CORNERSTONE
EARTH GROUP

TYPE OF SERVICES	Geotechnical Investigation
PROJECT NAME	435 East 3rd Mixed-Use Development
LOCATION	435 East 3rd Avenue, San Mateo, California
CLIENT	Windy Hill Property Ventures
PROJECT NUMBER	803-10-1
DATE	August 27, 2021

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Type of Services	**Geotechnical Investigation**
Project Name	**435 East 3rd Mixed-Use Development**
Location	**435 East 3rd Avenue San Mateo, California**
Client	**Windy Hill Property Ventures**
Client Address	**530 Emerson Street, Suite 150 Palo Alto, California**
Project Number	**803-10-1**
Date	

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SECTION 1: INTRODUCTION

This geotechnical report was prepared for the sole use of Windy Hill Property Ventures for the 435 E 3rd Office Development in San Mateo, California. The location of the site is shown on the Vicinity Map, Figure 1. For our use, we were provided with the following documents:

- A set of architectural plans titled “A Planning Application for 435 E. Third Avenue, San Mateo,” prepared by Arc Tec, Inc. dated Prelim Planning Revisions, May 14, 2021.
- A preliminary civil utility plan titled “A Planning Application for 435 E. Third Avenue, San Mateo,” prepared by BKF, dated Prelim Planning Revisions, May 14, 2021.

1.1 PROJECT DESCRIPTION

The planned development will be a 5-story at-grade mixed-use structure, and likely of steel- -frame construction. The ground through fourth floor will be for commercial office use with a balcony on the fourth floor. The fifth floor will for residential and will include exterior patios and a mechanical yard. The planned development will have a first floor footprint of approximate 9,819 square feet and will take up most of the existing 11,035 square foot property. Appurtenant utilities, landscaping and other improvements necessary for site development are also planned.

Structural loads are not currently known for the proposed structure; however, structural loads are expected to be typical for similar type structures. Minor cuts and fills on the order of 1 to 3 feet are expected for general site grading with potentially deeper cuts for undocumented fill removal anticipated in isolated areas of the site.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated June 29, 2021 and consisted of field and laboratory programs to evaluate physical and engineering properties of the subsurface soils, engineering analysis to prepare recommendations for site work and grading, building foundations, flatwork, 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 two borings drilled on July 24, 2021 with truck-mounted hollow-stem auger drilling equipment. The borings were drilled to depths ranging from approximately 40 to 60 feet. The borings were backfilled with cement grout in accordance with local requirements; exploration permits were obtained as required by local jurisdictions.

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

1.4 LABORATORY TESTING PROGRAM

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

1.5 ENVIRONMENTAL SERVICES

Environmental services were not requested for this project. If environmental concerns are determined to be present during future evaluations, the project environmental consultant should review our geotechnical recommendations for compatibility with the environmental concerns.

SECTION 2: REGIONAL SETTING

2.1 GEOLOGICAL SETTING

2.1.1 Regional Geology

The relatively flat-lying plain along the western edge of the San Francisco Bay is bounded by the Santa Cruz Mountains on the west and the San Francisco Bay to the east. The Coast Ranges is a geomorphic province of California that stretches from the Oregon border nearly to Point Conception. In the San Francisco Bay area, most of the Coast Ranges have developed on a basement of tectonically mixed Cretaceous- and Jurassic-age (70- to 200-million years old) rocks of the Franciscan Complex. Younger sedimentary and volcanic units locally cap these basement rocks. Still younger surficial deposits that reflect geologic conditions of the last million years or so cover most of the Coast Ranges.

Movement on the many splays of the San Andreas Fault system has produced the dominant northwest-oriented structural and topographic trend seen throughout the Coast Ranges today. This trend reflects the boundary between two of the Earth's major tectonic plates: the North American plate to the east and the Pacific plate to the west. The San Andreas Fault system and its major branching faults is about 40 miles wide in the Bay area and extends from the San

Gregorio Fault near the coastline to the Coast Ranges-Central Valley blind thrust at the western edge of the Great Central Valley as shown on the Regional Fault Map, Figure 3. The San Andreas Fault is the dominant structure in the system, nearly spanning the length of California, and capable of producing the highest magnitude earthquakes. Many other subparallel or branch faults within the San Andreas system are equally active and nearly as capable of generating large earthquakes. Right-lateral movement dominates on these faults but an increasingly large amount of thrust faulting resulting from compression across the system is now being identified also.

2.1.2 Local Geology

Roughly half the San Mateo 7.5-Minute Quadrangle and adjacent areas are covered by Quaternary alluvial sediment shed from the northwest-trending Santa Cruz Mountains that occupy the area west of the site (Pampeyan, 1994) as seen on Figure 4, Vicinity Geologic Map. The site is in an area adjacent to the San Francisco Bay where Holocene age (11,000 years or less before present) alluvial fan deposits account for the majority of Quaternary sediment deposited in the area, and is shown as underlain by medium-grained alluvium (Qam) of Holocene age over older alluvium (Qoa) of Pleistocene age.

The Qam unit is described as “unconsolidated to moderately consolidated, moderately sorted fine sand, silt and clayey silt.” The Qam unit is generally less than 20 feet thick, was deposited at the edge of coarse-grained alluviual fans (Qac) and locally interfingers with coarse and fine-grained alluvium (Qaf). It forms much of the flatland alluvial plain along the western edge of the Bay in the San Mateo quadrangle. The Qoa unit is designated as “(Late Pleistocene) older alluvial fan deposits” and is described as “unconsolidated to moderately consolidated gravel, sand and silt.”

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.

Table 1: Approximate Fault Distances

Fault Name	Distance	
	(miles)	(kilometers)
San Andreas (1906)	3.5	5.6
Monte Vista-Shannon	9.8	15.8
San Gregorio	10.4	16.7
Hayward (Total Length)	14.9	24.0

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 SITE BACKGROUND

We reviewed historical aerial imagery provided online by Historical Aerials (http://www.historicaerials.com). A summary of pertinent surface changes at and in the near vicinity of the site is as follows:

- 1956: The property appears to be vacant. The surrounding vicinity is developed with residential homes and commercial buildings. The existing street layout is observed in this aerial image.
- 1982: The existing auto repair shop building and canopy appear to be built. The asphalt parking lot also is observed on the property. The surrounding vicinity appears more developed.
- 1993: No pertinent surface changes are observed.
- 2018: No pertinent surface changes are observed.

From our research on the California State GeoTracker website, a case history is present at the 435 East 3rd Avenue address. A case closure letter report titled "Case Closure of One 10,000 Gallon Unleaded Gasoline, One 7,500 Gallon Regular Gasoline, One 5,000 Gallon Premium Gasoline, and One 280 Gallon Waste Oil Underground Storage Tank at Accu-Tune, 435 East 3rd Avenue, San Mateo, California" dated April 6, 1998 states that four separate leaky underground storage tanks were removed from the site in 1989. An inspection report dated July 10, 1989 by San Mateo County Environmental includes a summary of the observed conditions of the tanks as well as a rough drawn map denoting soil sampling locations and approximate sample depths. Soil sampling depths denoted on the inspection report ranged from 8 to 15 feet. The inspection report also includes a plan which shows the approximate locations of the four tanks prior to removal.

It is unclear how the excavations resulting from the tank and soil removal were backfilled and what material was used to bring the areas up to grade. As records of compaction have not been provided, any fills encountered are considered undocumented. We anticipate that undocumented fill within the vicinity of the removed tanks could extend as deep as 14 to 15 feet below existing grades. Detailed recommendations for mitigation of undocumented fills are present in the “Earthwork” section of this report.

3.2 SURFACE DESCRIPTION

The site is set in a commercial/residential area of San Mateo adjacent to the downtown district. The site is bounded by a commercial building to the southwest, a vacant lot to the northwest, East 3rd Avenue to the southeast, and South Claremont Street to the northeast. The site is relatively level but graded to drain to storm drainage facilities. A single-story auto repair shop is situated on the northern portion of the site with a canopy structure extending from the repair shop to the sidewalk along East 3rd Avenue. Underneath the canopy, is a concrete pad with vehicle lift stations anchored into the concrete. An asphalt parking lot extends around the building and canopy with landscaping planters on the edges of the site. Multiple trees are observed in the planters. No signs of underground storage tanks were observed at the surface.

Surface pavements generally consisted of 1 to 1½ inches of asphalt concrete over 4 to 4½ inches of aggregate base. Based on visual observations, the existing pavements are in poor shape with moderate to significant alligator cracking.

3.3 SUBSURFACE CONDITIONS

Below the surface pavements, our explorations generally encountered existing undocumented fill underlain by over interbedded native alluvial soils to the maximum depths explored during this investigation. A more detailed description of the subsurface conditions are presented in the following sections.

3.3.1 Undocumented Fills

Below the surface pavements, our borings generally encountered up to approximately 1½ feet of undocumented fill which generally consisted of stiff to hard sandy lean clay. We note that neither of our explorations were performed within the vicinity of the previous underground storage tanks.

