
GEOTECHNICAL INVESTIGATION

The Harker School Bucknall Campus

4300 Bucknall Road

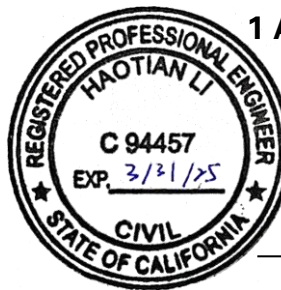
San Jose, California

Prepared For:

The Harker School
500 Saratoga Avenue
San Jose, California 95129

Prepared By:

Langan CA, Inc.
1 Almaden Boulevard, Suite 590
San Jose, California 95113



A handwritten signature in black ink that reads "Haotian Li".

Haotian Li, PE #C94457
Senior Staff Engineer



A handwritten signature in blue ink that reads "Serena T. Jang".

Serena T. Jang, GE #2702
Principal

21 March 2025
770703801

LANGAN

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1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by Langan CA, Inc. (Langan) for the proposed development at the Harker School Bucknall Campus, located at 4300 Bucknall Road in San Jose, California (Santa Clara County APN 403-59-012). The approximate location of the project site is shown on Figure 1.

The campus is bound by Bucknall Road on the north, South San Tomas Aquino Road on the east, Rincon Avenue on the south, and multiple private residential properties on the west, as shown on Figure 2. It is occupied by several 1- to 2-story classroom buildings (designated 100, 200, 300 and 500 Building), gymnasium, library, portable orchestra/music classroom building, swimming pool, soccer field and parking lot.

The 50% Schematic Design Plans (Studio Bondy Architecture, 2025) indicate the proposed development will include the demolition of some of the existing buildings including the library building and the portable orchestra/music classroom building, and the construction of the following:

- a new two-story multi-purpose building above one basement level for parking; the proposed basement will extend to approximately 15 feet below the first-floor level, excluding the thickness of foundations.
- a one-story at grade building addition to the existing 500 Building.

In addition, improvements to the existing driveways, parking lots, drop-off zones and sports fields are planned. The approximate locations of the proposed structures, and the approximate extent of the proposed basement are shown on Figure 2.

According to the topographic survey of the site (Kier+Wright, 2025), the existing ground surface within the project site is generally flat, with the ground surface elevation ranging between Elevation 221 and 227 feet¹. The finished first floor (ground floor) elevations of the proposed multi-purpose building and the proposed 500 Building addition will both be at Elevation

¹ All elevations referenced to NGVD29 vertical datum.

224.6 feet. The finished floor elevation for the multi-purpose building's basement is Elevation 213 feet (Studio Bondy Architecture, 2025). Except for the excavation for the multi-purpose building's basement, which is anticipated to be on the order of approximately 15 feet, we anticipate that minor cuts and fill (generally less than one foot) will be needed to rough grade the site. plan (Kier+Wright, 2025).

According to our communication with IMEG, the project structural engineer, we understand that the preliminary design dead plus live area loads of the proposed multi-purpose building will be on the order of 650 pounds per square foot (psf). Building loads for the 500 Building addition are currently not available.

2.0 SCOPE OF SERVICES

Our services were performed in general accordance with our proposal dated 30 October 2024. The purpose of our investigation was to provide conclusions and design-level recommendations regarding the geotechnical aspects of the proposed development. Our services included evaluating subsurface conditions and potential geologic hazards at the project site, and performing engineering analyses to develop conclusions and recommendations regarding:

- subsurface conditions including groundwater levels;
- expansion potential and infiltration rate of near surface soil;
- 2022 California Building Code (CBC) site classification, mapped values SS and S1, modification factors Fa and Fv and SMS and SM1 assuming the structure meets the exceptions of section 11.4.8 of ASCE 7-16 or the site is classified as Site Class C;
- site seismicity and potential for seismic hazards including liquefaction, lateral spreading, seismic densification induced settlement, and fault rupture, if any;
- most appropriate foundation type(s) for the proposed structure including deep foundations, if necessary;
- estimates of foundation settlement;
- design criteria for the recommended foundation type(s), including allowable bearing capacity, passive pressure, and coefficient of base friction for shallow foundations and vertical and lateral resistance of deep foundations, as appropriate;
- appropriate temporary shoring type and associated lateral earth pressures;
- static and dynamic lateral earth pressures on permanent walls;

- subgrade preparation for slab-on-grade floors and exterior slabs and flatwork, including sidewalks;
- site preparation, grading, drainage, and excavation, including criteria for fill quality and compaction;
- flexible and rigid pavements;
- soil corrosivity with brief evaluation; and
- construction considerations.

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

We began our investigation by reviewing the results of geotechnical investigations previously performed at the site. To further evaluate subsurface conditions at the site and obtain additional data, we drilled three borings at the site. To evaluate the infiltration rate for the soccer field area, we performed one infiltration rate test. We also collected four samples of the asphalt pavement and baserock for asbestos testing. The approximate locations of the borings, infiltration test and pavement/baserock sampling locations are presented on Figure 2.

Prior to performing our field exploration, we notified Underground Service Alert (USA) and checked the boring locations for detectable underground utilities using a private utility locator.

Details of the previous and current field exploration and laboratory testing are discussed in the remainder of this section.

3.1 Current Investigation Borings

Three borings, designated as LB-1 through LB-3, were drilled on 22 January 2025 using a truck-mounted drill rig equipped with hollow stem auger equipment and operated by Exploration Geoservices, Inc. of San Jose, California. The hollow stem auger borings were drilled to depths of approximately 20 to 30 feet below the existing ground surface (bgs).

Soil samples were obtained using one of two different driven split-barrel samplers. The sampler types are as follows:

- Sprague & Henwood (S&H) sampler with a 3.0-inch outside diameter and 2.5-inch inside diameter, lined with steel or brass tubes with an inside diameter of 2.43 inches.
- Standard Penetration Test (SPT) sampler with a 2.0-inch outside diameter and 1.38-inch

inside diameter.

The sampler types were chosen on the basis of soil type being sampled and desired sample quality for laboratory testing. In general, the S&H sampler was used to obtain samples in medium stiff to very stiff cohesive soil and the SPT sampler was used to evaluate the penetration resistance of sandy soil.

During drilling, the SPT and S&H samplers were driven with a 140-pound, automatic hammer falling 30 inches. The samplers were driven up to 18 inches and the hammer blows required to drive the samplers every six inches of penetration were recorded and are presented on the boring logs. A "blow count" is defined as the number of hammer blows per six inches of penetration or 50 blows for six inches or less of penetration. The driving of samplers was discontinued if the observed (recorded) blow count was 50 for six inches or less of penetration. The blow counts required to drive the S&H and SPT samplers with the automatic hammer were converted to approximate SPT N-values using factors of 0.8 and 1.3, respectively, to account for sampler type and hammer energy. The converted blow counts are shown on the boring logs.

Our engineer logged all the borings (LB-1 through LB-3) and obtained samples of the material encountered for visual classification and laboratory testing. Logs of the borings are presented in Appendix A as Figures A-1 through A-3. The soil encountered in the borings was classified in accordance with the Classification Chart presented on Figure A-4.

Upon completion, the boreholes were backfilled with soil cuttings.

The soil cuttings from the borings were collected in a 55-gallon drum, which was moved to a specific on-site temporary storage location, tested, and transported off-site for proper disposal.

3.2 Borings from Previous Investigations

We reviewed the results of a geotechnical investigation previously performed by others for the school campus (United Soil Engineering, Inc., 1997). The geotechnical investigation report included four soil borings, designated as B-1 through B-4, which were drilled to depths of approximately 20 feet bgs on 19 November 1997. We utilized the subsurface information from these borings to supplement our study of the subsurface conditions. The approximate locations of borings B-1 through B-4 are shown on Figure 2. The borings logs and laboratory testing results from the United Soil Engineering (USE) geotechnical investigation are included in Appendix B.

3.3 Infiltration Rate Test

We performed one in-situ infiltration rate test at the existing soccer field at the location designated as I-1 as shown on Figure 2. The test consisted of using a hand auger to excavate an approximately 3½-inch diameter borehole to a depth of about three feet bgs. A 1½-inch inner diameter PVC pipe was placed in the borehole and embedded two to four inches below the bottom of the boring. Then the borehole was filled with water and allowed to presoak for a period of 24 hours. After the presoak period was complete, water level readings were taken every 30 minutes. The water level readings were used to estimate a stabilized infiltration rate for the soil. The results of the infiltration rate tests are discussed in Section 7.10.

3.4 Laboratory Testing

The samples recovered from the current field investigation were examined to confirm their soil classification, and representative samples were selected for laboratory testing. Samples were tested to measure moisture content, dry density, fines content, shear strength, plasticity (Atterberg Limits), and R-value, as appropriate. Results of the laboratory tests are included on the boring logs and in Appendix C.

3.5 Soil Corrosivity Testing

To evaluate the corrosivity of the soil, corrosivity tests were performed on a soil sample obtained from a depth at approximately four feet bgs. The corrosivity of the soil sample was evaluated by CERCO Analytical using the following ASTM Test Methods:

- Redox – ASTM D1498
- pH – ASTM D4972
- Resistivity (100% Saturation) – ASTM G57
- Chloride – ASTM D4327
- Sulfate – ASTM D4327

The laboratory corrosion test results and brief corrosivity evaluations are presented in Appendix D and briefly discussed in Section 6.5.

3.6 Asbestos Testing

To determine the presence of asbestos in the site's pavement and baserock layer, we collected samples of asphalt concrete and the underlying baserock and submitted them to McCampbell Analytical, Inc. (MAI) of Pittsburg, California for analysis using the California Air Resources Board Method 435 (CARB 435). The approximate locations of samples AC-1 through AC-4 are shown on Figure 2, and the analytical report is included in Appendix E.

4.0 SUBSURFACE CONDITIONS

Where explored, the subsurface information indicates the site is underlain by approximately 9 to 10 feet of medium stiff to stiff clay and very loose to medium dense sand and gravel with varying amount of silt and clay. These layers are generally underlain by dense to very dense sand and gravel layers with varying amounts of silt and clay, interbedded with hard silt and clay layers, extending to the maximum depth explored of about 30 feet bgs. During the 1997 USE investigation, USE's borings indicated that very hard sandstone was encountered at depths of approximately 17 feet bgs. However, during our current investigation, sandstone was not encountered at the maximum depth explored.

Results of the Atterberg limits tests performed on two near-surface clay samples resulted in plasticity indexes (PIs) of 14 and 15, indicating relatively low expansion potential². The 1997 USE investigation reported a PI of 5 on a near surface silt sample. Where tested and where pocket penetrometers were used on cohesive samples, the undrained shear strength of the near-surface medium stiff to stiff clay ranged between 500 and 2,300 psf.

Groundwater was not encountered during the previous and current investigation. According to our review of the California Geological Survey (CGS) Seismic Hazard Zone Report for the San Jose West Quadrangle (CGS, 2002), historic high groundwater level in the project vicinity is on the order of 45 to 50 feet bgs. The groundwater level is likely influenced significantly by seasonal rainfall, groundwater pumping and drought conditions.

² Soil with low expansion potential undergoes minimal to no volume changes with changes in moisture content.

5.0 SEISMIC AND GEOLOGICAL HAZARDS

5.1 Regional Seismicity

The project site is in a seismically active region. Numerous earthquakes have been recorded in the region in the past, and moderate to large earthquakes should be anticipated during the service life of the proposed development. The Monte Vista – Shannon, San Andreas and Hayward faults are the major faults closest to the site. These and other faults of the region are shown on Figure 3. For each of these faults, as well as other active faults within about 50 kilometers (km) of the site, the distance from the site and estimated mean Moment magnitude³ [2014 Working Group on California Earthquake Probabilities (WGCEP) (2015) and Uniform California Earthquake Rupture Forecast Version 3 (UCERF3) as detailed in the United States Geological Survey Open File Report 2013-1165] are summarized in Table 1. The mean Moment magnitude presented on Table 1 was computed assuming full rupture of the segment using Hanks and Bakun (2008) relationship.

TABLE 2
Regional Faults and Seismicity

Fault Segment	Approx. Distance from fault (km)	Direction from Site	Mean Moment Magnitude
Monte Vista - Shannon	2.4	Southwest	7.0
San Andreas 1906 event	9.5	Southwest	8.1
Butano	13	Southwest	6.7
Sargent	16	South	6.8
Pilarcitos	17	West	6.7
Total Hayward-Rodgers Creek Healdsburg	19	East	7.6
Total Calaveras	22	East	7.5
Mission (connected)	24	Northeast	6.1
Total San Gregorio	33	West	7.6
Monterey Bay-Tularcitos	42	Southwest	7.2
Greenville	44	East	7.1

Note:

1. The table above is a summary and does not include all the fault segmentation, alternate traces and low activity faults included in the UCERF3 model.

³ Moment magnitude is an energy-based scale and provides a physically meaningful measure of the size of a faulting event. Moment magnitude is directly related to average slip and fault rupture area.

Figure 3 also shows the earthquake epicenters for events with magnitude greater than 5.0 from January 1800 through August 2014. Since 1800, four major earthquakes have been recorded on the San Andreas fault. In 1836 an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale (Figure 4) occurred east of Monterey Bay on the San Andreas fault (Topozada and Borchardt 1998). The estimated Moment magnitude, M_w , for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to an M_w of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas fault from Shelter Cove to San Juan Bautista approximately 470 kilometers in length. It had a maximum intensity of XI (MM), an M_w of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The Loma Prieta Earthquake occurred on 17 October 1989 in the Santa Cruz Mountains with an M_w of 6.9, the epicenter of which is approximately 29 km from the site.

In 1868 an earthquake with an estimated maximum intensity of X on the MM scale occurred on the southern segment (between San Leandro and Fremont) of the Hayward fault. The estimated M_w for the earthquake is 7.0. In 1861, an earthquake of unknown magnitude (probably an M_w of about 6.5) was reported on the Calaveras fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill earthquake ($M_w = 6.2$).

The 2016 U.S. Geologic Survey (USGS) predicted a 72 percent chance of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Area in 30 years (Aagaard et al. 2016). More specific estimates of the probabilities for different faults in the Bay Area are presented in Table 2.

TABLE 2
Estimates of 30-Year Probability (2014 to 2043) of a
Magnitude 6.7 or Greater Earthquake

Fault	Probability (percent)
Hayward-Rodgers Creek	33
Calaveras	26
N. San Andreas	22
Greenville	16
San Gregorio	6

5.2 Fault Rupture

Historically, ground surface fault rupture closely follows the traces of geologically young faults. The site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act and no known active or potentially active faults exist on the site. Additionally, the site is not located within a Fault Rupture Hazard Zone by maps prepared by Santa Clara County (2012). We therefore conclude the risk of fault offset rupture at the site from a known active fault is low. In a seismically active area, the remote possibility exists for future faulting in areas where no faults previously existed; however, we conclude the risk of surface faulting and consequent secondary ground failure is low.

5.3 Liquefaction and Associated Hazards

The site is in a seismically active area and will likely be subjected to strong shaking during a major earthquake. Strong shaking during an earthquake can result in ground failures such as those associated with soil liquefaction⁴, lateral spreading⁵, and seismic densification⁶. Each of these conditions has been evaluated based on our review of available information, field investigation, and results of our analysis, and is discussed in this section.

5.3.1 Liquefaction and Lateral Spreading

When a saturated soil with little to no cohesion liquefies during a major earthquake, it experiences a temporary loss of shear strength as a result of a transient rise in excess pore water pressure generated by strong ground motion. Flow failure, lateral spreading, differential settlement, loss of bearing, ground fissures, and sand boils are evidence of excess pore pressure generation and liquefaction.

The site is not located within a zone designated with the potential for liquefaction, as identified by the California Geological Survey (formerly the California Division of Mines and Geology), on map titled, "Earthquake Zones of Required Investigation, San Jose West Quadrangle" (CGS,

⁴ Liquefaction is a phenomenon in which saturated (submerged), cohesionless soil experiences a temporary loss of strength because of the buildup of excess pore water pressure, especially during cyclic loading such as those induced by earthquake. Soils most susceptible to liquefaction are loose, clean, saturated, uniformly graded, fine-grained sand.

⁵ Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported downslope or in the direction of a free face by earthquake and gravitational forces.

⁶ Seismic densification (also referred to as cyclic densification or differential compaction) is a phenomenon in which non-saturated, cohesionless soil is densified by earthquake vibrations, causing ground surface settlement.

2002). Additionally, the site is also not located within a Liquefaction Hazard Zone by maps prepared by Santa Clara County (2012).

Following the 1989 Loma Prieta Earthquake and 1906 San Francisco Earthquake studies of the Bay Area were conducted to quantify and record damages caused by liquefaction and lateral spreading. These reports, by Holzer in 1998 and Youd & Hoose in 1978, do not indicate evidence of liquefaction was observed at or near the site. The groundwater at the site is deep, which is anticipated to be greater than 45 to 50 feet bgs. Therefore, we conclude the potential for liquefaction and lateral spreading to be low for this site.

