# Appendix E

# Geotechnical Report

CORNERSTONE EARTH GROUP

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|---|---|
| **TYPE OF SERVICES** | Geotechnical Investigation |
| **PROJECT NAME** | 644 & 675 Piercy Road Industrial |
| **LOCATION** | 644 & 675 Piercy Road<br>San Jose, California |
| **CLIENT** | HPA, Inc. |
| **PROJECT NUMBER** | 1300-2-1 |
| **DATE** | December 12, 2022 |



CORNERSTONE EARTH GROUP

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|---|---|
| **Type of Services** | **Design-Level Geotechnical Investigation and Geologic Hazards Evaluation** |
| **Project Name** | **644 & 675 Piercy Road Industrial** |
| **Location** | **644 & 675 Piercy Road<br>San Jose, California** |
| **Client** | **HPA, Inc.** |
| **Client Address** | **600 Grand Avenue, Suite 302<br>Oakland, California** |
| **Project Number** | **1300-2-1** |
| **Date** | **December 12, 2022** |




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CORNERSTONE EARTH GROUP

| | |
|---|---|
| **Type of Services** | **Design-Level Geotechnical Investigation and Geologic Hazards Evaluation** |
| **Project Name** | **644 & 675 Piercy Road Industrial** |
| **Location** | **644 & 675 Piercy Road<br>San Jose, California** |

## SECTION 1: INTRODUCTION

This geotechnical report was prepared for the sole use of HPA, Inc. for the 644 & 675 Piercy Road Industrial project in San Jose, California. As you know, we previously prepared a draft preliminary geotechnical report for western half of the site titled, "Preliminary Geotechnical Investigation and Geologic Hazards Evaluation, Piercy Road Industrial, 550 and 644 Piercy Road, San Jose, California," dated November 5, 2021. The location of the site is shown on the Vicinity Map, Figure 1. For our use, we were provided with the following documents:

- A site plan titled "Conceptual Site plan 644 & 675 Piercy Road," prepared by HPA, dated October 21, 2022.
- A topographic plan titled "TS-A21611," prepared by Kier+Wright, undated.

### 1.1 PROJECT DESCRIPTION

The 644 and 675 Piercy Road Industrial project will include redeveloping the approximately 16-acre site for industrial use. The new development will include a warehouse totaling about 209,000 square feet including about 35,000 square feet of office and mezzanine space. We anticipate the building will be single-story with an interior clear height of 36 to 40 feet and consist of concrete tilt-up construction. The truck court is located along the southwest side of the building with about 34 dock doors currently planned. The proposed building is currently sloped with planned finished floor at Elevation 249.4 feet on the east side and Elevation 253.9 feet on the west side. Detention basins are planned along the east and south sides of the site. At-grade auto and trailer parking and drive aisles will cover the remainder of the site. A 50-foot-wide PG&E easement for an existing gas main runs northwest to southeast through the center of the project site. Appurtenant utilities, landscaping, storm water management areas, and other improvements necessary for overall site development will also be constructed.

Building loads are expected to be typical of this type of construction. We understand site grading will include significant cuts of up to 22 to 24 feet and fills of up to 10 to 15 feet; final cut and fill depths will be based on the final project design.

## 1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated September 28, 2022 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 EXPLORATION PROGRAM

Field exploration consisted of six borings drilled on November 7 and 8, 2022 with truck-mounted hollow-stem auger drilling equipment. The borings were drilled to depths ranging from about 20 to 44 feet. The borings were backfilled with cement grout in accordance with local requirements; exploration permits were obtained as required by local jurisdictions.

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

## 1.4 PREVIOUS FIELD PROGRAM

Our previous field exploration consisted of one boring drilled on September 27, 2021 with track-mounted, limited-access hollow-stem auger drilling equipment. The boring was drilled to a depth of about 19 feet. The boring was backfilled with cement grout in accordance with local jurisdictions.

In addition to our previous exploratory boring, three fault trenches were excavated on April 20 through 23, 2022 with conventional back-hoe equipped tractor equipment. The trenches were excavated to approximate depths ranging from about 6 to 8 feet and extended approximately 75 to 100 feet in length. The trenches were surveyed by Kier+Wright before being loosely backfilled with soil. As the exploratory trenches were loosely backfilled, they will need to be re-excavated and backfilled with engineered fill prior to development. Detailed trench logs (T-1 through T-3) are presented as Figures 5 through 7. Additional information and details regarding the fault investigation are included in Appendix C of this report.

The approximate locations of our previous explorations and trenches are shown on the Site Exploration Plan, Figure 2. Details regarding our previous field program are included in Appendix A.

## 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 analyses, and Plasticity Index tests. Details regarding our laboratory program are included in Appendix B.

### 1.6 ENVIRONMENTAL SERVICES

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

## SECTION 2: REGIONAL SETTING

### 2.1 GEOLOGICAL SETTING

The site is located within the Santa Clara Valley, which is a broad alluvial plane between the Santa Cruz Mountains to the southwest and west, and the Diablo Range to the northeast. The San Andreas Fault system, including the Monte Vista-Shannon Fault, exists within the Santa Cruz Mountains and the Hayward and Calaveras Fault systems exist within the Diablo Range. Additional geologic information is provided in the Fault Investigation Letter attached as Appendix C. A pedochronological report was also performed for the site and Fault Trenches T-1 and T-2 by Soil Tectonics, dated May 12, 2022, and is attached as part of Appendix E.

### 2.2 REGIONAL SEISMICITY

While seismologists cannot predict earthquake events, geologists from the U.S. Geological Survey have recently updated (in 2015) earlier estimates from their 2014 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%), 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 Fault.

The faults considered capable of generating significant earthquakes are generally associated with the well-defined areas of crustal movement, which trend northwesterly. The table below presents the State-considered active faults within 25 kilometers of the site. Please refer to Appendix C for additional regional seismicity discussion.

**Table 1: Approximate Fault Distances**

| Fault Name | Distance | |
|---|---|---|
| | (miles) | (kilometers) |
| Hayward (Southeast Extension) | 3.2 | 5.2 |
| Monte Vista-Shannon | 4.5 | 7.3 |
| Calaveras | 5.4 | 8.7 |
| Sargent | 10.7 | 17.3 |
| San Andreas (1906) | 12.3 | 19.8 |
| Hayward (Total Length) | 13.2 | 21.2 |

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

## SECTION 3: SITE CONDITIONS

### 3.1 SURFACE DESCRIPTION AND SITE HISTORY

At the time of our explorations, the site generally consisted of an open field covered in low grasses with one to four foot tall weeds, numerous palm trees in the northern portion of the site along Piercy Road, and a chain link fence dividing the 644 and 675 Piercy Road parcels. Two single story structures, a gravel access road, and appurtenant concrete structures are also located within the southern corner of the 675 Piercy Road parcel. Based on elevations shown on the topographic plan, the site generally slopes from north to south at approximately Elevation 295 in the northern corner to approximately 220 feet along the southern side of the site adjacent to Hellyer Road. The site is bounded by Piercy Road to the north, Tennant Avenue to the east, Hellyer Avenue to the south, and an undeveloped lot to the west.

To evaluate site history, we reviewed historic aerial imagery provided online by Historical Aerials (www.historicaerials.com) between 1948 and 2020, and our previous work in the area. A summary of pertinent surface changes at and near the vicinity of the site are as follows:

- 1948: 644 Piercy Road appears occupied by an orchard and 675 Piercy appears undeveloped. Piercy Road and Tennant Avenue appear in their current configuration.
- 1980: The orchard on 644 Piercy is removed and a residential structure appears in the northeastern corner of the parcel. The 644 and 675 Piercy Road parcels appear to be used for agricultural purposes.
- 1982: Several single-story structures are visible in the southern corner of 675 Piercy.
- 2002: Hellyer Avenue appears in its current configuration to the south. The existing residential development is observed constructed to the south of Tennant Avenue.
- 2020: No pertinent surface changes are observed.

We understand that while the latest aerial imagery shows residential structures within the northern portion of the 644 Piercy Road parcel, these structures have since been removed prior to our investigation.

## 3.2 SUBSURFACE CONDITIONS

Below the existing ground surface, our explorations generally encountered undocumented fill underlain by native colluvium soil and Santa Clara Formation to the maximum depths explored during this investigation. A more detailed description of the subsurface conditions are presented in the following sections. Cross sections of the subsurface conditions within the building pad are presented as Figures 9, 10, and 11.

### 3.2.1 Undocumented Fills

Below the existing surface, our Borings EB-1 and EB-4 generally encountered approximately 3 to 6 feet of undocumented fill. The fills were highly variable in content and generally consisted of medium dense clayey sand with gravel and hard lean clay with sand.

### 3.2.2 Colluvium Soils

Below the undocumented fills or existing surface where no fill was encountered, Borings EB-1, EB-3, EB-6, and EB-7 generally encountered colluvium soils (Qcol) consisting of very stiff to hard fat clay with sand to depths of about 5 to 11 feet. Below the undocumented fills or existing ground surface where no fill was encountered, Borings EB-2, EB-4, and EB-5 generally encountered colluvium soils consisting of hard lean clay with variable amounts of sand to depths of 7 to 12 feet.

### 3.2.3 Santa Clara Formation

Below the surficial colluvium soils, our Borings EB-1, EB-2, and EB-4 were underlain by Santa Clara Formation (QTsc) material consisting of interbedded layers of very dense clayey sand with gravel and hard lean clay with variable amounts of sand to the terminal boring depths of about 19 to 40 feet. Beneath the surficial colluvium, Boring EB-3 generally encountered Santa Clara Formation material consisting of hard lean clay to a depth of 8 feet below existing site grades and siltstone/mudstone to the terminal boring depth of approximately 24½ feet. Below the surficial colluvium soils, Boring EB-5 was underlain by Santa Clara Formation material comprised of interbedded layers of hard elastic silt, hard sandy lean clay, and very dense clayey sand to the maximum depth explored of about 44 feet. Beneath the surficial colluvium, Borings EB-6 and EB-7 generally encountered moderately to extremely expansive Santa Clara Formation siltstone and claystone to the terminal boring depths of 25 to 40 feet.

A summary of the subsurface conditions encountered in our fault trenches is provided in the Fault Investigation report included as Appendix C.

### 3.2.4 Plasticity/Expansion Potential

We performed seven Plasticity Index (PI) tests on representative samples. Test results were used to evaluate expansion potential of surficial soils, the underlying claystone, and for classification purposes. The results of the PI tests performed in surficial soils resulted in PIs ranging from 17 to 57 indicating a moderate to extreme expansion potential to wetting and drying cycles. The results of the PI tests performed in claystone and siltstone resulted in PIs ranging from 19 to 86, indicating moderate to extreme expansion potential to wetting and drying cycles. The result of the PI test performed for classification of deeper soils resulted in a PI of 13.

### 3.2.5 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents within the upper 25 feet range from about 5 percent under to about 13 percent over the estimated laboratory optimum moisture.

## 3.3 GROUNDWATER

Groundwater was encountered in our Boring EB-2 at a depth of 29 feet below current grades. The historic groundwater levels are not mapped in the immediate area around the site. Based on our experience in the area, we judge the depth to free groundwater to be on the order of 30 feet below the ground surface with some localized shallower perched water conditions. Fluctuations in groundwater levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors.

## 3.4 CORROSION SCREENING

We tested four samples collected at depths of 3½, 5½, and 13½ feet for resistivity, pH, soluble sulfates, and chlorides. The laboratory test results are summarized in Table 2A.

**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-2 | 3½ | 6.9 | 1,163 | 3 | 56 |
| EB-3 | 5½ | 7.0 | 651 | 76 | 300 |
| EB-4 | 13½ | 7.0 | 1,005 | 45 | 41 |
| EB-7 | 5½ | 7.0 | 1,284 | 20 | 154 |

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.

### 3.4.1 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 to very severely corrosive to buried metallic improvements (Chaker and Palmer, 1989).

In accordance with the 2019 CBC Section 1904.1, alternative cementitious materials for different exposure categories and classes shall be determined in accordance with ACI 318-14 Table 19.3.1.1, Table R19.3.1, and Table 19.3.2.1. Based on the laboratory sulfate test results, a cement type restriction is not required, although, in our opinion, it is generally a good idea to include some sulfate resistance and to maintain a relatively low water-cement ratio. We have summarized applicable exposure categories and classes from ACI 318-14, Table 19.3.1.1 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] | C0[4] |

1 (F0) “Concrete not exposed to freezing-and-thawing cycles” (ACI 318-14)
2 (S0) “Water soluble sulfate in soil, percent by mass” (ACI 318-14)
3 (W0) “Concrete dry in service” (ACI 318-14)
4 (C0) “Concrete dry or protected from moisture” (ACI 318-14)

We recommend the structural engineer and a corrosion engineer be retained to confirm the above information and provide additional recommendations, as needed.

## SECTION 4: GEOLOGIC HAZARDS

### 4.1 FAULT SURFACE RUPTURE

As discussed above several significant faults are located within 25 kilometers of the site. The site is not located within a State-designated Alquist Priolo Earthquake Fault Zone, but is mapped in a City of San Jose Potential Hazard Zone. Additional information on fault surface rupture hazard potential is presented in Appendix C of this report.

### 4.1.1 Geophysical Fault Study

A geophysical study spanning the length of the site trending from north to south and east to west was performed by JR Associates. The study included four seismic refraction surveys and one shear wave profiling survey. Additional discussion regarding the results of this study is included in Appendices C and D of this report.

## 4.2 ESTIMATED GROUND SHAKING

Moderate to severe (design-level) earthquakes can cause strong ground shaking, which is the case for most sites within the Bay Area. A site specific peak ground acceleration $(PGA)_M$ was estimated for analysis using a value equal to $F_{PGA}$ x PGA, as allowed in the 2019 edition of the California Building Code. For our liquefaction analysis we used a $PGA_M$ of 0.873g.

## 4.3 LIQUEFACTION POTENTIAL

The site is located within a State-designated Liquefaction Hazard Zone (CGS, San Jose East Quadrangle, 2003) and a Santa Clara County Liquefaction Hazard Zone (Santa Clara County, 2003). However, a laterally extensive groundwater table has not been identified in compilations (CGS, 2000), groundwater was not encountered in our exploratory borings, except for EB-2, and groundwater conditions are likely quite variable near the east hills. As part of our design-level geotechnical investigation, we screened the site for liquefaction during our site explorations by retrieving samples from the site, performing visual classification on sampled materials, and performing various tests to further classify the soil properties.

During strong seismic shaking, cyclically induced stresses can cause increased pore pressures within the soil matrix that can result in liquefaction triggering, soil softening due to shear stress loss, potentially significant ground deformation due to settlement within sandy liquefiable layers as pore pressures dissipate, and/or flow failures in sloping ground or where open faces are present (lateral spreading) (NCEER 1998). Limited field and laboratory data is available regarding ground deformation due to settlement; however, in clean sand layers settlement on the order of 2 to 4 percent of the liquefied layer thickness can occur. Soils most susceptible to liquefaction are loose, non-cohesive soils that are saturated and are bedded with poor drainage, such as sand and silt layers bedded with a cohesive cap.

As discussed in the “Subsurface” section above, we primarily encountered very stiff to hard lean and fat clays, dense to very dense clayey sands with gravel, and bedrock consisting of claystone and sandstone/mudstone. In addition, the design groundwater level is anticipated to be below any loose to medium dense granular soils. Based on the above, our screening of the site for liquefaction indicates a low potential for liquefaction.

## 4.4 LATERAL SPREADING

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

The Coyote Creek channel runs approximately 500 feet to the south of the site; however, due to the deep groundwater and low potential for liquefaction, 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 very stiff to hard lean and fat clays, siltstone and claystone, and dense to very dense clayey sands, in our opinion, the potential for significant differential seismic settlement affecting the proposed improvements is low.

## 4.6 TSUNAMI/SEICHE

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

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

A tsunami or seiche originating in the Pacific Ocean would lose much of its energy passing through San Francisco Bay. Based on the mapping of tsunami inundation potential for the San Francisco Bay Area by CGS (conservation.ca.gov/cgs/tsunami/maps), areas most likely to be inundated are marshlands, tidal flats, and former bay margin lands that are now artificially filled, but are still at or below sea level, and are generally within 1½ miles of the shoreline. The site is approximately 20 miles inland from the San Francisco Bay shoreline and is approximately 220 to 295 feet above mean sea level. Therefore, the potential for inundation due to tsunami or seiche is considered low.

## 4.7 FLOODING

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

The Department of Water Resources (DWR), Division of Safety of Dams (DSOD) compiled a database of Dam Failure Inundation Hazard Maps (DSOD, 2015). The generalized hazard maps were prepared by dam owners as required by the State Office of Emergency Services; they are intended for planning purposes only. Based on our review of these maps, the site is not located within a dam failure inundation area.

# 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.

- Presence of extremely expansive claystone
- Presence of highly expansive surficial soils
- Presence of undocumented fill
- Differing foundation support conditions (cut and fill transition)

### 5.1.1 Presence of Extremely Expansive Claystone

As discussed, moderately to extremely expansive claystone and siltstone was encountered in Borings EB-6 and EB-7 within the planned building area. Care should be taken to reduce water infiltration into this stratum as well as reducing or eliminating cuts in this material where possible. Expansive siltstone and claystone can undergo significant volume change with changes in overburden pressure and moisture content and can shrink and harden when dried and expand and soften when wetted. In addition, we would anticipate that extremely expansive claystone will likely be sensitive to load relief (i.e. cuts). For example, expansive claystone bedrock could expand on the order 2 to 5 inches for a 10-foot-cut above a 30-foot-thick claystone bed, and expansion may not be apparent until 1 to 2 years after the cuts are completed (Meehan, et al., 1975). The addition of water will increase the amount of heave. Differential movement may occur between the claystone beds and Santa Clara Formation soils if heave within the claystone beds occurs.

The primary geotechnical concern is the high to extreme expansion potential of the underlying claystone of Santa Clara Formation encountered at the site. On a preliminary basis, we anticipate potential heaving/swell of the bedrock due to overburden removal could occur while excavating the building pad, roughly estimated to be on the order of ½-inch per foot of cut or less. We recommend that care be taken and additional observation and monitoring be performed during cut operations that expose the underlying claystone in order to document any potential heave that may occur during construction as a result of load relief. If desired to reduce the potential for impact of overburden relief in the cut areas, the excavations should be left open and monitoring performed until rebound of the underlying claystone subsides.

Due to the potential heave and extreme expansion potential of the shallow claystone, we recommend the entire proposed structure and foundations be supported on at least 48 inches of non-expansive fill (NEF). Consideration should be given to the slabs being tied into the structure at egress points to reduce trip hazards due to potential heave from the claystone. Additional consideration should be given to the installation of rebar within slab-on-grade to reduce potential impact of any heave from the claystone. Detailed grading and foundation recommendations addressing this concern are presented in the following sections.

Pavement recommendations are provided in Section 10 below; however, if desired to provide additional mitigation of the extreme expansion potential of the claystone in areas of cut, the pavement sections may be supported on additional NEF. If this option is desired, we can provide additional recommendations as needed.

Any gravity utility pipes entering areas of high cut/claystone should be oversteepened and designed to accommodate the expected differential settlement due to variations in heave along the utility alignment to avoid sags and backflow of gravity lines.

### 5.1.2 Presence of Highly Expansive Surficial Soils

Highly expansive soils were also encountered in our borings and appear to generally blanket the site. Expansive soils can undergo significant volume change with changes in moisture content. They shrink and harden when dried and expand and soften when wetted. To reduce the potential for damage to the planned structures, slabs-on-grade should have sufficient reinforcement and be supported on a layer of non-expansive fill; footings should extend below the zone of seasonal moisture fluctuation. In addition, it is important to limit moisture changes in the surficial soils by using positive drainage away from buildings as well as limiting landscaping watering. Evaluation of potential import sources for the site should consider the acceptable range of plasticity, especially in the upper 5 feet of fill. 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. Detailed grading and foundation recommendations addressing this concern are presented in the following sections.

### 5.1.3 Presence of Undocumented Fill

As discussed above, we encountered undocumented fill within our borings at the site ranging in depth from about 3 to 5 feet. In addition, our Trench T-3 encountered between 6 to 7½ feet of undocumented fill. Based on past development and observations onsite, we anticipate deep fills may be encountered in other areas across the site. Undocumented fill 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. 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.

We note that three approximately 75- to 100-foot-long trenches, T-1, T-2, and T-3, were excavated across the site as part of our previous field exploration program and were loosely backfilled upon completion. The exact location and depths of Trenches T-1, T-2, and T-3 have been surveyed by Kier+Wright. All trench backfill should be removed and replaced as engineered fill.

### 5.1.4 Differing Foundation Support Conditions (Cut and Fill Transitions)

Based on site topography, discussions with the project civil engineer Kier+Wright, and planned finished floor elevations, we understand that cuts of up to 20 to 24 feet and fills of up to 10 to 15 feet may be required for the building pad. In addition, depth to bedrock is anticipated to be variable and was encountered on the northwestern side of the site in Borings EB-6 and EB-7 and trenches T-1 and T-2.

Foundations bearing on different material (soil/rock or cut/fill transitions) can settle unevenly and may result in increased differential settlement between the different materials. On a preliminary basis, we recommend finished pads with differing foundation support conditions (i.e. overlying cuts vs overlying fill) be over-excavated about 4 feet and replaced with engineered fill to provide a more uniform support of foundations. This requirement will likely be met by the requirement for non-expansive fill beneath slabs-on-grade and foundations as discussed above and in subsequent report sections. Additional recommendations addressing this concern are presented in the “Earthwork” and “Foundations” sections of this report.

## 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: EARTHWORK

## 6.1 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, which may be present on 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.

### 6.1.1 Demolition of Existing Slabs, Foundations and Pavements

All slabs, foundations, and pavements should be completely removed from within planned building areas.

As an owner value-engineered option, existing slabs, foundations, and pavements that extend into planned flatwork, pavement, or landscape areas may be left in place provided there is at least 3 feet of engineered fill overlying the remaining materials, they are shown not to conflict with new utilities, and that asphalt and concrete more than 10 feet square is broken up to allow subsurface drainage. Future distress and/or higher maintenance may result from leaving these prior improvements in place. A discussion of recycling existing improvements is provided later in this report.

### 6.1.2 Abandonment of Existing Utilities

All utilities 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.

## 6.2 SITE CLEARING AND PREPARATION

### 6.2.1 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 4 to 6 inches below existing grade in vegetated areas.

### 6.2.2 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.

## 6.3 MITIGATION OF UNDOCUMENTED FILLS

As discussed, undocumented fill was encountered at depths ranging from about 3 to 7½ feet. 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.

As previously discussed, two approximately 75 to 100 foot long fault trenches, T-1, T-2, and T-3, were excavated in the northwest corner of the site as part of our field exploration program and were loosely backfilled upon completion. The exact location and depths of fault trenches T-1, T-2, and T-3 have been surveyed by Kier+Wright. All trench backfill should be removed and replaced as engineered fill.

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.

## 6.4 CLAYSTONE NON-EXPANSIVE FILL AND CUT/FILL TRANSITION OVER-EXCAVATION

As discussed above, to reduce the potential of the extremely expansive claystone we recommend the entire building slab-on-grade and foundations be supported on at least 48 inches of non-expansive fill. Non-expansive fill should also extend at least 2 feet horizontally outside of building footprint. Foundations bearing on different material (soil/rock or cut/fill) can

settle unevenly and may result in increased differential settlement between the different materials. To reduce the potential for increased differential settlement due to differing foundation support conditions (cut/fill or soil/rock transitions), on a preliminary basis, we recommend that the finished building pads be over-excavated approximately 4 feet and replaced with engineered fill to provide a more uniform support of the foundations. This criteria will already be met as a result of the recommendation for non-expansive fill beneath slabs-on-grade and foundations to mitigate for the extremely expansive soils/bedrock at the site. Transition between areas of deeper cut and fills should not transition at a rate faster than 3:1 (h:v). These estimates should be further evaluated once final building loading and grading plans are available. Additional recommendations addressing this concern are presented in the "Foundation" section of this report.

If desired, where highly expansive claystone encountered, additional over-excavation can be performed.

## 6.5 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 20 feet at the site may be classified as OSHA Soil Type C materials.

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.

## 6.6 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.

Due to the potential for sandy soils to be encountered at the subgrade elevation, we recommend that subgrade compaction and proof rolling be performed within 24 hours of capillary break layer or slab-on-grade construction.

## 6.7 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.

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 site conditions.

### 6.7.1 Scarification and Drying

The subgrade may be scarified to a depth of 6 to 8 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.

### 6.7.2 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.

### 6.7.3 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.

## 6.8 MATERIAL FOR FILL

### 6.8.1 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. Highly expansive claystone bedrock or soils should not be used as retaining wall backfill.

It may be possible to reuse some of the site strippings as part of the engineered fill. Strippings should be spread thinly over a section of on-site materials, and thoroughly blended by disking in two directions, resulting in a material that does not exceed 3 percent organic content by weight. The stripping layer thickness will depend on the stripping organic content and the disking depth capabilities. A Cornerstone representative should collect samples of the blended material to check the organic content prior to approving the mix for use and approving the mixing operation.

### 6.8.2 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.

## 6.9 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 and 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 "Wet Soil Stabilization Guidelines" section of this report. Where the soil's PI is 20 or greater, the expansive soil criteria should be used.

**Table 3: 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)

### 6.9.1 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.

## 6.10 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 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.

### 6.11 PERMANENT CUT AND FILL SLOPES

All permanent cut and fill slopes in soil should have a maximum inclination of 2:1 (horizontal:vertical) for slopes up to 10 feet high; slopes greater than 10 feet should be inclined at no greater than 2½:1, and incorporate surface drainage and benching in accordance with local jurisdiction requirements. Our certified engineering geologist should review all bedrock cut slopes at the time of construction. If adverse conditions are observed, the slope should be over-excavated and rebuilt as an engineered fill slope. Fill slopes should be overbuilt and trimmed back, exposing engineered fill when complete. Refer to the "Erosion Control" section of this report for a discussion regarding protection of slope surfaces.

### 6.12 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. At a minimum, an intermediate bench should be incorporated where fill slopes exceed 25 feet. 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.

Upslope sources of water should be evaluated. If upslope irrigation of is present or planned, additional surface and subsurface drainage, or construction of drained buttress fills may be needed to protect site improvements. We should be consulted if this issue will affect the project.

### 6.13 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 and is anticipated to be 30 feet below grade, and therefore is expected to be deeper than 10 feet below the base of any infiltration measures.

- The site has a known geotechnical hazard consisting of steep slopes; therefore, stormwater infiltration facilities may not be feasible.

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

### 6.13.1 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.

#### 6.13.1.1 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.

#### 6.13.1.2 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 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.

- 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.

#### 6.13.1.3 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.

### 6.14 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.

### 6.15 LANDSCAPE CONSIDERATIONS

Since the near-surface soils, claystone and siltstone are moderately to extremely 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 7: 2019 CBC SEISMIC DESIGN CRITERIA

### 7.1 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.

Our explorations generally encountered very stiff to hard clays, claystone and siltstone, and dense to very dense sand deposits to a depth of 44 feet, the maximum depth explored. Based on our borings and review of local geology, the site is underlain by shallow alluvial soils underlain by dense to very dense sands and shallow rock 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 4. The table below lists the various factors used to determine the seismic coefficients and other parameters.

**Table 4: CBC Site Categorization and Site Coefficients**

| Classification/Coefficient | Design Value |
|---|---|
| Site Class | C |
| Site Latitude | 37.252380° |
| Site Longitude | -121.770394° |
| 0.2-second Period Mapped Spectral Acceleration[1], $S_S$ | 1.729g |
| 1-second Period Mapped Spectral Acceleration[1], $S_1$ | 0.656g |
| 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 - $S_{MS}$ | 2.075g |
| 1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – $S_{M1}$ | 0.918g |
| 0.2-second Period, Design Earthquake Spectral Response Acceleration – $S_{DS}$ | 1.383g |
| 1-second Period, Design Earthquake Spectral Response Acceleration – $S_{D1}$ | 0.612g |
| Site Amplification Factor at PGA – $F_{PGA}$ | 1.2 |
| Site Modified Peak Ground Acceleration – $PGA_M$ | 0.873g |

# 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 SHALLOW FOUNDATIONS

### 8.2.1 Conventional Shallow Footings

Conventional shallow footings should bear on 48 inches of non-expansive engineered fill (NEF), be at least 18 inches wide, and extend at least 18 inches below the lowest adjacent grade. Lowest adjacent grade is defined as the deeper of the following: 1) bottom of the adjacent interior slab-on-grade, or 2) finished exterior grade, excluding landscaping topsoil. The non-expansive fill section beneath the footing is due to the presence of highly to extremely expansive claystone and surficial soils and is intended to reduce the impact of significant seasonal moisture fluctuation and the potential for differential movement.

