APPENDIX D

Geotechnical Report

CORNERSTONE
EARTH GROUP

TYPE OF SERVICES	Preliminary Geotechnical Investigation
PROJECT NAME	95 South Almaden Avenue
LOCATION	95 South Almaden Avenue San Jose, California
CLIENT	JP DiNapoli Companies Inc.
PROJECT NUMBER	510-29-3
DATE	November 5, 2019

GEOTECHNICAL

CORNERSTONE
EARTH GROUP

Type of Services	**Preliminary Geotechnical Investigation**
Project Name	**95 South Almaden Avenue**
Location	**95 South Almaden Avenue** **San Jose, California**
Client	**JP DiNapoli Companies Inc.**
Client Address	**99 Almaden Boulevard, Suite 565** **San Jose, CA**
Project Number	**510-29-3**
Date	**November 5, 2019**

Renee S Morales

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SECTION 1: INTRODUCTION

This preliminary geotechnical report was prepared for the sole use of JP DiNapoli Companies Inc. for the 95 South Almaden Avenue project 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

We understand the project is still in the early planning stage. However, based on the information provided, the project will include redeveloping the approximately 1.3-acre site for a new mixed-use office development. The new development will include a 19-story tower over a podium level with three to four levels of below-grade parking. We estimate the building will encompass the entire site.

Structural loads are not available at this time; however, structural loads are expected to be typical for similar structures. Cuts on the order 40 to 50 feet are expected for below-grade parking.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated September 6, 2019 and consisted of field and laboratory programs to evaluate physical and engineering properties of the subsurface soils, engineering analysis to prepare preliminary recommendations for site work and grading, building foundations, and preparation of this report. Brief descriptions of our exploration and laboratory programs are presented below.

1.3 EXPLORATION PROGRAM

Field exploration consisted of four borings drilled on September 28th and 29th, 2019 with truck-mounted hollow-stem auger drilling equipment. The borings were drilled to depths ranging from 40 to 70 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, Plasticity Index tests, triaxial compression tests, and consolidation tests. Details regarding our laboratory program are included in Appendix B.

1.5 ENVIRONMENTAL SERVICES

Cornerstone Earth Group also provided environmental services for this project, including Phase 1 and 2 site assessments; environmental findings and conclusions are being 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. The alluvial in the area of the site is mapped to be greater than 500 feet thick (Rogers & Williams, 1974).

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, geologists from the U.S. Geological Survey have recently updated earlier estimates from their 2015 Uniform California Earthquake Rupture Forecast (Version 3) publication. The estimated probability of one or more magnitude 6.7 earthquakes (the size of the destructive 1994 Northridge earthquake) expected to occur somewhere in the San Francisco Bay Area has been revised (increased) to 72 percent for the period 2014 to 2043 (Aagaard et al., 2016). The faults in the region with the highest estimated probability of generating damaging earthquakes between 2014 and 2043 are the Hayward (33%), Rodgers Creek (33%), Calaveras (26%), and San Andreas Faults (22%). In this 30-year period, the probability of an earthquake of magnitude 6.7 or larger occurring is 22 percent along the San Andreas Fault and 33 percent for the Hayward or Rodgers Creek Faults.

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)
Hayward (Southeast Extension)	6.2	10
Monte Vista-Shannon	7.2	11.6
Calaveras	9.0	14.5
Hayward (Total Length)	9.1	14.6
San Andreas (1906)	11.4	18.4

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 currently occupied by an asphalt parking lot and planters with small to medium trees. The site is relatively level with site elevations ranging from approximately Elevation 84 to 85 (Google Earth, WGS84). The site is bounded by Post Street to the north, South Almaden Avenue to the east, South Almaden Avenue to the west, and an existing structure to the south.

At our boring locations, surface pavements consisted of 2 to 3 inches of asphalt concrete over 1 to 8 inches of aggregate base. Based on visual observations, the existing pavements are in poor condition with significant alligator cracking.

3.2 SUBSURFACE CONDITIONS

Below the surface pavements, our explorations generally encountered undocumented fill to depths of approximately 3 to 5 feet below existing ground surface. The fills consisted of lean clay with variable amounts of sand and gravel. The fill is generally underlain by medium stiff to hard lean clay with variable amounts of silt and sand interbedded with loose to medium dense sand with variable amounts of silt and clay, dense to very dense poorly graded sand with variable amounts of silt and sand, medium stiff to stiff silt, and stiff to very stiff fat clay to the maximum depth explored of 70 feet below existing ground surface. See Appendix A for additional details.

3.2.1 Plasticity/Expansion Potential

We performed 10 Plasticity Index (PI) tests on representative samples. Test results were used to evaluate expansion potential of surficial soils, and the plasticity of the fines in potentially liquefiable layers. The near surface tests resulted in PIs of 8 and 12, indicating low expansion potential. Deeper PI testing resulted in PIs ranging from 6 to 42, indicating low to high 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 40 feet range from 9 to 45 percent moisture. In our opinion, we estimated this corresponds to about near optimum to 20 percent above the estimated laboratory optimum moisture.

3.3 GROUNDWATER

Groundwater was encountered in our borings at depths ranging from 20 to 24 feet below existing ground surface. All measurements were taken at the time of drilling and may not represent the stabilized levels that can be higher than the initial levels encountered.

Historic high groundwater levels are mapped at a depth of approximately 15 feet below current grades (CGS, San Jose West, 7.5-minute Quadrangle, 2002). In general, fluctuations in groundwater levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors. On a preliminary basis, based on our explorations, review of historic depth to groundwater maps, and our experience within the site area, we anticipate a high groundwater level of 15 feet below existing grades and recommend a design groundwater depth of 15 feet below the existing site grades be used.

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 Santa Clara County Fault Hazard 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. A peak ground acceleration (PGA) was estimated for analysis using value equal to F_{PGA} x PGA, as allowed in the 2016 edition of the California Building Code. For our liquefaction analysis we used a PGA_M of 0.5g.

4.3 LIQUEFACTION POTENTIAL

The site is located within a State-designated Liquefaction Hazard Zone (CGS, San Jose West Quadrangle, 2002). Our preliminary investigation addressed this issue by performing a boring exploration that extended to a depth of at least 50 feet.

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 4 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 previously discussed, historic high groundwater in the area is mapped to be on the order of15 feet below the ground surface. In addition, the site is underlain by alluvial deposits consisting of clayey, silty, and sandy soils. The granular materials, including sandy soils, are anticipated to be generally medium dense to dense in consistency. As a result, there is the potential for liquefaction to impact site development. Currently up to a 4-level below grade parking garage is planned with cuts extending to approximately 40 feet below existing grades. As such, we anticipate much of the potentially liquefiable soils beneath the surface will likely be removed for the basement excavation. Based on our exploration below the bottom of the proposed basement we anticipate the potential for liquefaction below 40 feet to be low.

We recommend the potential for liquefaction be evaluated during the design-level geotechnical investigation once the project plans are finalized.

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.

Guadalupe Creek is located approximately 1000 feet to the west of the site and is about 20 feet deep. However, the creek channel bottom and sides are concrete lined. In our opinion, the concrete lining of the creek would likely prevent lateral spreading from occurring and affecting improvements.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose to medium dense unsaturated sandy soils can settle during strong seismic shaking. If loose to medium dense unsaturated sandy soils are present at the site, these soils could experience differential seismic settlement after strong seismic shaking. Currently, up to a 4-level below-grade parking garage is planned. As such, we anticipate all the unsaturated soils beneath the surface will likely be removed for the basement excavation.

We recommend the potential for unsaturated sand shaking settlement to affect the surface improvements be evaluated during the design-level geotechnical investigation once the project plans are finalized.

4.6 TSUNAMI/SEICHE

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 7½ miles inland from the San Francisco Bay shoreline, and is approximately 84 to 85 feet above mean sea level according to Google Earth®. 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 X, an area described as, "Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood." 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 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.

- Shallow groundwater
- Presence of undocumented fill
- Potential for static settlement
- Wet, unstable excavation subgrade

- Shoring considerations for the below-grade excavation
- Surcharge loading on basement walls
- Differential movement at on-grade to on-structure transitions
- Presence of granular soils

5.1.1 Shallow Groundwater

Shallow groundwater was measured in our borings at depths ranging from approximately 20 to 24 feet below the existing ground surface. Historic high groundwater is mapped at depths of approximately 15 feet below the existing ground surface. Our experience with similar sites in the vicinity indicates that shallow groundwater could significantly impact grading and underground construction. These impacts typically consist of potentially wet and unstable pavement subgrade, difficulty achieving compaction, and difficult underground utility installation. As discussed above, a basement on the order of 40 feet is proposed for the site, this basement will extend below design groundwater depths and as such will need to be designed to resist hydrostatic pressures. Significantly wet or potentially unstable subgrade at the bottom of the excavation should be anticipated. This should be further evaluated during the design level investigation.