5.3.2 Seismic Densification

Seismic densification can occur during strong ground shaking in loose, granular deposits above the water table, resulting in ground surface settlement. The degree of susceptibility to seismic densification is directly related to the relative density of the existing granular soils. Seismic densification of non-saturated, cohesionless soil following a major earthquake was analyzed using Pradel (1998). At borings LB-1 and LB-2 (at the new multi-purpose building), the potential for seismic densification is low because the sand and gravel layers encountered above the groundwater table are sufficiently dense or have sufficient fines. However, in boring LB-3 (at the 500 Building addition), a four-foot-thick layer of loose to medium dense gravel with clay and sand was encountered at a depth of 5½ feet bgs. We estimate seismic densification settlements in this layer will be less than ¾ inch during a major earthquake with differential settlement less than ½ inch over 30 feet.

5.4 Tsunami

Recent maps published by the California Department of Conservation (2021) indicate the project site is not within the tsunami inundation zone; therefore, we conclude the potential risk by inundation from tsunami to be low within the project site. However, these maps may not account for the impact of sea level rise on the potential risk of inundation from a tsunami; the project civil engineer should evaluate the impact of sea level rise on the potential risk of inundation from a tsunami over the life of the buildings.

5.5 Flooding

The project site is located within Federal Emergency Management Agency (FEMA) Flood Zone D, designated as “areas in which flood hazards are undetermined, but possible,” (FEMA 2009). The project civil engineer should further evaluate the potential risk of inundation from flooding.

6.0 DISCUSSION AND CONCLUSIONS

Based on our understanding of the proposed development and the subsurface conditions at the site, we conclude the project may be developed as planned. The primary geotechnical issues for this project include:

- selection of appropriate foundation systems to support the building loads and accommodate estimated static and seismic settlements;
- selection of appropriate temporary shoring type for the basement construction and associated lateral earth pressures.

Our conclusions and discussions regarding these and other geotechnical issues are discussed in the remainder of this report.

6.1 Foundations and Settlement

The primary considerations related to the selection of a safe, economical foundation system for the proposed structure are the final excavation depths, final building loads, and the anticipated building settlements under the new building loads and during an earthquake.

According to our communication with IMEG, the project structural engineer, we understand that the preliminary design dead plus live area loads for the proposed multi-purpose building will be on the order of 650 psf. The design building loads of the proposed at-grade building addition are currently not available, but based on the building type and level, we estimate the general column load would likely be on the order of 100 to 150 kips.

6.1.1 Multi-Purpose Building with Basement

Assuming foundation thickness of approximately three feet, we anticipate that a one-level basement will require an excavation depth of about 15 feet bgs. Because the excavation depth for the basement will result in a small net increase in overburden pressure, we anticipate settlements in the clay layers below will consist primarily of recompression.

At the excavation subgrade, we anticipate the soil exposed will consist of very dense gravel and clayey gravel layers. We conclude the building can be supported on shallow foundations. According to our discussion with IMEG, we understand spread footings and a mat foundation are both being considered for the multi-purpose building. Localized soft soil, if encountered under

the footing locations or mat slab, should be excavated and recompact with lean or structural concrete.

Design recommendations for the spread footing and mat foundations are presented in Section 7.2. Footings and mat foundations designed in accordance with these recommendations should settle less than $\frac{1}{2}$ inch; differential settlement between adjacent footings, typically 20 to 25 feet apart, should not exceed $\frac{1}{4}$ inch.

6.1.2 500 Building Addition (At-Grade)

For the at-grade 500 Building addition, we anticipate the soil at the foundation subgrade will consist of medium stiff to stiff sandy clay. We conclude that the building can be supported on shallow footing foundations. Localized soft soil, if encountered under footing locations, should be excavated and recompact with lean or structural concrete.

Design recommendations for the footings are presented in Section 7.2. Footings designed in accordance with these recommendations should not settlement more than $\frac{1}{2}$ inch; differential settlement between adjacent footings, typically 20 feet apart, should not exceed $\frac{1}{4}$ inch. In addition, as previously discussed in Section 5.3, we anticipate up to $\frac{3}{4}$ inch of seismically induced settlement.

6.2 Groundwater Considerations

Groundwater was not encountered at the site in the current and previous borings up to the maximum explored depths of about 30 feet bgs. Based on our review of the CGS Seismic Hazard Zone Report for the San Jose West Quadrangle (CGS, 2002), historic high groundwater level in the project vicinity is on the order of 45 to 50 feet bgs, corresponding to Elevation 182 feet to Elevation 171 feet.

On the basis of our knowledge of groundwater in the area, we conclude Elevation 182 feet can be used as the design groundwater elevation.

6.3 Temporary Shoring and Excavations

Our discussion regarding shoring and excavation are presented in the following subsections.

6.3.1 Excavation and Temporary Shoring

Where sufficient space is available, the excavation for the basement may be sloped back. If there is insufficient space to slope the proposed excavation, the excavation should be shored. There are several key considerations in selecting a suitable shoring system. Those we consider of primary concerns are:

- protection of surrounding improvements, including buildings, roadways, and utilities;
- the ability of the shoring system to reduce potential for ground movement;
- presence of existing foundations or other obstructions;
- cost.

Several shoring systems may be considered for the project, including sheet piles and soldier beams and lagging.

Sheet piles consist of interlocking steel sheets vibrated into the ground to create a continuous shoring wall. Once the basement excavations are made and the basement structural concrete is at design strength (as determined by the structural engineer), the sheets typically can be vibrated out, saving on material costs. However, it may not be possible to drive sheet piles through areas where dense to very dense clayey sand or gravel is present. In addition, because sheet piles are installed and removed using vibration, we would not typically recommend their use close to adjacent buildings, as the vibrations can cause some ground settlement.

A cantilever soldier-pile-and-lagging system consists of steel beams that are placed in predrilled holes; the annulus between the beams and sides of the hole are backfilled with concrete. Alternatively, the steel beams may be 'wet-set' in soil-cement mixed columns. Wood lagging is placed between the soldier beams as excavation proceeds.

If a cantilever soldier-pile-and-lagging shoring system does not provide adequate lateral resistance, we can provide additional recommendations for tiebacks or internal bracing to provide additional lateral resistance and limit shoring deflections. If tiebacks extend below neighboring properties or streets, encroachment permits/permission from the neighbor(s) or the City of San Jose will be required.

The shoring contractor should review the subsurface conditions and decide which shoring system is best suited for the project.

6.3.2 Excavation Considerations and Monitoring

During excavation, shoring systems are expected to move and deform, which could cause surrounding improvements to settle. The magnitude of shoring movements and resulting settlements are difficult to estimate because they depend on many factors, including the method of installation and the contractor's skill in installing the shoring. Typical maximum movement for a properly designed and constructed shoring system should be less than one inch. A monitoring program should be established to evaluate the effects of the construction on surrounding improvements. The Contractor should install surveying points to monitor the movement of shoring and settlement of adjacent structures and the ground surface during excavation. The monitoring should provide timely data which can be used to modify the shoring system if needed.

6.4 Construction Considerations

We anticipate soil to be excavated from the site can be removed with conventional earthmoving equipment such as excavators and backhoes. The areas of the proposed development are currently occupied by part of a soccer field, pavement, and some student play structures. Buried foundations from these structures and possibly previously existing structures may require the use of jackhammers or hoe-rams to break apart and remove.

If site grading is performed during wet weather, the exposed soil subgrade may become wet and difficult to compact; it may need to be aerated during dry weather. The grading contractor should be prepared to repair weak and wet subgrade, if required. In addition, heavy vibratory equipment should not be used during site preparation and grading, especially near the foundation subgrade elevation.

6.5 Corrosion Potential

Because corrosive soil can adversely affect underground utilities and foundation elements, laboratory testing was performed to evaluate the corrosivity of the soil at the project site.

CERCO Analytical performed tests on soil samples to evaluate corrosion potential to buried metals and concrete. The results of the tests are presented in Table 3 and Appendix D.

TABLE 3
Summary of Corrosivity Test Results

Test Boring	Sample Depth (feet)	pH	Sulfates (mg/kg)	Resistivity (ohms-cm)	Redox (mV)	Chlorides (mg/kg)
LB-1	4	7.10	47	1,700	190	38

Based upon resistivity measurement, the soil sample tested is classified as “corrosive” to buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron.

A brief evaluation of the corrosivity of the soil sample obtained during our investigation is presented in Appendix D. For more detailed recommendations regarding the corrosion protection of buried metals and concrete, a licensed corrosion consultant should be consulted.

7.0 RECOMMENDATIONS

From a geotechnical standpoint, the site can be developed as planned, provided the recommendations presented in this section of the report are incorporated into the design and contract documents. Criteria for foundation design, together with recommendations for site preparation, fill placement, shoring design, below-grade walls and seismic design are presented in this section of the report.

7.1 Earthwork

The following subsections present recommendations for site preparation, grading, and fill placement for the project.

7.1.1 Site Preparation and Grading

Demolition in areas to be developed should include removal of existing pavement and underground obstructions, including foundations of former structures. The resulting excavation should be properly backfilled based on the recommendations presented in this section.

Any vegetation and organic topsoil should be stripped in areas to receive new site improvements. Stripped organic soil can be stockpiled for later use in landscaped areas, if approved by the owner and architect; organic topsoil should not be used as compacted fill.

If encountered, any demolished asphalt and concrete at the site may be crushed to provide recycled construction materials, including sand, free-draining crushed rock, and potentially

Class 2 aggregate base (AB) provided it is properly crushed and screened and acceptable from an environmental standpoint. Where recycled material will be used as a crushed rock in applications where free-draining materials are required (except for capillary break), it should have no greater than six percent of material passing the 3/8-inch sieve. Where recycled Class 2 AB will be used beneath pavements, it should meet requirements of the State of California Department of Transportation (Caltrans) Standard Specifications. Recycled Class 2 AB that does not meet the Caltrans specifications should not be used beneath streets but could be acceptable for use as general site fills within building pads and beneath concrete flatwork, provided it meets the requirements for site fills as presented later in this section.

Existing underground utilities beneath areas to receive new improvements should be removed or abandoned in-place by filling them with grout. The procedure for in-place abandonment of utilities should be evaluated on a case-by-case basis and will depend on location of utilities relative to new improvements. However, in general, existing utilities within four feet of final grades should be removed, and the resulting excavation should be properly backfilled based on the recommendations presented in this section.

7.1.2 Fill Placement

All site fill should meet the following criteria:

- be free of organic matter;
- contain no rocks or lumps larger than three inches in greatest dimension;
- have a low expansion potential (defined by a liquid limit of less than 40 and plasticity index lower than 12);
- have a low corrosion potential⁷;
- be approved by the geotechnical engineer.

In general, the existing surface soil is not expected to meet the criteria for fill. A sample of proposed fill(s) should be submitted to the geotechnical engineer for testing at least three business days prior to use at the site.

⁷ Low corrosion potential is defined as a minimum resistivity of 2,000 ohms-cm and maximum sulfate and chloride concentrations of 250 parts per million.

Fill should be placed in horizontal lifts not exceeding eight inches in loose thickness, moisture-conditioned to near or above the optimum moisture content and compacted to at least 90 percent relative compaction⁸. For fill placed in the upper six inches of the soil subgrade within pavement areas, it should be compacted to at least 95 percent relative compaction. If used, sand containing less than 10 percent fines (particles passing the No. 200 sieve) should be compacted to at least 95 percent relative compaction. In areas where the fill will be thicker than five feet, the fill in this area should be compacted to at least 95 percent relative compaction for its entirety. Samples of on-site and proposed import fill materials should be submitted to the geotechnical engineer for approval at least three business days prior to use at the site.

7.1.3 At-Grade Improvements

Areas that will receive site improvements (e.g., sidewalks and exterior concrete flatwork) should be stripped of existing improvements or organics. Prior to placing fill or new at-grade improvements, the subgrade exposed after stripping should be scarified to a depth of at least eight inches, moisture-conditioned to near or above the optimum moisture content and compacted to at least 90 percent relative compaction.

The upper six inches at the final proposed soil subgrade within vehicular pavement areas should be compacted to at least 95 percent relative compaction.

A minimum of six inches of Class 2 AB should be placed beneath exterior vehicular concrete flatwork. Class 2 AB should be compacted to at least 95 percent relative compaction.

7.2 Foundations

The following section provides recommendations for spread footings and mat foundations.

7.2.1 Spread Footing Foundations

A firm subgrade should be exposed at the bottom of the proposed footing excavations. If isolated areas of soft material are encountered in the bottom of the excavation, they should be removed to expose firm material. Resulting overexcavations should be backfilled with lean or structural concrete.

⁸ Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557 laboratory compaction procedure.

We recommend spread footings within the excavation for the multi-purpose building (assumes an excavation depth of at least 10 feet bgs) have a minimum embedment of 18-inches below the lowest adjacent soil subgrade. With the recommended minimum embedment depth, we recommend an allowable dead plus live load bearing capacities of 5,000 psf. The bearing pressure may have a one-third increase for total loads, including wind and/or seismic loads.

For footings supporting the at-grade 500 Building addition, we recommend a minimum embedment of 18-inches below the lowest adjacent soil subgrade. For the recommended minimum embedment, footings bearing on firm native soil or engineered fill may be designed for an allowable bearing pressure of 2,500 psf for dead plus live loads, with a one-third increase for total loads, including wind and/or seismic loads.

Footings should be at least 18 inches wide for continuous footings and 24 inches for isolated spread footings. Footings adjacent to utility trenches (or other footings) should bear below an imaginary 1.5:1 (horizontal to vertical) plane projected upward from the bottom edge of the utility trench (or adjacent footings).

Lateral forces can be resisted by a combination of friction along the base of the footing, and passive resistance against the vertical faces of the foundation and, where applicable, the basement walls perpendicular to the direction of earthquake shaking. Frictional resistance should be computed using a base friction coefficient of 0.30. To calculate the passive resistance against the vertical faces of footings supporting at-grade structures, we recommend an equivalent fluid weight of 290 pounds per cubic foot (pcf). The upper foot should be ignored unless confined by a slab. The values for the friction coefficient and passive pressures include a factor of safety of 1.5.

If weak soil is encountered at the bottom of the footing excavation, it should be overexcavated and replaced with lean or structural concrete. We should observe the footing subgrade prior to placement of reinforcing steel. The excavation for the footings should be free of standing water, debris, and disturbed materials prior to placing concrete.

7.2.2 Mat Foundation

For the multi-purpose building, we understand a mat foundation is being considered. We estimated settlement for a proposed mat foundation using a building foundation pressure of 650 psf provided to us by the structural engineer and assumed to be uniform over the building footprint. Our settlement analyses indicate total settlement under the anticipated building loads

should be less than $\frac{1}{2}$ inch. Differential settlement will depend on the rigidity of the mat. To design the mat using the modulus of subgrade reaction method, we recommend a modulus of subgrade reaction of 75 kips per cubic foot (kcf). The modulus value is representative of the anticipated settlement under the building loads. After the mat analysis is completed, we should review the computed settlement and bearing pressure profiles to check that the modulus value is appropriate.

Resistance to lateral loads can be mobilized by a combination of passive pressure acting against the vertical faces of the mat and friction along the base of the mat. Passive resistance may be calculated using lateral pressures corresponding to an equivalent fluid weight of 290 pcf. Frictional resistance should be computed using a base friction coefficient of 0.30. These values include a factor of safety of about 1.5 and may be used in combination without reduction.

If weak or non-engineered fill is encountered in the mat excavation bottom, it should be over-excavated and replaced with lean or structural concrete. We should observe mat subgrade prior to placement of reinforcing steel. The excavation for the mat should be free of standing water, debris, and disturbed materials prior to placing concrete.

7.3 Below-Grade and Retaining Walls

We recommend all below-grade walls (basement and other retaining walls) be designed to resist lateral pressures imposed by the adjacent soil, vehicles, and any surcharge loads. Walls that are free to rotate should be designed for the active condition; walls that are restrained should be designed for the at-rest condition. The active and at-rest pressures are presented in Table 5 for both drained and undrained conditions.

Because the site is in a seismically active area, the design for structures assigned to Seismic Design Category D, E, or F and retaining more than six feet of backfill height should be checked for seismic conditions⁹. Under seismic loading conditions, there will be a seismic pressure increment that should be added to active earth pressures (Sitar et al., 2012). We used the procedures outlined in Sitar et al. (2012) and the peak ground acceleration based on the Design Earthquake (DE) and Maximum Considered Earthquake (MCE) ground motion level to compute the seismic pressure increment. For seismic conditions, we recommend basement walls be designed for the more critical loading condition of restrained (at-rest) pressure or total

⁹ California Building Code (2022) Section 1803.5.12.

pressures (active plus seismic increment) using the equivalent fluid weights and pressures presented in Table 5.

