Appendix D:
Geology and Soils Supporting Information

THIS PAGE INTENTIONALLY LEFT BLANK

Appendix D-1: Geological Hazard Evaluation and Design-Level Geotechnical Investigation

THIS PAGE INTENTIONALLY LEFT BLANK

CORNERSTONE EARTH GROUP

Type of Services	**Geologic Hazard Evaluation and Design-Level Geotechnical Investigation**
Project Name	**455 Piercy Road**
Location	**455 Piercy Road** **San Jose, California**
Client	**InSite Property Group**
Client Address	**811 Catalina Avenue, Suite 1306** **Redondo Beach, California**
Project Number	**1270-1-1**
Date	**March 22, 2021**

Prepared by **Stephen C. Ohlsen, P.E.**
Project Engineer
Geotechnical Project Manager

CERTIFIED ENGINEERING GEOLOGIST
CRAIG S. HARWOOD
No. 2275
STATE OF CALIFORNIA

Prepared by **Craig S. Harwood, P.G., C.E.G.**
Senior Project Geologist

REGISTERED PROFESSIONAL ENGINEER
NICHOLAS S. DEVLIN
No. 75613
Exp. 6-30-2022
CIVIL
STATE OF CALIFORNIA

Nicholas S. Devlin, P.E.
Principal Engineer
Quality Assurance Reviewer

CORNERSTONE EARTH GROUP

TABLE OF CONTENTS

TABLE OF CONTENTS I

SECTION 1: INTRODUCTION 1

1.1 Project Description -- 2

1.2 Scope of Services -- 3

1.3 Field Exploration Programs -- 3
1.3.1 Cornerstone Exploration Program 2021 3
1.3.2 Cornerstone Exploration Programs 2016 and 2017 3
1.3.3 Kleinfelder Exploration Program 2000 4
1.3.4 Hydro-Geo Consultants, Inc. (HGC) Exploration Programs 2000 4
1.3.5 Alliance Env. & Soil Engineering (AES) Exploration Program 2000 4

1.4 Previous Studies -- 4

1.5 Laboratory Testing Program -- 5

1.6 Environmental Services -- 5

SECTION 2: REGIONAL SETTING 5

2.1 Regional Seismicity -- 5
Table 1: Approximate Fault Distances 6

2.2 Regional Geologic Setting -- 6

2.3 Site Geology -- 7
2.3.1 Summary of Previous Fault Investigations 8
2.3.2 Review of Aerial Photographs 8

SECTION 3: SITE CONDITIONS 9

3.1 Geomorphology -- 9

3.2 Surface Description -- 9

3.3 Subsurface Conditions -- 10
3.3.1 EB-1 through EB-9 (CEG, 2017 and 2021) 10
3.3.2 Test Pit 1 and Trenches 1A, 1B, and 2 (CEG, 2016) 10
3.3.3 Exploratory Borings K-1, K-1A, K-2 and Test Pits TP-1 through TP-8 (Kleinfelder, 2000) 10

3.3.4 Exploratory Trenches ET-1 through ET-3 (HGC, 2000)....10
3.3.5 Exploratory Borings B-1 through B-6 (AES, 2000)....11
3.3.6 Summary 11
3.3.1 Plasticity/Expansion Potential 11
3.3.2 In-Situ Moisture Contents 11

3.4 Groundwater ---- 11

3.5 Corrosion Screening ---- 12
Table 2A: Summary of Corrosion Test Results....12
3.5.1 Preliminary Soil Corrosion Screening....12
Table 2B: ACI 318-19 Table 19.3.1.1 Exposure Categories and Classes 13
Table 2C: ACI 318-19 Table 19.3.2.1 Requirements for Concrete by Exposure Class 13

SECTION 4: GEOLOGIC HAZARDS....13

4.1 Fault Surface Rupture---- 13

4.2 Estimated Ground Shaking ---- 14

4.3 Liquefaction Potential---- 14
4.3.1 Ground Deformation and Surficial Cracking Potential....15

4.4 Lateral Spreading ---- 15

4.5 Seismic Settlement/Unsaturated Sand Shaking ---- 15

4.6 Landsliding ---- 15
Table 3: Debris Flow Scenarios....17
4.6.1 Slope Stability 18
Table 4: Summary of Soil and Bedrock Strength Properties....19

4.7 Soil Creep ---- 20

4.8 Flooding ---- 20

SECTION 5: CONCLUSIONS....20

5.1 Summary ---- 20
5.1.1 Fault Surface Rupture....21
5.1.2 Landsliding (Debris Flows) 21
5.1.3 Presence of Expansive Soil 21
5.1.4 Presence of Undocumented Fill and Existing Utilities....22
5.1.5 Strong Ground Shaking....22
5.1.6 Soil Corrosion Potential 22

5.2 Plans and Specifications Review---- 23

5.3 Construction Observation and Testing ---------- 23

SECTION 6: DEBRIS FLOW RETENTION WALLS 23

SECTION 7: EARTHWORK..........24

7.1 Site Demolition ---------- 24
7.1.1 Demolition of Existing Slabs, Foundations and Pavements..........24
7.1.2 Abandonment of Existing Utilities..........24

7.2 Site Clearing and Preparation ---------- 25
7.2.1 Site Stripping 25
7.2.2 Tree and Shrub Removal..........25

7.3 Mitigation of undocumented Fill ---------- 25

7.4 Temporary Cut and Fill Slopes ---------- 26

7.5 Subgrade Preparation ---------- 26

7.6 Wet Soil Stabilization Guidelines ---------- 26
7.6.1 Scarification and Drying..........27
7.6.2 Removal and Replacement 27
7.6.3 Chemical Treatment..........27

7.7 Material for Fill ---------- 27
7.7.1 Re-Use of On-site Soils 27
7.7.2 Potential Import Sources..........28
7.7.3 Non-Expansive Fill Using Lime Treatment..........28

7.8 Compaction Requirements ---------- 28
Table 5: Compaction Requirements..........29
7.8.1 Construction Moisture Conditioning 29

7.9 Trench Backfill ---------- 30

7.10 Permanent Cut and Fill Slopes ---------- 30

7.11 Site Drainage ---------- 30

7.12 Low-Impact Development (LID) Improvements ---------- 31
7.12.1 Storm Water Treatment Design Considerations 32

7.13 Permanent Erosion Control Measures ---------- 33

7.14 Landscape Considerations ---------- 34

SECTION 8: FOUNDATIONS 34

8.1 Summary of Recommendations ---------- 34

8.2 Seismic Design Criteria ---------- 34
Table 6: CBC Site Categorization and Site Coefficients 35

8.3 Shallow Foundations ---------- 35
8.3.1 Conventional Footings 35
8.3.2 Footing Settlement 36
Table 8: Assumed Structural Loading 36
8.3.3 Lateral Loading 36
8.3.4 Conventional Footing Construction Considerations..........36

SECTION 9: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS 37

9.1 Interior Slabs-on-Grade ---------- 37

9.2 Warehouse Slabs-on-Grade---------- 37

9.3 Interior Slabs Moisture Protection Considerations---------- 37

9.4 Exterior Flatwork ---------- 38

SECTION 10: VEHICULAR PAVEMENTS..........39

10.1 Asphalt Concrete ---------- 39
Table 9A: Asphalt Concrete Pavement Recommendations, Untreated Subgrade 39
Table 9B: Asphalt Concrete Pavement Alternatives – Treated Subgrade 40

10.2 Portland Cement Concrete ---------- 41
10.2.1 Truck Access and Parking Areas..........41
10.2.2 Trash Enclosures..........41

10.3 Pavement Cutoff ---------- 42

SECTION 11: RETAINING WALLS 42

11.1 Static Lateral Earth Pressures ---------- 42
Table 10: Recommended Lateral Earth Pressures..........42

11.2 Seismic Lateral Earth Pressures ---------- 43

11.3 Wall Drainage ---------- 43

11.4 Backfill ---------- 43

11.5 Foundations---------- 44

SECTION 12: LIMITATIONS 44

SECTION 13: REFERENCES 45

FIGURE 1: VICINITY MAP
FIGURE 2: SITE PLAN
FIGURE 3: REGIONAL FAULT MAP
FIGURE 4: VICINITY GEOLOGIC MAP
FIGURE 5: SITE GEOLOGIC MAP
FIGURE 6: GEOLOGIC CROSS SECTION A-A'
FIGURE 7: GEOLOGIC CROSS SECTION B-B'
FIGURE 8: SEISMIC HAZARD MAP
FIGURE 9: DEBRIS FLOW MAP

APPENDIX A: FIELD INVESTIGATION
APPENDIX B: LABORATORY TEST PROGRAM
APPENDIX C: PREVIOUS FAULT INVESTIGATION – CORNERSTONE EARTH GROUP, 2016
APPENDIX D: PREVIOUS EXPLORATION DATA
APPENDIX E: SLOPE STABILITY ANALYSIS

Type of Services	**Geologic Hazard Evaluation and Design-Level Geotechnical Investigation**
Project Name	**455 Piercy Road**
Location	**455 Piercy Road San Jose, California**

SECTION 1: INTRODUCTION

This geotechnical report was prepared for the sole use of InSite Property Group for the warehouse project located at 455 Piercy Road project in San Jose, California. The location of the site is shown on the Vicinity Map, Figure 1. For our use, we were provided with the following documents:

- Architectural site plan titled "Piercy Rd. Light Industrial, Piercy Road, San Jose, CA, Preliminary Site Plan – Scheme 01", prepared by RGA Office of Architectural Design, dated April 12, 2021.

- Preliminary grading plan titled "Piercy Road – Preliminary Earthwork", prepared by Kimley Horn, dated April 2021.

- Preliminary concept site plan titled "Piercy Road Warehouse, Piercy Road near Hellyer Avenue, City of San Jose, California" prepared by RMW Architecture and Interiors (RMW), dated June 13, 2018.

- A previous review letter titled, "Preliminary Geologic/Seismic Hazard Review, Proposed Industrial Building, 455 Piercy Road; APN 678-93-030, Project No: 16-014496-GC (3-14392)," prepared by the City of San Jose Department of Public Works, dated May 25, 2017.

- A Slope Stability and Subsurface Investigation Report, titled "Landslide Subsurface Investigation and Slope Stability Analysis for Silver Creek Valley Corporate Center, San Jose, CA", dated, October 27, 2000, prepared by Kleinfelder, Inc.

- A supplemental evaluation titled "Supplemental Evaluation To Our 6/7/2000 Report, Location of the Piercy Fault at Proposed Silver Creek Valley Corporate Center, Piercy Road, San Jose, California, Project 3509-A", dated September 2000, prepared by Hydro-Geo Consultants, Inc. (HGC).

- A geologic evaluation titled "Geologic Evaluation, Proposed Silver Creek Valley Corporate Center, Piercy Road, San Jose, California, Project 3509-A", dated June 2000, prepared by Hydro-Geo Consultants, Inc. (HGC).

- A geotechnical investigation report, titled "Geotechnical Investigation Report for Proposed Industrial Development at Piercy Road, San Jose, CA", dated May 24, 2000, prepared by Alliance Env. & Soil Engineering (AES).

- An untitled, undated topographic survey of the site.

1.1 PROJECT DESCRIPTION

The project site is located at 455 Piercy Road (APN 678-93-030), approximately 350 feet northeast of the intersection of Piercy Road and Hellyer Road in the City of San Jose, California (Figure 1). The planned development will consist of one, single-story, at-grade, high-bay warehouse with office space and a mezzanine. The proposed building will have a footprint of 116,800 square feet and is anticipated to be of concrete tilt-up construction. Appurtenant loading dock areas, auto, truck and trailer parking, utilities, site walls, landscaping, and other improvements necessary for site development are also planned.

The site is currently not in use but previously received some improvements in the form of underground utilities. The northeast half of the site was also previously graded for commercial use in 2001. This grading consisted largely of cutting into the base of the natural hillside along the northeast perimeter of the site. Utility lines were installed (storm drain, electrical conduit, water, and gas) and were left in place. Two active gas lines appear to be located on the site. A gas transmission line is adjacent to the southwest edge of the property, and a gas supply line trends northwest extending from Piercy Road to Silver Creek Road. Older improvements are present, including a former irrigation water conveyance channel ("Evergreen Canal"), and an asphalt concrete paved access road along the northern and eastern edges of the property which is maintained for access to a water tank owned by the Santa Clara Valley Water District. There are localized stockpiles of soil remaining from the previous grading and/or utility installation activities. The remainder of the site is generally covered by vegetation consisting of grass and weeds, and small native shrubs. The site ranges from approximately Elevation 201 feet World Geodetic System 1984 (WSG84) at the southwest property line, to Elevation 355 feet at the eastern corner of the site. Elevations were based on interpolation of plan contours from the above referenced plans. The property includes southwest facing slopes on the northeast portion of the site, and a relatively level graded pad on the southwestern portion.

Previous reports from adjacent properties (Cornerstone, 2016) and two previous geologic fault and hazards evaluation investigations performed by our firm at this site (Cornerstone 2016 and 2017) characterized landslide hazards and fault surface rupture hazards at the site. The findings of those investigations have been included in relevant sections of this report. Additionally, our fault investigation report dated November 23, 2016 is attached to this report as Appendix C.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated February 4, 2021 and consisted of field and laboratory programs to evaluate physical and engineering properties of the subsurface soils, engineering analysis to prepare recommendations for site work and grading, building foundations, flatwork, retaining walls, and pavements, and preparation of this report. Brief descriptions of our exploration and laboratory programs are presented below.

1.3 FIELD EXPLORATION PROGRAMS

As discussed, Cornerstone previously performed explorations for a geologic fault investigation and geologic hazards clearance reports in 2016 and 2017, respectively. In addition, other consultants have previously performed explorations at the site prior to our work.

1.3.1 Cornerstone Exploration Program 2021

Our geotechnical field exploration program consisted of three exploratory borings drilled on February 22, 2021 with truck-mounted, hollow-stem auger drilling equipment. The borings were intended to supplement the explorations previously performed by Cornerstone and other consultants and were drilled to depths of 29 to 50 feet below the existing grades. The borings were backfilled with cement grout in accordance with local requirements.

In addition, and as part of the environmental investigation by Innovative and Creative Environmental Solutions (ICES), 12 test pits were excavated by a rubber-tired backhoe to depths of 4 to 5 feet below existing grades on March 4, 2021. The test pit excavations were observed by Cornerstone to evaluate existing fill depths across the site. The test pits were loosely backfilled with the excavation spoils, and the loose fills will need to be re-worked during site grading.

1.3.2 Cornerstone Exploration Programs 2016 and 2017

Our previous field exploration at the site consisted of one test pit excavated by a back-hoe equipped tractor on September 6, 2016 to a depth of 15 feet below the existing grade, and three trenches excavated with a track-mounted excavator on September 7 and 8 and October 7, 2016 to a depth of up to 17 feet below the existing grades. Our field exploration at the site in 2017 consisted of six exploratory borings drilled on September 11 and 12, 2017 with track-mounted, solid-stem auger drilling equipment. The borings were drilled to depths ranging from 14 to 35 feet. The borings were backfilled with cement grout in accordance with local requirements.

Previously as part of our fault investigation, one test pit was excavated by a track-mounted excavator to depths of 5 to 16 feet. The pits were loosely backfilled, and the loose fills will need to be re-worked during site grading unless the pits are outside the limits of sensitive improvements.

1.3.3 Kleinfelder Exploration Program 2000

Kleinfelder's previous field exploration at the site consisted of three exploratory borings drilled with truck-mounted drilling equipment on September 29, 2000, and eight test pits excavated with a back-hoe equipped tractor on October 3, 2000. The borings and test pits were drilled/excavated to depths of 20½ to 26½ feet and 5 to 15 feet, respectively, below the existing grades.

1.3.4 Hydro-Geo Consultants, Inc. (HGC) Exploration Programs 2000

HGC's previous field explorations consisted of two exploratory trenches excavated with a back-hoe equipped tractor on May 16, 2000 to a depth of up to 15 feet below the existing grades, and one exploratory trench excavated on September 11, 2000 to a depth of 20 feet below the existing grades.

1.3.5 Alliance Env. & Soil Engineering (AES) Exploration Program 2000

AES' field exploration program consisted of six exploratory borings drilled with truck-mounted, hollow-stem auger drilling equipment on March 6, 2000. The borings were drilled to depths of 18 to 31 feet below the existing grades.

The approximate locations of our and others' explorations discussed above are shown on the Site Plan and Site Geologic Map, Figures 2 and 5, respectively. Graphic logs and details regarding our and other consultants' field programs are included in Appendix A and D, respectively.

1.4 PREVIOUS STUDIES

Published and unpublished geologic and geotechnical reports conducted onsite and nearby were researched and reviewed for this investigation and are listed above and in the "References" section of this report. As discussed, geotechnical/geologic investigations were previously performed at the site and include AES, May 2000; HGC, June and September 2000; and Kleinfelder, October 2000. In addition, our firm performed two onsite evaluations including a geologic fault investigation in 2016, and a geologic hazards clearance report in 2017. These site studies were for a previous development concept that is different from the current concept; however, the information from those and other consultants' previous studies are included in this report. In addition, our fault investigation report is attached to this report as Appendix C.

The adjacent site to the northwest (5880 Hellyer Avenue) was the subject of other consultant studies that were particularly relevant to the current investigation. Those adjacent studies include a fault investigation by Associated Terra Consultants (ATC,1989), as well as two studies conducted by our firm; a feasibility-level geologic/geotechnical study (CEG, 2015), and a design-level geologic/geotechnical investigation (CEG, 2016).

A more detailed discussion of previous geologic and geotechnical investigation studies performed at the site and in the vicinity of the site is presented in “Geological Setting” section of this report and in our previous fault investigation report for the site included as Appendix C.

1.5 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 analysis, Plasticity Index tests, Consolidated-Undrained Triaxial Compression tests, and preliminary soil corrosion screening. Details regarding our laboratory program are included in Appendix B.

1.6 ENVIRONMENTAL SERVICES

We understand that environmental services for this project are being provided by ICES. If environmental concerns are determined to be present during their evaluations, ICES should review our geotechnical recommendations for compatibility with the environmental concerns.

SECTION 2: REGIONAL SETTING

2.1 REGIONAL SEISMICITY

The San Andreas Fault is the dominant structural feature in the region and is a fundamental geologic boundary between two of the earth’s tectonic plates. The fault system follows a northwest-trending path through most of California, arising in the south from a set of transform faults in the Gulf of California and joining, to the north, the Mendocino Fracture Zone offshore of the northern part of the state. The San Francisco Bay region is within a zone of distributed active deformation associated with the North America-Pacific plate boundary. The plate boundary zone has had a complex history that has involved over time plate subduction, and crustal extension and contraction in association with dextral (right-lateral) strike-slip movements along faults within the boundary zone. The present-day seismotectonic setting of the region is marked by high rates of earthquake occurrence, right-lateral shear deformation along the San Andreas Fault system, and components of contractional strain, both oblique and normal to the San Andreas.

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; UCERF3) 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 San Andreas Fault generated the great San Francisco earthquake of 1906 and the Loma Prieta earthquake of 1989 and passes 4.6 miles southwest of the proposed commercial site. Two other major active faults in the Bay area are the Hayward Fault (southeast extension), and the Calaveras Fault. The Calaveras Fault is located about 5.3 miles northeast of the site and the southeast segment of the Hayward Fault is located about 2.5 miles east. The nearest mapped surface trace of the Monte Vista-Shannon Fault is located approximately 4.1 miles south of the subject site.

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)	2.5	4.0
Monte Vista-Shannon	4.1	6.6
Calaveras (South)	5.3	8.5
Sargent	11.1	17.9
San Andreas (1906)	12.4	19.9
Hayward Fault	13.8	22.2
Calaveras (North)	16.1	25.9

*Distances are from estimated surface projection of each fault.

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

2.2 REGIONAL GEOLOGIC SETTING

The subject property is located within the southeastern portion of the City of San Jose, along the margin between the Santa Clara Valley (on the west) and the Silver Creek Hills (on the east). The interface between these two physiographic regions is defined by a band of front-range faults, along which the mountains have risen and been thrust over the valley over the past 5 to 10 million years.

Within the region, the San Andreas Fault system distributes shearing across a complex assemblage of primarily right lateral, strike-slip, parallel and sub-parallel faults, and includes the Hayward and Calaveras faults. Western traces of a segment of the Calaveras Fault occur within the San Jose Foothills in the northeastern corner of the quadrangle. The Hayward Fault is farther west, near the base of the San Jose Foothills. The northwest-trending Silver Creek Thrust Fault bisects the Silver Creek Hills in the southeastern part of the quadrangle. Several smaller transpressive faults also are mapped within the quadrangle, primarily along the base of the San Jose Foothills. They include the Evergreen, Quimby, Piercy, and Clayton Faults

(Bryant, 1981a, 1981b; California Division of Mines and Geology, 1982; Wentworth et al., 1999; Hitchcock and Brankman, 2002; USGS, 2006).

The northwest-trending Silver Creek Fault is a 40-km-long strike-slip fault in the eastern Santa Clara Valley, California, that has exhibited different behaviors within a changing San Andreas Fault system over the past 10-15 Ma (Wentworth, et al., 2010). Evidence concerning continuation into the Holocene of this second Quaternary phase of deformation on the Silver Creek Fault is conflicting (Wentworth et al., 2010). The Silver Creek Fault does not exhibit clear evidence of topographic or Holocene stratigraphic offset and is no longer zoned by the State (Bryant, 1981a, 1981b; California Division of Mines and Geology, 1982). Some regional studies (including some of the most recent) also suggests there is lack of evidence or in some cases, inconclusive and conflicting evidence of the Silver Creek Fault being active in the Holocene, although most of these workers have expressed the need for further study (Bryant, 1981a, 1981b; Wiegers and Tryhorn, 1992; Fenton and Hitchcock, 2001; Hitchcock and Brankman, 2002; Wentworth et al., 2010).

2.3 SITE GEOLOGY

Regional scale mapping covers the area of the site. These include the mapping of Dibblee (1972, 2005), Helley and Wesling (1990), Wentworth et al., (1999), Knudsen et al., (2000) and the California Geological Survey (CGS, 2000). The attached Local Geologic Map, Figure 4 is a partial reproduction from the map of Wentworth, et al., (1999). These maps present interpretations based on sparse data, and projection. These maps were published prior to the grading of the site in the early 2000's. These maps suggest that Quaternary-age alluvial deposits are prevalent within in the southwestern portion of the site and Jurassic serpentinite is shown on the sloping portions of the site. As noted in subsequent sections, site-specific investigations at and near the site show that the geology in the immediate area is somewhat more complex than shown on the published maps. Site specific studies and studies of the adjacent site on the northwest indicate the following earth material units are present at the site: Artificial fill (Af), Slope wash deposits (Qsr), Holocene Alluvial fan deposits (Qhf), Holocene Alluvial fan levee deposits (Qhl), Melange of the Franciscan Complex (sandstone and shale; KJfm), and Jurassic serpentinite (sp) (Hydro-Geo Consultants, 2000a, 2000b, Kleinfelder, 2000, and Alliance Environmental and Soil Engineering, 2000; Cornerstone 2016, 2017). Additionally, a dormant bedrock landslide (Ols/sp) is mapped just beyond the north property corner.

The CGS (2000) has identified the quaternary alluvium as "Holocene alluvial fan deposits (Qhf)". A compilation of 357 geotechnical tests conducted within this unit in the San Jose East Quadrangle indicates that the Qhf unit was found to generally consist of 41% lean clay, 29% silt, 17% silty sand, and 11% other constituents. Wentworth et al. (1999) indicate that the Qhf mapping unit consists of loose and moderately to well-sorted, sandy clay to silty clay, which may contain lenses of silt and fine gravel. Witer et al., describe the levee deposits (Qhl) as long, low sinuous ridges oriented down-fan and are composed of overbank materials.

The Mélange consists primarily of shale but with minor graywacke sandstone and ranges from light gray to light olive gray (N7 to 5Y 6/1) greywacke, and grayish black to olive gray (N2 to 5Y 4/1). The Mélange also consists of thickly bedded graywacke and thinly interbedded shales that are friable, have low to moderate strength, and are moderately weathering. The Serpentinite has a generally low strength and is highly sheared with occasional pockets of highly weathered material and non-deformed clasts. Associated mafic and ultramafic rocks are generally moderate to hard, and moderately weathered. Both bedrock units (Serpentinite and Mélange) have higher degrees of weathering and weaker strength in the more highly sheared and faulted zones. Refer to "Subsurface Conditions" section for more information on site specific conditions.

2.3.1 Summary of Previous Fault Investigations

Traditionally the Piercy Fault was projected to cross the site with a variety of trends (depending on source). Several previous consultant's fault studies and geotechnical investigations have been performed in the vicinity of the site and two of the fault investigations were conducted at the subject site. The results of the most recent fault investigation were conducted by Cornerstone in a report dated November 23, 2016. Cornerstone directed a subsurface investigation involving two trenches and one test pit along a projection of the fault zone based on our findings at the adjacent site (Edenvale Storage at 5880 Hellyer Road). Our fault investigation of the subject site confirmed an approximately 70-foot-wide zone of faulting extending through the subject site. The fault zone is bounded on the northeast by a fault with a thrust geometry and on the southwest by a fault that has a reverse geometry. The southwest edge of the fault zone (southwesterly bounding fault) is dipping toward the northeast and therefore is inclined away from the most probable area of development for habitable structures. A 35-foot setback along the hanging wall (northeast boundary) and a 25-foot setback along the footwall (southwest edge) was recommended. The trench and test pit locations, fault zone and recommended building setbacks are presented in Figures 2 and 5. A copy of our previous fault investigation report (Cornerstone, 2016) for the site is attached to this report as Appendix C.

2.3.2 Review of Aerial Photographs

Six sets of black and white, stereo-paired aerial photographs were reviewed, and one pair of color infrared photos were reviewed as a part of our study. These photographs were taken during the years from 1948 to 1981 and range in scale from 1:12,000 to 1:30,000. In addition to the stereoscopic pairs of aerial photographs, we also reviewed selected individual (non-stereoscopic) aerial photographs. A complete listing of the stereoscopic pairs of photographs reviewed is included in the "References" section. Additionally, we reviewed Google Earth® images spanning from 1998 through 2012. A summary of our observations is provided below.

At the time of the 1948 photos the site contained one to two structures, likely agriculturally related or a residence, located in what is now the northwest portion of the site. Trees to the northwest of the site and in line with trees on the east side of the site form a slight lineament potentially pertaining to a geologic structure (fault, contact, variation in soil), or more likely planted along a former property line, or potentially corresponding to the agriculturally developed limits. A tonal contrast on the eastern portion of the site in the flatlands extends from the base of the eastern valley in the direction water would flow. Aerial photos from 1956 show the flatlands to be mowed or graded, accentuating the base of the steeper foothill slopes. Dark linear features are noted in the upper reaches of the axes of the valleys. The Evergreen Canal is a distinct feature with dark vegetation growing on the uphill edges of the structure. Aerial photos from the 1940s to 1960s show distinct orchards in the lowlands southwest and southeast of the site.

Between 2000 and 2002 the site experienced significant grading for a proposed development that was never completed. The low-lying valley bottom was graded for two structures. The toe of slopes of the alluvial fan deposits were cut creating a linear slope trending northwest, bisecting the property. The graded, partially developed site, with installed (stubbed-out) underground utilities, has generally appeared to remain in the same condition until the current investigation.

SECTION 3: SITE CONDITIONS

3.1 GEOMORPHOLOGY

The property exists near the base of a hillside where the Silver Creek Hills transition to the valley bottom. Approximately one half (southwest one half) of the site is nearly level but with minor terracing from previous grading activities. Lowland portions of the site have been graded to three pads that decrease in elevation to the southwest. The slopes in the upland area (northeast half of the site) are southwest facing and variably inclined from moderate to locally steep. The toe area of the hillside had been cut down by approximately 7 feet to 25 feet (vertically) during previous site grading. The immediate area around the site extends from an area dominated by bedrock (northeastern portion) and transitions into an abandoned flood plain of the ancestral Coyote Creek (within the southwestern portion). The overall topographic relief at the site ranges from about Elevation 201 feet near the southwest property line to Elevation 355 feet at the northeast property line.

3.2 SURFACE DESCRIPTION

The site is overgrown with weeds, grasses, and Coyote Bush. A paved road is located parallel to the northwest and northern property lines. The road crosses the Evergreen Canal and leads further uphill to a water tank. Piercy Road skirts the curving southeast property boundary. An existing commercial property lies to the west. Partially to undeveloped lands lie beyond the northwest and southwest property edges.

The lands upslope (northeast) of the site is open, undeveloped land. The Evergreen Canal contours the uplands at approximately Elevation 300 feet, crossing in and out of the site twice. The canal is concrete lined and appears to be in relatively good condition (no observed cracking or breach), except for one area where the fill berm on the outboard edge of the canal was compromised in the eastern valley (Figure 2). A 30-foot section of metal culvert replaces the failed portion of the canal.

The lowland portion of the site has been graded into three terraces. The upper terrace was cut into the hill exposing bedrock and alluvial fan deposits. Storm drain inlet boxes were notched into the base of the cut slope.

3.3 SUBSURFACE CONDITIONS

As discussed, Cornerstone and other consultants previously performed field explorations at the site consisting of test pits, trenches, and borings. The subsurface conditions encountered in those explorations as well as the recent explorations are discussed in the following sections.

3.3.1 EB-1 through EB-9 (CEG, 2017 and 2021)

Our Exploratory Borings EB-1, EB-2, EB-5, and EB-6 performed in 2017 were located on the existing hillside portion of the site; however, EB-3, EB-4, and EB-7 through EB-9 performed in 2017 and 2021 were located within the lower graded portion of the site and generally encountered fill and residual soil to depths of 2½ to 3 feet and 3 to 5 feet, respectively, underlain alluvial deposits, consisting of very stiff to hard, moderately to high plasticity clay with sand and very dense, clayey sand. The fill and residual soil encountered generally consisted of hard, high plasticity clay, clay with gravel, and sandy clay; and hard, high plasticity clay, respectively. Our Borings EB-3 and EB-4 encountered bedrock consisting of serpentinite and graywacke and shale at depths of 13 to 14 feet below the existing grades.

Additionally, the Test Pits (TP-1 through TP-12) performed by Innovative and Creative Environmental Solutions (ICES, 2021) generally encountered fill underlain by moderate to high plasticity clay with sand and dense, clayey sand with gravel. The depths of fill encountered ranged from 1½ to 3 feet below the existing grades.

3.3.2 Test Pit 1 and Trenches 1A, 1B, and 2 (CEG, 2016)

Our exploratory Test Pit (TP-1) and Trenches 1A, 1B, and 2 performed in 2016 encountered colluvium, slope wash deposits, serpentinite and Franciscan mélange bedrock. Our exploratory borings of 2017 encountered colluvium, slope wash deposits (Qsr), bedrock (serpentinite and sedimentary rock units) and locally, fill. The soil and other surficial deposits were found to consist of very stiff to hard, moderately to high plasticity clay with sand and very dense, clayey sand with gravel. The bedrock was found to be weak to moderately hard in terms of rock hardness description.

3.3.3 Exploratory Borings K-1, K-1A, K-2 and Test Pits TP-1 through TP-8 (Kleinfelder, 2000)

Kleinfelder’s exploratory borings K-1, K-1A, and K-2 and Test Pits TP-1 through TP-8 were mainly located on the hillside in the eastern portion of the site and generally encountered colluvium consisting of lean clay with sand, sandy lean clay, and lean clay with silt underlain by bedrock consisting of serpentinite to a depth of 26½ feet below the existing grades.

3.3.4 Exploratory Trenches ET-1 through ET-3 (HGC, 2000)

HGC’s Exploratory Trenches ET-1 through ET-3 were located in the central and southern portions of the site and generally encountered alluvium consisting of high plasticity clay and colluvium consisting of sandy and silty clay underlain by bedrock to a depth of 20 feet below the existing grades.

3.3.5 Exploratory Borings B-1 through B-6 (AES, 2000)

AES' Boring B-1 through B-6 were located in the lower portion of the site and generally encountered alluvium and colluvium consisting of stiff to very stiff, high plasticity clay, hard, sandy to silty clay, and dense to very dense, clayey sand and clayey sand with gravel underlain by bedrock encountered at depths of 10 to 18 feet below the existing grades; however, bedrock was not encountered within B-2 which terminated at a depth of 22 feet below the existing grades.

3.3.6 Summary

Overall, the fill (af) consisted of intermixed site soil (well-sorted sands and fine gravels and serpentinite rock fragments. The slope wash unit (Qsr) consisted of clay with sands and gravels. Angular fine to course gravels are found throughout. Sporadic stony horizons of gravels demarcate probable former colluvial and/or slope wash events. The alluvial fan deposits (Qhf) consisted of clay with varying amounts of sand, silt and gravel. The alluvial fan Levee deposits (Qfl) consisted of primarily clays with some minor thin sand interbeds.

The Franciscan mélange bedrock (KJfm) consisted of fractured shale, mudstone and sandstone. This material was found to be in a weak to moderately strong condition, while the serpentinite bedrock varied considerably from weak to moderately hard. The distribution and thickness of various earth materials and subsurface structural features are depicted on our Site Geologic Map, Figure 5 and the Geologic Cross Sections A-A' and B-B', Figures 6 and 7, respectively.

3.3.1 Plasticity/Expansion Potential

We performed one Plasticity Index (PI) tests on a representative sample. Test results were used to evaluate expansion potential of surficial soils. The result of the surficial PI test indicated a PI of 36, indicating high expansion potential to wetting and drying cycles.

3.3.2 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents within the upper 10 feet range from 4 to 14 percent over the estimated laboratory optimum moisture. Moisture contents in the undocumented fill encountered in the test pits by CES ranged from 2 percent under to 18 percent over the estimated laboratory optimum moisture.

3.4 GROUNDWATER

The San Jose East Seismic Hazard Evaluation report (CGS, 2000) does not indicate or provide mapped historic high groundwater levels in the immediate area of the site. Groundwater was not encountered in our field explorations performed in 2016 and 2017 that extended up to a depth of 50 feet below the existing grades, and also it was not encountered in the previous consultants' explorations, some of which extended to a depth of 31 feet (AES, 2000; HGC, 2000; and Kleinfelder, 2000) . However, localized perched groundwater was encountered within our Boring EB-8 at a depth of 25 feet.

Therefore, it is our opinion the depth to groundwater is greater than 50 feet at the site. However, fluctuations in groundwater levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors.

3.5 CORROSION SCREENING

We tested one sample collected at a depth of 3½ feet for resistivity, pH, soluble sulfates, and chlorides. The laboratory test results are summarized in Table 2.

Table 2A: Summary of Corrosion Test Results

Sample Location	Depth (feet)	Soil pH[1]	Resistivity[2] (ohm-cm)	Chloride[3] (mg/kg)	Sulfate[4,5] (mg/kg)
EB-9	3½	6.8	1,407	9	38

Notes: [1]ASTM G51
[2]ASTM G57 - 100% saturation
[3]ASTM D3427/Cal 422 Modified
[4]ASTM D3427/Cal 417 Modified
[5]1 mg/kg = 0.0001 % by dry weight

Many factors can affect the corrosion potential of soil including moisture content, resistivity, permeability, and pH, as well as chloride and sulfate concentration. Typically, soil resistivity, which is a measurement of how easily electrical current flows through a medium (soil and/or water), is the most influential factor. In addition to soil resistivity, chloride and sulfate ion concentrations, and pH also contribute in affecting corrosion potential.

3.5.1 Preliminary Soil Corrosion Screening

Based on the laboratory test results summarized in Table 2A and published correlations between resistivity and corrosion potential, the soils may be considered severely corrosive to buried metallic improvements (Chaker and Palmer, 1989).

In accordance with the 2019 CBC Section 1904A.1, alternative cementitious materials for different exposure categories and classes shall be determined in accordance with ACI 318-19 Table 19.3.1.1, Table R19.3.1, and Table 19.3.2.1. We have summarized applicable exposure categories and classes from ACI 318-19, Table 19.3.1.1 for both sites below in Table 2B.

Table 2B: ACI 318-19 Table 19.3.1.1 Exposure Categories and Classes

Freezing and Thawing (F)	Sulfate (S, soil)	In Contact with Water (W)	Corrosion Protection of Reinforcement (C)
F0[1]	S0[2]	W0[3]	C1[4]

1 (F0) "Concrete not exposed to freezing-and-thawing cycles" (ACI 318-14)
2 (S0) "Water soluble sulfate in soil, percent by mass" is less than 0.10 (ACI 318-14)
3 (W0) "Concrete dry in service. Concrete not in contact with water and low permeability is not required" (ACI 318-14)
4 (C1) "Concrete exposed to moisture and but not to an external source of chlorides" (ACI 318-14)

In addition, ACI 318-19, Table 19.3.2.1 provides requirements for concrete by exposure class. Table 2C below indicates different requirements that we recommend be followed for the concrete design.

Table 2C: ACI 318-19 Table 19.3.2.1 Requirements for Concrete by Exposure Class

Exposure Class	Maximum water:cement ratio	Minimum Compressive Strength (psi)	Maximum Water-Soluble Chloride Ion Content (% wt)
F0	N/A	2,500	N/A
S0 (soil)	N/A	2,500	N/A
W0	N/A	2,500	N/A
C1	N/A	2,500	0.3 (0.06)[1]

1 Maximum water-soluble chloride ion content for non-pre-stressed concrete, (value for pre-stressed concrete).

We recommend the structural engineer and a corrosion engineer be retained to confirm the information provided and for additional recommendations, as required.

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT SURFACE RUPTURE

As discussed above several significant faults are located within 25 kilometers of the site. Fault surface rupture is a manifestation of the fault displacement at the ground surface and is usually associated with moderate to large magnitude earthquakes (Mw > 6.5); however, more recent paleoseismic studies of faults suggest that in some scenarios earthquake magnitudes as low as Mw 5.0 can produce fault surface rupture (tang, et al., 2015, Champenois, et al., 2017). The amount of surface-fault displacement depends on the earthquake magnitude and other factors. The displacements associated with surface fault rupture can have devastating effects to structures and lifelines situated astride the zone of rupture. Evaluation of surface fault rupture is based on the premise that future fault rupture will most likely occur along previous ruptures. Consequently, accurate determination of the location and character of previous fault surface rupture is required for surface fault hazard assessment. In terms of fault rupture hazard evaluations, faults are defined by the state as "active" if they display evidence of movement within Holocene time (the last 11,000 years), and "potentially active" if they display evidence of movement within Quaternary time (i.e., within the last 1.6 million years). Information on the Piercy Fault is rather obscure in the published literature.

Although consultant's studies on adjacent parcels (ATC, 1989; Edenvale Storage site) and United Soil Engineering/E2C, Inc. (2000, Foxcon Property on southeast – 550 Piercy Road) asserted that the Piercy Fault exhibits Holocene offset, our review of their trench logs resulted in a different conclusion regarding recency of fault offset. In their fault investigation of the subject site, Hydro-Geo Consultants (2000a) concluded the Piercy Fault is not active based on their observation that Holocene age sediments overlying the fault (exposed in two of their trenches) were not cut by faulting. Our own exploration of the subject site and adjacent Edenvale Storage site on the north (Cornerstone, 2015) suggests the fault zone does not cut Holocene soils and we believe the interpretations of ATC (Edenvale site) and United Soil Engineering/E2C, Inc. (550 Piercy Road site) concerning Holocene offset is inconsistent with the geologic field relations that they documented within their trench logs. We concluded the most recent offset within the Piercy Fault zone predates the Holocene and occurred within the Pleistocene epoch. This and other aspects of the faulting topic are presented in our 2016 fault report for the site. Nearby site-specific studies in the immediate vicinity for the Piercy Fault zone have resulted in recommended building exclusion zones along the surface traces of the fault zone. Our 2016 fault investigation of the subject site identified a 70-foot-wide zone of faulting located approximately parallel to the major break in slope in the middle of the site (Figure 5). The fault zone is bounded on the northeast by a fault with a thrust geometry and on the southwest by a fault that has a reverse geometry. Our recommended building setback lines that are depicted in Figures 2 and 5, extend along the northeastern edge and southwest bounding limits of the fault zone and apply to any future habitable structures at the site. These setbacks are equal to 35 feet along the northeastern edge and 25 feet along the southwestern edge of the mapped fault zone surface trace.

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 following the ground motion hazard analysis procedure presented in Chapter 16 and 18 and Appendix J of the 2019 California Building Code (CBC) and Chapter 21, Section 21.2 of ASCE 7-16 and Supplement No. 1. For our analysis we used a PGA_M of 0.84g which was determined in accordance with Section 21.5 of ASCE 7-16.

4.3 LIQUEFACTION POTENTIAL

The western portion of the site is within a State-designated Liquefaction Hazard Zone (CGS, San Jose East Quadrangle, 2000); however, a laterally extensive groundwater table has not been identified in published groundwater-themed compilations (CGS, 2000). With the exception of localized perched water within our Boring EB-8, groundwater has not been encountered in the previous site-specific studies at and adjacent to the site, and groundwater conditions are likely quite variable near the east hills. We screened the site for liquefaction during our site exploration by retrieving samples from the site, performing visual classification on sampled materials, and performing various tests to further classify the soil properties.

As discussed in the "Subsurface" section above, our subsurface explorations encountered very stiff to hard, clay and very dense, clayey sand with gravel underlain by Franciscan bedrock. In

In addition, the groundwater encountered with EB-8 is concluded to be perched and not indicative of aquifer groundwater levels. Therefore, it is our opinion that the potential for liquefaction to impact the proposed improvements at the site is negligible.

4.3.1 Ground Deformation and Surficial Cracking Potential

The methods used to estimate liquefaction settlements assume that there is a sufficient cap of non-liquefiable material to prevent ground deformation or sand boils. For ground deformation or surficial cracking to occur, the pore water pressure within the liquefiable soil layer will need to be great enough to break through the overlying non-liquefiable layer, which could cause significant ground deformation and settlement. The work of Youd and Garris (1995) is typically used to estimate the potential for ground deformation; however, the potential for the site to be susceptible to liquefaction is very low, therefore, the potential for ground deformation and surficial cracking due to liquefaction is also very low.

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.

The potential for liquefaction is considered low, and there are no open faces within a distance considered susceptible to lateral spreading; therefore, in our opinion, the potential for lateral spreading to affect the site is low.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose unsaturated sandy soils can settle during strong seismic shaking. As the soils encountered at the site were predominantly stiff to very stiff to hard clays and medium dense to dense sands, in our opinion, the potential for significant differential seismic settlement affecting the proposed improvements is low.

4.6 LANDSLIDING

Published landslide-themed compilation maps depicting landslides that have occurred throughout the bay area region in the El Nino events of 1982, and 1997-98 do not show any landslides at or adjacent to the site (Ellen and Weiczorek, 1988; Wentworth et al, 1997). The steep slopes beyond the north property corner and slopes further uphill to the northeast are included within county and state designated landslide hazard regulatory zones (Santa Clara County, 2002; CGS, 2000). See Figure 8, Seismic Hazard Map.

An area located just above the Evergreen Canal is mapped as a dormant rockfall landslide (1,400 feet long by 1,000 feet wide) upslope of the northern portion of the parcel. Specifically, the toe area of the landslide lies upslope of the northern portion of the site (CGS, 2000;

Wiegers/CGS, 2011; CGS, 2016). This mapped landslide is indicated on Figure 5. Due to this mapping, the sloping areas located just upslope of the northern property corner are located within a state-designated earthquake induced landslide zone. The state-designation is interpretive based on remote sensing techniques rather than site-specific information. Weigers characterized this feature as a “dormant landslide” consisting primarily of rock debris (“A landslide involving bedrock in which the rock that moves remains largely intact for at least a portion of the movement”). Weigers (2011) clarified the term dormant as “[t]he observed landforms related to the landslide have been greatly eroded, including significant gullies or canyons cut into the landslide mass and/or main scarp by small streams.”

We have observed this feature in aerial photographs, and in the field. The slope mass appears to be primarily Serpentinite, which has been displaced somewhat downslope toward the southwest in a large-scale slope movement which occupies a lower (and more gently inclined) position on the slope than it did originally. Exposures in the toe area located adjacent to the upslope edge of the canal indicate it includes hard resistant bedrock which is sheared. It is our judgment that this feature is dormant and has been subject to slope processes (deeply eroded) over an extended portion of the Holocene, perhaps up to several thousand years. This feature is unrelated to the debris flow lobes (mapped as Qsr: Slope Wash deposits) that were identified on site by our firm in 2017 and previous investigations (Kleinfelder, 2002; and Hydro-Geo, 2000a).

A previous investigation of the site by Hydro-Geo Consultants (HGC, June 2000) identified the potential for debris flows from multiple sources: “poorly consolidated mine tailings… [b]lockage within the Evergreen Canal by the accumulation of sediments and vegetation… and erosion of the non-engineered fill along the outer edge of the canal.” Our field investigation confirmed these conditions and potential sources; however, we have refined the interpretation of these potential debris flow source areas. A debris flow was also identified in the eastern edge of the site by Hydro-Geo Consultants resulting from a breach in the canal, which was in a reported state of disrepair.

We did not identify evidence of past debris flows at the site and the canal was observed to be repaired with an approximately 20-foot-long metal culvert section, which has subsequently become undercut by erosion beneath the edges of the pipe.

