Appendix D:
Geotechnical Supporting Information

CORNERSTONE
EARTH GROUP

Type of Services	**Preliminary Geotechnical Investigation**
Project Name	**Moorpark Residential Development**
Location	**2323 Moorpark Avenue** **San Jose, California**
Client	**TTLC Management, Inc.**
Client Address	**12647 Alcosta Blvd., Suite 470** **San Ramon, CA**
Project Number	**648-20-2**
Date	**December 2, 2019**

Prepared by

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Type of Services	**Preliminary Geotechnical Investigation**
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SECTION 1: INTRODUCTION

This preliminary geotechnical report was prepared for the sole use of TTLC Management, Inc. for the parcels located at Moorpark Avenue in San Jose, California. The purpose of this study was to evaluate the existing subsurface conditions and develop an opinion regarding potential geotechnical concerns that could impact the proposed development. The preliminary geotechnical recommendations contained in this report are for your forward planning, cost estimating, and preliminary project design.

1.1 PROJECT DESCRIPTION

The project is located at 2323 Moorpark Ave in San Jose, California. The site is currently occupied by various businesses, apartments, and vacant lots. We understand that residential townhome units are being considered for this development. From prior experience, we anticipate the planned development will be three to four-story, at-grade structures. We anticipate the townhomes will be of wood-frame construction. Structural loads are not currently known for the proposed townhomes; however, structural loads are expected to be typical of similar type structures. Grading will likely consist of minor cuts and fills on the order of 2 feet or less.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated October 15, 2019 and consisted of field and laboratory programs to evaluate physical and engineering properties of the subsurface soils, preliminary engineering analysis to prepare recommendations for site work and grading, building foundations, flatwork, and pavements, and preparation of this preliminary report. Brief descriptions of our exploration and laboratory programs are presented below.

1.3 EXPLORATION PROGRAM

Field exploration consisted of three borings drilled on October 25, 2019 with truck-mounted, hollow-stem auger drilling equipment. The borings were drilled to depth of approximately 20 feet. The borings were backfilled with cement grout in accordance with local requirements; exploration permits were obtained as required by local jurisdictions.

The approximate locations of our exploratory borings are shown on the Site Plan, Figure 2. Details regarding our field program are included in Appendix A.

1.4 LABORATORY TESTING PROGRAM

In addition to visual classification of samples, the laboratory program focused on obtaining data for foundation design and seismic ground deformation estimates. Testing included moisture contents, dry densities, washed sieve analyses and Plasticity Index tests. Details regarding our laboratory program are included in Appendix B.

1.5 CORROSION EVALUATION

Two samples from our borings from depths from 3½ to 6 feet were tested for saturated resistivity, pH, and soluble sulfates and chlorides. JDH Corrosion Consultants prepared a brief corrosion evaluation based on the laboratory data, which is attached to this report in Appendix C. In general, the on-site soils can be characterized as mildly corrosive to corrosive to buried metal, and non-corrosive to buried concrete.

1.6 ENVIRONMENTAL SERVICES

Cornerstone Earth Group also provided environmental services for this project, including Phase 1 site assessment; environmental findings and conclusions are provided under separate covers.

SECTION 2: REGIONAL SETTING

2.1 GEOLOGICAL SETTING

The site is located within the Santa Clara Valley, which is a broad alluvial plane between the Santa Cruz Mountains to the southwest and west, and the Diablo Range to the northeast. The San Andreas Fault system, including the Monte Vista-Shannon Fault, exists within the Santa Cruz Mountains and the Hayward and Calaveras Fault systems exist within the Diablo Range.

2.2 REGIONAL SEISMICITY

The San Francisco Bay area region is one of the most seismically active areas in the Country. While seismologists cannot predict earthquake events, the U.S. Geological Survey's Working Group on California Earthquake Probabilities 2015 revises earlier estimates from their 2008 (2008, UCERF2) publication. Compared to the previous assessment issued in 2008, the estimated rate of earthquakes around magnitude 6.7 (the size of the destructive 1994 Northridge earthquake) has gone down by about 30 percent. The expected frequency of such events statewide has dropped from an average of one per 4.8 years to about one per 6.3 years. However, in the new study, the estimate for the likelihood that California will experience a magnitude 8 or larger earthquake in the next 30 years has increased from about 4.7 percent for UCERF2 to about 7.0 percent for UCERF3.

UCERF3 estimates that each region of California will experience a magnitude 6.7 or larger earthquake in the next 30 years. Additionally, there is a 63 percent chance of at least one magnitude 6.7 or greater earthquake occurring in the Bay Area region between 2007 and 2036.

The faults considered capable of generating significant earthquakes are generally associated with the well-defined areas of crustal movement, which trend northwesterly. The table below presents the State-considered active faults within 25 kilometers of the site.

Table 1: Approximate Fault Distances

Fault Name	Distance	
	(miles)	(kilometers)
Monte Vista-Shannon	4.8	7.8
Hayward (Southeast Extension)	8.3	13.4
San Andreas (1906)	9.3	14.9
Calaveras	11.2	18
Hayward (Total Length)	11.3	18.2
Sargent	12.4	20

A regional fault map is presented as Figure 3, illustrating the relative distances of the site to significant fault zones.

SECTION 3: SITE CONDITIONS

3.1 SURFACE DESCRIPTION

The site is bounded by residential housing to the east and west, Interstate Highway 280 to the north, and Moorpark Avenue to the south. The southern part site is occupied by an apartment complex ranging from one to two stories high and is surrounded by asphalt and concrete pavement. The northern part of the site, which is adjacent to Interstate 280, is occupied by an undeveloped lot with vegetation varying from 1 to 4 feet high.

Surface pavements, where encountered, generally consisted of 2 inches of asphalt concrete over 4 inches of aggregate base. Based on visual observations, the existing pavements are in poor shape with significant alligator cracking.

3.2 SUBSURFACE CONDITIONS

Boring EB-1 encountered undocumented fill consisting of medium dense clayey sands to a depth of approximately 3½ feet. The fill is underlain by hard sandy lean clays to 5 feet underlain by loose to medium dense clayey and silty sands to 12½ feet, which is underlain by very stiff lean clay to the maximum depth explored of 20 feet. Below the surface pavement, Boring EB-2 encountered hard lean clay with various amounts of sands to a depth of 8 feet underlain by loose to medium dense silty sands to a depth of 12½ feet. The sand layer is underlain by very stiff lean clay to a depth of 17 feet underlain by loose clayey sand to the maximum depth explored of 20 feet. Boring EB-3 encountered undocumented fill to a depth of approximately 2 feet consisting of medium depth silty sand. The fill is underlain by hard sandy lean clay to 4 feet underlain by loose to medium dense poorly-graded and clayey sands to a depth of about 13 feet. The upper sands are underlain by very stiff lean clay to 18 feet underlain by loose clayey sands to the maximum depth explored of 20 feet.

3.2.1 Plasticity/Expansion Potential

We performed one Plasticity Index (PI) test on a representative sample. Test result were used to evaluate expansion potential of surficial soils. The result of the surficial PI test indicated the PI of 14, indicating low plasticity and expansion potential to wetting and drying cycles.

