Geotechnical Evaluation and Geologic Hazards Assessment

Viticulture Facility – Alternative Location No. 2
3000 Campus Hill Drive

Livermore, California

Chabot Las Positas Community College District

7600 Dublin Boulevard | Dublin, California 94568

January 14, 2021 | Project No. 401294037

Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

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January 14, 2021
Project No. 401294037

Ms. Ann Kroll
Project Planner, Manager/Facilities Bond Program
Las Positas College
3000 Campus Hill Drive
Livermore, California 94551

Subject: Geotechnical Evaluation and Geologic Hazards Assessment
Viticulture Facility – Alternative Location No. 2
3000 Campus Hill Drive
Livermore, California

Dear Ms. Kroll:

In accordance with your authorization, we have performed a geotechnical evaluation and geologic hazards assessment for the proposed Viticulture Facility at Alternative Location No. 2 on the campus of Las Positas College in Livermore, California. This report presents the findings from our evaluation, the conclusions from our assessment, and our geotechnical recommendations regarding the proposed project.

As an integral part of our role as the geotechnical engineer-of-record, we request the opportunity to review the construction plans before they go to bid and to provide follow-up construction observation and testing services.

Ninyo & Moore appreciates the opportunity to be of service to you on this project.

Sincerely,
NINYO & MOORE

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CONTENTS

TABLES

FIGURES

APPENDICES

1 INTRODUCTION

In accordance with your request, Ninyo & Moore has performed a Geotechnical Evaluation and Geologic Hazards Assessment for the proposed Viticulture Facility on the campus of Las Positas College at 3000 Campus Hill Drive in Livermore, California (Figure 1). The purpose of our study was to evaluate the potential geologic hazards and geotechnical conditions at the proposed site, and provide recommendations for the design and construction of the project improvements.

In March 2020, we completed a geotechnical evaluation and geologic hazards assessment for the Viticulture Facility at another location on the campus, now designated as Alternative Location No. 1. Alternative Location No. 1 is at the southern end of the main solar array site above the intersection of Campus Hill Drive and Campus Loop Road (Ninyo & Moore, 2020a). Alternative Location No. 2, which is the focus of the current study, is east of the northeastern end of the track and field facility and south of the proposed Horticulture Facility (Ninyo & Moore, 2020b). Data from our Horticulture Facility evaluation (Ninyo & Moore, 2020b) is incorporated into our current study for the Viticulture Facility (Figure 2).

2 SCOPE OF SERVICES

Our scope of services included the following:

- Reviewed previous geotechnical reports and readily available geologic literature pertinent to the project area including geologic maps and reports, regional fault maps, seismic hazard maps, and aerial photography.
- Reviewed as-built plans and proposed site plans provided by the District.
- Site reconnaissance to observe the general site conditions and 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 to check the exploration locations for underground utility conflicts.
- Procurement of a boring permit from the Zone 7 Water Agency.
- Subsurface evaluation consisting of drilling, logging, and sampling of three solid stem auger borings. A representative of Ninyo & Moore logged the subsurface conditions exposed in the borings, and collected bulk and relatively undisturbed samples for laboratory testing. The exploratory borings were backfilled in accordance with the requirements of the Zone 7 Water Agency.

- Laboratory testing on selected soil samples to evaluate in-place soil moisture content and density, particle size distribution, Atterberg limits, unconfined compressive strength, soil corrosivity, and R-value as appropriate for the subsurface materials encountered.
- Engineering analysis of the gathered data to evaluate geotechnical considerations for the proposed improvements, including seismic parameters, liquefaction potential, foundation design criteria, and earthwork guidelines.
- Preparation of this geotechnical report presenting our findings regarding the geotechnical conditions encountered at the project site, the conclusions from our geologic hazards assessment, and our recommendations for the design and construction of the project.

3 SITE DESCRIPTION AND BACKGROUND

The Las Positas Community College campus is an irregularly-shaped parcel covering approximately 147 acres located at 3000 Campus Hill Drive in Livermore, California (Figure 1). The campus is located approximately 3,000 feet north of Highway 580 and east of Collier Canyon Road. The proposed project site is located at approximately 37.7150º north latitude and 121.7941º west longitude on the Livermore 7.5-minute topographic quadrangle (USGS, 2018).

The currently proposed location for the Viticulture Facility is east of the northeast end of the track and field facility and south of the proposed Horticulture Facility (Figure 2). This site is bounded to the north, east, and south by open space, and to the west by the track and field facility. The ground elevation at the site ranges between approximately 540 and 545 feet above mean sea level (MSL) with gradients that vary from near level to approximately 3 percent sloping down to the east (Sandis, 2019). The site is laterally offset approximately 45 feet from the crown of a composite slope to the east that consists of about 27 vertical feet of engineered fill on a natural slope approximately 23 feet high (Bohley, 2008). Gradients on the engineered fill slope and natural slope are up to approximately, 3:1 (horizontal to vertical) and 7:1 (horizontal to vertical), respectively. Existing vegetation on site generally consists of low grasses and weeds, and most of the surface has been plowed to control vegetation growth. Due to plowing and other activities, the upper 12 inches of the surface are disturbed.

Although currently undeveloped, the proposed site was previously graded for the future construction of a softball field as part of the Phase III Athletic Complex project (Ninyo & Moore, 2008). Grading for the Phase III Athletic Complex project included the removal of inadequately compacted fill material placed in the area from other earthwork projects on the campus, cutting portions of the athletic complex site which generally comprised the portion of the campus east of the Campus Loop and north of Campus Hill Drive, excavating keyways and benches into suitable materials, and placement of engineered fill to raise the grade and construct a fill slope along the eastern margin of the future athletic complex. Ninyo & Moore provided geotechnical observation and compaction testing services during earthwork for the Phase III Athletic Complex project which began in October of 2009 and was largely completed by September of 2010 (Ninyo & Moore, 2011). Earthwork on the proposed site for the Viticulture Facility during the grading the Athletic Complex project consisted of the removal of old fill and other unsuitable materials, and the placement of engineered fill to raise the grade on site up to approximately 12 feet.

4 PROJECT DESCRIPTION

Based upon information provided by tBP Architecture, the Viticulture Facility will consist of a classroom building with a footprint area of about 1,700 square feet and a winemaking facility with adjacent crush pad on a combined footprint of approximately 3,900 square feet. The approximate limits of these structures are shown Figure 2. For the purposes of this study, we assume that the proposed structures will be one- to two-stories supported on shallow foundations. Other features may include hardscaping, vehicle parking areas, and retaining walls. We anticipate that finish grades will be within a foot or so of the existing grades.

5 FIELD EXPLORATION AND LABORATORY TESTING

Our field exploration for this study included a site reconnaissance and subsurface exploration conducted on October 30, 2020 and November 18, 2020, respectively. The subsurface exploration consisted of three (3) auger borings designated as Borings B-4 through B-6 on Figure 2. Three borings, Borings B-1 through B-3, drilled during a previous study (Ninyo & Moore, 2008) for the athletic complex were incorporated into our current study and are also shown on Figure 2.

Prior to commencing the subsurface exploration, we contacted Underground Service Alert (USA) to notify utility owners and retained a private utility locater to locate and mark existing utilities on site. The exploratory borings were drilled to depths of up to approximately 30 feet below the ground surface using a CME D-50 track-mounted rig equipped with solid-stem augers. A representative of Ninyo & Moore logged the subsurface conditions exposed in the borings and collected relatively undisturbed and bulk soil samples from the borings. The samples were transported to our geotechnical laboratory for testing. The borings were backfilled with grout after drilling in conformance with the Zone 7 permit. Descriptions of the subsurface materials encountered are presented in the following sections. Detailed logs of the borings and soil sampling procedures are presented in Appendix A.

Laboratory testing of soil samples recovered from the borings included tests to evaluate in-situ soil moisture content and dry density, particle size distribution, Atterberg limits, unconfined compressive strength, and R-value. A soil sample was submitted to CERCO Analytical for corrosivity evaluation. The results of the in-situ moisture content and dry density tests are presented on the boring logs in Appendix A. The results of the laboratory tests performed are presented in Appendix B. The results of the corrosivity tests are presented in Appendix C.

6 PREVIOUS STUDIES

Ninyo & Moore previously performed a geologic hazards assessment and geotechnical evaluation for the athletic complex project (Ninyo & Moore, 2008). Three borings, Borings B-1 through B-3 were drilled in close proximity to the location currently proposed for the Viticulture Facility (Figure 2). The logs for these borings are included in Appendix A.

On February 1, 2020, a geophysical Refraction Microtremor (ReMi) survey was performed on for the adjacent proposed Horticulture Facility (Ninyo & Moore, 2020b). The purpose of the survey was to evaluate the subsurface shear-wave velocity at the site in order to select the appropriate seismic site class. The ReMi survey used the passive seismic method of Microtremor Array Measurements (MAM) and consisted of one linear profile of seismic data collection as shown on Figure 2. The method provided a shear wave velocity model to a depth of approximately 100 feet which was then used to calculate the weighted harmonic mean of the shear wave velocity (V_{s100}) over that interval to select the seismic site class. The seismic model results are provided in Appendix D.

7 GEOLOGIC AND SUBSURFACE CONDITIONS

7.1 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.

7.2 Site Geology

Regional maps (Dibblee & Minch, 2006; Graymer et al, 1996) indicate that the project site is underlain by the Livermore Gravel of Pliocene to Pleistocene age. The Livermore Gravel is described as poorly to moderately consolidated, indistinctly bedded, cobble conglomerate, gray conglomeratic sandstone, and gray coarse-grained sandstone with some siltstone and claystone (Graymer et al, 1996). A regional geologic map prepared by Dibblee & Minch (2006) is provided as Figure 4. This unit is described as alluvium on the boring logs in this report.

7.3 Subsurface Conditions

The following sections provide a generalized description of the units encountered during our subsurface evaluation. More detailed descriptions are presented on the boring logs in Appendix A. Cross sections depicting our interpretation of the subsurface geologic conditions are provided on Figures 5 and 6.

7.3.1 Artificial Fill

Fill placed under the geotechnical observation and testing of Ninyo & Moore (2011) was observed in Borings B-4 through B-6. The fill encountered in these borings generally consisted of dark brown to light brown and grayish brown, moist, stiff to hard clay and sandy clay with occasional layers of medium dense to dense clayey sand with gravel. The upper foot or so of the fill has been disturbed by plowing and is considered unsuitable for support of foundations and other improvements. Based on the extent of the fill encountered in Borings B-4 through B-6 and the elevation of the field density tests conducted during grading for the Phase III Athletic Complex, the thickness of the engineered fill on the site is estimated to range between approximately 8 and 25 feet. The fill described on the logs for Borings B-1 and B-2, drilled before grading for the Phase III Athletic Complex project, was removed and replaced with engineered fill as part of the grading for that project.

7.3.2 Alluvium

Alluvium was encountered in the borings from below the fill, where encountered, to the depths explored. As encountered, the alluvium generally consisted of light brown, moist, very stiff to hard, sandy clay with scattered caliche cementation. Alluvial overbank deposits consisting of dark brown, stiff to very stiff fat clay was encountered in Borings B-1 through B-3, drilled before grading for the Phase III Athletic Complex project. The overbank deposits on site were largely removed during grading for that project, and mixed into the fill excluding select fill areas under buildings and hardscape. The currently proposed location for the Viticulture

Facility was to be a softball field as part of the Phase III Athletic Complex project and generally did not include buildings or significant hardscaping.

7.4 Groundwater

Groundwater was not encountered during our subsurface exploration. The California Geological Survey (CGS) indicates that the project site is within an area where information regarding the depth to the historical high groundwater level is uncertain (CGS, 2008a).

Fluctuations in the level of groundwater may occur due to variations in ground surface topography, subsurface stratification, rainfall, irrigation practices, groundwater pumping, and other factors which may not have been evident at the time of our field evaluation. In addition, seeps may be encountered at elevations above the groundwater levels encountered due to perched groundwater conditions, leaking pipes, preferential drainage, or other factors not evident at the time of our exploration.

8 GEOLOGIC HAZARDS AND CONSIDERATIONS

This study considered a number of potential issues relevant to the proposed construction on the subject site, including seismic hazards, flood hazards, landsliding, unsuitable materials, settlement of compressible soil layers, naturally occurring asbestos, expansive soil, potential for on-site soil to corrode ferrous metals and promote sulfate attack on concrete, and excavation considerations. These issues are discussed in the following subsections.

8.1 Seismic Hazards

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

8.1.1 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, etc.) compiled by Knudsen et al. (2000), indicate that no ground effects related to historic seismic activity have been reported for the site.

8.1.2 Faulting and Ground Surface Rupture

The site is not located within an Alquist-Priolo Fault Rupture Hazard Zone (AP Zone) established by the State Geologist (CGS, 1982) to delineate regions of potential ground surface rupture adjacent to active faults. As defined by the California Geological Survey (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 the one associated with the Greenville Fault, which is within approximately 4¼ miles of the site to the northeast.

Regional geologic maps by Crane (1995), Dibblee and Minch (2006), Graymer et al. (1996), and Jennings and Bryant (2010) depict a fault within approximately 400 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, the fault located approximately 400 feet northwest of the project site would not be considered active for purposes of potential surface fault rupture with low probability of damage to structures due to surface rupture for this fault.

Additionally, Ninyo & Moore performed a subsurface fault trenching study in 2007 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 that was excavated 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 are not 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.

8.1.3 Strong Ground Motion

Based on historic activity, the potential for future strong ground motion at the site is considered significant. A site-specific ground motion hazard analysis was performed in accordance with Chapter 21 of the American Society of Civil Engineers (ASCE) Standard 7-16 to evaluate the peak ground acceleration (PGA) associated with the Maximum Considered Earthquake Geometric Mean (MCE_G) in accordance with the 2019 California Building Code (CBC). The site-specific ground motion analysis consisted of a probabilistic seismic hazard analysis (PSHA) using the Open Seismic Hazard Analysis (OpenSHA) Hazard Spectrum Application (Field et al., 2003) and a deterministic seismic hazard analysis (DSHA) using 2014 next generation attenuation (NGA) relationships (Seyhan, 2015). The earthquake rupture forecast considered in the analysis and the specific attenuation relationships utilized are listed on Figure 7. An average shear wave velocity of 984 feet per second (fps) to a depth of 100 feet (V_{s100}), based on the results of the geophysical survey for this study (Appendix D), was assumed for this analysis with a corresponding Class D seismic site classification. Basin characteristics were interpreted from Version 8.3.0 of the USGS Bay Area Velocity Model using the OpenSHA Site Data Application (Field et al., 2003). Assumed fault characteristics and site-to-rupture distances are based on the Caltrans Fault Database (Caltrans, 2019) and the Fault Section Data adopted by Version 3 of the Uniform California Earthquake Rupture Forecast (WGCEP, 2013). Our analysis indicates that the DSHA is controlled by either a magnitude 6.9 event on the Greenville fault with a site-to-rupture distance of about 6.4 kilometers, a magnitude 6.6 event on the Mount Diablo Thrust with a site-to-rupture distance of about 8.1 kilometers, or a magnitude 6.4 event on the Mount Diablo Thrust South with a site-to-rupture distance of about 1.6 kilometers. The results of our site-specific ground motion hazard analysis indicate that the MCE_G peak ground acceleration with adjustment for site class effects (PGA_M) is 0.838g. Ground motion calculations and assumed parameters are presented in Appendix E. Seismic design criteria to address ground shaking are provided in Section 10.1.

8.1.4 Liquefaction and Strain 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 soils (strain 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 generate sand boils leading to subsidence at the ground surface.

The site is not located within a seismic hazard zone for liquefaction (Figure 8) as mapped by the California Geological Survey (CGS, 2008). Moreover, we did not encounter saturated, loose, granular soils during our subsurface exploration. The fine-grained soil (silt and clay) encountered during our subsurface exploration generally did not conform to the characteristics of liquefiable soils published by Bray and Sancio (2006). The cohesive soils encountered were not particularly wet or sensitive. As such, we do not regard liquefaction, strain softening, or related hazards including lateral spreading or sand-boil induced ground subsidence as design considerations.

8.1.5 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.

8.1.6 Seismic Slope Stability

The site is not located within a hazard zone for earthquake-induced landslides (Figure 8) established by the California Geological Survey (CGS, 2008) and the project does not include the construction of significant slopes. As such, we do not regard seismic slope stability as a design consideration.

8.1.7 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 inundation area as shown on the Tsunami Inundation Map for Emergency Planning Map (State of California, 2009). Seiches are waves generated in a large enclosed body of water. Based on the inland location of the site, and the lack or a large body of water nearby, the potential for damage due to tsunamis or seiches is not a design consideration.

8.2 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 (Zone X). As such, the potential for flooding at the site is low.

8.3 Landsliding and Slope Stability

The site of the proposed project and surrounding area is relatively flat and the proposed grading does not include the construction of significant slopes. Based on the topographic site conditions and the proposed grading, we do not regard landsliding or slope stability as a design consideration.

8.4 Unsuitable Materials

The site is covered with engineered fill (Ninyo & Moore, 2011). Existing vegetation generally consists of low grasses and weeds. The upper foot or so of the engineered fill has been disturbed by plowing to control vegetation and is considered unsuitable for support of foundations and other improvements. Recommendations for remedial grading to mitigate the unsuitable support characteristics of the disturbed fill materials are presented later in this report.

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, vegetation, or other organic matter should be removed as part of the clearing and grubbing operations.

8.5 Static Settlement

No significant increase in pad elevations are anticipated for the project and the subsurface conditions encountered in our borings, below the upper foot or so of disturbed fill, generally consisted of stiff to hard clay. We estimate that static settlement will be approximately ½ inch for sustained wall and column loads of up to 6 kips per foot and 100 kips, respectively, with footings and pad grading that conform to the recommendations in this report.

8.6 Naturally Occurring Asbestos

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 and Hill (2000) indicates that no ultramafic rocks have been mapped in the general vicinity of the project site. Therefore, it is unlikely that significant concentrations of naturally occurring asbestos will be encountered at the site.

8.7 Expansive Soil

Some clay minerals undergo volume changes upon wetting or drying. Unsaturated soil containing those minerals will shrink/swell with the removal/addition of water. The heaving pressures associated with this expansion can damage structures and flatwork. Laboratory testing was performed on selected limited samples of near-surface soil to evaluate the expansion characteristics of the site. The tests were performed in in accordance with ASTM D 4318 to evaluate the liquid limit, plastic limit, and plasticity index of selected samples. The test results, presented in Appendix B, indicate that the plasticity index of samples tested ranged between 14 and 36. These results are indicative of soil with a low to very high expansion characteristic.

To reduce the potential for heave and differential movement due to shrink/swell behavior, recommendations are provided for remedial grading with select import fill or chemical-treatment of on-site soil to create a zone of material with low expansion characteristics below buildings.

8.8 Corrosive and Deleterious Soil

An evaluation of the corrosivity of the on-site material was conducted to assess the impact to concrete and metals. The corrosion impact was evaluated using the results of limited laboratory testing on samples obtained during our subsurface study. Laboratory testing to quantify pH, redox potential, electrical resistivity, chloride content, and soluble sulfate content was performed on a sample of the near-surface soil. The results of the corrosivity tests are presented in Appendix C.

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, 2018). The criteria used to evaluate the deleterious nature of soil on concrete are listed in Table 1. 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. The sample tested is corrosive to ferrous metals based on the resistivity test results and slightly corrosive based on the redox potential as noted in Appendix C. 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 may be consulted to provide recommendations to mitigate corrosion. Recommendations to mitigate the impact of corrosive soil on concrete structures are presented in Section 10.6.

Table 1 – 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)

8.9 Excavation Considerations

We anticipate that the proposed project may involve excavations of up to several feet for footings, utility trenches, and remedial grading, and possibly deeper excavations of up to 20 feet for drilled pier foundations to support light poles, canopies, or other minor structures. The geologic units encountered over this interval during our subsurface evaluation included engineered fill and alluvium that generally consisted of stiff to hard clay and sandy clay with occasional layers of medium dense to dense clayey sand. We anticipate that heavy earthmoving and drilling equipment in good working condition should be able to make the proposed excavations.