Sporadic stony horizons of gravel encountered in our test pits and trenches demarcate previous slope wash events (CEG, 2016). These horizons consist of fine to medium gravel, define thin beds, and were not well developed. This suggests relatively minor slope wash events, which could fall within the definition of debris flows but are more accurately described as slope wash deposits resulting from sheet wash in heavy rainfall rather than true debris flows. The steepest adjacent slopes have little soil cover and the troughs of the two swales contain generally clayey soils on slopes that are relatively gently inclined. It is our judgment that although previous slope wash events have impacted the site, these events were relatively minor and due to the stratigraphy exposed in the excavations, and probably predate the construction of the canal, which occurred decades ago. Whereas naturally derived, high volume/high velocity debris flow events are not likely a site concern, the fill material that supports the Evergreen Canal could produce debris flows as these materials are generally loose and are located on steep slopes.

The mound of mine tailings discussed by HGC (May 2000) is located 670 feet upslope (east) of the site (320 feet upslope of the canal) and consists of several cubic yards of primarily sandy angular gravel with cobbles. We consider it unlikely that this material could become mobilized, however, we also consider that the swale located downslope of the tailings area breaks to gentler grades well upslope of the canal, which would serve as a catchment structure in the event that it became mobilized.

Four hypothetical debris flow scenarios were analyzed to anticipate the effect they may have on planned retaining walls that will be located at the approximate upper edge of the cut slope at the base of the hills (Figure 9). Hypothetical volumes were estimated to calculate the potential velocities, run-out distances, and impact forces produced by debris flows. Table 3 presents the four hypothetical scenarios.

Table 3: Debris Flow Scenarios

Scenario*	Area (ft^2) / Thickness (ft) / Volume (ft^3)	Velocity[1] (ft/sec)	Run-Out Distance[2] (ft)	Impact Force[3] (lbs/ft^3)
W	1125 / 1 1125	20.7	32.3	125
X	1225 / 1.5 1838	36.8	102.2	125
Y	2455 / 3 7365	52.1	204.4	125
Z	1830 / 2 3660	42.5	136.2	125

*Hypothetical debris flows presented in Figure 8.

The hypothetical failures of fill material supporting the Evergreen Canal were considered in that they are capable of traveling the distance to the top of the cut slope and beyond onto the proposed planned development area.

The Evergreen Canal, which we understand is owned by the Santa Clara Valley Water District, is no longer used for conveyance of storm water, but still accumulates (and conveys) runoff in peak storm events. The canal remnant functions as a surface drainage interceptor ditch and fills with water and overflows during winter storms (Kleinfelder, 2000 and HGC, 2000). The potential for piping beneath the concrete shell or overtopping from blockages or during high flow events should be mitigated to reduce the potential for failure of the fill or native material adjacent to the canal. The canal should be periodically inspected to assure that it is has not fallen into a state of disrepair and repaired to reduce the risk of a breach and debris flow.

---

1 Semina and Turbino, 1990

2 Kang, 1997

3 Hollingsworth and Kovaks, 1981

The construction of diversion drains that convey runoff to approved drainage facilities in the lower part of the site could be considered to limit the risks.

4.6.1 Slope Stability

To screen the slopes adjacent to potential development at the site for slope stability, a cross section was prepared for each swale based on the existing topography and typical properties for the underlying soil and bedrock based on our review of our field and laboratory data, our previous work in similar formations, and published data for the underlying bedrock. The geologic cross sections used for our analysis is shown as Figures 6 and 7, Geologic Cross Sections A-A' and B-B', respectively. Geologic mapping at the site indicates that the structural orientation of discontinuities within the Serpentinite is generally favorable for the purpose of slope stability screening.

4.6.1.1 Method of Analysis

The stability of a slope is influenced by many factors including but not limited to the geologic structure and composition, inclination, and height of a slope, groundwater, climatic factors such as rainfall, and irrigation. In geotechnical engineering, "stability" is expressed as a ratio of resisting moments and forces divided by driving moments and forces termed the factor of safety (FS). Factors of safety can be calculated for static and seismic conditions. In performing the slope stability analysis, we followed the guidelines set forth by CGS in special publications 117A (2008).

Because the site is generally underlain by weathered Franciscan rock and serpentinite, we judged using slope stability analysis based on limit equilibrium methods and rotational failure modes appropriate based on the site conditions encountered and typical failure modes observed in the field at similar sites.

The stability of the geologic cross sections taken through the primary site locations of interest (Cross Sections A-A' and B-B' on Figures 2 and 5), the predominant swales on site mapped as dormant landslides, were evaluated using the computer program SLIDE, and circular modes of failure. Input parameters for the analyses include slope geometry, soil/bedrock layers or zones, total and saturated unit weights and strength parameters, and groundwater conditions.

In evaluating the stability of slopes under seismic conditions, SLIDE uses a "pseudo-static" method of analysis. The pseudo-static method models the effects of transient or pulsating earthquake loading on a potential slide mass by using an "equivalent" static horizontal acceleration acting on the mass of the potential landslide in a limit-equilibrium analysis. The ground motion parameter used in a pseudo-static analysis is referred to as the seismic coefficient "k". CGS (2008) has published recommendations for the selection of the "k" value in a publication titled, "Guidelines for Evaluation and Mitigation of Seismic Hazards in California, SP 117A." The site is located several faults in the Bay Area, and strong ground shaking can be expected during a seismic event near the site. In accordance with the CGS Guidelines, we have performed our pseudo-static analysis using simplified design procedures in accordance with Stewart and others (2003) to develop a "screen analysis procedure," based on a pseudo-static approach that accounts for the anticipated seismicity at the site and allows for different levels of acceptable displacements. For the controlling earthquake magnitude, and limiting displacements to a 15-centimeter threshold, we obtained a “k” value of 0.25 for our analysis.

Based on current procedures recommended in SP 117A, the minimum allowable factor of safety with respect to slope stability is 1.5 for static conditions. Slopes that have a factor of safety greater than 1.0 for seismic conditions using a pseudo-static seismic coefficient derived from the screening analysis procedure of Stewart and others (2003) can be considered stable.

4.6.1.2 Selection of Soil and Bedrock Properties

Laboratory testing was performed on selected samples to aid in selecting parameters for the soil and bedrock materials. In addition, we have also previously performed laboratory testing on similar materials in the area, and have reviewed previous analysis performed for the site, as well as published data for the bedrock types at the site. A summary of the soil and bedrock parameters used in our analyses are presented in the table below.

Table 4: Summary of Soil and Bedrock Strength Properties

Material Description	Total Unit Weight (pcf)	Saturated Unit Weight (pcf)	Cohesion (psf)	Friction Angle (degrees)
Colluvium/Slope Debris (Qsr)	110	115.9	150	27.1
Serpentinite (sp)	127.6	134.7	320	30.4

4.6.1.3 Selection of Groundwater Depth for Slope Stability Screening Analysis

Springs and evidence of seasonal spring activity was noted at the site, and perched water was encountered in one of our borings. For this reason, we modeled the slope under static conditions with a shallow piezometric surface generally 5 to 8 feet below the ground surface and within the colluvium.

4.6.1.4 Results of Slope Stability Screening Analysis

Static Conditions

Static analysis was performed based on laboratory testing results for remolded samples of the colluvium and Serpentinite, published shear strength values for the local formations (CGS), other analysis performed for the site, and engineering judgement, which fall within the range of expected values based on our testing of similar materials. Our analyses indicate that the factor of safety with respect to upper slope where a restrained soil nail wall is proposed is greater than a factor of safety of 1.5, implying global stability. Copies of the stability output for each section analyzed, also illustrating the assumed soil parameters and slope geometry, are attached in Appendix E. Our analysis was a review of global stability and did not include local stability analysis for any additional grading or cuts at the toe, or the stability of any potential wall or berm constructed as mitigation. If proposed, we should be contacted to review those improvements or grading for stability, if needed.

Seismic Conditions

For the seismic case, we performed a pseudo-static analysis using methods described in CSG publication SP 117A. Our analyses indicate that the factor of safety with respect to slope movement under seismic conditions for the site development grades appear stable, with a factor of safety of greater than 1.0. Copies of the stability output for the static and seismic cases analyzed, also illustrating the assumed soil parameters and slope geometry, are attached in Appendix E.

4.6.1.5 Slope Stability Conclusions

Based on the assumed properties, geometry, and our analysis, it appears that slope stability at the site is adequate under post-construction static and seismic conditions. However, care should be taken during construction to maintain stability. At this time, we are not aware of any new cuts being included in the planned development of the site. However, we should be contacted to review potential impacts to slope stability for any planned cuts and/or grading as part of any future development.

4.7 SOIL CREEP

It is feasible that in addition to other modes of slope movement, some soil creep could occur of the mantle of colluvium located over the serpentinite bedrock slopes. Based on the geometry of the site, and the recommended fault setbacks, we are of the opinion that those fault setbacks automatically place the proposed habitable structures far enough from the cut and natural slopes. The potential for soil creep to affect any future development is low.

4.8 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. Descriptions of each concern with brief outlines of our recommendations follow the listed concerns.

- Fault surface rupture
- Landsliding (debris flows)

- Presence of expansive soil
- Presence of undocumented fill and existing utilities
- Strong ground shaking
- Soil corrosion potential

5.1.1 Fault Surface Rupture

As previously discussed, a fault investigation conducted by our firm in 2016 (Appendix C) has resulted in a recommended building exclusion zone trending through the central portion of the site due to the presence of the Piercy Fault trending through the site. Please refer to that report for a complete presentation of the findings and Figure 2 for a depiction of the building exclusion zone. A 70-foot-wide zone of faulting is bounded on the northeast by a fault with a thrust geometry and on the southwest by a fault that has a reverse geometry. Our recommended building setback lines extend along the northeastern and southwest limits of the fault zone and apply to any future habitable structures at the site. These setbacks are equal to 35 feet along the northeastern bounding fault and 25 feet along the southwestern surface trace that bounds the southwest edge of the fault zone. Our setbacks are approximately shown on Figure 5, and represent the above setbacks from the fault zone, as mapped in our 2016 investigation. In our opinion, provided the appropriate setbacks are implemented during development, the potential for fault surface rupture affecting habitable structures should be low.

5.1.2 Landsliding (Debris Flows)

A portion of the hillside located just beyond the north property line is located within a landslide hazard regulatory zone and has been mapped as a large landslide by the CGS (2011, 2016). Based on our research, investigation, mapping and field observations, the large-scale landslide mapped on this portion of the slope is interpreted as a dormant landslide and is unlikely to move in the future. Hydro-Geo (2000) interpreted broad swales or convex-up hollows located on the hillside within the northeast portion of the site as landslides. This interpretation was based on observing surface features only. This mapping was adopted by Kleinfelder during their subsequent investigation of the site. Our subsurface investigation of the site indicated the toe areas of these features have formed in response to innumerable small scale, slope wash events rather than a catastrophic landslide mass movement. Old fills associated with the Evergreen Canal have been identified as potential debris flow sources. Hypothetical debris flow scenarios of involving this material were evaluated and found to be a source material for debris flows that could potentially impact future development. A debris flow retention wall is recommended within the sloping portion of the site in order to mitigate this potential constraint. A hypothetical location for the wall is shown on Figure 9 for the purposes of forward mitigation planning. Preliminary design recommendations for a debris flow retention wall is presented in Section 6 of this report.

5.1.3 Presence of Expansive Soil

Highly expansive surficial soils 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. Evaluation of potential import sources for the site should consider the acceptable range of plasticity, especially in the upper 3 to 5 feet of fill. Detailed grading and foundation recommendations addressing this concern are presented in the following sections.

5.1.4 Presence of Undocumented Fill and Existing Utilities

As discussed above, approximately 2½ to 3 feet of undocumented fills consisting of highly plastic, hard, fat clay with gravel was encountered in our Borings EB-7, EB-8, and EB-9. We understand that this undocumented fill is a result of previous grading activities when the southwest half of the site was graded for commercial use in 2001. This grading consisted largely of cutting into the base of the natural hillside along the northeast perimeter of the site. Utility lines were installed (storm drain, electrical conduit, water, and gas) and were left in place. Two active gas lines appear to be located on the site. A gas transmission line is adjacent to the southwest edge of the property, and a gas supply line trends northwest extending from Piercy Road to Silver Creek Road. Older improvements are present, including a former irrigation-water conveyance channel (“Evergreen Canal”), and an asphalt road along the northern edges of the property maintained for access to a water tank owned by the Santa Clara Valley Water District. There are localized stockpiles of soil remaining from the previous grading. These previous operations resulted in undocumented fill from trench backfill, grading, and localized stockpiles. Any undocumented fills encountered during site grading should be completely removed from within building areas and to a lateral distance of at least 5 feet beyond the building footprint or to a lateral distance equal to fill depth below the perimeter footing, whichever is greater. If observation and testing records are available for the existing fill materials, they may potentially be considered to be engineered fill and potentially re-used. Provided undocumented fills are mitigated by removal and replacement as engineered fill, the potential impact due to undocumented fill should be low. Detailed recommendations addressing this concern are presented in the “Mitigation of Undocumented Fills” section below.

5.1.5 Strong Ground Shaking

Strong ground shaking is expected at this site, as with most sites in the Bay Area, during a major earthquake in the area. To mitigate the effects of strong ground shaking, all planned structures should be designed in accordance with the most recent applicable Building Code for the specific development type. In our opinion, provided the appropriate procedures are used for design during development, the potential for ground shaking having a significant effect on the structures should be low.

5.1.6 Soil Corrosion Potential

Our testing indicates sulfate exposure at the site is low and therefore no cement-type restrictions to buried concrete are required. However, the corrosion potential for buried metallic structures, such as metal pipes, is considered severely corrosive. Based on the results of the preliminary soil corrosion screening, special requirements for corrosion control will likely be required to protect metal pipes and fittings.

5.2 PLANS AND SPECIFICATIONS REVIEW

We recommend that we be retained 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.

5.3 CONSTRUCTION OBSERVATION AND TESTING

As site conditions may vary significantly between the small-diameter borings performed during this investigation, we also recommend that a Cornerstone representative be present to provide geotechnical observation and testing during earthwork and foundation construction. This will allow us to form an opinion and prepare a letter at the end of construction regarding contractor compliance with project plans and specifications, and with the recommendations in our report. We will also be allowed to evaluate any conditions differing from those encountered during our investigation and provide supplemental recommendations as necessary. For these reasons, the recommendations in this report are contingent of Cornerstone providing observation and testing during construction. Contractors should provide at least a 48-hour notice when scheduling our field personnel.

SECTION 6: DEBRIS FLOW RETENTION WALLS

As previously discussed, whereas naturally derived, high volume debris flow events are not likely a site concern at the site, debris flow events could be produced from a failure in the fill material that forms the downslope edge of the Evergreen Canal. Four hypothetical debris flow scenarios were analyzed to anticipate the effect they may have on planned retaining walls that will be located at the approximate upper edge of the cut slope at the base of the hills (Figure 9). Hypothetical volumes were estimated to calculate the potential velocities, run-out distances, and impact forces produced by a debris flow.

In our opinion, there are several methods that could be used to provide a debris barrier to reduce the potential impact to development, including swales/berms, cable/mesh systems, and rigid debris-resisting barriers. Based on our review, debris flow scenarios, and our analysis, we estimate that any debris flow retention wall would need to be designed to retain a sufficient amount of debris to reduce the risk for potential impact to development. In our opinion, there are several locations where a debris-resisting barrier could be built. Figure 9 indicates a potential location for the wall; however, the wall could also be built near the toe of the existing hillside (and cut slope) but should not be located closer than 5 feet from the toe of the slope.

In our opinion, the wall should likely be supported on drilled piers, be at least 5 feet in height, have a deflector at the top of wall as part of the wall height, or include an additional foot of free board, and be designed for an equivalent fluid pressure of 125 pcf (impact force). While we recommend the wall be constructed at the top of the existing slope, we understand an option with the wall near the toe of the existing slope is being considered. If this alternative is desired or selected, we should be contacted to provide additional analysis and recommendations.

SECTION 7: EARTHWORK

7.1 SITE DEMOLITION

All existing improvements not to be reused for the current development, including all foundations, flatwork, pavements, utilities, and other improvements should be demolished and removed from the site. Recommendations in this section apply to the removal of these improvements, some of which are known to be present at the site, prior to the start of mass grading or the construction of new improvements for the project.

Cornerstone should be notified prior to the start of demolition and should be present on at least a part-time basis during all backfill and mass grading as a result of demolition. Occasionally, other types of buried structures (wells, cisterns, debris pits, etc.) can be found on sites with prior development. If encountered, Cornerstone should be contacted to address these types of structures on a case-by-case basis.

7.1.1 Demolition of Existing Slabs, Foundations and Pavements

All slabs, foundations, and pavements should be completely removed from within planned building areas. A discussion of recycling existing improvements is provided later in this report.

Special care should be taken during the demolition and removal of any existing floor slabs, foundations, utilities and pavements to minimize disturbance of the subgrade. Excessive disturbance of the subgrade, which includes either native or previously placed engineered fill, resulting from demolition activities can have serious detrimental effects on planned foundation and paving elements.

Existing foundations are typically mat-slabs, shallow footings, or piers/piles. If slab or shallow footings are encountered, they should be completely removed. If drilled piers are encountered, they should be cut off at an elevation at least 60-inches below proposed footings or the final subgrade elevation, whichever is deeper. The remainder of the drilled pier could remain in place. Foundation elements to remain in place should be surveyed and superimposed on the proposed development plans to determine the potential for conflicts or detrimental impacts to the planned construction. Following review, additional mitigation or planned foundation elements may need to be modified.

7.1.2 Abandonment of Existing Utilities

As discussed above, during previous site grading in 2001 utility lines were installed (storm drain, electrical conduit, water, and gas) and were left in place. All utilities not intended for use should be completely removed from within planned building areas. For any utility line to be considered acceptable to remain within building areas, the utility line must be completely backfilled with grout or sand-cement slurry (sand slurry is not acceptable), the ends outside the building area capped with concrete, and the trench fills either removed and replaced as engineered fill with the trench side slopes flattened to at least 1:1, or the trench fills are determined not to be a risk to the structure. The assessment of the level of risk posed by the particular utility line will determine whether the utility may be abandoned in place or needs to be completely removed. The contractor should assume that all utilities will be removed from within building areas unless provided written confirmation from both the owner and the geotechnical engineer.

Utilities extending beyond the building area may be abandoned in place provided the ends are plugged with concrete, they do not conflict with planned improvements, and that the trench fills do not pose significant risk to the planned surface improvements.

The risk for owners associated with abandoning utilities in place include the potential for future differential settlement of existing trench fills, and/or partial collapse and potential ground loss into utility lines that are not completely filled with grout.

7.2 SITE CLEARING AND PREPARATION

7.2.1 Site Stripping

The site should be stripped of all surface vegetation, and surface and subsurface improvements to be removed within the proposed development area. Demolition of existing improvements is discussed in the prior paragraphs. A detailed discussion of removal of existing fills is provided later in this report. Surface vegetation and topsoil should be stripped to a sufficient depth to remove all material greater than 3 percent organic content by weight. Based on our site observations, surficial stripping should extend about 6 to 8 inches below existing grade in vegetated areas.

7.2.2 Tree and Shrub Removal

Trees and shrubs designated for removal should have the root balls and any roots greater than ½-inch diameter removed completely. Mature trees are estimated to have root balls extending to depths of 2 to 4 feet, depending on the tree size. Significant root zones are anticipated to extend to the diameter of the tree canopy. Grade depressions resulting from root ball removal should be cleaned of loose material and backfilled in accordance with the recommendations in the “Compaction” section of this report.

7.3 MITIGATION OF UNDOCUMENTED FILL

As discussed, undocumented fills consisting of hard, fat clay with gravel was encountered in our Borings EB-7, EB-8, and EB-9. We understand that the southwestern half of the site was previously graded for commercial use in 2001. These operations resulted in undocumented fill from trench backfill, stockpiling, and grading operations. All undocumented fills should be completely removed from within building areas and to a lateral distance of at least 5 feet beyond the building footprint or to a lateral distance equal to fill depth below the perimeter footing, whichever is greater. Provided the fills meet the “Material for Fill” requirements below, the fills may be reused when backfilling the excavations. Based on review of the samples collected from our borings, it appears that the fill may be reused. If materials are encountered that do not meet the requirements, such as debris, wood, trash, those materials should be screened out of the remaining material and be removed from the site. Backfill of excavations should be placed in lifts and compacted in accordance with the "Compaction" section below.

If observations and testing records are available for the existing fill materials, they may potentially be considered to be engineered fill and potentially re-used. Provided undocumented fills are mitigated by removal and replacement as engineered fill, the potential impact due to undocumented fill should be low.

Fills extending into planned pavement and flatwork areas may be left in place provided they are determined to be a low risk for future differential settlement and that the upper 12 to 18 inches of fill below pavement subgrade is re-worked and compacted as discussed in the "Compaction" section below.

Exploratory trenches and test pits previously performed to explore the site, including the presence of faulting, were generally loosely backfilled. These exploration locations, where they extend into pavement and/or planned building areas, should be over-excavated and backfilled with engineered fill during grading.

7.4 TEMPORARY CUT AND FILL SLOPES

The contractor is responsible for maintaining all temporary slopes and providing temporary shoring where required. Temporary shoring, bracing, and cuts/fills should be performed in accordance with the strictest government safety standards. On a preliminary basis, the upper 10 feet at the site may be classified as OSHA Soil Type C materials. A Cornerstone representative should be retained to confirm the preliminary site classification.

Excavations performed during site demolition and fill removal should be sloped at 2:1 (horizontal:vertical) within the upper 5 feet below building subgrade. Actual excavation inclinations should be reviewed in the field during construction, as needed. Excavations below building subgrade and excavations in pavement and flatwork areas should be sloped in accordance with OSHA soil classification requirements.

7.5 SUBGRADE PREPARATION

After site clearing and demolition is complete, and prior to backfilling any excavations resulting from fill removal or demolition, the excavation subgrade and subgrade within areas to receive additional site fills, slabs-on-grade and/or pavements should be scarified to a depth of 6 inches, moisture conditioned, and compacted in accordance with the "Compaction" section below.

7.6 WET SOIL STABILIZATION GUIDELINES

Native soil and fill materials, especially soils with high fines contents such as clays and silty soils, can become unstable due to high moisture content, whether from high in-situ moisture

contents or from winter rains. As the moisture content increases over the laboratory optimum, it becomes more likely the materials will be subject to softening and yielding (pumping) from construction loading or become unworkable during placement and compaction.
As discussed in the "Subsurface" section in this report, the in-situ moisture contents are about 4 to 14 percent over the estimated laboratory optimum in the upper 10 feet of the soil profile. The contractor should anticipate drying the soils prior to reusing them as fill. In addition, repetitive rubber-tire loading will likely de-stabilize the soils.

There are several methods to address potential unstable soil conditions and facilitate fill placement and trench backfill. Some of the methods are briefly discussed below. Implementation of the appropriate stabilization measures should be evaluated on a case-by-case basis according to the project construction goals and the particular site conditions.

7.6.1 Scarification and Drying

The subgrade may be scarified to a depth of 12 to 18 inches and allowed to dry to near optimum conditions, if sufficient dry weather is anticipated to allow sufficient drying. More than one round of scarification may be needed to break up the soil clods.

7.6.2 Removal and Replacement

As an alternative to scarification, the contractor may choose to over-excavate the unstable soils and replace them with dry on-site or import materials. A Cornerstone representative should be present to provide recommendations regarding the appropriate depth of over-excavation, whether a geosynthetic (stabilization fabric or geogrid) is recommended, and what materials are recommended for backfill.

7.6.3 Chemical Treatment

Where the unstable area exceeds about 5,000 to 10,000 square feet and/or site winterization is desired, chemical treatment with quicklime (CaO), kiln-dust, or cement may be more cost-effective than removal and replacement. Recommended chemical treatment depths will typically range from 12 to 18 inches depending on the magnitude of the instability.

7.7 MATERIAL FOR FILL

7.7.1 Re-Use of On-site Soils

On-site soils with an organic content less than 3 percent by weight may be reused as general fill. General fill should not have lumps, clods or cobble pieces larger than 6 inches in diameter; 85 percent of the fill should be smaller than 2½ inches in diameter. Minor amounts of oversize material (smaller than 12 inches in diameter) may be allowed provided the oversized pieces are not allowed to nest together and the compaction method will allow for loosely placed lifts not exceeding 12 inches.

7.7.2 Potential Import Sources

Non-expansive material should be inorganic with a Plasticity Index (PI) of 15 or less, and not contain recycled asphalt concrete where it will be used within the habitable building areas. Imported soil for use as general fill material should be inorganic with a Plasticity Index (PI) of 20 or less, and not contain recycled asphalt concrete where it will be used within the habitable building areas. To prevent significant caving during trenching or foundation construction, imported material should have sufficient fines. Samples of potential import sources should be delivered to our office at least 10 days prior to the desired import start date. Information regarding the import source should be provided, such as any site geotechnical reports. If the material will be derived from an excavation rather than a stockpile, potholes will likely be required to collect samples from throughout the depth of the planned cut that will be imported. At a minimum, laboratory testing will include PI tests. Material data sheets for select fill materials (Class 2 aggregate base, ¾-inch crushed rock, quarry fines, etc.) listing current laboratory testing data (not older than 6 months from the import date) may be provided for our review without providing a sample. If current data is not available, specification testing will need to be completed prior to approval.

Environmental and soil corrosion characterization should also be considered by the project team prior to acceptance. Suitable environmental laboratory data to the planned import quantity should be provided to the project environmental consultant; additional laboratory testing may be required based on the project environmental consultant’s review. The potential import source should also not be more corrosive than the on-site soils, based on pH, saturated resistivity, and soluble sulfate and chloride testing.

7.7.3 Non-Expansive Fill Using Lime Treatment

As discussed above, non-expansive fill should have a Plasticity Index (PI) of 15 or less. Due to the high clay content and PI of the on-site soil and bedrock materials, it is not likely that sufficient quantities of non-expansive fill would be generated from cut materials. As an alternative to importing non-expansive fill, chemical treatment can be considered to create non-expansive fill. It has been our experience that for high PI clayey soil and bedrock materials will likely need to be mixed with at least 3 to 4 percent quicklime (CaO) or approved equivalent to adequately reduce the PI of the on-site soils to 15 or less. If this option is considered, additional laboratory tests should be performed during initial site grading to further evaluate the optimum percentage of quicklime required.

7.8 COMPACTION REQUIREMENTS

All fills, and subgrade areas where fill, slabs-on-grade, and pavements are planned, should be placed in loose lifts 8 inches thick or less and compacted in accordance with ASTM D1557 (latest version) requirements as shown in the table below. In general, clayey soils should be compacted with sheepsfoot equipment and sandy/gravelly soils with vibratory equipment; open-graded materials such as crushed rock should be placed in lifts no thicker than 18 inches consolidated in place with vibratory equipment. Each lift of fill and all subgrade should be firm and unyielding under construction equipment loading in addition to meeting the compaction requirements to be approved. The contractor (with input from a Cornerstone representative) should evaluate the in-situ moisture conditions, as the use of vibratory equipment on soils with high moistures can cause unstable conditions. General recommendations for soil stabilization are provided in the "Subgrade Stabilization Measures" section of this report. Where the soil's PI is 20 or greater, the expansive soil criteria should be used.

Table 5: Compaction Requirements

Description	Material Description	Minimum Relative[1] Compaction (percent)	Moisture[2] Content (percent)
General Fill (within upper 5 feet)	On-Site Expansive Soils	87 – 92	>3
	Low Expansion Soils	90	>1
General Fill (below a depth of 5 feet)	On-Site Expansive Soils	95	>3
	Low Expansion Soils	95	>1
Trench Backfill	On-Site Expansive Soils	87 – 92	>3
	Low Expansion Soils	90	>1
Trench Backfill (upper 6 inches of subgrade)	On-Site Low Expansion Soils	95	>1
Crushed Rock Fill	¾-inch Clean Crushed Rock	Consolidate In-Place	NA
Non-Expansive Fill	Imported Non-Expansive Fill	90	Optimum
Flatwork Subgrade	On-Site Expansive Soils	87 - 92	>3
	Low Expansion Soils	90	>1
Flatwork Aggregate Base	Class 2 Aggregate Base[3]	90	Optimum
Pavement Subgrade	On-Site Expansive Soils	87 - 92	>3
	Low Expansion Soils	95	>1
Pavement Aggregate Base	Class 2 Aggregate Base[3]	95	Optimum
Asphalt Concrete	Asphalt Concrete	95 (Marshall)	NA

1 – Relative compaction based on maximum density determined by ASTM D1557 (latest version)
2 – Moisture content based on optimum moisture content determined by ASTM D1557 (latest version)
3 – Class 2 aggregate base shall conform to Caltrans Standard Specifications, latest edition, except that the relative compaction should be determined by ASTM D1557 (latest version)

7.8.1 Construction Moisture Conditioning

Expansive soils can undergo significant volume change when dried then wetted. The contractor should keep all exposed expansive soil subgrade (and also trench excavation side walls) moist until protected by overlying improvements (or trenches are backfilled). If expansive soils are allowed to dry out significantly, re-moisture conditioning may require several days of re-wetting (flooding is not recommended), or deep scarification, moisture conditioning, and re-compaction.

7.9 TRENCH BACKFILL

Utility lines constructed within public right-of-way should be trenched, bedded and shaded, and backfilled in accordance with the local or governing jurisdictional requirements. Utility lines in private improvement areas should be constructed in accordance with the following requirements unless superseded by other governing requirements.

All utility lines should be bedded and shaded to at least 6 inches over the top of the lines with crushed rock (⅜-inch-diameter or greater) or well-graded sand and gravel materials conforming to the pipe manufacturer's requirements. Open-graded shading materials should be consolidated in place with vibratory equipment and well-graded materials should be compacted to at least 90 percent relative compaction with vibratory equipment prior to placing subsequent backfill materials.

General backfill over shading materials may consist of on-site native materials provided they meet the requirements in the "Material for Fill" section, and are moisture conditioned and compacted in accordance with the requirements in the "Compaction" section.

Where utility lines will cross perpendicular to strip footings, the footing should be deepened to encase the utility line, providing sleeves or flexible cushions to protect the pipes from anticipated foundation settlement, or the utility lines should be backfilled to the bottom of footing with sand-cement slurry or lean concrete. Where utility lines will parallel footings and will extend below the "foundation plane of influence," an imaginary 1:1 plane projected down from the bottom edge of the footing, either the footing will need to be deepened so that the pipe is above the foundation plane of influence or the utility trench will need to be backfilled with sand-cement slurry or lean concrete within the influence zone. Sand-cement slurry used within foundation influence zones should have a minimum compressive strength of 75 psi.

On expansive soils sites such this, it is desirable to reduce the potential for water migration into building and pavement areas through the granular shading materials. We recommend that a plug of low-permeability clay soil, sand-cement slurry, or lean concrete be placed within trenches just outside where the trenches pass into building and pavement areas.

7.10 PERMANENT CUT AND FILL SLOPES

From our review of the project scope, we do not anticipate any new slopes will be constructed nor that there will be any change to existing terrace slopes. Should this change, we should be contacted to provide keyway, benching, and drainage recommendations.

7.11 SITE DRAINAGE

Surface runoff should not be allowed to flow over the top of or pond at the top or toe of engineered slopes or retaining walls. Ponding should also not be allowed on or 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 towards suitable discharge facilities. Roof runoff should be directed away from building areas in closed conduits, to approved infiltration facilities, or on to hardscaped surfaces that drain to suitable facilities. Retention, detention or infiltration facilities should be spaced at least 10 feet from buildings, and preferably at least 5 feet from slabs-on-grade or pavements. These facilities are not recommended where stormwater infiltration may affect slopes at lower elevations on or adjacent to the site. However, if slopes are not present at lower elevations that could potentially be affected, and if retention, detention or infiltration facilities are located within these zones, we recommend that these treatment facilities meet the requirements in the Storm Water Treatment Design Considerations section of this report.

Lined v-ditches should be included at the top of slopes and intermediate benches, and at the toe of slopes or behind retaining walls adjacent to planned or existing development. All v-ditches and drain inlets should be sized to accommodate the design storm events for the upslope tributary area. Concrete-lined v-ditches should be reinforced as required and have adequate control and construction joints and should be constructed neat in excavations; backfill around formed ditches should not be allowed.

7.12 LOW-IMPACT DEVELOPMENT (LID) IMPROVEMENTS

The Municipal Regional Permit (MRP) requires regulated projects to treat 100 percent of the amount of runoff identified in Provision C.3.d from a regulated project's drainage area with low impact development (LID) treatment measures onsite or at a joint stormwater treatment facility. LID treatment measures are defined as rainwater harvesting and use, infiltration, evapotranspiration, or biotreatment. A biotreatment system may only be used if it is infeasible to implement harvesting and use, infiltration, or evapotranspiration at a project site.

Technical infeasibility of infiltration may result from site conditions that restrict the operability of infiltration measures and devices. Various factors affecting the feasibility of infiltration treatment may create an environmental risk, structural stability risk, or physically restrict infiltration. The presence of any of these limiting factors may render infiltration technically infeasible for a proposed project. To aid in determining if infiltration may be feasible at the site, we provide the following site information regarding factors that may aid in determining the feasibility of infiltration facilities at the site.

- The near-surface soils at the site are clayey, and categorized as Hydrologic Soil Group D, and is expected to have infiltration rates of less than 0.2 inches per hour. In our opinion, these clayey soils will significantly limit the infiltration of stormwater.

- Locally seasonal high groundwater is not mapped in the area, but perched groundwater was encountered as high as 21½ feet below grade in our borings, and therefore is expected to be at least 10 feet below the base of the infiltration measure.

- In our opinion, infiltration locations within 10 feet of the buildings would create a geotechnical hazard.

7.12.1 Storm Water Treatment Design Considerations

If storm water treatment improvements, such as shallow bio-retention swales, basins or pervious pavements, are required as part of the site improvements to satisfy Storm Water Quality (C.3) requirements, we recommend the following items be considered for design and construction.

7.12.1.1 General Bioswale Design Guidelines

- If possible, avoid placing bioswales or basins within 10 feet of the building perimeter or within 5 feet of exterior flatwork or pavements. If bioswales must be constructed within these setbacks, the side(s) and bottom of the trench excavation should be lined with 10-mil visqueen to reduce water infiltration into the surrounding expansive clay.

- Bioswales constructed within 3 feet of proposed buildings may be within the foundation zone of influence for perimeter wall loads. Therefore, where bioswales will parallel foundations and will extend below the "foundation plane of influence," an imaginary 1:1 plane projected down from the bottom edge of the foundation, the foundation will need to be deepened so that the bottom edge of the bioswale filter material is above the foundation plane of influence.

- The bottom of bioswale or detention areas should include a perforated drain placed at a low point, such as a shallow trench or sloped bottom, to reduce water infiltration into the surrounding soils near structural improvements, and to address the low infiltration capacity of the on-site clay soils.

7.12.1.2 Bioswale Infiltration Material

- Gradation specifications for bioswale filter material, if required, should be specified on the grading and improvement plans.

- Compaction requirements for bioswale filter material in non-landscaped areas or in pervious pavement areas, if any, should be indicated on the plans and specifications to satisfy the anticipated use of the infiltration area.

- If required, infiltration (percolation) testing should be performed on representative samples of potential bioswale materials prior to construction to check for general conformance with the specified infiltration rates.

- It should be noted that multiple laboratory tests may be required to evaluate the properties of the bioswale materials, including percolation, landscape suitability and possibly environmental analytical testing depending on the source of the material. We recommend that the landscape architect provide input on the required landscape suitability tests if bioswales are to be planted.

- If bioswales are to be vegetated, the landscape architect should select planting materials that do not reduce or inhibit the water infiltration rate, such as covering the bioswale with grass sod containing a clayey soil base.

- If required by governing agencies, field infiltration testing should be specified on the grading and improvement plans. The appropriate infiltration test method, duration and frequency of testing should be specified in accordance with local requirements.

- Due to the relatively loose consistency and/or high organic content of many bioswale filter materials, long-term settlement of the bioswale medium should be anticipated. To reduce initial volume loss, bioswale filter material should be wetted in 12-inch lifts during placement to pre-consolidate the material. Mechanical compaction should not be allowed, unless specified on the grading and improvement plans, since this could significantly decrease the infiltration rate of the bioswale materials.

- It should be noted that the volume of bioswale filter material may decrease over time depending on the organic content of the material. Additional filter material may need to be added to bioswales after the initial exposure to winter rains and periodically over the life of the bioswale areas, as needed.

7.12.1.3 Bioswale Construction Adjacent to Pavements

If bio-infiltration swales or basins are considered adjacent to proposed parking lots or exterior flatwork, we recommend that mitigative measures be considered in the design and construction of these facilities to reduce potential impacts to flatwork or pavements. Exterior flatwork, concrete curbs, and pavements located directly adjacent to bio-swales may be susceptible to settlement or lateral movement, depending on the configuration of the bioswale and the setback between the improvements and edge of the swale. To reduce the potential for distress to these improvements due to vertical or lateral movement, the following options should be considered by the project civil engineer:

- Improvements should be setback from the vertical edge of a bioswale such that there is at least 1 foot of horizontal distance between the edge of improvements and the top edge of the bioswale excavation for every 1 foot of vertical bioswale depth, or

- Concrete curbs for pavements, or lateral restraint for exterior flatwork, located directly adjacent to a vertical bioswale cut should be designed to resist lateral earth pressures in accordance with the recommendations in the “Retaining Walls” section of this report, or concrete curbs or edge restraint should be adequately keyed into the native soil or engineered to reduce the potential for rotation or lateral movement of the curbs.

7.13 PERMANENT EROSION CONTROL MEASURES

Hillside grading will require periodic maintenance after construction to reduce the potential for erosion and sloughing. At a minimum all slopes should be vegetated by hydroseeding or other landscape ground cover. The establishment of vegetation will help reduce runoff velocities, allow some infiltration and transpiration, trap sediment within runoff, and protect the soil from raindrop impact. Depending on the exposed material type and the slope inclination, more aggressive erosion control measures may be needed to protect slopes for one or more winter seasons while vegetation is establishing. For slopes with inclinations of 2:1 (horizontal:vertical) or greater, erosion control may consist of jute netting, straw matting, or erosion control blankets used in combination with hydroseeding.

Both construction and post-construction Storm Water Pollution Prevention Plans (SWPPPs) should be prepared for the project-specific requirements. We recommend that final grading plans be provided for our review.

7.14 LANDSCAPE CONSIDERATIONS

Since the near-surface soils are moderately to highly expansive, we recommend greatly reducing the amount of surface water infiltrating these soils near foundations and exterior slabs-on-grade. This can typically be achieved by:

- Using drip irrigation
- Avoiding open planting within 3 feet of the building perimeter or near the top of existing slopes
- Regulating the amount of water distributed to lawns or planter areas by using irrigation timers
- Selecting landscaping that requires little or no watering, especially near foundations.

We recommend that the landscape architect consider these items when developing landscaping plans.

SECTION 8: FOUNDATIONS

8.1 SUMMARY OF RECOMMENDATIONS

In our opinion, the proposed structures may be supported on shallow foundations provided the recommendations in the “Earthwork” section and the sections below are followed.

8.2 SEISMIC DESIGN CRITERIA

We understand that the project structural design will 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 shallow fills soils underlain by very dense sands and hard clays with typical SPT

"N" values above 50 blows per foot. Therefore, we have classified the site as Soil Classification C. 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 for design are presented in Table 6. The table below lists the various factors used to determine the seismic coefficients and other parameters.

Table 6: CBC Site Categorization and Site Coefficients

Classification/Coefficient	Design Value
Site Class	C
Site Latitude	37.260611°
Site Longitude	-121.780618°
0.2-second Period Mapped Spectral Acceleration[1], S_S	1.666g
1-second Period Mapped Spectral Acceleration[1], S_1	0.63g
Short-Period Site Coefficient – Fa	1.2
Long-Period Site Coefficient – Fv	1.4
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S_{MS}	1.999g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	0.882g
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.333g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	0.588g

8.3 SHALLOW FOUNDATIONS

8.3.1 Conventional Footings

Conventional footings should bear on natural, undisturbed soil or engineered fill, be at least 15 inches wide, 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 due to the presence of highly expansive soils and is intended to embed the footing below the zone of significant seasonal moisture fluctuation, reducing the potential for differential movement.

Footings constructed to the above dimensions and in accordance with the "Earthwork" recommendations of this report are capable of supporting maximum allowable bearing pressures of 3,000 psf for dead loads, 4,500 psf for combined dead plus live loads, and 6,000 psf for all loads including wind and seismic. These pressures are based on factors of safety of 3.0, 2.0, and 1.5 applied to the ultimate bearing pressure for dead, dead plus live, and all loads, respectively. These pressures are net values; the weight of the footing may be neglected for the portion of the footing extending below grade (typically, the full footing depth). Top and bottom mats of reinforcing steel should be included in continuous footings to help span irregularities and differential settlement.

8.3.2 Footing Settlement

Structural loads were not provided to us at the time this report was prepared; therefore, we assumed the typical loading in the following table.

Table 8: Assumed Structural Loading

Foundation Area	Range of Assumed Loads
Interior Isolated Column Footing	50 to 75 kips
Exterior Isolated Column Footing	50 to 75 kips
Perimeter Strip Footing	4 to 6 kips per lineal foot

Based on the above loading and the allowable bearing pressures presented above, we estimate that the total static footing settlement will be on the order of ½ inch, with about ¼ inch of post-construction differential settlement between adjacent foundation elements, or over a lateral distance of 50 feet. As our footing loads were assumed, we recommend we be retained to review the final footing layout and loading, and to verify the settlement estimates above.

8.3.3 Lateral Loading

Lateral loads may be resisted by friction between the bottom of footing and the supporting subgrade, and also by passive pressures generated against footing sidewalls. An ultimate frictional resistance of 0.45 applied to the footing dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 450 pcf may be used in design. The structural engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate values above. Where footings are adjacent to landscape areas without hardscape, the upper 12 inches of soil should be neglected when determining passive pressure capacity.

8.3.4 Conventional Footing Construction Considerations

Where utility lines will cross perpendicular to strip footings, the footing should be deepened to encase the utility line, providing sleeves or flexible cushions to protect the pipes from anticipated foundation settlement, or the utility lines should be backfilled to the bottom of footing with sand-cement slurry or lean concrete. Where utility lines will parallel footings and will extend below the “foundation plane of influence,” an imaginary 1:1 plane projected down from the bottom edge of the footing, either the footing will need to be deepened so that the pipe is above the foundation plane of influence or the utility trench will need to be backfilled with sand-cement slurry or lean concrete within the influence zone. Sand-cement slurry used within foundation influence zones should have a minimum compressive strength of 75 psi.

Footing excavations should be filled as soon as possible or be kept moist until concrete placement by regular sprinkling to prevent desiccation. A Cornerstone representative should observe all footing excavations prior to placing reinforcing steel and concrete. If there is a significant schedule delay between our initial observation and concrete placement, we may need to re-observe the excavations.

SECTION 9: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

9.1 INTERIOR SLABS-ON-GRADE

As the Plasticity Index (PI) of the surficial soils are high, the proposed slabs-on-grade should be supported on at least 24 inches of non-expansive fill (NEF) to reduce the potential for slab damage due to soil heave. The NEF layer should be constructed over subgrade prepared in accordance with the recommendations in the “Earthwork” section of this report. If moisture-sensitive floor coverings are planned, the recommendations in the “Interior Slabs Moisture Protection Considerations” section below may be incorporated in the project design if desired. If significant time elapses between initial subgrade preparation and NEF construction, the subgrade should be proof-rolled to confirm subgrade stability, and if the soil has been allowed to dry out, the subgrade should be re-moisture conditioned to at least three percent over the optimum moisture content.

The structural engineer should determine the appropriate slab reinforcement for the loading requirements and considering the expansion potential of the underlying soils. For unreinforced concrete slabs, ACI 302.1R recommends limiting control joint spacing to 24 to 36 times the slab thickness in each direction, or a maximum of 18 feet.

9.2 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. The warehouse slab should also be supported on at least 6 inches of non-expansive, 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. Due to the high plasticity of the surficial soils, an additional 18 inches of non-expansive fill (NEF) should underlie the upper granular base. All base and sub-base materials should be placed and compacted in accordance with the “Compaction” section of this report. If there will be areas within the warehouse that are moisture sensitive, such as equipment and elevator rooms, a vapor barrier may be placed over the upper granular base prior to slab construction. Please refer to the recommendations in the “Interior Slabs Moisture Protection Considerations” section for vapor barrier construction. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness.

9.3 INTERIOR SLABS MOISTURE PROTECTION CONSIDERATIONS

The following general guidelines for concrete slab-on-grade construction where floor coverings are planned are presented for the consideration by the developer, design team, and contractor. These guidelines are based on information obtained from a variety of sources, including the

American Concrete Institute (ACI) and are intended to reduce the potential for moisture-related problems causing floor covering failures, and may be supplemented as necessary based on project-specific requirements. The application of these guidelines or not will not affect the geotechnical aspects of the slab-on-grade performance.

