

Appendix D
Geotechnical Report



Type of Services	Preliminary Geotechnical Investigation and Geologic Hazards Evaluation
Project Name	Piercy Road Industrial
Location	550 Piercy Road San Jose, California
Client	Hines
Client Address	101 California Street, Suite 1000 San Francisco, California
Project Number	1076-4-2
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Type of Services	Preliminary Geotechnical Investigation and Geologic Hazards Evaluation
Project Name	Piercy Road Industrial
Location	550 Piercy Road San Jose, California

SECTION 1: INTRODUCTION

This preliminary geotechnical investigation and geologic hazards evaluation was prepared for the sole use of Hines for the Piercy Road Industrial 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. For our use, we have reviewed several Conceptual Site Plans prepared by Ware Malcomb.

1.1 PROJECT DESCRIPTION

Currently, the project will include redeveloping the approximately 29½-acre site for a new industrial complex. We understand the current plan will for the new complex will include two buildings totaling about 164,970 to 242,730 square feet each. Exact building layouts are currently being evaluated. However, we understand the building layouts will consider the existing fault setbacks. The buildings will have 36 feet clear height. Dock high and grade level doors will also be planned but the locations are still being finalized. Detention basins are currently planned along the north and south sides of the site. Buildings will consist of concrete tilt-up construction.

Building loads are expected to be typical of this type of construction. Site grading may include significant cuts and fills based on the final design.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated September 14, 2021 and consisted of limited field and laboratory programs to provide a preliminary evaluation of the geologic and geotechnical hazards and constraints for the site. Brief descriptions of our exploration and laboratory programs are presented below.

1.3 EXPLORATION PROGRAM

Field exploration consisted of two borings drilled on September 27, 2021 with track-mounted, limited-access hollow-stem auger drilling equipment. The borings were drilled to depths ranging from 25 to 40 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 preliminary data for future foundation design and potential geotechnical constraints. 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 ENVIRONMENTAL SERVICES

Cornerstone Earth Group also provided environmental services for this project, including Phase 1 site assessments; 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. Please refer to Appendix C for additional geologic information provide in the Phase 1 Preliminary Geologic Consultation Letter.

2.2 REGIONAL SEISMICITY

The San Francisco Bay area 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 2007 estimates 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. As seen with damage in San Francisco and Oakland due to the 1989 Loma Prieta earthquake that was centered about 50 miles south of San Francisco, significant damage can occur at considerable distances. Higher levels of shaking and damage would be expected for earthquakes occurring at closer distances.

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. Please refer to Appendix C for additional regional seismicity discussion.

Table 1: Approximate Fault Distances

Fault Name	Distance	
	(miles)	(kilometers)
Hayward (Southeast Extension)	3.2	5.1
Monte Vista-Shannon	4.5	7.2
Calaveras	5.3	8.5
Sargent	10.6	17.1
San Andreas (1906)	12.2	19.7
Hayward (Total Length)	13.2	21.2

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

At the time of our explorations, the site consisted of an open field covered in low grasses and one to four foot tall weeds. Gravel access roads surround the perimeter of the site along with two access roads that cross through the site. Based on elevations shown on the topographic plan, the site generally slopes from north to south at approximately Elevation 295 feet in the northern corner to approximately 220 feet along the southern side of the site adjacent to Hellyer Road. The site is bounded by Piercy Road to the north, undeveloped lots to the east, Hellyer Avenue to the south, and industrial developments to the west.

We reviewed historic aerial photographs provided on www.historicaerials.com dating back to 1948. The site vicinity was generally used as an orchard from 1948 to 1968. Between the 1968 and 1980, the site appears to have been developed for agriculture purposed. The site was used for agriculture purposes until 1987, and then the site appears relatively unchanged since the 1987 photo.

3.2 SUBSURFACE CONDITIONS

Below the ground surface, our explorations generally encountered 4½ to 14½ feet of undocumented fill consisting of hard sandy fat clay and very stiff to hard sandy lean clay. Beneath the undocumented fill, our exploratory boring EB-2 generally encountered very stiff to hard fat clay with varying amounts of sand to depths of about 9 to 11 feet. Beneath the fat clay, Boring EB-2 encountered hard sandy lean clay to a depth of about 16 feet underlain by dense clayey sand to a depth of 28 feet and hard elastic silt to the terminal boring depth of 40 feet. Beneath the undocumented fill, Boring EB-3 encountered dense silty sand to the terminal boring depth of 25 feet.

3.2.1 Plasticity/Expansion Potential

We performed two Plasticity Index (PI) tests on representative samples. Test results were used to evaluate expansion potential of surficial soils. The surficial PI tests resulted in PIs ranging from 26 to 39, indicating moderate 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 10 feet range from about 2 percent below to about 18 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. As predominantly clays were encountered, the borings were not left open long enough for water to seep into the boring holes. The CGS East San Jose Seismic Hazard Evaluation report (2000) does not show historic groundwater levels in the immediate area around the site. Based on our experience, we judge depth to free groundwater to be on the order of 30 to 40 feet below the ground surface with some localized shallower perched water conditions. 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, but is mapped in a Santa Clara County Fault Hazard Zone. Additional information on fault rupture hazard is presented in Appendix C of this report.

4.1.2 Geophysical Fault Study

A geophysical study spanning the length of the site trending from north to south was performed by JR Associates. The study included two seismic refraction surveys and two shear wave profiling surveys. Additional discussion about the results of this study is included in geologic hazards study, Appendix C, of this report. Additional information about this study and results are included in Appendix D of this report.

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_M) was estimated for analysis using a value equal to $F_{PGA} \times PGA$, as allowed in the 2019 edition of the California Building Code when an exception has been taken per ASCE 7-16, Section 11.4.8. For our preliminary liquefaction analysis we used a PGA_M of 0.794g.

4.3 LIQUEFACTION POTENTIAL

The site is located within a State-designated Liquefaction Hazard Zone (CGS, San Jose East Quadrangle, 2003) and a Santa Clara County Liquefaction Hazard Zone (Santa Clara County, 2003). However, a laterally extensive groundwater table has not been identified in compilations (CGS, 2000), groundwater was not encountered in our exploratory borings, and groundwater conditions are likely quite variable near the east hills. Our preliminary geotechnical investigation does not include a full liquefaction analysis or evaluation of liquefaction potential; however, we screened the site for liquefaction during our preliminary 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 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 discussed in the "Subsurface" section above, we primarily encountered very stiff to hard clays and elastic silts and dense to very dense sands. Based on the above, our preliminary screening of the site for liquefaction indicates a low potential for liquefaction. Our preliminary screening of the site will be verified with additional exploration and analysis during a final design-level geotechnical investigation.

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.

On a preliminary basis, the potential for liquefaction is considered low; therefore, on a preliminary basis, the potential for lateral spreading to affect the site is low. However, the Coyote Creek channel runs to the south of the site; therefore, we recommend the potential for lateral spreading be further evaluated during a final design-level geotechnical investigation.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose unsaturated sandy soils can settle during strong seismic shaking. As the soils encountered at the site were predominantly very stiff to hard clays and elastic silts and dense to very dense sands, in our opinion, the potential for significant differential seismic settlement affecting the proposed improvements is low. We recommend the potential for seismic settlement of unsaturated sands be verified with additional exploration and analysis during the final design-level geotechnical investigation.

