Appendix E:
Geology and Soils Supporting Information

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E.1 - Geotechnical Evaluation

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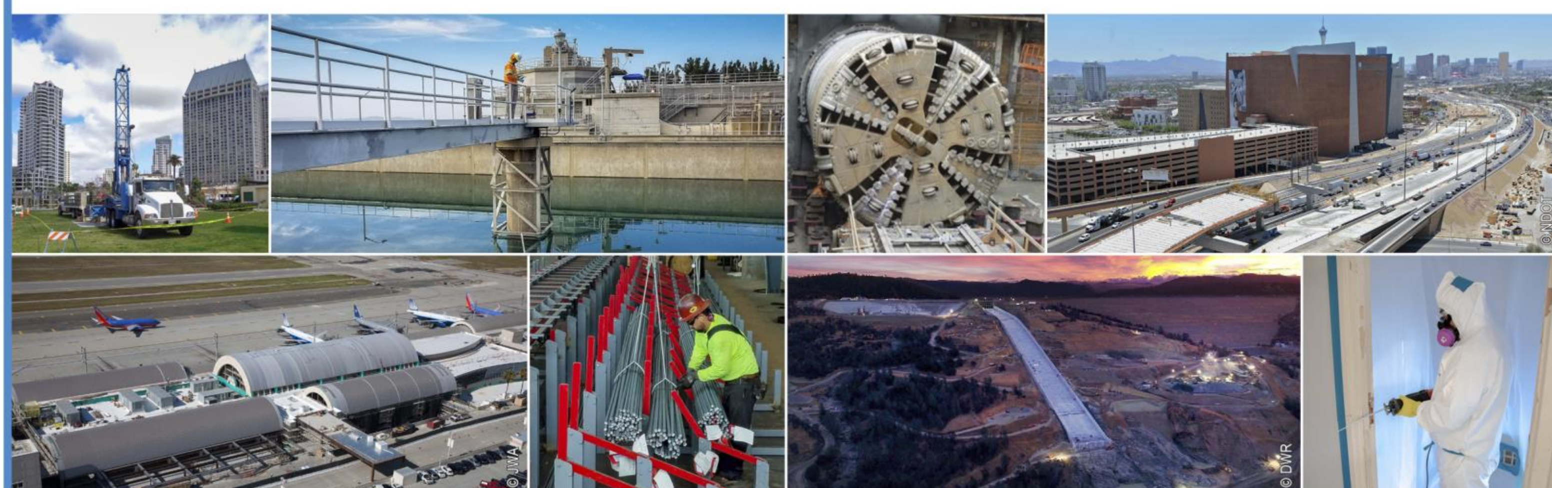
Preliminary Geotechnical Evaluation
Shirk & Riggin Industrial Park
West Riggin Avenue and North Kelsey Street
Visalia, California

Seefried Industrial Properties

2201 East Camelback Road, Suite 222 | Phoenix, Arizona 85016

August 2, 2022 | Project No. 211987001



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Preliminary Geotechnical Evaluation
Shirk & Riggin Industrial Park
West Riggin Avenue and North Kelsey Street
Visalia, California

Mr. Bryan Frarey
Seefried Industrial Properties
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1 INTRODUCTION

In accordance with your request and authorization, we have performed a preliminary geotechnical evaluation for the proposed Shirk & Riggin Industrial Park to be located at the northeast corner of West Riggin Avenue and North Kelsey Street in Visalia, California (Figure 1). Based on our our review of conceptual project plans (4Creeks, 2022), we understand that the project will generally include the construction of multiple buildings, loading docks, parking lots and driveway areas, stormwater infiltration basins, and improvements to adjacent streets. The purpose of our study was to evaluate the soil, geologic, and seismic conditions of the site in order to provide preliminary geotechnical design and construction recommendations for the conceptual design of the project. This report presents our preliminary findings, conclusions, and recommendations for the project. We understand that a design-level geotechnical evaluation will be performed once the final building, basin, and improvement layout is selected.

Ninyo & Moore prepared a Phase I Environmental Site Assessment (ESA) of the project site concurrently with the subject preliminary geotechnical evaluation. The results of the ESA were submitted in a separate report dated July 20, 2022.

2 SCOPE OF SERVICES

Our scope of services included the following:

- Project coordination, planning, and scheduling of the subsurface exploration.
- Review of readily available background materials, including published geologic maps and literature, in-house information, aerial photographs, and plans provided by the client.
- Site reconnaissance to observe the site conditions, mark the proposed boring locations, and coordinate with Underground Service Alert for utility clearance.
- Subsurface exploration consisting of drilling, logging, and sampling of twelve small-diameter, hollow-stem auger borings to depths ranging from approximately 5½ to 51½ feet below the existing ground surface. The borings were logged by a representative of our firm and relatively undisturbed and bulk soil samples were collected at selected intervals for laboratory testing.
- In-situ percolation testing in four of the borings during our field exploration.
- Laboratory testing of representative soil samples. Laboratory tests included evaluation of in-situ moisture content and dry density, gradation, percentage of particles finer than the No. 200 sieve, Atterberg limits, consolidation characteristics, direct shear strength, expansion index, R-value, and soil corrosivity.
- Data compilation and engineering analysis of the information obtained from our background review, subsurface exploration, percolation testing, and laboratory testing.

- Preparation of this report presenting our preliminary findings, conclusions, and preliminary geotechnical recommendations for the project.