Footings constructed to the above dimensions, bearing on engineered soil and in accordance with the “Earthwork” recommendations of this report are capable of supporting maximum allowable bearing pressures of 2,000 psf for dead loads, 3,000 psf for combined dead plus live loads, and 4,000 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.

### 8.2.2 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 5: Assumed Structural Loading**

| Foundation Area | Range of Assumed Loads |
|---|---|
| Interior Isolated Column Footing | 100 to 150 kips |
| Perimeter Strip Footing | 5 to 7 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 less than ½-inch of post-construction differential settlement between adjacent foundation elements, assumed to be on the order of 40 to 60 feet. As our footing loads were assumed, we recommend we be retained to review the final footing layout and loading, and verify the settlement estimates above.

As discussed above, to reduce the potential of the extremely expansive claystone we recommended the slab-on-grade and foundations be supported on at least 48 inches of non-expansive fill. Non-expansive fill should also extend at least 2 feet horizontally outside of footings (interior and exterior). Transition between areas of deeper cut and fills should not transition at a rate faster than 3:1 (h:v). These estimates should be further evaluated once final building loading and grading plans are available. Additional recommendations are presented in the “Earthwork” section of this report.

### 8.2.3 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 0.4 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.

### 8.2.4 Conventional Shallow 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.

Due to the presence of clean sand and gravel, footing excavation walls may not stand vertical and may need to be sloped to a minimum 1:1 inclination or Stay-Form or similar may need to be placed within the footing excavations as they are excavated during construction of the foundation elements. Granular material encountered in the footing bottoms will likely be disturbed to a depth of 6 to 8 inches following excavation and will need to be compacted to 90 percent relative compaction prior to steel placement. Care should be taken to not disturb the compacted granular material during steel placement. We should re-observe the footing excavations in granular materials after reinforcing steel has been placed and just prior to concrete placement. Footing excavations should also be kept moist by regular sprinkling with water to prevent desiccation and potential raveling of the granular materials. As an alternative, a rat slab can be placed over the granular material after we have observed the footing excavation to protect the granular material prior to steel placement.

## SECTION 9: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

As discussed above, to reduce the potential for increased differential settlement due to differing foundation support conditions (cut/fill or soil/rock) and to help mitigate the extreme expansive potential of the claystone, on a preliminary basis, we recommend that the finished building pad and foundations be underlain by at least 48 inches of non-expansive fill. Additional recommendations are presented in the "Earthwork" section of this report.

### 9.1 INTERIOR OFFICE SLABS-ON-GRADE

Due to the high to extreme expansion potential of the surficial soils, claystone, and siltstone, the proposed slabs-on-grade should be at least 5 inches thick and be supported on at least 48 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 slab-on-grade 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 3 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, and should be designed for the specific warehouse loading (i.e. forklifts, rack loads, etc.). At this time, rack loading information, etc. was not available. The slab should also be designed to accommodate potential slab settlement beneath heavily loaded slab areas. We recommend we be retained to review the final heavily loaded (i.e. rack loading) layout and loading and provide estimated settlements.

Warehouse slabs-on-grade should also be supported on at least 6 inches of crushed granular base having an R-value of at least 50 and no more than 10 percent passing the No. 200 sieve, such as Class 2 aggregate base. Due to the moderate to very high expansion potential of the surficial soils, an additional 42 inches of non-expansive fill (NEF) should be placed below the upper granular base.

To summarize, the warehouse slab-on-grade should be underlain by the materials and alternatives outlined below. Additional details follow these points.

- Zone A Fill Material – Upper 6 inches of pad subgrade should consist of crushed granular base having an R-value of at least 50 and no more than 10 percent passing the No. 200 sieve, such as Class 2 aggregate base.

- Zone B Fill Material – From 6 to 48 inches below pad subgrade (or below Zone A noted above), the soil/material should also consist of a non-expansive fill (NEF) with a Plasticity Index (PI) of 15, or less.

    - Zone B Fill Material Alternative 1 – From 6 to 48 inches below pad subgrade, the non-expansive fill material can also consist of the same material used for Zone A.

    - Zone B Fill Material Alternative 2 – From 6 to 48 inches below pad subgrade, the onsite material can be chemically treated to meet the non-expansive fill requirement with a PI of 15, or less.

- Zone B Fill Material Alternative 3 – From 6 to 48 inches below pad subgrade, the non-expansive fill material can consist of imported soil that meets the "Materials for Fill" section above and has a PI of 15, or less.

All base and sub-base materials should be placed and compacted in accordance with the "Compaction" section of this report. If there are 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.

Provided the full NEF section is placed and compacted, a preliminary modulus of subgrade reaction of 150 pounds per cubic inch (pci) may be used for preliminary design for slab design.

## 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 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 |
| No. 200 | 0 – 5 |

  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 flatwork, such as pedestrian walkways, patios, driveways, and sidewalks, may experience seasonal movement due to the native expansive soils; therefore, some cracking or vertical movement of conventional slabs should be anticipated where imported fill is not planned in flatwork areas. There are several alternatives for mitigating the impacts of expansive soils beneath concrete flatwork. We are providing recommendations to reduce distress to concrete flatwork that includes moisture conditioning the subgrade soils, using non-expansive fill, and providing adequate construction and control joints to control cracks that do occur. It should be noted that minor slab movement or localized cracking and/or distress could still occur.

- The minimum recommendation for concrete flatwork constructed on highly to extremely expansive soils is to properly prepare the clayey soils prior to placing concrete. This is typically achieved by scarifying, moisture conditioning, and re-compacting the subgrade soil. Subgrade soil should be moisture conditioned to at least 3 percent over the laboratory optimum and compacted using moderate compaction effort to a relative compaction of 87 to 92 percent (ASTM Test Method D1557). Since the near surface soils may have been previously compacted and tested, the subgrade soils could possibly be moisture conditioned by gradually wetting the soil, depending on the time of year slab construction occurs. This should not include flooding or excessively watering the soil, which would likely result in a soft, unstable subgrade condition, and possible delays in the construction while waiting for the soil to dry out. In general, the subgrade should be relatively firm and non-yielding prior to construction.

- Concrete flatwork, excluding pavements that would be subject to wheel loads, should be at least 4 inches thick and underlain by at least 18 inches of non-expansive fill. Non-expansive fill may include aggregate base, crushed rock, or imported soil with a PI of 15 or less. Non-expansive fill should be compacted to at least 90 percent relative compaction. 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.

- We recommend a maximum control joint spacing of about 2 feet in each direction for each inch of concrete thickness and a construction joint spacing of 10 to 12 feet. Construction joints that abut the foundations or slabs should include a felt strip, or

approved equivalent, that extends the full depth of the exterior slab. This will help to reduce the potential for permanent vertical offset between the slabs due to friction between the concrete edges. We recommend that exterior slabs be isolated from adjacent foundations.

At the owner's option, if desired to reduce the potential for vertical offset or widening of concrete cracks, consideration should be given to using reinforcing steel, such as No. 3 rebar spaced at 18 inches on center each direction.

## SECTION 10: VEHICULAR PAVEMENTS

### 10.1 ASPHALT CONCRETE

The following asphalt concrete pavement recommendations tabulated below are based on Procedure 608 of the Caltrans Highway Design Manual, estimated traffic indices for various pavement-loading conditions, and on a design R-value of 5. The design R-value was chosen based on engineering judgement considering the variable and expansive soil conditions. Additionally, due to the presence of highly to extremely expansive soils, we have also included an option for 12 inches of lime-treated subgrade soils using an estimated design R-value of 50.

**Table 6: Asphalt Concrete Pavement Recommendations**

| 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 | 13.0 | 16.5 |
| 6.5 | 4.0 | 13.5 | 17.5 |
| 7.0 | 4.0 | 15.5 | 19.5 |
| 7.5 | 4.5 | 16.5 | 21.0 |
| 8.0 | 5.0 | 17.5 | 22.5 |
| 8.5 | 5.0 | 19.5 | 24.5 |
| 9.0 | 5.5 | 20.5 | 26.0 |
| 9.5 | 6.0 | 21.5 | 27.5 |
| 10.0 | 6.5 | 22.5 | 29.0 |
| 10.5 | 6.5 | 24.5 | 31.0 |
| 11.0 | 7.0 | 25.5 | 32.5 |

[1]Caltrans Class 2 aggregate base; minimum R-value of 78.

**Table 7: Asphalt Concrete Pavement Recommendations, Lime Treated Subgrade**

| Design Traffic Index (TI) | Asphalt Concrete (inches) | Class 2 Aggregate Base[1] (inches) | Total Pavement Section Thickness (inches) |
|---|---|---|---|
| 4.0/4.5 | 2.5 | 4.0 | 6.5 |
| 5.0/5.5 | 3.0 | 4.0 | 7.0 |
| 6.0 | 3.5 | 4.0 | 7.5 |
| 6.5 | 4.0 | 4.0 | 8.0 |
| 7.0 | 4.0 | 4.5 | 8.5 |
| 7.5 | 4.5 | 5.0 | 9.5 |
| 8.0 | 5.0 | 5.0 | 10.0 |
| 8.5 | 5.0 | 6.0 | 11.0 |
| 9.0 | 5.5 | 6.5 | 12.0 |
| 9.5 | 6.0 | 7.0 | 13.0 |
| 10.0 | 6.5 | 7.0 | 13.5 |
| 10.5 | 6.5 | 8.0 | 14.5 |
| 11.0 | 7.0 | 8.0 | 15.5 |

[1]Caltrans Class 2 aggregate base; minimum R-value of 78; subgrade R-value of 78; minimum 12 inch chemical-treated subgrade R-value assumed to be 50

As discussed above, to provide additional mitigation of the extreme expansion potential of the claystone in areas of cut, the pavement sections may be supported on additional NEF, if this option is desired, we can provide additional recommendations as needed.

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

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). In our opinion, the truck loading areas where trucks turn, break, or stop should be constructed of reinforced PCC pavement. We have provided a few pavement alternatives as an anticipated number of trucks and number of load repetitions per day on a given location of the pavement has not been provided at this time. An alternative should be chosen that is greater than what is expected for the development. At this time, we have assumed trucks will consist of tractor trailers. When a more specific truck loading information is available, additional information can be provided, as needed.

**Table 8: PCC Pavement Recommendations (Untreated Subgrade)**

| Traffic Category | Minimum PCC Thickness[1] (inches) |
|---|---|
| Maximum ADTT = 20 | 7.0 |
| Maximum ADTT = 80 | 7.5 |
| Maximum ADTT = 400 | 8.0 |

[1]Subgrade design R-Value = 5

The PCC thickness recommended above are based on a concrete flexural strength of at least 550 psi, minimum 28-day compressive strength of 3,500 psi, laterally restraining the PCC with curbs or concrete shoulders, 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 of our report. The aggregate base should be constructed over compacted subgrade prepared in accordance with the "Compaction" section of this report. 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 can 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; joint sealing should be included due to the expansive soils at the site.

We note that a PCC thickness of 7 inches overlying 6 inches of Class 2 aggregate base as discussed above, is sufficient to support a traffic index (TI) of 7 or 8.

If the subgrade soils are to be chemically treated (lime/cement), we have provided some pavement alternatives below. For chemically treated subgrade, we recommend the upper 12

inches of subgrade soil be treated and the treated subgrade should obtain a minimum R-value of 50. If chemical treatment of the subgrade is performed, the underlying Class 2 aggregate base section may be reduced from 6 inches to 4 inches, provided it is compacted to a minimum relative compaction of 95 percent (ASTM D1557) as recommended in the "Earthwork" section of our report. 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.

**Table 9: PCC Pavement Recommendations (Chemical-Treated Subgrade)**

| Traffic Category | Minimum PCC Thickness[1] (inches) |
|---|---|
| Maximum ADTT = 20 | 6.5 |
| Maximum ADTT = 80 | 7.0 |
| Maximum ADTT = 400 | 7.5 |

[1]Subgrade design R-Value = 50, minimum 12 inches of chemically treated subgrade

We note that a PCC thickness of 6.5 inches overlying 6 inches of Class 2 aggregate base over 12 inches of chemically treated subgrade, as discussed above, is sufficient to support a traffic index (TI) of 7 or 8. As discussed above, to provide additional mitigation of the extreme expansion potential of the claystone in areas of cut, the pavement sections may be supported on additional NEF, if this option is desired, we can provide additional recommendations as needed.

### 10.2.1 Stress Pads for Trash Enclosures

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

## 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 24 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** | 1/3 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

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.

Highly expansive claystone bedrock or soils should not be used as retaining wall backfill.

### 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. Because walls greater than about 6 feet are planned, and peak ground accelerations greater than 0.40g are expected, we recommend checking the walls for the seismic condition in accordance with the interim recommendations of the above referenced paper and the 2019 CBC.

The CBC prescribes basic load combinations for structures, components and foundations with the intention that their design strength equals or exceeds the effects of the factored loads. With

respect to the load from lateral earth pressure and groundwater pressure, the CBC prescribes the basic combinations shown in CBC equations 16-2 and 16-7 below.

$1.2(D + F) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$ [Eq. 16-2]

In Eq. 16-2: H - should represent the total static lateral earth pressure, which for the site walls will be unrestrained (use 45 pcf)

$0.9(D + F) + 1.0E + 1.6H$ [Eq. 16-7]

In Eq. 16-7: H - should represent the static “active” earth pressure component under seismic loading conditions (use 45 pcf)

E - should represent the seismic increment component in Eq. 16-7, a triangular load with a resultant force of $14H^2$, which should be applied one third of the height up from the base of the wall (and which can also be expressed as an equivalent fluid pressure equal to 28 pcf).

The interim recommendations in the SEAOC paper more appropriately split out "active" earth pressure from the seismic earth pressure increment so that different load factors can be applied in accordance with different risk levels.

## 11.3 WALL DRAINAGE

### 11.3.1 At-Grade Site Walls

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 93 percent relative compaction at 3 percent over optimum moisture content 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 and or spread footing designed in accordance with the recommendations presented in the “Foundations” section of this report. Once building and grading plans are finalized, it may be necessary to modify retaining wall foundation embedment depths depending on the anticipated subsurface conditions.

## SECTION 12: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of HPA, Inc. specifically to support the design of the 644 & 675 Piercy Road Industrial 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.

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

Cornerstone prepared this report with the understanding that it is the responsibility of the owner or their 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

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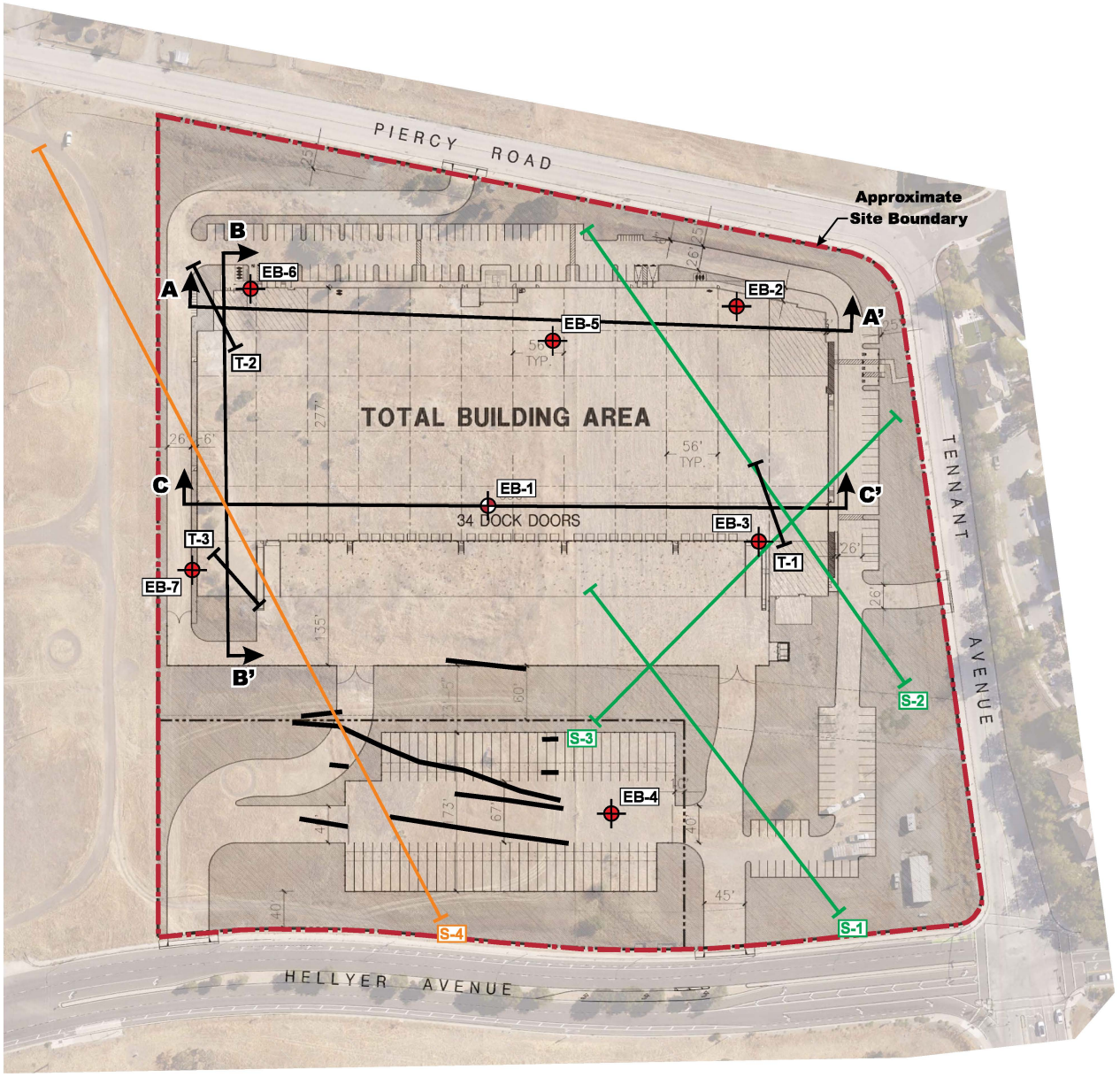



CORNERSTONE EARTH GROUP

**Vicinity Map**

**644 & 675 Piercy Road Industrial**
**San Jose, CA**

| | |
|---|---|
| Project Number | 1300-2-1 |
| Figure Number | Figure 1 |
| Date | November 2022 |
| Drawn By | FLL |




**Legend**

- Approximate location of faults by 2021 Alta Survey
- S-1 — Seismic Refraction (S-1) (JRA, 2022)
- S-4 — Seismic Refraction/MASW (S-4) (JRA, 2021)
- T-1 — Approximate location of fault trench (T-1)
- EB-1 — Approximate location of exploratory boring (EB) (Cornerstone, October 2021)
- EB-2 — Approximate location of exploratory boring (EB) (Cornerstone, current investigation)
- A A' — Approximate location of cross section

0 120 240

APPROXIMATE SCALE (FEET)

Base by DroneDeploy, dated 08/19/2022
Overlay by HPA Architecture, Conceptual Site Plan, dated 10/21/2022

Project Number 1300-2-1

Figure Number Figure 2

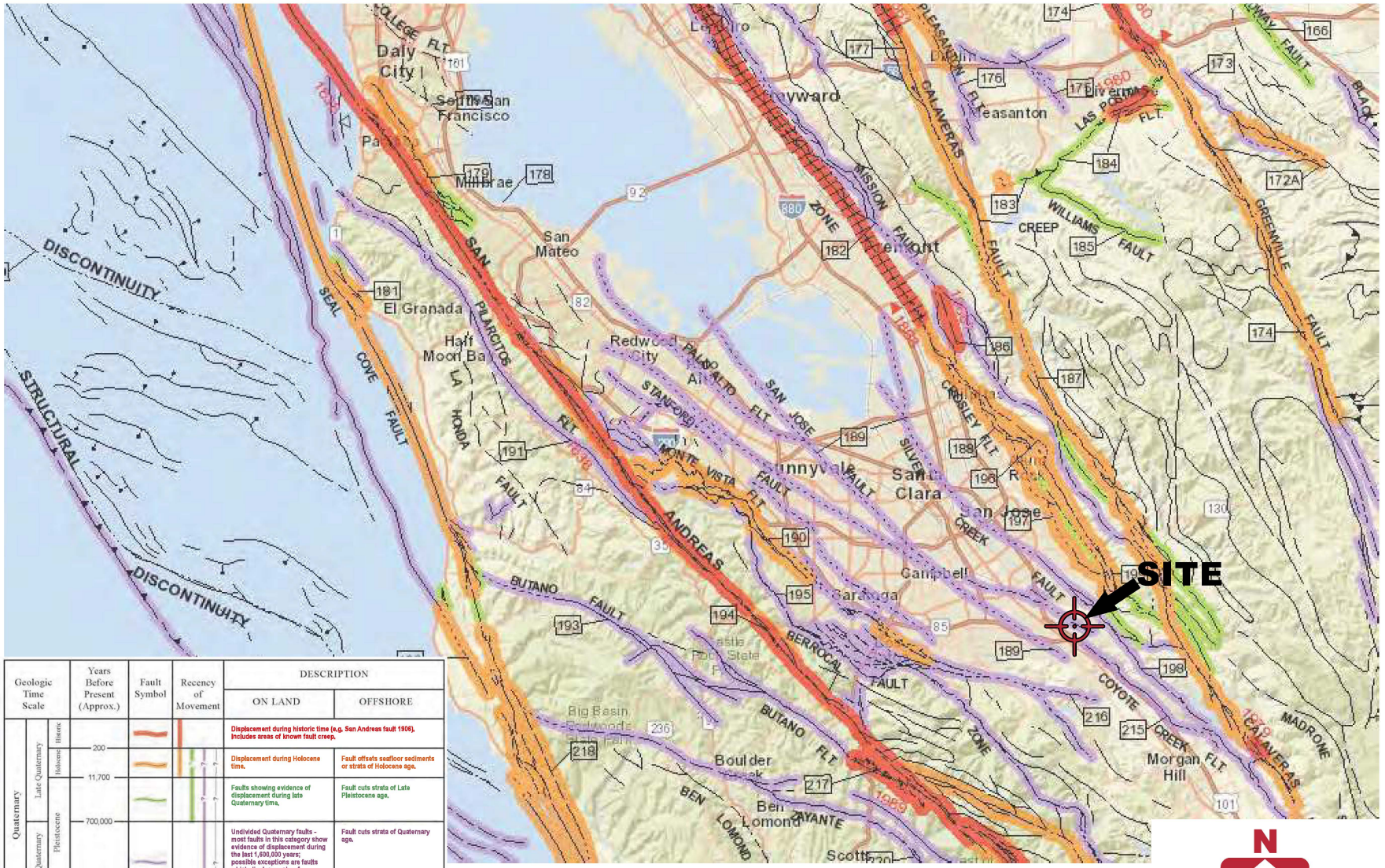
Date December 2022

Drawn By RRN

Site Exploration Plan

644 & 675 Piercy Road Industrial
San Jose, CA

CORNERSTONE EARTH GROUP




| Geologic Time Scale | Years Before Present (Approx.) | Fault Symbol | Recency of Movement | DESCRIPTION: ON LAND | DESCRIPTION: OFFSHORE |
|---|---|---|---|---|---|
| Quaternary – Historic | 200 | | | Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep. | |
| Quaternary – Holocene | 11,700 | | | Displacement during Holocene time. | Fault offsets seafloor sediments or strata of Holocene age. |
| Quaternary – Late Quaternary | | | | Faults showing evidence of displacement during late Quaternary time. | Fault cuts strata of Late Pleistocene age. |
| Quaternary – Pleistocene / Early Quaternary | 700,000 | | | Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age. | Fault cuts strata of Quaternary age. |
| Pre-Quaternary | 1,600,000 | | | 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. |
| | 4.5 billion (Age of Earth) | | | | |

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

Project Number: 1300-2-1

Figure Number: Figure 3

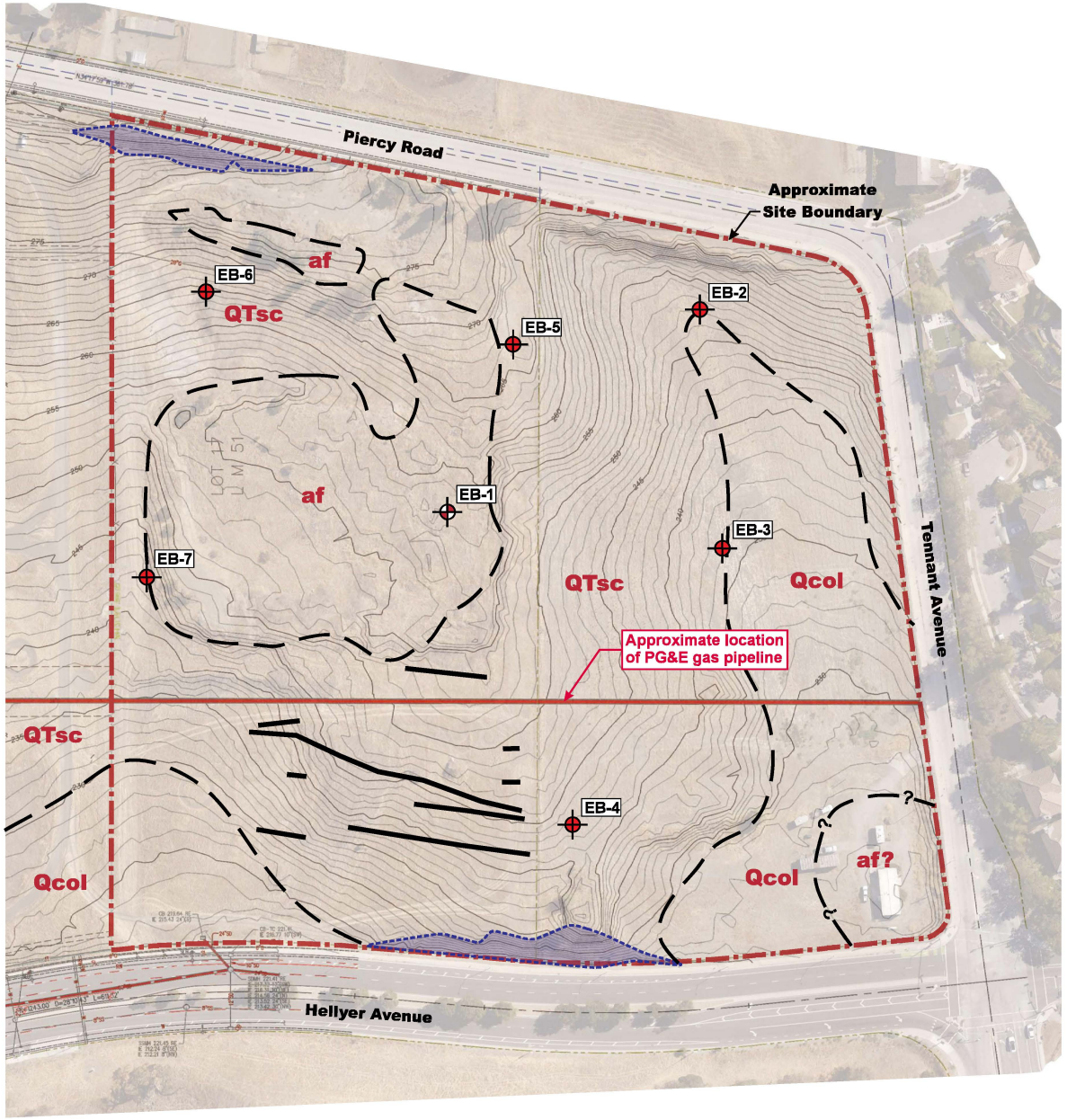
Drawn By: RRN

Date: November 2022

**Regional Fault Map**

**644 & 675 Piercy Road Industrial**
**San Jose, CA**

CORNERSTONE EARTH GROUP




## Explanation

### Geologic Units

- af — Artificial Fill
- Qcol — Colluvium
- QTsc — Santa Clara Formation

### Geologic Symbols

- Approximate location of cut slope
- Approximate location of geologic contact
- Approximate location of fault or shear mapped by E2C, 2000 in parcel 550 only; parcel 644 undated E2C mapping
- EB-1 — Approximate location of exploratory boring (EB) (Cornerstone, October 2021)
- EB-2 — Approximate location of exploratory boring (EB) (Cornerstone, current investigation)