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 20 miles

inland from the San Francisco Bay shoreline, and is approximately 220 to 290 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.

- Undocumented fill
- Presence of moderately to highly expansive soils
- Differing foundation support conditions (cut and fill transition)
- Naturally occurring asbestos (NOA)

5.1.1 Undocumented Fill

As discussed above, we encountered up to about 14½ feet of undocumented fill across the site. The exact fill quality, placement, and compaction is not known at this time. However, even well compacted fill can still settle due to the weight of new fill and building loads. If determined feasible and structures are supported on shallow foundations, we recommend the fill be removed from within the building footprints and replaced as engineered fill. Undocumented fill should be further evaluated as a part of the design-level geotechnical investigation.

5.1.2 Presence of Moderately to Highly Expansive Soils

Moderately to highly expansive surficial soils were encountered in our borings across the site and appear to generally blanket the site. Expansive soils can undergo significant volume change with changes in moisture content. They shrink and harden when dried and expand and soften when wetted. To reduce the potential for damage to the planned structures, slabs-on-grade should have sufficient reinforcement and be supported on a layer of non-expansive fill; footings should extend below the zone of seasonal moisture fluctuation. In addition, it is

important to limit moisture changes in the surficial soils by using positive drainage away from buildings as well as limiting landscaping watering.

5.1.3 Differing Foundation Support Conditions (Cut and Fill Transition)

Based on site topography and planned finished elevations, we anticipate some deeper cut and fill pads will be required. In addition, shallow bedrock may also be present closer along Piercy Road and within the western corner of the site next to Hellyer Avenue.

Foundation bearing on different material (soil/rock or cut/fill) can settle unevenly. This may result in increased differential settlement between the different materials. On a preliminary basis, we recommend finished pads with differing foundation support conditions be over-excavated about 3 to 5 feet and replaced with engineered fill to provide a more uniform support of foundations. These estimates should be further evaluated during the design-level geotechnical investigation and once final grading plans are available.

5.1.4 Naturally Occurring Asbestos (NOA)

Asbestos is commonly used to refer to several types of naturally occurring fibrous minerals that can be found in serpentinite rock. Although not encountered in our borings, serpentinite is mapped north of Piercy Road and could potentially outcrop or be encountered below surficial soils along the northern edge of the site. During site grading and if rock containing asbestos is exposed, special precautions will be necessary. Earthwork and excavation contractors should be made aware of this potential concern and measures be implemented, as required by governing agencies. Typical construction and grading activities will require a Naturally-Occurring Asbestos Dust Mitigation Plan (ADMP).

The presence of NOA and associated concerns should be further evaluated during the design-level geotechnical investigation.

5.2 DESIGN-LEVEL GEOTECHNICAL INVESTIGATION

The preliminary recommendations contained in this preliminary report were based on limited site development information, 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, 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 any existing foundations, debris, slabs, and/or abandoned underground utilities be removed entirely and the resulting excavations backfilled with engineered fill. Additionally, any native soils that are disturbed during demolition of the existing improvement should also be removed and replaced as engineered fill.

As discussed above, we encountered up to about 14½ feet of undocumented fill in our borings. For preliminary planning purposes, we recommend all undocumented fill be completely removed from within building areas and to a lateral distance of at least 5 feet beyond the building footprint. We anticipate the fills may be reused when backfilling the excavations. The actual lateral extent and depth of undocumented fill should be confirmed during the design-level investigation.

Surface water runoff should not be allowed to pond adjacent to the 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. Bio-treatment basins should be kept at least 10 feet away from buildings and, where possible, at least 3 feet away from pavements and flatwork.

SECTION 7: FOUNDATIONS

7.1 SUMMARY OF RECOMMENDATIONS

On a preliminary basis, provided the above potential concerns are evaluated as part of the design-level geotechnical investigations, the proposed structures can likely be supported on conventional continuous and/or isolated spread footings bearing entirely on natural, undisturbed soil, or engineered fill.

7.2 SEISMIC DESIGN CRITERIA

We understand that the project structural design may be based on the 2019 California Building Code (CBC), which provides criteria for the seismic design of buildings in Chapter 16. If a new Building Code comes into effect, these recommendations should be reviewed and updated. 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 alluvial soils with typical SPT "N" values between 15 and 50 blows per foot underlain by bedrock at varying depths. On a preliminary basis and based on our explorations to date, we have classified the site as Soil Classification D. Because we used site specific data from our explorations and laboratory testing, the site class should be considered as "determined" for the purposes of estimating the seismic design parameters from the code outlined below. The mapped spectral acceleration parameters S_s and S_1 were calculated using

the web-based program ATC Hazards by Locations, located at <https://hazards.atcouncil.org/>, based on the site coordinates presented below and the site classification. Recommended values in Table 3 may only be used for design if in the judgement of the project structural engineer an exception can be taken per ASCE 7-16 Section 11.4.8. If an exception is not taken, we should be notified so that we can perform a Ground Motion Hazard Analysis during the design-level geotechnical investigation. The table below lists the various factors used to determine the seismic coefficients and other parameters. We recommend the preliminary site classification be further evaluated and verified during the design-level geotechnical investigation.

Table 2: Preliminary CBC Site Categorization and Site Coefficients

Classification/Coefficient	Design Value
Site Class	D
Site Latitude	37.25336°
Site Longitude	-121.77229°
0.2-second Period Mapped Spectral Acceleration ¹ , S_s	1.716g
1-second Period Mapped Spectral Acceleration ¹ , S_1	0.650g
Short-Period Site Coefficient – F_a	1.0
Long-Period Site Coefficient – F_v	1.7
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{MS}	1.716g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	1.105g
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.144g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	0.737g

¹For Site Class B, 5 percent damped.

7.3 SHALLOW FOUNDATIONS

7.3.1 Conventional Shallow Footings

On a preliminary basis, the planned structures may be supported on conventional shallow footings. On a preliminary basis, footings should bear entirely on natural, undisturbed soil, or engineered fill, and extend at least 24 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. The deeper footing embedment is recommended due to the presence of moderately to highly expansive soils encountered in our preliminary explorations, and is intended to embed the footing below the zone of significant seasonal moisture fluctuation, reducing the potential for differential movement.

For planning purposes, maximum allowable bearing pressures of 3,000 psf for combined dead plus live loads may be feasible.

SECTION 8: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

8.1 WAREHOUSE SLABS-ON-GRADE

Warehouse slabs-on-grade should be at least 6 inches thick, should have a minimum compressive strength of 3,500 psi, and should be designed for the specific warehouse loading (i.e. Forklifts, rack loads, etc.).

The warehouse slab-on-grade should be supported by the materials and alternatives outlined below.

- Zone A Fill Material – Upper 6 inches of pad subgrade should consist of crushed granular base having an R-value of at least 50 and no more than 10 percent passing the No. 200 sieve, such as Class 2 aggregate base or subbase.
- Zone B Fill Material – from 6 to 24 inches below pad subgrade (or below the Zone A noted above), the soil/material should also consist of a non-expansive fill (NEF) with a Plasticity Index (PI) of 15, or less.
- Zone B Fill Material Alternative 1 – from 6 to 24 inches below pad subgrade (or below the Zone A noted above), the non-expansive fill material also can consist of the same material used for Zone A
- Zone B Fill Material Alternative 2 – from 6 to 24 inches below pad subgrade (or below the Zone A noted above), the onsite material also can be chemically treated to meet non-expansive requirement with a PI of 15, or less

SECTION 9: VEHICULAR PAVEMENTS

9.1 ASPHALT CONCRETE

Based on the clayey nature of the surficial soils, we have estimated an R-value of 5 for pavement design. Table 3 and 4 presents preliminary asphalt concrete pavement recommendations for a range of Traffic Indices for untreated and lime-treated subgrade.