- Place a minimum 15-mil vapor retarder conforming to ASTM E 1745, Class C requirements or better directly below the concrete slab; the vapor retarder should extend to the slab edges and be sealed at all seams and penetrations in accordance with manufacturer's recommendations and ASTM E 1643 requirements. A 4-inch-thick capillary break, consisting of crushed rock should be placed below the vapor retarder and consolidated in place with vibratory equipment. The mineral aggregate shall be of such size that the percentage composition by dry weight as determined by laboratory sieves will conform to the following gradation:

Sieve Size	Percentage Passing Sieve
1"	100
¾"	90 – 100
No. 4	0 - 10

  The capillary break rock may be considered as the upper 4 inches of the non-expansive fill previously recommended.

- The concrete water:cement ratio should be 0.45 or less. Mid-range plasticizers may be used to increase concrete workability and facilitate pumping and placement.

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

- Polishing the concrete surface with metal trowels is not recommended.

- Where floor coverings are planned, all concrete surfaces should be properly cured.

- Water vapor emission levels and concrete pH should be determined in accordance with ASTM F1869-98 and F710-98 requirements and evaluated against the floor covering manufacturer's requirements prior to installation.

9.4 EXTERIOR FLATWORK

Exterior concrete flatwork subject to pedestrian and/or occasional light pick up loading should be at least 4 inches thick and supported 12 inches of non-expansive fill overlying subgrade prepared in accordance with the "Earthwork" recommendations of this report. Flatwork that will be subject to heavier or frequent vehicular loading should be designed in accordance with the recommendations in the "Vehicular Pavements" section below. To help reduce the potential for uncontrolled shrinkage cracking, adequate expansion and control joints should be included. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness. Flatwork should be isolated from adjacent foundations or retaining walls except where limited sections of structural slabs are included to help span irregularities in retaining wall backfill at the transitions between at-grade and on-structure flatwork.

SECTION 10: VEHICULAR PAVEMENTS

10.1 ASPHALT CONCRETE

The following asphalt concrete pavement recommendations tabulated below are based on the Procedure 608 of the Caltrans Highway Design Manual, estimated traffic indices for various pavement-loading conditions, and on a design R-value of 5. The design R-value was chosen based on the results of the laboratory testing performed on a surficial sample collected from the proposed pavement area and engineering judgment considering the variable surface conditions.

Table 9A: Asphalt Concrete Pavement Recommendations, Untreated Subgrade

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base[1] (inches)	Total Pavement Section Thickness (inches)
4.0	2.5	7.5	10.0
4.5	2.5	9.5	12.0
5.0	3.0	10.0	13.0
5.5	3.0	12.0	15.0
6.0	3.5	12.5	16.0
6.5	4.0	14.0	18.0
7.0	4.0	15.5	19.5
7.5	4.5	17.0	21.5
8.0	5.0	17.5	22.5
8.5	5.0	19.0	24.0
9.0	5.5	20.5	26.0
9.5	6.0	22.0	28.0
10.0	6.0	23.5	29.5
10.5	7.0	24.0	31.0
11.0	7.0	26.0	33.0

Note: 1 – Caltrans Class 2 aggregate base; minimum R-value of 78.

Because surface soil may be improved using chemical treatment with Quicklime or Quicklime Plus, we estimated an improved subgrade R-value of 50 for pavement design. This improved R-value is based on treating the upper 12 inches of finished subgrade. The pavement sections presented in Table 9B are based on an improved subgrade R-value of 50.

Table 9B: Asphalt Concrete Pavement Alternatives – Treated Subgrade

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base[1] (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	4.0	7.0
5.5	3.0	4.5	7.5
6.0	3.5	4.5	8.0
6.5	4.0	5.0	9.0
7.0	4.0	5.0	9.0
7.5	4.5	5.0	9.5
8.0	5.0	5.5	10.5
8.5	5.0	6.0	11.0
9.0	5.5	7.0	12.5
9.5	6.0	7.0	13.0
10.0	6.0	8.5	14.5
10.5	7.0	8.5	15.5
11.0	7.0	9.0	16.0

Note: 1 – Caltrans Class 2 aggregate base; minimum R-value of 78.

Frequently, the full asphalt concrete section is not constructed prior to construction traffic loading. This can result in significant loss of asphalt concrete layer life, rutting, or other pavement failures. To improve the pavement life and reduce the potential for pavement distress through construction, we recommend the full design asphalt concrete section be constructed prior to construction traffic loading. Alternatively, a higher traffic index may be chosen for the areas where construction traffic will use the pavements.

Asphalt concrete pavements constructed on expansive subgrade where the adjacent areas will not be irrigated for several months after the pavements are constructed may experience longitudinal cracking parallel to the pavement edge. These cracks typically form within a few feet of the pavement edge and are due to seasonal wetting and drying of the adjacent soil. The cracking may also occur during construction where the adjacent grade is allowed to significantly dry during the summer, pulling moisture out of the pavement subgrade. Any cracks that form should be sealed with bituminous sealant prior to the start of winter rains. One alternative to reduce the potential for this type of cracking is to install a moisture barrier at least 24 inches deep behind the pavement curb.

10.2 PORTLAND CEMENT CONCRETE

10.2.1 Truck Access and Parking Areas

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). The exterior bus parking pads and stationing areas where trucks turn, brake, or stop should be constructed of reinforced Portland Cement Concrete (PCC) pavement. We recommend PCC parking areas and stationing areas be at least 7 inches thick and reinforced with continuous, distributed reinforcing in accordance with industry standards, and no less than a minimum of #4 bars at 18 inches on-center in both directions to help control shrinkage and to provide load transfer throughout the pads.

The PCC thickness recommended above is based on a concrete flexural strength of at least 550 psi, minimum 28-day compressive strength of 4,000 psi, and supporting the PCC on at least 6 inches of Class 2 aggregate base compacted to a minimum relative compaction of 95 percent (ASTM D1557) as recommended in the "Earthwork" section, and laterally restraining the PCC with curbs or concrete shoulders. The aggregate base should be constructed over compacted subgrade prepared in accordance with previous recommendations in this letter. The PCC should have a low water-cement ratio (industry standards), and in no case exceed a water-cement plus pozzolan ratio of 0.53.

Adequate expansion and control joints should be included. Joint spacing should not exceed about 24 times the pavement thickness, in approximately square panels for unreinforced concrete. Joint spacing could be increased somewhat for reinforced concrete where light reinforcing steel is provided to help hold together intermediate cracks that may form in panels. Construction and expansion joints, and dowels, should be designed in accordance with industry standards and joint sealing should be included due to the expansive soils at the site.

If the subgrade soils are to be chemically treated (lime/cement), the PCC thickness recommended above could be decreased to 6 inches, and the aggregate base section could be reduced to 4 inches. For chemically treated subgrade, we recommend the upper 15 inches of subgrade soil be treated and the treated subgrade should obtain a minimum R-value of 50 (or minimum unconfined strength target). Additional testing will need to be performed to determine the appropriate lime/cement percentage to be mixed with the subgrade soil to achieve an estimated R-value of 50. The remaining design and construction details should be consistent with the above recommendations for PCC pavements over untreated subgrade.

10.2.2 Trash Enclosures

Pads where trash containers will be stored, and where garbage trucks will park while emptying trash containers, should be constructed as PCC as well. We recommend that the trash enclosure pads and stress (landing) pads where garbage trucks will store, pick up, and empty trash be a minimum PCC thickness of 7 inches. The compressive strength, underlayment, and construction details should be consistent with the above recommendations for PCC pavements over untreated subgrade.

10.3 PAVEMENT CUTOFF

Surface water penetration into the pavement section can significantly reduce the pavement life, due to the native expansive clays. While quantifying the life reduction is difficult, a normal 20-year pavement design could be reduced to less than 10 years; therefore, increased long-term maintenance may be required.

It would be beneficial to include a pavement cut-off, such as deepened curbs, redwood-headers, or "Deep-Root Moisture Barriers" that are keyed at least 4 inches into the pavement subgrade. This will help limit the additional long-term maintenance.

SECTION 11: RETAINING WALLS

11.1 STATIC LATERAL EARTH PRESSURES

The structural design of any site retaining wall should include resistance to lateral earth pressures that develop from the soil behind the wall, any undrained water pressure, and surcharge loads acting behind the wall. Provided a drainage system is constructed behind the wall to prevent the build-up of hydrostatic pressures as discussed in the section below, we recommend that the walls be designed for the following pressures:

Table 10: Recommended Lateral Earth Pressures

Sloping Backfill Inclination (horizontal:vertical)	**Lateral Earth Pressure***	
	Unrestrained – Cantilever Wall	Restrained – Braced Wall
Level	45 pcf	45 pcf + 8H
3:1	55 pcf	55 pcf + 8H
2½:1	60 pcf	60 pcf + 8H
2:1	65 pcf	65 pcf + 8H
Additional Surcharge Loads	⅓ of vertical loads at top of wall	½ of vertical loads at top of wall

* Lateral earth pressures are based on an equivalent fluid pressure
** H is the distance in feet between the bottom of footing and top of retained soil

Basement walls should be designed as restrained walls. If adequate drainage cannot be provided behind the wall, an additional equivalent fluid pressure of 40 pcf should be added to the values above for both restrained and unrestrained walls for the portion of the wall that will not have drainage. Damp proofing or waterproofing of the walls may be considered where moisture penetration and/or efflorescence are not desired.

11.2 SEISMIC LATERAL EARTH PRESSURES

The 2019 CBC states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. At this time, we are not aware of any retaining walls for the project. However, minor landscaping walls (i.e. walls 6 feet or less in height) may be proposed. In our opinion, design of these walls for seismic lateral earth pressures in addition to static earth pressures is not warranted.

11.3 WALL DRAINAGE

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

Miradrain, Geotech Drainage Panels, or equivalent drainage matting can be used for wall drainage as an alternative to the Class 2 Permeable Material or drain rock backfill. Horizontal strip drains connecting to the vertical drainage matting may be used in lieu of the perforated pipe and crushed rock section. The vertical drainage panel should be connected to the perforated pipe or horizontal drainage strip at the base of the wall, or to some other closed or through-wall system such as the TotalDrain system from AmerDrain. Sections of horizontal drainage strips should be connected with either the manufacturer's connector pieces or by pulling back the filter fabric, overlapping the panel dimples, and replacing the filter fabric over the connection. At corners, a corner guard, corner connection insert, or a section of crushed rock covered with filter fabric must be used to maintain the drainage path.

Drainage panels should terminate 18 to 24 inches from final exterior grade. The Miradrain panel filter fabric should be extended over the top of and behind the panel to protect it from intrusion of the adjacent soil.

11.4 BACKFILL

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

11.5 FOUNDATIONS

Retaining walls may be supported on a continuous spread footing designed in accordance with the recommendations presented in the "Foundations" section of this report.

SECTION 12: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of InSite Property Group specifically to support the design of the 455 Piercy Road project in San Jose, California. The opinions, conclusions, and recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in Northern California at the time this report was prepared. No warranty, expressed or implied, is made or should be inferred.

Recommendations in this report are based upon the soil and groundwater conditions encountered during our subsurface exploration. If variations or unsuitable conditions are encountered during construction, Cornerstone must be contacted to provide supplemental recommendations, as needed.

InSite Property Group may have provided Cornerstone with plans, reports and other documents prepared by others. InSite Property Group 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 13: REFERENCES

Aagaard, B.T., Blair, J.L., Boatwright, J., Garcia, S.H., Harris, R.A., Michael, A.J., Schwartz, D.P., and DiLeo, J.S., 2016, Earthquake outlook for the San Francisco Bay region 2014–2043 (ver. 1.1, August 2016): U.S. Geological Survey Fact Sheet 2016–3020, 6 p., http://dx.doi.org/10.3133/fs20163020.

Associated Terra Consultants, June 30, 1989, Geologic Fault Investigation, APN: 678-14-069, Hellyer Avenue, San Jose, California: Job No. 105211.

Alliance Environmental and Soil Engineering, May 24, 2000, Geotechnical Investigation Report for Proposed Industrial Development at Piercy Road San Jose, California: File No. 00-255-GS.

Bailey, E.H. and Everhart, D.L., 1964, Geology and quicksilver deposits of the New Almaden district, Santa Clara County, California: U.S. Geological Survey, Professional Paper 360, scale 1:24,000.

Bryant, W.R., 1981a, Calaveras, Coyote Creek, Animas, San Felipe, and Silver Creek Faults, California Division of Mines and Geology: Fault Evaluation Report 122, 13p.

Bryant, W.R., 1981b, Fault Evaluation Report FER-106, SE Segment of the Hayward Fault, Evergreen Fault, Quimby Fault, Silver Creek Fault, and Piercy Fault, March 16, 6 pl.
California Building Code, 2019, Structural Engineering Design Provisions, Vol. 2.

California Division of Mines and Geology, 1982, Revised official map of Alquist-Priolo Earthquake Fault Hazard Zones, San Jose East Quadrangle: California Division of Mines and Geology, scale 1:24,000.

California Division of Mines and Geology (2008), "Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117A, September.

California Geological Survey, 2000, State of California Seismic Hazard Zones, San Jose East 7.5-Minute Quadrangle, California: Seismic Hazard Zone Report 044.

California Geological Survey, updated 2016, Landslide Inventory and Deep-Seated Landslide Susceptibility Interactive Mapping (Beta version), San Jose East Quadrangle, http://maps.conservation.ca.gov/cgs/lsi/, accessed on January 12, 2021.

Catchings, R.D., Goldman, M.R., Gandhok, G., Rymer, M.J., Underwood, D.H., 2000, US Geological Survey Open-File Report: 2000-125.

Champenois, J., Baize, S., Vallee, M., Jomard, H., Alvarado, A. P., Espin, P., Audin, L., 2017, Evidences of Surface Rupture Associated with a Low-magnitude (M_w5.0) Shallow Earthquake in the Ecuadorian Andes. *Journal of Geophysical Research: Solid Earth*, 122, 8446– 8458. https://doi.org/10.1002/2017JB013928

City of San Jose, 1983, Fault Hazard Maps (San Jose East Quadrangle, California): City of San Jose, California, 1:24,000 scale.

Cooper-Clark & Associates, 1974, Technical report, Geotechnical Investigation, City of San Jose's Sphere of Influence: Report submitted to City of San Jose Department of Public Works, 185 p., 26 plates, scale 1:48,000.

Cornerstone Earth Group, 2015, Fault Investigation, Silver Creek Valley Road Development, 5888 Hellyer Avenue, San Jose, California, dated December 3, 2015.

Cornerstone Earth Group, 2016, Design Level Geotechnical Investigation and Geologic Hazards Clearance, Edenvale Self Storage, 5888 Hellyer Avenue, San Jose, California, dated December 8, 2016.

Cornerstone Earth Group, 2016, Fault Investigation, 455 Piercy Road Fault Investigation, San Jose, California, dated November 23, 2016.

Dibblee, T.W. Jr., 1972, Preliminary geologic map of the San Jose East Quadrangle, California: U.S. Geological Survey Open File Report 72-92, scale 1:24,000.

Dibblee, T.W. and Minch, J.A., 2005, Geologic map of the San Jose East quadrangle, Santa Clara County, California: Dibblee Geological Foundation, Dibblee Foundation Map DF-155, scale 1:24,000.

Ellen, Stephen D, and Weiczorek, 1988, Landslides, Floods and Marine Effects of the Storm of January 3-5, 1982, in the San Francisco Bay Region, California, U.S. Geological Survey Paper 1434.

E2C, Inc., 2000, Geologic Hazard Assessment (Fault Mapping), 550 Piercy Road, San Jose, California, Proj. No. 1572SC01, dated September 21, 2000, unpublished consultant's report.

E2C, Inc., 2000, Addendum Geologic Hazard Assessment (Fault Mapping), 550 Piercy Road, San Jose, California, Proj. No. 1572SC01, dated November 8, 2000, unpublished consultant's report.

Fenton, C. H., and Hitchcock, C. S., 2001, Recent Geomorphic and Paleoseismic Investigations of Thrust Faults in Santa Clara Valley, California, in Ferriz, H., and Anderson, R., (Eds.) Engineering Geologic Practice in Northern California; California Department of Conservation Division of Miones and Geology, Bulletin 210

Graymer, R.W., Bryant, W.A., McCabe, C.A., Hecker, S., and Prentice, C.S., 2006, Map of Quaternary-active faults in the San Francisco Bay region: U.S. Geological Survey Scientific Investigations Map 2919.

Helley, E.J. and Wesling, J.R., 1990, Quaternary geologic map of the San Jose East Quadrangle, Santa Clara County, California: U.S. Geological Survey Open-File Report 90-427.

Helley, E.J., Graymer, R.W., Phelps, G.A., Showalter, P.K. and Wentworth, C.M., 1994, Preliminary Quaternary geologic maps of Santa Clara Valley, Santa Clara, Alameda, and San Mateo Counties, California, a digital database: U.S. Geological Survey Open-File Report 94-231, 8 p.

Hitchcock, S.H., Kelson, K.I., and Thompson, S.C., 1994, Geormorphic Investigations of Deformation Along the Northeastern Margin of the Santa Cruz Mountains, U.S. Geological Survey Open-File Report 94-187.

Hitchcock, C.S., Brankman, C. M., 2002, FINAL TECHNICAL REPORT: Assessment of late Quaternary deformation, eastern Santa Clara Valley, San Francisco Bay region, U. S. Geological Survey National Earthquake Hazards Reduction Program Award Number 01HQGR0034

Hollingsworth, R. and Kovacs, G.S., 1981. Soil Slumps and Debris Flows: Prediction and Protection: Bulletin of the Association of Engineering Geologists, Vol. 18, No. 1, pp. 17-28.

Hydro-Geo Consultants (HGC), Inc., June 2000, Geologic Evaluation Proposed Silver Creek Valley Corporation Center Piercy Road, San Jose, California, Project 3509-A.

Hydro-Geo Consultants (HGC), Inc., September 2000, Supplemental Evaluation to our 6/7/00 Report Location of the Piercy Fault at Proposed Silver Creek Valley Corporation Center Piercy Road, San Jose, California, Project 3509-A.

Kang, Z., 1997. Kinetic analysis on the deceleration and deposition process of viscous debris flows. In: Chen, C.L. (Ed.), Proceedings of the First International Conference on Debris Flow Hazard Mitigation: Mechanics, Prediction, and Assessment, San Francisco. ASCE, New York, pp.153-157.

Kleinfelder, Inc., October 27, 2000, Landslide Subsurface Investigation and Slope Stability Analysis Silver Creek Valley Corporate Center, San Jose, California: File No. 12-3077-00/GEO.

Knudsen, K.L., Sowers, J.M., Witter, R.C., Wentworth, C.M., and Helley, E.J., 2000, Preliminary maps of Quaternary deposits and liquefaction susceptibility, nine-county San Francisco Bay region, California: a digital database, U.S. Geological Survey, Open-File Report 00-444.

Nilsen, T.H. and Brabb, E.E., 1972, Preliminary photointerpretation and damage maps of landslide and other surficial deposits in of the San Jose East 7.5' Quadrangle, Santa Clara County, California: U.S. Geological Survey Miscellaneous File Map MF-75-277-49, scale 1:24,000.

Portland Cement Association, 1984, Thickness Design for Concrete Highway and Street Pavements: report.
Rogers, T.H., and J.W. Williams, 1974 Potential Seismic Hazards in Santa Clara County, California, Special Report No. 107: California Division of Mines and Geology.

Seminara G. and Tubino, M., 1990. Debris Flow: Meccanica, Controllo e Previsione. Roma, CNR-GNDCI, pgs.1-220.

Soil Test Trenches Survey, Hellyer Ave & Piercy Road, October 13, 2016, Scale 1" = 40' (Cornerstone, 2016).

Southern California Earthquake Center (SCEC), 1999, Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction Hazards in California, March.

State of California Department of Transportation, 2020, Highway Design Manual, Latest Edition.

Tang, M.-Y & Liu, J. & Shao, Y.-X & Wang, P. & Yuan, Z.-D., 2015, Analysis About the Minimum Magnitude Earthquake Associated with Surface Ruptures, Seismology and Geology, 37. 1193-1214.

U.S. Geological Survey and California Geological Survey, 2006, Quaternary fault and fold database for the United States, accessed November 13, 2016, from USGS web site: http://earthquake.usgs.gov/regional/qfaults/

USGS, 2011, Earthquake Ground Motion Parameters, Version 5.1.0, revision date February 10, 2011 - A Computer Program for determining mapped ground motion parameters for use with IBC 2006 available at http://earthquake.usgs.gov/research/hazmaps/design/index.php.

Wahrhaftig, C. and Sloan D., editors, 1989, Geology of San Francisco and Vicinity: Field Trip Guidebook of the 28th International Geological Congress.

USGS, 2011, Earthquake Ground Motion Parameters, Version 5.1.0, revision date February 10, 2011 - A Computer Program for determining mapped ground motion parameters for use with IBC 2006 available at http://earthquake.usgs.gov/research/hazmaps/design/index.php.

Weigers, M.O., and Tryhorn, A.D., 1992, Fault rupture studies along the Silver Creek fault, south San Jose, California: in Borchardt, G., Hirschfield, S.E., Lienkemper, J.J., McClellan, P., Williams, P.L., and Wong, I.G. (eds), Proceedings of the Second Conference on Earthquake Hazards in the Eastern San Francisco Bay Area, California Department of Conservation, Division of Mines and Geology, p. 225-230.

Weigers, M.O., 2011, Landslide Inventory of the San Jose East Quadrangle, California Geological Survey, 1:24,000 scale.

Wentworth et al., 1999, Preliminary Description of the San Jose 30x60 Minute Quadrangle, California: U.S. Geological Survey Open-File Report 98-795.

Wentworth, C.M., Blake, M.C. Jr., McLaughlin, R.J. and Graymer, R.W., 1999, Preliminary geologic map of the San Jose 30 X 60-minute Quadrangle, California: U.S. Geological Survey Open-File Report 98-795, 14 p.

Wentworth, C. M., Williams, R. A., Jachens, R.C., Graymer, R. W., and Stephenson, W.J., 2010, The Quaternary Silver Creek Fault Beneath the Santa Clara Valley, California, U.S. Geological Survey Open File Report 2010-1010.

Wentworth, C. M., Graham, S. E., Pike, R. J., Beukelman, G. S., Ramsey, D.W., and Barron, A. D., 1997, Summary Distribution of Slides and Earth Flows in the San Francisco Bay Region, California - U.S. Geological Survey Open File Report No. 97-745C, scale 1:62,500.

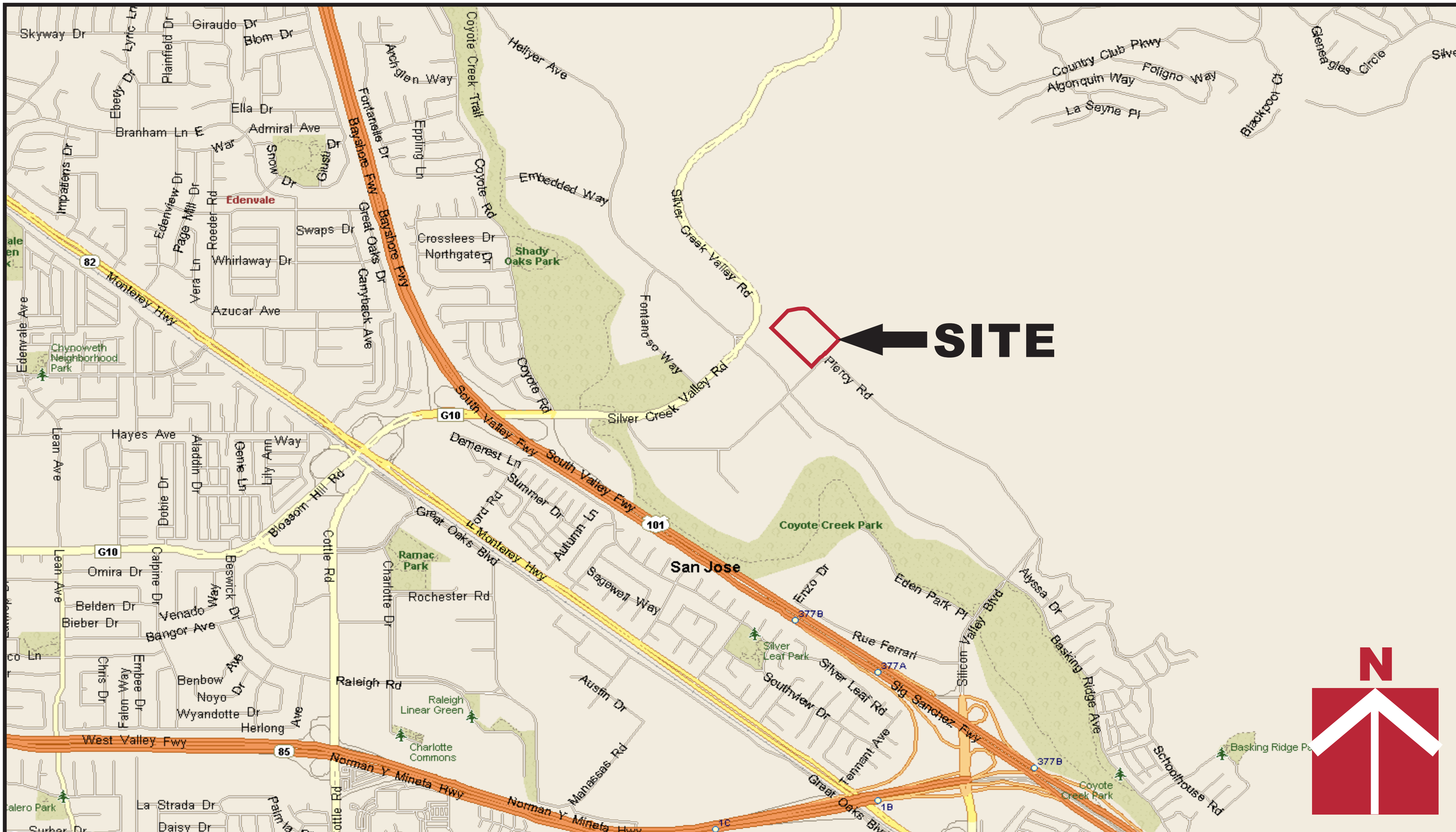
Witter, R.C., Knudsen, K.L, Sowers, J.M., Wentworth, C.M., Koehler, R.D., Randolph, C.E., Brooks, S.K., and Gans, K.D., 2006, Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, California: U.S. Geological Survey Open-File Report 2006-1037 (http://pubs.usgs.gov/of/2006/1037/)

Working Group on California Earthquake Probabilities, 2015, The Third Uniform California Earthquake Rupture Forecast, Version 3 (UCERF), U.S. Geological Survey Open File Report 2013-1165 (CGS Special Report 228). KMZ files available at: www.scec.org/ucerf/images/ucerf3_timedep_30yr_probs.kmz

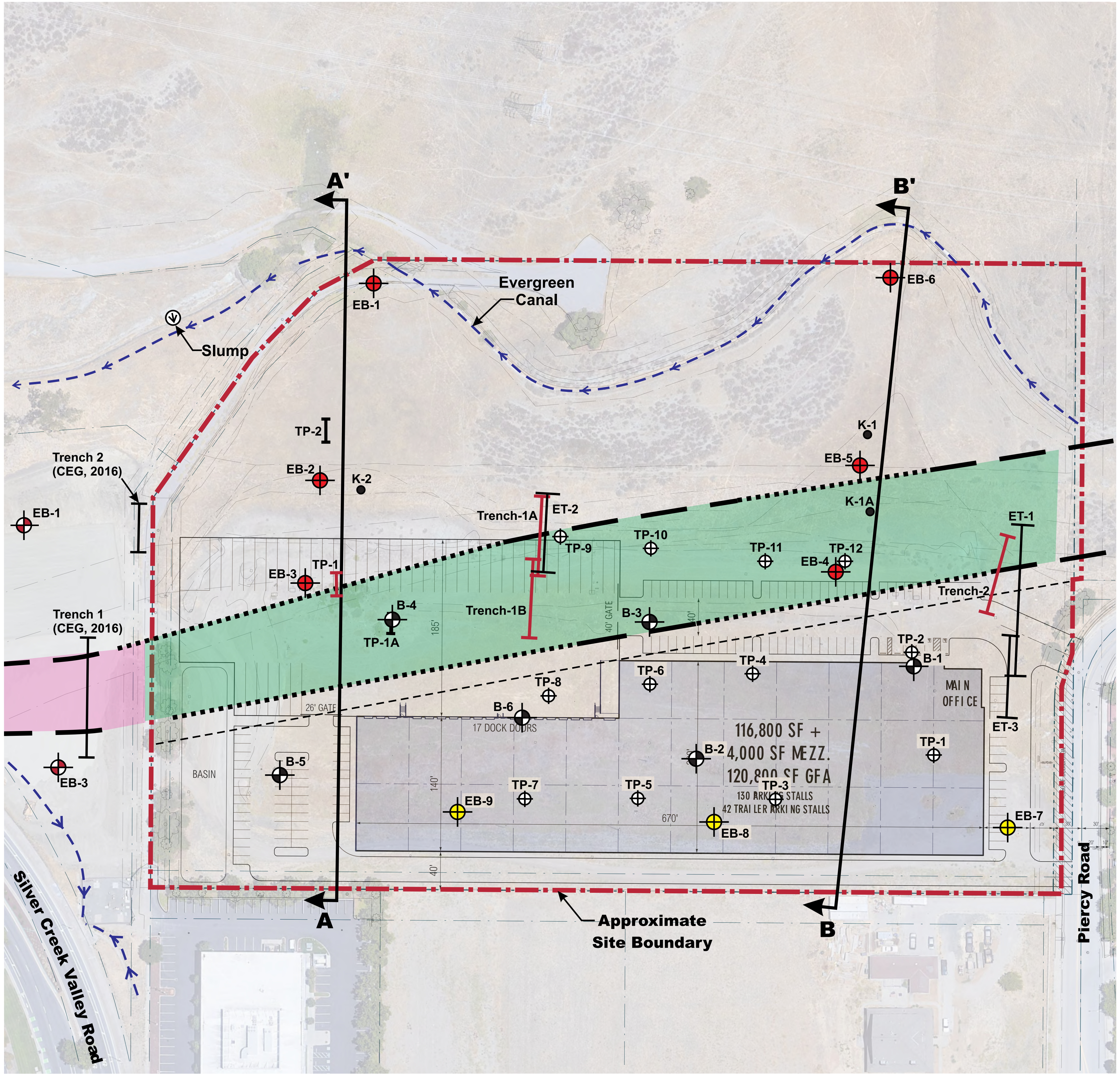
Stereoscopic Aerial Photographs

Date	Scale	Photo ID	Type	Source
09/26/48	1:24,000	GS-HR 2-164, GS-HR 2-165	B/W	USGS
08/23/60	1:30,000	GS-VACY 2-198 GS-VACY 2-199	B/W	USGS
09/28/63	1:20,000	CIV 95 CIV 96	B/W	USGS
05/27/65	1:12,000	SCL 22-174 SCL 22-175	BW	USGS
06/13/68	1:30,000	GS-VBZK 2-93 GS-VBZK 2-94	B/W	USGS
07/12/74	1:20,000	SFB-Area 4 11-169 SFB-Area 4 11-170	Color	USGS
02/22/81	1:24,000	GS-VEZR 3-141 GS-VEZR 3-142	BW	USGS

Note: USGS refers to U.S. Geological Survey, Menlo Park, California



CORNERSTONE EARTH GROUP	Vicinity Map	Project Number 1270-1-1
	455 Piercy Road San Jose, CA	Figure Number Figure 1
		Date March 2021 / Drawn By RRN



References for Legend

1. Fault Investigation, Silver Creek Valley Road Development, 5880 Hellyer Avenue, San Jose, California, Cornerstone Earth Group, December 2015.
2. Fault Investigation, 455 Piercy Road Fault Investigation, San Jose, CA, Cornerstone Earth Group, November 2016.

Legend

- Approximate location of exploratory boring (EB) (Cornerstone, 2021)
- Approximate location of exploratory boring (EB) (Cornerstone, 2017)
- Approximate location of exploratory boring (EB) (Cornerstone, 2016)
- Approximate location of test pit/trench (TP) (Cornerstone, 2016)
- Drainage ditch (< flow)
- 25 ft setback
- Approximate location of trench (Hydro-Geo Consultants, Inc., June 2000)
- Approximate location of test pit (ICES, 2021)
- Approximate location of soil boring (AES, 2000)
- Approximate location of soil boring (Kleinfelder, 2000)
- Approximate location of test pit (Kleinfelder, 2000)
- A A' Approximate location of cross section

Piercy fault zone
1
2

N

0 120 240

APPROXIMATE SCALE (FEET)

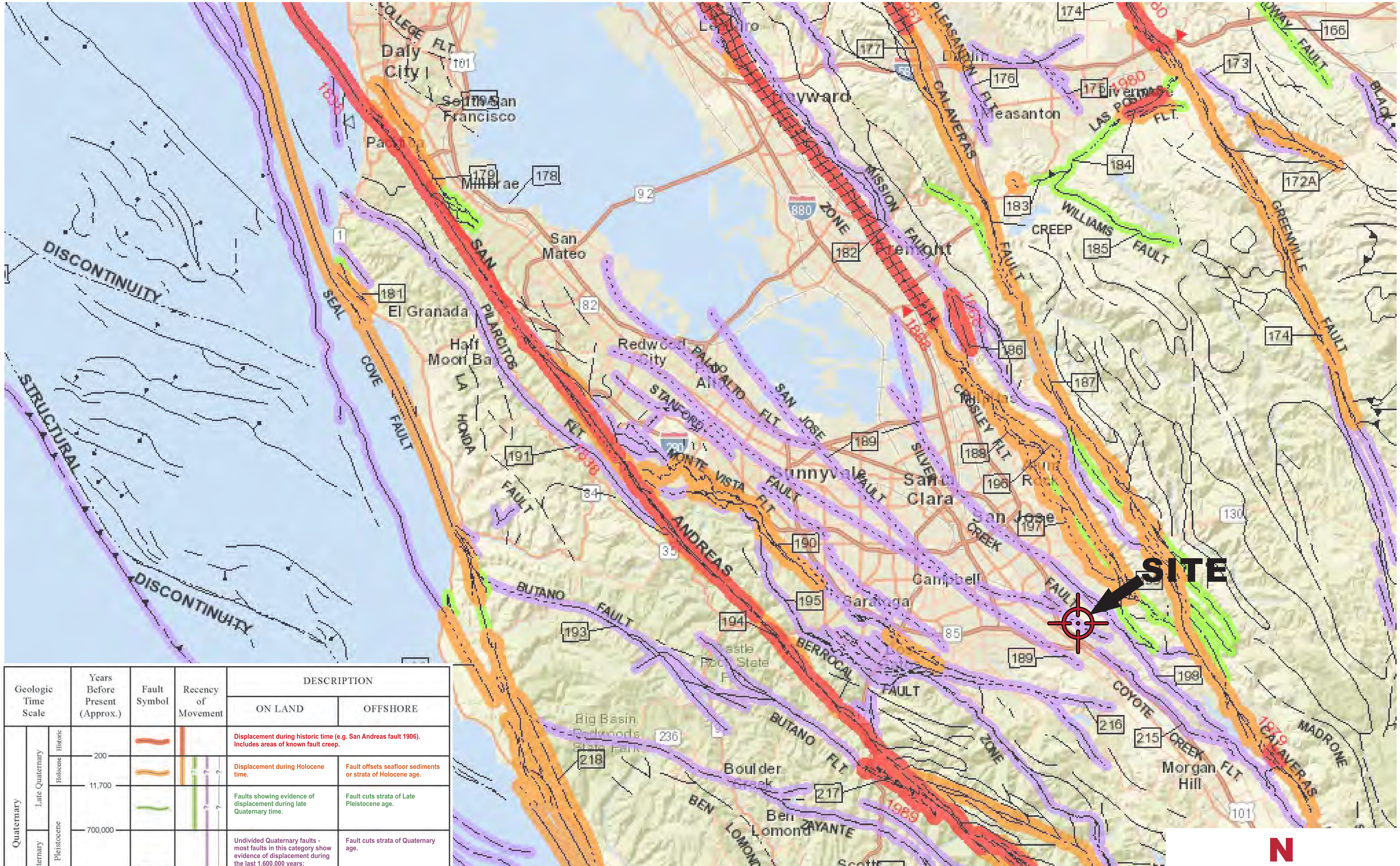
Base by Google Earth, dated 06/19/2019
Overlay by RGA, Preliminary Site Plan - Scheme 01 - A1-01, dated 04/12/2020

Project Number 1270-1-1
Figure Number Figure 2
Drawn By RRN
Date April 2021

Site Plan

455 Piercy Road
San Jose, CA

CORNERSTONE EARTH GROUP



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION: ON LAND	DESCRIPTION: OFFSHORE
Quaternary	Late Quaternary	Historic	200			Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	11,700		? ? ?	Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	700,000		? ?	Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		1,600,000		?	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			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 1270-1-1

Figure Number Figure 3

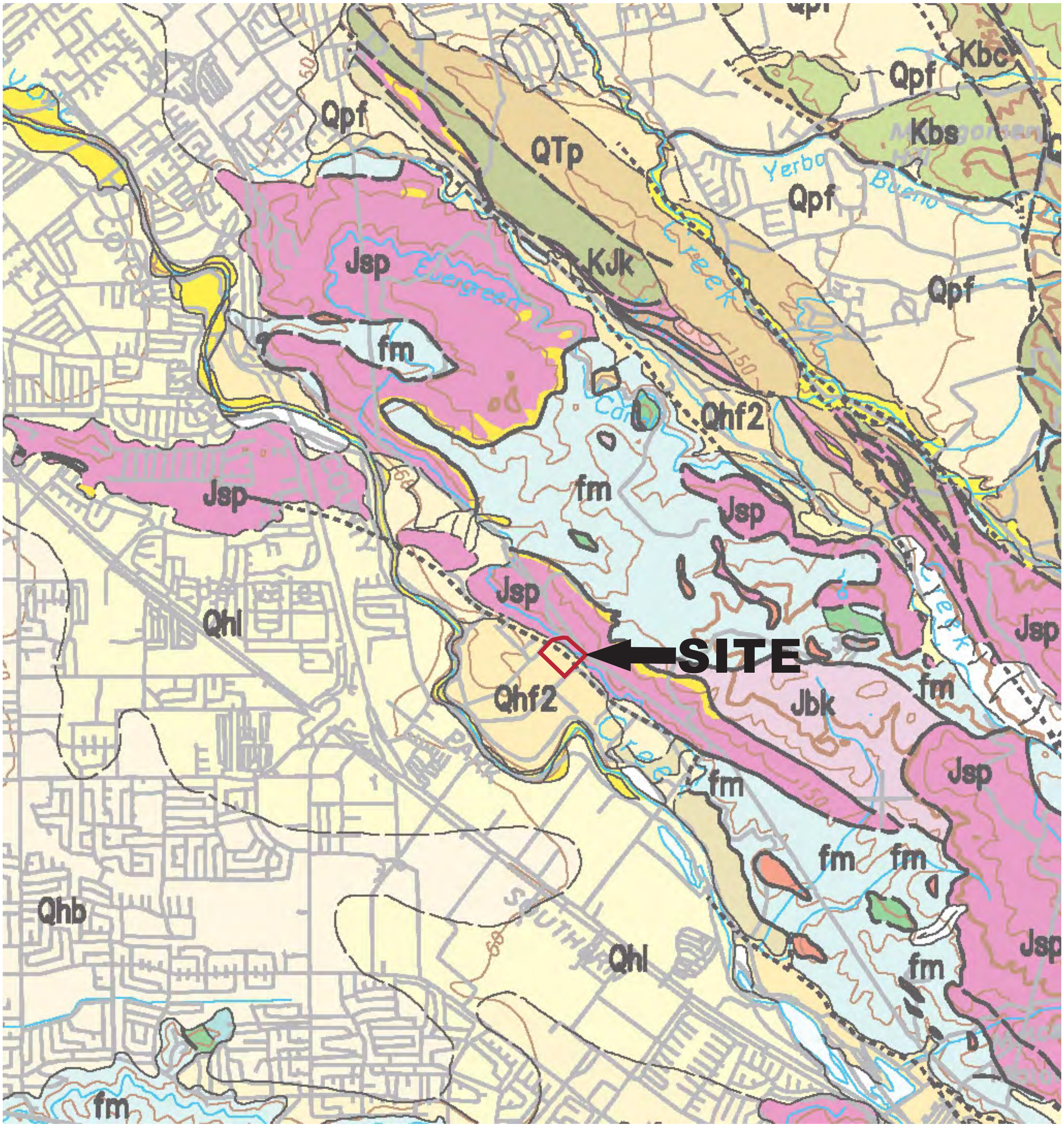
Date March 2021

Drawn By RRN

Regional Fault Map

455 Piercy Road
San Jose, CA

CORNERSTONE EARTH GROUP



Geologic Units

Qht Stream terrace deposits (Holocene)

Qhf2 Older alluvial fan deposits (Holocene)

Jsp Serpentinized harzburgite and dunite

Explanation

Contact - dashed where approximate, dotted where concealed

N

0 4000 8000

APPROXIMATE SCALE (FEET)

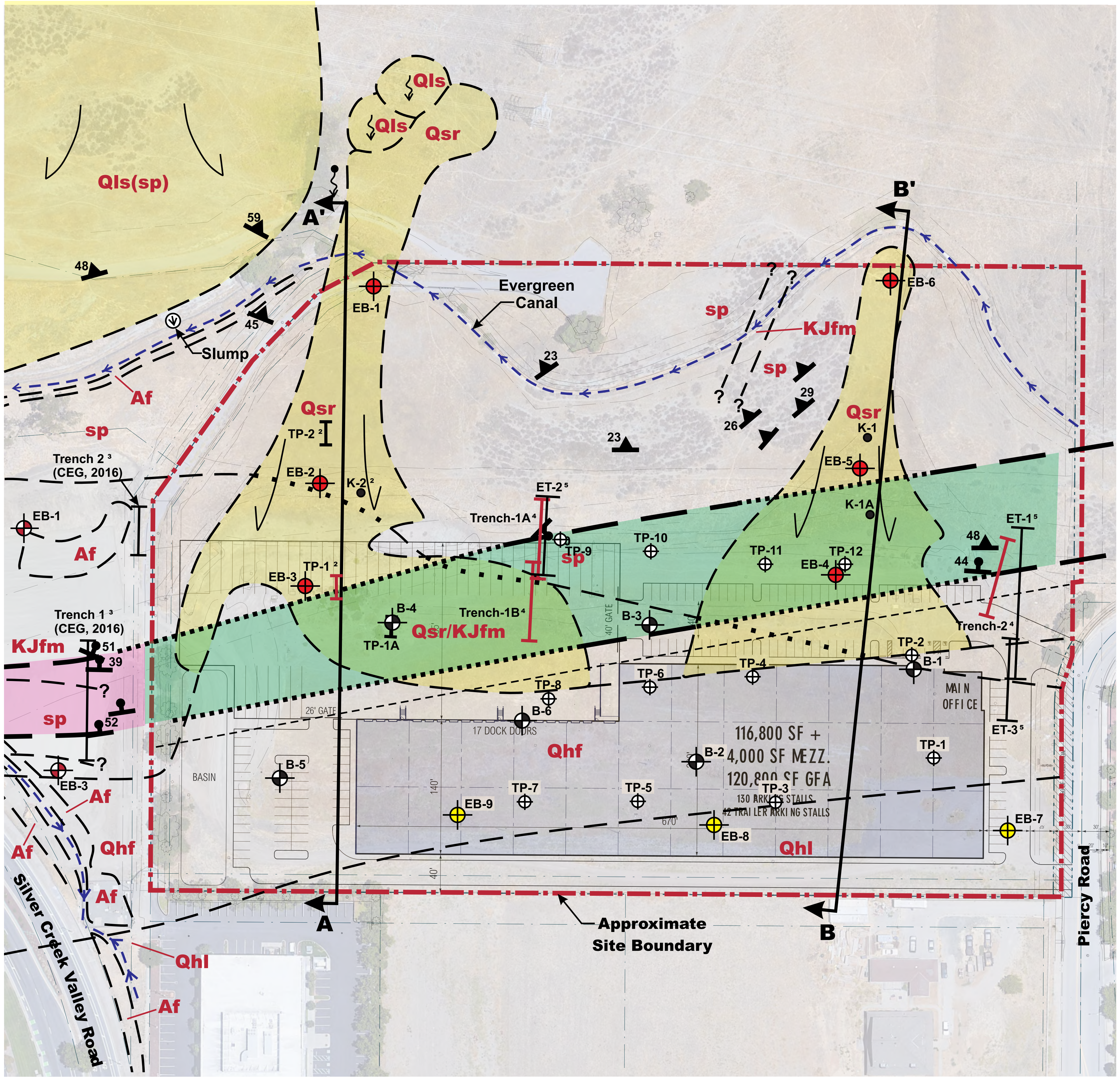
Base: USGS, Preliminary Geologic Map of the San Jose 30x60-Minute Quadrangle, California, compiled by Wentworth, Blake, McLaughlin, and Graymer, 1999

CORNERSTONE EARTH GROUP

Vicinity Geologic Map

455 Piercy Road
San Jose, CA

Project Number	1270-1-1
Figure Number	Figure 4
Date	March 2021
Drawn By	RRN



References for Legend

1. Landslide Inventory Map of the San Jose East Quadrangle Santa Clara County, California, June 2011.
2. Landslide Subsurface Investigation and Slope Stability Analysis, Silver Creek Valley Corporate Center, San Jose, California. Kleinfelder, October 27, 2000.
3. Fault Investigation, Silver Creek Valley Road Development, 5880 Hellyer Avenue, San Jose, California, Cornerstone Earth Group, December 2015.
4. Fault Investigation, 455 Piercy Road Fault Investigation, San Jose, CA, Cornerstone Earth Group, November 2016.
5. Hydro-Geo Consultants, Inc., June 2000, Geologic Evualation Proposed Silver Creek Valley Corporation Center Piercy Road, San Jose, California, Project 3509-A; and September 2000 Supplemental Evualation.