0 120 240

APPROXIMATE SCALE (FEET)

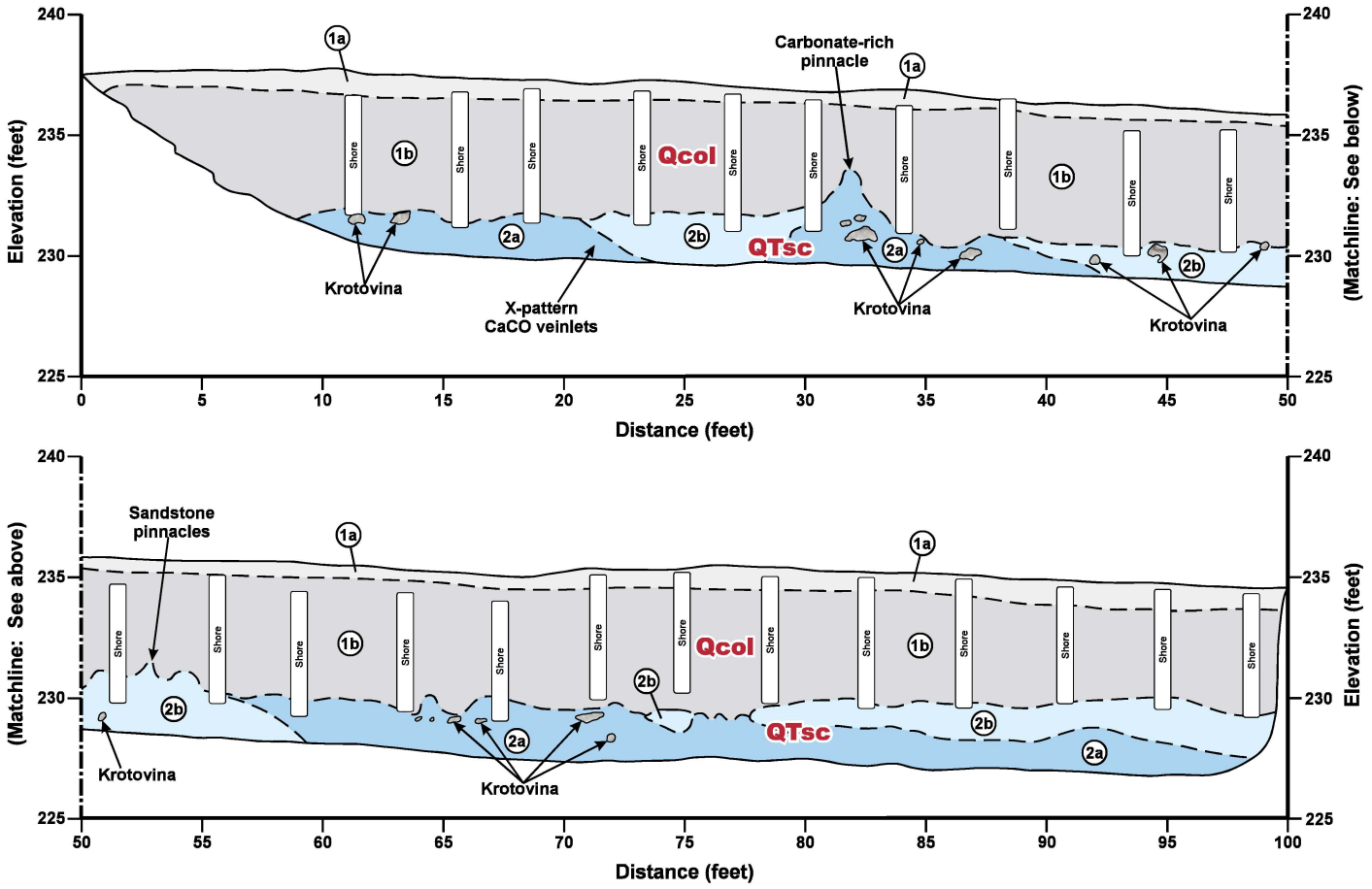
Base by DroneDeploy, dated 08/19/2022
Overlay by Kier & Wright, dwg 2021-10-20 A21611-Model-Layout

Project Number 1300-2-1
Figure Number Figure 4
Date November 2022
Drawn By RRN

Geologic and Exploration Plan

644 & 675 Piercy Road Industrial
San Jose, CA

CORNERSTONE EARTH GROUP




**Unit 1 (Colluvium, Qcol)**

(1a) **Lean Silty Clay with Sand (CL-SM):** Dark Brown to Black, loose to moderately stiff, trace gravel, dry to damp, disturbed by tilling. Increased in rounded gravels towards south end of trench (0-30cm, A-horizon).

(1b) **Fat Clay (CH):** Dark Brown to Black, stiff to very stiff, damp. Lower plasticity on SW end, occasional large grain sand, CaCO nodules, expansion (dessication) cracking throughout (mostly vertical). Absence of roots and shears. (30-60cm, A/B to 2 Bkt horizon). Transitional boundary with Unit 2 below.

**Unit 2 (Santa Clara Formation, QTsc)**

(2a) **Sandy Siltstone:** Light Brown to Tan, (Qtsc) friable, massive, locally hard (cementation); CaCo veinlets and nodules, contains abundant Krotovina burrows; consistency of a non-lithified silt (ML) at SW end of trench. (60-125cm Bbk horizon).

(2b) **Sandy Siltstone:** Yellow Brown to medium Brown and Olive Grey, prominent thin interbeds of sandstone, locally consistency of silty clay, damp. CaCo veinlets and locally cemented, Appears to occupy scour channels in subunit 2a. Krotovina burrows in central portion of trench, increase in gravels at SW end.

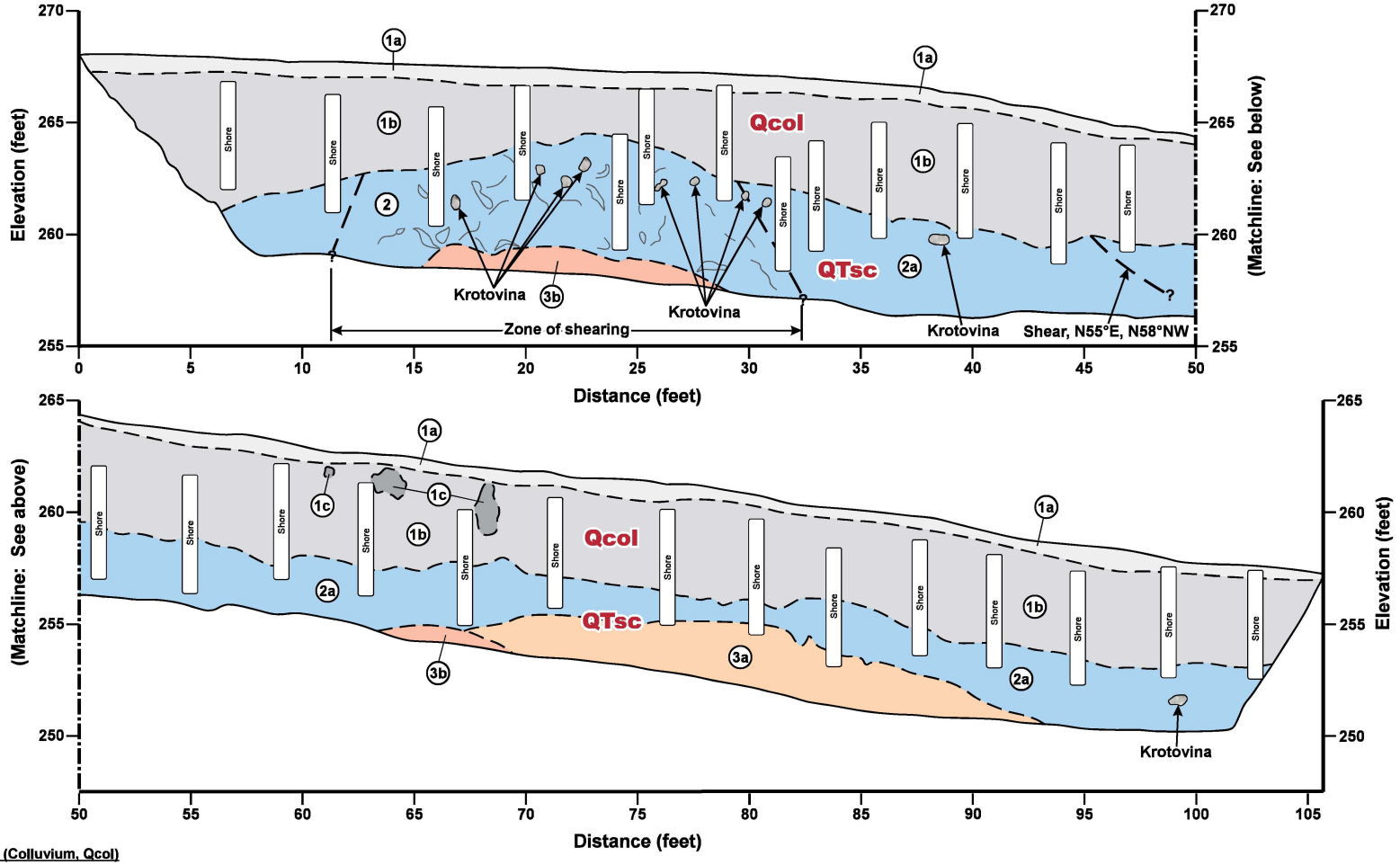
**Trench Log T-1**
(View Looking East)
1"=5' H:V

| | |
|---|---|
| Project Number | 1300-2-1 |
| Figure Number | Figure 5 |
| Drawn By | RRN |
| Date | November 2022 |

Trench Log T-1

644 & 675 Piercy Road Industrial
San Jose, CA

CORNERSTONE EARTH GROUP




**Unit 1 (Colluvium, Qcol)**

(1a) **Silty Clay (CL):** Dark Brown, medium stiff, dry; uniform thickness (0-30cm, A-horizon). Transitional contact with Subunit 1b below.

(1b) **Silty Fat Clay (CH):** Dark Brown to Black, dry, stiff to very still, contains some subangular gravels, CaCO nodules and patches (see Subunit 1c). Vertical expansion (dessication) cracking with interconnected horizontal cracking. (30-130cm, A/B-horizon over Bsskt horizon).

(1c) **Clayey Sandy Gravel (SC)** in irregular 2ft x 0.5ft patches: White to light Gray, calcareous coatings.

**Unit 2 (Santa Clara Formation, QTsc):** White to Buff Siltstone to Greenish-Gray Claystone. Moderately strong, intensely fractured to locally crushed, contains Krotovina burrows and trace gravels. (130-170cm 2BCrkt -horizon). At Sta 32+00 transitions to Subunit 2a.

(2a) **Claystone:** Light Green with white mottled Greenish-Gray, moderately strong, intensely fractured, contains abundant CaCO nodules and coatings. Increase and abundance of very thin, discontinuous shears surfaces of clay and silt. Random orientation with a few oriented to dip moderately to the SW (noted on log). Shears to not extend upward into Subunit 1b.

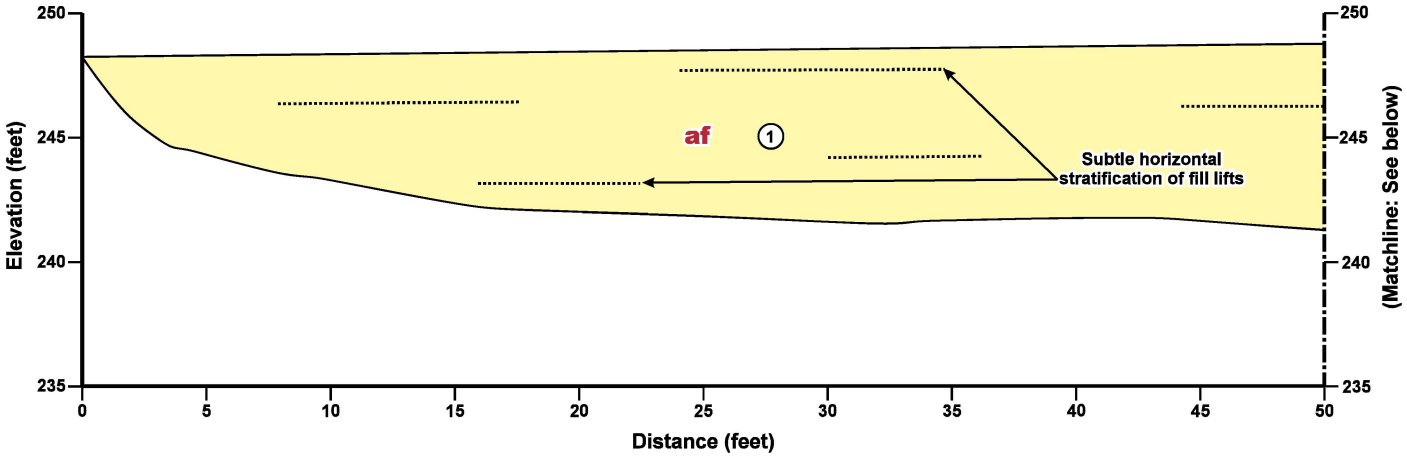
(3) **Claystone:** Light Olive Green, weak, intensely fractured, CaCO coatings, transitional contact with Subunit 2a above. (170-250+cm 2Crk -horizon).

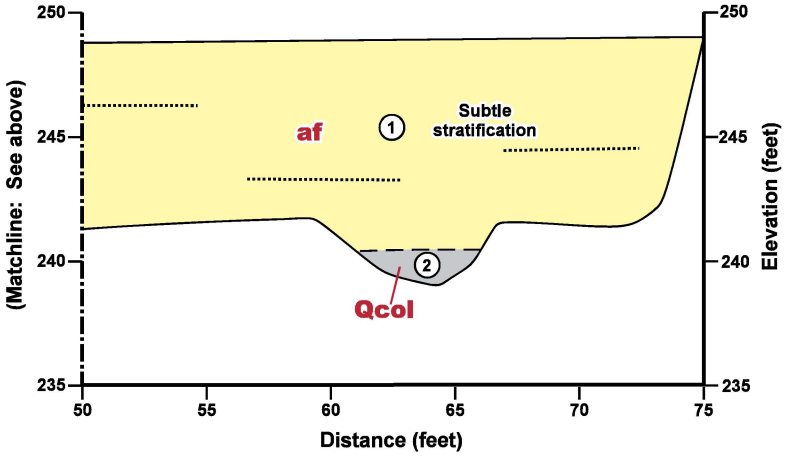
**Trench Log T-2**
(View Looking East)
1"=5' H:V

Project Number 1300-2-1
Figure Number Figure 6
Drawn By RRN
Date November 2022

Trench Log T-2
644 & 675 Piercy Road Industrial
San Jose, CA

CORNERSTONE EARTH GROUP




**Unit 1 (Fill, af)**

(1) **Sandy Clay with Gravel (SM):** Light Brown and Dark Gray with mottle red brown and white Silty, med stiff, damp.

**Unit 2 (Colluvium, Qcol)**

(2) **Fat Clay (CH):** Dark Brown to Gray, stiff, damp.

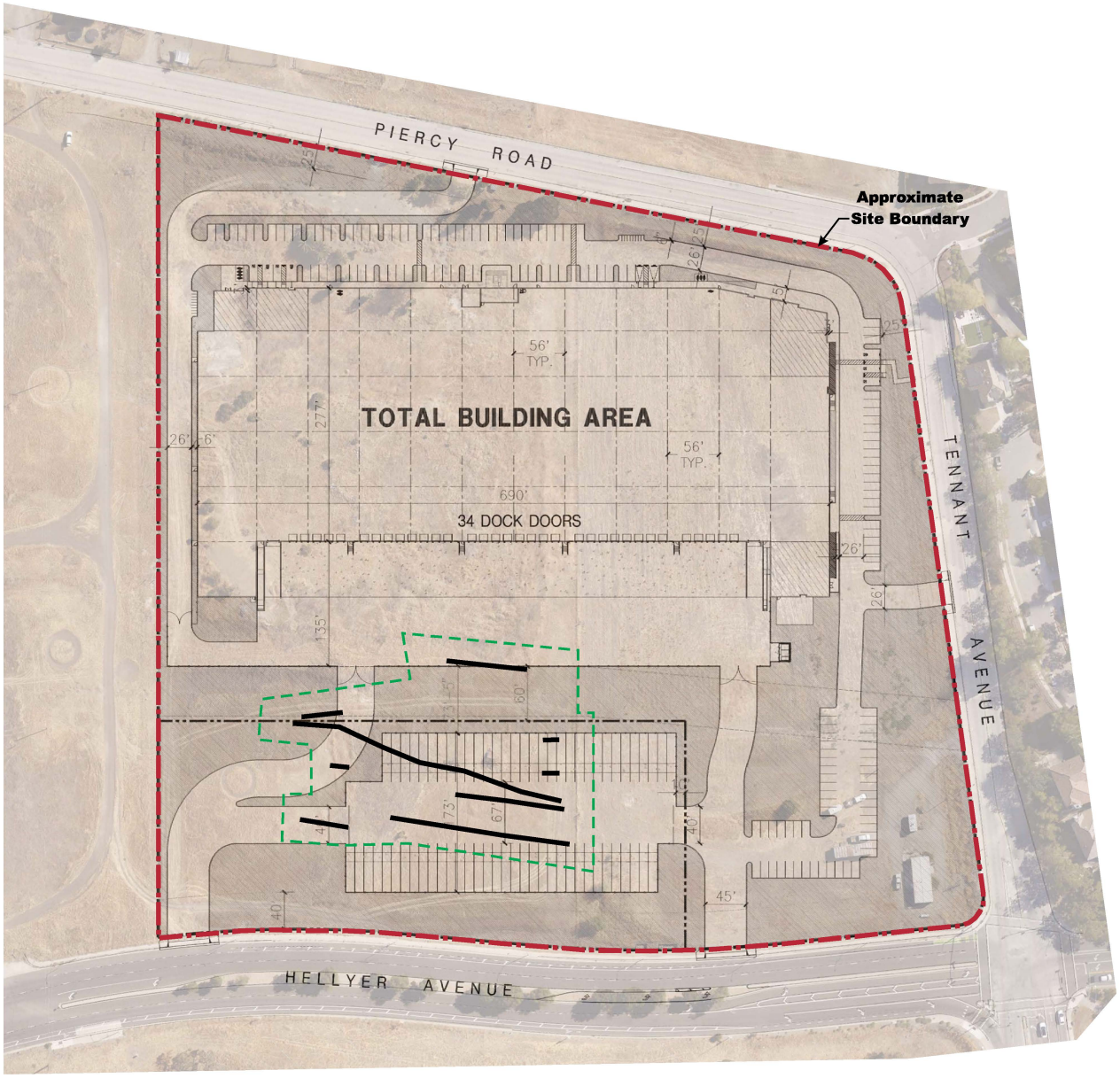
Note: Trench T-3 was not shored or entered. Contact between Unit 1 and 2 by tape measurement. Unit soils description from spoils pile.

**Trench Log T-3**
(View Looking East)
1"=5' H:V

Project Number 1300-2-1
Figure Number Figure 7
Date November 2022
Drawn By RRN

Trench Log T-3
644 & 675 Piercy Road Industrial
San Jose, CA

CORNERSTONE EARTH GROUP




**Legend**

Approximate location of faults by 2021 Alta Survey

Approximate location of proposed fault setback

N

0 120 240

APPROXIMATE SCALE (FEET)

Base by DroneDeploy, dated 08/19/2022
Overlay by HPA Architecture, Conceptual Site Plan, dated 10/21/2022

Project Number 1300-2-1

Figure Number Figure 8

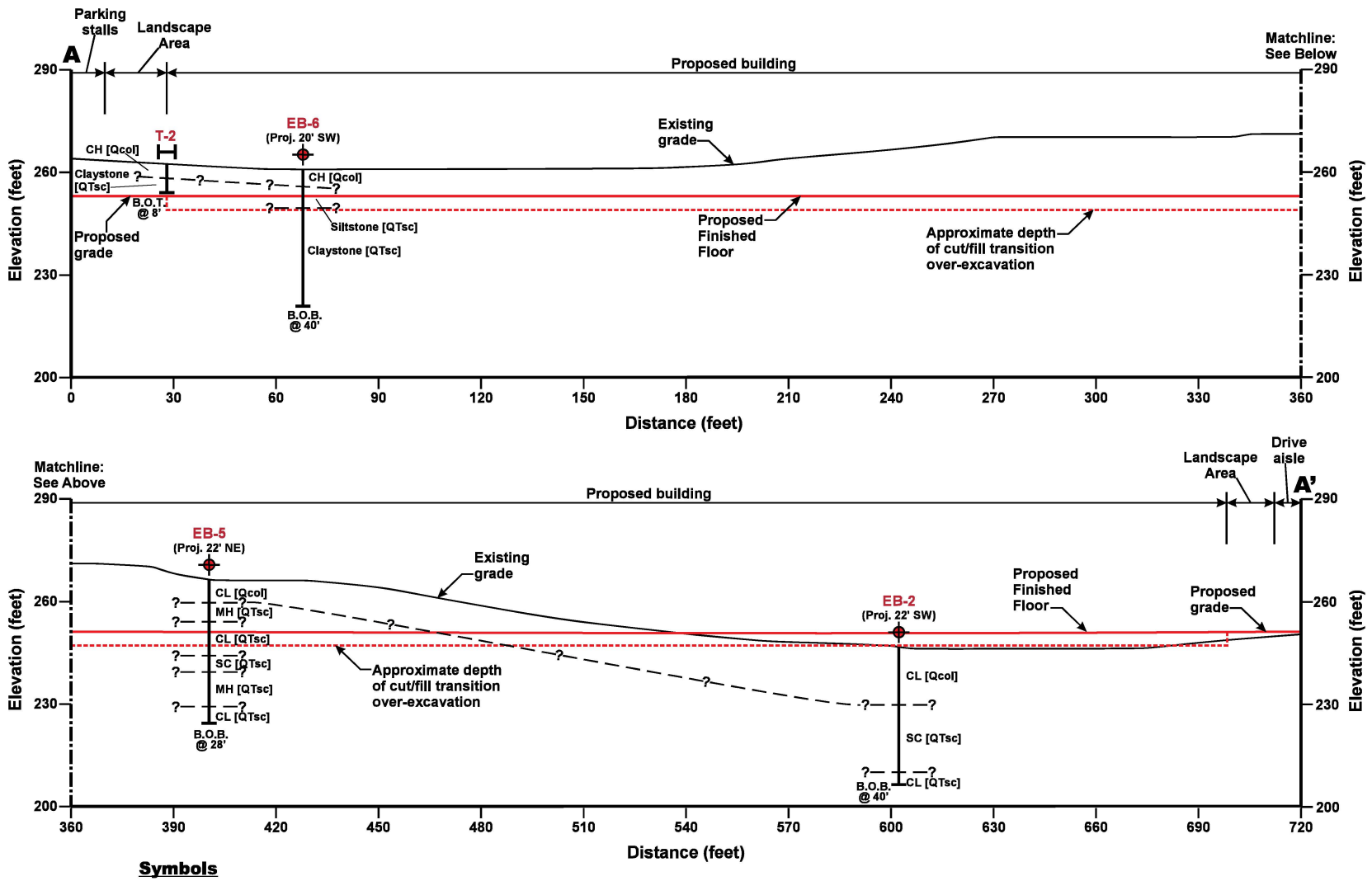
Date November 2022

Drawn By RRN

**Proposed Fault Setback Plan**

**644 & 675 Piercy Road Industrial**
**San Jose, CA**

CORNERSTONE EARTH GROUP




## Symbols

| | |
|---|---|
| CL | Lean Clay |
| CH | Fat Clay |
| MH | Expansive Silt |
| SC | Clayey Sand |
| [Qcol] | Colluvium |
| [QTsc] | Santa Clara Formation |

Approximate location of fault trench (T-2)

Approximate location of exploratory boring (EB) (Cornerstone, current investigation)

## Section A-A'

(View Looking Northeast)
1"=30' H:V

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.

Project Number 1300-2-1

Figure Number Figure 9

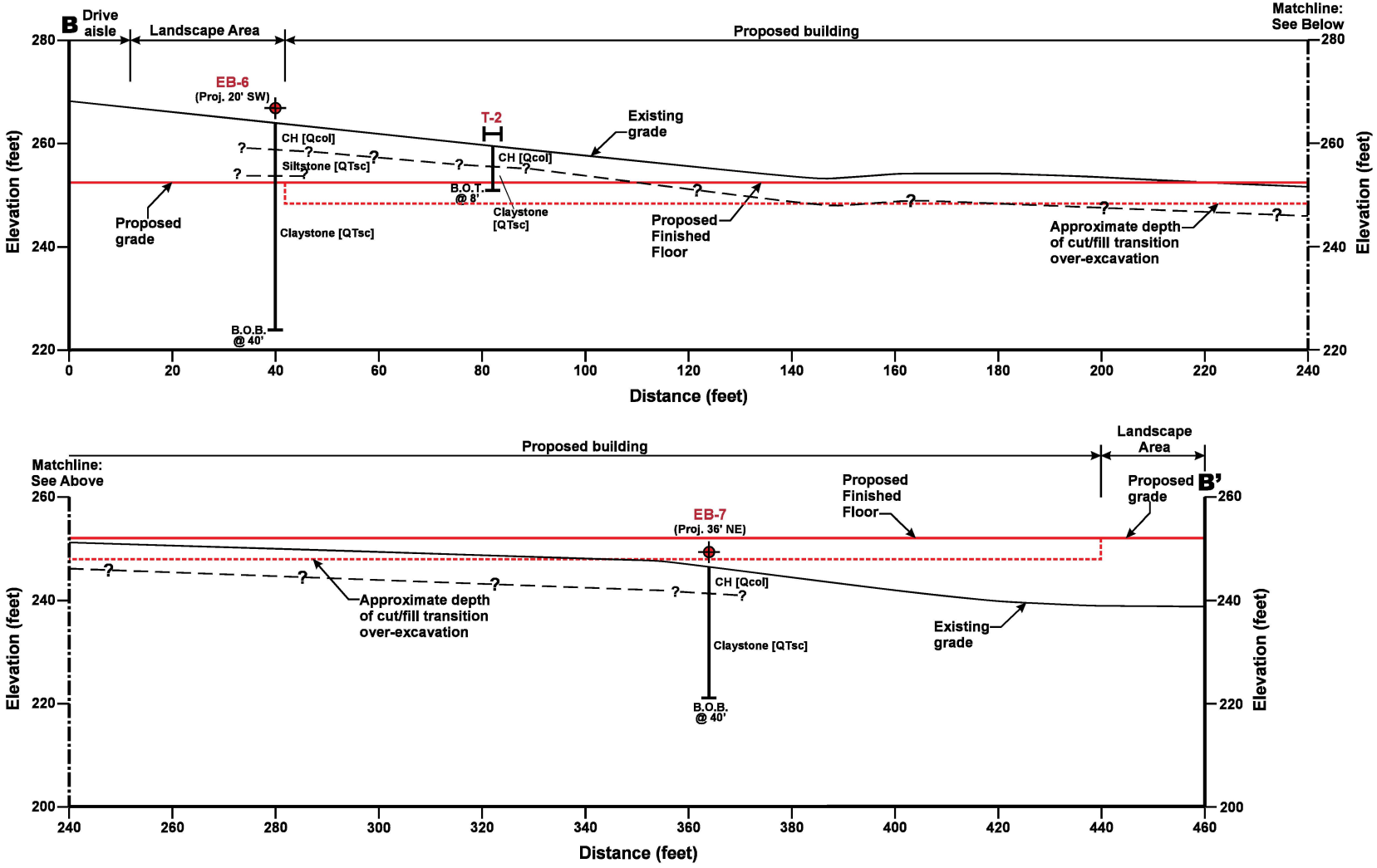
Drawn By RRN

Date December 2022

Geologic Cross Section A-A'

644 & 675 Piercy Road Industrial
San Jose, CA

CORNERSTONE EARTH GROUP




**Symbols**

| | | | |
|---|---|---|---|
| CL | Fat Clay | H | Approximate location of fault trench (T-2) |
| [Qcol] | Colluvium | ⊕ | Approximate location of exploratory boring (EB) (Cornerstone, current investigation) |
| [QTsc] | Santa Clara Formation | | |