5.1.2 Presence of Undocumented Fill

As discussed above, three to five feet of undocumented fills were encountered in our exploratory borings, however we anticipate that fills within the building footprint will be removed during the proposed basement excavation. Any undocumented fill that is not removed as part of the basement excavation may impact at-grade improvements. The impacts of undocumented fill on at-grade improvements should be further discussed in the design level investigation.

5.1.3 Potential for Static Settlement

Basement foundations will bear well below the groundwater table. A mat foundation may be feasible provided maximum allowable bearing capacities are not exceeded, and total static settlement estimates are tolerable. Our preliminary analysis of foundation settlement, based on estimated structural loads, result in static settlement on the order of 1 to 2 inches beneath the structure. As these values are based on preliminary analysis, the static settlement and feasibility of a mat foundation should be further evaluated during the design-level investigation.

5.1.4 Wet, Unstable Excavation Subgrade

The proposed building excavation will extend into saturated clay and sand with varying strength. Due to the high moisture content of this material, it will likely be unstable under the weight of track-mounted or rubber-tired construction equipment. To provide a firm working base for construction of the foundation, it may be necessary to stabilize the bottom of the excavation prior to construction of the mat or other foundation elements. Stabilization alternatives should be further discussed in a design level investigation.

5.1.5 Shoring Considerations for the Below-Grade Excavation

An excavation of up to about 40 feet deep is being considered for the structure. The primary considerations in selecting a suitable shoring system typically include 1) control of vertical and lateral ground surface or wall movements, 2) constructability, 3) dewatering and 4) cost. Shoring considerations should be further evaluated during the design level investigation.

5.1.6 Surcharge Loading on Basement Walls

Existing improvements and new at-grade improvements may generate additional surcharge loads onto the basement walls that extend below the improvement or foundation. These surcharge loads should be accounted for in the design of the shoring and basement walls.

5.1.7 Differential Movement at On-grade to On-Structure Transitions

Some of the proposed improvements will transition from on-grade support to overlying the basements. Where the depth of soil cover overlying the basement roof is thin or where basement walls extend to within inches of finished grade, these transition areas typically experience increased differential movement due to a variety of causes, including difficulty in achieving compaction of retaining wall backfill closest to the wall. We recommend consideration be given to where engineered fill is placed behind retaining walls extending to near finished grade, and that subslabs be included beneath flatwork or pavers that can cantilever at least 3 feet beyond the wall. If surface improvements are included that are highly sensitive to differential movement, additional measures may be necessary.

5.1.8 Presence of Granular Soils

As discussed, layers of granular soils with variable amounts of clay and silt fines and gravels were encountered at various depths between about 6 to 54 feet in some of our explorations. Contractors should plan on needing to form excavations within the zones where sands with low fines contents are encountered, as well as other similar construction issues as relates to temporary shoring, utility excavations, and granular material at the base of below-grade excavations. The impact of granular soils at the bottom of the basement should be further evaluated during the design level investigation.

5.2 DESIGN-LEVEL GEOTECHNICAL INVESTIGATION

The preliminary recommendations contained in this study were based on preliminary site development information and limited information from our field investigation. As site conditions may vary significantly between the small-diameter borings performed during this investigation, 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, as discussed above we understand a basement on the order of 40 feet below grade will is planned over the entire site, therefore fills within the building footprint are anticipated to be removed during excavation.

Shallow groundwater is present at depths as shallow as 15 feet below grade and could potentially perch at shallower depths. Temporary dewatering should be anticipated for the basement excavation. Near saturated to saturated soils should be expected that will require drying back to be re-used as engineered fill. These soils will also be subject to destabilization by rubber-tired and other construction equipment.

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 3 percent away from buildings. Biotreatment basins should be kept at least 10 feet away from buildings and, where possible, at least 3 feet from pavements and flatwork.

SECTION 7: FOUNDATIONS

7.1 SUMMARY OF RECOMMENDATIONS

On a preliminary basis, the mid-rise tower building may potentially be supported on reinforced concrete mat foundations. If static settlements are not tolerable for a reinforced concrete mat foundation, the proposed building may also be supported by ground improvement or deep foundations. Preliminary recommendations are discussed in the following sections.

7.2 SEISMIC DESIGN CRITERIA

We understand that the project structural design will likely be based on the 2019 California Building Code (CBC), which provides criteria for the seismic design of buildings in Chapter 16. The “Seismic Coefficients” used to design buildings are established based on a series of tables and figures addressing different site factors, including the soil profile in the upper 100 feet below grade and mapped spectral acceleration parameters based on distance to the controlling seismic source/fault system. Based on our borings and review of local geology, the site is underlain by deep alluvial soils with typical SPT “N” values between 15 and 50 blows per foot. Therefore, on a preliminary basis, the site may be classified as Site Class D. The mapped spectral acceleration parameters S_S and S_1 were calculated using the SEAOC web-based program Seismic Design Maps, located at https://seismicmaps.org/, based on the site coordinates presented below and the site classification.

ASCE 7-16 Section 11.4.8 requires that a ground motion hazard analysis be performed for Site Class D sites with mapped S_1 values greater than 0.2. Based on our review of the 2019 CBC, a site-specific hazard analysis in accordance with ASCE 7-16 Chapter 21.2 may be required using the UCERF3 model, which is used in the USGS model. This is outside of our current scope of work. A site-specific analysis should be planned for and performed during the design-level investigation. The values in Table 3 should not be used for design. Values are provided for determination of Seismic Design Category and comparison with minimum code requirements in future site-specific ground motion hazard analysis, as required.

Table 3: CBC Site Categorization and Site Coefficients

Classification/Coefficient	**Design Value**
Site Class	D
Site Latitude	37.33278990°
Site Longitude	- 121.89317020°
0.2-second Period Mapped Spectral Acceleration[1], S_S	1.5 g
1-second Period Mapped Spectral Acceleration[1], S_1	0.6 g
Short-Period Site Coefficient – Fa	1.0
Long-Period Site Coefficient – Fv[2]	null[3]
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S_{MS}	1.5 g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	null[3]
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.0 g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	null[3]

[1] For Site Class B, 5 percent damped.

[2] Fv determined based on criteria in ASCE 7-16 Section 21.2.2 and 21.3 for calculation of deterministic lower limit and calculation of code-based spectrum for comparison with site-specific values.

[3] Site-specific analysis required or exceptions taken.

7.3 REINFORCED CONCRETE MAT FOUNDATIONS

On a preliminary basis, the proposed structure may be supported on a mat foundation bearing on natural soil or engineered fill provided the estimate static settlements are tolerable. Structural loads are not yet available; however, we estimated average areal pressures for a 12-story above grade with 4-story below grade building to be on the order of 2,000 to 2,500 psf. Based on this assumed loading, on a preliminary basis we estimate static settlements for the above loading would be on the order of 1 to 2 inches. In addition to estimated static settlements, the mats would also need to be designed to accommodate estimated seismic settlements.

For your project planning, we recommend preliminary maximum allowable bearing pressure on the order of 4,000 to 5,000 psf for dead plus live loads in isolated heavily loaded areas of the mat. When evaluating wind and seismic conditions, allowable bearing pressures may be increased by one-third. These pressures are net values; the weight of the mat may be neglected for the portion of the mat extending below grade. Top and bottom mats of reinforcing steel should be included as required to help span irregularities and differential settlement.

The feasibility of mat foundations should be evaluated during the design-level geotechnical investigation. We will revise our analysis and recommendations during the design-level geotechnical investigation to adjust for any update to the project scope.

On a preliminary basis, consideration should be given to ground improvement or deep foundations if the design loads exceed the preliminary allowable bearing pressures above or if the estimated settlements exceed allowable settlements.

7.3.1 Hydrostatic Uplift and Waterproofing

Where portions of the structures extend below the design groundwater level, including bottoms of slabs-on-grade and mat foundations, they should be designed to resist potential hydrostatic uplift pressures. Retaining walls extending below design groundwater should be waterproofed and designed to resist hydrostatic pressure for the full wall height. Where portions of the walls extend above the design groundwater level, a drainage system may be added. Waterproofing and retaining walls should be further evaluated during the design-level investigation.

SECTION 8: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of JP DiNapoli Companies Inc. specifically to support the design of the 95 South Almaden Avenue 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 groundwater 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.

