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GEOTECHNICAL REPORT ON

EAST PALO ALTO TRASH CAPTURE DEVICE
NEWBRIDGE STREET AND WILLOW ROAD
EAST PALO ALTO, CALIFORNIA

by
Haley & Aldrich, Inc.
San Jose, California

for
Schaaf and Wheeler
Santa Clara, California

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1. Introduction

1.1 GENERAL

Haley & Aldrich, Inc. (Haley & Aldrich) has provided geotechnical design services to Schaaf and Wheeler for the City of East Palo Alto Trash Capture Device (TCD) Project in East Palo Alto, California. Our work has been completed to provide geotechnical recommendations for the design and construction of the new TCD.

1.2 PROJECT DESCRIPTION

The project consists of installing a new TCD associated with an existing storm drain system beneath Newbridge Street, between Willow Road and Saratoga Avenue, located in East Palo Alto, California. Planned improvements will include a new TCD that will tie into the existing storm drain system beneath the westbound lanes on Newbridge Street. The proposed TCD is a Debris Separating Baffle Box (DSBB) unit and will have a total structure weight of approximately 179,000 pounds. The excavation depth required for the device installation is anticipated to be approximately 17.3 feet below the ground surface (bgs).

1.3 PURPOSE AND SCOPE OF SERVICES

The purpose of Haley & Aldrich's geotechnical investigation was to assess the surface and subsurface soil and groundwater conditions in the immediate vicinity of the proposed TCD and to provide geotechnical recommendations for design and construction. The scope of work completed for the geotechnical investigation and report included:

- consulting and coordinating with Schaaf and Wheeler staff;
- performing reconnaissance to observe current site conditions and to mark for Underground Service Alert (USA);
- organizing a subsurface exploration program consisting of one boring using a truck-mounted drill rig;
- ordering laboratory testing of selected soil samples to determine key engineering properties;
- performing engineering analysis; and
- developing geotechnical design recommendations and preparation of this report.

2. Site Description

2.1 SITE DESCRIPTION

The project site is located in a relatively flat area of East Palo Alto on the southeastern part of the San Francisco Peninsula, just southwest of the San Francisco Bay (Figure 1). The site itself is located in the westbound lanes of Newbridge Street, between Willow Road and Saratoga Avenue. In this area, Newbridge Street is asphalt-paved and bounded by commercial and residential properties to the north, east, and south, and by Willow Road to the west. The proposed location of the new TCD is shown on Figure 2.

The ground surface is at approximately an elevation of 15 feet with respect to the North American Vertical Datum of 1988 (NAVD88).

2.2 INFORMATION PROVIDED

Before beginning work, the Schaaf and Wheeler team provided Haley & Aldrich with a standard detail of the DSBB, the location and depth of the existing sewer system, and a utility map of the area surrounding the project site.

3. Geologic Conditions

3.1 REGIONAL GEOLOGIC SETTING

The project site lies within the Coast Range geomorphic province of California. This province is characterized by northwest-southeast trending mountain ranges and intervening valleys, such as that occupied by San Francisco Bay and Santa Clara Valley. The site is located on very gently sloping land near the southeastern margin of San Francisco Bay, with the Diablo Range to the northeast and the Santa Cruz Mountains to the southwest.

3.2 SITE GEOLOGY

The geologic setting is shown on the Regional Geology Map, Figure 3. The distribution of geologic materials underlying the site has much to do with the site's vicinity to the San Francisco Bay and the Santa Cruz Mountains.

Geologic mapping by Witter and others (2006) shows the site as being underlain by alluvial fan sediments. Their detailed mapping shows geologic contact along and parallel to the centerline of Newbridge Street. The north portion of the road, where the TCD will be installed, is mapped as being underlain by fine-grained alluvial fan deposits (Holocene in age), which primarily consist of clays and silts with interbeds of sand and gravel. The southern portion is mapped as being underlain by alluvial fan deposits (Holocene), which generally include greater ranges of sediments of gravels, sands, silts, and clays that are poorly sorted and moderately to poorly bedded (Witter and others, 2006). Earlier mapping by Brabb and others (1998) shows the site as being underlain by floodplain deposits (Holocene in age) that are described as "medium- to dark-gray, dense, sandy to silty clay".

3.3 SEISMICITY

The project site is located within the greater San Francisco Bay Area, which is recognized as one of California's more seismically active regions. The seismic activity in this region results from the complex movements along the transform boundary between the Pacific Plate and the North American Plate. Along this transform boundary, the Pacific Plate is slowly moving to the northwest relative to the more stable North American Plate at approximately 40 millimeters per year in the Bay Area (Page, 1992). The differential movements between the two crustal plates caused the formation of a series of active fault systems within the transform boundary. The transform boundary between the two plates extends across a broad zone of the North American Plate, within which right-lateral strike-slip faulting predominates. In this broad transform boundary, the San Andreas fault accommodates less than half of the average total relative plate motion. Much of the remainder of the motion in the Bay Area is distributed across faults such as the Hayward, Calaveras, San Gregorio, Monte Vista-Shannon, Sargent, Greenville, Green Valley, West-Napa fault zones, etc.

Due to the project site's location in the seismically active San Francisco Bay Area, it will likely experience strong ground shaking from a large (Moment Magnitude [Mw] 6.7) or greater earthquake along one or more of the nearby active faults during the design lifetime of the project (Working Group on California Earthquake Probabilities [WGCEP], 2003; 2013). It should be noted that the third Uniform California Earthquake Rupture Forecast (UCERF3) time-independent model supports a magnitude-dependent methodology that accounts for historical open intervals on faults without a date of last event constraint.

The exact factors influencing differences between the second Uniform California Earthquake Rupture Forecast (UCERF2) and UCERF3 vary throughout the region and depend on evaluating specific seismogenic sources. For example, with the 30-year Mw greater than or equal to 6.7 probabilities, the most significant changes from UCERF2 are a threefold increase on the Calaveras fault and a threefold decrease on the San Jacinto fault. The model also suggests that the average time between 6.7 Mw or larger events has increased. The UCERF3 model indicates that Mw greater than or equal to 6.7 probabilities may not represent other hazard or loss measures. The applicability of UCERF3 should be evaluated on a case-by-case basis if required during site-specific ground motion analyses or at the behest of the regulatory agencies (WGCEP, 2014).

Some contributors to seismic risk for the project include the San Andreas, Hayward, Monte-Vista Shannon, and Calaveras faults. A large-magnitude earthquake on any of these fault systems has the potential to cause significant ground shaking in the vicinity of the project site. The intensity of ground shaking likely to occur in the area generally depends upon the earthquake's magnitude and the distance to the epicenter.

Relevant seismic sources in the San Francisco Bay area and their distances from the project site are summarized in Table 3-1.

Table 3-1. Distances to Selected Major Active Fault Surface Traces

Fault Name	Distance and Direction from Site to Surface Fault Traces
Monte Vista-Shannon	8.1 kilometers (km) southwest
San Andreas	10.7 km southwest
Hayward	18.8 km northeast
Calaveras	25.5 km northeast
San Gregorio	25.8 km southwest

3.4 GEOHAZARD MAPPING

3.4.1 Active Faulting

According to the California Geological Survey (CGS, 2018), a Holocene-active fault is defined as a fault that has had surface displacement within Holocene time (the last 11,700 years), and a pre-Holocene fault is defined as a fault whose recency of past movement is older than 11,700 years. The Alquist-Priolo Earthquake Fault Zoning Act only addresses the hazard of surface fault rupture for Holocene-active faults. However, pre-Holocene-active faults may also have the potential for future surface fault rupture (CGS, 2018). The Alquist-Priolo Earthquake Fault Zoning Act's main purpose is to prevent the construction of buildings used for human occupancy on the surface trace of active faults. Before a new project is permitted, cities and counties require a geologic investigation to demonstrate that proposed buildings will not be constructed on active faults. According to CGS (2006), the project site is not located within an Alquist-Priolo earthquake fault zone. Refer to the fault activity map on Figure 4.

According to the U.S. Geological Survey (USGS) Quaternary Fault and Fold Database (2017), no active faults are mapped as crossing through the project site.

CGS (2006) has prepared a map showing Zones of Required Investigation (e.g., liquefaction, landslide, and earthquake fault zones) for the Palo Alto 7.5-minute Quadrangle that encompasses the site.

3.4.2 Liquefaction Hazards

Witter and others (2006) have generated a map showing liquefaction susceptibility for the San Francisco Bay Area with a 5-class scale that includes very low (essentially bedrock areas), low, moderate, high, and very high liquefaction susceptibility classes (Figure 5). The soils underlying the planned improvements are mapped as having moderate liquefaction susceptibility (Witter and others, 2006).

The project area is shown within a liquefaction hazard zone on the State of California Seismic Hazard Zone Map produced by the CGS for the Palo Alto 7.5-minute quadrangle (CGS, 2006).

3.5 REGIONAL GROUNDWATER

The Seismic Hazard Zone Report for the Palo Alto 7.5-minute quadrangle shows historically high groundwater levels near the project site to be approximately 10 feet bgs (CGS, 2006).

4. Field Investigations

4.1 SITE RECONNAISSANCE

Haley & Aldrich performed a site reconnaissance on 26 November 2024 and 11 December 2024, in advance of performing a subsurface exploration. Site reconnaissance consisted of photographic documentation of the project area, identifying and marking the boring location for USA, and determining traffic control needs for the boring location. A private utility locator (GeoTech Utility Locating, LLC) was also used to clear the marked boring location of existing utilities.

4.2 SUBSURFACE EXPLORATION

Subsurface exploration consisted of drilling one geotechnical boring (B-1) near the planned improvement to assess the subsurface conditions. The approximate location of the boring is shown in Figure 2.

The boring was drilled by Exploration GeoServices, Inc. on 16 December 2024, using a Mobile B-53 truck-mounted drill rig, equipped with 8-inch-diameter, hollow-stem augers. The boring was drilled to a depth of 41.5 feet bgs.