## Section B-B'

(View Looking Southeast)
1"=20' H:V

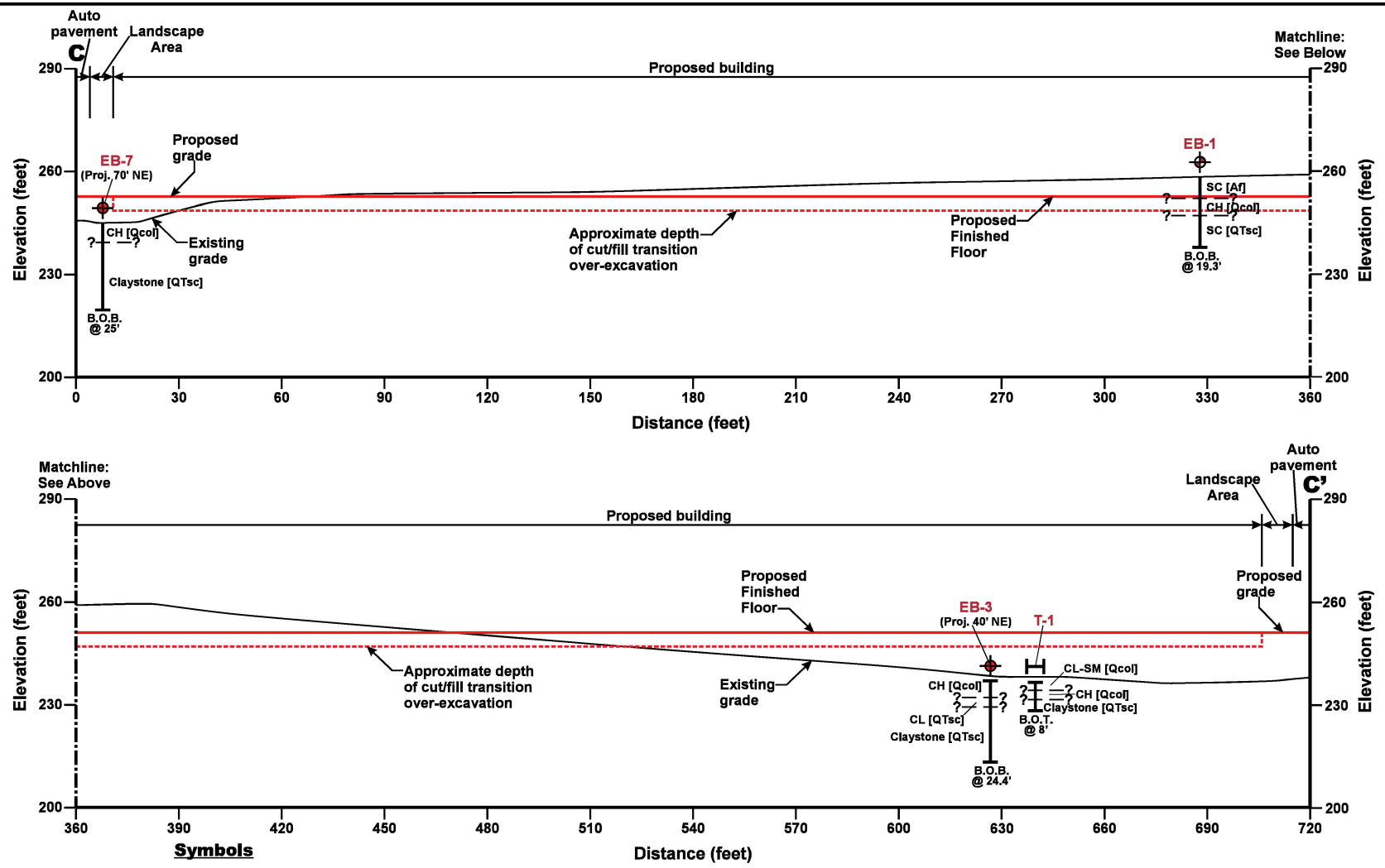
Notes:
1) Surficial fills associated with existing pavements, landscaping or utilities are not shown.
2) The subsurface profile is conceptual and is based on limited subsurface data obtained from widely spaced borings. Actual subsurface conditions may vary significantly between borings.
3) See Figure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

Project Number 1300-2-1
Figure Number Figure 10
Drawn By RRN
Date December 2022

Geologic Cross Section B-B'

644 & 675 Piercy Road Industrial
San Jose, CA

CORNERSTONE EARTH GROUP




**Symbols**

| | |
|---|---|
| CL | Lean Clay |
| CH | Fat Clay |
| CL-SM | Lean Silty Clay with Sand |
| SC | Clayey Sand with Gravel |
| [Af] | Artificial fill |
| [Qcol] | Colluvium |
| [QTsc] | Santa Clara Formation |

- Approximate location of fault trench (T-1)
- Approximate location of exploratory boring (EB) (Cornerstone, October 2021)
- Approximate location of exploratory boring (EB) (Cornerstone, current investigation)

## Section C-C'

(View Looking Northeast)
1"=30' H:V

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.

Project Number 1300-2-1
Figure Number Figure 11
Drawn By RRN
Date December 2022

Geologic Cross Section C-C'

644 & 675 Piercy Road Industrial
San Jose, CA

CORNERSTONE EARTH GROUP

## APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using a combination of truck-mounted and track-mounted, hollow-stem, limited-access auger drilling equipment. Seven 8-inch-diameter exploratory borings were drilled on September 27, 2021, and November 7 and 8, 2022 to depths of 19 to 44 feet. The approximate locations of exploratory borings are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Boring logs, as well as a key to the classification of the soil and bedrock, are included as part of this appendix.

Boring locations were approximated using existing site boundaries, a hand held GPS unit, and other site features as references. Boring elevations were 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. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows required to drive the last 12 inches. The various samplers are denoted at the appropriate depth on the boring logs.

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

Attached boring logs and related information depict subsurface conditions at the locations indicated and on the date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these boring locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.

# UNIFIED SOIL CLASSIFICATION (ASTM D-2487-10)

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

## OTHER MATERIAL SYMBOLS

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

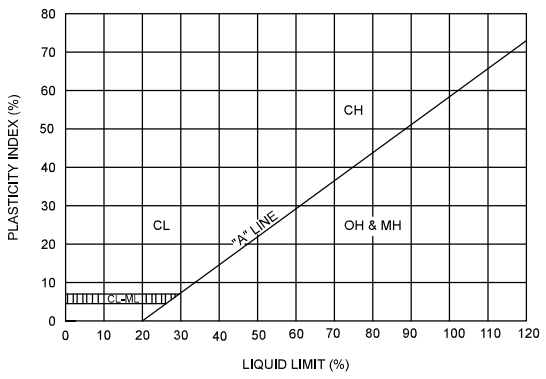
## SAMPLER TYPES

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

## ADDITIONAL TESTS

| | | | | | |
|---|---|---|---|---|---|
| CA | - | CHEMICAL ANALYSIS (CORROSIVITY) | 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

**Soft –** Reserved for plastic material alone.
**Low hardness –** Can be gouged deeply or carved easily with a knife blade.
**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.
**Hard –** Can be scratched with difficulty: scratch produces little powder and is often faintly visible.
**Very hard –** Cannot be scratched with knife blade: leaves a metallic streak.

## STRENGTH

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

**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.
**Moderate –** Slight change or partial decomposition of minerals: little disintegration: cementation little to unaffected. Moderate to occasionally intense discoloration. Moderately coated fractures.
**Little –** No megascopic decomposition of minerals: little or no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains or fracture surfaces.
**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** 550 Piercy Road
**PROJECT NUMBER** 1076-4-2
**PROJECT LOCATION** San Jose, CA

**DATE STARTED** 9/27/21 **DATE COMPLETED** 9/27/21
**GROUND ELEVATION** 258 FT +/- **BORING DEPTH** 19.3 ft.
**DRILLING CONTRACTOR** Cuesta Geo **LATITUDE** 37.25237° **LONGITUDE** -122.77084°
**DRILLING METHOD** MPP LAD Track Rig, 6½ inch Hollow-Stem Auger **GROUND WATER LEVELS:**
**LOGGED BY** EA ▽ **AT TIME OF DRILLING** Not Encountered
**NOTES** ▼ **AT END OF DRILLING** Not Encountered

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| 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 |
|---|---|---|---|---|---|---|---|---|
| 258.0 | 0 | **Clayey Sand with Gravel (SC) [Fill]** medium dense, moist, brown and gray mottled, fine to coarse sand, fine to coarse subangular gravel, some concrete debris | 30 | MC-1B | 113 | 8 | | |
| | | | 24 | MC-2B | 118 | 10 | | |
| | 5 | | 18 | MC-3B | 98 | 20 | | |
| 252.0 | | **Fat Clay with Sand (CH) [Qcol]** very stiff, moist, dark grayish brown to brown, fine sand, high plasticity | | | | | | |
| | | | 22 | MC-4B | 93 | 28 | | |
| | 10 | | | | | | | |
| 247.0 | | **Clayey Sand (SC) [QTsc]** very dense, moist, brown with light brown mottles, fine sand | 50/6" | MC-5B | 82 | 32 | | 21 |
| | 15 | | | | | | | |
| | | | 50/3" | SPT-6 | | 20 | | |
| 238.7 | | Bottom of Boring at 19.3 feet. | | | | | | |
| | 20 | | | | | | | |
| | 25 | | | | | | | |

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

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 12/1/22 09:14 - P:\DRAFTING\GINT FILES\1076-4-2 550 PIERCY ROAD.GPJ

# BORING NUMBER EB-2



CORNERSTONE EARTH GROUP

**PROJECT NAME** 644-675 Piercy Road
**PROJECT NUMBER** 1300-2-1
**PROJECT LOCATION** San Jose, CA

**DATE STARTED** 11/7/22 **DATE COMPLETED** 11/7/22
**GROUND ELEVATION** 247 FT +/- **BORING DEPTH** 40 ft.
**DRILLING CONTRACTOR** Exploration Geoservices Inc. **LATITUDE** 37.252288° **LONGITUDE** -121.769660°
**DRILLING METHOD** Mobile B-61, 8 inch Hollow-Stem Auger **GROUND WATER LEVELS:**
**LOGGED BY** RA ▽ **AT TIME OF DRILLING** 35 ft.
**NOTES** ▼ **AT END OF DRILLING** 29 ft.

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| 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 (○ HAND PENETROMETER, △ TORVANE, ● UNCONFINED COMPRESSION, ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL) |
|---|---|---|---|---|---|---|---|---|---|
| 247.0 | 0 | **Lean Clay with Sand (CL) [Qcol]**<br>hard, moist, dark brown, fine to medium sand, some fine subangular gravel, moderate plasticity | 26 | MC-1B | 105 | 12 | | | >4.5 ○ |
| | | Liquid Limit = 47, Plastic Limit = 15 | 29 | MC-2B | 116 | 15 | 32 | | >4.5 ○ |
| | 5 | | 27 | MC | | | | | >4.5 ○ |
| | | | 59 | MC-4B | 112 | 17 | | | >4.5 ○ |
| | 10 | | | | | | | | |
| 235.0 | | **Sandy Lean Clay (CL) [QTsc]**<br>hard, moist, gray and brown mottled, fine to medium sand, trace fine gravel, moderate plasticity | 70 | MC-5 | 104 | 16 | | | >4.5 ○ |
| | 15 | | | | | | | | |
| 230.0 | | **Clayey Sand (SC) [QTsc]**<br>dense, moist, brown, fine to medium sand, trace fine gravel | 60 | MC-6B | 96 | 23 | | | |
| | 20 | | | | | | | | |
| | | becomes very dense | 56 | SPT | | | | | |
| | 25 | | | | | | | | |
| 221.0 | | *Continued Next Page* | | | | | | | |

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# BORING NUMBER EB-2



CORNERSTONE EARTH GROUP

**PROJECT NAME** 644-675 Piercy Road

**PROJECT NUMBER** 1300-2-1

**PROJECT LOCATION** San Jose, CA

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| ELEVATION (ft) | DEPTH (ft) | SYMBOL | DESCRIPTION | N-Value (uncorrected) blows per foot | SAMPLES TYPE AND NUMBER | DRY UNIT WEIGHT PCF | NATURAL MOISTURE CONTENT | PLASTICITY INDEX, % | PERCENT PASSING No. 200 SIEVE | UNDRAINED SHEAR STRENGTH, ksf |
|---|---|---|---|---|---|---|---|---|---|---|
| 221.0 | | | **Clayey Sand (SC) [QTsc]**<br>medium dense, moist, brown, fine to medium sand, trace fine gravel | | | | | | | |
| | 30 | | | 40 | MC-8B | 105 | 19 | | | |
| 215.0 | | | **Clayey Sand with Gravel (SC) [QTsc]**<br>very dense, moist, brown, fine to medium sand, fine subangular gravel | | | | | | | |
| | 35 | | | 50/6" | SPT-9 | | 14 | | | |
| 210.0 | | | **Lean Clay with Sand (CL) [QTsc]**<br>hard, moist, brown, fine to medium sand, moderate plasticity | | | | | | | |
| 207.0 | 40 | | Bottom of Boring at 40.0 feet. | 50/6" | SPT-10 | | 16 | | | >4.5 |
| | 45 | | | | | | | | | |
| | 50 | | | | | | | | | |
| | 55 | | | | | | | | | |

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

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# BORING NUMBER EB-3



CORNERSTONE EARTH GROUP

**PROJECT NAME** 644-675 Piercy Road
**PROJECT NUMBER** 1300-2-1
**PROJECT LOCATION** San Jose, CA

**DATE STARTED** 11/7/22 **DATE COMPLETED** 11/7/22
**GROUND ELEVATION** 237.5 FT +/- **BORING DEPTH** 24.4 ft.
**DRILLING CONTRACTOR** Exploration Geoservices Inc. **LATITUDE** 37.251732° **LONGITUDE** -121.770220°
**DRILLING METHOD** Mobile B-61, 8 inch Hollow-Stem Auger **GROUND WATER LEVELS:**
**LOGGED BY** RA **AT TIME OF DRILLING** Not Encountered
**NOTES** **AT END OF DRILLING** Not Encountered

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| 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 |
|---|---|---|---|---|---|---|---|---|---|
| 237.5 | 0 | **Fat Clay with Sand (CH) [Qcol]**<br>very stiff to hard, moist, dark brown, fine to medium sand, trace fine subangular gravel, high plasticity | 22 | MC-1B | 100 | 26 | | | >4.5 |
| | | Liquid Limit = 81, Plastic Limit = 24 | 44 | MC-2B | 94 | 25 | 57 | | >4.5 |
| 232.5 | 5 | **Lean Clay (CL) [QTsc]**<br>hard, moist, brown with light brown mottles, some fine to medium sand, trace fine gravel, moderate plasticity | 40 | MC-3B | 93 | 21 | | | >4.5 |
| 229.5 | | **Siltstone / Mudstone [QTsc]**<br>low hardness, friable, deep weathering, brown gray mottled, moderate plasticity | 50/3" | MC-4 | | 23 | | | >4.5 |
| | 10 | | 50/6" | SPT-5 | | 28 | | | >4.5 |
| | 15 | | 50/6" | SPT-6 | | 26 | | | >4.5 |
| | 20 | | 50/5" | SPT-7 | | 30 | | | >4.5 |
| 213.1 | 25 | Bottom of Boring at 24.4 feet. | | | | | | | |

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

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# BORING NUMBER EB-4



CORNERSTONE EARTH GROUP

**PROJECT NAME** 644-675 Piercy Road
**PROJECT NUMBER** 1300-2-1
**PROJECT LOCATION** San Jose, CA

**DATE STARTED** 11/7/22 **DATE COMPLETED** 11/7/22
**GROUND ELEVATION** 242.5 FT +/- **BORING DEPTH** 20 ft.
**DRILLING CONTRACTOR** Exploration Geoservices Inc.
**LATITUDE** 37.251480° **LONGITUDE** -121.771346°
**DRILLING METHOD** Mobile B-61, 8 inch Hollow-Stem Auger
**GROUND WATER LEVELS:**
**LOGGED BY** RA
**AT TIME OF DRILLING** Not Encountered
**NOTES**
**AT END OF DRILLING** Not Encountered

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| 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 |
|---|---|---|---|---|---|---|---|---|---|
| 242.5 | 0 | **Lean Clay with Sand (CL) [Fill]** hard, moist, brown and light brown mottled, fine to medium sand, trace fine subangular gravel, moderate plasticity | 54 | MC-1B | 107 | 15 | | | >4.5 |
| 239.5 | | **Lean Clay (CL) [Qcol]** hard, moist, brown, some fine to medium sand, moderate plasticity | 59 | MC | | | | | >4.5 |
| | 5 | | 50/6" | MC-3B | 113 | 17 | | | >4.5 |
| 235.0 | | **Sandy Lean Clay (CL) [QTsc]** hard, moist, brown, fine to medium sand, some fine subangular to subrounded gravel, moderate plasticity | 74 | MC | | | | | >4.5 |
| | 10 | | | | | | | | |
| | 15 | | 54 | SPT-5 | | 12 | | | >4.5 |
| 225.5 | | **Lean Clay with Sand (CL) [QTsc]** hard, moist, brown, fine to medium sand, moderate plasticity | 49 | SPT-6 | | 18 | | | >4.5 |
| 222.5 | 20 | Bottom of Boring at 20.0 feet. | | | | | | | |
| | 25 | | | | | | | | |

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

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# BORING NUMBER EB-5



CORNERSTONE EARTH GROUP

**PROJECT NAME** 644-675 Piercy Road
**PROJECT NUMBER** 1300-2-1
**PROJECT LOCATION** San Jose, CA

**DATE STARTED** 11/8/22 **DATE COMPLETED** 11/8/22
**GROUND ELEVATION** 266 FT +/- **BORING DEPTH** 43.9 ft.
**DRILLING CONTRACTOR** Exploration Geoservices Inc. **LATITUDE** 37.252590° **LONGITUDE** -121.770241°
**DRILLING METHOD** Mobile B-61, 8 inch Hollow-Stem Auger **GROUND WATER LEVELS:**
**LOGGED BY** RA ▽ **AT TIME OF DRILLING** Not Encountered
**NOTES** ▼ **AT END OF DRILLING** Not Encountered

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| 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 (○ HAND PENETROMETER, △ TORVANE, ● UNCONFINED COMPRESSION, ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL) |
|---|---|---|---|---|---|---|---|---|---|
| 266.0 | 0 | **Sandy Lean Clay (CL) [Qcol]**<br>hard, moist, brown with light brown mottles, fine to medium sand, some fine subangular to subrounded gravel, moderate plasticity | 50/1" | MC | | | | | >4.5 ○ |
| | | | 50/5" | MC-2B | 104 | 19 | | | >4.5 ○ |
| | 5 | | 50/5" | MC-3B | 110 | 17 | | | >4.5 ○ |
| 259.0 | | **Elastic Silt with Sand (MH) [QTsc]**<br>hard, moist, brown, fine to medium sand, high plasticity<br>Liquid Limit = 55, Plastic Limit = 38 | 50/6" | SPT-4 | | 31 | 17 | | >4.5 ○ |
| | 10 | | | | | | | | |
| 254.0 | | **Sandy Lean Clay (CL) [QTsc]**<br>hard, moist, brown, fine to medium sand, some fine subangular to subrounded gravel, moderate plasticity | 50/5" | MC-5B | 98 | 22 | | | >4.5 ○ |
| | 15 | | | | | | | | |
| | | | 50/6" | SPT-6 | | 19 | | | >4.5 ○ |
| | 20 | | | | | | | | |
| 244.0 | | **Clayey Sand (SC) [QTsc]**<br>very dense, moist, brown, fine to coarse sand, some fine subangular gravel, moderate plasticity<br>Liquid Limit = 34, Plastic Limit = 21 | 64 | SPT-7 | | 14 | 13 | | |
| | 25 | | | | | | | | |
| 240.0 | | | | | | | | | |

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# BORING NUMBER EB-5



CORNERSTONE EARTH GROUP

**PROJECT NAME** 644-675 Piercy Road

**PROJECT NUMBER** 1300-2-1

**PROJECT LOCATION** San Jose, CA

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| 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 |
|---|---|---|---|---|---|---|---|---|---|
| 240.0 | | | | | | | | | |
| 239.0 | | **Elastic Silt with Sand (MH) [QTsc]** hard, moist, brown, fine to medium sand, high plasticity | | | | | | | |
| | 30 | | 60 | SPT-8 | | 29 | | | >4.5 |
| | | | 50/6" | SPT-9 | | 25 | | | >4.5 |
| | 35 | | | | | | | | |
| 229.0 | | **Sandy Lean Clay (CL) [QTsc]** hard, moist, brown, fine to medium sand, some fine subangular gravel, moderate plasticity | 50/5" | SPT-10 | | 17 | | | >4.5 |
| | 40 | | | | | | | | |
| 222.1 | | Bottom of Boring at 43.9 feet. | 50/5" | SPT-11 | | 20 | | | >4.5 |
| | 45 | | | | | | | | |
| | 50 | | | | | | | | |
| | 55 | | | | | | | | |

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

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 12/12/22 11:19 - P:\DRAFTING\GINT FILES\1300-2-1 644-675 PIERCY RD.GPJ

# BORING NUMBER EB-6



CORNERSTONE EARTH GROUP

**PROJECT NAME** 644-675 Piercy Road
**PROJECT NUMBER** 1300-2-1
**PROJECT LOCATION** San Jose, CA

**DATE STARTED** 11/8/22 **DATE COMPLETED** 11/8/22
**GROUND ELEVATION** 263.5 FT +/- **BORING DEPTH** 40 ft.
**DRILLING CONTRACTOR** Exploration Geoservices Inc. **LATITUDE** 37.253406° **LONGITUDE** -121.771116°
**DRILLING METHOD** Mobile B-61, 8 inch Hollow-Stem Auger **GROUND WATER LEVELS:**
**LOGGED BY** RA **AT TIME OF DRILLING** Not Encountered
**NOTES** **AT END OF DRILLING** Not Encountered

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| 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 |
|---|---|---|---|---|---|---|---|---|---|
| 263.5 | 0 | **Fat Clay with Sand (CH) [Qcol]**<br>very stiff, moist, dark brown, fine to coarse sand, trace fine subangular gravel, high plasticity<br>Liquid Limit = 68, Plastic Limit = 23 | 22 | MC-1B | 97 | 20 | 45 | | >4.5 |
| | | some light brown mottles | 33 | MC-2B | 101 | 20 | | | >4.5 |
| 258.5 | 5 | **Siltstone / Mudstone [QTsc]**<br>low hardness, friable, deep weathering, brown gray mottled, moderate plasticity | 50/6" | MC-3B | 96 | 25 | | | |
| | | Liquid Limit = 69, Plastic Limit = 50 | 50/5" | MC-4B | 91 | 26 | 19 | | |
| 251.5 | 10 | **Claystone [QTsc]**<br>low hardness, friable, deep weathering, light gray and brown mottled, high plasticity | 87 | MC-5 | | 37 | | | |
| | 15 | | 50/3" | MC-6B | 82 | 30 | | | |
| | 20 | | 50/4" | SPT-7 | | 33 | | | |
| 237.5 | 25 | | | | | | | | |

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

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# BORING NUMBER EB-6



CORNERSTONE EARTH GROUP

**PROJECT NAME** 644-675 Piercy Road

**PROJECT NUMBER** 1300-2-1

**PROJECT LOCATION** San Jose, CA

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| ELEVATION (ft) | DEPTH (ft) | SYMBOL | DESCRIPTION | N-Value (uncorrected) blows per foot | SAMPLES TYPE AND NUMBER | DRY UNIT WEIGHT PCF | NATURAL MOISTURE CONTENT | PLASTICITY INDEX, % | PERCENT PASSING No. 200 SIEVE |
|---|---|---|---|---|---|---|---|---|---|
| 237.5 | | | **Claystone [QTsc]**<br>low hardness, friable, deep weathering, light gray and brown mottled, high plasticity | | | | | | |
| | 30 | | | 50/3" | SPT-8 | | 26 | | |
| | | | | 50/6" | SPT-9 | | 38 | | |
| | 35 | | | | | | | | |
| | | | | 48 | SPT-10 | | 44 | | |
| 223.5 | 40 | | Bottom of Boring at 40.0 feet. | | | | | | |
| | 45 | | | | | | | | |
| | 50 | | | | | | | | |
| | 55 | | | | | | | | |

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

1.0 2.0 3.0 4.0

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# BORING NUMBER EB-7



CORNERSTONE EARTH GROUP

**PROJECT NAME** 644-675 Piercy Road
**PROJECT NUMBER** 1300-2-1
**PROJECT LOCATION** San Jose, CA

**DATE STARTED** 11/8/22 **DATE COMPLETED** 11/8/22
**GROUND ELEVATION** 246 FT +/- **BORING DEPTH** 25 ft.
**DRILLING CONTRACTOR** Exploration Geoservices Inc.
**LATITUDE** 37.252875° **LONGITUDE** -121.771834°
**DRILLING METHOD** Mobile B-61, 8 inch Hollow-Stem Auger
**GROUND WATER LEVELS:**
**LOGGED BY** RA
**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 |
|---|---|---|---|---|---|---|---|---|---|
| 246.0 | 0 | **Fat Clay with Sand (CH) [Qcol]**<br>very stiff, moist, dark brown, fine to medium sand, high plasticity | 43 | MC | | | | | >4.5 |
| | | | 24 | MC-2B | 75 | 26 | | | >4.5 |
| 241.0 | 5 | **Claystone [QTsc]**<br>soft to low hardness, plastic, deep weathering, gray and light gray mottled, high plasticity<br>Liquid Limit = 128, Plastic Limit = 42 | 28 | MC-3B | 75 | 39 | 86 | | |
| | | | 30 | MC-4B | 64 | 58 | | | |
| | 10 | | | | | | | | |
| | 15 | | 50 | MC-5B | 83 | 39 | | | |
| | | | 55 | MC-6B | 84 | 36 | | | |
| | 20 | | | | | | | | |
| | | | 50/6" | MC-7B | 91 | 31 | | | |
| 221.0 | 25 | Bottom of Boring at 25.0 feet. | | | | | | | |

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

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 12/12/22 11:19 - P:\DRAFTING\GINT FILES\1300-2-1 644-675 PIERCY RD.GPJ

## APPENDIX B: LABORATORY TEST PROGRAM

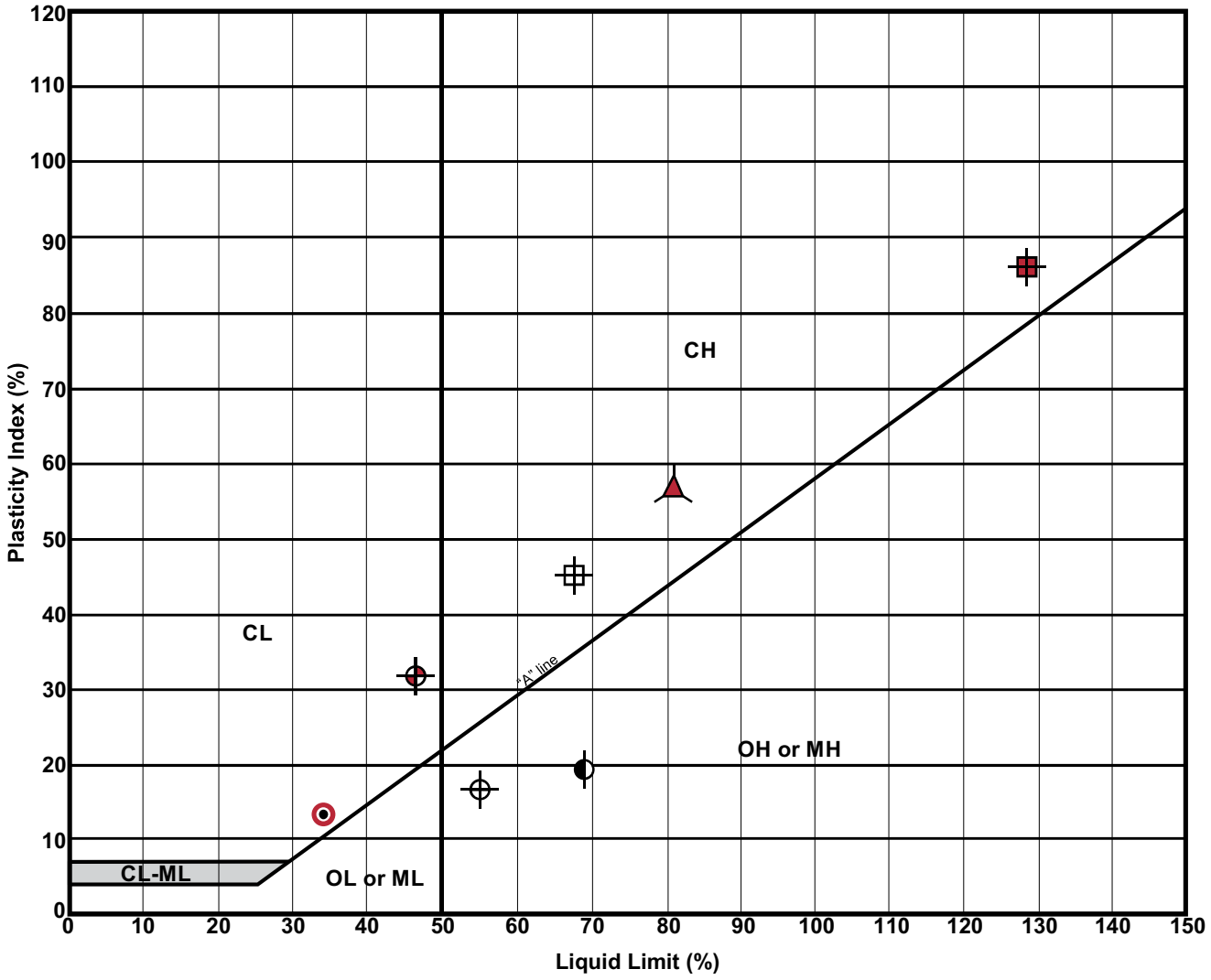
The laboratory testing program was performed to evaluate the physical and mechanical properties of the soils retrieved from the site to aid in verifying soil classification.