JP DiNapoli Companies Inc. may have provided Cornerstone with plans, reports and other documents prepared by others. JP DiNapoli Companies 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 9: REFERENCES

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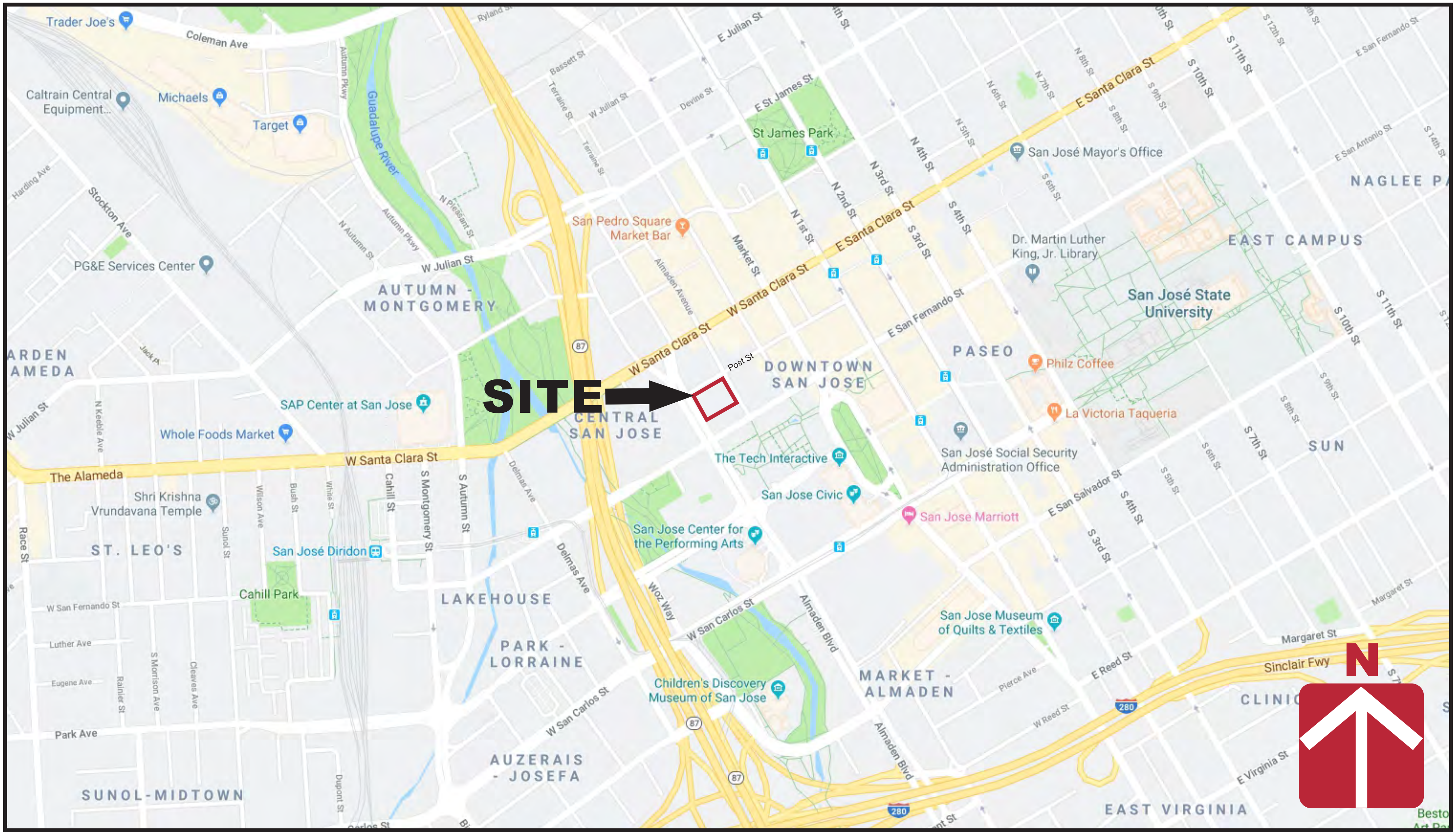

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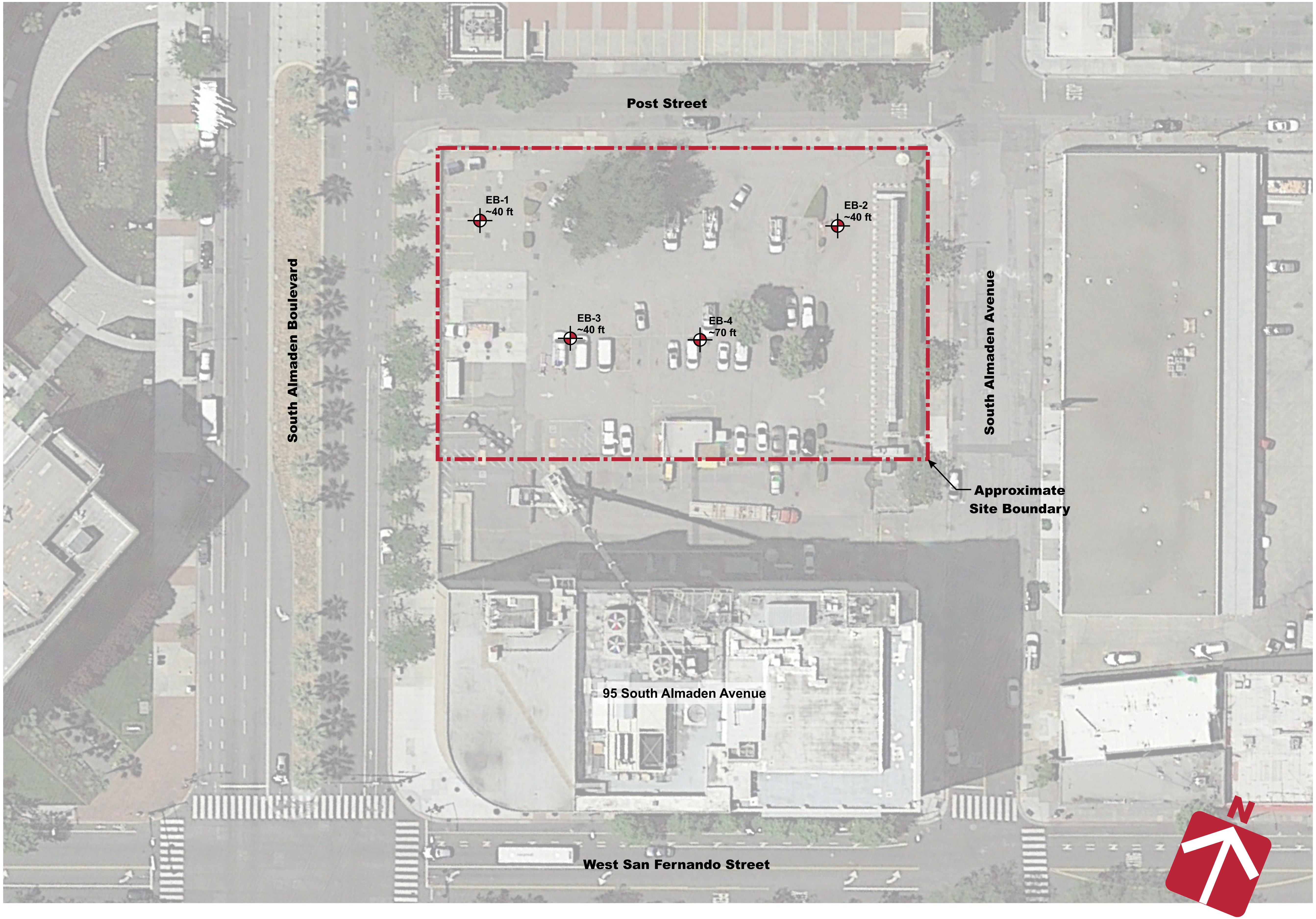
Vicinity Map

95 South Almaden Avenue
San Jose, CA

Project Number	510-29-3
Figure Number	Figure 1
Date	October 2019
Drawn By	RRN



Legend

⌖ Approximate location of environmental boring (EB)

0 50 100

APPROXIMATE SCALE (FEET)

