CHABOT-LAS POSITAS COMMUNITY COLLEGE DISTRICT
PURCHASING DEPARTMENT

May 15, 2026

Addendum No. 01
Request for Proposals (RFP) B25/26-12
Las Positas College STEAM Project – Inc 2 Special Inspection

To: All Prospective Bidders

This Addendum ONE (01) is issued to incorporate the following changes, additions or deletions to the RFP B25/26-12. Any modifications/changes made by this addendum affect only the portions or paragraphs specifically identified herein; all remaining portions of the RFP B25/26-12 to remain in force. It is the responsibility of all responders to conform to this addendum.

A. RFI QUESTIONS AND RESPONSES:

QUESTION 1: The RFP states that proposals shall be limited to twenty-five (25) single-sided pages, excluding the cover letter, tabs, and resumes.

a) Please confirm whether the Fee section, including Exhibit A and Exhibit B (fee forms), is included in or excluded from the 25-page proposal limit.

b) Please also confirm whether Attachment A (Drug-Free Workplace Certification) and Attachment B (Non-Collusion Affidavit) may be submitted as an appendix and do not count toward the 25-page limit.

RESPONSE 1:
A) Yes, this is part of the 25-page limit
B) Yes, these two forms can be included as a appendix and will not count towards the page limit

QUESTION 2: Will the current geotechnical engineer for this project be retained for consulting, recommendations, and earthwork testing? Will the selected lab of record be expected to assume the responsibilities of the geotechnical engineer of record for the project?

RESPONSE 2: It is the intent for the selected LOR to assume responsibilities of the Geotechnical engineer.

QUESTION 3: If the selected LOR will be performing earthwork testing, please provide the geotechnical investigation report.

RESPONSE 3: Report is included in this addendum.

QUESTION 4: Since this is Increment 2, could you please provide us with the full completed work from Increment 1? Is the pad 100% complete and certified? Are all underground utilities installed**?**

RESPONSE 4: INC. 1 Utility work is 95% complete; Building pad, lime treatment to start in next two weeks, build out of the electrical pad for the new Inc 2 switchgear and transformer to commence in late May. This work will be overseen by Inc 1, LOR

QUESTION 5: What are the locations of the rebar and structural steel fabrication shops?

RESPONSE 5:
Structural Steel at Golden State Steel - 2250 S Golden State Blvd, Fowler, CA 93625
Reinforcements (Rebar) at CMC Steel, 120 W Larch Rd, Tracy, CA 95304

QUESTION 6: Are we to include Attachment A (Drug-Free Workplace Certification) and Attachment B (Non-Collusion Affidavit) in the quals package?

RESPONSE 6: See response 1B.

QUESTION 7: If so, can we add a section titled "Appendix" or "Forms?"

RESPONSE 7: See response 1B.

QUESTION 8: Do you have a soils report for this site?

RESPONSE 8: Yes, Geotechnical Report Included in Addendum

QUESTION 9: In the Insurance section, you indicate Builders Risk for the Full Value of the Work. This is not something that is typically applicable to the materials testing firm. Was this left in by error?

RESPONSE 9: Yes, this is a mistake. Please disregard.

QUESTION 10: Would it be acceptable to submit our fee breakdown using our own format in lieu of Exhibit B? Exhibit B does not appear to capture all inspection items outlined in the DSA 103 form.

RESPONSE 10: If there are inspection items missing from our form that appear on the DSA 103 you may include those items at the end of Exhibit B or as an attachment to exhibit B.

QUESTION 11: Should geotechnical observation and soil compaction testing services be included in our scope? If so, could you please share the geotechnical report for the project site?

RESPONSE 11: Yes, Geotechnical Report Included in Addendum.

QUESTION 12: Has a contractor been selected? If so, could you please provide the current project schedule, if available?

RESPONSE 12: Yes, schedule is included in RFP package. See pages 12 – 24 in the RFP.

QUESTION 13: Do you have information on the location and anticipated schedule of the structural steel fabrication shop to assist with estimating shop welding inspection visits?

RESPONSE 13: Golden State Steel in Fresno is the steel subcontractor.

QUESTION 14: Is there a sample agreement available for our review?

RESPONSE 14: See attached Draft Professional Service Contract.

QUESTION 15: Which firm provided special inspections and materials testing services for Increment 01?

RESPONSE 15: CONSTRUCTION TESTING SERVICES

***All other terms and conditions remain unchanged*.**

Michael McClung - Buyer, Purchasing and Warehouse Services
Chabot-Las Positas Community College District

Geotechnical Evaluation and Geologic Hazards Assessment

Las Positas College – STEAM Building

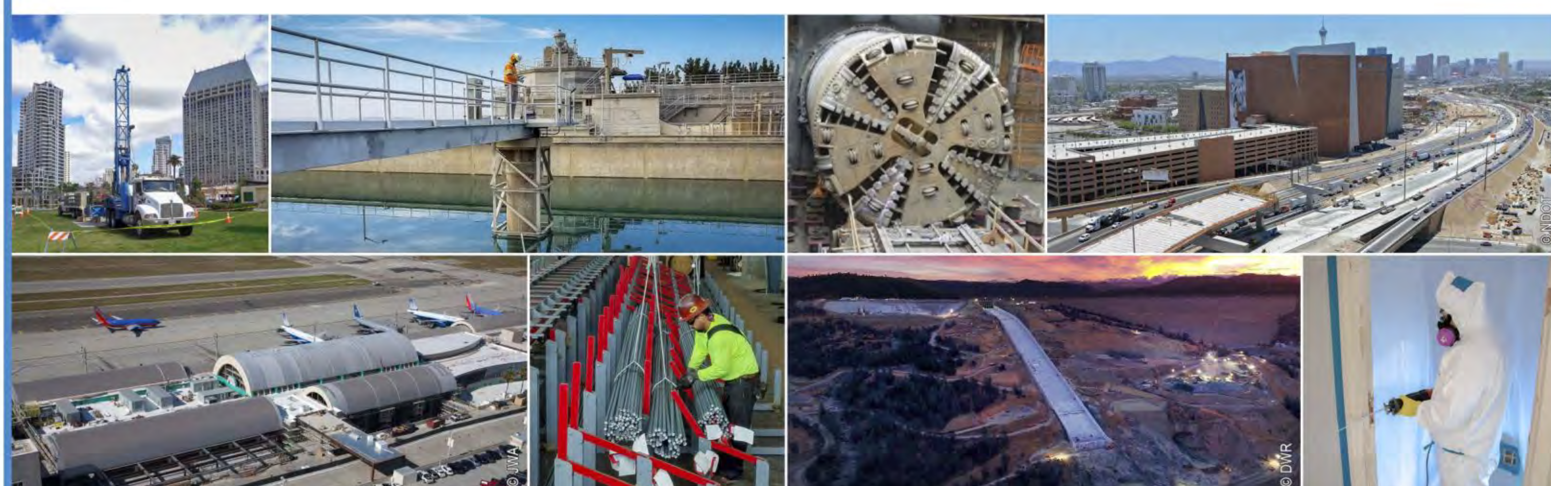
3000 Campus Hill Drive

Livermore, California

Chabot Las Positas Community College District

7600 Dublin Boulevard | Dublin, California 94568

November 22, 2023 | Project No. 401294038



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Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

35+ YEARS

Geotechnical Evaluation and Geologic Hazards Assessment
Las Positas College – STEAM Building
3000 Campus Hill Drive
Livermore, California

Chabot Las Positas Community College District
7600 Dublin Boulevard | Dublin, California 94568

November 22, 2023 | Project No. 401294038

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1 INTRODUCTION

In accordance with your authorization and our agreement dated September 15, 2023, Ninyo & Moore has performed a Geotechnical Evaluation and Geologic Hazards Assessment, based on our proposal dated September 8, 2023, for the proposed STEAM Building project at Las Positas College located at 3000 Campus Hill Drive in Livermore, California (Figure 1). These services were performed to evaluate the subsurface conditions at site and to assess the potential impact of regional geologic and seismic hazards on the project under consideration. This report presents the findings and conclusions from our geologic hazards assessment and subsurface evaluation, and our geotechnical recommendations for the design and construction of the project.

2 SCOPE OF SERVICES

Our scope of services included the following:

- Review of readily available geologic and seismic literature pertinent to the project area including regional geologic maps, fault maps, seismic hazard maps and aerial photography.
- Site reconnaissance to observe the general site conditions and to mark the proposed locations for subsurface exploration.
- Coordination with Underground Service Alert to locate the underground utilities in the vicinity of the proposed exploration locations.
- A private utility survey by electro-magnetic scanning to evaluate the exploration locations for underground utility conflicts.
- Procurement of a boring permit from the Zone 7 Water Agency.
- Subsurface exploration consisting of four (4) auger borings drilled to depths of up to 26½ feet below the ground surface and five (5) cone penetration test (CPT) soundings advanced to depths of up to approximately 51 feet below the ground surface.
- Performance of a geophysical Refraction Microtremor (ReMi) survey along one line to evaluate seismic site classification.
- Infiltration testing at one (1) location using the double ring infiltrometer method.
- Laboratory testing on selected soil samples to evaluate in-situ soil moisture content and dry density, fines content, particle size distribution, Atterberg limits, expansion index, shear strength, unconfined compressive strength, and soil corrosivity.
- Compilation and engineering analysis of the information obtained from the background review, subsurface exploration for this study, subsurface exploration from a previous study for Building 1900 at Las Positas College (Ninyo & Moore, 2008), and the results of the laboratory testing.

- Preparation of this report presenting our findings regarding the geotechnical conditions encountered at the project site, the conclusions from our geologic hazards assessment, and our geotechnical recommendations for the design and construction of the project.

3 PROJECT DESCRIPTION

The proposed project will construct a comprehensive center for the arts and sciences, divided into two separate major components: STEAM-Arts and STEAM-Sciences. The project is in the preliminary design phase and may be designed as an integrated building or as two separate buildings with a combined gross square footage of approximately 87,000 square feet that will include classrooms, studios, laboratories, faculty offices, and an arts gallery. It is assumed that the proposed STEAM building(s) will be two- to three-story structures, constructed at (or within a couple feet of) the existing grade, with a combined footprint area of approximately 40,000 square feet. The project will also include an approximately 5,000-square-foot, truss-supported shade structure over the amphitheater stage associated with the Mertes Center for the Arts (Building 4000), and renovate portions of Building 1800. It is assumed that renovations to Building 1800 will not include new foundations or modification of existing foundations.

4 SITE DESCRIPTION

The campus of Las Positas Community College occupies an irregularly-shaped parcel of approximately 147 acres located at 3000 Campus Hill Drive in Livermore, California (Figure 1). The campus is located approximately 3,500 feet north of Highway 580 and east of Collier Canyon Road on a gently inclined southwest facing slope. A perimeter road circles the campus, linking the parking lots located around the campus buildings.

The STEAM building is proposed to be constructed southwest of Building 1800 (Figure 2) in the central portion of the campus along the northern edge of the campus at the current location of Buildings 600 and 800, which will be demolished as part of this project. The amphitheater stage where the shade canopy will be constructed is northeast of Building 4000, southeast of Parking Lot G, and about 100 feet west of the proposed location for the STEAM building. The central portion of the campus encircled by the Campus Loop Road generally slopes down to the southwest at an average overall gradient of approximately 2 percent across the proposed location for the STEAM building and Amphitheater stage canopy. The existing grade at the proposed site for the STEAM building ranges from approximately 483 feet above mean sea level (MSL) at the edge of the pad for Building 1800, to approximately 476 feet above MSL around at the southern portion of the pad for Building 800 to approximately 468 feet above MSL at the southern edge of the pad around Building 600 (Google, 2023) with an approximately 3-foot high retaining wall between Buildings 600 and 800. The ground around the southern portion of the building pad for

Building 800 slopes up to the east and down to the west at gradients of up to approximately 6:1 (horizontal to vertical). The amphitheater is generally recessed below the adjacent grade. The top of the amphitheater stage, at approximately 456 feet above MSL, is about 15 feet below the adjacent grade in Parking Lot G to the northwest with a concrete wall retaining a grade differential of approximately 10 feet and a backfill slope of approximately 2:1 (horizontal to vertical). The slope gradient of the exposed ground northeast and southwest of the stage is approximately 2½:1 (horizontal to vertical) or flatter where not terraced by short concrete walls. The amphitheater seating area located south and east of the stage is generally paved and terraced.

5 FIELD EXPLORATION AND LABORATORY TESTING

The field exploration for this study included a site reconnaissance conducted on September 27, 2023, and subsurface exploration that consisted of four exploratory borings, five CPT soundings, one infiltration test, and a geophysical survey. The log from a boring drilled in 2008 for a previous evaluation (Ninyo & Moore, 2008) was also reviewed as part of this study. The approximate locations of the borings, CPT soundings, infiltration test, and geophysical survey for this study are noted on Figure 2 along with the location of the boring from the previous evaluation designated as Boring B-5. Prior to commencing the subsurface exploration, Ninyo & Moore reviewed utility plans provided by the District, notified Underground Service Alert for field marking of the existing utilities, and arranged for a private utility survey to check the exploration locations for underground utility conflicts. The borings and soundings were backfilled with neat cement grout on October 6, 2023 and drilled holes in the pavement were patched.

5.1 Cone Penetration Testing

Ninyo & Moore retained Middle Earth Geo Testing to perform the cone penetration testing. The CPT soundings were advanced to depths of up to approximately 51 feet below the ground surface on October 6, 2023 using a truck-mounted rig with 20-ton reaction capacity. Tip resistance, sleeve friction, and pore water pressure data were collected and recorded electronically at intervals of approximately 2 inches while the cone was advanced. The tip resistance and sleeve friction were used to evaluate the soil behavior type (Robertson, 1986). Pore pressure dissipation tests were also performed in the soundings but stable pore pressure readings indicative of static groundwater levels could not be achieved within the time available for testing. The CPT data, and the interpreted soil behavior types are presented on the logs in Appendix A.

5.2 Exploratory Borings

Ninyo & Moore retained California Geotech Services to drill four exploratory borings designated as Boring B-1 through B-4. The borings were advanced to depths of up to approximately 26½ feet below the ground surface on October 6, 2023 using a Mobile B-24 truck-mounted rig equipped with solid stem augers. Exploration Geoservices drilled Boring B-5 as part of our previous study (Ninyo & Moore, 2008) on March 12, 2008 to a depth of approximately 50 feet below the ground surface using a Mobile B-53 truck-mounted rig equipped with hollow stem augers. Ninyo & Moore logged the subsurface conditions exposed in the borings and collected bulk and relatively undisturbed soil samples from the borings. Soil was field-classified in accordance with the Unified Soil Classification System (USCS) using the visual-manual procedures in Standard D 2488 by the American Society for Testing and Materials (ASTM). Bulk soil samples were collected from the augers and from a split-barrel Standard Penetration Test (SPT) sampler with an external diameter of 2-inches and an unlined internal diameter of 1⅜ inches. Relatively undisturbed soil samples were collected using a 3-inch external-diameter Modified California split-barrel sampler with stainless steel liners having an inside diameter of approximately 2.4 inches. The split-barrel samplers were driven into the soil at the bottom of the borehole to a depth of 18 inches with a 140-pound safety hammer lifted for 30-inch freefall using a rope and cathead where noted or with a 140-pound wireline hammer falling 30 inches on a spooling cable. Sampler penetration resistance, expressed as hammer blows per foot of penetration over the last 12 inches of the 18-inch drive, is recorded on the boring logs. The sampler penetration resistance recorded on the logs has not been corrected for the effects of overburden stress, sampler size or hammer efficiency. Detailed logs of the borings are presented in Appendix B along with a description of the sampling procedures utilized.

5.3 Laboratory Testing

Ninyo & Moore performed geotechnical laboratory testing on soil samples collected from the borings to evaluate in-situ soil moisture content and dry density, percent passing the #200 sieve, particle size distribution, Atterberg limits, expansion index, shear strength, and unconfined compressive strength. The results of the in-situ moisture and dry density tests are presented on the boring logs in Appendix B. The results of the other laboratory tests are presented in Appendix C. One soil sample collected from Boring B-2 was submitted to CERCO Analytical for corrosivity testing and evaluation. The results of the testing from the evaluation are presented in Appendix D.

5.4 Infiltration Testing

Ninyo & Moore performed field testing at the site on October 20, 2023 to evaluate the infiltration rate of near surface soil for consideration in the selection and design of storm water management systems. The double ring infiltrometer method was used to evaluate the infiltration rate at one test location designated as P-1 located near Boring B-1. The test was performed at the ground surface.

The double-ring infiltrometer method limits the influence that the lateral exfiltration of water can have on the measured infiltration rate. Subsurface conditions at the test location consisted of lean clay. After clearing the test location of loose material, a 6-inch diameter metallic ring was driven into the subgrade about 4 inches. A second 4-inch diameter metallic ring was inserted in the center of the 6-inch ring and driven about 3¾ inches into the subgrade. Water was added in the annular space and the inner ring. The water level of about 6 inches was maintained in the annular space and inner ring by adding water using two Marriotte tubes having the capacity of 3,000 and 10,000 cubic centimeter (cc). The drop in the water level in the Marriotte tubes were recorded over periodic intervals. The infiltration rate reported is the average infiltration rate in the inner ring over the last four measurement intervals. Test results are summarized in Table 1 and presented in Appendix E. The test results indicate that the infiltration rate of near-surface soil at the location tested is relatively slow. Due to the variability of subsurface materials encountered during our exploration, variability in infiltration rates should be anticipated. Additional testing to refine estimates of infiltration rate may be advisable where infiltration-type stormwater management facilities are proposed, particularly where overflow of the facility would be problematic.

Table 1 – Infiltration Test Results

Test	Test Depth (feet)	Subsurface Conditions	Infiltration Rate (inches/hour)
P-1	At Surface	Lean Clay	0.3

5.5 Geophysical Survey

Ninyo & Moore performed a geophysical survey at the project site using the refraction microtremor (ReMi) method on October 16, 2023 to evaluate the characteristic shear wave velocity at the site to a depth of 100 feet below the ground surface (V_{s100}) for seismic site classification. The survey consisted of collecting microtremor array measurements (MAM) from surface waves using ambient noise as a passive source and an array of geophones along one seismic line. The survey line location is illustrated on Figure 2. The survey results are provided in Appendix F along with a description of the survey procedure.

6 GEOLOGIC AND SUBSURFACE CONDITIONS

Our findings regarding regional geologic setting, site geology, subsurface stratigraphy, and groundwater conditions at the subject site are provided in the following sections.

6.1 Regional Geologic Setting

The site is located east of San Francisco Bay in the Coast Ranges geomorphic province of California. The Coast Ranges are comprised of several mountain ranges and structural valleys formed by tectonic processes commonly found around the Circum-Pacific belt. Basement rocks have been sheared, faulted, metamorphosed, and uplifted, and are separated by thick blankets of Cretaceous and Cenozoic sediments that fill structural valleys and line continental margins. The San Francisco Bay area has several ranges that trend northwest, parallel to major strike-slip faults such as the San Andreas, Hayward, and Calaveras (Figure 3). Major tectonic activity associated with these and other faults within this regional tectonic framework consists primarily of right-lateral, strike-slip movement.

6.2 Site Geology

Regional mapping (Figure 4) by Dibblee & Minch (2006) based on previous mapping by Dibblee, (1980) indicates that the site is underlain by alluvial gravel, sand, and clay, of Holocene age, near a contact with Pleistocene-age Livermore Gravel. The Livermore Gravel is described as poorly to moderately consolidated, indistinctly bedded, cobble conglomerate, gray conglomeratic sandstone, and gray coarse-grained sandstone (Graymer et al, 1996); and also includes some siltstone and claystone. Regional mapping of Quaternary deposits by Knusden et al. (2000) indicates that the site is underlain by alluvial fan deposits of late Pleistocene to Holocene age. Later mapping of Quaternary deposits by Witter et al. (2006) indicates that the site is underlain by alluvial fan deposits of late Pleistocene age.

6.3 Stratigraphy

Our subsurface exploration at the site encountered asphalt pavement, fill, topsoil, and alluvium below asphalt pavement. The following sections provide a generalized description of the geologic units encountered during the subsurface evaluation. More detailed descriptions are presented on the boring logs in Appendix B. Our interpretation of the subsurface conditions at the site is depicted on cross sections AA', BB', and CC' (Figures 5 through 7).

6.3.1 Pavement

With the exception of Borings B-1 and B-2, the borings and CPT soundings for this evaluation were drilled or pushed through asphalt concrete pavement. The pavement section encountered in the borings generally consisted of asphalt concrete over aggregate base. The asphalt concrete section encountered in the borings ranged in thickness from approximately 2 inches (Boring B-3) to approximately 3 inches (Boring B-5). The aggregate base section encountered in the borings ranged in thickness from approximately 3 inches (Boring B-3) to approximately 9 inches (Boring B-4). Variations in the thickness of the asphalt concrete or aggregate base layers may be encountered due to past maintenance, utility work, or other factors.

6.3.2 Fill

Fill was encountered in Boring B-2 at the ground surface to a depth of approximately 3 feet. The fill, as encountered, consisted of gray, moist, stiff, lean clay. Additional fill may be present near the existing buildings and underground utilities.

6.3.3 Topsoil

Native topsoil was encountered in Boring B-1 at the ground surface, and below the pavement in Borings B-3 and B-5 below the pavement. The topsoil extended to depths that ranged from approximately 3 inches below the ground surface (Boring B-1) to approximately 3 feet below the ground surface (Boring B-5). As encountered in the borings, the topsoil generally consisted of dark brown to black, soft to very stiff, lean to fat clay, and loose gravel. Organic matter was observed in the topsoil at Boring B-1.

6.3.4 Alluvium

Alluvium was encountered below the topsoil in Borings B-1, B-3, and B-5, below the fill in Boring B-2, and below the pavement section in Boring B-4 to the depths explored. The alluvium, as encountered, generally consisted of light brown to brown, moist, firm to hard, sandy clay and lean to fat clay with trace to little amounts of sand and gravel; with occasional layers of light brown to brown, moist to wet, loose to very dense sand and clayey sand, and loose to medium dense gravel and clayey gravel. Scattered veins of caliche were encountered in the alluvium.

6.4 Groundwater

Groundwater was not encountered in Borings B-1 through B-4 which extended to depths of approximately 21 to 26½ feet below the ground surface. Pore pressure dissipation tests conducted as part of the cone penetration testing at depths between approximately 36 and 43 feet below the ground surface did not converge on stable readings indicative of static groundwater levels. Groundwater was encountered in Boring B-5 at a depth of approximately 46½ feet below the ground surface during our previous subsurface exploration (Ninyo & Moore, 2008). Groundwater may rise to a higher elevation than was encountered in the borings due to the relatively slow seepage rates in clay and the short time available for seepage into the boring. Regional groundwater records in the seismic hazard zone report for the Livermore 7.5-minute

USGS quadrangle prepared by the California Geological Survey (CGS, 2008a), indicate that the historic high groundwater level at the site is more than 30 feet below the ground surface.

Variations or fluctuations in the groundwater level across the site and over time may occur due to seasonal precipitation, spatial variations in topography or subsurface hydrogeologic conditions, or as a result of changes to nearby irrigation practices or groundwater pumping. In addition, seeps may be encountered at elevations above the observed groundwater levels due to perched groundwater conditions, leaking pipes, preferential drainage, or other factors not evident at the time of our exploration.

7 GEOLOGIC HAZARDS AND CONSIDERATIONS

This study considered a number of geologic and seismic hazards relevant to the proposed project including seismic hazards, landsliding and slope stability, naturally occurring asbestos, unsuitable materials, collapsible soil, settlement of compressible soil layers from static loading, corrosive soil, expansive soil, and excavation characteristics. These issues are discussed in the following subsections.

7.1 Seismic Hazards

The seismic hazards considered in this study include the potential for ground surface rupture due to faulting, seismic ground shaking, liquefaction and strain softening, dynamic settlement, seismic slope stability, and tsunamis and seiches. These potential hazards are discussed in the following subsections.

7.1.1 Historical Seismicity

The site is located in a seismically active region. Figure 3 presents the location of the site relative to the epicenters of historic earthquakes with magnitudes of 5.5 or more from 1800 to 2000. Records of historic ground effects related to seismic activity (e.g. liquefaction, sand boils, lateral spreading, ground cracking) compiled by Knudsen et al. (2000), indicate that ground effects related to historic seismic activity have not been reported for the site vicinity.

7.1.2 Ground Surface Fault Rupture

The site is not located within an Earthquake Fault Zone established by the state geologist (CGS, 1982) (formerly known as Alquist-Priolo Special Studies Zones) to delineate regions of potential ground surface rupture adjacent to active faults. As defined by the CGS, active faults are faults that have caused surface displacement within Holocene time, or within approximately the last 11,700 years (CGS, 2018). The closest fault rupture hazard zone is associated with the Greenville Fault, which is located within approximately 4½ miles of the site to the northeast (CGS, 1982a).

Regional geologic maps by Crane (1995), Dibblee and Minch (2006), Graymer et al. (1996), and Jennings and Bryant (2010) depict a fault within approximately 350 feet of the project site to the northwest (Figure 4). Jennings & Bryant (2010) refer to this fault, perpendicular and north of the Livermore fault, as a Quaternary fault with evidence of displacement in the last 1.6 million years. Graymer et al. (1996), Crane (1995), and Majmunder (1991) interpret the fault as a thrust feature, with the hanging wall to the north of the fault trace, while Dibblee & Minch (2006) indicate that the north side of the fault is moving up relative to the south side. Detailed mapping by Dibblee & Minch (2006) and Majmunder (1991) indicates that the fault is exposed in the Pliocene to Pleistocene Livermore Gravels, but is generally concealed by Holocene alluvium. Based upon the information presented above, this fault would not be considered active for purposes of potential surface fault rupture with low probability of damage to structures due to surface rupture.

Additionally, in 2007, Ninyo & Moore performed a subsurface fault trenching study to evaluate if northwest-trending lineaments observed in aerial photographs as projecting onto the Las Positas Community College Campus were related to faulting. No evidence of faulting was found within the approximately 660-foot-long trench excavation performed across these lineaments (Ninyo & Moore, 2007).

Based on our review of the referenced geologic maps and the results of our previous fault trenching study, known active faults have not been mapped on the site and the site is not located within a fault-rupture hazard zone. Therefore, the probability of damage from surface fault rupture is considered to be low.

7.1.3 Seismic Ground Motion and Site Classification

Considering the proximity of the site to historic and Holocene active faults (Figure 3), the potential for future strong seismic ground shaking at the site is significant. Seismic design criteria to address ground shaking are provided in Section 9.1. The peak ground acceleration (PGA) associated with the Maximum Considered Earthquake Geometric Mean (MCE_G) was calculated in accordance with the American Society of Civil Engineers (ASCE) 7-16 Standard and the 2022 California Building Code (CBC). The MCE_G peak ground acceleration with adjustment for site class effects (PGA_M) was calculated as 0.773g using the seismic design tool developed by the Structural Engineers Association of California in conjunction with the Office of Statewide Health Planning and Development (SEAOC & OSHPD, 2023). The calculated PGA_M is based a mapped MCE_G peak ground acceleration of 0.703g for the site and a site coefficient (F_{PGA}) of 1.1 for Site Class D. Site Class D was selected based on results from our refraction microtremor (ReMi) survey at the site (Appendix F) that computed (Appendix G) an interval-weighted, harmonic-mean shear wave velocity 1,027 feet per second to a depth of 100 feet (V_{s100}) that is approximately equivalent to a Vs30 of 313 meters per second.