3.3.2 Alluvial Soils

Below the undocumented fills, the native alluvial soils generally consisted of very stiff to hard sandy lean clay to lean clay with sand to depths of approximately 5 to 14 feet below existing site grades. Below the clays, dense to very dense clayey sand with gravel were observed to approximate depths of 21½ to 22 feet. A very dense layer of well graded sand with clay and gravel was encountered in Boring EB-2 to a depth of 29 feet followed by dense to very dense clayey sand with gravel to about 40 feet below existing grades, the terminal depth of the boring. Below the clayey sand in EB-1, hard sandy lean clay to lean clay with sand was observed to a depth of about 41½ feet followed by very dense clayey sand with gravel to the maximum depth explored of about 60 feet below existing grades.

3.3.3 Plasticity/Expansion Potential

We performed one Plasticity Index (PI) test on a representative sample. Test results were used to evaluate expansion potential of surficial soils. The result of the surficial PI test indicated a PI of 11, indicating low to moderate low expansion potential to wetting and drying cycles.

3.3.4 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents within the upper 15 feet range from 7 to 20 percent moisture. In our opinion, we estimated this corresponds to about 5 percent under to 6 percent above the estimated laboratory optimum moisture content.

3.4 GROUNDWATER

Groundwater was encountered in our explorations, EB-1 & EB-2, at depths ranging from 27½ to 28½ feet below current grades. All measurements were taken at the time of drilling and may not represent the stabilized levels that can be higher than the initial levels encountered.

Published data (CGS, San Mateo 7.5-minute Quadrangle, 2018) indicated that seasonal and/or historical high groundwater levels in the vicinity of the site are on the order of 12 to 13 feet below the ground surface.

In general, fluctuations in groundwater levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors.
Based on the above information and our experience in the area, we recommend a design groundwater depth of 12 feet below current grades.

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. As shown in Figure 3, no known surface expression of fault traces is thought to cross the site; therefore, fault surface rupture hazard is not a significant geologic hazard at the site.

4.2 ESTIMATED GROUND SHAKING

Moderate to severe (design-level) earthquakes can cause strong ground shaking, which is the case for most sites within the Bay Area. A peak ground acceleration (PGA_M) was estimated following the ground motion hazard analysis procedure presented in Chapter 16 and 18 and Appendix J of the 2019 California Building Code (CBC) and Chapter 21, Section 21.2 of ASCE 7-16 and Supplement No. 1. For our analysis we used a PGA_M of 0.89g which was determined in accordance with Section 21.5 of ASCE 7-16.

4.3 LIQUEFACTION POTENTIAL

The site is not located within a State-designated Liquefaction Hazard Zone (CGS, San Mateo 7.5-minute Quadrangle, 2018). However, we screened the site for liquefaction during our site exploration by retrieving samples from the site, performing visual classification on sampled materials, and performing various tests to further classify the soil properties.

During strong seismic shaking, cyclically induced stresses can cause increased pore pressures within the soil matrix that can result in liquefaction triggering, soil softening due to shear stress loss, potentially significant ground deformation due to settlement within sandy liquefiable layers as pore pressures dissipate, and/or flow failures in sloping ground or where open faces are present (lateral spreading) (NCEER 1998). Limited field and laboratory data is available regarding ground deformation due to settlement; however, in clean sand layers settlement on the order of 2 to 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 stiff cohesive and dense granular soils below the design groundwater depth. Based on the above and our experience in the area, our screening of the site for liquefaction indicates a low potential for liquefaction.

4.4 LATERAL SPREADING

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

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

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

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

4.6 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 1-mile inland from the San Francisco Bay shoreline and is approximately 24 to 26 feet above mean sea level (Google Earth, WGS84). According to published maps (CGS, County of San Mateo Tsunami Hazard Area Map, 2021), the site is not within a tsunami inundation zone. Therefore, the potential for inundation due to tsunami or seiche is considered low.

4.7 FLOODING

Based on our internet search of the Federal Emergency Management Agency (FEMA) flood map public database, the site is located within Zone X, an area of minimal flood hazard. We recommend the project civil engineer be retained to confirm this information.

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 undocumented fill
- Redevelopment considerations
- Depth to groundwater

5.1.1 Presence of Undocumented Fill

While our explorations encountered up to approximately 1½ feet of undocumented fill, additional undocumented fill may be present as a result of prior development grading. As discussed above, we understand that several underground storage tanks were previously removed from the site in 1989 and backfilled to grade. We anticipate that undocumented fills could be encountered up to 12 to 15 feet below existing grade in the vicinity of the previous tank excavations. Any fills encountered during site grading should be completely removed from within the building area 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.

The new building will span over backfill of the fill from the tank removal and undisturbed native soil areas, which could result in increased differential movement under static and seismic loading conditions. Therefore, to reduce the potential for differential movement beneath the structure, the side slopes of former tank excavations should be over-excavated or benched to inclinations no steeper than 2:1 (horizontal:vertical) within the new building areas to reduce abrupt fill thickness transitions. We recommend that engineered fill placed within new building at-grade areas be compacted to 95 percent relative compaction.

Provided undocumented fills are mitigated by removal and replacement as engineered fill, the potential impact due to undocumented fill should be low. Refer to Section 6.3, Mitigation of Undocumented Fills, below for further recommendations.

5.1.2 Redevelopment Considerations

As discussed, the site is currently occupied by an existing single-story building, a canopy structure, and appurtenant flatwork, site fixtures, and landscaping. We understand that all the existing improvements will be demolished for the construction of the new building. Potential issues that are often associated with redeveloping sites include demolition of existing improvements, abandonment of existing utilities, and undocumented fills. Please refer to the “Earthwork” section below for further recommendations.

5.1.3 Depth to Groundwater

As discussed above, based on our experience in the project vicinity and historic high groundwater maps, we recommend a design groundwater depth of 12 feet below the ground surface. Groundwater may be encountered during deeper excavations for elevator pits, utility installation, foundations, or mitigation of undocumented fills. Our experience with similar sites indicates that shallow ground water could significantly impact grading and underground construction. These impacts typically consist of potentially wet and unstable subgrade, difficulty achieving compaction, and difficult underground utility installation. Dewatering and shoring of utility trenches and demolition excavations may be required in some isolated areas of the site. Detailed recommendations addressing this concern are presented in the “Earthwork” section 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 are currently 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.

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

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

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 3 to 4 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 earlier, undocumented fills were encountered in our borings ranging up to 1½ feet in depth. Additionally, up to 15 feet of undocumented fill may be present from the removal of former underground storage tanks, as mentioned above.

All 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, however we do not know the composition of the material used to backfill the excavations resulting from the removal of the USTs and surrounding soil. 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.

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.

6.3.1 Supplemental Exploratory Test Pits

As mentioned, compaction records for the backfill of the prior underground storage tank excavations are not available for review. We recommend additional exploration be performed to further evaluate the undocumented fill material within the previous tank areas and define the lateral limits of over-excavation and re-compaction and depth of over-excavation within the building areas.

6.4 TEMPORARY CUT AND FILL SLOPES

The contractor is responsible for maintaining all temporary slopes and providing temporary shoring where required. Temporary shoring, bracing, and cuts/fills should be performed in accordance with the strictest government safety standards. On a preliminary basis, the upper 10 feet at the site may be classified as OSHA Site 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.

6.5 SUBGRADE PREPARATION

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

6.6 WET SOIL STABILIZATION GUIDELINES

Native soil and fill materials, especially soils with high fines contents such as clays and silty soils, can become unstable due to high moisture content, whether from high in-situ moisture contents or from winter rains. As the moisture content increases over the laboratory optimum, it becomes more likely the materials will be subject to softening and yielding (pumping) from construction loading or become unworkable during placement and compaction.

As discussed in the “Subsurface” section in this report, the in-situ moisture contents can range up to 6 percent over the estimated laboratory optimum in the upper 15 feet of the soil profile. The contractor should anticipate drying the soils prior to reusing them as fill. In addition, repetitive rubber-tire loading will likely de-stabilize the soils.

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

6.6.1 Scarification and Drying

The subgrade may be scarified to a depth of 6 to 12 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.

6.6.2 Removal and Replacement

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

6.6.3 Chemical Treatment

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

6.7 MATERIAL FOR FILL

6.7.1 Re-Use of On-site Soils

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

6.7.2 Re-Use of On-Site Site Improvements

We anticipate that moderate quantities of asphalt concrete (AC) grindings and aggregate base (AB) and some Portland Cement Concrete (PCC) will be generated during site demolition. If the AC grindings are mixed with the underlying AB to meet Class 2 AB specifications, they may be reused within the new pavement and flatwork structural sections, but AC grindings may not be reused within the habitable building areas. Laboratory testing will be required to confirm the grindings meet project specifications. Due to the existing alligator cracking of the AC pavements, it is likely that the grinding operation will leave significant oversize chunks and will not likely meet the Class 2 AB gradation requirements but may meet Caltrans subbase requirements. Depending on the quantities of oversized material, the grindings may still be used within the structural section; however, the pavement design will need to be modified to account for the difference, typically resulting in the addition of about 1 inch to the structural section.