Near-vertical temporary cuts in these deposits up to 4 feet in depth should remain stable for a limited period of time. Sloughing of the sidewalls may occur particularly if the excavation encounters granular soil or seeping conditions, is exposed to water, or if the sidewalls are disturbed during construction operations. Excavation subgrade may become unstable if exposed to wet conditions. Appropriate temporary slopes or shoring may be needed to stabilize excavation sidewalls. Recommendations for excavation stabilization are presented.

9 CONCLUSIONS

Based on our review of the referenced background data, site field reconnaissance, subsurface evaluation, and laboratory testing, it is our opinion that the proposed construction is feasible from a geotechnical standpoint. Geotechnical considerations include the following:

- The subsurface exploration for this study encountered engineered fill (Ninyo & Moore, 2011) and alluvium. The fill, as encountered in the borings for this study, generally consisted of stiff to hard clay and sandy clay with occasional layers of medium dense to dense clayey sand with gravel. The upper foot or so of the fill was loose/soft as a result of disturbance by plowing. The alluvium underlying the engineered fill generally consisted of very stiff to hard sandy clay with scattered caliche cementation.

- Groundwater was not encountered during our subsurface exploration. Variations in the groundwater level across the site and over time should be anticipated.

- The site could experience a relatively large degree of ground shaking during a significant earthquake on a nearby fault.
- The project site is outside the 500-year flood zone.
- Tsunamis, seiches, landslides, slope stability, 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.
- We estimate that static settlement will be approximately ½ inch for the proposed improvements on footings with remedial grading to mitigate the upper layer of loose/soft fill disturbed by plowing.
- Laboratory testing indicates that site soil has a low to very high expansion characteristic. Recommendations for remedial grading are provided to mitigate the potential impact of expansive soils.
- 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 corrosion testing of a soil sample collected during the subsurface exploration for this study indicates that the site does not meet the definition of a corrosive environment for structures (Caltrans, 2018) but the samples tested are considered slightly corrosive to corrosive for ferrous metals based on the electrical resistivity and redox potential testing as noted in Appendix C. A corrosion engineer may 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.

10 RECOMMENDATIONS

The following sections present our geotechnical recommendations for the design and construction of the proposed improvements. The project improvements should be designed and constructed in accordance with these recommendations, applicable codes, and appropriate construction practices.

10.1 Seismic Design Criteria

Ninyo & Moore performed a site-specific ground motion analysis in accordance with the procedure in Chapter 21 of ASCE Standard 7-16. The assumptions and methodology for this analysis are discussed in Section 7.1.3. Seismic Site Class D was selected based on the results of the geophysical survey performed for this study (Appendix D). The design response spectrum based on the site-specific ground motion analysis is presented on Figure 7 and the corresponding seismic design criteria are summarized in Table 2. Calculations from the analysis are presented in Appendix E.

Table 2 – 2019 California Building Code Seismic Design Criteria		
Seismic Design Parameter Evaluated for 37.7150° North Latitude, 121.7941°West Longitude	**Site Specific**	**Section 11.4 ASCE 7-16**
Site Class	D	D
Site Coefficient, Fa	---	1.0
Site Coefficient, Fv	---	1.7
Mapped Spectral Response Acceleration at 0.2-second period, S_S	---	1.815g
Mapped Spectral Response Acceleration at 1.0-second period, S_1	---	0.600g
Site-Adjusted Spectral Acceleration at 0.2-second period, S_{MS}	1.865g	1.815g
Site-Adjusted Spectral Acceleration at 1.0-second period, S_{M1}	1.716g	1.020g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	1.244g	1.210g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	1.144g	0.680g
Seismic Design Category for Risk Category I, II, or III	D	D

The spectral ordinates and seismic coefficients based on the mapped values of the risk-targeted spectral response acceleration, consistent with Section 11.4 of ASCE Standard 7-16, are also presented in the table (SEAOC & OSHPD, 2019). In conformance with the 2019 California Building Code, the spectral ordinates and seismic coefficients consistent with Section 11.4 of ASCE Standard 7-16 may be used for seismic design presuming that the seismic response coefficient is calculated from equation 12.8-2 of ASCE Standard 7-16 for structures with a fundamental period of 0.84 seconds or less in accordance with Exception 2 in Section 11.4.8 of ASCE Standard 7-16. Otherwise, the seismic design criteria and design response spectrum consistent with the site-specific ground motion analysis in Table 2 and Figure 7, respectively, should be used for seismic design per the 2019 California Building Code.

10.2 Foundations

The proposed buildings may be supported on footings presuming that remedial grading is performed per the recommendations in Section 10.3.4 to mitigate concerns related to unsuitable subgrade materials and expansive soil. Recommendations for footings to support site retaining walls are provided in Section 10.4. Light poles, free-standing canopies, and other minor structures may be supported on drilled piers as an alternative to footings.

Foundations should be designed in accordance with structural considerations and the following recommendations. In addition, requirements of the appropriate governing jurisdictions and applicable building codes should be considered in design of the structures. The foundation design parameters provided in the following sections are not intended to preclude differential movement of foundations. Minor cracking (considered tolerable) may occur.

10.2.1 Footings

Footings on pads prepared in accordance with the recommendations in Section 10.3.4 may be designed using the criteria listed in Table 3. The geotechnical engineer should observe the footing excavations to evaluate bearing materials and subgrade condition before the exposed subgrade is covered.

Structures supported on footings consistent with these recommendations should be designed for the total and differential settlements listed in Table 3 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 4. The designer may interpolate between the values in the table for intermediate footing widths.

Table 3 – Recommended Bearing Design Parameters for Footings

Footing	Sustained Loads	Footing Widths	Bearing Depth[1]	Allowable Bearing Capacity[2]	Static Settlement
Wall Footing	6 kips/foot or less	18 inches or more	24 inches or more	2,500 psf	½ inch total ¼ inch differential over 30 feet
Column Footing	100 kips or less	24 inches or more	24 inches or more	2,500 psf	½ inch total ¼ inch differential over 30 feet

Notes:

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.

Table 4 – Footing Modulus of Subgrade Reaction

Footing	Footing Width				
	1.5 feet	2 feet	3 feet	5 feet	7 feet
Wall Footing	67 pci	52 pci	39 pci	28 pci	--
Column Footing	--	93 pci	61 pci	40 pci	29 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. Footings should be deepened or excavation depths reduced as-needed. 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 up to 3,000 psf 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 for alternative basic load combinations with loads of short duration such as wind or seismic forces. 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.

10.2.2 Drilled Piers

Drilled piers used as foundations for light poles, free-standing canopies, and other minor structures embedded up to 20 feet below grade may be designed for an allowable side friction of up to 600 pounds per square foot (psf) at 60 psf per foot of embedment depth to evaluate resistance to downward axial loads and up to 400 psf at 40 psf per foot depth for upward axial loads. The recommended values for allowable skin friction include a safety factor 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 settlement due to sustained loads of approximately ¼ inch with a differential 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 1806 of the California Building Code with a one-third increase for wind or seismic loading conditions. 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.

10.2.3 Slab-on-Grade Floors

Building slab-on-grade floors should be designed by the structural engineer based on the anticipated loading conditions. The slab should be reinforced with deformed steel bars with a nominal diameter of ⅜-inch or more. We recommend that masonry briquettes or plastic chairs be used to aid in the correct placement of slab reinforcement in the upper half of the slab. Refer to Section 10.6 for the recommended concrete cover over reinforcing steel. Joints consistent with ACI guidelines (ACI, 2016) may be constructed at periodic intervals to reduce the potential for random cracking of the slab. A vapor retarder is recommended in areas where moisture-sensitive floor coverings or conditioned environments are anticipated. See Section 10.7 for vapor retarding system recommendations. Where a vapor retarding system is not used, slabs should be constructed on 6 inches of compacted aggregate base conforming to Sections 10.3.6 and 10.3.8. Slab subgrade should be prepared in accordance with Section 10.3.7.

10.3 Earthwork

Earthwork should be performed in accordance with the requirements of applicable governing agencies and the recommendations presented below. The geotechnical consultant should observe earthwork operations. Evaluations performed by the geotechnical consultant during the course of operations may result in new recommendations, which could supersede the recommendations in this section.

10.3.1 Pre-Construction Conference

We recommend that a pre-construction conference be held to discuss the grading recommendations presented in the report. Representatives of the District, the architect, the engineer, Ninyo & Moore, and the contractor should be in attendance to discuss project schedule and earthwork requirements.

10.3.2 Site Preparation

Prior to performing earthwork operations, the site should be cleared of vegetation, surface soils containing roots or other organic matter, surface obstructions (e.g., pavements, aggregate base, curb/gutter, foundations, slabs-on-grade etc.), 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 disposed of in an appropriate landfill. Soils containing roots or other organic matter may be stockpiled for later use as landscaping fill, as authorized by the owner's representative. Stockpiled soil that cannot be used as landscaping fill or processed to meet criteria for general fill should be hauled to an appropriate landfill for disposal. Active utilities within the project limits, if any, should be re-routed or protected from damage by construction activities. Existing utilities or underground tanks/vaults to be abandoned should be excavated and removed. 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.

10.3.3 Subgrade Observation

Prior to placement of fill or the erection of forms, the District should request an evaluation of the exposed subgrade by Ninyo & Moore. Materials that are considered unsuitable shall be excavated under the observation of Ninyo & Moore 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 soils; and undocumented or otherwise deleterious fill materials. Unsuitable materials should be removed from below footings, slabs, and areas to receive fill to the depth of suitable material as evaluated by the geotechnical engineer in the field.

Laboratory testing indicates that the site soil has a low to very high expansion characteristic. In addition, the upper foot or so of the ground surface has been disturbed by plowing and is considered unsuitable for the support of buildings and other significant foundations. Recommendations for remedial grading to mitigate concerns related to unsuitable subgrade materials and expansive soils are provided in the following section.

10.3.4 Remedial Grading

To mitigate the variable support characteristics related to the loose/disturbed condition of the upper fill observed during our exploration, and the potential for shrink or swell due to expansive soil, the building should be constructed over a pad of fill with low expansion characteristics. The pad of low expansion fill should extend to 3 feet below the nominal bottom of slab and one foot outside the building footprint. The pad of low expansion fill may be constructed by removing the existing subgrade soil at the building location and backfilling with imported select fill. After excavation, Ninyo & Moore should observe the condition of the exposed subgrade to evaluate if additional excavation is needed. After this evaluation, the exposed subgrade should be scarified and moisture conditioned, as needed, to achieve a moisture content above the optimum. The conditioned subgrade should be compacted to 90 percent of the reference density as evaluated by ASTM D1557. The excavation may then be backfilled with imported select fill that that conforms with the criteria in Section 10.3.6, and is placed and compacted in lifts per the recommendations in Section 10.3.8. 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 that conforms with the criteria for general fill may be chemically treated per the recommendations in Section 10.3.5 to create the layer with low expansion characteristics below the building. In general, the materials removed from the remedial excavations should be suitable for reuse as general fill, provided that the material is screened for rocks or lumps in excess of 3 inches in diameter, trash, debris, roots, vegetation, or deleterious materials.

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

10.3.5 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. Please note that chemical treatment of on-site soils may not be suitable for landscape areas or areas where permeable pavement is proposed.

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 quicklime or cement, 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 3 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.

To improve the subgrade support characteristics, quicklime should be mixed into the soil as described above. Following the 16-hour mellowing period after the initial mixing, cement should be mixed into the soil at a rate of 3 percent or more by dry weight of soil. The moisture content of the soil should not exceed the optimum moisture content of the material, as evaluated by ASTM D1557, when the cement is spread and initially mixed. The subgrade should be mixed and aerated as-needed to reduce the moisture content. If additional water is needed to achieve the optimum moisture, the water should be added during a re-mixing operation after the cement has been initially mixed into the subgrade so as to reduce the potential for the formation of cement balls when water is applied. The cement-treated soil should be compacted within 2 hours of initial mixing to achieve 95 percent of the reference density as evaluated by ASTM D1557 on a wet density basis. Vehicular traffic and heavy construction equipment should not be allowed on the treated material for a 1-hour period after compaction. The cement-treated material should be maintained in a moist condition for a 7-day curing period by routinely sprinkling water, covering the treated material with moist straw, or placing fill over the treated subgrade. Treated subgrade for pavements should be proof-rolled with a loaded water truck to check for yielding conditions. Mitigation of yielding areas by pulverizing and re-mixing with additional stabilizing agent should be anticipated.

10.3.6 Material Recommendations

Materials used during earthwork, grading, and paving operations should comply with the requirements listed in Table 5. Materials should be evaluated by the geotechnical engineer for suitability prior to use. The contractor should notify the geotechnical consultant 72 hours prior to import of materials or use of on-site materials to permit time for sampling, testing, and evaluation of the proposed materials. On-site materials may need to be dried out before re-use as fill. The contractor should be responsible for the uniformity of import material brought to the site.

Table 5 – Geotechnical Recommendations for Materials		
Material and Use	**Source**	**Requirements[1,2,3]**
Asphalt Concrete	Import	Type A; CSS[5] Section 39-2
Aggregate Base	Import	Class II; CSS[5] Section 39-2
Select (Low Expansion) Fill - Top of building pad - Behind retaining walls [4]	Import	Close-graded with 35 percent or more passing No. 4 sieve and either: Expansion Index of 50 or less, Plasticity Index of 12 or less, or less than 10 percent, by dry weight, passing No. 200 sieve
	On-site borrow	As per general fill and treated with lime per Section 10.3.5
General Fill -For uses not otherwise specified	On-site borrow	Organic content 3 percent by dry weight or less
Permeable Aggregate - Capillary break gravel - Retaining wall backdrain	Import	Open-graded, clean, compactable crushed rock or angular gravel; nominal size 3/4" or less
Pipe/Conduit Bedding Material - Below conduit invert to12 inches above conduit	Import	90 to 100 percent (by mass) should pass No. 4 sieve, and 5 percent or less should pass No. 200 sieve
Trench Backfill - Above pipe zone material and in top 3 feet of building pad or top foot below flatwork	Import or lime-treated site soil	As per select fill and excluding rock/lumps retained on 4-inch sieve or 2-inch sieve in top 12 inches
Trench Backfill - Above pipe zone material in other locations	Import or on-site borrow	As per general fill and excluding rock/lumps retained on 4-inch sieve or 2-inch sieve in top 12 inches
Controlled Low Strength Material (CLSM)	Import	CSS[5] Section 19-3.02G

Notes:

1 In general, fill should not consist of pea-gravel and should be free of rocks or lumps in excess of 6 inches in diameter, trash, debris, roots, vegetation or other deleterious material.
2 In general, import fill should be tested or documented to be non-corrosive and free from hazardous materials in concentrations above levels of concern.
3 Non-corrosive as defined by the Corrosion Guidelines (Caltrans, 2018).
4 Above a plane extending up and away from the heel of wall footing at 1:1 (horizontal to vertical) angle.
5 CSS is California Standard Specifications (Caltrans, 2018).

10.3.7 Subgrade Preparation

Subgrade below footings, slabs, pavement, walkways or fill, should be prepared as per the recommendations in Table 6. Recommendations for subgrade preparation for footings bearing on expansive subgrade are provided for retaining walls and deepened structural footings.

Table 6 – Subgrade Preparation Recommendations	
Subgrade Location	**Source**
Below pavement	• Clear and grub as per Section 10.3.2. • Check for unsuitable materials as per Section 10.3.3. • Scarify top 8 inches then moisture condition and compact as per Section 10.3.8. • Proof roll compacted subgrade with loaded water truck under the observation of the geotechnical engineer. Mitigate yielding areas in accordance with the recommendations of the engineer. • Keep in moist but not saturated condition by sprinkling water.
Below building pads	• Clear and grub as per Section 10.3.2. • Perform remedial grading as per Section 10.3.4. • Keep in moist condition by sprinkling water
Utility trenches	• Check for unsuitable materials as per Section 10.3.3. • Remove or compact loose/soft material.
Below flatwork	• Clear and grub as per Section 10.3.2. • Perform remedial grading as per Section 10.3.4. • Keep in moist but not saturated condition by sprinkling water.
Below retaining walls	• Check for unsuitable materials as per Section 10.3.3. • Scarify and moisture condition exposed subgrade as-needed to achieve a moisture content approximately 2 points above the optimum as evaluated by ASTM D1557. Compact moisture-conditioned subgrade per Section 10.3.8. • Keep in moist condition by sprinkling water.
Below fill	• Clear and grub as per Section 10.3.2. • Check for unsuitable materials as per Section 10.3.3. • Scarify top 8 inches then moisture condition and compact as per Section 10.3.8. • Keep in moist but not saturated condition by sprinkling water.

10.3.8 Fill Placement and Compaction

Fill and backfill should be compacted in horizontal lifts in conformance with the recommendations presented in Table 7. The allowable uncompacted thickness of each lift of fill depends on the type of compaction equipment utilized, but generally should not exceed 8 inches in loose thickness. Heavy compaction equipment should not be used in the zone of influence behind retaining walls. The zone of influence is the region above a plane extending up and away from the heel of the wall at a slope of about 2:1 (horizontal to vertical).

Table 7 – Recommended Compaction Criteria			
Fill Type	**Location**	**Compacted Density[1]**	**Moisture Content[2]**
Asphalt Concrete	Pavement section	91 percent	Not Applicable
Aggregate Base	Flatwork and hardscape underlayment	95 percent	Near Optimum
Subgrade (not lime-treated)	Upper 12 inches below pavement for vehicles	95 percent	+ 2 percent
	In locations not already specified	90 percent	+ 2 percent
Bedding and Pipe Zone Fill	Material below invert to 12 inches above pipe	90 percent	Near Optimum
Trench Backfill	Below pavement (within 2 feet of finished grade)	95 percent	+ 2 percent
	In locations not already specified	90 percent	+ 2 percent
Lime- or cement-treated subgrade or fill	In locations not already specified	95 percent	+ 2 percent
Select or General Fill (not lime-treated)	Behind retaining walls	90 percent	+ 2 percent
	In locations not already specified	90 percent	+ 2 percent

Notes:

1 Expressed as percent relative compaction or ratio of field density to reference density (typically on a dry density basis for soil and aggregate and on a wet density basis for asphalt concrete and lime treated subgrade). The reference density of soil, lime-treated subgrade, and aggregate should be evaluated by ASTM D 1557. The reference density of asphalt concrete should be evaluated by ASTM D 2041.

2 Target moisture content at compaction relative to the optimum as evaluated by ASTM D 1557.

Compacted fill should be maintained in a moist (but not saturated) condition by the periodic sprinkling of water prior to placement of additional overlying fill. Fill that has been permitted to dry out and loosen or develop desiccation cracking, should be scarified, moisture conditioned, and recompacted as per the requirements above.

10.3.9 Temporary Excavations and Shoring

Trench excavations shall be stabilized in accordance with the Excavation Rules and Regulations (29 Code of Federal Regulations [CFR], Part 1926) stipulated by the Occupational Safety and Health Administration (OSHA). Stabilization shall consist of shoring sidewalls or laying slopes back.

Dewatering pits or sumps should be used to depress the groundwater level (if encountered) below the bottom of the excavation. Table 8 lists the OSHA material type classifications and corresponding allowable temporary slope layback inclinations for soil deposits that may be encountered on site. Alternatively, an internally-braced shoring system or trench shield conforming to the OSHA Excavation Rules and Regulations (29 CFR, Part 1926) may be used to stabilize excavation sidewalls during construction. The lateral earth pressures listed in Table 8 may be used to design or select the internally-braced shoring system or trench shield. 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 our exploration. 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.