Base by Google Earth, dated 5/9/2018

Project Number 510-29-3

Figure Number Figure 2

Date October 2019

Drawn By RRN

Site Plan

95 South Almaden Avenue
San Jose, CA

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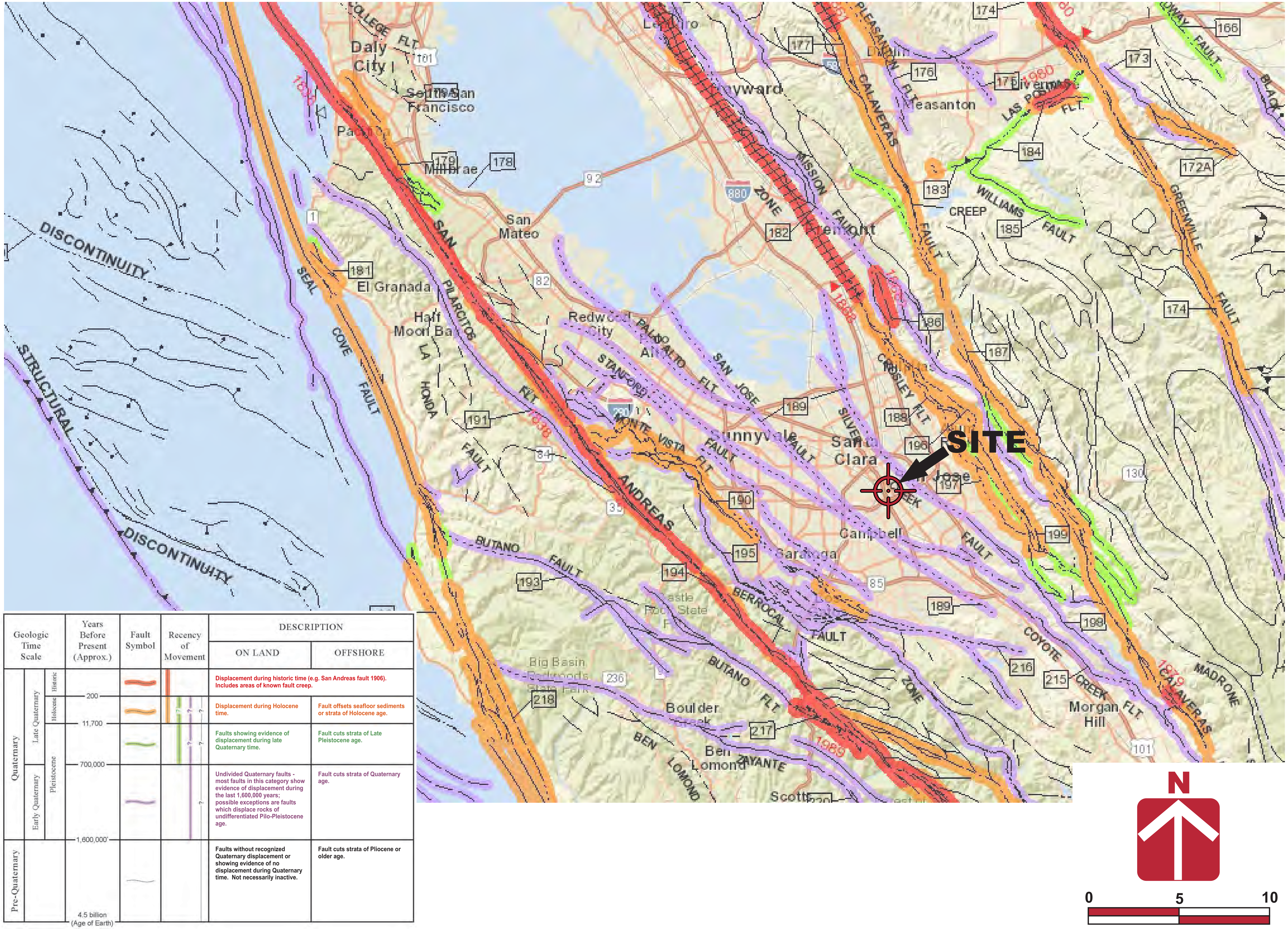


Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION	
						ON LAND	OFFSHORE
Quaternary	Late Quaternary	Historic		🟥		Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	200	🟧	?	Displacement during 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
510-29-3

Figure Number
Figure 3

Drawn By
RRN

Date
October 2019

Regional Fault Map

95 South Almaden 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. Four 8-inch-diameter exploratory borings were drilled on September 28 and 29, 2019 to depths of 40 to 70 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 and bedrock, are included as part of this appendix.

Boring locations were approximated using existing site boundaries, a hand held GPS unit, and other site features as references. Boring elevations were not determined. The locations of the borings should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. 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. Relatively undisturbed samples were also obtained with 2.875-inch I.D. Shelby Tube sampler which were hydraulically pushed. 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.

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-10)

MATERIAL TYPES	CRITERIA FOR ASSIGNING SOIL GROUP NAMES			GROUP SYMBOL	SOIL GROUP NAMES & LEGEND
COARSE-GRAINED SOILS >50% RETAINED ON NO. 200 SIEVE	GRAVELS >50% OF COARSE FRACTION RETAINED ON NO 4. SIEVE	CLEAN GRAVELS <5% FINES	Cu>4 AND 1<Cc<3	GW	WELL-GRADED GRAVEL
			Cu>4 AND 1>Cc>3	GP	POORLY-GRADED GRAVEL
		GRAVELS WITH FINES >12% FINES	FINES CLASSIFY AS ML OR CL	GM	SILTY GRAVEL
			FINES CLASSIFY AS CL OR CH	GC	CLAYEY GRAVEL
	SANDS >50% OF COARSE FRACTION PASSES ON NO 4. SIEVE	CLEAN SANDS <5% FINES	Cu>6 AND 1<Cc<3	SW	WELL-GRADED SAND
			Cu>6 AND 1>Cc>3	SP	POORLY-GRADED SAND
		SANDS AND FINES >12% FINES	FINES CLASSIFY AS ML OR CL	SM	SILTY SAND
			FINES CLASSIFY AS CL OR CH	SC	CLAYEY SAND
FINE-GRAINED SOILS >50% PASSES NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT<50	INORGANIC	PI>7 AND PLOTS>"A" LINE	CL	LEAN CLAY
			PI>4 AND PLOTS<"A" LINE	ML	SILT
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OL	ORGANIC CLAY OR SILT
	SILTS AND CLAYS LIQUID LIMIT>50	INORGANIC	PI PLOTS >"A" LINE	CH	FAT CLAY
			PI PLOTS <"A" LINE	MH	ELASTIC SILT
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OH	ORGANIC CLAY OR SILT
HIGHLY ORGANIC SOILS		PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR		PT	PEAT

OTHER MATERIAL SYMBOLS

Poorly-Graded Sand with Clay
Clayey Sand
Sandy Silt
Artificial/Undocumented Fill
Poorly-Graded Gravelly Sand
Topsoil
Well-Graded Gravel with Clay
Well-Graded Gravel with Silt
Sand
Silt
Well Graded Gravelly Sand
Gravelly Silt
Asphalt
Boulders and Cobble

SAMPLER TYPES

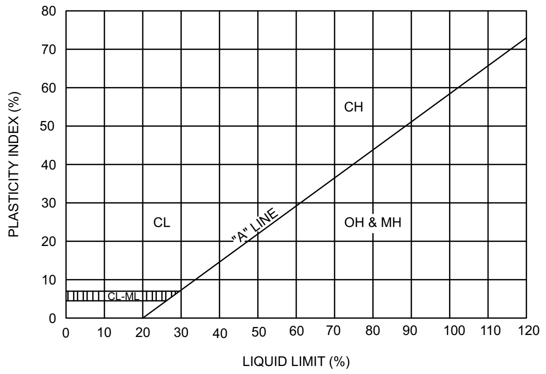
☒ SPT
⧗ Modified California (2.5" I.D.)
▯ Rock Core
■ Shelby Tube
○ No Recovery
Grab Sample

ADDITIONAL TESTS

CA - CHEMICAL ANALYSIS (CORROSIVITY)
CD - CONSOLIDATED DRAINED TRIAXIAL
CN - CONSOLIDATION
CU - CONSOLIDATED UNDRAINED TRIAXIAL
DS - DIRECT SHEAR
PP - POCKET PENETROMETER (TSF)
(3.0) - (WITH SHEAR STRENGTH IN KSF)
RV - R-VALUE
SA - SIEVE ANALYSIS: % PASSING #200 SIEVE
▼ - WATER LEVEL

PI - PLASTICITY INDEX
SW SWELL TEST
TC - CYCLIC TRIAXIAL
TV - TORVANE SHEAR
UC - UNCONFINED COMPRESSION
(1.5) - (WITH SHEAR STRENGTH IN KSF)
UU - UNCONSOLIDATED UNDRAINED TRIAXIAL

PLASTICITY CHART



PENETRATION RESISTANCE (RECORDED AS BLOWS / FOOT)				
SAND & GRAVEL		SILT & CLAY		
RELATIVE DENSITY	BLOWS/FOOT*	CONSISTENCY	BLOWS/FOOT*	STRENGTH** (KSF)
VERY LOOSE	0 - 4	VERY SOFT	0 - 2	0 - 0.25
LOOSE	4 - 10	SOFT	2 - 4	0.25 - 0.5
MEDIUM DENSE	10 - 30	MEDIUM STIFF	4 - 8	0.5-1.0
DENSE	30 - 50	STIFF	8 - 15	1.0 - 2.0
VERY DENSE	OVER 50	VERY STIFF	15 - 30	2.0 - 4.0
		HARD	OVER 30	OVER 4.0

* NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1-3/8 INCH I.D.) SPLIT-BARREL SAMPLER THE LAST 12 INCHES OF AN 18-INCH DRIVE (ASTM-1586 STANDARD PENETRATION TEST).

** UNDRAINED SHEAR STRENGTH IN KIPS/SQ. FT. AS DETERMINED BY LABORATORY TESTING OR APPROXIMATED BY THE STANDARD PENETRATION TEST, POCKET PENETROMETER, TORVANE, OR VISUAL OBSERVATION.

CORNERSTONE
EARTH GROUP

LEGEND TO SOIL DESCRIPTIONS

Figure Number
A-1

CORNERSTONE
EARTH GROUP

BORING NUMBER EB-1

PAGE 1 OF 2

PROJECT NAME 95 South Almaden Boulevard
PROJECT NUMBER 510-29-2
PROJECT LOCATION San Jose, CA
DATE STARTED 9/28/19 **DATE COMPLETED** 9/28/19
GROUND ELEVATION ______ **BORING DEPTH** 40 ft.
DRILLING CONTRACTOR Geoservices Exploration Inc.
LATITUDE 37.333227° **LONGITUDE** -121.893993°
DRILLING METHOD Mobile B-61, 8 inch Hollow-Stem Auger
GROUNDWATER LEVELS:
LOGGED BY BCG
▽ **AT TIME OF DRILLING** 22 ft.
NOTES ______
▼ **AT END OF DRILLING** 22 ft.