Table 3: Asphalt Concrete Pavement Recommendations (Untreated Subgrade)

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base* (inches)	Total Pavement Section Thickness (inches)
4.0	2.5	7.5	9.5
4.5	2.5	8.5	11.0
5.0	3.0	9.0	12.0
5.5	3.0	10.5	13.5
6.0	3.5	11.5	15.0
6.5	4.0	12.5	16.5
7.0	4.0	14.0	18.0
7.5	4.5	15.5	20.0
8.0	5.0	17.0	21.5

* Caltrans Class 2 aggregate base or recycled crushed concrete with a minimum R-value of 78.

Table 4: Asphalt Concrete Pavement Recommendations (Lime-Treated Subgrade)

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base* (inches)	Total Pavement Section Thickness (inches)
4.0	2.5	4.0	6.5
4.5	2.5	4.0	6.5
5.0	3.0	4.5	7.5
5.5	3.0	4.5	7.5
6.0	3.5	4.5	8.0
6.5	3.5	5.0	8.5
7.0	4.0	5.0	9.0
7.5	4.5	5.5	10.0
8.0	4.5	6.5	11.0

* Caltrans Class 2 aggregate base or recycled crushed concrete with a minimum R-value of 78; minimum cement-treated subgrade R-value assumed to be 50

The AC section alternatives in Table 4 is based on a minimum designed R-value of 50 and treatment depth of 12 inches.

9.2 PORTLAND CEMENT CONCRETE

The exterior Portland Cement Concrete (PCC) pavement recommendations tabulated below are based on methods presented in "Concrete Pavement for Trucking Facilities" by the American

Concrete Pavement Association (ACPA, 1995), and the Portland Cement Association (PCA) design manual (PCA, 1984).

Table 5: PCC Pavement Recommendations (Untreated Subgrade)

Traffic Category	Minimum PCC Thickness¹ (inches)
Trucks per day per lane = 20	7.0
Trucks per day per lane = 80	7.5
Trucks per day per lane = 400	8.0

Note: 1 – Subgrade design R-Value = 5

Chemically treated (e.g. lime-treated) subgrade alternatives are provided in Table 6. For chemically treated subgrade, we recommend the upper 12 inches of subgrade soil be treated and the treated subgrade should obtain a minimum R-value of 50. If chemical treatment of the subgrade is performed, the underlying Class 2 aggregate base section may be reduced from 6 inches to 4 inches.

Table 6: PCC Pavement Recommendations (Chemical-Treated Subgrade)

Traffic Category	Minimum PCC Thickness¹ (inches)
Trucks per day per lane = 20	6.5
Trucks per day per lane = 80	7.0
Trucks per day per lane = 400	7.5

Note: 1 – Subgrade design R-Value = 50, minimum 12 inches of chemically treated subgrade

SECTION 10: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of Hines specifically to support the design of the Piercy Road Industrial 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.

Hines may have provided Cornerstone with plans, reports and other documents prepared by others. Hines 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.

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Vicinity Map

**550 Piercy Road Industrial
San Jose, CA**

Project Number

1076-4-2

Figure Number

Figure 1

Date

May 2022

Drawn By

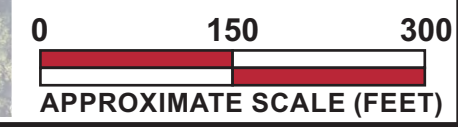
RRN



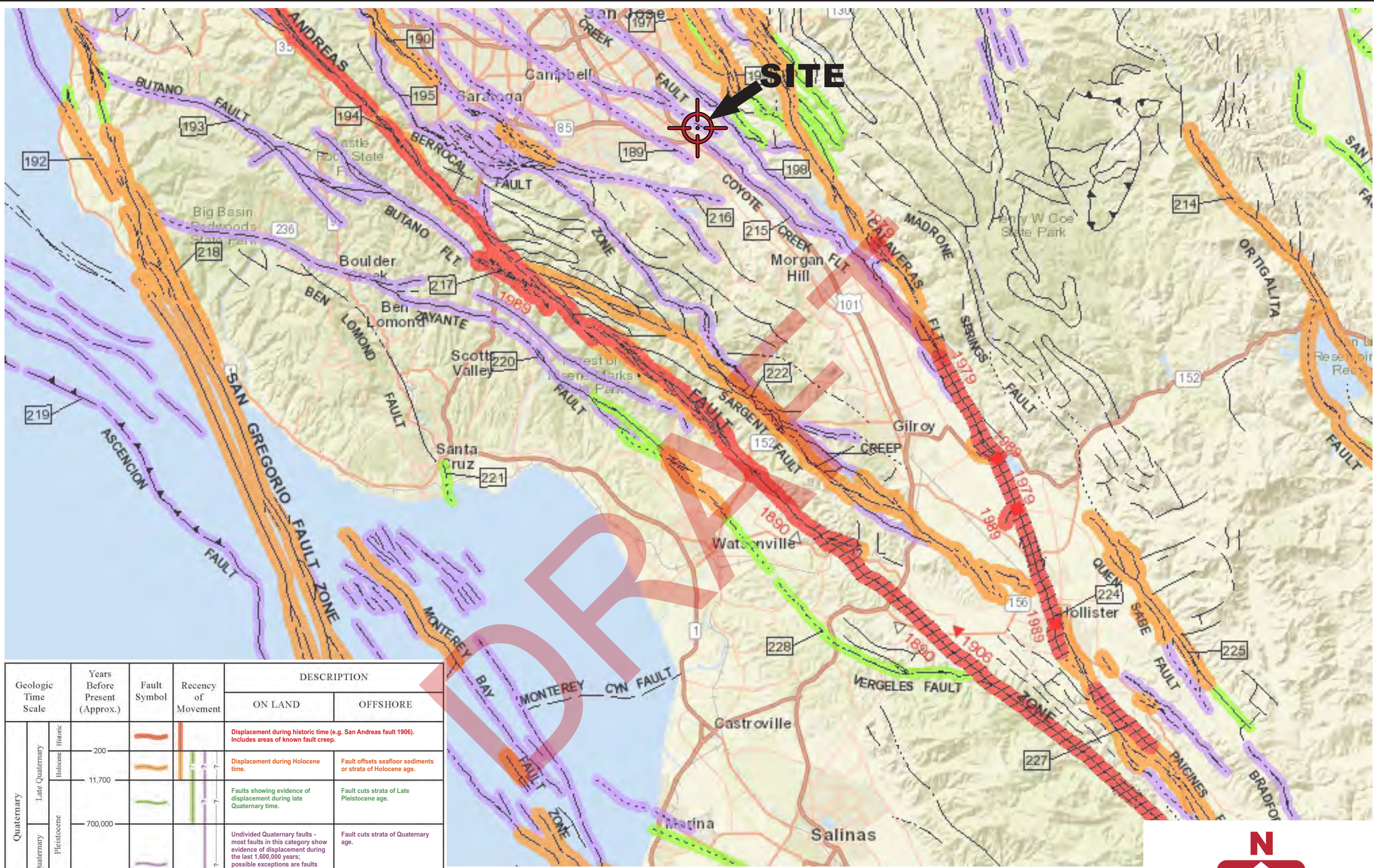
Legend
 Approximate location of exploratory boring (EB)