7.1.4 Liquefaction and Cyclic Softening

The strong vibratory motions generated by earthquakes can trigger a rapid loss of shear strength in saturated, loose, granular soils of low plasticity (liquefaction) or in wet, sensitive, cohesive soil (cyclic softening). Liquefaction and strain softening can result in a loss of foundation bearing capacity or lateral spreading of sloping or unconfined ground. Liquefaction can also cause settlement of buildings on shallow foundation and generate sand boils leading to subsidence at the ground surface. Liquefaction (or cyclic softening) is generally not a concern at depths more than 50 feet below ground surface.

The site is not located within a seismic hazard zone for liquefaction (Figure 8) as mapped by the CGS (CGS, 2008). Regional groundwater records and the findings from our subsurface exploration indicate that the static groundwater level at the site is more than 30 feet below the ground surface and the alluvium encountered at that depth generally consists of hard clay with occasional layers of very dense sand. Based on these findings, we do not regard liquefaction, cyclic softening, or related hazards including lateral spreading or sand-boil induced ground subsidence as design considerations.

7.1.5 Dynamic Settlement

The strong vibratory motion associated with earthquakes can also dynamically compact loose granular soil leading to surficial settlements. Dynamic settlement is not limited to the near-surface environment and may occur in both dry and saturated sand and silt. Cohesive soils are not typically susceptible to dynamic settlement.

Based on the generally stiff to hard consistency and cohesive nature of the on-site materials, we do not regard dynamic settlement as a design consideration.

7.1.6 Tsunamis and Seiches

Tsunamis are long wavelength seismic sea waves (long compared to ocean depth) generated by the sudden movements of the ocean floor during submarine earthquakes, landslides, or volcanic activity. The project location is not within a tsunami hazard area as shown on the

Tsunami Hazard Area Map (State of California, 2021). Seiches are waves generated in a large enclosed body of water. Based on the mapped distance of the site to San Francisco Bay, and the lack of nearby large enclosed bodies of water, the potential for damage due to tsunamis or seiches is not a design consideration.

7.1.7 Seismic Slope Stability

The site is not located within a hazard zone for earthquake-induced landslides on the Seismic Hazard Zones Map (Figure 8) prepared by the CGS (2008). As such, we do not regard seismic slope stability as a design consideration. Slope stability and landsliding are further addressed in Section 7.3.

7.2 Flood Hazards

Our review of Federal Emergency Management Agency (FEMA) Flood Insurance Rate Maps (FEMA, 2009) indicates that the project site is outside the 500-year flood zone in Zone X, an area of minimal flood hazard. As such, the potential for flooding at the site is low.

7.3 Landsliding and Slope Stability

Existing unsupported slopes at the site for the proposed improvements are not steeper than 2:1 (horizontal to vertical) and existing slopes steeper than 5:1 (horizontal to vertical) are not more than 10 feet high. We did not observe indications of unstable conditions on the existing slopes during the site reconnaissance. It is assumed that new slopes associated with the proposed construction will be consistent with the existing slopes. As such, we do not regard landsliding or slope stability as a design consideration for the project.

7.4 Unsuitable Materials

Fill materials that were not placed and compacted in lifts with geotechnical observation and testing, or fill materials lacking documentation of such observation and testing, are considered non-engineered or undocumented fill. Non-engineered or undocumented fill is generally unsuitable as a bearing material below foundations and new fills due to the potential for differential settlement resulting from variable support characteristics or the potential inclusion of deleterious materials. Undocumented fill may be present as backfill at utility trenches and around foundations for the existing structures at the site that will be demolished to make room for the proposed improvements. Recommendations for remedial grading to mitigate the unsuitable support characteristics of undocumented fill are presented in Section 9.

Soil containing roots or other organic matter are not suitable as fill or subgrade material below structures, walls, pavements, flatwork, or engineered fill. Surficial soil containing roots or other organic matter should be removed as part of the clearing and grubbing operations.

7.5 Collapsible Soil

Loose, dry, low-density soil can "collapse" or compact with the addition of water under foundation loads or the weight of overlying soil. Ground settlement occurs when the collapsible soil is first saturated or is saturated to depths greater than those achieved by typical rain events. Non-engineered or undocumented fill, young alluvial fans, debris flow sediments, and deposits of wind-blown soil may include collapsible soils, particularly in arid or semi-arid environments. Subsurface exploration for this study encountered moist alluvium that generally consisted of firm to hard clay and medium dense to very dense clayey sand, with occasional layers of medium dense well-graded and clayey gravel. Ninyo & Moore inundated a sample of relatively undisturbed alluvium under a vertical stress as part of a consolidation test. The consolidation of the test specimen upon inundation without an increase in the vertical stress was negligible, as illustrated on the test results in Appendix C, indicating that that collapsible soil is not a consideration for the site provided that remedial grading is performed in accordance with the recommendations in this report to remove and replace undocumented fill (where encountered) below finish pad subgrade with engineered fill.

7.6 Consolidation and Static Settlement

Compression or consolidation of loose or soft soil due to new overburden fill, structural loads, or increased effective stresses from dewatering can result in ground settlement. The subsurface conditions encountered in our borings, at 3 feet or more below the ground surface, generally consisted of stiff to hard clay with occasional layers of medium to very dense clayey sand. Layers of soft clay and loose sand and gravel were encountered in the upper three feet of the borings. Recommendations for remedial grading are provided to mitigate loose, soft, or otherwise unsuitable material below shallow foundations. Based on the proposed project and observed subsurface conditions presuming that remedial grading is performed in accordance with the recommendations in this report, it is estimated that the static settlement due to consolidation or compression will be approximately 1½ inches for sustained wall loads of up to 10 kips per foot on strip footings and for sustained column loads of up to 100 kips on spread footings. The corresponding differential static settlement is estimated to be approximately ¾ inches over a lateral distance of about 30 feet. Based on the proposed project and regional records of historic groundwater levels, settlement from dewatering is not expected as a result of the proposed project.

The estimated differential settlement conforms with the differential settlement threshold criteria in Table 12.13-3 of ASCE 7-16 for multi-story structures of Risk Category I or II. Recommendations for footings are provided in Section 9.

7.7 Naturally Occurring Asbestos

Based on guidelines developed by the California Department of Toxic Substances and Control (2004 and 2005), a Preliminary Environmental Assessment (PEA) is recommended for school sites that are located within a 10-mile radius of a rock formation that may contain naturally occurring asbestos (NOA). Natural occurrences of asbestos are more likely to be encountered in, and immediately adjacent to, outcrops of ultramafic rocks. Ultramafic rock was not encountered during our subsurface exploration. Regional mapping by Churchill & Hill (2000) and by Graymer et al. (1996) indicates that outcrops of ultramafic rocks have, in general, not been mapped within a 10-mile radius except for an outcrop of serpentinite, which commonly contains NOA, that is mapped about 8¼ miles south of the site near the San Antonio Reservoir. Arroyo Las Positas, which drains to the west, is between this serpentinite outcrop and the site. Based on these conditions, it is unlikely that the natural soil at the site contains significant concentrations of NOA.

7.8 Corrosive and Deleterious Soil

Laboratory testing was performed to evaluate the corrosivity of on-site soil. A soil sample collected during the subsurface exploration was submitted to CERCO Analytical of Concord, California to perform laboratory testing and evaluate the corrosivity of the samples on the basis of tests to quantify pH, redox potential, electrical resistivity, chloride content, and soluble sulfate content. The results of the testing and the findings from the corrosivity evaluation are presented in Appendix D.

California Department of Transportation (Caltrans) defines a corrosive environment for structures as an area where the soil has a chloride concentration of 500 parts per million (ppm) or greater, soluble sulfate concentration of 0.15 percent (1,500 ppm) or greater, or a pH of 5.5 or less (Caltrans, 2021). The criteria used to evaluate the deleterious nature of soil on concrete are listed in Table 2. Based on these criteria and the results of the testing, the near-surface soil at the site does not meet the definition of a corrosive environment for structures, and the sulfate exposure to concrete is negligible with an exposure classification for sulfate of S0. As noted in Appendix D, the test results indicate that the site soil is corrosive to ferrous metals based on the resistivity test results and moderately corrosive based on the redox potential. Buried iron, steel, cast iron, ductile iron, galvanized steel, and dielectric coated steel should be appropriately protected against corrosion depending on the importance or expected service life of the element. A corrosion engineer should be consulted to provide recommendations to mitigate corrosion. Recommendations to mitigate the impact of corrosive soil on concrete structures are presented in Section 9.6.

Table 2 – Criteria for Deleterious Soil on Concrete

Sulfate Content Percent by Weight	Sulfate Exposure	Exposure Class
0.0 to 0.1	Negligible	S0
0.1 to 0.2	Moderate	S1
0.2 to 2.0	Severe	S2
> 2.0	Very Severe	S3

Reference: American Concrete Institute (ACI) Committee 318 Table 19.3.1.1 (ACI, 2016)

7.9 Expansive Soil

Some clay minerals undergo volume changes upon wetting or drying. Unsaturated soils containing those minerals will shrink/swell with the removal/addition of water. The heaving pressures and differential movement associated with expansion and changes in soil moisture can damage structures and flatwork. Laboratory testing was performed on selected samples to evaluate the expansion index of the near-surface soil in accordance with the American Society of Testing and Materials (ASTM) Standard D 4829. The results of the testing, presented in Appendix C, indicates that the Expansion Index of the samples tested ranges from 43 to 113, which is consistent with a low to high expansion characteristic. To reduce the potential for heave and differential movement due to shrink/swell behavior, recommendations are provided as part of the remedial grading and pad preparation to create a layer with low expansion characteristics understructures using imported fill. Site soil may be chemically treated with quicklime to reduce the expansion characteristic of the soil as an alternative to importing select fill.

7.10 Excavation Characteristics

We anticipate that the project will involve excavations of up to a few feet for foundations, remedial grading, and utility installation with deeper excavations of up to approximately 20 feet for drilled pier foundations to support the shade canopy at the amphitheater stage. The geologic units encountered over this interval during the subsurface exploration included fill, topsoil, and alluvium that generally consisted of moist to wet, soft to hard clay, and loose to medium dense clayey sand, clayey gravel, and gravel. We anticipate that conventional earthmoving and foundation drilling equipment in good working condition should be able to make the proposed excavations. Excavations in fill, where present, may encounter obstructions consisting of debris, rubble, abandoned structures, or over-sized materials that may require special handling or demolition equipment for removal.

Near-vertical temporary cuts in the near surface deposits up to 4 feet in depth should remain stable for a limited period of time. However, sloughing of the materials exposed on the excavation sidewall may occur, particularly if the excavation encounters seepage or granular soil, is exposed to water, or if the sidewall is disturbed during construction operations. Excavation subgrade may become unstable if exposed to wet conditions. Excavations for drilled piers may encounter saturated granular soils. Recommendations for excavation stabilization are provided in Section 9. Excavated materials may also be wet and need to be dried out before reuse as fill.

8 CONCLUSIONS

Based on our review of the referenced background data, our site field reconnaissance, subsurface evaluation, and laboratory testing, it is our opinion that there are no known geotechnical constraints that would preclude construction of the project provided that the recommendations presented in this report are incorporated into the design and construction of the proposed improvements. Key findings from our geotechnical evaluation and subsurface exploration include the following:

- The subsurface exploration for this study encountered fill, topsoil, alluvium, and asphalt concrete pavement. The fill, as encountered, consisted of gray, stiff clay. The topsoil, as encountered, generally consisted of dark brown to black, soft to very stiff lean to fat clay, and loose gravel with visible organic matter in Boring B-1. The alluvium, as encountered, generally consisted of light brown to brown, firm to hard, sandy clay and lean to fat clay, with occasional layers of medium to very dense sand and clayey sand, and loose to medium dense gravel and clayey gravel. Scattered veins of caliche cementation were encountered in the alluvium.

- Groundwater was encountered in Boring B-5 at a depth of approximately 46½ feet below the ground surface. Variations in the groundwater level across the site and over time should be anticipated as discussed in Section 6.4. The historic high groundwater level at the site based on regional mapping (CGS, 2008a) is more than 30 feet below the ground surface.

- Based on historic activity, the potential for future seismic ground motion at the site is considered significant. Seismic parameters for use in structural design are provided. Designing to the requirements of the current California Building Code will address the impacts of shaking on the structure but will not preclude damage or complete loss of the building. Code design is based on life-safety.

- The project site is outside the 500-year flood zone.

- Landslides, tsunamis, seiches, and ground surface rupture due to faulting are not design considerations based on the location of the project.

- Dynamic settlement, seismic strain softening, liquefaction and related hazards are not design considerations based on the subsurface conditions encountered.

- The proposed location for the project improvements is currently occupied by existing buildings that will be demolished. Undocumented fill may be present as backfill at utility trenches and around foundations for the existing structures. Layers of loose sand and gravel, soft clay, and topsoil with visible organic matter were encountered in the shallow soil during the subsurface exploration. Recommendations for site preparation and remedial pad grading are provided below to mitigate loose, soft, or otherwise unsuitable material below shallow foundations and concerns related to undocumented and potentially collapsible fill for the proposed construction.

- The total estimated static settlement, presuming that remedial grading is performed in accordance with the recommendations in this report, is approximately 1½ inches for sustained wall and column loads of up to 10 kips per foot and 100 kips, respectively, with a differential static settlement of approximately ¾ inches over a lateral distance of about 30 feet. Recommendations for footings are provided.

- The results of our laboratory testing indicate that the subsurface materials have a low to high expansion characteristic. Recommendations to create a layer with low expansion characteristics on the building pads by using imported select fill or chemically treating site soil with quicklime, are provided in Section 9.3.8 .

- It is unlikely that significant concentrations of naturally occurring asbestos will be encountered at the site based on the site location and the subsurface conditions encountered.

- Laboratory testing of soil samples collected during the subsurface exploration for this study indicates that the site does not meet the definition of a corrosive environment for structures (Caltrans, 2021) and the sulfate exposure to concrete is negligible (Class S0). Based on electrical resistivity testing, the samples tested are considered corrosive to ferrous metals, as noted in Appendix D. A corrosion engineer should be consulted to provide specific guidance on protective measures to mitigate corrosion.

- Excavations that remain unsupported and are exposed to water or encounter granular soil may be unstable and prone to sloughing. Recommendations for excavation stabilization are provided.

- Infiltration testing performed for this study indicates that the infiltration rate of the near-surface soil is relatively slow.

9 RECOMMENDATIONS

The following sections present our geotechnical recommendations for the design and construction of the proposed improvements. The project improvements and earthwork should be designed and constructed in accordance with these recommendations, applicable codes, relevant grading ordinances, and appropriate construction practices. The geotechnical consultant should observe and evaluate excavations for earthwork and foundation constructions. Evaluations performed by the geotechnical consultant during the course of construction may result in new recommendations, which could supersede the recommendations provided in this section.

9.1 Seismic Design Criteria

Table 3 presents the Risk-Targeted, Maximum Considered Earthquake (MCE_R) spectral response accelerations consistent with the 2022 California Building Code (CBC) and corresponding site-adjusted and design level spectral response accelerations based on the USGS seismic design maps (SEAOC/OSHPD, 2023). Seismic Site Class D was selected based on the results of the ReMi Survey (Appendix F) for structures with a fundamental period of ½ second or less such that the exception to Site Class F in Section 20.3.1-1 of ASCE Standard 7-16 is applicable.

Table 3 – California Building Code Seismic Design Criteria	
Seismic Design Parameter Evaluated for 37.7110ºNorth latitude, 121.8017ºWest longitude	**Value**
Site Class	D
Site Coefficient, Fa	1.0
Site Coefficient, Fv	1.7
Mapped Spectral Acceleration at 0.2-second Period, S_s	1.727 g
Mapped Spectral Acceleration at 1.0-second Period, S_1	0.600 g
Spectral Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	1.727 g
Spectral Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	1.530 g*
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	1.151 g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	1.020 g*
Seismic Design Category for Risk Category I, II, or III	D

Note: *S_{M1} and S_{D1} parameters include 50 percent increase per ASCE 7-16 Section 11.4.8 Item 1 Exception

The values provided in the table may be used may be used for seismic design of structures with a fundamental period of ½ second or less in accordance with the exception to item 1 in Section 11.4.8 of ASCE 7-16 Supplement 3.

9.2 Foundations

Foundations should be designed in accordance with structural considerations and the following recommendations. In addition, requirements of the appropriate governing jurisdictions, practices of the Structural Engineers Association of California, and applicable building codes should be considered in the design of the structures. The proposed STEAM building may be supported on spread and perimeter footings provided that remedial grading is performed in accordance with the recommendations in Section 9.3.8 to mitigate undocumented fill, expansive, and loose surficial soil. The proposed shade canopy at the amphitheater stage may be supported on drilled piers to provide additional overturning and uplift resistance. The foundation design parameters provided in the following sections are not intended to preclude differential movement of foundations. Minor cracking may occur.

9.2.1 Footings

Footings for the proposed buildings may be designed using the criteria in Table 4. The geotechnical engineer should observe the footing excavations to evaluate bearing materials and subgrade condition.

Table 4 – Recommended Design Parameters for Footings

Footing	Sustained Loads	Footing Width	Bearing Depth[1]	Allowable Bearing Capacity[2]	Static Settlement[3]
Wall Footing	10 kips/foot or less	18 inches or more	2 feet or more	2,000 psf	1½ inches total¾ inches differential
Column Footing	100 kips or less	24 inches or more	2 feet or more	2,000 psf	1½ inches total¾ inches differential over 30 feet

[1] Below the adjacent finish grade.
[2] Net allowable bearing capacity in pounds per square foot with Safety Factor of 3 or more. Allowable bearing capacity may be increased by one-third for wind or seismic alternative basic load combinations.

Structures supported on spread or perimeter footings consistent with these recommendations should be designed for the total and differential settlements listed in Table 4 for sustained loads. Footing settlement due to static loads may be further evaluated using a modulus of subgrade reaction. Recommended values for the modulus of subgrade reaction in pounds per cubic inch (pci) are provided in Table 5. The designer may interpolate between the values in the table for intermediate footing widths.

Table 5 – Footing Modulus of Subgrade Reaction

Footing	Footing Width					
	1.5 feet	2 feet	3 feet	5 feet	7 feet	10 feet
Wall Footing	20 pci	17 pci	14 pci	11 pci	10 pci	--
Column Footing	--	19 pci	16 pci	13 pci	11 pci	9 pci

The footings should be reinforced with deformed steel bars as detailed by the project structural engineer. Where footings are located adjacent to utility trenches or other excavations, the footing bearing surfaces should bear below an imaginary plane extending upward from the bottom edge of the adjacent trench/excavation at a 2:1 (horizontal to vertical) angle above the bottom edge of the footing. Footing bottoms should not be sloped more than 1 unit vertical to 10 units horizontal. Wall footings may be stepped provided that the bearing grade differential between adjacent steps does not exceed 18 inches and the slope of a series of such steps does not exceed 1 unit vertical to 2 units horizontal.

A lateral bearing pressure of 300 psf per foot of depth may be used to evaluate the resistance of footings to lateral loads. The recommended lateral bearing pressure is for level and gently sloping ground conditions where the ground slope adjacent to the foundation is 5 percent or less. The lateral bearing pressure should be neglected to a depth of 12 inches where the ground adjacent to the foundation is not covered by a slab or pavement. The lateral bearing pressure may be increased by one-third when considering wind or seismic alternative basic load combinations. A friction coefficient of 0.35 may be assumed for evaluating frictional resistance to lateral loads. The weight of the material above a plane rising up and away from the bottom edges of the footings at 20 degrees off plumb may be considered, along with the weight of the footing and the material over the footing, when evaluating footing resistance to uplift. A unit weight of 120 pounds per cubic foot (pcf) for soil or aggregate and 150 pcf for normal weight concrete may be assumed for this evaluation.

9.2.2 Drilled Piers

Drilled piers for the proposed shade canopy structure embedded up to 20 feet below grade may be designed for an allowable side friction of 60 pounds per square foot per foot depth of embedment to evaluate resistance to downward axial loads and 40 psf per foot depth for upward axial loads. The recommended values for allowable skin friction include a factor of safety of 2 for downward loading and 3 for upward loading. The allowable side friction may be increased by one-third for alternative basic load combinations with loads of short duration such as wind or seismic loads. The spacing between adjacent piers should be equivalent to three pier diameters or more to mitigate reduction in axial resistance due to group effects. Structures supported on shallow pier foundations should be designed for a total and differential settlement due to sustained loads of approximately ¼ inch over a horizontal distance of 30 feet.

A lateral bearing pressure of 100 pounds per square foot (psf) per foot depth up to 1,500 psf may be used to evaluate resistance to lateral loads and overturning moments in accordance with Section 1807 of the California Building Code with a one-third increase for alternative basic load combinations with wind or seismic loads. The allowable lateral bearing pressure may be increased by a factor of two for structures that can accommodate ½ inch of lateral deflection of the top of the pier foundation.

The spacing between adjacent piers should be equivalent to three pier diameters or more to avoid a reduction in lateral load resistance due to group effects for piers in a row perpendicular to the direction of lateral loading. For piers in a row parallel to the direction of lateral loading, the contribution of trailing piers to the lateral load resistance of the group should be neglected where the center to center spacing is less than eight pier diameters.

Drilled pier excavations should be cleaned of loose material prior to pouring concrete. Drilled pier excavations that encounter groundwater or cohesionless soil may be unstable and may need to be stabilized by temporary casing or use of drilling mud. Standing water should be removed from the pier excavation or the concrete should be delivered to the bottom of the excavation, below the water surface, by tremie pipe. Casing should be removed from the excavation as the concrete is placed. Concrete should be placed in the piers in a manner that reduces the potential for segregation of the components.

9.2.3 Slabs-on-Grade

Slab-on-grade floors should be designed by the project structural engineer based on the anticipated loading conditions. A vapor retarder is recommended in areas where moisture-sensitive floor coverings or conditioned environments are anticipated. Where a vapor retarding system is not used, slabs should be constructed on 4 inches of compacted aggregate base. Slab-on-grade floors subject to vehicular loading consisting of passenger cars and trucks should be no less than 6 inches thick on 6 inches of compacted aggregate base. The slab should be reinforced with deformed steel bars with a nominal diameter of ⅜-inch or more as designed by the project structural engineer. Masonry briquettes or plastic chairs should be used to maintain the position of slab reinforcement, during concrete placement, in the upper half of the slab with appropriate concrete cover over the reinforcing steel. Refer to Section 9.6 for the recommended concrete cover over reinforcing steel. Joints consistent with the guidelines of ACI Committee 302 may be constructed at periodic intervals to reduce the potential for random cracking of the slab.

9.2.4 Moisture Vapor Retarder

The migration of moisture through slabs underlying enclosed spaces or overlain by moisture sensitive floor coverings should be discouraged by providing a moisture vapor retarding system between the subgrade soil and the bottom of slabs. We recommend that the moisture vapor retarding system consist of a 4-inch-thick capillary break, overlain by a 15-mil-thick plastic membrane. Sand should not be placed over the vapor retarder. The capillary break should be constructed of clean, compacted, open-graded crushed rock or angular gravel of ¾-inch nominal size. The crushed rock or angular gravel should be compacted with a vibratory plate compactor or roller to reduce the potential for damage to the vapor retarder by rock puncture during placement of reinforcement and concrete. The plastic membrane should conform to the requirements in the latest version of ASTM Standard E 1745 for a Class A membrane. To reduce the potential for slab curling and cracking, an appropriate concrete mix with low shrinkage characteristics and a low water-to-cementitious-materials ratio should be specified. In addition, the concrete should be delivered and placed in accordance with ASTM C94 with attention to concrete temperature and elapsed time from batching to placement, and the slab should be cured in accordance with the guidelines of ACI Committee 302.

Where the exterior grade is at a higher elevation than the moisture vapor retarding system (including the capillary break layer), consideration should be given to constructing a subdrain around the foundation perimeter. The subdrain should consist of ¾-inch crushed rock wrapped in filter fabric (Mirafi 140N, or equivalent). The subdrain should be capped by a pavement or 12 inches of native soil and drained by a perforated pipe (Schedule 40 polyvinyl chloride pipe, or similar). The pipe should be sloped at 1 percent or more to discharge at an appropriate outlet away from the foundation. The pipe should be located below the bottom elevation of the moisture vapor retarding system but above a plane extending down and away from the bottom edge of the foundation at a 2:1 (horizontal to vertical) gradient. A sump may be used where gravity drainage is not feasible.

9.3 Earthwork

Earthwork associated with the proposed project is expected to include demolition, site clearing, remedial grading to remove undocumented fill, unsuitable material, and highly expansive soil, placement of fill to achieve the proposed pad grades, excavation and placement of trench backfill for new utilities, preparation of subgrade for flatwork, and finish grading to establish site drainage. Earthwork should be performed in accordance with the requirements of applicable governing agencies and the recommendations presented below.

9.3.1 Pre-Construction Conference

Holding a pre-construction conference to discuss earthwork is recommended. Representatives of the owner and the governing agency should be in attendance with the civil engineer, the geotechnical engineer, and the contractor to discuss the work plan and project schedule, site safety considerations, regulatory agency requirements, and the geotechnical recommendations for earthwork, construction observation, and testing.

9.3.2 Site Preparation

Site preparation should begin with the demolition of the designated existing improvements and removal of vegetation, utility lines, surface obstructions (e.g., pavements, aggregate base, curb/gutter, foundations), rubble and debris, and other deleterious materials from areas to be graded. Vegetation should be removed to such a depth that organic material is generally not present. Clearing and grubbing should extend to the outside of the proposed excavation and fill areas. Rubble and excavated materials that do not meet criteria for use as fill should be removed from the site for disposal in an appropriate landfill. Soil containing roots or other organic matter may be stockpiled for later use as landscaping fill, as authorized by the owner's representative. Active utilities within the project limits, if any, should be re-routed or protected from damage by construction activities. Existing utilities to be abandoned should be removed, crushed in place, or backfilled with grout. Excavations resulting from removal of buried utilities, tree stumps, or obstructions should be backfilled with compacted fill in accordance with the recommendations in the following sections.

9.3.3 Excavation Stabilization

Excavations should be stabilized by benching or laying slopes back in accordance with the Occupational Safety and Health Administration (OSHA) Excavation Rules and Regulations. Alternatively, an internally-braced shoring system or trench shield conforming to the OSHA Excavation Rules and Regulations may be used to stabilize excavation sidewalls. Recommended OSHA material type classifications for site soil are provided in Table 6 along the corresponding allowable temporary slope inclinations and lateral earth pressures for design or selection of excavation shoring. The recommendations listed in this table are based upon the limited subsurface data provided by our subsurface exploration and reflect the influence of the environmental conditions that existed at the time of the exploration. The recommendations also presume that the excavations will not extend below a plane extending down and away from the foundation bearing surfaces of existing adjacent structures at an angle of 2:1 (horizontal to vertical). Dewatering should be performed as-needed to depress groundwater levels below the bottom of excavations.