Upon completion of drilling, the borehole was backfilled with neat cement grout in accordance with San Mateo County Environmental Health Services permit requirements. The upper 2 feet of the borehole was backfilled with concrete, troweled smooth, and dyed black to blend in with the existing pavement.

4.2.1 Logging and Sampling

The materials encountered in the boring were logged in the field by a Haley & Aldrich engineer. The soils encountered were sampled and visually classified in the field, office, and laboratory according to the Unified Soil Classification System (USCS) in general accordance with ASTM International (ASTM) standards D2487 and D2488.

During the drilling operations, soil samples were obtained using the following sampling methods:

- California Modified (CM) Sampler; 3.0-inch outer-diameter (O.D.), 2.5-inch inner-diameter (I.D.) (ASTM D1586)
- Standard Penetration Test (SPT) Split Spoon Sampler; 2.0-inch O.D., 1.375-inch I.D. (ASTM D1586)

The CM and SPT samplers were driven 18 inches (unless otherwise noted on the boring log) with a 140-pound hammer dropped from a height of 30 inches using a cable-drop, safety hammer. The number of blows required to drive the samplers through each 6-inch interval was recorded for each sample and is included on the boring logs in Appendix A. The blow counts included on the boring logs represent the field values and have not been corrected.

Soil samples obtained from the boring were packaged and sealed in the field to reduce the potential for moisture loss and disturbance. The samples were delivered to Cooper Testing Labs in Palo Alto, California, for further analysis and storage.

4.2.2 Soil Conditions Encountered

Subsurface soil conditions encountered in the boring were generally consistent with regional geologic mapping.

The boring encountered asphaltic pavement to a depth of approximately 13 inches. The fill layer below the asphaltic pavement consisted of dry silty sand with traces of clay and gravel and was approximately 2.5 feet thick. Alluvial soils were then encountered underlying the fill and consisted of the five following layers:

1. sandy silt with traces of clay to a depth of 11 feet
2. approximately 5 feet of poorly graded sand to a depth of 16 feet
3. medium sandy lean clay to an approximate depth of 28.5 feet
4. silty sand layer to an approximate depth of 33.5 feet
5. medium sandy lean clay layer to the final depth of the boring (41.5 feet)

4.2.3 Groundwater Conditions Encountered

Groundwater was encountered in the boring at approximately 18 feet bgs.

4.3 GEOTECHNICAL LABORATORY TESTING

Testing was performed to obtain information concerning the qualitative and quantitative physical properties of the samples recovered during the subsurface exploration program. Tests were performed at Cooper Testing Laboratory in Palo Alto, California in general conformance with applicable ASTM standards. The following tests were performed:

- Moisture Content and Dry Unit Weight (ASTM D7263b)
- Particle Size Analysis (ASTM D422 and D1140)
- Atterberg Limits (ASTM D4318; dry method)
- Unconsolidated-Undrained Triaxial Compression (ASTM D2850)
- Caltrans Corrosion Package including:
  - Resistivity (Minimum) (Caltrans 643)
  - pH (Caltrans 643)
  - Chloride (Caltrans 422m)
  - Sulfate (Caltrans 417m)

The results of the laboratory testing program are summarized on the boring log in Appendix A and on the laboratory test data sheets presented in Appendix B.

5. Conclusions and Discussion

5.1 GENERAL SUMMARY

The existing storm drain at the proposed location of the TCD consists of a 36-inch-diameter reinforced concrete pipe (RCP) with an invert depth of approximately 12.2 feet bgs. The depth of the TCD extends approximately 5.2 feet below the pipe invert depth to a total depth of approximately 17.3 feet bgs.

The primary geotechnical factors to be considered in the design of the proposed TCD construction project include the following:

- Excavatability of encountered materials;
- Shoring and excavation stability;
- High groundwater level affecting foundation design and construction through buoyancy forces;
- Liquefaction potential below the site; and
- Corrosion potential of the surrounding soils.

5.2 EXCAVATABILITY

Subsurface exploration was completed using solid-stem augers and did not encounter auger refusal to the depth explored (41.5 feet bgs). Based on the subsurface conditions encountered, Haley & Aldrich anticipates that an appropriately sized backhoe or excavator will be capable of excavating the soil underlying the project area.

5.3 SHORING AND EXCAVATION STABILITY

The excavation for the TCD structure is planned to extend to a minimum depth of approximately 19 to 20 feet bgs. Soils at the bottom of the excavation are anticipated to be wet and comprised of soft to medium, low plasticity clays with varying amounts of sand. Due to the depth of excavation and the presence of sands, silty sands, and sandy silts above the excavation subgrade, the sides of the excavation will require shoring.

The design of excavation shoring should be made the responsibility of the contractor. Shoring design should be completed for the contractor by a qualified California-registered civil engineer and then submitted to the engineer of record for review and approval before construction. The shoring design should be completed in coordination with the dewatering designer. Refer to Section 6.3 for the design of shoring recommendations.

5.4 HIGH GROUNDWATER AND BUOYANCY UPLIFT

Based on the available historical data for the groundwater depth, the potential for buoyancy uplift is moderate at the project location. Careful consideration by the contractor for shoring and dewatering design and construction will be required to mitigate buoyancy uplift.

Design and construction of the TCD will require a means of preventing uplift. This may be accomplished with an extended base around the perimeter of the structure over which soil backfill is placed. Other means of preventing buoyancy uplift include considering friction on the sides of the structure and installing tension piles or micropiles at the base of the structure.

5.5 LIQUEFACTION POTENTIAL

Liquefaction is a phenomenon caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles, resulting in the sudden loss of shear strength in the soil. Granular soils, which rely on interparticle friction for strength, are susceptible to liquefaction until the excess pore pressures can dissipate. Sand boils and flows observed at the ground surface after an earthquake are the result of excess pore pressures dissipating upward, carrying soil particles with the draining water. In general, loose, saturated sand soils with low silt and clay content are the most susceptible to liquefaction. Silty soils with low plasticity are moderately susceptible to liquefaction under relatively higher levels of ground shaking. For any soil type, the soil must be saturated for liquefaction to occur.

The Boulanger and Idriss (2014) and Idriss and Boulanger (2008) methods were used to evaluate soil liquefaction at boring B-1 and estimate the liquefaction-induced settlement. Based on our analysis, in the absence of any excavation, the estimated liquefaction-induced settlement at the ground surface is 0.4 inches. This estimate is based on potentially liquefiable soils at approximately 10 feet bgs, assuming these soils are fully saturated. Since these soils will be excavated as part of the TCD construction, Haley & Aldrich does not anticipate liquefaction-induced settlements below the structure.

5.6 CORROSION

One soil sample was collected and tested for corrosivity characteristics by Cooper Testing Lab in Palo Alto, California. The sample (from 20.5 to 21 feet bgs) was tested for resistivity, sulfate and chloride ion concentration, and pH. The test results indicate that soils near the bottom of the TCD excavation are not considered corrosive to steel or concrete based on Caltrans corrosion guidelines (2021) and ACI (2019), respectively.

6. Design and Construction Considerations

6.1 DESIGN GROUNDWATER LEVEL

For the design of the proposed TCD, a design groundwater level of 10 feet bgs is recommended for design and construction. Groundwater at the time of drilling was encountered at 18 feet bgs, with historical data indicating higher groundwater levels up to approximately 10 feet bgs. The contractor and shoring designer should consult the boring log provided in Appendix A.

6.2 DEWATERING

The excavation of the TCD bearing on a shallow foundation will likely extend the excavation depth to approximately 19 to 20 feet bgs.

Based on our review of the subsurface conditions, dewatering may be required during the excavation and compaction of backfill materials. The soils encountered in boring B-1 included mostly fine-grained silts and clays, which have low permeability. A poorly graded sand layer was encountered between the depths of 11 and 16 feet bgs. This sand layer is above the groundwater encountered in the boring. However, due to seasonal variations of the groundwater, this sand layer may be fully saturated depending on the time of year of the construction. In addition, this sand layer is susceptible to caving. Soils at the bottom of the excavation are anticipated to be moist to wet and comprised of homogeneous sandy clays.

Haley & Aldrich anticipates that dewatering may be accomplished with a sump pit at the bottom of the excavation and that dewatering wells may not be required. We recommend dewatering to a minimum of 2 feet below the bottom of the excavation.

6.3 TEMPORARY SHORING

Shoring design should be based on Occupational Safety and Health Administration (OSHA) Type B Soil. The impact of elevated groundwater conditions on the temporary shoring can be mitigated by implementing contractor-designed dewatering measures and designing the shoring to be watertight and to account for the loading imposed by the groundwater in accordance with the recommendations provided herein. It is recommended that all temporary shoring be designed in conformance with the State of California, Department of Transportation, Trenching and Shoring Manual (latest edition).

Due to the presence of sand layers and high groundwater, sheet piles are recommended for shoring of the excavation required for the installation of the TCD. The use of speed shores, trench boxes, or slide rail systems is not recommended.

6.3.1 Lateral Earth Pressures

Static lateral earth pressure will be imposed on all shored excavations. Table 6-1 summarizes the lateral earth pressures recommended for use in the design of unbraced temporary shoring. Active pressure should be assumed for conditions where the top of the wall is free to deflect up to 1/2 inch. Passive pressure should be ignored for a depth of 24 inches and may be utilized to resist overturning and sliding. Where structures will be located below groundwater, hydrostatic pressures should be added to the passive lateral earth pressure values, as shown in Table 6-1. As noted previously, the design of unbraced shoring will likely be controlled by deflections; as a result, calculations should also consider allowable ground deformations.

Table 6-1. Lateral Earth Pressures - Unbraced Shoring		
Pressure Type	**Above Groundwater Level (Equivalent Fluid Pressure) (pcf)**	**Below Groundwater Level (Buoyant Equivalent Fluid Pressure + Hydrostatic) (pcf)**
Active (0 to 16 ft)	40	80
At-Rest (0 to 16 ft)	60	90
Passive (0 to 16 ft)	425	275
Active (16 to 30 ft)	50	90
At-Rest (16 to 30 ft)	75	100
Passive (16 to 30 ft)	315	225
Notes: pcf = pounds per cubic foot		

If the temporary shoring is braced, a rectangular or trapezoidal loading diagram, such as those recommended by Terzaghi and Peck (1967), Tschebotarioff (1973), Caltrans (2021), and the Federal Highway Administration (FHWA; 1999), should be used. These methods generally correlate the earth pressure load to a percentage of the unit weight of the soil times the height of the excavation. The method and loading should be determined by the contractor and provided to the engineer of record for review.