Site Plan
 550 Piercy Road Industrial
 San Jose, CA

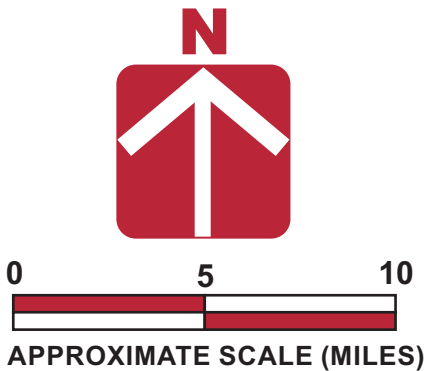


Project Number		1076-4-2
Figure Number		Figure 2
Date	May 2022	Drawn By
		RRN, FLL



Geologic Time Scale		Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION	
					ON LAND	OFFSHORE
Quaternary	Late Quaternary	Holocene			Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		200			Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
	11,700			Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.	
	Pleistocene	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 Plio-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 Number1076-4-2

Figure NumberFigure 3

DateMay 2022

Drawn ByRRN, FLL

Regional Fault Map

550 Piercy Road Industrial
San Jose, CA

CORNERSTONE
EARTH GROUP

APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using track-mounted, hollow-stem auger drilling equipment. Two 6½-inch-diameter exploratory borings were drilled on September 27, 2021 to depths of 25 to 40 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. 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-2487-98)

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

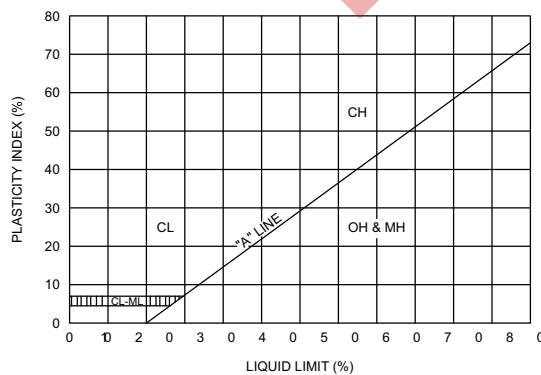
SAMPLER TYPES

	SPT		Shelby Tube
	Modified California (2.5" I.D.)		No Recovery
	Rock Core		Grab Sample

ADDITIONAL TESTS

CA - CHEMICAL ANALYSIS (CORROSIVITY)	PI - PLASTICITY INDEX
CD - CONSOLIDATED DRAINED TRIAXIAL	SW - SWELL TEST
CN - CONSOLIDATION	TC - CYCLIC TRIAXIAL
CU - CONSOLIDATED UNDRAINED TRIAXIAL	TV - TORVANE SHEAR
DS - DIRECT SHEAR	UC - UNCONFINED COMPRESSION
PP - POCKET PENETROMETER (TSF)	(1.5) - (WITH SHEAR STRENGTH IN KSF)
(3.0) - (WITH SHEAR STRENGTH IN KSF)	
RV - R-VALUE	UU - UNCONSOLIDATED UNDRAINED TRIAXIAL
SA - SIEVE ANALYSIS: % PASSING #200 SIEVE	
- WATER LEVEL	

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

BORING NUMBER EB-2

PAGE 1 OF 2

DATE STARTED 9/27/21 DATE COMPLETED 9/27/21

DRILLING CONTRACTOR Cuesta Geo

DRILLING METHOD MPP LAD Track Rig, 6½ inch Hollow-Stem Auger

LOGGED BY EA

NOTES

PROJECT NAME 550 Piercy Road

PROJECT NUMBER 1076-4-2

PROJECT LOCATION San Jose, CA

GROUND ELEVATION BORING DEPTH 40 ft.

LATITUDE 37.25397° LONGITUDE -122.77503°

GROUND WATER LEVELS:

▽ AT TIME OF DRILLING Not Encountered

▼ 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.

ELEVATION (ft)

DEPTH (ft)

SYMBOL

DESCRIPTION**Sandy Fat Clay (CH) [Fill]**

hard, moist, dark brown, fine sand, some fine gravel, high plasticity
Liquid Limit = 59, Plastic Limit = 20

Fat Clay with Sand (CH)

hard, moist, brown, fine sand, high plasticity

Sandy Lean Clay (CL)

hard, moist, brown with light brown mottles, fine sand, moderate plasticity

Clayey Sand (SC)

dense, moist, brown, fine to medium sand

N-Value (uncorrected)
blows per footSAMPLES
TYPE AND NUMBERDRY UNIT WEIGHT
PCFNATURAL
MOISTURE CONTENT

PLASTICITY INDEX, %

PERCENT PASSING
No. 200 SIEVEUNDRAINED SHEAR STRENGTH,
ksf

○ HAND PENETROMETER

△ TORVANE

● UNCONFINED COMPRESSION

▲ UNCONSOLIDATED-UNDRAINED
TRIAXIAL

1.0 2.0 3.0 4.0

>4.5

>4.5

>4.5

>4.5

>4.5

Continued Next Page



PROJECT NAME 550 Piercy Road

PROJECT NUMBER 1076-4-2

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

Clayey Sand (SC)
dense, moist, brown, fine to medium sand

Elastic Silt (MH)
hard, moist, brown, fine to medium sand, high plasticity

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

UNDRAINED SHEAR STRENGTH,
ksf

○ HAND PENETROMETER

△ TORVANE

● UNCONFINED COMPRESSION

▲ UNCONSOLIDATED-UNDRAINED
TRIAXIAL

1.0 2.0 3.0 4.0

33

SPT-8

49

>4.5

50

MC-9B

73

50

>4.5

6"

16

SPT-10

60

>4.5



CORNERSTONE EARTH GROUP

BORING NUMBER EB-3

PAGE 1 OF 1

DATE STARTED 9/27/21 DATE COMPLETED 9/27/21

DRILLING CONTRACTOR Cuesta Geo

DRILLING METHOD MPP LAD Track Rig, 6½ inch Hollow-Stem Auger

LOGGED BY EA

NOTES

PROJECT NAME 550 Piercy Road

PROJECT NUMBER 1076-4-2

PROJECT LOCATION San Jose, CA

GROUND ELEVATION BORING DEPTH 25 ft.

LATITUDE 37.25436° LONGITUDE -122.77154°

GROUND WATER LEVELS:

▽ AT TIME OF DRILLING Not Encountered

▼ 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.

ELEVATION (ft)

DEPTH (ft)

SYMBOL

DESCRIPTION

Sandy Lean Clay (CL) [Fill]

hard, moist, dark brown and brown mottled, fine to medium sand, some fine to coarse subangular gravel, moderate plasticity
Liquid Limit = 45, Plastic Limit = 19

some cobbles

becomes very stiff

Silty Sand (SM)

dense, moist, brown, fine to medium sand

Bottom of Boring at 25.0 feet.

N-Value (uncorrected)
blows per footSAMPLES
TYPE AND NUMBERDRY UNIT WEIGHT
pcfNATURAL
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

17

MC-1B

99

13

26

>4.5

30

MC-2B

14

9

MC-3B

92

15

13

MC-4B

105

17

15

MC-5B

85

17

31

SPT

32

SPT-7

12

26

35

SPT-8

6

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.