3.2.2 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents within the upper 10 feet range from 5 percent below optimum to 5 percent over the estimated laboratory optimum moisture.

3.3 GROUNDWATER

Groundwater was not encountered in any of our borings during drilling; however, the borings were not left open but were immediately backfilled when the boring was completed. Based on our previous experience in the area and review of historic high ground water maps (CGS, 2003), we anticipate that the high ground water level will be greater than 50 feet below current grades. Due to the presence of shallow sand layers, perched groundwater could be potentially be encountered following periods of heavy rainfall due to surface water infiltration.

Fluctuations in groundwater levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors.

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT RUPTURE

As discussed above several significant faults are located within 25 kilometers of the site. The site is not located within a State-designated Alquist Priolo Earthquake Fault Zone or a City of San Jose Potential Hazard Zone. As shown in Figure 3, no known surface expression of fault traces is thought to cross the site; therefore, fault rupture hazard is not a significant geologic hazard at the site.

4.2 ESTIMATED GROUND SHAKING

Moderate to severe (design-level) earthquakes can cause strong ground shaking, which is the case for most sites within the Bay Area. On a preliminary basis, we utilized a site modified peak ground acceleration (PGA_M) provided by ATC Hazards by Location on-line calculator (https://hazards.atcouncil.org), based on the site conditions listed below and the site classification. For our preliminary analysis we used a PGA_M of 0.69g.

4.3 LIQUEFACTION POTENTIAL

The site is not located within a State-designated Liquefaction Hazard Zone (CGS, San Jose West Quadrangle, 2003) or a Santa Clara County Liquefaction Hazard Zone (Santa Clara County, 2003). However, we screened the site for liquefaction during our site exploration by retrieving samples from the site, performing visual classification on sampled materials, and performing various tests to further classify the soil properties.

During strong seismic shaking, cyclically induced stresses can cause increased pore pressures within the soil matrix that can result in liquefaction triggering, soil softening due to shear stress loss, potentially significant ground deformation due to settlement within sandy liquefiable layers as pore pressures dissipate, and/or flow failures in sloping ground or where open faces are present (lateral spreading) (NCEER 1998). Limited field and laboratory data is available regarding ground deformation due to settlement; however, in clean sand layers settlement on the order of 2 to 3 percent of the liquefied layer thickness can occur. Soils most susceptible to liquefaction are loose, non-cohesive soils that are saturated and are bedded with poor drainage, such as sand and silt layers bedded with a cohesive cap.

As discussed in the "Subsurface" section above, we primarily encountered stiff cohesive and medium dense granular soils. In addition, the design ground water level is anticipated to be below a depth of 50 feet. Based on the above, our screening of the site for liquefaction indicates a low potential for liquefaction.

4.4 LATERAL SPREADING

Lateral spreading is horizontal/lateral ground movement of relatively flat-lying soil deposits towards a free face such as an excavation, channel, or open body of water; typically lateral spreading is associated with liquefaction of one or more subsurface layers near the bottom of the exposed slope. As failure tends to propagate as block failures, it is difficult to analyze and estimate where the first tension crack will form.

There are no open faces within 200 feet of the site where lateral spreading could occur; therefore, in our opinion, the potential for lateral spreading to affect the site is low.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose unsaturated sandy soils can settle during strong seismic shaking. Localized loose sand layers encountered at the site generally greater than 25 percent silt and clay fines. Based on our preliminary analysis, the potential for differential seismic settlement affecting the proposed improvements is low to moderate, with localized seismic settlement on the order of ¼ inch or less. Additional analysis should be performed during the design-level geotechnical investigation to confirm these preliminary estimates.

4.6 TSUNAMI/SEICHE

The terms tsunami or seiche are described as ocean waves or similar waves usually created by undersea fault movement or by a coastal or submerged landslide. Tsunamis may be generated at great distance from shore (far field events) or nearby (near field events). Waves are formed, as the displaced water moves to regain equilibrium, and radiates across the open water, similar to ripples from a rock being thrown into a pond. When the waveform reaches the coastline, it quickly raises the water level, with water velocities as high as 15 to 20 knots. The water mass, as well as vessels, vehicles, or other objects in its path create tremendous forces as they impact coastal structures.

Tsunamis have affected the coastline along the Pacific Northwest during historic times. The Fort Point tide gauge in San Francisco recorded approximately 21 tsunamis between 1854 and 1964. The 1964 Alaska earthquake generated a recorded wave height of 7.4 feet and drowned eleven people in Crescent City, California. For the case of a far-field event, the Bay area would have hours of warning; for a near field event, there may be only a few minutes of warning, if any.

A tsunami or seiche originating in the Pacific Ocean would lose much of its energy passing through San Francisco Bay. Based on the study of tsunami inundation potential for the San Francisco Bay Area (Ritter and Dupre, 1972), areas most likely to be inundated are marshlands, tidal flats, and former bay margin lands that are now artificially filled, but are still at or below sea level, and are generally within 1½ miles of the shoreline. The site is approximately 8 miles inland from the San Francisco Bay shoreline and is approximately 136 to 144 feet above mean sea level. Therefore, the potential for inundation due to tsunami or seiche is considered low.

4.7 FLOODING

Based on our internet search of the Federal Emergency Management Agency (FEMA) flood map public database, the site is located within Zone D, an area of undetermined, but possible flood hazard. We recommend the project civil engineer be retained to confirm this information and verify the base flood elevation, if appropriate.

SECTION 5: CONCLUSIONS

5.1 SUMMARY

From a geotechnical viewpoint, the project is feasible provided the concerns listed below are addressed in the project design. The preliminary recommendations that follow are intended for conceptual planning and preliminary design. A design-level geotechnical investigation should be performed once site development plans are prepared indicating where proposed structures are planned. The design-level investigation findings will be used to confirm the preliminary recommendations and develop detailed recommendations for design and construction. Descriptions of each geotechnical concern with brief outlines of our preliminary recommendations follow the listed concerns.

- Presence of undocumented fills
- Shallow loose sand layers
- Soil corrosion potential

5.1.1 Undocumented Fill

As previously mentioned, undocumented fill was encountered in Borings EB-1 and EB-2 to depths of approximately 2 to 3½ feet and may vary in different depths throughout the site. The fill was likely placed during original site development; however, records of placement and compaction of this fill material are not available. The fill may be highly variable following site demolition and may not uniformly support the proposed residential structures and adjacent improvements. The presence of and distribution of undocumented fill should be further evaluated during the design-level geotechnical investigation. On a preliminary basis, we recommend that undocumented fills, where encountered, be over-excavated and re-compacted during site grading.

5.1.2 Loose Sand Layer

Our borings encountered localized loose sand layers below a depth of approximately 5 feet. If encountered during site excavations, such as utility trenching or below-grade vaults, localized sloughing or caving could potentially occur along excavation sidewalls. Excavations performed within existing city streets could undermine existing improvements if improper sloping or shoring techniques are used. Contractors may be required to lay back excavations or utilize shoring designed to reduce the potential for sidewall caving.