3 SITE DESCRIPTION

The site is located north of the California State Route 198 and east of the California State Route 99 in the city of Visalia, California (Figure 1) and consists of a relatively flat, approximately 280-acre agricultural field currently used as an almond orchard. The field is located in an area of mixed-use, including similar agricultural fields to the north and east, industrial properties to the west and south, and residential neighborhoods to the southeast. The site is intersected by Road 89 and it is bounded on the north by the Modoc Ditch, on the south by West Riggin Avenue, on the west by North Kelsey Street, and on the east by North Shirk Road (Figure 2). Topographically, the project site is gently sloping towards the east with a ground surface elevation of approximately 300 to 305 feet above mean sea level (USGS, 2021a, b).

4 PROPOSED CONSTRUCTION

It is our understanding that the final project layout, including building sizes and locations, are still being developed and are subject to change. Our review of conceptual plans prepared by 4Creeks (2022), indicates that the project may include the construction of multiple single-story structures, including eight industrial buildings with loading docks, 15 flex industrial buildings, a convenience store, car wash, and two smaller auxiliary buildings. The conceptual plans indicate that the industrial buildings may have footprints ranging from approximately 109,890 to 1,078,440 square-feet (sf), the flex buildings may range from approximately 10,220 to 14,080 sf, the convenience store may be approximately 6,900 sf, the car wash may be approximately 4,500 sf, and the two auxiliary buildings may be less than 3,000 sf. Based on our experience with similar projects, it is anticipated that the new structures will be supported by shallow spread foundations with Portland cement concrete (PCC) slabs-on-grade. Conceptually, the project will also include the construction of new asphalt and PCC pavement parking lots and driveways around the buildings, including areas for trailer and automobile parking, a new road through the middle of the site (Clancy Street), landscape areas, associated underground utilities, seven stormwater detention basins up to approximately 5 feet in depth, and improvements to the adjacent roadways (West Riggin Avenue and North Shirk Street). The conceptual building, roadway, and stormwater basin layouts are shown on Figure 3.

5 SUBSURFACE EVALUATION AND LABORATORY TESTING

Our subsurface exploration was performed on June 13 through 16, 2022, and consisted of drilling, logging, and sampling of twelve small-diameter borings (B-1 through B-8 and P-1 through P-4) using a truck-mounted drill rig with 8-inch-diameter hollow-stem augers. The borings were drilled to depths ranging from approximately 5½ to 51½ feet below the ground surface at the approximate locations shown on Figures 2 and 3. The borings were logged by a representative from our firm. Bulk and relatively undisturbed soil samples were obtained at selected depths for laboratory testing. Percolation testing was performed in borings P-1 through P-4 as further discussed below. The logs of the exploratory borings are presented in Appendix A.

Laboratory testing was performed on representative samples to evaluate in-situ moisture content and dry density, gradation, percentage of particles finer than the No. 200 sieve, Atterberg limits, consolidation characteristics, direct shear strength, expansion index, R-value, and soil corrosivity. The results of the in-situ moisture content and dry density tests are presented on the boring logs in Appendix A. The remaining laboratory test results are presented in Appendix B.

6 GEOLOGY AND SUBSURFACE CONDITIONS

The project site is located within the San Joaquin Valley of the Central Valley geomorphic province of California (Norris and Webb, 1990). The San Joaquin Valley is a north to south trending synclinal trough generally underlain by sequences of Tertiary- to Pleistocene-age marine and non-marine sedimentary rock. The sedimentary rock is underlain by Cretaceous-age granitic rock associated with the Sierra Nevada batholith along the east side of the valley and Cretaceous-age Franciscan basement rock of the Coast Ranges along the west side of the valley. The sedimentary rock is generally covered by Pleistocene to Holocene alluvium that has infilled the valley. Regional geologic mapping by Matthews and Burnett (1965) indicate that the project site is generally underlain by Holocene-age alluvial fan deposits (Figure 3). Croft and Gordon (1968) describe these deposits as older alluvium consisting of a blanket of feldspathic gravel, sand, and silty sand derived from the Sierra Nevada Mountains.

The results of our subsurface evaluation indicate that the site is underlain by undocumented fill soils and alluvium. Undocumented fill was encountered in our borings to depths of up to approximately 2 feet that generally consisted of tilled surface materials of brown and grayish brown, moist, loose to medium dense, silty sand and sandy silt. Alluvium was encountered below the fill to the total depths explored in our borings up to approximately 51½ feet. The alluvium generally consisted of interbedded deposits of light brown, brown, light grayish brown, grayish brown, moist, loose to very dense, silt with sand, sandy silt, silty sand, poorly graded sand with silt, and poorly graded sand with variable amounts of gravel and occasional interbeds of brown to light brown, moist, hard, lean clay. Detailed descriptions of the materials encountered in our borings are presented on the boring logs in Appendix A.

7 GROUNDWATER

Groundwater was not encountered in our borings at the time of drilling. According to Croft and Gordon (1968), the depth to groundwater at the site is approximately 200 feet or more below the ground surface. However, groundwater monitoring well data from the State of California Water Resources Control Board's GeoTracker website (2022) indicate that the depth to groundwater ranges from approximately 100 to 200 feet at monitoring wells located approximately 0.5 mile south and 2.0 miles southwest of the site. It is anticipated that similar depths to groundwater are present at the site. Seepage may be encountered during site excavation due to irrigation at the site. Fluctuations in groundwater levels may occur due to variations in precipitation, ground surface topography, subsurface stratification, irrigation, groundwater pumping, and other factors that may not have been evident at the time of our field evaluation.

8 FLOOD HAZARDS

Based on our review of flood insurance rate maps for the project area (Federal Emergency Management Agency [FEMA], 2009), the project site is not located in the 100-year Flood Hazard Zone, A99. Zone A99 includes areas to be protected from a 100-year flood by the Federal Flood Protection System under construction at the time of publication of the FEMA map. The project site is located within FEMA's designated Other Flood Areas - Zone X, which includes areas assigned to be within the 500-year floodplain.