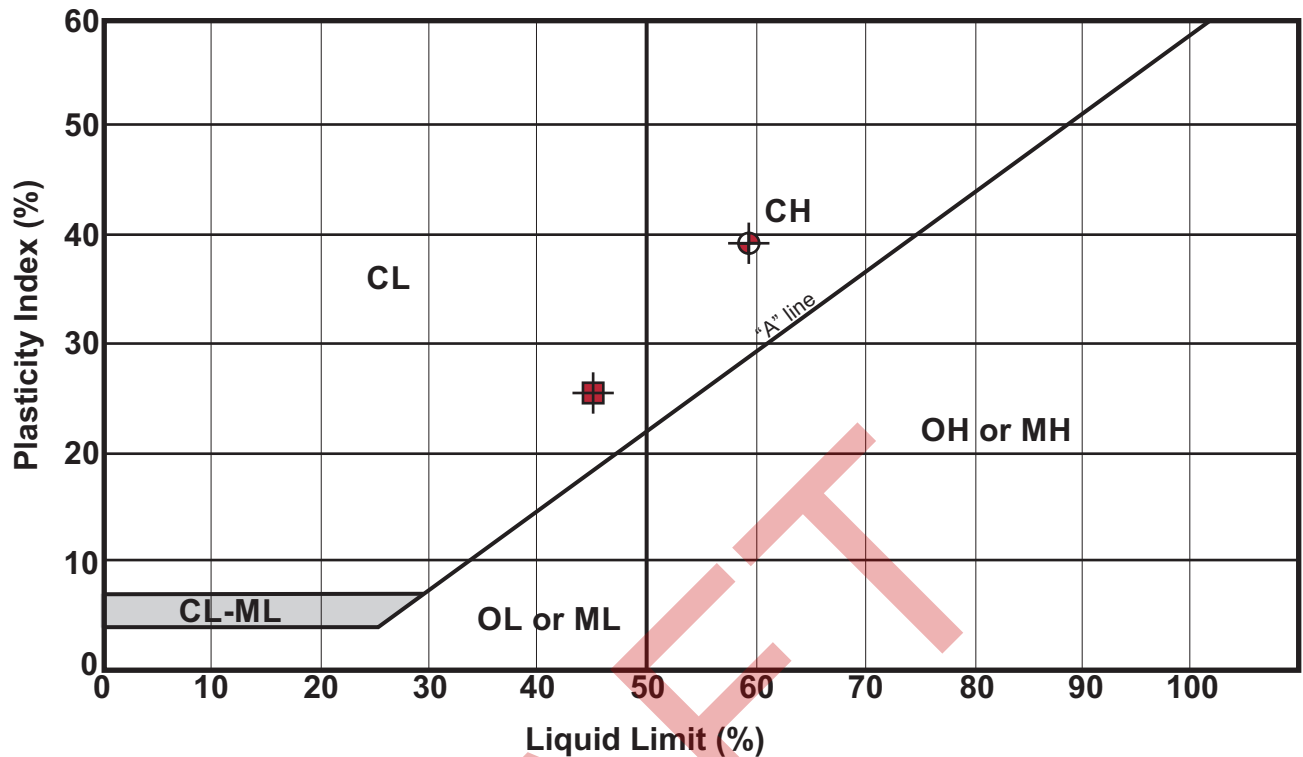
Moisture Content: The natural water content was determined (ASTM D2216) on 17 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities: In place dry density determinations (ASTM D2937) were performed on 10 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 two 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: Two 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.

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	59	20	39	—	Sandy Fat Clay (CH) [Fill]
■	EB-3	2.0	13	45	19	26	—	Sandy Lean Clay (CL) [Fill]

Samples prepared in accordance with ASTM D421

APPENDIX C: DRAFT PHASE 1 GEOHAZARD EVALUATION

WILLIAM GODWIN, C.E.G, CONSULTANTS REPORT DATED OCTOBER 23, 2021

DRAFT

Date: May 25, 2022

To: Mr. Dahn Tran, P.E.
Senior Principal Engineer
Cornerstone Earth Group

Subject: Phase I Preliminary Geologic Consultation, Piercy Road Industrial Development
550 Piercy Road, San Jose, California 95132. Cornerstone Job No. 1076-4-2

Dear Mr. Tran,

This letter presents a discussion of my initial Phase 1 assessment of geologic information concerning the project site at an undeveloped parcels, located at 550 Piercy Road, in San Jose, hereafter referred to as the project (Site). The assessment of the Site geologic setting was obtained through a desktop study of all available published literature, selected historic aerial photographs, consultants reports and some limited site exploration. The geologic setting and potential hazards of the site will form the basis of future geotechnical investigations and refinement of a recent conceptual building layout (Drawing Ware Malcolm drawing 002-S, dated 4/11/2022).

Background

The scope for future geotechnical design-level investigations and subsequent approvals by the City of San Jose will depend on the presentation and interpretation of some site specific consultant reports. Key among these is a figure dated 7/25/2004 presumably from an earlier report prepared by Environmental/Engineering Consultants (E2C) which depicts proposed fault locations and setbacks for an adjacent parcel to the southeast. I was unable to obtain a copy of the E2C report, am not aware of the date of the report and when I contacted Mr. Mike Shimamoto, geologist for the City of San Jose, he informed us the report does not reside in their files. It is possible that the report was never submitted to Mr. Shimamoto. To my knowledge there has not been a Preliminary Hazards Review Letter discussing the hazards mapping pertinent to the location of the site.

The site is located within a geologically complex area within the Edenvale area of the foothills of southeastern San Jose, as shown on Figure 1. The site is located within city-designated geologic hazard zone for fault surface rupture, primarily based on the location of the Piercy Fault, a reverse or thrust fault which trends along the base of the nearby foothills and adjacent to Piercy Road. The Site is just outside the City hazard zone for landsliding. A number of regional scale geologic maps and hazard themed maps have been published covering the general area of the site. These published maps were reviewed as part of my Phase 1 work and offer broad interpretations of the geologic setting. They are included in attached Reference list and consist of:

Geologic maps: Dibblee, OFR 72-92 (1972); Dibblee and Minch, Map DF-155 (2005); Graymer et al, OFR-95-597 (1998); Jennings and Bryant (2010); and Wentworth et al. (1999).

Published geologic hazard-themed maps and reports: Bryant, W.R, Fault Evaluation Rpt. 122 (1981a); Bryant, W.R, Fault Evaluation Rpt. 106 (1981b); Hitchcock and Brankman, NHERP study (2002); City of San Jose, Fault Hazard Maps, San Jose East Quad, (1983); CDMG, revised Alquist Priolo Map, San Jose East Quad (1982); Santa Clara County Geologic Hazards sheet 37 (2021); and California Geological Survey, San Jose East EZRIM Report 044, (2000).

A number of consultant's site-specific geotechnical and fault investigations have been conducted for sites located within a mile of the subject site, as shown on a Google Earth basemap from 2000 (Figure 1). The consultant, location and (year) of each investigation include: AES, Piercy Rd (2000); AST, 514 Piercy Rd (2016); Cornerstone Earth Group, 455 Piercy Rd (2016); Earthtech, Edenvale Redevelopment (2000); ENGEO, 715 Piercy Rd (2006); E2C Inc. 550 Piercy Rd (2000) and Enzo Dr. (2001); Kleinfelder, Levin site (1994) and PGE Stone Metcalf (2018); Romig, 469 and 471 Piercy Rd (2017-18); Terrasearch, unnamed parcel (2008); and USE, 498, 500, 502 Piercy Rd (1998).