5.1.3 Soil Corrosion Potential

A preliminary soil corrosion screening was performed by JDH Corrosion Consultants based on the results of analytical tests on samples of the near-surface soil. In general, the JDH report concludes that the corrosion potential for buried concrete does not warrant the use of sulfate resistant concrete. However, the corrosion potential for buried metallic structures, such as metal pipes, is considered mildly corrosive to corrosive. JDH recommends that special requirements for corrosion control be made to protect metal pipes. A more detailed discussion of the site corrosion evaluation is presented in Appendix C.

5.2 DESIGN-LEVEL GEOTECHNICAL INVESTIGATION

The preliminary recommendations contained in this preliminary study were based on limited site development information and limited exploration and our experience in the area with similar projects. As site conditions may vary significantly between the small-diameter borings performed during this investigation from the assumed conditions, we also recommend that we be retained to 1) perform a design-level geotechnical investigation, once detailed site development plans are available; 2) to review the geotechnical aspects of the project structural, civil, and landscape plans and specifications, allowing sufficient time to provide the design team with any comments prior to issuing the plans for construction; and 3) be present to provide geotechnical observation and testing during earthwork and foundation construction.

SECTION 6: EARTHWORK

6.1 ANTICIPATED EARTHWORK MEASURES

On a preliminary basis, we recommend that existing foundations, slabs, and/or abandoned underground utilities be removed entirely and the resulting excavations backfilled with engineered fill. Additionally, native soils that are disturbed during demolition of the existing improvements should also be removed and replaced as engineered fill. Any existing undocumented fill encountered during grading should be over-excavated down to native soils within the proposed building footprints and 5 feet laterally beyond. On a preliminary basis and for conceptual planning and cost estimating, undocumented fill over-excavation on the order of 3 feet below current site grades should be considered.

On-site soils below the stripped layer appear to be suitable for use as fill at the site, provided they are determined to be suitable for re-use from an environmental viewpoint. Imported fill material for use as general fill should have a Plasticity Index of 15 or less and have at least 10 percent silt or clay fines to prevent sloughing or caving during construction. Existing asphalt and concrete materials can likely be pulverized and re-used as granular base material at the site; however, asphalt grindings should not be re-used beneath residential lots or building footprints. All fill as well as scarified surface soils in areas to receive fill or slabs-on-grade, and subgrade, and trench backfill, should be compacted to at least 90 percent relative compaction as determined by ASTM Test Designation D-1557, latest edition; and be at least 2 percent above optimum moisture. Aggregate materials should be compacted to at least 95 percent in pavement areas, and 90 percent in flatwork areas.

It should be noted that excavations extending greater than 5 feet below current site grades may encountered loose to medium dense sands that could be susceptible to slough or caving. Excavation contractors may need to plan to lay back deeper trench excavations at inclinations of 1:1 (horizontal:vertical) or flatter in accordance with OSHA requirements.

6.2 SURFACE DRAINAGE

Surface water runoff should not be allowed to pond adjacent to building foundations, slabs-on-grade, or pavements. Hardscape surfaces should slope at least 2 percent towards suitable discharge facilities; landscape areas should slope at least 2 to 3 percent away from buildings. Due to the high expansion potential of the near-surface soils, retention, detention, or infiltration facilities should be spaced at least 10 feet from buildings, and preferably at least 5 feet from slabs-on-grade or pavements. The near-surface soils at the site consist of interbedded clays and clayey sands that would likely be categorized as Hydrologic Soil Group C or D, which are expected to have infiltration rates on the order of 0.1 to 0.5 inches per hour or less. In our opinion, the shallow clayey soils will significantly limit stormwater infiltration.

SECTION 7: FOUNDATIONS

7.1 SUMMARY OF RECOMMENDATIONS

On a preliminary basis, the proposed residential structures could be supported on shallow foundations consisting of rigid mat foundations or conventional shallow footings.

7.2 SEISMIC DESIGN CRITERIA

On a preliminary basis, we are providing Seismic Design Parameters for the project in accordance with the 2019 California Building Code (2019 CBC), which will be effective for projects that are submitted to the local building department starting January 1, 2020. The new 2019 CBC includes major changes to the procedures used determine the seismic design parameters and has added new requirements for design of foundations for structures constructed on sites with liquefiable soils. The analysis used to provide the requested 2019 CBC seismic design parameters was based on Chapters 16/16A and 18/18A of 2019 CBC and Chapter 11 of ASCE 7-16 (including Supplement 1).

The analysis considered mapped spectral acceleration parameters based on distance to the controlling seismic source/fault system. Based on review of local geology, the site is underlain by deep alluvial soils with typical SPT "N" values between 15 and 50 blows per foot which classifies the site as Site Class D. Effects of local soil conditions are incorporated in the initial (i.e., before any ground shaking) average shear wave velocity for the upper 30 m (100 ft) of the site soils. Shear wave velocity data was not collected for this preliminary study but will need to be collected during the design-level investigation. On a preliminary basis and with some preliminary assumptions on the site coordinates and classification, as well as the Risk Category, Building Period and Importance factor, the following table lists the various factors or site data used for this analysis.

Table 2: Preliminary 2019 CBC Site Data

Site Data	Design Value
Site Class (Per Chapter 20 ASCE 7-16)	D
Design Shear Wave Velocity, V_{S30}	N/A
Site Latitude	37.316394°
Site Longitude	-121.934378°
Risk Category	II
Importance Factor, I_e	1
0.2-second Period Mapped Spectral Acceleration[1], S_S	1.500g
1-second Period Mapped Spectral Acceleration[1], S_1	0.600g

[1] Assumed for Site Class B, 5 percent damped.

A detailed ground motion hazards analysis in accordance with Chapter 21, Section 21.2 of ASCE 7-16 should be performed in the design-level phase.

7.3 POST-TENSIONED MAT FOUNDATIONS

The planned at-grade residential structures may be supported on post-tensioned (PT) concrete mat foundations designed in accordance with the procedures developed by the Post-Tensioning Institute (latest edition) and the 2019 California Building Code.

To reduce potential differential movement, on a preliminary basis, mats should be designed for a maximum average areal bearing pressure of 750 psf for dead plus live loads; at column or wall loading, the maximum localized allowable bearing pressure should be limited to about 3,000 psf. When evaluating wind and seismic conditions, allowable bearing pressure may be increased by one-third. Additional reinforcing steel may be required to help span irregularities and differential settlement.

On a preliminary basis, we estimate that differential static settlements will be on the order of ½ to ¾ inch or less across a typical mat foundation. Final settlement and foundation deflection criteria should be determined during the design-level geotechnical investigation.

7.4 CONVENTIONAL FOOTINGS

On a preliminary basis, the planned structures may also be supported on conventional shallow footings. On a preliminary basis, footings should bear on natural, undisturbed soil or engineered fill, be at least 15 to 18 inches wide, and extend at least 18 inches below the lowest adjacent grade. Lowest adjacent grade is defined as the deeper of the following: 1) bottom of the adjacent interior slab-on-grade, or 2) finished exterior grade, excluding landscaping topsoil.