Geologic Units

Af	Artificial fill
Qls	Landslide
Qls(sp)	Dormant landslide (serpentinite) [1]
Qhf	(Holocene) Alluvial fan deposits
Qhl	(Holocene) Alluvial fan levee depsits
Qsr	Slope wash
KJfm	Jurassic Cretaceous Melange of the Franciscan Complex (sandstone and shale)
sp	Jurassic Cretaceous serpentinite

Legend

- Approximate location of exploratory boring (EB) (Cornerstone, 2021)
- Approximate location of exploratory boring (EB) (Cornerstone, 2017)
- Approximate location of exploratory boring (EB) (Cornerstone, 2016)
- Approximate location of test pit/trench (TP) (Cornerstone, 2016)
- Drainage ditch (< flow)
- Landslide/ debris flow [1]
- 25 ft setback
- 55 Shear foliation
- 51 Fault
- Approximate location of test pit (Hydro-Geo Consultants, Inc., June 2000)
- Approximate location of test pit (ICES, 2021)
- Approximate location of soil boring (AES, 2000)
- Approximate location of soil boring (Kleinfelder, 2000)
- Approximate location of test pit (Kleinfelder, 2000)
- A A' Approximate location of cross section
- Piercy fault zone 3 4

N

0 120 240

APPROXIMATE SCALE (FEET)

Base by Google Earth, dated 06/19/2019
Overlay by RGA, Preliminary Site Plan - Scheme 01 - A1-01, dated 04/12/2020

Project Number 1270-1-1

Figure Number Figure 5

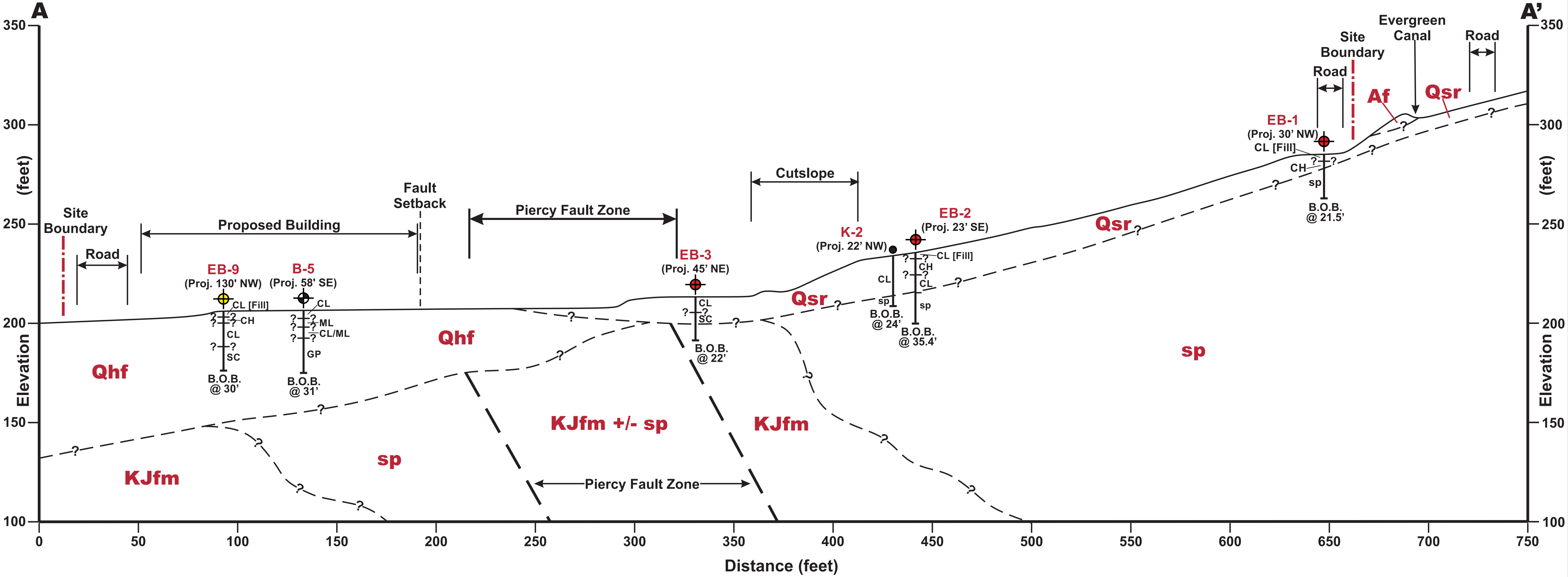
Drawn By RRN

Date April 2021

Site Geologic Map

455 Piercy Road
San Jose, CA

CORNERSTONE EARTH GROUP



Section A-A'

(View Looking Northwest)
1"=50' H:V

Explanation

Geologic Units

Af	Artificial fill
Qhf	(Holocene) alluvial fan deposits
Qsr	Slope wash
KJfm	Jurassic Cretaceous Melange of the Franciscan Complex (sandstone and shale)
sp	Jurassic Cretaceous serpentinite

Symbols

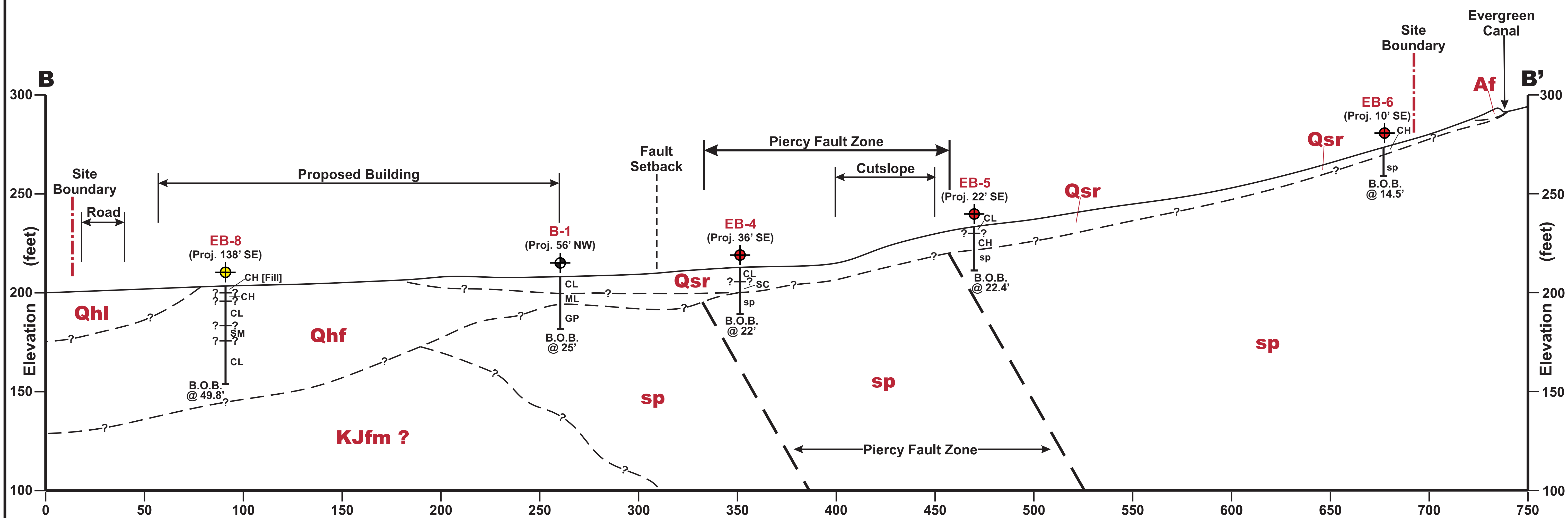
- Approximate location of exploratory boring (EB) (Cornerstone, 2021)
- Approximate location of exploratory boring (EB) (Cornerstone, 2017)
- Approximate location of soil boring (AES, 2000)
- Approximate location of soil boring (Kleinfelder, 2000)

CL	Lean Clay or sandy clay
CH	Fat Clay
SC	Clayey Sand
ML	Silt
GP	Poorly Graded Gravel

Notes:
1) Surficial fills associated with existing pavements, landscaping or utilities are not shown.
2) The subsurface profile is conceptual and is based on limited subsurface data obtained from widely spaced borings. Actual subsurface conditions may vary significantly between borings.
3) See Figure 2 for location of cross section.

CORNERSTONE EARTH GROUP

Geologic Cross Section A-A'	Project Number 1270-1-1
455 Piercy Road San Jose, CA	Figure Number Figure 6
	Date March 2021 / Drawn By RRN



Section B-B'

(View Looking Northwest)
1"=50' H:V

Explanation

Geologic Units

Af Artificial fill

Qhf (Holocene) alluvial fan deposits

Qhl (Holocene) alluvial fan levee deposits

Qsr Slope wash

KJfm Jurassic Cretaceous Melange of the Franciscan Complex (sandstone and shale)

sp Jurassic Cretaceous serpentinite

Symbols

Approximate location of exploratory boring (EB) (Cornerstone, 2021)

Approximate location of exploratory boring (EB) (Cornerstone, 2017)

Approximate location of soil boring (AES, 2000)

CL Lean Clay or sandy clay

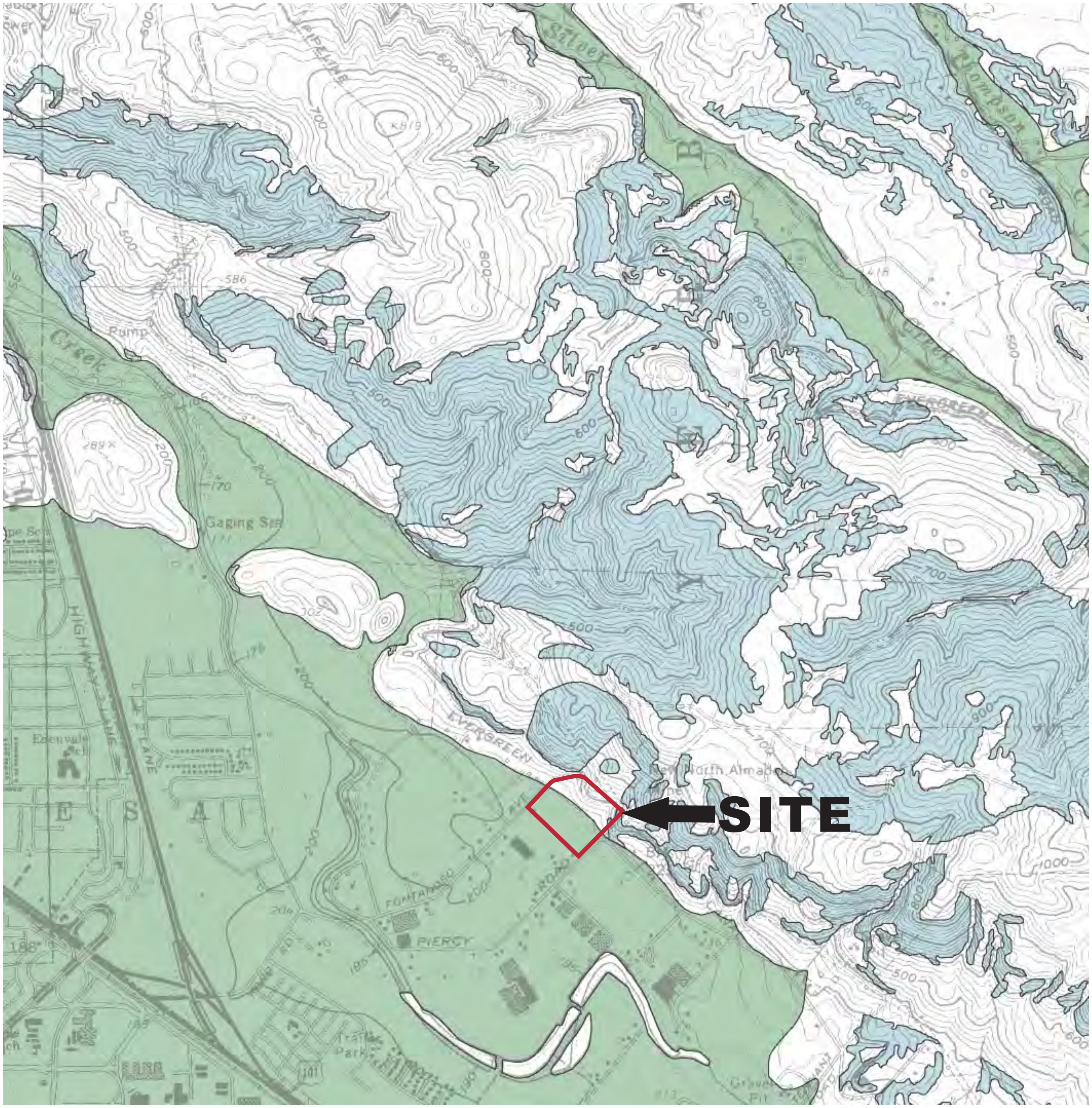
CH Fat Clay

SC Clayey Sand

GP Poorly Graded Gravel

Notes:
1) Surficial fills associated with existing pavements, landscaping or utilities are not shown.
2) The subsurface profile is conceptual and is based on limited subsurface data obtained from widely spaced borings. Actual subsurface conditions may vary significantly between borings.
3) See Figure 2 for location of cross section.

CORNERSTONE EARTH GROUP	Geologic Cross Section B-B'	Project Number 1270-1-1
	455 Piercy Road San Jose, CA	Figure Number Figure 7
		Date March 2021 / Drawn By RRN



Explanation

Liquefaction

Areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslides

Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

N

0 2000 4000

APPROXIMATE SCALE (FEET)

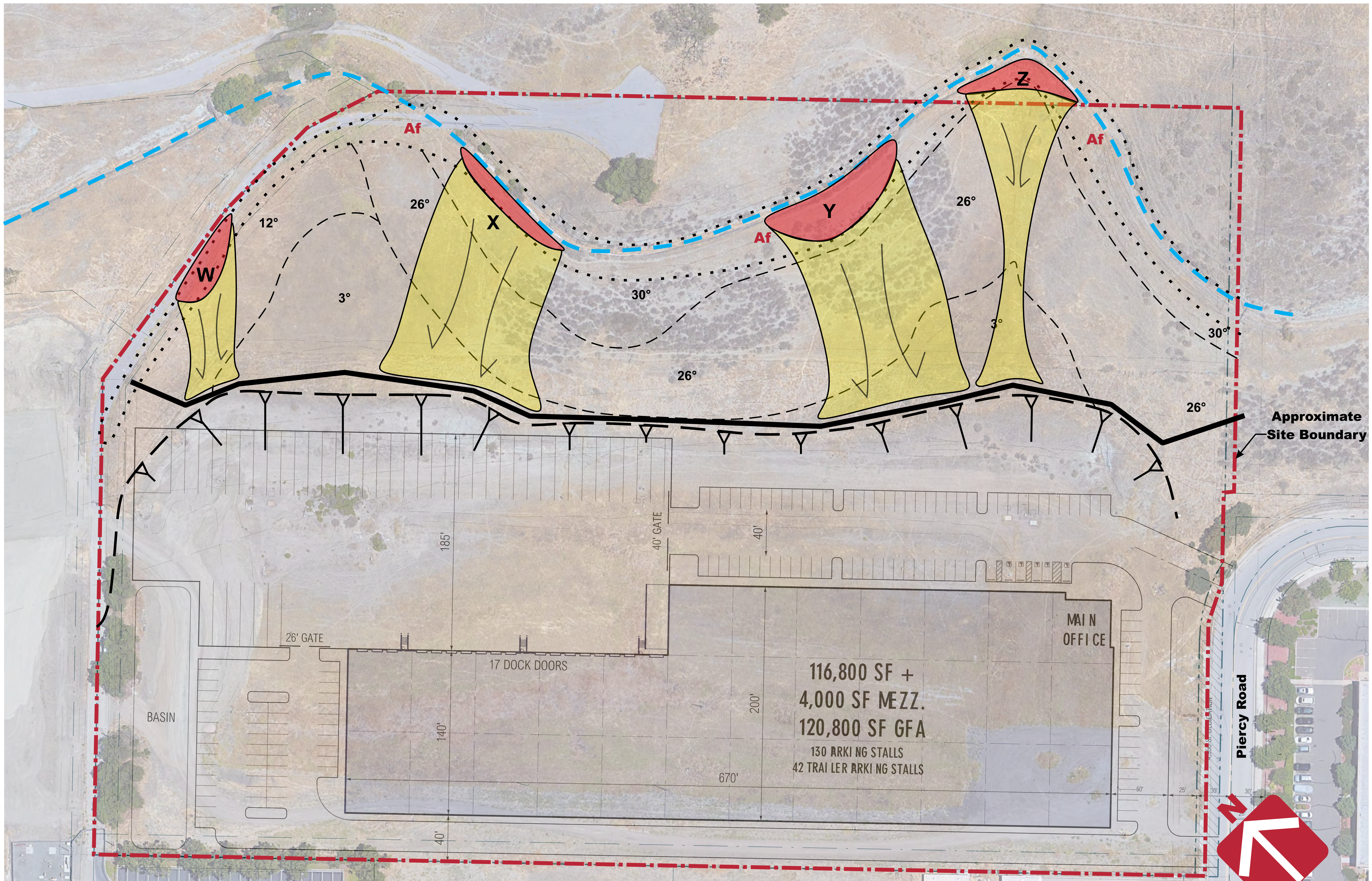
Base: State of California, Seismic Hazard Zones,
San Jose East Quadrangle, released: January 17, 2001

CORNERSTONE EARTH GROUP

Seismic Hazard Map

455 Piercy Road
San Jose, CA

Project Number	1270-1-1
Figure Number	Figure 8
Date	March 2021
Drawn By	RRN



Geologic Units

Af Artificial fill

Legend

26° Area of approximate similar slope

Engineered cut slope

X Hypothetical failure

Hypothetical debris flow

Proposed debris flow wall

0 80 160

APPROXIMATE SCALE (FEET)

Base by Google Earth, dated 06/19/2019
Overlay by RGA, Preliminary Site Plan - Scheme 01 - A1-01, dated 04/12/2020

Project Number 1270-1-1

Figure Number Figure 9

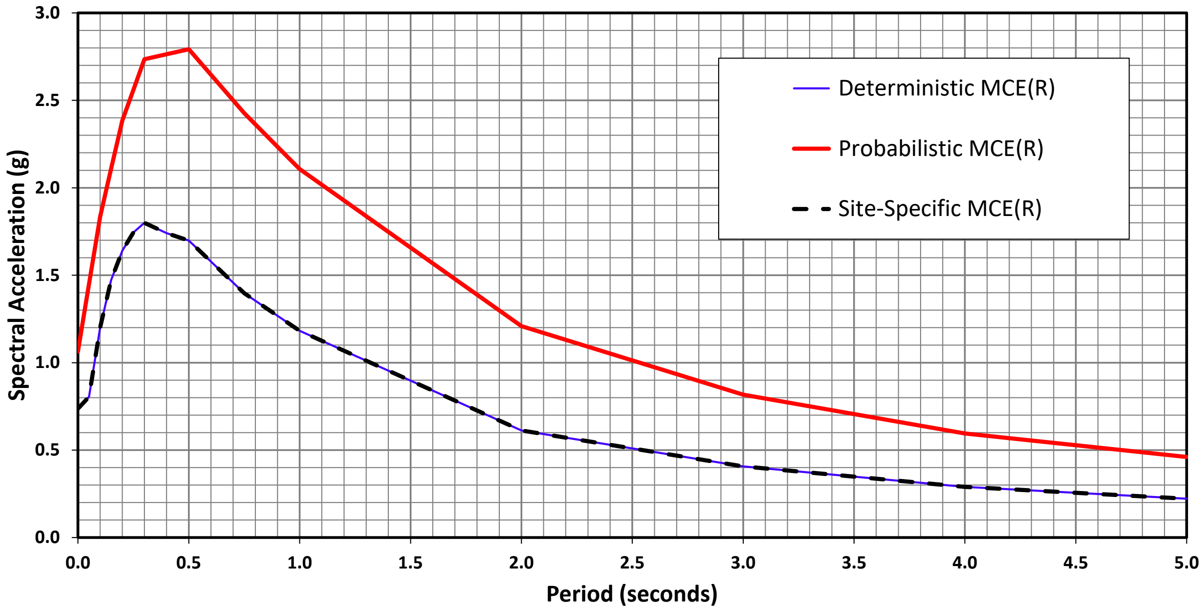
Drawn By RRN

Date April 2021

Debris Flow Analysis Map

455 Piercy Road
San Jose, CA

CORNERSTONE EARTH GROUP



The Site-Specific Maximum Considered Earthquake (MCE_R) is defined as the lesser of the following at all periods:

- Deterministic MCE_R – maximum 84th percentile deterministic, or
- Probabilistic MCE_R – defined as the 2,475–year ground motion.

Site-Specific MCE_R	
Period (Seconds)	Spectral Acceleration (g)
0.00	0.737
0.05	0.805
0.10	1.201
0.15	1.472
0.19	1.607
0.20	1.640
0.25	1.745
0.30	1.800
0.40	1.741
0.50	1.698
0.75	1.397
0.95	1.226
1.00	1.182
2.00	0.612
3.00	0.406
4.00	0.289
5.00	0.222

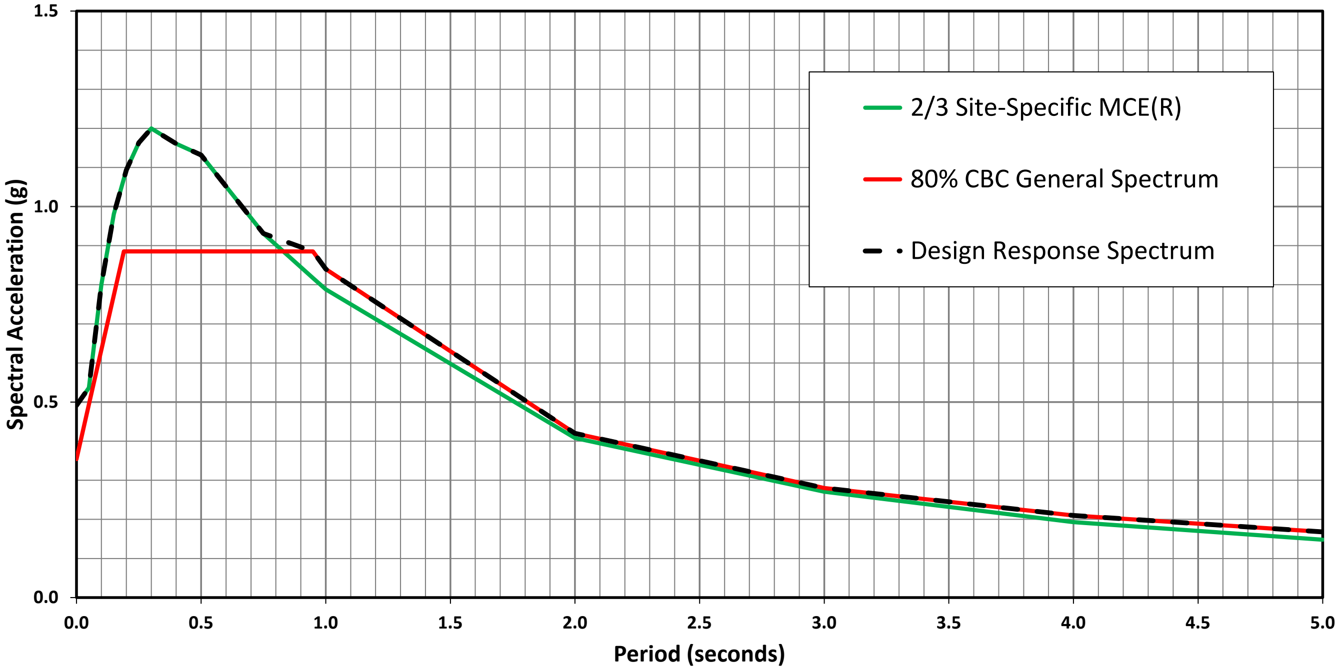
References:

ASCE/SEI 7-16: Minimum Design Loads and Associated Criteria for Buildings and Other Strutures with Supplement No. 1.

2019 California Building Code, Title 24, Part 2, Volume 2

CORNERSTONE EARTH GROUP

MCE_R RESPONSE SPECTRA	FIGURE 10	
455 Piercy Road San Jose, CA	PROJECT NO. 1270-1-1	
	March 16, 2021	NSD



The Site-Specific Design Response Spectrum per Section 21.2, 21.3 and 21.4 of ASCE 7-16 is defined as the greater of the following at all periods:

- 2/3 of the Site-Specific MCE_R, or
- 80% of the CBC General Spectrum.

Design Response Spectra	
Period (Seconds)	Spectral Acceleration (g)
0.00	0.491
0.05	0.537
0.10	0.801
0.15	0.981
0.19	1.071
0.20	1.094
0.25	1.163
0.30	1.200
0.40	1.161
0.50	1.132
0.75	0.931
0.95	0.885
1.00	0.840
2.00	0.420
3.00	0.280
4.00	0.210
5.00	0.168

Site Design	Design Values
Site Class (Per Chapter 20 ASCE 7-16)	D
Shear Wave Velocity, V_{S30} (m/sec)	250
Site Latitude (degrees)	37.260611
Site Longitude (degrees)	-121.780618
Risk Category	II
Building Period (sec)	Unknown
Importance Factor, I_e	1
[1]Site Specific PGA_M (g)	0.67

[1] Lower of Deterministic and Probabilistic, but not less than 80% of mapped value of FM x PGA, determined in accordance with Section 21.5 of ASCE 7-16.

Design Acceleration Parameters[1]	
S_{DS}	1.080
S_{D1}	0.840
S_{MS}	1.620
S_{M1}	1.260

References:

ASCE/SEI 7-16: Minimum Design Loads and Associated Criteria for Buildings and Other Strutures with Supplement No. 1.
2019 California Building Code, Title 24, Part 2, Volume 2

CORNERSTONE EARTH GROUP

DESIGN RESPONSE SPECTRA	FIGURE 11	
455 Piercy Road San Jose, CA	PROJECT NO. 1270-1-1	
	March 16, 2021	NSD

APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using track-mounted, hollow-stem auger, limited-access drilling equipment. Six 6-inch-diameter exploratory borings were drilled on September 11 and 12, 2017 to depths of 14½ to 35½ feet. Three 8-inch-diameter exploratory borings were drilled using truck-mounted, hollow-stem auger, drilling equipment on February 1, 2021 to depths of 29 to 50 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, and other site features as references. Boring elevations were based on interpolation of plan contours. The locations and elevations 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	Sand
Clayey Sand	Silt
Sandy Silt	Well Graded Gravelly Sand
Artificial/Undocumented Fill	Gravelly Silt
Poorly-Graded Gravelly Sand	Asphalt
Topsoil	Boulders and Cobble
Well-Graded Gravel with Clay	
Well-Graded Gravel with Silt	

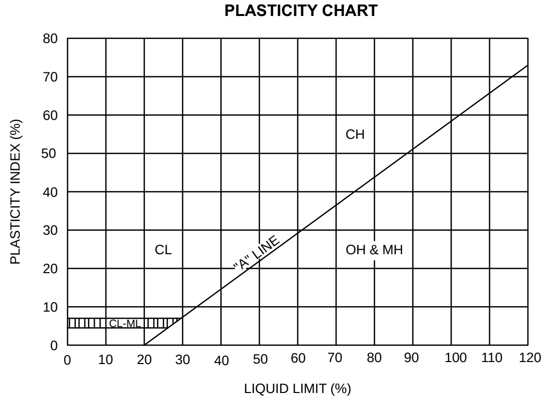
SAMPLER TYPES

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	**LEGEND TO SOIL DESCRIPTIONS**	Figure Number A-1

HARDNESS

1. **Soft** – Reserved for plastic material alone.
2. **Low hardness** – Can be gouged deeply or carved easily with a knife blade.
3. **Moderately hard** – Can be readily scratched by a knife blade: scratch leaves a heavy trace of dust and is readily visible after the powder has been blown away.
4. **Hard** – Can be scratched with difficulty: scratch produces little powder and is often faintly visible.
5. **Very hard** – Cannot be scratched with knife blade: leaves a metallic streak.

STRENGTH

1. **Plastic** or very low strength.
2. **Friable –** Crumbles easily by rubbing with fingers.
3. **Weak –** An unfractured specimen of such material will crumble under light hammer blows.
4. **Moderately strong –** Specimen will withstand a few heavy hammer blows before breaking.
5. **Strong –** Specimen will withstand a few heavy ringing blows and will yield with difficulty only dust and small flying fragments.
6. **Very strong –** Specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.

WEATHERING – The physical and chemical disintegration and decomposition of rocks and minerals by natural processes such as oxidation, reduction, hydration, solution, carbonation, and freezing and thawing.

- **D. Deep –** Moderate to complete mineral decomposition: extensive disintegration: deep and thorough discoloration: many fractures, all extensively coated or filled with oxides, carbonates and/or clay or silt.
- **M. Moderate –** Slight change or partial decomposition of minerals: little disintegration: cementation little to unaffected. Moderate to occasionally intense discoloration. Moderately coated fractures.
- **L. Little –** No megascopic decomposition of minerals: little or no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains or fracture surfaces.
- **F. Fresh –** Unaffected by weathering agents. No disintegration or discoloration. Fractures usually less numerous than joints.

FRACTURING

Intensity	**Size of Pieces in Feet**
Very little fractured	Greater than 4.0
Occasionally fractured	1.0 to 4.0
Moderately fractured	0.5 to 1.0
Closely fractured	0.1 to 0.5
Intensely fractured	0.05 to 0.1
Crushed	Less than 0.05

BEDDING OF SEDIMENTARY ROCKS

Splitting Property	**Thickness**	**Stratification**
Massive	Greater than 4.0 feet	very thick-bedded
Blocky	2.0 to 4.0 feet	thick-bedded
Slabby	0.2 to 2.0 feet	thin-bedded
Flaggy	0.05 to 0.2 feet	very thin-bedded
Shaly or Platy	0.01 to 0.05 feet	laminated
Papery	less than 0.01 feet	thinly laminated

BORING NUMBER EB-1

CORNERSTONE EARTH GROUP

PROJECT NAME 455 Piercy Road
PROJECT NUMBER 920-1-2
PROJECT LOCATION San Jose, CA
DATE STARTED 9/11/17 **DATE COMPLETED** 9/11/17
GROUND ELEVATION 287 FT +/- **BORING DEPTH** 21.5 ft.
DRILLING CONTRACTOR Britton Exploration, Inc.
LATITUDE 37.2618° **LONGITUDE** -121.7805°
DRILLING METHOD CME 55 Track Rig, 6 inch solid stem auger
GROUND WATER LEVELS:
LOGGED BY CSH
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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

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

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
287.0	0	**Sandy Lean Clay (CL) [Fill]** hard, moist, dark gray with brown mottles, fine to medium sand, some fine to coarse subangular to subrounded gravel, moderate plasticity	24	MC-2	94	19			>4.5
284.5		**Fat Clay with Sand (CH)** hard to very stiff, moist, dark brown, fine to medium sand, trace fine subangular gravel, high plasticity	5	MC-4	77	29			>4.5
	5		9	MC-5	77	34			3.0
280.0		**Serpentinite [sp]** low hardness, weak, deep weathering, greenish gray to gray, disintigrated	21	SPT-6		25			
	10		39	SPT-7		14			
	15		27	SPT-8		23			
	20		36	SPT-9		21			
265.5		Bottom of Boring at 21.5 feet.							
	25								
	30								
	35								

BORING NUMBER EB-2

CORNERSTONE EARTH GROUP

PROJECT NAME 455 Piercy Road
PROJECT NUMBER 920-1-2
PROJECT LOCATION San Jose, CA
DATE STARTED 9/11/17 **DATE COMPLETED** 9/11/17
GROUND ELEVATION 232 FT +/- **BORING DEPTH** 35.4 ft.
DRILLING CONTRACTOR Britton Exploration, Inc.
LATITUDE 37.2615° **LONGITUDE** -121.7811°
DRILLING METHOD CME 55 Track Rig, 6 inch solid stem auger
GROUND WATER LEVELS:
LOGGED BY CSH
∇ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION
232.0	0	**Sandy Lean Clay (CL) [Fill]** hard, moist, dark gray with brown mottles, fine to medium sand, some fine to coarse subangular to subrounded gravel, moderate plasticity
229.5		**Fat Clay with Sand (CH)** hard to very stiff, moist, dark brown, fine to medium sand, trace fine subangular gravel, high plasticity
221.0		**Sandy Lean Clay (CL)** hard to very stiff, moist, brown, fine to medium sand, some fine to coarse subangular to subrounded gravel, moderate plasticity
212.5		**Serpentinite [sp]** low hardness, weak, deep weathering, greenish gray to gray, totally decomposed
208.5		**Serpentinite [sp]** moderately hard, weak to moderately strong, deep to moderate weathering, greenish gray to gray, with mafic inclusions
196.6		Bottom of Boring at 35.4 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
12	MC-2		25			>4.5 (hand penetrometer)
16	MC-3	80	35			>4.5 (hand penetrometer)
18	SPT-4		35			>4.5 (hand penetrometer)
14	SPT-5		31			
20	SPT-6		28			>4.5 (hand penetrometer)
24	SPT-7		26			
15	SPT-8		13			
20	SPT-9		11			
50/6"	SPT-10		9			
50/5"	SPT-11		13			

Legend: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL. Scale: 1.0, 2.0, 3.0, 4.0

BORING NUMBER EB-3

CORNERSTONE EARTH GROUP

PROJECT NAME 455 Piercy Road
PROJECT NUMBER 920-1-2
PROJECT LOCATION San Jose, CA

DATE STARTED 9/12/17 **DATE COMPLETED** 9/12/17
GROUND ELEVATION 207 FT +/- **BORING DEPTH** 22 ft.
DRILLING CONTRACTOR Britton Exploration, Inc.
LATITUDE 37.2612° **LONGITUDE** -121.7815°
DRILLING METHOD CME 55 Track Rig, 6 inch solid stem auger
GROUND WATER LEVELS:
LOGGED BY CSH
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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

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

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
207.0	0	**Sandy Lean Clay (CL)** hard, moist, brown with olive gray mottles, fine to medium sand, some fine subangular gravel, moderate plasticity. Liquid Limit = 43, Plastic Limit = 24							
			25	MC-1	102	18	19		>4.5 ○
			58	MC-4	99	21			>4.5 ○
	5		60	SPT-5		20			>4.5 ○
200.0		**Clayey Sand with Gravel (SC)** very dense, moist, gray and brown mottled, fine to medium sand, fine to coarse subangular to subrounded gravel, some cobbles	50/3"	SPT-6		11			
	10		50/5"	SPT-7		7			
			50/6"	SPT-8		7			
193.0	15	**Graywacke and Shale [KJfm]** moderately hard, moderately strong, little to moderate weathering, gray							
			50/6"	SPT-9		8			
	20	locally intensely sheared	29	SPT-10		12			
185.0			50/6"	SPT-11		15			
		Bottom of Boring at 22.0 feet.							
	25								
	30								
	35								

BORING NUMBER EB-4

CORNERSTONE EARTH GROUP

PROJECT NAME 455 Piercy Road
PROJECT NUMBER 920-1-2
PROJECT LOCATION San Jose, CA

DATE STARTED 9/11/17 **DATE COMPLETED** 9/11/17
GROUND ELEVATION 210 FT +/- **BORING DEPTH** 22 ft.
DRILLING CONTRACTOR Britton Exploration, Inc.
LATITUDE 37.2602° **LONGITUDE** -121.7800°
DRILLING METHOD CME 55 Track Rig, 6 inch solid stem auger
GROUND WATER LEVELS:
LOGGED BY CSH
AT TIME OF DRILLING Not Encountered
NOTES
AT END OF DRILLING Not Encountered

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

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

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
210.0	0	**Sandy Lean Clay (CL)** hard, moist, brown, fine to medium sand, some fine to coarse subangular gravel, moderate plasticity	52	MC-2	91	24			>4.5 ○
			45	MC-4	95	23			>4.5 ○
	5		36	SPT-5		20			>4.5 ○
203.0		**Clayey Sand with Gravel (SC)** very dense to medium dense, moist, gray and brown mottled, fine to medium sand, fine to coarse subangular gravel, some cobbles	51	SPT-6		21			
	10		41	MC-7	105	16			
			60	SPT-8		19			
197.0		**Serpentinite [sp]** moderately hard, weak to moderately strong, deep to moderate weathering, greenish gray to gray, with mafic inclusions							
	15		50/4"	SPT-9		14			
	20		50/5"	SPT-10		33			
			53	SPT-11		47			
188.0		Bottom of Boring at 22.0 feet.							
	25								
	30								
	35								

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 10/31/17 11:18 - P:\DRAFTING\GINT FILES\920-1-2 PIERCY RD.GPJ

BORING NUMBER EB-5

CORNERSTONE EARTH GROUP

PROJECT NAME 455 Piercy Road

PROJECT NUMBER 920-1-2

PROJECT LOCATION San Jose, CA

DATE STARTED 9/11/17 **DATE COMPLETED** 9/11/17

GROUND ELEVATION 234 FT +/- **BORING DEPTH** 22.4 ft.

DRILLING CONTRACTOR Britton Exploration, Inc.

LATITUDE 37.2603° **LONGITUDE** -121.7797°

DRILLING METHOD CME 55 Track Rig, 6 inch solid stem auger

GROUND WATER LEVELS:

LOGGED BY CSH

▽ **AT TIME OF DRILLING** Not Encountered

NOTES

▼ **AT END OF DRILLING** Not Encountered

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

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION
234.0	0	**Sandy Lean Clay (CL)** hard, moist, dark brown, fine to medium sand, some fine subangular gravel, moderate plasticity
231.0		**Fat Clay with Sand (CH)** hard, moist, brown, fine to medium sand, some fine subangular gravel, high plasticity
222.0		**Serpentinite [sp]** moderately hard, weak to moderately strong, deep to moderate weathering, greenish gray to gray, pervasively sheared
211.6		Bottom of Boring at 22.4 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
4	MC-2		20			
10	MC-4	90	28			>4.5
23	MC-6	85	29			>4.5
23	SPT-7		28			>4.5
21	SPT-8		27			>4.5
50 6"	SPT-9		29			
50 3"	SPT-10		44			
50 4"	SPT-11		30			

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

Depth scale (ft): 0, 5, 10, 15, 20, 25, 30, 35

BORING NUMBER EB-6

CORNERSTONE EARTH GROUP

PROJECT NAME 455 Piercy Road
PROJECT NUMBER 920-1-2
PROJECT LOCATION San Jose, CA

DATE STARTED 9/11/17 **DATE COMPLETED** 9/11/17
DRILLING CONTRACTOR Britton Exploration, Inc.
DRILLING METHOD CME 55 Track Rig, 6 inch solid stem auger
LOGGED BY CSH
NOTES

GROUND ELEVATION 277 FT +/- **BORING DEPTH** 14.3 ft.
LATITUDE 37.2606° **LONGITUDE** -121.7794°
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.

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
277.0	0	**Fat Clay with Sand (CH)** hard, moist, dark brown to brown, fine to medium sand, some fine subangular gravel, high plasticity	14	MC-1	83	26			>4.5 ○
			37	SPT-2		25			>4.5 ○
272.5	5	**Serpentinite [sp]** moderately hard, moderately strong, moderate weathering, greenish gray to gray	50/4"	SPT-3		10			
			50/6"	SPT-4		10			
	10		50/6"	SPT-5		7			
262.7			50/6"	SPT-6		3			
	15	Bottom of Boring at 14.3 feet.							
	20								
	25								
	30								
	35								

CORNERSTONE EARTH GROUP

DATE STARTED 2/22/21	DATE COMPLETED 2/22/21
DRILLING CONTRACTOR Exploration Goservices, Inc.	
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger	
LOGGED BY CSH	
NOTES	
PROJECT NAME 455 Piercy Road	
PROJECT NUMBER 1270-1-1	
PROJECT LOCATION San Jose, CA	
GROUND ELEVATION	BORING DEPTH 29.3 ft.
LATITUDE	LONGITUDE
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.

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

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	1 inch aggregate base							
	Fat Clay with Gravel (CH) [Fill] hard, dry to moist, dark brown, fine to coarse subangular gravel, some fine to medium sand, high plasticity; Liquid Limit = 62, Plastic Limit = 26	37	MC-1B	92	18	36		>4.5
	Fat Clay (CH) [Residual soil] hard, dry to moist, dark brown, some fine to medium sand, high plasticity	29	MC-2B	94	11			>4.5
5	**Lean Clay with Sand (CL) [Alluvium]** hard, moist, brown, fine to medium sand, trace fine subangular gravel, moderate plasticity	54	MC-3B	110	9			>4.5
		58	SPT					>4.5
10								
		76	SPT-5		18			>4.5
15								
	Sandy Lean Clay (CL) [Alluvium] hard, moist, brown, fine to medium sand, trace fine subangular gravel, moderate plasticity	50/6"	MC-6B	96	26			>4.5
20								
	Lean Clay with Sand (CL) [Alluvium] hard, moist, brown, fine to medium sand, trace fine subangular gravel, moderate plasticity	50/4"	SPT-7		27			>4.5
25								

Continued Next Page

CORNERSTONE EARTH GROUP

PROJECT NAME 455 Piercy Road

PROJECT NUMBER 1270-1-1

PROJECT LOCATION San Jose, CA

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
			Lean Clay with Sand (CL) [Alluvium] hard, moist, brown, fine to medium sand, trace fine subangular gravel, moderate plasticity	50/3"	MC-8	93	28			>4.5 ○
			Bottom of Boring at 29.3 feet.							
	30									
	35									
	40									
	45									
	50									
	55									

BORING NUMBER EB-8

CORNERSTONE EARTH GROUP

PROJECT NAME 455 Piercy Road
PROJECT NUMBER 1270-1-1
PROJECT LOCATION San Jose, CA

DATE STARTED 2/22/21 **DATE COMPLETED** 2/22/21
GROUND ELEVATION **BORING DEPTH** 49.8 ft.
DRILLING CONTRACTOR Exploration Goservices, Inc.
LATITUDE **LONGITUDE**
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY CSH
▽ **AT TIME OF DRILLING** 25 ft.
NOTES
▼ **AT END OF DRILLING** 21.5 ft.