Faults identified in specific fault investigations by these consultants are depicted in dashed, colored lines in Figure 1. The work by Earthtech (2000) for the Edenvale Area includes geologic mapping and perimeter borings for the initial proposed redevelopment area which I have adopted as a base for the Vicinity Geologic Map (Figure 2) and which provides an overview of the geologic complexity of the vicinity. Of note, landslides are not shown on the map as it focuses on the lower and flatter slopes adjacent to and southwest of Piercy Road. Landslides do occur upslope of Piercy Road but not in the immediate vicinity.

The work by E2C in 2000 (9/21 and 11/18) was specifically for 550 Piercy Road and included a significant amount of trenching (not shown for clarity). It resulted in the establishment of fault setbacks around observed faults and shears some of which offset Holocene-age sediments. As mentioned earlier the unavailable report by E2C on the adjacent parcel also established setbacks based on trenching.

Aerial Photography Review and Site Reconnaissance

An additional reference commonly used by engineering geologists is aerial photographs taken over a period of time to identify subtle features not readily seen on the ground. These features include scarps, concave surfaces, topographic expression and seepage indicative of potential landslides. Faults are commonly associated with topographic and geomorphic expression, tonal differences, lineaments and offset drainages. Findings from the aerial photo analysis are presented in the following sections of this letter report.

For this project, a total of 12 aerial photo sets (stereo-paired) were evaluated that represented aircraft flights between 1960 and 1998. Flight dates included a range of months from April to November, representative of both wet and dry terrain conditions. These photos were viewed through a stereoscope to provide a timeline of site development. In addition, Google Earth imagery more recent than 1998 were also reviewed. A summary of the reviewed photography is included as Table 1 in this letter report.

I performed a geologic site reconnaissance on October 7th 2021 which revealed similar geology and earth materials to those shown on Figure 2 although some areas have been further developed

since 2000. Older ultramafic rocks of the Franciscan Complex occur upslope of Piercy Road and are predominantly serpentinite (symbol sp on Figure 2). A large quarry was observed close to the site and near the abandoned Evergreen canal. Areas between Piercy Road and Hellyer Avenue contain younger rocks of the Santa Clara Formation (symbol QTsc) on the site and parcels on either side, in addition to a low hill along Hellyer that exposes the Santa Clara Formation in the cut slope. Alluvium (Qal) is found on the lower flanks of the slopes and is associated with meandering channel of Coyote Creek. No significant fill was observed other than along the perimeter of Piercy Road associated with grading.

Subsurface Exploration and Surface Geophysical Surveys

Figure 3 was prepared using the recent topography from Kier & Wright (2021) to show recent exploration and geology as discussed below. I reviewed logs of two exploratory borings drilled on September 27th at the Site and shown as EB-2 and EB-3 on Figure 3. Boring EB-2 was located in the northwestern corner of 550 Piercy and also had a few feet of Sandy Fat Clay fill over a sequence of Lean Clay, Clayey Sand and Elastic Silt to a depth of 40ft. Boring EB-3 was drilled on the upslope side of 550 Piercy Road and penetrated some coarse grained Sandy Lean Clay Fill and Silty Sand to 25 feet in depth. The earth materials directly beneath the fill in these three areas are part of the Santa Clara Formation.

A seismic survey was conducted by JR Associates in October, 2021 utilizing both seismic refraction and Multi-Spectral Analysis of Surface Wave (MASW) techniques. The survey consisted of two lines oriented in a North-South configuration in order to intercept previously mapped fault and shear traces by E2C, as shown in green on Figure 3. The purpose of the surveys was to try to see if there was a relationship between low shear wave velocity and zones of faulting and shearing. The purpose of the refraction surveys was to see if there were bedrock highs or zones of more competent bedrock beneath the Site. The results are presented in the report dated October 14th and can be summarized as follows:

The refraction data indicated the materials at the site consist of a thin layer of less dense, looser, younger soils overlying denser Santa Clara formation sediments. There were no indications of competent Franciscan bedrock in the upper 40 feet of soil.

The shear wave data indicated two areas with low shear wave velocities consistent with the previous E2C maps. The first was approximately 545 feet from the start of Line 1. This area had a narrow zone of low S-wave velocity that could be an indication of soil shearing in a fault zone. The second zone was approximately 918 feet from the start of Line 2. This was a narrow zone of slightly lower S-wave velocities that coincided with the alleged fault traces noted by previous investigations. There was one other low S-wave velocity zone between 610 and 790 feet from the start of Line 2. This zone corresponded to an area of fill material.

In general, the low velocity zones mimic the location of the previously mapped faults and shears but the velocity panels are not as definitive as to show other areas of shearing or the lateral continuity of shears projecting towards the survey transects.

Discussion of Geologic Hazards

Geologic Hazards in the general vicinity includes landsliding in areas adjacent to the toe of steeper slopes as addressed in CEG (2016), expansive soil along portions of Piercy Rd as shown on Figure 3 by Earthtech (2000) and liquefaction in areas within the younger alluvium in the Coyote Creek area (ENGEO, 2006 and E2C, 2001). However, none of these geologic hazards appear to be a major concern for the project Site. The primary geologic hazard that may impact future development of the Site is the potential of fault rupture from a splay of the Piercy Fault.

Dibblee (1972) had mapped the Piercy Fault and considered it active and as a result it initially was included as a Special Studies Zone (SSZ) that required site-specific investigation as part of the Alquist-Priolo Act. Later work by the California Division of Mines and Geology (now the CGS) removed the SSZ designation for lack of evidence (Bryant, 1981). While it is no longer included in the State of California Earthquake Fault Zone (1982) it is still included in the City Special Studies zone (1983). The US Geological Survey (USGS, 2009) locates the Piercy Fault (called out as the Coyote Creek Fault) along a portion of Piercy Road as shown on Figure 2. Figure 2 also shows fault traces identified in selected fault investigations mentioned above. Most of these have been interpreted as splays from the Piercy Fault that extend towards the project site.

As mentioned previously, E2C conducted separate fault investigations on the Site and the adjacent parcel. The faults from these investigations are shown on Figure 3. Fault setback zones from these traces were provided by E2C on maps of 550 Piercy Road and the adjacent parcel (undated drawing by Ware Malcomb, referred to as the Alta Land Title Survey). These have been transferred onto Figure 3.

While fault trenching on the project Site appears to support the presence of several faults shown on Figure 3, faults extending from the adjacent parcel (with the exception of the one near the "Line 2" label) were not noted in the E2C report. I am willing to discuss the results of this Phase 1 letter report with Mr. Shimamoto, with the objective of obtaining a Preliminary Hazards Review Letter, representing agreement on a future proposed scope for the design-level investigation of the site.

I hope this provides the information you need at this time.