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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	2 inches asphalt concrete over 1 inches aggregate base							
		Lean Clay with Sand (CL) [Fill] very stiff, moist, dark brown, fine sand, low plasticity		GB-1		16			
		brick fragments from 2-3'							
		Sandy Silty Clay (CL-ML) hard, moist, brown to light brown, fine sand, low plasticity		GB-2		15			
	5	Liquid Limit = 24, Plastic Limit = 18	27	MC-3B	111	16	6		>4.5 ○
		Silty Sand (SM) medium dense, moist, brown, fine sand	25	MC-4B	84	11			
		Lean Clay with Sand (CL) very stiff, moist, brown and gray mottled, fine sand, moderate plasticity	18	MC-5B	95	25			○
	10								
		Lean Clay (CL) hard, gray to dark gray, some fine sand, moderate plasticity	43	MC-6A	104	22			○
	15								
		Silty Sand (SM) medium dense, moist, gray, fine to medium sand, some fine subangular to subrounded gravel	27	MC-7B	113	14			
	20								
▼		**Poorly Graded Sand with Silt (SP-SM)** dense, moist, gray, fine to coarse sand, some fine subangular to subrounded gravel							
		See sieve analysis results.	77	MC-8B	114	15		11	
	25								

Continued Next Page

CORNERSTONE
EARTH GROUP

BORING NUMBER EB-1

PAGE 2 OF 2

PROJECT NAME 95 South Almaden Boulevard
PROJECT NUMBER 510-29-2
PROJECT LOCATION San Jose, CA

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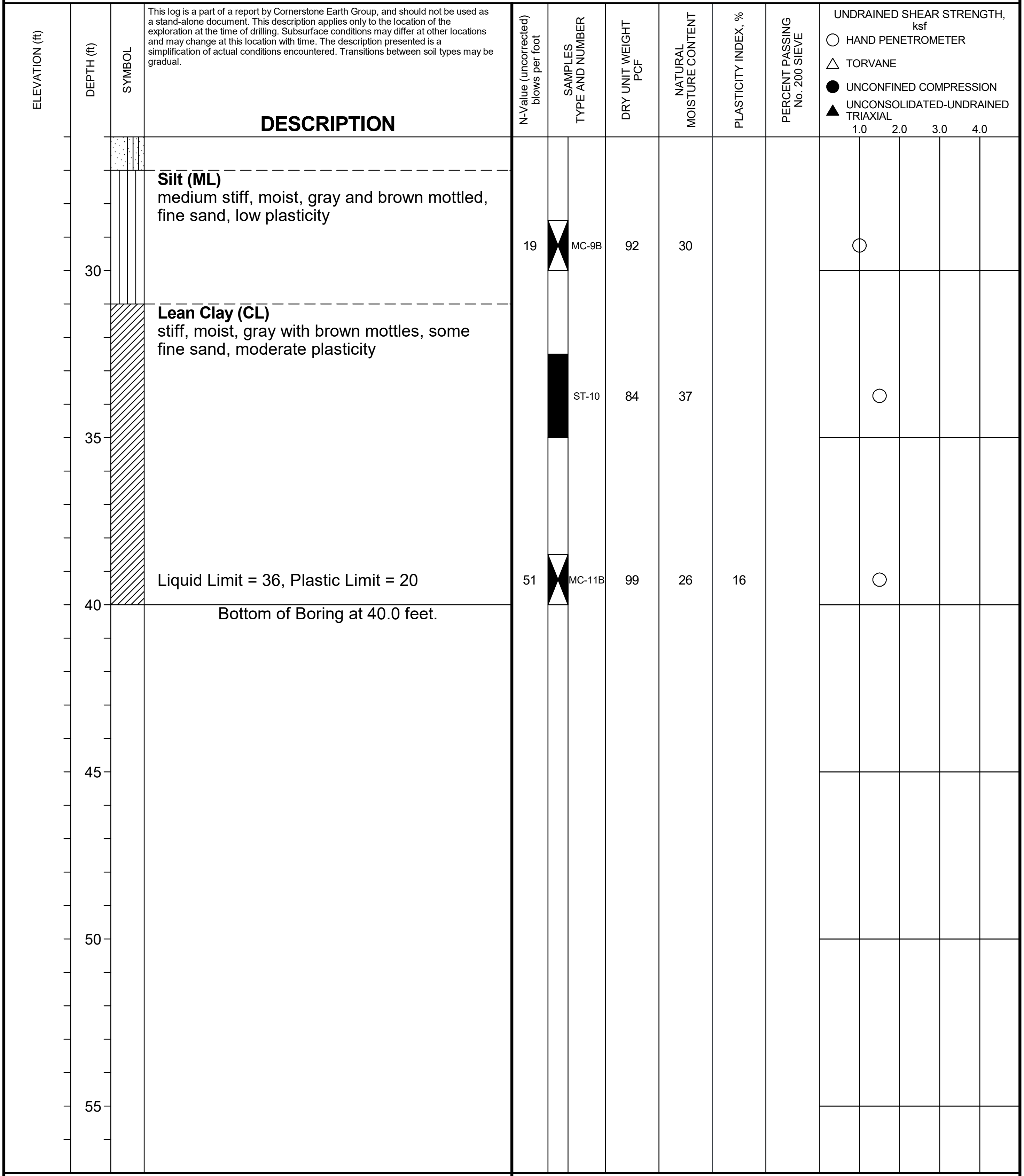

Silt (ML)
medium stiff, moist, gray and brown mottled, fine sand, low plasticity

Lean Clay (CL)
stiff, moist, gray with brown mottles, some fine sand, moderate plasticity

Liquid Limit = 36, Plastic Limit = 20

Bottom of Boring at 40.0 feet.

N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
19	MC-9B	92	30		
	ST-10	84	37		
51	MC-11B	99	26	16	

BORING NUMBER EB-2

PAGE 1 OF 2

CORNERSTONE
EARTH GROUP

PROJECT NAME 95 South Almaden Boulevard
PROJECT NUMBER 510-29-2
PROJECT LOCATION San Jose, CA
DATE STARTED 9/29/19 **DATE COMPLETED** 9/29/19
GROUND ELEVATION **BORING DEPTH** 40 ft.
DRILLING CONTRACTOR Geoservices Exploration Inc.
LATITUDE 37.333464° **LONGITUDE** -121.893360°
DRILLING METHOD Mobile B-61, 8 inch Hollow-Stem Auger
GROUNDWATER LEVELS:
LOGGED BY BCG
▽ **AT TIME OF DRILLING** 24 ft.
NOTES
▼ **AT END OF DRILLING** 31 ft.

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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 8 inches aggregate base							
		Sandy Lean Clay (CL) [Fill] very stiff, moist, brown, fine sand, fine to coarse subangular to subrounded gravel, low plasticity Liquid Limit = 29, Plastic Limit = 17 See sieve analysis results.		GB-1		16	12	69	
				GB-2		16			
	5	**Sandy Silty Clay (CL-ML)** stiff, moist, brown, fine sand, low plasticity	21	MC-3B	94	9			
		Silty Sand (SM) medium dense, moist, brown, fine sand, trace fine subangular to subrounded gravel							
		Sandy Lean Clay (CL) stiff, moist, brown, fine sand, low plasticity							
		Silty Sand (SM) medium dense, moist, reddish brown, fine to medium sand See sieve analysis results.	26	MC-4B	88	11		16	
	10								
		Lean Clay (CL) stiff, moist, brown and gray mottled, some fine sand, moderate plasticity Liquid Limit = 42, Plastic Limit = 26	21	MC-5B	77	40	16		○
	15								
		Silty, Clayey Sand (SC-SM) medium dense, wet, brown and gray mottled, fine to medium sand, some fine subangular to subrounded gravel Liquid Limit = 26, Plastic Limit = 19 See sieve analysis results.	29	MC-6B	92	37	7	36	○
	20								
▽		**Poorly Graded Sand with Silt (SP-SM)** dense, wet, gray, fine to coarse sand, some fine subangular to subrounded gravel	86	MC-7B	124	13			
	25								

Continued Next Page

CORNERSTONE
EARTH GROUP

BORING NUMBER EB-2

PAGE 2 OF 2

PROJECT NAME 95 South Almaden Boulevard

PROJECT NUMBER 510-29-2

PROJECT LOCATION San Jose, CA

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ELEVATION (ft)	DEPTH (ft)	SYMBOL	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
		▨	**Poorly Graded Sand with Silt (SP-SM)** dense, wet, gray, fine to coarse sand, some fine subangular to subrounded gravel						
			becomes medium dense	28	MC				
	30	▨	**Lean Clay (CL)** stiff, moist, gray with brown mottles, some fine sand, moderate plasticity						
	▼								
				38	MC-9B	94	30		
	35								
				43	MC-10B	96	27		
	40		Bottom of Boring at 40.0 feet.						
	45								
	50								
	55								