It is recommended that the contractor’s shoring design engineer evaluate high and low groundwater cases to confirm which case governs the design.

Surcharge loading from traffic on the adjacent pavement and construction equipment can be modeled as a minimum uniform ground pressure of 250 pounds per square foot (psf) or higher, as otherwise determined by the contractor's shoring design engineer.

6.3.2 Minimum Depth of Shoring

The shoring for installation of the TCD should extend a minimum of 5 feet below the base of the planned excavation for cantilever shoring systems. The actual depth of the shoring should be determined by the contractor's shoring design engineer, based on the lateral loads, surcharge loads, bracing requirements, calculated displacements, and the depth of penetration required to control stability, including the heave of the excavation bottom. The determination of minimum depth should also consider soil and groundwater conditions at the time of excavation.

6.3.3 Installation and Removal of Shoring

To reduce the potential for vibration-induced settlements during construction, it is recommended that the contractor monitor the soils encountered during excavation and, at a minimum, avoid the generation of vibrations at locations where loose cohesionless soils are encountered. Additionally, the contractor should use static force to remove sheet pile sections (if used for shoring) or other embedded elements used for temporary shoring to avoid generating vibrations and possible settlement of the structure, adjacent pipeline, and/or ground surface. Settlement of adjacent structures during the removal of shoring should not be allowed and should be monitored during removal.

6.4 BEARING SUPPORT

Haley & Aldrich estimated the existing weight of soil and fill at the proposed TCD location and compared these values to the estimated weight of the structure. The purpose of this estimate was to evaluate whether the soils beneath the proposed structure will experience a net increase or net decrease in effective overburden stresses. Based on our analyses, we concluded that the overall weight of the proposed structure will be less than the weight of the soil removed. Therefore, we expect either the same or a net reduction in overburden pressure on the soils below the proposed structure. Settlement associated with the weight of the proposed structure will primarily consist of recompression of clayey soil layers or disturbed sandy clay soils at or below the bottom of the planned structure. Recompression of these soils is estimated to be less than 1/2 inch and is expected to occur during construction.

For the proposed structure, an allowable vertical bearing pressure of 1,500 psf may be used, which accounts for the sum of dead and live loads. In-place densities of 125 pcf may be assumed for existing soils and for soil backfill for use in determining the weight of soil removed and of backfill placed.

6.5 EARTHWORK

6.5.1 Subgrade Preparation

The bottom of the excavation for the TCD will likely encounter moist to wet, medium sandy clays. To provide a stable surface on which to construct forms, place concrete, and compact backfill, the bottom of the excavation should be lined with a minimum of 18 inches of Caltrans Class 1 Aggregate Subbase material or 3/4-inch clean crushed rock. A layer of geotextile, such as Mirafi 500X (or equivalent), may be needed underneath the crushed rock if soft/loose materials are encountered. The crushed rock should be compacted with a manual vibratory compaction plate by making a minimum of three passes until a firm non-yielding surface results.

If groundwater and soft soil is encountered at the bottom of the excavation, a mud slab may be constructed at the base of the excavation. Before the construction of the mud slab, a layer of high-strength geogrid (Tensar BX1200 or equivalent) should be installed directly on the subgrade at the base of the excavation. The geogrid should also extend up the sides of the excavation to cover the geotextile/filter fabric and to provide protection in the event the geotextile is damaged during shoring removal. The mud slab should consist of approximately 6 inches of Portland Cement Concrete, Lean Concrete Base, or Controlled Density Fill (CDF; also known as controlled low strength material [CLSM] or flowable fill). Above the mud slab, a minimum 1-foot-thick layer of Caltrans Class 1 Aggregate Subbase, covered with a separation fabric such as Mirafi 500X (or equivalent), should be placed.

6.5.2 Structure Backfill

The onsite soils may be suitable for use as structure backfill, upon approval by the project geotechnical engineer. Structure backfill shall be compacted to at least 90 percent relative compaction below the upper 12 inches of the street subgrade followed by 95 percent in the upper 12 inches. Backfill material should be placed in lifts not exceeding 8 inches in uncompacted thickness. Thinner lifts may be necessary to achieve the recommended level of compaction of the backfill due to equipment limitations.

Compaction should be performed by mechanical means only. Water jetting to attain compaction shall not be permitted.

If the space between the sides of the excavation and TCD is too narrow for effective compaction of structure backfill, we recommend CDF be used as backfill around the TCD. The CDF is comprised of cementitious material, sand or fine aggregate, and water, and typically has a compressive strength between 100 and 200 pounds per square inch. The primary advantage of using CDF is that it is placed relatively quickly and does not require compaction.

6.5.3 Import Fill

Import fill may be anticipated for the project for backfilling around the TDC. If import material is brought in to replace onsite material, it should be noted that all potential imported fill must be reviewed and approved by the geotechnical engineer before importation to the site. A minimum of five days will be required to evaluate and test the suitability of all planned imported materials. The imported materials should be non-expansive and have a Plasticity Index of less than 15 percent and a Liquid Limit of 30 percent or less. The imported material shall be free of organic debris or contaminated materials.

6.5.4 Pavements

Pavement sections should be replaced in kind. In general, pavement subgrade and aggregate base rock should be compacted to a minimum of 95 percent relative compaction (ASTM D1557 latest edition) within 2 percent of optimum moisture content.

6.6 CODE-BASED SEISMIC DESIGN VALUES

Due to the proximity of the site to the numerous active fault systems that traverse the greater San Francisco Bay Area, the project site will likely be subjected to the effects of a major earthquake during the design life of the proposed improvements. The effects are likely to consist of significant ground accelerations. These ground-type movements may cause damage to the proposed improvements. Haley & Aldrich, therefore, recommends that, at a minimum, the structural systems for the proposed improvements be designed in accordance with the requirements of Chapter 16 of the 2022 California Building Code and ASCE 7-16, Supplement 3. The California Building Code seismic design parameters for the site are included in Table 6-2.

Table 6-2. 2022 CBC Seismic Design Parameters	
Item	**Design Value**
Site Soil Class Definition	D
Ss – 0.2 Second Spectral Response Acceleration	1.5
S1 – 1.0 Second Spectral Response Acceleration	0.6
Fa – Values of Site Coefficient	1.0
Fv – Value of Site Coefficient	1.7*
SMS – MCE_R Spectral Response Acceleration Parameter (g)	1.5
SM1 – MCE_R Spectral Response Acceleration Parameter (g)	1.53**
SDS – Designed Spectral Response Acceleration for Short Periods	1.0
SD1 – Designed Spectral Response Acceleration for 1-Sec Periods	1.02
Peak Ground Acceleration (PGA)	0.60
Maximum Peak Ground Acceleration (PGA_M)	0.66

Notes:

Reference: https://asce7hazardtool.online/ (ASCE, 2025).

** - This value shall only be used for the calculation of T_S, determination of Seismic Design Category, linear interpolation for intermediate values of S_1, and when taking the exceptions under Items 1 and 2 of Section 11.4.8 of ASCE 7-16 for the calculation of S_{D1}.*

*** - In accordance with the EXCEPTION to ASCE 7-16, Section 11.4.8, a ground motion hazard analysis is not required since the value of the parameter S_{M1} determined by Eq. (11.4-2) has been increased by 50 percent for all applications of S_{M1}.*

6.7 TECHNICAL REVIEW AND CONSTRUCTION OBSERVATION

Before construction, the geotechnical engineer should review the project plans and specifications for conformance with the intent of the recommendations presented in this report. The geotechnical engineer should be contacted a minimum of 48 hours in advance of excavation operations to observe the subsurface conditions.

7. Limitations

The conclusions and recommendations presented in this report are based on the information provided regarding the planned construction, and the results of the geologic mapping, subsurface exploration, and testing. Site conditions described in the text of this report are those existing at the time of our last field reconnaissance and are not necessarily representative of the site conditions at other times or locations. This information notwithstanding, the nature and extent of subsurface variations may not become evident until construction. If variations are encountered during construction, Haley & Aldrich should be notified promptly so that conditions can be reviewed and recommendations reconsidered, as appropriate.

It is the City of East Palo Alto's responsibility to ensure that the recommendations contained in this report are carried out during the construction phases of the project. This report was prepared based on preliminary design information provided, which is subject to change during the design process. At approximately the 90 percent design level, Haley & Aldrich should review the design assumptions made in this report and prepare addenda or memoranda as appropriate. Any modifications included in these addenda or memoranda should be carefully reviewed by the project designers to make sure that any conclusions or recommendations that are modified are accounted for in the final design of the project.

The findings of this report should be considered valid for a period of three years unless the conditions of the site change. After a period of three years, Haley & Aldrich should be contacted to review the site conditions and prepare a letter regarding the applicability of this report.

This report presents the results of a geotechnical and geologic investigation only and should not be construed as an environmental audit or study. The evaluation or identification of the potential presence of hazardous materials at the site was not requested and was beyond the scope of this investigation and report.

The conclusions and recommendations contained in this report are valid only for the project described in this report. We have employed accepted geotechnical engineering procedures, and our professional opinions and conclusions are made in accordance with generally accepted geotechnical engineering principles and practices. This standard is in lieu of all other warranties, either expressed or implied.