TABLE 5
Below-Grade Wall Design Earth Pressures

	Static Conditions		Seismic Conditions ¹	
	Unrestrained Walls (Active)	Restrained Walls (At-Rest)	Total – Active Plus Seismic Pressure Increment	
			DE	MCE
Drained Conditions ²	35 pcf ³	55 pcf	80 pcf	100 pcf
Undrained Conditions	80 pcf	90 pcf	100 pcf	115 pcf

Notes:

1. The more critical condition of either at-rest pressure for static conditions or active pressure plus a seismic pressure increment for seismic conditions should be checked for basement walls retaining more than six feet of backfill height.
2. If the drained condition values are used in design, walls will need to be back drained to reduce the potential for the buildup of hydrostatic pressures and should not be located within 10 feet of a bio-retention or other storm water infiltration systems.
3. pcf = pounds per cubic foot

Where traffic will pass within 10 feet of basement walls, temporary traffic loads should be considered in the design of the walls. Traffic loads may be modeled by a uniform pressure of 100 psf applied in the upper 10 feet of the walls.

If surcharge loads occur above an imaginary 1½:1 (horizontal: vertical) line projected up from the bottom of a below-grade wall, a surcharge pressure should be included in the wall design. If this condition exists, we should be consulted to estimate the surcharge pressure on a case-by-case basis.

If basement walls are designed to resist the lateral forces from the building, we recommend using passive pressures corresponding to an equivalent fluid unit weight of 290 pcf; the structural capacity of the walls should be checked for this loading condition.

The lateral earth pressures recommended for the drained condition are applicable to walls that are above the groundwater level and are backdrained to prevent the buildup of hydrostatic pressure, and the walls should not be located within 10 feet of a bio-retention or other storm water infiltration systems. If any of these conditions are not met, the walls should be designed using lateral earth pressures for the undrained condition.

For back-draining walls, one acceptable method is to place a prefabricated drainage panel against the back side of the walls. The drainage panel should extend down to a four-inch-diameter perforated PVC collector pipe at the base of the walls. The pipe should be surrounded on all sides by at least four inches of Caltrans Class 2 permeable material (see Caltrans Standard Specifications Section 68-1.025) or wrapped in filter fabric (Mirafi 140N or equivalent). We should check the manufacturer's specifications regarding the proposed prefabricated drainage panel material to verify it is appropriate for its intended use. The pipe should be connected to a suitable discharge point. As an alternative to using prefabricated drainage panel, the wall may be drained using Caltrans Class 2 permeable material (Caltrans Standard Specifications Section 68-1.025) or clean drain rock wrapped in a geotextile filter fabric (Mirafi 140N or equivalent). The gravel drain should be at least 12 inches wide and should extend up the back of the wall to about 2 feet below the ground surface; the upper 2 feet should be covered with a cap of native soil to reduce infiltration of surface water. A four-inch-diameter perforated PVC collector pipe should be placed within the gravel blanket near the base of the wall to drain the water to a suitable discharge. The pipe should be surrounded on all sides by at least four inches of Caltrans Class 2 permeable material or drain rock.

Wall backfill should be compacted to at least 90 percent relative compaction using light compaction equipment. Wall backfill with less than 10 percent fines, or deeper than five feet, should be compacted to at least 95 percent relative compaction for its entirety. If heavy equipment is used, the wall should be appropriately designed to withstand loads exerted by the equipment and/or temporarily braced.

To protect against moisture migration, below-grade walls should be waterproofed, and water stops placed at all construction joints. Waterproofing, if used, should be placed directly against the backside of the walls and according to manufacturer's specifications.

7.4 Excavations

Excavations that will be deeper than five feet and will be entered by workers should be shored or sloped in accordance with the Occupational Safety and Health Administration (OSHA standards (29 CFR Part 1926). For the proposed basement, we anticipate excavations on the order of 15 feet bgs may be required. This excavation can be made with temporary slopes of about 1½:1 (horizontal to vertical), if space permits. If sloped cuts are greater than 15 feet in height, they should be analyzed on a case-by-case basis. Additionally, if excavations with temporary

slopes are left open through the winter, proper drainage at the top of the slope should be maintained and hydro-seeding of the slope may be necessary to reduce erosion.

7.5 Shoring Design

Where there is inadequate space for temporary slopes, we recommend a cantilever soldier-pile-and-lagging shoring system be used. The shoring system should be designed to resist an active earth pressure of 35 pcf. The shoring should be designed to limit ground deformations to less than one inch.

The passive resistance of the shoring system may be designed using an equivalent fluid weight of 290 pcf. Passive pressures can be assumed to act on an area of three pile diameters assuming the compressive strength of the lean concrete in the toe of the soldier pile is greater than 100 psi and that the soldier piles are spaced at least three times the pile diameter (including concrete). The passive pressure includes a factor of safety of about 1.5.

The shoring designer should evaluate the required embedment depth of the soldier piles. Furthermore, the designer should determine the type and size of shoring members required to resist the earth pressures presented in this section.

If the shoring will be designed to provide lateral support of adjacent structures, then the pressure will need to be revised to account for building surcharges.

If traffic is within a distance equal to the shoring depth, a uniform surcharge load of 100 psf acting on the upper 10 feet should be used in the design. An increase in lateral design pressure for the shoring may be required where heavy construction equipment or stockpiled materials will be within a distance equal to the shoring depth in feet. The increase in pressure should be determined after the surcharge loads are known. The anticipated deflections of the shoring system should be estimated to check if they are acceptable. The shoring system should be sufficiently rigid to prevent detrimental movement of the temporary shoring and possible damage to adjacent improvements.

The shoring system should be designed by a licensed civil engineer experienced in the design of retaining systems and installed by an experienced shoring specialty contractor. The shoring engineer should be responsible for the design of temporary shoring in accordance with applicable regulatory requirements. Control of ground movement will depend on the timeliness of

installation of lateral restraint and on the design. We should review the shoring plans and a representative from our office should observe the installation of the shoring.

The contractor should establish survey points on the shoring and on adjacent buildings and streets within 100 feet of the excavation perimeter prior to the start of excavation. These survey points should be used to monitor the vertical and horizontal movements of the shoring and surrounding structures and streets during construction. The contractor should also survey and take photographs of the adjacent buildings prior to the start of excavation and immediately after its completion.

7.6 Floor Slab

Floor slabs may be designed to bear on grade. We recommend that the slab-on-grade should have a minimum thickness of six inches. Where soft or loose soil is present at the subgrade elevation, the weak soil should be removed and replaced with engineered fill or lean concrete.

If the subgrade is disturbed during excavation for footings and utilities, it should be re-rolled. Loose, disturbed materials should be excavated, removed, and replaced with engineered fill during final subgrade preparation.

Moisture is likely to condense on the underside of the at-grade floor slabs, basement slabs or mat, even though they will be above the groundwater table. Consequently, a moisture barrier should be considered if movement of water vapor through the slab/mat would be detrimental to its intended use. A typical moisture barrier consists of a capillary moisture break and a water vapor retarder. The moisture barrier should be placed directly beneath the slab/mat. A moisture barrier is generally not required beneath parking garage slabs, except for areas beneath mechanical, electrical, and storage rooms. If moisture is acceptable on the slab/mat, the vapor retarder and capillary break can be replaced with at least six inches of Class 2 aggregate base, compacted to at least 95 percent relative compaction.

The vapor retarder should meet the requirements for Class C vapor retarders stated in ASTM E1745. The vapor retarder should be placed in accordance with the requirements of ASTM E1643. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder. The capillary moisture break should consist of at least four inches of clean, free-draining gravel or crushed rock; recycled material should not be used for the capillary break. The particle size of the gravel/crushed rock should meet the gradation requirements presented in Table 6.

TABLE 6
Gradation Requirements for Capillary Moisture Break

Sieve Size	Percentage Passing Sieve
<i>Gravel or Crushed Rock</i>	
1 inch	90 – 100
3/4 inch	30 – 100
1/2 inch	5 – 25
3/8 inch	0 – 6

Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab. Therefore, concrete for the floor slab should have a low w/c ratio - less than 0.45. The slab should be properly cured. Before the floor covering is placed, the contractor should check that the concrete surface and the moisture emission levels (if emission testing is required) meet the manufacturer's requirements.

7.7 Seismic Design

The site is classified as a stiff soil site, Site Class D. A site-specific ground motion analysis is required for structures on Site Class D sites with S_1 greater than or equal to 0.2, in accordance with the 2022 CBC and by reference ASCE 7-16, unless the criteria listed in the exception in Section 11.4.8 of ASCE 7-16 as modified by Supplement 3 of ASCE 7-16 are met. If the project structural engineer determined that these criteria will be met for the project, for seismic design in accordance with the provisions of 2022 CBC/ASCE 7-16 and Supplement 3, we recommend the following:

- Risk-Targeted Maximum Considered Earthquake (MCE_R) ground motion parameter spectral response acceleration (5% critical damping) for 0.2 seconds (S_s) and 1 second (S_1) of 2.050g and 0.733g, respectively.
- Site Class D.
- Site Coefficients F_a and F_v of 1.0 and 1.7, respectively.
- MCE_R and DE spectral response acceleration parameters at short periods, S_{MS} and S_{DS} , of 2.050g and 1.367g, respectively.
- MCE_R and DE spectral response acceleration parameters at one-second period, S_{M1} and

S_{D1} , of 1.869g and 1.246g, respectively (these values have been increased by 50percent in accordance with ASCE 7-16 Supplement 3).

7.8 Asphalt Pavement

The State of California flexible pavement design method was used to develop the recommended asphalt concrete pavement sections. We expect the final soil subgrade in asphalt-paved areas will generally consist of on-site soil. From the laboratory test results, we selected an R-value of 10 for design.

For our calculations, we assumed a Traffic Index (TI) of 4 for automobile parking areas with occasional trucks, and 5 to 6 for driveways and truck-use areas; these TIs should be confirmed by the project civil engineer. Table 7 presents our recommendations for asphalt pavement sections.

TABLE 7
Pavement Section Design

TI	Asphalt Concrete (inches)	Class 2 Aggregate Base R = 78 (inches)
4	2.5	7
5	3	9
6	3.5	11.5

Pavement components should conform to the current Caltrans Standard Specifications. The upper six inches of the soil subgrade in pavement areas should be moisture-conditioned to above optimum and compacted to at least 95 percent relative compaction and rolled to provide a smooth non-yielding surface. Aggregate base should be compacted to at least 95 percent relative compaction.

7.9 Concrete Pavement and Exterior Slabs

Where non-vehicular concrete flatwork is supported on soil, we recommend it be underlain by at least four inches of Class 2 AB or City of San Jose minimum standards. Class 2 AB should be compacted to at least 90 percent relative compaction. The subgrade should be prepared as recommended in Section 7.1.3.

For loading and service areas, we recommend a minimum of six inches of concrete for medium traffic and a minimum of eight inches of concrete for heavy traffic. The upper six inches of soil subgrade should be compacted to at least 95 percent relative compaction and should provide a smooth, non-yielding surface. The concrete should be underlain by at least six inches of Class 2 AB compacted to at least 95 percent relative compaction.

For vehicular concrete pavements, contraction joints should be constructed at 15-foot spacing. Where the outer edge of a concrete pavement meets asphalt pavement or the existing structure, the concrete slab should be thickened by 50 percent at a taper not to exceed a slope of 1 to 10. The slab edges should be confined by curbs or pavement, and slabs should have dowels connecting adjacent slabs. In addition, at areas subject to vehicles with heavy axle loads, we recommend the slabs be reinforced with a minimum of No. 4 bars at 16-inch-spacing in both directions.

7.10 Site Infiltration

The infiltration rate of the soil at the I-1 test location is summarized in Table 8 below.

TABLE 8
Infiltration Rate Test Result

Test ID	Test Depth (feet)	Soil Type	Measured Infiltration Rate (inches/hour)
I-1	3	CL	0.48

Based on the infiltration rate test result, we recommend using an initial design value of about 0.5 inch per hour for the drainage system design for the soccer field, where infiltration depths begin approximately three feet bgs.

7.11 Utilities

The corrosivity test results provided in Appendix D should be reviewed and corrosion protection measures used, if needed. A corrosion engineer should be retained if additional recommendations are needed.

Utility trenches should be excavated a minimum of four inches below the bottom of pipes or conduits and have clearances of at least four inches on both sides. Where necessary, trench excavations should be sloped or shored and braced, in accordance with all safety regulations, to

prevent cave-ins. Section 7.4 provides recommendations for sloping excavations. If sheet piling is used as shoring, and is to be removed after backfilling, it should be placed a minimum of two feet away from the pipes or conduits to prevent disturbance to them as the sheet piles are extracted. It may be difficult to drive sheet piles if cobbles, coarse grained gravel layers or buried obstructions are encountered.

Backfill for utility trenches should be compacted according to the recommendations presented for the general site fill (Section 7.1.2). Jetting of trench backfill should not be permitted.

To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of sand or fine gravel. After pipes and conduits are tested, inspected (if required), and approved, they should be covered to a depth of six inches with sand or fine gravel, which should then be mechanically tamped or compacted with a vibratory plate. Backfill should be placed in lifts of eight inches or less, moisture-conditioned, and compacted to at least 90 percent relative compaction. If sand or gravel with less than 10 percent fines (particles passing the No. 200 sieve) is used, it should be compacted to 95 percent relative compaction. If the backfill will be thicker than five feet, the backfill should be compacted to at least 95 percent relative compaction for its entirety.

Special care should be taken in controlling utility backfilling in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to exterior improvements.

7.12 Surface Drainage

Positive surface drainage should be provided around the proposed structures to direct surface water away from the structures or slopes. To reduce the potential for water ponding adjacent to the buildings, we recommend the ground surface within a horizontal distance of ten feet measured perpendicular to the face of the wall be designed to slope down and away from the buildings with a surface gradient of at least five percent in unpaved areas and two percent in paved areas in accordance with California Building Code (CBC) Section 1804.4. In addition, roof downspouts should be discharged into controlled drainage facilities to keep the water away from the buildings.

Drainage control design should include provisions for positive surface gradients so that surface runoff is not permitted to pond, particularly adjacent to structures and slopes, or on roadways and pavements. Surface runoff should be directed away from the building and slopes to properly designed and installed drop inlets.

7.13 Stormwater Treatment Methods

Bioretention areas are landscaping features used to treat stormwater runoff within a development site. They are commonly located in parking lot islands and landscape areas. Surface runoff is directed into shallow, landscaped depressions, which usually include mulch and a prepared soil mix. Typically, the filtered runoff is collected in a perforated underdrain beneath the bioretention system and returned to the storm drain system. For larger storms, runoff will generally overflow the bioretention areas and is diverted to the storm drain system.

The soil within a bioretention system should typically have an infiltration rate sufficient to draw down any pooled water within 48 hours after a storm event. Bioretention soil should be installed in accordance with the Santa Clara County's C.3 stormwater technical guidelines. Because the soil subgrade beneath bioretention areas could be clay, which has a relatively low infiltration rate, an underdrain system with a waterproof liner on the sides and bottom of the bioretention swale should be installed.

Underdrains are typically at the invert of the bioretention system to intercept water that does not infiltrate into the surrounding soils. Underdrains consist of a perforated PVC pipe surrounded by at least two inches of Class 2 Permeable material (Caltrans Standard Specifications Section 68-2.02F(3)). The perforated PVC pipe cross section area should be determined based on the desired hydraulic conductivity of the underdrain. Underdrains should be installed in accordance with the Santa Clara County's C.3 stormwater technical guidelines.

Overflow from bioretention areas should be directed to the storm drain system away from building foundations and slabs. Where bioretention systems are closer than five feet to foundations, passive resistance of the foundation elements should be neglected.

8.0 ADDITIONAL GEOTECHNICAL SERVICES

During final design we should be retained to consult with the design team as geotechnical questions arise. Technical specifications and design drawings should incorporate Langan's recommendations. When authorized, Langan will assist the design team in reviewing specification sections related to geotechnical issues such as earthwork, shoring, shallow foundations, and backfill. Langan should also, when authorized, review the project plans, as well as Contractor submittals relating to materials and construction procedures for geotechnical work, to confirm the designs incorporate the intent of our recommendations.

Langan has investigated and interpreted the site subsurface conditions and developed the foundation design recommendations contained herein and is therefore best suited to perform quality assurance observation and testing of geotechnical-related work during construction. The work requiring quality assurance confirmation and/or special inspections per the California Building Code includes, but is not limited to, earthwork, backfill, subgrade preparation, shoring and shallow foundations.

Recognizing that construction observation is the final stage of geotechnical design, quality control observation during construction by Langan is necessary to confirm the design assumptions and design elements, to maintain our continuity of responsibility on this project, and allow us to make changes to our recommendations, as necessary. The foundation system and general geotechnical construction methods recommended herein are predicated upon Langan assisting with the final design and providing construction observation services for the Owner. Should Langan not be retained for these services, we cannot assume the role of geotechnical engineer of record, and the entity providing the final design and construction observation services must serve as the engineer of record.