Sincerely,



William Godwin, C.E.G. 2199
Engineering Geologist

Attachments:

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Aerial Photographs Reviewed

Figure 1 Location and Fault Trace Map

Figure 2 Vicinity Geologic Map

Figure 3 Geologic and Exploration Map

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Aerial Photographs Reviewed

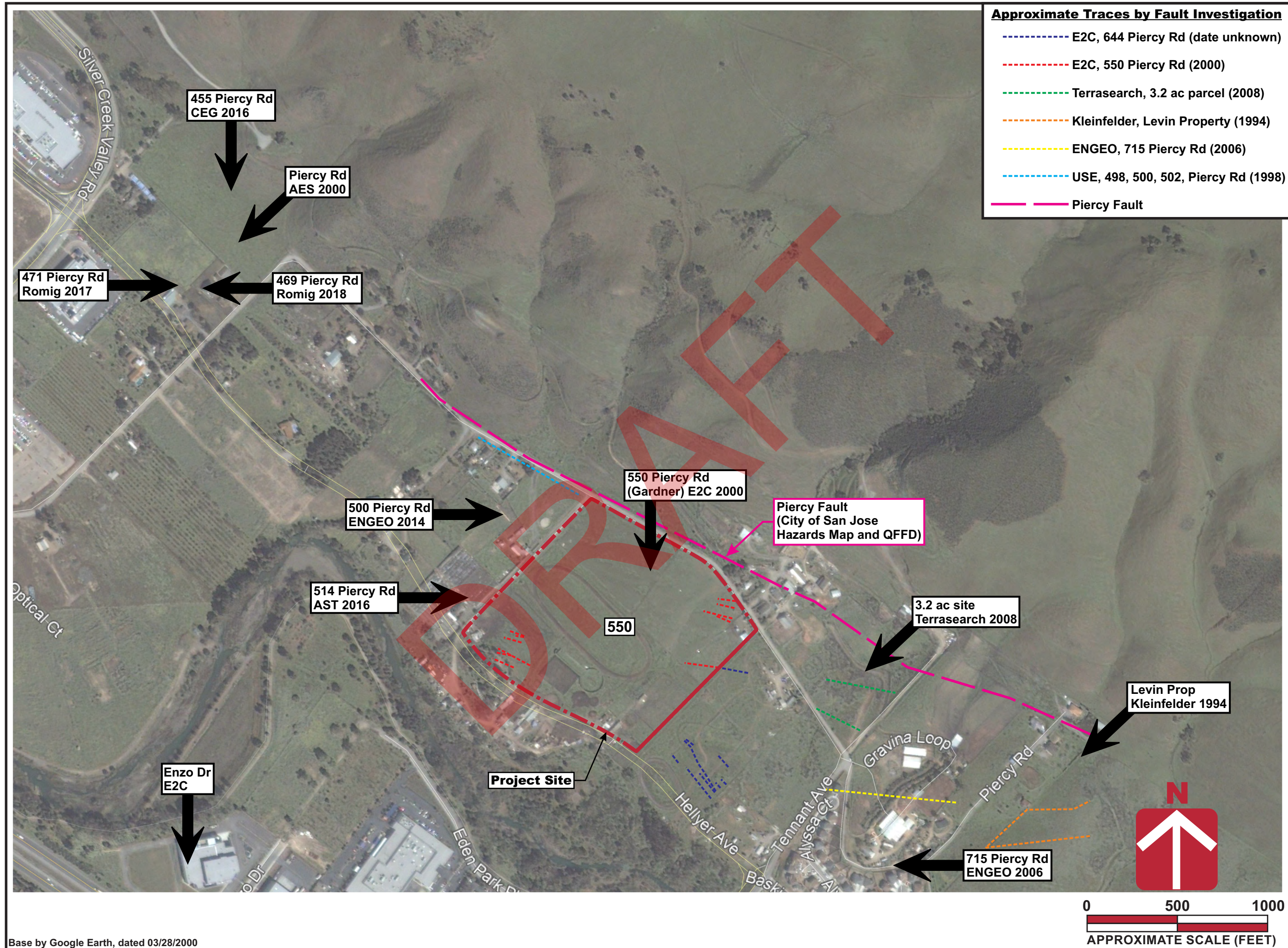
Table 1 - List of Reviewed Aerial Photos

Source	Form	Scale	Date	Flight	Line	Frames
Google Earth	Color	Various	8/6/2020	N/A	N/A	N/A
Google Earth	Color	Various	4/5/2016	N/A	N/A	N/A
Google Earth	Color	Various	5/31/2007	N/A	N/A	N/A
Google Earth	Color	Various	10/5/2004	N/A	N/A	N/A
Google Earth	Color	Various	9/30/2002	N/A	N/A	N/A
Google Earth	Color	Various	3/28/2000	N/A	N/A	N/A
PAS	Black & White	1:12000	8/17/1998	6100	233	19, 20
PAS	Black & White	1:12000	7/31/1996	5200	33	81, 82
PAS	Black & White	1:12000	7/22/1992	4230	233	23, 24
PAS	Black & White	1:12000	6/29/1988	3324	19	25, 26
PAS	Black & White	1:12000	4/28/1982	2135	19	20, 21
PAS	Black & White	1:12000	4/28/1982	2135	20	21, 22, 23
PAS	Black & White	1:12000	7/23/1980	1905	20	19, 20
PAS	Black & White	1:12000	11/4/1976	1277	20	19, 20
PAS	Black & White	1:12000	5/23/1974	1138	20	19, 20
PAS	Black & White	1:12000	10/12/1971	1006	20	16, 17
PAS	Black & White	1:36000	7/23/1963	550	17	38, 39
PAS	Black & White	1:30000	8/22/1960	385	9	3, 4, 5

Notes: PAS – Pacific Aerial Surveys

FIGURES

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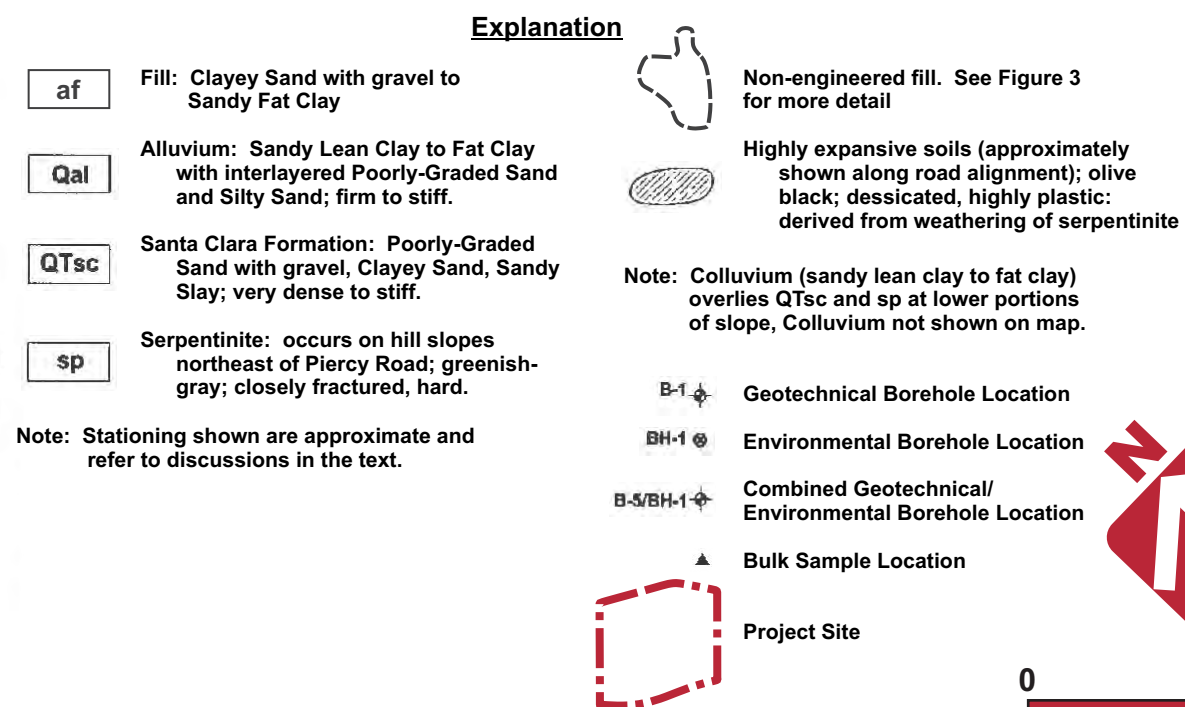