References

1. American Society of Civil Engineers (ASCE), 2025. American Society of Civil Engineers 7 Hazard Tool. Data Source: USGS Seismic Design Maps based on ASCE/SEI 7-16. https://asce7hazardtool.online/. Accessed January 2025.
2. ASCE, 2017. *Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE/SEI 7-16)*. Reston, VA: American Society of Civil Engineers.
3. American Concrete Institute (ACI) Committee 318, 2019. *Building Code Requirements for Structural Concrete (ACI 318-19): An ACI Standard; Commentary on Building Code Requirements for Structural Concrete (ACI 318R-19)*. Farmington Hills, MI: American Concrete Institute.
4. Boulanger, R. W. and I. M. Idriss, 2014. CPT and SPT Based Liquefaction Triggering Procedures. Report UCD/CGM-14/01, Department of Civil and Environmental Engineering, University of California, Davis, CA, 138 pp.
5. Brabb, E. E., R.W. Graymer, and D. L. Jones, 1998. *Geology of the onshore part of San Mateo County, California: a digital database.* U.S. Geological Survey.
6. California Building Standards Commission, 2022. *California Building Code, Title 24, Part 2 (Volumes 1 & 2)*. Sacramento, CA: California Building Standards Commission.
7. California Department of Transportation, 2011, "Trenching and Shoring Manual"; published by the California Department of Transportation. Revision 1, August.
8. California Department of Transportation, 2021. *Corrosion Guidelines, Version 3.2*. Division of Engineering Services, Materials Engineering and Testing Services, Corrosion Branch.
9. CGS, 2018. Earthquake Fault Zones: A Guide for Government Agencies, Property Owners/Developers, and Geoscience Practitioners for Assessing Fault Rupture Hazards in California. CGS Special Publication 42.
10. CGS, 2006. Earthquake Zones of Required Investigation Map for the Palo Alto 7.5 Minute Quadrangle, San Mateo and Santa Clara Counties, California. California Geological Survey Seismic Hazard Zone Report 111.
11. Caltrans-California Department of Transportation Division of Engineering Services Materials Engineering and Testing Services Corrosion and Structural Concrete Field Investigation Branch, 2021. Corrosion Guidelines, Version 3.2. May 2021.
12. Federal Highway Administration, 1999. FHWA-IF-99-015, *Geotechnical Engineering Circular No. 4 Ground Anchors and Anchored Systems*, June.
13. Idriss, I.M. and R.W. Boulanger, 2008. *Soil Liquefaction during Earthquakes*. Earthquake Engineering Research Institute, Monograph MNO-12, Oakland, CA.
14. Terzaghi, K., and R.B. Peck, 1967. "Soil Mechanics in Engineering Practice, Second Edition" Published by John Wiley & Sons, New York. 729 pp.
15. Tschebotarioff, G.P., 1973. "Foundations, Retaining and Earth Structures, Second Edition" Published by McGraw-Hill, New York, 642 pp.

16. I Page, B. M., 1992, Tectonic setting of the San Francisco Bay Region, in Proceedings on the Second Conference on Earthquake Hazards in the eastern San Francisco Bay Area, edited by G. Borchardt, Spec. Publ. Calif. Div. Mines Geol., 113, 1-7.

17. U.S. Geological Survey, 2017. Quaternary fault and fold database for the United States. Accessed December 2024.

18. Witter, R. C., Knudsen, K. L., Sowers, J. M., Wentworth, C. M., Koehler, R. D., Randolph, C. E., and Gans, K. D., 2006. *Maps of Quaternary deposits and liquefaction susceptibility in the central San Francisco Bay region, California (No. 2006-1037)*. Digital Database: U.S. Geological Survey Open-File Report 00-444.

19. Working Group on California Earthquake Probabilities (WGCEP), 2003. Earthquake Probabilities in the San Francisco Bay Region: 2002-2031: U.S. Geological Survey Open File Report 2003-214.

20. WGCEP, 2013. The Uniform California Earthquake Rupture Forecast Version 3 (UCERF3) – Time Independent Model: U.S. Geological Survey Open-File Report 2013-1165.

21. WGCEP, 2014. Long-Term Time-Dependent Probabilities for the Third Uniform California Earthquake Rupture Forecast (UCERF3): Bulletin of the Seismology Society of America.

https://haleyaldrich.sharepoint.com/sites/CityofEastPaloAlto/Shared Documents/0211503.East Palo Alto Trash Capture/Deliverables/Geotech Report/Final/2025_0123_HAI_EastPaloAltoTrashCaptureDevice_F.docx

FIGURES

MAP SOURCE: ESRI
SITE COORDINATES: 37°28'22"N, 122°9'14"W

HALEY ALDRICH

EAST PALO ALTO TRASH CAPTURE DEVICE
NEWBRIDGE STREET
EAST PALO ALTO, CALIFORNIA

SITE LOCATION MAP

APPROXIMATE SCALE: 1 IN = 0.5 MILE
JANUARY 2025

FIGURE 1

GIS: \\haleyaldrich.com\share\CF\Projects\0211503\GIS\ArcGIS\0211503-EastPaloAlto-TrashCapture.aprx - kdrozynska - 12/10/2024 8:33 PM

LEGEND

- BORING LOCATION BY H&A, DRILLED ON 12/16/2024
- (M) EXISTING MANHOLE
- — T — EXISTING TELCO FIBER LINE
- — SD — EXISTING STORM DRAIN LINE
- — G — EXISTING GAS LINE
- — E — EXISTING ELECTRIC LINE
- PROPERTY LINES

NOTES

1. ALL LOCATIONS AND DIMENSIONS ARE APPROXIMATE.

2. LOCATIONS OF THE PROPOSED TRASH CAPTURE DEVICE AND EXISTING UTILITIES FROM SCHAAF AND WHEELER, GIS DATA PROVIDED ON 11/25/2024.

3.PARCEL DATA FROM COUNTY OF SAN MATEO INFORMATION SERVICES, ACCESSED ONLINE ON 1/19/2024.

3. AERIAL IMAGERY FROM ESRI (COUNTY OF SANTA CLARA, 11/19/2022).

N

0 40 80
SCALE IN FEET

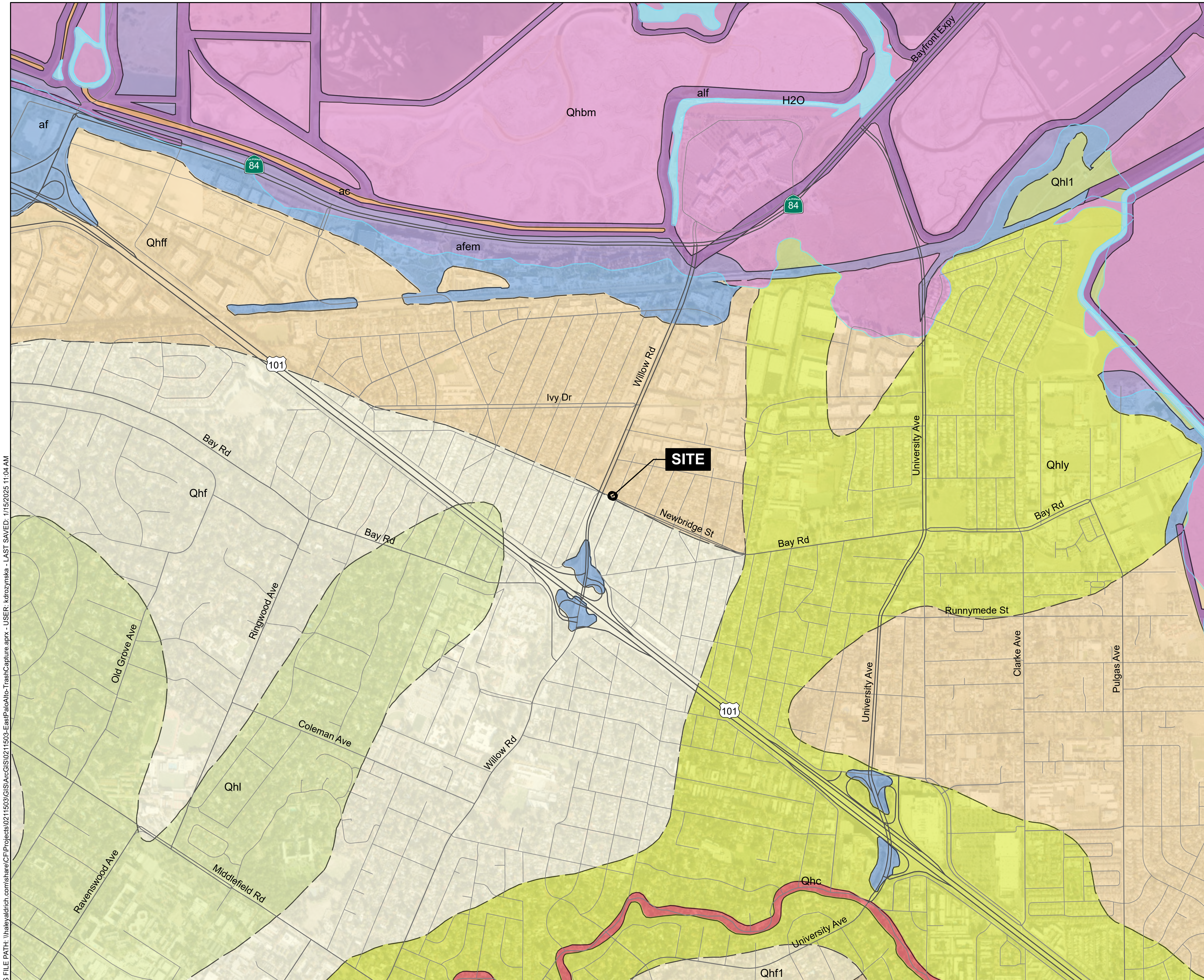
HALEY ALDRICH

EAST PALO ALTO TRASH CAPTURE DEVICE
NEWBRIDGE STREET
EAST PALO ALTO, CALIFORNIA

SITE PLAN

JANUARY 2025

FIGURE 2



LEGEND

af	ARTIFICIAL FILL (HISTORICAL)
afem	ARTIFICIAL FILL OVER ESTUARINE MUD (HISTORICAL)
alf	ARTIFICIAL LEVEE FILL (HISTORICAL)
ac	ARTIFICIAL STREAM CHANNEL (HISTORICAL)
Qhc	HISTORICAL STREAM CHANNEL DEPOSITS (HISTORICAL)
Qhly	ALLUVIAL FAN LEVEE DEPOSITS (LATEST HOLOCENE)
Qhbm	SAN FRANCISCO BAY MUD (HOLOCENE)
Qhf	ALLUVIAL FAN DEPOSITS (HOLOCENE)
Qhf1	YOUNGER ALLUVIAL FAN DEPOSITS (HOLOCENE)
Qhff	ALLUVIAL FAN DEPOSITS, FINE FACIES (HOLOCENE)
Qhl	ALLUVIAL FAN LEVEE DEPOSITS (HOLOCENE)
Qhl1	YOUNGER ALLUVIAL FAN LEVEE DEPOSITS (HOLOCENE)