Table 8 – OSHA Material Classifications and Allowable Slopes

Formation	OSHA Classification	Allowable Temporary Slope[1,2,3]	Lateral Earth Pressure on Shoring[4] (psf)
Cohesive Fill & Alluvium (above groundwater)	Type B	1h:1v (45°)	45×D + 72
Granular Fill & Alluvium (above groundwater)	Type C	1½h:1v (34°)	80×D + 72

Notes:

1 Allowable slope for excavations less than 20 feet deep. Excavation sidewalls in cohesive soil may be benched to meet the allowable slope criteria (measured from the bottom edge of the excavation). The allowable bench height is 4 feet. The bench at the bottom of the excavation may protrude above the allowable slope criteria.

2 In layered soil, layers shall not be sloped steeper than the layer below.

3 Temporary excavations less than 4 feet deep may be made with vertical side slopes and remain unshored if judged to be stable by a competent person (29 CFR, Part 1926.650).

4 'D' is depth of excavation for excavations up to 20 feet deep. Includes a surface surcharge equivalent to two feet of soil.

The shoring system should be designed or selected by a suitably qualified individual or specialty subcontractor. 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. We recommend that the contractor take appropriate measures to protect workers. OSHA requirements pertaining to worker safety should be observed.

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).

10.3.10 Construction Dewatering

Groundwater was not encountered in our exploratory borings. However, significant fluctuations in the groundwater level may occur as a result of variations in seasonal precipitation and other factors. 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. 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.

10.3.11 Utility Trenches

Trenches constructed for the installation of underground utilities should be stabilized in accordance with our recommendations in Section 10.3.9. Utility trenches should be backfilled with materials that conform to our recommendations in Section 10.3.6. Trench backfill, bedding, and pipe zone fill should be compacted in accordance with Section 10.3.8 of this report. Bedding and pipe zone fill should be shoveled under pipe haunches and compacted by manual or mechanical tampers. Trench backfill should be compacted by mechanical means. Densification of trench backfill by flooding or jetting should not be permitted.

Trenches should not be excavated adjacent to footings. If trenches are to be excavated near a footing, the bottom of the trench should be located above a 2:1 (horizontal to vertical) plane projected downward from the bottom of the footing. Utility lines that cross beneath footings should be encased in concrete or CLSM below the footing for a distance equivalent to the depth of the excavation.

10.3.12 Rainy Weather Considerations

Earthwork and foundation construction should be performed during the period between approximately April 15 and October 15 to avoid the rainy season. 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 agency 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. The geotechnical consultant should be consulted for recommendations to stabilize the site as-needed.

Subgrade exposed to water may soften and be subject to pumping under equipment loads. The contractor should be prepared to stabilize exposed subgrade and the bottom of the excavations. In general, unstable subgrade may be mitigated by scarification and aeration of the soil to achieve a moisture content near the optimum, removal of accumulated water or dewatering to depress groundwater levels below the bottom of the excavation, overexcavating to a suitable depth and replacing the wet material with suitable fill, compacting a layer of crushed rock fill into the subgrade, or using geogrid to stabilize additional fill. Specific recommendations for excavation stabilization will be influenced by the nature of the excavation and the conditions encountered during construction.

10.4 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, respectively, 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, respectively, 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. Vaults or other below grade walls that are restrained by framing, floor diaphragms, or abutting walls should be designed to resist at-rest earth pressures. For rising backfill conditions, the active or at-rest equivalent fluid earth pressures may be increased by 1 psf per foot depth per degree of inclination. An additional equivalent fluid pressure of 33 psf per foot depth may be used to evaluate seismic earth pressure on retaining walls, as appropriate, for consideration with active earth pressures.

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 backfill surcharge and lateral earth pressure for adjacent footings should be considered, as applicable, where the adjacent footings bear above an imaginary plane that rises up and away from the bottom edge of the wall at a 2:1 (horizontal to vertical) gradient.

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. The passive earth pressure for walls on ground sloping more than 5 percent should be reduced by 5 psf per foot depth per degree of inclination. 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 200 psf per additional foot of width and 600 psf per additional foot of embedment up to 4,000 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.

10.5 Pavements and Flatwork

Recommendations for pavement and exterior flatwork are presented in the following sections. A design R-value of 5 was selected based on laboratory testing of a site soil sample. Recommendations for preparation of subgrade are presented in Section 10.3.7. Pavement sections were evaluated for a range of traffic indexes or loading conditions. The designer may interpolate between the values provided once a traffic index or loading condition has been selected.

10.5.1 Asphalt Pavement

Ninyo & Moore conducted an analysis to evaluate appropriate asphalt pavement structural sections following the methodology presented in the Highway Design Manual (Caltrans, 2020). Alternative sections were evaluated. The potential degree of differential movement from shrinkage/swelling of expansive subgrade soil can be reduced, where desirable, by using the alternative sections with treated subgrade or sections with 12 or more inches of combined aggregate base and aggregate subbase. The pavement sections were designed for a 20-year service life presuming that periodic maintenance, including crack sealing and resurfacing will be performed during the service life of the pavement. Premature deterioration may occur without periodic maintenance. Our recommendations for the pavement sections are presented in Table 9.

Table 9 – Asphalt Concrete Pavement Structural Sections				
Design R-Value	Traffic Index	Alternative 1	Alternative 2	Alternative 3
5	5	3 inches AC 5 inches AB 7 inches ASB	3 inches AC 8 inches AB SEG	3 inches AC 6 inches AB 12 inches TS
5	6	3½ inches AC 13 inches AB	3½ inches AC 10 inches AB SEG	3½ inches AC 8 inches AB 12 inches TS
5	8	5 inches AC 18 inches AB	5 inches AC 14 inches AB SEG	5 inches AC 10 inches AB 12 inches TS
5	10	6½ inches AC 23 inches AB	6½ inches AC 18 inches AB SEG	6½ inches AC 13 inches AB 12 inches TS

Notes:

1 AC is Type A, Dense-Graded Hot Mix Asphalt complying with Caltrans Standard Specification 39-2 (2018).
2 AB is Class II Aggregate Base complying with Caltrans Standard Specification 26-1.02 (2018).
3 ASB is Class II Aggregate Subbase complying with Caltrans Standard Specification 25-1.02 (2018).
4 SEG is subgrade enhancement geotextile such as Mirafi 600X.
5 TS is chemically treated subgrade consistent with the recommendations in Section 10.3.5.

Paving operations and base preparation should be observed and tested by Ninyo & Moore. Subgrade enhancement geotextiles, where utilized, should be rolled out flat and tight, without folds or wrinkles, over prepared subgrade in the direction of travel. The geotextile should be pinned to the subgrade with nails and washers or u-shaped sod staples. Adjacent rolls should overlap 12 inches or more. Abutting rolls should overlap in the direction of fill placement to reduce the potential for peeling of the geotextile during fill placement. Aggregate base fill should be pushed over the geotextile into position and compacted. To reduce the potential for displacement of the geotextile or deterioration of the subgrade, construction equipment should not operate on the geotextile with 6 inches of aggregate base cover.

Aggregate base for pavement should be placed in lifts of no more than 8 inches in loose thickness and compacted per Section 10.3.8. Asphalt concrete should be placed and compacted per Section 10.3.8. Pavements should be sloped so that runoff is diverted to an appropriate collector (concrete gutter, swale, or area drain) to reduce the potential for ponding of water on the pavement. Concentration of runoff over asphalt pavement should be discouraged. Cracks that form in the asphalt concrete surface should be periodically sealed to reduce moisture intrusion into the aggregate base section. Deep curbs that extend 6 inches below the aggregate base section may be used to reduce the potential moisture intrusion into the aggregate base section adjacent to landscaped areas or the bottom of slopes. Subdrains may be considered as a supplement or alternative means of the mitigating moisture in the aggregate base section. Underlayment with SEG below the aggregate base section should be considered to mitigate cracking near the edge of pavement due to differential shrink/swell behavior of expansive subgrade soil where lateral confinement from adjacent curbs or pavements is not provided. Root barriers adjacent to trees may be considered to reduce the potential for pavement heave from root growth.

10.5.2 Exterior Flatwork

Concrete walkways and other exterior flatwork not subject to vehicular loading should be 4 inches thick (or more) over 6 inches of aggregate base. Concrete thickness should be increased to 6 inches at driveways for vehicular traffic up to periodic garbage trucks and emergency vehicles. The aggregate base should conform to and be compacted in accordance with our recommendations in Sections 10.3.6 and 10.3.8, respectively. Flatwork and driveway subgrade should be prepared in accordance with the recommendations in Section 10.3.7.

Laboratory testing for this study indicates that site soil has a low to very 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 below, 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 10.3.6. Recommendations for chemical treatment of subgrade with quicklime are provided in Section 10.3.5.

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 should be detailed and constructed in accordance with the guidelines of ACI Committee 302 (ACI, 2016). The lateral spacing between contraction joints should be 8 feet or less for a 4-inch thick slab and 12 feet or less for a 6-inch thick slab. 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.

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 6 inches from contraction joints and should consist of No. 3 deformed bars at 18 inches on center, both ways. Slabs reinforced with distributed steel should be 6 inches thick (or more). 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. 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. Root barriers adjacent to trees may be considered to reduce the potential for pavement heave from root growth.

10.6 Concrete

Laboratory testing indicated that the concentration of sulfate and corresponding potential for sulfate attack on concrete is negligible for the soil tested. However, due to the 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 concrete exposed to soil should have 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 cast-in-place against soil. Concrete cover over reinforcing steel for other exposure conditions should conform to ACI guidelines (ACI, 2016).

To reduce the potential for shrinkage cracks in the concrete during curing, we recommend that the 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 ACI Manual of Concrete Practice (MCP) 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.

10.7 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. The bottom of the moisture barrier system should be higher in elevation than the exterior grade, if possible. Positive drainage should be established and maintained adjacent to foundations and flatwork.

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.

10.8 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 a distance of 5 feet or more from the structure for impervious surfaces and 5 percent or more 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 be limited 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. Stormwater management facilities that percolate water into the subgrade should not be located within a distance of 20 feet from structure foundations.

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. Future alteration of the established drainage patterns may impact the constructed improvements.

10.9 Review of Construction Plans

The recommendations provided in this report are based on preliminary design information for the proposed construction. We recommend that a copy of the plans be provided to Ninyo & Moore for review before bidding to check the interpretation of our recommendations and that the designed improvements are consistent with our assumptions. It should be noted that, upon review of these documents, some recommendations presented in this report might be revised or modified to meet the project requirements.

10.10 Construction Observation and Testing

The recommendations provided in this report are based on subsurface conditions encountered in discrete borings. During construction, the geotechnical engineer should be retained to perform construction observation and testing as follows to check that the work performed conforms with the geotechnical recommendations and the subsurface conditions exposed in the construction excavations are consistent with the assumed conditions from the exploratory borings:

- Observe removal of unsuitable materials, chemical treatment, and remedial grading.
- Observe preparation and compaction of subgrade.
- Check and test imported materials prior to use as fill.
- Observe placement and compaction of fill.
- Perform field density tests to evaluate fill and subgrade compaction.
- Observe pier drilling and placement of concrete.

- Observe the condition of the water vapor retarding system before concrete placement.
- Observe foundation excavations for bearing materials and cleaning 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.

11 LIMITATIONS

The field evaluation, laboratory testing, 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 additional subsurface exploration. Additional 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.

This report is intended for design purposes only. It does not provide sufficient data to prepare an accurate bid by contractors. It is suggested that the bidders and their geotechnical consultant perform an independent evaluation of the subsurface conditions in the project areas. The independent evaluations may include, but not be limited to, review of other geotechnical reports prepared for the adjacent areas, site reconnaissance, and additional exploration and laboratory testing.

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 could 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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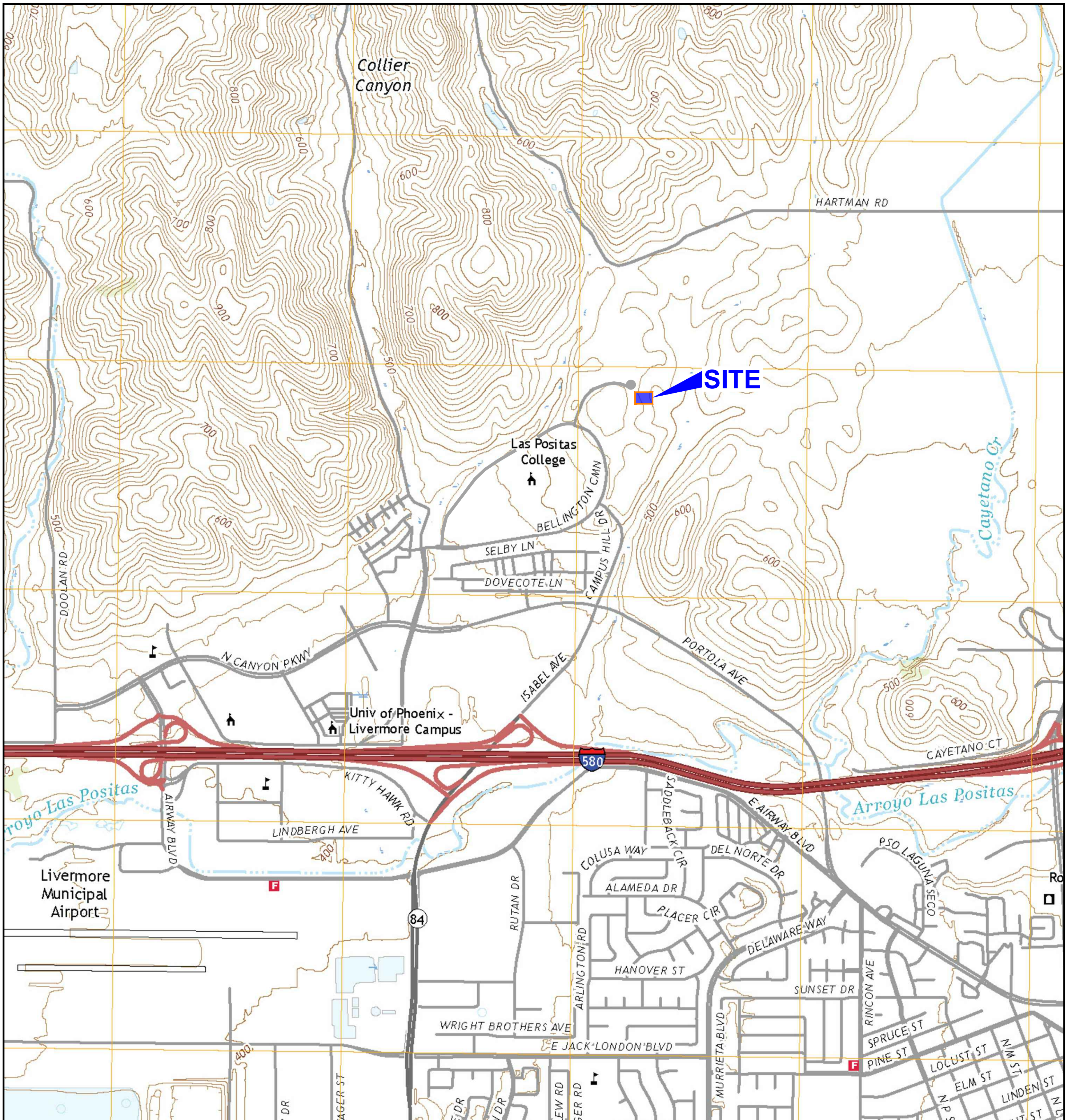
FIGURES



NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE I REFERENCE: USGS, 2018

N

SCALE (FEET)

0 2,000 4,000

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FIGURE 1

Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

SITE LOCATION

LAS POSITAS COLLEGE - VITICULTURE ALTERNATIVE LOCATION NO. 2
3000 CAMPUS HILL DRIVE, LIVERMORE, CALIFORNIA

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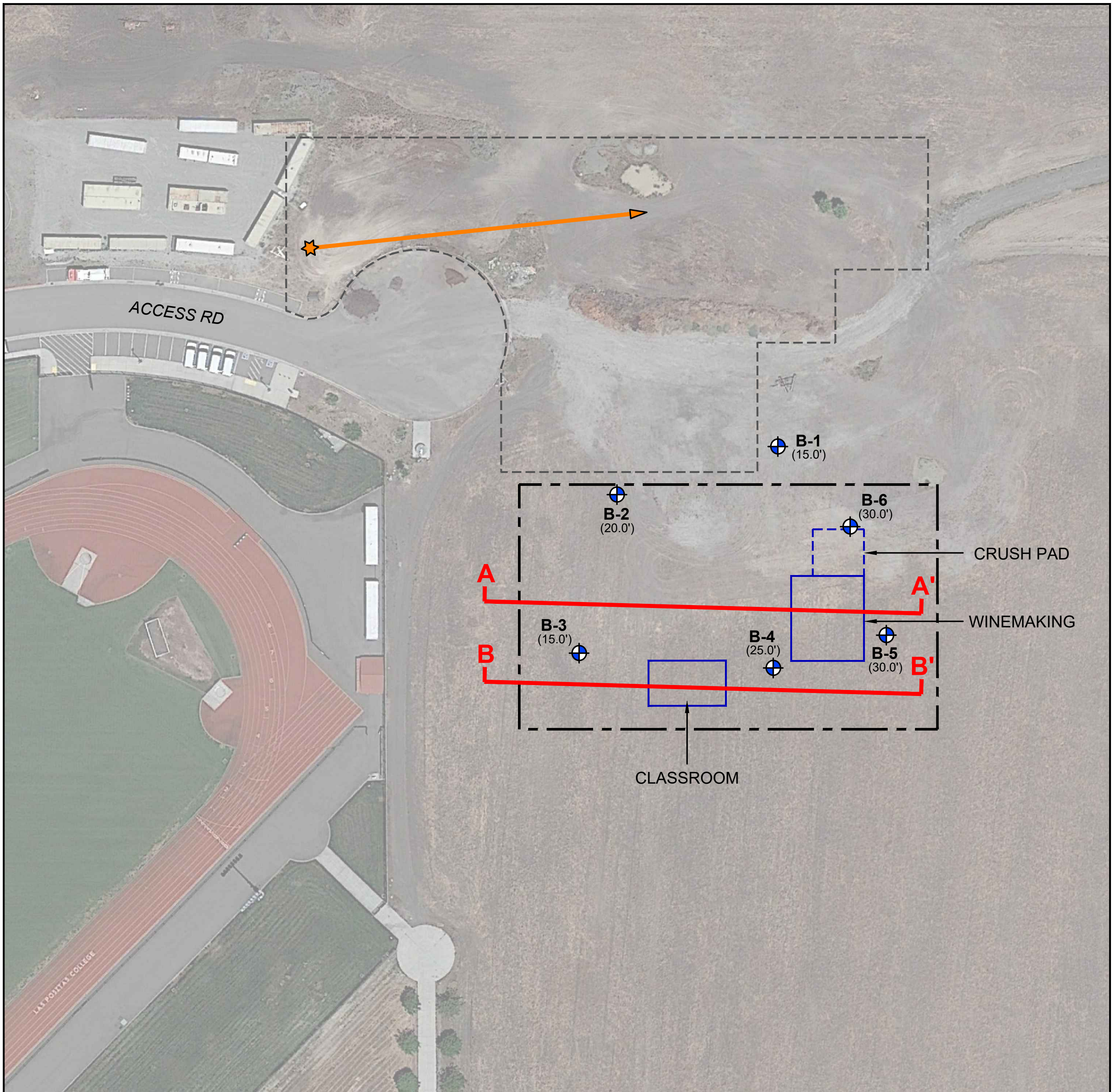

LEGEND

VITICULTURAL FACILITY PROJECT BOUNDARY

HORTICULTURAL FACILITY PROJECT BOUNDARY

PROPOSED BUILDING FOOTPRINT

B-1 (15.0') BORING LOCATION TOTAL DEPTH (FEET)