Table 6 – OSHA Material Classifications and Allowable Slopes

Material	OSHA Classification	Allowable Temporary Slope	Lateral Earth Pressure on Shoring (psf)
Fill, Topsoil, & Alluvium (above groundwater)	Type C	1½ h:1v (34°)	80×D + 72

Note: 'D' is depth of excavation in feet

Excavation stability, material classifications, allowable slopes, and shoring pressures should be re-evaluated and revised, as-needed, during construction. Excavations, shoring systems and the surrounding areas should be evaluated daily by a competent person for indications of possible instability or collapse. The shoring system should be designed or selected by a suitably qualified individual or specialty subcontractor. The contractor should take appropriate measures to protect workers. OSHA requirements pertaining to worker safety should be observed. The shoring parameters presented in this report are preliminary design criteria, and the designer should evaluate the adequacy of these parameters and make appropriate modifications for their design.

Excavations made in close proximity to existing structures may undermine the foundation of those structures and/or cause soil movement related distress to the existing structures. Stabilization techniques for excavations in close proximity to existing structures will need to account for the additional loads imposed on the shoring system and appropriate setback distances for temporary slopes. The geotechnical engineer should be consulted for additional recommendations if the proposed excavations cross below a plane extending down and away from the foundation bearing surfaces of the adjacent structure at an angle of 2:1 (horizontal to vertical).

Excavation bottoms or subgrade, if exposed to wet conditions, may be subject to pumping under load. The contractor should be prepared to stabilize subgrades after exposure to water. In general, unstable subgrade conditions may be mitigated by scarification and aeration to dry the soil to the optimum moisture content or treating the soil with quicklime. Alternatively, unstable subgrade may be removed and replaced with engineered fill. Construction of a bridging layer consisting of crushed rock or granular fill with geotextile may be needed to support the engineered fill layer so that the specified compaction can be achieved. Appropriate mitigation measures will be influenced by the conditions encountered. The geotechnical consultant should be consulted for recommendations to stabilize the site as-needed.

9.3.4 Construction Dewatering

Groundwater was encountered in the Boring B-5 at a depth of approximately 46½ feet below the ground surface. Variations in groundwater levels across the site and over time should be anticipated. Water intrusion into the excavations may occur as a result of groundwater intrusion or surface runoff. The contractor should be prepared to take appropriate dewatering measures in the event that water intrudes into the excavations. Sump pits, trenches, or similar measures should be used to depress the water level below the bottom of the excavation.

Considerations for construction dewatering should include anticipated drawdown, volume of pumping, potential for settlement, and groundwater discharge. Disposal of groundwater should be performed in accordance with the guidelines of the Regional Water Quality Control Board.

9.3.5 Subgrade Observation

Prior to placement of fill, erection of forms or placement of reinforcement for foundations, the project geotechnical engineer should be notified to evaluate the exposed subgrade. Materials that are considered unsuitable shall be excavated under the observation of the geotechnical engineer in accordance with the recommendations in this section or the field recommendations of the geotechnical engineer.

Unsuitable materials include, but may not be limited to dry, loose, soft, wet, expansive, organic, or compressible natural soil; and undocumented or otherwise deleterious fill materials. Unless otherwise noted, unsuitable materials should be removed to a depth at which suitable foundation subgrade, as evaluated in the field by the geotechnical engineer, is exposed.

9.3.6 Fill Material

In general, fill should not consist of pea gravel and should be free of rocks or lumps in excess of 3-inches in median dimension, hazardous materials, trash, debris, and vegetation or other deleterious material. The on-site soil is generally suitable, from a geotechnical perspective, for reuse as general fill, including use as engineered fill for the building pad, provided that it is processed as-needed to meet the preceding criteria, and moisture conditioned to achieve a moisture content approximately 2 percentage points above the optimum. Moisture conditioning by adding water or by mixing and aerating to reduce the moisture content should be anticipated. Site soil, if saturated due to exposure to groundwater or rainy conditions may need an extended drying time to achieve the optimum moisture content.

In addition to the preceding criteria, imported fill should be close graded with 35 percent or more by dry weight passing the No. 4 sieve and either: an expansion index of 50 or less, a plasticity index of 12 or less, or less than 10 percent by dry weight passing the No. 200 sieve. Imported fill should also meet the Caltrans (2021) criteria for non-corrosive soils (i.e., soils having a chloride concentration of 500 parts per million [ppm] or less, a soluble sulfate content of approximately 0.15 percent [1,500 ppm] or less, and a pH value of 5.5 or more). Material considered for use as fill should be evaluated before it is imported to the site. The contractor should be responsible for the uniformity of import material brought to the site.

9.3.7 Fill Placement and Compaction

The exposed subgrade in areas to receive fill should be scarified and moisture conditioned as needed to achieve a moisture content about 2 percentage points above the optimum, before compaction, by mechanical means, to 90 percent or more of the reference density as evaluated by ASTM D1557 on a dry density basis. Fill should then be placed and compacted in lifts by hand tampers or mechanical means to 90 percent of the ASTM D1557 reference density. The fill should be moisture conditioned to approximately 2 percentage points above the optimum before it is compacted. The allowable lift thickness is influenced by the type of compaction equipment utilized but generally should not exceed 8 inches in loose thickness. Compacted fill should be maintained in a moist condition by sprinkling water or covering with plastic. Compacted fill that has dried out and loosened or developed desiccation cracking should be scarified, moisture-conditioned, and recompacted as per the requirements above before additional fill is placed.

9.3.8 Remedial Grading and Pad Preparation

To mitigate loose, soft, or otherwise unsuitable material below shallow foundations and concerns related to undocumented and potentially collapsible fill for the proposed construction, remedial grading should be performed to prepare the building pads. The remedial grading should consist of overexcavating the pad and backfilling the pad with engineered fill. Based on the subsurface conditions encountered in the borings, the anticipated depth of removal is 3 feet below the existing grade at the proposed building footprint. Ninyo & Moore should inspect the remedial excavations and evaluate if additional excavation is needed. The lateral limits of the remedial excavation should extend 3 feet beyond the building footprint. The anticipated extent of the overexcavation should be detailed on the construction plans to reduce the potential that these recommendations are overlooked during construction bidding.

After remedial excavation, the exposed subgrade should be scarified and moisture conditioned as needed to achieve a moisture content about 2 percentage points above the optimum, before compaction, by mechanical means, to 90 percent or more of the reference density as evaluated by ASTM D1557 on a dry density basis. The excavations may then be backfilled with general fill or imported fill conforming with the recommendations in Section 9.3.6, that is placed and compacted in lifts per the recommendations in Section 9.3.7. Material removed from the remedial excavations may be reused as general fill to backfill the excavations provided that the material is processed, as needed, to remove vegetation, debris, deleterious material, and rocks or clods that cannot pass the 3-inch sieve.

Laboratory testing for this evaluation indicated that some of the near-surface site soil is highly expansive. To mitigate the potential for swell or heave due to expansive soil, the building should be constructed over a pad of fill with low expansion characteristics using imported fill conforming with the recommendations in Section 9.3.6. The pad of low expansion fill should extend to 3 feet below the nominal bottom of slab and three feet outside the building footprint. A crushed rock capillary break layer or aggregate base layer under the slab may be considered as part of the layer with low expansion characteristics. As an alternative to importing select fill, site soil may be chemically treated as per the recommendations in Section 9.3.9 to reduce the expansion characteristic of the soil. Chemically treated site soil used as low expansion fill that extends outside the building footing into landscaping areas should be removed to a depth of 2 feet and replaced with suitable landscaping fill.

The anticipated extent of the overexcavation and the zone with low expansion fill characteristics should be detailed on the construction plans to reduce the potential that these recommendations are overlooked during construction bidding.

Finish subgrade should be maintained in a moist condition by sprinkling water or covering with plastic. Finish subgrade that has dried out and loosened or developed desiccation cracking should be scarified, moisture-conditioned, and recompacted before it is covered.

9.3.9 Chemical Treatment

The on-site soil may be chemically treated with quicklime to reduce the expansion characteristic of the soil as an alternative to importing select fill. The quicklime should conform with ASTM Standard C977.

On-site materials containing roots or organic matter exceeding 3 percent of the soil by dry weight are not suitable for chemical treatment and should be stripped from the area where the treatment is to be performed. The chemical treatment should be performed by an experienced contractor that specializes in the chemical treatment of soil. The chemical agent should be proportioned and spread with a mechanical spreader and mixed into the soil on a mixing table or in place to produce consistent distribution of the agent within the treated layer. The depth of mixing should not exceed 18 inches per lift or the capacity of the mixer if less. Precautions to reduce the potential for dusting of the agent, such as scheduling or suspending operations to avoid windy weather, should be taken. Casting or tailgating of the chemical agent should not be permitted. The mixer should be equipped with a rotary cutting/mixing assembly, grade checker, and an automatic water distribution system. Mixing or spreading operations should not be performed during inclement weather or when the ambient temperature is less than 35 degrees Fahrenheit or during foggy or rainy weather. Adjacent passes of the mixer should overlap by 4 inches or more.

To reduce the expansion characteristic of the soil, quicklime should be mixed into the soil at a rate of 5 percent or more by dry weight of soil. Mixing and pulverizing should continue until the treated soil does not contain untreated soil clods larger than 1 inch and the quantity of untreated soil clods retained on the No. 4 sieve is less than 40 percent of the dry soil mass. Water should be added as-needed during the mixing process to achieve a moisture content above the optimum, as evaluated by ASTM D1557, for the lime-soil mixture. The lime-soil mixture should be re-mixed following a 16-hour mellowing period after the initial mixing. The lime-soil mixture should be compacted within 3 days after initial mixing to achieve 95 percent of the reference density as evaluated by ASTM D1557 on a wet density basis.

9.3.10 Utility Trenches

Trenches constructed for the installation of underground utilities should be stabilized in accordance with the recommendations in Section 9.3.3. Where possible, utility trenches adjacent to exiting or proposed footings should not extend below a 2:1 (horizontal to vertical) plane projected down and away from the bottom edge of the footing. Loose or soft soil exposed at the bottom of the trench should be removed or compacted to achieve a firm condition. Conduits and pipelines should be supported on granular bedding material that extends from the springline of the pipe to 6 inches below the pipe. Granular bedding and pipe zone material should consist of aggregates ordinarily used for highway base and subbase with 100 percent by dry weight passing the 1-inch sieve, 90 to 100 percent passing the No. 4 sieve, and no more than 5 percent passing the No. 200 sieve. Controlled low strength material (CLSM) may be used as an alternative to granular bedding and pipe zone material. CLSM should consist of a mixture of water, Portland cement, fly ash, and sound aggregate that flows without segregation of aggregates and produces an unconfined compressive strength of 50 to 150 pounds per square inch (psi) with a compressive strength of 50 psi developed about 1 hour after placement.

Prior to placement of bedding material, excavation subgrade should be compacted to a firm condition as needed. Debris, organic matter, or other unsuitable material exposed at the bottom of the excavation should be removed and replaced with additional bedding. Where a firm subgrade condition cannot be achieved by compaction before placement of bedding, the unstable subgrade should be overexcavated to a depth of 6 inches and backfilled with 6 inches of ¾-inch crushed rock that is compacted into the subgrade followed by 6 inches of granular bedding material that is compacted to a firm condition. Granular bedding material placed on firm subgrade or bedding material should be shoveled under pipe haunches, as needed, and compacted to 90 percent of the ASTM D1557 reference density on a dry density basis by manual tampers or mechanical compactors.

Additional granular bedding that is moisture conditioned to approximately 2 percentage points above the optimum should be placed as pipe zone material and compacted in lifts to 90 percent of the ASTM D1557 reference density by manual tampers or mechanical compactors to 12 inches over the pipe or conduit. Trench backfill that conforms with the recommendations for general fill in Section 9.3.6 and is moisture conditioned to approximately 2 percentage points above the optimum should be placed above the pipe zone fill and compacted in lifts to 90 percent of the ASTM D1557 reference density by manual tampers or mechanical compactors. Material excavated for the trench can be reused as trench backfill provided that the material is moisture conditioned to approximately 2 percentage points above the optimum and processed, as needed, to remove vegetation, debris, deleterious material, and rocks or clods that cannot pass the 2-inch sieve. Trench backfill under pavement in the public right of way should consist of aggregate base that conforms with Section 26 of the California Standard Specifications (Caltrans, 2022) and is compacted to 95 percent of the ASTM D1557 reference density on a dry density basis after the material is moisture conditioned to approximately 2 percentage points above the optimum. The allowable lift thickness for pipe zone and trench backfill is influenced by the type of compaction equipment utilized but generally should not exceed 8 inches in loose thickness. Special care should be exercised to avoid damaging the pipe during compaction of the backfill. Densification of pipe zone and trench backfill by flooding or jetting should not be permitted. CLSM may be used as pipe zone and trench backfill. Lift thickness for CLSM should not exceed 3 feet. Measures to mitigate flotation of the pipe or conduit should be taken, as needed, where CLSM is used as bedding or pipe zone fill.

To reduce potential for moisture intrusion into the building envelope, utility trenches should be plugged at locations where the trench excavations cross under the building perimeter. The trench plug should be constructed of a compacted, fine-grained, cohesive soil that fills the cross-sectional area of the trench for a distance equivalent to the depth of the excavation. Alternatively, the plug may be constructed of concrete or CLSM.

9.3.11 Rainy Weather Considerations

Scheduling earthwork and foundation construction for the period-between approximately April 15 and October 15 to avoid the rainy season is recommended. In the event that grading is performed during the rainy season, the plans for the project should be supplemented to include a stormwater management plan prepared in accordance with the requirements of the relevant agencies having jurisdiction. The plan should include details of measures to protect the subject property and adjoining off-site properties from damage by erosion, flooding or the deposition of mud, debris, or construction-related pollutants, which may originate from the site or result from the grading operation. The protective measures should be installed by the commencement of grading, or prior to the start of the rainy season. The protective measures should be maintained in good working order unless the project drainage system is installed by that date and approval has been granted by the building official to remove the temporary devices.

In addition, construction activities performed during rainy weather may impact the stability of excavation subgrade and exposed ground. Temporary swales should be constructed to divert surface runoff away from excavations and slopes. Steep temporary slopes should be covered with plastic sheeting during significant rains.

9.4 Retaining Walls

Walls backfilled with imported select fill or lime-treated on-site soil and retaining up to 10 feet of soil above the wall footing may be designed for active or at-rest equivalent fluid earth pressures of 83 or 93 psf per foot depth for undrained conditions with level backfill. Walls with drained backfill conditions may be designed for active or at-rest equivalent fluid earth pressures of 40 or 60 psf per foot depth with level backfill. Walls that yield or deflect may be designed for active earth pressures. Wall deflection equivalent to about 1 percent of wall height may be needed to reduce at-rest earth pressures to active earth pressures. Walls that are restrained by abutting walls should be designed to resist at-rest earth pressures. An additional equivalent fluid pressure of 14 psf per foot depth may be used to evaluate seismic earth pressure on retaining walls, as appropriate, for consideration with active earth pressures. Seismic earth pressures may be neglected for walls retaining less than 6 feet of soil above the foundation.

Walls retaining level ground should be designed to resist construction or live load surcharges on the backfill. The lateral earth pressure due to a backfill surcharge of 240 psf should be a uniform horizontal surcharge of 80 psf for yielding conditions and 120 psf for at-rest conditions. An additional dead load backfill surcharge and lateral earth pressure should be considered, as applicable, for a backfill embankment, terraced wall, or adjacent structure above an imaginary plane that rises up and away from the bottom edge of the wall at a 2:1 (horizontal to vertical) gradient. The lateral earth pressure for an irregular dead load backfill surcharge is a uniform lateral earth pressure equivalent to 33 percent of the average backfill surcharge for yielding conditions and 50 percent of the average backfill surcharge for at-rest conditions. The average backfill surcharge may be evaluated as the sum of the structure surcharge above the 2:1 plane and the weight of the backfill above the 2:1 plane and above top of fill at the back of the wall divided by twice the total height of the wall measured as the top of fill at the back of the wall to the bottom of the footing or top of drilled pier foundation. A unit weight of 120 pounds per cubic foot (pcf) for soil or aggregate and 150 pcf for normal weight concrete may be assumed for this evaluation. The structure surcharge may be neglected for this calculation where the structure loads are supported on foundations that bear below the 2:1 plane.

Hydrostatic pressures may be neglected, provided that suitable drainage of the retained soil is provided. The retained soil should be drained by weep holes or a subdrain at the base of the wall stem consisting of ¾-inch crushed rock wrapped in filter fabric (Mirafi 140N, or equivalent). The subdrain should be capped by a pavement or 12 inches of native soil and drained by a perforated pipe (Schedule 40 polyvinyl chloride pipe, or similar). The pipe should be sloped at 1 percent or more to discharge at an appropriate outlet away from the wall. Alternatively, geocomposite drain panels (Miradrain 6000XL, or similar) placed against the back of the wall may be used to supplement a smaller subdrain located near the base of the wall. Measures to reduce the rate of moisture or vapor intrusion through the wall may be advisable for walls where the discoloration resulting from moisture intrusion would be undesirable. Such measures might include use of concrete with a low water-to-cementitious-materials ratio, and/or the placement of an asphalt emulsion or 10-mil thick plastic membrane to the back surface of the wall.

Lateral forces may be resisted by friction at the base of the wall footing for gravity and semi-gravity walls, and passive earth pressure acting on the embedded wall, wall footing, or wall key, if present, for semi-gravity and cantilever walls. Semi-gravity and cantilever walls on near level ground may be designed for a passive equivalent fluid lateral earth pressure of 300 psf per foot depth presuming a lateral deflection equivalent to 1 percent of the wall embedment depth to mobilize the passive condition. The passive earth pressure may be proportionally reduced for lower levels of lateral deflection as desired. Passive earth pressure should be neglected to a depth of 1 foot below the ground surface when evaluating lateral load resistance where the ground surface is not covered by pavement or flatwork. Gravity and semi-gravity walls may be designed for a coefficient of friction of 0.35 to resist lateral loads and a net allowable bearing capacity of 1,300 psf for a 12-inch footing width and 12 inches of embedment below the adjacent grade plus 100 psf per additional foot of width and 600 psf per additional foot of embedment up to 5,500 psf. The allowable bearing capacity may be increased by one-third for seismic load combinations. The coefficient of friction may be increased to 0.50 where the footing is constructed over 6 inches of aggregate base compacted to 95 percent of the reference density as evaluated by ASTM D1557.

Footing bottoms should not be sloped more than 1 unit vertical to 10 units horizontal. Wall footings may be stepped provided that the bearing grade differential between adjacent steps does not exceed 18 inches and the slope of a series of such steps does not exceed 1 unit vertical to 2 units horizontal. Walls should be designed to withstand a total static settlement of 1 inch with a differential of ½ inch over a 20-foot span.

9.5 Exterior Flatwork

Walkways and other exterior flatwork not subject to vehicular loading should be 4 inches thick (or more) over 4 inches of aggregate base. Concrete and aggregate base thickness should be increased to 6 inches or more for flatwork subject to vehicular traffic including periodic garbage trucks and emergency vehicles. Flatwork subject to impact from unloading of dumpsters should be 8 inches thick or more.

Appropriate jointing of concrete flatwork can encourage cracks to form at joints, reducing the potential for crack development between joints. Joints should be laid out in a square pattern at consistent intervals. Contraction and construction joints should be detailed and constructed in accordance with the guidelines of ACI Committee 302. The ratio of lateral spacing between contraction joints to the nominal thickness of the slab should not exceed 24 for jointed plain concrete. Contraction joints formed by premolded inserts, grooving plastic concrete, or saw-cutting at initial hardening, should extend to a depth equivalent to 25 percent of the slab thickness and 1 inch or more for thin slabs. The joint location and layout of new or reconstructed flatwork abutting existing flatwork should be consistent with joint location/layout of the existing flatwork.

Flatwork may be reinforced with distributed steel to reduce the potential for differential slab movement where cracking occurs. The distributed reinforcing steel should be terminated about 3 inches from contraction joints and should consist of No. 3 deformed bars at 18 inches on center, both ways, or with 6x6-D4/D4 welded wire fabric supplied as sheets (not rolls). Slabs reinforced with distributed steel should be 6 inches thick (or more) for No. 3 bar reinforcement and 5 inches thick (or more) for 6x6-D4/D4 reinforcement to provide adequate concrete cover for the steel. To reduce the potential for differential slab movement across joints, the distributed steel may be extended through the joints. This improvement will be balanced by a reduction in the functionality of the contraction joint to encourage crack formation at joints. Flatwork subject to impact from unloading of dumpsters should be reinforced with No. 4 deformed bars at 12 inches on center, both ways extending through contraction joints, if present. Masonry briquettes or plastic chairs should be used to maintain the position of the reinforcement in the upper half of the slab with 1½ inches of cover over the steel and 3 inches of cover under the steel. Root barriers adjacent to trees may be considered to reduce the potential for pavement heave from root growth.

After demolition, clearing, and rough grading, finish subgrade for flatwork should be scarified and moisture conditioned as needed to achieve a moisture content about 2 percentage points above the optimum, before compaction, by mechanical means, to 90 percent or more of ASTM D1557 reference density on a dry density basis for flatwork that is not subject to vehicular loading and 95 percent of the reference density for flatwork that is subject to vehicular loading. Aggregate base should conform to the criteria for Class 2 aggregate base in Section 26-1.02 of the California Standard Specifications (Caltrans, 2022) and should be placed and compacted in lifts by vibrating plates, smooth drum rollers, or mechanical tampers, to 95 percent of the ASTM D1557 reference density on a dry density basis after moisture conditioning to near the optimum moisture content. The allowable lift thickness is influenced by the type of compaction equipment utilized but generally should not exceed 8 inches in loose thickness. Compacted subgrade and aggregate base should be maintained in a moist condition by sprinkling water. Compacted subgrade or aggregate base that has dried out and loosened or developed desiccation cracking should be scarified and moisture-conditioned, as needed, and recompacted as per the requirements above before the material is covered with concrete or aggregate base.

Laboratory testing for this study indicates that site soils have a medium to high expansion characteristic. Seasonal variations in soil moisture, particularly near the perimeter of the flatwork, may result in differential vertical and lateral movement with seasonal shrinkage and swelling of the expansive soil. Where not restrained by curbs, the potential for longitudinal cracking and joint separation from differential lateral movement can be mitigated by extending distributed reinforcing steel though flatwork joints, as discussed above, or by placing a layer of geotextile (Mirafi 600X or equivalent) below the aggregate base layer. Where desirable, the potential degree of differential vertical movement from shrinkage/swelling can by reduced by chemically treating the subgrade to a depth of 12-inches with quicklime to reduce the expansion characteristic, or by replacing the top 12 inches of subgrade below the flatwork with low expansion fill consisting of additional aggregate base or imported select fill conforming to the criteria in Section 9.3.6. Recommendations for chemical treatment of subgrade with quicklime are provided in Section 9.3.9.

9.6 Concrete

Due to the potential variability in the on-site soil and the potential future use of reclaimed water at the site~~,~~ we recommend that Type II/V or Type V cement be used for concrete structures in contact with soil. In addition, we recommend a water-to-cement ratio of no more than 0.45. A 3-inch thick, or thicker, concrete cover should be maintained over reinforcing steel where concrete is in contact with soil in accordance with recommendations of ACI Committee 318.

To reduce the potential for shrinkage cracks in the concrete during curing, concrete for slabs and flatwork should not contain large quantities of water or accelerating admixtures containing calcium chloride. Higher compressive strengths may be achieved by using larger aggregates in lieu of increasing the cement content and corresponding water demand. Additional workability, if desired, may be obtained by including water-reducing or air-entraining admixtures. Concrete should be placed in accordance with the guidelines of ACI Committee 302 and project specifications. Particular attention should be given to curing techniques and curing duration. Slabs that do not receive adequate curing have a more pronounced tendency to curl upwards at edges and corners, and to develop random shrinkage cracks and other defects.

In the event that contraction joints are used to influence the location of crack development in slabs and the joints are to be constructed by saw cutting of the slabs, saw cuts should be made by soft-cut sawing within 4 to 12 hours after the initial hardening (not curing) of the concrete, as required by atmospheric conditions. The contractor should be responsible for monitoring of the concrete during initial set or hardening and to determine the optimal timing for cutting of the slabs.

9.7 Surface Drainage and Site Maintenance

Surface drainage on the site should generally be provided so that water is diverted away from structures and is not permitted to pond. Positive drainage should be established adjacent to structures to divert surface water to an appropriate collector (graded swale, v-ditch, or area drain) with a suitable outlet. Drainage gradients should be 2 percent or more for a distance of 5 feet or more from the structure for impervious surfaces and 5 percent or more for a distance of 10 feet or more from the structure for pervious surfaces. Slope, pad, and roof drainage (from adjacent structures) should be collected and diverted to suitable discharge areas away from structures or other slopes by non-erodible devices (e.g., gutters, downspouts, concrete swales, etc.). Graded swales, v-ditches, or curb and gutter should be provided at the site perimeter to restrict flow of surface water onto and off of the site. Slopes should be vegetated or otherwise armored to reduce potential for erosion of soil. Drainage structures should be periodically cleaned out and repaired, as-needed, to maintain appropriate site drainage patterns.

Landscaping adjacent to foundations should include vegetation with low-water demands and irrigation should limit to that which is needed to sustain the plants. Trees should be restricted from the areas adjacent to foundations a distance equivalent to the canopy radius of the mature tree. Infiltration basins, dry wells, and other stormwater management measures that rely on infiltration without a liner and subdrain should not be located within 20 feet of structure foundations. Bioretention planters located within 20 feet of structure foundations should be lined with concrete or a plastic membrane and include a subdrain.

Care should be taken by the contractor during grading to preserve any berms, drainage terraces, interceptor swales or other drainage devices on or adjacent to the project area. Drainage patterns established at the time of grading should be maintained for the life of the project. The property owner and maintenance personnel should be made aware that altering drainage patterns might be detrimental to wall performance.

9.8 Geotechnical Engineer of Record Services

The recommendations provided in this report are based on preliminary design information for the proposed construction. The Geotechnical Engineer-of-Record (GEOR) should review the plans that are developed by the design team before construction bidding, to check that the scope of the project as designed is consistent with the assumed basis of this report and evaluate conformance with the geotechnical recommendations.

During construction, the GEOR should evaluate the exposed subsurface conditions for consistency with the conditions encountered in the discrete borings performed for this study, and to check that the work conforms with the geotechnical recommendations. Specifically, the geotechnical engineer should be retained to:

- Observe preparation and compaction of subgrade.
- Observe remedial grading and building pad preparation.
- Check and test imported materials prior to use as fill.
- Observe placement and compaction of fill and aggregate base.
- Perform field density tests to evaluate fill and subgrade compaction.
- Check the foundation excavations for bearing materials and cleaning prior to placement of reinforcing steel and concrete.
- Check the condition of the moisture vapor retarding system prior to placement of reinforcing steel and concrete.

The recommendations provided in this report assume that Ninyo & Moore will be retained as the geotechnical consultant during the construction phase of the project. If another geotechnical consultant is selected, we request that the selected consultant provide a letter to the architect and the owner (with a copy to Ninyo & Moore) indicating that they fully understand Ninyo & Moore's recommendations, and that they are in full agreement with the recommendations contained in this report.