9.0 LIMITATIONS

The conclusions and recommendations provided in this report result from our interpretation of the geotechnical conditions existing at the site inferred from a limited number of borings, as well as project information provided by you. Actual subsurface conditions could vary. Recommendations provided are dependent upon one another and no recommendation should be followed independent of the others. Any proposed changes in structures, depths of excavation, or their locations should be brought to Langan's attention as soon as possible so that we can determine whether such changes affect our recommendations. Information on subsurface strata and groundwater levels shown on the logs represent conditions encountered only at the locations indicated and at the time of investigation. If different conditions are encountered during construction, they should immediately be brought to Langan's attention for evaluation, as they may affect our recommendations.

This report has been prepared to assist the Owner, architect, structural engineer and civil engineer in the design process and is only applicable to the design of the specific project identified. The information in this report cannot be utilized or depended on by engineers or contractors who are involved in evaluations or designs of facilities on adjacent properties which are beyond the limits of that which is the specific subject of this report.

REFERENCES

- 2014 Working Group on California Earthquake Probabilities. (2015). "UCERF3: A new Earthquake Forecast for California's Complex Fault System", U.S. Geological Survey 2015–3009. <http://dx.doi.org/10.3133/fs20153009>
- Aagaard, B. T., Blair, J. L., Boatwright, J., Garcia, S. H., Harris, R. A., Michael, A. J., Schwartz, D. P., and DiLeo, J.S. (2016). "Earthquake Outlook for the San Francisco Bay Region 2014–2043 (ver. 1.1, August 2016), U.S. Geological Survey Fact Sheet 2016–3020, 6 p. <http://dx.doi.org/10.3133/fs20163020>
- ASCE/SEI 7-16 (2016). Minimum Design Loads for Buildings and Other Structures.
- ASCE 7-16 Supplement No. 1 Proposal Form (2018). ASCE/SEI 7-16 Section 21.2.2 and the commentary to Sections 21.2.2 and 21.3. Revised 9 April.
- ASCE 7-16 Supplement No. 3 (2021). ASCE/SEI 7-16 Section 11.4.8 and the commentary to Sections 11.4.8. Effective 5 November 2021.
- Boulanger, R. W. and Idriss, I. M. (2014). "CPT and SPT Based Liquefaction Triggering Procedures." Department of Civil & Environmental Engineering, College of Engineering, University of California at Davis, Center for Geotechnical Modeling, Report No. UCD/CGM-14/01. April.
- California Building Standards Commission (2022). California Building Code.
- California Division of Mines and Geology (2008). "Guidelines for Evaluating and Mitigating Seismic Hazards in California." Special Publication 117A.
- California Division of Mines and Geology (1996). "Probabilistic seismic hazard assessment for the State of California." DMG Open-File Report 96-08.
- California Geological Survey (2002). "Earthquake Zones of Required Investigation, San Jose West Quadrangle." Released 7 February 2002.
- California Geological Survey (2002). "Seismic Hazard Zone Report for the San Jose West 7.5-Minute Quadrangle, Santa Clara County, California."
- Cao, T., Bryant W.A., Rowshandel, B., Branum D. and Wills, C.J. (2003). "The revised 2002 California probabilistic seismic hazard maps June 2003," California Geological Survey.
- Federal Emergency Management Agency (2009). "Flood Insurance Rate Map, Santa Clara County, California, Panel 236 of 830, Map Number 06085C0236H," Effective May 18, 2009.
- Hanks, T. C., and W. H. Bakun (2008). "M-log A observations of recent large earthquakes". Bulletin of Seismological Society of America, Vol. 98, No. 1, pp 490-503.

REFERENCES (Continued)

- Holzer, T.L. et al. (2008). "Liquefaction Hazard Maps for Three Earthquake Scenarios for the Communities of San Jose, Campbell, Cupertino, Los Altos, Los Gatos, Milpitas, Mountain View, Palo Alto, Santa Clara, Saratoga and Sunnyvale, Northern Santa Clara County." USGS Open File Report 2008-1270. ICBO (1997). Uniform Building Code, Volume 2, Structural Engineering Design Provisions.
- Idriss, I.M. and R.W. Boulanger (2008). "Soil Liquefaction During Earthquakes," EERI, MNO-12.
- Lienkaemper, J. J. (1992). "Map of recently active traces of the Hayward Fault, Alameda and Contra Costa counties, California." Miscellaneous Field Studies Map MF-2196.
- Kier+Wright (2025). "Harker Bucknall Multi-Purpose Building, Topographic Survey," Sheet C1.1, dated 3 February 2025.
- Kier+Wright (2025). "Harker Bucknall Multi-Purpose Building, Preliminary Grading, Drainage and Utility Plan", Sheets C3.1 to C3.4, dated 3 February 2025.
- Pradel, D. (1998). "Procedure to Evaluate Earthquake -Induced Settlements in Dry Sandy Soils," ASCE, Journal of Geotechnical & Geoenvironmental Engineering, Vol. 124, No. 4, 364-368.
- Santa Clara County (2012). "Santa Clara County Geologic Hazard Zones." Map Page 27.
- Seed, H.B. and Idriss, I.M. (1971). "Simplified Procedure for Evaluating Soil Liquefaction during Earthquakes," Journal of Geotechnical Engineering Division, ASCE, 97 (9), 1249-1273.
- Sitar, N., E.G. Cahill and J.R. Cahill (2012). "Seismically Induced Lateral Earth Pressures on Retaining Structures and Basement Walls."
- State of California, (2021). Tsunami Hazard Area Map, Santa Clara County; produced by the California Geological Survey and the California Governor's Office of Emergency Services; dated 2021, displayed at multiple scales.
- Studio Bondy Architecture (2025), "Harker Bucknall multi-Purpose Building, 50% Schematic Design", dated 3 February 2025.
- Toppozada, T. R. and Borchardt G. (1998). "Re-Evaluation of the 1836 "Hayward Fault" and the 1838 San Andreas Fault earthquakes," *Bulletin of Seismological Society of America*, 88(1), 140-159.
- United Soil Engineering, Inc. (1997). "Geotechnical Investigation & Geologic Hazard Assessment, Proposed Harker School, Southwest Corner of Bucknall Road & San Tomas Aquino Road, San Jose, California," dated 1 December 1997.

REFERENCES (Continued)

Valley Water. Historical Groundwater Elevation Data; Interactive Map Application. <https://gis.valleywater.org/GroundwaterElevations/map.php>. Accessed February 2025.

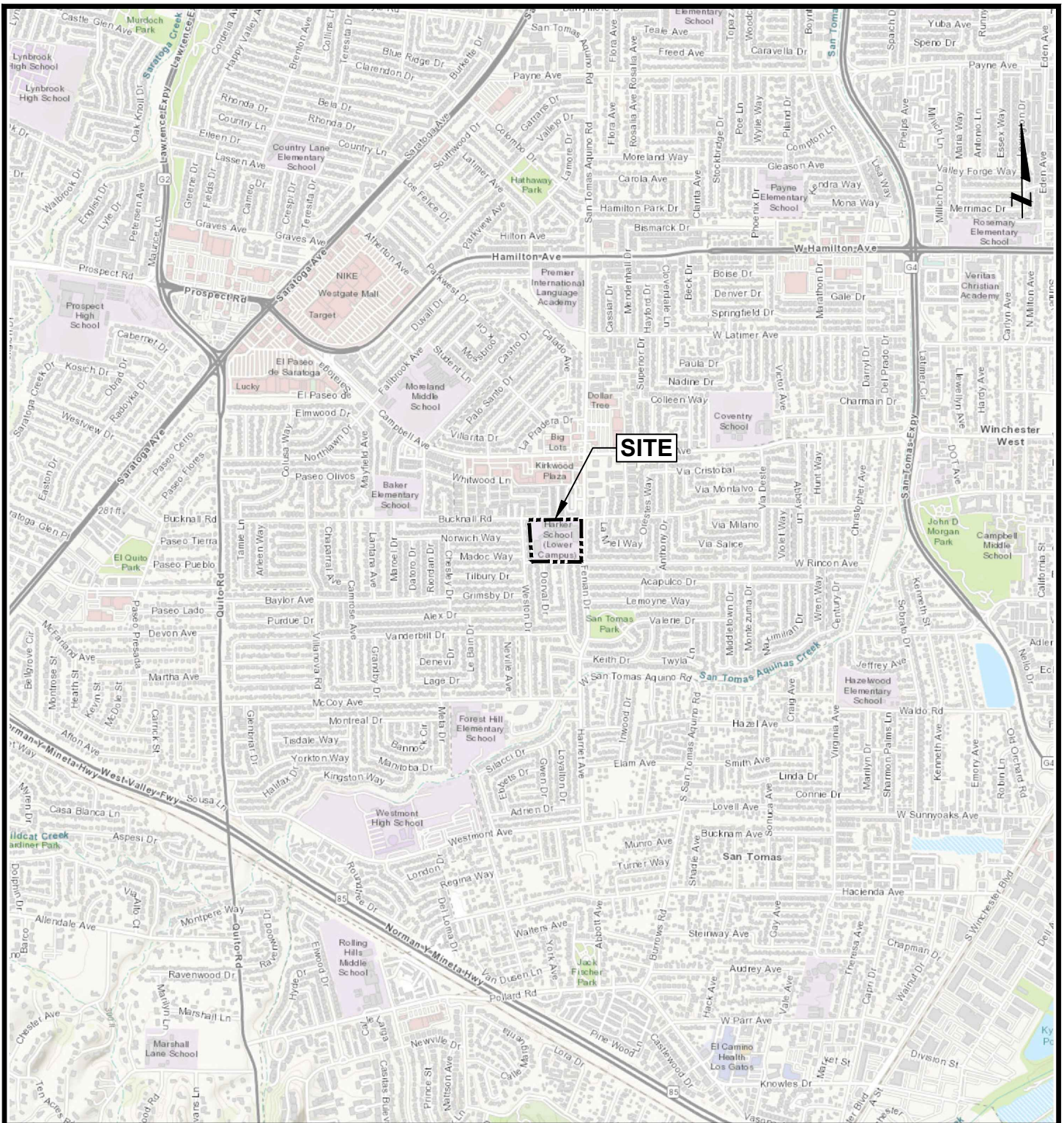
Youd, T.L., and Garris, C.T. (1995). "Liquefaction-induced ground-surface disruption." *Journal of Geotechnical Engineering*, American Society of Civil Engineers, Vol. 121, 805-809.

Youd, T. L., Hansen, C. M., and Bartlett, S. F., (2002). Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement, *Journal of Geotechnical and Geoenvironmental Engineering*, December.

Youd, T.L., and Hoose, S.N. (1978). "Historic Ground Failures in Northern California Triggered by Earthquakes," United States Geological Survey Professional Paper 993.

Youd, T.L., and Idriss, I.M. (2001). "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 127, No. 4.

FIGURES



Note:
Base map is provided through Langan's Esri
Arc GIS software licensing and Arc GIS online,
National Geographic Society, i-cubed.

0 2,000 Feet
Approximate scale

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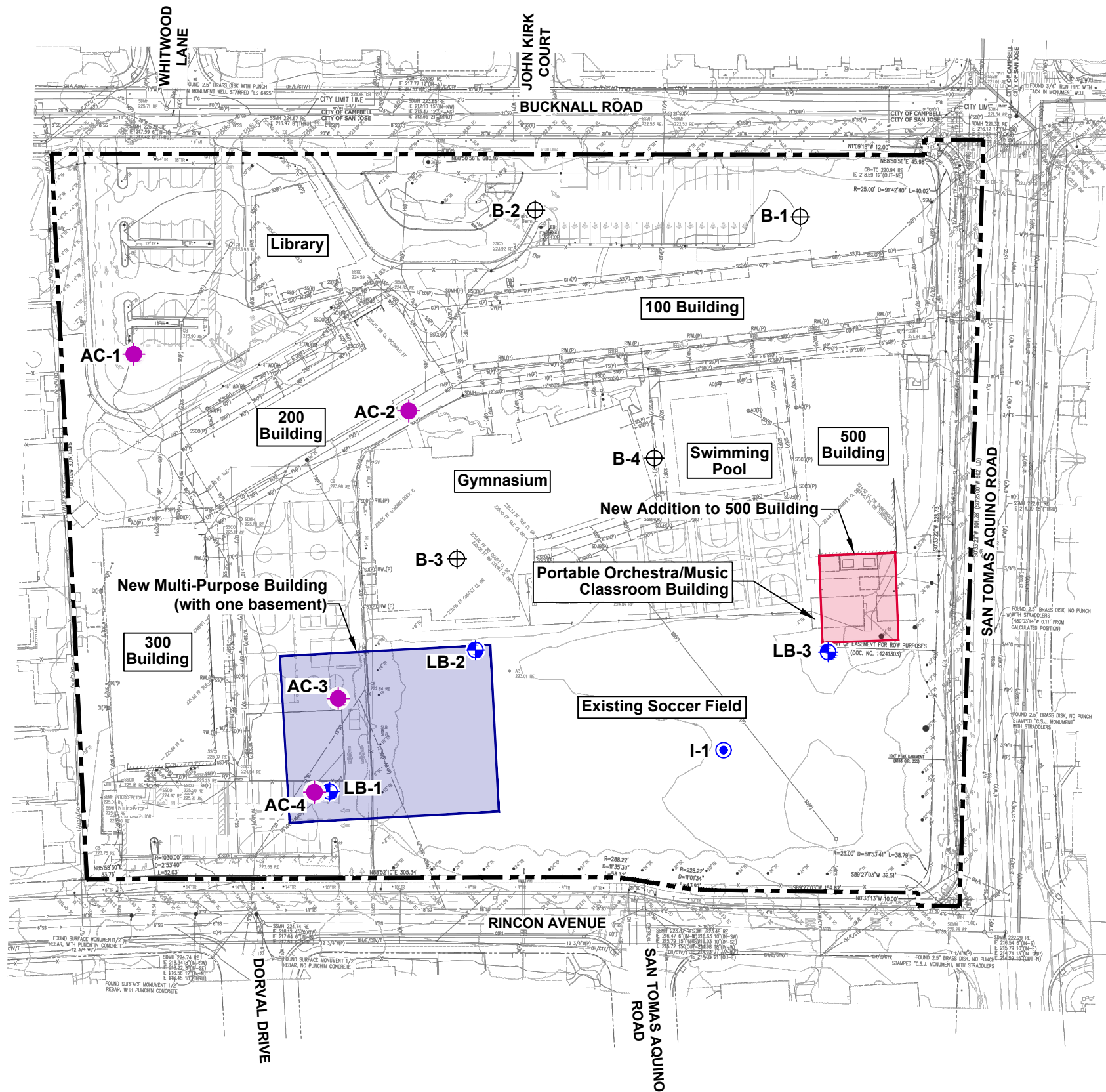
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Project
**THE HARKER SCHOOL
BUCKNALL CAMPUS**
SAN JOSE
SANTA CLARA COUNTY CALIFORNIA

Figure Title
**SITE
LOCATION MAP**

Project No.
770703801
Date
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AG
Checked By
HL

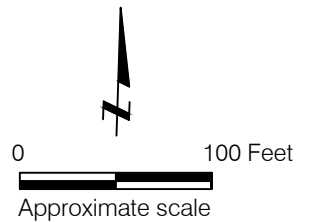
Figure
1



EXPLANATION

- LB-1**  Approximate location of boring by Langan, January 2025
- I-1**  Approximate location of infiltration test by Langan, January 2025
- AC-1**  Approximate location of asphalt concrete and baserock sampling by Langan, January 2025
- B-1**  Approximate location of boring by United Soil Engineering, Inc., November 1997
-  Site boundary

Reference:
1. Base map "Topographic Survey - Harker Bucknall MP Building", Sheet C1.1, by Kier+Wright, dated 3 February 2025.
2. Proposed building locations based on "Preliminary Civil Site Plan - Harker Bucknall MP Building", Sheet C2.1, by Kier+Wright, dated 3 February 2025.