Base by Google Earth, dated 03/28/2000

- Approximate Traces by Fault Investigation**
- E2C, 644 Piercy Rd (date unknown)
 - E2C, 550 Piercy Rd (2000)
 - Terrasearch, 3.2 ac parcel (2008)
 - Kleinfelder, Levin Property (1994)
 - ENGEO, 715 Piercy Rd (2006)
 - USE, 498, 500, 502, Piercy Rd (1998)
 - Piercy Fault

Project Number 1076-4-2		Figure Number Figure 1	Date May 2022	Drawn By RRN
Location and Fault Trace Map				
550 Piercy Road San Jose, CA				
				

- - - - - E2C, 644 Piercy Rd (date unknown)
 - - - - - E2C, 550 Piercy Rd (2000)
 - - - - - Terrasearch, 3.2 ac parcel (2008)
 - - - - - Kleinfelder, Levin Property (1994)
 - - - - - ENGEO, 715 Piercy Rd (2006)
 - - - - - USE, 498, 500, 502, Piercy Rd (1998)
 ————— Piercy Fault



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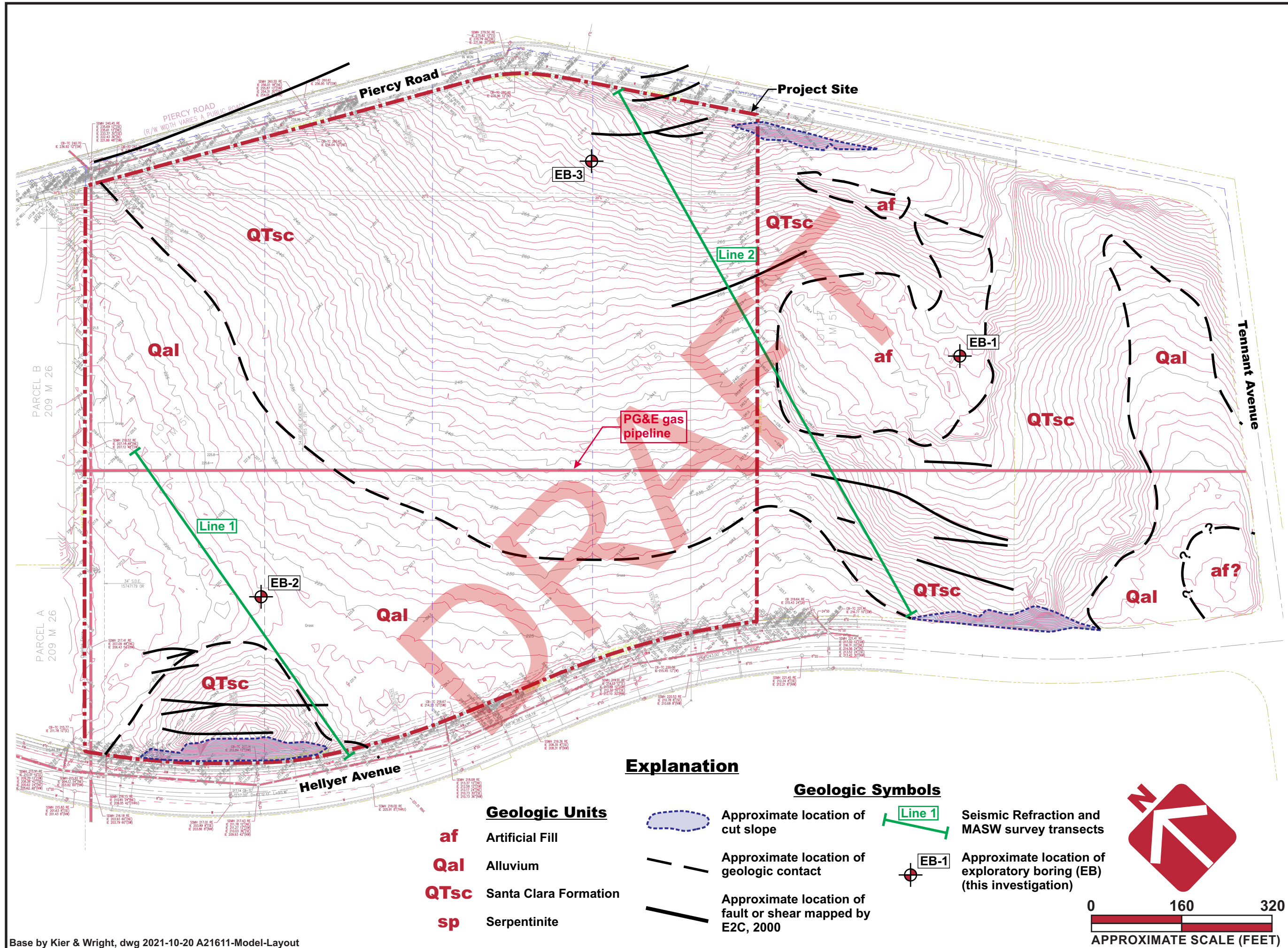
APPROXIMATE SCALE (FEET)



CORNERSTONE
EARTH GROUP

Figure Number

Date	May 2022	Drawn By	RRN
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Base by Kier & Wright, dwg 2021-10-20 A21611-Model-Layout

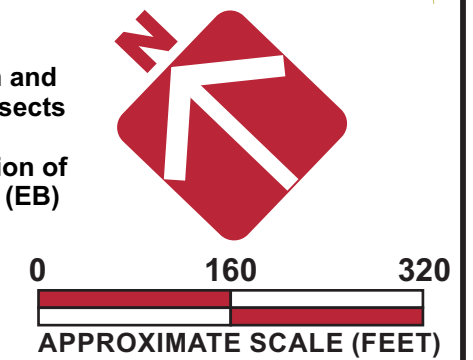
- Geologic Units**
- af Artificial Fill
 - Qal Alluvium
 - QTsc Santa Clara Formation
 - sp Serpentine

Explanation

- Approximate location of cut slope
- Approximate location of geologic contact
- Approximate location of fault or shear mapped by E2C, 2000

Geologic Symbols

- Line 1 Seismic Refraction and MASW survey transects
- EB-1 Approximate location of exploratory boring (EB) (this investigation)



Project Number	1076-4-2
	Figure Number
Figure 3	
Date	May 2022
Drawn By	RRN
Geologic and Exploration Plan	
550 Piercy Road San Jose, CA	
CORNERSTONE EARTH GROUP	

APPENDIX D: GEOPHYSICAL FAULT INVESTIGATION

JR ASSOCIATES REPORT DATED OCTOBER 14, 2021

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