9 FIELD PERCOLATION TESTING

At the time of our evaluation, Tulare County did not have established percolation testing procedures. Therefore, in-situ percolation testing was performed in general accordance with the Percolation Test Procedure outlined in the County of Riverside's Design Handbook for Low Impact Development Best Management Practices for borings less than 10 feet deep (Riverside County, 2014). Percolation testing was performed in borings P-1 through P-4 to evaluate the infiltration rates of the on-site soils at the anticipated invert depths of the proposed stormwater infiltration basins. The approximate locations of the percolation test borings are shown on Figures 2 and 3.

Preparation of each boring for percolation testing included the placement of a 2-inch-diameter, slotted, polyvinyl chloride (PVC) pipe in the borehole, and backfilling the annulus between the borehole wall and pipe with pea gravel. The tested zone was pre-soaked with clean water prior to performing the percolation testing. After the pre-soaking, falling-head percolation testing was performed using the test procedure for sandy soil. The test involved placing clean water into the PVC pipe to establish a head of water and measuring the rate at which the water level dropped in the pipe at consecutive 10-minute time intervals for one hour. The test infiltration rate was calculated from the measured percolation rates using a conversion formula (based on the Porchet Method) that was provided in the County of Riverside guidelines (Riverside County, 2014). Per the guidelines (Riverside County, 2014), a safety factor of 3 was applied to the converted infiltration rates to account for site variations and calculate the design infiltration rate. The approximate depth of testing, converted infiltration rate, and calculated design infiltration rate for each boring are provided in Table 1.

Table 1 – Percolation Test Results

Test Boring	Basin	Approximate Depth of Tested Zone (feet)	Soil Type at Test Interval (USCS)	Converted Infiltration Rate, I_t (inches/hour)	Design Infiltration Rate, FS=3 (inches/hour)
P-1	1	3.5 to 5.5	SM	4.76	1.59
P-2	1	3.5 to 5.5	SM	11.06	3.69
P-3	1	3.5 to 5.5	SM	4.76	1.59
P-4	1	3.5 to 5.6	SM	3.38	1.13

Notes:
USCS – Unified Soil Classification System
I_t – Test Infiltration Rate per County of Riverside, Appendix B, Section 2.3 (2014)
FS – Factor of Safety per County of Riverside, Appendix B, Section 1.6 (2014)

10 FAULTING, SEISMICITY, AND GEOLOGIC HAZARDS

The project site is located in a seismically active area, as is the majority of southern California. The numerous faults in southern California include active, potentially active, and inactive faults. As defined by the California Geological Survey (CGS), active faults are faults that have ruptured within Holocene time, or within approximately the last 11,000 years. Potentially active faults are those that show evidence of movement during Quaternary time (approximately the last 1.6 million years) but for which evidence of Holocene movement has not been established. Inactive faults have not ruptured in the last approximately 1.6 million years. The approximate locations of major active faults in the region and their geographic relationship to the project site are shown on Figure 5.

Based on our review of seismic hazard maps, geologic literature, and geologic maps, the site is not located within a State of California Earthquake Fault Zone (formerly known as Alquist-Priolo Special Studies Zone) (Hart and Bryant, 2018), and no active faults are known to cross the subject site. The principal seismic and geologic hazards evaluated at the subject site are surface ground rupture, ground motion, liquefaction, dynamic compaction of dry soils, and regional subsidence. A brief description of these hazards and the potential for their occurrences on site are discussed in the following sections.

10.1 Surface Ground Rupture

Based on our review of the referenced literature and our site reconnaissance, no active faults are known to cross the project site. Therefore, the probability of damage from surface fault rupture is considered to be low. However, lurching or cracking of the ground surface as a result of nearby seismic events is possible.

10.2 Site-Specific Seismic Ground Motion

Considering the proximity of the site to active faults capable of producing a maximum moment magnitude of 6.0 or more, the project area has a high potential for experiencing strong ground motion. The 2019 California Building Code (CBC) specifies that the risk-targeted maximum considered earthquake (MCE_R) ground motion response accelerations be used to evaluate seismic loads for design of buildings and other structures. Per the 2019 CBC, a site-specific ground motion hazard analysis shall be performed for structures on Site Class D with a mapped MCE_R, 5 percent damped, spectral response acceleration parameter at a period of 1 second (S_1) greater than or equal to 0.2g in accordance with Sections 21.2 and 21.3 of the American Society of Civil Engineers (ASCE) Publication 7-16 (2016) for the Minimum Design Loads and Associated Criteria for Building and Other Structures. We calculated that the S_1 for the site is equal to 0.222g using the 2022 Applied Technology Council (ATC) seismic design tool (web-based); therefore, a site-specific ground motion hazard analysis was performed for the project area.

The site-specific ground motion hazard analysis consisted of the review of available seismologic information for nearby faults and performance of probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA) to develop acceleration response spectrum curves corresponding to the MCE_R for 5 percent damping. Prior to the site-specific ground motion hazard analysis, we obtained the mapped seismic ground motion values and developed the general MCE_R response spectrum for 5 percent damping in accordance with Section 11.4 of ASCE 7-16 (ATC, 2022). The average shear wave velocity (V_S) for the upper 30 meters of soil (V_{S30}) is assumed to be 228 meters per second (m/s) (CGS/Wills et al., 2017) and the depths to V_S = 1,000 m/s and V_S = 2,500 m/s are assumed to be 60 meters and 6,550 meters, respectively (Southern California Earthquake Center [SCEC] Community Velocity Model).