MAM SURVEY LINE

A A' GEOLOGIC CROSS SECTION LINE

NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE
REFERENCES: SANDIS, 2019; GOOGLE EARTH, 2020

N

SCALE (FEET)
0 100 200

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FIGURE 2

EXPLORATION LOCATIONS

LAS POSITAS COLLEGE - VITICULTURE ALTERNATIVE LOCATION NO. 2
3000 CAMPUS HILL DRIVE, LIVERMORE, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

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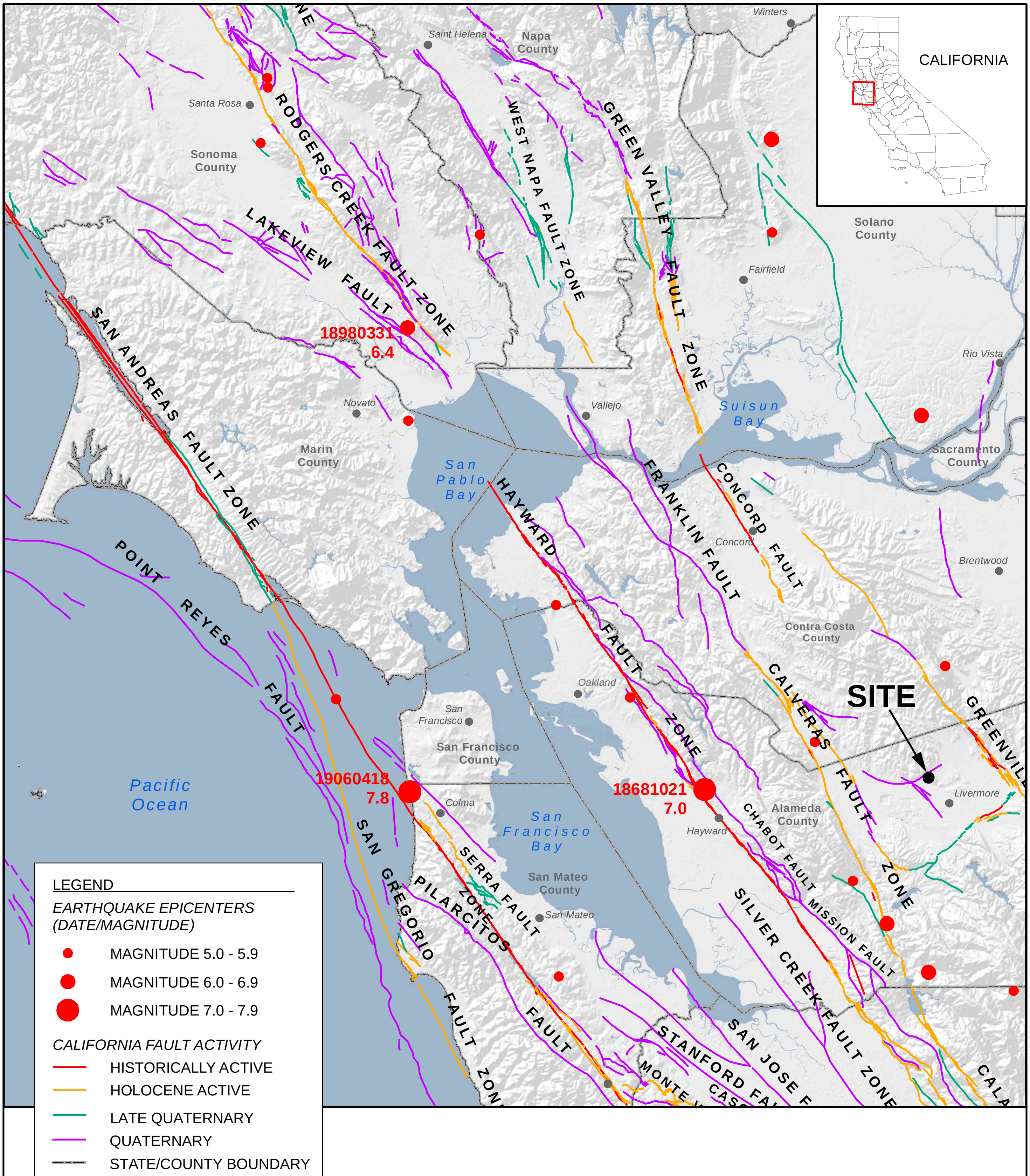

NOTE: DIRECTIONS, DIMENSIONS, AND LOCATIONS ARE APPROXIMATE

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

N

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0 10 20

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FIGURE 3

FAULT LOCATIONS AND EARTHQUAKE EPICENTERS

LAS POSITAS COLLEGE - VITICULTURE ALTERNATIVE LOCATION NO. 2
3000 CAMPUS HILL DRIVE, LIVERMORE, CALIFORNIA
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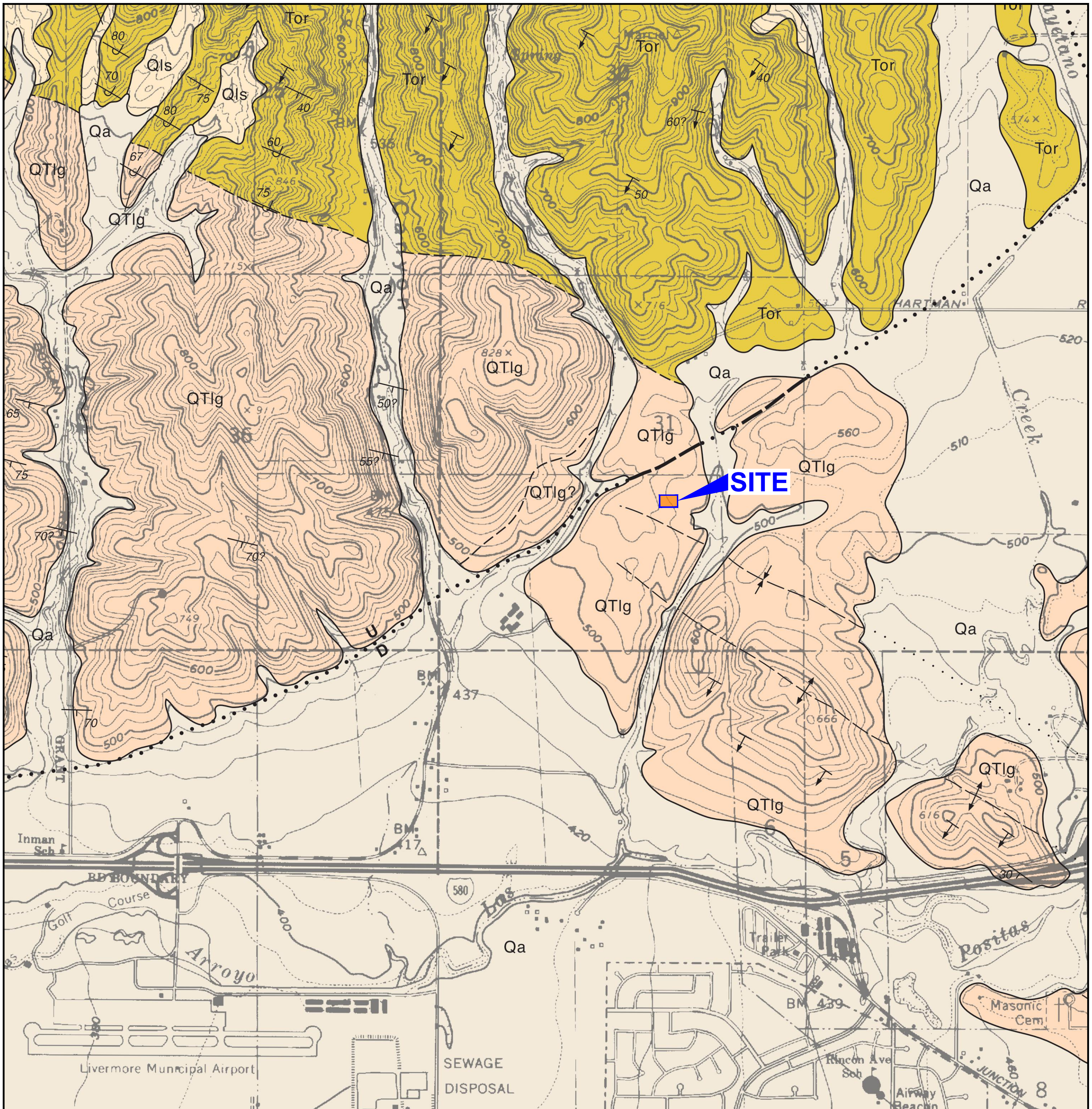
Geotechnical & Environmental Sciences Consultants



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

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FIGURE 4

Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

REGIONAL GEOLOGY

LAS POSITAS COLLEGE - VITICULTURE ALTERNATIVE LOCATION NO. 2
3000 CAMPUS HILL DRIVE, LIVERMORE, CALIFORNIA

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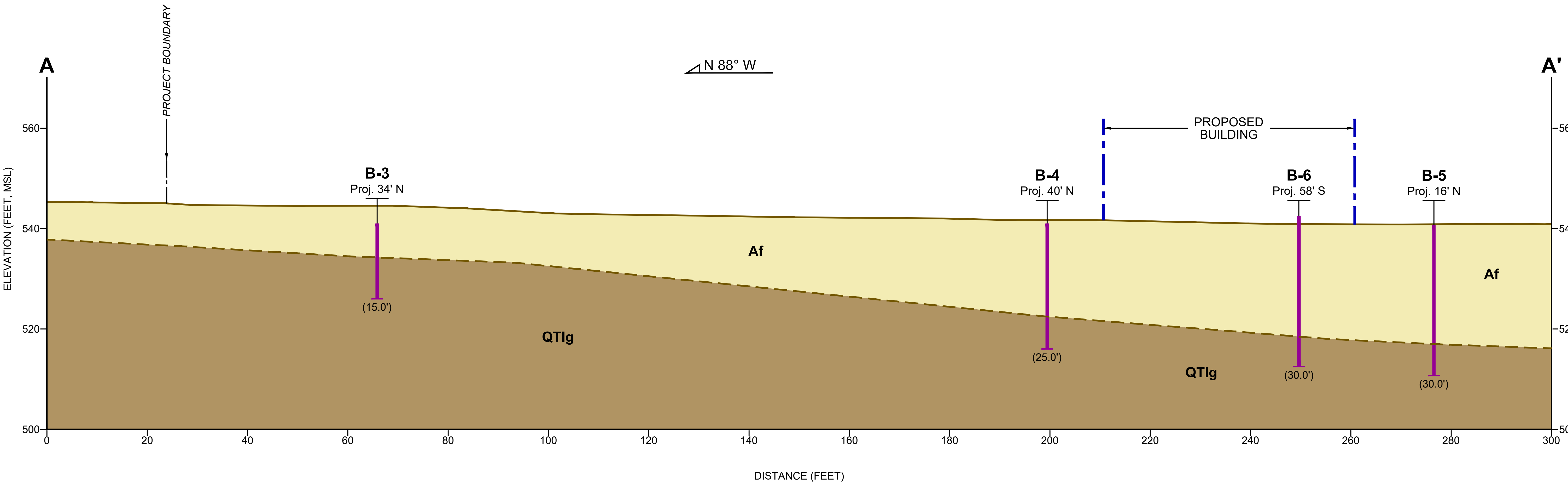


LEGEND

Af	ARTIFICIAL FILL (NINYO & MOORE, 2011)	?-	GEOLOGIC CONTACT: APPROXIMATE, QUERIED WHERE UNCERTAIN	B-3	BORING
QTlg	ALLUVIUM (LIVERMORE GRAVELS)	MSL	MEAN SEA LEVEL	(15.0')	TOTAL DEPTH (FEET)

NOTE:
BORING B-3 WAS ADVANCED IN MAY 2008 BEFORE ARTIFICIAL FILL WAS PLACED IN 2011. THE SOILS DESCRIBED IN THE UPPER PORTION OF THE BORING LOG WERE REMOVED DURING THE 2011 EARTHWORK OPERATIONS.

NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE

401294037.dwg 01/12/2021 AEK

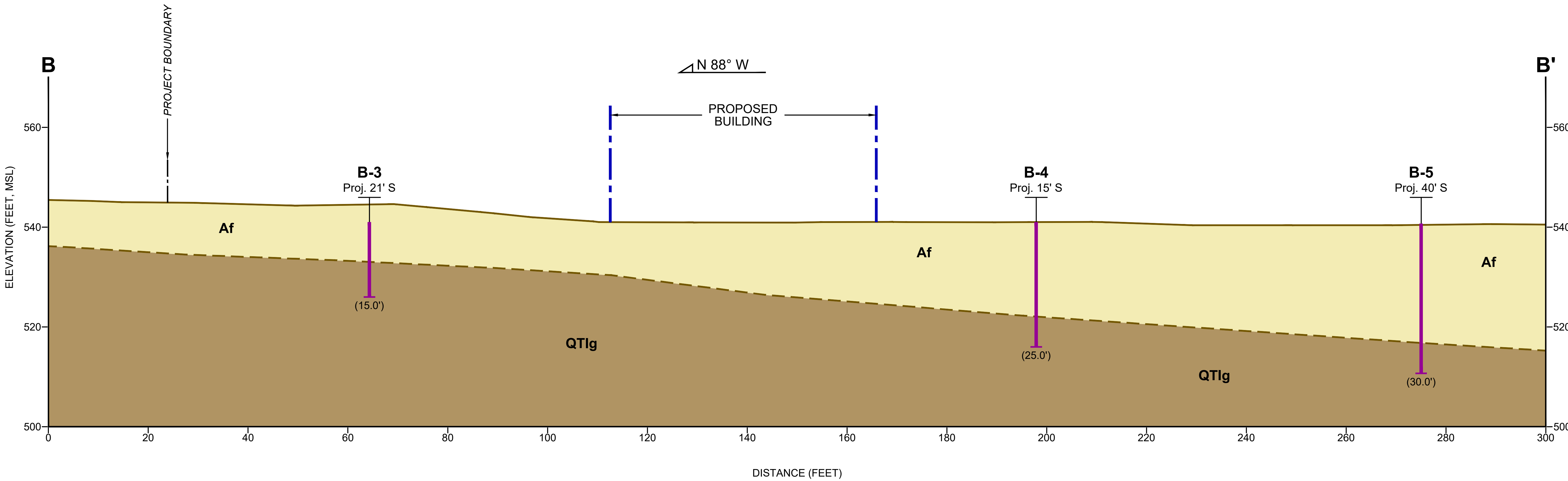
FIGURE 5

GEOLOGIC CROSS SECTION A-A'

LAS POSITAS COLLEGE - VITICULTURE ALTERNATIVE LOCATION NO. 2
3000 CAMPUS HILL DRIVE, LIVERMORE, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

401294037 | 01/21



LEGEND

- Af — ARTIFICIAL FILL (NINYO & MOORE, 2011)
- QTlg — ALLUVIUM (LIVERMORE GRAVELS)
- ?— GEOLOGIC CONTACT: APPROXIMATE, QUERIED WHERE UNCERTAIN
- MSL — MEAN SEA LEVEL
- B-3 — BORING
- (15.0') — TOTAL DEPTH (FEET)

NOTE:
BORING B-3 WAS ADVANCED IN MAY 2008 BEFORE ARTIFICIAL FILL WAS PLACED IN 2011. THE SOILS DESCRIBED IN THE UPPER PORTION OF THE BORING LOG WERE REMOVED DURING THE 2011 EARTHWORK OPERATIONS.

NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE

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FIGURE 6

GEOLOGIC CROSS SECTION B-B'

LAS POSITAS COLLEGE - VITICULTURE ALTERNATIVE LOCATION NO. 2
3000 CAMPUS HILL DRIVE, LIVERMORE, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

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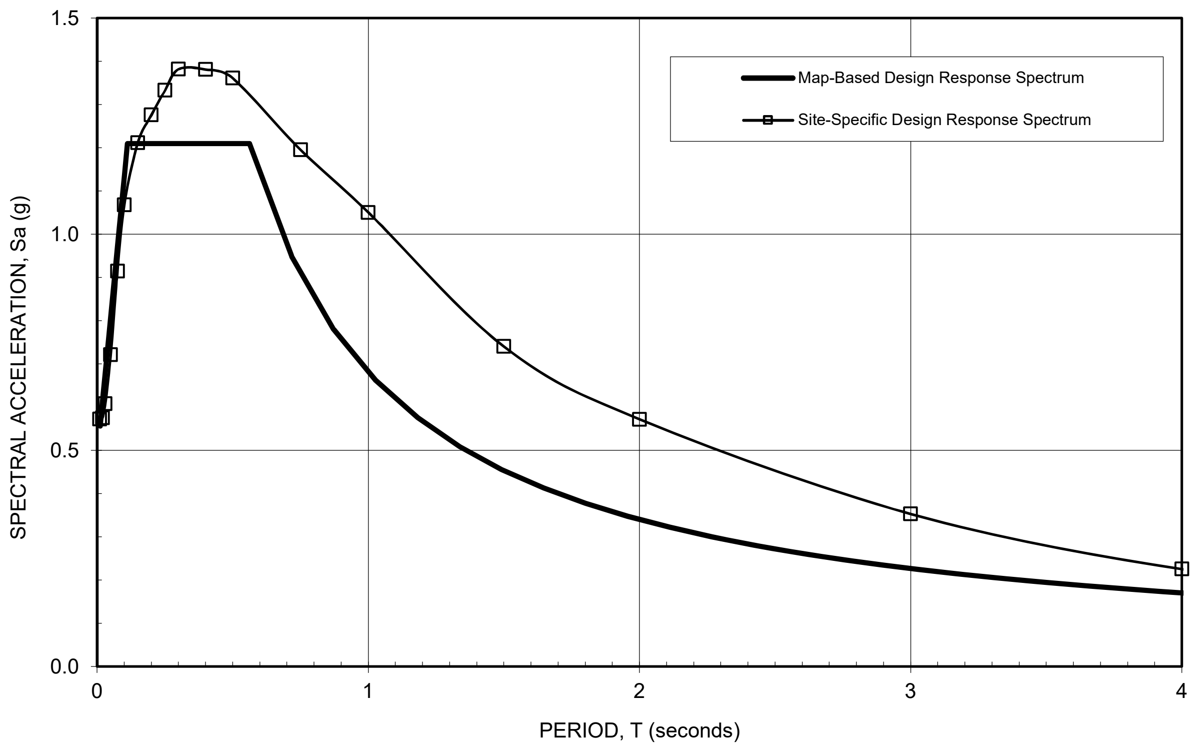
PERIOD (seconds)	MAP-BASED DESIGN RESPONSE SPECTRUM Sa, (g)	SITE-SPECIFIC DESIGN RESPONSE SPECTRUM Sa, (g)
0.010	0.549	0.572
0.050	0.807	0.721
0.100	1.130	1.068
0.150	1.210	1.212
0.200	1.210	1.276
0.250	1.210	1.333
0.300	1.210	1.382
0.400	1.210	1.381
0.500	1.210	1.361
0.750	0.907	1.195
1.000	0.680	1.050

S_{DS} = 1.244	S_{D1} = 1.144	S_{MS} = 1.865	S_{M1} = 1.716	PGA_M = 0.838



NOTES:

1 Site specific design response spectrum is two-thirds of the acceleration response spectrum that is associated with the maximum considered earthquake and is expected to achieve a 1% probability of collapse in a 50-year period (MCE_R). The MCE_R spectrum is computed as the lesser of probablistic and deterministic spectral response accelerations at each period per ASCE 7-16 Section 21.2.3. The site specific design response spectrum conforms with the lower bound limit in ASCE 7-16 Section 21.3.

2 Probabilistic response spectrum is the 5% damped acceleration in the direction of maximum horizontal response associated with a ground motion having a 2% probability of exceedance in 50 years and adjusted for risk of collapse per Method 1 of ASCE 7-16 Section 21.2.1. Spectrum computed using UCERF3 single branch earthquake forecast and the CY14, CB14, and BSSA14 attenuation relationships.