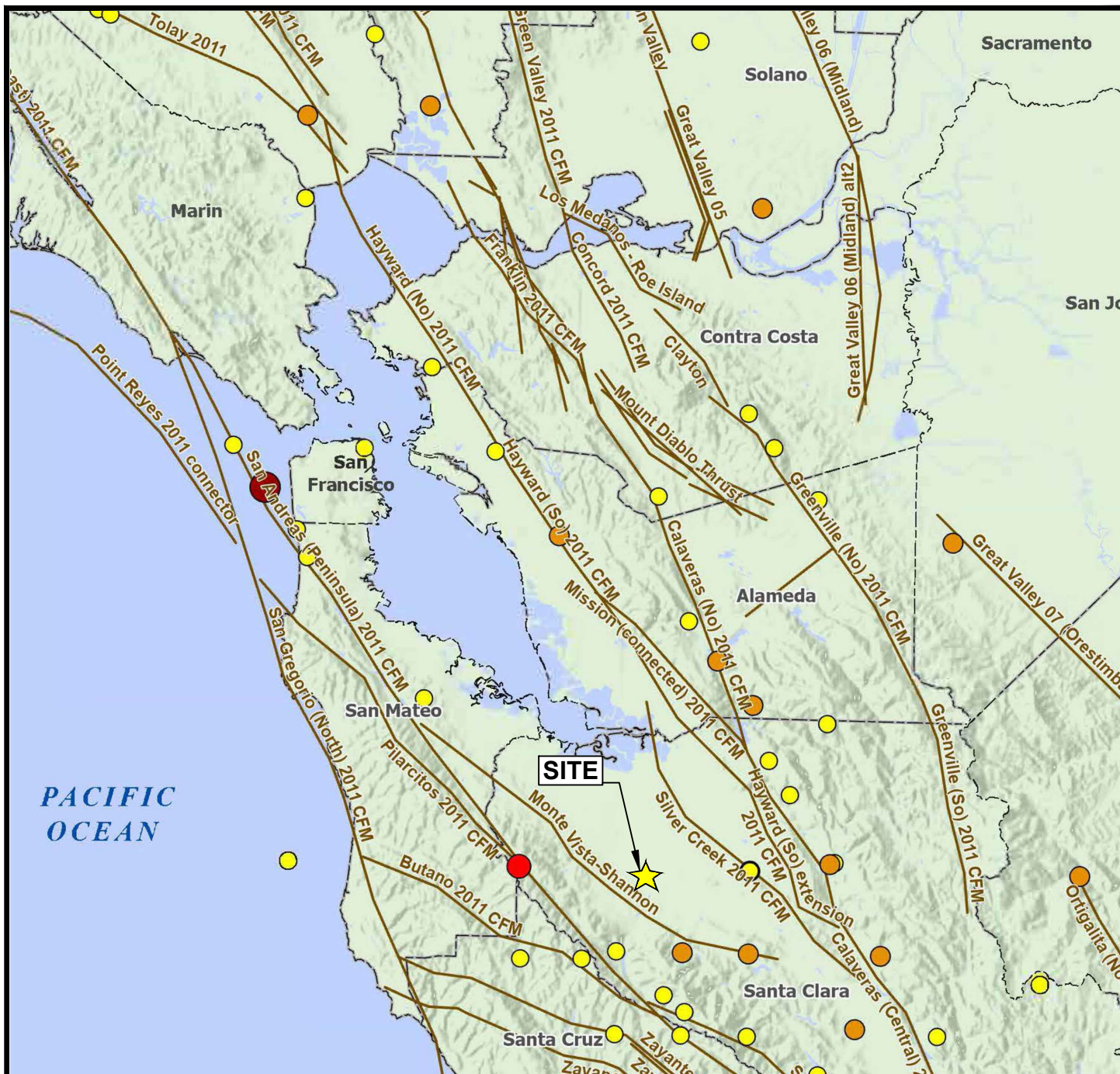
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Project
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Figure Title
SITE PLAN

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Figure
2



LEGEND



Site Location



County Boundary

Fault

Earthquake Epicenter Magnitude

Magnitude 5 to 5.9

Magnitude 6 to 6.9

Magnitude 7 to 7.4

Magnitude 7.5 to 8

0 20 km



Approximate scale

NOTES:

1. QUATERNARY FAULT DATA DISPLAYED ARE PROVIDED BY THE CGS MAP SHEET 48: FAULT BASED SEISMIC SOURCES USED IN THE UNIFORM CALIFORNIA EARTHQUAKE RUPTURE FORECAST, VERSION 3 (UCERF3).
2. EARTHQUAKE EPICENTER (MAGNITUDE) DATA QUERIED WITH A MAGNITUDE OF 5+ FROM 01/01/1800 TO PRESENT, FROM THE ANSS COMPREHENSIVE EARTHQUAKE CATALOG (COMCAT), DOWNLOADED 02/18/2024.
3. BASEMAP AND COUNTY BOUNDARIES PROVIDED BY USGS AND CALIFORNIA DEPARTMENT OF TRANSPORTATION.
4. MAP DISPLAYED IN CALIFORNIA STATE COORDINATE SYSTEM, CALIFORNIA (TEALE) ALBERS, NORTH AMERICAN DATUM OF 1983 (NAD83), METERS.

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San Jose, CA 95113

T: 408.283.3600 F: 408.283.3601 www.langan.com

Project

**THE HARKER SCHOOL
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SAN JOSE

SANTA CLARA COUNTY

CALIFORNIA

Figure Title

**MAP OF MAJOR
FAULTS AND EARTHQUAKE
EPICENTERS IN THE
SAN FRANCISCO BAY AREA**

Project No.
770703801

Date
02/18/2025

Drawn By
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Checked By
HL

Figure

3

I Not felt by people, except under especially favorable circumstances. However, dizziness or nausea may be experienced.

Sometimes birds and animals are uneasy or disturbed. Trees, structures, liquids, bodies of water may sway gently, and doors may swing very slowly.

II Felt indoors by a few people, especially on upper floors of multi-story buildings, and by sensitive or nervous persons.

As in Grade I, birds and animals are disturbed, and trees, structures, liquids and bodies of water may sway. Hanging objects swing, especially if they are delicately suspended.

III Felt indoors by several people, usually as a rapid vibration that may not be recognized as an earthquake at first. Vibration is similar to that of a light, or lightly loaded trucks, or heavy trucks some distance away. Duration may be estimated in some cases.

Movements may be appreciable on upper levels of tall structures. Standing motor cars may rock slightly.

IV Felt indoors by many, outdoors by a few. Awakens a few individuals, particularly light sleepers, but frightens no one except those apprehensive from previous experience. Vibration like that due to passing of heavy, or heavily loaded trucks. Sensation like a heavy body striking building, or the falling of heavy objects inside.

Dishes, windows and doors rattle; glassware and crockery clink and clash. Walls and house frames creak, especially if intensity is in the upper range of this grade. Hanging objects often swing. Liquids in open vessels are disturbed slightly. Stationary automobiles rock noticeably.

V Felt indoors by practically everyone, outdoors by most people. Direction can often be estimated by those outdoors. Awakens many, or most sleepers. Frightens a few people, with slight excitement; some persons run outdoors.

Buildings tremble throughout. Dishes and glassware break to some extent. Windows crack in some cases, but not generally. Vases and small or unstable objects overturn in many instances, and a few fall. Hanging objects and doors swing generally or considerably. Pictures knock against walls, or swing out of place. Doors and shutters open or close abruptly. Pendulum clocks stop, or run fast or slow. Small objects move, and furnishings may shift to a slight extent. Small amounts of liquids spill from well-filled open containers. Trees and bushes shake slightly.

VI Felt by everyone, indoors and outdoors. Awakens all sleepers. Frightens many people; general excitement, and some persons run outdoors.

Persons move unsteadily. Trees and bushes shake slightly to moderately. Liquids are set in strong motion. Small bells in churches and schools ring. Poorly built buildings may be damaged. Plaster falls in small amounts. Other plaster cracks somewhat. Many dishes and glasses, and a few windows break. Knickknacks, books and pictures fall. Furniture overturns in many instances. Heavy furnishings move.

VII Frightens everyone. General alarm, and everyone runs outdoors.

People find it difficult to stand. Persons driving cars notice shaking. Trees and bushes shake moderately to strongly. Waves form on ponds, lakes and streams. Water is muddied. Gravel or sand stream banks cave in. Large church bells ring. Suspended objects quiver. Damage is negligible in buildings of good design and construction; slight to moderate in well-built ordinary buildings; considerable in poorly built or badly designed buildings, adobe houses, old walls (especially where laid up without mortar), spires, etc. Plaster and some stucco fall. Many windows and some furniture break. Loosened brickwork and tiles shake down. Weak chimneys break at the roofline. Cornices fall from towers and high buildings. Bricks and stones are dislodged. Heavy furniture overturns. Concrete irrigation ditches are considerably damaged.

VIII General fright, and alarm approaches panic.

Persons driving cars are disturbed. Trees shake strongly, and branches and trunks break off (especially palm trees). Sand and mud erupts in small amounts. Flow of springs and wells is temporarily and sometimes permanently changed. Dry wells renew flow. Temperatures of spring and well waters varies. Damage slight in brick structures built especially to withstand earthquakes; considerable in ordinary substantial buildings, with some partial collapse; heavy in some wooden houses, with some tumbling down. Panel walls break away in frame structures. Decayed pilings break off. Walls fall. Solid stone walls crack and break seriously. Wet grounds and steep slopes crack to some extent. Chimneys, columns, monuments and factory stacks and towers twist and fall. Very heavy furniture moves conspicuously or overturns.

IX Panic is general.

Ground cracks conspicuously. Damage is considerable in masonry structures built especially to withstand earthquakes; great in other masonry buildings - some collapse in large part. Some wood frame houses built especially to withstand earthquakes are thrown out of plumb, others are shifted wholly off foundations. Reservoirs are seriously damaged and underground pipes sometimes break.

X Panic is general.

Ground, especially when loose and wet, cracks up to widths of several inches; fissures up to a yard in width run parallel to canal and stream banks. Landsliding is considerable from river banks and steep coasts. Sand and mud shifts horizontally on beaches and flat land. Water level changes in wells. Water is thrown on banks of canals, lakes, rivers, etc. Dams, dikes, embankments are seriously damaged. Well-built wooden structures and bridges are severely damaged, and some collapse. Dangerous cracks develop in excellent brick walls. Most masonry and frame structures, and their foundations are destroyed. Railroad rails bend slightly. Pipe lines buried in earth tear apart or are crushed endwise. Open cracks and broad wavy folds open in cement pavements and asphalt road surfaces.

XI Panic is general.

Disturbances in ground are many and widespread, varying with the ground material. Broad fissures, earth slumps, and land slips develop in soft, wet ground. Water charged with sand and mud is ejected in large amounts. Sea waves of significant magnitude may develop. Damage is severe to wood frame structures, especially near shock centers, great to dams, dikes and embankments, even at long distances. Few if any masonry structures remain standing. Supporting piers or pillars of large, well-built bridges are wrecked. Wooden bridges that "give" are less affected. Railroad rails bend greatly and some thrust endwise. Pipe lines buried in earth are put completely out of service.

XII Panic is general.

Damage is total, and practically all works of construction are damaged greatly or destroyed. Disturbances in the ground are great and varied, and numerous shearing cracks develop. Landslides, rock falls, and slumps in river banks are numerous and extensive. Large rock masses are wrenched loose and torn off. Fault slips develop in firm rock, and horizontal and vertical offset displacements are notable. Water channels, both surface and underground, are disturbed and modified greatly. Lakes are dammed, new waterfalls are produced, rivers are deflected, etc. Surface waves are seen on ground surfaces. Lines of sight and level are distorted. Objects are thrown upward into the air.

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San Jose, CA 95113

T: 408.283.3600 F: 408.283.3601 www.langan.com

Project

**THE HARKER SCHOOL
BUCKNALL CAMPUS**

SAN JOSE

SANTA CLARA COUNTY

CALIFORNIA

Figure Title

**MODIFIED MERCALLI
INTENSITY SCALE**

Project No.

770703801

Date

02/18/2025

Drawn By

AG

Checked By

HL

Figure

4

APPENDIX A
LOGS OF BORINGS BY LANGAN

PROJECT: THE HARKER SCHOOL BUCKNALL CAMPUS San Jose, California						Log of Boring LB-1 PAGE 1 OF 1						
Boring location: See Site Plan, Figure 2						Logged by: H. Li Drilled By: Exploration Geoservices, Inc.						
Date started: 1/22/25			Date finished: 1/22/25									
Drilling method: Hollow Stem Auger												
Hammer weight/drop: 140 lbs./30 inches			Hammer type: Automatic			LABORATORY TEST DATA						
Samplers: Sprague & Henwood (S&H), Standard Penetration Test (SPT)												
DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
	Sampler Type	Sample	Blows/ 6"	SPT N-Value ¹								
Ground Surface Elevation: 224 feet ²												
1					AB	3 inches asphalt concrete (AC)						
2	HA					8 inches aggregate base (AB)						
3					CL	SANDY CLAY (CL) brown, medium stiff to stiff, moist, fine to coarse sand						
4	HA					at 2 feet: LL = 30, PI = 14, see Figure C-2						
5						increased sand content						
6	S&H		3 2 2	3	SC	CLAYEY SAND (SC) brown, very loose, moist, fine-grained, trace fine rounded gravel Triaxial Test, see Figure C-1	TxUU	700	540	49.6	16.9	99
7												
8	S&H		4 14 18	26								
9					SM	SILTY SAND with GRAVEL (SM) brown, medium dense, moist, fine- to coarse-grained, fine to coarse subrounded gravel					6.6	109
10												
11	S&H		14 18 20	30		GRAVEL with CLAY and SAND (GP-GC) brown, dense, moist, fine to coarse, subangular to angular, fine to coarse sand					5.1	117
12												
13												
14					GP-GC							
15	SPT		36 50/5"	65/5"		brown to gray, very dense						
16												
17												
18												
19						SILT (ML) olive with orange mottling, hard, moist						
20												
21	S&H		10 20 22	34	ML		PP		>4500		20.1	107
22												
23												
24						CLAYEY SAND (SC) yellow-brown, very dense, moist, fine-grained						
25												
26	SPT		18 22 36	75	SC							
27												
28						CLAY with SAND (CL) yellow-brown. hard, moist, fine sand						
29	SPT		9 15 15	39	CL							
30												
Boring terminated at a depth of 30 feet below ground surface. Boring backfilled with soil cuttings. Groundwater not encountered during drilling. PP = Pocket penetrometer.						LANGAN Project No.: 770703801 Figure: A-1						
						¹ S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.8 and 1.3, respectively to account for sampler type and hammer energy. ² Elevations referenced to NGVD 29 vertical datum and estimated from topographic survey plan prepared by Kier+Wright, dated 3 February 2025.						

TEST GEOTECH LOG 770703801 HARKER SCHOOL.GPJ TEMPLATE CA-MODIFIED.GDT 3/21/25

¹ S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.8 and 1.3, respectively to account for sampler type and hammer energy.
² Elevations referenced to NGVD 29 vertical datum and estimated from topographic survey plan prepared by Kier+Wright, dated 3 February 2025.