10 LIMITATIONS

The field evaluation and geotechnical analyses presented in this geotechnical report have been conducted in general accordance with current practice and the standard of care exercised by geotechnical consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be encountered during construction. Uncertainties relative to subsurface conditions can be reduced through subsurface exploration. Subsurface evaluation will be performed upon request.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified and additional recommendations, if warranted, will be provided upon request. It should be understood that the conditions of a site can change with time as a result of natural processes or the activities of man at the subject site or nearby sites. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur due to government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

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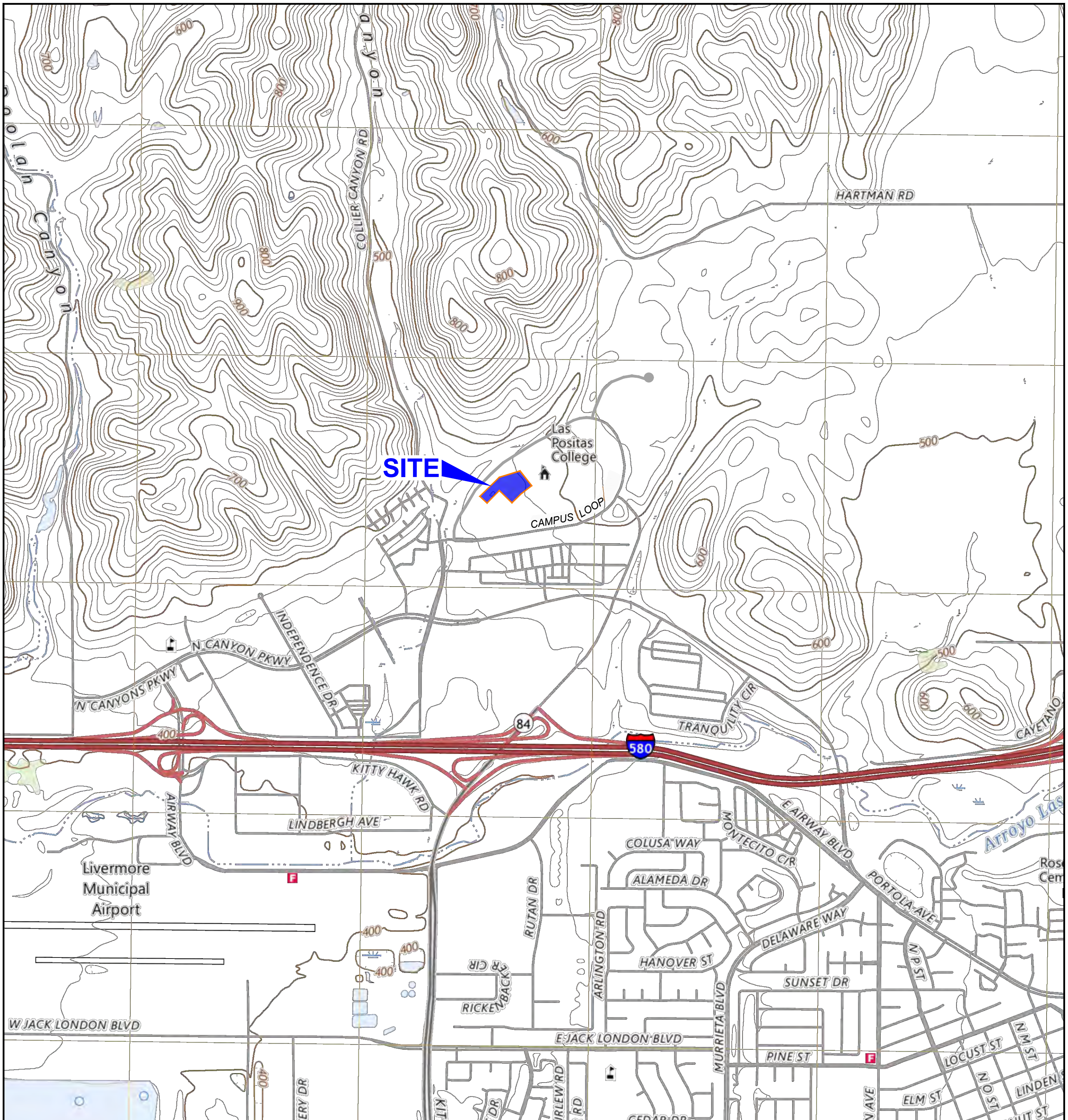
FIGURES



NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE | REFERENCE: USGS, 2023

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SCALE (FEET)
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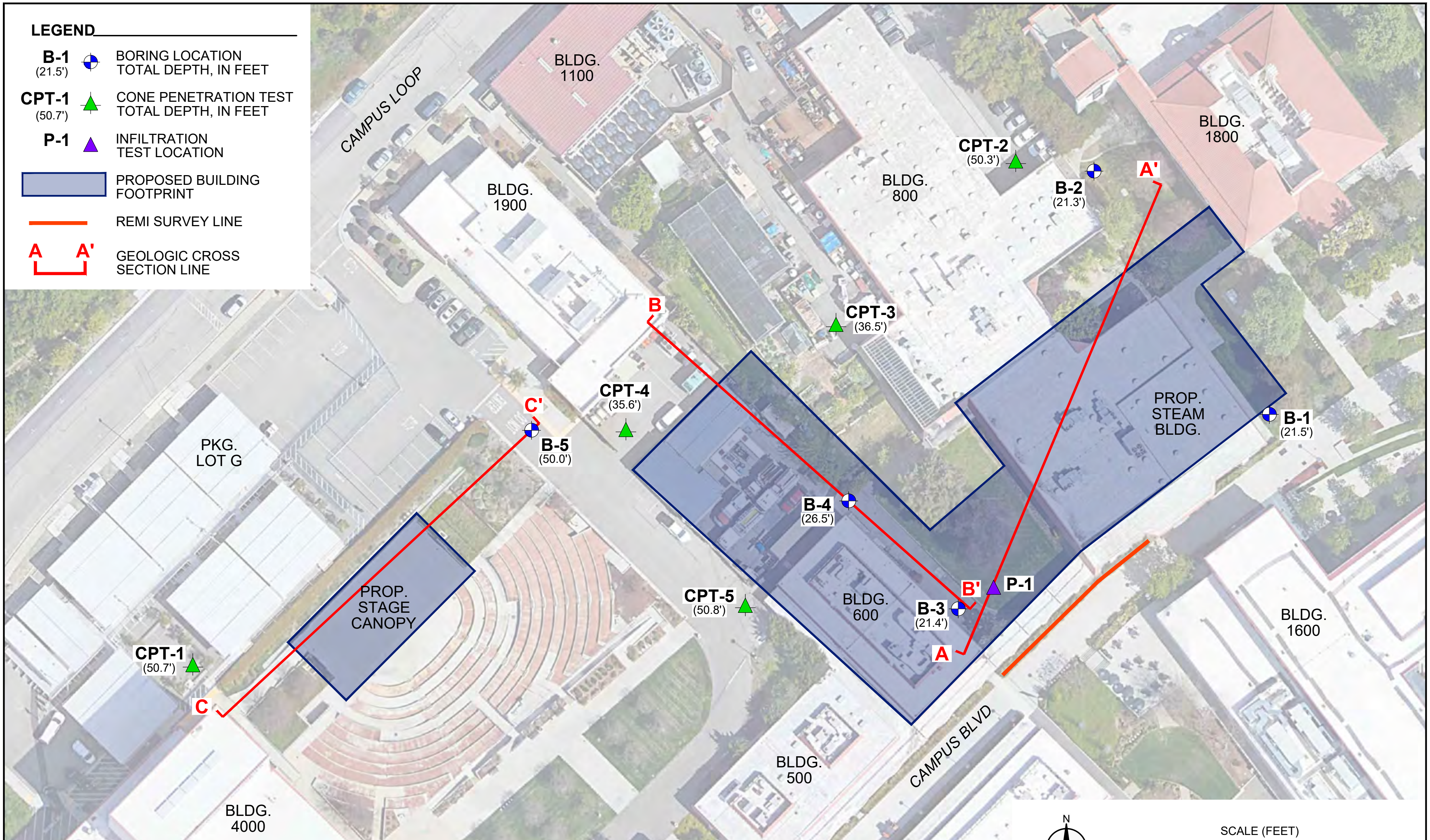
FIGURE 1

SITE LOCATION

LAS POSITAS COLLEGE - STEAM BUILDING
3000 CAMPUS HILL DRIVE
LIVERMORE, CALIFORNIA

401294038 | 11/23

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants



NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE | REFERENCE: GOOGLE EARTH, 2023

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FIGURE 2

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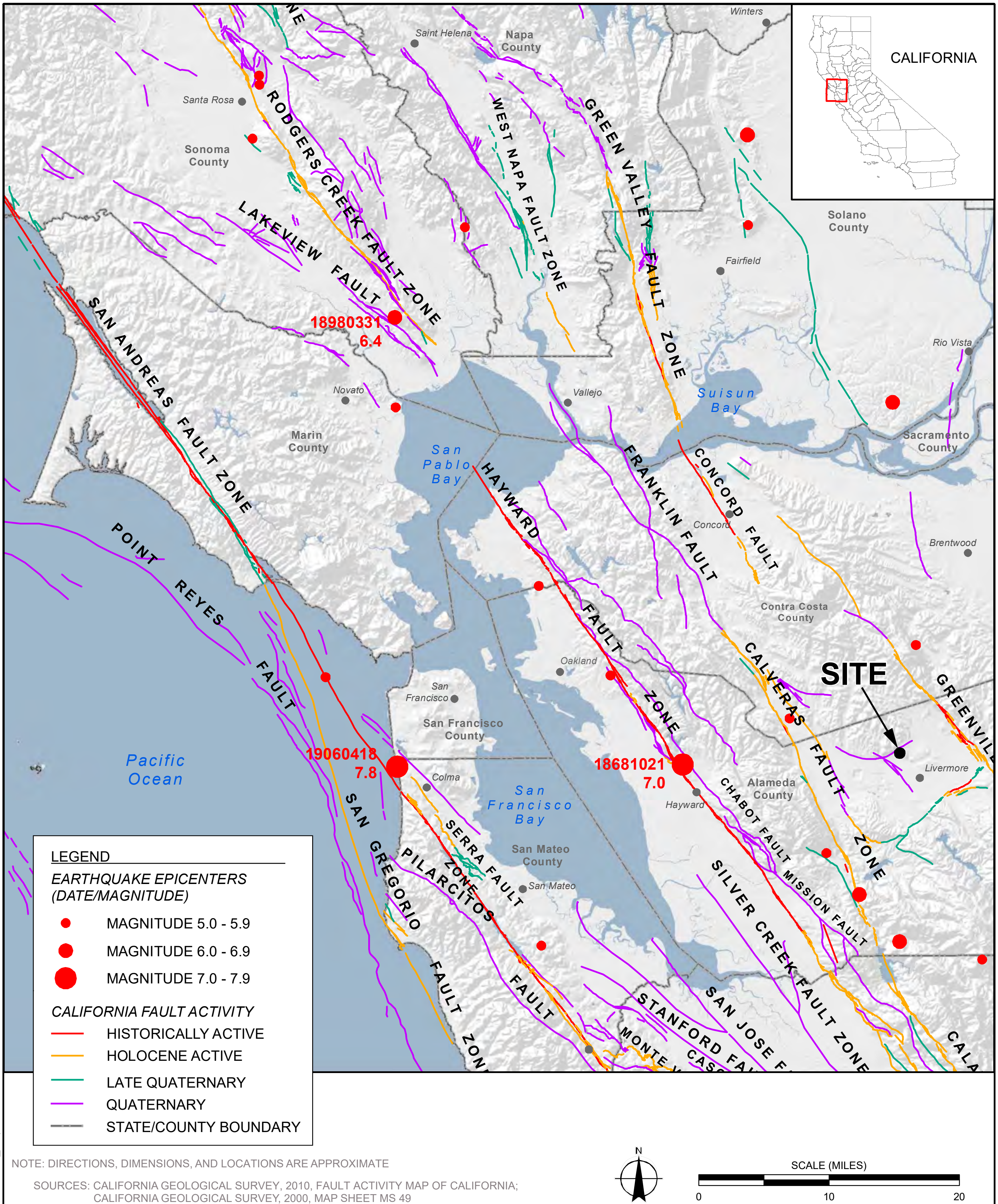
Geotechnical & Environmental Sciences Consultants

EXPLORATION LOCATIONS

LAS POSITAS COLLEGE - STEAM BUILDING
3000 CAMPUS HILL DRIVE
LIVERMORE, CALIFORNIA

401294038 I 11/23



NOTE: DIRECTIONS, DIMENSIONS, AND LOCATIONS ARE APPROXIMATE

SOURCES: CALIFORNIA GEOLOGICAL SURVEY, 2010, FAULT ACTIVITY MAP OF CALIFORNIA; CALIFORNIA GEOLOGICAL SURVEY, 2000, MAP SHEET MS 49

401294038_FL.pdf 11/14/2023 AEK

FIGURE 3

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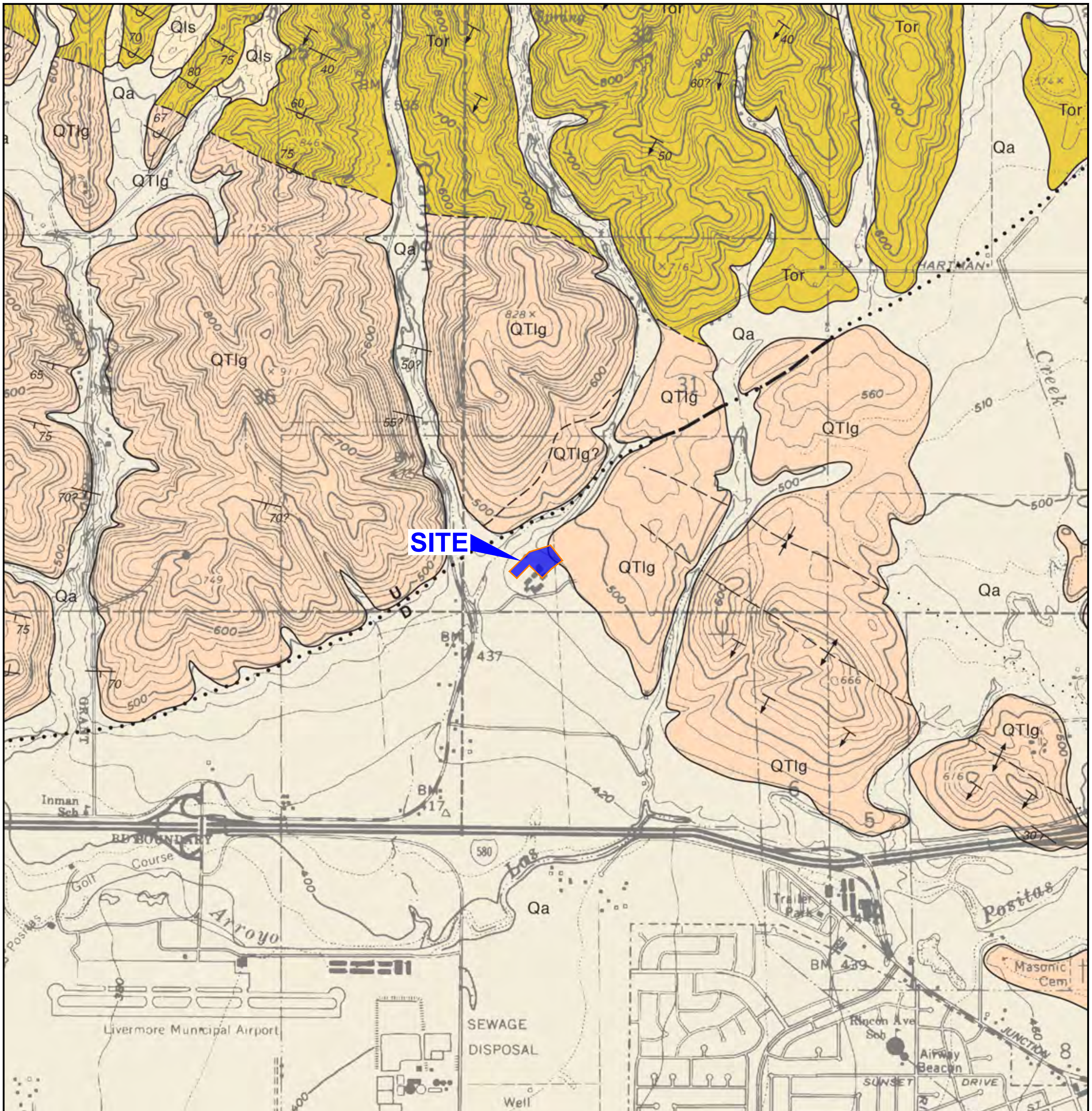
FAULT LOCATIONS AND EARTHQUAKE EPICENTERS

LAS POSITAS COLLEGE - STEAM BUILDING
3000 CAMPUS HILL DRIVE
LIVERMORE, CALIFORNIA
401294038 | 11/23



LEGEND

Qa	ALLUVIAL GRAVEL, SAND, & CLAY OF VALLEY AREAS (HOLOCENE)
Qls	LANDSLIDE RUBBLE (HOLOCENE)
QTlg	LIVERMORE GRAVELS (PLEISTOCENE)
Tor	ORINDA FORMATION: PEBBLE CONGLOMERATE, SANDSTONE, & CLAYSTONE (PLIOCENE)

THRUST FAULT

FAULT

GEOLOGIC CONTACT

STRIKE AND DIP OF BEDDING

N

SCALE (FEET)

0 2,000 4,000

NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE I REFERENCE: DIBBLEE, 2006

401294038.dwg 11/14/2023 AEK

FIGURE 4

REGIONAL GEOLOGY

LAS POSITAS COLLEGE - STEAM BUILDING
3000 CAMPUS HILL DRIVE
LIVERMORE, CALIFORNIA

401294038 I 11/23

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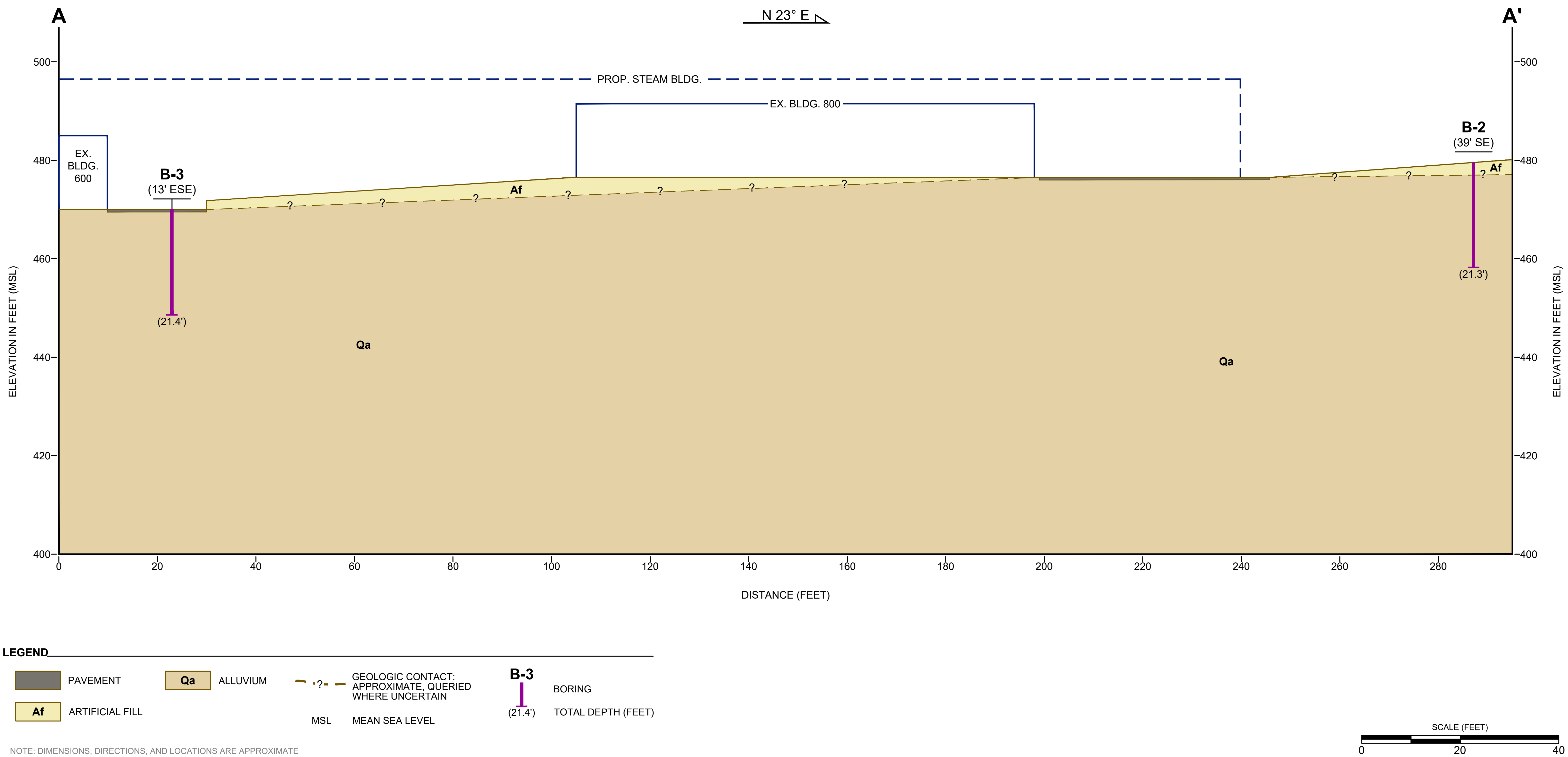

NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE

401294038.dwg 11/21/2023 AEK

FIGURE 5

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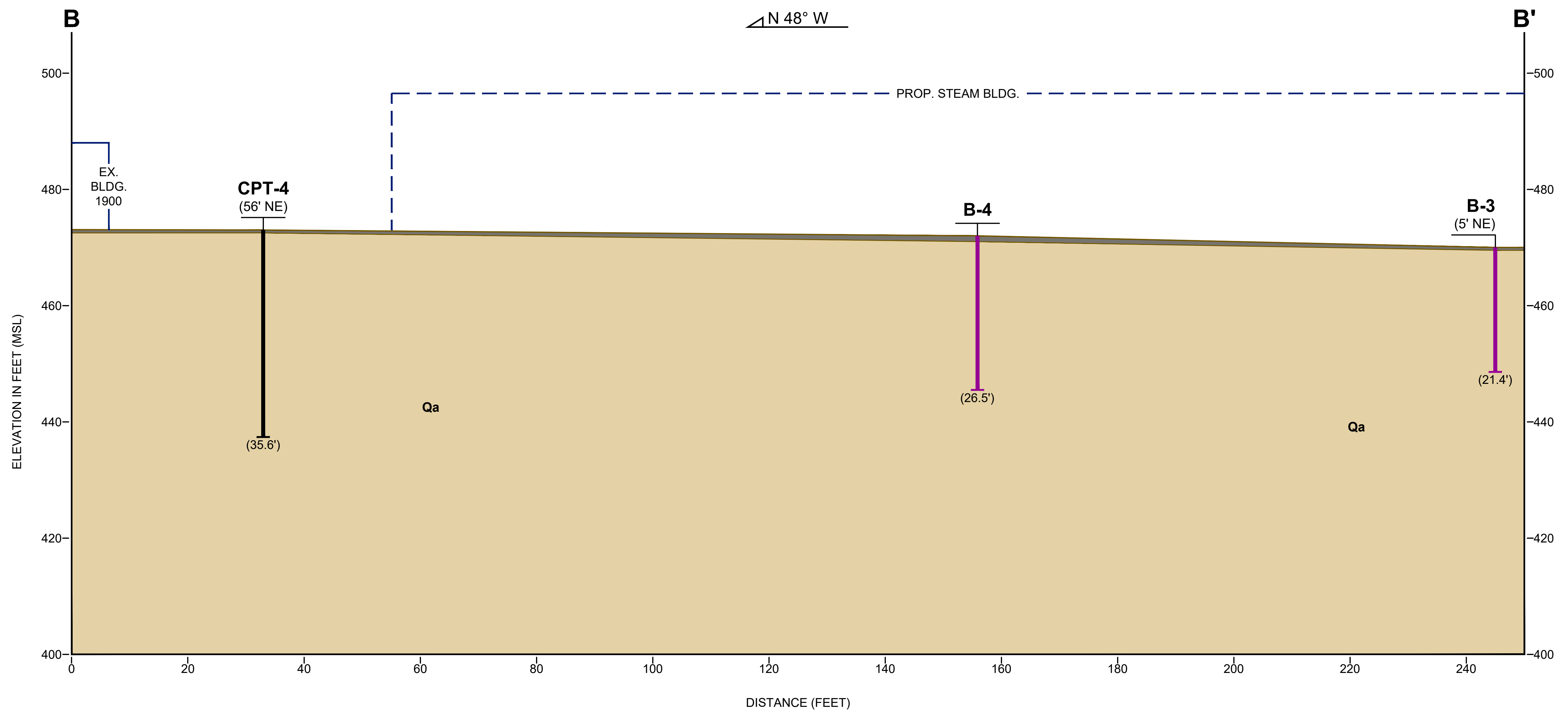
GEOLOGIC CROSS SECTION A-A'

LAS POSITAS COLLEGE - STEAM BUILDING
3000 CAMPUS HILL DRIVE
LIVERMORE, CALIFORNIA

401294038 I 11/23



LEGEND

PAVEMENT

Qa ALLUVIUM

GEOLOGIC CONTACT: APPROXIMATE, QUERIED WHERE UNCERTAIN

MSL MEAN SEA LEVEL

CPT-4 CPT (35.6') TOTAL DEPTH (FEET)

B-4 BORING (26.5') TOTAL DEPTH (FEET)

NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE

SCALE (FEET) 0 20 40

401294038.dwg 11/21/2023 AEK

FIGURE 6

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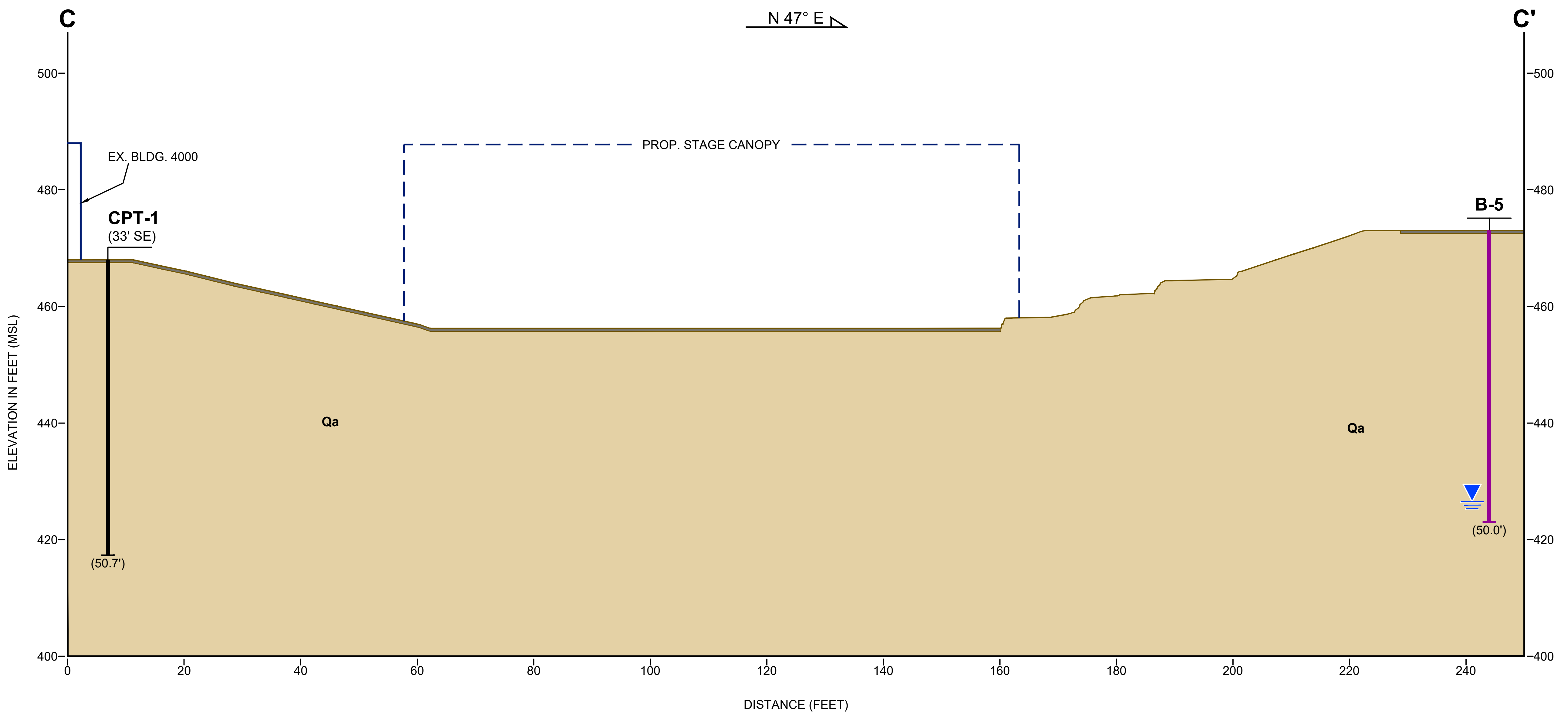
GEOLOGIC CROSS SECTION B-B'