NOTES

1. ALL LOCATIONS AND DIMENSIONS ARE APPROXIMATE.
2. REGIONAL GEOLOGY FROM WITTER, 2006.

N

0 0.25 0.5
SCALE IN MILES

HALEY ALDRICH

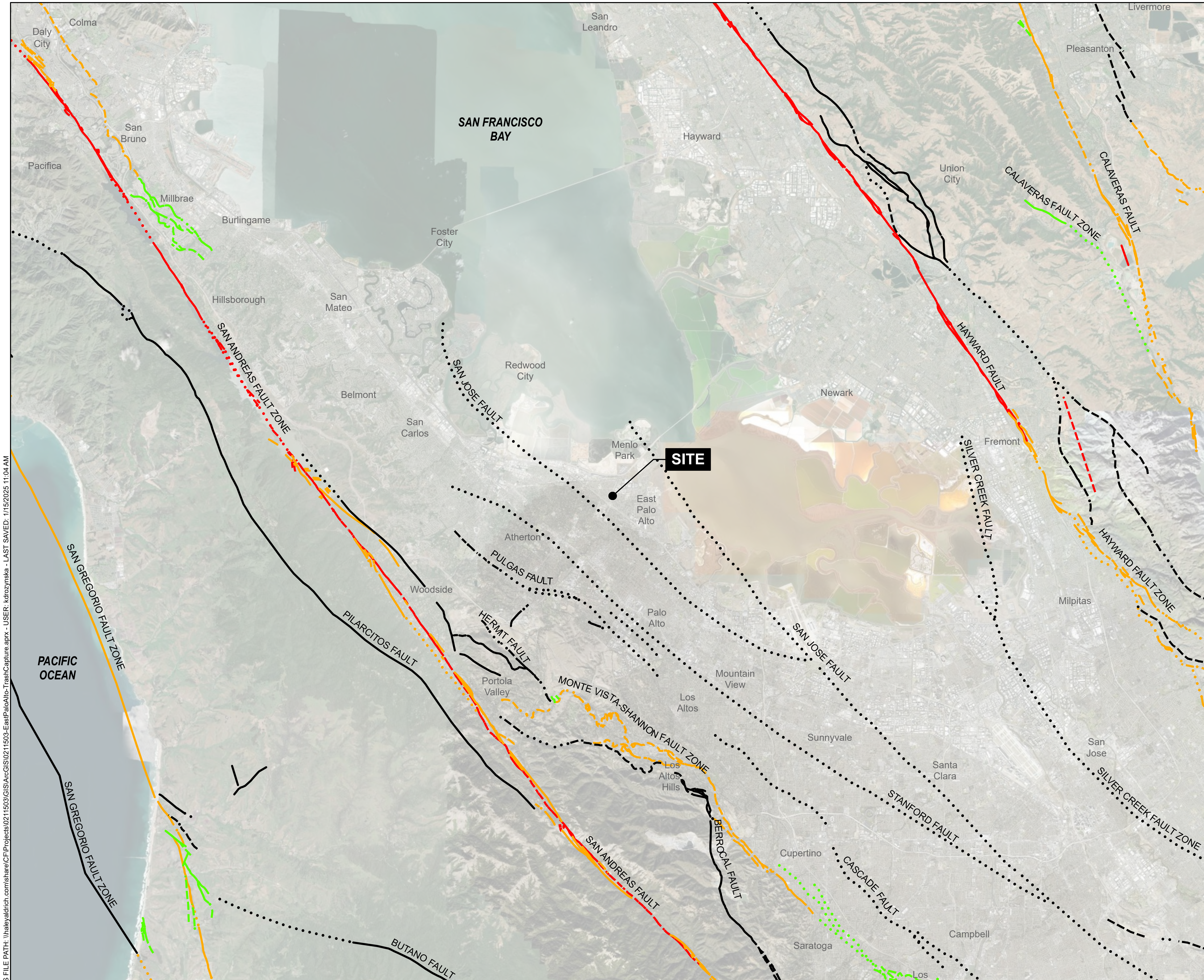
EAST PALO ALTO TRASH CAPTURE DEVICE
NEWBRIDGE STREET
EAST PALO ALTO, CALIFORNIA

REGIONAL GEOLOGY MAP

JANUARY 2025

FIGURE 3

GIS FILE PATH: \\haleyaldrich.com\share\CF\Projects\0211503\GIS\ArcGIS\0211503-EastPaloAlto-TrashCapture.aprx - USER: kdrozynska - LAST SAVED: 1/15/2025 11:04 AM



GIS FILE PATH: \\haleyaldrich.com\share\CF\Projects\0211503\GIS\ArcGIS\0211503-EastPaloAlto-TrashCapture.aprx - USER: kdrozynska - LAST SAVED: 1/15/2025 11:04 AM

LEGEND

FAULT ACTIVITY BASED ON TIME OF MOST RECENT SURFACE DEFORMATION:

- HISTORICAL (<150 YEARS), WELL CONSTRAINED LOCATION
- HISTORICAL (<150 YEARS), MODERATELY CONSTRAINED LOCATION
- HISTORICAL (<150 YEARS), INFERRED LOCATION
- LATEST QUATERNARY (<15,000 YEARS), WELL CONSTRAINED LOCATION
- LATEST QUATERNARY (<15,000 YEARS), MODERATELY CONSTRAINED LOCATION
- LATEST QUATERNARY (<15,000 YEARS), INFERRED LOCATION
- LATE QUATERNARY (<130,000 YEARS), WELL CONSTRAINED LOCATION
- LATE QUATERNARY (<130,000 YEARS), MODERATELY CONSTRAINED LOCATION
- LATE QUATERNARY (<130,000 YEARS), INFERRED LOCATION
- UNDIFFERENTIATED QUATERNARY(<1.6 MILLION YEARS), WELL CONSTRAINED LOCATION
- UNDIFFERENTIATED QUATERNARY(<1.6 MILLION YEARS), MODERATELY CONSTRAINED LOCATION
- UNDIFFERENTIATED QUATERNARY(<1.6 MILLION YEARS), INFERRED LOCATION

NOTES

1. ALL LOCATIONS AND DIMENSIONS ARE APPROXIMATE.
2. FAULT LOCATIONS FROM US GEOLOGICAL SURVEY QUATERNARY FAULTS AND FOLDS DATABASE, ACCESSED ONLINE ON 30 JULY 2021.