**Moisture Content:** The natural water content was determined (ASTM D2216) on 51 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 30 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:** Seven Plasticity Index determinations (ASTM D4318) were performed on samples of the subsurface soils to measure the range of water contents over which this material exhibits plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential. Results of these tests are shown on the boring logs at the appropriate sample depths.

# Plasticity Index (ASTM D4318) Testing Summary




| Symbol | Boring No. | Depth (ft) | Natural Water Content (%) | Liquid Limit (%) | Plastic Limit (%) | Plasticity Index | Passing No. 200 (%) | Group Name (USCS - ASTM D2487) |
|---|---|---|---|---|---|---|---|---|
| ⊕ | EB-2 | 4.0 | 12 | 47 | 15 | 32 | — | Lean Clay with Sand (CL) |
| ▲ | EB-3 | 4.0 | 25 | 81 | 24 | 57 | — | Fat Clay with Sand (CH) |
| ⊕ | EB-5 | 8.5 | 31 | 55 | 38 | 17 | — | Elastic Silt with Sand (MH) |
| ◉ | EB-5 | 23.5 | 14 | 34 | 21 | 13 | — | Clayey Sand (SC) [QTsc] |
| ⊞ | EB-6 | 2.0 | 20 | 68 | 23 | 45 | — | Fat Clay with Sand (CH) |
| ◐ | EB-6 | 8.5 | 26 | 69 | 50 | 19 | — | Siltstone / Mudstone [QTsc] |
| ■ | EB-7 | 6.0 | 39 | 128 | 42 | 86 | — | Claystone [Qtsc] |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |

CORNERSTONE EARTH GROUP

**Plasticity Index Testing Summary**

**644 & 675 Piercy Road Industrial Fault Investigation**
**San Jose, CA**

Project Number: 1300-2-1

Figure Number: Figure B1

Date: November 2022

Drawn By: FLL

# APPENDIX C: FAULT INVESTIGATION

## WILLIAM GODWIN, P.G., C.E.G., CONSULTANTS REPORT DATED DECEMBER 1, 2022

| | |
|---|---|
| **Type of Services** | **Fault Investigation** |
| **Project Name** | **644/675 Piercy Road Fault Investigation** |
| **Location** | **644/675 Piercy Road**<br>**San Jose, California** |
| **Client** | **HPA, Inc.** |
| **Client Address** | **600 Grand Avenue, Suite 302**<br>**Oakland, California** |
| **Project Number** | **1300-2-1** |
| **Date** | **December 1, 2022** |




**Prepared by** **William H. Godwin, C.E.G.**
Certified Engineering Geologist

**Reviewed by** **Stephen C. Ohlsen, P.E.**
Project Engineer

CORNERSTONE EARTH GROUP

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CORNERSTONE EARTH GROUP

| | |
|---|---|
| **Type of Services** | **Fault Investigation** |
| **Project Name** | **644 & 675 Piercy Road Industrial, Fault Investigation** |
| **Location** | **644 & 675 Piercy Road**<br>**San Jose, California** |

## SECTION 1: INTRODUCTION

This fault investigation was prepared for the sole use of HPA, Inc. for the property located at 644 and 675 Piercy Road in San Jose, California (Figure 1). The purpose of this study was to delineate whether a trace of the Piercy Fault crosses the combined site and to 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 my use, we were provided with the following documents:

- A Topographic Survey of 644 & 675 Piercy Road for Hines, prepared by Kier + Wright and dated May, 2020 at a Scale 1" = 50'.
- A Preliminary Grading Plan of 644 & 675 Piercy Road for Hines, prepared by Kier + Wright and dated May, 2020 at a Scale 1" = 50'.

### 1.1 PROJECT DESCRIPTION

The project site is located on two contiguous parcels at 644 and 675 Piercy Road bounded by Piercy Road to the northeast, Tennant Avenue on the southeast and Hellyer Road on the southwest in San Jose, California (Figure 1). The project site, hereafter referring to both combined parcels currently does not contain any permanent structure but was previously partially developed (644 Piercy) and contains trailers and equipment (675 Piercy). Several abandoned utility lines still cross the site. Of importance is an active 2-inch diameter PG&E gas transmission line that crosses both parcels in a northwest to southeast direction approximately two-thirds of the distance from Piercy Road. The remainder of the site is generally covered by grasses and small native shrubs and various debris piles. The site ranges from approximately Elevation 225 feet at the southwest property line, to Elevation 294 feet at the northeastern corner. The property consists of southwest facing slopes leading down to Hellyer Avenue

We understand that the site is to be developed as a storage facility with interior offices. The main structure of approximately 210,000 square feet will occupy the eastern portion of the site northeast of the gas pipeline with parking and roadways occupying the remained of the site.

## 1.2 SCOPE OF SERVICES

The scope of services for this fault investigation was to address the potential for surface fault rupture from the Piercy Fault and various projections and splays from offsite. In accordance with the California Geologic Survey's Note 49 (CGS, 2002) which provides guidelines for evaluating hazards from surface fault rupture, the plan included review of historic geologic and seismic data, aerial photo interpretation, conducting field geologic mapping, surface geophysical surveys, selected subsurface borings and fault trenching. Cornerstone Earth Group (Cornerstone) assisted in procuring geophysical, drilling, trenching and pedochronology contractors and consultants to perform aspects of this work. Deliverables include this report, which will be included as part of a design-level geotechnical investigation for the 644/675 Piercy Road development.

## 1.3 PREVIOUS STUDIES

The site is located within a geologically complex area within the Edenvale area of the foothills of southeastern San Jose, as shown on Figure 1. The site is located within city-designated geologic hazard zone for fault surface rupture; however, it is just outside the hazard zone for landsliding. A number of regional scale geologic maps and hazard themed maps have been published covering the general area of the site. These published maps were reviewed as part of our Phase 1 work and offer broad interpretations of the geologic setting and include:

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

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

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

The work by Earthtech (2000) includes geologic mapping and perimeter borings for the proposed redevelopment area which we have adopted as a base for our Vicinity Geologic Map (Figure 1) and which provides an overview of the geologic complexity of the vicinity. Of note, landslides are not shown on the map as it focuses on the lower and flatter slopes adjacent to and southwest of Piercy Road. Landslides do occur upslope of Piercy Rd but not in the immediate vicinity.

Our recent geologic site reconnaissance revealed similar geology and earth materials to those shown on Figure 1 although some sites have been developed since 2000. Older ultramafic rocks

of the Franciscan Complex occur upslope of Piercy Road and are predominantly serpentinite. A large quarry was observed close to the site and near the abandoned Evergreen canal. Areas between Piercy Road and Hellyer Avenue contain younger rocks of the Santa Clara Formation (symbol QTsc) both on the site and parcels on either side. Alluvium (Qal) is found on the lower flanks of the slopes and is associated with meandering channel of Coyote Creek. Although not mapped at the scale shown in Figure 3, our reconnaissance did find evidence of fill on the site associated with historic grading of pads for buildings (since removed) near the Piercy Road entrance. Additional fill of unknown thickness may underly the center of the project site as shown by topographic highs. Some debris-laden fill was observed in the northeast portion of the site.

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

Of the various previous geotechnical/geologic investigations performed in the general vicinity of the site, three were particularly relevant to the current investigation and are summarized below:

1) Phase I Preliminary Geologic Consultation, Piercy Road Industrial Development, 644 Piercy Road, San Jose, California, 95132. This letter report, dated April 1, 2021 addressed previous work by E2C in 2000 on the 644 parcel and surrounding parcels. It included a thorough literature review, historic aerial photo interpretation and a site geologic reconnaissance. It summarized the regulatory framework of the Piercy Fault at a City, County and State and concluded that a future fault investigation would be necessary to address potential surface fault rupture and to address fault setbacks provided by E2C in the context of future development.

2) Phase I Preliminary Geologic Consultation, Piercy Road Industrial Development, 550/644 Piercy Road, San Jose, California, 95132. This letter report dated October 23, 2021 covered many of the same scope items as in 1), however it also covered an additional adjacent parcel (550 Piercy Road) to the northwest and included subsurface drilling and surface geophysical (refraction and MASW methods). Of particular emphasis was the fault investigation work by E2C in 2000, and fault setbacks established for the Piercy fault and other fault shears on both parcels. The conclusion was that proposed grading and development of the parcels may require trenching and interpretation of faulting.

3) Reevaluation of Fault Rupture Hazard, 550 Piercy Road, San Jose, California, 95132. This letter report dated September 27, 2022 addressed the northwest portion of the 550 Piercy Road parcel with the intent of reviewing geologic structures in new fault trenches to determine if the fault shear features described in the E2C 2000 report truly represented a fault rupture hazard. The letter report documented additional scope including new subsurface borings, geophysical surveys and two fault trenches excavated in the vicinity of selected E2C trenches. The trenches were field reviewed with a pedochronological study performed by Dr. Glenn Borchardt. The letter report recommended that fault

setbacks established by E2C for most of the site were still valid, with the exception of two areas in the northwestern portion of the site which exposed minor shearing that did not offset Holocene or younger sediments. The shears do not constitute a fault rupture hazard to the proposed development and the corresponding fault setback restriction should be removed.

### 1.4 EXPLORATION PROGRAM

Exploration for this fault investigation consisted of four seismic survey lines, a few borings and three trenches excavated with a rubber-tire 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

#### 2.1.1 Seismic Investigations

Two seismic surveys were conducted by JR Associates that provided subsurface information crucial to the planning and selection of fault trenching. A summary of these two reports is below:

JR Associates, 2021 Seismic Survey for 550 Piercy Road, San Jose, California Consultants Report for Cornerstone. This survey employed seismic refraction and multichannel analysis of surface waves (MASW) methods. One survey line (line S-4, shown on Figure 2) extended from 550 Piercy onto the western portion of the 644 Piercy parcel and to near the Hellyer Avenue property line. A low velocity zone was observed on this line which corresponded to the eastern corner of the proposed building.

JR Associates, 2022 Seismic Survey for Piercy Road, San Jose, California Consultants Report for Cornerstone. While not in the title, this geophysical report was conducted specifically for 675 Piercy as there had previously been no exploration of this parcel other than work by Earthtech (2000) along the Tennant Road as part of the original Edenvale development. Three survey transects (S-1, S-2, S-3) were position in an “H” pattern to convey subsurface velocity profiles. A single low velocity zone was observed at the intersection of S-2 and S-3, and two others were observed in S-1

#### 2.1.2 Subsurface Borings

One boring, EB-1 was drilled in 2021 on the 644 parcel to a depth of 19.3 feet near the boundary of the two parcels, as shown on Figures 2 and 4. The boring, although not near a fault trench was meant to provide subsurface stratigraphy and penetration data for preliminary geotechnical planning purposes. It showed 6 feet of fill over 5 feet of fat clay and 8 feet of clayey sand. An additional six borings have since been drilled on parcels 644 and 675 and that information is provided in the design-level geotechnical investigation.

### 2.1.3 Fault Trenching

A fault trenching program was implemented to excavate three trenches, one on the 675 Piercy parcel and two on the 644 Piercy parcel, as shown on Figure 2. Trench locations were chosen to correspond with low velocity zones observed in the geophysical lines, the footprint of the proposed development buildings and projection of faults from the 550 Piercy parcel to the north and faults from other projects to the east. Two of the trenches (T-1 and T-2) were excavated, cleaned, logged and inspected under the supervision of Cornerstone's Certified Engineering Geologist. A third trench (T-3) encountered a thick fill section and could not safety shored or entered. The trenches were excavated to depths ranging from 8 to 10 feet. 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 walls but followed certain features across to the opposing trench wall where they were exposed further by cleaning.

## 2.2 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 the 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).

Dibblee (1972) and Bryant (1981) had mapped the Piercy Fault and considered it active and as a result it initially was included as a Special Studies Zone that required site-specific investigation as part of the Alquist-Priolo Act. It is no longer included in the State of California Earthquake Fault Zone (1982) however it is still included in the City Special Studies zone (1983). Figure 1 shows fault traces identified in selected fault investigations mentioned above. Most of these have been interpreted as splays from the Piercy Fault that extend towards the project site.

As mentioned above, E2C presumably conducted a fault investigation on the site as a basis for the traces and fault setback zones located on figure 8 of an undated (and unavailable) report.

## 2.3 CURRENT FAULT INVESTIGATION

### 2.3.1 Trenching Observations

Trench 1 was excavated to depths of 8 to 10 feet and oriented in a northeast to southwest direction within parcel 675 Piercy. The location corresponds with to a low velocity zone in Seismic Line S2 as well as the projection of a fault observed in an offsite fault study (ENGEO, 2014). Approximately 100ft in length, Trench 1 followed a very gentle slope to the southwest as shown on Figure 4 and was oriented normal to the trend of the Piercy Fault. The majority of the trench exposed a fat clay (colluvial unit 1) overlying a sandy siltstone (Santa Clara Formation unit 2). The near surface soils consisted of disturbed (tilled) silty clay with sand with some rounded gravels possibly from ancestral deposits from the nearby Coyote Creek. Unit 2 contained locally concentrated calcium

carbonate stringers and veinlets as well as pervasive krotovina (rodent) burrows. Unit 2 also exhibited alternating and moderately dipping beds of more cemented sandstone (Unit 2b) in the northern end of the trench with the same unit occupying what appeared to be scour channels in Unit 2a and the southern end of the trench. Of significance was the absence of shearing or fault in the entire exposed trench walls.

Trench 2 was excavated to depths of 8 to 11 feet and oriented in a northeast to southwest direction within parcel 644. The location was chosen to shadow the northern end of the proposed and potentially intercept the eastern projection of a subparallel Piercy fault splay observed in the adjacent 550 Piercy parcel by E2C. Approximately 105 feet in length, Trench 2 followed a moderately dipping slope to the southwest as shown on Figure 5. The majority of the trench exposed a silty fat clay (colluvial unit 1) overlying a two claystone units (Santa Clara Formation unit 2a and 3). Unit 1a constitutes a thin top soil with transitional contact with the underlying fat clay (Unit 1b) which was very consistent in texture and lithology with the exception of subunit Unit 1c, a localized pocket of clayey sandy gravel found between stations 60 and 70. One unique feature in Unit 2a, a moderately strong, intensely fractured claystone was the localized occurrence of micro-shear structures between stations 12 and 32. The structures were discontinuous, random in orientation and were associated with calcium carbonate stringers and nodules. One prominent shear, striking N58oNW and dipping 55oE was observed at station 47. This shear did not project up into unit 1b, the perceived A/B horizon. Krotovina (rodent) burrows were found throughout Unit 1b. A basal claystone (Unit 3) was observed that was in transitional contact with Unit 2a but was noticeably weaker and devoid of shearing.

Trench 3 was excavated to depths of 7 to 10+ feet and oriented in a northeast to southwest direction, in line with seismic line S-4. Trench 3 revealed upwards of 8.5 feet of fill over fat clay. The fill thickness precluded extending the trench depth another 10 feet to expose native soils and the Santa Clara Formation bedrock. The quality of the fill also posed a risk of caving so the trench was not shored, cleaned and logged. Instead trench spoil was logged prior to backfilling. The trench was positioned to look at low velocity interval seen in the survey line S-4 (JRA, 2021).

### 2.3.2 Pedochronological Observations

Soil Tectonics, Inc. conducted a pedochronological evaluation as detailed in Appendix E. Dr. Glenn Borchardt analyzed a profile in Trench 2, representative of the entire project site. The objective of this study was to characterize observed soil development and age to support interpretations of potential fault rupture with respect to displacement of Holocene-age soil horizon. Below are excerpts from his analysis.

> A soil profile was studied to assess the age of the soils in Trenches T-1 and T-2 excavated southwest of the Piercy fault southwest of Piercy Road in San Jose, California (Table 1). Soil Profile No. 1 had A and AB horizons overlying Bkt and 2BCrkt and 2Crk horizons. The three upper horizons were derived primarily from colluvium, while the two lower, extremely calcitic horizons were derived from the Santa Clara Formation.
>
> The thickness of this soil (135-170 cm) and the clay film development in the Bkt horizon is typical of Pleistocene soils. The salt accumulation at its base also attests to its great

age. The calcite in this soil appears to be entirely derived from the underlying Santa Clara Formation. The nodules in the soil appear to have been brought up toward the surface during shrink-swell common in Vertisols like this one. They also appear to be disintegrating and are amenable to deposition by illuvial clay films.

1. The clayey soils in the colluvium at in Trench T-2 are all older than 11,700 years, and likely developed during the Late Pleistocene over 80,000 years ago.
2. Although somewhat thicker, the soils in Trench T-1 appear to be the same age.
3. There was no evidence of surface fault rupture in Trenches T-1 and T-2.
4. The few shears in the underlying Santa Clara Formation appear ancient and intraformational.

### 2.2 MITIGATING FAULT SURFACE RUPTURE

A final development concept for the site is shown on Figure 2. 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 either side of the surface trace) was previously recommended for the subject site by E2C (2000a, 2000b), although the report itself is not available to determine specific details of their analysis. 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.

## 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, site-specific fault studies by E2C on parcel 644 Piercy have resulted in recommended building exclusion zones due to a potential for surface fault rupture. Despite the absence of trench logs and the E2C fault investigation report, it is likely that there is evidence that surface traces of the Piercy Fault project through the western portion site. However site-specific trenching in the footprint of the proposed structure as part of this investigation does not suggest that faulting of near surface Holocene-age soils is present. Localized shearing was observed but it is contained within Late Pleistocene soils as supported by a pedochronologic soils analysis.

Our recommended building setback lines that are depicted in Figure 8, extend along the boundaries of shorter fault segments found northeast of the building and represent the bounding

limits of fault zone and apply to any future habitable structures at the site. These setbacks are inline with those proposed by and reflected on the ALTA survey drawings and are equal to 25 feet along either side of the mapped fault zone surface trace. We recommend that previous projection of fault traces from the 550 Piercy investigation depicted on Figure 2 be removed and that setbacks associated with it be removed.

## SECTION 4: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of HPA, Inc. specifically to support the design of 644/675 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.

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

Cornerstone prepared this report with the understanding that it is the responsibility of the owner or their 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

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## Aerial Photographs Reviewed

**Table 1 - List of Reviewed Aerial Photos**

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

Notes: PAS – Pacific Aerial Surveys

# APPENDIX D: GEOPHYSICAL FAULT INVESTIGATION

## JR ASSOCIATES REPORT DATED OCTOBER 11, 2021

J R ASSOCIATES

Engineering Geophysics
17040 Oak Leaf Drive
Morgan Hill, CA 95037
(408) 293-7390

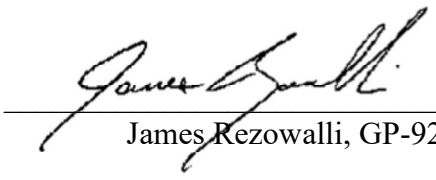
# SEISMIC SURVEY FOR [redacted]
# SAN JOSE, CALIFORNIA

October 11, 2021

for

Cornerstone Earth Group
1259 Oakmead Parkway
Sunnyvale, CA 94085

by



James Rezowalli, GP-921

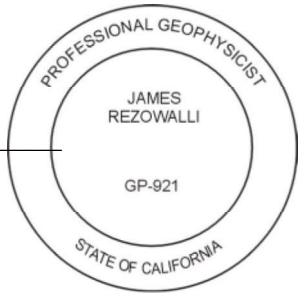


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Drawing 2 Fault Traces Mapped by Others

Drawing 3 Site Map

Drawing 4 Piercy Road Seismic Profiles

Drawing 5 Piercy Road Seismic Profiling Results

# I INTRODUCTION

This report presents the results of a seismic investigation performed at Piercy Road in San Jose, California (Drawing 1). The investigation was performed for Cornerstone Earth Group by J R Associates. The purpose of the investigation was to measure the depth to bedrock if present and to look for geophysical evidence indicative of faulting beneath the property. James Rezowalli, Principal Geophysicist, and Brian Rezowalli, Field Manager, of J R Associates performed the field work in October of 2021.

## A. Site Conditions

The site consists of two vacant lots at [redacted]644 Piercy Road. The sites were grass covered and gently sloping to the southwest. There was an area of fill material near the middle of 644 Piercy Road. A gas pipeline crosses the middle of both sites from the southeast to the northwest. Cornerstone Earth Group provided us information from previous fault investigations performed by others that indicated the site had faults. Drawing 2 shows the approximate locations of the alleged fault traces. The purpose of this investigation was to look for geophysical evidence of potential faults in the vicinity of the alleged traces noted in the previous investigations.

## II METHODOLOGY

We used two different seismic techniques to measure the depth to bedrock and to look for geophysical evidence of faulting, seismic refraction and shear wave profiling. Seismic refraction uses compressional (P-) waves that can refract off the top of the bedrock. We used the refraction technique to look for shallow bedrock that could be difficult to rip. Also faults can offset the top of bedrock and an abrupt change in bedrock elevations can be an indication of faulting. Because the top of the bedrock could be at about the same elevation on both sides of a fault we performed shear (S-) wave profiling. Shear wave profiles show changes in S-wave velocity with depth. We used it to look for changes in shear velocity that may either be caused by different S-wave velocities in the materials on either side of the fault or a low S-wave velocity zone caused by shearing and gouge within the fault zone.

### A. Field Procedures

Refraction data were collected along two seismic lines, one crossing each site from north to south and crossing the alleged fault traces (Drawing 3) The refraction survey contained geophones on ten-foot centers and multiple shot points. The shot points were at the beginning, the end, and every 125 feet along the line. A twelve-pound sledge hammer striking an aluminum plate was used to create P-waves at the shot point locations.

An array consisting of a shot point placed 30 feet away from a string of 24 geophones was used to collect data for the shear wave profiling. The geophones were spaced 3 feet apart and a twelve-pound sledge hammer was used to generate the surface wave. . Once a measurement was collected the array was moved 12 feet along the line and the process repeated. A measurement of

shear wave velocity with depth was calculated using the multichannel analysis of surface waves (MASW) technique developed by the University of Kansas at Lawrence. S-wave versus depth measurements were collected at twelve-foot intervals along the two seismic lines. The S-wave velocity versus depth measurements were concatenated together to create a two dimensional S-wave profile.

The depth of investigation for both the refraction and S-wave profiling was approximately 40 feet below the ground surface.

B. Data Reduction

Refraction data reduction began by picking the arrival times from the seismograph recordings. An arrival time is the time a P-wave spent traveling from shot point to geophone. The wave could either travel along the ground surface or be refracted from an interface between materials. For a refraction to occur, the materials below the interface must have a greater P-wave velocity than the materials above the interface. The arrival times were entered into a computer program with elevation, location, and layer control information. The absolute elevation at the start of the lines was arbitrarily set at [redacted] 200 feet for line 2. Elevation changes along the lines were obtained using a hand level. The information was entered into software provided by Geometrics Incorporated to calculate the depth and velocities of the seismic layers beneath the lines.

The program Surfseis developed by the Kansas Geological Survey was used to process the seismic records into S-wave profiles. From each seismic recording a fundamental-mode dispersion curve was extracted. The dispersion curve is related to the shear wave velocities of the different wave lengths contained in the surface wave. Longer wave lengths are related to the S-wave velocity of deeper soils and shorter wave lengths are related to the S-wave velocities of

near surface soils. The dispersion curves are inverted into a series of one-dimensional S-wave velocity profiles. More information of the MASW can be found at the Kansas Geological Survey's web site at www.kgs.edu/software/surfseis/.

## III RESULTS

A. Refraction Results

The results of the computer analysis of the refraction data are presented in Drawing 3. The refraction data indicated the upper 40 feet of soil consist of a relatively thin layer of younger less dense soils overlying older denser soils. The surface layer was [redacted] between a few feet to 18 feet beneath line 2. The compressional wave velocities ranged from 800 feet per second (fps) near the surface to 4800 fps for the deepest layers. The relatively low compressional wave velocities suggest that competent bedrock is deeper than approximately 40 feet below the ground surface.

B. Shear Wave Results

The results of the MASW shear wave profiling are presented in Drawing 3. The shear wave velocity ranged from 800 fps to 1800 fps and in general increased with depth. The range is typical of soil materials and also indicates that competent bedrock is below 40 feet beneath the ground surface. [redacted]

Line 2 had two zones with lower S-wave velocities (Drawing 3). The first was between 610 and 790 feet from the start of the line. This is in the area of the fill material which is a likely cause of the low S-wave velocities. The second zone was a thin zone approximately 918 feet

from the start of the line. The S-wave velocities in this zone are slightly lower than the adjacent soils but this is an area were previous investigations alleged one of the fault traces.

C. Summary

The refraction data indicated the materials at the site consist of a thin layer of less dense, looser, younger soils overlying denser older material. There were no indications of competent bedrock in the upper 40 feet of soil. The shear wave data indicated two areas with low shear wave velocities that we suggest be investigated further. [redacted] The second zone was approximately 918 feet from the start of Line 2. This was a narrow zone of slightly lower S-wave velocities that coincided with one of the alleged fault traces noted by previous investigations. Drawing 5 shows the location of the two low S-wave velocity zones. There was one other low S-wave velocity zone between 610 and 790 feet from the start of Line 2. This zone corresponded to an area of fill material. Drawing 5 shows the locations of the low S-wave velocity zones.

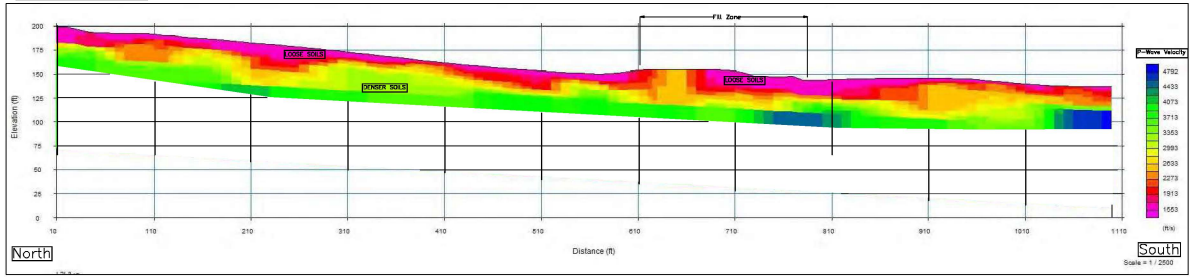
D. Limitations

Seismic layers do not always correspond directly to lithologic changes that might be found in borehole or trenching data. A seismic layer is an interface between materials with different P- or S-wave velocities. Factors such as weathering, cementation, induration, and saturation as well as lithologic changes can create changes in seismic velocities. Other geologic structures, such as buried stream channels, can cause anomalous or low velocity zones in seismic data. There can be lithologic changes without velocity changes. However, our field experience indicates that seismic layers often correspond to major changes in lithology or saturation to within ±20% of the depth to the interface. In order to detect a fault there must be a change in the physical rock properties

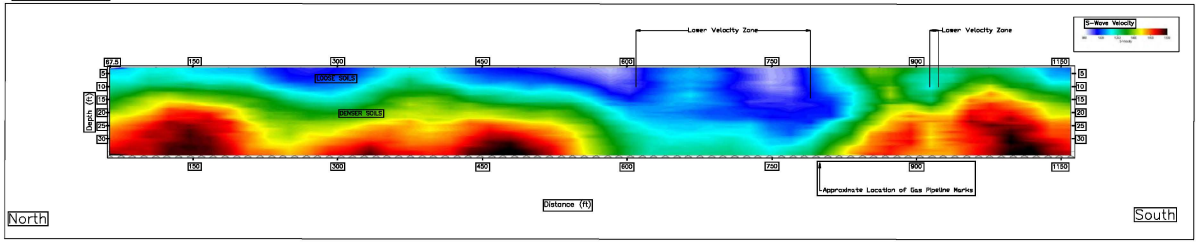
across the fault. If there is no change in physical properties across a fault then it will not be detected. Our data should be compared with available geologic and other data before conclusions are drawn.

