage Floor Slabs

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

If desired to limit moisture rise through garage slabs, the guidelines presented in the “Moisture Protection Considerations” Section 7.8 below should be considered.

7.8 Moisture Protection Considerations for Slabs-on-Grade

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

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

8.0 RETAINING WALLS

8.1 Lateral Earth Pressures

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

with adequate drainage should be designed to resist an equivalent fluid pressure of 45 pcf plus one-third of any surcharge loads.

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

8.2 Seismic Lateral Earth Pressures

We understand the below-grade 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. Our calculations show that if the below-grade walls are designed using the static lateral earth pressures for restrained wall conditions as provided above, there is no need to consider additional seismic lateral earth pressures in seismic design of the walls.

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, 1/2- to 3/4-inch crushed rock may be used in place of the Class 2 Permeable Material provided the crushed rock and pipe are enclosed in filter fabric, such as Mirafi 140N or equivalent. The upper 2 feet of wall backfill should consist of relatively low permeable compacted on-site clayey soil. The subdrain outlet should be connected to a free-draining outlet or sump.

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

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

9.0 PAVEMENTS

9.1 Asphalt Concrete

Based on the subsurface soils encountered, we estimated an R-value to provide data for pavement design. We judge an R-value of 15 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 12.

Table 12. Recommended Asphalt Concrete Pavement Design Alternatives
Pavement Components
Design R-Value = 15

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

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

The traffic indices used in our pavement design are considered reasonable values for the proposed development and should provide a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance. 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 13. 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 13. Recommended Minimum PCC Pavement Thickness

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

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

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 First Community Housing, specifically for design of the proposed Mixed Use Development at 21 North 21st Street project in San Jose, California. The

opinions, conclusions, and recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in the San Francisco Bay Area at the time this report was written. No other warranty, expressed or implied, is made or should be inferred.

The opinions, conclusions and recommendations contained in this report are based upon the information obtained from our 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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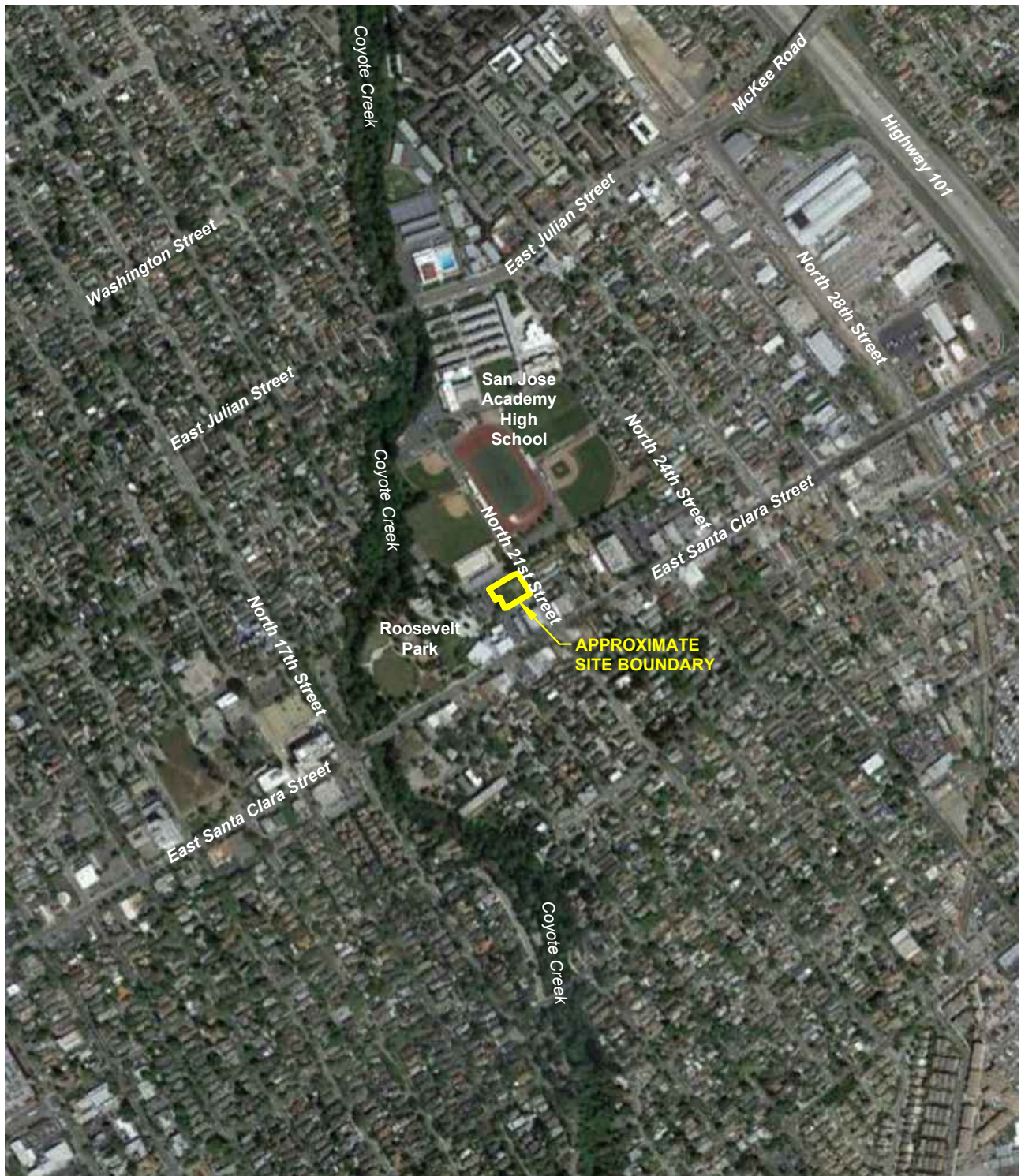
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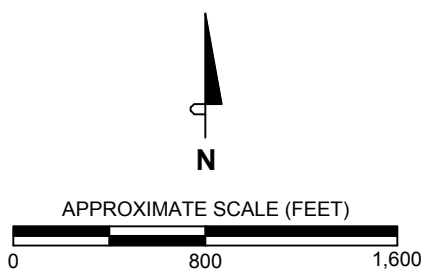
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* * * * *



SOURCE AERIAL PHOTO: Google Earth, April 2016.



VICINITY MAP

Mixed Use Development
21 North 21st Street
San Jose, California



264960



FIGURE 1

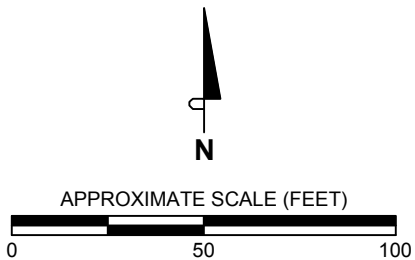


SOURCE AERIAL PHOTO: Google Earth, April 2016.

LEGEND

Approximate locations of:

-  Cone penetration test (CPT)
-  Exploratory boring



SITE PLAN

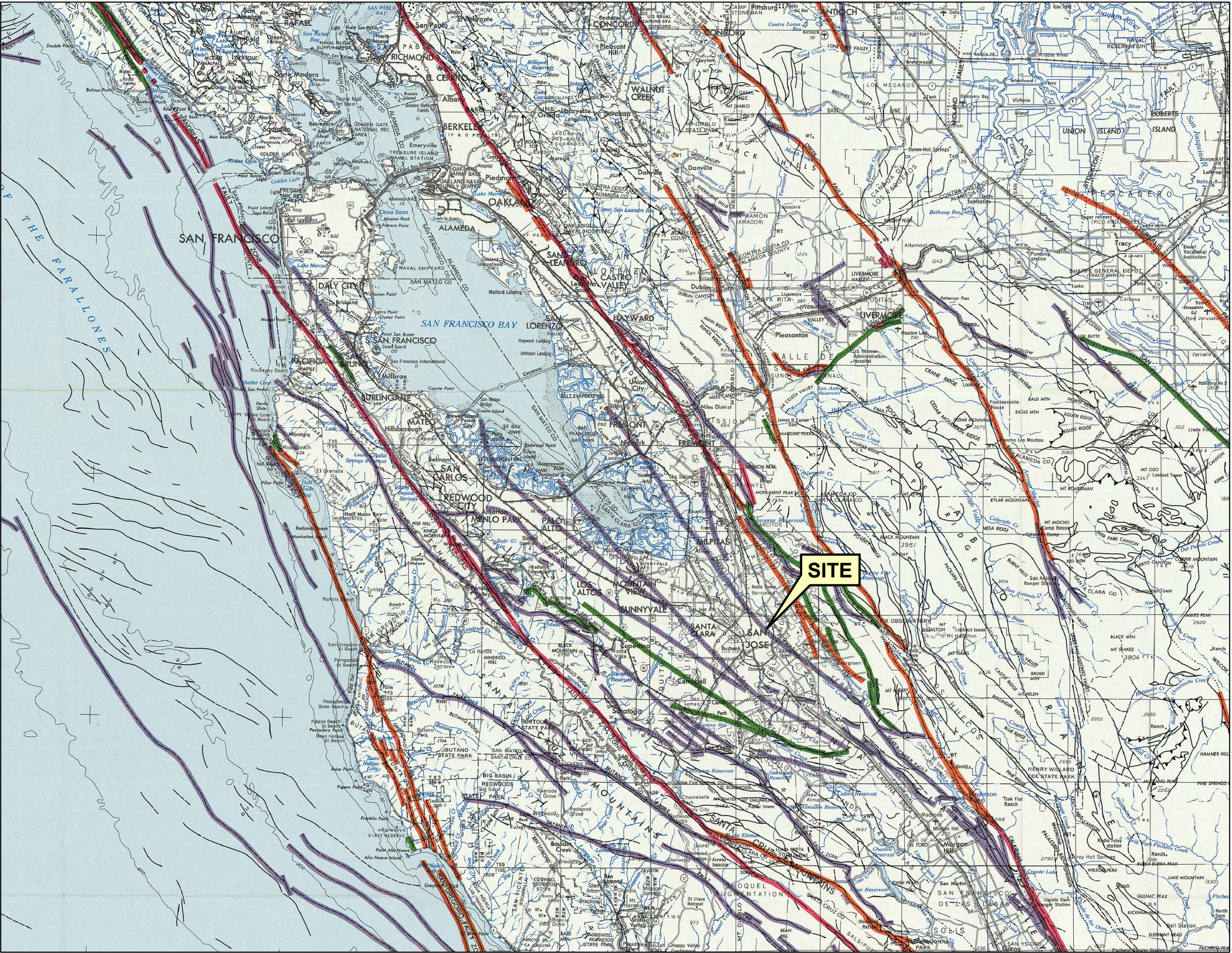
Mixed Use Development
21 North 21st Street
San Jose, California