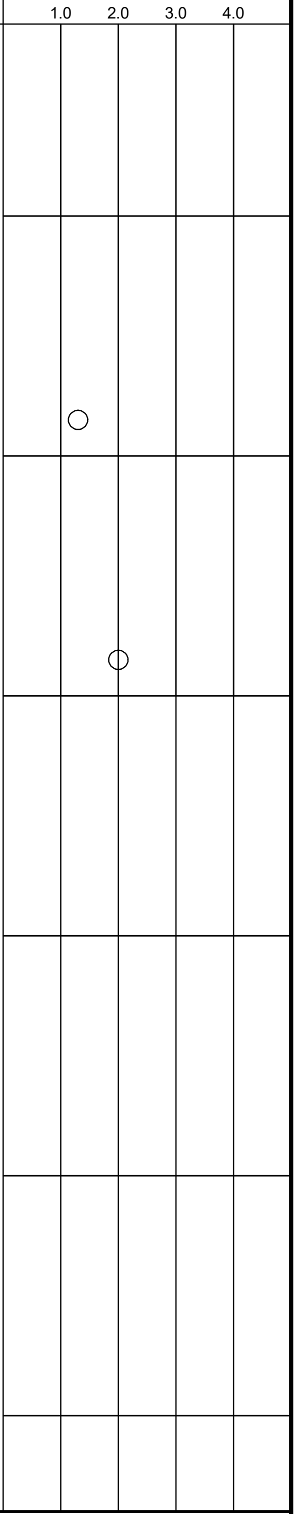
UNDRAINED SHEAR STRENGTH, ksf

○ HAND PENETROMETER

△ TORVANE

● UNCONFINED COMPRESSION

▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL



BORING NUMBER EB-3

PAGE 1 OF 2

CORNERSTONE EARTH GROUP

PROJECT NAME 95 South Almaden Boulevard
PROJECT NUMBER 510-29-2
PROJECT LOCATION San Jose, CA
DATE STARTED 9/29/19 **DATE COMPLETED** 9/29/19
GROUND ELEVATION **BORING DEPTH** 41.5 ft.
DRILLING CONTRACTOR Geoservices Exploration Inc.
LATITUDE 37.333123° **LONGITUDE** -121.893904°
DRILLING METHOD Mobile B-61, 8 inch Hollow-Stem Auger
GROUNDWATER LEVELS:
LOGGED BY BCG
▽ **AT TIME OF DRILLING** 22 ft.
NOTES
▼ **AT END OF DRILLING** 33 ft.

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ELEVATION (ft)	DEPTH (ft)	SYMBOL	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 ○ HAND PENETROMETER △ TORVANE ● UNCONFINED COMPRESSION ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL 1.0 2.0 3.0 4.0
	0		3 inches asphalt concrete over 8 inches aggregate base							
			Clayey Sand with Gravel (SC) [Fill] medium dense, moist, brown, fine to coarse sand, fine subangular to subrounded gravel, trace brick and concrete fragments		GB					
			Lean Clay with Sand (CL) [Fill] very stiff, moist, dark grayish brown, fine sand, some brick fragments, low plasticity		GB					
	5		**Sandy Lean Clay (CL)** very stiff, moist, brown, fine sand, low plasticity	26	MC-3B	106	13			○
			Silty Sand (SM) medium dense, moist, brown to light brown, fine sand							
			Sandy Silt (ML) stiff, moist, gray with brown mottles, fine sand, low plasticity	30	MC-4B	91	28			○
	10		**Fat Clay (CH)** stiff, moist, brown and gray mottled, some fine sand, high plasticity Liquid Limit = 54, Plastic Limit = 27		MC-5B	86	34	27		○
			Lean Clay with Sand (CL) hard, moist, dark grayish brown, fine to medium sand, some fine subangular to subrounded gravel, moderate plasticity Liquid Limit = 47, Plastic Limit = 24	80	MC-6B	96	28	23		○
	15									
			Clayey Sand with Gravel (SC) loose, moist, brown, fine to medium sand, fine subangular to subrounded gravel See sieve analysis results.	16	MC-7	113	18		29	
	20									
▽										
			Silty Sand (SM) medium dense, moist, gray, fine to medium sand See sieve analysis results.	29	MC-8B	106	23		17	
	25									

Continued Next Page

CORNERSTONE EARTH GROUP

BORING NUMBER EB-3

PAGE 2 OF 2

PROJECT NAME 95 South Almaden Boulevard

PROJECT NUMBER 510-29-2

PROJECT LOCATION San Jose, CA

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
Silty Sand (SM) medium dense, moist, gray, fine to medium sand	37	MC-9B	106	21		
Fat Clay (CH) stiff, moist, dark gray, some fine sand, high plasticity Liquid Limit = 65, Plastic Limit = 23	46	MC-10B	83	37	42	
Lean Clay with Sand (CL) stiff, moist, gray with brown mottles, fine sand, moderate plasticity	50	MC-11B	108	20		
Poorly Graded Sand with Silt (SP-SM) very dense, moist, brown, fine to medium sand	84	SPT-12		23		

Bottom of Boring at 41.5 feet.

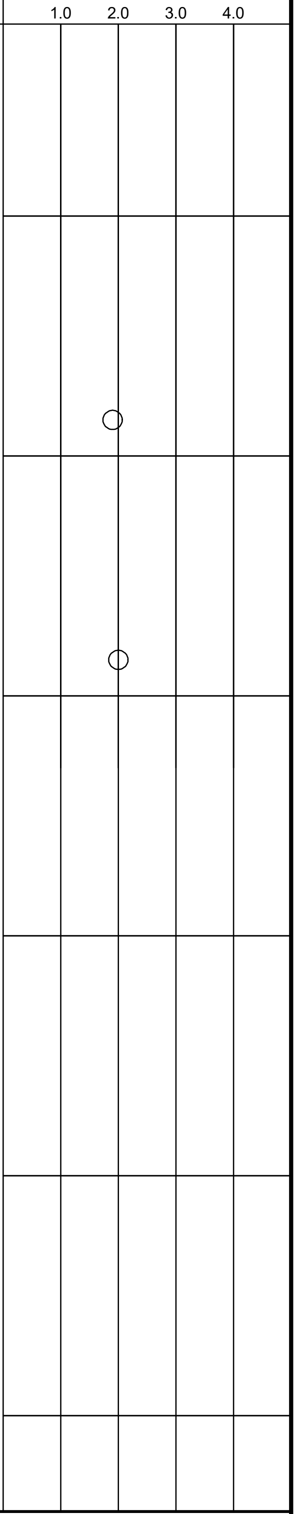
UNDRAINED SHEAR STRENGTH, ksf

○ HAND PENETROMETER

△ TORVANE

● UNCONFINED COMPRESSION

▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL



ELEVATION (ft) · DEPTH (ft) · SYMBOL

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 10/15/19 08:42 - P:\DRAFTING\GINT FILES\510-29-2 95 S ALMADEN.GPJ

CORNERSTONE EARTH GROUP

BORING NUMBER EB-4

PAGE 1 OF 3

PROJECT NAME 95 South Almaden Boulevard
PROJECT NUMBER 510-29-2
PROJECT LOCATION San Jose, CA
DATE STARTED 9/28/19 **DATE COMPLETED** 9/28/19
GROUND ELEVATION **BORING DEPTH** 70 ft.
DRILLING CONTRACTOR Geoservices Exploration Inc.
LATITUDE 37.333226° **LONGITUDE** -121.893484°
DRILLING METHOD Mobile B-61, 8 inch Hollow-Stem Auger
GROUNDWATER LEVELS:
LOGGED BY BCG
∇ **AT TIME OF DRILLING** 20 ft.
NOTES
▼ **AT END OF DRILLING** 50 ft.