If the site area allows for on-site pulverization of PCC and provided the PCC is pulverized to meet the “Material for Fill” requirements of this report, it may be used as select fill within the habitable building areas, excluding the capillary break layer; as typically pulverized PCC comes close to or meets Class 2 AB specifications, the recycled PCC may likely be used within the pavement structural sections. PCC grindings also make good winter construction access roads, similar to a cement-treated base (CTB) section.

6.7.3 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.

6.8 COMPACTION REQUIREMENTS

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

Table 2: Compaction Requirements

Description	**Material Description**	**Minimum Relative[1] Compaction (percent)**	**Moisture[2] Content (percent)**
General Fill (within upper 5 feet)	On-Site Soils	90	>1
General Fill (below a depth of 5 feet)	On-Site Soils	95	>1
Fill Beneath Footings	Low Expansion Soils	95	>1
Trench Backfill	On-Site Soils	90	>1
Trench Backfill (upper 6 inches of subgrade)	On-Site 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 Soils	90	>1
Flatwork Aggregate Base	Class 2 Aggregate Base[3]	90	Optimum

Table 2 continues

Table 2: Compaction Requirements (Continued)

Description	Material Description	Minimum Relative[1] Compaction (percent)	Moisture[2] Content (percent)
Pavement Subgrade	On-Site Soils	95	>1
Pavement Aggregate Base	Class 2 Aggregate Base[3]	95	Optimum

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 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.

6.10 SITE DRAINAGE

6.10.1 Surface Drainage

Ponding should not be allowed 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. However, 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.

6.11 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 ground water is mapped at a depth of 12 feet, and therefore is expected to be within 10 feet of the base of the infiltration measure.

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

SECTION 7: 2019 CBC SEISMIC DESIGN CRITERIA

7.1 SEISMIC DESIGN CRITERIA

We developed site-specific seismic design parameters in accordance with Chapter 16, Chapter 18 and Appendix J of the 2019 California Building Code (CBC) and Chapters 11, 12, 20, and 21 and Supplement No. 1 of ASCE 7-16.

7.1.1 Site Location and Provided Data For 2019 CBC Seismic Design

The project is located at latitude 37.566567° and longitude -122.321192°, which is based on Google Earth (WGS84) coordinates at the approximate center of site in San Mateo, California. We have assumed that a Seismic Importance Factor (I_e) of 1.00 has been assigned to the structure in accordance with Table 1.5-2 of ASCE 7-16 for structures classified as Risk Category II. The building period has not been provided by the project structural engineer.

7.2 2019 CBC SEISMIC DESIGN CRITERIA

As discussed in the "Subsurface" of our report, our exploratory borings encountered dense sands and stiff to hard clays deposits to a depth of 60 feet, the maximum depth explored.

Based on our experience in the project vicinity, geologic mapping, and the alluvial soils encountered within our exploratory borings, we have classified the site as Soil Classification D, which is described as a "stiff soil" profile. Because we used site specific data from our explorations and laboratory testing, the site class should be considered as "determined" for the purposes of estimating the seismic design parameters from the code. Our site-specific ground motion hazard analysis considered a V_{S30} of 280 m/s (918 ft/s).

In accordance with Section 11.4.8 of ASCE 7-16, we performed a ground motion hazard analysis following Chapter 21, Section 21.2 of ASCE 7-16. We evaluated both Probabilistic MCE_R Ground Motions, in accordance with Method 1 and Method 2, and Deterministic MCE_R Ground Motions to generate our recommended design response spectrum for the project, see Figure 5. The recommended design spectral accelerations and associated periods are provided in graphically on Figure 6.

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 natural, undisturbed soil or engineered fill, 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.

Footings constructed to the above dimensions and in accordance with the "Earthwork" recommendations of this report are capable of supporting maximum allowable bearing pressures of 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 3: Assumed Structural Loading

Foundation Area	Range of Assumed Loads
Interior Isolated Column Footing	600 to 650 kips
Exterior Isolated Column Footing	300 to 325 kips
Perimeter Strip Footing	20 to 22 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 ⅔-inch, with about ⅓-inch of post-construction differential settlement between adjacent foundation elements, assumed to be on the order of 30 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.

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 0.40 applied to the footing dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 400 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

9.1 INTERIOR SLABS-ON-GRADE

As the Plasticity Index (PI) of the surficial soils is 15 or less, the proposed slabs-on-grade may be supported directly on 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 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 near 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 INTERIOR SLABS MOISTURE PROTECTION CONSIDERATIONS

The following general guidelines for concrete slab-on-grade construction where floor coverings are planned are presented for the consideration by the developer, design team, and contractor. These guidelines are based on information obtained from a variety of sources, including the American Concrete Institute (ACI) and are intended to reduce the potential for moisture-related problems causing floor covering failures, and may be supplemented as necessary based on project-specific requirements. The application of these guidelines or not will not affect the geotechnical aspects of the slab-on-grade performance.

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

Sieve Size	Percentage Passing Sieve
1”	100
¾”	90 – 100
No. 4	0 – 10
No. 200	0 – 5

- 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.

9.3 EXTERIOR FLATWORK

9.3.1 Pedestrian Concrete Flatwork

Exterior concrete flatwork subject to pedestrian and/or occasional light pick up loading should be at least 4 inches thick and supported on at least 4 inches of Class 2 aggregate base overlying subgrade prepared in accordance with the “Earthwork” recommendations of this report. Flatwork that will be subject to heavier or frequent vehicular loading should be designed in accordance with the recommendations in the “Vehicular Pavements” section below. To help reduce the potential for uncontrolled shrinkage cracking, adequate expansion and control joints should be included. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness. Flatwork should be isolated from adjacent foundations or retaining walls.

SECTION 10: VEHICULAR PAVEMENTS

10.1 ASPHALT CONCRETE

Patching of existing asphalt concrete pavements in the public right-of-way should match in kind the existing structural section, or conform to a minimum section provided by the City. If structural sections are desired, we can provide those once the City provides their criteria and requirements.

10.2 PORTLAND CEMENT CONCRETE

Portland Cement Concrete (PCC) driveway entrances to the site in the public right-of-way should be designed and constructed in accordance with City requirements. Any portion of a concrete driveway on grade within private property should have a structural section of at least 6 inches of concrete overlying at least 6 inches of Class 2 aggregate base compacted as recommended in the “Earthwork” section. The concrete should have a compressive strength of at least 3,500 psi and be laterally restrained with curbs or concrete shoulders. Adequate expansion and control joints should be included. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness. If there is an at-grade concrete trash enclosure slab where the large dumpsters are stored, it should be at least 8 inches thick overlying at least 6 inches of Class 2 aggregate base.

10.3 EXTERIOR FLATWORK

Exterior concrete flatwork subject to pedestrian and/or occasional light pick up loading should be at least 4 inches thick and supported on at least 6 inches of Class 2 aggregate base overlying subgrade prepared in accordance with the “Earthwork” recommendations of this report. To help reduce the potential for uncontrolled shrinkage cracking, adequate expansion and control joints should be included. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness.

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 with level backfill be designed for the following pressures:

Table 6: Recommended Lateral Earth Pressures

Wall Condition	Lateral Earth Pressure*	Additional Surcharge Loads
Unrestrained – Cantilever Wall	45 pcf	⅓ of vertical loads at top of wall
Restrained – Braced Wall	45 pcf + 8H** psf	½ of vertical loads at top of wall

* Lateral earth pressures are based on an equivalent fluid pressure for level backfill conditions
** 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.

11.2 SEISMIC LATERAL EARTH PRESSURES

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

11.2 WALL DRAINAGE

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

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

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

11.3 BACKFILL

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

11.4 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.

SECTION 12: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of Windy Hill Property Ventures specifically to support the design of the 435 E 3rd Office Development project in San Mateo, 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 ground water 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.

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

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

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

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

Recommendations provided in this report are based on the assumption that Cornerstone will be retained to provide observation and testing services during construction to confirm that conditions are similar to that assumed for design, and to form an opinion as to whether the work has been performed in accordance with the project plans and specifications. If we are not retained for these services, Cornerstone cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of Cornerstone's report by others. Furthermore, Cornerstone will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services.