PROJECT: THE HARKER SCHOOL BUCKNALL CAMPUS San Jose, California						Log of Boring LB-2 PAGE 1 OF 1						
Boring location: See Site Plan, Figure 2						Logged by: H. Li Drilled By: Exploration Geoservices, Inc.						
Date started: 1/22/25 Date finished: 1/22/25												
Drilling method: Hollow Stem Auger												
Hammer weight/drop: 140 lbs./30 inches Hammer type: Automatic						LABORATORY TEST DATA						
Samplers: Sprague & Henwood (S&H), Standard Penetration Test (SPT)												
DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
	Sampler Type	Sample	Blows/ 6"	SPT N-Value ¹								
Ground Surface Elevation: 223.5 feet ²												
1	HA				CL	CLAY with SAND (CL) brown, stiff, moist, fine to coarse sand, trace fine gravel	PP		2,250		18.1	110
2						0 to 5 feet: R-Value Test, see Figure C-3						
3												
4												
5	S&H		3 6 5	9		trace fine to coarse gravel						
6												
7	S&H		3 8 10	14		increased sand and gravel content	PP		2,000			
8												
9	S&H		13 18 33	41	SM	SILTY SAND with GRAVEL (SM) brown, dense, moist, fine- to coarse-grained, fine to coarse rounded to subangular gravel					5.1	116
10												
11												
12	SPT		10 20 30	65	GC	CLAYEY GRAVEL with SAND (GC) yellow, very dense, moist, fine to coarse, subangular gravel, fine to coarse sand						
13												
14												
15												
16	SPT		25 35 50	111	SC	CLAYEY SAND with GRAVEL (SC) brown, very dense, moist, fine- to coarse-grained, fine to coarse subangular to rounded gravel						
17												
18												
19												
20	SPT		25 35 50/4"	111/ 10"	CL	SANDY CLAY with GRAVEL (CL) olive-gray with orange mottling to gray, hard, moist, fine to medium sand, fine rounded to subangular gravel						
21												
22												
23												
24	SPT		30 40 45	111	SC	CLAYEY SAND with GRAVEL (SC) brown, very dense, moist, fine- to coarse-grained, fine subangular to rounded gravel						
25												
26												
27												
28	SPT											
29												
30												
Boring terminated at a depth of 30 feet below ground surface. Boring backfilled with soil cuttings. Groundwater not encountered during drilling. PP = Pocket penetrometer.						¹ S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.8 and 1.3, respectively to account for sampler type and hammer energy. ² Elevations referenced to NGVD 29 vertical datum and estimated from topographic survey plan prepared by Kier+Wright, dated 3 February 2025.						
						LANGAN						
						Project No.: 770703801 Figure: A-2						

TEST GEOTECH LOG 770703801 HARKER SCHOOL.GPJ TEMPLATE CA-MODIFIED.GDT 3/21/25

TEST GEOTECH LOG 770703801 HARKER SCHOOL.GPJ TEMPLATE CA-MODIFIED.GDT 3/21/25

PROJECT: THE HARKER SCHOOL BUCKNALL CAMPUS San Jose, California						Log of Boring LB-3 PAGE 1 OF 1							
Boring location: See Site Plan, Figure 2						Logged by: H. Li Drilled By: Exploration Geoservices, Inc.							
Date started: 1/22/25 Date finished: 1/22/25													
Drilling method: Hollow Stem Auger													
Hammer weight/drop: 140 lbs./30 inches Hammer type: Automatic						LABORATORY TEST DATA							
Samplers: Sprague & Henwood (S&H), Standard Penetration Test (SPT)													
DEPTH (feet)	SAMPLES			SPT N-value ¹	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft	
	Sampler Type	Sample	Blows/ 6"										
Ground Surface Elevation: 223 feet ²													
1					CL	SANDY CLAY (CL) brown, medium stiff to stiff, moist, fine sand, trace fine rounded gravel, trace rootlets LL = 34, PI = 15, see Figure C-2							
2	HA												
3													
4	HA												
5					GP-GC	GRAVEL with CLAY and SAND (GP-GC) brown, loose to medium dense, moist, fine rounded gravel, fine to coarse sand at 6 feet: LL = 24, PI = 8, see Figure C-2 fine to coarse angular to rounded gravel dense, fine subangular gravel							
6	S&H		5 6 7	10							10.0	7.3	
7													
8	S&H		8 10 14	19							9.9	6.0	124
9					SC	CLAYEY SAND with GRAVEL (SC) yellow to light brown, very dense, moist, fine- to coarse-grained, fine subangular gravel							
10													
11	SPT		7 14 21	46									
12													
13					SC	CLAYEY SAND with GRAVEL (SC) yellow to light brown, very dense, moist, fine- to coarse-grained, fine subangular gravel							
14													
15													
16	SPT		14 21 21	55									
17					SC	CLAYEY SAND with GRAVEL (SC) yellow to light brown, very dense, moist, fine- to coarse-grained, fine subangular gravel							
18													
19	SPT		30 38 37	98									
20													
21													
22													
23													
24													
25													
26													
27													
28													
29													
30													
Boring terminated at a depth of 20 feet below ground surface. Boring backfilled with soil cuttings. Groundwater not encountered during drilling. PP = Pocket penetrometer.						¹ S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.8 and 1.3, respectively to account for sampler type and hammer energy. ² Elevations referenced to NGVD 29 vertical datum and estimated from topographic survey plan prepared by Kier+Wright, dated 3 February 2025.							
						Project No.: 770703801 Figure: A-3							

UNIFIED SOIL CLASSIFICATION SYSTEM		
Major Divisions	Symbols	Typical Names
Coarse-Grained Soils (more than half of soil > no. 200 sieve size)	GW	Well-graded gravels or gravel-sand mixtures, little or no fines
	GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
	GM	Silty gravels, gravel-sand-silt mixtures
	GC	Clayey gravels, gravel-sand-clay mixtures
	SW	Well-graded sands or gravelly sands, little or no fines
	SP	Poorly-graded sands or gravelly sands, little or no fines
	SM	Silty sands, sand-silt mixtures
	SC	Clayey sands, sand-clay mixtures
Fine -Grained Soils (more than half of soil < no. 200 sieve size)	Silts and Clays LL = < 50	ML Inorganic silts and clayey silts of low plasticity, sandy silts, gravelly silts
		CL Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
		OL Organic silts and organic silt-clays of low plasticity
	Silts and Clays LL = > 50	MH Inorganic silts of high plasticity
		CH Inorganic clays of high plasticity, fat clays
		OH Organic silts and clays of high plasticity
Highly Organic Soils	PT	Peat and other highly organic soils

SAMPLE DESIGNATIONS/SYMBOLS

GRAIN SIZE CHART		
Classification	Range of Grain Sizes	
	U.S. Standard Sieve Size	Grain Size in Millimeters
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel coarse fine	3" to No. 4	76.2 to 4.76
	3" to 3/4" 3/4" to No. 4	76.2 to 19.1 19.1 to 4.76
Sand coarse medium fine	No. 4 to No. 200	4.76 to 0.075
	No. 4 to No. 10	4.76 to 2.00
	No. 10 to No. 40 No. 40 to No. 200	2.00 to 0.420 0.420 to 0.075
Silt and Clay	Below No. 200	Below 0.075

 Unstabilized groundwater level

 Stabilized groundwater level

PP = Pocket Penetrometer

TV = Torvane



Sample taken with Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter. Darkened area indicates soil recovered



Classification sample taken with Standard Penetration Test sampler



Undisturbed sample taken with thin-walled tube



Disturbed sample



Sampling attempted with no recovery



Core sample



Analytical laboratory sample



Sample taken with Direct Push or Drive sampler



Sonic

SAMPLER TYPE

C Core barrel

CA California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter

D&M Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube

O Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube

PT Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube

S&H Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter

SPT Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.38- or 1.5-inch inside diameter - see report text

ST Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

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Langan CA, Inc.
1 Almaden Boulevard, Suite 590
San Jose, CA 95113

T: 408.283.3600 F: 408.283.3601 www.langan.com

Project
**THE HARKER SCHOOL
BUCKNALL CAMPUS**
SAN JOSE
SANTA CLARA COUNTY CALIFORNIA

Figure Title
SOIL CLASSIFICATION CHART

Project No.
770703801
Date
02/12/2025
Drawn By
JPC
Checked By
HL

Figure
A-4

APPENDIX B

PREVIOUS FIELD EXPLORATION BY OTHERS

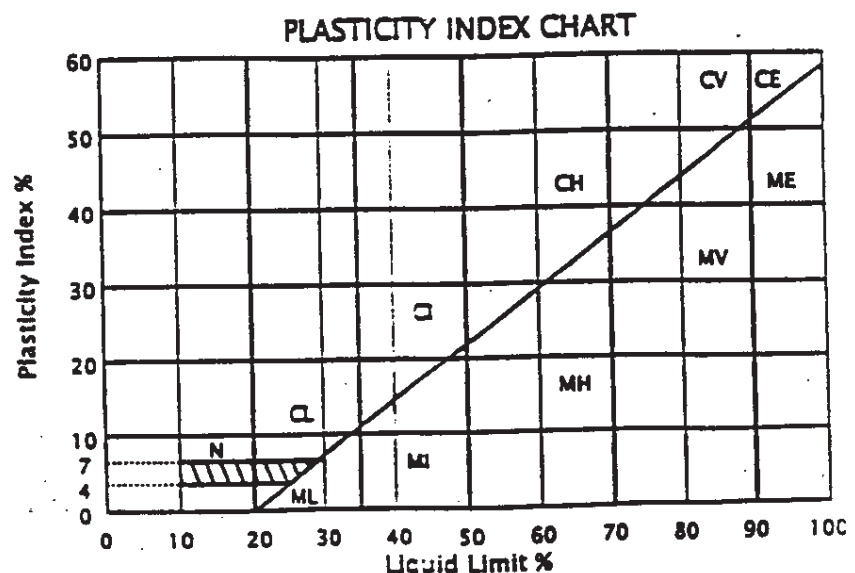
METHOD OF SOIL CLASSIFICATION

MAJOR DIVISIONS		SYMBOL		TYPICAL NAMES	
COARSE GRAINED SOILS (More than 1/2 of soil no. 200 sieve size)	<u>GRAVELS</u> (More than 1/2 of coarse fraction > no. 4 sieve size)	GW		Well graded gravel or gravel-sand mixtures, little or no fines	
		GP		Poorly graded gravel or gravel-sand mixtures, little or no fines	
		GM		Silty gravels, gravel-sand-silt mixtures	
		GC		Clayey Gravels, gravel-sand-clay mixtures	
	<u>SANDS</u> (More than 1/2 of coarse fraction < no. 4 sieve size)	SW		Well graded sands or gravelly sands, no fines	
		SP		Poorly graded sands or gravelly sands, no fines	
		SM		Silty sands, sand-silt mixtures	
		SC		Clayey sands, sand-clay mixtures	
FINE GRAINED SOILS (More than 1/2 of soil > no. 200 sieve size)	<u>SILTS & CLAYS</u> <u>LL < 50</u>	ML		Inorganic silts and very fine sand, rock flour, silty or clayey fine sand or clayey silt/slight plasticity	
		CL		Inorganic clay of low to medium plasticity, gravelly clays, sandy clay, silty clay, lean clays	
		OL		Organic silts and organic silty clay of low plasticity	
	<u>SILTS & CLAYS</u> <u>LL > 50</u>	MH		Inorganic silts, micaceous or diazocaceous fines and, or silty soils, elastic silt	
		CH		Inorganic clays of high plasticity, fat clays	
		OH		Organic clays of medium to high plasticity, organic silty clays, organic silts	
<u>HIGHLY ORGANIC SOIL</u>		PT		Peat and other highly organic soils	

CLASSIFICATION CHART - (UNIFIED SOIL CLASSIFICATION SYSTEM)

99-018

CLASSIFICATION	RANGE OF GRAIN SIZES	
	U.S. Standard Sieve Size	Grain Size In Millimeters
BOULDERS	Above 12"	Above 305
COBBLES	12" to 3"	305 to 76.2
GRAVELS Coarse Fine	3" to No. 4	76.2 to 4.76
	3" to 3/4"	76.2 to 19.1
	3/4" to No. 4	19.1 to 4.76
SAND Coarse Medium Fine	No. 4 to No. 200	4.76 to 0.074
	No. 4 to No. 10	4.76 to 2.00
	No. 10 to No. 40	2.00 to 0.420
	No. 40 to No. 200	0.420 to 0.074
SILT AND CLAY	Below No. 200	Below 0.074



Logged By:		EXPLANATION OF TEST BORING SYMBOLS						Boring No.	
Date Drilled:									
Dry Density p.c.f.	Moisture Content %	Penet. Resist. Blows/ft.	Unconf. Comp. Strength, k.s.f.	Direct Shear Test		Sample Number	Depth In Feet	Boring Log	File No. 4481-S1
				"C" k.s.f.	"φ" Degree				 STATIC GROUNDWATER  GROUNDWATER FIRST NOTED
									DESCRIPTION
									CLAY
									SILT
									SAND
									GRAVEL
									clayey
									silty
									sandy
									gravelly
						1-1			Sample Taken with sample number and lab results given
									Sample Attempt - Unsuccessful No Sample Number
									Refusal noted in Remarks
									Brown SANDSTONE

Remarks:

99 - 018

Logged By: V.V.				EXPLORATORY BORING LOG				Boring No.	
Date Drilled: 11/19/97								B-1	
File No. 4481-S1									
Dry Density p.c.f.	Moisture Content %	Penet. Resist. Blows/ft.	Unconf. Comp. Strength, k.s.f.	Direct Shear Test		Sample Number	Depth In Feet	Boring Log	DESCRIPTION
				"C" k.s.f.	"φ" Degree				
108.2	4.7	18		0.2	32	1-1			3 inches of organic material, Light Brown Sandy Silty GRAVEL, moist, stiff
101.6	5.0	23				1-2	5		Brownish Gray Sandy GRAVEL, damp, very hard
121.4	10.3	16				1-3	10		Brown Sandy Clayey GRAVEL, moist, hard
127.3	6.8	60				1-4	15		very hard drilling fractured rock pieces
									Brown SANDSTONE, very hard
117.5	7.8	*				1-5	20		REFUSAL
									Boring Terminated at 20 Feet
Remarks: * = 50 Blows for 5 Inches									
99-018									

Logged By: V.V.		EXPLORATORY BORING LOG					Boring No. B-2		
Date Drilled: 11/19/97		File No. 4481-S1							
Dry Density p.c.f.	Moisture Content %	Penet. Resist. Blows/ft.	Unconf. Comp. Strength, k.s.f.	Direct Shear Test "C" k.s.f. "Ø" Degree		Sample Number	Depth in Feet	Boring Log	DESCRIPTION
98.8	7.9	19				2-1	3		3 inches of organic material, Light Brown Sandy Silty GRAVEL, moist, stiff
106.6	8.2	15				2-2	5		Reddish Gray Sandy GRAVEL, damp, hard
107.3	9.5	26				2-3	10		Brown Sandy Silty GRAVEL, moist, hard
126.9	6.5	57				2-4	15		very hard drilling fractured rock
		*							Brown SANDSTONE, damp, very hard REFUSAL
							20		Boring Terminated at 19 Feet
Remarks: * = 50 Blows for 5½ Inches 99 - 018									

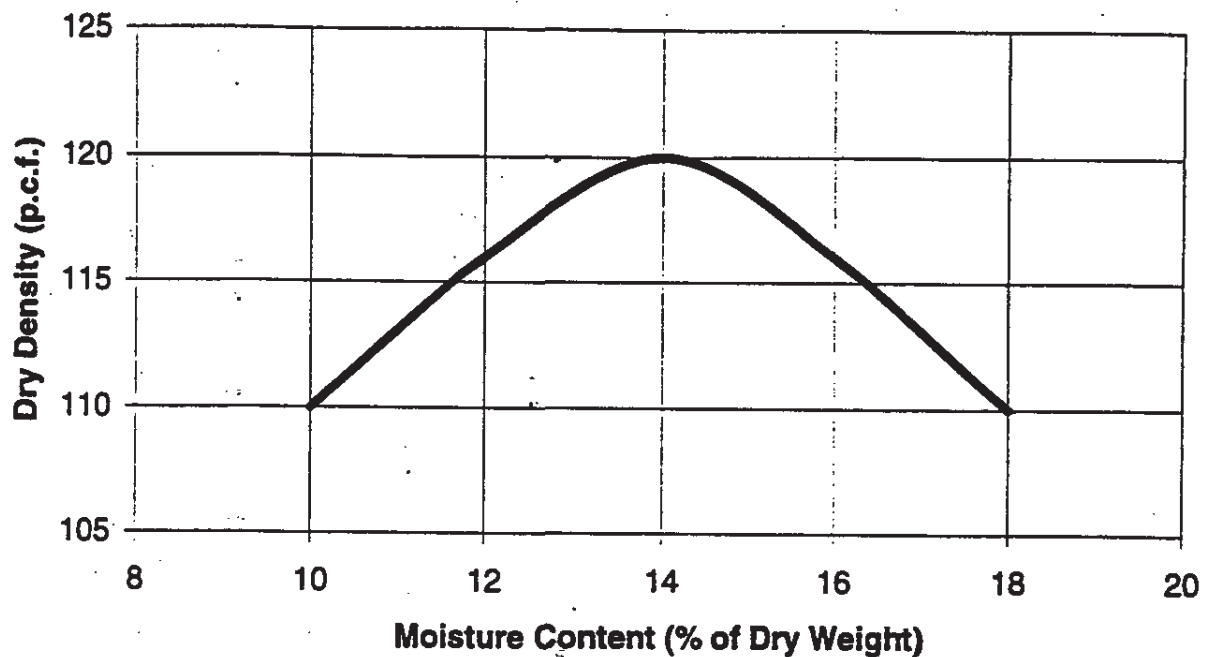
Logged By: V.V.		EXPLORATORY BORING LOG					Boring No. B-3		
Date Drilled: 11/19/97		File No. 4481-S1							
Dry Density p.c.f.	Molsture Content %	Penet. Resist. Blows/ft.	Unconf. Comp. Strength, k.s.f.	Direct Shear Test "C" k.s.f. "φ" Degree		Sample Number	Depth in Feet	Boring Log	DESCRIPTION
105.1	10.8	27				3-1	5		2 inches of organic material, Tan Brown Sandy CLAY, dry, stiff
121.9	6.5	40				3-2	10		Reddish Brown Sandy GRAVEL, moist, very dense
125.2	9.8	42				3-3	15		
							20		Brown SANDSTONE, damp, very hard REFUSAL
									Boring Terminated at 20 Feet
Remarks:									
99 - 018									

Logged By: V.V.		EXPLORATORY BORING LOG					Boring No. B-4	
Date Drilled: 11/19/97		File No. 4481-S1						
Dry Density p.c.f.	Moisture Content %	Penet. Resist. Blows/ft.	Unconf. Comp. Strength, k.s.f.	Direct Shear Test "C" k.s.f. "Ø" Degree		Sample Number	Depth In Feet	Boring Log
								DESCRIPTION
105.7	8.4	28				4-1	5	 3 inches of organic material, Tan Brown Sandy CLAY, moist, very dense
128.2	5.6	41				4-2	10	 Reddish Brown Sandy GRAVEL, moist, very dense
129.0	11.2	46				4-3	15	 Brown SANDSTONE, very hard
								REFUSAL
								Boring Terminated at 20 Feet
Remarks: 99 - 018								