The 2014 new generation attenuation (NGA) West-2 relationships were used to evaluate the site-specific ground motions. The NGA relationships that we used for developing the probabilistic and deterministic response spectra are by Chiou and Youngs (2014), Campbell and Bozorgnia (2014), Boore, Stewart, Seyhan, and Atkinson (2014), and Abrahamson, Silva, and Kamai (2014). The Open Seismic Hazard Analysis software developed by United States Geological Survey (USGS,

2020) was used for performing the PSHA. The Calculation of Weighted Average 2014 NGA Models spreadsheet by the Pacific Earthquake Engineering Research Center was used for performing the DSHA (Seyhan, 2014).

PSHA was performed for earthquake hazards having a 2 percent chance of being exceeded in 50 years multiplied by the risk coefficients per ASCE 7-16. The maximum rotated components of ground motions were considered in PSHA with 5 percent damping. For the DSHA, we analyzed accelerations from characteristic earthquakes on active faults within the region using the hazard curves and deaggregation plots at the site obtained from the USGS Unified Hazard Tool application (USGS, 2021). A magnitude 8.0 event on the San Andreas Fault with a rupture distance of 104 kilometers from the site was evaluated to be the controlling earthquake. Hence, the deterministic seismic hazard analysis was performed for the site using this event and corrections were made to the spectral accelerations for the 84th percentile of the maximum rotated component of ground motion with 5 percent damping.

The site-specific MCE_R response spectrum was taken as the lesser of the spectral response acceleration at any period from the PSHA and DSHA, and the site-specific general response spectrum was determined by taking two-thirds of the MCE_R response spectrum with some conditions in accordance with Section 21.3 of ASCE 7-16. Figure 6 presents the site-specific MCE_R response spectrum and the site-specific design response spectrum. The general mapped design response spectrum calculated in accordance with Section 11.4 of ASCE 7-16 is also presented on Figure 6 for comparison. The site-specific spectral response acceleration parameters, consistent with the 2019 CBC, are provided in Section 12.2 of this report for the evaluation of seismic loads on buildings and other structures. The site-specific maximum considered earthquake geometric mean (MCE_G) peak ground acceleration, PGA_M, was calculated as 0.365g.

10.3 Liquefaction Potential

Liquefaction is the phenomenon in which loosely deposited granular soils and non-plastic silts located below the water table undergo rapid loss of shear strength when subjected to strong earthquake-induced ground shaking. Ground shaking of sufficient duration results in the loss of grain-to-grain contact due to a rapid rise in pore water pressure, and causes the soil to behave as a fluid for a short period of time. Liquefaction is known generally to occur in saturated or near-saturated cohesionless soils at depths shallower than 50 feet below the ground surface. Liquefaction is also known to occur in relatively fine-grained soils (i.e., sandy silt and clayey silt) with a plasticity index (PI) of less than 12 and an in-place moisture content more than 85 percent of the liquid limit (LL) and sensitive silts and clays with a PI more than 18. Factors known to influence liquefaction potential include composition and thickness of soil layers, grain size, relative density, groundwater level, degree of saturation, and both intensity and duration of ground shaking.

The project site is not located in an area mapped as a liquefaction hazard zone and groundwater in the vicinity of the site is more than approximately 100 feet below the ground surface (Croft and Gordon, 1968, and GeoTracker, 2022). Accordingly, it is our opinion that liquefaction and liquefaction-related seismic hazards (e.g., dynamic settlement of saturated soils and/or lateral spreading) are not design considerations for the project.

10.4 Dynamic Compaction of Dry Soils

Relatively dry soils (e.g., soils above the groundwater table) with low density or softer consistency tend to undergo a degree of compaction during a seismic event. Earthquake shaking often induces significant cyclic shear strain in a soil mass, which responds to the vibration by undergoing volumetric changes. Volumetric changes in dry soils take place primarily through changes in the void ratio (usually contraction in loose or normally consolidated soft soils, and dilation in dense or overconsolidated stiff soils) and secondarily through particle reorientation. Such volumetric changes are generally non-recoverable.

Based on our subsurface exploration, the alluvium at the site generally consists of very loose to very dense granular materials. Dry soil dynamic settlement was evaluated using the computer program LiquefyPro (CivilTech, 2008) based on the data from deep borings B-1 and B-8 and using PGA_M of 0.365g for a design earthquake magnitude of 8.0. Based on our evaluation, the dynamic settlement of dry soils is estimated to be on the order of 1-inch or less. The differential settlement may be assumed as ½-inch in a horizontal distance of 40 feet.

10.5 Regional Subsidence

Subsidence is characterized as a sinking of the ground surface relative to surrounding areas, and can generally occur where deep, unconsolidated soil deposits are present. Regional land subsidence is typically associated with regional groundwater withdrawal, oil and/or natural gas withdrawal, hydro-compaction, or oxidation and compaction of peat soils following drainage of marshland (Faunt, 2009). Subsidence can result in the development of ground surface cracks, larger scale ground failures, such as surface faulting and/or earth fissuring, ground surface drainage problems due to permanent subsurface aquifer compaction, increased incidences of flooding, and sinkholes, differential settlement, and damaged wells (Baum et al., 2008).

According to the USGS, the site is located within a portion of the San Joaquin Valley that has been subject to historical subsidence due to groundwater pumping with some areas experiencing as much as 28 feet of subsidence (USGS, 2018). In general, groundwater management policies and practices have improved over the years to help reduce groundwater pumping and the potential for future land subsidence. However, periods of drought and/or reduced surface water availability could result in renewed groundwater pumping and subsidence in the San Joaquin Valley.