# IV DRAWINGS

Refraction Line 2




MASW Line 2

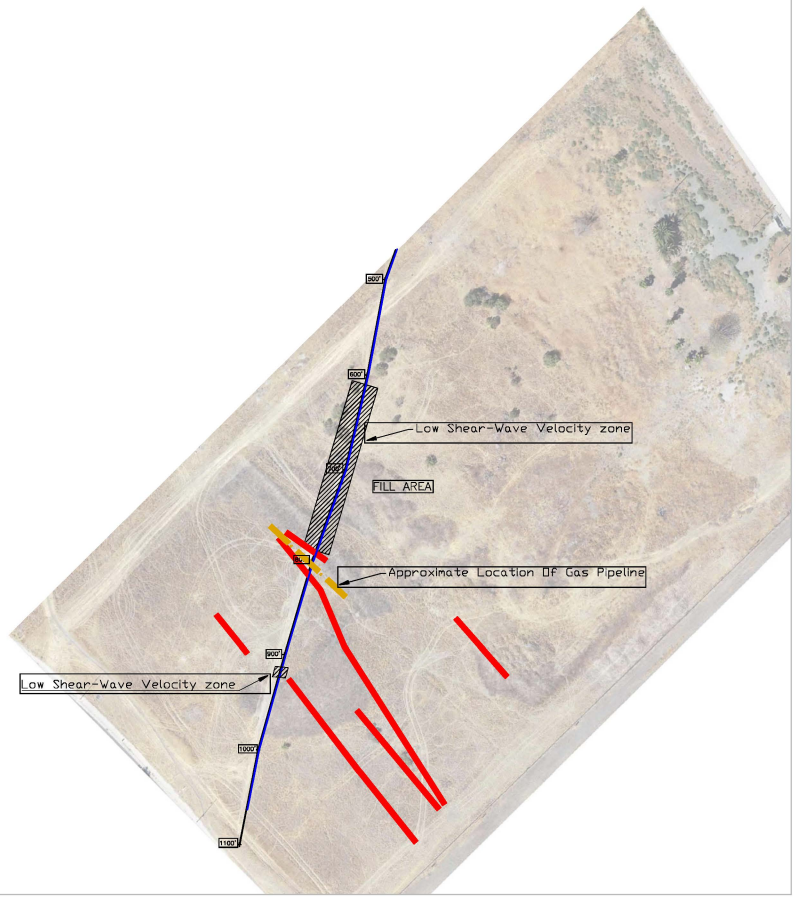



3LHUFiSRDG8HLVPLF3URILOHV
550 Piercy Road
San Jose, California

| | | | |
|---|---|---|---|
| SCALE: See Diagrams | | | DRAWN BY: J.J.R. |
| DATE: 10-07-2121 | JOB NUMBER: 139-010-21 | | REVISED |

-5$VVRFLDWHV, Civil and Environmental Geophysics
17040 Oak Leaf Drive, Morgan Hill, CA (408) 293-7390

DRAWING NUMBER: 4 of 5




3LHUFi5RDG8HLVPLF3URILOLQJ5HVXOWV
550 Piercy Road
San Jose, California

SCALE: 1" = 80'
DRAWN BY: J.J.R.
DATE: 10-07-2121
JOB NUMBER: 139-010-21
REVISED

-5$VVRFLDWHV Soil and Environmental Geophysics
17040 Oak Leaf Drive, Morgan Hill, CA (408) 293-7390

DRAWING NUMBER: 5 of 5

J R ASSOCIATES

Engineering Geophysics
17040 Oak Leaf Drive
Morgan Hill, CA 95037
(408) 293-7390

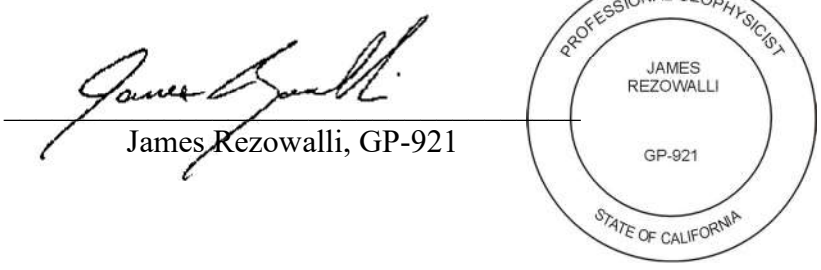
# SEISMIC SURVEY FOR PIERCY ROAD SAN JOSE, CALIFORNIA

March 9, 2022

for

Cornerstone Earth Group
1259 Oakmead Parkway
Sunnyvale, CA 94085

by




James Rezowalli, GP-921

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LIST OF ILLUSTRATIONS

Drawing 1 Vicinity Map

Drawing 2 Site Map

Drawing 3 Piercy Road Seismic Profiles

Drawing 4 Piercy Road Seismic Profiling Results

# I INTRODUCTION

This report presents the results of a seismic investigation performed at a site off Piercy Road in San Jose, California (Drawing 1). The investigation was performed for Cornerstone Earth Group by J R Associates. The purpose of the investigation was to measure the depth to bedrock if present and to look for geophysical evidence indicative of faulting beneath the property. James Rezowalli, Principal Geophysicist, and Brian Rezowalli, Field Manager, of J R Associates performed the field work in March of 2022.

## A. Site Conditions

The site was a lot adjacent to 644 Piercy Road (Drawing 1). The site was grass covered and gently sloping to the southwest. There were trailers and construction equipment in the southeast corner of the site. A gas pipeline crosses the middle of the site from the southeast to the northwest. Cornerstone Earth Group provided us logs from three exploratory borings drilled recently near the site. They also provided several boring logs from previous investigations near the site. The logs indicate the materials at the site consist of a thin layer of mixed sands and clays overlying denser poorly indurated sands and clays of the Santa Clara Formation. Nearby borings offsite indicated Franciscan bedrock underlies the Santa Clara formation.

Cornerstone Earth Group provided us drawings showing the results from several previously performed geotechnical investigations. The available drawings showed trench and possible fault

locations. Drawing 2 shows the approximate traces noted in the reports. The purpose of our study was to look for geophysical evidence of potential fault traces at the current site.

## II METHODOLOGY

We used two different seismic techniques to measure the depth to bedrock and to look for geophysical evidence of faulting, seismic refraction and shear wave profiling. Seismic refraction uses compressional (P-) waves that can refract off the top of the bedrock or off the water table. Faults can offset bedrock or the top of the water table and an abrupt change in bedrock elevation can be an indication of faulting. The refraction technique is also used to look for shallow bedrock that could be difficult to rip. Because the top of the bedrock could be at about the same elevation on both sides of a fault we performed shear (S-) wave profiling. Shear wave profiles show changes in S-wave velocity with depth. We used it to look for changes in shear velocity that may either be caused by different S-wave velocities in the materials on either side of the fault or a low S-wave velocity zone caused by shearing and gouge within the fault zone.

### A. Field Procedures

Refraction data were collected along three seismic lines, two crossing the site from north to south and crossing projections of the alleged fault traces (Drawing 2). The third line ran from east to west near the middle of the site. The refraction survey contained geophones on ten-foot centers and multiple shot points. The shot points were at the beginning, the end, and every 125 feet along the line. A twelve-pound sledge hammer striking an aluminum plate was used to create P-waves at the shot point locations. Data were recorded with a Geometrics 24-channel Geode seismograph.

An array consisting of a shot point placed 30 feet away from a string of 24 geophones was used to collect data for the shear wave profiling. The geophones were spaced 3 feet apart and a

twelve-pound sledge hammer was used to generate the surface wave. Once a measurement was collected the array was moved 12 feet along the line and the process repeated. A measurement of shear wave velocity with depth was calculated using the multichannel analysis of surface waves (MASW) technique developed by the University of Kansas at Lawrence. S-wave versus depth measurements were collected at twelve-foot intervals along the three seismic lines. The S-wave velocity versus depth measurements were concatenated together to create a two dimensional S-wave profile.

The depth of investigation for both the refraction and S-wave profiling was approximately 40 feet below the ground surface.

B. Data Reduction

The program Plotrefra developed by Geometrics Incorporated was used to process the refraction data. Refraction data reduction began by picking the arrival times from the seismograph recordings. An arrival time is the time a P-wave spent traveling from shot point to geophone. The wave could either travel along the ground surface or be refracted from an interface between materials. For a refraction to occur, the materials below the interface must have a greater P-wave velocity than the materials above the interface. The arrival times were entered into a computer program with elevation, location, and layer control information. The absolute elevations at the start of the lines was obtained from Google maps and are approximate. Elevation changes along the lines were obtained using a hand level. The information was entered into software provided by Geometrics Incorporated to calculate the depth and velocities of the seismic layers beneath the lines.

The program Surfseis developed by the Kansas Geological Survey was used to process the seismic records into S-wave profiles. From each seismic recording a fundamental-mode

dispersion curve was extracted. The dispersion curve is related to the shear wave velocities of the different wave lengths contained in the surface wave. Longer wave lengths are related to the S-wave velocity of deeper soils and shorter wave lengths are related to the S-wave velocities of near surface soils. The dispersion curves are inverted into a series of one-dimensional S-wave velocity profiles. More information of the MASW can be found at the Kansas Geological Survey's web site at www.kgs.edu/software/surfseis/.

## III RESULTS

### A. Refraction Results

The results of the computer analysis of the refraction data are presented in Drawing 3. The refraction data indicated the upper 40 feet of soil consist of a relatively thin layer of younger less dense surficial soils overlying older denser Santa Clara formation. The surface layer ranged from 3 feet to 15 feet deep beneath the lines. The compressional wave velocities ranged from 1000 feet per second (fps) near the surface to 5000 fps for the second layer. The relatively low compressional wave velocities suggest that competent Franciscan bedrock is deeper than approximately 40 feet below the ground surface. P-wave velocities around 5000 fps often indicate the top of a water table.

At approximately 200 feet and 345 feet along refraction Line 1 there is a thickening of the transitional velocities between the near surface soils and the deeper layer. Also there are offsets in the refracting horizon at these two locations. This area roughly coincides with the southernmost suspected fault traces noted in the earlier geotechnical reports for the adjoining property (Drawing 4).

### B. Shear Wave Results

The results of the MASW shear wave profiling are presented in Drawing 3. The shear wave velocity ranged from 800 fps to 2200 fps and in general increased with depth. The range is typical of poorly indurated materials like the Santa Clara formation sediments and also indicates that Franciscan bedrock is below 40 feet beneath the ground surface. At approximately 192 feet and at 342 feet along line 1 there are zones with low S-wave velocities that could be due to

shearing in a fault zone. The low S-wave velocity zones roughly coincide with the southernmost suspected fault traces noted in the earlier reports regarding the adjoining property (Drawing 4).

At approximately 295 feet along Line 2 there was a zone with low S-wave velocities (Drawing 3). The S-wave velocities in this zone are slightly lower than the adjacent soils and the zone roughly coincides with a fault trace mapped to the east of the site (Drawing 4).

There was nothing significant seen beneath Line 3.

C. Summary

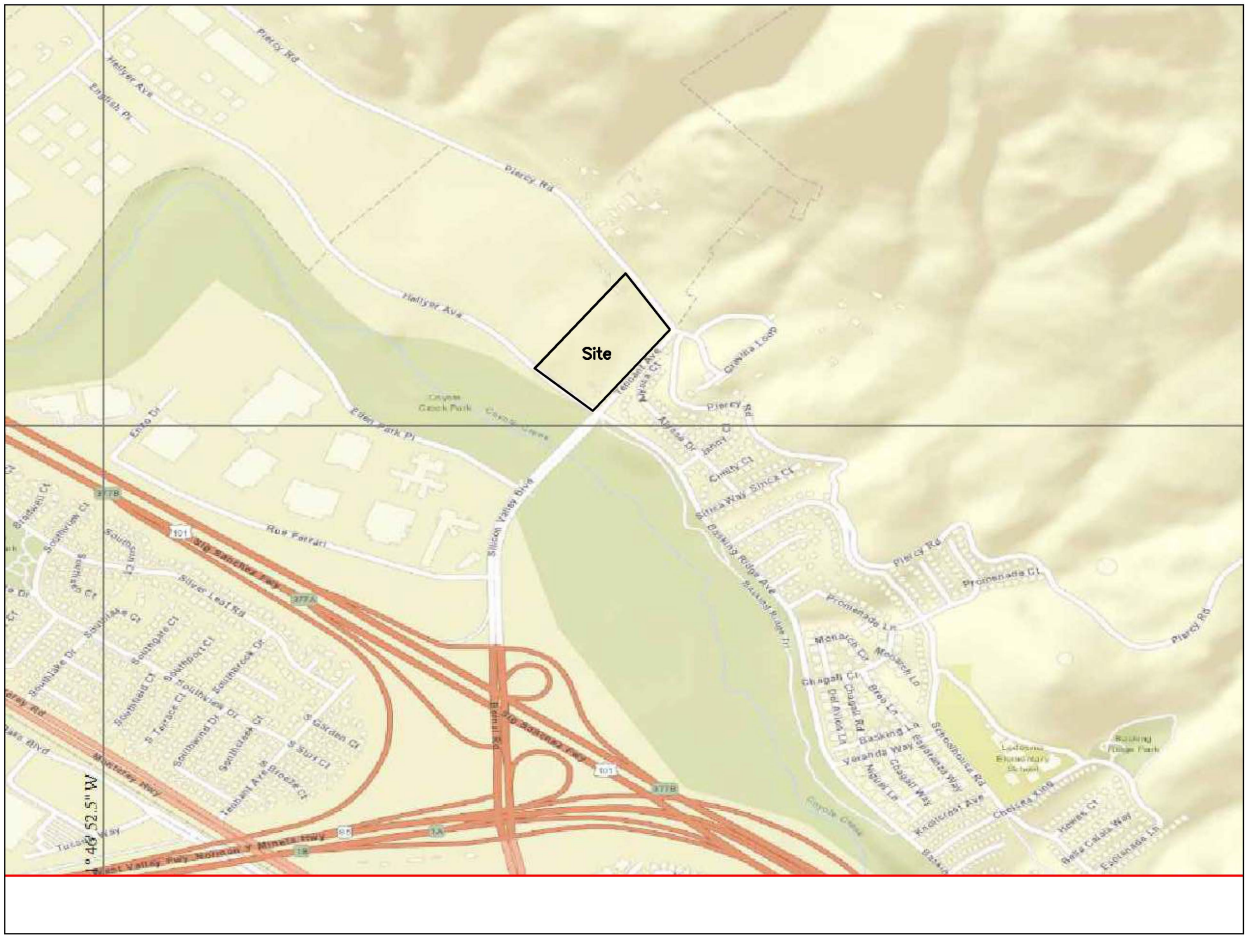
The refraction data indicated the materials at the site consist of a thin layer of less dense, looser, younger soils overlying denser Santa Clara formation sediments. There were no indications of competent Franciscan bedrock in the upper 40 feet of soil. The shear wave data indicated two areas along Line 1 with low shear wave velocities. These two areas coincided with a thickening of the near surface soils and offsets in the refracting horizon beneath Line 1 (Drawing 3). These zones roughly correlate to the projection of the southernmost fault traces noted in the previous investigations (Drawing 4). There was a low S-wave velocity zone approximately 295 feet from the start of Line 2. This was a narrow zone of slightly lower S-wave velocities that coincided with the alleged fault trace noted to the east of the site by previous investigations (Drawing 4). The three low velocity zones should be compared with available geologic and other data before conclusions regarding faulting are made.

D. Limitations

Seismic layers do not always correspond directly to lithologic changes that might be found in borehole or trenching data. A seismic layer is an interface between materials with different P- or

S-wave velocities. Factors such as weathering, cementation, induration, and saturation as well as lithologic changes can create changes in seismic velocities. Other geologic structures, such as buried stream channels, can cause anomalous or low velocity zones in seismic data. There can be lithologic changes without velocity changes. However, our field experience indicates that seismic layers often correspond to major changes in lithology or saturation to within ±20% of the depth to the interface. In order to detect a fault there must be a change in the physical rock properties across the fault. If there is no change in physical properties across a fault then it will not be detected. The three low velocity zones found in our data should be compared with available geologic and other data before conclusions are drawn.

# IV DRAWINGS





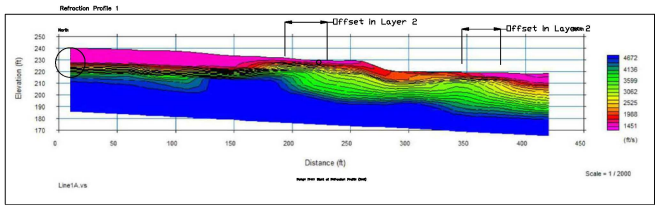


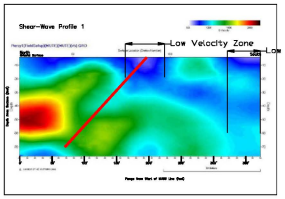
**Vicinity Map**
Piercy Road
San Jose, California

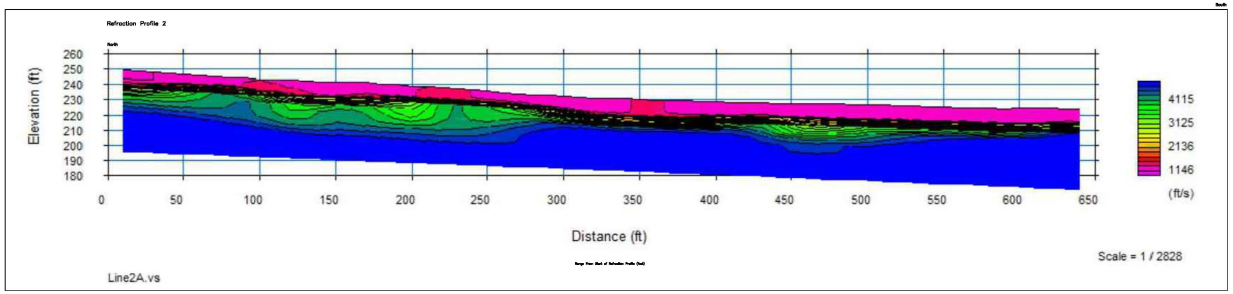
| | | | |
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| DATE: 2-07-2022 | JOB NUMBER: | 112-010-21 | REVISED: |

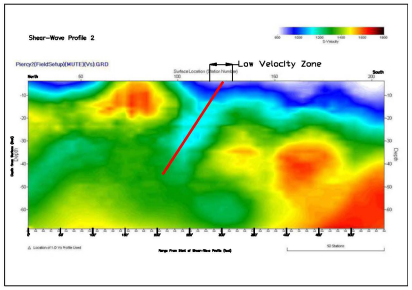
J R Associates Civil and Environmental Geophysics
17040 Oak Leaf Drive Morgan Hill, CA (408) 293-7390

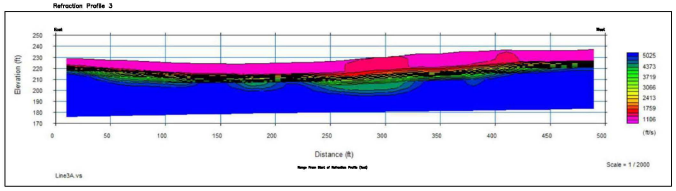
DRAWING NUMBER: 1 of 4

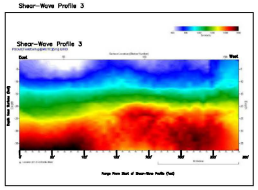


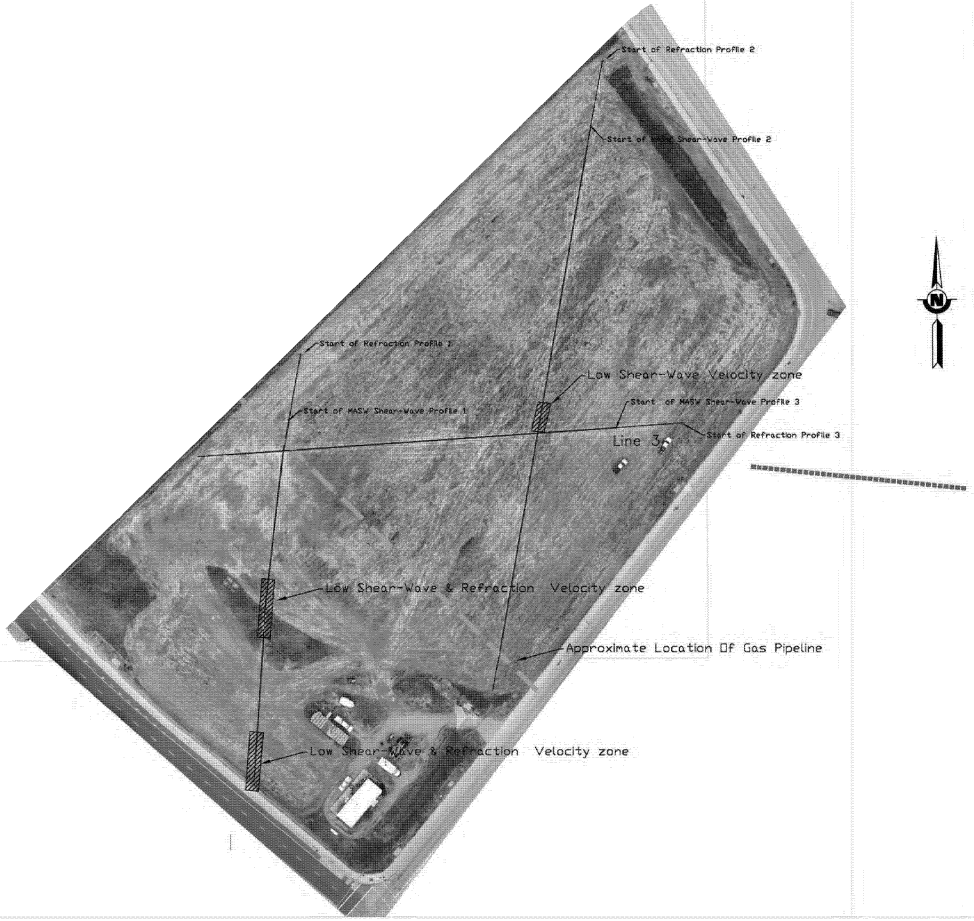




**Piercy Road Seismic Profiles**
665 Piercy Road
San Jose, California

SCALE: See Diagrams | DRAWN BY: J.J.R.
DATE: 3-07-2022 | JOB NUMBER: 112-010-22 | REVISED

J R Associates Civil and Environmental Geophysics
17040 Oak Leaf Drive, Morgan Hill, CA (408) 293-7390




EXPLANATION:

- Approximate Locations of Fault Traces Mapped By Others
- Refraction Line Profile
- Shear-Wave Profile
- Approximate Location Of Gas Pipeline Markings
- Zone With Low Shear-Wave Velocities

**Piercy Road Seismic Profiling Results**
665 Piercy Road
San Jose, California

| | | | |
|---|---|---|---|
| SCALE: 1" = 60' | | | DRAWN BY: J.J.R. |
| DATE: 3-07-2022 | JOB NUMBER: 112-010-22 | | REVISED: |

J R Associates Civil and Environmental Geophysics
17040 Oak Leaf Drive, Morgan Hill, CA (408) 293-7390

DRAWING NUMBER: 4 of 4

# APPENDIX E: PEDOCHRONOLOGICAL REPORT FOR 644 AND 675 PIERCY ROAD (GLENN BORCHARDT 2022)

## GLENN BORCHARDT PEDOCHRONOLOGICAL REPORT DATED MAY 12, 2022

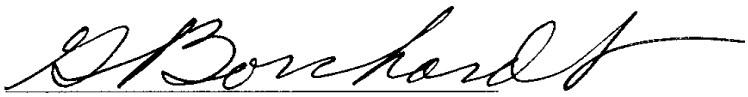
APPENDIX E

# PEDOCHRONOLOGICAL REPORT FOR 644 AND 675 PIERCY ROAD, SAN JOSE, CALIFORNIA

Prepared for Cornerstone Earth Group, Inc., Sunnyvale, California, Project 1300-2-1

May 12, 2022

Soil Tectonics
P.O. Box 5335
Berkeley, CA 94705



Glenn Borchardt

Principal Soil Scientist
Certified Professional Soil Scientist No. 24836

# PEDOCHRONOLOGICAL REPORT FOR 644 AND 675 PIERCY ROAD, SAN JOSE, CALIFORNIA

Prepared for Cornerstone Earth Group, Inc., Sunnyvale, California, Project 1300-2-1

May 12, 2022

Glenn Borchardt

## INTRODUCTION

An assessment of seismic and landslide risk due to ground movement can be aided greatly by the techniques of pedochronology (Borchardt, 1992, 1998a), soil dating. This is because the youngest geological unit overlying fault traces and headwall grabens is generally a soil horizon. The age and relative activity of ground movement often can be estimated by evaluating the age and relative disturbance of overlying soil units, as well as buried soils called paleosols. Terms, prefixes, and suffixes are defined in the Soils Glossary at the end of this report.

Soil horizons exhibit a wide range of physical, chemical, and mineralogical properties that evolve at varying rates. Soil scientists use various terms to describe these properties. A black, highly organic "A" horizon, for example, may form within a few centuries, while a dark brown, clayey "Bt" horizon may take up to 40 kyr (40,000 years) to form. Certain soil properties are invariably absent in young soils. For instance, soils developed in granitic alluvium of the San

Joaquin Valley do not have Munsell hues redder than 10YR unless they have formed after 100 ka (100,000 years ago) (Birkeland, 1999; Harden, 1982). Still other properties, such as the movement and deposition of clay-size particles and the precipitation of calcium carbonate at extraordinary depths, indicate soil formation during a climate much wetter than at present. In the absence of a radiometric age date for the material from which a particular soil formed, an estimate of its age must take into account all the known properties of the soil and the landscape and climate in which it evolved.

# METHOD

The first step in studying a soil is the compilation of the data necessary for describing it (Birkeland, 1999; Borchardt, 2020). At minimum, this requires a Munsell colorimeter or color chart, hand lens, acid bottle, and instruments for 1:1 soil:water pH and conductivity measurements. The second step may involve collecting samples of each horizon of the soil profile column for laboratory analysis of particle size. This is done to check the textural classifications made in the field and to evaluate the genetic relationships between horizons and between different soils in the landscape. When warranted, the clay mineralogy and chemistry of the soil also is analyzed to provide additional information on the changes undergone by the initial material from which the soil weathered. The last step is the comparison of this accumulated soil data with that for soils having developed under similar conditions, preferably in the same region. Such information is scattered in soil survey reports (e.g., Welch, 1981), soil science journals, and consulting reports. In a particular locality, there is seldom enough comparative data available for this purpose. That is why, at the very least, the study of one soil profile always makes the evaluation of the next that much easier.

# RESULTS OF THIS EVALUATION

A soil profile was studied to assess the age of the soils in Trenches T-1 and T-2 excavated southwest of the Piercy fault southwest of Piercy Road in San Jose, California (Table 1). Soil Profile No. 1 had A and AB horizons overlying Bkt and 2BCrkt and 2Crk horizons. The three upper horizons were derived primarily from colluvium, while the two lower, extremely calcitic horizons were derived from the Santa Clara Formation.