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

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

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
	0	**Fat Clay with Gravel (CH) [Fill]** hard, dry to moist, dark brown, some fine to medium sand, fine to coarse subangular gravel, high plasticity	68	MC-1B	84	32			>4.5 ○
		Fat Clay (CH) [Residual soil] hard, dry to moist, dark brown, some fine to medium sand, high plasticity	62	2A MC 2B	93 93	21 24			>4.5 ○ ●
	5	**Lean Clay with Sand (CL) [Alluvium]** hard, moist, brown, fine to medium sand, moderate plasticity	50	MC-3B	111	17			>4.5 ○
			38	MC-4B	107	13			>4.5 ○
	10								
			50/6"	MC-5B	98	21			>4.5 ○
	15	**Sandy Lean Clay (CL) [Alluvium]** hard, moist, brown, fine to medium sand, moderate plasticity							
			50	MC-6B	102	16			
	20	**Silty Sand (SM) [Alluvium]** medium dense, moist, brown, fine to medium sand							
	▼								
			28	SPT-7		25		39	
	▽ 25								

Continued Next Page

CORNERSTONE EARTH GROUP

PROJECT NAME 455 Piercy Road
PROJECT NUMBER 1270-1-1
PROJECT LOCATION San Jose, CA

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
		Lean Clay with Sand (CL) [Alluvium] hard, moist, brown, fine to medium sand, trace fine subangular gravel, moderate plasticity							
	30		50/6"	SPT-8		33			>4.5 ○
		Sandy Lean Clay (CL) [Alluvium] hard, moist, brown, fine to medium sand, trace fine subangular gravel, moderate plasticity							
	35		50/6"	SPT-9		26			>4.5 ○
		Lean Clay with Sand (CL) [Alluvium] hard, moist, brown, fine to medium sand, trace fine subangular gravel, moderate plasticity							
	40		50/6"	SPT-10		34			>4.5 ○
			50/5"	SPT-11		32			>4.5 ○
	45	**Sandy Lean Clay (CL) [Alluvium]** hard, moist, brown, fine to medium sand, trace fine subangular gravel, moderate plasticity							
			50/4"	SPT-12		32			>4.5 ○
	50	Bottom of Boring at 49.8 feet.							
	55								

BORING NUMBER EB-9

CORNERSTONE EARTH GROUP

PROJECT NAME 455 Piercy Road
PROJECT NUMBER 1270-1-1
PROJECT LOCATION San Jose, CA

DATE STARTED 2/22/21 **DATE COMPLETED** 2/22/21
GROUND ELEVATION **BORING DEPTH** 30 ft.
DRILLING CONTRACTOR Exploration Goservices, Inc.
LATITUDE **LONGITUDE**
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY CSH
AT TIME OF DRILLING Not Encountered
NOTES
AT END OF DRILLING Not Encountered

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

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	**Fat Clay (CH) [Fill]** hard, dry to moist, dark brown, some fine to medium sand, some fine to coarse subangular gravel, high plasticity	78	MC-1B	95	21			>4.5 (hand penetrometer)
		Fat Clay (CH) [Residual soil] hard, dry to moist, dark brown, some fine to medium sand, high plasticity	61	MC-2B	88	15			>4.5 (hand penetrometer)
	5	**Lean Clay with Sand (CL) [Alluvium]** very stiff, moist, brown, fine to medium sand, moderate plasticity	50	MC-3B	99	12			
			36	4A MC	91	11			unconfined compression
	10			4B	95	13			
			50/6"	MC-5B	96	24			
	15								
		Sandy Lean Clay (CL) [Alluvium] hard, moist, brown, fine to medium sand, trace fine subangular gravel, moderate plasticity	50/4"	MC-6	97	26			
	20								
			50/4"	SPT-7		23			>4.5 (hand penetrometer)
	25								

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

Continued Next Page

CORNERSTONE EARTH GROUP

PROJECT NAME 455 Piercy Road

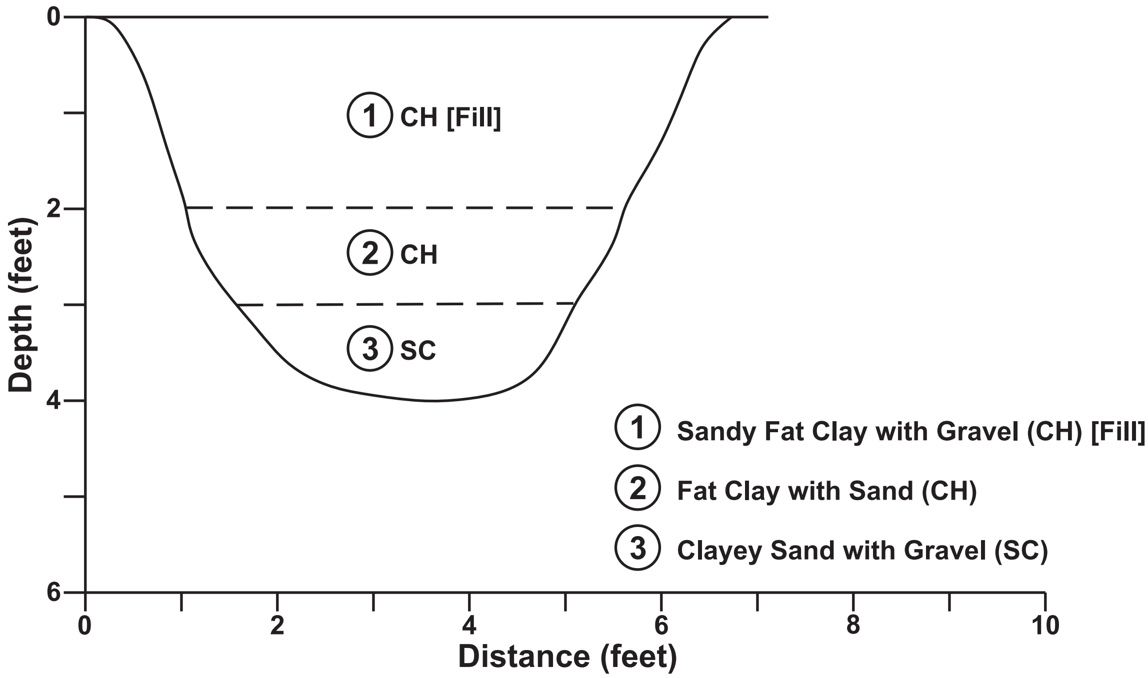
PROJECT NUMBER 1270-1-1

PROJECT LOCATION San Jose, CA

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
			Sandy Lean Clay (CL) [Alluvium] hard, moist, brown, fine to medium sand, trace fine subangular gravel, moderate plasticity	50/6"	MC-8	93	24			
	30		Bottom of Boring at 30.0 feet.							
	35									
	40									
	45									
	50									
	55									

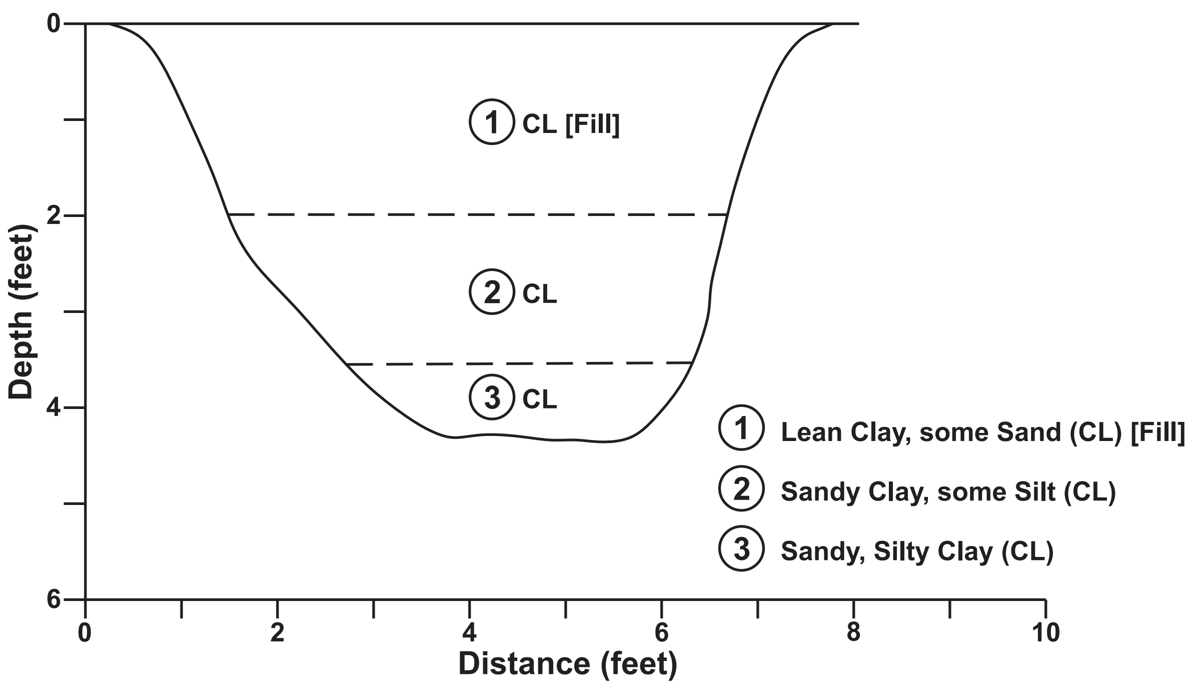
TP-1

(Looking West)



TP-2

(Looking West)

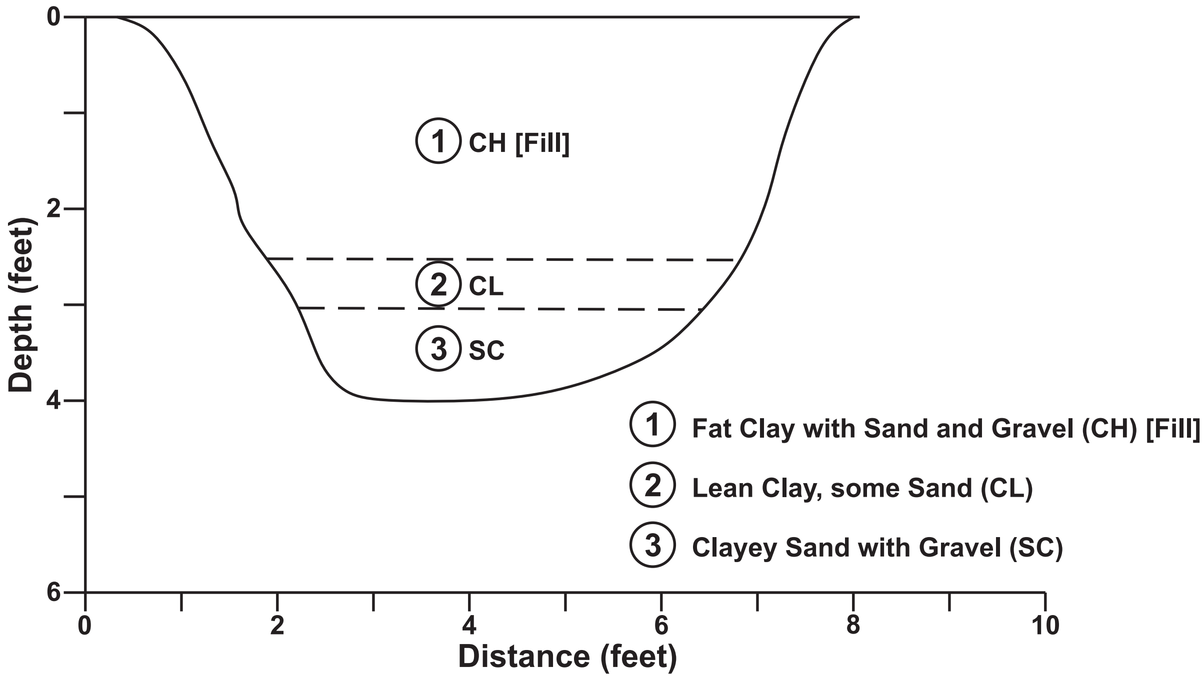


Scale: 1" = 2'

CORNERSTONE EARTH GROUP	Test Pit 1 and 2	Project Number 1270-1-1
	455 Piercy Road San Jose, CA	Figure Number Figure A-1
		Date March 2021 / Drawn By RRN

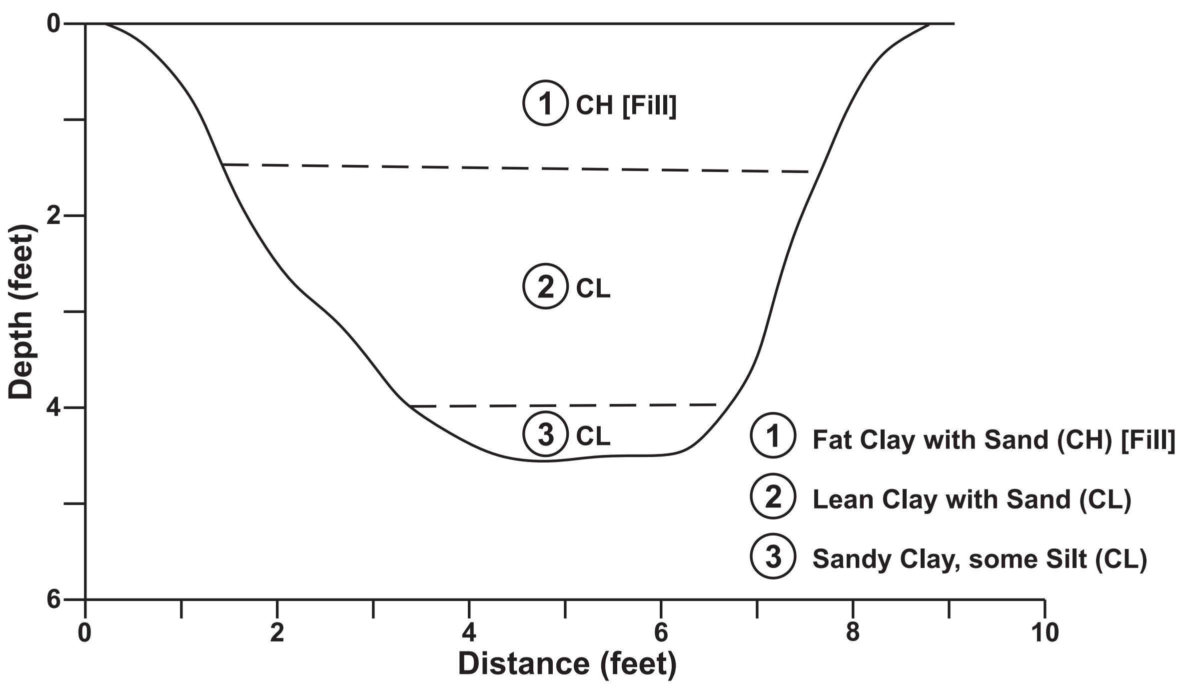
TP-3

(Looking West)



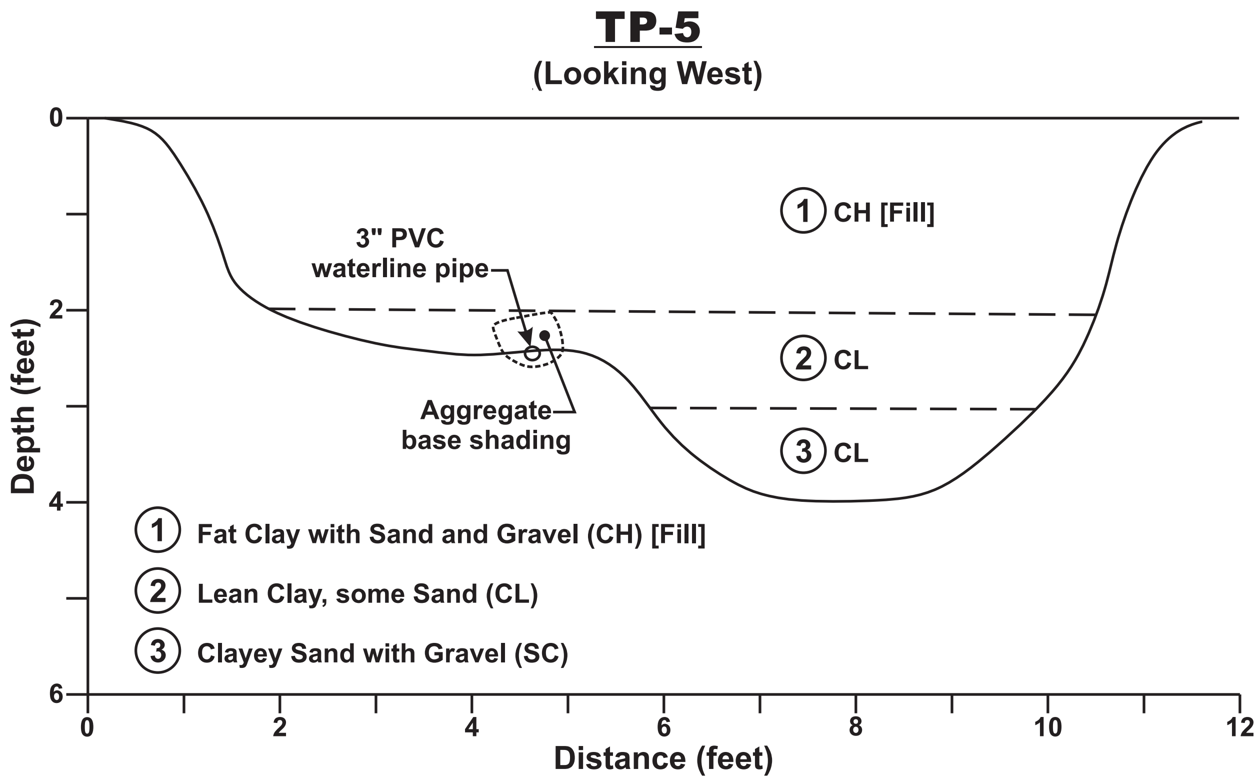
TP-4

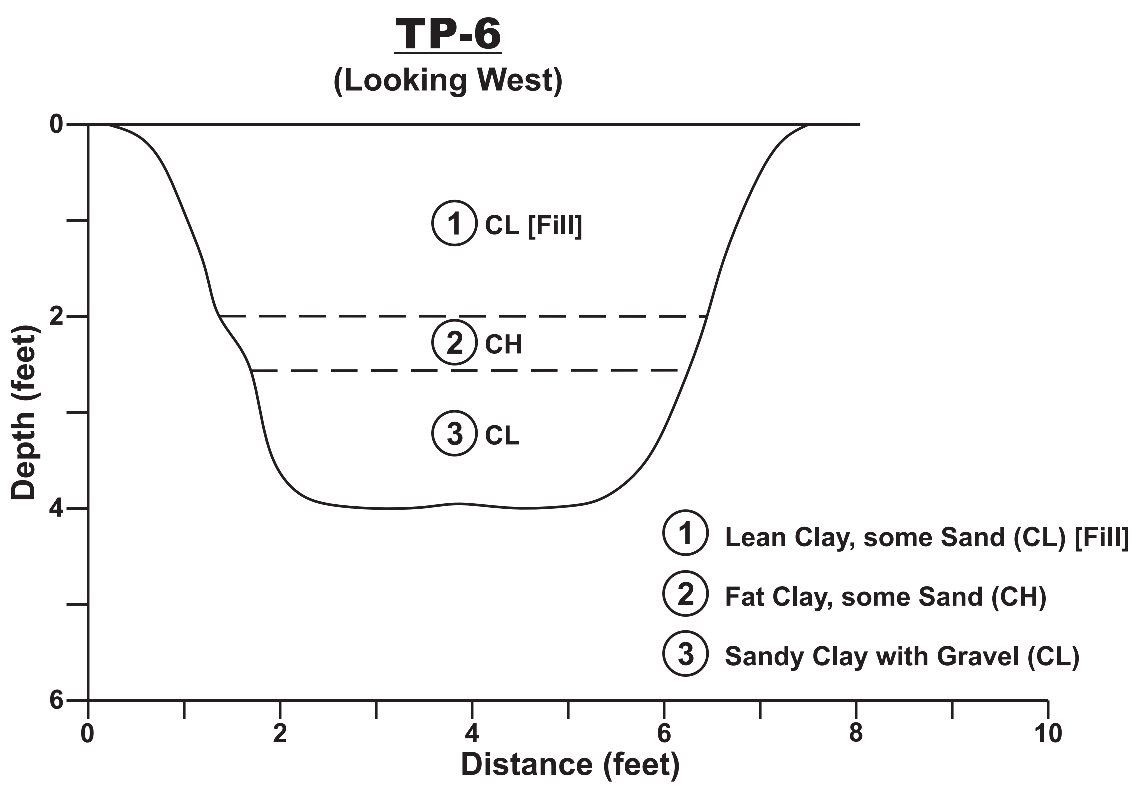
(Looking West)



Scale: 1" = 2'

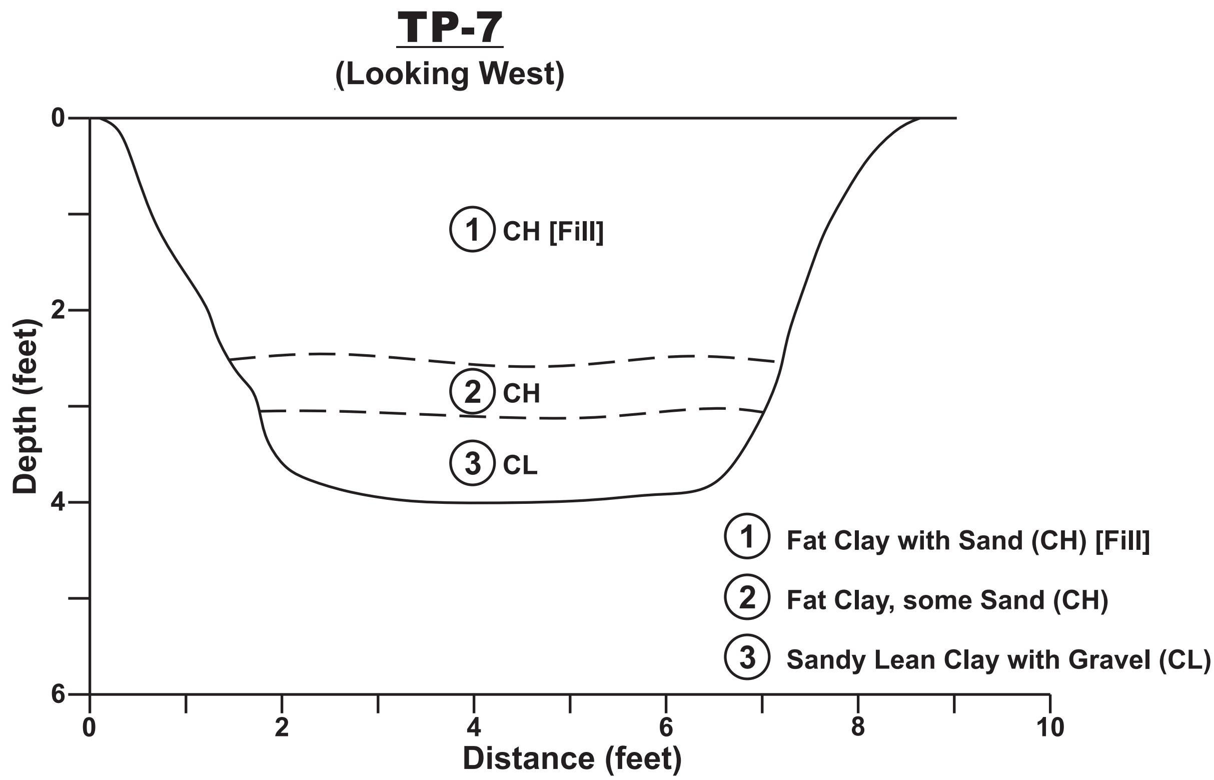
CORNERSTONE EARTH GROUP	Test Pit 3 and 4	Project Number 1270-1-1
	455 Piercy Road San Jose, CA	Figure Number Figure A-2
		Date March 2021 / Drawn By RRN

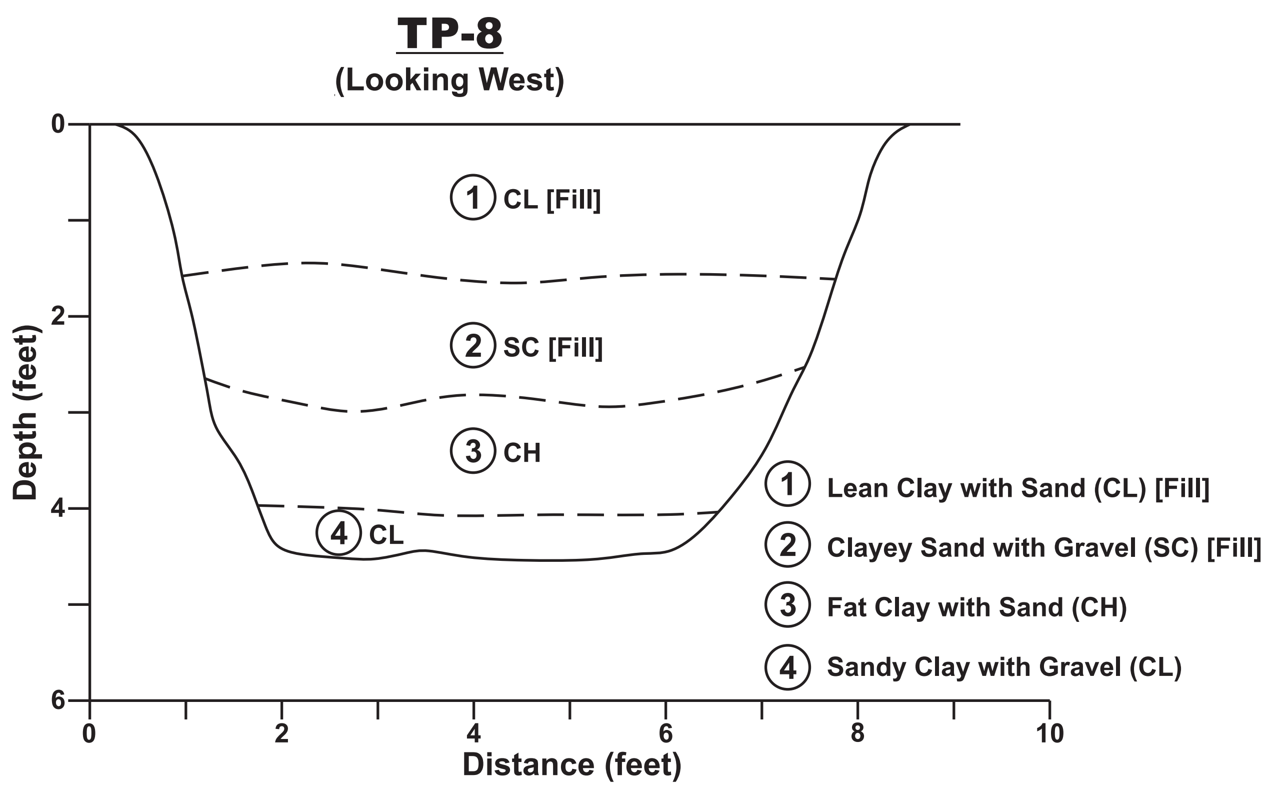




Scale: 1” = 2’

CORNERSTONE EARTH GROUP	Test Pit 5 and 6	Project Number 1270-1-1
	455 Piercy Road San Jose, CA	Figure Number Figure A-3
		Date March 2021 / Drawn By RRN





Scale: 1" = 2'

CORNERSTONE EARTH GROUP

Test Pit 7 and 8

455 Piercy Road
San Jose, CA

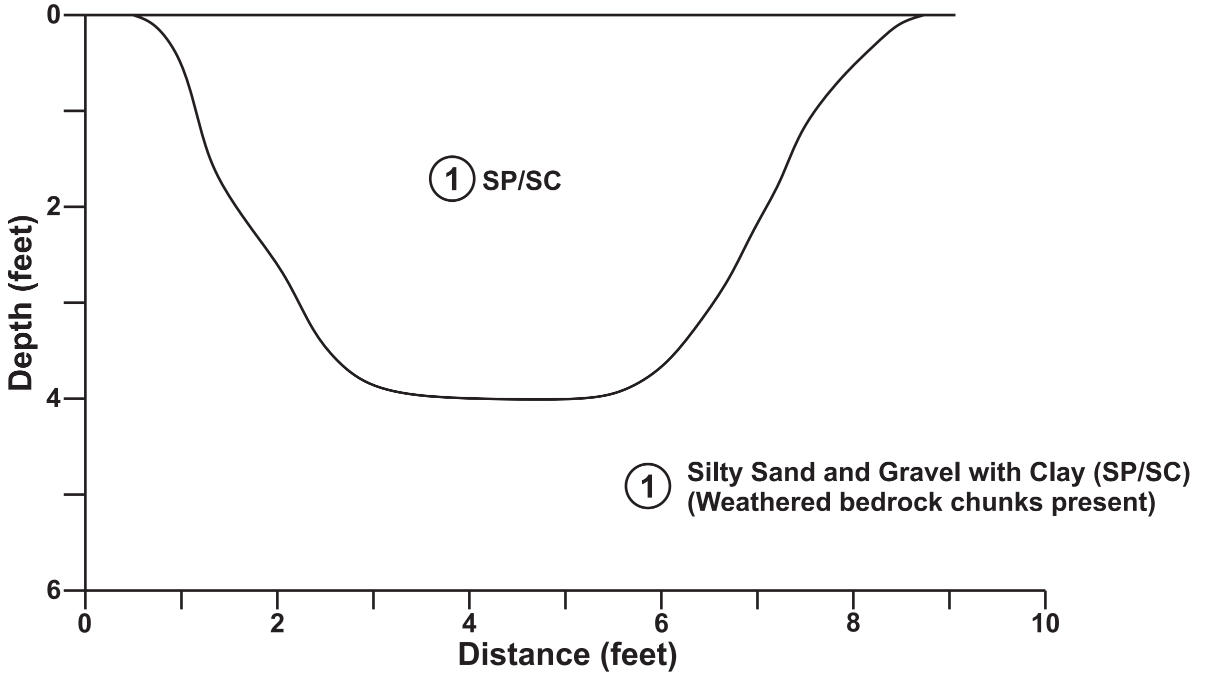
Project Number 1270-1-1

Figure Number Figure A-4

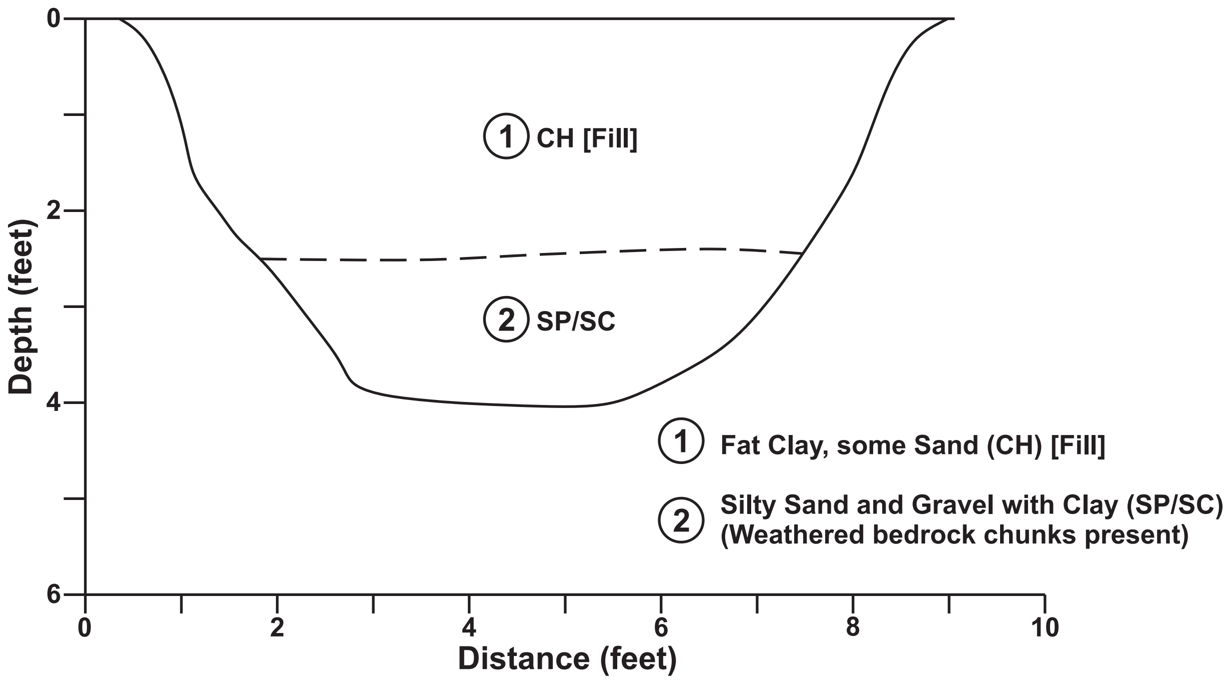
Date March 2021

Drawn By RRN

TP-9
(Looking West)

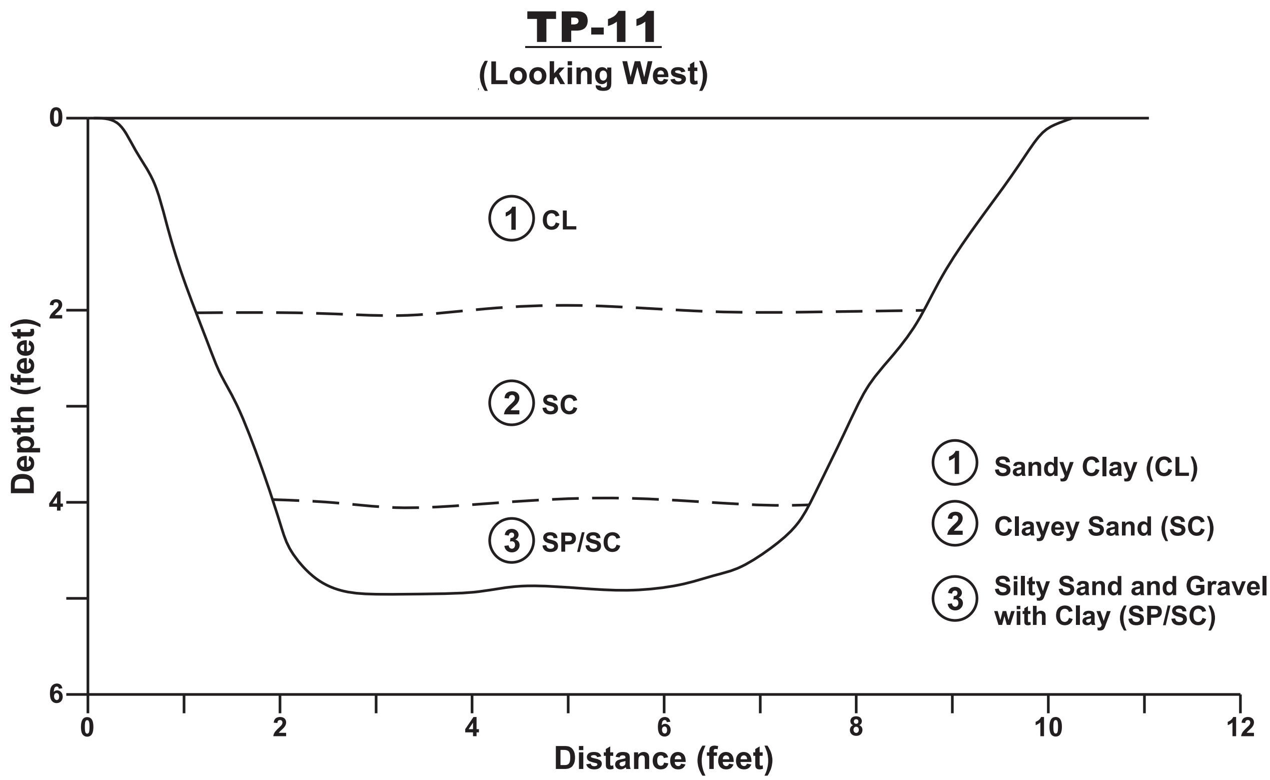


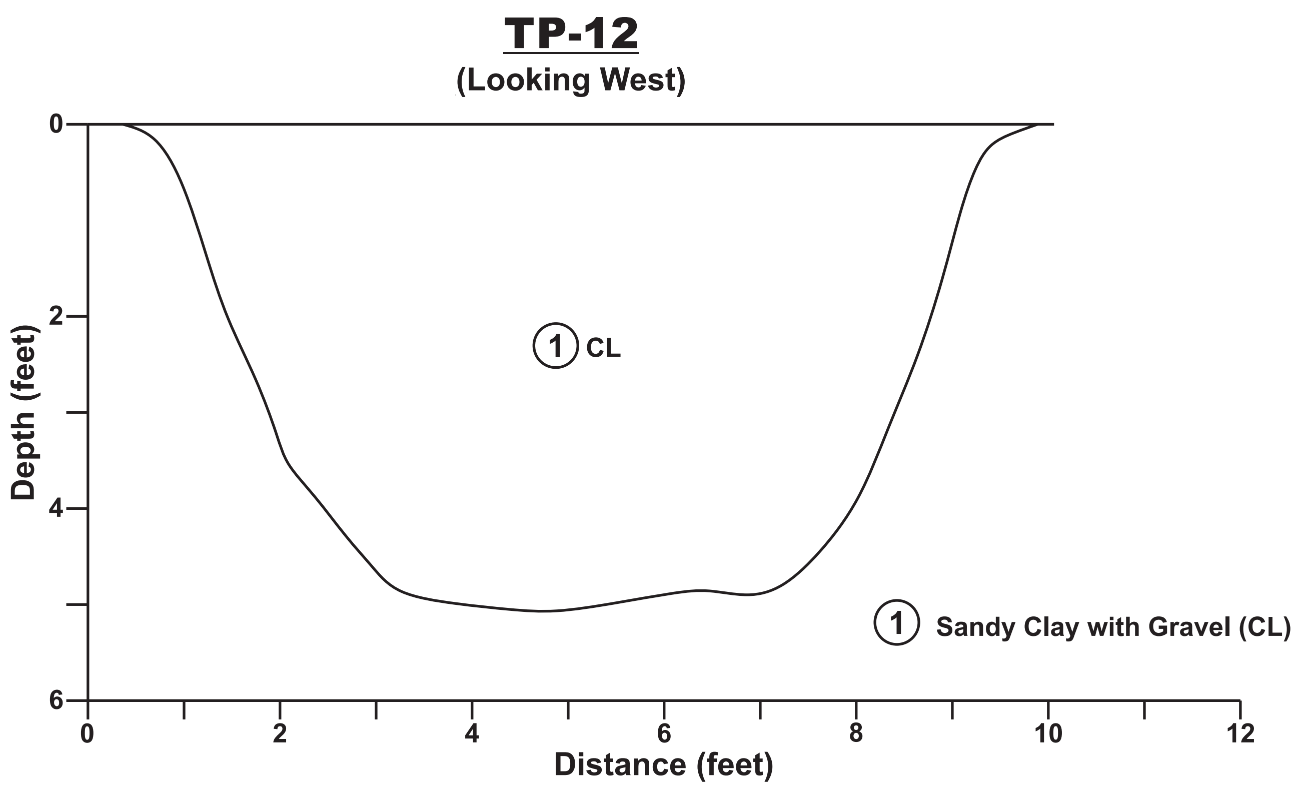
TP-10
(Looking West)



Scale: 1” = 2’

CORNERSTONE EARTH GROUP	Test Pit 9 and 10	Project Number 1270-1-1
	455 Piercy Road San Jose, CA	Figure Number Figure A-5
		Date March 2021 / Drawn By RRN





Scale: 1" = 2'

CORNERSTONE EARTH GROUP	Test Pit 11 and 12	Project Number 1270-1-1
	455 Piercy Road San Jose, CA	Figure Number Figure A-6
		Date March 2021 / Drawn By RRN

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 78 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 41 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.

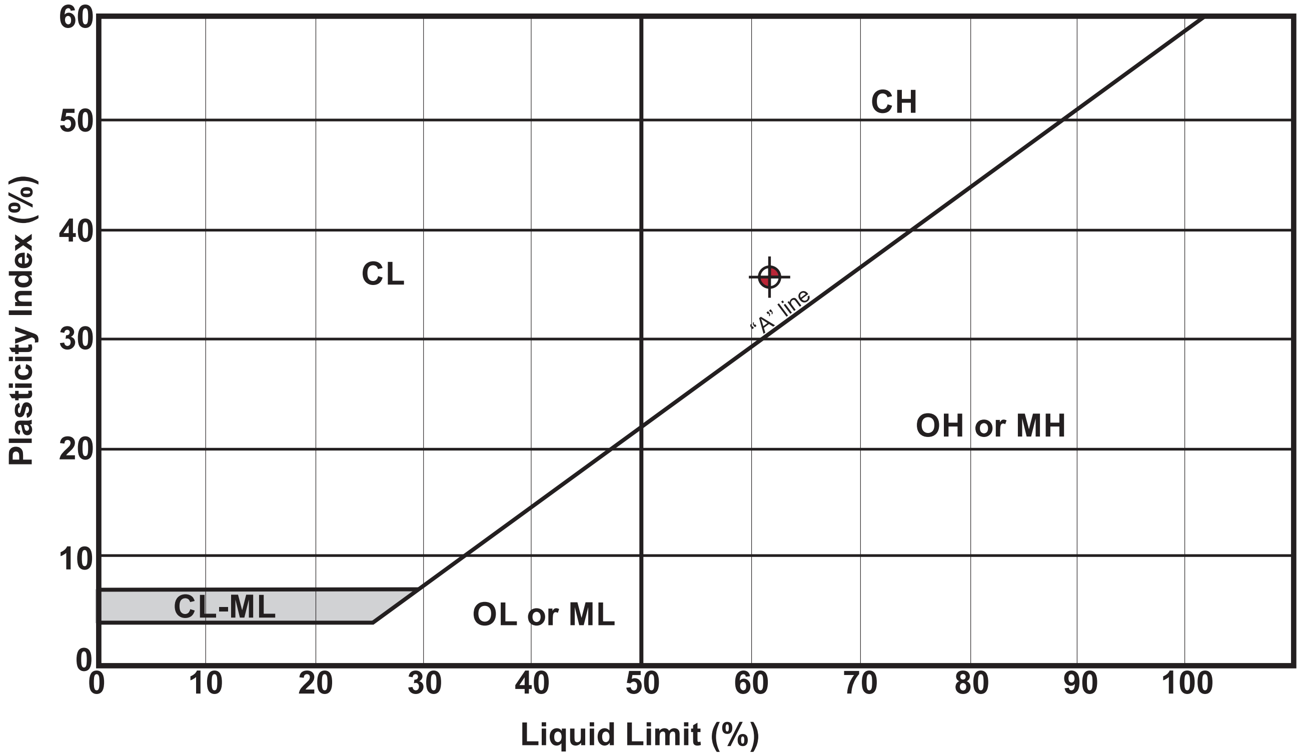
Plasticity Index: Two Plasticity Index determinations (ASTM D4318) were performed on samples of the subsurface soils to measure the range of water contents over which this material exhibits plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential. Results of these tests are shown on the boring log at the appropriate sample depths.

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

Consolidated-Undrained Triaxial Compression with Pore Pressure Measurements: The undrained shear strength was determined on 6 remolded samples of colluvium and bedrock material by consolidated-undrained triaxial shear strength testing with pore pressure measurements (ASTM D4767). The results of these test are included as part of this appendix.

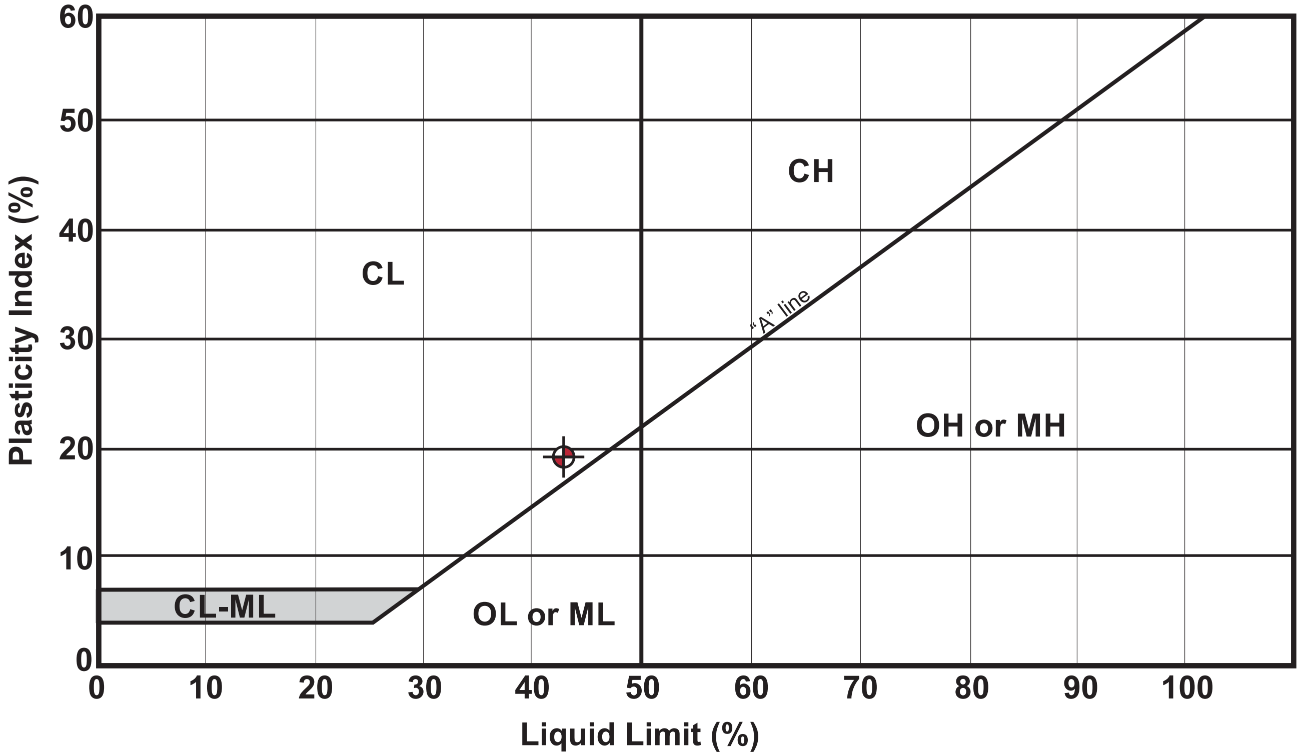
Corrosion: One soluble sulfate determination (ASTM D4327), one resistivity test (ASTM G57), one chloride determination (ASTM D4327), and one pH determination (ASTM G51) were performed on samples of the subsurface soil. Results of these tests are attached in this appendix.