11 CONCLUSIONS

Based on the results of our evaluation, it is our opinion that the proposed project is feasible from a geotechnical perspective, provided that the following recommendations, and recommendations from our future design level geotechnical evaluation, are incorporated into the design and construction of the project. In general, the following preliminary conclusions were made:

- The site is generally underlain by undocumented fill soils and alluvium. The fill soils generally consisted of moist, loose to medium dense, sandy silt and silty sand. The alluvium generally consisted of granular interbedded deposits of moist, loose to very dense, silt with sand, sandy silt, silty sand, poorly graded sand with silt, and poorly graded sand with variable amounts of gravel land occasional interbeds of moist, hard, lean clay. The fill is considered to be potentially compressible.

- Based on our explorations, we anticipate that excavations within fill and alluvial materials during site grading will be feasible with earthmoving equipment in good working order.

- We anticipate that the materials generated during the excavation of the existing undocumented fill and alluvial soils should be generally suitable for reuse as compacted fill provided they are free of trash, debris, roots, vegetation, other deleterious materials, and contamination and meet the requirements in the Fill Material section of this report.

- The site soils should be considered as Type C soils in accordance with Occupational Safety and Health Administration (OSHA) regulations. Granular alluvial soils encountered at the site may be subject to caving.

- Groundwater was not encountered during our subsurface exploration. Groundwater levels in the vicinity of the site range from approximately 100 to 200 feet below the ground surface (Croft and Gordon, 1968, and GeoTracker, 2022). Seepage may be encountered during site excavation due to irrigation at the site. The depth to groundwater varies due to site topography, seasonal precipitation, subsurface conditions, irrigation, groundwater pumping, and other factors.

- Our laboratory consolidation tests indicate that some of the relatively dry and loose near-surface alluvial soils may have up to approximately 1 percent of hydro-collapse potential under the existing overburden pressure if the soils become inundated due to various factors, such as broken water pipelines, rising groundwater levels, or excessive irrigation. Hydro-collapse may cause distress, such as excessive differential settlement to the proposed structures and cracks in the hardscape, which will require additional maintenance.

- The site is not located within an area subject to earthquake-induced liquefaction.

- Based on our evaluation, the dynamic settlement of dry soils is estimated to be on the order of 1-inch or less. The differential settlement may be assumed as ½-inch in a horizontal distance of 40 feet.

- The subject site is not located within a State of California Earthquake Fault Zone (Hart and Bryant, 2018). The probability of surface fault rupture at the site is considered to be low.

- The site is not located within a designated flood inundation zones for 100-year flood events. However, the site is located within an area considered to be subject to 500-year flood events (FEMA, 2009).

- The design PGA_M was calculated as 0.365g for the site.

- Based on our laboratory corrosion testing, the on-site soil may be classified as non-corrosive per the 2021 Caltrans Corrosion Guidelines.

- Ninyo & Moore prepared a Phase I ESA for the site. The presence of pesticides or herbicides were recognized environmental conditions (RECs) at this site. A Phase II ESA maybe needed prior to site development.

12 PRELIMINARY RECOMMENDATIONS

The following sections provide our preliminary geotechnical recommendations for use in the conceptual design and construction of the proposed buildings, site pavements, underground utilities, and street improvements. These recommendations are based on our evaluation of the site geotechnical conditions and our understanding of the conceptual planned construction. The project's final design and construction should be based on a design-level geotechnical evaluation performed of the selected building and improvement locations and should be in accordance with the requirements of applicable governing agencies.

12.1 Earthwork

Earthwork at the site is anticipated to consist of remedial grading of the near-surface soils, fill placement, foundation and basin excavations, trenching and backfilling for new utilities, pavement construction, and finish grading for establishment of site drainage. Earthwork operations should be performed in accordance with the requirements of the applicable governing agencies and the recommendations presented in the following sections of this report.

12.1.1 Soil Management

RECs consisting of pesticides and/or herbicides were reported in our Phase I ESA (Ninyo & Moore, 2022) that may warrant further evaluation at the site. Depending on the findings of the additional work, a Soil Management Plan and dust abatement program to reduce fugitive dust generation may be required for the project.

12.1.2 Pre-Construction Conference

We recommend that a pre-construction conference be held. The owner and/or their representative, the governing agencies' representatives, the civil engineer, Ninyo & Moore, and the contractor should be in attendance to discuss the work plan and project schedule and earthwork requirements.

12.1.3 Site Preparation

Prior to performing excavations or other earthwork, the site should be cleared of surface obstructions, deleterious materials, and vegetation, as well as surface soils containing organic materials. Existing utilities within the project limits (if any) should be re-routed or protected from damage by construction activities. Obstructions that extend below the finished grade should be removed and the resulting holes filled with compacted soil. Materials generated from the clearing operations should be removed from the project site and disposed of at a legal dump site.

12.1.4 Excavation Characteristics

Based on our field exploration, we anticipate that excavations within fill and alluvial materials at the site may be accomplished with conventional earthmoving equipment in good working condition. Based on our site reconnaissance and review of geologic literature, we anticipate that the materials encountered will be comprised predominantly of moist, loose to dense, sandy silt and sand with variable amounts of silt and gravel with occasional interbeds of moist, hard, lean clay. Oversize cobbles and boulders, if encountered during excavation operations, are not considered to be suitable for backfill and should be disposed of offsite. Contractors should make their own independent evaluation of the excavatability of the on-site materials prior to submitting their bids.

12.1.5 Treatment of Near-Surface Soils

The remedial grading recommendations provided in this section are based on a limited number of borings and laboratory tests. The recommendations presented herein may be revised when additional borings and laboratory test data is available during the future design level geotechnical evaluation.