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

ELEVATION (ft)	DEPTH (ft)	SYMBOL	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		3 inches asphalt concrete over 3 inches aggregate base							
			Sandy Lean Clay (CL) [Fill] stiff, moist, dark brown, fine sand, some fine subangular to subrounded gravel, low plasticity		GB-1		15			
			Lean Clay with Sand (CL) stiff, moist, brown to light brown, fine sand, low plasticity Liquid Limit = 26, Plastic Limit = 18 See sieve analysis results.		GB-2		14	8	78	
	5			24	MC-3B	99	21			○ 3.5
			Sandy Lean Clay (CL) very stiff, moist, brown, fine sand, low plasticity							
			Silty Sand (SM) medium dense, moist, brown to light brown, fine sand	35	MC-4B	93	26	6	80	○ 1.5
	10		**Silt with Sand (ML)** stiff, moist, gray with brown mottles, fine sand, low plasticity Liquid Limit = 28, Plastic Limit = 22 See sieve analysis results.							
			Fat Clay (CH) very stiff, moist, dark gray, some fine sand, some roots, high plasticity	38	MC-5B	73	45			○ 2.5
	15									
				26	MC-6B	102	23			○ 2.5
	∇ 20		**Clayey Sand (SC)** medium dense, moist, gray brown, fine to medium sand							
			Silty Sand (SM) medium dense, moist, brown, fine to medium sand	52	MC-7B	106	22			
	25									

Continued Next Page

BORING NUMBER EB-4

PAGE 2 OF 3

CORNERSTONE EARTH GROUP

PROJECT NAME 95 South Almaden Boulevard

PROJECT NUMBER 510-29-2

PROJECT LOCATION San Jose, CA

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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)	SYMBOL	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
			Silty Sand (SM) medium dense, moist, brown, fine to medium sand							
			See sieve analysis results.	32	MC-8B	104	22		19	
	30									
			Lean Clay (CL) stiff, moist, brown and gray, some fine sand, moderate plasticity							
				38	MC-9B	92	30			○
	35									
			Lean Clay with Sand (CL) stiff, moist, brown and gray, fine sand, low to moderate plasticity							
				41	MC-10B	98	23			○
	40									
					ST-11	103 103	24 22			▲
			Silty Sand (SM) medium dense, moist, gray, fine to medium sand, some fine subangular gravel							
			Poorly Graded Sand with Silt and Gravel (SP-SM) very dense, moist, gray, fine to medium sand, fine subangular to subrounded gravel	72	SPT-12		10		9	
	45									
			Poorly Graded Gravel with Sand (GP) very dense, wet, brown, fine to coarse subangular to subrounded gravel, fine to medium sand	62	SPT-13		7			
	▼ 50									
			Poorly Graded Sand with Silt (SP-SM) very dense, moist, brown, fine to medium sand	64	SPT-14A		17			○
	55									

Continued Next Page

BORING NUMBER EB-4

PAGE 3 OF 3

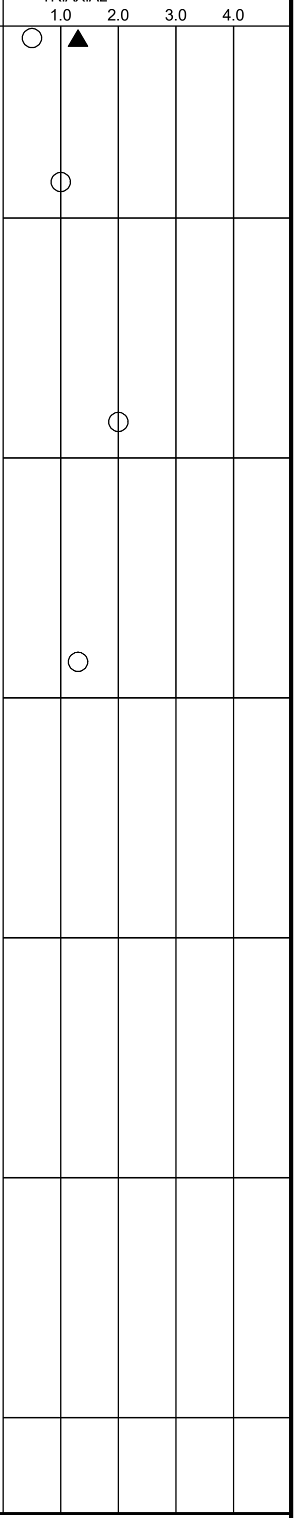
CORNERSTONE EARTH GROUP

PROJECT NAME 95 South Almaden Boulevard
PROJECT NUMBER 510-29-2
PROJECT LOCATION San Jose, CA

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.

ELEVATION (ft)	DEPTH (ft)	SYMBOL	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
		▨	**Lean Clay (CL)** stiff, moist, brown, some fine sand, moderate plasticity		ST-15	97	27		
						98	26		
		▨	**Sandy Lean Clay (CL)** medium stiff, moist, gray, fine sand, low plasticity	62	MC-16B	100	26		
	60								
		▨	**Lean Clay (CL)** stiff, moist, gray, some fine sand, moderate plasticity	55	MC-17B	97	27		
	65								
				38	MC-18B	105	22		
	70		Bottom of Boring at 70.0 feet.						
	75								
	80								
	85								

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



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 50 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 40 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.

Grain Size Analyses: The particle size distribution (ASTM D422) was determined on 10 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.

Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on one 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: Ten Plasticity Index determinations (ASTM D4318) were performed on samples of the subsurface soils 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 these tests are shown on the boring logs at the appropriate sample depths.

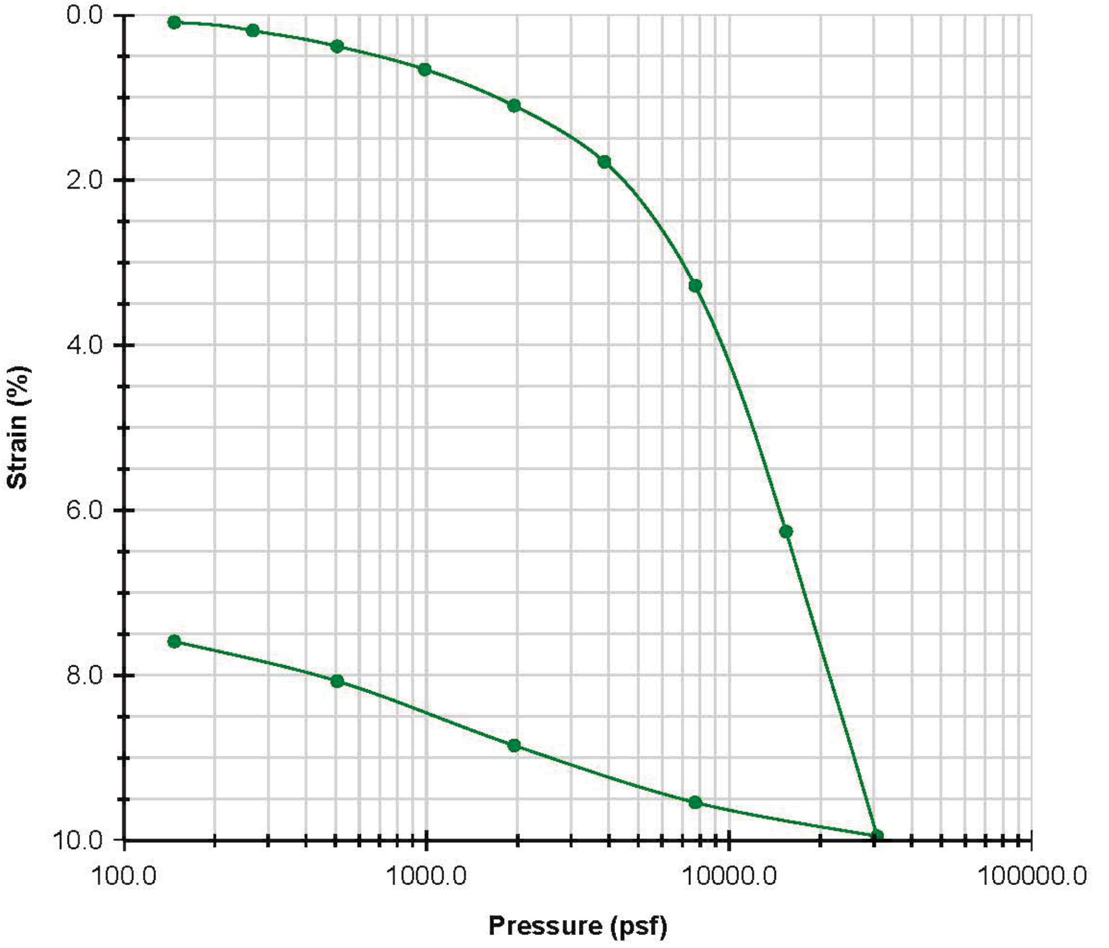
Undrained-Unconsolidated Triaxial Shear Strength: The undrained shear strength was determined on two relatively undisturbed sample(s) by unconsolidated-undrained triaxial shear strength testing (ASTM D2850). The results of this test are included as part of this appendix.

Consolidation: Two consolidation tests (ASTM D2435) were performed on relatively undisturbed samples of the subsurface clayey soils to assist in evaluating the compressibility property of this soil. Results of the consolidation tests are presented graphically in this appendix.