TABLE I

**SUMMARY OF MOISTURE, DENSITY,
UNCONFINED COMPRESSION AND DIRECT SHEAR TESTING**

Sample No.	Depth Ft.	In-Place Conditions		Unconfined Compressive Strength k.s.f.	Direct Shear Testing	
		Dry Density p.c.f.	Moisture Content % Dry Wt.		Angle of Internal Friction Degrees	Unit Cohesion k.s.f.
1-1	3	108.2	4.7		32	0.2
1-2	5	101.6	5.0			
1-3	10	121.4	10.3			
1-4	15	127.3	6.8			
1-5	20	117.5	7.8			
				99 - 018	=	
2-1	3	98.8	7.9			
2-2	5	106.6	8.2			
2-3	10	107.3	9.5			
2-4	15	126.9	6.5			
3-1	5	105.1	10.8			
3-2	10	121.9	6.5			
3-3	15	125.2	9.8			
4-1	5	105.7	8.4			
4-2	10	128.2	5.6			
4-3	15	129.0	11.2			



SAMPLE: A

DESCRIPTION: Brown Sandy CLAY

LABORATORY TEST PROCEDURE: - ASTM D1557-91

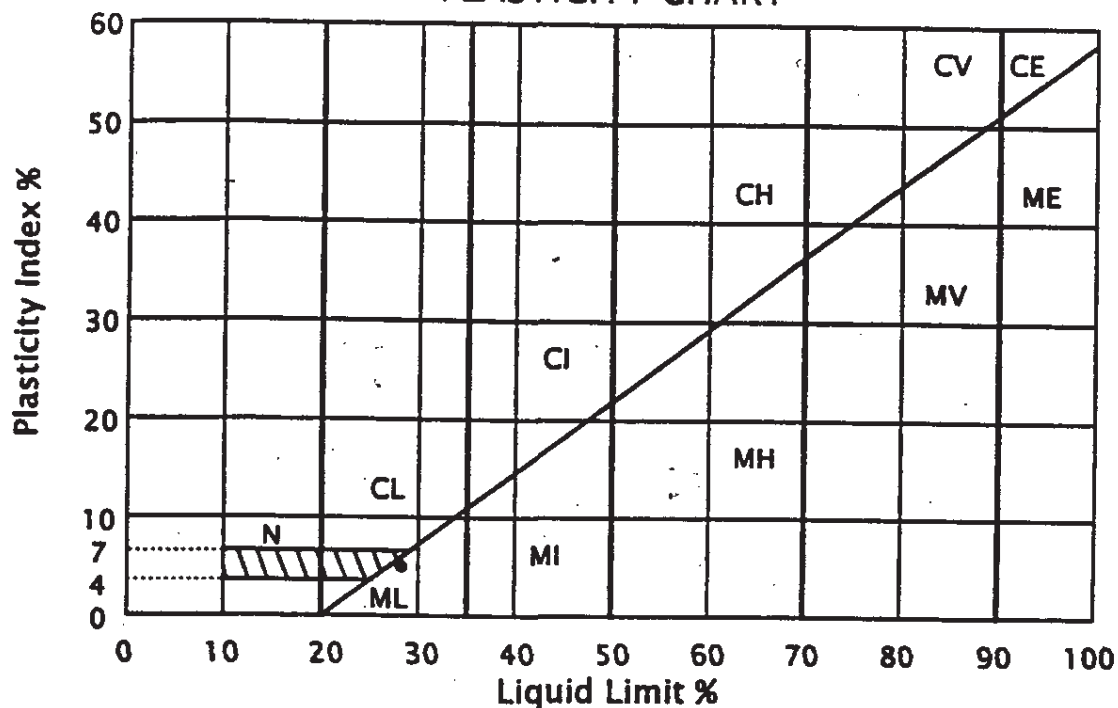
MAXIMUM DRY DENSITY: 120.0 p.c.f.

OPTIMUM MOISTURE CONTENT: 14 %

99 - 018

United Soil Engineering, Inc. 3476 Edward Avenue Santa Clara, CA 95054 (408) 988-2990	COMPACTION TEST A Proposed Harker School SW Corner of Bucknall Road & San Tomas Aquino Road San Jose, California	File No.: 4481-S1	FIGURE
		Drawn by: T. R.	
		Scale: N/A	3 Dec. 1997

PLASTICITY CHART



PLASTICITY DATA

Key Symbol	Hole No.	Depth ft.	Liquid Limit %	Plasticity Index %	Unified Soil Classification Symbol *
●	BAG A	0-1	28	5	ML

*Soil type classification Based on British suggested revisions to Unified Soil Classification System

United Soil Engineering, Inc.

3476 Edward Avenue
Santa Clara, CA 95054
(408) 988-2990

PLASTICITY INDEX

Proposed Harker School
SW Corner of Bucknall Road
& San Tomas Aquino Road
San Jose, California

File No.: 4481-S1

Drawn by: T. R.

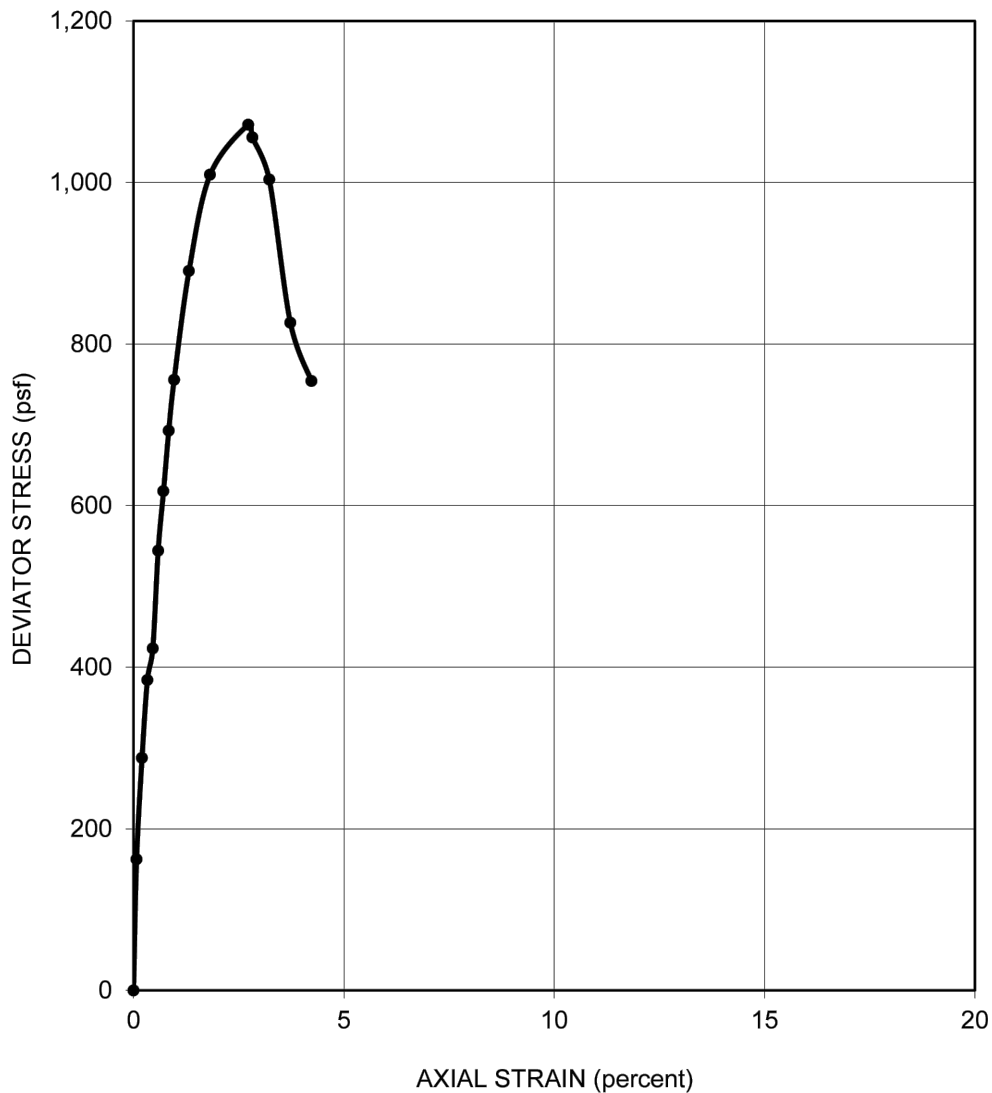
Scale: N/A

FIGURE

4

Dec.
1997

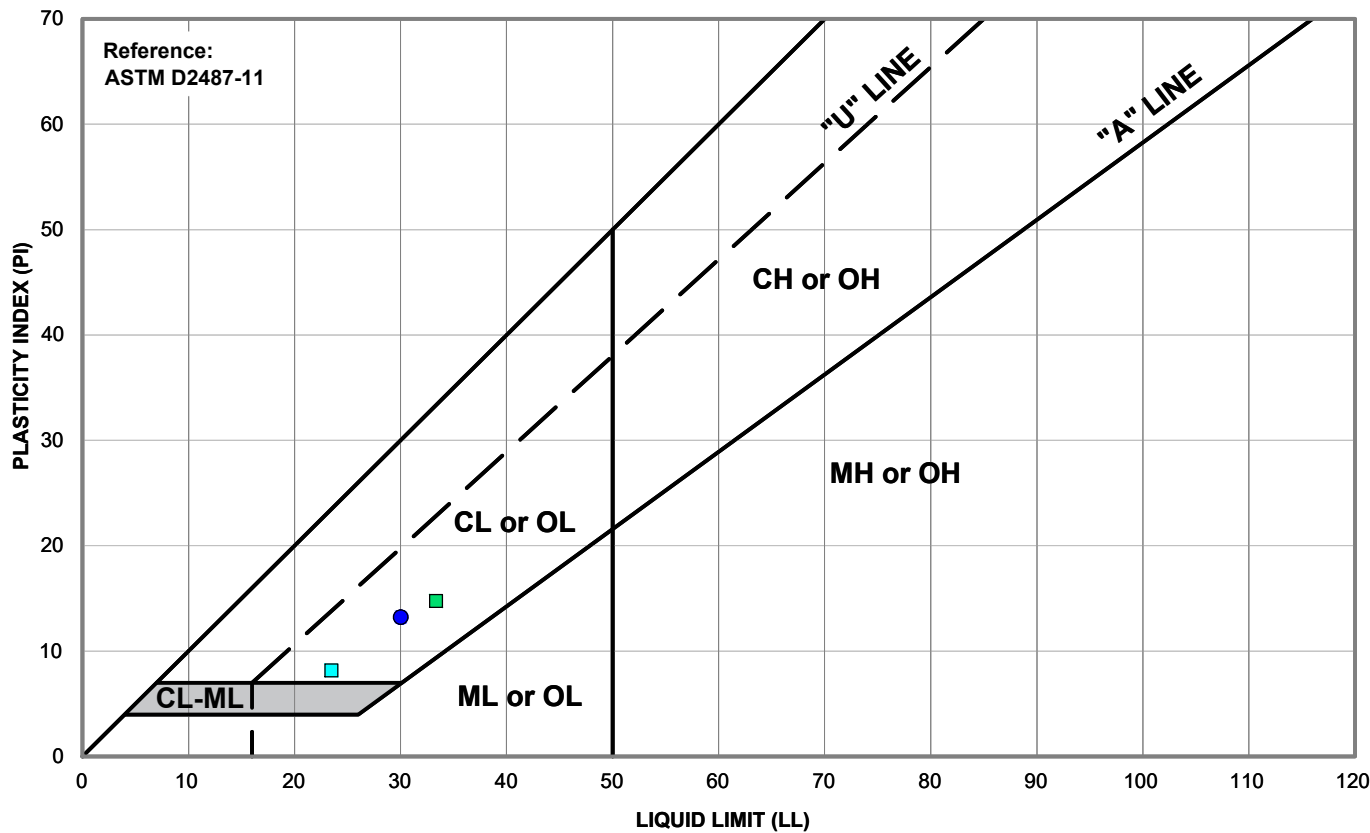
APPENDIX C
LABORATORY TEST RESULTS



SAMPLER TYPE: Sprague & Henwood		SHEAR STRENGTH: 540 psf	
DIAMETER (in.): 2.38	HEIGHT (in.): 5.62	STRAIN AT FAILURE: 2.7 %	
MOISTURE CONTENT: 16.9 %		CONFINING PRESSURE: 700 psf	
DRY DENSITY: 99 pcf		STRAIN RATE: 0.50 % / min	
DESCRIPTION: CLAYEY SAND (SC), brown			SOURCE: LB-1 at 6 feet

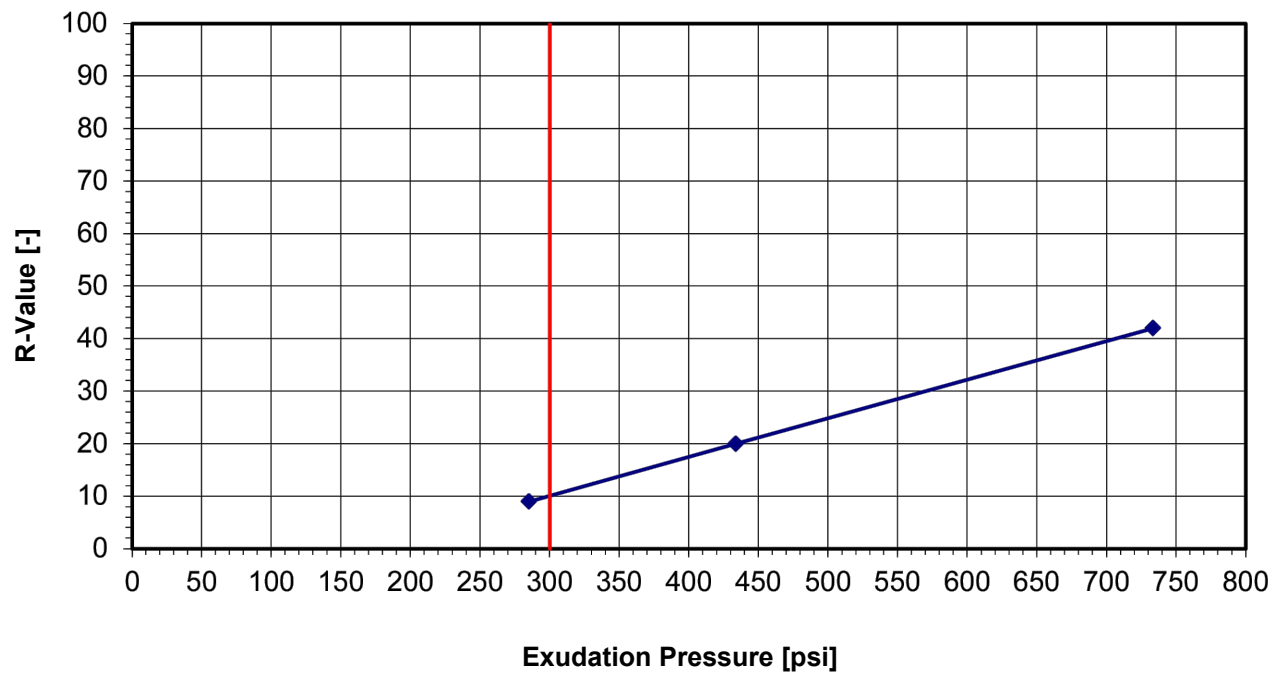
LANGAN Langan CA Inc. 1 Almaden Boulevard, Suite 590 San Jose, CA 95113 T: 408.283.3600 F: 408.283.3601 www.langan.com	Project THE HARKER SCHOOL BUCKNALL CAMPUS SAN JOSE SANTA CLARA COUNTY CALIFORNIA	Figure Title UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST	Project No. 770703801	Figure C-1
			Date 2/27/2025	
			Drawn By JPC	
			Checked By HL	

PLASTICITY CHART



Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	LB-1 at 2 feet	SANDY CLAY (CL), brown	--	30	14	--
■	LB-3 at 2 feet	SANDY CLAY (CL), brown	--	34	15	--
■	LB-3 at 6 feet	GRAVEL with CLAY and SAND (GP-GC), brown	7.3	24	8	10.0

LANGAN Langan CA, Inc. 1 Almaden Boulevard, Suite 590 San Jose, CA 95113 T: 408.283.3600 F: 408.283.3601 www.langan.com	Project THE HARKER SCHOOL BUCKNALL CAMPUS SAN JOSE SANTA CLARA COUNTY CALIFORNIA	Figure Title PLASTICITY CHART	Project No. 770703801	Figure C-2
			Date 02/18/2025	
			Drawn By AG	
			Checked By HL	



—◆— R-Value

— 300 psi exudation pressure

Sample ID	A	B	C	D
Water Content (%)	11.9	10.4	9.8	--
Dry Density (pcf)	127.4	129.5	130.9	--
Exudation Pressure (psi)	285	434	733	--
Expansion Pressure (psf)	0.0	0.0	0.0	--
Resistance Value (R)	9	20	42	--

Sample Source	Sample Description	Sand Equivalent	Expansion Pressure	R-Value
LB-2 at 0 to 5 feet	CLAY with SAND (CL), brown	--	--	10

LANGAN

Langan CA, Inc.
1 Almaden Boulevard, Suite 590
San Jose, CA 95113

T: 408.283.3600 F: 408.283.3601 www.langan.com

Project
**THE HARKER SCHOOL
BUCKNALL CAMPUS**
SAN JOSE
SANTA CLARA COUNTY CALIFORNIA

Figure Title
**RESISTANCE VALUE
TEST REPORT**

Project No.
770703801
Date
2/27/2025
Drawn By
JPC
Checked By
HL

Figure
C-3

APPENDIX D

SOIL CORROSIVITY TEST RESULTS



1100 Willow Pass Court, Suite A
Concord, CA 94520-1006

925 462 2771 Fax. 925 462 2775

www.cercoanalytical.com

6 February, 2025

Job No. 2502003
Cust. No. 12242

Mr. Haotian Li
Langan
1 Almaden Blvd., Suite 590
San Jose, CA 95113

Subject: Subject: Project No.: 770703801/700/USR4
Project Name: Harker School Bucknall Campus
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Li:

Pursuant to your request, CERCO Analytical has analyzed the soil sample submitted on February 04, 2025. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurement, the sample is classified as “corrosive”. All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentration is 38 mg/kg and is determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentration is 47 mg/kg and is determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at this location.