SECTION 13: REFERENCES

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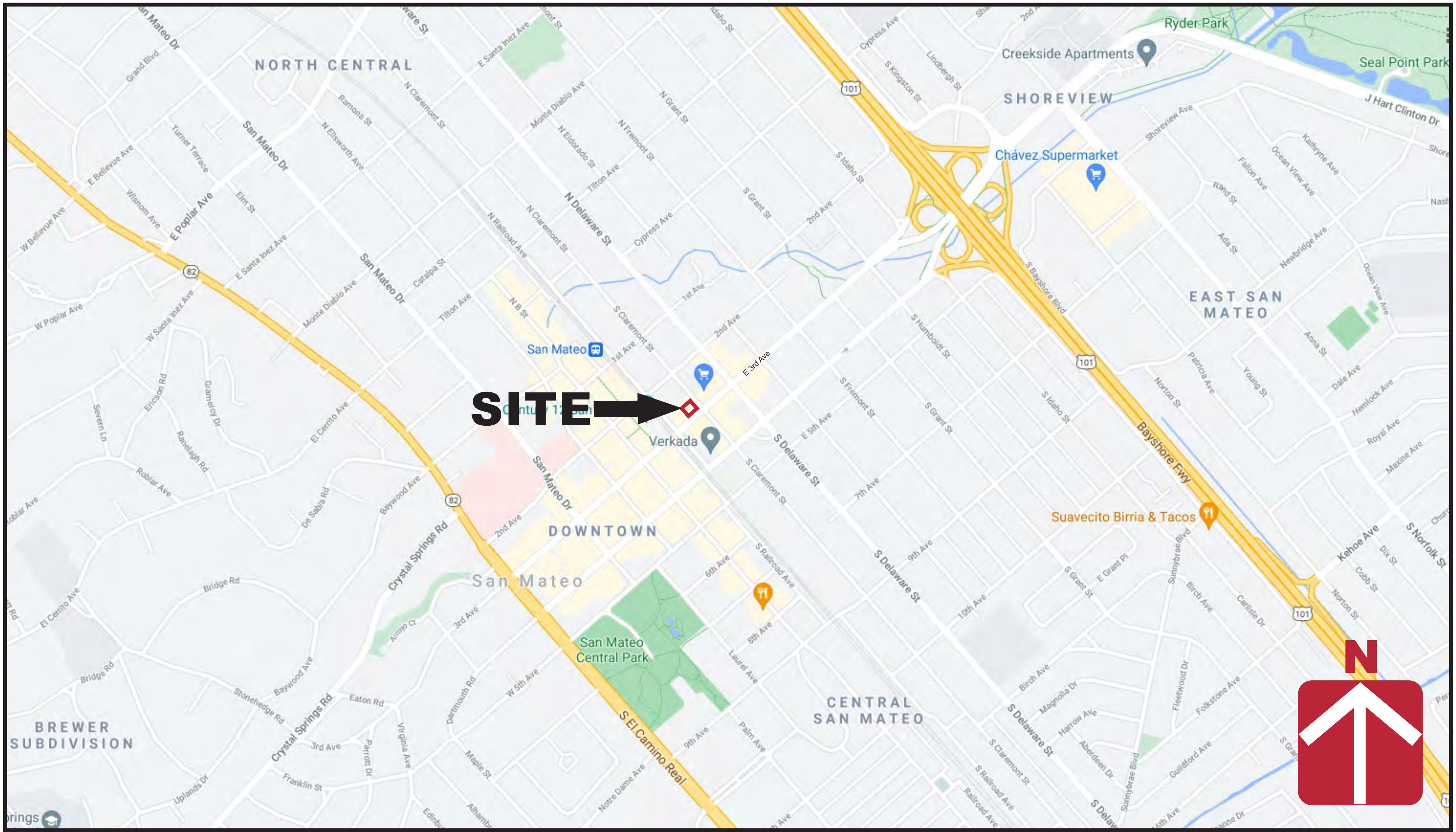


CORNERSTONE EARTH GROUP

Vicinity Map

435 East 3rd Mixed-Use Development
San Mateo, CA

Project Number
803-10-1

Figure Number
Figure 1

Date
August 2021

Drawn By
RRN

Legend

⌖ Approximate location of exploratory boring (EB)

0 20 40
APPROXIMATE SCALE (FEET)

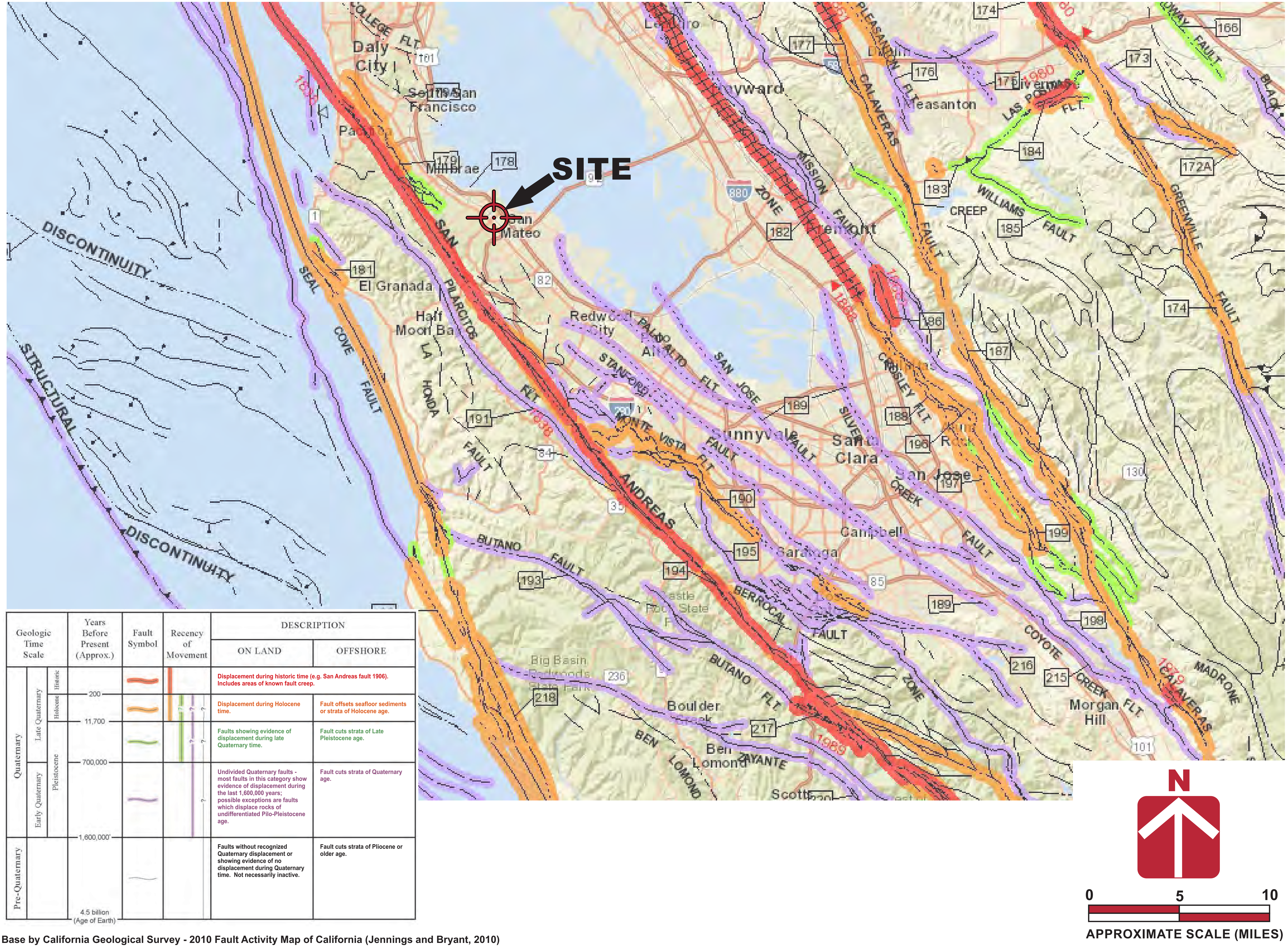
Base by Google Earth, dated 09/26/2020
Overlay by Architectural Technologies, Site Plan - A 1.01, dated 05/14/2021

Project Number	803-10-1
Figure Number	Figure 2
Drawn By	RRN
Date	August 2021

Site Plan

435 East 3rd Mixed-Use Development
San Mateo, CA

CORNERSTONE
EARTH GROUP



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION	
						ON LAND	OFFSHORE
Quaternary	Late Quaternary	Historic		🟥		Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
			200				
		Holocene		🟧	? ? ?	Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
			11,700				
		Pleistocene		🟩	? ?	Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
			700,000				
	Early Quaternary			🟪	?	Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Pilo-Pleistocene age.	Fault cuts strata of Quaternary age.
			1,600,000				
Pre-Quaternary				—		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
803-10-1