In order to provide suitable support and reduce the potential for settlement of proposed buildings, we recommend that the building pads be overexcavated and recompacted to depths of approximately 6 feet below the existing ground surface or 4 feet beneath the bottom of the footings, whichever is deeper. The over-excavation should remove the undocumented fill and upper loose alluvial deposits, and should expose relatively dense alluvial deposits. Additional overexcavation of loose, soft, and/or wet areas, and/or undocumented fill may be appropriate. The limits of the excavation should extend laterally so that the overexcavation is approximately 6 feet beyond the perimeter of the building or a distance equal to the depth of the overexcavation, whichever is greater. The excavation bottom should be evaluated by our representative during the excavation work. The exposed subgrade should be scarified to a depth of approximately 8 inches, moisture-conditioned, and compacted prior to the placement of fill.

In areas of pavements subject to vehicle traffic and exterior flatwork, the near-surface soils should be removed to a depth of approximately 2 feet below the pavement or flatwork subgrade. The actual depths of overexcavation should be evaluated by our representative based on the materials exposed at the time of construction. The limits of overexcavation should extend laterally beyond the improvements to a distance equal to the depth of over-excavation. The subgrade at the bottoms of the overexcavation should be scarified to a depth of 8 inches, moisture-conditioned to slightly above the laboratory optimum moisture content, and compacted to a relative compaction of 90 percent or more. The overexcavated areas should be backfilled to the finished grade with the on-site soils with very low expansion potential and compacted to a relative compaction of 90 percent or more.

12.1.6 Fill Material

In general, the on-site granular soils should be suitable for re-use as structural fill and trench backfill provided they are free of trash, debris, roots, vegetation, or other deleterious materials. The onsite silt materials may be reused as fill, provided they are mixed with onsite sandy materials prior to being used as fill. Fill should generally be free of rocks or solid lumps of material in excess of 4 inches in diameter. Rocks or solid lumps larger than approximately 4 inches in diameter should be broken into smaller pieces or should be removed from the site. On-site soils used as fill will involve moisture-conditioning to achieve appropriate moisture content for compaction.

Imported materials, if used, should consist of clean, non-expansive, granular material that conforms to the latest edition of “Greenbook” Standard Specifications for Public Works Construction for structure backfill. Soil should also be tested for corrosive properties prior to importing. We recommend that the imported materials comply with the Caltrans (2021) criteria for non-corrosive soils (i.e., soils having a chloride concentration of less than 500 parts per million (ppm), a soluble sulfate content of less than 0.15 percent (1,500 ppm), a pH value higher than 5.5, and an electrical resistivity of more than 1,500 ohm-centimeters [ohm-cm]). Import materials for use as fill should be evaluated by the geotechnical consultant prior to importing. The contractor should be responsible for the uniformity of import material brought to the site.

12.1.7 Fill Placement and Compaction

Fill placed for support of the new buildings, pavements, and other miscellaneous improvements should be compacted in horizontal lifts to a relative compaction of 90 percent or more as evaluated by ASTM International (ASTM) D 1557. Fill soils should be placed at slightly above the optimum moisture content as evaluated by ASTM D 1557. The optimum lift thickness of fill will depend on the type of compaction equipment used but generally should not exceed 8 inches in loose thickness. Placement and compaction of the fill soils should be in general accordance with appropriate governing agency grading ordinances and good construction practice. Special care should be taken to avoid damage to wet and dry utility lines when compacting fill and subgrade materials.

12.1.8 Temporary Excavations

Trenches and excavations should be designed and constructed in accordance with OSHA regulations. These regulations provide trench sloping and shoring design parameters for trenches up to 20 feet deep based on the soil types encountered. Trenches over 20 feet deep should be designed by the contractor’s engineer based on site-specific geotechnical analyses. For planning purposes, we recommend that the materials on site be considered as OSHA soil Type C.

Temporary excavations should be constructed in accordance with OSHA recommendations. For trench or other excavations, OSHA requirements regarding personnel safety should be met by using appropriate shoring (including trench boxes) or by laying back the slopes no steeper than 1.5:1 (horizontal to vertical). Temporary excavations that encounter seepage may need shoring or may be mitigated by placing sandbags or gravel along the base of the seepage zone. Excavations encountering seepage should be evaluated on a case-by-case basis. On-site safety of personnel is the responsibility of the contractor. Recommendations for temporary shoring can be provided during the future design level geotechnical evaluation, if requested. Care should be taken by the contractor to avoid undermining adjacent existing foundations and improvements. New excavations should not extend within the “zone of influence” of existing foundations, which is defined as a 1:1 (horizontal to vertical) plane projecting out from the bottom outside edge of the foundations. In the event that excavations will extend within the “zone of influence” of existing foundations, our office should be notified and appropriate recommendations provided, such as temporary underpinning of impacted foundations and/or temporary shoring.

12.2 Site-Specific Seismic Design Considerations

Design of the proposed improvements should be performed in accordance with the requirements of the governing jurisdictions and applicable building codes. Table 2 presents the site-specific spectral response acceleration parameters in accordance with the CBC (2019) guidelines.

Table 2 – 2019 California Building Code Seismic Design Criteria

Site Coefficients and Spectral Response Acceleration Parameters	Values
Site Class	D
Mapped Spectral Response Acceleration at 0.2-second Period, S_S	0.564g
Mapped Spectral Response Acceleration at 1.0-second Period, S_1	0.222g
Site-Specific Spectral Response Acceleration at 0.2-second Period, S_{MS}	0.915g
Site-Specific Spectral Response Acceleration at 1.0-second Period, S_{M1}	0.592g
Site-Specific Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	0.610g
Site-Specific Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.395g
Site-Specific Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Acceleration, PGA_M	0.365g

12.3 Foundations

The proposed buildings may be supported on shallow, spread footings bearing on compacted engineered fill in accordance with the recommendations presented in this report. 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 the design of the structures.

12.3.1 Footings

Footings should extend 24 inches or more below the lowest adjacent finished grade and have a width of 24 inches or more. Spread footings should be reinforced with a minimum of two No. 4 steel reinforcing bars, one placed near the top and one placed near the bottom of the footings, and further detailed in accordance with the recommendations of the structural engineer.