The pH of the soil is 7.10, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potential is 190-mV and is indicative of potentially “moderately corrosive” soils resulting from anaerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc.* at (925) 927-6630.

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

A handwritten signature in blue ink, appearing to read 'J. Darby Howard, Jr.'.

J. Darby Howard, Jr., P.E.
President

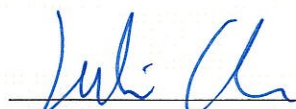
JDH/jdl
Enclosure

Client: Langan
Client's Project No.: 770703801/700/USR4
Client's Project Name: Harker School Bucknall Campus
Date Sampled: 22-Jan-25
Date Received: 4-Feb-25
Matrix: Soil
Authorization: Chain of Custody

Date of Report: 6-Feb-2025

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
2502003-001	LB-1, 4'	190	7.10	-	1,700	-	38	47

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G187	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	4-Feb-2025	5-Feb-2025	-	5-Feb-2025	-	5-Feb-2025	5-Feb-2025


Julia Clauson
Chemist

* Results Reported on "As Received" Basis

N.D. - None Detected

2502003

 CERCO
analytical

Page 1 of 1

[illegible]

APPENDIX E

CARB 435 Analytical Report



McC Campbell Analytical, Inc.

"When Quality Counts"

Analytical Report

WorkOrder: 2501E81

Report Created for: Langan

1 Almaden Blvd, Suite 590
San Jose, CA 95113

Project Contact: Haotian Li

Project P.O.:

Project: 770703801-700-USR 6; Harker School Buckhall
Campus

Project Location:

Project Received: 01/27/2025

Analytical Report reviewed & approved for release on 02/03/2025 by:

Ana Venegas
Project Manager

McC Campbell Analytical Inc. is not accredited for all the testing in this report. The report shall not be reproduced except in full, without the written approval of the laboratory. The analytical results relate only to the items tested.



Glossary of Terms & Qualifier Definitions

Client: Langan

WorkOrder: 2501E81

Project: 770703801-700-USR 6; Harker School Buckhall Campus

Glossary Abbreviation

%D	Serial Dilution Percent Difference
95% Interval	95% Confident Interval
CCV	Continuing Calibration Verification.
CCV REC (%)	% recovery of Continuing Calibration Verification.
CPT	Consumer Product Testing not NELAP Accredited
DF	Dilution Factor
DI WET	(DISTLC) Waste Extraction Test using DI water
DISS	Dissolved (direct analysis of 0.45 µm filtered and acidified water sample)
DLT	Dilution Test (Serial Dilution)
DUP	Duplicate
EDL	Estimated Detection Limit
ERS	External reference sample. Second source calibration verification.
ITEF	International Toxicity Equivalence Factor
LCS	Laboratory Control Sample
LCS2	Second LCS for the batch. Spike level is lower than that for the first LCS; applicable to method 1633.
LQL	Lowest Quantitation Level
MB	Method Blank
MB IS/SS % Rec	% Recovery of Internal Standard or Surrogate in Method Blank, if applicable
MB SS % Rec	% Recovery of Surrogate in Method Blank, if applicable
MDL	Method Detection Limit ¹
ML	Minimum Level of Quantitation
MS	Matrix Spike
MSD	Matrix Spike Duplicate
NA	Not Applicable
ND	Not detected at or above the indicated MDL or RL
NR	Data Not Reported due to matrix interference or insufficient sample amount.
PDS	Post Digestion Spike
PF	Prep Factor
RD	Relative Difference
RL	Reporting Limit ²
RPD	Relative Percent Difference
RRT	Relative Retention Time
RSD	Relative Standard Deviation
SNR	Surrogate is diluted out of the calibration range

¹ MDL is the minimum measured concentration of a substance that can be reported with 99% confidence that the measured concentration is distinguishable from method blank results. Definition and Procedure for the Determination of the Method Detection Limit, Revision 2, 40CFR, Part 136, Appendix B, EPA 821-R-16-006, December 2016. Values are based upon our default extraction volume/amount and are subject to change.

² RL is the lowest level that can be reliably determined within specified limits of precision and accuracy during routine laboratory operating conditions. (The RL cannot be lower than the lowest calibration standard used in the initial calibration of the instrument and must be greater than the MDL.) Values are based upon our default extraction volume/amount and are subject to change.



Glossary of Terms & Qualifier Definitions

Client: Langan

WorkOrder: 2501E81

Project: 770703801-700-USR 6; Harker School Buckhall Campus

SPK Val	Spike Value
SPKRef Val	Spike Reference Value
SPLP	Synthetic Precipitation Leachate Procedure
ST	Sorbent Tube
TCLP	Toxicity Characteristic Leachate Procedure
TEQ	Toxicity Equivalents
TNTC	"Too Numerous to Count;" greater than 250 colonies observed on the plate.
TZA	TimeZone Net Adjustment for sample collected outside of MAI's Coordinated Universal Time (UTC). (Adjustment for Daylight Saving is not accounted.)
WET (STLC)	Waste Extraction Test (Soluble Threshold Limit Concentration)

Analytical Qualifiers

k10	CARB 435 Exception 1 - No asbestos detected. The limit of quantitation (LOQ) = 0.25%.
k12	Asbestos observed but no asbestos points counted
k15	Chrysotile



Analytical Report

Client: Langan
Date Received: 01/27/2025 14:00
Date Prepared: 01/28/2025
Project: 770703801-700-USR 6; Harker School Buckhall Campus

WorkOrder: 2501E81
Extraction Method: CARB 435 Asbestos
Analytical Method: 435 CARB
Unit: %

Asbestos (CARB 435) 400 Point Count

Client ID	Lab ID	Matrix	Date Collected	Instrument	Batch ID
AC-1	2501E81-001A	Soil	01/22/2025 16:00	Microscope	310482

<u>Analytes</u>	<u>Result</u>	<u>RL</u>	<u>DE</u>	<u>Date Analyzed</u>
Asbestos	ND	0.25	1	01/30/2025 11:00

Analyst(s): DA

Analytical Comments: k10

Client ID	Lab ID	Matrix	Date Collected	Instrument	Batch ID
AC-2	2501E81-002A	Soil	01/22/2025 08:00	Microscope	310482

<u>Analytes</u>	<u>Result</u>	<u>RL</u>	<u>DE</u>	<u>Date Analyzed</u>
Asbestos	ND	0.25	1	01/30/2025 11:20

Analyst(s): DA

Analytical Comments: k10

Client ID	Lab ID	Matrix	Date Collected	Instrument	Batch ID
AC-3	2501E81-003A	Soil	01/22/2025 08:30	Microscope	310482

<u>Analytes</u>	<u>Result</u>	<u>RL</u>	<u>DE</u>	<u>Date Analyzed</u>
Asbestos	< 0.25	0.25	1	01/30/2025 11:40

Analyst(s): DA

Analytical Comments: k12,k15

(Cont.)



Analytical Report

Client: Langan
Date Received: 01/27/2025 14:00
Date Prepared: 01/28/2025
Project: 770703801-700-USR 6; Harker School Buckhall Campus

WorkOrder: 2501E81
Extraction Method: CARB 435 Asbestos
Analytical Method: 435 CARB
Unit: %

Asbestos (CARB 435) 400 Point Count

Client ID	Lab ID	Matrix	Date Collected	Instrument	Batch ID
AC-4	2501E81-004A	Soil	01/22/2025 09:00	Microscope	310482

Analytes	Result	RL	DE	Date Analyzed
Asbestos	< 0.25	0.25	1	01/30/2025 13:05

Analyst(s): DA

Analytical Comments: k12,k15



1534 Willow Pass Rd
Pittsburg, CA 94565-1701
(925) 252-9262

☐ WaterTrax☐ CLIP☐ EDF

CHAIN-OF-CUSTODY RECORD

Page 1 of 1

WorkOrder: 2501E81

ClientCode: TRSJ

☒ EQuIS☐ Dry-Weight☒ Email☐ HardCopy☐ ThirdParty☐ J-flag☐ Detection Summary☐ Excel**Report to:**

Haotian Li
Langan
1 Almaden Blvd, Suite 590
San Jose, CA 95113
(407) 494-8451 FAX:

Email: hli@langan.com
cc/3rd Party:
PO:
Project: 770703801-700-USR 6; Harker School
Buckhall Campus

Bill to:

Accounts Payable
Langan
135 Main St, Suite 1500
San Francisco, CA 94105
Langan_InvoiceCapture@concursolutio

Requested TAT: 5 days;*Date Received:* **01/27/2025***Date Logged:* **01/27/2025**

Lab ID	ClientSampID	Matrix	Collection Date	Hold	Requested Tests (See legend below)											
					1	2	3	4	5	6	7	8	9	10	11	12
2501E81-001	AC-1	Soil	1/22/2025 16:00	☐	A	A										
2501E81-002	AC-2	Soil	1/22/2025 08:00	☐	A	A										
2501E81-003	AC-3	Soil	1/22/2025 08:30	☐	A	A										
2501E81-004	AC-4	Soil	1/22/2025 09:00	☐	A	A										

Test Legend:

1	CARB435_400
5	
9	

2	PRDisposal Fee
6	
10	

3	
7	
11	

4	
8	
12	

Prepared by: Emily Perez**Comments:**

NOTE: Soil samples are discarded 60 days after receipt unless other arrangements are made (Water samples are 30 days).
Hazardous samples will be returned to client or disposed of at client expense.



WORK ORDER SUMMARY

Client Name: LANGAN
Client Contact: Haotian Li
Contact's Email: hli@langan.com

Project: 770703801-700-USR 6; Harker School Buckhall Campus

Work Order: 2501E81
QC Level: LEVEL 2
Date Logged: 1/27/2025

Comments:

☐ WaterTrax ☐ CLIP ☐ EDF ☐ Excel ☒ EQUIS ☒ Email ☐ HardCopy ☐ ThirdParty ☐ J-flag

LabID	ClientSampID	Matrix	Test Name	Cont./ Comp.	Bottle & Preservative	U**	Head Space	Dry- Weight	Collection Date & Time	TAT	Test Due Date	Sediment Content	Hold	Sub Out
001A	AC-1	Soil	Asbestos, CARB 435, 400 Point	1	Plastic Baggie, Medium	☐	☐	☐	1/22/2025 16:00	5 days	2/3/2025		☐	☐
002A	AC-2	Soil	Asbestos, CARB 435, 400 Point	1	Plastic Baggie, Medium	☐	☐	☐	1/22/2025 8:00	5 days	2/3/2025		☐	☐
003A	AC-3	Soil	Asbestos, CARB 435, 400 Point	1	Plastic Baggie, Medium	☐	☐	☐	1/22/2025 8:30	5 days	2/3/2025		☐	☐
004A	AC-4	Soil	Asbestos, CARB 435, 400 Point	1	Plastic Baggie, Medium	☐	☐	☐	1/22/2025 9:00	5 days	2/3/2025		☐	☐

NOTES: * STLC and TCLP extractions require 2 days to complete; therefore, all TATs begin after the extraction is completed (i.e., One-day TAT yields results in 3 days from sample submission).

- ISM prep requires 5 to 10 days to complete; therefore, all TATs begin after the extraction is completed (i.e., One-day TAT yields results in 6 to 11 days from sample submission). Due date listed on WO summary will not accurately reflect the time needed for sample preparation.

- Organic extracts are held for 40 days before disposal; Inorganic extract are held for 30 days.

- MAI assumes that all material present in the provided sampling container is considered part of the sample - MAI does not exclude any material from the sample prior to sample preparation unless requested in writing by the client.

U** = An unpreserved container was received for a method that suggests a preservation in order to extend hold time for analysis.

16444

Page ___ of ____

LANGAN

CHAIN OF CUSTODY RECORD

Page ___ of ____

☐	135 Main Street, Suite 1500, San Francisco, CA 94105
☐	501 14th Street, Third Floor, Oakland, CA 94612
☐	3320 Data Drive, Suite 350, Rancho Cordova, CA 95670-7982
☒	1 Almaden Boulevard, Suite 590, San Jose, CA 95113

Site Name: Marker School Bucknall Campus
Job Number: 770703801-700-USR6
Project Manager/Contact: Haotian Li hli@langan.com
Samplers: Baggies
Recorder (Signature Required): Haotian Li Sholtb

Analysis Requested

Turnaround Time

STANDARD

No. Containers

Matrix	& Preservative
--------	----------------

[illegible]

White Copy - Original

Yellow Copy - Laboratory

Pink Copy - Field

COC Number:

0.9 net
11233



Sample Receipt Checklist

Client Name: Langan
Project: 770703801-700-USR 6; Harker School Buckhall Campus
WorkOrder №: 2501E81 Matrix: Soil
Carrier: Laurie Moore (MAI Courier)

Date and Time Received: 1/27/2025 14:00
Date Logged: 1/27/2025
Received by: Emily Perez
Logged by: Emily Perez

Chain of Custody (COC) Information

Chain of custody present?	Yes ☒	No ☐	
Chain of custody signed when relinquished and received?	Yes ☒	No ☐	
Chain of custody agrees with sample labels?	Yes ☒	No ☐	
Sample IDs noted by Client on COC?	Yes ☒	No ☐	
Date and Time of collection noted by Client on COC?	Yes ☒	No ☐	
Sampler's name noted on COC?	Yes ☒	No ☐	
COC agrees with Quote?	Yes ☐	No ☐	NA ☒

Sample Receipt Information

Custody seals intact on shipping container/cooler?	Yes ☐	No ☐	NA ☒
Custody seals intact on sample bottles?	Yes ☐	No ☐	NA ☒
Shipping container/cooler in good condition?	Yes ☒	No ☐	
Samples in proper containers/bottles?	Yes ☒	No ☐	
Sample containers intact?	Yes ☒	No ☐	
Sufficient sample volume for indicated test?	Yes ☒	No ☐	

Sample Preservation and Hold Time (HT) Information

All samples received within holding time?	Yes ☒	No ☐	NA ☐
Samples Received on Ice?	Yes ☒	No ☐	

(Ice Type: WET ICE)

Sample/Temp Blank temperature	Temp: 0.9°C	NA ☐	
ZHS conditional analyses: VOA meets zero headspace requirement (VOCs, TPHg/BTEX, RSK)?	Yes ☐	No ☐	NA ☒
Sample labels checked for correct preservation?	Yes ☒	No ☐	
pH acceptable upon receipt (Metal: <2)?	Yes ☐	No ☐	NA ☒

UCMR Samples:

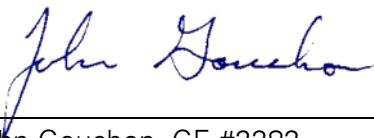
pH tested and acceptable upon receipt (200.7: ≤2; 533: 6 - 8; 537.1: 6 - 8)?	Yes ☐	No ☐	NA ☒
Free Chlorine tested and acceptable upon receipt (<0.1mg/L) [not applicable to 200.7]?	Yes ☐	No ☐	NA ☒

Comments:

DISTRIBUTION

Electronic copies: The Harker School
500 Saratoga Avenue
San Jose, California 95129

QUALITY CONTROL REVIEWER

A handwritten signature in blue ink, reading "John Gouchon". The signature is written in a cursive style with a horizontal line underneath it.

John Gouchon, GE #2282
Principal