LAS POSITAS COLLEGE - STEAM BUILDING
3000 CAMPUS HILL DRIVE
LIVERMORE, CALIFORNIA

401294038 I 11/23



LEGEND

PAVEMENT	GEOLOGIC CONTACT: APPROXIMATE, QUERIED WHERE UNCERTAIN	GROUNDWATER DEPTH (MEASURED AT TIME OF EXPLORATION)	CPT-1	CPT	B-5 BORING
Qa ALLUVIUM	MSL MEAN SEA LEVEL		(50.7')	TOTAL DEPTH (FEET)	(50.0') TOTAL DEPTH (FEET)

NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE

SCALE (FEET) 0 20 40

FIGURE 7

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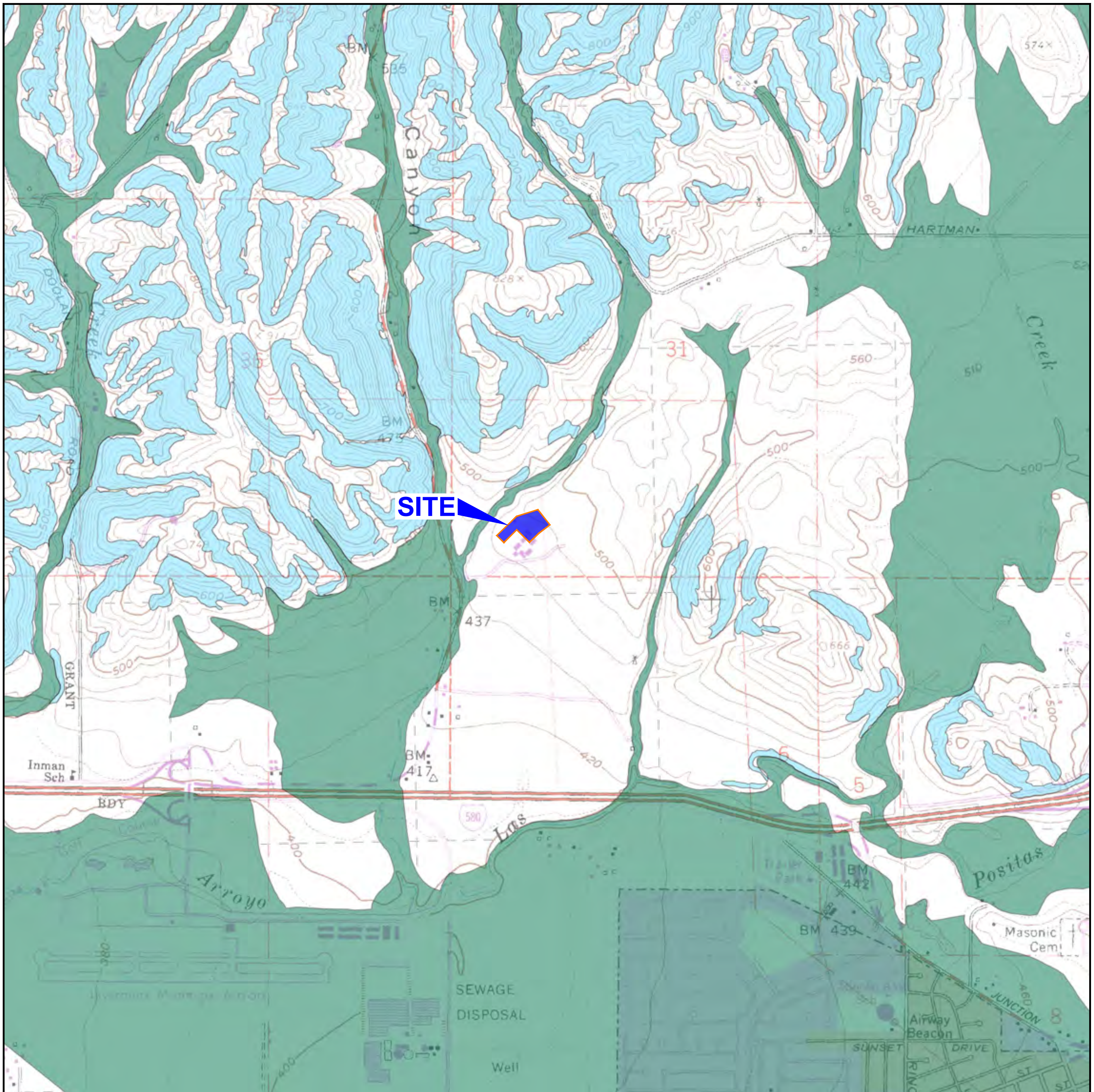
GEOLOGIC CROSS SECTION C-C'

LAS POSITAS COLLEGE - STEAM BUILDING
3000 CAMPUS HILL DRIVE
LIVERMORE, CALIFORNIA

401294038 I 11/23

401294038.dwg 11/21/2023 AEK



LEGEND

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

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

N

SCALE (FEET)
0 2,000 4,000

NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE
REFERENCE: CGS, 1982, 2008

401294038.dwg 11/21/2023 AEK

FIGURE 8

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SEISMIC HAZARD ZONES

LAS POSITAS COLLEGE - STEAM BUILDING
3000 CAMPUS HILL DRIVE
LIVERMORE, CALIFORNIA

401294038 I 11/23

APPENDIX A

Cone Penetration Testing

APPENDIX A

CONE PENETRATION TESTING

Field Procedure for Cone Penetration Testing

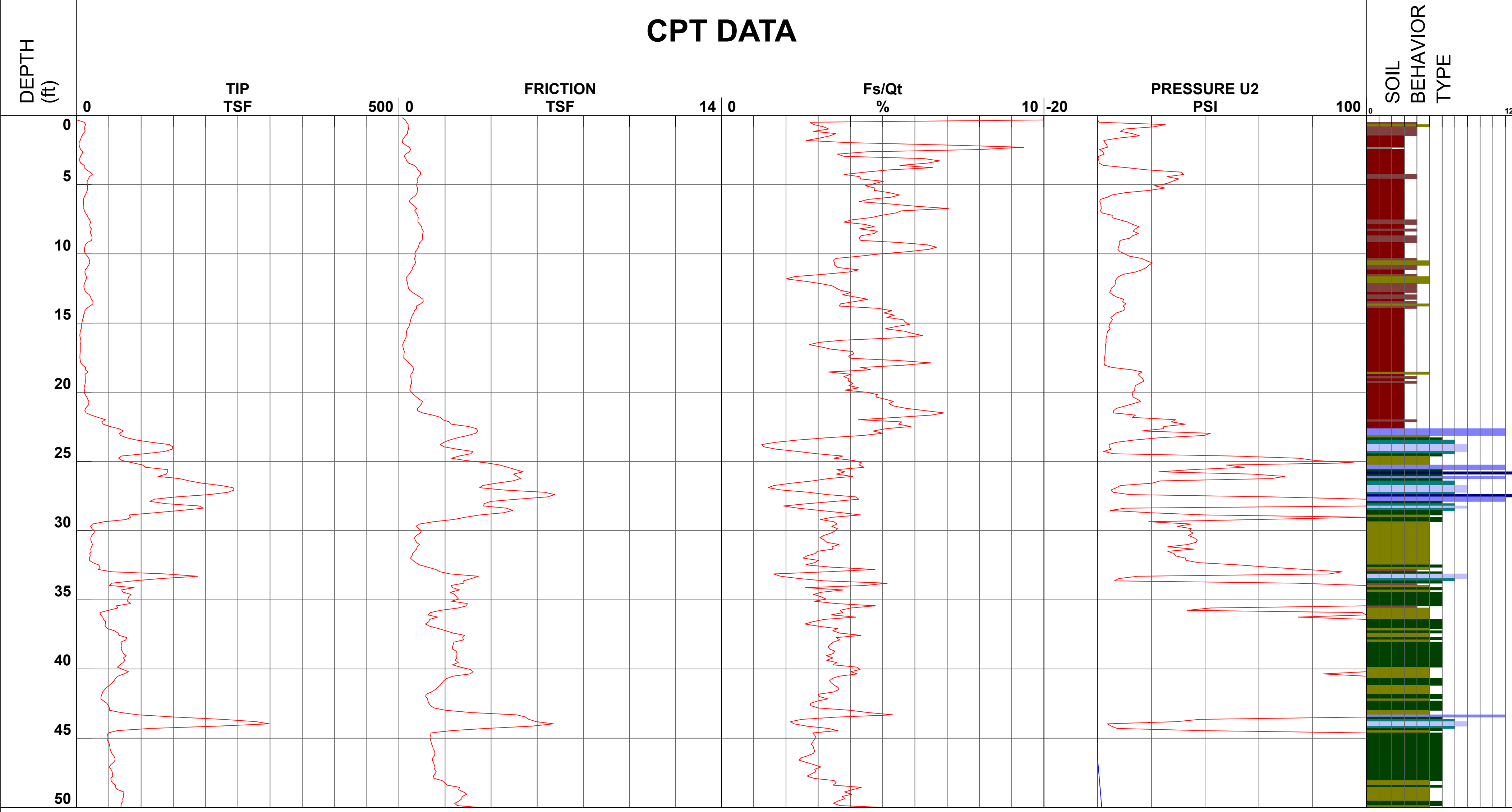
A penetrometer with a conical tip having an apex angle of 60 degrees and a cone base area of 15 square centimeters was hydraulically pushed through the soil using the reaction mass of a 20-ton rig at a constant rate of about 20 millimeters per second in accordance with ASTM D 5778. The penetrometer was instrumented to measure, by electronic methods, the water pressure acting on a transducer near the cone tip, the force on the conical point required to penetrate the soil, and the force on a friction sleeve behind the cone tip as the penetrometer was advanced. Tip resistance, sleeve friction, and pore water pressure data were collected and recorded electronically at intervals of approximately 2 inches while the cone was advanced. Cone tip resistance corrected for pore pressure (Qt) was calculated by dividing the measured force of penetration by the cone base area and adding a fraction of the recorded pore pressure (U2). Friction sleeve resistance (Fs) was calculated by dividing the measured force on the friction sleeve by the surface area of the sleeve. The friction ratio was calculated as the ratio of the tip resistance to the sleeve friction (Fs/Qt). The tip resistance and friction ratio were used to evaluate the soil behavior type (Robertson, 1986). Graphs of tip resistance, sleeve friction, friction ratio, pore pressure, and interpreted soil behavior type are presented on the logs of the CPT Soundings.

Ninyo & Moore

Project	Las Positas College-STEAM Building	Operator	AJ-ER	Filename	SDF(049).cpt
Job Number	401294038	Cone Number	DDG1587	GPS	
Hole Number	CPT-01	Date and Time	10/6/2023 10:28:36 AM	Maximum Depth	50.69 ft
EST GW Depth During Test		46.50 ft			

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

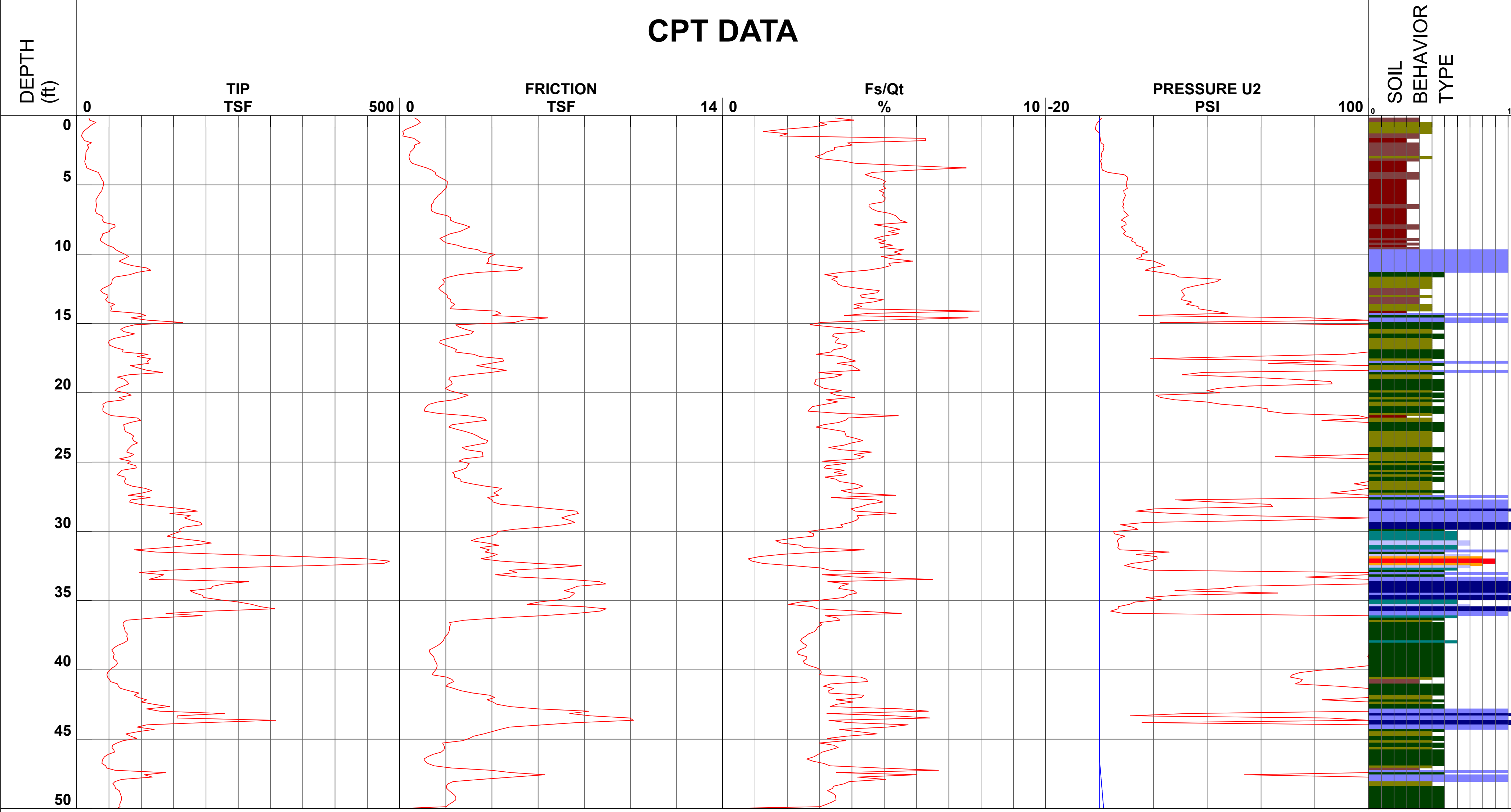
Cone Size 15cm²

S*Soil behavior type and SPT based on data from UBC-1983

Ninyo & Moore

Project	Las Positas College-STEAM Building	Operator	AJ-ER	Filename	SDF(053).cpt
Job Number	401294038	Cone Number	DDG1587	GPS	
Hole Number	CPT-02	Date and Time	10/6/2023 5:09:19 PM	Maximum Depth	50.36 ft
EST GW Depth During Test			46.50 ft		

Net Area Ratio .8



- 1 - sensitive fine grained
- 2 - organic material
- 3 - clay
- 4 - silty clay to clay
- 5 - clayey silt to silty clay
- 6 - sandy silt to clayey silt
- 7 - silty sand to sandy silt
- 8 - sand to silty sand
- 9 - sand
- 10 - gravelly sand to sand
- 11 - very stiff fine grained (*)
- 12 - sand to clayey sand (*)

Cone Size 15cm²

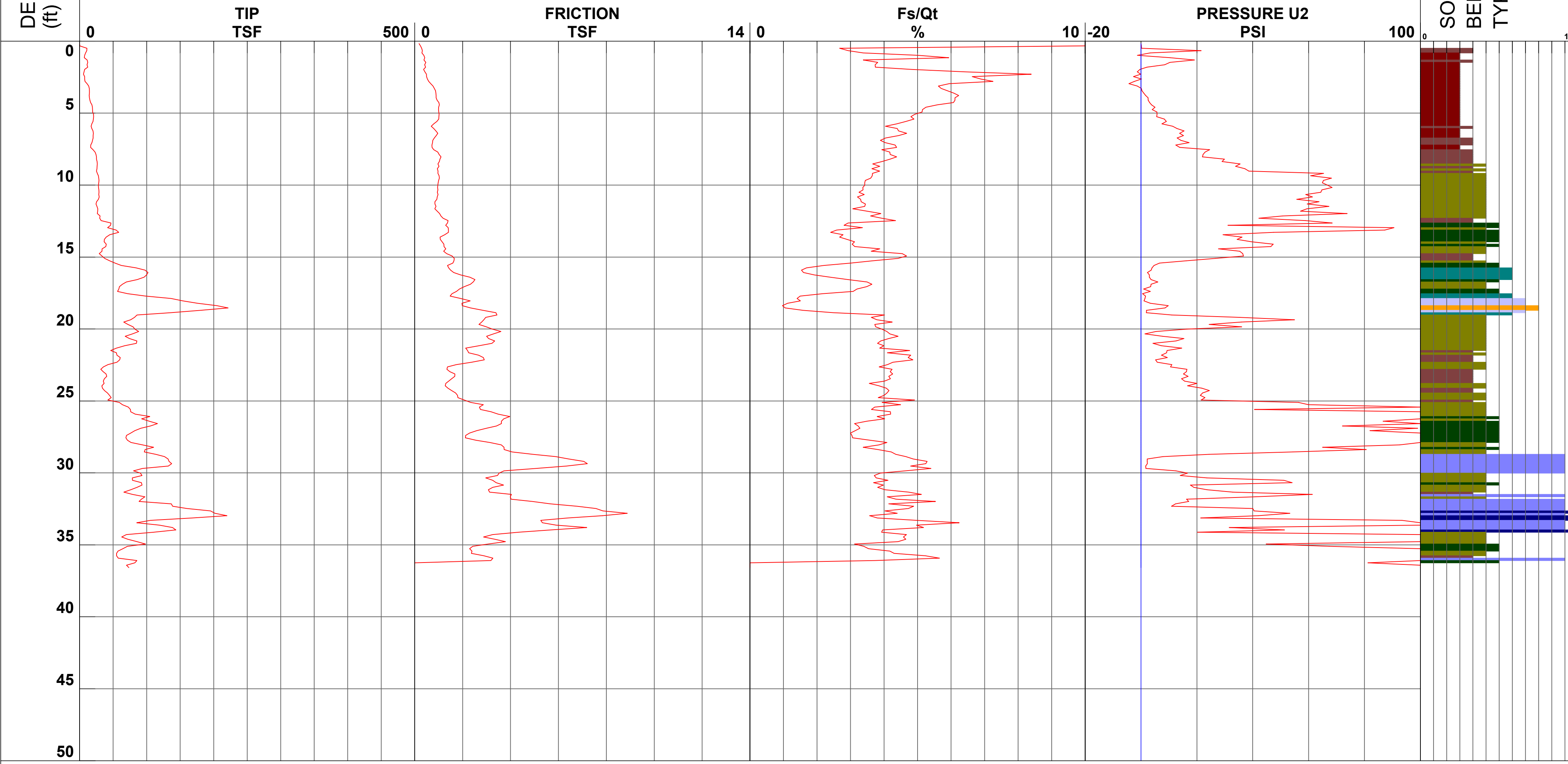
S*Soil behavior type and SPT based on data from UBC-1983

Ninyo & Moore

Project	Las Positas College-STEAM Building	Operator	AJ-ER	Filename	SDF(052).cpt
Job Number	401294038	Cone Number	DDG1587	GPS	
Hole Number	CPT-03	Date and Time	10/6/2023 3:43:20 PM	Maximum Depth	36.58 ft
EST GW Depth During Test		46.50 ft			

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 15cm²

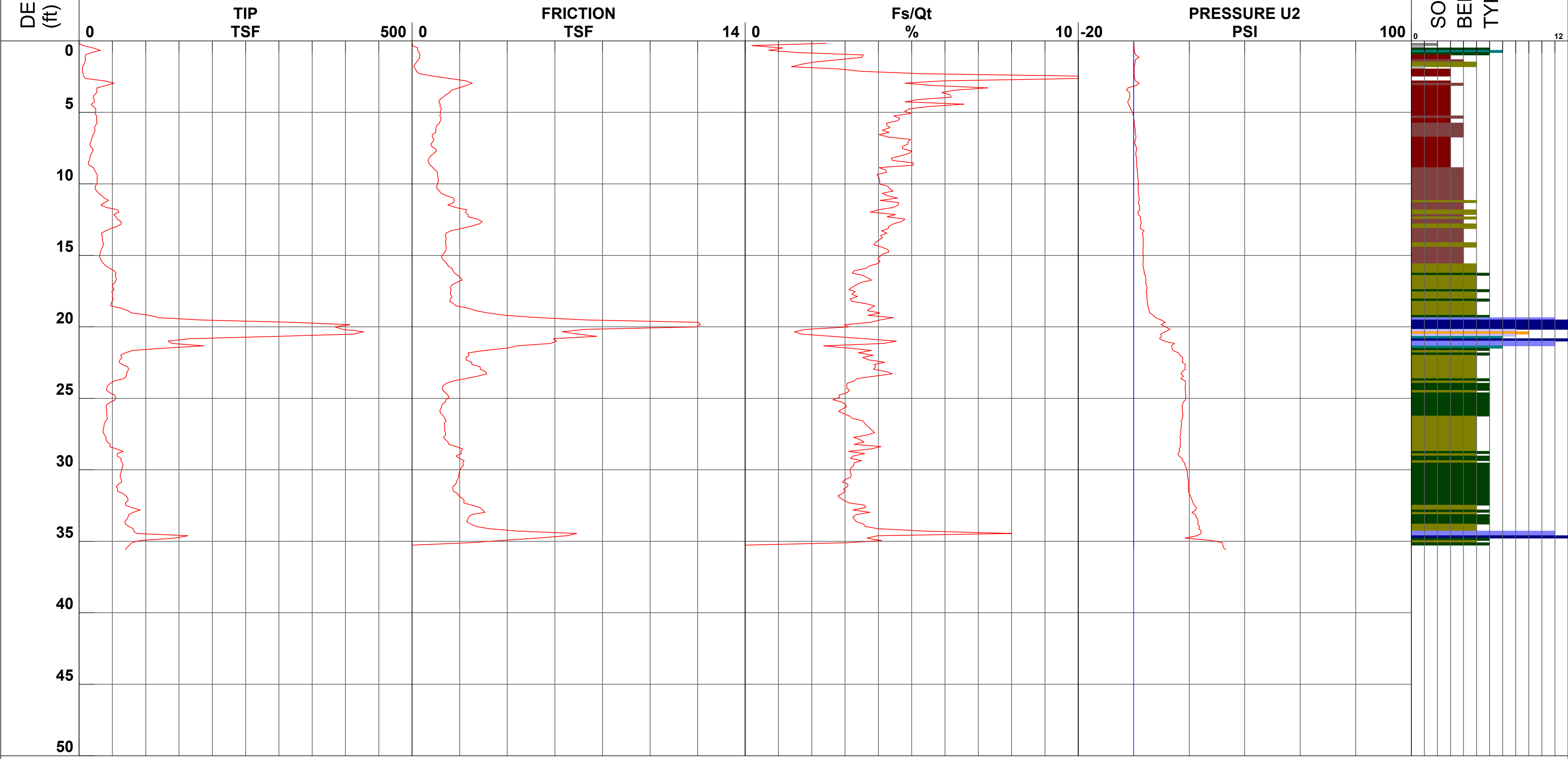
S*Soil behavior type and SPT based on data from UBC-1983

Ninyo & Moore

Project	Las Positas College-STEAM Building	Operator	AJ-ER	Filename	SDF(050).cpt
Job Number	401294038	Cone Number	DDG1587	GPS	
Hole Number	CPT-04	Date and Time	10/6/2023 12:14:16 PM	Maximum Depth	35.60 ft
EST GW Depth During Test		46.50 ft			

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

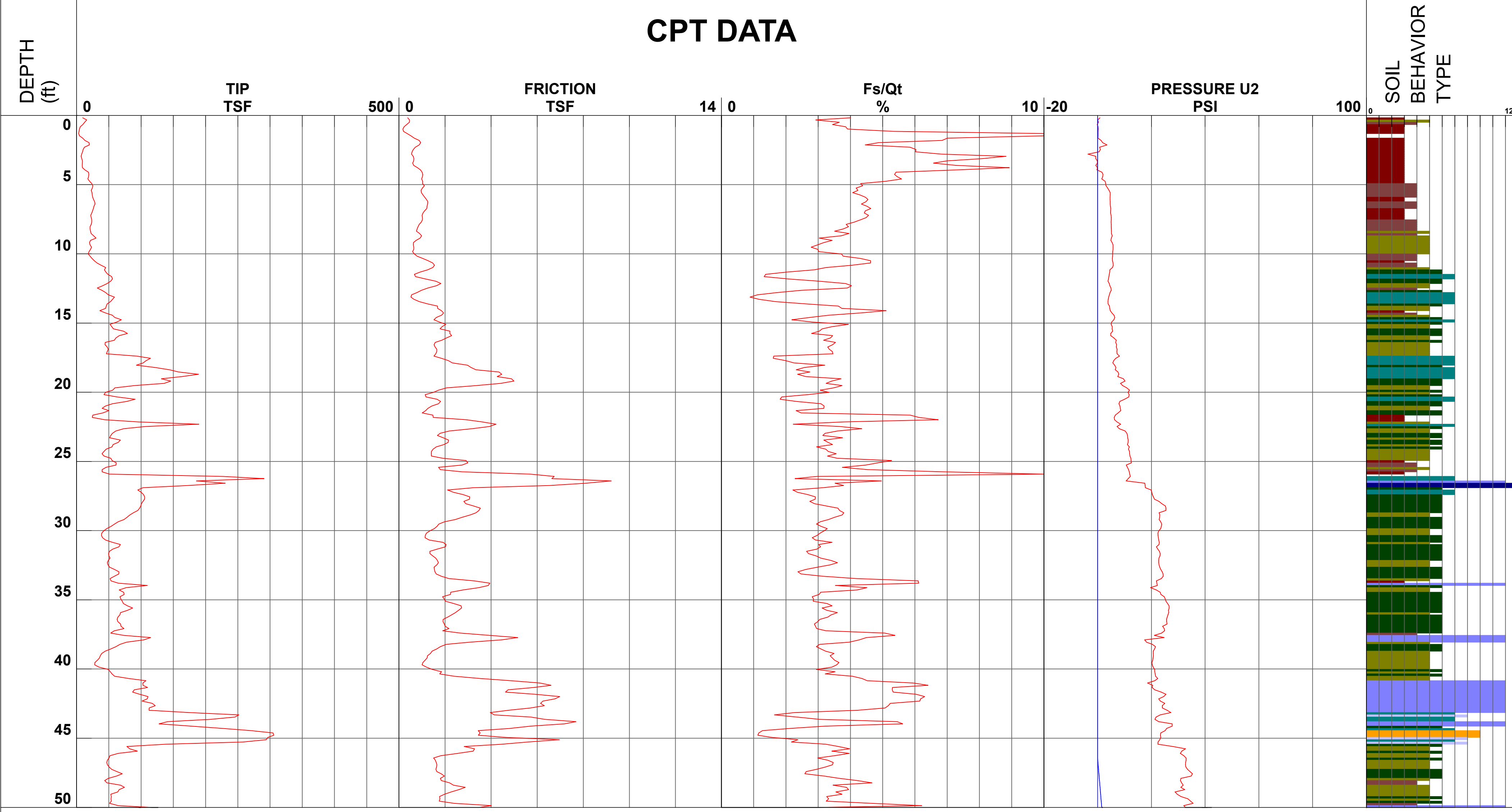
Cone Size 15cm²

S*Soil behavior type and SPT based on data from UBC-1983

Ninyo & Moore

Project	Las Positas College-STEAM Building	Operator	AJ-ER	Filename	SDF(051).cpt
Job Number	401294038	Cone Number	DDG1587	GPS	
Hole Number	CPT-05	Date and Time	10/6/2023 1:58:34 PM	Maximum Depth	50.85 ft
EST GW Depth During Test			46.50 ft		

Net Area Ratio .8



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 15cm²

S*Soil behavior type and SPT based on data from UBC-1983

APPENDIX B

Boring Logs

APPENDIX B

BORING LOGS

Field Procedure for the Collection of Disturbed Samples
Disturbed soil samples were obtained in the field using the following methods.