On a preliminary basis, footings as described above and bearing on compacted fill soils as discussed in this report, may be designed using a net allowable bearing capacity of 2,250 pounds per square foot (psf). The bearing capacity may be increased by 150 psf and 250 psf for every additional foot of increase in width and depth, respectively, up to a value of 3,250 psf.

Total and differential settlements for footings designed and constructed in accordance with the above recommendations are estimated to be less than 1 inch and ½ inch over a horizontal span of 40 feet, respectively.

Footings bearing on compacted fill may be designed using a coefficient of friction of 0.35, where the total frictional resistance equals the coefficient of friction times the dead load. Footings may be designed using a passive resistance of 350 psf per foot of depth for level ground condition up to a value of 3,500 psf. The allowable lateral resistance can be taken as the sum of the frictional resistance and passive resistance provided the passive resistance does not exceed one-half of the total allowable resistance. The passive resistance may be increased by one-third when considering loads of short duration such as wind or seismic forces.

12.3.2 Building Floor Slabs

Building floor slabs should be designed by the project structural engineer based on the anticipated loading conditions. Building floor slabs should be underlain by compacted soil prepared in accordance with the recommendations presented in this report. We recommend that slabs be, at a minimum, 5 inches thick and reinforced with No. 4 steel reinforcing bars placed 18 inches on-center (each way) placed near the mid-height of the slab. The placement of the reinforcement in the slab is vital for satisfactory performance. The slab should be underlain by a vapor retarder and capillary break system consisting of a polyethylene vapor retarder (with a thickness of 10 mil or more) membrane placed over 4 inches of medium to coarse, clean sand or pea gravel. As an alternative, the slab underlayment may consist of a 15-mil Stego Wrap vapor barrier (or equivalent) placed over 4 inches of crushed gravel. The steel reinforcements for the floor slab shall be placed on the vapor retarder using chairs, as appropriate. The vapor retarder is recommended in areas where moisture-sensitive floor coverings are anticipated. Soils underlying the slabs should be moisture-conditioned and compacted in accordance with the recommendations presented in this report prior to concrete placement. A long-term modulus of subgrade reaction of 50 pounds per cubic inch (pci) may be used in design for the compacted slab subgrade. Joints should be constructed at intervals designed by the structural engineer to help reduce random cracking of the slab.

12.4 Exterior Flatwork

Exterior flatwork should be supported on subgrade soils prepared in accordance with the remedial recommendations presented in Section 12.1 of this report. Exterior flatwork should have a thickness of 4 inches or more. The flatwork should be reinforced with No. 4 steel reinforcing bars placed 24 inches on-center (each way) near the mid-height of the slab.

To reduce the potential for distress to exterior concrete flatwork due to movement of the underlying soil, we recommend that flatwork be installed with crack-control joints at appropriate spacing as designed by the structural engineer. Exterior flatwork should be underlain by 4 inches of clean sand. Positive drainage should be established and maintained adjacent to flatwork.

12.5 Underground Utilities

We anticipate that utility pipelines will be supported on compacted fill or native alluvium. The depths of the pipelines are not known; however, we anticipate that the pipe invert depths will not exceed 10 feet.

12.5.1 Pipe Bedding

We recommend that pipelines be supported on 6 inches or more of granular bedding material such as sand with a sand equivalent (SE) value of 30 or more. Bedding material should be placed and compacted around the pipe, and 12 inches or more above the top of the pipe in accordance with the current "Greenbook" Standard Specifications for Public Works. We do not recommend the use of crushed rock for bedding material. It has been our experience that the voids within a crushed rock material are sufficiently large enough to allow fines to migrate into the voids, thereby creating the potential for sinkholes and depressions to develop at the ground surface.

Special care should be taken not to allow voids beneath and around the pipe. Bedding material and compaction requirements should be in accordance with the recommendations of this report, the project specifications, and applicable requirements of the appropriate agencies. Compaction of the bedding material and backfill should proceed along both sides of the pipe concurrently and be compacted to 90 percent or more relative compaction as evaluated by ASTM D 1557.

12.5.2 Modulus of Soil Reaction

The modulus of soil reaction is used to characterize the stiffness of soil backfill placed on the sides of buried flexible pipelines for the purpose of evaluating lateral deflection caused by the weight of the backfill above the pipe. We recommend that a modulus of soil reaction of 400 pounds per square inch (psi) be used for design, provided that granular bedding material is placed adjacent to the pipe, as recommended in this report.

12.6 Preliminary Pavement Recommendations

Paved access roads and parking stalls will be part of the proposed improvements for this project. Accordingly, the preliminary pavement sections were designed based on the subgrade soil conditions and our laboratory testing. Laboratory testing of representative near-surface soil samples were performed and indicated R-values of approximately 53, 59, 69, and 72. An R-value of 50 was used in our analyses for the preliminary pavement design. The preliminary pavement sections presented herein may be revised, as appropriate, based on the additional R-value testing to be performed during the future design level geotechnical evaluation. We have evaluated pavement structural sections for Traffic Indices (TI) of 5, 6, 7, 8, 9, 10, and 11. Our asphalt concrete (AC) pavement analysis was performed using the methodology outlined in the Caltrans Highway Design Manual (Caltrans, 2019b). The AC pavement analysis assumes an approximate 20-year design life for new pavements. For the design of Portland Cement Concrete (PCC) pavements, we used the methodology presented in the Navy Pavement Design Manual (1979). Based on the design R-value and TI, our preliminary estimates of pavement structural sections consisting of AC over an aggregate base, full depth AC, or non-reinforced full depth PCC, are provided below in Table 3.