3 Deterministic response spectrum is the 84th percentile, 5% damped spectral reponse acceleration in the maximum horizontal direction computed using CY14, CB14, and BSSA14 attenuation relationships and considering a Mw 6.9 event on the Greenville fault about 6.4 km from the site, a Mw 6.6 event on the Mount Diablo Thrust about 8.1 km, and a Mw 6.4 event on the Mount Diablo Thrust South about 1.6km from the site. Scaled to 1.5*Fa where appropriate.

4 Map-based design response spectrum is computed from mapped spectral ordinates, modified for Site Class D (Stiff Soil) conditions, in accordance with ASCE 7-16 Section 11.4. It is presented for comparison.

FIGURE 7

ACCELERATION RESPONSE SPECTRA

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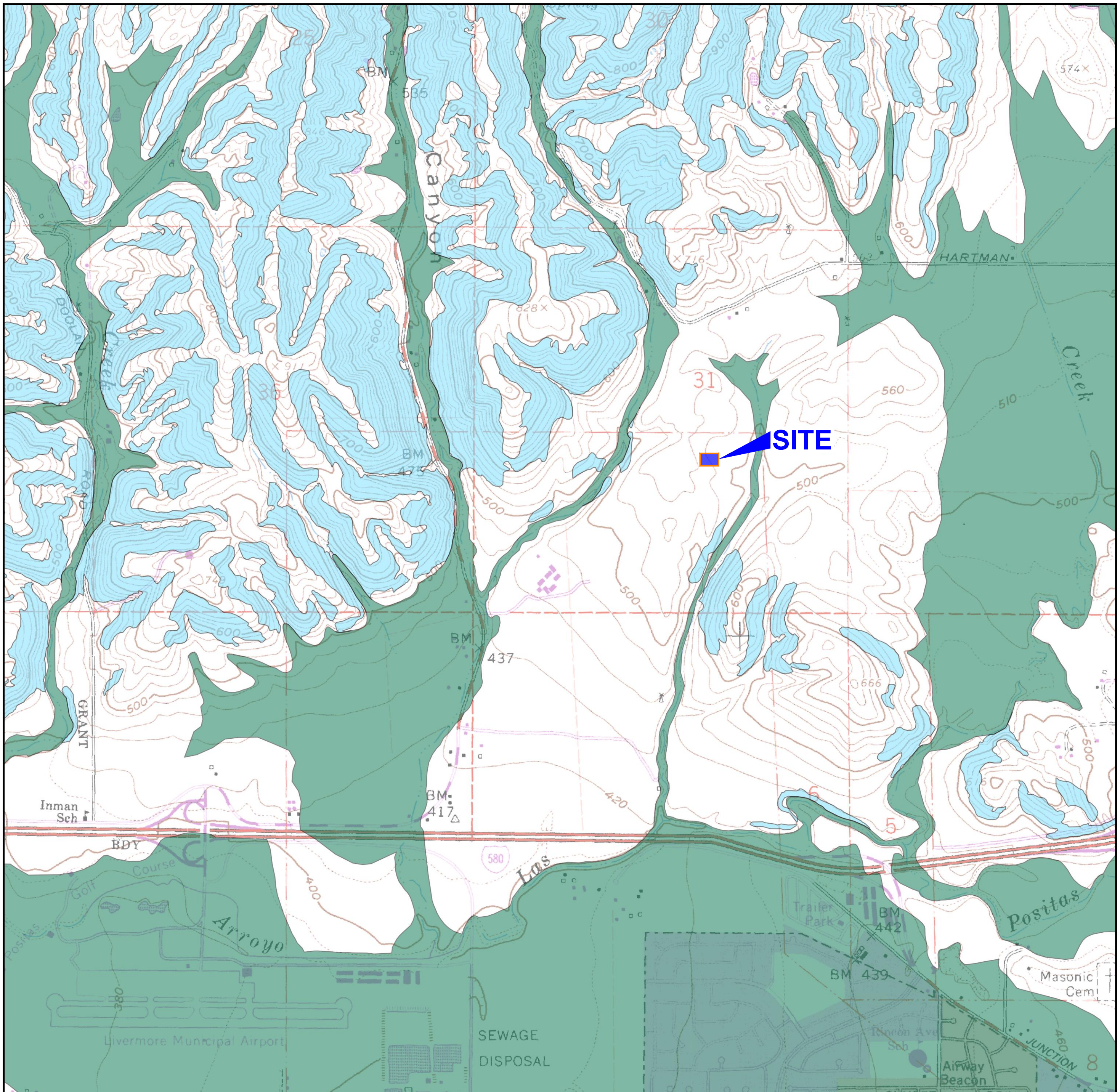
LAS POTISAS COLLEGE - VITICULTURE ALTERNATIVE LOCATION NO. 2
3000 CAMPUS HILL DR., LIVERMORE, CALIFORNIA



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

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FIGURE 8

SEISMIC HAZARD ZONES

LAS POSITAS COLLEGE - VITICULTURE ALTERNATIVE LOCATION NO. 2
3000 CAMPUS HILL DRIVE, LIVERMORE, CALIFORNIA

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401294037 I 01/21

APPENDIX A

Boring Logs

APPENDIX A

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 12 to 18 inches with a 140-pound hammer free-falling 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. 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 methods.

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 brass liners with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with the weight of a hammer in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer, and the number of blows per foot of driving are presented on the boring logs as an index to the relative resistance of the materials sampled. The samples were removed from the sample barrel in the brass liners, sealed, and transported to the laboratory for testing.

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	<u>MAJOR MATERIAL TYPE (SOIL)</u>: 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								
20								The total depth line is a solid line that is drawn at the bottom of the boring.

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BORING LOG

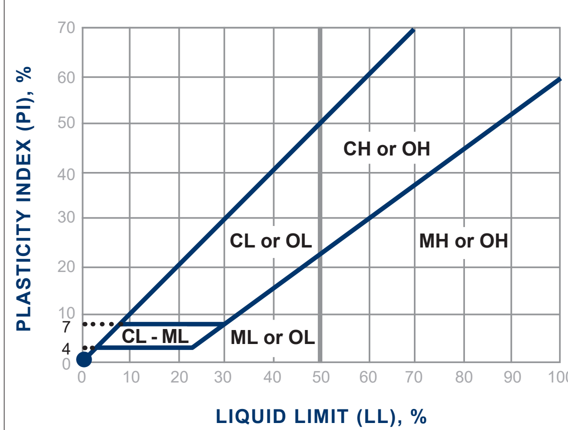
Soil Classification Chart Per ASTM D 2488

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

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USCS METHOD OF SOIL CLASSIFICATION

DATE DRILLED	5/15/08	BORING NO.	B-1
GROUND ELEVATION	532.14' ±MSL	SHEET 1 OF 1	
METHOD OF DRILLING	8" Hollow Stem Auger, Mobile B-53 (Exploration Geoservices)		
DRIVE WEIGHT	140 lbs. (Wire Line)	DROP	30"
SAMPLED BY	KAG	LOGGED BY KAG	REVIEWED BY PKB

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0		■	22 19				CL	FILL: Dark brown and light brown, dry to damp, very stiff, sandy CLAY; little gravel; trace caliche; upper 12" disturbed by discing.
		■	31				SM	Brown, damp, medium dense, silty SAND; little clay; trace caliche.
10							CH	ALLUVIAL OVERBANK DEPOSIT: Dark brown, damp, stiff to very stiff, fat CLAY.
		■	30				CL	ALLUVIUM: Light brown, damp, very stiff, CLAY.
								Total depth = 15 feet. Backfilled with cuttings on 5/15/08. 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.
20								
30								
40								

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LAS POSITAS COLLEGE - VITICULTURE ALT LOCATION NO. 2
3000 CAMPUS HILL DRIVE, LIVERMORE, CALIFORNIA

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
								DATE DRILLED 5/15/08 BORING NO. B-2 GROUND ELEVATION 539.11' ±MSL SHEET 1 OF 1 METHOD OF DRILLING 8" Hollow Stem Auger, Mobile B-53 (Exploration Geoservices) DRIVE WEIGHT 140 lbs. (Wire Line) DROP 30" SAMPLED BY KAG LOGGED BY KAG REVIEWED BY PKB
0							CL	FILL: Yellowish brown, damp, very stiff, CLAY; few gravel; some sand (fine to coarse grained); little silt; upper 12" disturbed by discing.
			21	18.4	98.4		SM	Brown, damp, medium dense, silty SAND; fine to coarse grained; little clay; trace gravel.
			18					
							CH	ALLUVIAL OVERBANK DEPOSIT: Dark grayish brown, damp, very stiff, fat CLAY; trace sand (fine to medium grained); trace caliche.
10			34				CL	ALLUVIUM: Yellowish brown, damp, very stiff, CLAY; few caliche.
			50/5.5"					Hard; trace caliche.
			59					
20								Total depth = 20 feet. Backfilled with cuttings on 5/15/08. 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.
30								
40								

FIGURE A- 2

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LAS POSITAS COLLEGE - VITICULTURE ALT LOCATION NO. 2
3000 CAMPUS HILL DRIVE, LIVERMORE, CALIFORNIA

DATE DRILLED	5/15/08	BORING NO.	B-3
GROUND ELEVATION	541.43' ±MSL	SHEET 1	OF 1
METHOD OF DRILLING	8" Hollow Stem Auger, Mobile B-53 (Exploration Geoservices)		
DRIVE WEIGHT	140 lbs. (Wire Line)	DROP	30"
SAMPLED BY KAG	LOGGED BY KAG	REVIEWED BY	PKB

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							CH	ALLUVIAL OVERBANK DEPOSIT: Dark brown, damp, very stiff, fat CLAY; upper 12" disturbed by discing.
			28					
			13					Brown, stiff.
							CL	ALLUVIUM: Light brown, damp, stiff, sandy CLAY.
							ML	Reddish brown, damp, dense, SILT.
10			50/6"					Very dense.
			60					
								Total depth = 15 feet.
								Backfilled with cuttings on 5/15/08.
20								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.
30								
40								

FIGURE A- 3

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LAS POSITAS COLLEGE - VITICULTURE ALT LOCATION NO. 2
3000 CAMPUS HILL DRIVE, LIVERMORE, CALIFORNIA

DATE DRILLED 11/18/2020 BORING NO. B-4

GROUND ELEVATION 541' + (MSL) SHEET 1 OF 1

METHOD OF DRILLING 4.5" SSA, CME D-50 Track Rig (GeoEx), 3" HA top 2'

DRIVE WEIGHT 140 lbs (automatic trip hammer) DROP 30 inches

SAMPLED BY KCC LOGGED BY KCC REVIEWED BY DCS

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							CL	FILL: Dark brown, moist, soft, lean CLAY; few dry grass; loose and disturbed (cultivated) Light brown; hard; some sand; few gravel
			35	20.0	103.1			
			10					Stiff.
								Dark brown.
			18	19.1	100.8			Light brown to dark brown; very stiff; trace gravel; trace roots.
10								
			18	19.1	103.6			Dark brown. Few gravel; few sand.
			37	16.1	110.0		CL	ALLUVIUM: Light brown, moist, hard, sandy lean CLAY; trace gravel.
20								
			56	23.9	98.6			Some sand; cementation in the matrix.
								Total Depth = 25.0 feet.
								Backfilled with cement grout shortly after drilling on 11/18/2020.
30								Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to relatively low rate of seepage in clay 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, 2020).
40								

DATE DRILLED 11/18/2020 BORING NO. B-5

GROUND ELEVATION 541' + (MSL) SHEET 1 OF 1

METHOD OF DRILLING 4.5" SSA, CME D-50 Track Rig (GeoEx), 3" HA top 2'

DRIVE WEIGHT 140 lbs (automatic trip hammer) DROP 30 inches

SAMPLED BY KCC LOGGED BY KCC REVIEWED BY DCS

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							CL	FILL: Dark brown, moist, soft, lean CLAY; few dry grass; loose and disturbed (cultivated).
			31				SC	Light brown, moist, medium dense, clayey SAND.
							CL	Dark brown, moist, very stiff, lean CLAY.
			26					Some sand.
10			34					Light brown and dark brown; hard; some sand.
			30					Dark brown. Light brown; some sand.
20			26					Grayish brown; few sand.
								Dark brown; some sand.
			43				CL	ALLUVIUM: Light brown, moist, hard, sandy lean CLAY; trace gravel.
30			25					
								Total Depth = 30.0 feet. Backfilled with cement grout shortly after drilling on 11/18/2020.
40								

Notes:
Groundwater, though not encountered at the time of drilling, may rise to a higher level due to relatively low rate of seepage in clay 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, 2020).

DATE DRILLED	11/18/2020	BORING NO.	B-6
GROUND ELEVATION	542' ± (MSL)	SHEET 1 OF 1	
METHOD OF DRILLING	4.5" SSA, CME D-50 Track Rig (GeoEx), 3" HA top 2'		
DRIVE WEIGHT	140 lbs (automatic trip hammer)	DROP	30 inches
SAMPLED BY	KCC	LOGGED BY KCC	REVIEWED BY DCS

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0	Bulk					SC	FILL: Light brown, moist, very loose, clayey SAND with gravel
	Driven	43					Dense.
						CL	Dark brown, moist, very stiff, lean CLAY; trace gravel.
	Driven	25	23.1	102.9			Some sand.
	Driven	39					Gray to light brown; hard; some sand.
10							
	Driven	37					Dark brown.
	Driven	50					
20							
	Driven	28				CL	ALLUVIUM: Light brown, moist, hard, sandy lean CLAY; trace gravel.
	Driven	25					
30							Total Depth = 30.0 feet. Backfilled with cement grout shortly after drilling on 11/18/2020.
40							

Notes:
Groundwater, though not encountered at the time of drilling, may rise to a higher level due to relatively low rate of seepage in clay 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, 2020).

APPENDIX B

Laboratory Testing

APPENDIX B

LABORATORY TESTING

Classification
Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488. Soil classifications are indicated on the logs of the exploratory borings in Appendix A.

In-Place Moisture and Density Tests
The moisture content of samples obtained from the exploratory borings was evaluated in accordance with ASTM D 2216. The dry density of relatively undisturbed samples obtained from the exploratory borings was evaluated in accordance with ASTM D 2937. The test results are presented on the boring logs in Appendix A.

Gradation Analysis
Gradation analysis tests were performed on selected representative soil samples in accordance with ASTM D 422/D 6913. The grain-size distribution curves are shown on Figures B-1 through B-3. These test results were utilized in evaluating the soil classifications in accordance with the USCS.

Atterberg Limits
Tests were performed on selected representative fine-grained soil samples to evaluate the liquid limit, plastic limit, and plasticity index in accordance with ASTM D 4318. These test results were utilized to evaluate the soil classification in accordance with the USCS. The test results and classifications are shown on Figure B-4.

Unconfined Compression Test
An unconfined compression test was performed on a relatively undisturbed sample in accordance with ASTM D 2166. The test results are shown on Figure B-5.

R-Value
The resistance value, or R-value, for site soils was evaluated in general accordance with California Test (CT) 301. Samples were prepared and evaluated for exudation pressure and expansion pressure. The equilibrium R-value is reported as the lesser or more conservative of the two calculated results. The test results are shown on Figure B-6.

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-4	3.5-4.0	32	18	14	--	--	0.08	--	--	60	CL

PERFORMED IN ACCORDANCE WITH ASTM D 422 / D6913

GRADATION TEST RESULTS

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3000 CAMPUS HILL DRIVE, LIVERMORE, CALIFORNIA

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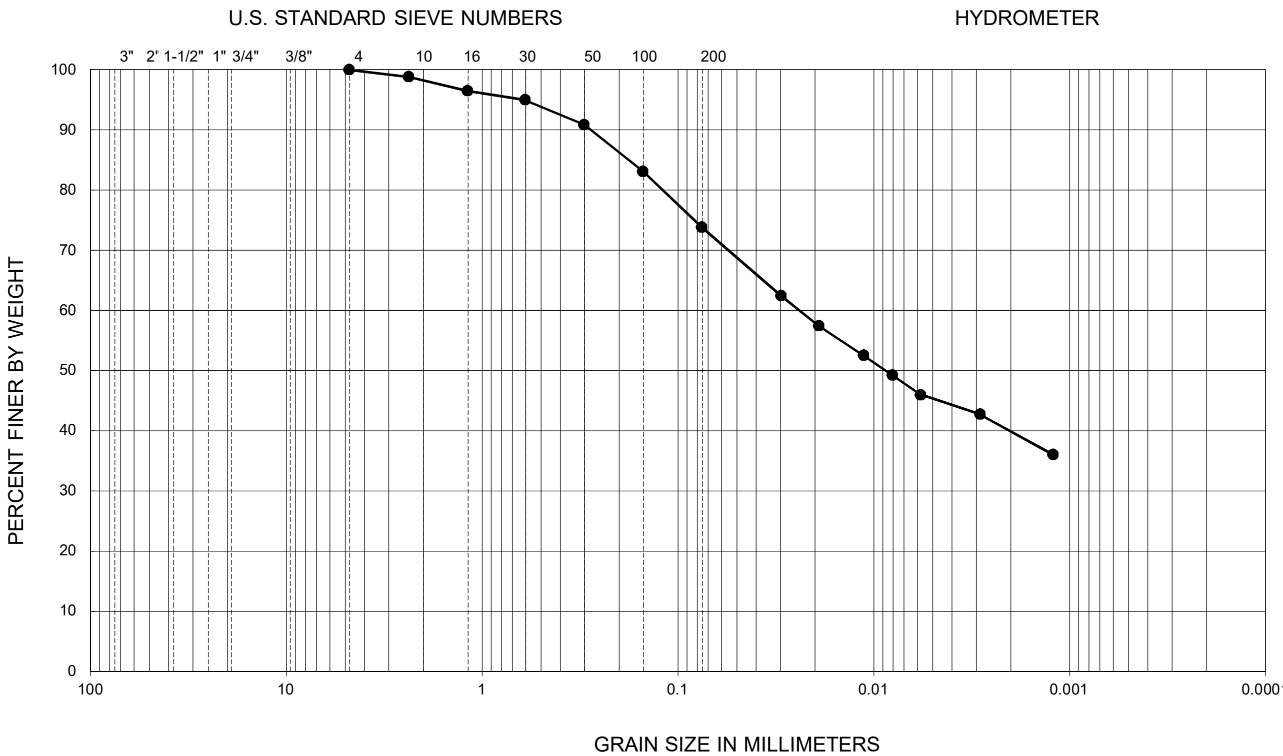
GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY

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-4	19.5-20.0	--	--	--	--	--	0.09	--	--	58	CL

PERFORMED IN ACCORDANCE WITH ASTM D 422 / D6913

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 (%)	USCS
●	B-6	6.0-6.5	--	--	--	--	--	0.025	--	--	74	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE B-3

GRADATION TEST RESULTS

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LAS POSITAS COLLEGE - VITICULTURE ALTERNATE LOCATION NO. 2
3000 CAMPUS HILL DRIVE, LIVERMORE, CALIFORNIAA