On a preliminary basis, footings should be designed for allowable bearing pressures of 3,000 psf for combined dead plus live loads. Provided undocumented fills are adequately re-compacted during site grading, differential footing settlement will likely be on the order of ½ inch or less between footings.

SECTION 8: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

8.1 INTERIOR SLABS-ON-GRADE

As the Plasticity Index (PI) of the surficial soils ranges up to 14, conventional slabs-on-grade used in conjunction with shallow footings could likely be supported directly on native subgrade soils that have been adequately re-compacted during site grading. Additional laboratory testing will need to be performed during the design-level geotechnical investigation to confirm these preliminary findings.

8.2 EXTERIOR FLATWORK

On a preliminary basis, exterior concrete flatwork subject to pedestrian and/or occasional light pick up loading should be at least 4 inches thick and supported on at least 4 inches of granular base material overlying prepared subgrade.

SECTION 9: VEHICULAR PAVEMENTS

9.1 ASPHALT CONCRETE

The following asphalt concrete pavement recommendations tabulated below are based on the Procedure 608 of the Caltrans Highway Design Manual, estimated traffic indices for various pavement-loading conditions, and on a design R-value of 5. The design R-value was chosen based on engineering judgment considering the native clay soils and variable surface conditions.

Table 3: Asphalt Concrete Pavement Recommendations, Design R-value = 5

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	Total Pavement Section Thickness (inches)
4.0	2.5	8.0	10.5
4.5	2.5	10.0	12.5
5.0	3.0	10.0	13.0
5.5	3.0	12.0	14.0
6.0	3.5	13.0	16.5
6.5	4.0	14.0	18.0

Note: Caltrans Class 2 aggregate base; minimum R-value of 78
Preliminary subgrade R-value assumed to be 5

SECTION 10: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of TTLC Management, Inc. specifically to support the design of the Moorpark Residential Development project in San Jose, California. The opinions, conclusions, and preliminary recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in Northern California at the time this report was prepared. No warranty, expressed or implied, is made or should be inferred.

Preliminary recommendations in this report are based upon the soil and ground water conditions encountered during our limited subsurface exploration. Preparation of a design-level investigation is anticipated to provide additional information and refine the preliminary recommendations presented herein. If variations or unsuitable conditions are encountered during the construction phase, Cornerstone must be contacted to provide supplemental recommendations, as needed.

TTLC Management, Inc. may have provided Cornerstone with plans, reports and other documents prepared by others. TTLC Management, Inc. understands that Cornerstone reviewed and relied on the information presented in these documents and cannot be responsible for their accuracy.

Cornerstone prepared this report with the understanding that it is the responsibility of the owner or his representatives to see that the recommendations contained in this report are presented to other members of the design team and incorporated into the project plans and specifications, and that appropriate actions are taken to implement the geotechnical recommendations during construction.

Conclusions and recommendations presented in this report are valid as of the present time for the development as currently planned. Changes in the condition of the property or adjacent properties may occur with the passage of time, whether by natural processes or the acts of other persons. In addition, changes in applicable or appropriate standards may occur through legislation or the broadening of knowledge. Therefore, the conclusions and recommendations presented in this report may be invalidated, wholly or in part, by changes beyond Cornerstone's control. This report should be reviewed by Cornerstone after a period of three (3) years has elapsed from the date of this report. In addition, if the current project design is changed, then Cornerstone must review the proposed changes and provide supplemental recommendations, as needed.

An electronic transmission of this report may also have been issued. While Cornerstone has taken precautions to produce a complete and secure electronic transmission, please check the electronic transmission against the hard copy version for conformity.

Recommendations provided in this report are based on the assumption that Cornerstone 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, Cornerstone cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of Cornerstone's report by others. Furthermore, Cornerstone will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services.

SECTION 11: REFERENCES

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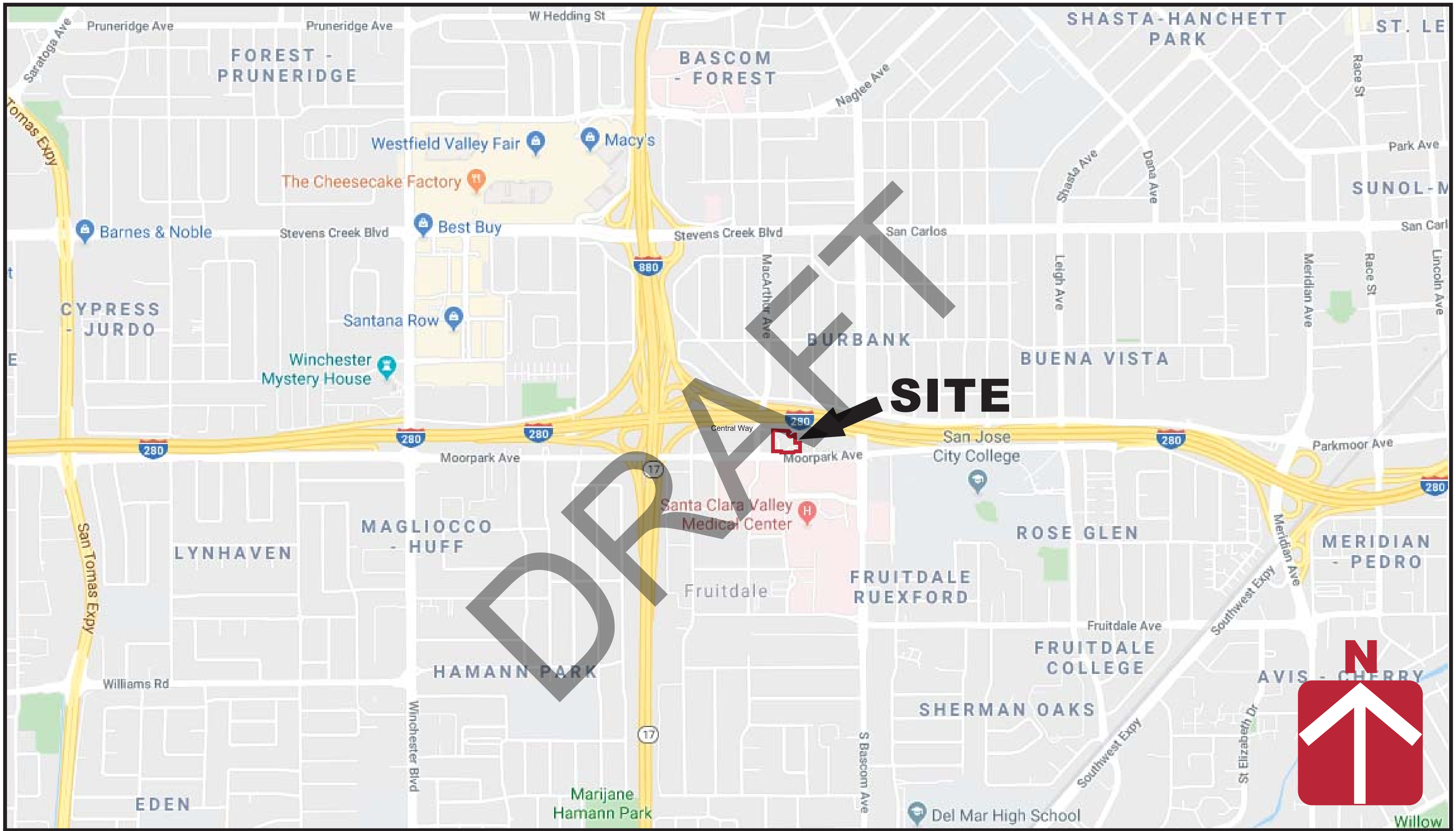