# *Soil Profile No. 1*

This profile was described at station 73' in Trench T-2 across an area sloping about 12% SW (Table 1). All horizons in the solum were relatively moist with the upper three horizons being 135-cm thick very dark gray clay and containing a few angular clasts indicating colluviation (Figures 1 and 2). Detrital calcite nodules exist in the AB and Bsskt horizons, with concentrations increasing with depth (Figures 2 and 3). The Bsskt horizon has strong angular blocky structure and slickensides indicating seasonal shrink-swell (Figure 3). The upper portion of the underlying 2BCrkt horizon is dark gray, while the lower portion is yellowish, reflecting clay film deposits lacking organic matter (Figure 4). The underlying 2Crk horizon contains predominantly calcite intermixed with clay presumably of the Santa Clara Formation.

The detrital calcite nodules appear to have been brought up into the colluvium via the self-plowing action of the Vertisol. Rare nodules in the AB horizon appear to be disintegrating and amenable to clay film penetration (Figure 5). Those in the Bsskt horizon have patchy thin to medium thick clay films attesting to the illuvial nature of that horizon (Figure 6). A closeup of the 2BCrkt horizon shows the penetration of the B-horizon material into the top of the Santa Clara Formation (Figure 7).

# *Soil pH and Conductivity*

Soil pH measurements are part of a proper soil description. They help to establish that the material observed actually underwent pedogenesis. Unweathered bedrock and sediments normally have a pH generally around neutral with little change with depth. This soil is mildly alkaline in the surface, becoming strongly alkaline in the 2Crk horizon (Table 1 and Figure 8).

As expected for these clayey materials, salts dissolved from parent rocks trickle down through the soil, concentrating at its base (Borchardt, 2016). Soil electrical conductivity in this soil doubles as a result of this process (Figure 9). In fine soils like this, salts get trapped, producing conductivity peaks relatively commensurate with the duration of exposure to soil formation.

# *Soil Ages*

The thickness of this soil (135-170 cm) and the clay film development in the Bkt horizon is typical of Pleistocene soils. The salt accumulation at its base also attests to its great age. The

calcite in this soil appears to be entirely derived from the underlying Santa Clara Formation. The nodules in the soil appear to have been brought up toward the surface during shrink-swell common in Vertisols like this one. They also appear to be disintegrating and are amenable to deposition by illuvial clay films. That is why the horizon designation for the Bkt has the “k” preceding the “t,” implying the calcite existed before the translocated clay and not the other way around. Note also that soils actively forming pedogenic calcite tend to produce thin filaments, followed by soft nodules at ped intersections (Borchardt and Lienkaemper, 1999). The “k” designation in the profile description is therefore a bit of a misnomer as the calcite thus indicated is not pedogenic.

It is likely that the erosive episode that produced the colluvium at the site occurred during the Pleistocene glacial period when sea level was as much as 126 m lower than at present. Base level thus would have much lower than at present and precipitation would have been two to three times the 15 in/yr that occurs at present. Although the colluvial deposition appears to have been relatively gentle across the Santa Clara Formation there are a few places where some erosion has occurred along the contact. Serpentinite clasts right at the contact must have eroded from the outcrop to the northeast of the Piercy fault, which is mapped northeast of Piercy Road.

# CONCLUSIONS

1. The clayey soils in the colluvium at in Trench T-2 are all older than 11,700 years, and likely developed during the Late Pleistocene over 80,000 years ago.
2. Although somewhat thicker, the soils in Trench T-1 appear to be the same age.
3. There was no evidence of surface fault rupture in Trenches T-1 and T-2.
4. The few shears in the underlying Santa Clara Formation appear ancient and intraformational.

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Table 1. Soil profile described on lot 644 southwest of the Piercy fault southwest of Piercy Road, San Jose, California. Abbreviations and definitions are given in Schoeneberger and others (2012), Soil Science Division Staff (2017), and Soil Survey Staff (1999, 2010).

Description of soil developed southwest of the mapped trace of the Piercy fault by Glenn Borchardt, who measured and sampled the soil on April 25, 2022 in the southeast wall at station 73 in Trench T-2 at 37.2533°, -121.7710°. Elevation 257' GEP.** Mediterranean climate with mean annual precipitation of 15.09"/yr (383 mm/yr) at San Jose. Natural slope 12%. Aspect SW. Excellent drainage. Water table deep. Vegetation grass and weeds. The parent material is clayey colluvium overlying caliche presumably of the Santa Clara Formation. Moist throughout. Soil pH is mildly alkaline in the surface, becoming strongly alkaline in the 2Crk horizon. Soil in the area is mapped as Altamont clay, *Aridic Haploxererts.*

| Horizon | Depth, cm | Description |
|---|---|---|
| A | 0-32 | Very dark gray (10YR3/1m, 4/1d) clay; coarse strong granular to angular blocky structure; very sticky and very plastic when wet, firm when moist, and very hard when dry; many very fine roots; many very fine to medium continuous random tubular pores; very slight effervesce in matrix; clear smooth boundary; pH 7.86; conductivity 385 uS; Sample No. 22B001. |
| AB | 32-86 | Very dark gray (10YR3/1m, 5/1d) clay; coarse angular blocky structure; very sticky and very plastic when wet, firm when moist, and very hard when dry; common very fine roots; common very fine to medium continuous random tubular pores; very slight effervesce in matrix violent effervescence in rare distinct <1 cm white calcitic mottles; gradual smooth boundary; pH 8.09; conductivity 425 uS; Sample No. 22B002. |
| Bsskt | 86-135 | Very dark gray (10YR3/1m, 5/1d) clay; coarse granular to angular blocky structure; very sticky and very plastic when wet, firm when moist, and extremely hard when dry; few very fine roots; few very fine to medium continuous random tubular pores; very slight effervesce in matrix and extremely violent effervesce in rare distinct <1 cm angular white mottles due to calcite; few thin patchy clay films on calcite; clear wavy boundary; pH 8.75; conductivity 579 uS; Sample No. 22B003. |
| 2BCrkt | 135-170 | Dark gray (10YR4/1m, 6/1d) clay with many fine to coarse distinct |

white mottles due to calcite; fine to coarse granular structure; very sticky and very plastic when wet, firm to friable when moist, and extremely hard when dry; few very fine to medium continuous spherical and random tubular pores; extremely violent effervesce; many thin to medium thick patchy clay films on calcite; clear wavy boundary; pH 8.75; conductivity 854 uS; Sample No. 22B004.

2Crk 170-250+ Gray (10YR5/1m, 8/1d) clay with many fine to coarse distinct white mottles due to calcite; medium to coarse strong granular and angular blocky structure; very sticky and very plastic when wet, firm when moist, and very hard when dry; few very fine to medium continuous spherical and random tubular pores; extremely violent effervesce; pH 8.84; conductivity 692 uS; Sample No. 22B005.

| | | | | |
|---|---|---|---|---|
| *ESTIMATED AGE: | $t_o$ | = | 80 | ka |
| | $t_b$ | = | **0** | ka |
| | $t_d$ | = | 80 | kyr |

---

*Pedochronological estimates based on available information. All ages should be considered subject to ±50% variation unless otherwise indicated (Borchardt, 1992). Bold dates are absolute.

$t_o$ = date when soil formation or aggradation began, ka

$t_b$ = date when soil or strata was buried, ka

$t_d$ = duration of soil development or aggradation, kyr

**GEP = Google Earth Pro elevation

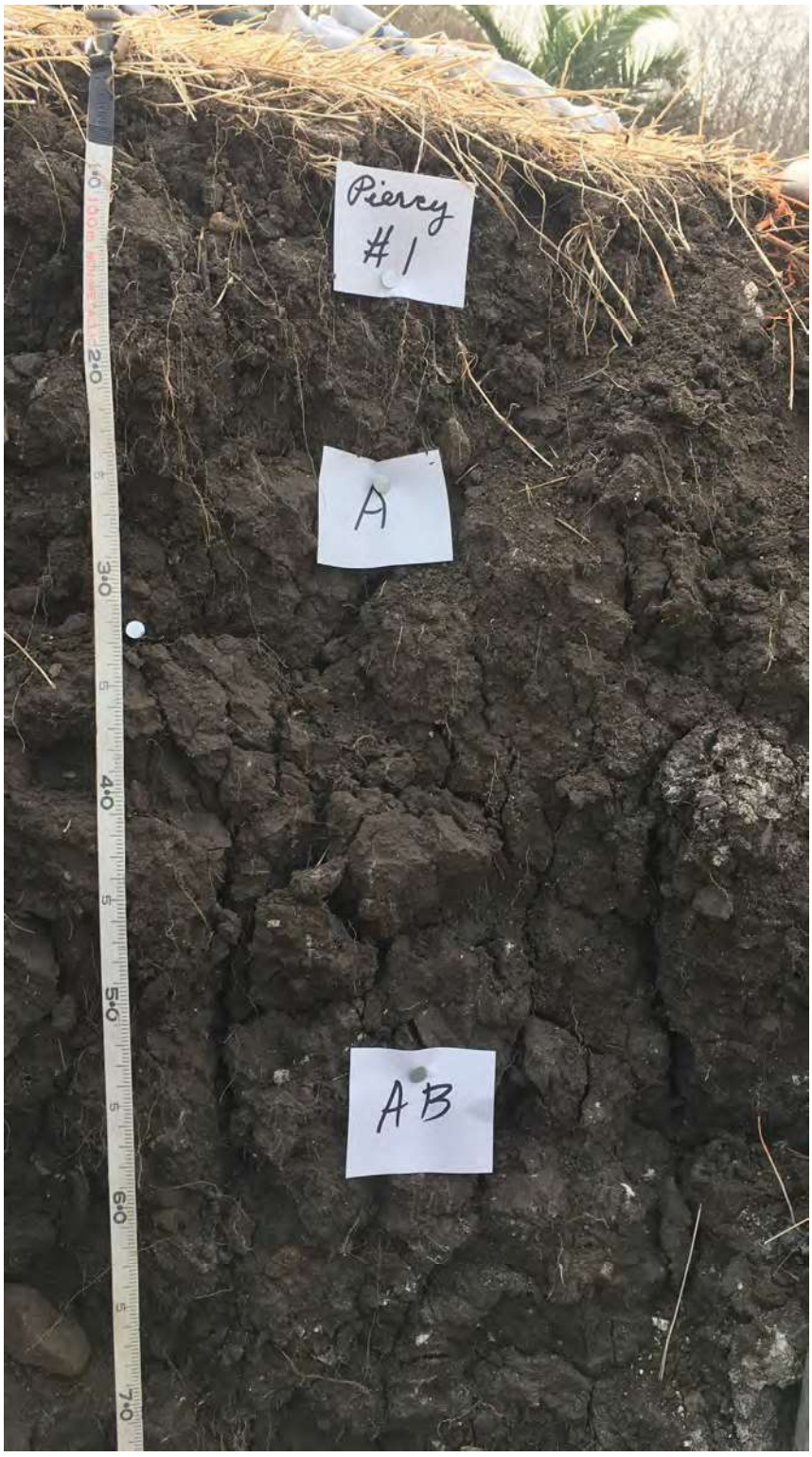



Figure 1. Soil Profile No. 1 in Trench T-1 at Piercy Road. The A horizon is 32 cm thick and is mostly devoid of detrital calcite nodules. Rare detrital calcite nodules exist in the underlying AB horizon extending to the 86-cm depth. View SE.




Figure 2. Soil Profile No. 1 in Trench T-1 at Piercy Road. The Bsskt horizon underlies the AB horizon at 86 cm. Detrital calcite nodules become more numerous along with the slickensides (ss) in the Bsskt. Note the angular clast at the 70-cm depth indicating colluvial influence. View SE.




Figure 3. Soil Profile No. 1 in Trench T-1 at Piercy Road. The Bsskt horizon extends to the 137-cm depth where it begins to penetrate the underlying 2BCrkt horizon. The “ss” designates slickensides, which commonly form in Vertisols as a result of seasonal shrink-swell. View SE.

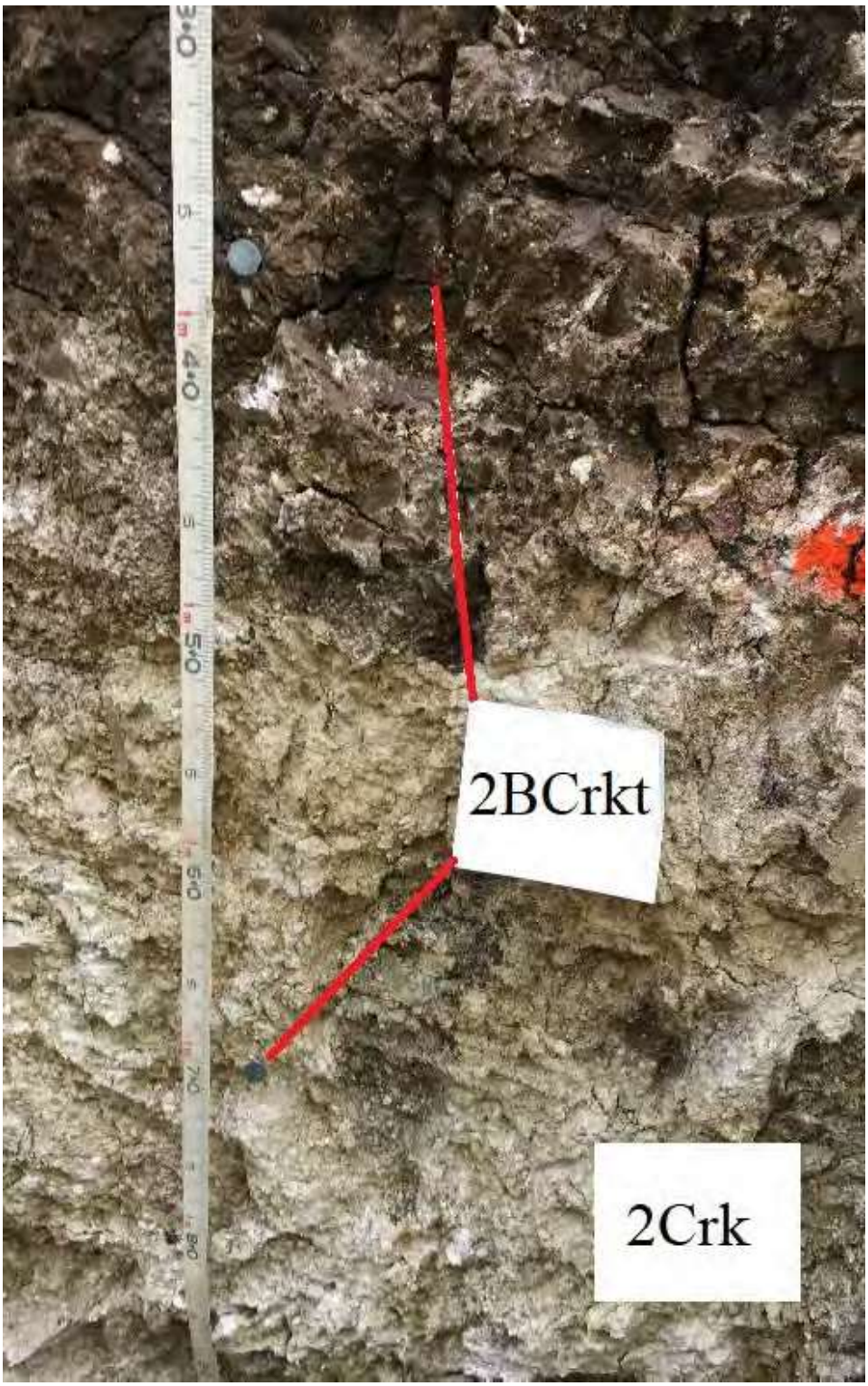



Figure 4. Soil Profile No. 1 in Trench T-1 at Piercy Road. The 2BCrkt horizon penetrates the underlying 2Crk horizon of the Santa Clara Formation along a clear wavy boundary. View SE.




Figure 5. Closeup of rare detrital calcite nodule in the AB horizon of Soil Profile No. 1. Note the presence of a few thin clay films within the degraded matrix of the nodule. Scale in mm.




Figure 6. Closeup of detrital calcite nodule in the Bsskt horizon of Soil Profile No. 1. Note the presence of thin to medium thick patchy clay films coating the nodule, indicating the detrital nodule existed in this horizon before clay illuviation occurred there. Scale in mm.




Figure 7. Closeup of the upper portion of the 2BCrkt horizon of Soil Profile No. 1. Note the interpenetration of clay films among the calcite clasts of the top of the Santa Clara Formation. Scale in mm labeled at 1-cm intervals.

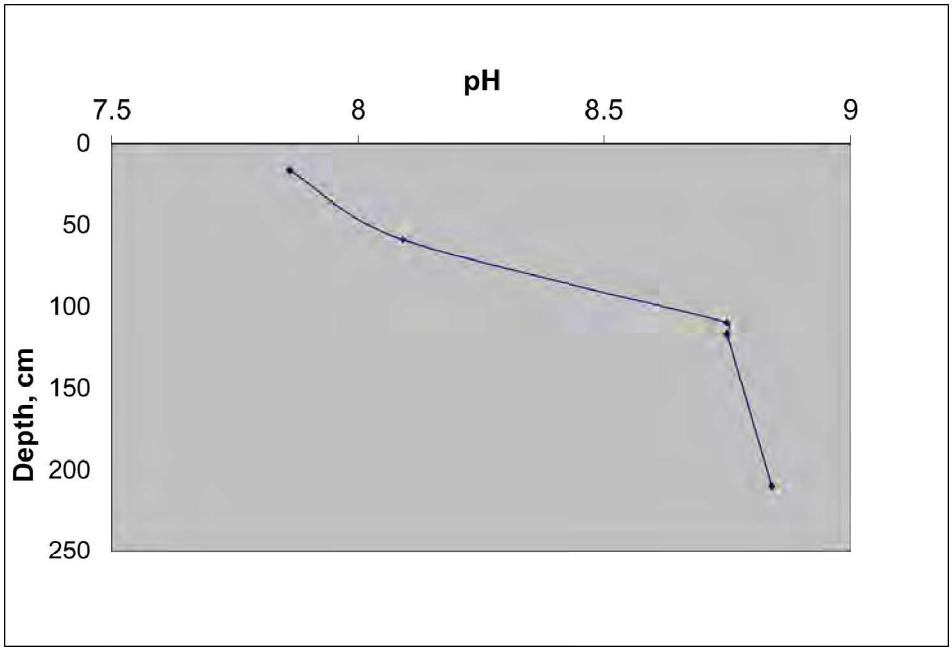



Figure 8. Depth function for pH in Soil Profile No. 1, showing the tendency for the upper horizons to become acidic, with the lowest horizons remaining alkaline.

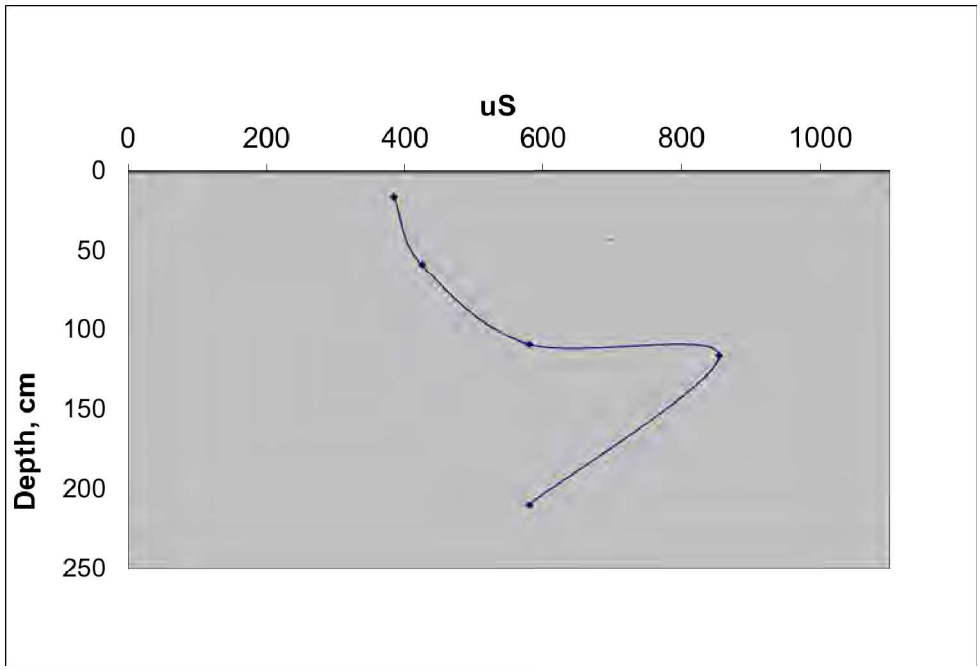



Figure 9. Depth function for electrical conductivity (a measure of salt content) in the soil profile showing the increasing conductivity with depth. Note that the presumed salt content at the base of the profile at 117 cm is twice what it is in the surface.

June 7, 2019

# SOILS GLOSSARY

AGE. Elapsed time in calendar years. Because the cosmic production of C-14 has varied during the Quaternary, radiocarbon years (expressed as kyr B.P.) must be corrected by using tree-ring and other data. Abbreviations used for corrected ages are: ka (kilo anno or years in thousands) or Ma (millions of years). Abbreviations used for intervals are: yr (years), kyr (thousands of years). radiocarbon ages = yr B.P. Calibrated ages are calculated from process assumptions, relative ages fit in a sequence, and correlated ages refer to a matching unit. (See also yr B.P., HOLOCENE, PLEISTOCENE, QUATERNARY, PEDOCHRONOLOGY).

AGGRADATION. Deposition on the earth's surface in the direction of uniformity of grade.

ALKALI (SODIC) SOIL. A soil having so high a degree of alkalinity (pH 8.5 or higher) or so high a percentage of exchangeable sodium (15 % or more of the total exchangeable bases) that plant growth is restricted.

ALKALINE SOIL. Any soil that has a pH greater than 7.3. (See Reaction, Soil.)

ANGULAR ORPHANS. Angular fragments separated from weathered, well-rounded cobbles in colluvium derived from conglomerate.

ARGILLAN. (See Clay Film.)

ARGILLIC horizon. A horizon containing clay either translocated from above or formed in place through pedogenesis.

ALLUVIATION. The process of building up of sediments by a stream at places where stream velocity is decreased. The coarsest particles settle first and the finest particles settle last.

ANOXIC. (See also GLEYED SOIL). A soil having a low redox potential.

AQUICLUDE. A saturated body of sediment or rock that is incapable of transmitting significant quantities of water under ordinary hydraulic gradients.

AQUITARD. A body of rock or sediment that retards but does not prevent the flow of water to or from an adjacent aquifer. It does not readily yield water to wells or springs but may serve as a storage unit for groundwater.

ATTERBERG LIMITS. The moisture content at which a soil passes from a semi-solid to a plastic state (plastic limit, PL) and from a plastic to a liquid state (liquid limit, LL). The plasticity index (PI) is the numerical difference between the LL and the PL.

BEDROCK. The solid rock that underlies the soil and other unconsolidated material or that is exposed at the surface.

BISEQUUM. Two soils in vertical sequence, each soil containing an eluvial horizon and its underlying B horizon.

BOUDIN, BOUDINAGE. From a French word for sausage, describes the way that layers of rock break up under extension. Imagine the hand, fingers together, flat on the table, encased in soft clay and being squeezed from above, as being like a layer of rock. As the spreading clay moves the fingers (sausages) apart, the most mobile rock fractions are drawn or squeezed into the developing gaps.

BURIED SOIL. A developed soil that was once exposed but is now overlain by a more recently formed soil.

CALCAREOUS SOIL. A soil containing enough calcium carbonate (commonly with magnesium carbonate) to effervesce (fizz) visibly when treated with cold, dilute hydrochloric acid. A soil having measurable amounts of calcium carbonate or magnesium carbonate.

CARBONATE MORPHOLOGY STAGES. Descriptive classes of calcite precipitation indicating increasing pedogenesis over time:

| Stage | Description | Percent Carbonate |
|---|---|---|
| I | Bk horizon with few filaments and coatings | <10 |
| I+ | Bk with common filaments and continuous clast coatings | <10 |
| II | Bk with continuous clast coatings, white masses, few nodules | >10 |
| II+ | Bk as above, but matrix is completely whitened, common nodules | >15 |
| >II | K horizon that is 90% white, many nodules | >20 |
| III+ | K that is completely plugged | >40 |
| IV | K as above, but upper part cemented and has weak platy structure | >50 |
| V | K same as above, but laminar layer is strong with incipient brecciation | >50 |
| VI | K brecciation and recementation, as well as pisoliths, are common | >50 |

CATENA. A sequence of soils of about the same age, derived from similar parent material and forming under similar climatic conditions, but having different characteristics due to variation in relief and drainage. (See also TOPOSEQUENCE.)

CEC. Cation exchange capacity. The amount of negative charge balanced by positively charged ions (cations) that are exchangeable by other cations in solution (meq/100 g soil = cmol(+)/kg soil).

CLAY. As a soil separate, the mineral soil particles are less than 0.002 mm in diameter. As a soil textural class, soil material that is 40 percent or more clay, less than 45 percent sand, and less than 40 percent silt.

CLAY FILM. A coating of oriented clay on the surface of a sand grain, pebble, soil aggregate, or ped. Clay films also line pores or root channels and bridge sand grains. Frequency classification is based on the percent of the ped faces and/or pores that contain films: very few--<5%; few--5-25%; common--25-50%; many--50-90%; and continuous--90-100%. Thickness classification is based on visibility of sand grains: thin--very fine sand grains standout; moderately thick--very fine sand grains impart microrelief to film; thick--fine sand grains enveloped by clay and films visible without magnification. Synonyms: clay skin, clay coat, argillan, illuviation cutan.

CLAY LAMELLAE. Thin, generally wavy subhorizontal bands that appear as multiple micro-Bt horizons at the base of the solum generally in sandy Holocene deposits. Each lamella generally is 1-3 cm in thickness. There may be two to six or more clay lamellae between 5 and 30 cm apart.

COBBLE. Rounded or partially rounded fragments of rock ranging from 7.5 to 25 cm in diameter.

COLLUVIUM. Any loose mass of soil or rock fragments that moves downslope largely by the force of gravity. Usually it is thicker at the base of the slope.

COLLUVIUM-FILLED SWALE. The prefailure topography of the source area of a debris flow.

COMPARATIVE PEDOLOGY. The comparison of soils, particularly through examination of features known to evolve through time.

CONCRETIONS. Grains, pellets, or nodules of various sizes, shapes, and colors consisting of concentrated compounds or cemented soil grains. The composition of most concretions is unlike that of the surrounding soil. Calcium carbonate and iron oxide are common compounds in concretions.

CONDUCTIVITY. The ability of a soil solution to conduct electricity, generally expressed as

the reciprocal of the electrical resistivity. Electrical conductance is the reciprocal of the resistance (1/R = 1/ohm = $ohm^{-1}$ = mho [reverse of ohm] = siemens = S), while electrical conductivity is the reciprocal of the electrical resistivity (EC = 1/r = 1/ohm-cm = mho/cm = S/cm or mmho/cm = dS/m). EC, expressed as uS/cm, is equivalent to the ppm of salt in solution when multiplied by 0.640. Pure rain water has an EC of 0, standard 0.01 N KCl is 1411.8 uS at 25C, and the growth of salt-sensitive crops is restricted in soils having saturation extracts with an EC greater than 2,000 uS/cm. Measurements in soils are usually performed on 1:1 suspensions containing one part by weight of soil and one part by weight of distilled water.

CONSISTENCE, SOIL. The feel of the soil and the ease with which a lump can be crushed by the fingers. Terms commonly used to describe consistence are --

Loose.--Noncoherent when dry or moist; does not hold together in a mass.

Friable.--When moist, crushes easily under gentle pressure between thumb and forefinger and can be pressed together into a lump.

Firm.--When moist, crushes under moderate pressure between thumb and forefinger, but resistance is distinctly noticeable.

Plastic.--When wet, readily deformed by moderate pressure but can be pressed into a lump; will form a "wire" when rolled between thumb and forefinger.

Sticky.--When wet, adheres to other material, and tends to stretch somewhat and pull apart, rather than to pull free from other material.

Hard.--When dry, moderately resistant to pressure; can be broken with difficulty between thumb and forefinger.

Soft.--When dry, breaks into powder or individual grains under very slight pressure.

Cemented.--Hard and brittle; little affected by moistening.

CTPOT. Easily remembered acronym for climate, topography, parent material, organisms, and time; the five factors of soil formation.

CUMULIC. A soil horizon that has undergone aggradation coincident with its active development.

CUTAN. (See Clay Film.)

DEBRIS FLOW. Incoherent or broken masses of rock, soil, and other debris that move downslope in a manner similar to a viscous fluid.