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-7	2.0	18	62	26	36	—	Fat Clay with Gravel (CH) [Fill]

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-3	2.0	18	43	24	19	---	Sandy Lean Clay (CL)

CORNERSTONE EARTH GROUP

Plasticity Index Testing Summary

455 Piercy Road
San Jose, CA

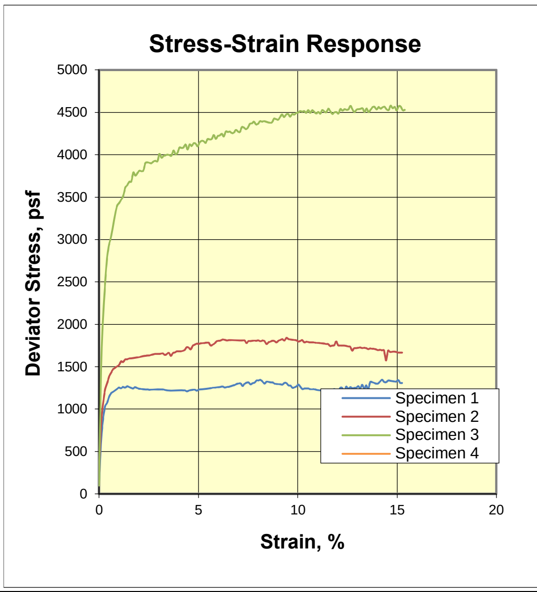
Project Number 920-1-2

Figure Number Figure B1

Date October 2017

Drawn By FLL

Consolidated Undrained Triaxial Compression with Pore Pressure
ASTM D4767

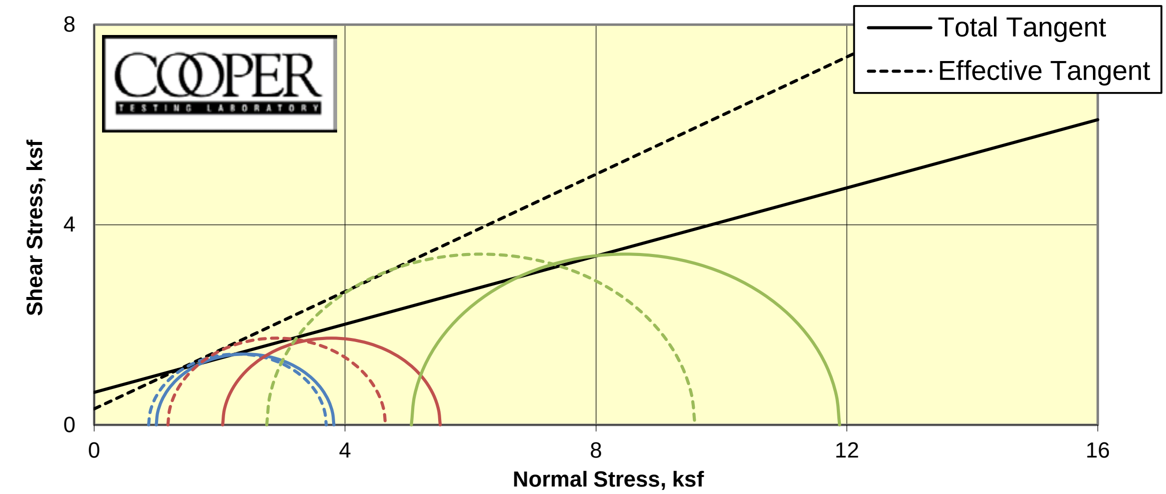


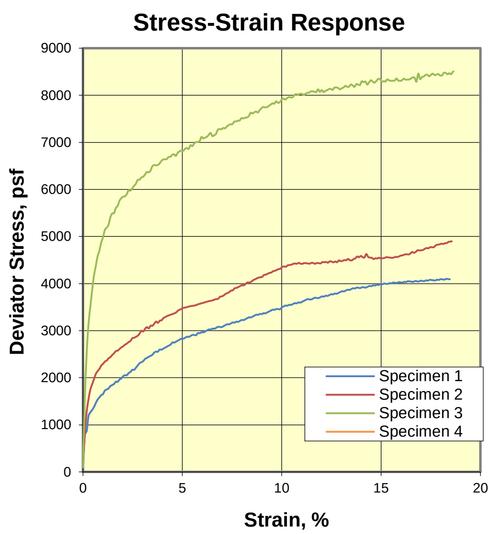
Specimen	1	2	3	4
Boring	Alluvium	Alluvium	Alluvium	
Sample	Bulk	Bulk	Bulk	
Depth				
Visual Description	Dark Grayish Brown CLAY w/ Sand	Dark Grayish Brown CLAY w/ Sand	Dark Grayish Brown CLAY w/ Sand	
MC (%)	20.4	20.1	20.3	
Dry Density (pcf)	94.9	95.1	95.0	
Saturation (%)	67.8	67.1	67.5	
Void Ratio	0.843	0.839	0.841	
Diameter (in)	2.38	2.38	2.38	
Height (in)	5.00	5.00	5.00	
	Final			
MC (%)	32.5	31.1	27.8	
Dry Density (pcf)	91.5	93.5	98.3	
Saturation (%)	100.0	100.0	100.0	
Void Ratio	0.910	0.870	0.778	
Diameter (in)	2.40	2.39	2.34	
Height (in)	5.06	5.04	4.95	
Cell Pressure (psi)	56.0	64.0	84.0	
Back Pressure (psi)	49.5	49.4	48.5	
	Effective Stresses At:			
Strain (%)	5.0	5.0	5.0	
Deviator (ksf)	1.217	1.772	4.099	
Excess PP (psi)	3.3	8.3	20.4	
Sigma 1 (ksf)	1.677	2.673	6.275	
Sigma 3 (ksf)	0.460	0.901	2.176	
P (ksf)	1.068	1.787	4.225	
Q (ksf)	0.609	0.886	2.049	
Stress Ratio	3.649	2.968	2.884	
Rate (in/min)	0.0005	0.0004	0.0004	

CTL Number:	640-1157		
Client Name:	Cornerstone Earth Group		
Project Name:	Piercy Road		
Project Number:	920-1-2		
Date:	10/10/2017	By:	MD/DC
Total C	**0.250**	**ksf**	
Total phi	**14.9**	**degrees**	
Eff. C	**0.150**	**ksf**	
Eff. Phi	**27.1**	**degrees**	©

Remarks: Remolded to 95pcf @ 20%. Expansion occurred during back pressure saturation.

Consolidated Undrained Triaxial Compression with Pore Pressure
ASTM D4767





Specimen	1	2	3	4
Boring	Serpentine	Serpentine	Serpentine	
Sample	Bulk	Bulk	Bulk	
Depth				
Visual Description	Dark Greenish Gray Sandy SILT (slightly plastic) (Serpentine)	Dark Greenish Gray Sandy SILT (slightly plastic) (Serpentine)	Dark Greenish Gray Sandy SILT (slightly plastic) (Serpentine)	
MC (%)	14.0	14.5	14.2	
Dry Density (pcf)	105.8	105.6	105.8	
Saturation (%)	63.7	65.5	64.6	
Void Ratio	0.593	0.597	0.592	
Diameter (in)	2.38	2.38	2.38	
Height (in)	5.45	5.44	5.44	
	Final			
MC (%)	26.9	26.4	24.8	
Dry Density (pcf)	97.6	98.4	100.9	
Saturation (%)	100.0	100.0	100.0	
Void Ratio	0.727	0.713	0.671	
Diameter (in)	2.45	2.45	2.42	
Height (in)	5.53	5.50	5.47	
Cell Pressure (psi)	56.0	64.0	84.0	
Back Pressure (psi)	49.1	49.8	48.9	
	Effective Stresses At:			
Strain (%)	5.0	5.0	5.0	
Deviator (ksf)	2.831	3.468	6.826	
Excess PP (psi)	0.8	6.1	16.0	
Sigma 1 (ksf)	3.700	4.644	9.577	
Sigma 3 (ksf)	0.868	1.176	2.751	
P (ksf)	2.284	2.910	6.164	
Q (ksf)	1.416	1.734	3.413	
Stress Ratio	4.260	3.950	3.481	
Rate (in/min)	0.0004	0.0004	0.0004	

CTL Number:	640-1157		
Client Name:	Cornerstone Earth Group		
Project Name:	Piercy Road		
Project Number:	920-1-2		
Date:	10/17/2017	By:	MD/DC
Total C	**0.650**	**ksf**	
Total phi	**18.8**	**degrees**	
Eff. C	**0.320**	**ksf**	
Eff. Phi	**30.4**	**degrees**	©

Remarks: Remolded to 115 pcf @ 11%. Unable to hit target density.

APPENDIX C: PREVIOUS FAULT INVESTIGATION- CORNERSTONE EARTH GROUP, 2016

CORNERSTONE
EARTH GROUP

Type of Services	Fault Investigation
Project Name	455 Piercy Road Fault Investigation
Location	455 Piercy Road San Jose, California
Client	Panattoni Development Company
Client Address	8775 Folsom Blvd; Suite 200 Sacramento, California 95826
Project Number	920-1-1
Date	November 23, 2016

Prepared by **C. Barry Butler, P.E., G.E.**
Senior Principal Engineer
Geotechnical Project Manager

Craig Harwood, C.E.G.
Certified Engineering Geologist

1259 Oakmead Parkway | Sunnyvale, CA 94085
T 408 245 4600 | F 408 245 4620

1270 Springbrook Road, Suite 101 | Walnut Creek, CA 94597
T 925 988 9500 | F 925 988 9501

www.cornerstoneearth.com

TABLE OF CONTENTS

SECTION 1: INTRODUCTION 1

1.1 Project Description ----- 1

1.2 Scope of Services ----- 2

1.3 Previous Studies ----- 2

1.4 Exploration Program ----- 2

SECTION 2: FAULT INVESTIGATION 2

2.1 Field Investigation ----- 2

2.1.1 Encountered Geologic Units 3

2.1.2 Fault History 4

2.1.2.1 Fault Investigation, Silver Creek Valley Road Development (Cornerstone, 2015) 4

2.1.2.2 Geologic Investigation for Michael Luu Property (Hydro Geo Consultants, 2000a and 2000b)5

2.1.2.3 Geologic Investigation Silver Creek Valley Road at Hellyer Avenue (Associated Terra Consultants, 1989) 6

2.1.3 Current Fault Investigation 7

2.2 Mitigating Fault Surface Rupture ----- 8

SECTION 3: CONCLUSIONS 8

3.1 Summary ----- 8

3.1.1 Fault Rupture 9

SECTION 4: LIMITATIONS 9

SECTION 5: REFERENCES 10

FIGURE 1: VICINITY MAP
FIGURE 2: SITE PLAN
FIGURE 3: SITE GEOLOGIC MAP
FIGURE 4: TEST PIT 1
FIGURE 5A: TRENCH 1A
FIGURE 5B: TRENCH 1B
FIGURE 6: TRENCH 2
FIGURE 7: GEOLOGIC CROSS SECTION A-A’

APPENDIX A: PREVIOUS STUDIES TRENCH LOGS

Type of Services	**Fault Investigation**
Project Name	**455 Piercy Road Fault Investigation**
Location	**455 Piercy Road**
	San Jose, California

SECTION 1: INTRODUCTION

This fault investigation was prepared for the sole use of Panattoni Development Company for the property located at 455 Piercy Road in San Jose, California (Figure 1). The purpose of this study was to delineate the trace of the Piercy Fault and define a geologic hazard setback zone for structures with human occupancy, as required by the City of San Jose and the State of California. The discussions of fault descriptions and set-back recommendations contained in this report are for your forward planning, cost estimating, and preliminary project design. For our use, we were provided with the following documents:

- An untitled, undated topographic survey of the site, Scale 1” = 40’.
- Conceptual Site Exhibit (Flipped Building) of 455 Piercy Road, May 4, 2016, Scale 1” = 30’.

1.1 PROJECT DESCRIPTION

The project site is located at 455 Piercy Road (APN 678-93-030), approximately 350 feet northeast of the intersection of Piercy Road and Hellyer Road in San Jose, California (Figure 1). The site is currently not in use but was previously partially developed. The southwest half of the site was previously graded for commercial use in 2001. Utility lines were installed (storm drain, electrical conduit, water, and gas) and were left in place. Two active gas lines appear to be located on the site. A gas transmission line is adjacent to the southwest edge of the property, and a 2-inch gas supply line trends northwest extending from Piercy Road to Silver Creek Road. Older improvements are present, including a former irrigation-water conveyance channel (Evergreen Canal), and an asphalt road along the northern edges of the property maintained for access to a water tank. The remainder of the site is generally covered by grasses and small native shrubs. The site ranges from approximately Elevation 201 feet at the southwest property line, to Elevation 351 feet at the eastern corner. The property consists of southwest facing slopes on the northeast half and a relatively level graded pad on the southwest half.

We understand that the site is to be developed as a storage facility with interior offices. Previous reports from adjacent properties anticipate the Piercy Fault crossing the Site. For this reason, a fault investigation was conducted to define the location, extent and potential setbacks for the Piercy Fault.

1.2 SCOPE OF SERVICES

Field work included directing a subsurface investigation consisting of trenching, logging the exposed subsurface material, and coordinating with PG&E, surveyors, and the City of San Jose. Deliverables include this report, maps, and trench logs defining the presence and extent of the Piercy Fault Zone.

1.3 PREVIOUS STUDIES

Of the various previous geotechnical/geologic investigations performed in the general vicinity of the site, three were particularly relevant to the current investigation; 1) Associated Terra Consultants (1989; Silver Creek Road at Hellyer Avenue), 2) Hydro-Geo Consultants, 2000a, 2000b; Luu Property), and 3) Cornerstone Earth Group (Cornerstone), 2015; Silver Creek Road Development. We have included the trench logs from those prior investigations in our Appendix B.

A more detailed discussion of previous fault investigation studies performed at the site and in the site vicinity is presented in Fault History section of this report (Section 2.1.2)

1.4 EXPLORATION PROGRAM

Exploration for our fault investigation consisted of one test pit excavated by a back-hoe and three trenches excavated with a track-mounted excavator, which were subsequently shored, cleaned, logged and backfilled. A detailed description of trenching activities follows in Section 2. Trench locations along with mapping of site-specific geologic features are presented in Figure 2.

SECTION 2: FAULT INVESTIGATION

2.1 FIELD INVESTIGATION

One test pit and three trenches were excavated, cleaned, logged and inspected under the supervision of Cornerstone's Certified Engineering Geologist. The test pit and trenches were excavated to depths ranging from 5 to 16 feet. The test pit was excavated with a backhoe and the trenches were excavated with a larger track-mounted excavator. Exposed walls were supported with hydraulic shoring, and cleaned with hand tools for logging of sub-surface conditions. We focused our logging on the southerly trench wall but followed certain features across to the opposing trench wall where they were exposed further by cleaning.

Identified faults from previous investigations on an adjacent site were projected onto the subject site. Those investigations were by Associated Terra Consultants (ATC, 1989) and Cornerstone (2015). Test Pit 1 was located to intersect with the surface projection of the identified Piercy Fault Zone. Test Pit 1 was initially intended as a longer trench but was found to not be located in a viable location, as the excavation reached 15 feet deep without exposing any substantive rock. A new location was proposed closer to the central ridge within the central portion of the site.

Trench 1A and 1B were located within the central portion of the site in order to intersect the projected surface trace of the Piercy Fault Zone (of ATC and Cornerstone). Trench 1A shadowed Exploratory Trench #2 (ET-2) logged by Hydro-Geo (2000). Trench 1B continued downslope of Trench 1A.

Trench 2 was located along the southeast edge of the site where previous mapping by Hydro-Geo (2000) and the City of San Jose, Fault Hazard Map (1983) locates the Piercy Fault. Trench 2 shadowed the northeast half of Hydro-Geo's (2000) Exploratory Trench #1 (ET-1). Our interpretation of the surface trace of the Percy Fault across the site differs somewhat from that of Hydro-Geo (see Faulting).

The approximate locations of the trenches are shown on the Site Plan and Site Geologic Map (Figures 2 and 3). Detailed trench logs are presented as Figures 4, 5A, 5B, and 6. A subsurface interpretation of the southeast end of the site is presented as Figure 7.

2.1.1 Encountered Geologic Units

Artificial fill is exposed in trenches where the site has been graded or where utilities were installed. Test Pit 1 revealed a surficial berm of artificial fill consisting of intermixed site soil and serpentinite rock fragments. Trench 1A contains shallow (1 to 2 feet) fills along the cut slope of the hill and on the previously graded pad at the base of the hill consisting predominately of intermixed crushed serpentinite and some soil. Trench 1B crossed multiple utility lines (storm drain, water, gas and electrical) where the imported trench fill consisted of well-sorted sands and moderately-sorted fine gravels (quarry fines). Significant site grading followed by terrace building was apparent where approximately six feet of site soil was placed. Fill material was exposed on the southwest portion of Trench 2.

Surficial soils exposed in all trenches and the test pit are predominantly dark brown clays with some sand of medium to high plasticity. Clay content, strength and moisture increased with depth. A-horizon soils (residual soil) are more granular and blocky in texture and lighter in color (2.5YR 3/2) compared to lower horizons. B-horizon soils (residual subsoil) have blocky to prismatic structure, very stiff with higher plasticity and are darker in color (5Y 2.5/2). Soils occasionally contain coarse sands and gravels of serpentinite, chert, and/or sandstone. The uphill (northeastern) portions of Trench 1A and Trench 2 exposed a thinner colluvial soil compared to the thicker valley filling soils to the southwest. Soils and bedrock contacts are distinct to gradational. Krotovina filled with soil (infilled animal burrows) accentuate the fractures and joints of the underlying rock.

An unconsolidated colluvial soil overlying intact bedrock below was encountered within Test Pit 1, Trench 1B and Trench 2. This unit thickens as a colluvial apron to the southwest and consists of clay with sands and gravels; the unit is brown (2.5YR 3/3), consistently massive, and cohesive with moderate to low PI. Angular fine to course gravels are found throughout. Sporadic stony horizons of gravels demarcate probable former colluvial and/or slope wash events.

Within our trenches two bedrock units were encountered: Serpentinite and Franciscan Complex Mélange. Test Pit 1 exposed a thick colluvial unit (>15 feet) overlying fractured dark shale. Trench 1A consisted of serpentinite that continues into the northeast portion of Trench 1B where it was found to be in contact with Mélange at a depth of 17 feet. Trench 2 exposed serpentinite bedrock throughout.

The Serpentinite is variegated in color ranging from a light olive gray (5Y 6/1), greenish black (5GY 2/1), yellowish gray (5Y 8/1), and olive gray (5Y 3/2 to 4/4). The Mélange consists primarily of shale but with minor graywacke sandstone and ranges from light gray to light olive gray (N7 to 5Y 6/1) greywacke, and grayish black to olive gray (N2 to 5Y 4/1). The Mélange also consists of thickly bedded graywacke and thinly interbedded shales that are friable, have low to moderate strength, and are moderately weathering. The Serpentinite has a generally low strength and is highly sheared with occasional pockets of highly weathered material and non-deformed clasts. Associated mafic and ultramafic rocks are generally moderate to hard, and moderately weathered. Both bedrock units (Serpentinite and Mélange) have higher degrees of weathering and weaker strength in the more highly sheared and faulted zones. Carbonate precipitation is ubiquitous within shears and faulted zones, and common within joints and fractures and intraformational shears.

Deeply weathered and fractured zones accommodate pockets of well-developed soil and also provide relatively softer zones where gopher burrowing is concentrated. In a single outcrop exposing these deep in-filled burrows (pockets) can give the impression of offset soil unless examined across a trench or test pit excavation (See discussion in “Faulting”).

2.1.2 Fault History

Information on the Piercy Fault is relatively obscure in the published literature. The fault was originally recognized by Dibblee (1972) based on an exposure of Serpentinite in fault contact with Santa Clara Formation just southeast of the subject site, although he shows the fault as concealed beneath alluvium (but not offsetting Holocene age alluvium) along most of its 3.9-mile-long mapped trace. Additional regional mapping has recognized the Piercy Fault trending through the area; (Bailey and Everhart, 1964; Dibblee, 1972; City of San Jose, 1983; Terratech 1983; Helley and Herd, 1990; Dibblee and Minch, 2005; Williams and Rogers, 1974). As already discussed, the Piercy Fault has previously been mapped as crossing the adjacent property on the northwest (Associated Terra Consultants, 1989) and by Cornerstone (2015), and more recently (this study) projected through the subject site near the top of the cut slope. Some discrepancies exist between the findings of Hydro-Geo Consultants (HGC; 2000a, 2000b),) and our investigation of the adjacent site (5880 Hellyer Avenue) and the current site-specific study in terms of the juxtaposition of units and the fault’s projection through the site.

2.1.2.1 Fault Investigation, Silver Creek Valley Road Development (Cornerstone, 2015)

The Silver Creek Valley Road Development is located to the northwest between Silver Creek Valley Road and the northwest property line of the subject site. The fault investigation on that property included logging two trenches and incorporating data from three borings from a previous Site Feasibility Investigation. The northeast Trench (T-2) was located to intersect a surface projection of the Piercy Fault along the upper edge of the cut slope, as depicted in the HGC investigation for this subject site from 2000. Cornerstone's logging of Trench 2 found no evidence of faulting. Cornerstone's Trench 1 identified a zone of faulting 75 feet wide composed of a zone of sheared and faulted sections of Serpentinite and greywacke. The zone of faulting was consistent in terms of it's surface projection and general character with the zone found 850 feet to the northwest by ATC.

As an aside, sometime between May of 2001 and November of 2002 the current subject site (southeast of the Silver Creek Valley Road Development) has received significant downcutting during mass grading, and such ground surface modification would be expected to make the westward arc of the fault surface trace more pronounced. Cornerstone interpreted the faulting as more complex than is shown on the map by HGC and actually arcs more westerly across the northwest end of the subject site where it intersects Cornerstone's (2015) Trench 1 somewhere within the identified zone of faulting confirmed in the 2015 investigation.

2.1.2.2 Geologic Investigation for Michael Luu Property (Hydro Geo Consultants, 2000a and 2000b)

The Micheal Luu Property is 14.25 acres of land adjacent to the southeast edge of the subject site. Investigations by Hydro Geo Consultants consisted of three trenches and one magnetometer traverse [only two of their trench logs (T-1 and T-2) were available for our review]. The Piercy Fault was initially identified on the Luu property in a magnetic survey near the break in slope where the foothills abut the valley floor. As an aside it would seem that given the field relations that characterize the fault (interleaved, highly sheared and moderately to steeply dipping bedrock) that a magnetometer survey would be unlikely to identify a fault in such a situation. Subsequently, two trench explorations excavated across that break in slope revealed a 5-foot wide zone of intense shearing ("several faults") within serpentinite associated with heavy Caliche. The log for their Exploratory Trench ET-2 (located 435 feet to the southeast of our Trench T-1) was reviewed as part of our research but the log for their Exploratory Trench ET-1 was not available for our review. One shear zone labeled as the "Piercy Fault" within their Trench ET-2 has a field measured attitude of N43°W, 40°N, which suggests a thrust geometry. We noted that an outcrop of the fault zone is locally exposed at that southeast property corner where we measured a low-angle reverse geometry (48° to the northeast). Their log shows a zone of intense shearing at the base of the hillside (their Station 52 to 68) where the Holocene alluvium deepens (toward the southwest) rather abruptly. They interpreted this to be the Piercy Fault and labeled it as such on their log. They also show a fault located up near the northeast end of the trench where a pocket of colluvial soil appears to terminate abruptly against this northeast dipping feature (at their station 15). We infer this is an example of colluvium forming on the upslope edge of a resistant outcrop, rather than truncation of a deposit. Although some features shown on their logs were not explained, they concluded the Piercy Fault is not active based on the observation that Holocene age sediments overlying the fault (exposed in two of their trenches) are not cut by faulting. They projected the fault along a consistently straight trend toward the subject site as shown on the current Site Geologic Map. We question this mapping as the fault has a 40° to 48° dip and crosses a lobate landform where it projects into current subject site. Given that fault geometry and the changing topography toward the northerly direction, the fault would normally arc toward a more westerly direction as it trends into the subject site. They further supported this mapping with the results of trenching on the adjacent site (on the southeast) by United Soils Engineering ("USE"). In 1998 USE conducted two twenty-foot long trenches on that site which encountered a deep section of Quaternary alluvium. They claimed to have encountered evidence of faulting in both of their trenches but their logs and the trench locations do not support this claim.

2.1.2.3 Geologic Investigation Silver Creek Valley Road at Hellyer Avenue (Associated Terra Consultants, 1989)

Associated Terra Consultants ("ATC") in 1989 conducted a site investigation on an adjacent site located approximately 200 feet northwest of the subject site. This part of the parcel is now part of the right of way for Silver Creek Valley Road. The ATC geological investigation included the logging of three test pits, three trenches, and surveying by way of seven seismic refraction lines focused on defining the trace of the Piercy Fault identified in previous published studies (Bailey and Everhart, 1964; Dibblee, 1972; ; Williams and Rogers, 1974; City of San Jose, 1983; Terratech 1983; Graymer et al, 1995; Wentworth et al., 1999; Hitchcock and Brankman, 2002; Dibblee and Minch, 2005). These sources invariable show the fault trending along the toe of the range front where it is shown as a thrust fault.

Hydro-Geo identified a zone of faulting 44 to 55 feet wide in two of their trenches (T-1 and T-3), which they projected to the southeast aligning with Cornerstone's (2015) Trench 1 and the western corner of the current subject site. The fault zone was characterized as a series of prominent, through-going shears. Their site geologic map suggests the fault zone is concentrated within a band of Serpentinite, which is bounded on the northeast and the southwest by Franciscan Mélange. However, their trench logs show a very similar series of structural and stratigraphic relationships as encountered in Cornerstone's (2015) investigation. Intra-formational shearing and distinct faulted zones were identified in their trenches. The relative trend of fault traces between the two trenches is approximately N45W to N50W, projecting through the southwest corner of the subject site in the area of Cornerstone's (2015) Trench T-1. The dips of fault traces range from 40° to 76° to the east. ATC indicated that some of the shears within the bedrock extended into and displaced overlying "A", "B" and "C" soil horizons. Due to their interpretation (ATC, 1989) of possible fault offset of Holocene soils, ATC recommended a 100-foot wide building exclusion zone (50 foot setback on either side of the fault zone) through the northern portion of the that site.

The trenches by ATC apparently did not follow these soil horizon offset features across their trenches to examine the apparent relative offsets and stratigraphic relationships on trend with these features. We believe these types of features, consistent with features observed in Cornerstone's (2015) Trench T-1, are more likely attributed to animal burrowing activity near resistant, steeply dipping beds or zones within the bedrock (see Current Field Investigation section of this report).

2.1.3 Current Fault Investigation

Test Pit 1 was excavated to a depth of 16 feet for a length of 11 feet exposing mostly colluvium overlying Mélange (shale). The shale was highly weathered with ubiquitous carbonate within the sheared matrix.

Trench 1A was excavated to depths of 4 to 8 feet and revealed intact serpentinite bedrock throughout. We identified a zone of faulting starting at 50 feet from the northeast uphill edge of the trench (denoted by a stake set at the ground surface). Significant fractures filled with illuviated clay and carbonate seems denote the eastern edge of faulting. Attitudes of identified fault traces measured across the trench vary in strike from S79°E to S73°E and vary in dip from 62°S to 80°S. This geometry is counter to northwesterly striking Piercy trace but are interpreted as associated high angle splays at the eastern edges of the Piercy Fault zone.

Trench 1B overlaps Trench 1A by approximately 14 feet. The zone of faulting identified at Station 50 in Trench 1A continues to the southwest in Trench 1B to a distance of 47 feet. Faulting may continue further to the southwest but bedrock was not exposed any further to the southwest in Trench 1B as the alluvium and colluvium thickens to the southeast. Trench 1B was excavated to depths of 7 to 15 feet. Sheared and fractured serpentinite was exposed from the northeastern edge to Station 16 where it comes in contact with fractured and sheared sandstones and shales (Mélange of the Franciscan Complex). An assortment of bedding attitudes were measured, which exhibited a convoluted structure. A wedge of rock and soil was not removed to keep an active gas (48 feet) line supported. Faulting may persist to the southwest but the fact that the bedrock dives further toward the southwest indicated it was beyond the range of conventional trenching techniques and the geologic field relations that define the fault led us to conclude that geophysical or exploratory boring arrays would be ineffective techniques in locating the southwestern limit of faulting. A combined distance of 70 feet of faulted rock is exposed in Trenches 1A and 1B but the zone of faulting likely continues to the southwest. In comparison, the Piercy Fault zone was measured to be 75 feet wide approximately 500 feet to the northwest (Cornerstone, 2015), and 44 feet wide approximately 100 feet beyond (ATC, 1989).

Trench 2 was excavated to depths of 4 to 16 feet revealing serpentinite at depth overlain by a colluvial apron and capped by 2 to 3 feet of soil. Fractures and shears with carbonate and illuviated clay were logged within the serpentinite from the uphill northeast end of the trench to Station 52 (marked at surface with a stake and surveyed). This stake marks the western extent of the Piercy Fault Zone based on our projection. More resistant serpentinite with hard, un-fractured ultramafic rock was exposed beyond to the southwest of Station 52 foot. Attitudes of the identified fault traces measured across the trench vary in strike from N30°W to N28°W and vary in dip from 43°NE to 45°NE suggesting primarily low-angle, reverse geometry but also thrust geometry locally.

Sigmoidal-shaped fault-bounded blocks of sheared shale, greywacke, and Serpentinite were encountered in Trenches 1A, 1B and 2 dip to the northeast, similar to previous investigation observations by HGC and ATC. No fractures or shear features were observed to cut Holocene soils or the colluvial apron between soil and bedrock. The soil or colluvium/bedrock contact is wavy to irregular and appears to be offset. However, after following the trend of these features across the trench and clearing the opposing trench wall the soil had an almost opposite sense of apparent offset. Upon closer examination we concluded these features were the result of soil and colluvium in abrupt contact (deposition) against a resistant portion of bedrock and also resulting in disturbance due to animal burrowing activity.

A surface projection of the Piercy Fault Zone was plotted in map view by extending the western and eastern extent in a southeasterly direction from the previously identified zone (Cornerstone, 2015) through the eastern and western edges that were located onsite in Trench 1A and Trench 2; respectively. The eastern edge southeast of Trench 1A was projected at a similar strike to the western edge and coincides with a subtle change in slope uphill of Trench 2 (Figure 3). This fault zone width is consistent with its width encountered by ATC and Cornerstone on the adjacent (northwest) site.

2.2 MITIGATING FAULT SURFACE RUPTURE

A final development concept for the site has not been conceived of at this time. It is the southwestern bounding fault of the overall identified fault zone that is relevant for establishing the building setback for potential habitable structures planned as part of any future development concepts for the site. As already mentioned, a 50-foot building exclusion zone (25-foot setback from the surface trace) was previously recommended for the subject site by Hydro-Geo Consultants (2000a, 2000b), although they had only identified a single surface trace and they had concluded the fault was “inactive” (i.e., a pre-quaternary fault). Previous mapping in the area suggest that the Piercy Fault is in fact Quaternary active but the latest movement is thought to have cut Pleistocene age material. Therefore, a building exclusion zone established along its surface trace is appropriate. The fault zone is bounded on the northeast by a fault with a thrust geometry and on the southwest by a fault that has a reverse geometry. We therefore recommend a 35-foot setback along the hanging wall (northeast boundary) and a 25-foot setback along the footwall (southwest edge). The southwest edge of the fault zone (southwesterly bounding fault) is dipping toward the northeast and away from the most probable area of development for habitable structures, therefore, a conventional 25-foot building setback is recommended for this side of the fault zone.

It should be noted that any proposed cutting (grading) on the northeast side of the fault zone can shift the fault surface traces further toward the northeast and therefore impact the setback on the graded side of the fault zone.

SECTION 3: CONCLUSIONS

3.1 SUMMARY

In accordance with project approval guidelines by the City of San Jose, once the development concept becomes available, faulting, and other geologic hazards, will be addressed in further detail as part of a required future design-level geotechnical and geologic hazards clearance investigation.

3.1.1 Fault Rupture

As previously discussed, nearby Site specific studies in the immediate vicinity have resulted in recommended building exclusion zones due to a potential for surface fault rupture. Despite the different interpretations concerning the location of the fault, there is compelling evidence that surface traces of the Piercy Fault project through the subject site. Our recommended building setback lines that are depicted in Figures 3, extend along the northeastern edge and southwest bounding limits of the fault zone and apply to any future habitable structures at the site. These setbacks are equal to 35 feet along the northeastern edge and 25 feet along the southwestern edge of the mapped fault zone surface trace.

SECTION 4: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of Panattoni Development Company (Panattoni) specifically to support the design of 455 Piercy Road 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 conditions encountered during our limited subsurface exploration. If variations or unsuitable conditions are encountered during the construction phase, Cornerstone must be contacted to provide supplemental recommendations, as needed.

Panattoni may have provided Cornerstone with plans, reports and other documents prepared by others. Panattoni 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 5: REFERENCES

Alliance Environmental and Soil Engineering, May 24, 2000, Geotechnical Investigation Report for Proposed Industrial Development at Piercy Road San Jose, California: File No. 00-255-GS.

Associated Terra Consultants, June 30, 1989, Geologic Fault Investigation, APN: 678-14-069, Hellyer Avenue, San Jose, California: Job No. 105211.

Bailey, E.H. and Everhart, D.L., 1964, Geology and quicksilver deposits of the New Almaden district, Santa Clara County, California: U.S. Geological Survey, Professional Paper 360, scale 1:24,000.

Batatian, L.D., and Nelson, C.V., 1999, Fault setback requirements to reduce rupture faulting hazards in Salt Lake County [abs.]: Association of Engineering Geologists, Program with abstracts, 42nd Annual Meeting, Salt Lake City, Utah, p. 59.

Bryant, W.R., 1981a, Calaveras, Coyote Creek, Animas, San Felipe, and Silver Creek Faults,, California Division of Mines and Geology: Fault Evaluation Report 122, 13p.

Bryant, W.R., 1981b, Fault Evaluation Report FER-106, SE Segment of the Hayward Fault, Evergreen Fault, Quimby Fault, Silver Creek Fault, and Piercy Fault, March 16, 6 pl.

California Division of Mines and Geology, 1982, Revised official map of Alquist-Priolo Earthquake Fault Hazard Zones, San Jose East Quadrangle: California Division of Mines and Geology, scale 1:24,000.

City of San Jose, 1983, Fault Hazard Maps (San Jose East Quadrangle, California): City of San Jose, California, 1:24,000 scale.

Cooper-Clark & Associates, 1974, Technical report, Geotechnical Investigation, City of San Jose's Sphere of Influence: Report submitted to City of San Jose Department of Public Works, 185 p., 26 plates, scale 1:48,000.

Cornerstone, 2015, Fault Investigation, Silver Creek Valley Road Development, 5880 Hellyer Avenue, San Jose, California.

Dibblee, T.W. Jr., 1972, Preliminary geologic map of the San Jose East Quadrangle, California: U.S. Geological Survey Open File Report 72-92, scale 1:24,000.

Dibblee, T.W. and Minch, J.A., 2005, Geologic map of the San Jose East quadrangle, Santa Clara County, California: Dibblee Geological Foundation, Dibblee Foundation Map DF-155, scale 1:24,000.

Graymer, R.W., Jones, D.L., and Brabb, E.E., 1998, Geologic map of the Hayward fault zone, Contra Costa, Alameda, and Santa Clara Counties, California: a digital database: U.S. Geological Survey, Open-File Report OF-95-597, scale 1:50,000.

Hitchcock, C.S., Brankman, C. M., 2002, FINAL TECHNICAL REPORT: Assessment of late Quaternary deformation, eastern Santa Clara Valley, San Francisco Bay region, U. S. Geological Survey National Earthquake Hazards Reduction Program Award Number 01HQGR0034

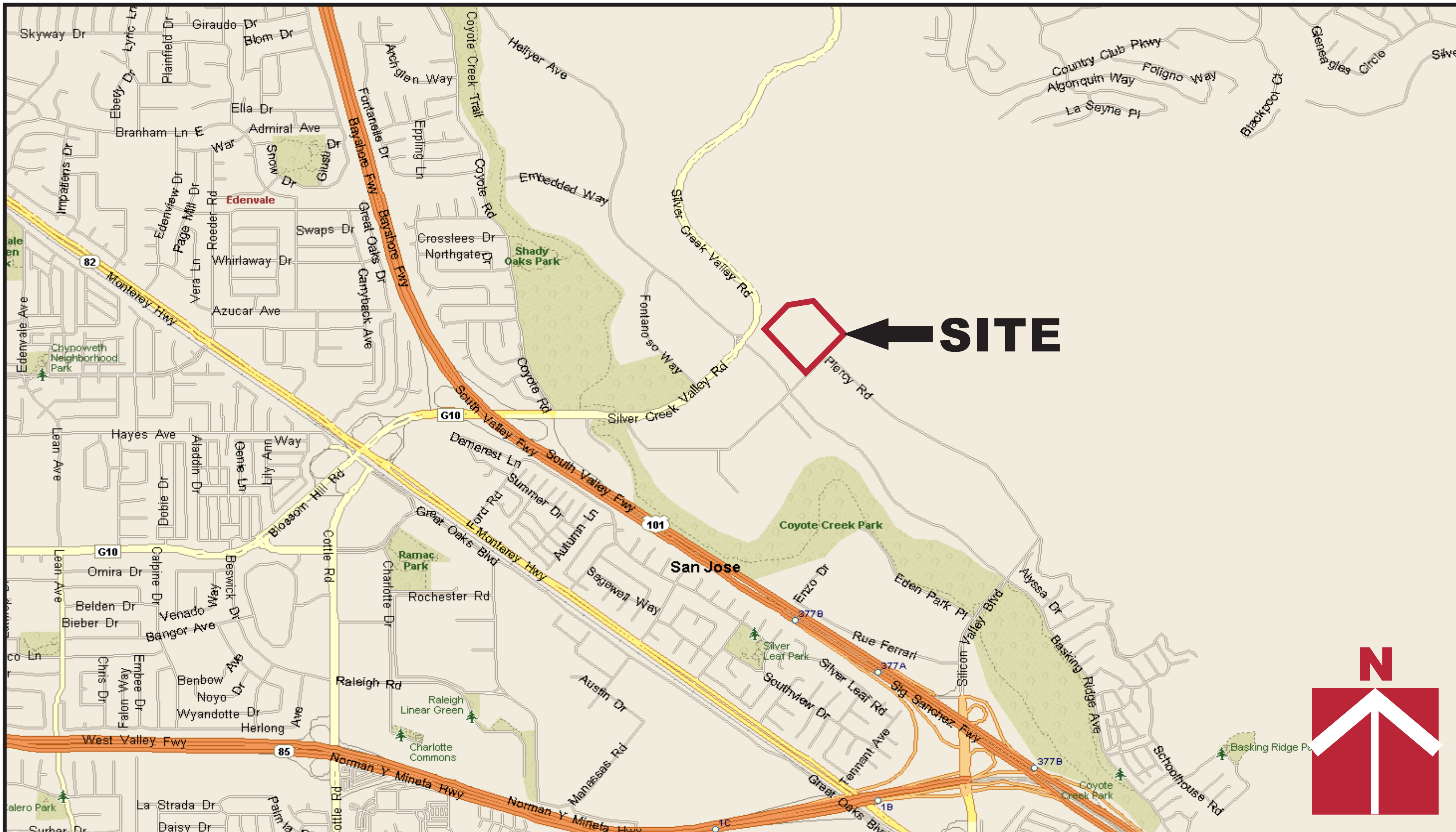
Hydro-Geo Consultants, Inc., June 2000, Geologic Evaluation Proposed Silver Creek Valley Corporation Center Piercy Road, San Jose, California, Project 3509-A.

Hydro-Geo Consultants, Inc., September 2000, Supplemental Evaluation to our 6/7/00 Report Location of the Piercy Fault at Proposed Silver Creek Valley Corporation Center Piercy Road, San Jose, California, Project 3509-A.

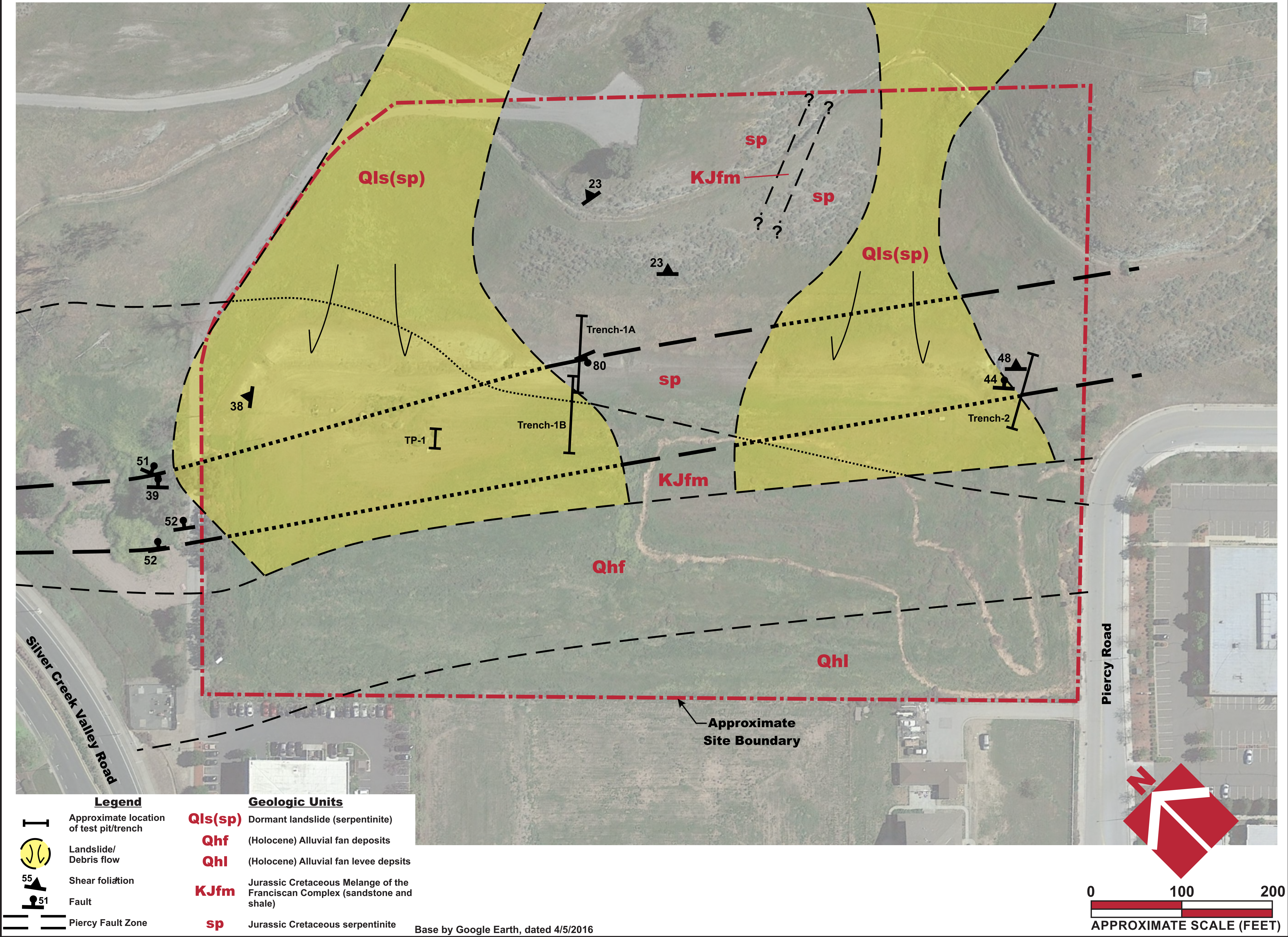
Rogers, T.H., and J.W. Williams, 1974 Potential Seismic Hazards in Santa Clara County, California, Special Report No. 107: California Division of Mines and Geology.

Santa Clara County Planning Office, Geologic Hazard Zones – Spatial Data http://www.sccgov.org/sites/planning/GIS/GeoHazardZones/Spatial_Data/Pages/County-Geologic-Hazard-Zones-Data.aspx

Wentworth et al, 1999, Preliminary Geologic Map of the San Jose 30 x 60 Minute Quadrangle, California.