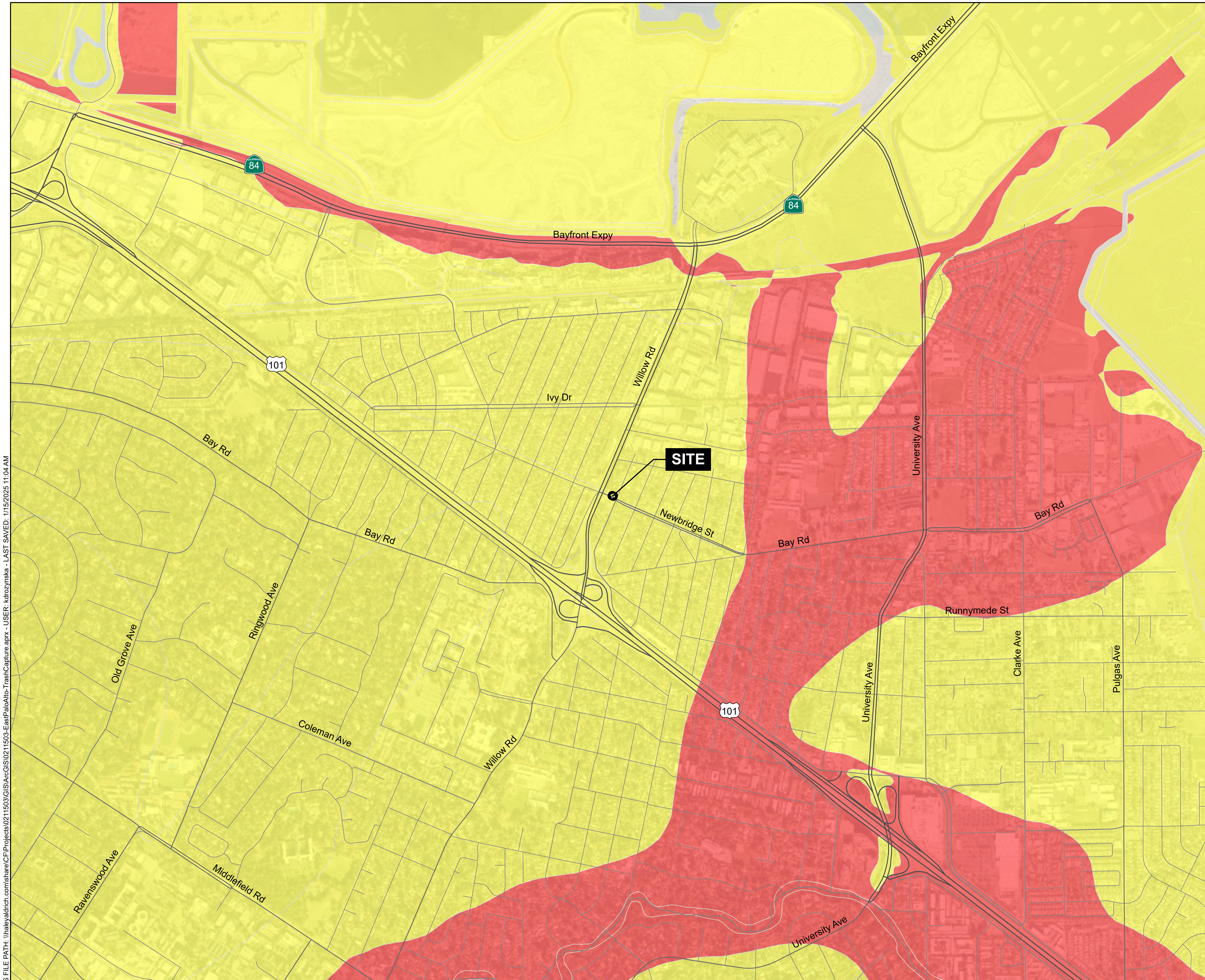
HALEY ALDRICH

EAST PALO ALTO TRASH CAPTURE DEVICE
NEWBRIDGE STREET
EAST PALO ALTO, CALIFORNIA

FAULT ACTIVITY MAP

JANUARY 2025

FIGURE 4



LIQUEFACTION SUSCEPTIBILITY

- VERY HIGH
- HIGH
- MODERATE
- LOW
- VERY LOW
- NOT MAPPED

NOTES

1. ALL LOCATIONS AND DIMENSIONS ARE APPROXIMATE.
2. LIQUEFACTION SUSCEPTIBILITY FROM WITTER, 2006.

N

0 0.25 0.5
SCALE IN MILES

HALEY ALDRICH

EAST PALO ALTO TRASH CAPTURE DEVICE
NEWBRIDGE STREET
EAST PALO ALTO, CALIFORNIA

LIQUEFACTION SUSCEPTIBILITY MAP

JANUARY 2025

FIGURE 5

GIS FILE PATH: \\haleyaldrich.com\share\CF\Projects\0211503\GIS\ArcGIS\0211503-EastPaloAlto-TrashCapture.aprx - USER: kdrozynska - LAST SAVED: 1/15/2025 11:04 AM

APPENDIX A
Boring Log

Sample Description

Identification of soils in this report is based on visual field and laboratory observations which include density/consistency, moisture condition, grain size, and plasticity estimates and should not be construed to imply field nor laboratory testing unless presented herein. ASTM D 2488 visual-manual identification methods were used as a guide. Where laboratory testing confirmed visual-manual identifications, then ASTM D 2487 was used to classify the soils.

Relative Density/Consistency

Soil density/consistency in borings is related primarily to the standard penetration resistance (N). Soil density/consistency in test pits and probes is estimated based on visual observation and is presented parenthetically on the logs.

SAND or GRAVEL Relative Density	N (Blows/Foot)	SILT or CLAY Consistency	N (Blows/Foot)
Very loose	0 to 4	Very soft	0 to 1
Loose	5 to 10	Soft	2 to 4
Medium dense	11 to 30	Medium stiff	5 to 8
Dense	31 to 50	Stiff	9 to 15
Very dense	>50	Very stiff	16 to 30
		Hard	>30

Moisture

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

USCS Soil Classification Chart (ASTM D 2487)

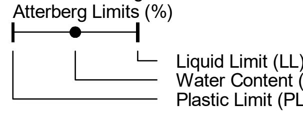
Major Divisions			Symbols: Graph	Symbols: USCS	Typical Descriptions
Coarse Grained Soils More than 50% of Material Retained on No. 200 Sieve	Gravel and Gravelly Soils More than 50% of Coarse Fraction Retained on No. 4 Sieve	Clean Gravels *(<5% fines)*		GW	Well-Graded Gravel; Well-Graded Gravel with Sand
				GP	Poorly Graded Gravel; Poorly Graded Gravel with Sand
		Gravels *(5-12% fines)*		GW-GM	Well-Graded Gravel with Silt; Well-Graded Gravel with Silt and Sand
				GW-GC	Well-Graded Gravel with Clay; Well-Graded Gravel with Clay and Sand
				GP-GM	Poorly Graded Gravel with Silt; Poorly Graded Gravel with Silt and Sand
				GP-GC	Poorly Graded Gravel with Clay; Poorly Graded Gravel with Clay and Sand
		Gravels with Fines *(>12% fines)*		GM	Silty Gravel; Silty Gravel with Sand
				GC	Clayey Gravel; Clayey Gravel with Sand
	Sand and Sandy Soils More than 50% of Coarse Fraction Passing No. 4 Sieve	Sands with few Fines *(<5% fines)*		SW	Well-Graded Sand; Well-Graded Sand with Gravel
				SP	Poorly Graded Sand; Poorly Graded Sand with Gravel
		Sands *(5-12% fines)*		SW-SM	Well-Graded Sand with Silt Well-Graded Sand with Silt and Gravel
				SW-SC	Well-Graded Sand with Clay; Well-Graded Sand with Clay and Gravel
				SP-SM	Poorly Graded Sand with Silt; Poorly Graded Sand with Silt and Gravel
				SP-SC	Poorly Graded Sand with Clay; Poorly Graded Sand with Clay and Gravel
		Sands with Fines *(>12% fines)*		SM	Silty Sand; Silty Sand with Gravel
				SC	Clayey Sand; Clayey Sand with Gravel
Fine Grained Soils More than 50% of Material Passing No. 200 Sieve	Silts			ML	Silt; Silt with Sand or Gravel; Sandy or Gravelly Silt
				MH	Elastic Silt; Elastic Silt with Sand or Gravel; Sandy or Gravelly Elastic Silt
	Silty Clay *(based on Atterberg Limits)*			CL-ML	Silty Clay; Silty Clay with Sand or Gravel; Gravelly or Sandy Silty Clay
	Clays			CL	Lean Clay; Lean Clay with Sand or Gravel; Sandy or Gravelly Lean Clay
				CH	Fat Clay; Fat Clay with Sand or Gravel; Sandy or Gravelly Fat Clay
	Organics			OL/OH	Organic Soil; Organic Soil with Sand or Gravel; Sandy or Gravelly Organic Soil
Highly Organic *(>50% organic material)*				PT	Peat - Decomposing Vegetation - Fibrous to Amorphous Texture

Minor Constituents

	Estimated Percentage
Sand, Gravel	
Trace	<5
Few	5 - 15
Cobbles, Boulders	
Trace	<5
Few	5 - 10
Little	15 - 25
Some	30 - 45

Soil Test Symbols

%F	Percent Passing No. 200 Sieve
AL	Atterberg Limits (%)



CA	Chemical Analysis
CAUC	Consolidated Anisotropic Undrained Compression
CAUE	Consolidated Anisotropic Undrained Extension
CBR	California Bearing Ratio
CIDC	Consolidated Drained Isotropic Triaxial Compression
CIUC	Consolidated Isotropic Undrained Compression
CK0DC	Consolidated Drained k0 Triaxial Compression
CK0DSS	Consolidated k0 Undrained Direct Simple Shear
CK0UC	Consolidated k0 Undrained Compression
CK0UE	Consolidated k0 Undrained Extension
CRSCN	Constant Rate of Strain Consolidation
DS	Direct Shear
DSS	Direct Simple Shear
DT	In Situ Density
GS	Grain Size Classification
HYD	Hydrometer
ILCN	Incremental Load Consolidation
K0CN	k0 Consolidation
kc	Constant Head Permeability
kf	Falling Head Permeability
MD	Moisture Density Relationship
OC	Organic Content
OT	Tests by Others
P	Pressuremeter
PID	Photoionization Detector Reading
PP	Pocket Penetrometer
SG	Specific Gravity
TRS	Torsional Ring Shear
TV	Torvane
UC	Unconfined Compression
UUC	Unconsolidated Undrained Triaxial Compression
VS	Vane Shear
WC	Water Content (%)

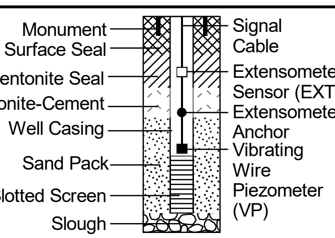
Groundwater Indicators

- Groundwater Level on Date or At Time of Drilling (ATD)
- Groundwater Level on Date Measured in Piezometer
- Groundwater Seepage (Test Pits)

Sample Symbols

- 1.5" I.D. Split Spoon
- 3.0" I.D. Split Spoon
- Modified California Sampler
- Rock Core Run
- Sonic Core
- Thin-walled Sampler
- Grab
- Cuttings
- Push Probe

Well Symbols



HALEY ALDRICH	Project: Trash Capture Project Location: East Palo Alto, California Project No.: 0211503-000	**Key to Exploration Logs**	Sheet **1 of 1**

BORING NUMBER B-1

CLIENT Schaaf and Wheeler, Inc.
PROJECT NAME East Palo Alto Trash Capture Device
PROJECT NUMBER 0211503
PROJECT LOCATION East Palo Alto, California
DATE STARTED 12/16/2024 **COMPLETED** 12/16/2024
GROUND ELEVATION 15 ft **DATUM** NAVD88 **HOLE SIZE** 8 in.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
COORDINATES: **LATITUDE** 37.472783 **LONGITUDE** -122.1539142
DRILLING RIG/METHOD Mobile B-53/8-in. Hollowstem Auger
∇ **GROUNDWATER AT TIME OF DRILLING** 18.0 ft / Elev -3.0 ft
LOGGED BY K. Amini **CHECKED BY** K. Loeb
GROUNDWATER AT END OF DRILLING --- Not Measured
HAMMER TYPE 140 lb hammer with 30 in. safety
GROUNDWATER AFTER DRILLING --- Not Measured

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
0	13" Asphaltic Concrete	GB								
	Silty SAND (SM): light gray and light brown, fine to medium sand, some silt, trace clay and gravel [FILL]									
	Sandy SILT (ML): brown, moist, medium dense, fine to medium sand, some clay [ALLUVIUM]									
5		CM	8-9-10		103	20				67
10		CM	8-12-16		110	15				
	Poorly Graded SAND w/ Silt (SP-SM): brown, moist, medium dense, fine to coarse sand, trace gravel, trace silt [ALLUVIUM]									
15	-becomes dense	CM	25-22-21			15				9
	Sandy Lean CLAY (CL): light brown, moist, stiff, low plasticity, fine sand [ALLUVIUM] ∇	SPT	9-11-15	1.25	106	21	30	19	11	
20	TXUU and Corrosion test at 20.5'	CM	13-16-18	2.5 1.25	96	28	35	18	17	
	-becomes medium stiff	SPT	9-10-13	1.0						
25	-becomes hard	CM	12-19-35	4.25 3.5						
30	Silty SAND (SM): brown , moist to wet, dense, fine to coarse sand, trace gravel [ALLUVIUM]									

(Continued Next Page)

HALEY ALDRICH

BORING NUMBER B-1

CLIENT Schaaf and Wheeler, Inc.