DEBRIS SLOPE. A constant slope with debris on it from the free face above.

DEGRADATION. A modification of the earth's surface by erosion.

DURIPAN. A subsurface soil horizon that is cemented by illuvial silica, generally deposited as opal or microcrystalline silica, to the degree that less than 50 percent of the volume of air-dry fragments will slake in water or HCl.

ELUVIATION. The removal of soluble material and solid particles, mostly clay and humus, from a soil horizon by percolating water.

EOLIAN. Deposits laid down by the wind, landforms eroded by the wind, or structures such as ripple marks made by the wind.

FAULT-LINE SCARP. A scarp that has been produced by differential erosion along an old fault line.

FAULTSLIDE. A landslide that shows physical evidence of its interaction with a fault.

FIRST-ORDER DRAINAGE. The most upstream, field-discernible concavity that conducts water and sediments to lower parts of a watershed.

FLOOD PLAIN. A nearly level alluvial plain that borders a stream and is subject to flooding unless protected artificially.

FOSSIL FISSURE. A buried rectilinear chamber associated with extension due to ground movement. The chamber must be oriented along the strike of the shear and must have vertical and horizontal dimensions greater than its width. It must show no evidence of faunal activity and its walls may have silt or clay coatings indicative of frequent temporary saturation with ground water. May be mistaken for an animal burrow. Also known as a paleofissure.

FRAGIC. Term for the tendency for a hard or extremely hard ped or clod to rupture suddenly rather than to undergo slow deformation when pressure is applied. Fragic peds slake in water within 10 minutes and display no cementation upon repeated wetting and drying. Fragic clays tend to be kaolinitic.

FRIABILITY. Term for the ease with which soil crumbles. A friable soil is one that crumbles easily.

GENESIS, SOIL. The mode of origin of the soil. Refers especially to the processes or soil-forming factors responsible for the formation of the solum (A and B horizons) from the unconsolidated parent material.

GEOMORPHIC. Pertaining to the form of the surface features of the earth. Specifically, geomorphology is the analysis of landforms and their mode of origin.

GLEYED SOIL. A soil having one or more neutral gray horizons as a result of water logging

and lack of oxygen. The term "gleyed" also designates gray horizons and horizons having yellow and gray mottles as a result of intermittent water logging.

GRAVEL. Rounded or angular fragments of rock 2 to 75 mm in diameter. Soil textures with >15% gravel have the prefix "gravelly" and those with >90% gravel have the suffix "gravel."

HIGHSTAND. The highest elevation reached by the ocean during an interglacial period.

HOLOCENE. The most recent epoch of geologic time, extending from 10 ka to the present.

HORIZON, SOIL. A layer of soil, approximately parallel to the surface, that has distinct characteristics produced by soil-forming processes. These are the major soil horizons:

> O horizon.--The layer of organic matter on the surface of a mineral soil. This layer consists of decaying plant residues.
>
> A horizon.--The mineral horizon at the surface or just below an O horizon. This horizon is the one in which living organisms are most active and therefore is marked by the accumulation of humus. The horizon may have lost one or more of soluble salts, clay, and sesquioxides (iron and aluminum oxides).
>
> E horizon -- This eluvial horizon is light in color, lying beneath the A horizon and above the B horizon. It is made up mostly of sand and silt, having lost most of its clay and iron oxides through reduction, chelation, and translocation.
>
> B horizon.--The mineral horizon below an A horizon. The B horizon is in part a layer of change from the overlying A to the underlying C horizon. The B horizon also has distinctive characteristics caused (1) by accumulation of clay, sesquioxides, humus, or some combination of these; (2) by prismatic or blocky structure; (3) by redder or stronger colors than the A horizon; or (4) by some combination of these.
>
> C horizon.--The relatively unweathered material immediately beneath the solum. Included are sediment, saprolite, organic matter, and bedrock excavatable with a spade. In most soils this material is presumed to be like that from which the overlying horizons were formed. If the material is known to be different from that in the solum, a number precedes the letter C.
>
> R horizon.--Consolidated rock not excavatable with a spade. It may contain a few cracks filled with roots or clay or oxides. The rock usually underlies a C horizon but may be immediately beneath an A or B horizon.

Major horizons may be further distinguished by applying prefix Arabic numbers to designate differences in parent materials as they are encountered (e.g., 2B, 2BC, 3C) or by applying suffix

numerals to designate minor changes (e.g., B1, B2).

The following is from the Natural Resources Conservation Service, except for the proposed addition of mn:

**“Suffix Symbols**

Lowercase letters are used as suffixes to designate specific kinds of master horizons and layers. The term “accumulation” is used in many of the definitions of such horizons to indicate that these horizons must contain more of the material in question than is presumed to have been present in the parent material. The suffix symbols and their meanings are as follows:

a Highly decomposed organic material

This symbol is used with O to indicate the most highly decomposed organic materials, which have a fiber content of less than 17 percent (by volume) after rubbing.

b Buried genetic horizon

This symbol is used in mineral soils to indicate identifiable buried horizons with major genetic features that were developed before burial. Genetic horizons may or may not have formed in the overlying material, which may be either like or unlike the assumed parent material of the buried soil. This symbol is not used in organic soils, nor is it used to separate an organic layer from a mineral layer.

c *Concretions or nodules*

This symbol indicates a significant accumulation of concretions or nodules. Cementation is required. The cementing agent commonly is iron, aluminum, manganese, or titanium. It cannot be silica, dolomite, calcite, or more soluble salts.

co *Coprogenous earth*

This symbol, used only with L, indicates a limnic layer of coprogenous earth (or sedimentary peat).

d *Physical root restriction*

This symbol indicates noncemented, root-restricting layers in natural or human-made sediments or materials. Examples are dense basal till, plowpans, and other mechanically compacted zones.

di *Diatomaceous earth*

This symbol, used only with L, indicates a limnic layer of diatomaceous earth.

e *Organic material of intermediate decomposition*

This symbol is used with O to indicate organic materials of intermediate decomposition. The fiber content of these materials is 17 to 40 percent (by volume) after rubbing.

f *Frozen soil or water*

This symbol indicates that a horizon or layer contains permanent ice. The symbol is not used for seasonally frozen layers or for dry permafrost.

ff *Dry permafrost*

This symbol indicates a horizon or layer that is continually colder than 0° C and does not contain enough ice to be cemented by ice. This suffix is not used for horizons or layers that have a temperature warmer than 0° C at some time of the year.

g *Strong gleying*

This symbol indicates either that iron has been reduced and removed during soil formation or that saturation with stagnant water has preserved it in a reduced state. Most of the affected layers have chroma of 2 or less, and many have redox concentrations. The low chroma can represent either the color of reduced iron or the color of uncoated sand and silt particles from which iron has been removed. The symbol g is not used for materials of low chroma that have no history of wetness, such as some slates or E horizons. If g is used with B, pedogenic change in addition to gleying is implied. If no other pedogenic change besides gleying has taken place, the horizon is designated Cg.

h *Illuvial accumulation of organic matter*

This symbol is used with B to indicate the accumulation of illuvial, amorphous, dispersible complexes of organic matter and sesquioxides if the sesquioxide component is dominated by aluminum but is present only in very small quantities. The organo-sesquioxide material coats sand and silt particles. In some horizons these coatings have coalesced, filled pores, and cemented the horizon. The symbol h is also used in combination with s as “Bhs” if the amount of the sesquioxide component is significant but the color value and chroma, moist, of the horizon are 3 or less.

i *Slightly decomposed organic material*

This symbol is used with O to indicate the least decomposed of the organic materials. The fiber content of these materials is 40 percent or more (by volume) after rubbing.

j *Accumulation of jarosite*

Jarosite is a potassium or iron sulfate mineral that is commonly an alteration product of

pyrite that has been exposed to an oxidizing environment. Jarosite has hue of 2.5Y or yellower and normally has chroma of 6 or more, although chromas as low as 3 or 4 have been reported. [Note: No longer used to indicate "juvenile."]

jj *Evidence of cryoturbation*

Evidence of cryoturbation includes irregular and broken horizon boundaries, sorted rock fragments, and organic soil materials existing as bodies and broken layers within and/or between mineral soil layers. The organic bodies and layers are most commonly at the contact between the active layer and the permafrost.

k *Accumulation of secondary carbonates*

This symbol indicates an accumulation of visible pedogenic calcium carbonate (less than 50 percent, by volume). Carbonate accumulations exist as carbonate filaments, coatings, masses, nodules, disseminated carbonate, or other forms.

kk *Engulfment of horizon by secondary carbonates*

This symbol indicates major accumulations of pedogenic calcium carbonate. The suffix kk is used when the soil fabric is plugged with fine grained pedogenic carbonate (50 percent or more, by volume) that exists as an essentially continuous medium. The suffix corresponds to the stage III plugged horizon or higher of the carbonate morphogenetic stages (Gile et al., 1966).

m *Cementation or induration*

This symbol indicates continuous or nearly continuous cementation. It is used only for horizons that are more than 90 percent cemented, although they may be fractured. The cemented layer is physically root-restrictive. The dominant cementing agent (or the two dominant ones) may be indicated by adding defined letter suffixes, singly or in pairs. The horizon suffix km or kkm indicates cementation by carbonates; qm, cementation by silica; sm, cementation by iron; yym, cementation by gypsum; kqm, cementation by lime and silica; and zm, cementation by salts more soluble than gypsum.

ma *Marl*

This symbol, used only with L, indicates a limnic layer of marl.

mn *Mangans*

This symbol indicates an accumulation of manganese oxide, generally as ped coatings called mangans (First used by Borchardt on 20130418.)

n *Accumulation of sodium*

This symbol indicates an accumulation of exchangeable sodium.

o *Residual accumulation of sesquioxides*

This symbol indicates a residual accumulation of sesquioxides.

p *Tillage or other disturbance*

This symbol indicates a disturbance of the surface layer by mechanical means, pasturing, or similar uses. A disturbed organic horizon is designated Op. A disturbed mineral horizon is designated Ap even though it is clearly a former E, B, or C horizon.

q *Accumulation of silica*

This symbol indicates an accumulation of secondary silica.

r *Weathered or soft bedrock*

This symbol is used with C to indicate cemented layers (moderately cemented or less cemented). Examples are weathered igneous rock and partly consolidated sandstone, siltstone, or slate. The excavation difficulty is low to high.

s *Illuvial accumulation of sesquioxides and organic matter*

This symbol is used with B to indicate an accumulation of illuvial, amorphous, dispersible complexes of organic matter and sesquioxides if both the organic-matter and sesquioxide components are significant and if either the color value or chroma, moist, of the horizon is 4 or more. The symbol is also used in combination with h as “Bhs” if both the organic-matter and sesquioxide components are significant and if the color value and chroma, moist, are 3 or less.

se *Presence of sulfides*

Typically dark colors (e.g., value <4, chroma <2); may have a sulphurous odor.

ss *Presence of slickensides*

This symbol indicates the presence of slickensides. Slickensides result directly from the swelling of clay minerals and shear failure, commonly at angles of 20 to 60 degrees above horizontal. They are indicators that other vertic characteristics, such as wedge-shaped peds and surface cracks, may be present.

t *Accumulation of silicate clay*

This symbol indicates an accumulation of silicate clay that either has formed *in situ* within a horizon or has been moved into the horizon by illuviation, or both. At least some part of the horizon should show evidence of clay accumulation either as coatings on surfaces of peds or in pores, as lamellae, or as bridges between mineral grains.

u *Presence of human-manufactured materials (artifacts)*

This symbol indicates the presence of manufactured artifacts that have been created or modified by humans, usually for a practical purpose in habitation, manufacturing, excavation, or construction activities. Examples of artifacts are processed wood products, liquid petroleum products, coal, combustion by-products, asphalt, fibers and fabrics, bricks, cinder blocks, concrete, plastic, glass, rubber, paper, cardboard, iron and steel, altered metals and minerals, sanitary and medical waste, garbage, and landfill waste.

v *Plinthite*

This symbol indicates the presence of iron-rich, humus-poor, reddish material that is firm or very firm when moist and hardens irreversibly when exposed to the atmosphere and to repeated wetting and drying.

w *Development of color or structure*

This symbol is used with B to indicate the development of color or structure, or both, with little or no apparent illuvial accumulation of material. It should not be used to indicate a transitional horizon.

x *Fragipan character*

This symbol indicates a genetically developed layer that has a combination of firmness and brittleness and commonly a higher bulk density than the adjacent layers. Some part of the layer is physically root-restrictive.

y *Accumulation of gypsum*

This symbol indicates an accumulation of gypsum (<50% by volume).

yy *Dominance of gypsum*

This symbol indicates an accumulation of gypsum (>50% by volume); light colored (e.g., value >7, chroma <4); may be pedogenically derived or inherited transformation of primary gypsum from parent material.

z *Accumulation of salts more soluble than gypsum*

This symbol indicates an accumulation of salts that are more soluble than gypsum; e.g.,

NaCl.

HUMUS. The well-decomposed, more or less stable part of the organic matter in mineral soils.

ILLUVIATION. The deposition by percolating water of solid particles, mostly clay or humus, within a soil horizon.

INTERFLUVE. The land lying between streams.

ISOCHRONOUS BOUNDARY. A gradational boundary between two sedimentary units indicating that they are approximately the same age. Opposed to a nonisochronous boundary, which by its abruptness indicates that it delineates units having significant age differences.

KROTOVINA. An animal burrow filled with soil.

LEACHING. The removal of soluble material from soil or other material by percolating water.

LOWSTAND. The lowest elevation reached by the ocean during a glacial period.

MANGAN. A thin coating of manganese oxide (cutan) on the surface of a sand grain, pebble, soil aggregate, or ped. Mangans also line pores or root channels and bridge sand grains.

MODERN SOIL. The portion of a soil section that is under the influence of current pedogenetic conditions. It generally refers to the uppermost soil regardless of age.

MODERN SOLUM. The combination of the A and B horizons in the modern soil.

MORPHOLOGY, SOIL. The physical make-up of the soil, including the texture, structure, porosity, consistence, color, and other physical, mineral, and biological properties of the various horizons, and the thickness and arrangement of those horizons in the soil profile.

MOTTLING, SOIL. Irregularly marked with spots of different colors that vary in number and size. Mottling in soils usually indicates poor aeration and lack of drainage. Descriptive terms are as follows: abundance--few, common, and many; size--fine, medium, and coarse; and contrast--faint, distinct and prominent. The size measurements are these: fine, less than 5 mm in diameter along the greatest dimension; medium, from 5 to 15 mm, and coarse, more than 15 mm.

MRT (MEAN RESIDENCE TIME.) The average age of the carbon atoms within a soil horizon. Under ideal reducing conditions, the humus in a soil will have a C-14 age that is half the true age of the soil. In oxic soils humus is typically destroyed as fast as it is produced, generally yielding MRT ages no older than 300-1000 years, regardless of the true age of the soil.

MUNSELL COLOR NOTATION. Scientific description of color determined by comparing soil to a Munsell Soil Color Chart (Available from Macbeth Division of Kollmorgen Corp., 2441 N. Calvert St., Baltimore, MD 21218). For example, dark yellowish brown is denoted as 10YR3/4m

in which the 10YR refers to the hue or proportions of yellow and red, 3 refers to value or lightness (0 is black and 10 is white), 4 refers to chroma (0 is pure black and white and 20 is the pure color), and m refers to the moist condition rather than the dry (d) condition.

OVERBANK DEPOSIT. Fine-grained alluvial sediments deposited from floodwaters outside of the fluvial channel.

OXIC. A soil having a high redox potential. Such soils typically are well drained, seldom being waterlogged or lacking in oxygen. Rubification in such soils tends to increase with age.

PALEO SOIL TONGUE. A soil tongue that formed during a previous soil-forming interval.

PALEOSEISMOLOGY. The study of prehistoric earthquakes through the examination of soils, sediments, and rocks.

PALEOSOL. A soil that formed on a landscape in the past with distinctive morphological features resulting from a soil-forming environment that no longer exists at the site. The former pedogenic process was either altered because of external environmental change or interrupted by burial.

PALINSPASTIC RECONSTRUCTION. Diagrammatic reconstruction used to obtain a picture of what geologic and/or soil units looked like before their tectonic deformation.

PARENT MATERIAL. The great variety of unconsolidated organic and mineral material in which soil forms. Consolidated bedrock is not yet parent material by this concept.

PED. An individual natural soil aggregate, such as a granule, a prism, or a block.

PEDOCHRONOLOGY. The study of pedogenesis with regard to the determination of when soil formation began, how long it occurred, and when it stopped. Also known as soil dating. Two ages and the calculated duration are important:

$t_o$ = age when soil formation or aggradation began, ka

$t_b$ = age when the soil or stratum was buried, ka

$t_d$ = duration of soil development or aggradation, kyr

Pedochronological estimates are based on available information. All ages should be considered subject to $\pm$50% variation unless otherwise indicated.

PEDOCHRONOPALEOSEISMOLOGY. The study of prehistoric earthquakes by using pedochronology.

PEDOLOGY. The study of the process through which rocks, sediments, and their constituent minerals are transformed into soils and their constituent minerals at or near the surface of the

earth.

PEDOGENESIS. The process through which rocks, sediments, and their constituent minerals are transformed into soils and their constituent minerals at or near the surface of the earth.

PERCOLATION. The downward movement of water through the soil.

pH VALUE. The negative log of the hydrogen ion concentration. Measurements in soils are usually performed on 1:1 suspensions containing one part by weight of soil and one part by weight of distilled water. A soil with a pH of 7.0 is precisely neutral in reaction because it is neither acid nor alkaline. An acid or "sour" soil is one that gives an acid reaction; an alkaline soil is one that gives an alkaline reaction. In words, the degrees of acidity or alkalinity are expressed as:

| | |
|---|---|
| Extremely acid | <4.5 |
| Very strongly acid | 4.5 to 5.0 |
| Strongly acid | 5.1 to 5.5 |
| Medium acid | 5.6 to 6.0 |
| Slightly acid | 6.1 to 6.5 |
| Neutral | 6.6 to 7.3 |
| Mildly alkaline | 7.4 to 7.8 |
| Moderately alkaline | 7.9 to 8.4 |
| Strongly alkaline | 8.5 to 9.0 |
| Very strongly alkaline | >9.0 |
| | |
| Used if significant: | |
| Very slightly acid | 6.6 to 6.9 |
| Very mildly alkaline | 7.1 to 7.3 |

PHREATIC SURFACE. (See Water Table.)

PLANATION. The process of erosion whereby a portion of the surface of the Earth is reduced to a fundamentally even, flat, or level surface by a meandering stream, waves, currents, glaciers, or wind.

PLEISTOCENE. An epoch of geologic time extending from 10 ka to 1.8 Ma; it includes the last Ice Age.

PRESSURE FACES. Shiny surfaces on soil peds subject to shrink-swell in clayey soils. Unlike slickensides and soil slicks, they show only perpendicular movement to ped surfaces.

PROFILE, SOIL. A vertical section of the soil through all its horizons and extending into the parent material.

PROGRADATION. The building outward toward the sea of a shoreline or coastline by nearshore deposition.

QUATERNARY. A period of geologic time that includes the past 1.8 Ma. It consists of two epochs--the Pleistocene and Holocene.

REFUGIUM. A place of refuge. Plants, animals, and soil minerals tend to accumulate only in the most ideal areas when surrounded by a hostile environment.

RELICT SOIL. A surface soil that was partly formed under climatic conditions significantly different from the present.

RUBIFICATION. The reddening of soils through the release and precipitation of iron as an oxide during weathering. Munsell hues and chromas of well-drained soils generally increase with soil age.

SALINE SOIL. A soil that contains soluble salts in amounts that impair the growth of crop plants but that does not contain excess exchangeable sodium.

SAND. Individual rock or mineral fragments in a soil that range in diameter from 0.05 to 2.0 mm. Most sand grains consist of quartz, but they may be of any mineral composition. The textural class name of any soil that contains 85 percent or more sand and not more than 10 percent clay.

SECONDARY FAULT. A minor fault that bifurcates from or is associated with a primary fault. Movement on a secondary fault never occurs independently of movement on the primary, seismogenic fault.

SHORELINE ANGLE. The line formed by the intersection of the wave-cut platform and the sea cliff. It approximates the position of sea level at the time the platform was formed.

SILT. Individual mineral particles in a soil that range in diameter from the upper limit of clay (0.002 mm) to the lower limit of very find sand (0.05 mm.) Soil of the silt textural class is 80 percent or more silt and less than 12 percent clay.

SLICKENSIDES. Polished and grooved surfaces produced by one mass sliding past another. In soils, slickensides may form along a fault plane; at the bases of slip surfaces on steep slopes; on faces of blocks, prisms, and columns undergoing shrink-swell. In tectonic slickensides, the striations are strictly parallel.

SLIP RATE. The rate at which the geologic materials on the two sides of a fault move past each other over geologic time. The slip rate is expressed in mm/yr, and the applicable duration is

stated. Faults having slip rates less than 0.01 mm/yr are generally considered inactive, while faults with Holocene slip rates greater than 0.1 mm/yr generally display tectonic geomorphology.

SMECTITE. A fine, platy, aluminosilicate clay mineral that expands and contracts with the absorption and loss of water. It has a high cation-exchange capacity and is plastic and sticky when moist.

SOIL. A natural, three-dimensional body at the earth's surface that is capable of supporting plants and has properties resulting from the integrated effect of climate and living matter acting on earthy parent material, as conditioned by relief over periods of time.

SOIL SEISMOLOGIST. Soil scientist who studies the effects of earthquakes on soils.

SOIL SLICKS. Curvilinear striations that form in swelling clayey soils where there is marked change in moisture content. Clayey slopes buttressed by rigid materials may allow minor amounts of gravitationally driven plastic flow, forming soil slicks sometimes mistaken for evidence of tectonism. Soil slicks disappear with depth and the striations are seldom strictly parallel as they are when movement is major. (See also SLICKENSIDES.)

SOIL TECTONICS. The study of the interactions between soil formation and tectonism.

SOIL TONGUE. That portion of a soil horizon extending into a lower horizon.

SOLUM. Combined A and B horizons. Also called the true soil. If a soil lacks a B horizon, the A horizon alone is the solum.

STONELINE. A thin, buried, planar layer of stones, cobbles, or bedrock fragments. Stonelines of geological origin may have been deposited upon a former land surface. The fragments are more often pebbles or cobbles than stones. A stoneline generally overlies material that was subject to weathering, soil formation, and erosion before deposition of the overlying material. Many stonelines seem to be buried erosion pavements, originally formed by running water on the land surface and concurrently covered by surficial sediment.

STRATH TERRACE. A gently sloping terrace surface bearing little evidence of aggradation.

STRUCTURE, SOIL. The arrangement of primary soil particles into compound particles or aggregates that are separated from adjoining aggregates. The principal forms of soil structure are--platy (laminated), prismatic (vertical axis of aggregates longer than horizontal), columnar (prisms with rounded tops), blocky (angular or subangular), and granular. Structureless soils are either single grained (each grain by itself, as in dune sand) or massive (the particles adhering without any regular cleavage, as in many hardpans).

SUBSIDIARY FAULT. A branch fault that extends a substantial distance from the main fault

zone.

TECTOTURBATION. Soil disturbance resulting from tectonic movement.

TEXTURE, SOIL. Particle size classification of a soil, generally given in terms of the USDA system which uses the term "loam" for a soil having equal properties of sand, silt, and clay. The basic textural classes, in order of their increasing proportions of fine particles are sand, loamy sand, sandy loam, loam, silt loam, silt, sandy clay loam, clay loam, silty clay loam, sand clay, silty clay, and clay. The sand, loamy sand, and sandy loam classes may be further divided by specifying "coarse," "fine," or "very fine."

TOPOSEQUENCE. A sequence of kinds of soil in relation to position on a slope. (See also CATENA.)

TRANSLOCATION. The physical movement of soil particles, particularly fine clay, from one soil horizon to another under the influence of gravity.

UNIFIED SOIL CLASSIFICATION SYSTEM. The particle size classification system used by the U.S. Army Corps of Engineers and the Bureau of Reclamation. Like the ASTM and AASHO systems, the sand/silt boundary is at 80 um instead of 50 um used by the USDA. Unlike all other systems, the gravel/sand boundary is at 4 mm instead of 2 mm and the silt/clay boundary is determined by using Atterberg limits.

VERTISOL. A soil with at least 30% clay, usually smectite, that fosters pronounced changes in volume with change in moisture. Cracks greater than 1 cm wide appear at a depth of 50 cm during the dry season each year. One of the ten USDA soil orders.

WATER TABLE. The upper limit of the soil or underlying rock material that is wholly saturated with water. Also called the phreatic surface.

WAVE-CUT PLATFORM. The relatively smooth, slightly seaward-dipping surface formed along the coast by the action of waves generally accompanied by abrasive materials.

WEATHERING. All physical and chemical changes produced in rocks or other deposits at or near the earth's surface by atmospheric agents. These changes result in disintegration and decomposition of the material.

WETTING FRONT. The greatest depth affected by moisture due to precipitation.

yr B.P. Uncorrected radiocarbon age expressed in years before present, calculated from 1950. Calendar-corrected ages are expressed in ka (or cal BP), or, if warranted, as AD or BCE (before the common era).

CORNERSTONE EARTH GROUP

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|---|---|
| Date: | December 22, 2022 |
| Project No.: | 1300-2-2 |
| Prepared For: | Ms. Teresa Goodwin<br>**HPA, INC.**<br>600 Grand Avenue, Suite 302<br>Oakland, California 94610 |
| Re: | Site Response Analysis<br>644 & 675 Piercy Road Industrial<br>San Jose, California |

Dear Ms. Goodwin:

As requested, this letter presents our updated seismic analysis for the above referenced project. As you know, we have completed a geotechnical investigation and provided recommendations in a geotechnical report titled, "Geotechnical Investigation, 644 & 675 Piercy Road Industrial, San Jose, California," dated December 12, 2022. That report should be referred to for additional recommendations not provided in this letter.

The previous seismic parameters presented in our 2022 report were in accordance with the Exception per ASCE 7-16 Section 11.4.8 and the 2022 California Building Code (CBC). This letter presents the Site Response Analysis (SRA) design parameters in accordance with Chapter 16, 18, and Appendix J of the 2022 CBC and Chapters 11, 12, 20, and 21 and Supplement No. 1 of ASCE 7-16.

## THE PROJECT

The 644 and 675 Piercy Road Industrial project will include redeveloping the approximately 16-acre site for industrial use. The new development will include a warehouse totaling about 209,000 square feet including about 35,000 square feet of office and mezzanine space. We anticipate the building will be single-story with an interior clear height of 36 to 40 feet and consist of concrete tilt-up construction. The truck court is located along the southwest side of the building with about 34 dock doors currently planned. The proposed building is currently sloped with planned finished floor at Elevation 249.4 feet on the east side and Elevation 253.9 feet on the west side. Detention basins are planned along the east and south sides of the site. At-grade auto and trailer parking and drive aisles will cover the remainder of the site. A 50-foot-wide PG&E easement for an existing gas main runs northwest to southeast through the center of the project site. Appurtenant utilities, landscaping, storm water management areas, and other improvements necessary for overall site development will also be constructed.

## SITE RESPONSE ANALYSIS

Following Section 11.4.8 of ASCE 7-16, our technical partner, Robert Pyke, PhD, G.E., performed a Site Response Analysis (SRA) in accordance with Chapter 21, Section 21.1. The results are presented in Appendix A. The recommended MCE Spectrum is shown graphically on Figure 7 and tabulated in Table 2 that report. We have summarized the recommended seismic design values in Table 1 below for your use.

**Table 1 – Site-Specific Design Acceleration Parameters**

| Parameter | Value |
|---|---|
| $S_{DS}$ | 1.15 |
| $S_{D1}$ | 0.48 |
| $S_{MS}$ | 1.73 |
| $S_{M1}$ | 0.73 |

## CLOSURE

We hope this provides the information you need at this time. Recommendations presented in this letter have been prepared for the sole use of HPA, Inc. specifically for the property at 644 & 675 Piercy Road in San Jose, California. Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices at this time and location. No warranties are either expressed or implied.

If you have any questions or need any additional information from us, please call and we will be glad to discuss them with you.

Sincerely,

**Cornerstone Earth Group, Inc.**

Stephen C. Ohlsen, P.E.
Project Engineer

REGISTERED PROFESSIONAL ENGINEER
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Attachments: Appendix A – Site Response Analysis (SRA)

Copies: Addressee (PDF by email)