264960

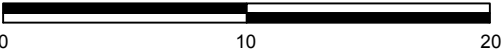
FIGURE 2

FILE NAME: Z:\CAD_DRAW\Current\Mixed Use Development_San Jose\Fig3_Regional Fault Map.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 | Holocene/Historic | 200 | | | Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep. |
| | | | 10,000 | | | Displacement during Holocene time. ² |
| | Early Quaternary | Pleistocene | 700,000 | | | Faults showing evidence of displacement during late Quaternary time. ^{3,4} |
| | | | 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 | | | 2,000,000 | | | |
| | | | 5,000,000 | | | Fault showing evidence of no displacement during Quaternary time or faults without recognized Quaternary displacement. |

NOTES:

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

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

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

From: Bortugno & others (1991)

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

REGIONAL FAULT MAP

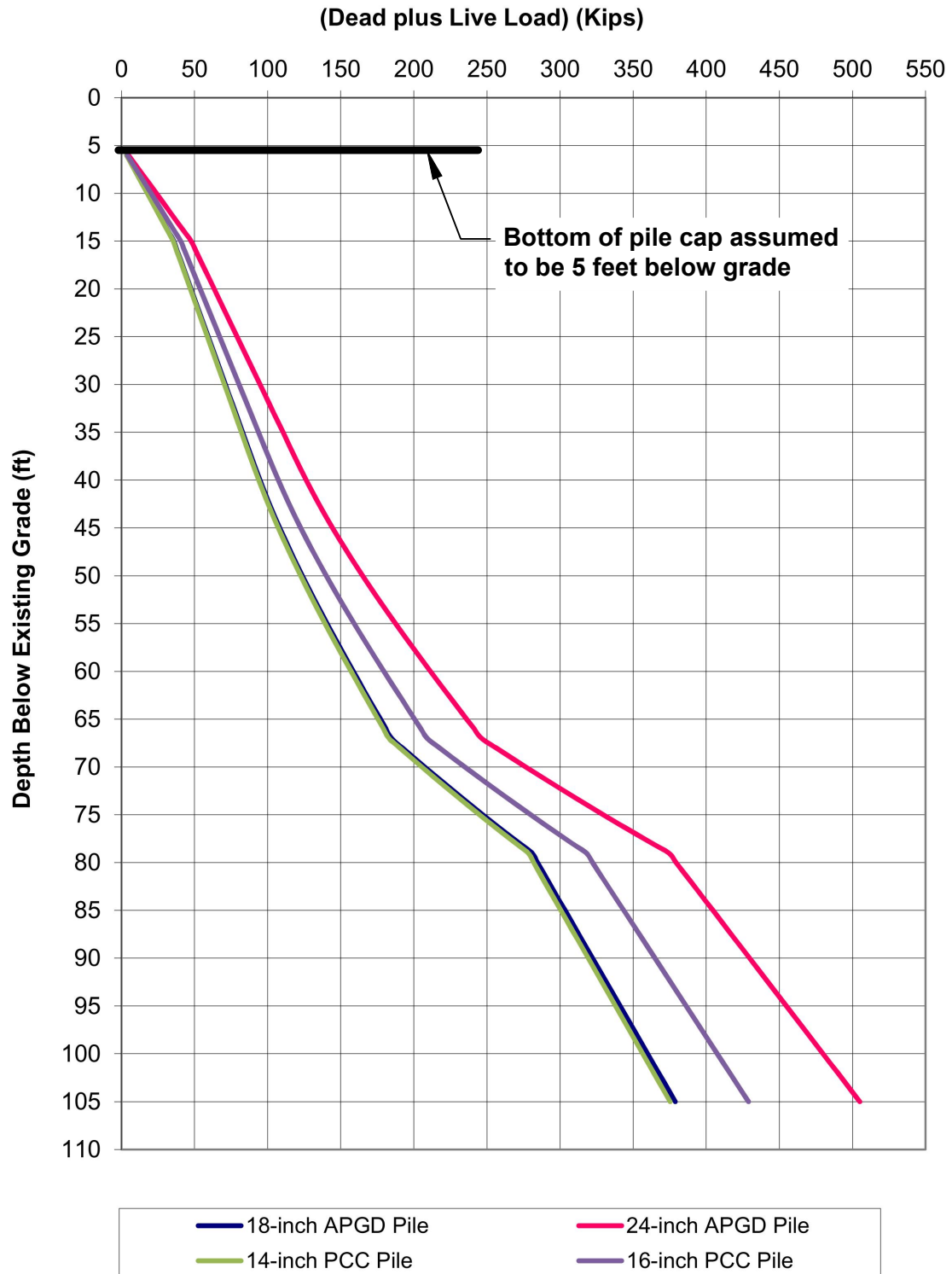
Mixed Use Development
21 North 21st Street
San Jose, California



264960

FIGURE 3

Allowable Downward Vertical Capacity



VERTICAL PILE CAPACITIES

Mixed Use Development
21 North 21st Street
San Jose, California



264960

FIGURE 4

APPENDIX A

FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using conventional, truck-mounted, hollow-stem auger drilling equipment and cone penetration test (CPT) equipment. Four 8-inch diameter exploratory borings were drilled on October 18, 2016 to a maximum depth of 75 feet. Two CPTs were advanced on October 26, 2016 to a maximum depth of 120 feet. The approximate locations of the exploratory borings and CPTs are shown on Figure 2. The soils encountered in the borings were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). The logs of the borings and CPTs, as well as a key to the classification of the soil and CPTs, are included as part of this appendix.

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

Representative soil samples were obtained from the 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.

* * * * *

EXPLORATORY BORING: EB-1

Sheet 1 of 3

DRILL RIG: MOBILE B-61

BORING TYPE: 8-INCH HOLLOW-STEM AUGER

LOGGED BY: AC

START DATE: 10-18-16

FINISH DATE: 10-18-16

PROJECT NO: 264960

PROJECT: MIXED USE DEVELOPMENT AT 21 N. 21ST ST.

LOCATION: SAN JOSE, CA

COMPLETION DEPTH: 75.0 FT.

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

| ELEVATION (FT) | DEPTH (FT) | SOIL LEGEND | MATERIAL DESCRIPTION AND REMARKS | SOIL TYPE | PENETRATION RESISTANCE (BLOWS/FT.) | SAMPLER | MOISTURE CONTENT (%) | DRY DENSITY (PCF) | PERCENT PASSING NO. 200 SIEVE | Undrained Shear Strength (ksf) |
|----------------|------------|-------------|---|-----------|------------------------------------|---------|----------------------|-------------------|-------------------------------|--------------------------------|
| | 0 | | SURFACE ELEVATION: | | | | | | | |
| | | | LEAN CLAY WITH SAND (CL) very stiff, moist, brown, moderate plasticity, fine sand, trace rootlets Liquid Limit = 31, Plasticity Index = 11 | CL | 13 | | 9 | 83 | | |
| | | | hard | | 14 | | 12 | 88 | | |
| | 5 | | SANDY LEAN CLAY (CL) very stiff, moist, brown, low to moderate plasticity, fine sand | CL | 13 | | 11 | 87 | | |
| | | | hard | | 13 | | 12 | 100 | | |
| | 10 | | | | | | | | | |
| | | | LEAN CLAY WITH SAND (CL) stiff, moist, brown, low to moderate plasticity, fine sand | CL | 17 | | 21 | 100 | | |
| | 15 | | | | | | | | | |
| | | | CLAYEY SAND (SC) medium dense, moist, dark brown, fine sand, low plasticity | SP-SC | | | | | | |
| | | | LEAN CLAY (CL) stiff, moist, dark brown, moderate plasticity | | 22 | | 21 | 97 | | |
| | 20 | | | | | | | | | |
| | | | dark olive brown | CL | 20 | | 27 | 92 | | |
| | 25 | | | | | | | | | |
| | | | medium stiff | CL | 12 | | 24 | 97 | | |
| | 30 | | | | | | | | | |

Continued Next Page

GROUND WATER OBSERVATIONS:

▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 22.0 FEET

LA CORP GDT 11/4/16 MV, CA*



EB-1
264960

EXPLORATORY BORING: EB-1 Cont'd

Sheet 2 of 3

DRILL RIG: MOBILE B-61

BORING TYPE: 8-INCH HOLLOW-STEM AUGER

LOGGED BY: AC

START DATE: 10-18-16

FINISH DATE: 10-18-16

PROJECT NO: 264960

PROJECT: MIXED USE DEVELOPMENT AT 21 N. 21ST ST.

LOCATION: SAN JOSE, CA

COMPLETION DEPTH: 75.0 FT.