PROJECT NAME East Palo Alto Trash Capture Device

PROJECT NUMBER 0211503

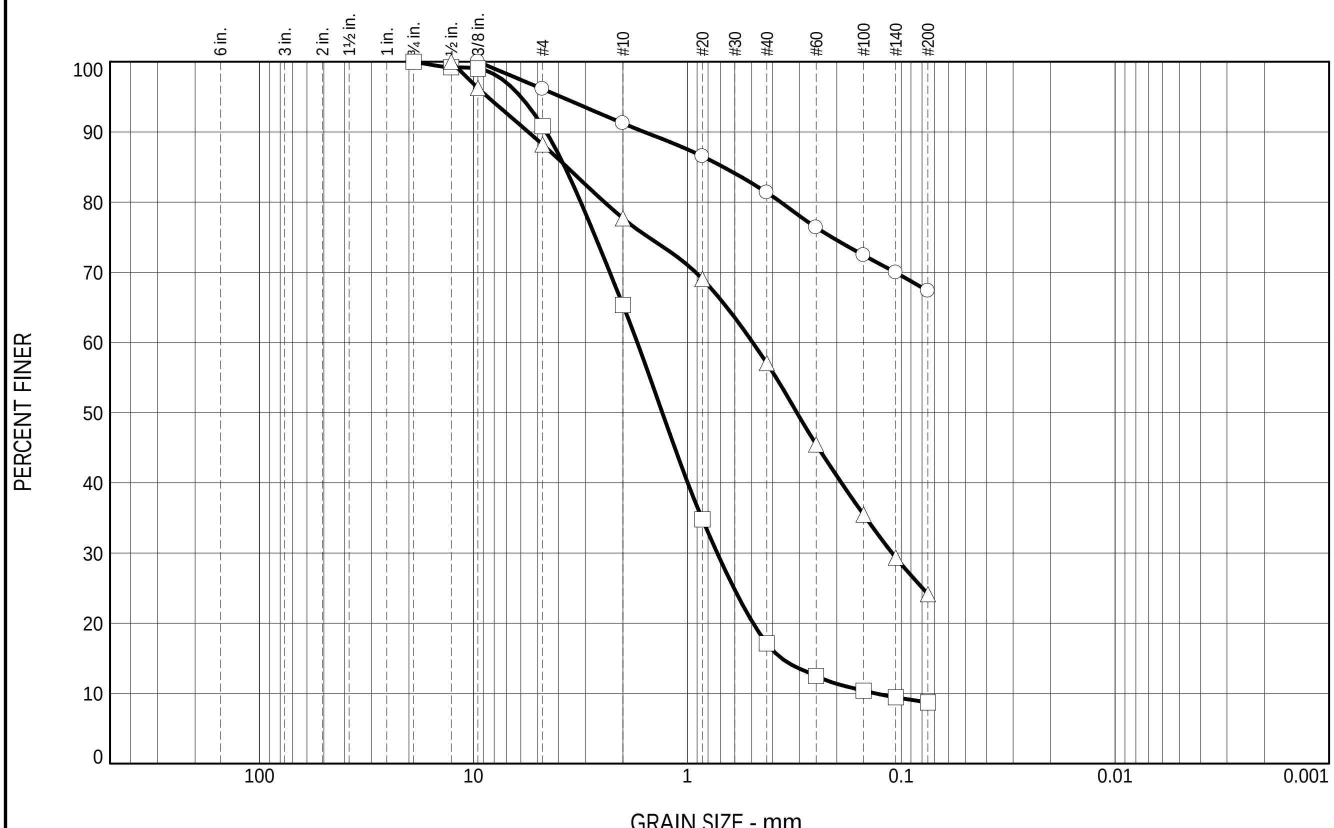
PROJECT LOCATION East Palo Alto, California

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
30		Silty SAND (SM): brown , moist to wet, dense, fine to coarse sand, trace gravel [ALLUVIUM] *(continued)* -3" soft clay lense, light brown	CM	13-18-23		115	17				24
		Sandy Lean CLAY (CL): gray, moist, stiff, low plasticity, trace fine sand [ALLUVIUM]									
35		TXUU test at 36'	CM	14-20-22	1.5 2.0	110	20	30	17	13	
40		-becomes medium stiff, light brown	CM	14-18-23	1.0 1.0	93	30	35	22	13	

Bottom of borehole at 41.5 ft. Borehole backfilled with neat cement grout.

APPENDIX B
Laboratory Testing Data

Particle Size Distribution Report



	+3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PL	PI
○	0.0	3.9	28.7	67.4					
□	0.0	9.2	82.1	8.7					
△	0.0	11.8	64.1	24.1					

SIEVE inches size	PERCENT FINER ○	□	△
3/4"		100.0	
1/2"		99.2	100.0
3/8"	100.0	99.0	96.3
GRAIN SIZE			
D_{60}		1.7122	0.4948
D_{30}		0.7243	0.1103
D_{10}		0.1327	
COEFFICIENTS			
C_c		2.31	
C_u		12.90	

SIEVE number size	PERCENT FINER ○	□	△
#4	96.1	90.8	88.2
#10	91.2	65.3	77.7
#20	86.5	34.8	69.1
#40	81.4	17.1	57.1
#60	76.4	12.5	45.5
#100	72.4	10.4	35.5
#140	70.0	9.4	29.3
#200	67.4	8.7	24.1

Material Description

○ Brown Sandy CLAY

□ Brown Well-Graded SAND w/ Silt

△ Yellowish Brown Clayey SAND

REMARKS:

○

□

△

○ Source of Sample: B-1 Depth: 5.5-6' Sample Number: 1-2

□ Source of Sample: B-1 Depth: 15.5-16' Sample Number: 1-6

△ Source of Sample: B-1 Depth: 30.5-31' Sample Number: 1-14

COOPER TESTING LABORATORY

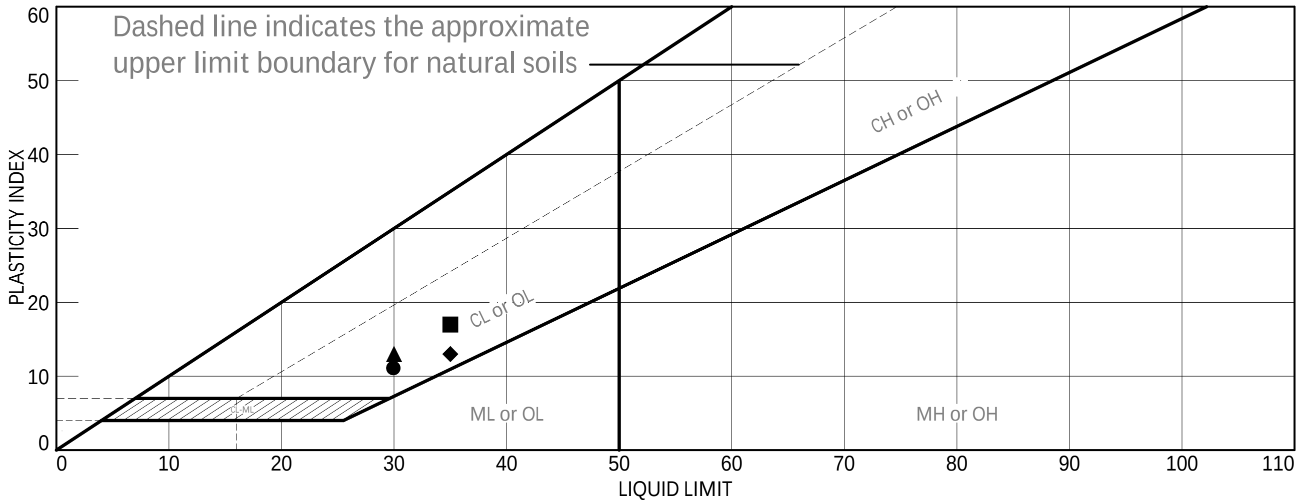
Client: Haley & Aldrich, Inc.

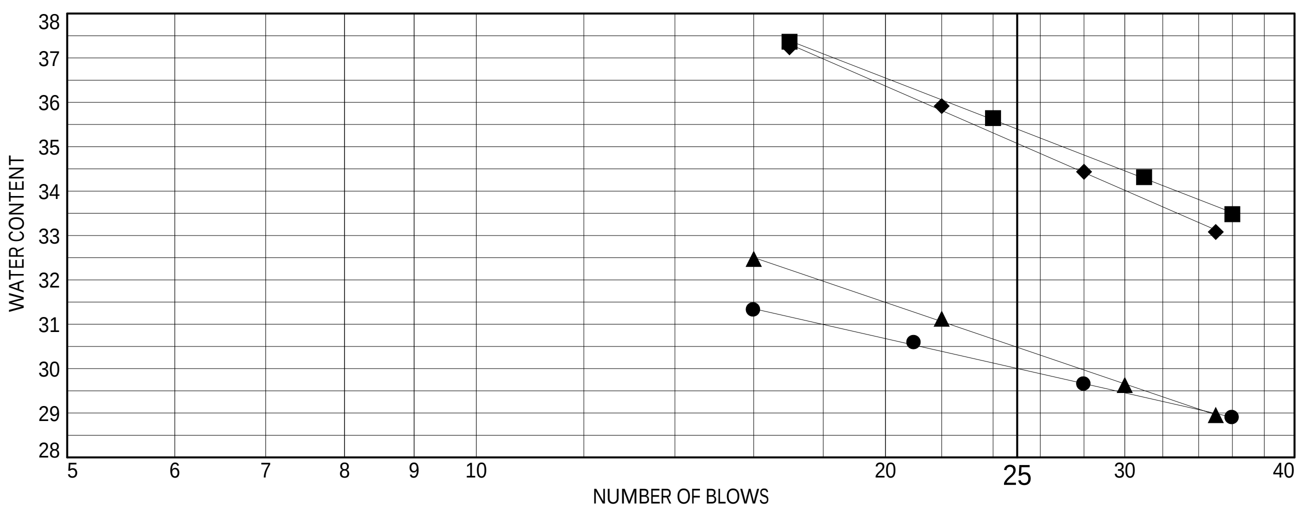
Project: E Palo Alto - 0211503

Project No.: 715-128

Figure

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Brown Lean Clayey SAND w/ Gravel	30	19	11			
■	Yellowish Brown Sandy Lean CLAY	35	18	17			
▲	Bluish Gray Sandy Lean CLAY w/ Gravel	30	17	13			
◆	Light Yellowish Brown Sandy Lean CLAY	35	22	13			

Project No. 715-128 Client: Haley & Aldrich, Inc.