Figure Number
Figure 3

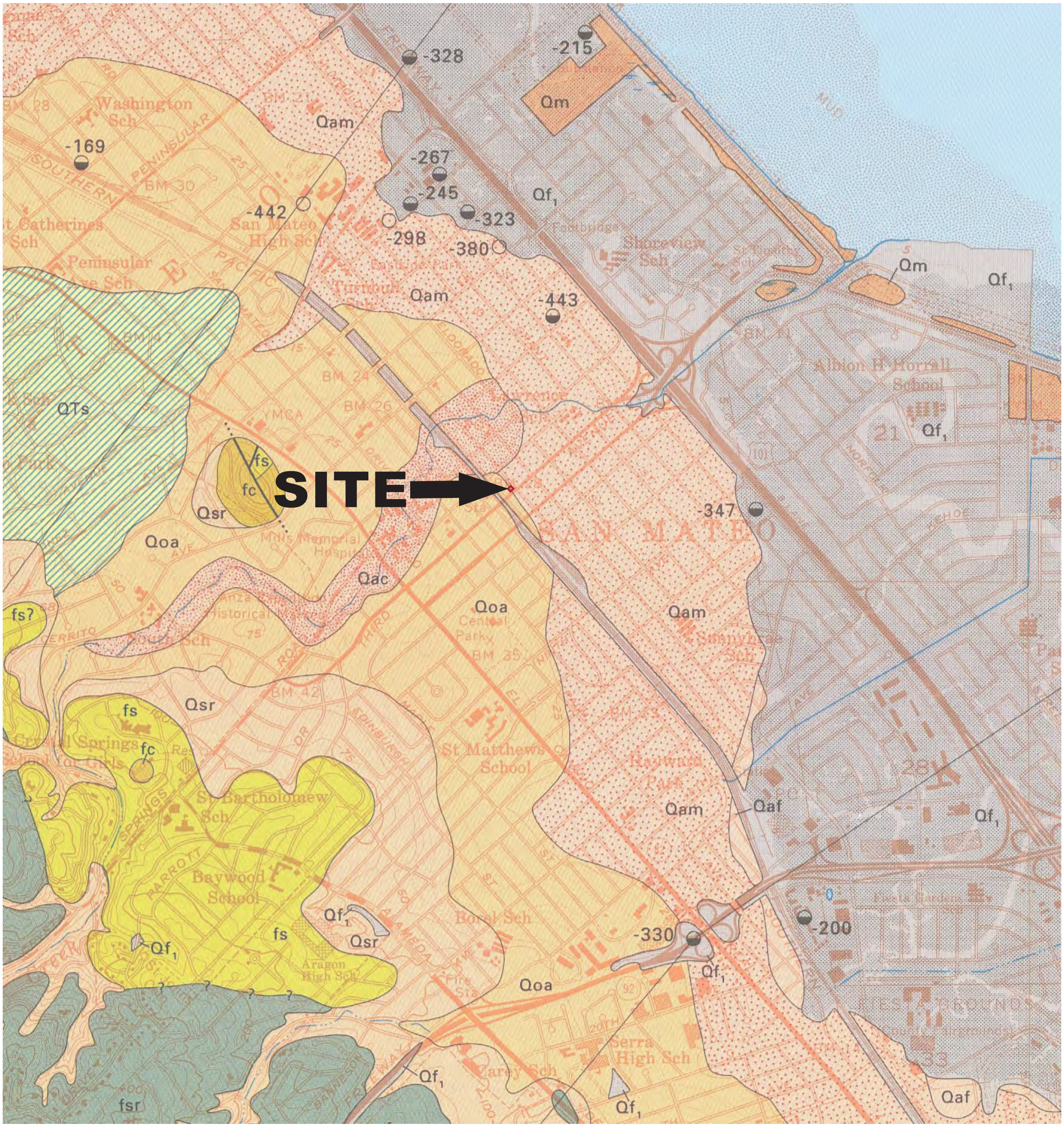
Date
August 2021

Drawn By
RRN

Regional Fault Map

435 East 3rd Mixed-Use Development
San Mateo, CA

CORNERSTONE
EARTH GROUP



Geologic Units

Qam	Medium-grained alluvium (Holocene)
Qac	Coarse-grained alluvium (Holocene)
Qf$_1$	Artificial fill (Holocene)
Qoa	Older alluvium (Pleistocene)

Explanation

Contact- dashed where approximate, dotted where concealed

N

0 2000 4000

APPROXIMATE SCALE (FEET)

Base: Geologic Map of the Montara Mountain and San Mateo 7.5'
Quadrangles, San Mateo County, California, by Pampeyan, 1994

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Vicinity Geologic Map

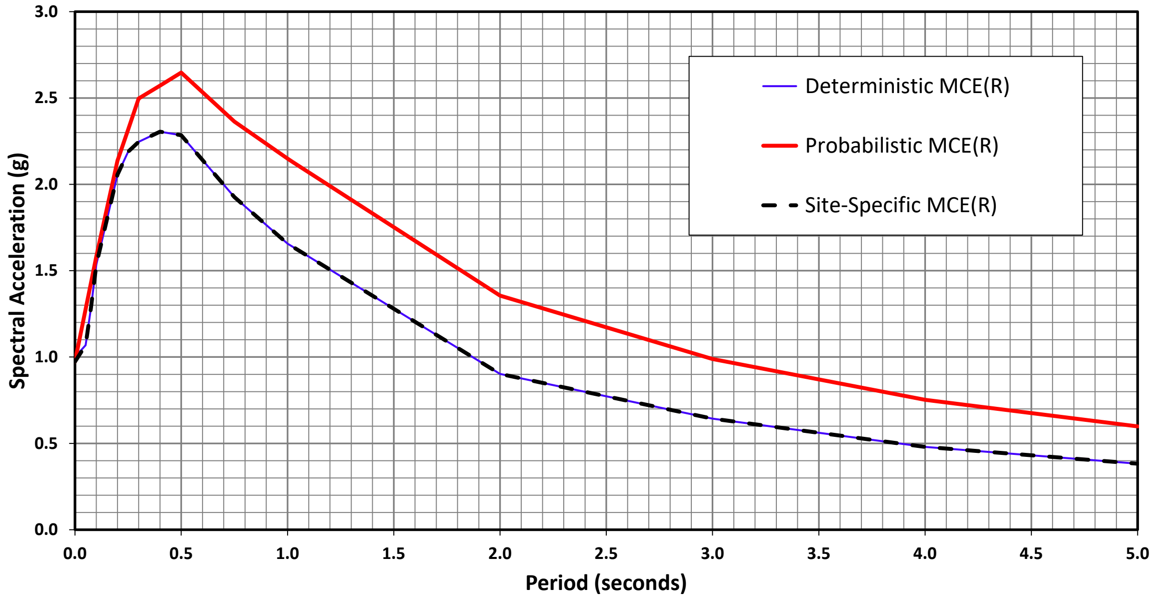
435 East 3rd Mixed-Use Development
San Mateo, CA

Project Number
803-10-1

Figure Number
Figure 4

Date
August 2021

Drawn By
RRN



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

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

Site-Specific MCE_R	
Period (Seconds)	Spectral Acceleration (g)
0.00	0.974
0.05	1.068
0.08	1.285
0.10	1.535
0.20	2.056
0.21	2.070
0.25	2.189
0.30	2.246
0.40	2.305
0.50	2.285
0.75	1.926
1.00	1.657
1.03	1.638
2.00	0.903
3.00	0.643
4.00	0.479
5.00	0.383

References:

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

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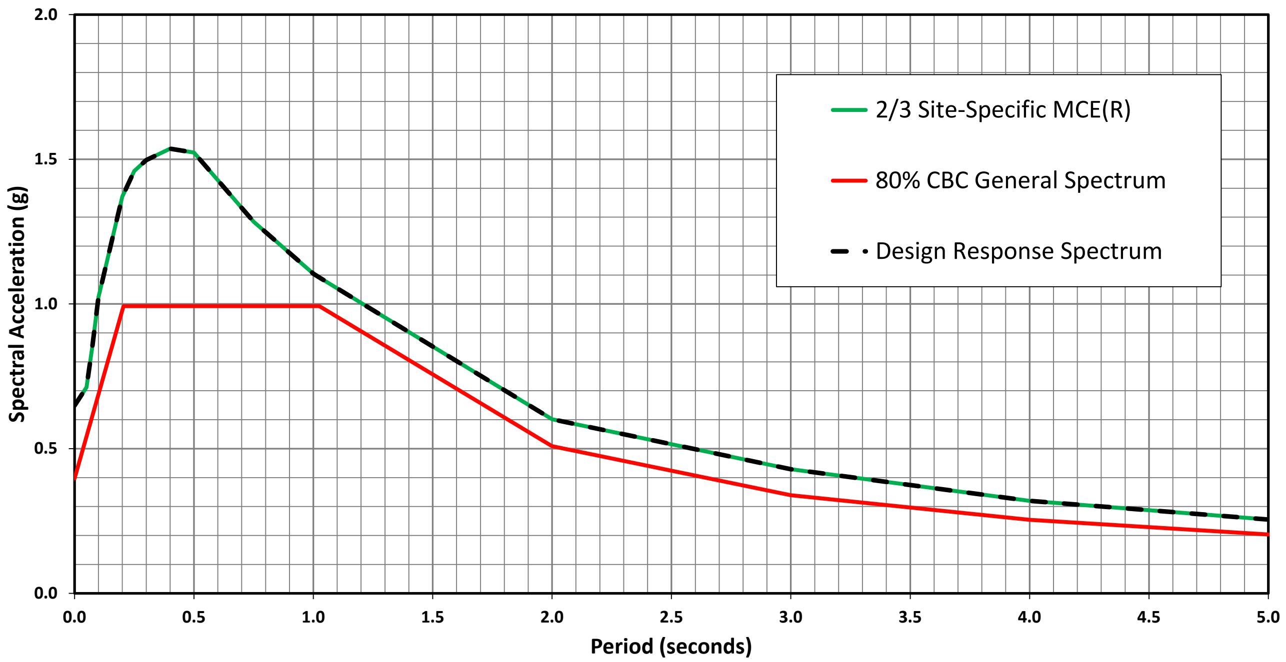
MCE_R RESPONSE SPECTRA