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

| ELEVATION (FT) | DEPTH (FT) | SOIL LEGEND | MATERIAL DESCRIPTION AND REMARKS | SOIL TYPE | PENETRATION RESISTANCE (BLOWS/FT.) | SAMPLER | MOISTURE CONTENT (%) | DRY DENSITY (PCF) | PERCENT PASSING NO. 200 SIEVE | Undrained Shear Strength (ksf) |
|----------------|------------|-------------|---|-----------|------------------------------------|---------|----------------------|-------------------|-------------------------------|--|
| | | | | | | | | | | ○ Pocket Penetrometer △ Torvane ● Unconfined Compression ▲ U-U Triaxial Compression |
| | 30 | | SANDY LEAN CLAY (CL) medium stiff, moist, dark brown, low to moderate plasticity, fine sand | CL | | | | | | 1.0 2.0 3.0 4.0 |
| | | | LEAN CLAY (CL) stiff, moist, olive brown to gray, moderate plasticity | | | | | | | |
| | 35 | | soft | CL | 13 | ✕ | 25 | 100 | | |
| | | | stiff | | | | | | | |
| | 40 | | CLAYEY SAND (SC) medium dense, wet, olive brown, fine sand | SC | 16 | ✕ | 26 | 93 | | |
| | | | LEAN CLAY (CL) medium stiff, moist, olive brown to gray, moderate plasticity | | | | | | | |
| | 45 | | dark gray | | 12 | ✕ | 34 | 86 | | |
| | | | stiff, greenish gray | CL | 22 | ✕ | 37 | 85 | | |
| | 55 | | | | 33 | ✕ | 34 | 88 | | |
| | 60 | | very stiff | | 30 | ✕ | 28 | 94 | | |

Continued Next Page

GROUND WATER OBSERVATIONS:

▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 22.0 FEET

LA CORP.GDT 11/4/16 MV, CA*



EB-1
264960

EXPLORATORY BORING: EB-1 Cont'd

Sheet 3 of 3

DRILL RIG: MOBILE B-61

BORING TYPE: 8-INCH HOLLOW-STEM AUGER

LOGGED BY: AC

START DATE: 10-18-16

FINISH DATE: 10-18-16

PROJECT NO: 264960

PROJECT: MIXED USE DEVELOPMENT AT 21 N. 21ST ST.

LOCATION: SAN JOSE, CA

COMPLETION DEPTH: 75.0 FT.

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

| ELEVATION (FT) | DEPTH (FT) | SOIL LEGEND | MATERIAL DESCRIPTION AND REMARKS | SOIL TYPE | PENETRATION RESISTANCE (BLOWS/FT.) | SAMPLER | MOISTURE CONTENT (%) | DRY DENSITY (PCF) | PERCENT PASSING NO. 200 SIEVE | Undrained Shear Strength (ksf) | | | | |
|----------------|------------|-------------|--|-----------|------------------------------------|---------|----------------------|-------------------|-------------------------------|--------------------------------|-----|-----|-----|--|
| | | | | | | | | | | 1.0 | 2.0 | 3.0 | 4.0 | |
| | 60 | | LEAN CLAY (CL) very stiff, moist, greenish gray, moderate plasticity | CL | | | | | | | | | | |
| | | | SANDY LEAN CLAY (CL) very stiff, moist, greenish gray, low to moderate plasticity, fine sand | CL | 35 | ✕ | 26 | 98 | | | | | | |
| | 65 | | LEAN CLAY (CL) stiff, moist, greenish gray, moderate plasticity | CL | | | | | | | | | | |
| | | | SANDY LEAN CLAY (CL) stiff, moist, dark olive brown to gray, low to moderate plasticity | CL | 14 | ✕ | 25 | 95 | | | | | | |
| | 70 | | CLAYEY SAND (SC) medium dense, wet, greenish gray, fine sand, low plasticity | SC | | | | | | | | | | |
| | | | POORLY GRADED SAND WITH CLAY (SP-SC) dense, wet, brown, fine to coarse sand, low plasticity | SP-SC | 31 | ✕ | | | | | | | | |
| | 75 | | Bottom of boring at 75 feet | | | | | | | | | | | |
| | 80 | | | | | | | | | | | | | |
| | 85 | | | | | | | | | | | | | |
| | 90 | | | | | | | | | | | | | |

GROUND WATER OBSERVATIONS:

▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 22.0 FEET

LA CORP.GDT 11/4/16 MV, CA*



EB-1
264960

EXPLORATORY BORING: EB-2

Sheet 1 of 2

DRILL RIG: MOBILE B-61

BORING TYPE: 8-INCH HOLLOW-STEM AUGER

LOGGED BY: AC

START DATE: 10-18-16

FINISH DATE: 10-18-16

PROJECT NO: 264960

PROJECT: MIXED USE DEVELOPMENT AT 21 N. 21ST ST.

LOCATION: SAN JOSE, CA

COMPLETION DEPTH: 35.0 FT.

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

| ELEVATION (FT) | DEPTH (FT) | SOIL LEGEND | MATERIAL DESCRIPTION AND REMARKS | SOIL TYPE | PENETRATION RESISTANCE (BLOWS/FT.) | SAMPLER | MOISTURE CONTENT (%) | DRY DENSITY (PCF) | PERCENT PASSING NO. 200 SIEVE | Undrained Shear Strength (ksf) |
|----------------|------------|-------------|--|-----------|------------------------------------|---------|----------------------|-------------------|-------------------------------|--|
| | 0 | | SURFACE ELEVATION: | | | | | | | ○ Pocket Penetrometer △ Torvane ● Unconfined Compression ▲ U-U Triaxial Compression |
| | 0 | | LEAN CLAY (CL) stiff, moist, light brown, low to moderate plasticity, trace fine sand and rootlets | | 8 | | 8 | 73 | | |
| | 1 | | | CL | 13 | | 12 | 88 | | |
| | 2 | | hard | | | | | | | |
| | 3 | | | | 12 | | 9 | 82 | | |
| | 4 | | stiff, brown | | | | | | | |
| | 5 | | | | | | | | | |
| | 6 | | SANDY LEAN CLAY (CL) very stiff, moist, brown, low to moderate plasticity, fine sand | | 15 | | 10 | 94 | | |
| | 7 | | | CL | | | | | | |
| | 8 | | | | | | | | | |
| | 9 | | | | | | | | | |
| | 10 | | | | | | | | | |
| | 11 | | | | | | | | | |
| | 12 | | | | | | | | | |
| | 13 | | | | | | | | | |
| | 14 | | | | | | | | | |
| | 15 | | LEAN CLAY (CL) very stiff, moist, brown, moderate plasticity, trace fine sand | | 15 | | 21 | 91 | | |
| | 16 | | | | | | | | | |
| | 17 | | | | | | | | | |
| | 18 | | | | | | | | | |
| | 19 | | | | | | | | | |
| | 20 | | hard, Liquid Limit = 32, Plasticity Index = 14 | | 23 | | 21 | 98 | | |
| | 21 | | | CL | | | | | | |
| | 22 | | | | | | | | | |
| | 23 | | | | | | | | | |
| | 24 | | stiff, dark brown | | 20 | | 20 | 101 | | |
| | 25 | | | | | | | | | |
| | 26 | | | | | | | | | |
| | 27 | | | | | | | | | |
| | 28 | | | | | | | | | |
| | 29 | | | | | | | | | |
| | 30 | | olive brown | | 16 | | 24 | 97 | | |
| | 31 | | | | | | | | | |
| | 32 | | | | | | | | | |
| | 33 | | | | | | | | | |
| | 34 | | | | | | | | | |
| | 35 | | | | | | | | | |

Continued Next Page

GROUND WATER OBSERVATIONS:

▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 23.0 FEET

LA CORP GDT 11/4/16 MV, CA*



EB-2
264960

Sheet 2 of 2

PROJECT NO: 264960

PROJECT: MIXED USE DEVELOPMENT AT 21 N. 21ST ST.

LOCATION: SAN JOSE, CA

FINISH DATE: 10-18-16

COMPLETION DEPTH: 35.0 FT.

[illegible]

▽: FREE GROUND WATER MEASURED DURING DRILLING AT 23.0 FEET

EXPLORATORY BORING: EB-3

Sheet 1 of 2

DRILL RIG: MOBILE B-61

BORING TYPE: 8-INCH HOLLOW-STEM AUGER

LOGGED BY: AC

START DATE: 10-19-16

FINISH DATE: 10-19-16

PROJECT NO: 264960

PROJECT: MIXED USE DEVELOPMENT AT 21 N. 21ST ST.

LOCATION: SAN JOSE, CA

COMPLETION DEPTH: 50.0 FT.

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

| ELEVATION (FT) | DEPTH (FT) | SOIL LEGEND | MATERIAL DESCRIPTION AND REMARKS | SOIL TYPE | PENETRATION RESISTANCE (BLOWS/FT.) | SAMPLER | MOISTURE CONTENT (%) | DRY DENSITY (PCF) | PERCENT PASSING NO. 200 SIEVE | Undrained Shear Strength (ksf) |
|----------------|------------|-------------|---|-----------|------------------------------------|---------|----------------------|-------------------|-------------------------------|--|
| | | | | | | | | | | ○ Pocket Penetrometer △ Torvane ● Unconfined Compression ▲ U-U Triaxial Compression |
| | 0 | | SURFACE ELEVATION: | | | | | | | 1.0 2.0 3.0 4.0 |
| | | | LEAN CLAY WITH SAND (CL) very stiff, moist, light brown, low to moderate plasticity, fine sand, trace medium to coarse sand and fine gravel (sub-angular/rounded) | CL | 22 | | 8 | 91 | | |
| | | | LEAN CLAY (CL) hard, moist, brown, low to moderate plasticity, trace fine sand | CL | 13 | | 12 | 90 | | |
| | 5 | | | CL | 9 | | 10 | 87 | | |
| | | | very stiff | CL | 15 | | 14 | 98 | | |
| | | | SANDY LEAN CLAY (CL) very stiff, moist, brown, low to moderate plasticity, fine sand, trace rootlets | CL | 14 | | 21 | 97 | | |
| | 15 | | | CL | 13 | | 24 | 98 | | |
| | | | stiff, dark olive brown | CL | 20 | | 27 | 95 | | |
| | 20 | | LEAN CLAY (CL) medium stiff, moist, dark olive brown, moderate plasticity | CL | 12 | | 22 | 100 | | |
| | | | stiff | CL | | | | | | |
| | 25 | | | CL | | | | | | |
| | | | medium stiff, trace fine sand | CL | | | | | | |
| | 30 | | | CL | | | | | | |

Continued Next Page

GROUND WATER OBSERVATIONS:

▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 28.0 FEET

LA CORP GDT 11/4/16 MV, CA*



EB-3
264960

EXPLORATORY BORING: EB-3 Cont'd

Sheet 2 of 2

DRILL RIG: MOBILE B-61

BORING TYPE: 8-INCH HOLLOW-STEM AUGER

LOGGED BY: AC

START DATE: 10-19-16

FINISH DATE: 10-19-16

PROJECT NO: 264960

PROJECT: MIXED USE DEVELOPMENT AT 21 N. 21ST ST.