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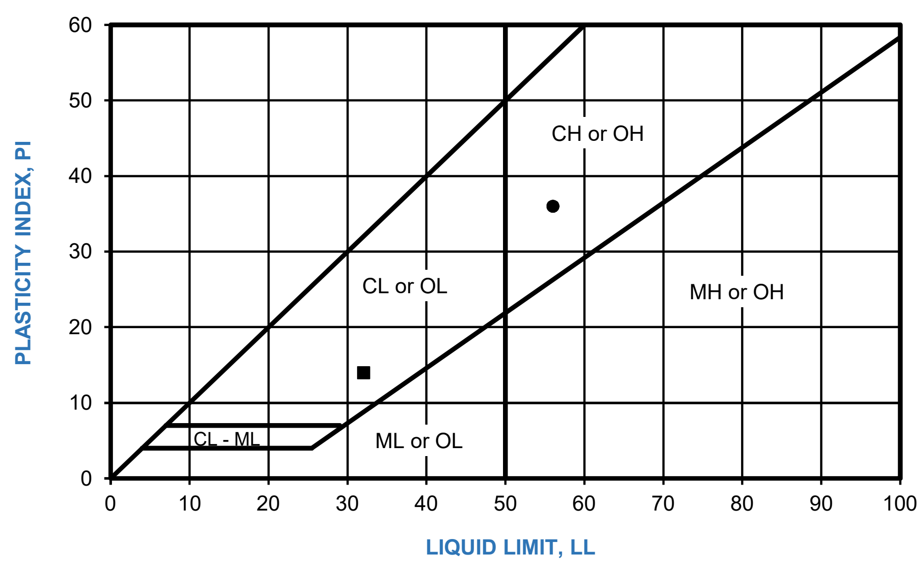
SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS
●	B-3	2.0-2.5	56	20	36	CH	CH
■	B-4	3.5-4.0	32	18	14	CL	CL



PERFORMED IN ACCORDANCE WITH ASTM D 4318

FIGURE B-4

ATTERBERG LIMITS TEST RESULTS

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3000 CAMPUS HILL DRIVE, LIVERMORE, CALIFORNIA

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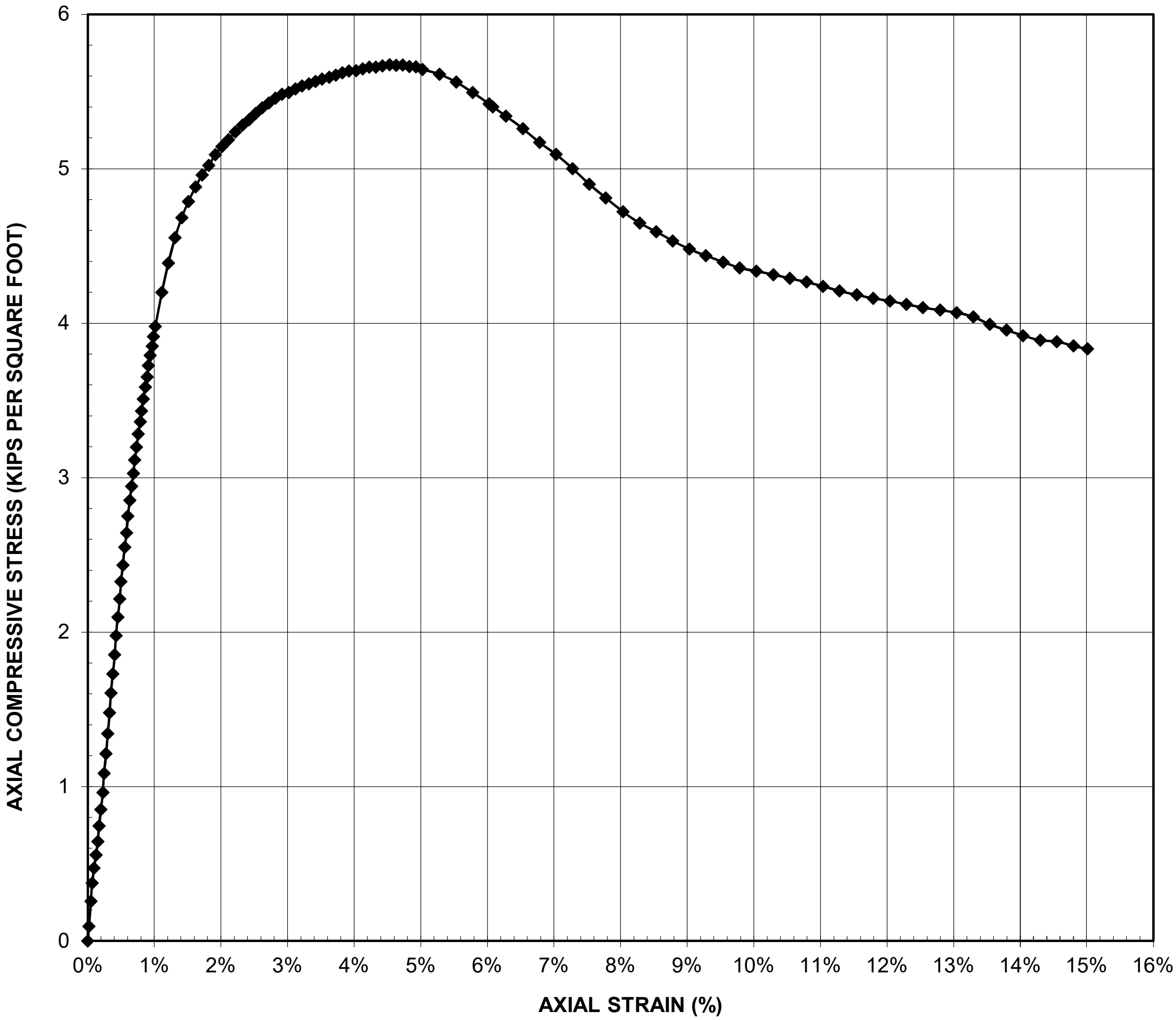

SYMBOL	DESCRIPTION	SOIL TYPE	SAMPLE LOCATION	SAMPLE DEPTH (ft.)	MOISTURE CONTENT *w*, (%)	DRY DENSITY γ_d, (pcf)	STRAIN RATE (%/min.)	UNDRAINED SHEAR STR s_u, (ksf)
◆	Sandy lean clay	CL	B-6	6.0-6.5	23.1	102.9	1.00	2.84

PERFORMED IN ACCORDANCE WITH ASTM D 2166

FIGURE B-5

UNCONFINED COMPRESSION RESULTS

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LAS POSITAS COLLEGE - VITICULTURE ALTERNATE LOCATION NO. 2
3000 CAMPUS HILL DRIVE, LIVERMORE, CALIFORNIA

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	SOIL TYPE	R-VALUE
B-4	0.0-1.0	Dark brown lean clay	<5

PERFORMED IN ACCORDANCE WITH ASTM D 2844/CT 301

FIGURE B-7

R-VALUE TEST RESULTS

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LAS POSITAS COLLEGE - VITICULTURE ALTERNATE LOCATION NO. 2
3000 CAMPUS HILL DRIVE, LIVERMORE, CALIFORNIA

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APPENDIX C

Corrosivity Testing (CERCO Analytical)

California State Certified Laboratory No. 2153

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

17 December, 2020

Job No. 2012028
Cust. No.13270

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

Subject: Project No.: 401294037
Project Name: Viticulture Alternative 2
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Seymour:

Pursuant to your request, CERCO Analytical has analyzed the soil sample submitted on December 2, 2020. 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 65 mg/kg. Because the chloride ion concentration is less than 300 mg/kg, it is determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentration was 22 mg/kg and is determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at this location.

The pH of the soil was 7.69 which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potential range was 300-mV which is indicative of potentially "slightly 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.

Cheryl McMillen for

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

JDH/jdl
Enclosure

California State Certified Laboratory No. 2153

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Client: Ninyo & Moore
Client's Project No.: 401294037
Client's Project Name: Viticulture Alternative 2
Date Sampled: 18-Nov-20
Date Received: 2-Dec-20
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 17-Dec-2020

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)*
2012028-001	B-5 @ 0-1.0'	300	7.69	-	1,100	-	65	22

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	14-Dec-2020	14-Dec-2020	-	14-Dec-2020	-	14-Dec-2020	14-Dec-2020

* Results Reported on "As Received" Basis
N.D. - None Detected

Cheryl McMillen
Laboratory Director

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

APPENDIX D

Geophysical Survey

APPENDIX D

GEOPHYSICAL SURVEY

Scope

A seismic survey using passive surface wave techniques was performed at the site on February 1, 2020. Surveys were performed along one line using passive techniques. The survey line location is noted on Figure 2. The purpose of the study was to evaluate the subsurface shear-wave velocity at a representative location.

Passive Surface Wave Techniques

The passive surface wave method provided a shear wave velocity model to a depth of approximately 100 feet below the ground surface (bgs) and V_{s100} for seismic site classification (CBC, 2019). The passive seismic method carried out included Microtremor Array Measurements (MAM) and consisted of one linear profile of seismic data collection. The following sections provide a summary of the methods and analyses used in our study. The seismic model results are provided on Figure D-1.

Field Methods

A Geode 24–Channel Seismograph (Geometrics Inc., San Jose, California) was used for the MAM survey, with 4.5 Hertz (Hz) vertical component geophone placed at intervals of approximately 10-feet for a total profile length of 230 feet. Approximately twenty records were collected, with a record length of 30 seconds (s) and a 2 millisecond (ms) sampling 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, California) seismic processing software. The dispersive characteristics of surface waves are used to evaluate the subsurface velocity at depth. Longer wavelength (longer-period and lower-frequency) surface waves travel deeper and thus contain more information about deeper velocity structure. Shorter wavelength (shorter-period and higher-frequency) surface waves travel relatively shallow within the earth and thus contain more information about velocity closer to the surface. 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 (Vs100).

- 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 Vs100 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. Our MAM seismic modeling results are provided on Figure D-1. The layered model in Figure D-1 indicates our interpretation of the approximate changes in shear wave velocity vertically with depth across the surveyed location.

The model results indicate a Vs100 value of 984 feet/sec. Accordingly, the site is interpreted to have a Seismic Site Classification of Class D.

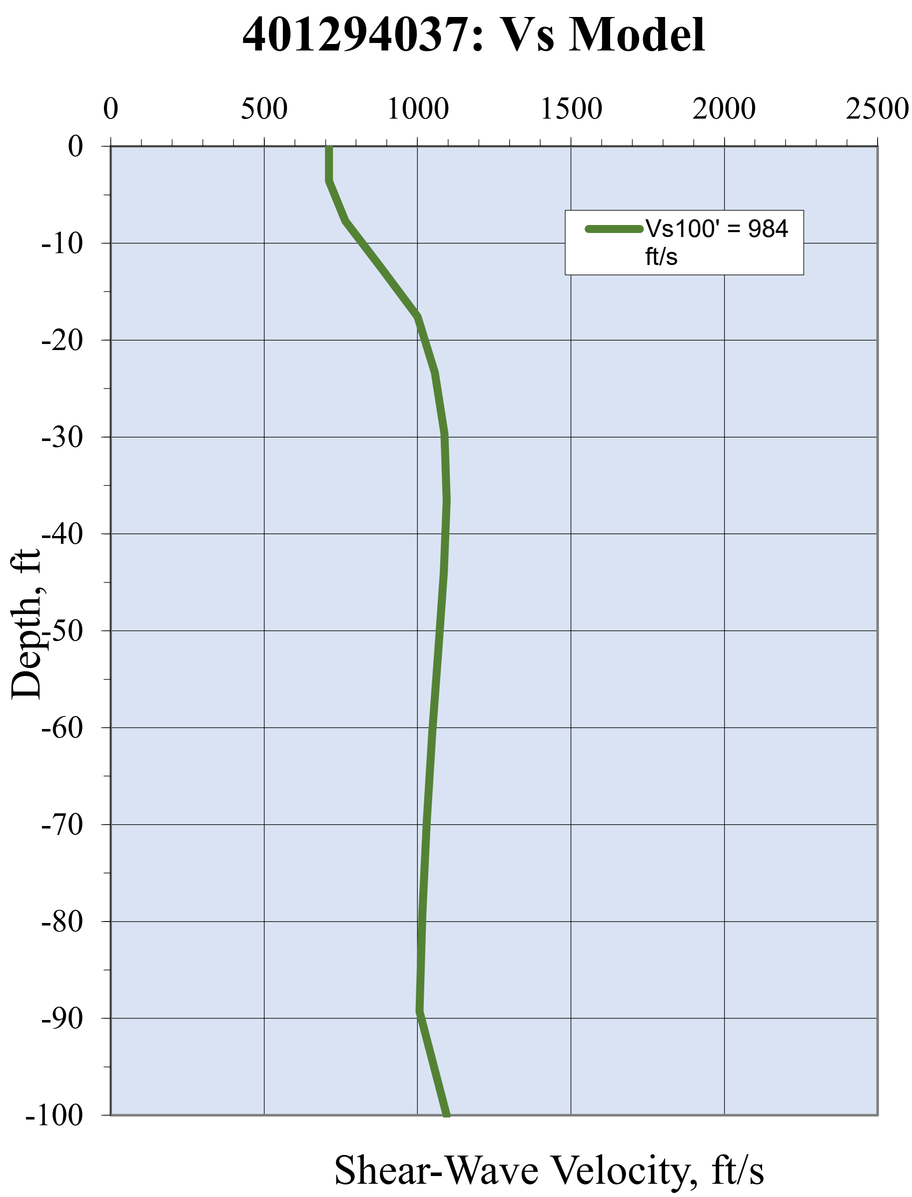


Figure D-1 MAM Shear Wave Velocity Model Results

APPENDIX E

Ground Motion Calculations

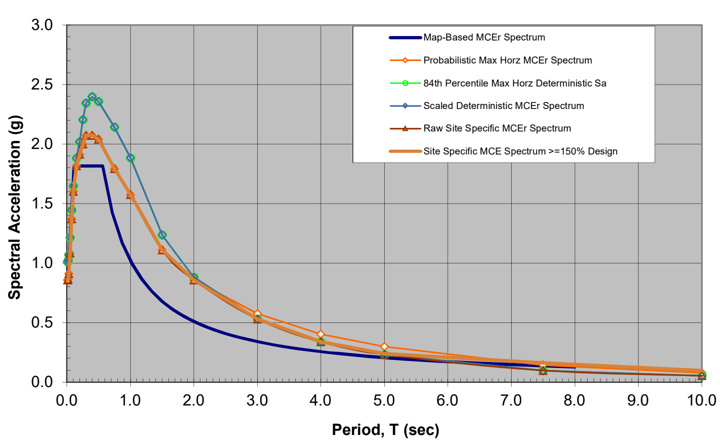
		Spectral Response Mapped Values		Site Coefficients		Spectral Response Adjusted for Site Effects		Design Spectral Response		Period at 0.2*Ts	Period at S_{D1}/S_{DS}	Long-Period Transition
Vs30 (mps)	Site Class	Ss	S1	Fa	Fv	Sms	Sm1	Sds	Sd1	To	Ts	TL
299.9	D	1.815	0.600	1.000	1.700	1.815	1.020	1.210	0.680	0.112	0.562	8.0
Default?	FALSE	Design Spectrum Limit:		1.000	2.500	1.815	1.500	1.210	1.000	0.165	0.826	8.0
		Deterministic Limit:		1.000	2.500							

Risk Coefficients		$(C_{R1}-C_{RS})$ 0.8	PGA Site Coefficient	Mapped Adjusted PGA	ASCE 7-16 Sec. 21.5
C_{RS}	C_{R1}	Ratio	F_{PGA}	PGA_M	PGA
0.931	0.923	-0.01	1.100	0.815	0.838

Probabilistic MCE Spectrum 2% chance of exceedance in 50 years with 5% damping using OPENSHA						Probabilistic MCE_R Spectrum		84th Percentile DSHA Spectral Response in Maximum Horizontal Direction					Deterministic MCE_R Spectrum 1.5Fa Scaled	Site-Specific MCE_R	SS MCE_R >= 150% of SS Design Sa
	Chiou & Youngs (2014) GMRotI50	Campbell & Bozorgnia (2014) GMRotI50	Boore et al (2014) GMRotI50	Max Horz Direction Response to Geomean	Average x Max Horz Direction to Geomean	Risk targeted for 1% chance of collapse in 50 years, using Method 1 (ASCE 7-16 Sec 21.2.1.1)		Chiou & Youngs (2014) 84%ile GM	Campbell & Bozorgnia (2014) 84%ile GM	Boore et al (2014) 84%ile GM	Max Horz Direction Response to Geomean	Average x Max Horz Direction to Geomean	Maximum between 84th percentile Sa and Det. Limit		
Period	CY_Sa	CB_Sa	BA_Sa	Max/Mean	PMH_Sa	Period	PMCER_Sa	CY84_Sa	CB84_Sa	BSSA84_Sa	Max/Mean	DMH84_Sa	DMCER_Sa	SaM	SaM
0.01	0.868	0.663	1.022	1.10	0.922	0.01	0.858	1.136	0.713	0.958	1.10	1.011	1.011	0.858	0.858
0.02	0.880	0.676	1.007	1.10	0.927	0.02	0.863	1.154	0.742	0.944	1.10	1.024	1.024	0.863	0.863
0.03	0.928	0.717	1.060	1.10	0.979	0.03	0.912	1.202	0.791	0.966	1.10	1.069	1.069	0.912	0.912
0.05	1.052	0.854	1.314	1.10	1.162	0.05	1.082	1.312	0.911	1.129	1.10	1.215	1.215	1.082	1.082
0.075	1.266	1.098	1.730	1.10	1.474	0.075	1.372	1.502	1.069	1.416	1.10	1.446	1.446	1.372	1.372
0.1	1.464	1.295	2.017	1.10	1.720	0.1	1.602	1.687	1.170	1.703	1.10	1.648	1.648	1.602	1.602
0.15	1.761	1.447	2.195	1.10	1.953	0.15	1.818	1.984	1.248	2.019	1.10	1.881	1.881	1.818	1.818
0.2	2.013	1.502	2.158	1.10	2.056	0.2	1.914	2.279	1.278	2.127	1.10	2.020	2.020	1.914	1.914
0.25	2.166	1.596	2.085	1.11	2.149	0.25	2.000	2.508	1.449	2.144	1.11	2.206	2.206	2.000	2.000
0.3	2.225	1.711	2.043	1.13	2.229	0.3	2.073	2.630	1.587	2.167	1.13	2.344	2.344	2.073	2.073
0.4	2.191	1.761	1.891	1.15	2.230	0.4	2.072	2.690	1.672	2.013	1.15	2.397	2.397	2.072	2.072
0.5	2.101	1.730	1.806	1.18	2.200	0.5	2.042	2.657	1.646	1.849	1.18	2.359	2.359	2.042	2.042
0.75	1.733	1.511	1.465	1.24	1.937	0.75	1.793	2.292	1.567	1.442	1.24	2.141	2.141	1.793	1.793
1	1.411	1.266	1.267	1.30	1.707	1	1.576	1.887	1.308	1.233	1.30	1.884	1.884	1.576	1.576
1.5	0.974	0.879	0.874	1.33	1.203	1.5	1.110	1.271	0.810	0.790	1.33	1.237	1.237	1.110	1.110
2	0.731	0.660	0.676	1.35	0.929	2	0.858	0.919	0.554	0.545	1.35	0.881	0.881	0.858	0.858
3	0.432	0.433	0.470	1.40	0.623	3	0.575	0.464	0.352	0.331	1.40	0.529	0.529	0.529	0.529
4	0.267	0.286	0.358	1.45	0.437	4	0.403	0.247	0.220	0.233	1.45	0.338	0.338	0.338	0.338
5	0.171	0.207	0.283	1.50	0.323	5	0.298	0.140	0.153	0.172	1.50	0.231	0.231	0.231	0.240
7.5	0.073	0.092	0.166	1.50	0.155	7.5	0.143	0.053	0.063	0.082	1.50	0.097	0.097	0.097	0.160
10	0.039	0.049	0.096	1.50	0.085	10	0.078	0.026	0.036	0.044	1.50	0.052	0.052	0.052	0.096