Consolidation Test ASTM D2435

Boring: EB-4 **Sample:** 11 **Depth:** 41.5’

Description: Lean Clay with Sand (CL)



	BEFORE	AFTER
Moisture (%)	21.7	18.9
Dry Density (pcf)	103.3	112.1
Saturation (%)	91.8	100.0
Void Ratio	0.64	0.51

(A) Stress Strain Curve

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Strain-Log Curve - EB-4 @ 41.5’

95 South Almaden Avenue
San Jose, CA

Project Number 510-29-2

Figure Number Figure B2

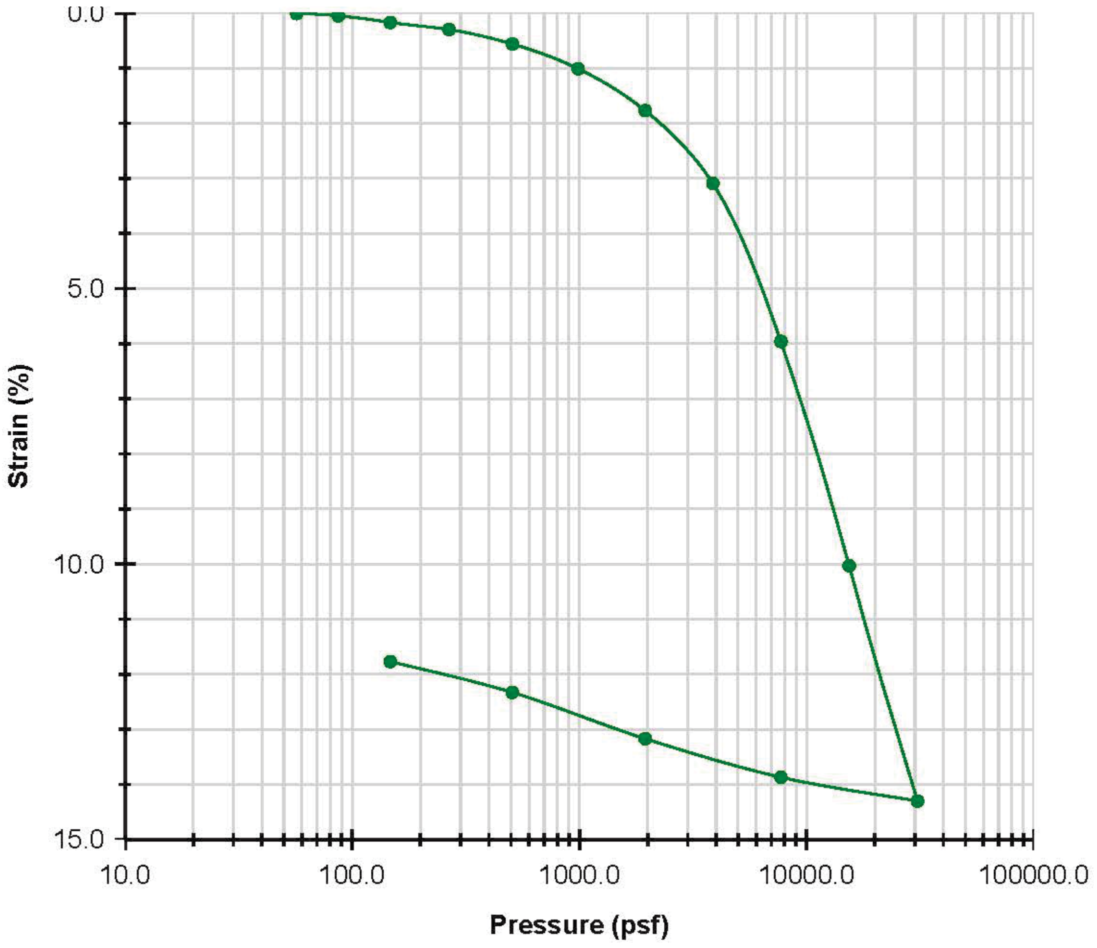
Date October 2019

Drawn By FLL

Consolidation Test ASTM D2435

Boring: EB-4 **Sample:** 15 **Depth:** 57.0’

Description: Lean Clay (CL)



	BEFORE	AFTER
Moisture (%)	26.0	20.2
Dry Density (pcf)	97.7	109.6
Saturation (%)	95.8	100.0
Void Ratio	0.74	0.55

—●— (A) Stress Strain Curve

CORNERSTONE
EARTH GROUP

Strain-Log Curve - EB-4 @ 57.0’

95 South Almaden Avenue
San Jose, CA

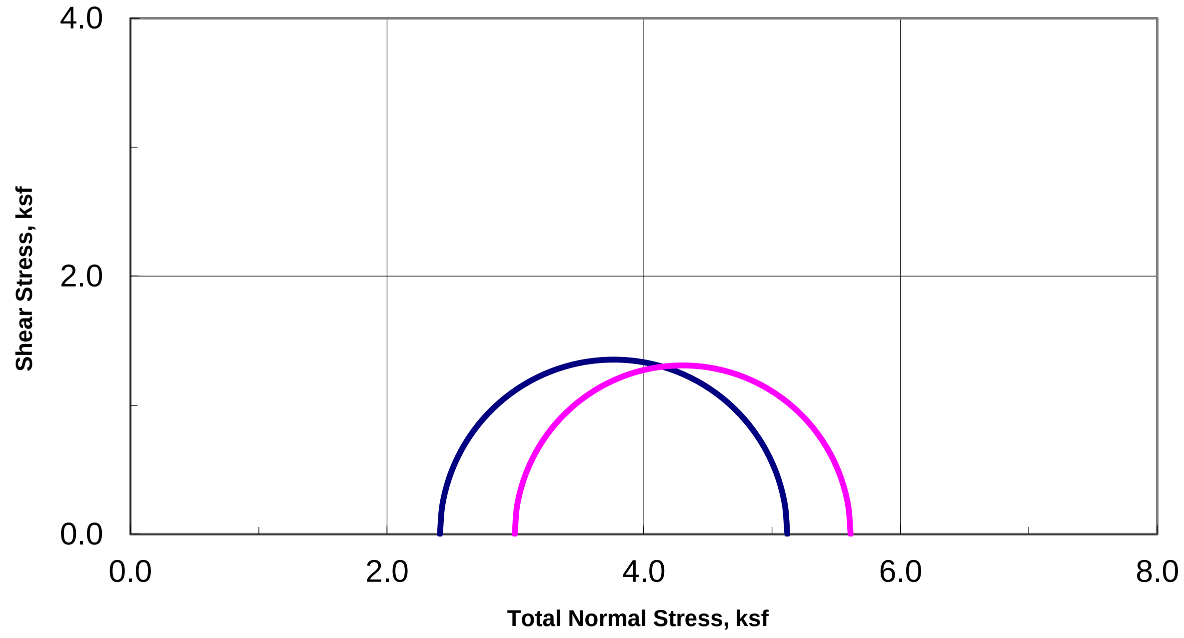
Project Number	510-29-2
Figure Number	Figure B3
Date	October 2019
Drawn By	FLL

Cooper Testing Labs, Inc.
937 Commercial Street
Palo Alto, CA 94303

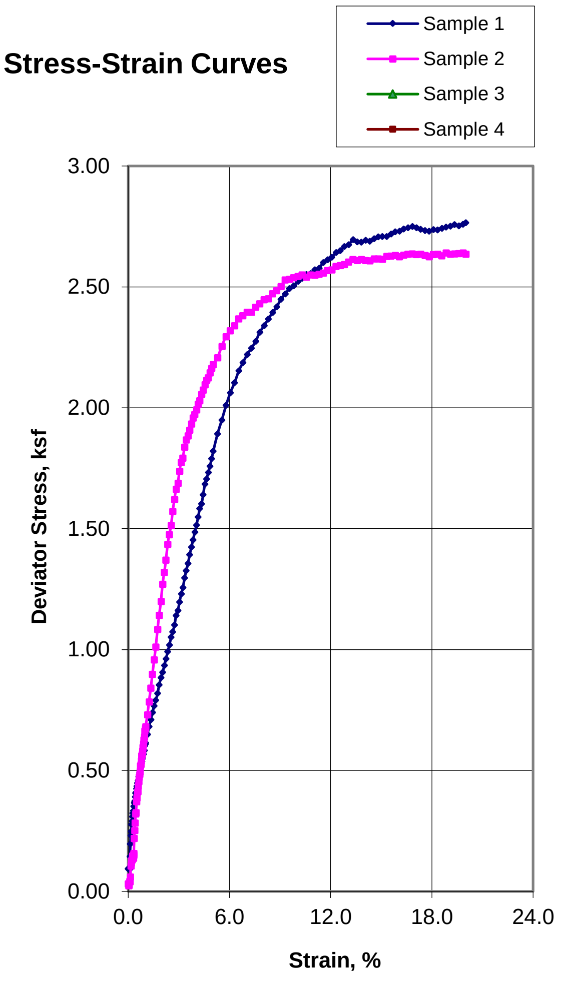
COOPER
TESTING LABORATORY

Unconsolidated-Undrained Triaxial Test

ASTM D2850



Stress-Strain Curves



Sample Data				
	1	2	3	4
Moisture %	23.5	27.1		
Dry Den,pcf	103.0	97.1		
Void Ratio	0.636	0.737		
Saturation %	99.6	99.5		
Height in	5.92	5.92		
Diameter in	2.87	2.87		
Cell psi	16.7	20.8		
Strain %	15.00	15.00		
Deviator, ksf	2.707	2.617		
Rate %/min	1.00	1.00		
in/min	0.059	0.059		
Job No.:	**640-1356**			
Client:	**Cornerstone Earth Group**			
Project:	**510-29-2**			
Boring:	EB-4	EB-4		
Sample:	11	15		
Depth ft:	40.0	55		

Visual Soil Description	
Sample #	
1	Gray Sandy CLAY
2	Gray CLAY
3	
4	

Remarks:

Note: Strengths are picked at the peak deviator stress or 15% strain which ever occurs first per ASTM D2850.