Project: E Palo Alto - 0211503

●Source of Sample: B-1 Depth: 16-16.5' Sample Number: 1-7

■Source of Sample: B-1 Depth: 20.5-21' Sample Number: 1-9

▲Source of Sample: B-1 Depth: 36-36.5' Sample Number: 1-17

◆Source of Sample: B-1 Depth: 41-41.5' Sample Number: 1-19

Remarks:

COOPER TESTING LABORATORY

Figure

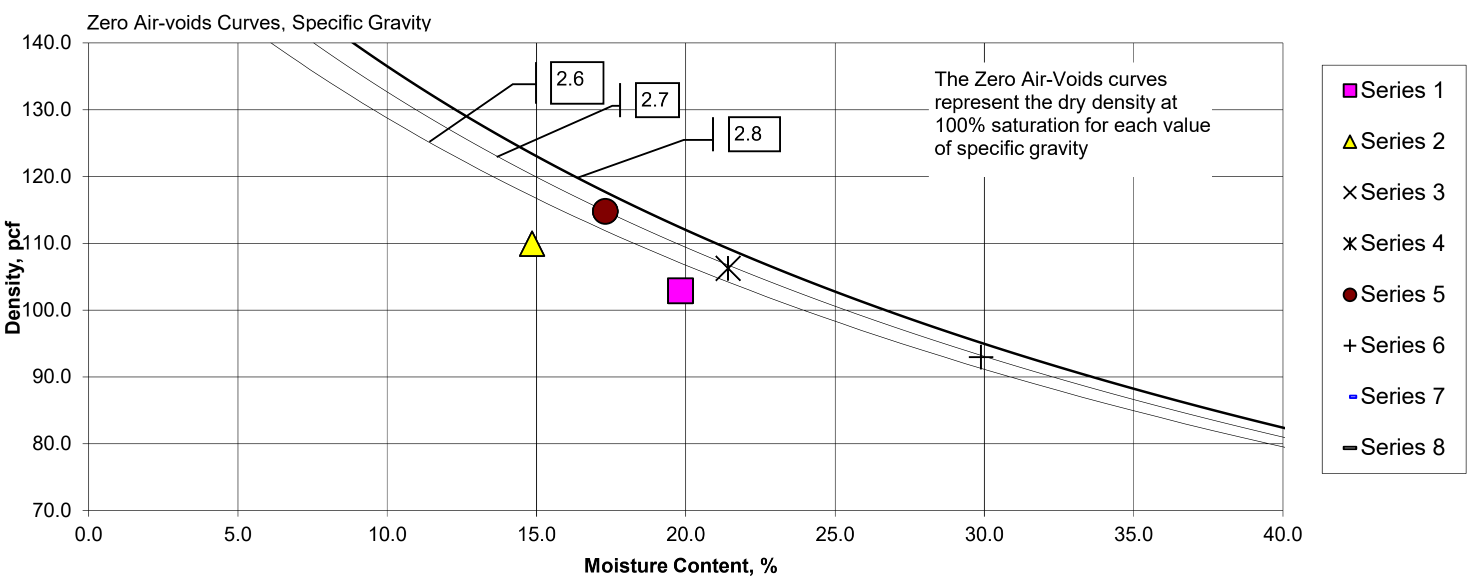
Moisture-Density-Porosity Report
Cooper Testing Labs, Inc. (ASTM D7263b)

CTL Job No:	715-128	**Project No.**	0211503	**By:**	RU
Client:	Haley & Aldrich	**Date:**	01/03/25		
Project Name:	E Palo Alto	**Remarks:**			

Boring:	B-1	B-1	B-1	B-1	B-1	B-1		
Sample:	1-2	1-5	1-6	1-7	1-14	1-19		
Depth, ft:	5.5-6	11-11.5	15.5-16	16-16.5	30.5-31	41-41.5		
Visual Description:	Brown Sandy CLAY	Brown Clayey SAND w/ Gravel	Brown Well-Graded SAND w/ Silt	Brown Lean Clayey SAND w/ Gravel	Yellowish Brown Clayey SAND	Light Yellowish Brown Sandy Lean CLAY		
Actual G_s								
Assumed G_s	2.70	2.70		2.70	2.70	2.70		
Moisture, %	19.8	14.8	14.8	21.4	17.3	29.9		
Wet Unit wt, pcf	123.3	126.3		129.0	134.6	120.8		
Dry Unit wt, pcf	102.9	110.0		106.3	114.8	93.0		
Dry Bulk Dens.pb, (g/cc)	1.65	1.76		1.70	1.84	1.49		
Saturation, %	83.9	75.3		98.6	99.7	99.3		
Total Porosity, %	39.0	34.8		37.0	31.9	44.8		
Volumetric Water Cont,Θw,%	32.7	26.2		36.5	31.8	44.5		
Volumetric Air Cont., Θa,%	6.3	8.6		0.5	0.1	0.3		
Void Ratio	0.64	0.53		0.59	0.47	0.81		
Series	1	2	3	4	5	6	7	8

Note: All reported parameters are from the as-received sample condition unless otherwise noted. If an assumed specific gravity (Gs) was used then the saturation, porosities, and void ratio should be considered approximate.

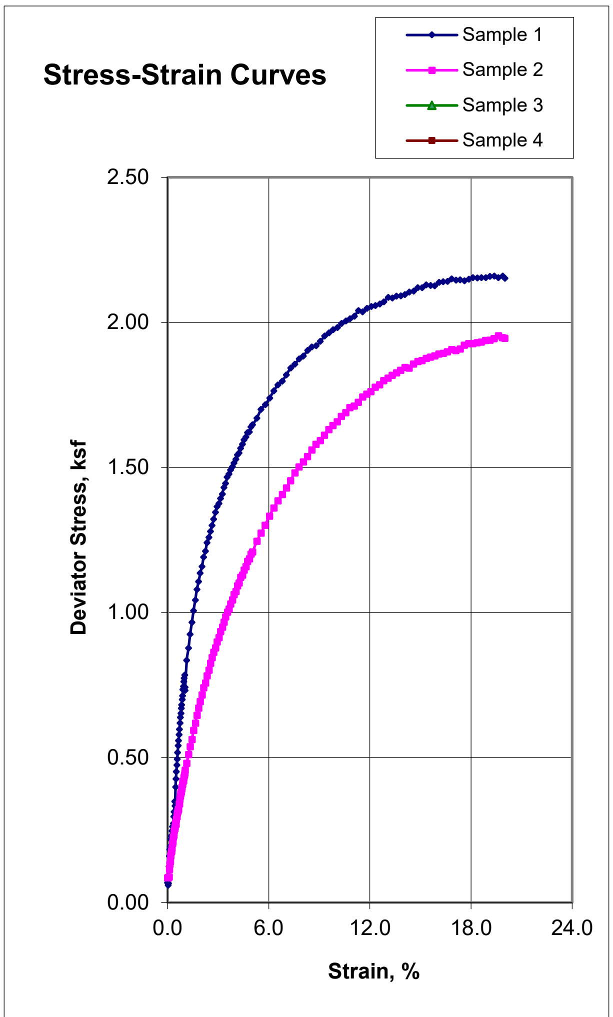
Moisture-Density



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

COOPER TESTING LABORATORY

Unconsolidated-Undrained Triaxial Test
ASTM D2850



Sample Data				
	1	2	3	4
Moisture %	27.9	19.5		
Dry Den,pcf	95.9	110.0		
Void Ratio	0.757	0.532		
Saturation %	99.5	98.9		
Height in	4.98	4.96		
Diameter in	2.41	2.41		
Cell psi	14.7	20.5		
Strain %	15.00	15.00		
Deviator, ksf	2.120	1.866		
Rate %/min	1.00	1.00		
in/min	0.050	0.050		
Job No.:	715-128			
Client:	Haley & Aldrich			
Project:	0211503			
Boring:	B-1	B-1		
Sample:	1-9	1-17		
Depth ft:	20.5-21	36-36.5		

Visual Soil Description	
Sample #	
1	Yellowish Brown Sandy Lean CLAY
2	Bluish Gray Sandy Lean CLAY w/ Gravel
3	
4	

Remarks:

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

COOPER TESTING LABORATORY

Corrosivity Test Summary

CTL # 715-128 **Date:** 1/8/2025 **Tested By:** PJ **Checked:** PJ
Client: Haley & Aldrich **Project:** E Palo Alto **Proj. No:** 0211503
Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP	Moisture	Soil Visual Description
Boring	Sample, No.	Depth, ft.	As Rec.	Minimum	Saturated	mg/kg	mg/kg	%		(Redox)	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		mv	%	
			ASTM G57	Cal 643	ASTM G57	Cal 422-mod.	Cal 417-mod.	Cal 417-mod.	Cal 643	SM 2580B	ASTM D2216	
B-1	1-9	20.5-21	-	2223	-	11	69	0.0069	8.1	-	4.0	Yellowish Brown Sandy Lean CLAY