CORNERSTONE EARTH GROUP	Vicinity Map	Project Number 9201-1
	455 Piercy Road San Jose, CA	Figure Number Figure 1
		Date November 2016 / Drawn By RRN



Project Number 920-1-1

Figure Number Figure 2

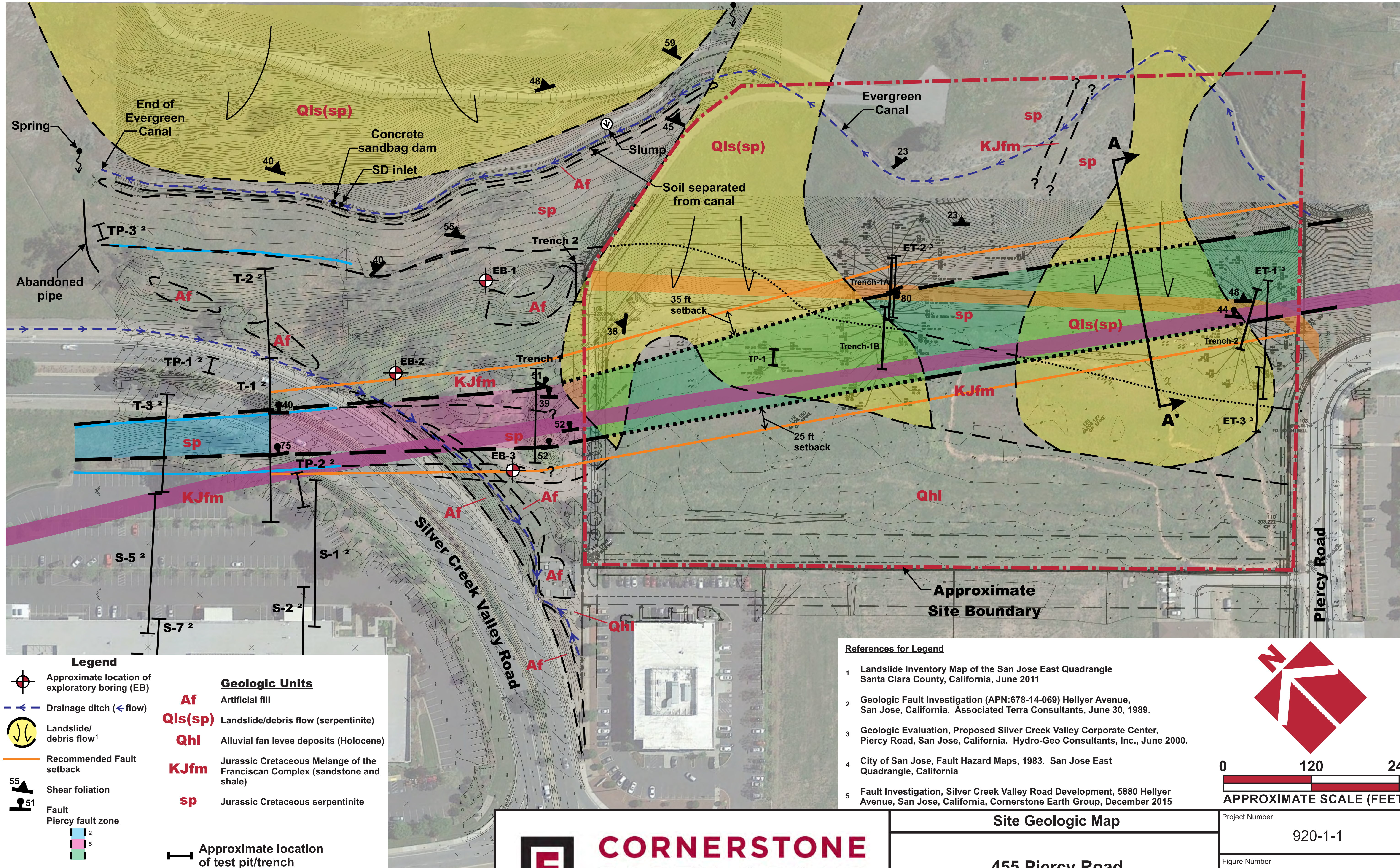
Drawn By RRN

Date November 2016

Site Plan

455 Piercy Road
San Jose, CA

CORNERSTONE
EARTH GROUP



Legend

- Approximate location of exploratory boring (EB)
- Drainage ditch (← flow)
- Landslide/ debris flow[1]
- Recommended Fault setback
- Shear foliation
- Fault
- Piercy fault zone [2] [5]
- Contact [2]
- Reported fault [3]
- Reported fault [4]
- Approximate location of test pit/trench
- A A' Approximate location of cross section

Geologic Units

Af	Artificial fill
Qls(sp)	Landslide/debris flow (serpentinite)
Qhl	Alluvial fan levee deposits (Holocene)
KJfm	Jurassic Cretaceous Melange of the Franciscan Complex (sandstone and shale)
sp	Jurassic Cretaceous serpentinite

Base by Google Earth, dated 3/28/2015

References for Legend

1. Landslide Inventory Map of the San Jose East Quadrangle Santa Clara County, California, June 2011
2. Geologic Fault Investigation (APN:678-14-069) Hellyer Avenue, San Jose, California. Associated Terra Consultants, June 30, 1989.
3. Geologic Evaluation, Proposed Silver Creek Valley Corporate Center, Piercy Road, San Jose, California. Hydro-Geo Consultants, Inc., June 2000.
4. City of San Jose, Fault Hazard Maps, 1983. San Jose East Quadrangle, California
5. Fault Investigation, Silver Creek Valley Road Development, 5880 Hellyer Avenue, San Jose, California, Cornerstone Earth Group, December 2015

0 120 240
APPROXIMATE SCALE (FEET)

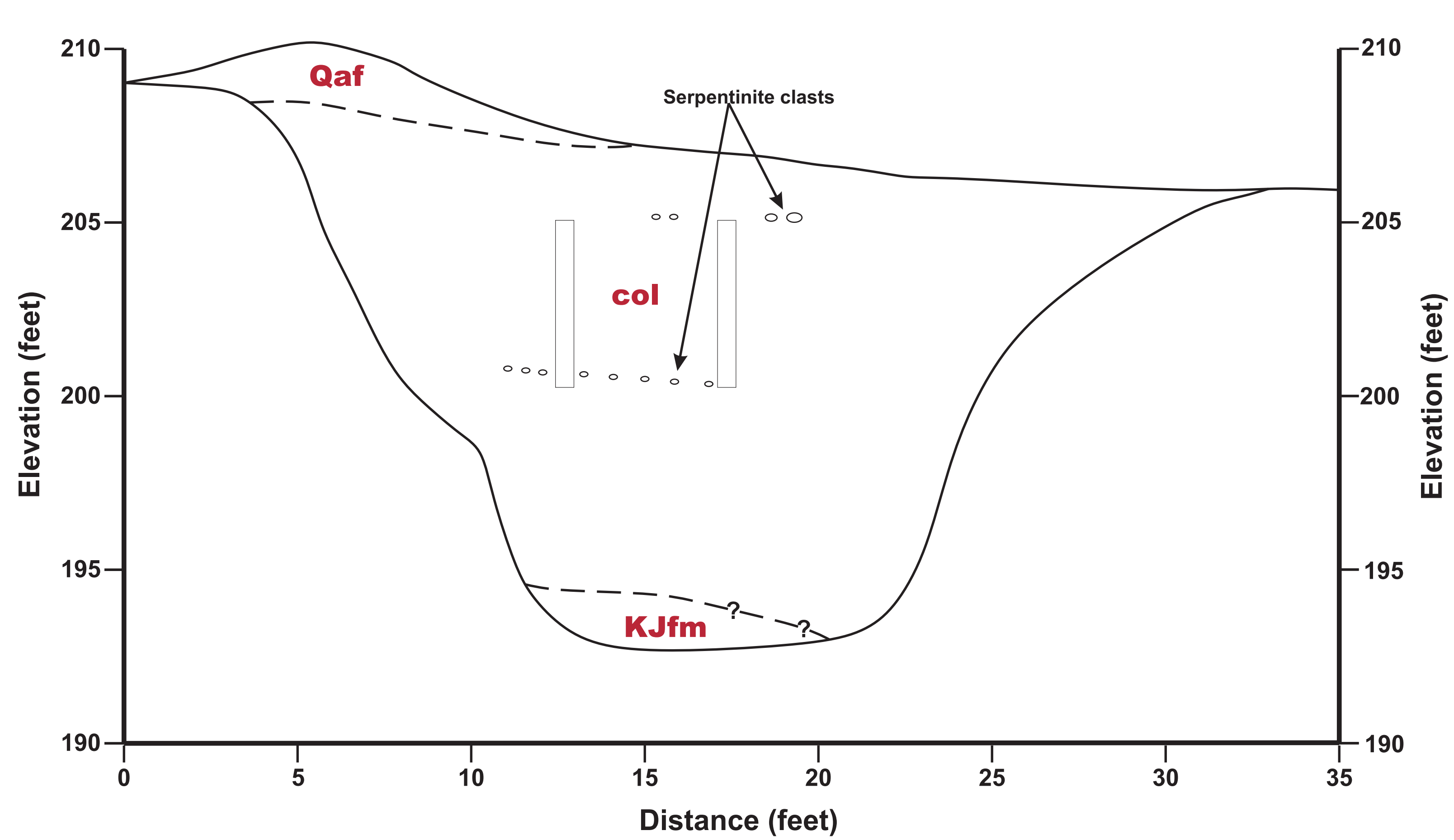
CORNERSTONE EARTH GROUP

Site Geologic Map	Project Number 920-1-1
455 Piercy Road San Jose, CA	Figure Number Figure 3
	Date November 2016 / Drawn By RRN

Test Pit 1

S 46° W

(Looking Southeast)
1"=5' H:V



Qaf — **Sandy Clay with gravel [CL][fill],** dark brown to dark gray (7.5YR 4/2 and 4/0), dry to moist, friable to hard, lacking structure to granular with some angular gravel, occasional roots; artificial fill and reworked/plowed native soil.

col — **Sandy Clay with Gravel [CL] (colluvial),** dark reddish brown (2.5Y 3/2 and 3/4), moist, stiff, medium plasticity, blocky structure with coarse angular to sub-angular gravel (serpentinite, greywacke, shale) with occasional slight downslope imbrication.

KJfm — **Graywacke and Shale,** low hardness to hard, friable to moderately strong, moderate to little weathering, Grawacke (sandstone): light gray to light olive gray (N7 to 5Y 6/1), Shale: grayish black (N2) and olive black to olive gray (5Y 2/1 to 5Y 4/1), moderately to intensly fractured, Graywacke (sandstone): thick- to very thick-bedded, Shale: very thin-bedded; Franciscan Mélange (sandstones and shales).

Project Number 920-1-1
Figure Number Figure 4
Drawn By RRN
Date November 2016

Test Pit 1

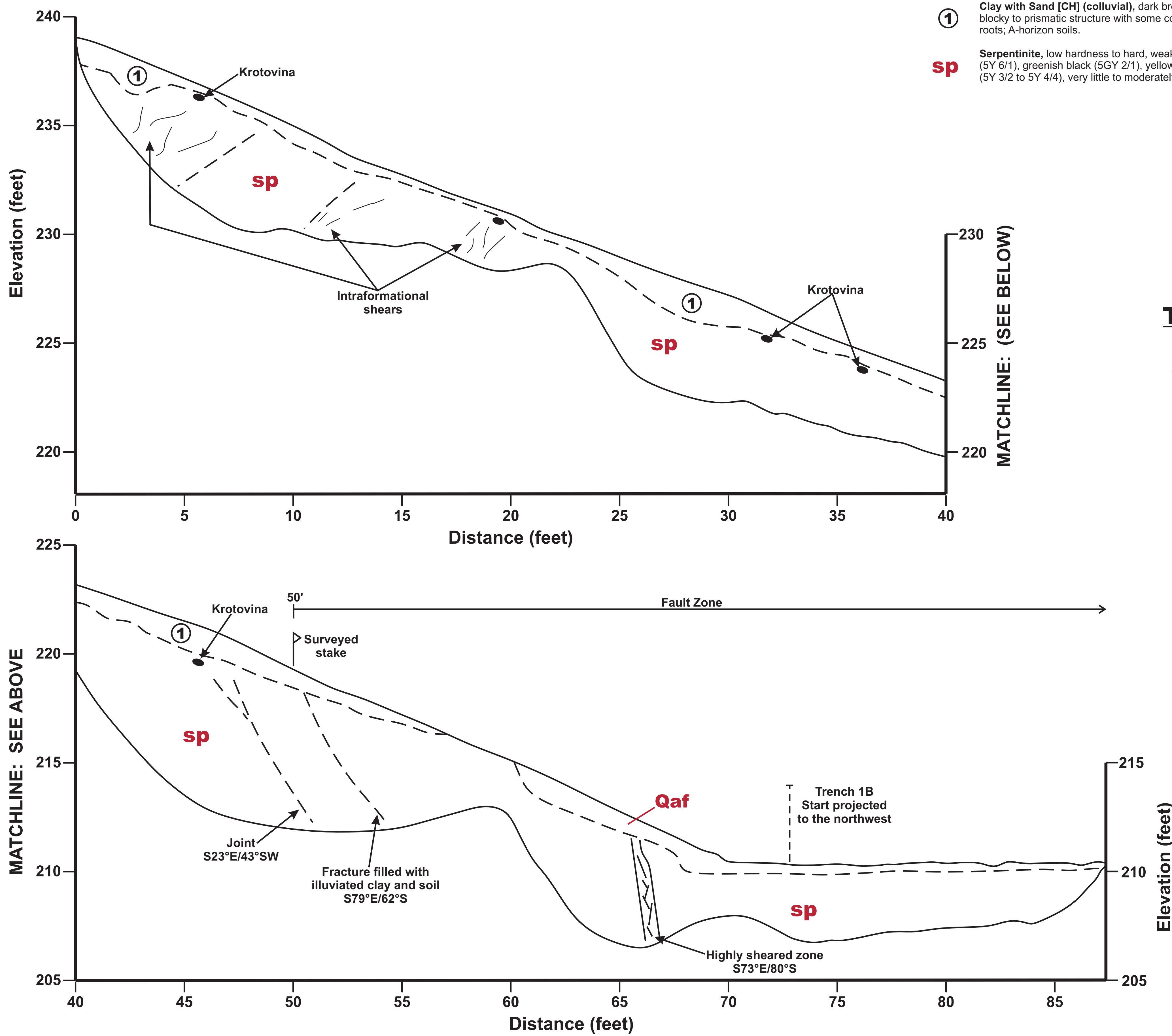
455 Piercy Road
San Jose, CA

CORNERSTONE EARTH GROUP

Qaf **Sandy Clay with gravel [CL][fill],** dark brown to dark gray (7.5YR 4/2 and 4/0), dry to moist, friable to hard, lacking structure to granular with some angular gravel, occasional roots; artificial fill and reworked/plowed native soil.

1 **Clay with Sand [CH] (colluvial),** dark brown (5Y 2.5/2), moist, stiff to hard, medium to high plasticity, blocky to prismatic structure with some coarse sands and occasional sub-angular gravel, some fine roots; A-horizon soils.

sp **Serpentinite,** low hardness to hard, weak to strong, deep to moderately weathered, light olive gray (5Y 6/1), greenish black (5GY 2/1), yellowish gray (5Y 8/1), and olive gray to moderately olive gray (5Y 3/2 to 5Y 4/4), very little to moderately fractured, massive.



Trench 1A

S 47° W

(Looking Southeast)
1"=5' H:V

Project Number 920-1-1

Figure Number Figure 5A

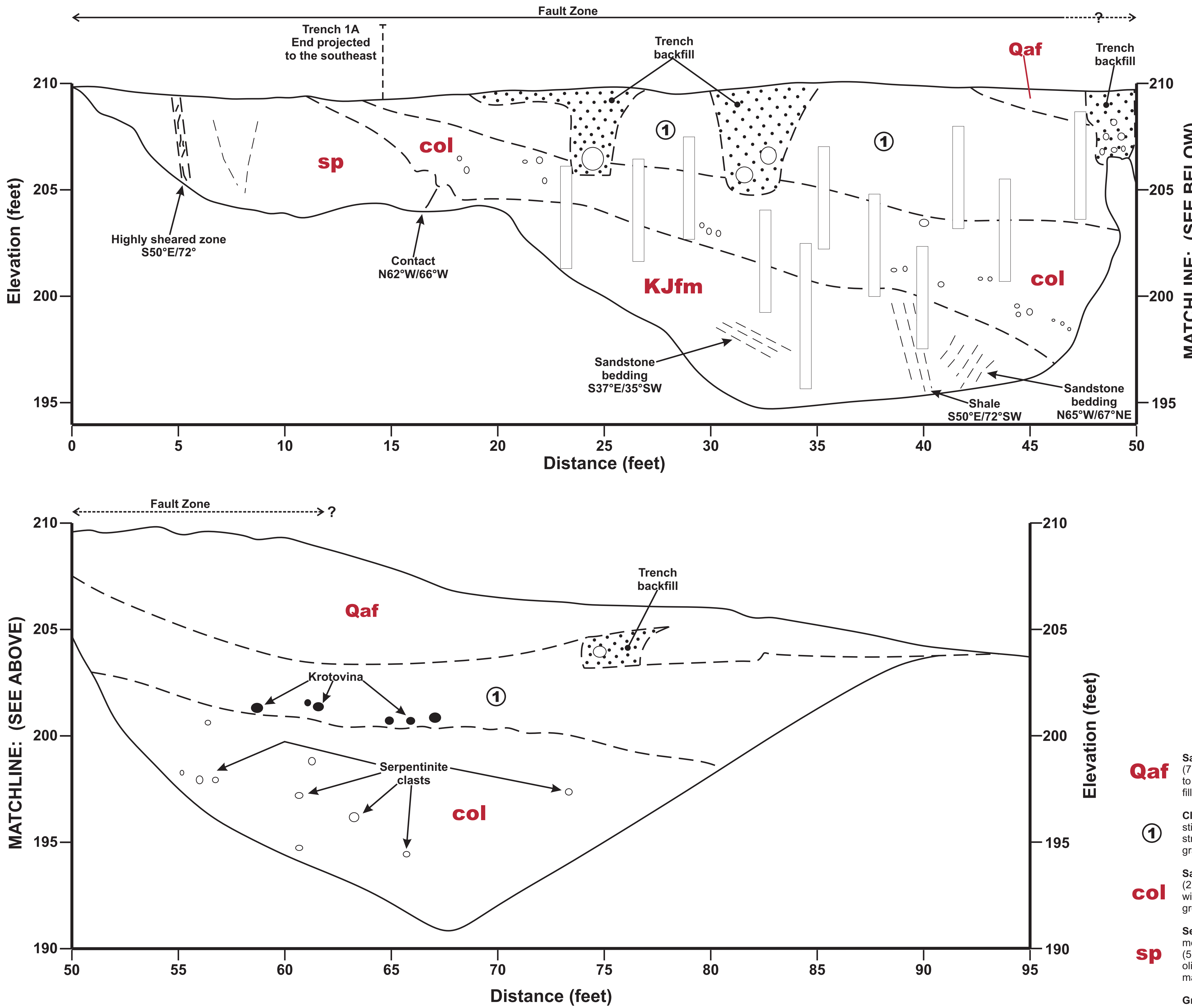
Drawn By RRN

Date November 2016

Trench 1A

455 Piercy Road
San Jose, CA

CORNERSTONE EARTH GROUP



Trench 1B

S 47° W

(Looking Southeast)
1"=5' H:V

Qaf — **Sandy Clay with gravel [CL][fill],** dark brown to dark gray (7.5YR 4/2 and 4/0), dry to moist, friable to hard, lacking structure to granular with some angular gravel, occasional roots; artificial fill and reworked/plowed native soil.

1 — **Clay with Sand [CH] (colluvial),** dark brown (5Y 2.5/2), moist, stiff to hard, medium to high plasticity, blocky to prismatic structure with some coarse sands and occasional sub-angular gravel, some fine roots; A-horizon soils.

col — **Sandy Clay with Gravel [CL] (colluvial),** dark reddish brown (2.5Y 3/2 and 3/4), moist, stiff, medium plasticity, blocky structure with coarse angular to sub-angular gravel (serpentinite, greywacke, shale) with occasional slight downslope imbrication.

sp — **Serpentinite,** low hardness to hard, weak to strong, deep to moderately weathered, light olive gray (5Y 6/1), greenish black (5GY 2/1), yellowish gray (5Y 8/1), and olive gray to moderately olive gray (5Y 3/2 to 5Y 4/4), very little to moderately fractured, massive.

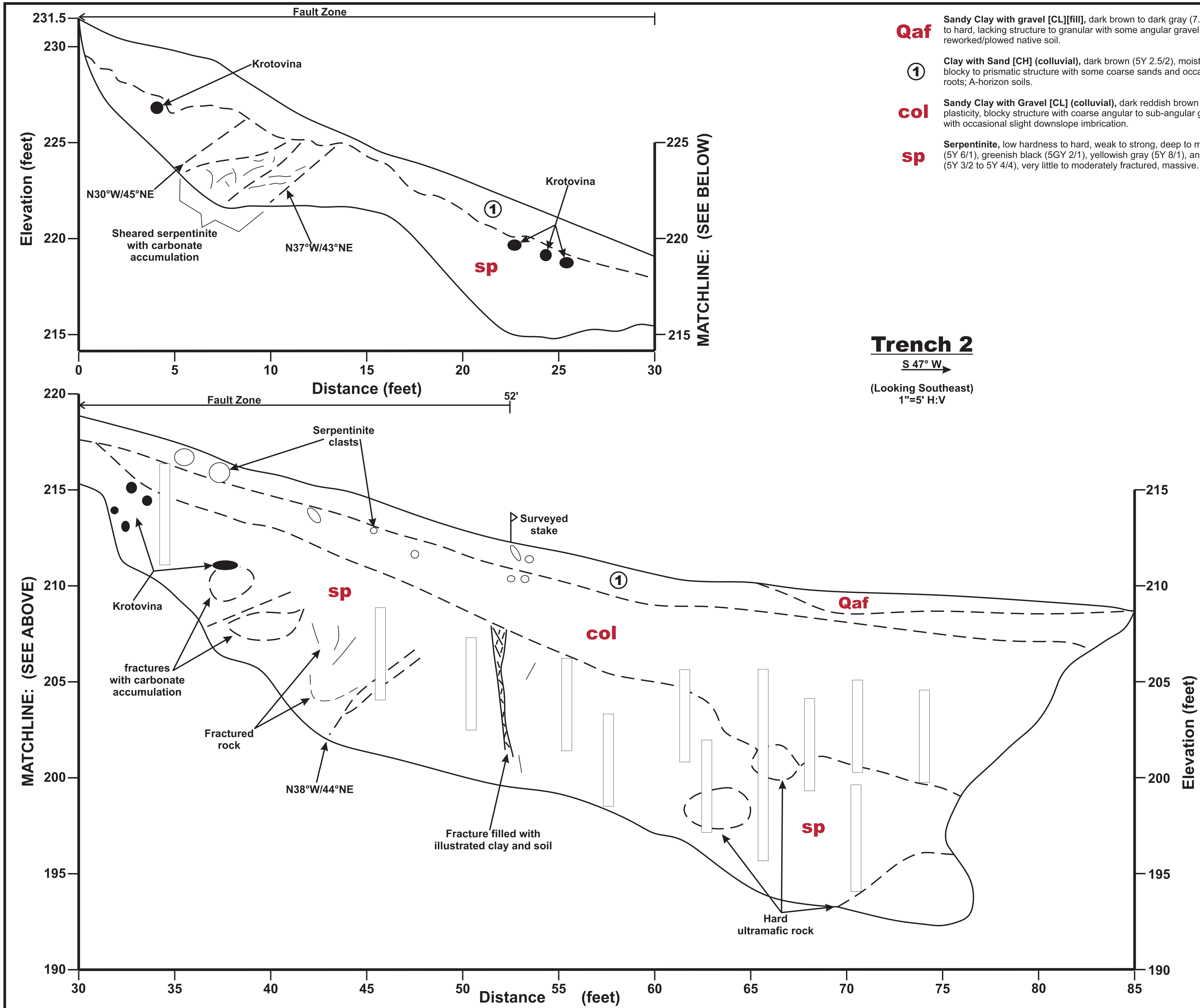
KJfm — **Graywacke and Shale,** low hardness to hard, friable to moderately strong, moderate to little weathering, Grawacke (sandstone): light gray to light olive gray (N7 to 5Y 6/1), Shale: grayish black (N2) and olive black to olive gray (5Y 2/1 to 5Y 4/1), moderately to intensely fractured, Graywacke (sandstone): thick to very thick-bedded, Shale: very thin-bedded; Franciscan Mélange (sandstones and shales).

Project Number: 920-1-1
Figure Number: Figure 5B
Date: November 2016
Drawn By: RRN

Trench 1B

455 Piercy Road
San Jose, CA

CORNERSTONE EARTH GROUP



Qaf **Sandy Clay with gravel [CL][fill],** dark brown to dark gray (7.5YR 4/2 and 4/0), dry to moist, friable to hard, lacking structure to granular with some angular gravel, occasional roots; artificial fill and reworked/plowed native soil.

1 **Clay with Sand [CH] (colluvial),** dark brown (5Y 2.5/2), moist, stiff to hard, medium to high plasticity, blocky to prismatic structure with some coarse sands and occasional sub-angular gravel, some fine roots; A-horizon soils.

col **Sandy Clay with Gravel [CL] (colluvial),** dark reddish brown (2.5Y 3/2 and 3/4), moist, stiff, medium plasticity, blocky structure with coarse angular to sub-angular gravel (serpentinite, greywacke, shale) with occasional slight downslope imbrication.

sp **Serpentinite,** low hardness to hard, weak to strong, deep to moderately weathered, light olive gray (5Y 6/1), greenish black (5GY 2/1), yellowish gray (5Y 8/1), and olive gray to moderately olive gray (5Y 3/2 to 5Y 4/4), very little to moderately fractured, massive.

Trench 2

S 47° W

(Looking Southeast)
1"=5' H:V

Project Number 920-1-1

Figure Number Figure 6

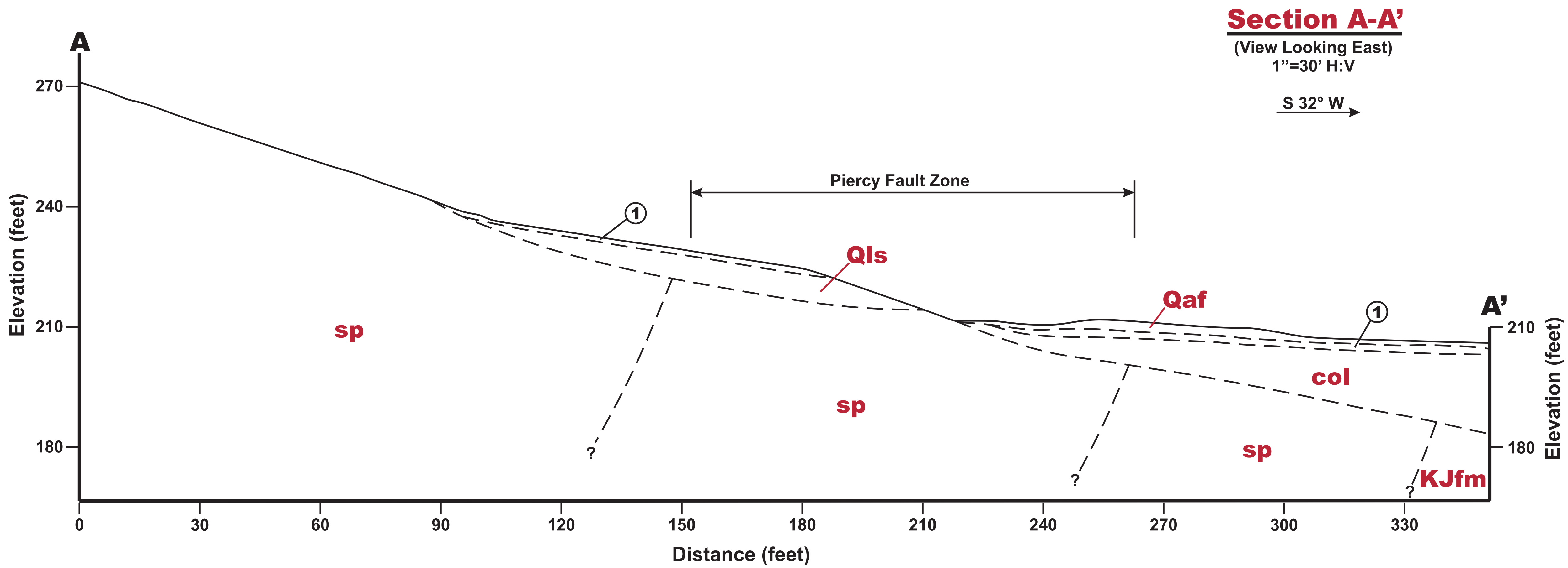
Drawn By RRN

Date November 2016

Trench 2

455 Piercy Road
San Jose, CA

CORNERSTONE EARTH GROUP



Qaf	**Sandy Clay with gravel [CL][fill],** dark brown to dark gray (7.5YR 4/2 and 4/0), dry to moist, friable to hard, lacking structure to granular with some angular gravel, occasional roots; artificial fill and reworked/plowed native soil.
1	**Clay with Sand [CH] (colluvial),** dark brown (5Y 2.5/2), moist, stiff to hard, medium to high plasticity, blocky to prismatic structure with some coarse sands and occasional sub-angular gravel, some fine roots; A-horizon soils.
Qls/col	**Sandy Clay with Gravel [CL] (colluvial),** dark reddish brown (2.5Y 3/2 and 3/4), moist, stiff, medium plasticity, blocky structure with coarse angular to sub-angular gravel (serpentinite, greywacke, shale) with occasional slight downslope imbrication.
sp	**Serpentinite,** low hardness to hard, weak to strong, deep to moderately weathered, light olive gray (5Y 6/1), greenish black (5GY 2/1), yellowish gray (5Y 8/1), and olive gray to moderately olive gray (5Y 3/2 to 5Y 4/4), very little to moderately fractured, massive.
KJfm	**Graywacke and Shale,** low hardness to hard, friable to moderately strong, moderate to little weathering, Grawacke (sandstone): light gray to light olive gray (N7 to 5Y 6/1), Shale: grayish black (N2) and olive black to olive gray (5Y 2/1 to 5Y 4/1), moderately to intensly fractured, Graywacke (sandstone): thick to very thick-bedded, Shale: very thin-bedded; Franciscan Mélange (sandstones and shales).

Notes:
1) Surficial fills associated with existing pavements, landscaping or utilities are not shown.
2) The subsurface profile is conceptual and is based on limited subsurface data obtained from widely spaced borings. Actual subsurface conditions may vary significantly between borings.
3) See Figure 3 for location of cross section.

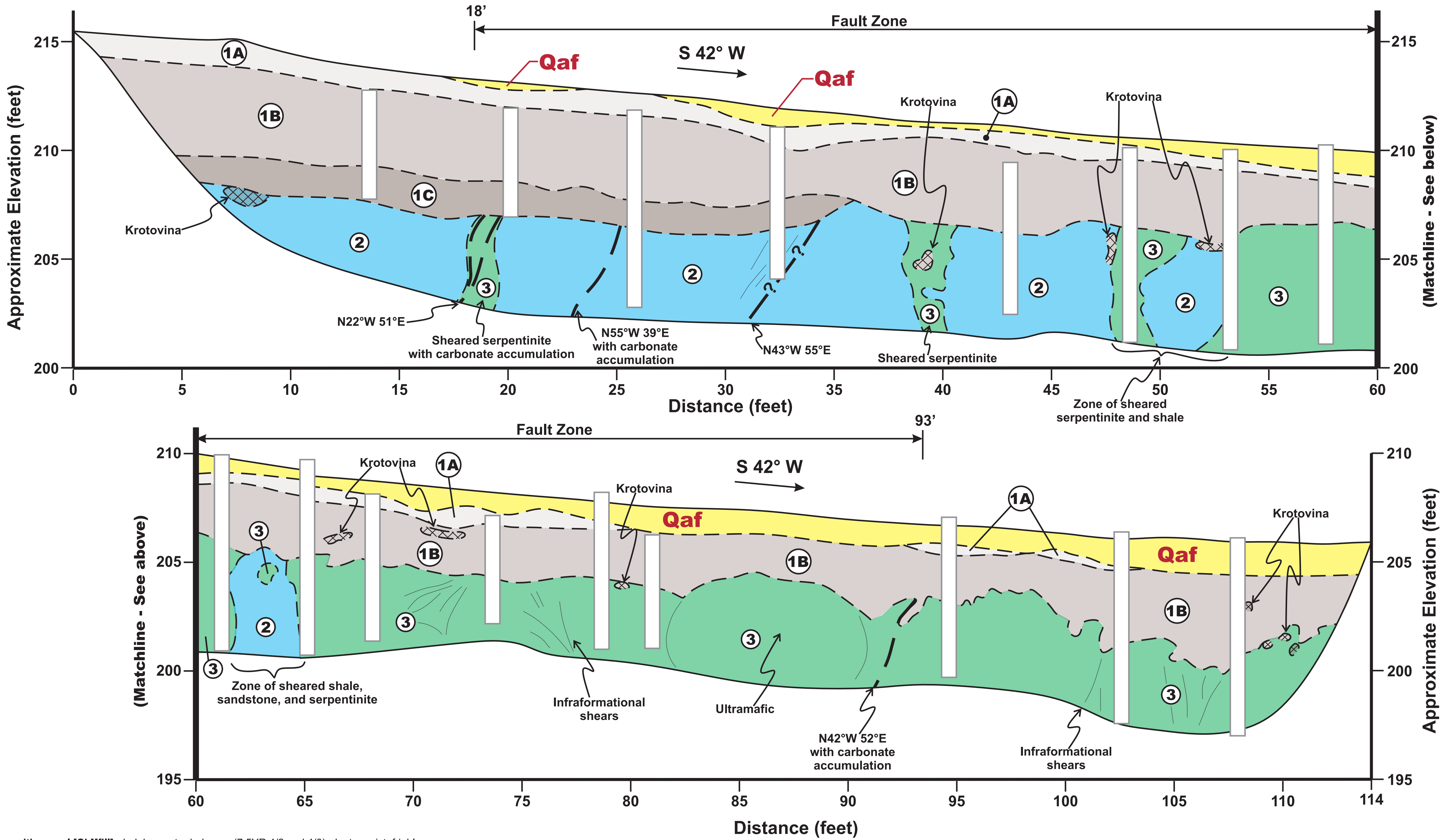
Project Number 920-1-1
Figure Number Figure 7
Drawn By RRN
Date November 2016

Geologic Cross Section A-A'

455 Piercy Road
San Jose, CA

CORNERSTONE EARTH GROUP

APPENDIX A: PREVIOUS INVESTIGATION TRENCH LOGS



Qaf **Sandy Clay with gravel [CL][fill],** dark brown to dark gray (7.5YR 4/2 and 4/0), dry to moist, friable to hard, lacking structure to granular with some angular gravel, occasional roots; artificial fill and reworked/plowed native soil.

1A **Clay with Sand [CL/CH] (colluvial),** dark brown (2.5YR 3.2), moist, stiff to hard, low to medium plasticity, granular to blocky structure with occasional sands and sub-rounded to sub-angular gravel (greywacke and serpentinite), some fine and coarse roots; A-horizon soils.

1B **Clay with Sand [CH] (colluvial),** dark brown (5Y 2.5/2), moist, stiff to hard, medium to high plasticity, blocky to prismatic structure with some coarse sands and occasional sub-angular gravel, some fine roots; B-horizon soils.

1C **Sandy Clay with Gravel [CL] (colluvial),** dark reddish brown (2.5Y 3/2 and 3/4), moist, stiff, medium plasticity, blocky structure with coarse angular to sub-angular gravel (serpentinite, greywacke, shale) with occasional slight downslope imbrication; B-horizon soils.

2 **Grawacke and Shale,** low hardness to hard, friable to moderately strong, moderate to little weathering, Grawacke (sandstone): light gray to light olive gray (N7 to 5Y 6/1), Shale: grayish black (N2) and olive black to olive gray (5Y 2/1 to 5Y 4/1), moderately to intensly fractured, Graywacke (sandstone): thick- to very thick-bedded, Shale: very thin-bedded; Franciscan Mélange (sandstones and shales).

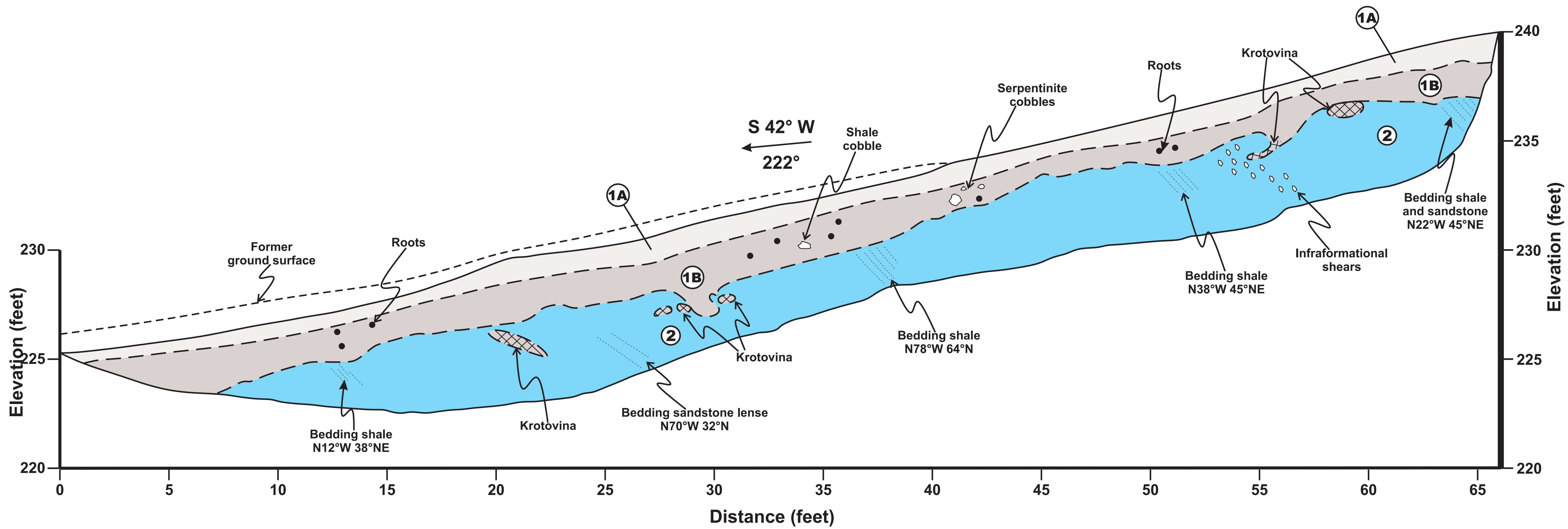
3 **Serpentinite,** low hardness to hard, weak to strong, deep to moderately weathered, light olive gray (5Y 6/1), greenish black (5GY 2/1), yellowish gray (5Y 8/1), and olive gray to moderately olive gray (5Y 3/2 to 5Y 4/4), very little to moderately fractured, massive.

Trench 1

(View Looking Southeast)
1"=5' H:V

CORNERSTONE EARTH GROUP

Trench 1	Project Number 336-7-1
Silver Creek Valley Road Development San Jose, CA	Figure Number Figure 4A
	Date October 2015 — Drawn By RRN



Trench 2

(View Looking Northwest)
1"=5' H:V

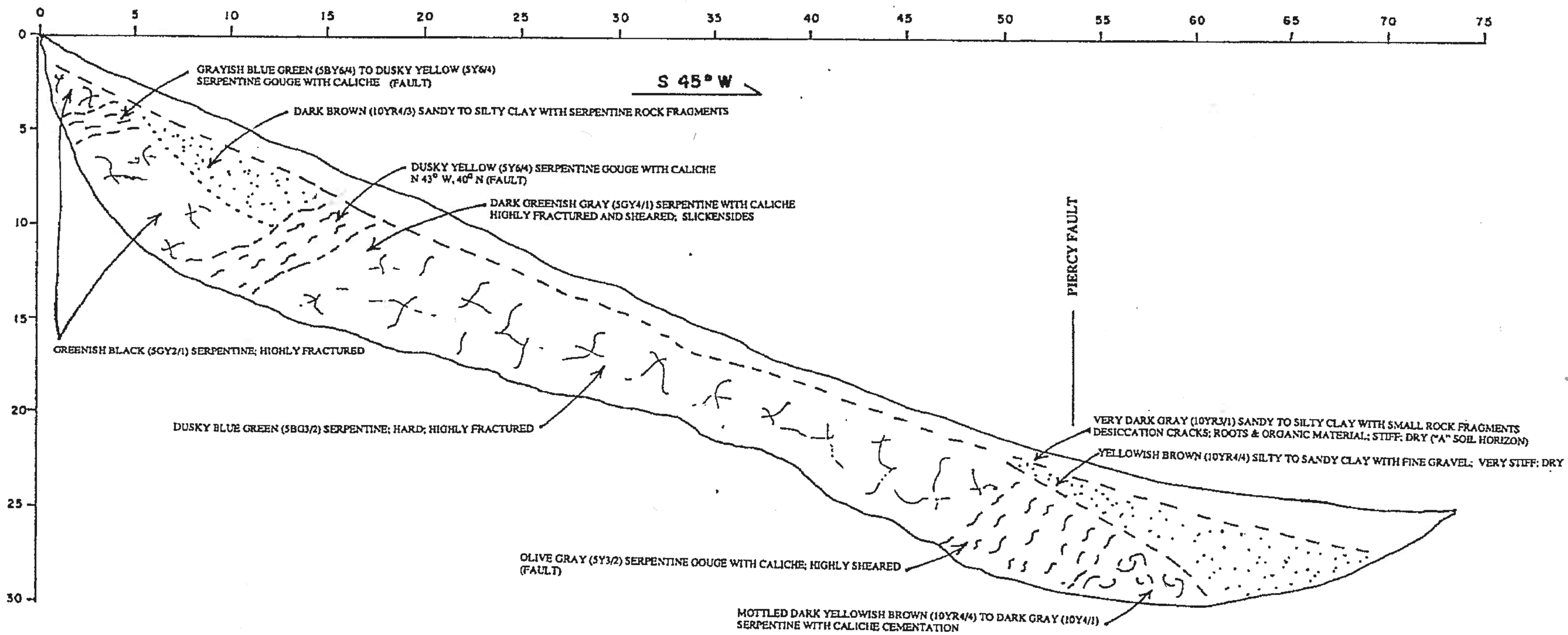
1A **Clay with Sand [CL/CH] (colluvial),** dark brown (2.5YR 3.2), moist, stiff to hard, low to medium plasticity, granular to blocky structure with occasional sands and sub-rounded to sub-angular gravel (greywacke and serpentinite), some fine and coarse roots; A-horizon soils.

1B **Clay with Sand [CH] (colluvial),** dark brown (5Y 2.5/2), moist, stiff to hard, medium to high plasticity, blocky to prismatic structure with some coarse sands and occasional sub-angular gravel, some fine roots; B-horizon soils.

2 **Grawacke and Shale,** low hardness to hard, friable to moderately strong, moderate to little weathering, Grawacke (sandstone): light gray to light olive gray (N7 to 5Y 6/1), Shale: grayish black (N2) and olive black to olive gray (5Y 2/1 to 5Y 4/1), moderately to intensly fractured, Graywacke (sandstone): thick- to very thick-bedded, Shale: very thin-bedded; Franciscan Mélange (sandstones and shales).

CORNERSTONE EARTH GROUP

Trench 2	Project Number 336-7-1
Silver Creek Valley Road Development San Jose, CA	Figure Number Figure 4B
	Date September 2015 / Drawn By RRN



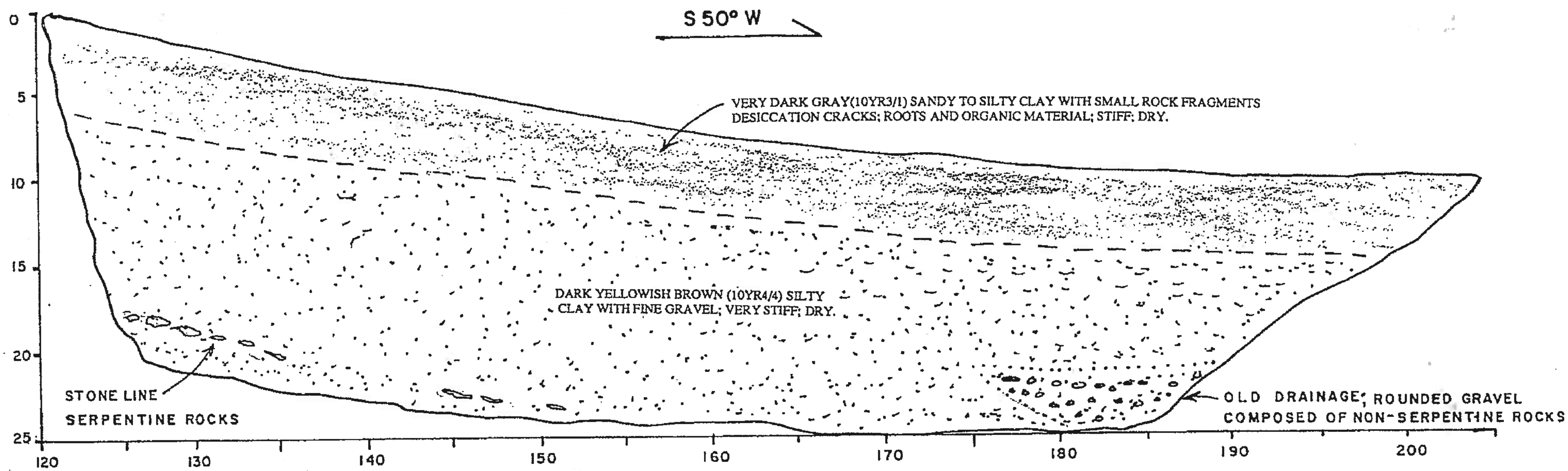
NOTES:

1. EXPLORATORY TRENCH EXCAVATED WITH A BACKHOE ON 5/16/00.
2. FREE WATER WAS NOT ENCOUNTERED TO THE DEPTH EXCAVATED.
3. EXPLORATORY TRENCH LOOSELY BACKED FILLED AT THE COMPLETION OF LOGGING.

EXPLORATORY TRENCH ET-2
HORIZONTAL AND VERTICAL SCALE IN FEET

PLATE 4-B

HYDRO-GEO CONSULTANTS, INC.