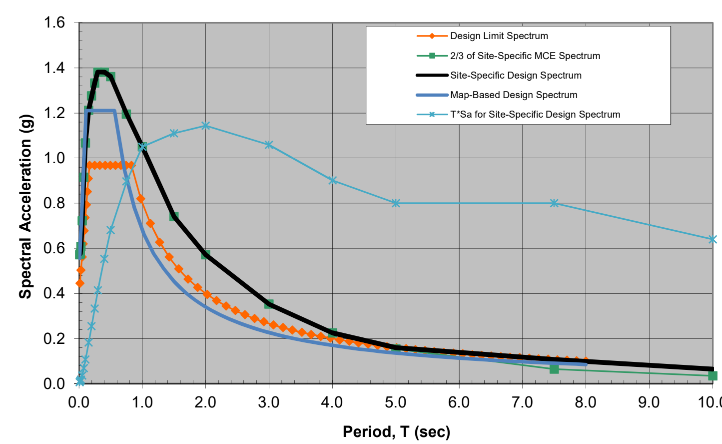
Site-Specific Spectral Response Acceleration Parameters						
	S_{DS}	1.244	S_{MS}	1.865	Risk Category:	II
	S_{D1}	1.144	S_{M1}	1.716	Seismic Design Category:	D

Summary of MCE Response Spectra



Map-Based Design Spectrum		Design Spectrum Limit (Sec 21.3)	2/3*Site-Specific MCE_R Spectrum	Site-Specific Design Spectrum
Period	Sa	Sa	Sa	Sa
0.01	0.549	0.422	0.572	0.572
0.02	0.613	0.457	0.575	0.575
0.03	0.678	0.493	0.608	0.608
0.05	0.807	0.563	0.721	0.721
0.075	0.968	0.651	0.915	0.915
0.1	1.130	0.739	1.068	1.068
0.15	1.210	0.914	1.212	1.212
0.2	1.210	0.968	1.276	1.276
0.25	1.210	0.968	1.333	1.333
0.3	1.210	0.968	1.382	1.382
0.4	1.210	0.968	1.381	1.381
0.5	1.210	0.968	1.361	1.361
0.75	0.907	0.968	1.195	1.195
1	0.680	0.800	1.050	1.050
1.5	0.453	0.533	0.740	0.740
2	0.340	0.400	0.572	0.572
3	0.227	0.267	0.353	0.353
4	0.170	0.200	0.225	0.225
5	0.136	0.160	0.154	0.160
7.5	0.091	0.107	0.065	0.107
10	0.054	0.064	0.035	0.064

Site-Specific Design Response Spectrum



X-Axis: Period (sec)
Y-Axis: SA (g)
Number of Data Sets: 4

DATASET #1

Name:
Num Points: 24
Info:
IMR Param List:

IMR = Chiou & Youngs (2014); Gaussian Truncation = None; Tectonic Region = Active Shallow Crust;
Component = RotD50; Std Dev Type = Total; Additional Epistemic Uncertainty = null

Site Param List:

Longitude = -121.794123; Latitude = 37.714984; Vs30 = 299.9; Vs30 Type = Measured; Depth 1.0 km/sec = 607.0; Depth 2.5 km/sec = 3.207

IML/Prob Param List:

Map Type = IML@Prob; Probability = 0.02

Forecast Param List:

Eqk Rup Forecast = UCERF3 Single Branch ERF; Fault Model = Fault Model 3.1;
Deformation Model = Average Block Model; Scaling Relationship = Ellsworth B;
Slip Along Rupture Model (Dsr) = Tapered Ends; Inversion Model = Characteristic (Constrained);
Total Mag 5 Rate = RATE_6p5; MMax Off Fault = MAG_7p3; Moment Rate Fixes = NONE;
Spatial Seismicity PDF = UCERF2; Apply Aftershock Filter = false; Aleatory Mag-Area StdDev = 0.0;
Background Seismicity = Include; Treat Background Seismicity As = Point Sources;
Fault Grid Spacing = 1.0; Probability Model = Poisson

TimeSpan Param List:

Duration = 50.0

Maximum Distance = 200.0; Pt Src Dist Corr = None

X, Y Data:
0.01   0.8681104
0.02   0.87992465
0.03   0.9281602
0.04   0.9808062
0.05   1.0518602

```
0.075   1.2663296
0.1     1.4644854
0.12    1.6062919
0.15    1.7609613
0.17    1.860161
0.2     2.0125618
0.25    2.1663897
0.3     2.2245038
0.4     2.190593
0.5     2.101092
0.75    1.7331814
1.0     1.4108397
1.5     0.97378516
2.0     0.73128957
3.0     0.43237144
4.0     0.26742998
5.0     0.17087811
7.5     0.07254352
10.0    0.038705517
```

DATASET #2

Name:
Num Points: 21
Info:
IMR Param List:

IMR = Campbell & Bozorgnia (2014); Gaussian Truncation = None;
Tectonic Region = Active Shallow Crust; Component = RotD50; Std Dev Type = Total;
Additional Epistemic Uncertainty = null

Site Param List:

Longitude = -121.794123; Latitude = 37.714984; Vs30 = 299.9; Vs30 Type = Measured;
Depth 1.0 km/sec = 607.0; Depth 2.5 km/sec = 3.2

IML/Prob Param List:

Map Type = IML@Prob; Probability = 0.02

Forecast Param List:

Eqk Rup Forecast = UCERF3 Single Branch ERF; Fault Model = Fault Model 3.1;
Deformation Model = Average Block Model; Scaling Relationship = Ellsworth B;
Slip Along Rupture Model (Dsr) = Tapered Ends; Inversion Model = Characteristic
(Constrained);
Total Mag 5 Rate = RATE_6p5; MMax Off Fault = MAG_7p3; Moment Rate Fixes = NONE;
Spatial Seismicity PDF = UCERF2; Apply Aftershock Filter = false; Aleatory Mag-Area

StdDev = 0.0;
Background Seismicity = Include; Treat Background Seismicity As = Point Sources;
Fault Grid Spacing = 1.0; Probability Model = Poisson

TimeSpan Param List:

Duration = 50.0

Maximum Distance = 200.0; Pt Src Dist Corr = None

X, Y Data:

0.01	0.66295874
0.02	0.6758375
0.03	0.7172501
0.05	0.85354817
0.075	1.097739
0.1	1.2951787
0.15	1.4474838
0.2	1.5023304
0.25	1.595553
0.3	1.7106943
0.4	1.7606803
0.5	1.7295703
0.75	1.5114663
1.0	1.2660912
1.5	0.8792805
2.0	0.660396
3.0	0.43264526
4.0	0.28591475
5.0	0.20697875
7.5	0.0923808
10.0	0.048547927

DATASET #3

Name:
Num Points: 21
Info:
IMR Param List:

IMR = Boore, Stewart, Seyhan & Atkinson (2014); Gaussian Truncation = None;
Tectonic Region = Active Shallow Crust; Component = RotD50; Std Dev Type = Total;
Additional Epistemic Uncertainty = null

Site Param List:

Longitude = -121.794123; Latitude = 37.714984; Vs30 = 299.9; Vs30 Type = Measured;
Depth 1.0 km/sec = 607.0; Depth 2.5 km/sec = 3.207

IML/Prob Param List:

Map Type = IML@Prob; Probability = 0.02

Forecast Param List:

Eqk Rup Forecast = UCERF3 Single Branch ERF; Fault Model = Fault Model 3.1; Deformation Model = Average Block Model; Scaling Relationship = Ellsworth B; Slip Along Rupture Model (Dsr) = Tapered Ends; Inversion Model = Characteristic (Constrained); Total Mag 5 Rate = RATE_6p5; MMax Off Fault = MAG_7p3; Moment Rate Fixes = NONE; Spatial Seismicity PDF = UCERF2; Apply Aftershock Filter = false; Aleatory Mag-Area StdDev = 0.0; Background Seismicity = Include; Treat Background Seismicity As = Point Sources; Fault Grid Spacing = 1.0; Probability Model = Poisson

TimeSpan Param List:

Duration = 50.0

Maximum Distance = 200.0; Pt Src Dist Corr = None

X, Y Data:

0.01	1.0216444
0.02	1.0069838
0.03	1.0604197
0.05	1.3141947
0.075	1.7301171
0.1	2.0166485
0.15	2.1948633
0.2	2.158365
0.25	2.0850265
0.3	2.0430202
0.4	1.8907502
0.5	1.8061986
0.75	1.464851
1.0	1.2673239
1.5	0.8740721
2.0	0.67562217
3.0	0.47000256
4.0	0.35776812
5.0	0.2831005
7.5	0.1659332
10.0	0.095916554

DATASET #4

Name:
Num Points: 22
Info:
IMR Param List:

IMR = Abrahamson, Silva & Kamai (2014); Gaussian Truncation = None;
Tectonic Region = Active Shallow Crust; Component = RotD50; Std Dev Type = Total;
Additional Epistemic Uncertainty = null

Site Param List:

Longitude = -121.794123; Latitude = 37.714984; Vs30 = 760.0; Vs30 Type = Measured;
Depth 1.0 km/sec = 607.0; Depth 2.5 km/sec = 3.207

IML/Prob Param List:

Map Type = IML@Prob; Probability = 0.02

Forecast Param List:

Eqk Rup Forecast = UCERF3 Single Branch ERF; Fault Model = Fault Model 3.1;
Deformation Model = Average Block Model; Scaling Relationship = Ellsworth B;
Slip Along Rupture Model (Dsr) = Tapered Ends; Inversion Model = Characteristic (Constrained);
Total Mag 5 Rate = RATE_6p5; MMax Off Fault = MAG_7p3; Moment Rate Fixes = NONE;
Spatial Seismicity PDF = UCERF2; Apply Aftershock Filter = false; Aleatory Mag-Area StdDev = 0.0;
Background Seismicity = Include; Treat Background Seismicity As = Point Sources;
Fault Grid Spacing = 1.0; Probability Model = Poisson

TimeSpan Param List:

Duration = 50.0

Maximum Distance = 200.0; Pt Src Dist Corr = None

X, Y Data:
```
0.01    0.7357724
0.02    0.75466037
0.03    0.80214065
0.05    0.9190473
0.075   1.18909
0.1     1.4409226
0.15    1.7682319
0.2     1.8198775
0.25    1.7391365
0.3     1.6115746
0.4     1.337436
0.5     1.2061359
0.75    0.924627
```

1.0	0.7177821
1.5	0.4829636
2.0	0.35350412
3.0	0.21503936
4.0	0.14687762
5.0	0.11251091
6.0	0.086804636
7.5	0.06776052
10.0	0.048704877

PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

WEIGHTED AVERAGE of 2014 NGA WEST-2 GMPEs

Last updated: 04 14 15

by Emel Seyhan, PhD, PEER & UCLA -- email: emel.seyhan@gmail.com, peer_center@berkeley.edu

This excel file will be updated as necessary on the PEER website to fix any typos or other errors. Please check the website frequently for new versions at: http://peer.berkeley.edu/ngawest2/databases/

Legend	Pre-defined option	Main input variable	Calculated variable	Input var. flag	Internal variable

GMPE averaging	Geometric	*Weighted average of the natural logarithm of the spectral values*

GMPEs	ASK14	BSSA14	CB14	CY14	I14
Weight	0	0.333333333	0.333333	0.333333	0

# of std. dev.	1	
Damping ratio (%)	5	*Modification factors are calculated in Sheet DSF*

ASK14 Abrahamson & Silva & Kamai 2014 NGA West-2 Model
BSSA14 Boore & Stewart & Seyhan & Atkinson 2014 NGA West-2 Model
CB14 Campbell & Bozorgnia 2014 NGA West-2 Model
CY14 Chiou & Youngs 2014 NGA West-2 Model
I14 Idriss 2014 NGA West-2 Model

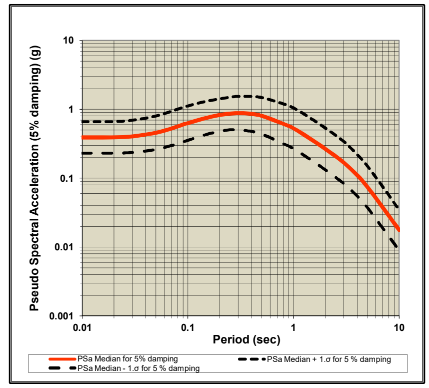
Input variables	Errors and warnings
M_w: 6.9	
R_{RUP} (km): 6.4	
R_{JB} (km): 6.4	
R_X (km): 6.4	
Ry0 (km): 999	*If unknown use 999*
V_{S30} (m/sec): 299.9	
U (BSSA13): 0	*1: Unspecified fault mech.*
F_{RV}: 0	*1: reverse fault*
F_{NM}: 0	*1: normal fault*
F_{HW}: 0	*1: hanging wall side*
Dip (deg): 90	
Z_{TOR} (km): 0	*If unknown use 999*
Z_{HYP} (km): 999	*If unknown use 999*
$Z_{1.0}$ (km): 0.607	*If unknown use 999*
$Z_{2.5}$ (km): 3.207	*If unknown use 999*
W (km): 999	*If unknown use 999*
Vs30Flag: measured	*Choose options for V_{s30} from the list*
F_{AS}: no	*Aftershock effect is not applicable.*
Region: California	*Choose region from the list*

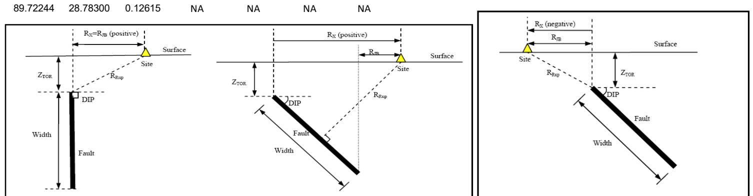
Calculated Variables/Flags

ΔDPP: 0	*Always 0 for median calcs.*
PGA_r (g): 0.305	
Z_{BOT} (km) (CB14): 15	*Enter for default W calcs*
SS: 1	*auto calculated*
$V_{s30Flag}$: 1	*measured*
F_{AS}: 0	*Aftershock effect is not applicable.*
Region: 0	*California*
Option for Sa value: 1	*Weighted average of the natural logarithm of the spectral values*

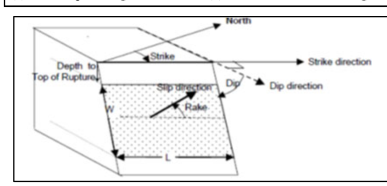
RotD50 Horizontal Component of PGA, PGV and *IMs*

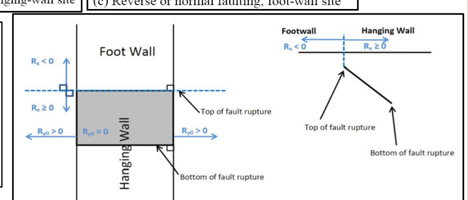
GMP	*T* (s)	Baseline: 5% Damping				User defined: 5% Damping			
		PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	0.01	0.39080	0.66076	0.23113	0.00097	0.39080	0.66076	0.23113	0.00097
	0.02	0.39209	0.66449	0.23136	0.00389	0.39209	0.66449	0.23136	0.00389
	0.03	0.40530	0.69455	0.23651	0.00905	0.40530	0.69455	0.23651	0.00905
	0.05	0.45722	0.79954	0.26146	0.02837	0.45722	0.79954	0.26146	0.02837
	0.075	0.54504	0.96945	0.30643	0.07611	0.54613	0.97138	0.30705	0.07626
	0.1	0.62956	1.11894	0.35421	0.15628	0.63144	1.12230	0.35527	0.15675
	0.15	0.75170	1.30841	0.43186	0.41985	0.75320	1.31102	0.43273	0.42069
	0.2	0.82275	1.41655	0.47786	0.81695	0.82439	1.41938	0.47882	0.81858
	0.25	0.86738	1.49580	0.50297	1.34572	0.86998	1.50028	0.50448	1.34976
	0.3	0.88091	1.54087	0.50361	1.96807	0.88179	1.54242	0.50412	1.97004
	0.4	0.85690	1.53679	0.47780	3.40342	0.85776	1.53833	0.47827	3.40683
	0.5	0.80469	1.48134	0.43712	4.99382	0.80549	1.48282	0.43756	4.99882
	0.75	0.64117	1.23940	0.33169	8.95291	0.64117	1.23940	0.33169	8.95291
	1	0.53097	1.04731	0.26919	13.18053	0.53043	1.04627	0.26892	13.16735
	1.5	0.35890	0.71822	0.17935	20.04584	0.35926	0.71894	0.17953	20.06589
	2	0.26682	0.53584	0.13286	26.49345	0.26628	0.53477	0.13259	26.44047
	3	0.16845	0.34011	0.08343	37.63326	0.16828	0.33977	0.08334	37.59562
	4	0.11058	0.22105	0.05532	43.92109	0.11047	0.22083	0.05526	43.87717
	5	0.07543	0.15109	0.03766	46.81434	0.07513	0.15048	0.03751	46.62709
	7.5	0.03257	0.06488	0.01636	45.48396	0.03251	0.06475	0.01632	45.39299
	10	0.01776	0.03482	0.00905	44.07570	0.01768	0.03468	0.00902	43.89940
PGA (g)	0	0.38803	0.65541	0.22973	0.00096	0.38803	0.65541	0.22973	0.00096
PGV (cm/s)	-1	50.81811	89.72244	28.78300	0.12615	NA	NA	NA	NA





(a) Strike slip faulting (b) Reverse or normal faulting, hanging-wall site (c) Reverse or normal faulting, foot-wall site





Courtesy: Jennifer Donahue

Definition of Parameters

Damping ratio = Viscous damping ratio (%) See Sanaz et al. (2012) PEER Report
PSA = Pseudo-absolute acceleration response spectrum (g)
PGA = Peak ground acceleration (g)
PGV = Peak ground velocity (cm/s)
S_d = Relative displacement response spectrum (cm)
M_w = Moment magnitude
R_{RUP} = Closest distance to coseismic rupture (km), used in ASK13, CB13 and CY13. See Figures a, b and c for illustration
R_{JB} = Closest distance to surface projection of coseismic rupture (km). See Figures a, b and c for illustration
R_X = Horizontal distance from top of rupture measured perpendicular to fault strike (km). See Figures a, b and c for illustration
R_{y0} = The horizontal distance off the end of the rupture measured parallel to strike (km)
V_{S30} = The average shear-wave velocity (m/s) over a subsurface depth of 30 m
U = Unspecified-mechanism factor: 1 for unspecified; 0 otherwise
F_{RV} = Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
F_{NM} = Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal
F_{HW} = Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise
Dip = Average dip of rupture plane (degrees)
Z_{TOR} = Depth to top of coseismic rupture (km)
Z_{HYP} = Hypocentral depth from the earthquake
$Z_{1.0}$ = Depth to Vs=1 km/sec
$Z_{2.5}$ = Depth to Vs=2.5 km/sec
W = Fault rupture width (km)
$V_{s30flag}$ = 1 for measured, 0 for inferred Vs30
F_{AS} = 0 for mainshock; 1 for aftershock
Region = Specific regions considered in the models, Click on Region to see codes
ΔDPP = Directivity term, direct point parameter; uses 0 for median predictions
PGA_r (g) = Peak ground acceleration on rock (g), this specific cell is updated in the cell for BSSA14 and CB14, for others it is taken account for in the macros
Z_{BOT} (km) = The depth to the bottom of the seismogenic crust
Z_{BOR} (km) = The depth to the bottom of the rupture plane
SS = 1 for strike slip, automatically updated in the cell

Input variables with defaults (If entered 999 as input):

Red colored value: The value is used in the code when input is unknown

DEFAULTs	USER defined	ASK14	BSSA14	CB14	CY14	I14
W (km)	999.00			**14.769**		
$Z_{1.0}$ (km)	0.607	0.607			0.459	
$\delta Z_{1.0}$ (km)	0.148		**0.148**			
$Z_{2.5}$ (V_{S30}=1100)(km)	3.207			0.398		
$Z_{2.5}$ (V_{S30})(km)	3.207			1.758		
Z_{hyp} (km)	999.00			**10.458**		
Z_{tor} (km)	0.00			0.231	0.231	
Z_{BOR} (km)	-			15.000		

ACKNOWLEDGEMENTS

Nick Gregor, Bechtel
Silvia Mazzoni, Consultant

All NGA West-2 participants are acknowledged for their constructive comments and feedback.

PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

WEIGHTED AVERAGE of 2014 NGA WEST-2 GMPEs

Last updated: 04 14 15

by Emel Seyhan, PhD, PEER & UCLA -- email: emel.seyhan@gmail.com, peer_center@berkeley.edu

This excel file will be updated as necessary on the PEER website to fix any typos or other errors. Please check the website frequently for new versions at: http://peer.berkeley.edu/ngawest2/databases/

Legend	Pre-defined option	Main input variable	Calculated variable	Input var. flag	Internal variable

GMPE averaging	Geometric	*Weighted average of the natural logarithm of the spectral values*

GMPEs	ASK14	BSSA14	CB14	CY14	I14
Weight	0	0.333333333	0.333333	0.333333	0

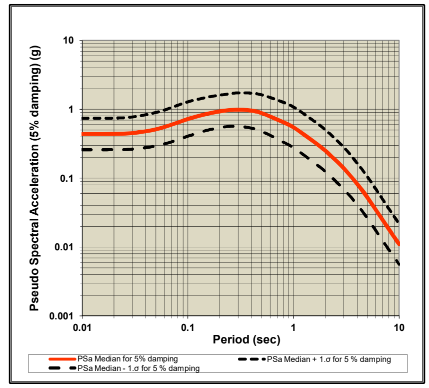
# of std. dev.	1	
Damping ratio (%)	5	*Modification factors are calculated in Sheet DSF*

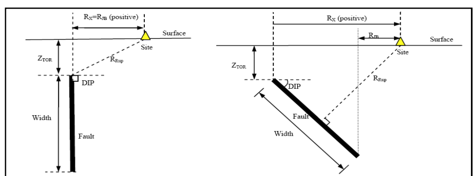
ASK14 Abrahamson & Silva & Kamai 2014 NGA West-2 Model
BSSA14 Boore & Stewart & Seyhan & Atkinson 2014 NGA West-2 Model
CB14 Campbell & Bozorgnia 2014 NGA West-2 Model
CY14 Chiou & Youngs 2014 NGA West-2 Model
I14 Idriss 2014 NGA West-2 Model

RotD50 Horizontal Component of PGA, PGV and *IMs*

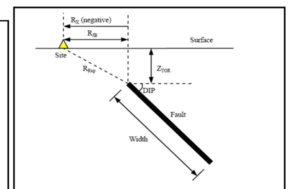
Input variables	Value	Errors and warnings
M_w	6.6	
R_{RUP} (km)	8.01	
R_{JB} (km)	0.01	
R_X (km)	0.36	
Ry0 (km)	999	*If unknown use 999*
V_{S30} (m/sec)	299.9	
U (BSSA13)	0	*1: Unspecified fault mech.*
F_{RV}	1	*1: reverse fault*
F_{NM}	0	*1: normal fault*
F_{HW}	1	*1: hanging wall side*
Dip (deg)	38	
Z_{TOR} (km)	8	*If unknown use 999*
Z_{HYP} (km)	999	*If unknown use 999*
$Z_{1.0}$ (km)	0.607	*If unknown use 999*
$Z_{2.5}$ (km)	3.207	*If unknown use 999*
W (km)	999	*If unknown use 999*
Vs30Flag	measured	*Choose options for V_{s30} from the list*
F_{AS}	no	*Aftershock effect is not applicable.*
Region	California	*Choose region from the list*

GMP	*T* (s)	Baseline: 5% Damping				User defined: 5% Damping			
		PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	0.01	0.43834	0.74117	0.25924	0.00109	0.43834	0.74117	0.25924	0.00109
	0.02	0.43997	0.74565	0.25960	0.00437	0.43997	0.74565	0.25960	0.00437
	0.03	0.45328	0.77679	0.26450	0.01013	0.45328	0.77679	0.26450	0.01013
	0.05	0.51361	0.89802	0.29375	0.03187	0.51361	0.89802	0.29375	0.03187
	0.075	0.61909	1.10077	0.34818	0.08644	0.62032	1.10297	0.34888	0.08662
	0.1	0.72209	1.28281	0.40646	0.17925	0.72425	1.28665	0.40768	0.17979
	0.15	0.85634	1.48984	0.49221	0.47829	0.85805	1.49282	0.49320	0.47925
	0.2	0.93197	1.60397	0.54151	0.92539	0.93290	1.60558	0.54205	0.92632
	0.25	0.97418	1.67943	0.56509	1.51142	0.97613	1.68279	0.56622	1.51444
	0.3	0.99083	1.73272	0.56660	2.21365	0.99182	1.73445	0.56716	2.21587
	0.4	0.95591	1.71410	0.53309	3.79667	0.95687	1.71581	0.53362	3.80047
	0.5	0.88164	1.62288	0.47895	5.47138	0.88252	1.62450	0.47943	5.47685
	0.75	0.67939	1.31332	0.35146	9.48658	0.67939	1.31332	0.35146	9.48658
	1	0.54755	1.08007	0.27758	13.59211	0.54755	1.08007	0.27758	13.59211
	1.5	0.35284	0.70611	0.17631	19.70708	0.35319	0.70682	0.17648	19.72679
	2	0.25168	0.50546	0.12532	24.99028	0.25117	0.50444	0.12507	24.94030
	3	0.13921	0.28110	0.06894	31.10220	0.13907	0.28082	0.06888	31.07109
	4	0.08284	0.16561	0.04144	32.90406	0.08276	0.16544	0.04140	32.87116
	5	0.05294	0.10602	0.02643	32.85134	0.05272	0.10560	0.02632	32.71994
	7.5	0.02104	0.04191	0.01057	29.38438	0.02100	0.04183	0.01054	29.32561
	10	0.01109	0.02175	0.00565	27.53057	0.01105	0.02166	0.00563	27.42044
PGA (g)	0	0.43509	0.73493	0.25758	0.00108	0.43509	0.73493	0.25758	0.00108
PGV (cm/s)	-1	49.87598	88.07228	28.24513	0.12381	NA	NA	NA	NA

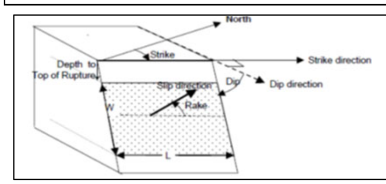


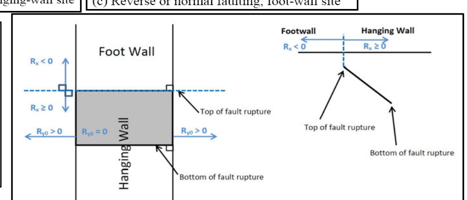


(a) Strike slip faulting (b) Reverse or normal faulting, hanging-wall site



(c) Reverse or normal faulting, foot-wall site





Courtesy: Jennifer Donahue

Calculated Variables/Flags

Variable	Value	Note
ΔDPP	0	*Always 0 for median calcs.*
PGA_r (g)	0.424	
Z_{BOT} (km) (CB14)	15	*Enter for default W calcs*
SS	0	*auto calculated*
$V_{s30Flag}$	1	*measured*
F_{AS}	0	*Aftershock effect is not applicable.*
Region	0	*California*
Option for Sa value	1	*Weighted average of the natural logarithm of the spectral values*

Definition of Parameters

Damping ratio = Viscous damping ratio (%) See Sanaz et al. (2012) PEER Report
PSA = Pseudo-absolute acceleration response spectrum (g)
PGA = Peak ground acceleration (g)
PGV = Peak ground velocity (cm/s)
S_d = Relative displacement response spectrum (cm)
M_w = Moment magnitude
R_{RUP} = Closest distance to coseismic rupture (km), used in ASK13, CB13 and CY13. See Figures a, b and c for illustation
R_{JB} = Closest distance to surface projection of coseismic rupture (km). See Figures a, b and c for illustration
R_X = Horizontal distance from top of rupture measured perpendicular to fault strike (km). See Figures a, b and c for illustration
R_{y0} = The horizontal distance off the end of the rupture measured parallel to strike (km)
V_{S30} = The average shear-wave velocity (m/s) over a subsurface depth of 30 m
U = Unspecified-mechanism factor: 1 for unspecified; 0 otherwise
F_{RV} = Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
F_{NM} = Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal
F_{HW} = Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise
Dip = Average dip of rupture plane (degrees)
Z_{TOR} = Depth to top of coseismic rupture (km)
Z_{HYP} = Hypocentral depth from the earthquake
$Z_{1.0}$ = Depth to Vs=1 km/sec
$Z_{2.5}$ = Depth to Vs=2.5 km/sec
W = Fault rupture width (km)
$V_{s30flag}$ = 1 for measured, 0 for inferred Vs30
F_{AS} = 0 for mainshock; 1 for aftershock
Region = Specific regions considered in the models, Click on Region to see codes
ΔDPP = Directivity term, direct point parameter; uses 0 for median predictions
PGA_r (g) = Peak ground acceleration on rock (g), this specific cell is updated in the cell for BSSA14 and CB14, for others it is taken account for in the macros
Z_{BOT} (km) = The depth to the bottom of the seismogenic crust
Z_{BOR} (km) = The depth to the bottom of the rupture plane
SS = 1 for strike slip, automatically updated in the cell

Input variables with defaults (If entered 999 as input):

Red colored value: The value is used in the code when input is unknown

DEFAULTs	USER defined	ASK14	BSSA14	CB14	CY14	I14
W (km)	999.00			19.199		
$Z_{1.0}$ (km)	0.607	0.607			0.459	
$\delta Z_{1.0}$ (km)	0.148		0.148			
$Z_{2.5}$ (V_{S30}=1100)(km)	3.207			0.398		
$Z_{2.5}$ (V_{S30})(km)	3.207			1.758		
Z_{hyp} (km)	999.00			11.252		
Z_{tor} (km)	8.00			3.180	3.180	
Z_{BOR} (km)	-			15.000		

ACKNOWLEDGEMENTS

Nick Gregor, Bechtel
Silvia Mazzoni, Consultant

All NGA West-2 participants are acknowledged for their constructive comments and feedback.

PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

WEIGHTED AVERAGE of 2014 NGA WEST-2 GMPEs

Last updated: 04 14 15

by Emel Seyhan, PhD, PEER & UCLA -- email: emel.seyhan@gmail.com, peer_center@berkeley.edu

This excel file will be updated as necessary on the PEER website to fix any typos or other errors. Please check the website frequently for new versions at: http://peer.berkeley.edu/ngawest2/databases/

Legend	Pre-defined option	Main input variable	Calculated variable	Input var. flag	Internal variable

GMPE averaging	Geometric	*Weighted average of the natural logarithm of the spectral values*

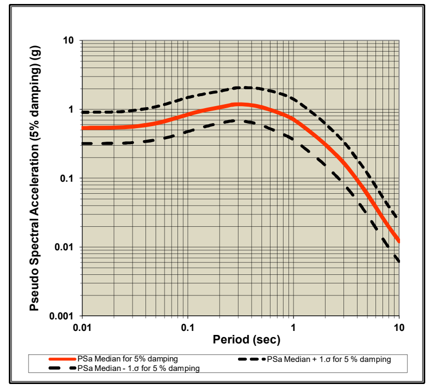
GMPEs	ASK14	BSSA14	CB14	CY14	I14
Weight	0	0.333333333	0.333333	0.333333	0

# of std. dev.	1	
Damping ratio (%)	5	*Modification factors are calculated in Sheet DSF*

ASK14 Abrahamson & Silva & Kamai 2014 NGA West-2 Model
BSSA14 Boore & Stewart & Seyhan & Atkinson 2014 NGA West-2 Model
CB14 Campbell & Bozorgnia 2014 NGA West-2 Model
CY14 Chiou & Youngs 2014 NGA West-2 Model
I14 Idriss 2014 NGA West-2 Model

RotD50 Horizontal Component of PGA, PGV and *IMs*

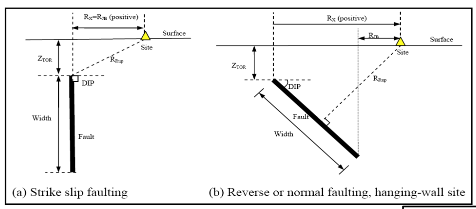
GMP	*T* (s)	Baseline: 5% Damping				User defined: 5% Damping			
		PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	0.01	0.53709	0.90543	0.31860	0.00133	0.53709	0.90543	0.31860	0.00133
	0.02	0.54326	0.91784	0.32155	0.00539	0.54326	0.91784	0.32155	0.00539
	0.03	0.56109	0.95827	0.32853	0.01254	0.56109	0.95827	0.32853	0.01254
	0.05	0.62622	1.09073	0.35953	0.03886	0.62622	1.09073	0.35953	0.03886
	0.075	0.73485	1.30055	0.41522	0.10261	0.73632	1.30316	0.41605	0.10282
	0.1	0.84005	1.48390	0.47556	0.20853	0.84173	1.48686	0.47651	0.20895
	0.15	0.98164	1.69585	0.56822	0.54828	0.98262	1.69755	0.56879	0.54883
	0.2	1.06552	1.82050	0.62363	1.05800	1.06658	1.82232	0.62426	1.05906
	0.25	1.14609	1.96217	0.66943	1.77814	1.15182	1.97198	0.67277	1.78703
	0.3	1.18449	2.05814	0.68169	2.64630	1.18449	2.05814	0.68169	2.64630
	0.4	1.14903	2.05055	0.64386	4.56370	1.15018	2.05260	0.64450	4.56827
	0.5	1.07302	1.96830	0.58496	6.65909	1.07410	1.97027	0.58555	6.66575
	0.75	0.87481	1.68855	0.45323	12.21531	0.87394	1.68687	0.45277	12.20310
	1	0.71406	1.40770	0.36221	17.72563	0.71263	1.40489	0.36149	17.69018
	1.5	0.44831	0.89712	0.22403	25.03966	0.44831	0.89712	0.22403	25.03966
	2	0.30881	0.62054	0.15368	30.66353	0.30820	0.61930	0.15337	30.60220
	3	0.16669	0.33697	0.08246	37.24153	0.16636	0.33629	0.08230	37.16705
	4	0.09440	0.18895	0.04716	37.49457	0.09431	0.18876	0.04712	37.45708
	5	0.05895	0.11822	0.02939	36.58392	0.05871	0.11775	0.02928	36.43759
	7.5	0.02252	0.04492	0.01129	31.44891	0.02255	0.04496	0.01130	31.48036
	10	0.01221	0.02398	0.00622	30.30525	0.01217	0.02390	0.00620	30.21434
PGA (g)	0	0.53300	0.89763	0.31649	0.00132	0.53300	0.89763	0.31649	0.00132
PGV (cm/s)	-1	65.05603	115.01353	36.79817	0.16149	NA	NA	NA	NA

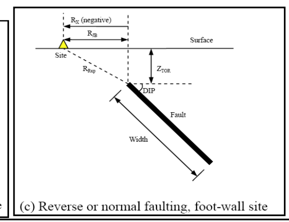


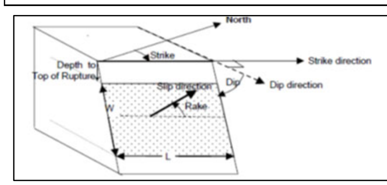
Input variables	Errors and warnings
M$_w$: 6.4	
R$_{RUP}$ ***(km)***: 1.59	
R$_{JB}$ ***(km)***: 1.23	
R$_X$ ***(km)***: -1.23	
Ry0 (km): 999	*If unknown use 999*
V$_{S30}$ ***(m/sec)***: 299.9	
U (BSSA13): 0	*1: Unspecified fault mech.*
F$_{RV}$: 1	*1: reverse fault*
F$_{NM}$: 0	*1: normal fault*
F$_{HW}$: 0	*1: hanging wall side*
Dip (deg): 40	
Z$_{TOR}$ ***(km)***: 8	*If unknown use 999*
Z$_{HYP}$ ***(km)***: 999	*If unknown use 999*
Z$_{1.0}$ ***(km)***: 0.607	*If unknown use 999*
Z$_{2.5}$ ***(km)***: 3.207	*If unknown use 999*
W (km): 999	*If unknown use 999*
Vs30Flag: measured	*Choose options for* V_{s30} *from the list*
F$_{AS}$: no	*Aftershock effect is not applicable.*
Region: California	*Choose region from the list*

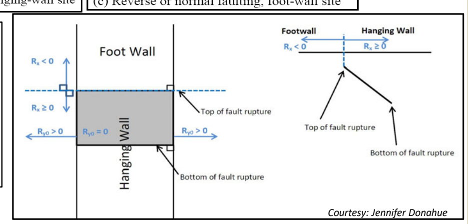
Calculated Variables/Flags

ΔDPP: 0	*Always 0 for median calcs.*
PGA$_r$ ***(g)***: 0.402	
Z$_{BOT}$ ***(km) (CB14)***: 15	*Enter for default W calcs*
SS: 0	*auto calculated*
V$_{s30Flag}$: 1	*measured*
F$_{AS}$: 0	*Aftershock effect is not applicable.*
Region: 0	*California*
Option for Sa value: 1	*Weighted average of the natural logarithm of the spectral values*









Definition of Parameters

Damping ratio = Viscous damping ratio (%) See Sanaz et al. (2012) PEER Report
PSA = Pseudo-absolute acceleration response spectrum (g)
PGA = Peak ground acceleration (g)
PGV = Peak ground velocity (cm/s)
S$_d$ = Relative displacement response spectrum (cm)
M$_w$ = Moment magnitude
R$_{RUP}$ = Closest distance to coseismic rupture (km), used in ASK13, CB13 and CY13. See Figures a, b and c for illustation
R$_{JB}$ = Closest distance to surface projection of coseismic rupture (km). See Figures a, b and c for illustation
R$_X$ = Horizontal distance from top of rupture measured perpendicular to fault strike (km). See Figures a, b and c for illustation
R$_{y0}$ = The horizontal distance off the end of the rupture measured parallel to strike (km)
V$_{S30}$ = The average shear-wave velocity (m/s) over a subsurface depth of 30 m
U = Unspecified-mechanism factor: 1 for unspecified; 0 otherwise
F$_{RV}$ = Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
F$_{NM}$ = Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal
F$_{HW}$ = Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise
Dip = Average dip of rupture plane (degrees)
Z$_{TOR}$ = Depth to top of coseismic rupture (km)
Z$_{HYP}$ = Hypocentral depth from the earthquake
Z$_{1.0}$ = Depth to Vs=1 km/sec
Z$_{2.5}$ = Depth to Vs=2.5 km/sec
W = Fault rupture width (km)
V$_{s30flag}$ = 1 for measured, 0 for inferred Vs30
F$_{AS}$ = 0 for mainshock; 1 for aftershock
Region = Specific regions considered in the models, Click on Region to see codes
ΔDPP = Directivity term, direct point parameter; uses 0 for median predictions
PGA$_r$ ***(g)*** = Peak ground acceleration on rock (g), this specific cell is updated in the cell for BSSA14 and CB14, for others it is taken account for in the macros
Z$_{BOT}$ ***(km)*** = The depth to the bottom of the seismogenic crust
Z$_{BOR}$ ***(km)*** = The depth to the bottom of the rupture plane
SS = 1 for strike slip, automatically updated in the cell

Input variables with defaults (If entered 999 as input):

DEFAULTs	USER defined	ASK14	BSSA14	CB14	CY14	I14
		Red colored value: The value is used in the code when input is unknown				
W (km)	999.00			15.445		
$Z_{1.0}$ (km)	0.607	0.607			0.459	
$\delta Z_{1.0}$ (km)	0.148		0.148			
$Z_{2.5}$ (V_{S30}=1100)(km)	3.207			0.398		
$Z_{2.5}$ (V_{S30})(km)	3.207			1.758		
Z_{hyp} (km)	999.00			11.362		
Z_{tor} (km)	8.00			4.115	4.115	
Z_{BOR} (km)	-			14.042		

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