CORNERSTONE EARTH GROUP

Vicinity Map

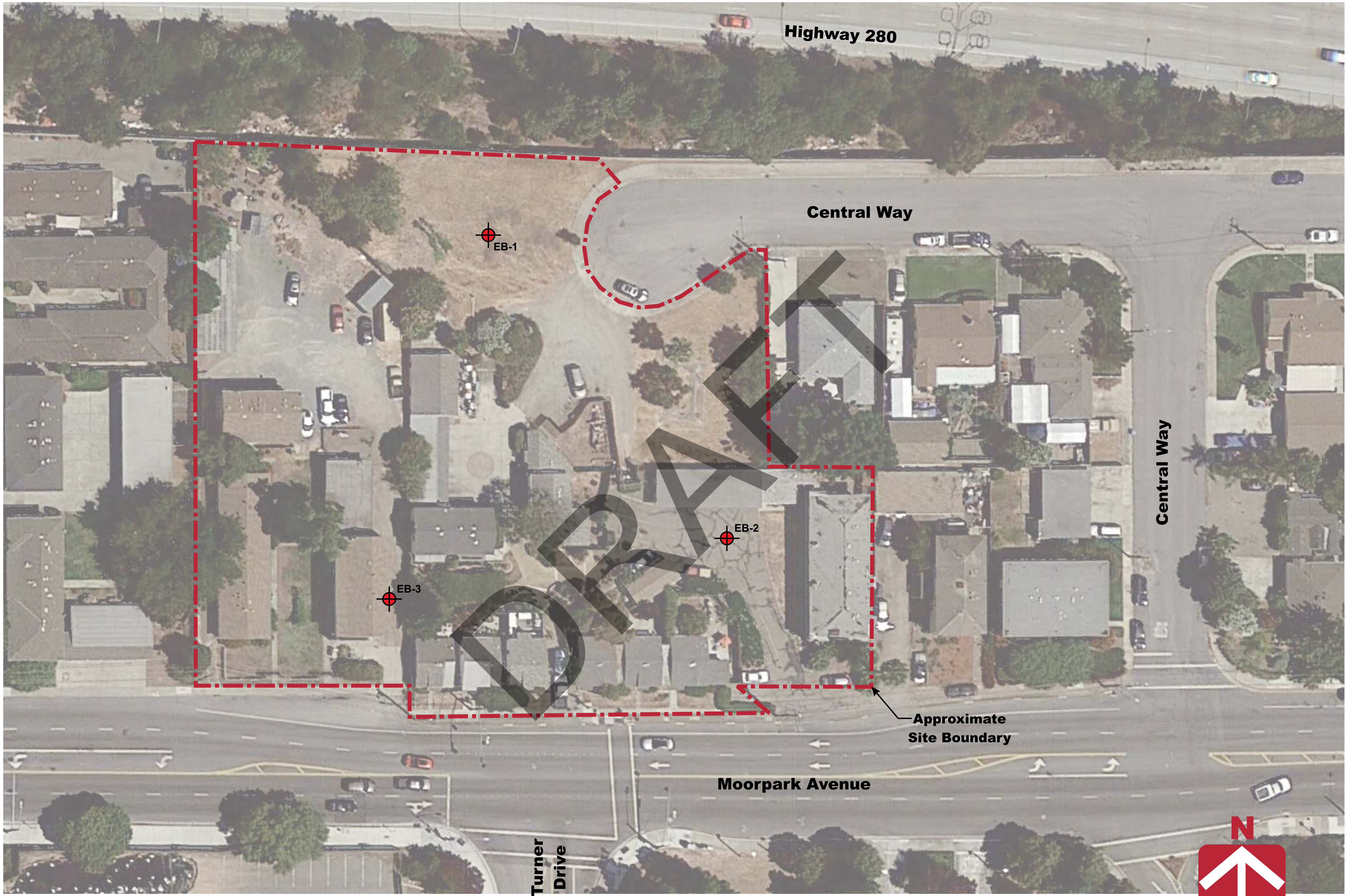
Moorpark Avenue
San Jose, CA

Project Number
648-20-2

Figure Number
Figure 1

Date November 2019

Drawn By RRN



Legend

⊕ Approximate location of exploratory boring (EB)

0 50 100
APPROXIMATE SCALE (FEET)