Bulk Samples
Bulk samples of representative earth materials were obtained from the exploratory borings. The samples were bagged and transported to the laboratory for testing.

The Standard Penetration Test (SPT) Sampler
Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of 1-3/8 inches. The sampler was driven into the ground 18 inches or the interval recorded on the boring log where driving refusal occurred, with a 140-pound hammer falling relatively freely from a height of 30 inches in general accordance with ASTM D 1586. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration or the interval reported. Soil samples were observed and removed from the sampler, bagged, sealed and transported to the laboratory for testing.

Field Procedure for the Collection of Relatively Undisturbed Samples
Relatively undisturbed soil samples were obtained in the field using the following method.

Modified Split-Barrel Drive Sampler
Relatively undisturbed soil samples were obtained in the field using a modified split-barrel drive sampler. The sampler, with an external diameter of 3.0 inches, was lined with 6-inch long, thin wall stainless steel liners with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with a 140-pound hammer with a drop height of 30 inches in general accordance with ASTM D 3550. The driving weight was permitted to fall relatively freely. The sampler was driven into the ground 18 inches or the interval recorded on the boring log where driving refusal occurred. The approximate length of the fall, the weight of the hammer, and the number of blows for the last 12 inches of penetration or the interval reported are presented on the boring logs. The blow counts are recorded as an index to the relative resistance of the materials sampled. The samples were removed from the sample barrel in the liners, sealed, and transported to the laboratory for testing.

Field Testing
The following tests were performed in the field to evaluate soil properties.

Static Cone Penetrometer
A penetrometer with a conical tip having an apex angle of 60 degrees and a cone base area of 1.5 square centimeters was manually pushed 6 inches into the soil. The penetrometer was instrumented to measure the Cone Penetration Index (Qc) computed as the peak force on the cone divided by the cone base area. The Cone Penetration Index is reported in kilograms per square centimeter (ksc) on the boring logs at the depth of the test as a measure of the relative density or consistency of the soil encountered.

BORING LOG EXPLANATION SHEET

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	
0								Bulk sample.
								Modified split-barrel drive sampler.
								No recovery with modified split-barrel drive sampler.
								Sample retained by others.
								Standard Penetration Test (SPT).
5								No recovery with a SPT.
			XX/XX					Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.
								No recovery with Shelby tube sampler.
								Continuous Push Sample.
								Seepage.
10								Groundwater encountered during drilling.
								Groundwater measured after drilling.
							SM	MAJOR MATERIAL TYPE (SOIL): Solid line denotes unit change.
							CL	Dashed line denotes material change. Attitudes: Strike/Dip b: Bedding c: Contact j: Joint f: Fracture F: Fault cs: Clay Seam s: Shear bss: Basal Slide Surface sf: Shear Fracture sz: Shear Zone sbs: Shear Bedding Surface
15								
								The total depth line is a solid line that is drawn at the bottom of the boring.
20								

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BORING LOG

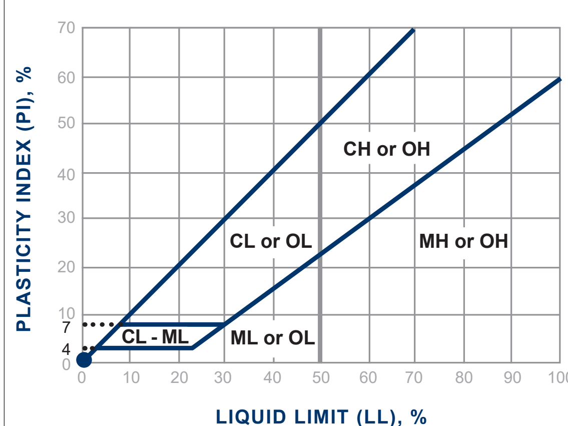
Soil Classification Chart Per ASTM D 2488

Primary Divisions			Secondary Divisions: Group Symbol	Secondary Divisions: Group Name
COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve	**GRAVEL** more than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVEL less than 5% fines	GW	well-graded GRAVEL
			GP	poorly graded GRAVEL
		GRAVEL with DUAL CLASSIFICATIONS 5% to 12% fines	GW-GM	well-graded GRAVEL with silt
			GP-GM	poorly graded GRAVEL with silt
			GW-GC	well-graded GRAVEL with clay
			GP-GC	poorly graded GRAVEL with
		GRAVEL with FINES more than 12% fines	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND 50% or more of coarse fraction passes No. 4 sieve	CLEAN SAND less than 5% fines	SW	well-graded SAND
			SP	poorly graded SAND
		SAND with DUAL CLASSIFICATIONS 5% to 12% fines	SW-SM	well-graded SAND with silt
			SP-SM	poorly graded SAND with silt
			SW-SC	well-graded SAND with clay
			SP-SC	poorly graded SAND with clay
		SAND with FINES more than 12% fines	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOILS 50% or more passes No. 200 sieve	**SILT and CLAY** liquid limit less than 50%	INORGANIC	CL	lean CLAY
			ML	SILT
			CL-ML	silty CLAY
		ORGANIC	OL (PI > 4)	organic CLAY
			OL (PI < 4)	organic SILT
	SILT and CLAY liquid limit 50% or more	INORGANIC	CH	fat CLAY
			MH	elastic SILT
		ORGANIC	OH (plots on or above "A"-line)	organic CLAY
			OH (plots below "A"-line)	organic SILT
	Highly Organic Soils		PT	Peat

Grain Size

Description		Sieve Size	Grain Size	Approximate Size
Boulders		> 12"	> 12"	Larger than basketball-sized
Cobbles		3 - 12"	3 - 12"	Fist-sized to basketball-sized
Gravel	Coarse	3/4 - 3"	3/4 - 3"	Thumb-sized to fist-sized
	Fine	#4 - 3/4"	0.19 - 0.75"	Pea-sized to thumb-sized
Sand	Coarse	#10 - #4	0.079 - 0.19"	Rock-salt-sized to pea-sized
	Medium	#40 - #10	0.017 - 0.079"	Sugar-sized to rock-salt-sized
	Fine	#200 - #40	0.0029 - 0.017"	Flour-sized to sugar-sized
Fines		Passing #200	< 0.0029"	Flour-sized and smaller

Plasticity Chart



Apparent Density - Coarse-Grained Soil

Apparent Density	Spooling Cable or Cathead: SPT (blows/foot)	Spooling Cable or Cathead: Modified Split Barrel (blows/foot)	Automatic Trip Hammer: SPT (blows/foot)	Automatic Trip Hammer: Modified Split Barrel (blows/foot)
Very Loose	≤ 4	≤ 8	≤ 3	≤ 5
Loose	5 - 10	9 - 21	4 - 7	6 - 14
Medium Dense	11 - 30	22 - 63	8 - 20	15 - 42
Dense	31 - 50	64 - 105	21 - 33	43 - 70
Very Dense	> 50	> 105	> 33	> 70

Consistency - Fine-Grained Soil

Consistency	Spooling Cable or Cathead: SPT (blows/foot)	Spooling Cable or Cathead: Modified Split Barrel (blows/foot)	Automatic Trip Hammer: SPT (blows/foot)	Automatic Trip Hammer: Modified Split Barrel (blows/foot)
Very Soft	< 2	< 3	< 1	< 2
Soft	2 - 4	3 - 5	1 - 3	2 - 3
Firm	5 - 8	6 - 10	4 - 5	4 - 6
Stiff	9 - 15	11 - 20	6 - 10	7 - 13
Very Stiff	16 - 30	21 - 39	11 - 20	14 - 26
Hard	> 30	> 39	> 20	> 26

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

USCS METHOD OF SOIL CLASSIFICATION

DATE DRILLED 10/6/2023 BORING NO. B-1

GROUND ELEVATION 477' ± MSL SHEET 1 OF 1

METHOD OF DRILLING 4" Solid Flight, B-24 Truck Mounted (California Geotech)

DRIVE WEIGHT 140 lbs (cathead) DROP 30 inches

SAMPLED BY DHL LOGGED BY SSA REVIEWED BY RPM/PCC

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0	Bulk					CL	TOPSOIL: Black, wet, lean CLAY with vegetation.
	Bulk	Qs=10.5 Qs=9.7 Qs=19.5				CL	ALLUVIUM: Brown, moist, firm, lean CLAY.
							Stiff.
	Driven	38	12.9	114.1		SC	Brown with black mottling, moist, medium dense, clayey SAND; trace gravel.
						CL	Brown, moist, very stiff, lean CLAY.
10	Driven	39	8.6	110.9		GW	Brown, moist, medium dense, well-graded GRAVEL.
						SC	Brown, moist, medium dense, clayey SAND; few gravel.
							Scattered caliche veins.
	Driven	53	16.4	139.2		CL	Brown, moist, hard, lean CLAY with sand; trace gravel.
20	Driven	78	14.7	117.8			Few sand; scattered caliche veins.
30							
40							

Total depth = 21.5 feet.

Qs is Static Cone Penetration Test, resistance in kg/cm2.

Backfilled with neat cement shortly after drilling.

Notes:

Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents (Google, 2023).

DATE DRILLED 10/6/2023 BORING NO. B-2

GROUND ELEVATION 475' ± MSL SHEET 1 OF 1

METHOD OF DRILLING 4" Solid Flight, B-24 Truck Mounted (California Geotech)

DRIVE WEIGHT 140 lbs (cathead) DROP 30 inches

SAMPLED BY DHL LOGGED BY SSA REVIEWED BY RPM/PCC

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0			Qs=28 Qs=21				CL	FILL: Gray, moist, stiff, lean CLAY.
			Qs=24 Qs=26				CL	ALLUVIUM: Light brown, moist, stiff, lean CLAY.
			33	16.0	117.8		SC	Light brown, moist, medium dense, clayey SAND.
10			34	20.1	107.1		CL	Light brown, moist, very stiff, lean CLAY with sand.
			49	19.4	109.0			Hard; few sand; scattered caliche veins.
20			74/9"	14.6	118.7			
30								Total depth = 21.3 feet. Qs is Static Cone Penetration Test, resistance in kg/cm2. Backfilled with neat cement shortly after drilling. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents (Google, 2023).
40								

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 2 inches thick.
								AGGREGATE BASE: Approximately 3 inches thick.
			Qs=22				CH	TOPSOIL: Black, moist, soft, fat CLAY.
			Qs=29				GW	Black, moist, loose, well-graded GRAVEL.
			Qs=27				CL	ALLUVIUM: Light brown, moist, stiff, lean CLAY.
			Qs=37				GC	Light brown, moist, loose, clayey GRAVEL.
			38	19.3	108.3		CL	Light brown, moist, very stiff, lean CLAY with sand; scatered caliche veins.
10			47	13.9	119.3			Hard; trace to few gravel.
			56	15.8	116.2			
20			80/11"	11.6	128.1		SC	Brown, moist, very dense, clayey SAND with gravel.
30								
40								

Total depth = 21.4 feet.

Qs is Static Cone Penetration Test, resistance in kg/cm2.

Backfilled with neat cement and patched with cold asphalt shortly after drilling.

Notes:

Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents (Google, 2023).

DATE DRILLED 10/6/2023 BORING NO. B-3

GROUND ELEVATION 468' ± MSL SHEET 1 OF 1

METHOD OF DRILLING 4" Solid Flight, B-24 Truck Mounted (California Geotech)

DRIVE WEIGHT 140 lbs (cathead) DROP 30 inches

SAMPLED BY DHL LOGGED BY SSA REVIEWED BY RPM/PCC

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 2.5 inches thick.
							SC	AGGREGATE BASE: Approximately 9 inches thick.
			Qs=28				CL	ALLUVIUM: Light brown, moist, loose, clayey SAND; trace gravel.
			Qs=26				SC	Light brown, moist, soft, lean CLAY; little gravel.
			Qs=25				CH	Light brown, moist, loose, clayey SAND with gravel.
			Qs=30					Brown to gray, moist, hard, fat CLAY with sand.
			40	20.9	107.1			
10							CL	Light brown, moist, hard, lean CLAY with sand.
			45	18.3	112.3			
			48	15.1	117.1			Trace to little sand.
20			48	11.5	116.3			
							SC	Light brown, moist, medium dense, clayey SAND.
			47	17.5	111.9		CL	Brown to light brown, moist, hard, lean CLAY with sand.
30								
40								

DATE DRILLED 10/6/2023 BORING NO. B-4

GROUND ELEVATION 471' ± MSL SHEET 1 OF 1

METHOD OF DRILLING 4" Solid Flight, B-24 Truck Mounted (California Geotech)

DRIVE WEIGHT 140 lbs (cathead) DROP 30 inches

SAMPLED BY DHL LOGGED BY SSA REVIEWED BY RPM/PCC

Total depth = 26.5 feet.

Qs is Static Cone Penetration Test, resistance in kg/cm2.

Backfilled with neat cement and patched with cold asphalt shortly after drilling.

Notes:

Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents (Google, 2023).

FIGURE B- 4

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

DATE DRILLED	3/12/2008	BORING NO.	B-5
GROUND ELEVATION	472' ± MSL	SHEET	1 OF 2
METHOD OF DRILLING	8" Hollow-Stem Auger, Mobile B-53		
DRIVE WEIGHT	140 lbs	DROP	30 inches
SAMPLED BY	KAG	LOGGED BY	SSA
REVIEWED BY	RPM/PCC		

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							ASPHALT CONCRETE: Approximately 3 inches thick.
							AGGREGATE BASE: Approximately 6.5 inches thick.
	X		20			CL	TOPSOIL: Dark brown, damp, stiff to very stiff CLAY; little fine sand.
		X	15			CL	ALLUVIUM: Brown, damp, stiff CLAY; some fine to medium sand.
		X	30				Very stiff; little fine sand.
		X	26				
10							
		X	42				Hard; some fine sand; scattered caliche.
		X	64				
20							
						SC	Brown, moist, very dense, fine SAND; some clay; scattered caliche.
		X	92				
30							
						CL	Brown, moist, hard, CLAY; some fine sand; scattered caliche.
		X	92				
40							

FIGURE B- 5

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Geotechnical & Environmental Sciences Consultants

LAS POSITAS COLLEGE - STEAM BUILDING
3000 CAMPUS HILL DRIVE, LIVERMORE, CALIFORNIA

401294038 | 11/23

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
40							CL	ALLUVIUM(Continued): Brown, moist, hard, CLAY; some fine sand; scattered caliche.
				▼			SP-SC	Brown, saturated, very dense, SAND; few clay.
		■	63					
50								Total depth = 50.0 feet. Backfilled with neat cement and patched with cold asphalt shortly after drilling. Notes: Groundwater was measured at a depth of approximately 46.5 feet in the borehole shortly after completion of drilling. Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents (Google, 2023).
60								
70								
80								

DATE DRILLED 3/12/2008 BORING NO. B-5

GROUND ELEVATION 472' ± MSL SHEET 2 OF 2

METHOD OF DRILLING 8" Hollow-Stem Auger, Mobile B-53

DRIVE WEIGHT 140 lbs DROP 30 inches

SAMPLED BY KAG LOGGED BY SSA REVIEWED BY RPM/PCC

APPENDIX C

Laboratory Testing

APPENDIX C

LABORATORY TESTING

Classification
Soil was classified using visual-manual procedures (ASTM D 2488). Soil classifications were updated in accordance with the Unified Soil Classification System (USCS) and ASTM D 2487 based on the results of laboratory tests to evaluate particle size characteristics Atterberg Limits. Soil classifications are indicated on the log of the exploratory boring in Appendix B.

Moisture Content and In-Place Density
The moisture content of samples obtained from the borings was evaluated in accordance with ASTM D 2216 by drying the samples in an oven at 110±5 degrees Celsius. The dry density of relatively undisturbed samples obtained from the borings was evaluated in accordance with ASTM D 2937. The test results are presented on the logs of the borings in Appendix B.

200 Wash Analysis
An evaluation of the percentage of particles finer than the No. 200 sieve in selected samples was performed in accordance with ASTM D 1140. The test results are presented on Figure C-1.

Gradation Analysis
Gradation analysis tests were performed on selected soil samples in accordance with ASTM D 422. The grain-size distribution curves are shown on Figures C-2 through Figure C-4. These test results were utilized in evaluating the soil classifications in accordance with the USCS.

Atterberg Limits
Tests were performed on selected samples to evaluate the liquid limit, plastic limit, and plasticity index in accordance with ASTM D 4318. These test results were used to evaluate the soil classification in accordance with the USCS. The test results and classifications are shown on Figure C-5.

Direct Shear Test
A direct shear test was performed on an undisturbed sample in accordance with ASTM D 3080 to evaluate the shear strength characteristics of the selected material. The sample was inundated during shearing to represent adverse field conditions. The results are shown on Figure C-6.

Consolidation Test
A consolidation test was performed on a relatively undisturbed soil sample in accordance with ASTM D 2435. The sample was inundated during testing to represent adverse field conditions. The percent of consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The results of the test are summarized on Figure C-7.

Expansion Index Tests
The Expansion Index was evaluated for selected materials in accordance with ASTM D 4829. The specimens were molded under a specified compactive energy at approximately 50 percent saturation (plus or minus 1 percent). The prepared 1-inch thick by 4-inch diameter specimens were loaded with a surcharge of 144 pounds per square foot and inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The test results are presented on Figure C-8.

Unconfined Compression Test
Unconfined compression tests were performed on relatively undisturbed samples in accordance with ASTM D 2166. The test results are shown on Figure C-9.

SAMPLE LOCATION	SAMPLE DEPTH (ft)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	USCS (TOTAL SAMPLE)
B-3	5.5-6.0	Lean CLAY with sand	100	85	CL
B-4	5.5-6.0	Lean CLAY with sand	100	79	CL
B-4	11.0-11.5	Lean CLAY with sand	100	79	CL

PERFORMED IN ACCORDANCE WITH 1140

NO. 200 SIEVE ANALYSIS TEST RESULTS

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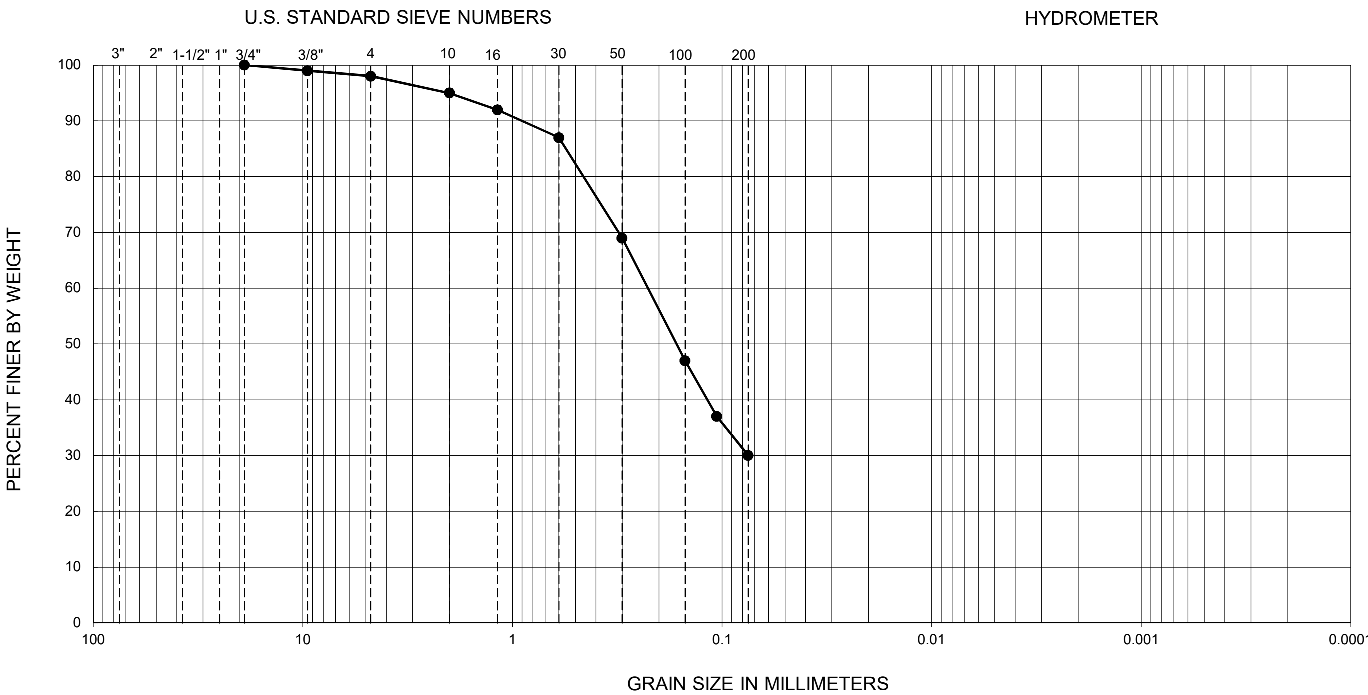
LAS POSITAS COLLEGE - STEAM BUILDING
3000 CAMPUS HILL DRIVE, LIVERMORE, CALIFORNIA

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-1	5.5-6.0	--	--	--	--	--	0.24	--	--	30	SC

PERFORMED IN ACCORDANCE WITH ASTM D 422 / D6913

Soak Time: 2.0

Group Name: Clayey SAND

% Gravel 2

% Sand 68

% Fines 30

FIGURE C-2

GRADATION TEST RESULTS

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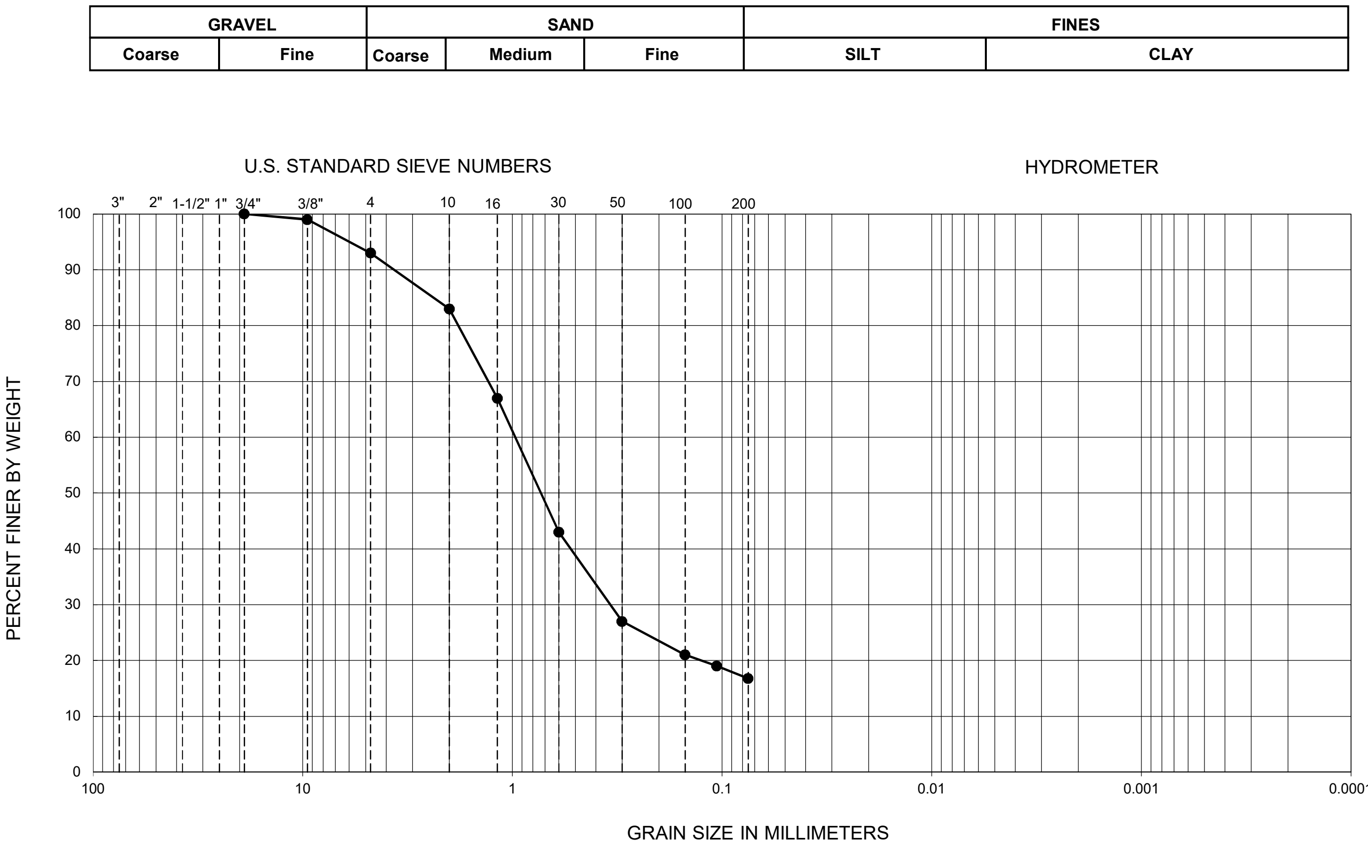
LAS POSITAS COLLEGE - STEAM BUILDING
3000 CAMPUS HILL DRIVE, LIVERMORE, CALIFORNIA