LOCATION: SAN JOSE, CA

COMPLETION DEPTH: 50.0 FT.

| ELEVATION (FT) | DEPTH (FT) | SOIL LEGEND | MATERIAL DESCRIPTION AND REMARKS | SOIL TYPE | PENETRATION RESISTANCE (BLOWS/FT.) | SAMPLER | MOISTURE CONTENT (%) | DRY DENSITY (PCF) | PERCENT PASSING NO. 200 SIEVE | Undrained Shear Strength (ksf) | | | |
|-------------------|---------------|-------------|---|-----------|--|---------|-------------------------|----------------------|----------------------------------|-----------------------------------|-----------|--------------------------|----------------------------|
| | | | | | | | | | | ○ Pocket Penetrometer | △ Torvane | ● Unconfined Compression | ▲ U-U Triaxial Compression |
| | 30 | | SANDY LEAN CLAY (CL) medium stiff, moist, dark brown, fine sand | CL | | | | | | | | | |
| | | | LEAN CLAY (CL) medium stiff, moist, dark greenish gray, moderate plasticity | | | | | | | | | | |
| | 35 | | | CL | 14 | ✕ | 25 | 95 | | ○ | | | |
| | | | stiff | | | | | | | | | | |
| | 40 | | CLAYEY SAND (SC) medium dense, wet, greenish gray, fine sand, low plasticity | SC | 13 | ✕ | 24 | 96 | | | ○ | | |
| | | | LEAN CLAY (CL) stiff, moist, dark greenish gray, moderate plasticity | CL | | | | | | | | | |
| | 45 | | CLAYEY SAND (SC) medium dense, wet, dark olive brown, fine sand, low plasticity | SC | 15 | ✕ | 27 | 91 | | | ○ | | |
| | | | LEAN CLAY (CL) stiff, moist, greenish gray, moderate plasticity | CL | | | | | | | | | |
| | 50 | | Bottom of boring at 50 feet | | 23 | ✕ | 35 | 85 | | | ○ | | |
| | 55 | | | | | | | | | | | | |
| | 60 | | | | | | | | | | | | |

GROUND WATER OBSERVATIONS:

▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 28.0 FEET

LA CORP.GDT 11/4/16 MV, CA*



EB-3
264960

EXPLORATORY BORING: EB-4

Sheet 1 of 2

DRILL RIG: MOBILE B-61

BORING TYPE: 8-INCH HOLLOW-STEM AUGER

LOGGED BY: AC

START DATE: 10-18-16

FINISH DATE: 10-18-16

PROJECT NO: 264960

PROJECT: MIXED USE DEVELOPMENT AT 21 N. 21ST ST.

LOCATION: SAN JOSE, CA

COMPLETION DEPTH: 35.0 FT.

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

| ELEVATION (FT) | DEPTH (FT) | SOIL LEGEND | MATERIAL DESCRIPTION AND REMARKS | SOIL TYPE | PENETRATION RESISTANCE (BLOWS/FT.) | SAMPLER | MOISTURE CONTENT (%) | DRY DENSITY (PCF) | PERCENT PASSING NO. 200 SIEVE | Undrained Shear Strength (ksf) |
|----------------|------------|-------------|--|-----------|------------------------------------|---------|----------------------|-------------------|-------------------------------|--------------------------------|
| | 0 | | SURFACE ELEVATION: | | | | | | | |
| | | | LEAN CLAY WITH SAND (CL) hard, moist, light brown, low to moderate plasticity, fine sand, trace rootlets | CL | 11 | | 7 | 81 | | |
| | | | brown | | 13 | | 8 | 90 | | |
| | 5 | | SANDY LEAN CLAY (CL) very stiff, moist, light brown, low to moderate plasticity, fine sand | CL | 15 | | 7 | 87 | | |
| | | | LEAN CLAY WITH SAND (CL) hard, moist, brown, low to moderate plasticity, fine sand | CL | 17 | | 12 | 95 | | |
| | 10 | | | | | | | | | |
| | | | stiff | | 16 | | 25 | 97 | | |
| | 15 | | LEAN CLAY (CL) stiff, moist, brown, low to moderate plasticity | CL | 19 | | 23 | 100 | | |
| | | | medium stiff, dark brown, moderate plasticity | | | | | | | |
| | 20 | | | | | | | | | |
| | | | very stiff, olive brown | | 20 | | 26 | 98 | | |
| | 25 | | | | | | | | | |
| | | | medium stiff | SC | | | | | | |
| | 30 | | | | | | | | | |

Continued Next Page

GROUND WATER OBSERVATIONS:

▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 23.5 FEET

LA CORP.GDT 11/4/16 MV, CA*



EB-4
264960

EXPLORATORY BORING: EB-4 Cont'd

Sheet 2 of 2

DRILL RIG: MOBILE B-61

BORING TYPE: 8-INCH HOLLOW-STEM AUGER

LOGGED BY: AC

START DATE: 10-18-16

FINISH DATE: 10-18-16

PROJECT NO: 264960

PROJECT: MIXED USE DEVELOPMENT AT 21 N. 21ST ST.

LOCATION: SAN JOSE, CA

COMPLETION DEPTH: 35.0 FT.

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

MATERIAL DESCRIPTION AND REMARKS

30

35

35

40

45

50

55

60

CLAYEY SAND (SC)
medium dense, wet, brown, fine sand, low plasticity

LEAN CLAY (CL)
stiff, moist, brown, moderate plasticity, trace fine sand
Liquid Limit = 31, Plasticity Index = 10

soft, dark olive brown to gray

SC

CL

19

11

30

23

102

Undrained Shear Strength (ksf)

○ Pocket Penetrometer

△ Torvane

● Unconfined Compression

▲ U-U Triaxial Compression

1.0 2.0 3.0 4.0

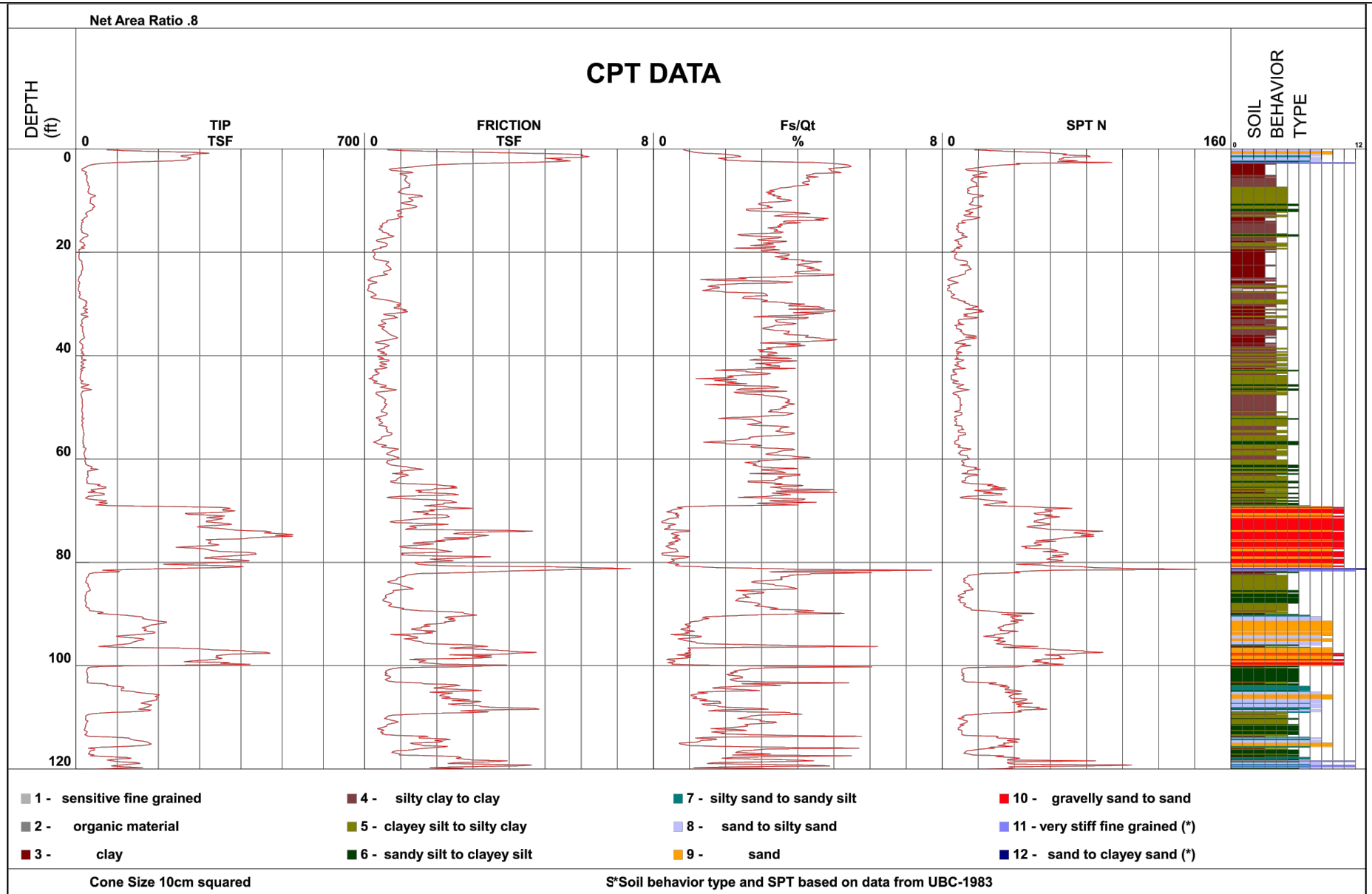
GROUND WATER OBSERVATIONS:

▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 23.5 FEET

LA CORP.GDT 11/4/16 MV, CA*



EB-4
264960

**CONE PENETRATION TEST - CPT-1**

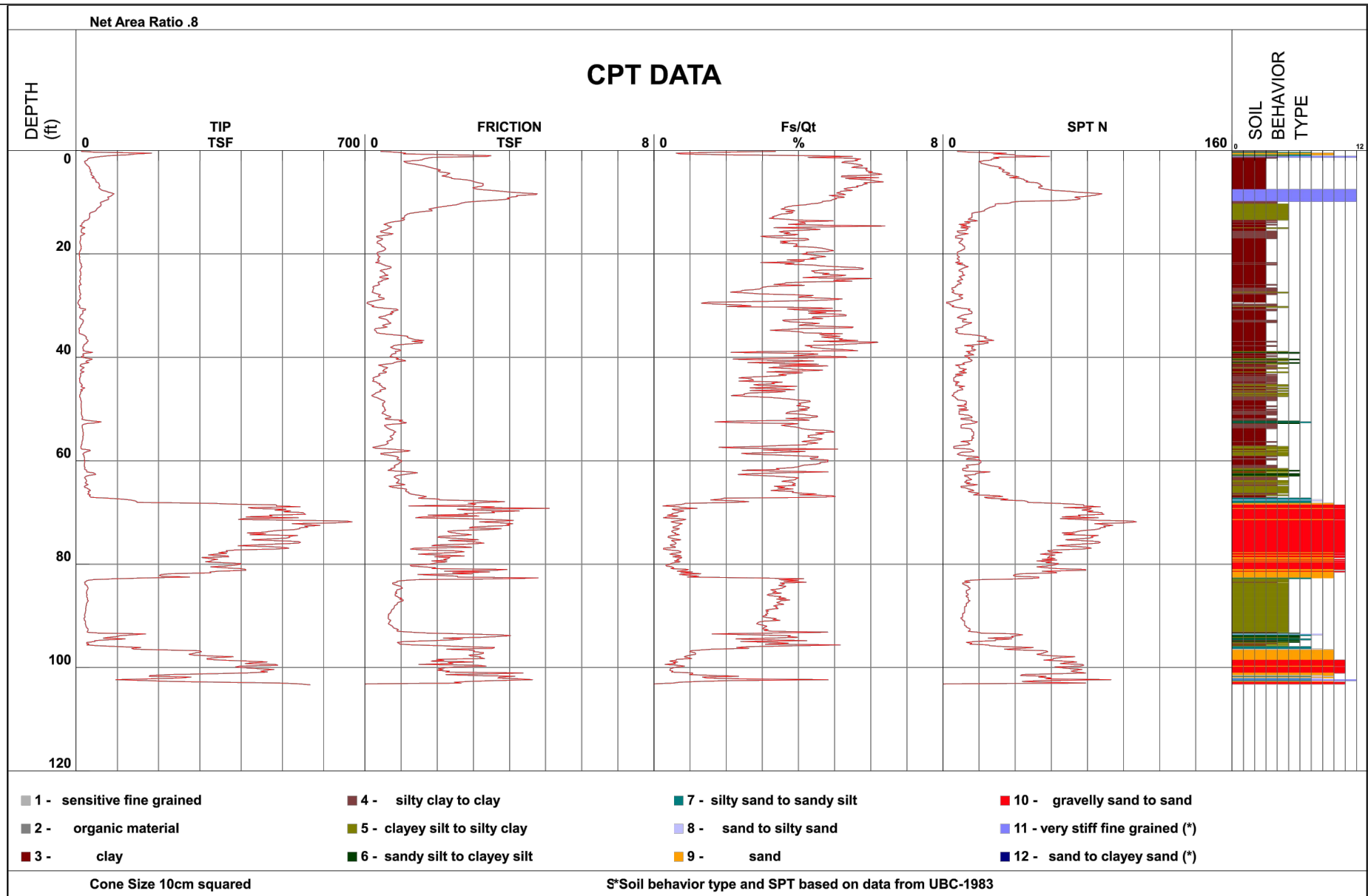
Mixed Use Development
21 North 21st Street
San Jose, California



264960

CPT-1

FILE NAME: Z:\CAD_DRAWING\Current\Mixed Use Development_San Jose\CPT-2.dwg | Layout Tab: 8x11



CONE PENETRATION TEST - CPT-2

Mixed Use Development
21 North 21st Street
San Jose, California



264960

CPT-2

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 the materials recovered from the boring. These water contents are recorded on the boring log at the appropriate sample depths.

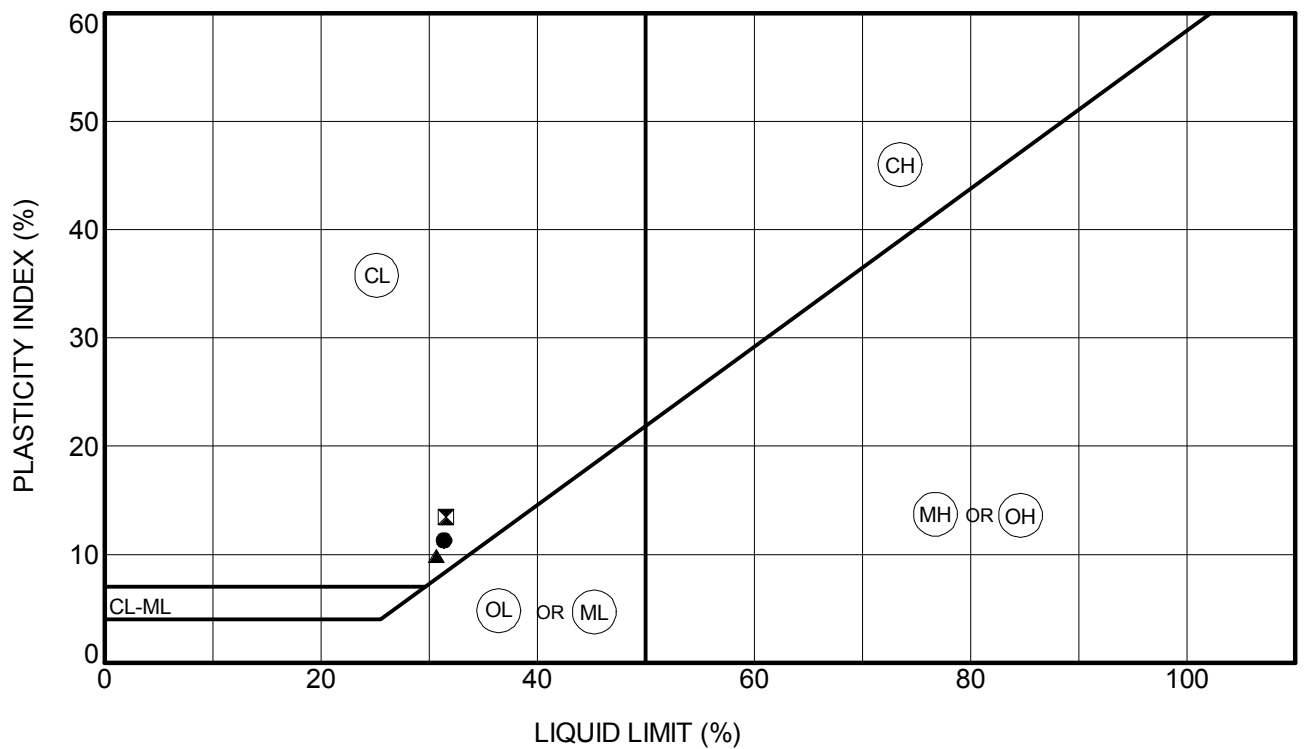
Dry Densities: In place dry density tests (ASTM D2937) were performed to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring log at the appropriate sample depths.

Plasticity Index: 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 samples of the subsurface soils to aid in the classification of these soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Consolidation: A consolidation test (ASTM D2435) was performed on a relatively undisturbed sample of the subsurface clayey soils to assist in evaluating the compressibility properties of the soils. Result of the consolidation test is presented graphically in this appendix.