Base by Google Earth, dated 8/9/2018

Project Number 648-20-2

Figure Number Figure 2

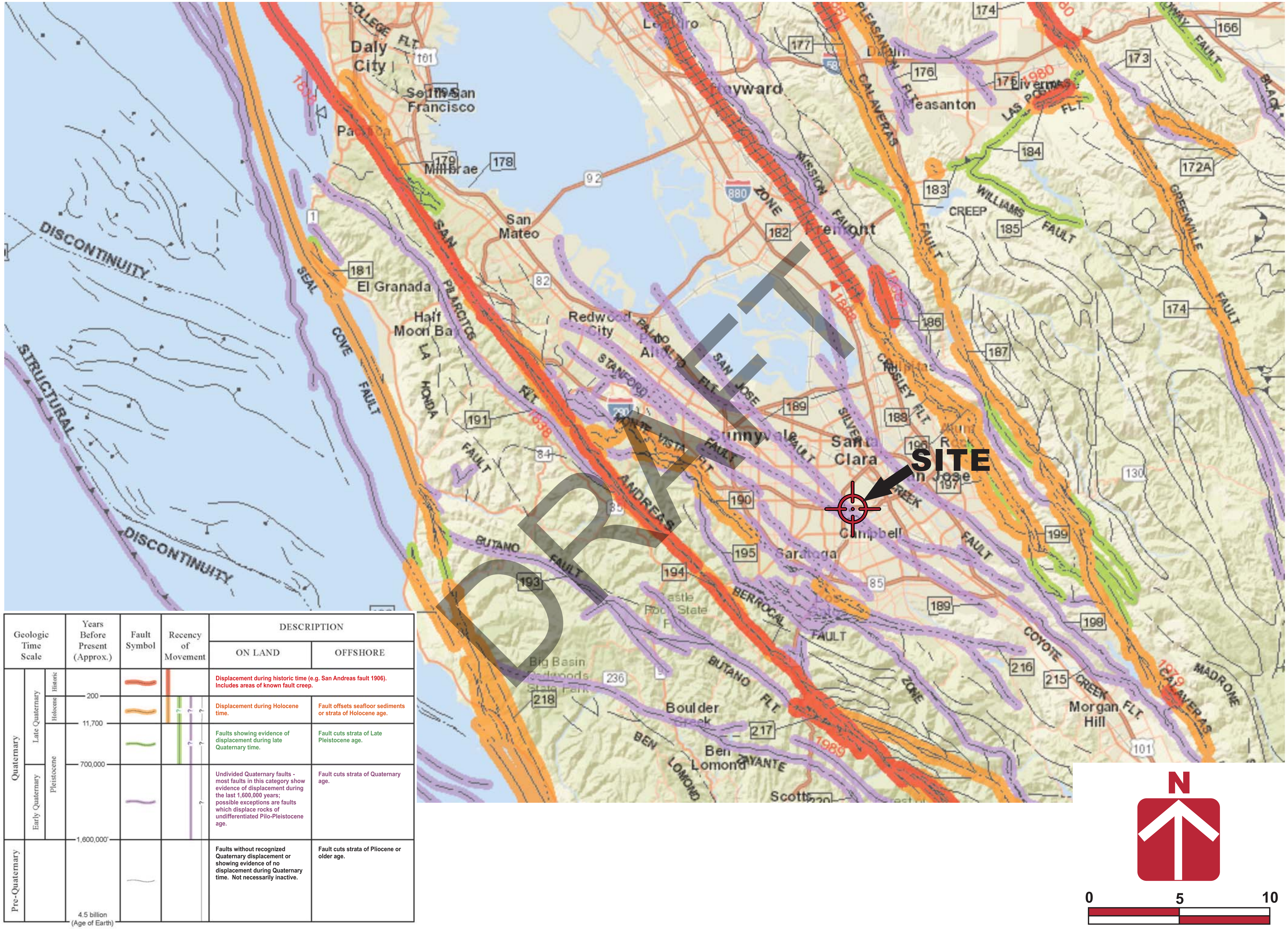
Drawn By RRN

Date November 2019

Site Plan

Moorpark Avenue
San Jose, CA

CORNERSTONE
EARTH GROUP



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION ON LAND	DESCRIPTION OFFSHORE
Quaternary	Late Quaternary	Historic		🟥		Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	200	🟧	? ? ?	Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	11,700	🟩	? ?	Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		700,000	🟪	?	Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Pilo-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary			1,600,000' 4.5 billion (Age of Earth)	—		Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

Base by California Geological Survey - 2010 Fault Activity Map of California (Jennings and Bryant, 2010)

Project Number
648-20-2

Figure Number
Figure 3

Drawn By
FLL

Date
November 2019

Regional Fault Map

Moorpark Avenue
San Jose, CA

CORNERSTONE EARTH GROUP

APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, hollow-stem, auger drilling equipment. Three 8-inch-diameter exploratory borings were drilled on October 25, 2019 to depths of approximately 20 feet. The approximate locations of exploratory borings are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Boring logs, as well as a key to the classification of the soil, are included as part of this appendix.

Boring locations were approximated using existing site boundaries and other site features as references. Boring elevations were based on interpolation of plan contours. The locations of the borings should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. The standard penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The 2-inch O.D. split-spoon sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration (ASTM D1586). 2.5-inch I.D. samples were obtained using a Modified California Sampler driven into the soil with the 140-pound hammer previously described. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows required to drive the last 12 inches. The various samplers are denoted at the appropriate depth on the boring logs.

Field tests included an evaluation of the unconfined compressive strength of the soil samples using a pocket penetrometer device. The results of these tests are presented on the individual boring logs at the appropriate sample depths.

Attached boring 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 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.

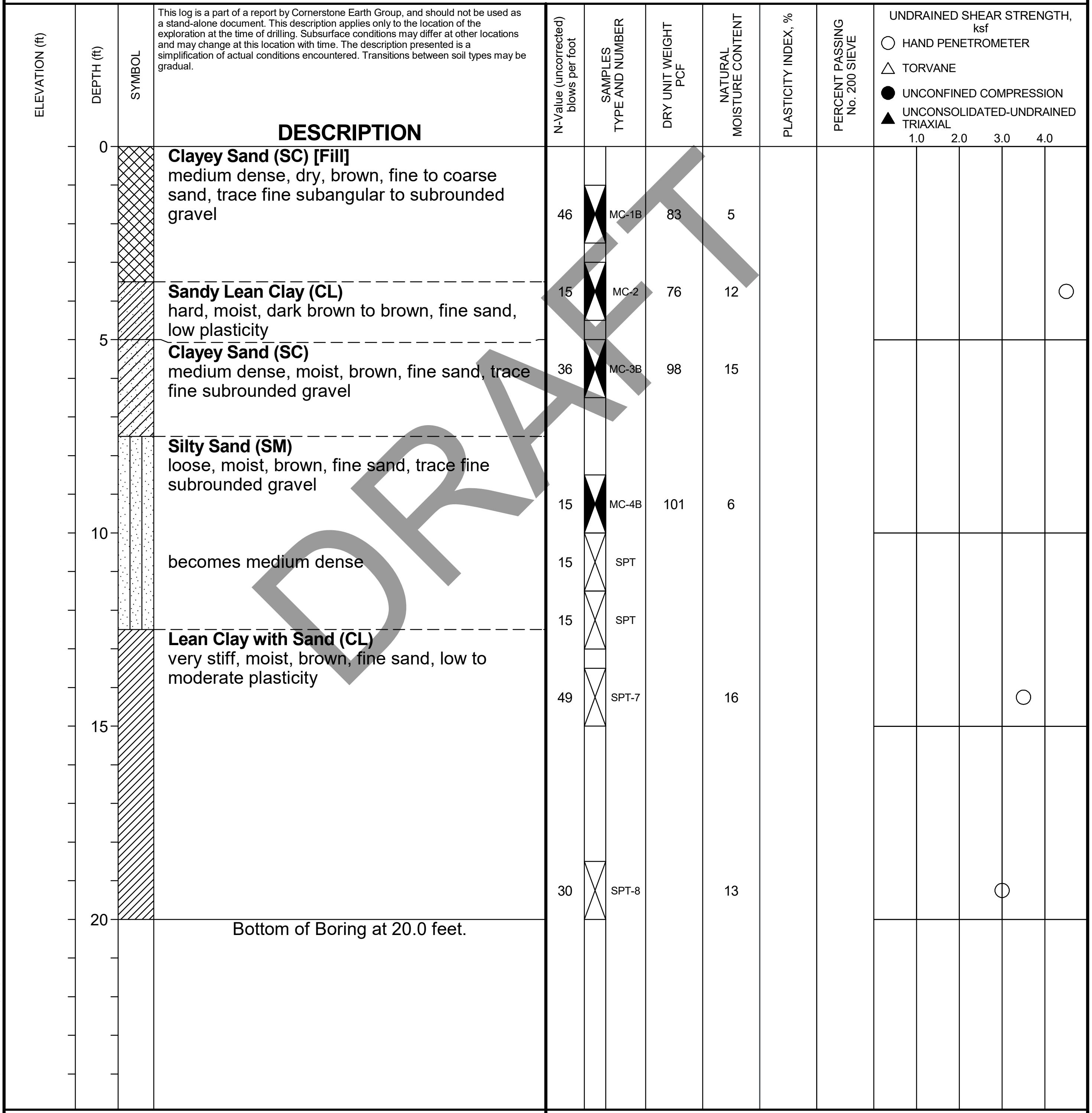
CORNERSTONE
EARTH GROUP

BORING NUMBER EB-1

PAGE 1 OF 1

PROJECT NAME 2323 Moorpark Avenue
PROJECT NUMBER 648-20-2
PROJECT LOCATION San Jose, CA
DATE STARTED 10/25/19 **DATE COMPLETED** 10/25/19
GROUND ELEVATION **BORING DEPTH** 20 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.316873° **LONGITUDE** -121.934999°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUNDWATER LEVELS:
LOGGED BY JLC
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

This log is a part of a report by Cornerstone Earth Group, 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.



DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
Clayey Sand (SC) [Fill] medium dense, dry, brown, fine to coarse sand, trace fine subangular to subrounded gravel	46	MC-1B	83	5		
Sandy Lean Clay (CL) hard, moist, dark brown to brown, fine sand, low plasticity	15	MC-2	76	12		
Clayey Sand (SC) medium dense, moist, brown, fine sand, trace fine subrounded gravel	36	MC-3B	98	15		
Silty Sand (SM) loose, moist, brown, fine sand, trace fine subrounded gravel	15	MC-4B	101	6		
becomes medium dense	15	SPT				
	15	SPT				
Lean Clay with Sand (CL) very stiff, moist, brown, fine sand, low to moderate plasticity	49	SPT-7		16		
	30	SPT-8		13		

Bottom of Boring at 20.0 feet.

BORING NUMBER EB-2

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME 2323 Moorpark Avenue
PROJECT NUMBER 648-20-2
PROJECT LOCATION San Jose, CA
DATE STARTED 10/25/19 **DATE COMPLETED** 10/25/19
GROUND ELEVATION **BORING DEPTH** 20 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.316464° **LONGITUDE** -121.934577°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUNDWATER LEVELS:
LOGGED BY JLC
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf (1.0 / 2.0 / 3.0 / 4.0)
	0	2 inches asphalt concrete over 4 inches aggregate base							
		Lean Clay with Sand (CL) hard, moist, dark brown to brown, fine sand, low plasticity Liquid Limit = 30, Plastic Limit = 16	16	MC-1B	107	14	14		>4.5 ○
			14	MC-2B	105	15			>4.5 ○
	5		8	MC-3B	92	13		52	>4.5 ○
		Sandy Silty Clay (CL-ML) hard, moist, brown, fine to medium sand, low plasticity							
		Silty Sand (SM) loose to medium dense, moist, brown, fine to medium sand, trace fine subrounded gravel	16	MC-4B	111	8			
	10		10	SPT-5		7		27	
		Lean Clay with Sand (CL) very stiff, moist, brown, fine sand, low to moderate plasticity	48	MC-6B	110	17			○
	15								
		Clayey Sand (SC) loose, moist, brown, fine to coarse sand, some fine to coarse subangular to subrounded gravel	17	MC-7B	118	7			
	20	Bottom of Boring at 20.0 feet.							

BORING NUMBER EB-3

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME 2323 Moorpark Avenue

PROJECT NUMBER 648-20-2

PROJECT LOCATION San Jose, CA

DATE STARTED 10/25/19 **DATE COMPLETED** 10/25/19

GROUND ELEVATION ______ **BORING DEPTH** 20 ft.

DRILLING CONTRACTOR Exploration Geoservices, Inc.

LATITUDE 37.316401° **LONGITUDE** -121.935204°

DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger

GROUNDWATER LEVELS:

LOGGED BY JLC

▽ **AT TIME OF DRILLING** Not Encountered

NOTES ______

▼ **AT END OF DRILLING** Not Encountered

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

UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0 2.0 3.0 4.0)

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
	0	**Silty Sand (SM) [Fill]** medium dense, moist, brown, fine to coarse sand, trace fine subangular to subrounded gravel	15	MC-1B	104	11			○ >4.5
		Sandy Lean Clay (CL) hard, moist, dark brown to brown, fine sand, low plasticity	13	MC-2B	92	10			○ >4.5
	5	**Poorly Graded Sand with Silt (SP-SM)** loose, dry to moist, brown, fine to medium sand, some fine subrounded to rounded gravel	17	MC-3B	110	3			
		becomes medium dense	24	MC-4A	115	4			
	10	**Clayey Sand (SC)** medium dense, moist, brown, fine to medium sand, trace fine subrounded gravel	12	SPT					
	15	**Lean Clay with Sand (CL)** very stiff, moist, brown, fine sand, low to moderate plasticity	16	SPT-6		15		75	○ 3.0
		Clayey Sand (SC) loose, moist, brown, fine to coarse sand, some fine to coarse subangular to subrounded gravel	17	MC-7B	111	12		43	
	20	Bottom of Boring at 20.0 feet.							

APPENDIX B: LABORATORY TEST PROGRAM

The laboratory testing program was performed to evaluate the physical and mechanical properties of the soils retrieved from the site to aid in verifying soil classification.

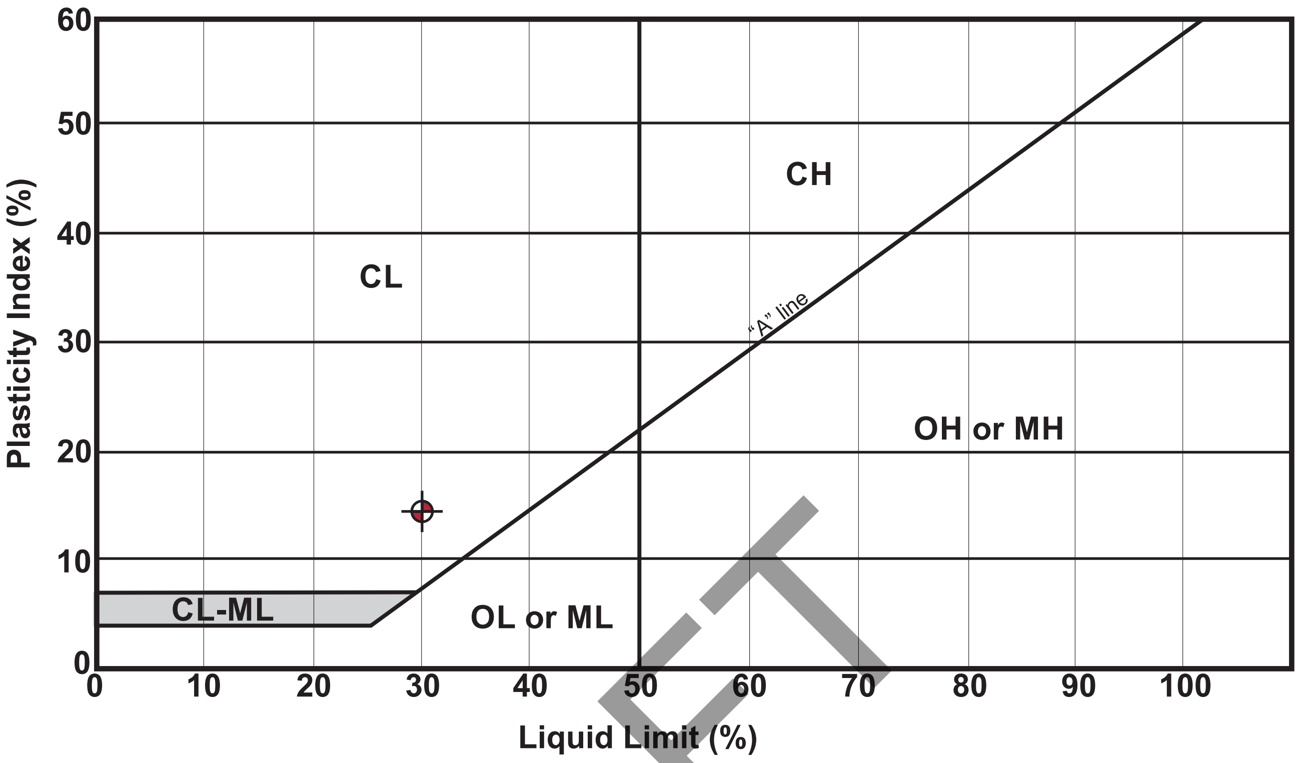
Moisture Content: The natural water content was determined (ASTM D2216) on 19 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities: In place dry density determinations (ASTM D2937) were performed on 15 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