435 East 3rd Mixed-Use Development
435 E 3rd Avenue
San Mateo, CA

FIGURE 5

PROJECT NO. 803-10-1

August 10, 2021 | SCO



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

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

Design Response Spectra	
Period (Seconds)	Spectral Acceleration (g)
0.00	0.649
0.05	0.712
0.08	0.857
0.10	1.023
0.20	1.371
0.21	1.380
0.25	1.459
0.30	1.497
0.40	1.537
0.50	1.523
0.75	1.284
1.00	1.105
1.03	1.092
2.00	0.602
3.00	0.429
4.00	0.320
5.00	0.255

Site Design	Design Values
Site Class (Per Chapter 20 ASCE 7-16)	D
Shear Wave Velocity, V_{S30} (m/sec)	280
Site Latitude (degrees)	37.566567
Site Longitude (degrees)	-122.321192
Risk Category	II
Building Period (sec)	Unknown
Importance Factor, I_e	1
[1]Site Specific PGA_M (g)	0.89

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

Design Acceleration Parameters[1]	
S_{DS}	1.383
S_{D1}	1.286
S_{MS}	2.074
S_{M1}	1.930

References:

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

CORNERSTONE EARTH GROUP

DESIGN RESPONSE SPECTRA

435 East 3rd Mixed-Use Development
435 E 3rd Avenue
San Mateo, CA

FIGURE 6

PROJECT NO. 803-10-1

August 10, 2021 | SCO

APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, hollow-stem auger drilling equipment. Two 8-inch-diameter exploratory borings were drilled on July 24, 2021 to depths of 40 to 60 feet. The approximate locations of exploratory borings are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Boring logs, as well as a key to the classification of the soil and bedrock, are included as part of this appendix.

Boring locations were approximated using existing site boundaries, and other site features as references. Boring elevations were based on interpolation of plan contours were not determined. The locations of the borings should be considered accurate only to the degree implied by the method used.

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

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

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

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-10)

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

OTHER MATERIAL SYMBOLS

▨	Poorly-Graded Sand with Clay	░	Sand
▨	Clayey Sand	▥	Silt
▥	Sandy Silt	⁘	Well Graded Gravelly Sand
▩	Artificial/Undocumented Fill	▥	Gravelly Silt
⁘	Poorly-Graded Gravelly Sand	■	Asphalt
░	Topsoil	●	Boulders and Cobble
●	Well-Graded Gravel with Clay		
●	Well-Graded Gravel with Silt		

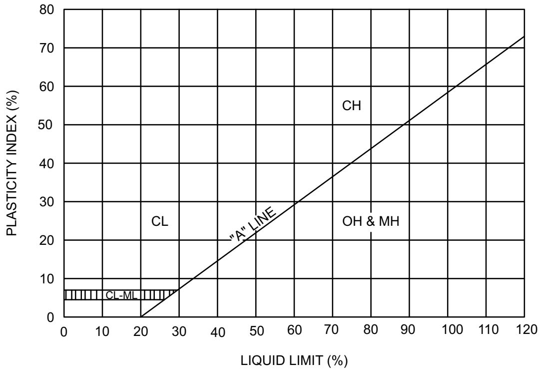
SAMPLER TYPES

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

ADDITIONAL TESTS

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

PLASTICITY CHART



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

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

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

CORNERSTONE
EARTH GROUP

LEGEND TO SOIL DESCRIPTIONS

Figure Number
A-1

CORNERSTONE EARTH GROUP

BORING NUMBER EB-1

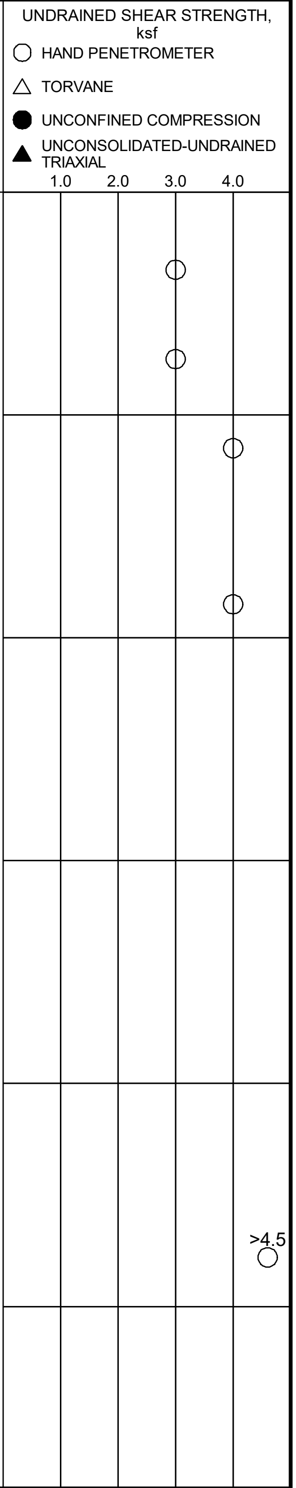
PAGE 1 OF 2

PROJECT NAME 435 East 3rd Avenue Office Development
PROJECT NUMBER 803-10-1
PROJECT LOCATION San Mateo, CA
DATE STARTED 7/24/21 **DATE COMPLETED** 7/24/21
GROUND ELEVATION ____ **BORING DEPTH** 59.9 ft.
DRILLING CONTRACTOR Exploration Geoservices Inc.
LATITUDE 37.566752° **LONGITUDE** -122.321154°
DRILLING METHOD Mobile B-61, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY CRS
▽ **AT TIME OF DRILLING** 28.5 ft.
NOTES ____
▼ **AT END OF DRILLING** 27 ft.

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

DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
1 inch asphalt concrete over 4 inches aggregate base							
Sandy Lean Clay (CL) [Fill] very stiff, moist, brown, fine to medium sand, low plasticity	47	MC-1C	114	15			
Sandy Lean Clay (CL) very stiff, moist, dark brown to brown, fine to medium sand, some fine subrounded to subangular gravel, low plasticity	61	MC-2B	117	15			
Lean Clay with Sand (CL) hard, moist, reddish brown, fine to medium sand, some subrounded to subangular gravel, moderate plasticity	74	MC-3B	109	19			
	72	MC-4B	106	20			
	50 6"	MC-5B	120	13			
Clayey Sand with Gravel (SC) very dense, moist, brown, fine to coarse sand, fine to coarse subrounded to subangular gravel	50 5"	MC-6B	116	14			
Lean Clay with Sand (CL) hard, moist, reddish brown, fine to medium sand, some subrounded to subangular gravel, moderate plasticity	50 4"	MC-7B	108	20			>4.5

Continued Next Page



CORNERSTONE EARTH GROUP

BORING NUMBER EB-1

PAGE 2 OF 2

PROJECT NAME 435 East 3rd Avenue Office Development

PROJECT NUMBER 803-10-1

PROJECT LOCATION San Mateo, CA

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

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

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
∇		▨	**Sandy Lean Clay (CL)** hard, moist, reddish brown, fine to coarse sand, some fine subrounded to subangular gravel, low plasticity	50/6"	MC-8B	112	18			>4.5 ○
	30									
			becomes very stiff	45	SPT-9		22			○
	35									
				58	SPT-10		19			○
	40									
		▨	**Clayey Sand with Gravel (SC)** very dense, moist, brown, fine to coarse sand, fine subrounded to subangular gravel							
				62	SPT-11		18			
	45									
				50/3"	SPT-12		13			
	50									
				83	SPT-13		20			
	55									
				50/5"	SPT-14		16			
	60		Bottom of Boring at 59.9 feet.							

BORING NUMBER EB-2

PAGE 1 OF 2

CORNERSTONE EARTH GROUP

PROJECT NAME 435 East 3rd Avenue Office Development
PROJECT NUMBER 803-10-1
PROJECT LOCATION San Mateo, CA
DATE STARTED 7/24/21 **DATE COMPLETED** 7/24/21
GROUND ELEVATION **BORING DEPTH** 40 ft.
DRILLING CONTRACTOR Exploration Geoservices Inc.
LATITUDE 37.566574° **LONGITUDE** -122.321329°
DRILLING METHOD Mobile B-61, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY CRS
∇ **AT TIME OF DRILLING** 28.5 ft.
NOTES
▼ **AT END OF DRILLING** 27.5 ft.

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

DESCRIPTION

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
	0	1½ inches asphalt concrete over 4½ inches aggregate base							
		Sandy Lean Clay (CL) hard, moist, reddish brown, fine to medium sand, low plasticity	33	MC-1B	108	7			>4.5 ○
		Sandy Lean Clay (CL) hard, moist, brown, fine to medium sand, trace fine subrounded to subangular gravel, low plasticity Liquid Limit = 25, Plastic Limit = 14	50 5"	MC-2B	118	9	11		>4.5 ○
	5		50 6"	MC-3	109	10			
		Clayey Sand with Gravel (SC) very dense, moist, brown, fine to coarse sand, fine to coarse subrounded to subangular gravel							
	10		50 5"	MC-4	116	11			
			50 6"	MC-5	116	12			
	15								
			50 5"	MC-6B	111	14			
	20								
		Well Graded Sand with Clay and Gravel (SW-SC) very dense, moist, brown, fine to coarse sand, fine to coarse subrounded to subangular gravel							
			50 5"	MC-7B	121	12		9	
	25								

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

Continued Next Page

CORNERSTONE EARTH GROUP

BORING NUMBER EB-2

PAGE 2 OF 2

PROJECT NAME 435 East 3rd Avenue Office Development

PROJECT NUMBER 803-10-1

PROJECT LOCATION San Mateo, CA

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

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

1.0 2.0 3.0 4.0

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
		Well Graded Sand with Clay and Gravel (SW-SC) very dense, moist, brown, fine to coarse sand, fine to coarse subrounded to subangular gravel						
		Clayey Sand with Gravel (SC) dense to very dense, moist, brown, fine to coarse sand, fine to coase subrounded to subangular gravel	59	SPT-8B		13		
	30							
			50/5"	SPT-15		15		
	35			ST				
			75	SPT-17		18		
	40	Bottom of Boring at 40.0 feet.						
	45							
	50							
	55							

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.