* * * * *



| Symbol | Boring No. | Depth (ft.) | Natural Water Content (%) | Liquid Limit (%) | Plastic Limit (%) | Plasticity Index (%) | Passing No. 200 Sieve | Unified Soil Classification Description |
|--------|------------|-------------|---------------------------|------------------|-------------------|----------------------|-----------------------|---|
| ● | EB-1 | 2.0 | | 31 | 20 | 11 | | |
| ⊠ | EB-2 | 19.5 | | 32 | 18 | 14 | | |
| ▲ | EB-4 | 31.0 | | 31 | 21 | 10 | | |
| | | | | | | | | |
| | | | | | | | | |
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PLASTICITY CHART AND DATA

Project: MIXED USE DEVELOPMENT AT 21 NORTH 21ST STREET

Location: SAN JOSE, CA

Project No.: 264960

FIGURE B-1





Remarks:

[illegible]

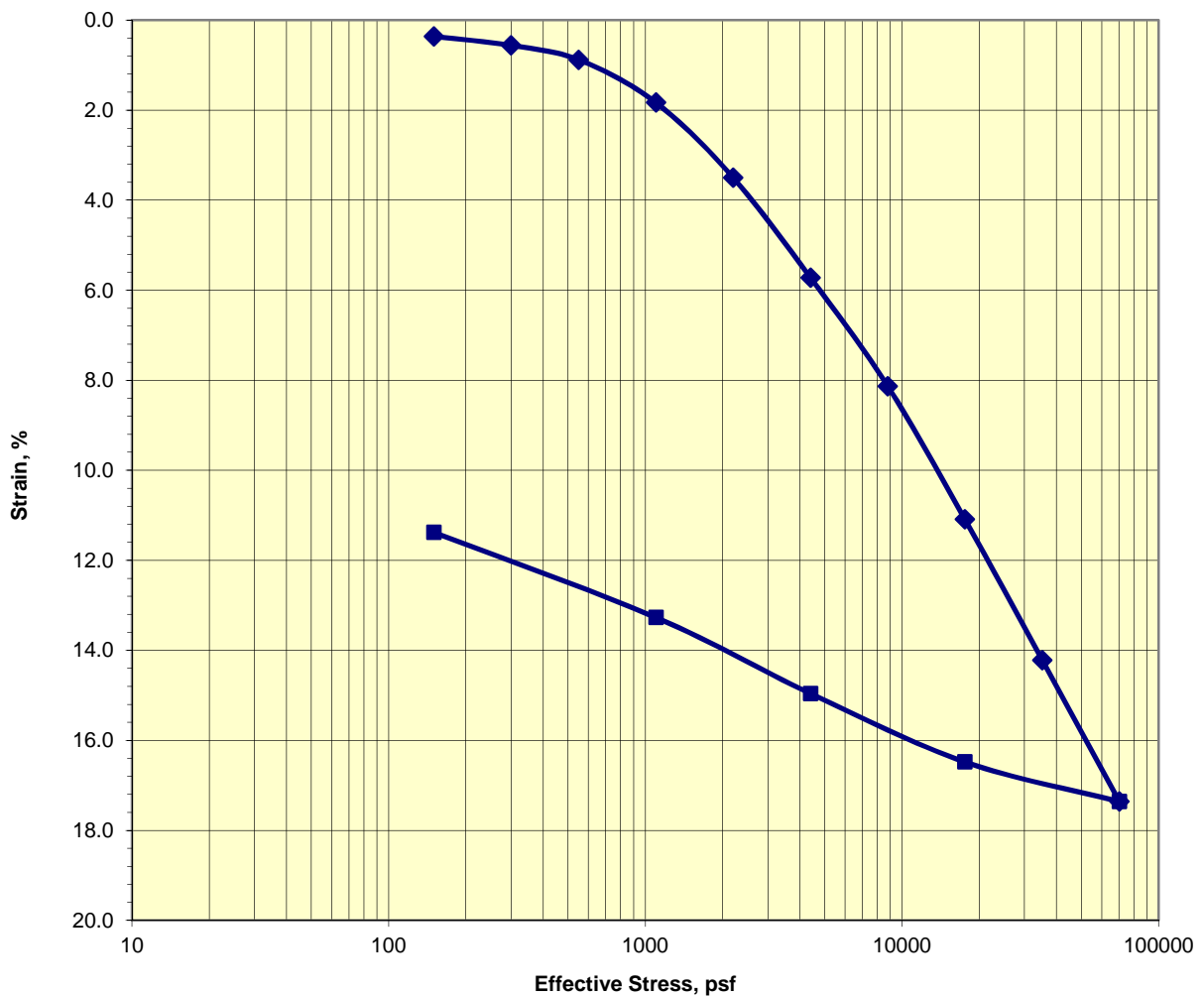


Consolidation Test

ASTM D2435

| | | |
|--|---------------------------|-----------------|
| Job No.: 028-2607 | Boring: EB-4 | Run By: MD |
| Client: TRC | Sample: 8A | Reduced: PJ |
| Project: Mixed Use 21 N 21st Street - 264960 | Depth, ft.: 28.5(Top-10") | Checked: PJ/DC |
| Soil Type: Grayish Brown Sandy CLAY | | Date: 11/7/2016 |

Strain-Log-P Curve



| | | | | |
|-------------------|-----|---------|-------|----------|
| Assumed Gs | 2.8 | Initial | Final | Remarks: |
| Moisture %: | | 25.1 | 19.2 | |
| Dry Density, pcf: | | 101.9 | 113.6 | |
| Void Ratio: | | 0.715 | 0.538 | |
| % Saturation: | | 98.2 | 100.0 | |

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).

* * * * * * * * * *

LIQUEFACTION ANALYSIS REPORT

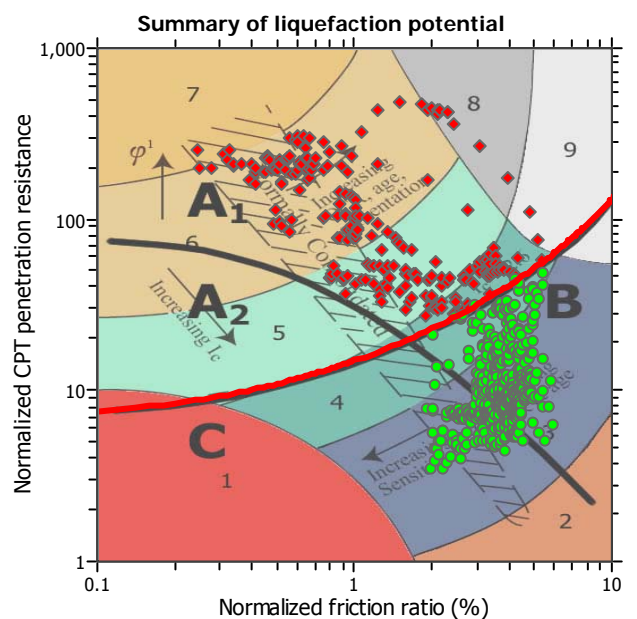
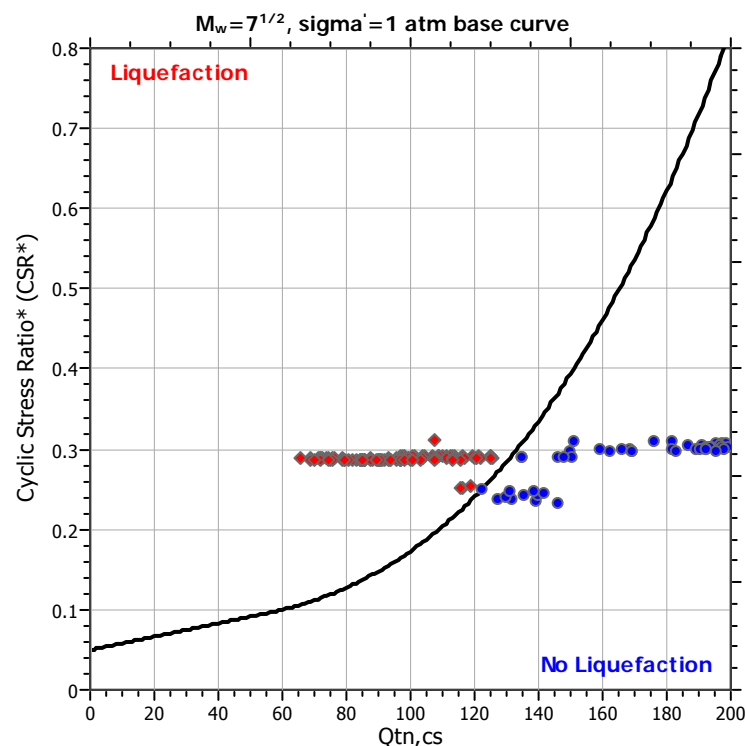
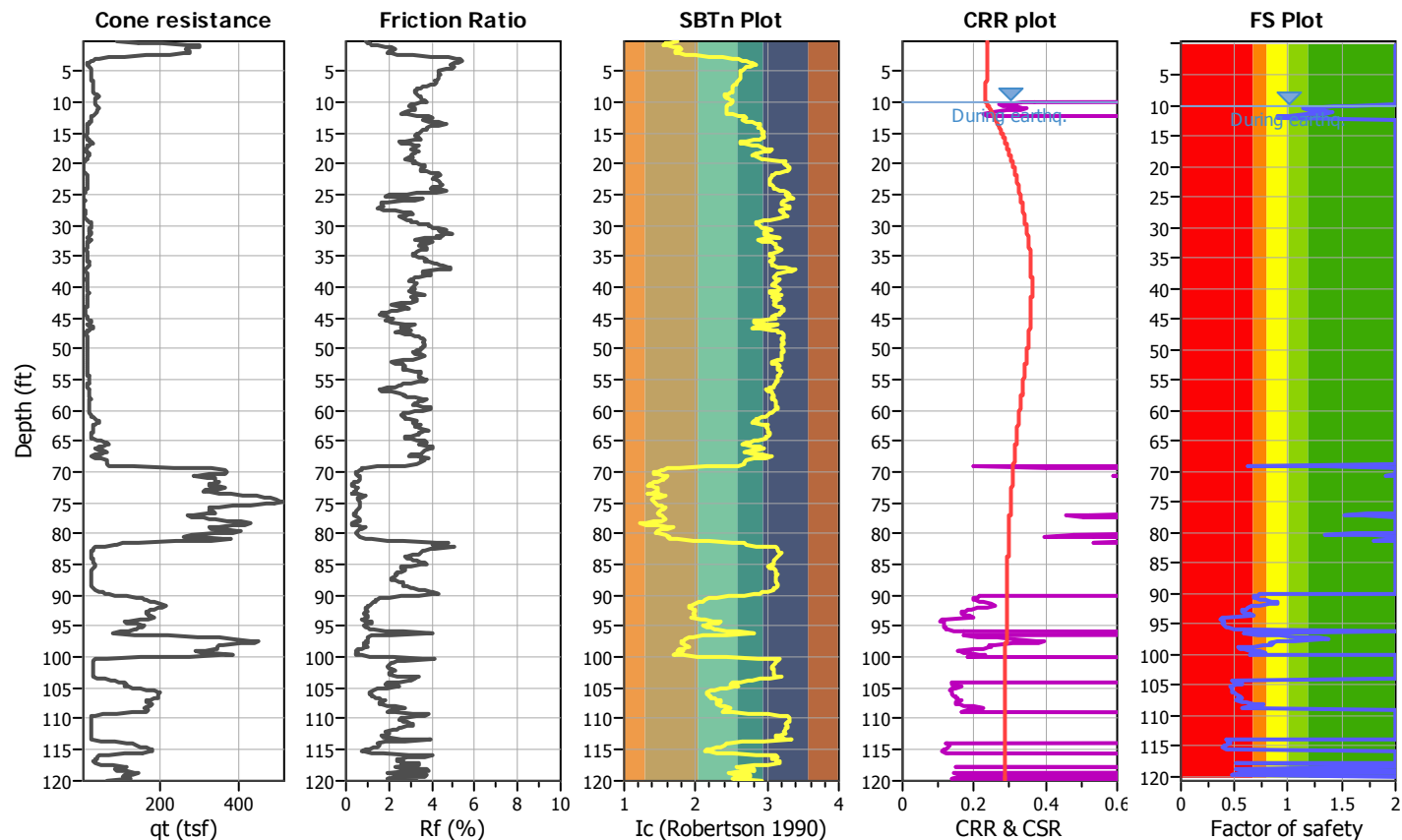
Project title : Montgomery 7