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

Plasticity Index: One Plasticity Index determination (ASTM D4318) was performed on a sample of the subsurface soil to measure the range of water contents over which this 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 this test are shown on the boring log at the appropriate sample depth.

Plasticity Index (ASTM D4318) Testing Summary



Symbol	Boring No.	Depth (ft)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index	Passing No. 200 (%)	Group Name (USCS - ASTM D2487)
⊕	EB-2	2.0	14	30	16	14	—	Lean Clay with Sand (CL)

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Plasticity Index Testing Summary

2323 Moorpark Avenue
San Jose, CA

Project Number 648-20-2

Figure Number Figure B1

Date November 2019

Drawn By FLL

APPENDIX C: SITE CORROSIVITY EVALUATION

JDH CORROSION CONSULTANTS REPORT DATED NOVEMBER 27, 2019

jdh corrosion
consultants, inc.

Protecting the infrastructure
through innovative
Corrosion Engineering Solutions

November 27, 2019

Cornerstone Earth Group, Inc.
1220 Oakland Blvd, Suite 220
Walnut Creek, California 94596

Attention: **John R. Dye, P.E., G.E.**
Principal Engineer

Subject: **Site Corrosivity Evaluation**
2323 Moorpark Ave
San Jose, CA
Project: 648-20-2

Dear John,

In accordance with your request, we have reviewed the laboratory soils data for the above referenced project site. Our evaluation of these results and our corresponding recommendations for corrosion control for the above referenced project foundations and buried site utilities are presented herein for your consideration.

Soil Testing & Analysis

Soil Chemical Analysis

Two (2) soil samples from the project site were chemically analyzed for corrosivity by **Cooper Testing Laboratories**. Each sample was analyzed for chloride and sulfate concentration, pH, resistivity at 100% saturation and moisture percentage. The test results are presented in Cooper Testing Laboratories Corrosivity Test Summary dated 11/14/2019. The results of the chemical analysis were as follows:

Soil Laboratory Analysis

Chemical Analysis	Range of Results	Corrosion Classification*
Chlorides	4 – 11 mg/kg	Non-corrosive*
Sulfates	32 – 92 mg/kg	Non-corrosive**
pH	7.7	Non-corrosive*
Moisture (%)	4.2 – 17.6 %	Not-applicable
Resistivity at 100% Saturation	2,249 – 34,274 ohm-cm	**Moderately Corrosive to Non-corrosive***

* With respect to bare steel or ductile iron.
** With respect to mortar coated steel

Discussion

Reinforced Concrete Foundations

Due to the low levels of water-soluble sulfates found in these soils, there is no special requirement for sulfate resistant concrete to be used at this site. The type of cement used should be in accordance with California Building Code (CBC) for soils which have less than 0.10 percent by weight of water soluble sulfate (SO_4) in soil and the minimum depth of cover for the reinforcing steel should be as specified in CBC as well.

Underground Metallic Pipelines

The soils at the project site are generally considered to be "corrosive" to ductile/cast iron, steel and dielectric coated steel based on the saturated resistivity measurements. Therefore, special requirements for corrosion control are required for buried metallic utilities at this site depending upon the critical nature of the piping. Pressure piping systems such as domestic and fire water should be provided with appropriate coating systems and cathodic protection, where warranted. In addition, all underground pipelines should be electrically isolated from above grade structures, reinforced concrete structures and copper lines in order to avoid potential galvanic corrosion problems.

LIMITATIONS

The conclusions and recommendations contained in this report are based on the information and assumptions referenced herein. All services provided herein were performed by persons who are experienced and skilled in providing these types of services and in accordance with the standards of workmanship in this profession. No other warrantees or guarantees, expressed or implied, is provided.

We thank you for the opportunity to be of service to **Cornerstone Earth Group** on this project and trust that you find the enclosed information satisfactory. If you have any questions, or if we can be of any additional assistance, please feel free to contact us at (925) 927-6630.

Respectfully submitted,

Brendon Hurley

Brendon Hurley
JDH Corrosion Consultants, Inc.
Field Technician

Mohammed Ali

Mohammed Ali, P.E.
JDH Corrosion Consultants, Inc.
Principal

REGISTERED PROFESSIONAL ENGINEER
MOHAMMED ALI
No. CR 1035
Exp. June 30, 2021
CORROSION
STATE OF CALIFORNIA

CC: File19295

COOPER
TESTING LABORATORY

Corrosivity Tests Summary

CTL # 640-1365 **Date:** 11/14/2019 **Tested By:** PJ **Checked:** PJ

Client: Cornerstone Earth Group **Project:** 2323 Moorpark Ave SJ **Proj. No:** 648-20-2

Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP		Sulfide	Moisture	Soil Visual Description
			As Rec.	Min	Sat.	mg/kg	mg/kg	%		(Redox)		Qualitative	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		E_H (mv)	At Test	by Lead	%	
Boring	**Sample, No.**	**Depth, ft.**	ASTM G57	Cal 643	ASTM G57	ASTM D4327	ASTM D4327	ASTM D4327	ASTM G51	ASTM G200	Temp °C	Acetate Paper	ASTM D2216	
EB-2	2A	3.5	-	-	2,249	11	92	0.0092	7.7	-	-	-	17.6	Brown Sandy CLAY (Silty)
EB-3	3B	6.0	-	-	34,274	4	32	0.0032	7.7	-	-	-	4.2	Dark Yellowish Brown Silty SAND w/ Gravel