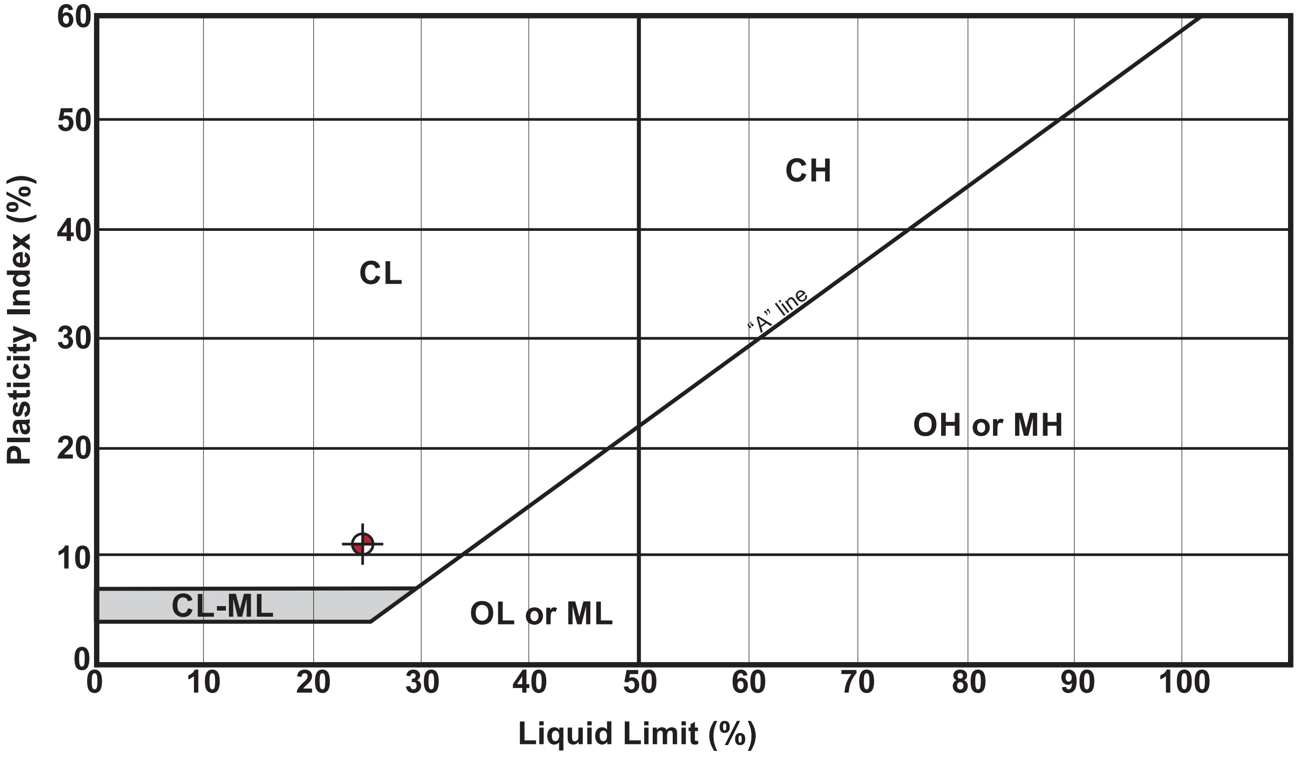
Moisture Content: The natural water content was determined (ASTM D2216) on 24 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

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

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

Plasticity Index: One Plasticity Index determination (ASTM D4318) was performed on a sample of the subsurface soil to measure the range of water contents over which this material exhibits plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential. Results of this test are shown on the boring log at the appropriate sample depth.

Plasticity Index (ASTM D4318) Testing Summary



Symbol	Boring No.	Depth (ft)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index	Passing No. 200 (%)	Group Name (USCS - ASTM D2487)
⊕	EB-2	3.5	9	25	14	11	—	Sandy Lean Clay (CL)

Samples prepared in accordance with ASTM D421

CORNERSTONE
EARTH GROUP

Plasticity Index Testing Summary

435 East 3rd Mixed-Use Development
San Mateo, CA

Project Number
803-10-1

Figure Number
Figure B1

Date
August 2021

Drawn By
FLL

APPENDIX C: CASE CLOSURE REPORT FOR REMOVED UNDERGROUND STORAGE TANKS AT 435 EAST 3RD AVENUE SAN MATEO CALIFORNIA

FILE NO: 2178.12

RB FILE NO: 41-0003

SAN MATEO COUNTY
SOIL ONLY CASE CLOSURE SUMMARY
LEAKING UNDERGROUND FUEL STORAGE TANKS PROGRAM

Response requested by

Not Applicable-Soil Only case

I. AGENCY INFORMATION

Address:	590 Hamilton Street, Redwood City, CA 94063
Project Manager:	Tanya Haeri-McCarroll
Title:	Hazardous Material Specialist
Telephone No.:	(650) 363-4957

II. CASE INFORMATION

Site Name: ACCU-TUNE

Site Address: 435 E. 3RD AVE.
SAN MATEO, CALIFORNIA

LUSTIS Case #:	Local Case #: 0003	
LOP Case #: 110048	URF Filing Date:	SWEEPS#:

Responsible Party Information

NAME	ADDRESS	PHONE #
Fred Strebel	25 Miranda Ct., Hillsborough, CA 94010	650-344-4905

Tank Information

TANK #	SIZE IN GALLON	CONTENTS	CLOSED IN PLACE/REMOVED	DATE
1	10,000	UNLEADED GASOLINE	REMOVED	JUL 1989
2	7,500	REGULAR GASOLINE	REMOVED	JUL 1989
3	5,000	PREMIUM GASOLINE	REMOVED	JUL 1989
4	280	WASTE OIL	REMOVED	JUL 1989

III. RELEASE AND SITE CHARACTERIZATION INFORMATION

Cause and Type of Release:

Site Characterization Complete? Yes

Date Approved by Oversight Agency: April 3, 1998

Number of Monitoring wells Installed: 0

Proper screened interval? NA

Highest GW depth BGS: NA Lowest Depth: NA

Flow Direction: Northeast
Most sensitive GW use: Drinking Water
Are Drinking Water affected? No   Aquifer Name: NA
Is Surface Water Affected? No   Nearest/Affected SW Name: NA
Off-Site Beneficial use Impacts(Location):
Report(s) on File? Yes   Where is it filed? 590 Hamilton St., Redwood City, CA 94063

Treatment and Disposal of Affected Material

MATERIAL	AMOUNT (INCLUDE UNITS)	TREATMENT OR DISPOSAL	DATE
Tank	4 tanks	disposed	1989
Piping	unknown	unknown	1989
Soil	unknown	disposed	1989

Maximum Documented Contaminant Concentrations - Before and After Cleanup

	SOIL (PPM)	
Contaminant	Before	After
Oil & Grease	94	94
TPH - G	13	13
Benzene	0.039	0.039
Toluene	0.035	0.035
Ethyl-benzene	0.092	0.092
Xylenes	0.170	0.170
MtBE	NA	NA
VOCs	NA	NA
Lead	NA	NA
Other metals	NA	NA

NA = NOT ANALYZED

COMMENTS: The release appears to be insignificant. Metals and VOCs were not required to be analyzed in 1989.

IV. CLOSURE

Does completed corrective action protect existing beneficial uses per the Regional Board Basin Plan? YES

Does completed corrective action protect potential beneficial uses per the Regional Board Basin Plan? YES

Does corrective action protect public health for current land use? YES

Site Management Requirements: NO

Should corrective action be reviewed if land use changes? NO

Monitoring Wells Decommissioned? Number Decommissioned: 0
Number Retained: 0

List Enforcement Actions Taken:

List Enforcement Action Rescinded:

V. RWQCB Notification

Date Submitted to RB:	RB Response:
RB Staff: Randy Lee	Title: Water Resource Control Engineer

Comments: This is a soil only case.