Location : 565 Lorraine Avenue

CPT file : CPT-01

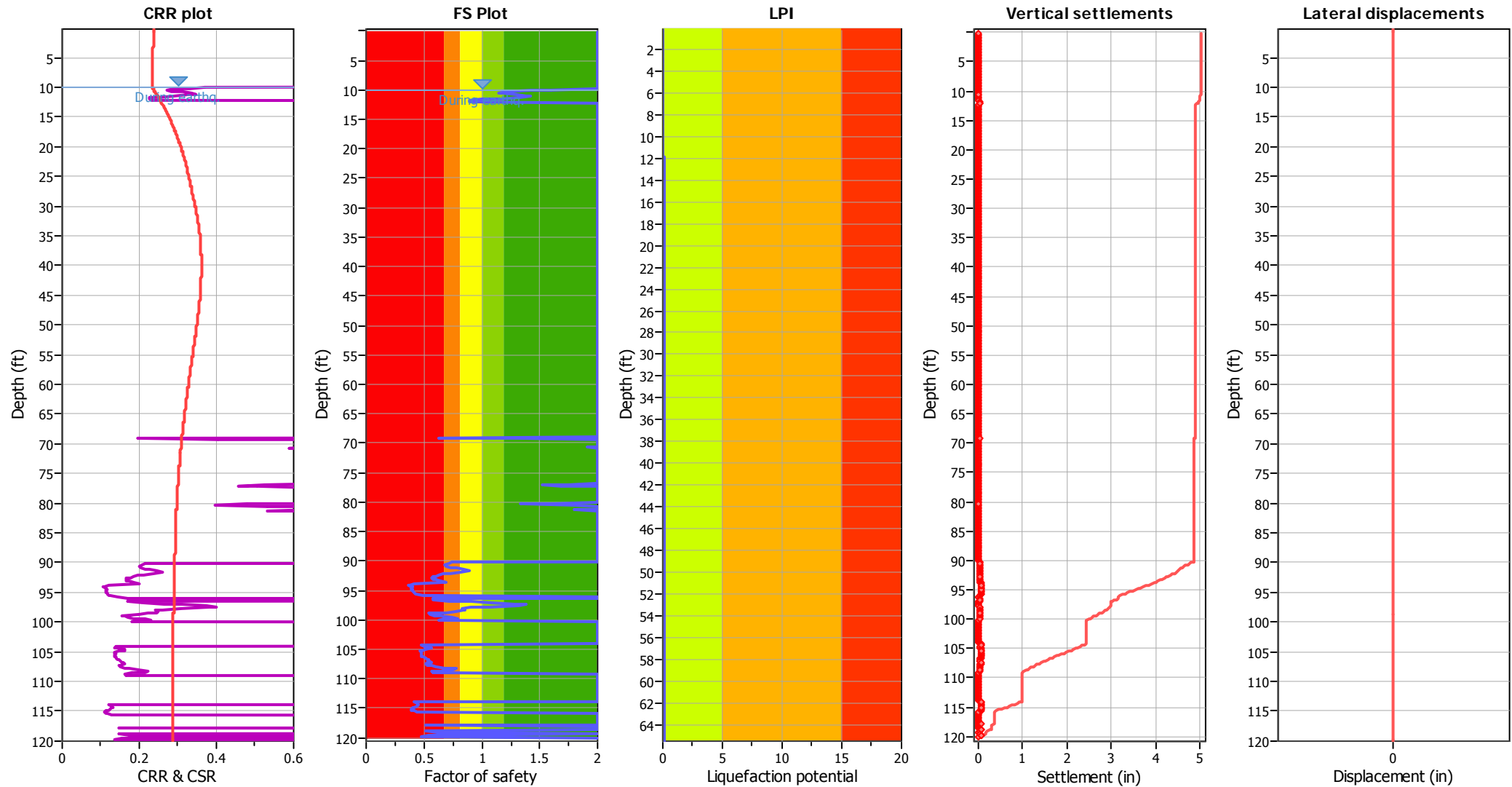
Input parameters and analysis data

| | | | | | | | |
|------------------------------|-------------------|---------------------------|--------------|-------------------------|-----|-----------------------------|--------------|
| Analysis method: | NCEER (1998) | G.W.T. (in-situ): | 17.40 ft | Use fill: | No | Clay like behavior applied: | Sands only |
| Fines correction method: | NCEER (1998) | G.W.T. (earthq.): | 10.00 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 : | 6.63 | 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_0 applied: | Yes | | |



Zone A₁: Cyclic liquefaction likely depending on size and duration of cyclic loading
 Zone A₂: 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.): | 10.00 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 ₀ applied: | Yes |
| Earthquake magnitude M _w : | 6.63 | 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): | 17.40 ft | Fill height: | N/A | Limit depth: | N/A |

F.S. color scheme

| | |
|-------------|---|
| Red | Almost certain it will liquefy |
| Orange | Very likely to liquefy |
| Yellow | Liquefaction and no liq. are equally likely |
| Light Green | Unlike to liquefy |
| Dark Green | Almost certain it will not liquefy |

LPI color scheme

| | |
|--------|----------------|
| Red | Very high risk |
| Orange | High risk |
| Yellow | Low risk |