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-1	11.0-11.5	--	--	--	--	0.36	1.01	--	--	17	SC

PERFORMED IN ACCORDANCE WITH ASTM D 422 / D6913

Soak Time: 2.1

Group Name: Clayey SAND

% Gravel 7

% Sand 76

% Fines 17

FIGURE C-3

GRADATION TEST RESULTS

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LAS POSITAS COLLEGE - STEAM BUILDING
3000 CAMPUS HILL DRIVE, LIVERMORE, CALIFORNIA

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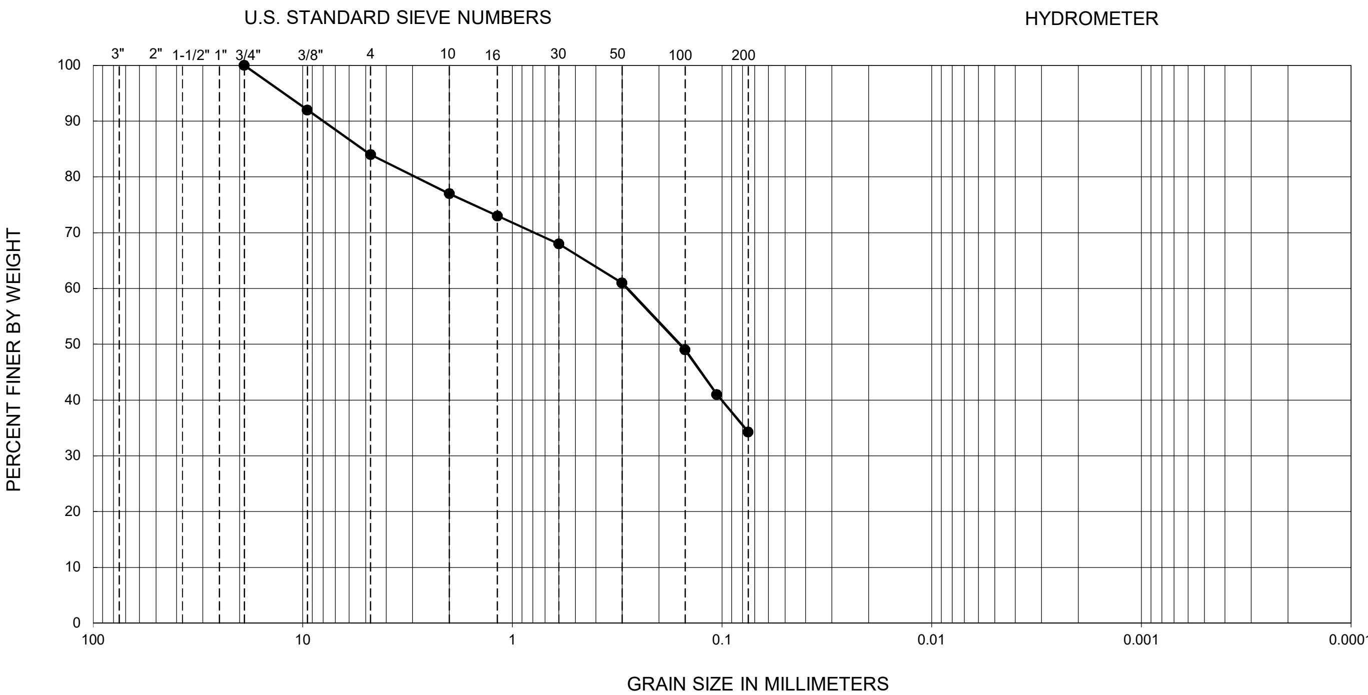
GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-3	21.0-21.5	--	--	--	--	--	0.29	--	--	34	SC

PERFORMED IN ACCORDANCE WITH ASTM D 422 / D6913

Soak Time: 2.2

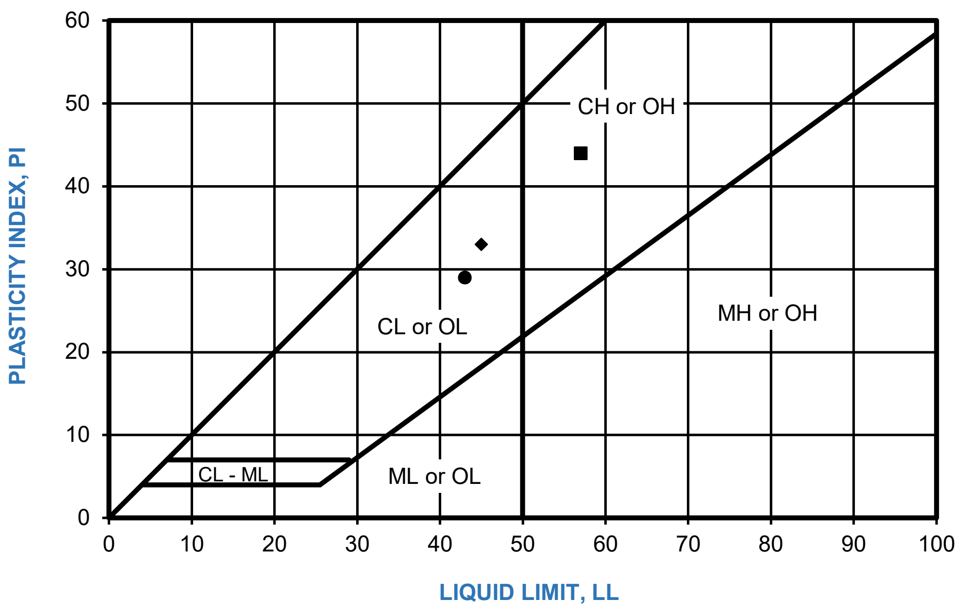
Group Name: Clayey SAND with gravel

% Gravel 16

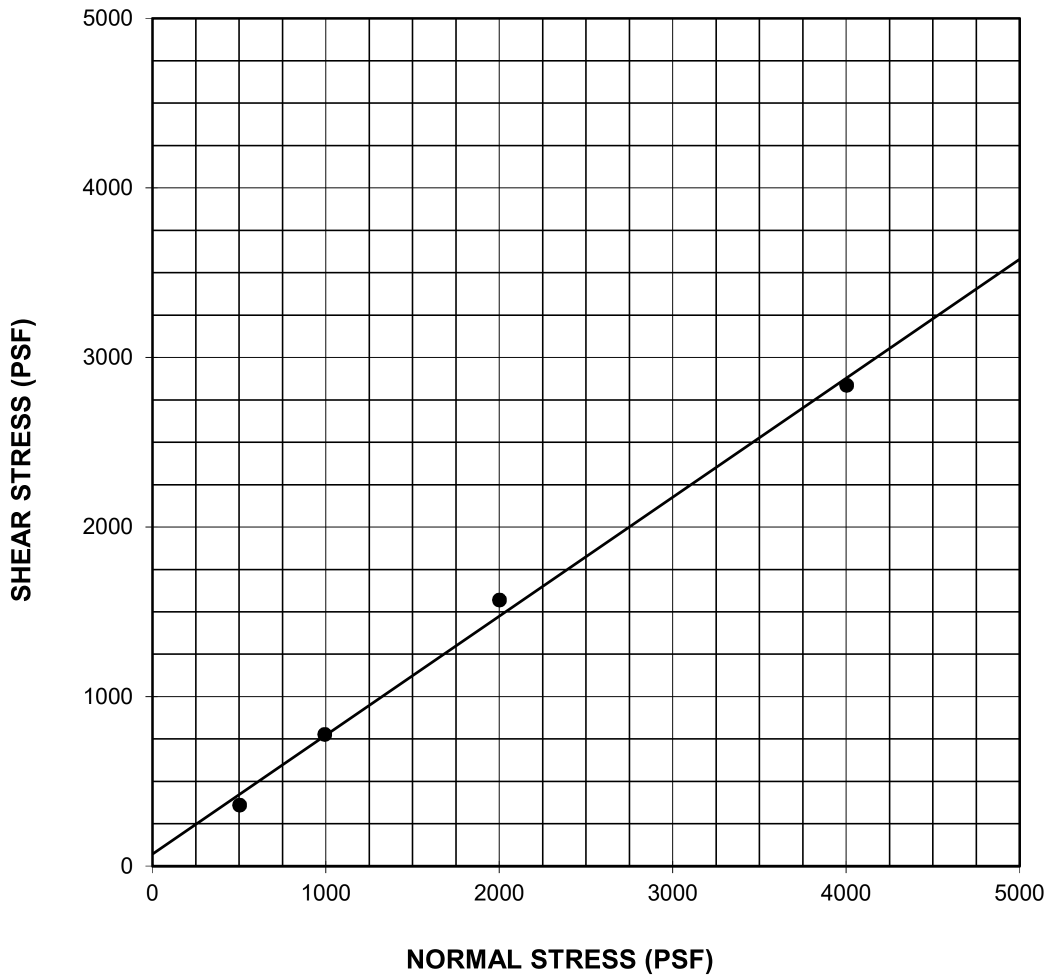
% Sand 50

% Fines 34

SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS
●	B-3	5.5-6.0	43	14	29	CL	CL
■	B-4	5.5-6.0	57	13	44	CH	CH
◆	B-4	11.0-11.5	45	12	33	CL	CL



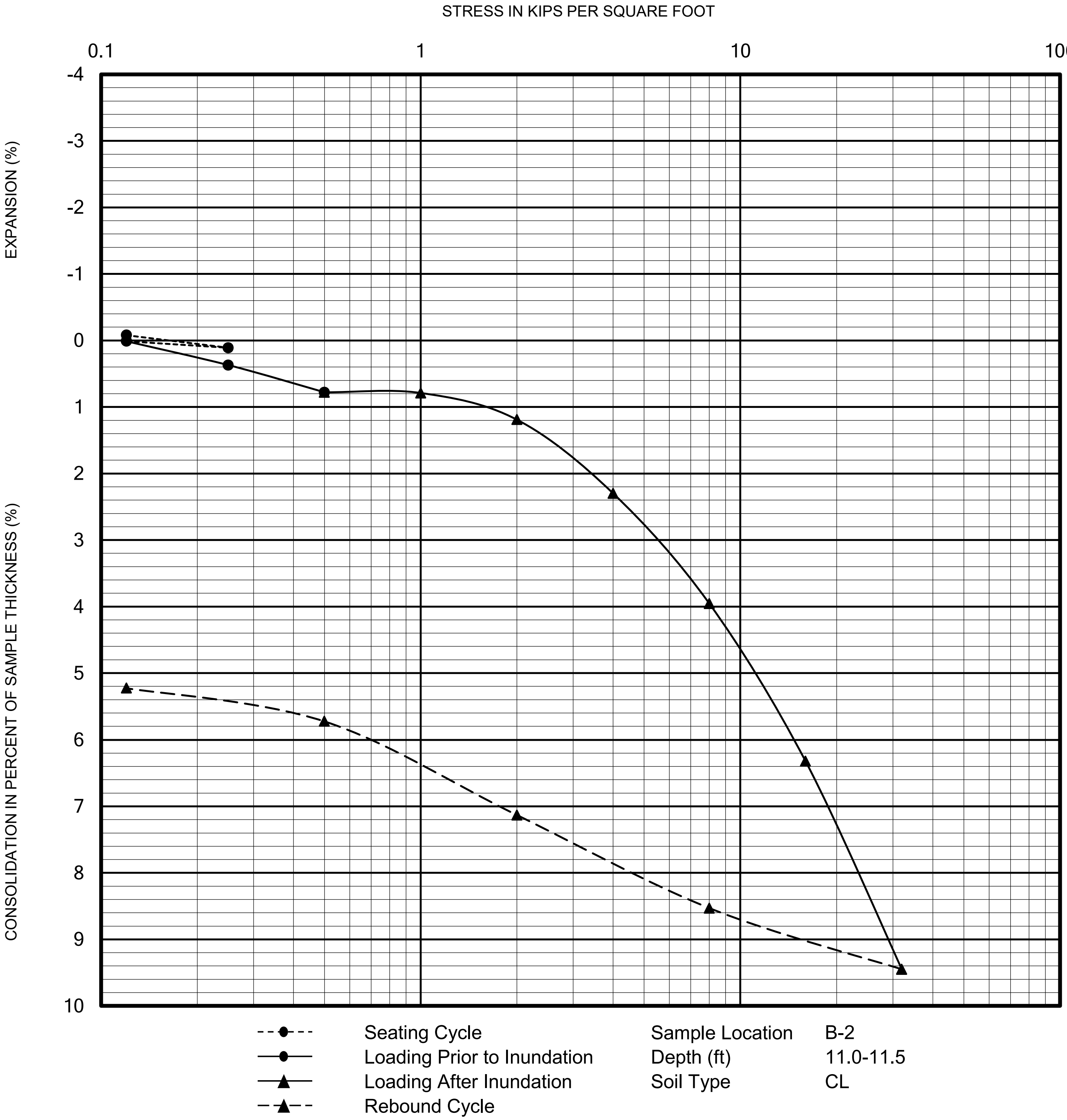
PERFORMED IN ACCORDANCE WITH ASTM D 4318



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
Clayey SAND	—●—	B-1	6.0-6.5	Peak	70	35	SC

PERFORMED IN ACCORDANCE WITH ASTM D 3080

DIRECT SHEAR TEST RESULTS



Sample Location	B-2
Depth (ft)	11.0-11.5
Soil Type	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

FIGURE C-7

CONSOLIDATION TEST RESULTS

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3000 CAMPUS HILL DRIVE, LIVERMORE, CALIFORNIA

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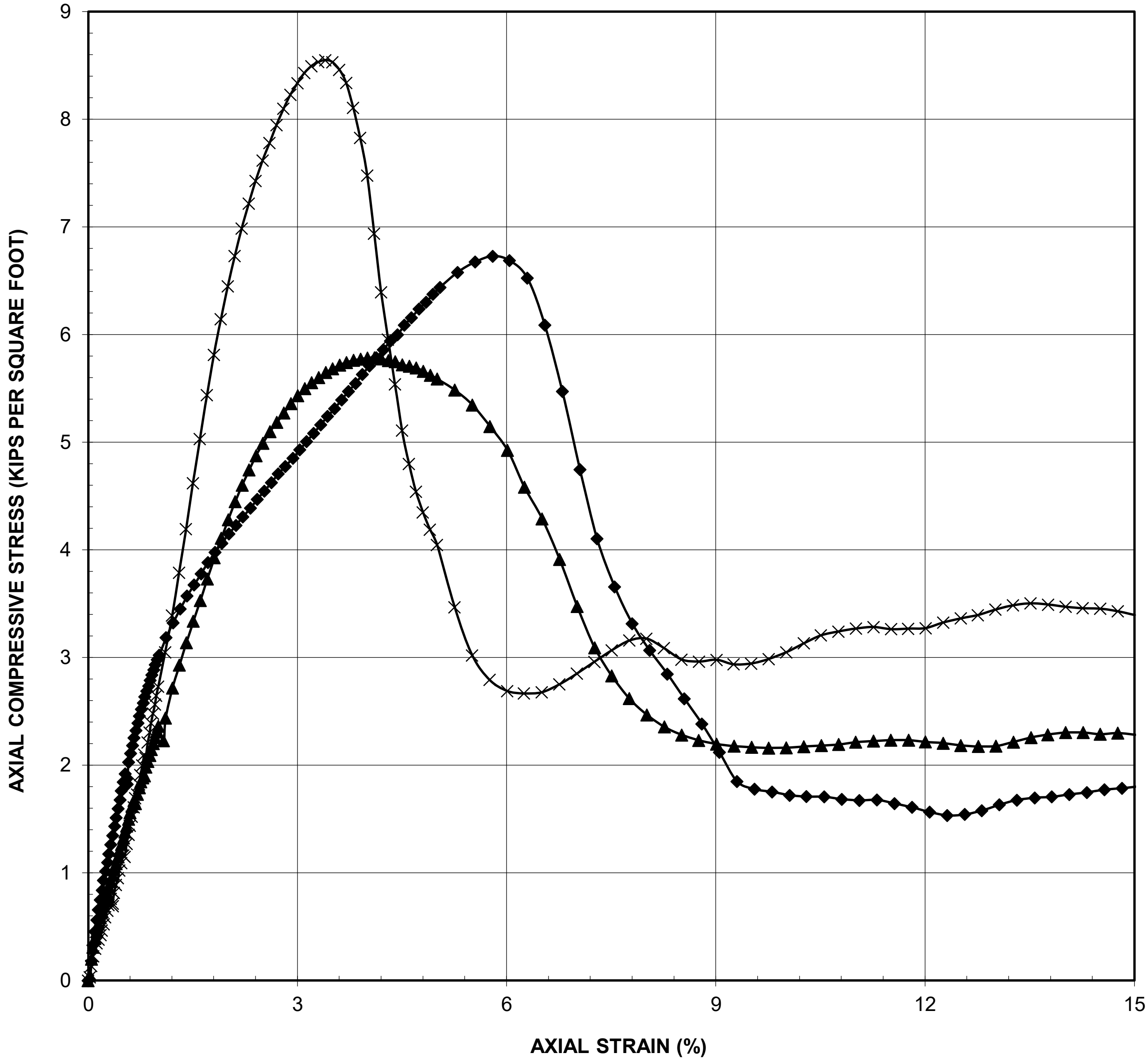
SAMPLE LOCATION	SAMPLE DEPTH (ft)	INITIAL MOISTURE (percent)	COMPACTED DRY DENSITY (pcf)	FINAL MOISTURE (percent)	VOLUMETRIC SWELL (in)	EXPANSION INDEX	POTENTIAL EXPANSION
B-1	0.0-5.0	11.3	103.7	23.3	0.043	43	Low
B-2	0.0-5.0	13.5	95.7	32.2	0.113	113	High
B-3	0.0-5.0	8.7	114.5	21.5	0.069	69	Medium
B-4	0.0-5.0	13.0	98.7	29.8	0.092	92	High
B-5	0.0-3.0	13.1	102.9	27.7	0.064	64	Medium

PERFORMED IN ACCORDANCE WITH ASTM D 4829



SYMBOL	DESCRIPTION	SOIL TYPE	SAMPLE LOCATION	SAMPLE DEPTH (ft.)	MOISTURE CONTENT w, (%)	DRY DENSITY γ_d, (pcf)	STRAIN RATE (%/min.)	UNDRAINED SHEAR STR s_u, (ksf)
◆	Clayey SAND	SC	B-2	6.0-6.5	16.0	117.8	1.00	3.36
▲	Lean CLAY with sand	CL	B-3	6.0-6.5	19.3	108.3	1.00	2.89
X	Fat CLAY	CH	B-4	6.0 - 6.5	20.9	107.1	1.00	4.28

PERFORMED IN ACCORDANCE WITH ASTM D 2166

FIGURE C-9

UNCONFINED COMPRESSION RESULTS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

LAS POSITAS COLLEGE - STEAM BUILDING
3000 CAMPUS HILL DRIVE, LIVERMORE, CALIFORNIA

401294038 | 11/23

APPENDIX D

Corrosivity Testing (CERCO Analytical)

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

20 October, 2023

Job No. 2310031
Cust. No.13270

Mr. Rathna Mothkuri
Ninyo & Moore
2149 O'Toole Avenue, Suite 30
San Jose, CA 95131

Subject: Project No.: 401294038
Project Name: Las Positas College, Steam Building, 300 Campus Hill Drive, Livermore, CA
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Mothkuri:

Pursuant to your request, CERCO Analytical has analyzed the soil sample submitted on October 17, 2023. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, 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 22 mg/kg and is determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentration is 27 mg/kg and is determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The pH of the soil is 8.59, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials 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.

for

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

JDH/jdl
Enclosure

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client: Ninyo & Moore
Client's Project No.: 401294038
Client's Project Name: Las Positas College-Steam Building, 300 Campus Hill Drive, Livermore, CA
Date Sampled: 6-Oct-23
Date Received: 17-Oct-23
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 20-Oct-2023

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)*
2310031-001	B-2/0.0-5.0'	190	8.59	-	540	-	22	27

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	-	-	-	15	15
Date Analyzed:	17-Oct-2023	18-Oct-2023	-	19-Oct-2023	-	18-Oct-2023	18-Oct-2023

* Results Reported on "As Received" Basis

Julia Clauson
Chemist

Quality Control Summary - All laboratory quality control parameters were found to be within established limits

APPENDIX E

Infiltration Test Results

Project =	LPC STEAM BUILDING	
Project No. =	401294038	
Test Location =		P-1
Liquid Use =	Tap Water	
Liquid level maintained using = Marriote Tubes		
Penetration of Rings (in) =	Inner: 3 3/4	Outer: 4
Historic Depth to Water Table =		30 feet
Date =	10/20/2023	
Tested By =	DHL	

Constants	Area (cm^2)	Depth to Liquid (cm)	Conversion 1 Div = # (cc)
Inner Ring	81	15.24	5.352
Annular Space	101	15.24	16.753

Average Infiltration Rate Inner Ring (in/hr)	0.3

Test No. (Hole No.)	Time (hr:min)	Elapsed Time (hr)	Flow Readings				Incremental Infiltration Rate	
			Inner Reading		Annular Space			
			Reading (cm)	Flow (cm^3)	Reading (cm)	Flow (cm^3)	Inner (in/hr)	Annular (in/hr)
P-1 (at Surface) [Lean CLAY]	1:39 PM	0.25	57.60	32.11	58.20	83.77	0.6	1.3
	1:54 PM		57.00		57.70			
	1:54 PM	0.25	57.00	16.06	57.70	16.75	0.3	0.3
	2:09 PM		56.70		57.60			
	2:09 PM	0.25	56.70	16.06	57.60	16.75	0.3	0.3
	2:24 PM		56.40		57.50			
	2:24 PM	0.25	56.40	16.06	57.50	16.75	0.3	0.3
	2:39 PM		56.10		57.40			
	2:39 PM	0.5	56.10	32.11	57.40	33.51	0.3	0.3
	3:09 PM		55.50		57.20			
	3:09 PM	0.5	55.50	32.11	57.20	33.51	0.3	0.3
	3:39 PM		54.90		57.00			

FIGURE E-1

INFILTRATION TEST RESULTS

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Geotechnical & Environmental Sciences Consultants

LAS POSITAS COLLEGE - STEAM BUILDING
3000 CAMPUS HILL DRIVE LIVERMORE, CALIFORNIA

APPENDIX F

Geophysical Survey

APPENDIX F

GEOPHYSICAL SURVEY

Scope

Ninyo & Moore performed a geophysical survey at the site using the refraction microtremor (ReMi) technique. The survey was performed on October 16, 2023 along one line. The survey line location is noted on Figure 2 of the report.

Refraction Microtremor Survey

The ReMi method was used to develop a one-dimensional profile of shear wave velocity and evaluate the characteristic shear wave velocity to a depth of 100 feet below the ground surface (V_{s100}) for seismic site classification. The survey consisted of collecting microtremor array measurements (MAM) from surface waves using ambient noise as a passive source and an array of geophones along one seismic lines. The following sections provide a summary of the methods and analyses used in the ReMi survey. The seismic model results are provided on Figure F-1.

Field Methods

A Geode 24–Channel Seismograph (Geometrics Inc., San Jose, CA) was used for MAM surveying, with 4.5 Hertz (Hz) vertical component geophone placement every 10 feet for a total profile length of 230 feet. Approximately twenty records were collected, with a record length of 30 seconds (s) and 2 millisecond (ms) sample interval. The field data were digitally recorded in SEG2 format, reviewed in the field for data quality, saved to a hard disk, and documented.

Data Processing and Modeling

The MAM seismic data were processed using SeisImager (Geometrics Inc., San Jose, CA) seismic processing software. The dispersive characteristics of surface waves are used to evaluate the subsurface velocity at depth. Longer wavelength (that is, longer-period and lower-frequency) surface waves travel deeper and thus contain more information about deeper velocity structure. Shorter wavelength (that is, shorter-period and higher-frequency) surface waves travel shallower and thus contain more information about shallower velocity structure. The dispersion is dependent on the material properties, such as surface wave velocity, relative material densities, and Poisson's ratio. An inversion is performed on the collected passive seismic shear wave records within SeisImager to produce a model of the variation in shear wave velocities with depth. The following data processing flow was used to calculate Average Shear-wave Velocities (AVS) to a depth of approximately 100 feet (V_{s100}).

- Collated records into list file and edited any bad channels or records,
- Applied 2D Spatial Auto Correlation (SPAC); using a linear array and 24 geophones at 10 feet spacing,
- Phase velocity frequency transformation from 2 to 20 Hz,
- Automated velocity picks of raw phase velocity were calculated and updated manually,
- Created an initial model and carried out a non-linear Least Squares Method (LSM) inversion to produce a final shear wave velocity model; convergence of the inversion was judged whether the model achieved an RMS <5% within 5-7 iterations,
- Calculated V_s100 using final shear wave velocity model.

Results

Shear wave data resolution generally decreases with depth, due to the loss of sensitivity of the dispersion curve to changes in shear wave velocity as depth increases. A figure showing our MAM seismic modeling result is provided on Figure F-1. The layered model on Figure F-1 depicts our interpretation of the approximate variation in shear wave velocity with depth across the surveyed location.

The characteristic shear wave velocity (V_{s100}) at the turf field site, calculated from the model, is approximately 1,027 feet per second which is consistent with a Class D seismic site classification for stiff soil (ASCE, 2016).