EXPLORATORY TRENCH LOG ET-3

HORIZONTAL AND VERTICAL SCALE IN FEET

PLATE 2

NOTES:

1. EXPLORATORY TRENCH EXCAVATED WITH AN EXCAVATOR ON 9/11/00.
2. FREE WATER WAS NOT ENCOUNTERED TO THE DEPTH EXCAVATED.
3. THE EXPLORATORY TRENCH WAS LEFT OPEN AND IS TO BE REFILLED BY THE PROPERTY OWNER.
4. THE TRENCH BACKFILL MUST BE ADEQUATELY COMPACTED TO MITIGATE POTENTIAL GROUND SETTLEMENT.

HYDRO-GEO CONSULTANTS, INC.

JOB NO: 105211

Qaf - Silty clay, black, dry, low plasticity, soft; some roots, contains some angular clasts of concrete up to one foot: artificial fill.

A - Silty clay, dark gray to black, dry to slightly damp, medium plasticity, soft to firm; abundant roots: A-soils horizon?

B - Silty clay, greenish-gray to dark brown to black, dry to slightly moist, low to high plasticity, soft to firm: B-soils horizon?

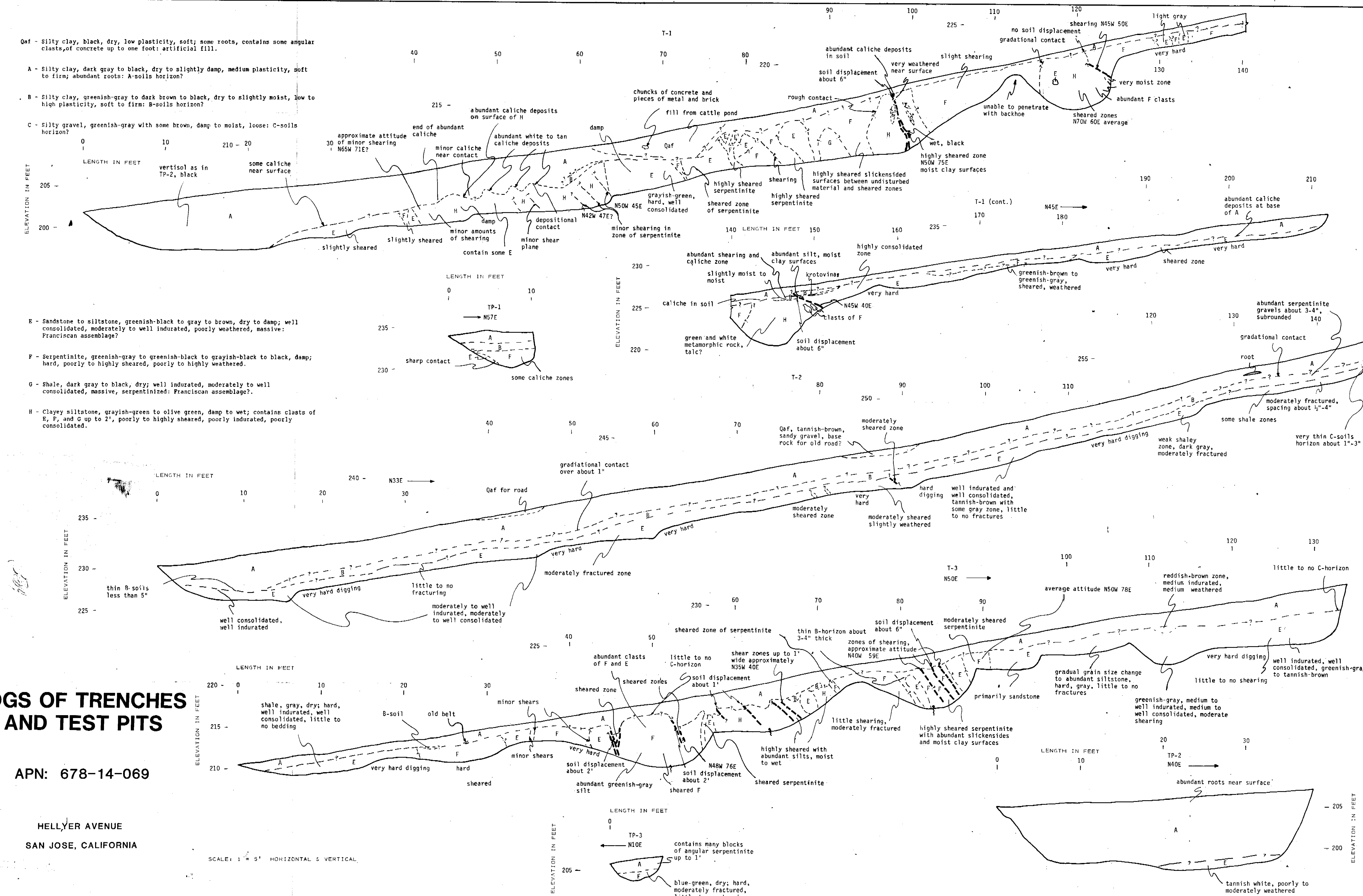
C - Silty gravel, greenish-gray with some brown, damp to moist, loose: C-soils horizon?

E - Sandstone to siltstone, greenish-black to gray to brown, dry to damp; well consolidated, moderately to well indurated, poorly weathered, massive: Franciscan assemblage?

F - Serpentinite, greenish-gray to greenish-black to grayish-black to black, damp; hard, poorly to highly sheared, poorly to highly weathered.

G - Shale, dark gray to black, dry; well indurated, moderately to well consolidated, massive, serpentinized: Franciscan assemblage?.

H - Clayey siltstone, grayish-green to olive green, damp to wet; contains clasts of E, F, and G up to 2', poorly to highly sheared, poorly indurated, poorly consolidated.



LOGS OF TRENCHES AND TEST PITS

APN: 678-14-069

HELLYER AVENUE

SAN JOSE, CALIFORNIA

ASSOCIATED TERRA CONSULTANTS

ENGINEERING GEOLOGY-ENVIRONMENTAL SERVICES

APPENDIX D: PREVIOUS EXPLORATION DATA

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS		LTR	ID	DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY	GW		Well-graded gravels or gravel with sand, little or no fines.
		GP		Poorly-graded gravels or gravel with sand, little or no fines.
		GM		Silty gravels, silty gravel with sand mixture.
		GC		Clayey gravels, clayey gravel with sand mixture.
	SAND AND SANDY	SW		Well-graded sands or gravelly sands, little or no fines.
		SP		Poorly-graded sands or gravelly sands, little or no fines.
		SM		Silty sand.
		SC		Clayey sand.

MAJOR DIVISIONS		LTR	ID	DESCRIPTION
FINE GRAINED SOILS	SILTS AND CLAYS	ML		Inorganic silts and very fine sands, rock flour or clayey silts with slight plasticity.
		CL		Inorganic lean clays of low to medium plasticity, gravelly clays, sandy clays, silty clays.
		OL		Organic silts and organic silt-clays of low plasticity.
	SILTS AND CLAYS	MH		Inorganic elastic silts, micaceolus or diatomaceous or silty soils.
		CH		Inorganic fat clays (high plasticity).
		OH		Organic clays of medium high to high plasticity.
HIGHLY ORGANIC SOILS		Pt		Peat and other highly organic soils.

Standard Penetration Split Spoon Sampler 2.0 inch, 1.4 inch I.D.

Modified California Sampler 2.5 inch O.D., 2.0 inch I.D.

Bulk Sample

California Sampler, 3.0 inch O.D., 2.5 inch I.D.

Shelby Tube 3.0 inch O.D.

0800, 5/31 Approximate water level first observed in boring. Time recorded in reference to a 24 hour clock.

0800, 5/31 Approximate water level observed in boring following drilling

PEN	Pocket Pentrometer reading, in tsf
TV:Su	Torvane shear strength, in ksf

LL	LIQUID LIMIT	TX	TRIAXIAL SHEAR
PI	PLASTICITY INDEX	CONSOL	CONSOLIDATION
%-#200	SIEVE ANALYSIS (#200 SCREEN)	R-Value	RESISTANCE VALUE
DS	DIRECT SHEAR	SE	SAND EQUIVALENT
C	COHESION (PSF)	EI	EXPANSION EQUIVLANT
PHI	FRCITION ANGLE	FS	FREE SWELL (U.S.B.R.)

Notes: Blow counts represent the number of blows a 140-pound hammer falling 30 inches required to drive a sampler through the last 12 inches of an 18 inch penetration, unless otherwise noted.

The lines separating strata on the logs represent approximate boundaries only. The actual transition may be gradual. No warranty is provided as to the continuity of soil strata between borings. Logs represent the soil section observed at the boring location on the date of drilling only.

KLEINFELDER

PROJECT NO. 12-3077-00/GEO

BORING LOG LEGEND

Sliver Creek Valley Corporate Center
Piercy Road
San Jose, California

PLATE **A-1**

10/20/2000 3:27:27 PM

Date Completed:	10/6/00	*Sampler:*	Modified California Sampler 2.5 inch O.D., 2.0 inch I.D.
Logged By:	B.Buhowsky	*Driller:*	Exploration Geoservices
Total Depth:	26.5 ft	*Method:*	6" Hollow Stem Auger
Hammer Wt:	140 lbs., 30" drop		

Depth,ft	FIELD Sample	FIELD Blows/ft	LABORATORY Dry Density pcf	LABORATORY Moisture Content %	LABORATORY Compress. Strength tsf	LABORATORY Other Tests	Pen, tsf	DESCRIPTION — *Surface Elevation:* Estimated 240 feet (Above MSL)
0								**SANDY LEAN CLAY (CL)** dark brown, very stiff, contains subangular, serpentine gravels, damp
5	■	56						**LEAN CLAY (CL)** with silt, sand and gravel, dark brown, hard, sand and gravel fragments are composed of serpentine and silica carbonate, serpentine gravels up to 1.5" diameter, moist
10	■	59						
11								**SERPENTINE (SERP)** greenisgh blue, highly weathered, moist, mottled, moderately strong to weak
15	■	49						
20	■	84						**SERPENTINE** bluish gray, dense, moderately weathered, weak, moist
25	■	75						
26.5								Boring terminated @ 26.5 No groundwater encountered
30								

KLEINFELDER

LOG OF BORING NO. K-1

Sliver Creek Valley Corporate Center
Piercy Road
San Jose, California

PLATE A-2

PROJECT NO. 12-3077-00/GEO

Date Completed:	10/6/00	*Sampler:*	Modified California Sampler 2.5 inch O.D., 2.0 inch I.D.
Logged By:	B. Buhowsky	*Driller:*	Exploration Geoservices
Total Depth:	20.5 ft	*Method:*	6" Hollow Stem Auger
Hammer Wt:	140 lbs., 30" drop		

Depth, ft	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density pcf	LABORATORY: Moisture Content %	LABORATORY: Compress. Strength tsf	LABORATORY: Other Tests	Pen, tsf	DESCRIPTION — *Surface Elevation:* Estimated 228 feet (Above MSL)
0								**LEAN CLAY (CL)** brown, with silt, sand and gravel, hard, contains serpentine and silica carbonate fragments, damp
5	■	39						
10	■	40						Serpentine gravels are subangular
15	■	82						
~16.5								**SERPENTINE (SERP)** - greenish gray,dense, weak, mottled, moist
20	■	50/5"						
20.5								Boring terminated @ 20.5 No groundwater encountered
25								
30								

KLEINFELDER

LOG OF BORING NO. K-1A

Sliver Creek Valley Corporate Center
Piercy Road
San Jose, California

PLATE A-3

PROJECT NO. 12-3077-00/GEO

C:\PROGRA~1\GINTW\PROJECTS\12307700.GPJ

Date Completed:	10/6/00	*Sampler:*	Modified California Sampler 2.5 inch O.D., 2.0 inch I.D.
Logged By:	B. Buhowsky	*Driller:*	Exploration Geoservices
Total Depth:	24.0 ft	*Method:*	6" Hollow Stem Auger
Hammer Wt:	140 lbs., 30" drop		

Depth, ft	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density pcf	LABORATORY: Moisture Content %	LABORATORY: Compress. Strength tsf	LABORATORY: Other Tests	Pen, tsf	DESCRIPTION — *Surface Elevation:* Estimated 232 feet (Above MSL)
								LEAN CLAY (CL) dark brown, with sand, very stiff, slightly prismatic, moist
	■	29						SANDY LEAN CLAY (CL) olive, hard, sand grains are composed of serpentine, moist
5								
	■	62						
10								
								subangular serpentine gravels present within
	■	93/10"						SANDY LEAN CLAY (CL) olive, hard, sand grains are composed of serpentine, moist
15								
	■	70	89	29				
20								SERPENTINE (SERP) greenish gray,dense, weak, mottled, moist
	■	97	106	13				
								Boring terminated @ 24 No groundwater encountered
25								
30								

KLEINFELDER

LOG OF BORING NO. K-2

Sliver Creek Valley Corporate Center
Piercy Road
San Jose, California

PLATE A-4

PROJECT NO. 12-3077-00/GEO

C:\PROGRA~1\GINTW\PROJECTS\1230700.GPJ

PRIMARY DIVISIONS			GROUP SYMBOL	SECONDARY DIVISIONS
COARSE GRAINED SOILS — MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS — MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (LESS THAN 5% FINES)	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GRAVEL WITH FINES	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines.
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines.
	SANDS — MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (LESS THAN 5% FINES)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES	SM	Silty sands, sand-silt mixtures, non-plastic fines
			SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS — MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS — LIQUID LIMIT IS LESS THAN 50%		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS — LIQUID LIMIT IS GREATER THAN 50%		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils

DEFINITION OF TERMS

SANDS AND GRAVELS	BLOWS/FOOT†
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

RELATIVE DENSITY

SILTS AND CLAYS	STRENGTH‡	BLOWS/FOOT†
VERY SOFT	0 - 1/4	0 - 2
SOFT	1/4 - 1/2	2 - 4
FIRM	1/2 - 1	4 - 8
STIFF	1 - 2	8 - 16
VERY STIFF	2 - 4	16 - 32
HARD	OVER 4	OVER 32

CONSISTENCY

† Number of blows of 140 pound hammer falling 30 inches to drive a 2 inch O.D. (1-3/8 inch I.D.) split spoon (ASTM D-1586).

‡ Unconfined compressive strength in tons/sq. ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D-1586), pocket penetrometer, torvane, or visual observation.

U.S. STANDARD SERIES SIEVE: 200, 40, 10, 4 — CLEAR SQUARE SIEVE OPENINGS: 3/4", 3", 12"

SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

GRAIN SIZES

Figure 3-Unified Soil Classification

AES Eng.

Date Drilled 3/6/00	Logged By : M.M.				Boring No. B-1			
							Direct Shear	
DESCRIPTION	Sample	Depth (feet)	Sample No.	Dry Density (p.c.f)	Water Content %	Penetration Resistance (Blows/Foot)	"0" Degree	"C" Cohesion
2-4" organic, top soil Black Clay, moist to wet, firm, plastic Dark Brown, Silty Clay, moist and stiff to very stiff CL Colluvial		3	1-1	89.3	26.4	21		
Brown, Sandy, Silty Clay to Silty Sandy Clay moist and very stiff ML		8	1-2	91.7	22.6	27		
Same Brown, Sandy Clay w/minor gravel moist to damp, hard CL/ML Rock at the shoe Franciscan Complex More gravel and rock		13	1-3	106.8	14.6	50/6"		
Multi color, Rock, Gravel and Sand, very dense, damp GP Bedrock material		18	1-4	111.7	12.2	73		
Gravel and Sand, damp and very dense GP		23	1-5	114.4	13.5	71		
Boring Terminated @ 25-feet Groundwater was not encountered								

EXPLORATION BORING LOG		AES ENGINEERING SANTA CLARA, CALIFORNIA
Project No. 00-255-S	Figure : 4	

Date Drilled 3/6/00		Logged By : M.M.			Boring No. B-2			
							Direct Shear	
DESCRIPTION	Sample	Depth (feet)	Sample No.	Dry Density (p.c.f)	Water Content %	Penetration Resistance (Blows/Foot	"0" Degree	"C" Cohesion
2-4" organic, top soil Dark Brown, Silty Clay, moist to wet, firm, plastic CL								
Dark Brown, Silty Clay, damp to moist and very stiff CL Lighter color		4	2-1	107.9	16.0	34		
Light Brown, Sandy, Silty Clay damp to moist and very stiff ML/CL Minor gravel, 1/2" in size		9	2-2	97.1	18.0	31		
Olive Brown, Sandy, Silty Clay w/minor gravel moist, stiff to very stiff CL/ML Hard to drill		14	2-3	102.3	19.6	26		
Mottled Brown, Gravely, Sandy Clay moist to damp, hard GP rock fragment, hard to drill Boring Terminated @ 22-feet Groundwater was not encountered		19	2-4	101.0	17.7	54		

EXPLORATION BORING LOG	AES ENGINEERING SANTA CLARA, CALIFORNIA
Project No. 00-255-S	Figure : 5

Date Drilled 3/6/00	Logged By : M.M.				Boring No. B-3			
DESCRIPTION	Sample	Depth (feet)	Sample No.	Dry Density (p.c.f)	Water Content %	Penetration Resistance (Blows/Foot	Direct Shear "0" Degree	Direct Shear "C" Cohesion
2-4" organic, top soil, Black Clay Dark Brown, Silty Clay, moist to wet, firm, plastic CL								
Dark Brown, Silty Clay, damp to moist and very stiff CL Colluvial		5	3-1	93.8	21.6	21		
Dark Brown, Sandy Clay to Clayey Sand damp to moist and very hard ML/CL Rock at the shoe Serpentine Rock Bedrock, Rock fragments		10	3-2	104.7	16.1	50/5"		
Dark Green, gravel and Rock, damp and very dense GP Very hard to drill Bedrock refusal Boring Terminated @ 18.5-feet Groundwater was not encountered		15	3-3	120.1	11.7	50/3"		

EXPLORATION BORING LOG

Project No. 00-255-S

Figure : 6

AES ENGINEERING

SANTA CLARA, CALIFORNIA

Date Drilled 3/6/00	Logged By : M.M.				Boring No. B-4			
							Direct Shear	
DESCRIPTION	Sample	Depth (feet)	Sample No.	Dry Density (p.c.f)	Water Content %	Penetration Resistance (Blows/Foot)	"0" Degree	"C" Cohesion
2-4" organic, top soil, Black Clay Dark Brown, Silty Clay, moist to wet, firm, plastic CL								
Dark Brown, Silty Clay, damp to moist and very stiff CL Color change minor small size angular gravel		5	4-1	102.8	19.5	32	11.0	1200 ps
Light Brown, Gravely, Sandy Clay damp to moist and very hard ML/CL		10	4-2	104.7	17.2	44		
Serpentine Rocks Bedrock, Rock fragments Black to Dark Green, gravel and Rock, damp and very dense GP Very hard to drill		15	4-3	118.6	6.9	56		
Dark Green, Rock, Gravel and Sand damp, very dense Boring Terminated @ 22-feet Groundwater was not encountered		20	4-4	122.9	8.6	50/4"		

EXPLORATION BORING LOG	AES ENGINEERING SANTA CLARA, CALIFORNIA
Project No. 00-255-S	Figure : 7

Date Drilled 3/6/00	Logged By : M.M.						Boring No. B-5	
DESCRIPTION	Sample	Depth (feet)	Sample No.	Dry Density (p.c.f)	Water Content %	Penetration Resistance (Blows/Foot)	Direct Shear "0" Degree	Direct Shear "C" Cohesion
2-4" organic, top soil Black Clay, moist to wet, firm, plastic Dark Brown to Black, Silty Clay, moist and stiff CL Color Change		3	5-1	85.4	22.7	14		
Light Brown, Sandy, Silty Clay moist and very stiff ML minor gravel		8	5-2	101.6	19.1	27		
Harder to drill Light Brown, Silty, Sandy Clay w/minor gravel moist to damp, hard CL/ML Rock at the shoe Hard to drill More gravel and rock		13	5-3	102.8	17.3	61		
Multi color, Rock, Gravel and Sand, very dense, damp GP Bedrock material		18	5-4	111.7	12.2	65		
Gravel and Sand, damp and very dense GP Rock Fragments		23	5-5	117.8	9.3	78		
Multi Color, Gravel, Sand and rock Fragments Bedrock Boring Terminated @ 31-feet Groundwater was not encountered		28	5-6	121.4	7.8	50/3.5"		

EXPLORATION BORING LOG

Project No. 00-255-S Figure : 8

AES ENGINEERING

SANTA CLARA, CALIFORNIA

Date Drilled 3/6/00	Logged By : M.M.	Boring No. B-6

DESCRIPTION	Sample	Depth (feet)	Sample No.	Dry Density (p.c.f)	Water Content %	Penetration Resistance (Blows/Foot	Direct Shear "0" Degree	Direct Shear "C" Cohesion
2-4" organic, top soil Black to Dark Brown, Silty Clay, moist to wet, firm, plastic CL								
Dark Brown, Silty Clay, moist and stiff CL Colluvial Same		4	6-1	98.9	18.6	27		
Light Brown, Sandy, Silty Clay damp to moist and very stiff ML/CL Minor angular gravel, 1/2" in size		9	6-2	103.1	15.5	26		
Mottled Brown, Sandy, Silty Clay w/minor gravel moist, stiff to very stiff CL/ML Serpentine Rock Fragments Harder to drill		14	6-3	106.6	14.3	44		
Dark Green, Rock, Sand & Gravel w/ Clay binder moist to damp, hard GP rock at shoe, hard to drill Boring Terminated @ 21-feet Groundwater was not encountered		19	6-4	115.3	9.5	50/5"		

EXPLORATION BORING LOG

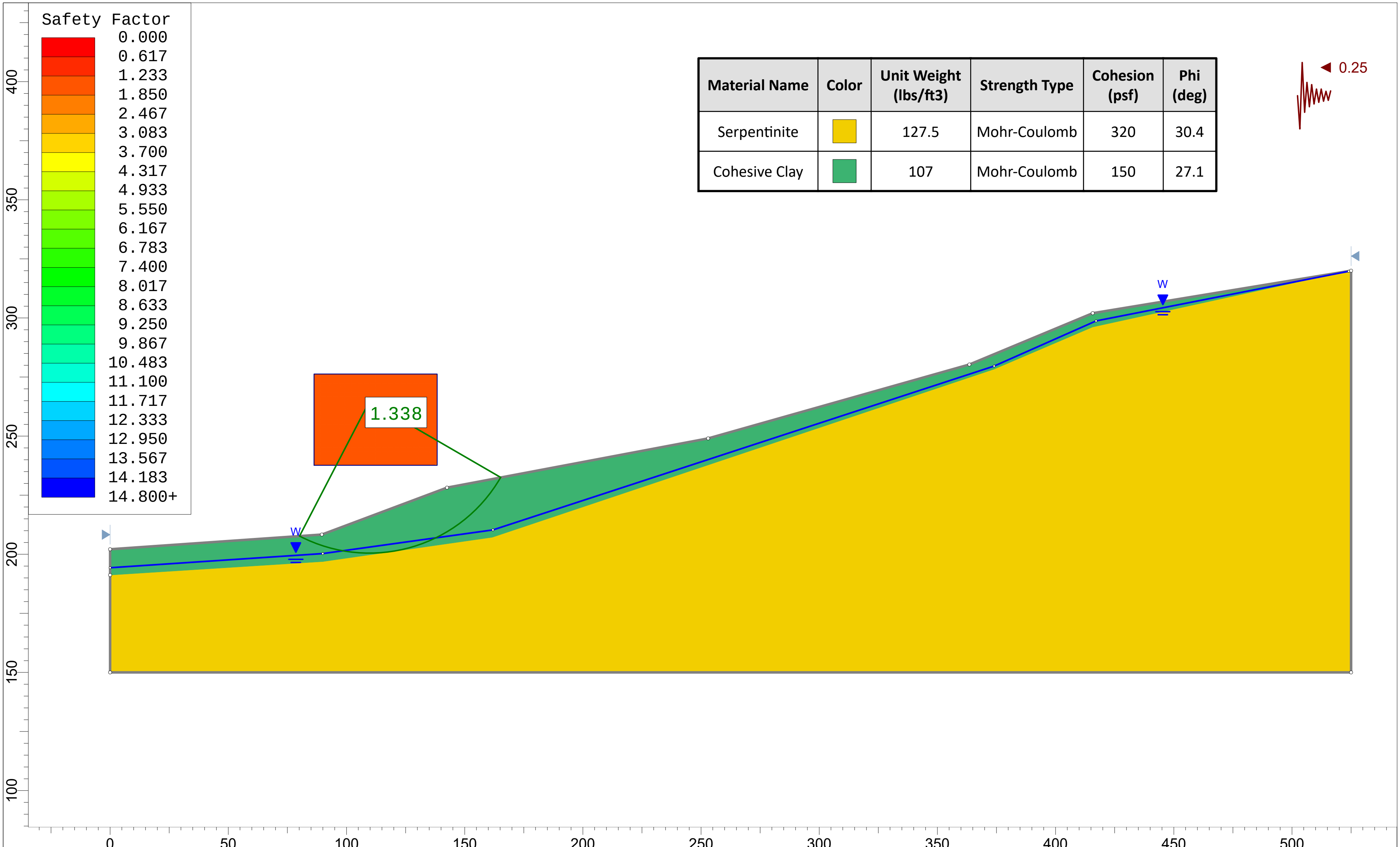
Project No. 00-255-S

Figure : 9

AES ENGINEERING

SANTA CLARA, CALIFORNIA

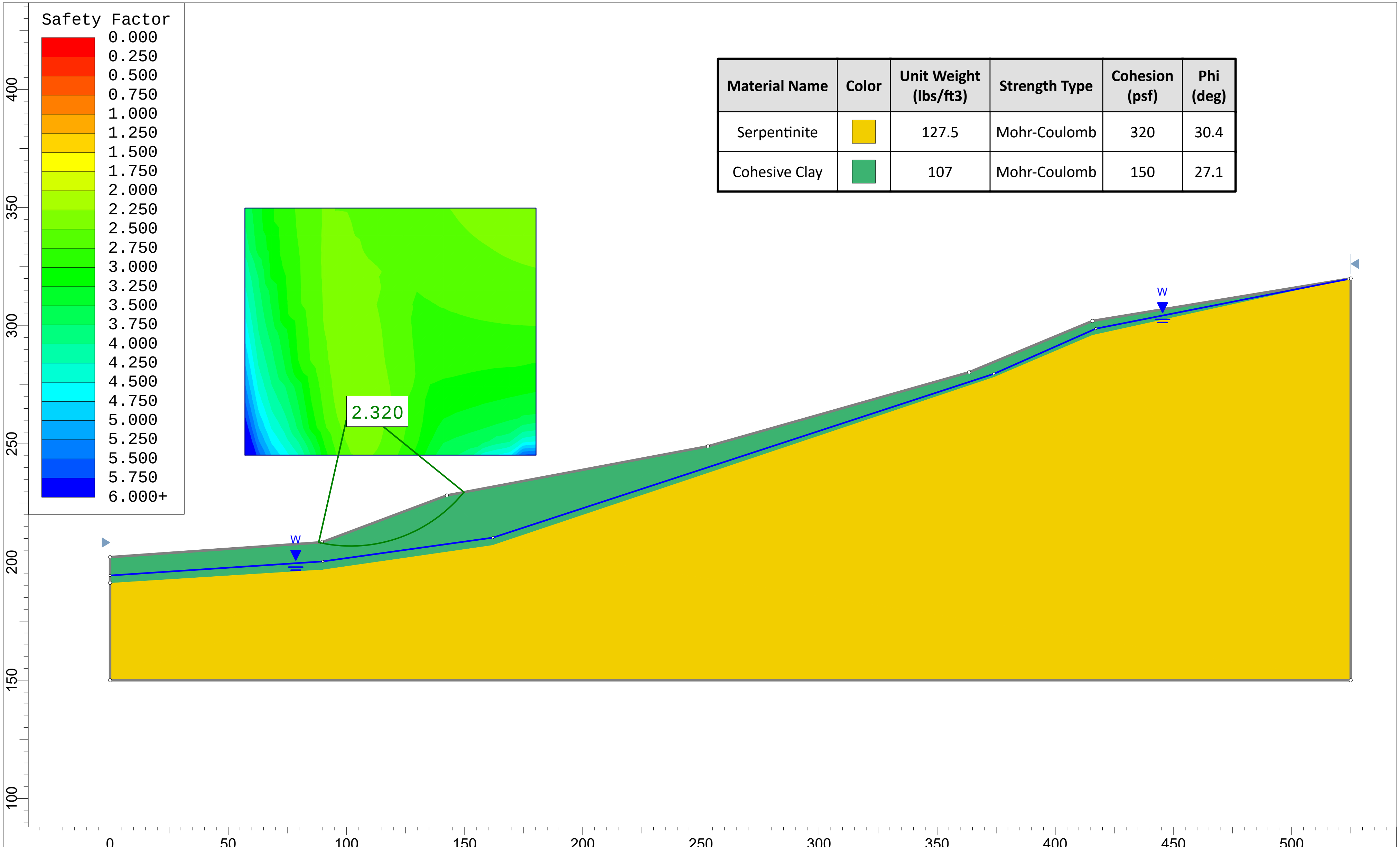
APPENDIX E: SLOPE STABILITY ANALYSIS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Serpentinite		127.5	Mohr-Coulomb	320	30.4
Cohesive Clay		107	Mohr-Coulomb	150	27.1

Project	455 Piercy Road, San Jose CA
Analysis Description	Slope Stability
Drawn By	AA/CB
Scale	1:674
Company	CEG
Date	11/6/2017, 10:35:18 AM
File Name	Section AA' Seismic.slim

SLIDEINTERPRET 7.023

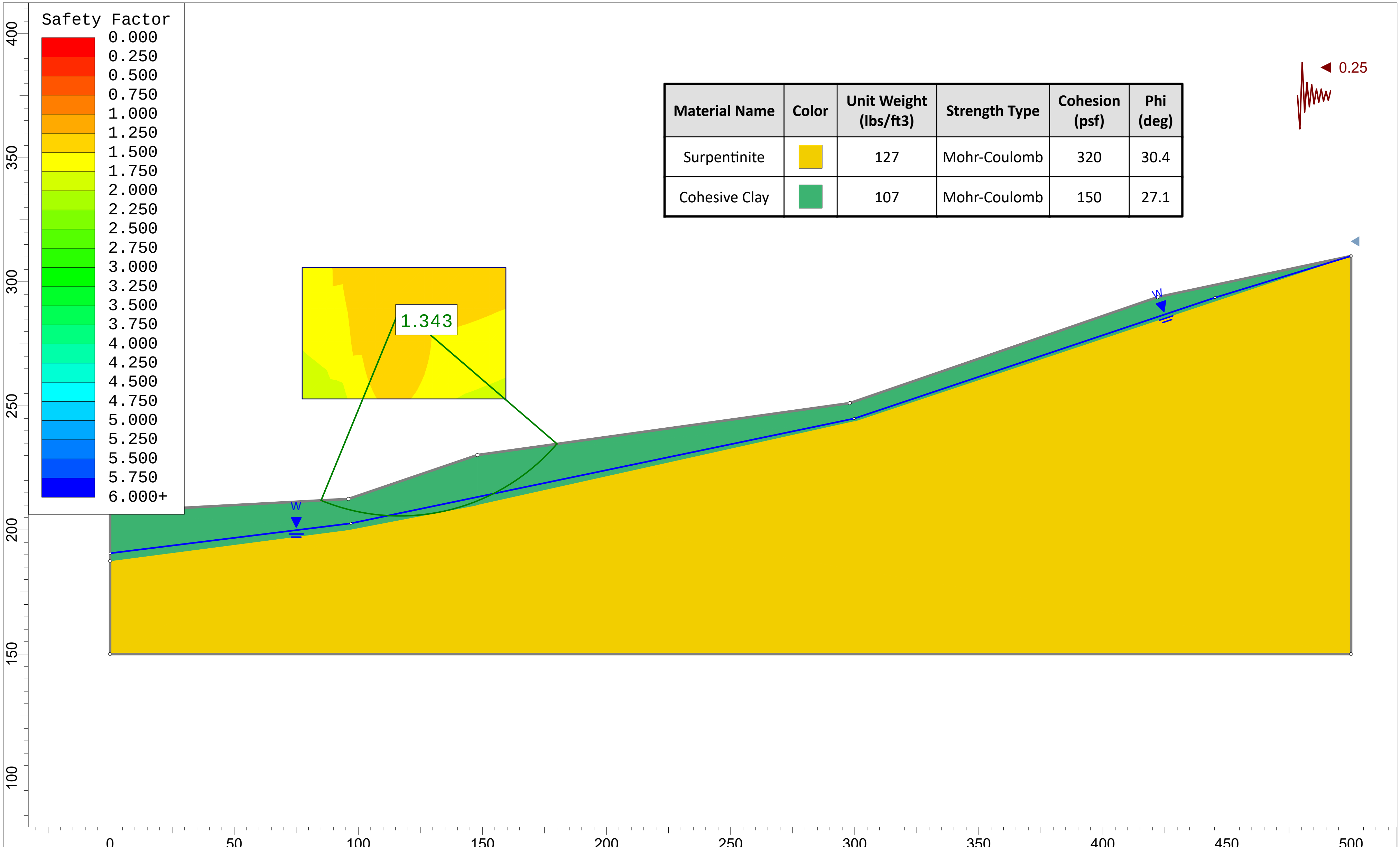


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Serpentinite		127.5	Mohr-Coulomb	320	30.4
Cohesive Clay		107	Mohr-Coulomb	150	27.1

rocscience

SLIDEINTERPRET 7.023

Project	455 Piercy Road, San Jose CA		
Analysis Description	Slope Stability		
Drawn By AA/CB	*Scale* 1:674	*Company*	CEG
Date	11/6/2017, 10:35:18 AM	*File Name*	Section AA' Static.slim

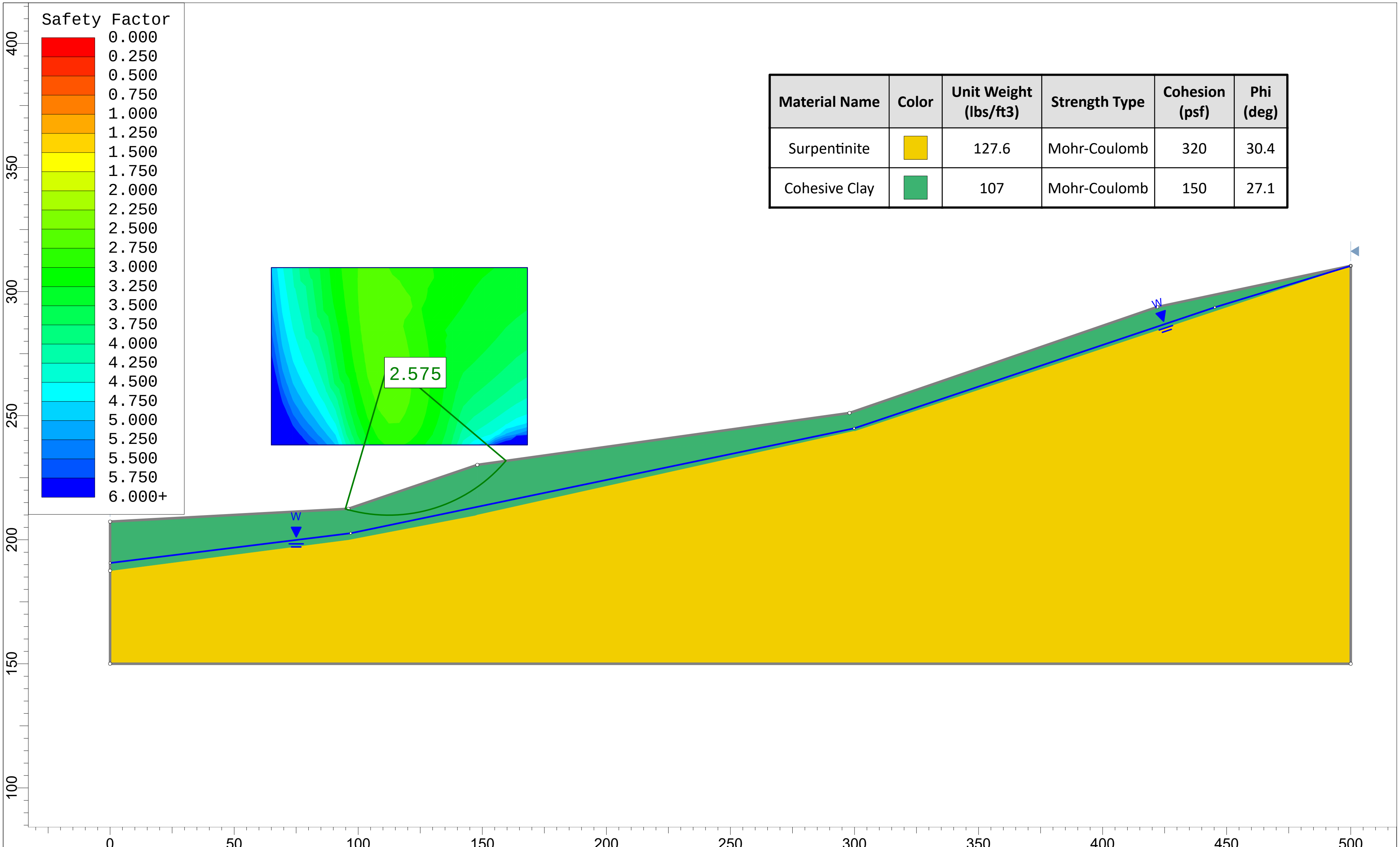


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Surpentinite		127	Mohr-Coulomb	320	30.4
Cohesive Clay		107	Mohr-Coulomb	150	27.1

rocscience

SLIDEINTERPRET 7.023

Project	455 Piercy Road, San Jose CA
Analysis Description	Slope Stability
Drawn By	AA/CB
Scale	1:642
Company	CEG
Date	11/17/2017, 1:28:17 PM
File Name	Section BB' Seismic.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Surpentinite		127.6	Mohr-Coulomb	320	30.4
Cohesive Clay		107	Mohr-Coulomb	150	27.1

rocscience

SLIDEINTERPRET 7.023

Project	455 Piercy Road, San Jose CA		
Analysis Description	Slope Stability		
Drawn By	AA/CB	*Scale*	1:642
Company	CEG		
Date	11/17/2017, 1:28:17 PM	*File Name*	Section BB' Static.slim

THIS PAGE INTENTIONALLY LEFT BLANK

Appendix D-2: Paleontological Records Search

THIS PAGE INTENTIONALLY LEFT BLANK

Kenneth L. Finger, Ph.D.
Consulting Paleontologist

18208 Judy St., Castro Valley, CA 94546-2306 510.305.1080 klfpaleo@comcast.net

September 17, 2021

Dana DePietro
FirstCarbon Solutions
1350 Treat Boulevard, Suite 380
Walnut Creek, CA 94597

Re: Paleontological Records Search: Piercey Road Industrial Warehouse Project (5458.0005), City of San Jose, Santa Clara County

Dear Dr. DePietro:

As per the request of Madelyn Dolan, I have performed a records search on the University of California Museum of Paleontology (UCMP) database for the Piercey Road Industrial Warehouse Project in San Jose. The project site extends from Fontanoso Avenue southwest to Piercy Road and extends from the edge of the flat valley onto the slope of the adjacent foothills. Its street address is 455 Piercey Road, which has a Public Land Survey (PLS) location of S½, SW¼, Sec. 5, T8S, R2E, San Jose East quadrangle (USGS 7.5-series topographic map). The applicant proposes construction of a light-industrial building on 121,580 square feet with loading docks and parking spaces that will be accessed via Piercy Road.

<u>Geologic Units</u>

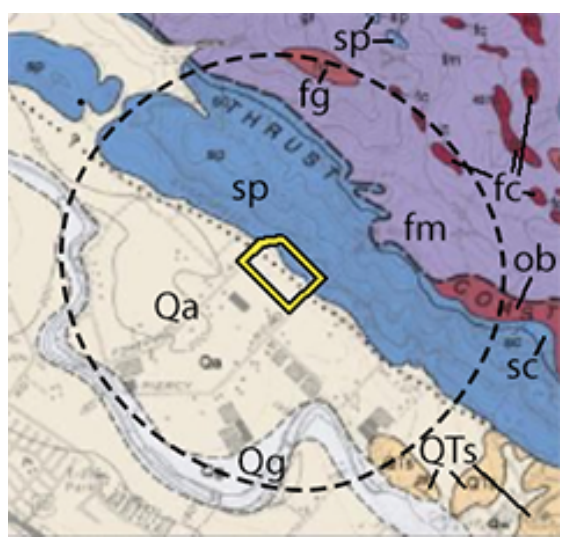
According to the part of the geologic map for the San Jose East quadrangle (Dibblee and Minch, 2005) shown here, the project site (yellow outline at center) is located on Holocene alluvium (Qa) and Jurassic–Cretaceous serpentinite (sp) of the Coast Range Ophiolite Complex.

MAP LEGEND

Qa Alluvium of valley areas & low-sloping fans near foothills (Holocene)
Qg Sand & gravel of Coyote Creek (Holocene)
QTs Santa Clara Fm. (late Pliocene–Pleistocene)

Coast Range Ophiolite Complex (Jurassic–Cretaceous)
sp Serpentinite
sc Serpentinite altered to ferruginous silica-carbonate rock
ob Basalt & keratophyre

Franciscan Assemblage (Jurassic–Cretaceous)
fm Mélange
fc Chert or metachert
fg Greenstone



Also within the half-mile search area (dashed outline) are Holocene sand and gravel of Coyote Creek (Qg), the late Pliocene–Pleistocene Santa Clara Formation (QTs), altered serpentinite (sc) and basalt and keratophyre (ob) of the Coast Range Ophiolite Complex, and Franciscan mélange (fm), chert or metachert (fc), and greenstone (fg). Holocene deposits are too young to be fossiliferous. Ophiolites originate as the mafic igneous rocks (i.e., basalt, keratophyre) that comprise-oceanic crust and the ultramafic rocks (e.g., peridotites and pyroxenites) that characterize the upper mantle; the former only rarely contain fossils, which are exclusively invertebrates, while the latter are metamorphosed into serpentinite, which is a nonfossiliferous.

Paleontological Records Search

The records search performed on the UCMP database focused on the Santa Clara Formation, the Coast Range Ophiolite Complex, and the Franciscan Assemblage. The Santa Clara Formation has five vertebrate and four plant localities listed. It yielded six vertebrate specimens, including boney fish, *Equus* (horse), Camelidae (camel), and *Bison latifrons* (long-horned bison). The camelid was found nearest to the project site at Anderson Lake, about five miles to the southeast. The composite floral assemblage consists of 19 specimens, 18 of which are recorded from PA160 (Calabazas Canyon), about five miles west of the project site; the latter include *Pteridium calabazensis* (bracken fern), *Pinus* (pine), *Prunus merriami* (stone fruit), *Calocedrus* (incense cedar), *Alnus merriami* (alder), *Quercus hannibali* (oak), *Ribes stanfordianum* (currant or gooseberry, *Cercocarpus cuneatus* (mountain mahogany), *Amelanchier* (serviceberry), and *Ceanothus chaneyi* (California lilac). The only other specimen is *Acer tyrellense* (maple), recorded from another site farther away. No vertebrate or plant localities are recorded from the Coast Range Ophiolite Complex. In sedimentary rocks of the Franciscan Assemblage, there are two plant localities (San Luis Obispo and Monterey counties) and two vertebrate localities (San Luis Obispo and San Joaquin counties). Santa Clara County is represented by a single invertebrate locality in the Franciscan.

Each geologic unit's paleontological potential and sensitivity for significant paleontological resources in Santa Clara County is summarized as follows:

UNIT	POTENTIAL	SENSITIVITY
Holocene deposits	none	none
Santa Clara Formation	moderate	high
Coast Range serpentinite	none	none
Franciscan Assemblage	very low	high

Remarks and Recommendations

A paleontological walkover survey is not recommended because the surface of the entire project site is on nonfossiliferous geologic units. The greatest potential of encountering significant paleontological resources would be if the Santa Clara Formation is present in the shallow subsurface of the project site. Being surficially mapped at the southeastern edge of the half-mile search area, however, it is more likely to be at a depth below that of the deepest project-related excavations. I therefore do not recommend paleontological monitoring of all onsite earth-disturbing activities.

In the unlikely event that any bones, teeth, or unusually abundant and well-preserved plant materials be unearthed, the construction crew should not attempt to remove them**,** as they could be extremely fragile and therefore prone to crumbling, and their *in situ* position needs to be properly recorded; instead, all work in the immediate vicinity of the discovery should be diverted at least 15 feet away from the find until it is assessed by a professional paleontologist and, if deemed significant, salvaged in a timely manner. Recovered fossils should be deposited in an appropriate repository, such as the UCMP, where they will be properly curated and made accessible for future study.

Sincerely,

Ken Finger

Reference Cited

Dibblee, T.W., and Minch, J.A., 2005. Geologic map of the San Jose East quadrangle, Santa Clara County, California: Dibblee Geology Center Geologic Map #DF-155. Scale 1:24,000.

THIS PAGE INTENTIONALLY LEFT BLANK