LIQUEFACTION ANALYSIS REPORT

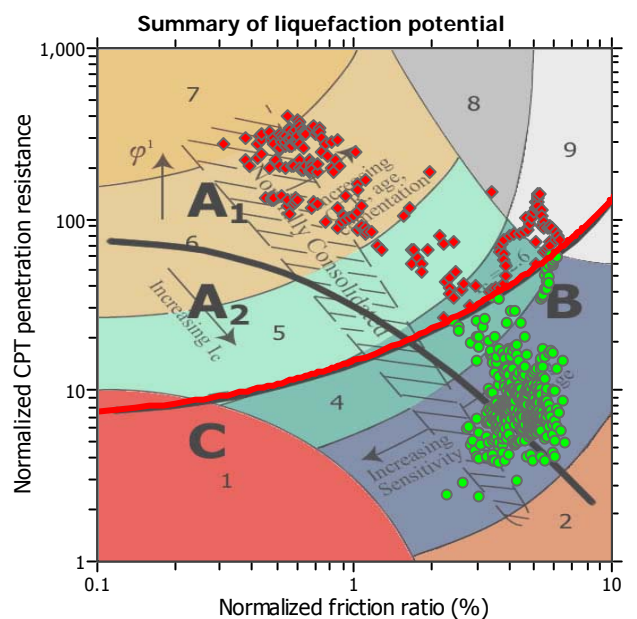
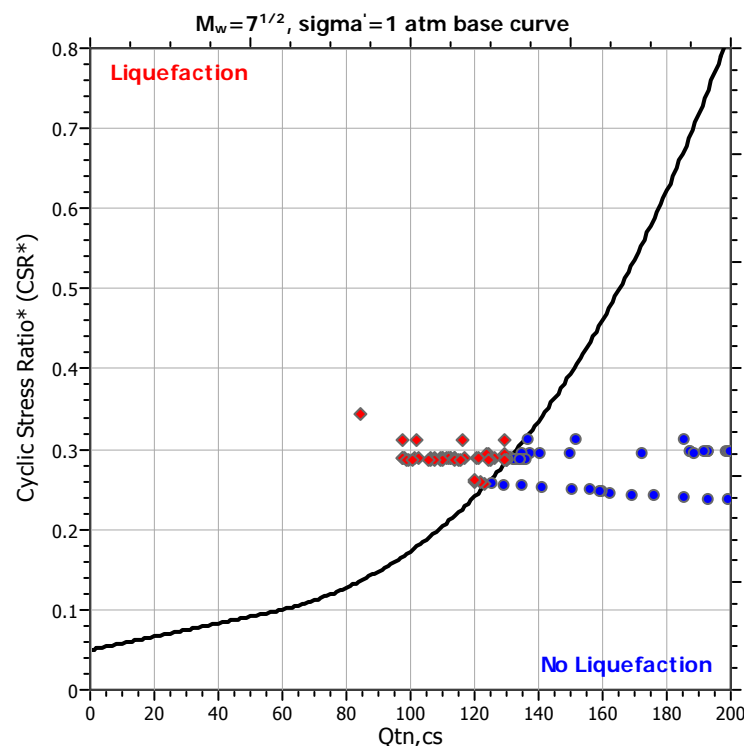
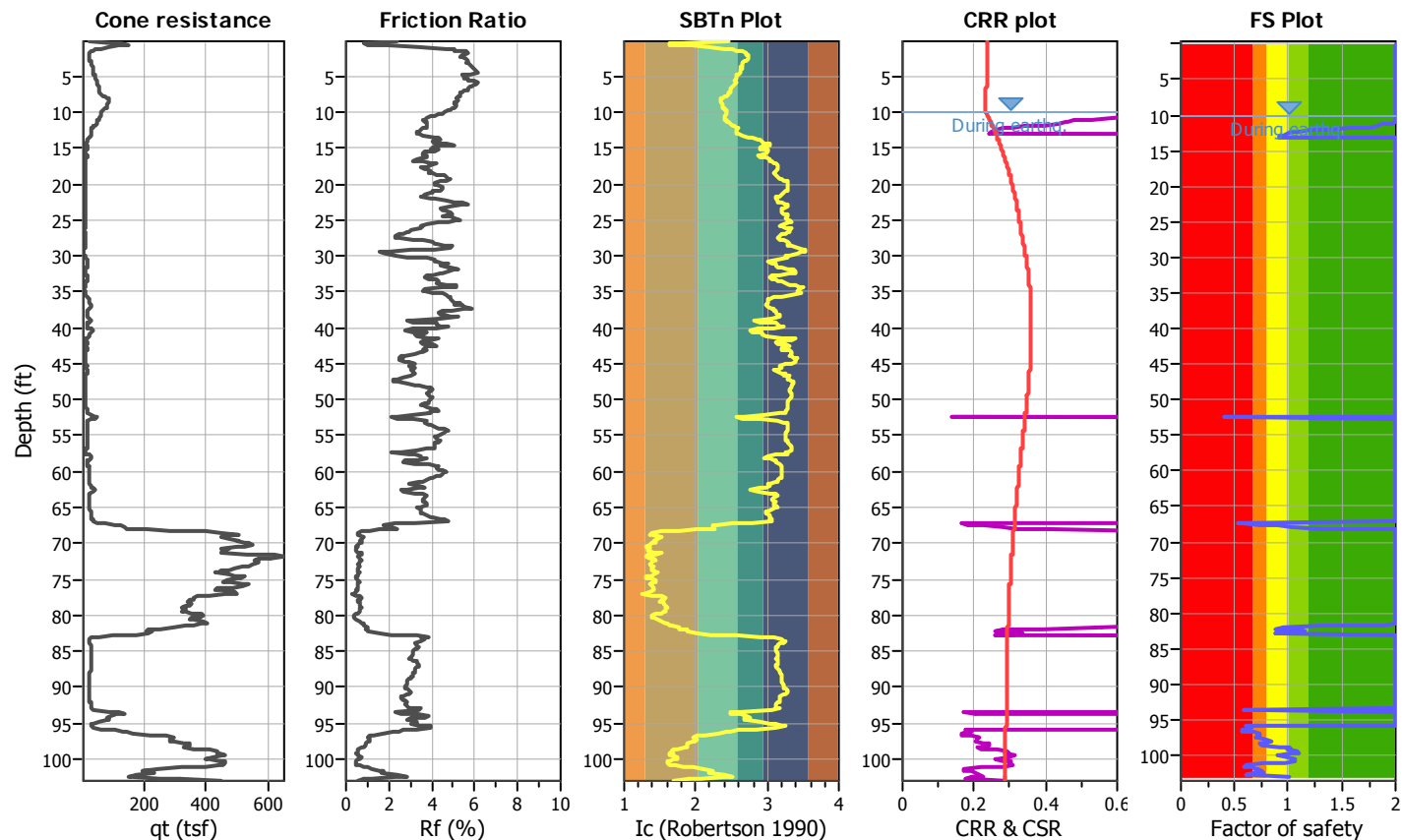
Project title : Montgomery 7

Location : 565 Lorraine Avenue

CPT file : CPT-02

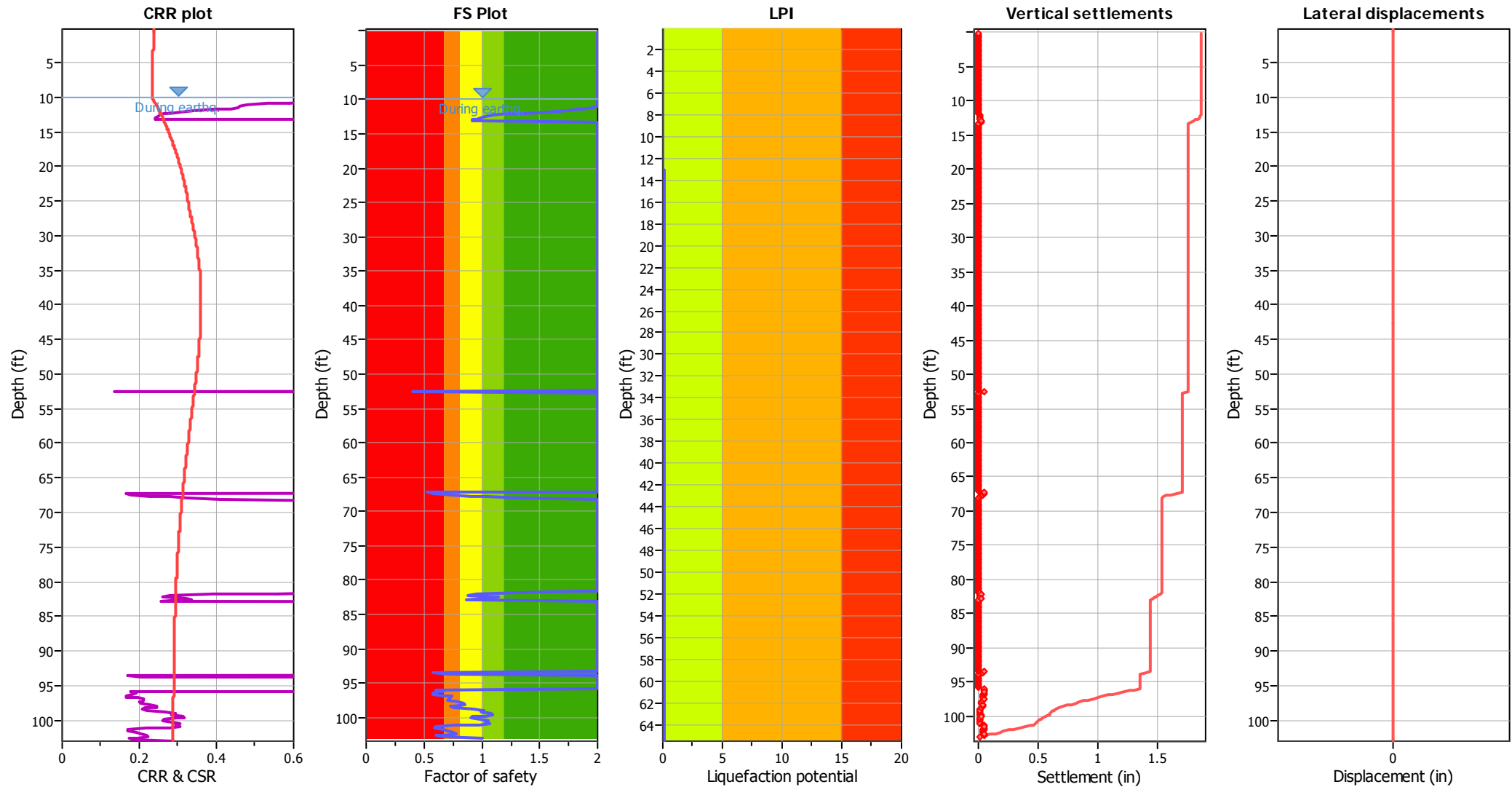
Input parameters and analysis data

| | | | | | | | |
|------------------------------|-------------------|---------------------------|--------------|-------------------------|-----|----------------------|--------------|
| Analysis method: | NCEER (1998) | G.W.T. (in-situ): | 17.40 ft | Use fill: | No | Clay like behavior | |
| Fines correction method: | NCEER (1998) | G.W.T. (earthq.): | 10.00 ft | Fill height: | N/A | applied: | Sands only |
| Points to test: | Based on Ic value | Average results interval: | 3 | Fill weight: | N/A | Limit depth applied: | No |
| Earthquake magnitude M_w : | 6.63 | Ic cut-off value: | 2.60 | Trans. detect. applied: | No | Limit depth: | N/A |
| Peak ground acceleration: | 0.50 | Unit weight calculation: | Based on SBT | K_0 applied: | Yes | MSF method: | Method based |



Zone A₁: Cyclic liquefaction likely depending on size and duration of cyclic loading
 Zone A₂: 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 (earthq.): | 10.00 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 ₀ applied: | Yes |
| Earthquake magnitude M _w : | 6.63 | 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): | 17.40 ft | Fill height: | N/A | Limit depth: | N/A |

F.S. color scheme

| | |
|-------------|---|
| Red | Almost certain it will liquefy |
| Orange | Very likely to liquefy |
| Yellow | Liquefaction and no liq. are equally likely |
| Light Green | Unlike to liquefy |
| Dark Green | Almost certain it will not liquefy |

LPI color scheme

| | |
|--------|----------------|
| Red | Very high risk |
| Orange | High risk |
| Yellow | Low risk |