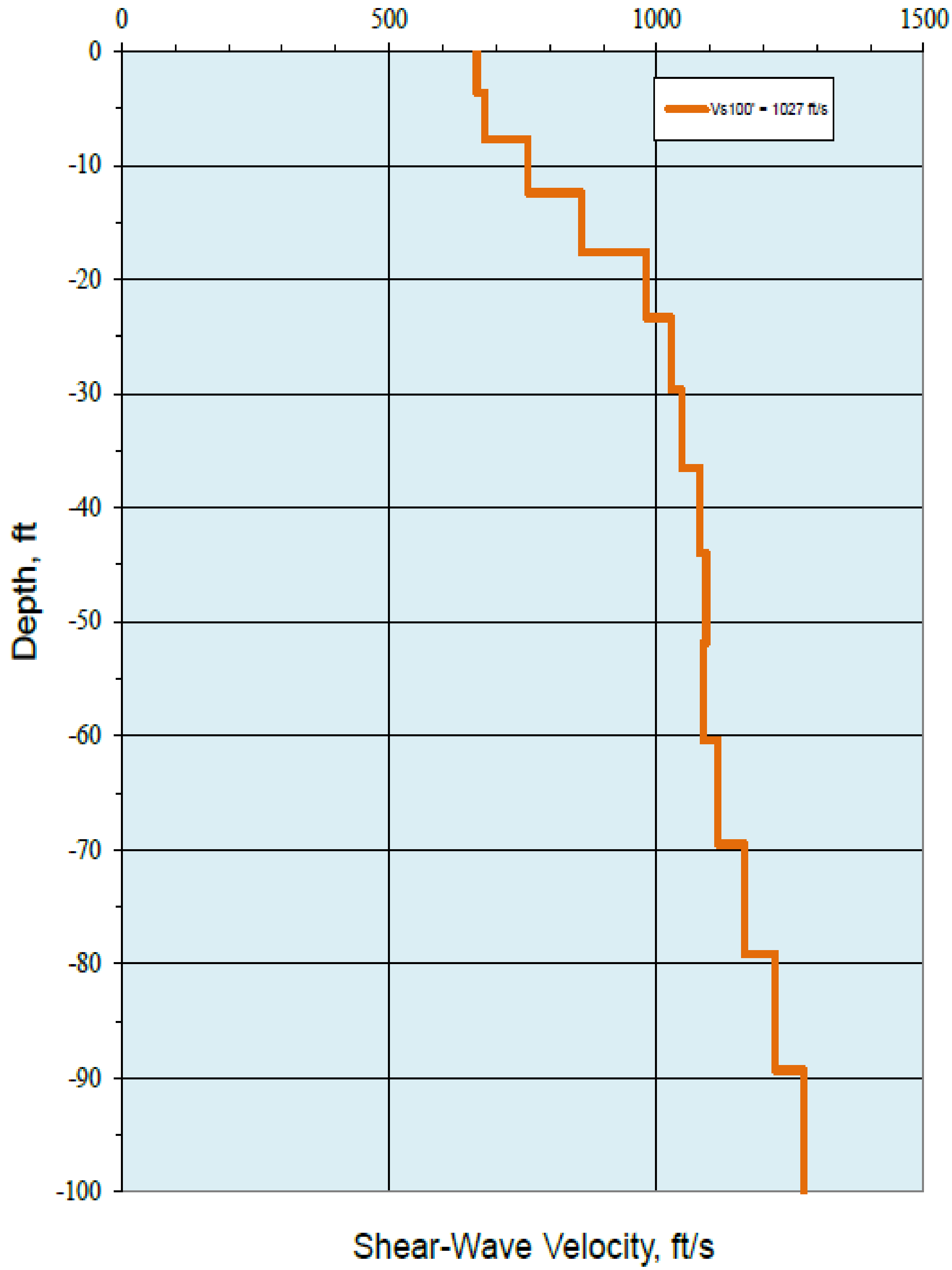


FIGURE F-1

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

SHEAR WAVE VELOCITY PROFILE

LAS POSITAS COLLEGE - STEAM BUILDING
3000 CAMPUS HILL DRIVE, LIVERMORE, CALIFORNIA

401294038 | 11/23

APPENDIX G

Calculations

Ninyo & Moore

REMI SHEAR WAVE CALCULATIONS

401294038

3000 Campus Hill Drive
Livermore, California

Line 1

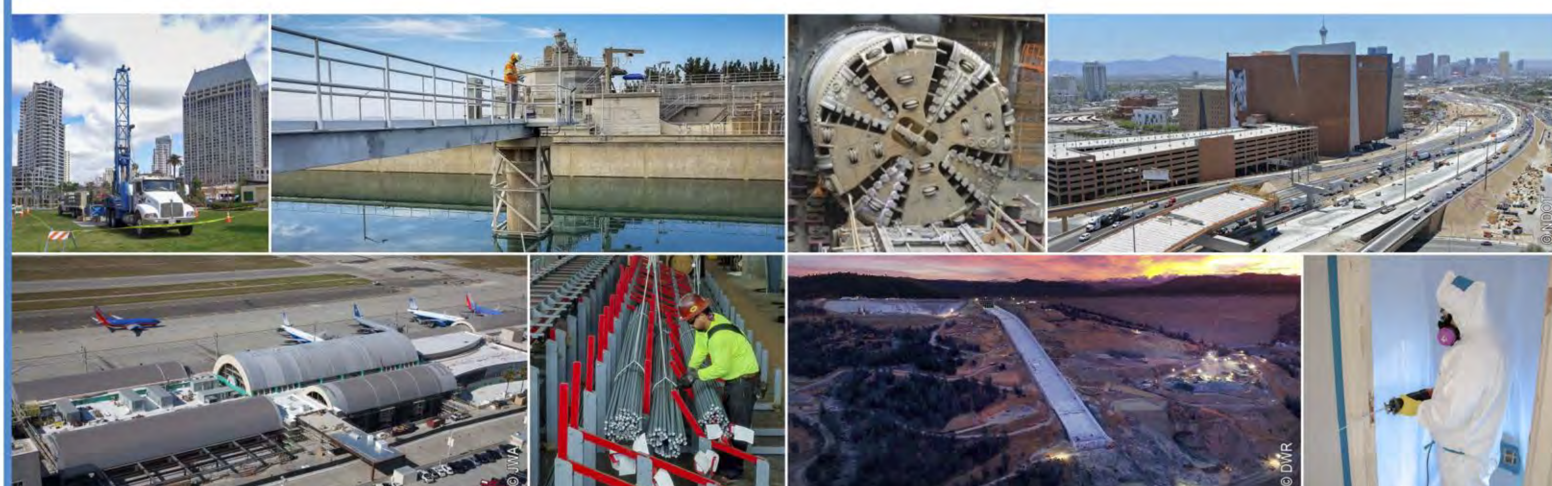
d_i	v_{si}	d_i/v_{si}
3.6	663.7	0.00538
4.1	678.6	0.00607
4.7	759.5	0.00615
5.2	859.9	0.00607
5.8	979.7	0.00589
6.3	1027.6	0.00615
6.9	1047.5	0.00656
7.4	1080.3	0.00687
8.0	1093.3	0.00729
8.5	1088.4	0.00782
9.1	1114.9	0.00813
9.6	1163.9	0.00826
10.2	1220.9	0.00833
10.7	1274.6	0.00841
	Average v_s	1027

$$\overline{v_s} = \frac{\sum d_i}{\sum \frac{d_i}{v_{si}}}$$

Per ASCE 7-22: Equation 20.4-1

d_i = thickness of any layer between 0 and 100 feet

v_{si} = shear wave velocity in feet per second



2149 O'Toole Ave, Suite 30 | San Jose, California 95131 | p. 408.435.9000

ARIZONA | CALIFORNIA | COLORADO | NEVADA | TEXAS | UTAH

ninyoandmoore.com

Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

PROFESSIONAL SERVICES AGREEMENT

THIS AGREEMENT is made and entered into this ***Date of Contract Here***, in the City of Dublin, County of Alameda, State of California, by and between CHABOT-LAS POSITAS COMMUNITY COLLEGE DISTRICT, a California Community College District, (hereinafter referred to as "DISTRICT") and ***Consultant Name Here***, Consultants (hereinafter referred to as "CONSULTANT") having its principal place of business at ***Consultant Address Here***.

WITNESSETH:

WHEREAS, DISTRICT desires to engage CONSULTANT to perform certain of the professional services, and

WHEREAS, CONSULTANT represents that it is fully qualified and willing to perform the services required hereunder, professional services for the "Materials Testing and Special Inspection Services for Temporary Faculty Village Project at Las Positas College", and

NOW THEREFORE, for and in consideration of the covenants and conditions hereinafter set forth, the parties do mutually agree as follows:

I. STATEMENT OF SERVICES
CONSULTANT hereby agrees to perform the tasks and services set forth in Exhibit "A", entitled "Statement of Services", attached hereto and made a part hereof, in accordance with the terms and conditions, sequence, time, and manner expressed herein.

II. COMPENSATION AND PAYMENT
For and in consideration of the services performed by CONSULTANT hereunder, DISTRICT agrees to pay CONSULTANT the sums set forth under Exhibit "B" entitled, " Cost of Work," attached hereto and made a part hereof.

III. TERMS AND CONDITIONS
CONSULTANT agrees to be bound by the "General Provisions for Professional Services Agreement" identified as Exhibit "C", also attached hereto and made a part hereof.

IV. TERM

The initial term of this Professional Services Agreement shall expire (9) weeks after the date upon which the DISTRICT and the CONSULTANT each execute the initial "Statement of Services" as identified in Exhibit "A", a counterpart copy hereof, deliver an executed counterpart copy hereof to the other. The Term of this Agreement may be renewed annually at the sole option and discretion of the DISTRICT ("the Compensation and Payment Terms"). Notwithstanding expiration of the Initial Term, if at such time, there are remaining Services to be performed by the CONSULTANT in connection with task assigned the CONSULTANT under this Agreement prior to expiration of the Term or Renewal Term, as applicable of this Agreement, the CONSULTANT shall continue to diligently perform and complete all such remaining services; notwithstanding expiration of the Term or Renewal Term of this Agreement, the DISTRICT will continue to make payment for the services performed in connection with such assigned tasks in accordance with the terms of the Project Assignment. If the DISTRICT elects to renew the Term for any Renewal Terms, the terms and conditions set forth herein shall be applicable and govern during the Renewal Term(s).

IN WITNESS WHEREOF, the authorized representatives of the parties hereto have executed this Agreement effective on the date first written above.

"DISTRICT" CHABOT-LAS POSITAS COMMUNITY COLLEGE DISTRICT	**"CONSULTANT" TBD**
By:_________________________ ________ Date	By: ________________________ ________ Date
Vice Chancellor, Business Services	Print Name:______________________
	Title:___________________________

EXHIBIT "A"
STATEMENT OF SERVICES

1. CONSULTANT represents that it has the expertise, experience, personnel, and resources to perform the desired services. The CONSULTANT further represents that CONSULTANT and all personnel engaged to provide/perform services hereunder are and shall remain fully qualified and authorized, permitted and/or licensed under applicable law or regulations to perform such services. None of the work or services shall be subcontracted without the prior written approval of DISTRICT.

2. CONSULTANT will perform or cause to be performed those services described below in accordance with all laws, regulations, and applicable codes and with the provisions of this agreement. CONSULTANT shall use its best efforts to conduct the services in an expeditious and timely manner. All services hereunder shall be provided/performed in accordance with the standard of care for consultants providing/performing similar services.

3. A written definition of the Services to be performed by the CONSULTANT is set forth below and Scope of Services as defined in **Attachment 1** – dated ***"Enter Date Here"*** herein by this reference made a part of this Agreement.

4. No other terms and conditions shall apply other than as specified in Exhibit "C", Section 17, "Extent of Agreement."

EXHIBIT "B"
COMPENSATION AND PAYMENT

1. For and in consideration of the performance and completion of the services hereunder, DISTRICT agrees to pay CONSULTANT for a LUMP SUM fee of ***Enter Fee Here*** Dollars ($00,000.00) as referenced in the "Proposal for Materials Testing and Special Inspection Services" as defined in Attachment 1.

2. A written definition of the compensation to be paid to the CONSULTANT will be as stipulated in those subsequent Consulting Assignment(s) issued by the District to the Consultant, pursuant to Paragraph 3, of Exhibit "A", above. Subject to the specific terms and conditions of any subsequent Consulting Assignment(s)/Task Order issued to the CONSULTANT, payment of fees will be on a Time and Materials/Fixed Price basis, inclusive of all related expenses.

   Once each month, CONSULTANT shall submit an invoice for services rendered during the previous month. Consultant invoice is to include the District Purchase Order number which will be provided independently by the District. Within thirty (30) days DISTRICT shall promptly pay CONSULTANT the amount due. If the consultant fails to timely and fully perform material obligations of the Consultant hereunder, notwithstanding any provision of the Agreement to the contrary, the DISTRICT may withhold from any amount due the CONSULTANT, with the withheld amounts being disbursed to the CONSULTANT after the CONSULTANT has fully cured such failure to performance, less costs, expenses, losses or damages sustained by the DISTRICT as a result of such failure to performance.

3. CONSULTANT shall not perform any additional service, or incur any additional expense in the performance of this Agreement without the prior written approval of DISTRICT.

4. DISTRICT shall not be responsible for payment or reimbursement of monies for additional services performed without the prior written approval of DISTRICT.

5. Should a change of scope or additional services be required, payment for such services will be determined at the time of DISTRICT's written approval, and such shall be amended to this Agreement.

6. DISTRICT will not be responsible for reimbursement for costs invoiced more than 90 days after the costs were incurred.

EXHIBIT "C"
GENERAL PROVISIONS FOR
PROFESSIONAL SERVICES AGREEMENT

1. Responsibility
CONSULTANT shall be solely responsible for the professional quality, technical accuracy and the coordination of all designs, drawings, specifications, calculations, data, reports or other Services to be provided hereunder, and shall, without any additional compensation, correct or revise any errors or deficiencies promptly upon notice or discovery thereof, provided that the CONSULTANT'S obligation to correct or revise errors/discrepancies in the services provided is in addition to and not in lieu of the consultant's liability to the DISTRICT for losses, costs, expenses or damages sustained by the DISTRICT as a result of such errors/deficiencies. Neither a review, approval or acceptance of, nor payment for, any of the Services required hereunder shall be construed as a waiver of any rights under this Agreement by DISTRICT or of any cause of action arising out of the performance of this Agreement, and CONSULTANT shall be liable for all damages caused by or arising out of CONSULTANT'S negligent performance of any Services provided or required hereunder.

2. Changes
DISTRICT may, upon ten (10) days written notice, make changes in the Scope of Services to be provided hereunder. If such changes result in an increase or a decrease in Services, the time required to performance thereof, or the compensation thereof, this Agreement shall be modified accordingly in writing in order for such changes to be valid.

3. Termination
A. Performance of the work and Services hereunder may be terminated by DISTRICT at any time, in whole or in part:

(1) Whenever CONSULTANT shall default in its obligations hereunder or fails to make progress in the prosecution of the work or Services; or
(2) For the convenience of DISTRICT.

B. Termination shall be effected by delivery to CONSULTANT of the Notice of Termination, specifying whether said termination is for default of CONSULTANT or for the convenience of DISTRICT, the extent to which performance of the work and Services is terminated; and the date upon which said termination is to become effective. If, after Notice of Termination for default, it is determined that CONSULTANT was not in default, or that CONSULTANT 's failure to fulfill its obligations was due to causes beyond its control and without its fault or negligence, the Notice of Termination shall be deemed to have been issued for the convenience of DISTRICT.

C. Following receipt of Notice of Termination, CONSULTANT shall discontinue performance on the date and to the extent specified therein, and deliver to DISTRICT the completed or partially completed plans, information, data, reports, estimates, summaries, materials, or other documents which, if performance had been completed, would be furnished to DISTRICT. CONSULTANT shall continue performance of such part of the work and Services which are not terminated by the Notice of Termination. CONSULTANT shall prepare and submit a termination claim for services satisfactorily performed, which shall include costs and expenses, reimbursable in accordance with the Terms of this Agreement, not previously paid to CONSULTANT, incurred prior to the effective date specified in the Notice of Termination, and DISTRICT may agree upon the whole or any part of the amount(s) claimed by CONSULTANT on account of the termination or partial termination.

D. In the event of termination for default, DISTRICT shall be entitled to complete the work and Services hereunder or engage others to do so and in addition to whatever remedies it may have at law if the expense of completing said work and Services is greater than the amount CONSULTANT was to receive as compensation therefore, DISTRICT shall be entitled to recover the difference from CONSULTANT.

4. <u>Confidentiality</u>

CONSULTANT hereby agrees that all information provided by DISTRICT relating to the Services hereunder shall be considered confidential and proprietary, and shall not be reproduced, transmitted, used or disclosed by the CONSULTANT without the written consent of DISTRICT, except as may be necessary for the non-disclosing party to fulfill its obligations hereunder; provided that the limitation shall not apply to any information or portion thereof, which is within the public domain at the time of its disclosure. The requirements of this provision shall survive the term of this Agreement.

5. <u>Ownership and Reuse of Documents</u>

All non-proprietary data, information, reports, drawings, renderings, or other documents or materials prepared by CONSULTANT hereunder shall become the property of DISTRICT whether or not the work covered thereby is executed; provided that CONSULTANT may at the CONSULTANT'S cost and expense reproduce such items to retain as a record copy for its files.

6. <u>Relationship</u>

The legal relationship of CONSULTANT to DISTRICT hereunder shall be that of an independent contractor and not that of an agent or employee.

7. Examination of Records

If the Services performed by CONSULTANT hereunder are in support of any government contract or program, or under a cost reimbursable type agreement, or for any authorized additional service or reimbursable expense, CONSULTANT shall until the expiration of six (6) years after final payment hereunder, maintain such books and records under generally recognized accounting methods and permit inspection by DISTRICT or any of its authorized representatives.

8. Compliance with Laws

CONSULTANT shall comply with all applicable federal, state, and local laws, ordinances, rules, regulations, and orders in effect throughout the term of this Agreement, including, but not limited to Executive Order No. 11246 of September 24, 1965, as amended (regarding Equal Employment Opportunity), and the orders of the Secretary of Labor pursuant thereto.

9. Insurance

Prior to commencing work, the CONSULTANT shall procure and maintain at CONSULTANT'S own cost and expense for the duration of this Agreement the following insurance against claims which may arise from or in connection with the performance of the work or services hereunder by the CONSULTANT, its agents, representatives, employees or sub consultants.

A. Minimum Limits of Insurance.

CONSULTANT shall maintain limits of no less than:

(1) Commercial General Liability

Two Million Dollars ($2,000,000) combined single limit per occurrence for bodily injury and property damage. Coverage shall be provided on an "occurrence" basis.

(2) Comprehensive Automobile Liability Insurance:

One Million Dollars ($1,000,000) combined single limit per accident for bodily injury or property damage. The following coverages shall be included:

(a) Owned Automobiles.

(b) Hired Automobiles.

(c) Non-Owned Automobiles.

(3) Professional Liability Errors and Omissions Insurance: With a limit of not less than One Million Dollars ($1,000,000).

(4) Workers' Compensation and Employer's Liability: Workers' compensation limits as required by the Labor Code of the State of California and Employer's Liability limits of One Million Dollars ($1,000,000) per accident.

B. <u>Deductibles and Self-insured Retentions</u>. Any deductibles or self-insured retentions must be declared to and approved by the DISTRICT. At the option of the DISTRICT, the insurer shall reduce or eliminate such deductibles (limited to general and automobile liability insurance only) or self-insured retentions with respect to the DISTRICT, its officials and employees, or the CONSULTANT shall procure a bond guaranteeing payment of losses and related investigation, claim administration, and defense expenses.

C. Other Insurance Provisions

(1) <u>General Liability and Automobile Liability Coverages Only</u>:

(a) The DISTRICT, members of its boards and commissions, officers, and employees are to be covered as insureds as respects: liability arising out of activities performed by or on behalf of the CONSULTANT; premises owned, leased, or used by the CONSULTANT; and premises on which CONSULTANT is performing services on behalf of the DISTRICT. The coverage shall contain no special limitations on the scope of protection afforded to the DISTRICT, members of its boards and commissions, officers, and employees.

(b) The CONSULTANT'S insurance coverage shall be primary insurance as respects the DISTRICT, members of its boards and commissions, officers, and employees. Any insurance or self-insurance maintained by the DISTRICT, its officials, and employees, shall be in excess of Consultant's insurance and shall not contribute with it.

(c) Any failure to comply with reporting provisions of the policies shall not affect coverage provided to the DISTRICT, members of its boards and commissions, officers, or employees.

d) Coverage shall state that CONSULTANT'S insurance shall apply separately to each insured against whom a claim is made or suit is brought, except with respect to the limits of the insurer's liability.

(2) <u>Workers' Compensation and Employer's Liability Coverages</u>: The insurer shall agree to waive all rights of subrogation against the DISTRICT, members of its boards and commissions, officers, and employees for losses arising from work performed by CONSULTANT for the DISTRICT.

(3) <u>All Coverages</u>.

(a) Each insurance policy required by this Agreement shall be endorsed to state that coverage shall not be suspended, voided, canceled, or reduced in coverage limits except after thirty (30) days prior written notice has been given to the DISTRICT.

(b) If CONSULTANT, for any reason, fails to maintain insurance coverage which is required pursuant to this Agreement, such failure shall be deemed a material breach of this Agreement. The DISTRICT, at its sole option, may terminate this Agreement in accordance with Provision Number 14, Termination. Alternatively, the DISTRICT may purchase such required insurance and may deduct that cost from sums owed to Consultant provided CONSULTANT does not obtain the insurance itself within five (5) days of receipt of the DISTRICT'S notice of intent.

(c) CONSULTANT agrees to add designated agents of the DISTRICT as additional insured under the above policies as mutually agreed.

D. Acceptability of Insurers.
Insurance is to be placed with insurers rated A: 6 or better by A.M. Best's rating-service.

E. Verification of Coverage.
CONSULTANT shall furnish the DISTRICT with written evidence acceptable to the DISTRICT of insurance and minimum coverage amounts required by this Agreement.

F. Sub-consultants.
Prior to authorizing work by a Sub-consultant to proceed, CONSULTANT shall provide to the DISTRICT evidence acceptable to the DISTRICT of insurance demonstrating satisfactory compliance by each Sub-consultant with the insurance requirements stated herein.

10. Indemnity
To the fullest extent permitted by law, the CONSULTANT shall indemnify, defend and hold harmless the District and its employees, officers, Board of Trustee, Trustees, agents and representatives from any and all claims, demands, losses, responsibilities or liabilities for: (i) injury or death of persons; (ii) damage to property or: (iii) other costs or charges, directly or indirectly arising out of or attributable, in whole or in part, to the negligent or willful acts, omissions, errors and/or other conduct of CONSULTANT, its Design Consultants or the employees, agents and representatives of CONSULTANT or any of its Design Consultants in the performance of obligations or services or in providing work product under this Agreement. The foregoing shall include without limitation, attorney's fees and costs incurred by the District. The provisions hereof shall apply during the period of CONSULTANT'S performance under this Agreement and shall survive the termination of this Agreement until any such claim, demand, loss, responsibility or liability covered by the provisions hereof is barred by the applicable Statue of Limitations.

11. Remedies.
The rights and remedies set forth herein shall be in addition to any other remedies provided by law, and waiver by DISTRICT of any provision hereunder or a breach thereof by DISTRICT shall not be deemed a waiver of future compliance thereof and such provision shall continue in full force and effect.

12. Severability.
In the event that any term or provision of this Agreement is held to be illegal, invalid, or unenforceable under the laws, regulations or ordinances of any federal, state, or other government to which this Agreement is subject, such term or provision shall be deemed severed from this Agreement and the remaining terms and provisions shall remain unaffected thereby and continue in full force.

13. Notices.
All notices required or permitted under this Agreement shall be considered as duly given to any party for all purposes hereof only if given in writing and hand delivered; or sent by registered or certified mail, postage prepaid and return receipt requested; or sent by electronic email; with confirming receipt; telex, or telegram, and also confirmed by registered mail, postage prepaid and return receipt requested, addressed as set forth below, or to such other address as may be designated by notice given as provided above. All notices shall be effective upon first receipt, unless otherwise specified herein.

DISTRICT:	Chabot-Las Positas Community College District 7600 Dublin Blvd., 3rd Floor Dublin, CA 94568 Attention: Owen Letcher, Vice Chancellor Facilities/Bond Program & Operations Cc to Ann Kroll, Project Planner, Manager Facilities/Bond Program Phone # (925) 424-1863
CONSULTANT:	Consultant Name Address 1 Address 2 Contact Name Phone # Email

14. Modification.
This Agreement may only be modified by a written amendment hereto, duly executed by both parties.

15. Successors and Assignment.
CONSULTANT binds itself, its successors, assigns, and legal representatives to DISTRICT with respect to all of the covenants of this Agreement and further agrees that it shall not sell, assign, transfer, mortgage, pledge or in any manner encumber its interests in this Agreement or in any proceeds from this Agreement without the prior written consent of DISTRICT. In the event that CONSULTANT violates the foregoing prohibition, or in the event that CONSULTANT without the prior written consent of DISTRICT, which consent shall not be unreasonably withheld, sells, assigns, transfers, mortgages, pledges or in any manner encumbers, except as security for credit agreements, all or substantially all of its corporate assets, or directly or indirectly undergoes a change in control of its ownership, DISTRICT shall be entitled, at its sole option:

A. To require the CONSULTANT'S successor to continue to perform under this Agreement and to continue to satisfactorily fulfill CONSULTANT'S obligations under this Agreement; or
B. To terminate this Agreement. In such case CONSULTANT shall be responsible for any and all liabilities arising from such termination. In the event that DISTRICT replaces CONSULTANT with another consultant after such termination, CONSULTANT shall be responsible for any and all costs, expenses and liabilities arising from such substitution. In any event, CONSULTANT shall remain liable for any and all work product or services provided by it prior to the termination.

This Agreement and the terms hereof are binding upon and inure to the benefit of the successors and assigns of both the District and the CONSULTANT.

16. Disputes.
A. Continuation of Consultant Assignment. Except in the event of the District's failure to make undisputed payment of the Contract Price due the Consultant, notwithstanding any disputes between District and Consultant hereunder, Consultant and District shall each continue to perform their respective obligations hereunder; including the obligation of the Consultant to continue to provide and perform services hereunder pending a subsequent resolution of such disputes.

B. Mandatory Mediation. All claims, disputes and other matters in controversy between the Consultant and the District arising out of or pertaining to this Agreement shall be submitted for resolution by non-binding mediation conducted under the auspices of the American Arbitration Association ("AAA") and the Construction Mediation Rules of the AAA in effect at the time that a Demand For Mediation is filed. The commencement and completion of mediation proceedings pursuant to the foregoing is a condition precedent to either the District or the Consultant commencing arbitration proceedings.

C. Binding Arbitration. Claims, disputes or other matters in question between the parties to this Agreement arising out of or relating to this Agreement or breach thereof which are not resolved through the mandatory mediation procedures set forth above shall be resolved by binding arbitration conducted in accordance with the Construction Industry Arbitration Rules of the American Arbitration Association in effect at the time of the filing of a Demand for Arbitration, provided that the Parties may by mutual agreement modify such Rules or adopt other rules governing the conduct of arbitration proceedings.

D. Demand for arbitration shall be filed in writing with the other party to this Agreement and with the American Arbitration Association. A demand for arbitration shall be made within a reasonable time after the claim; dispute or other matter in question has arisen. In no event shall the demand for arbitration be made after the date when institution of legal or equitable proceedings based on such claim, dispute or other matter in question would be barred by the applicable statues of limitations.

E. No arbitration arising out of or relating to this Agreement shall include, by consolidation, joiner or in any other manner, an additional person or entity not a party to this Agreement, except by written consent containing a specific reference to the Agreement signed by the District, CONSULTANT and any other person or entity sought to be joined. Consent to arbitration involving an additional person or entity shall not constitute consent to arbitration of any claim, dispute or other matter in question not described in the written consent or with a person or entity not named or described therein. The foregoing agreement to arbitrate and other agreements to arbitrate with an additional person or entity duly consented to by the parties to this Agreement shall be specifically enforceable in accordance with applicable law in any court having jurisdiction thereof.

F. The award rendered by the arbitrator or arbitrators shall be final, and judgment may be entered upon it in accordance with applicable law in any court having jurisdiction thereof.

17. Extent of Agreement.

The Agreement and Exhibit A “Statement of Services,” Exhibit B “Compensation and Payment,” and Exhibit C,” General Provisions for Professional Services Agreement,” contain all of the promises, representations and understandings of the parties hereto and supersedes any previous understandings, commitments, proposals or agreements, whether oral or written, and may only be modified as hereinbefore provided.

18. Governing Laws.
Unless otherwise specified herein, this Agreement shall be governed by the law of the State of California.

19. Professional Registration.
If the CONSULTANT's Services under this Agreement involve the production of documents or drawings that require signing or sealing by a registered professional, CONSULTANT warrants that it has such qualified person assigned to this Project who is registered in the State(s) of California.

20. Time.
Time is of the essence in the performance and completion of the CONSULTANT'S obligations under the Agreement.

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