Director, Environmental Health Services

APR 0 3 1998
Date

UNDERGROI ⁻⁻⁼ TANK PROGRAM OFFICIAL INSP⁻⁻ TION REPORT

COUNTY NAME San Mateo **COUNTY #** 41

SITE NAME: Accu-Tune INSPECTION DATE: 7/10/89

SITE ADDRESS: 435 E. 3rd ave CITY/STATE/ZIP San Mateo, CA 94402

	TANK 10 K	TANK 5K	TANK 7½ K	TANK 280 g
CHANGES SITE/OWNER/PERMIT? YES [✓ NO] FORM A AND/OR B SUBMITTED? NA	COMPUTER NUMBER	COMPUTER NUMBER	COMPUTER NUMBER	COMPUTER NUMBER
TYPE OF INSPECTION Final | SITE COMPUTER #	PER # EXP. DATE	PER # EXP. DATE	PER # EXP. DATE	PER # EXP. DATE

OPERATIONAL TANK

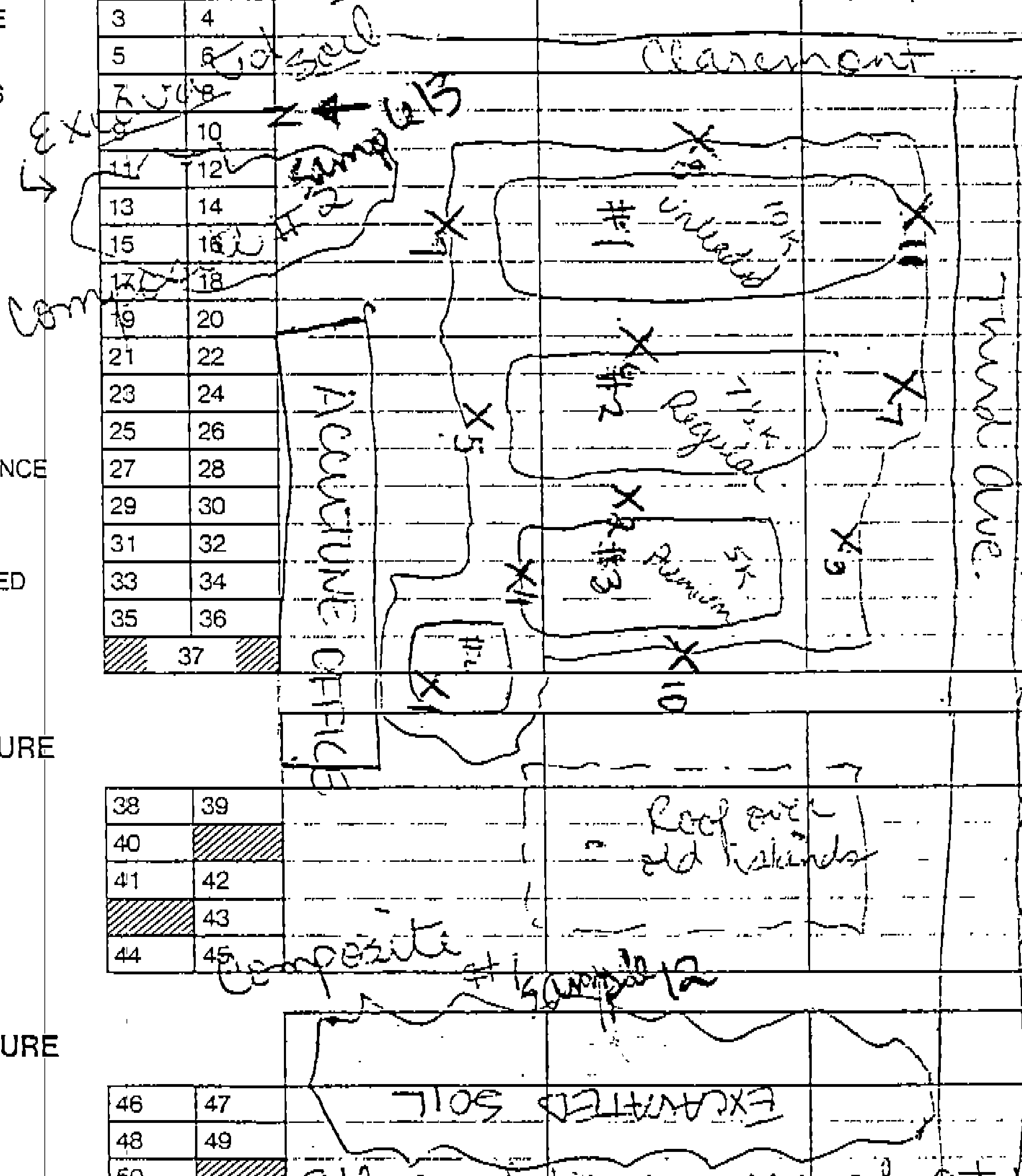
	TANK CONTENTS MAJOR	MINOR				WASTE OIL
PERMIT TO OPERATE	1	2				
CHANGE IN CONDITIONS TO OPERATE	3	4				
APPROVED CONSTRUCTION	5	6				
WRITTEN MONITORING PROCEDURES	7	8				
APPROVED MONITORING SYSTEM	9	10				
MONITORING SYSTEM	11	12				
APPROVED MONITOR FREQUENCY	13	14				
MONITORING RECORDS MAINTAINED	15	16				
ACCESS CASING SECURED	17	18				
PIPING	19	20				
INVENTORY RECONCILIATION	21	22				
TANK GAUGING	23	24				
APPROVED RESPONSE PLAN	25	26				
UNAUTHORIZED RELEASE OCCURRENCE	27	28				
SAMPLING	29	30				
APPROVED TANK REPAIRS	31	32				
UNAUTHORIZED RELEASES REPORTED	33	34				
SAFETY HAZARD	35	36				
CONDITIONS ABATED	▨ 37 ▨					

TEMPORARY TANK CLOSURE

REMOVAL OF RESIDUAL	38	39
FLAMMABLE VAPORS REMOVED	40	▨
ACCESS LOCATIONS SEALED	41	42
POWER DISCONNECTED	▨	43
OWNER/OPERATOR MONITORING	44	45

PERMANENT TANK CLOSURE

REMOVAL OF RESIDUAL MATERIALS	46	47
PIPING	48	49
FLAMMABLE VAPORS REMOVED	50	▨
UNAUTHORIZED RELEASE	51	52
SAMPLING	53	54
IMPROPER ABANDONMENT	55	56



X1 = sample in native soil under ~10 tank at 8 ft
X2 = sample in native soil 14 ft
X3 = sample taken in side at ~~5~~ ft (5K) 14' 6"
X4 = side wall sample at 13'
X5 = side wall sample at 14'. (7.5K N end)
X6 = ~~sample~~ sample taken from pit bottom at 15 ft
X7 = side wall sample taken at 15 ft.
X8 = side wall sample at 14'
X9 = side wall sample at 15'
X10 = side wall sample at 14'
X11 = side wall at

All conditions normal at time of removal Tank #3 (premium) rusty with several (~10 holes soil discolored and odorous. Tank #2 rusty with several holes (2 side seam, 3

SYSTEM STATUS (MUST MARK ONE)

MAJOR VIOL. 57 ☐ MINOR VIOL. 58 ☐ NO VIOL. 59 ☐

THE MARKED ITEMS REPRESENT VIOLATIONS AND MUST BE CORRECTED AS FOLLOWS: Tank #1 removed and in good condition with no overt holes Tank #4 removed (waste oil) rusty with no overt holes. No water observed in pit. Composite sample from excavated soil to be taken tomorrow (7/11/89) Two composites taken from both piles of excavated soil on 7/11/89

OFFICE: San Mateo County Environmental Health

INSP: Bilaskio / Gomar

TITLE: Hazardous Materials Specialists

RECEIVED BY: X Chuck

PHONE: 363- 318

RECK:

1087

Approximate location of previously removed UST

New Tank

COUNTY OF SAN MATEO

HEALTH SERVICES AGENCY

April 6, 1998

Fred Strebel
25 Miranda Ct.
Hillsborough, CA 94010

SMCo Site# 110048

SUBJECT: CASE CLOSURE OF ONE 10,000 GALLON UNLEADED GASOLINE, ONE 7,500 GALLON REGULAR GASOLINE, ONE 5,000 GALLON PREMIUM GASOLINE, AND ONE 280 GALLON WASTE OIL UNDERGROUND STORAGE TANK AT ACCU-TUNE, 435 EAST 3RD AVENUE, SAN MATEO, CALIFORNIA

Dear Mr. Strebel:

This letter confirms the completion of site investigation and remedial action for the underground storage tanks formerly located at the above-described location. Thank you for your cooperation throughout this investigation. Your willingness and promptness in responding to our inquiries concerning the former underground storage tanks are greatly appreciated.

Based on the information in the above referenced file and with the provision that the information provided to this agency was accurate and representative of site conditions, no further action related to the underground tank release is required.

This notice is issued pursuant to a regulation contained in Section 2721(e) of Title 23, California Code of Regulations.

Please contact Tanya Haeri-McCarroll at (650) 363-4957 if you have any questions regarding this matter.

Sincerely,

Brian J. Zamora, MPH, REHS
Director, Environmental Health and Public Health

cc: RWQCB
SWRCB

PUBLIC HEALTH AND ENVIRONMENTAL PROTECTION DIVISION
Board of Supervisors: Ruben Barrales • Richard S. Gordon • Mary Griffin • Tom Huening • Michael D. Nevin • **Health Services Director:** Margaret Taylor
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