Table 3 – Preliminary Structural Pavement Sections

Traffic Index	AC over CAB or AC over CMB (inches)	Full Depth AC (inches)	Full Depth Non-Reinforced PCC (inches)
≤5	3 over 4	4½	6
6	3 over 4½	6	6½
7	3½ over 5½	7	8½
8	5 over 5½	8	9½
9	5½ over 6½	9	11
10	6 over 8	10	12½
11	7 over 9	11	13

Notes:
AC – Asphalt Concrete
CAB – Crushed Aggregate Base
CMB – Crushed Miscellaneous Base
PCC – Portland Cement Concrete with a 28-day compressive strength of 2,500 psi

Pavement sections should be supported on subgrade soils prepared in accordance with the remedial recommendations presented in Section 12.1 of this report. Base material should be placed at a relative compaction of 95 percent or more as evaluated by ASTM D 1557. The subgrade soil should be compacted to 95 percent relative compaction. The concrete compressive strength for the PCC section should be 3,500 psi or more. No reinforcements will be needed for the PCC pavement, unless noted.

Aggregate base material should conform to the latest specifications in Section 200 2.2 for crushed aggregate base or Section 200 2.4 for crushed miscellaneous base of the Greenbook and should be compacted to a relative compaction of 95 percent in accordance with ASTM D 1557. AC should conform to Section 203.6 of the Greenbook and should be compacted to a relative compaction of 95 percent in accordance with ASTM D 1557.

12.7 Soil Corrosivity

Laboratory testing were performed on selected representative shallow soil samples collected from our borings to evaluate soil pH, electrical resistivity, water-soluble chloride content, and water-soluble sulfate content. The soil pH and electrical resistivity tests were performed in general accordance with California Test Method (CT) 643. Chloride content tests were performed in general accordance with CT 422. Sulfate testing were performed in general accordance with CT 417. The soil pH of the samples tested were measured to be 6.4 and 7.4 and the electrical resistivity were measured to be 4,990 and 2,110 ohm-centimeters. The chloride content of the samples were measured to be 20 and 30 ppm. The sulfate content of the sample was measured to be 0.001 percent by weight (i.e., 10 ppm). Based on the laboratory test results and 2021 Caltrans corrosion criteria, the soils at the project site may be classified as non-corrosive, which is defined as having earth materials with less than 500 ppm chlorides, less than 0.15 percent sulfates (i.e., 1,500 ppm), a pH of 5.5 or more, or an electrical resistivity of 1,500 ohm-cm or more. The corrosivity test results are presented in Appendix B.

12.8 Concrete Placement

Concrete in contact with soil or water that contains high concentrations of water-soluble sulfates can be subject to premature chemical and/or physical deterioration. Based on the CBC (2019), the potential for sulfate attack is negligible for water-soluble sulfate contents in soil ranging from 0.00 to 0.10 percent by weight, moderate for water-soluble sulfate contents ranging from 0.10 to 0.20 percent by weight, severe for water-soluble sulfate contents ranging from 0.20 to 2.00 percent by weight, and very severe for water-soluble sulfate contents over 2.00 percent by weight. The soil samples tested for this evaluation, using CT 417, indicate water-soluble sulfate contents of approximately 0.001 percent by weight (i.e., 10 ppm). Accordingly, the on-site soils are considered to have a negligible potential for sulfate attack. However, due to the potential variability of the soils on site, consideration should be given to using Type II/V cement for the project.

To reduce the potential for shrinkage cracks in the concrete during curing, we recommend that the concrete for the proposed improvements be placed with a slump of 4 inches based on ASTM C 143. The slump should be checked periodically at the site prior to concrete placement. We further recommend that concrete cover over reinforcing steel for foundations be provided in accordance with CBC (2019). The structural engineer should be consulted for additional concrete specifications.

12.9 Drainage

Positive surface drainage is imperative for satisfactory performance of the site. Positive drainage should be provided and maintained to direct surface water away from foundations and offsite. Positive drainage is defined as a slope of 2 percent or more for a distance of 10 feet or more away from foundations and other site improvements. Runoff should then be directed by the use of swales or pipes into a collective drainage system. Surface waters should not be allowed to pond adjacent to footings or on pavements. Concentrated runoff should not be allowed to flow over pavement as this can result in early deterioration of the pavement. We recommend that the top level of the structures have roof drains and downspouts installed to collect runoff. Area drains for landscaped and paved areas are recommended.

13 CONSTRUCTION OBSERVATION

The preliminary recommendations provided in this report are based on our understanding of the proposed project and our evaluation of the data collected based on subsurface conditions observed in our exploratory borings. It is imperative that the geotechnical consultant checks the subsurface conditions during construction.

During construction, we recommend that the duties of the geotechnical consultant include, but not be limited to:

- Observing clearing, grubbing, and removals.
- Observing excavation, placement, and compaction of fill, including structural and trench backfill.
- Evaluating on-site soil for suitability of its use as engineered fill/structural backfill prior to placement.
- Evaluating imported materials prior to their use as fill, if any.
- Performing field tests to evaluate fill compaction.
- Observing foundation excavations for bearing material prior to placement of reinforcing steel or concrete.

The recommendations provided in this report are based on the assumption that Ninyo & Moore will perform future design geotechnical evaluation and provide geotechnical observation and testing services during construction. In the event that the services of Ninyo & Moore are not used during construction, we request that the selected consultant provide the owner with a letter (with a copy to Ninyo & Moore) indicating that they fully understand Ninyo & Moore's recommendations and that they are in full agreement with the design parameters and recommendations contained in this report.

14 LIMITATIONS

The field evaluation, laboratory testing, and geotechnical analyses presented in this preliminary 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 preliminary 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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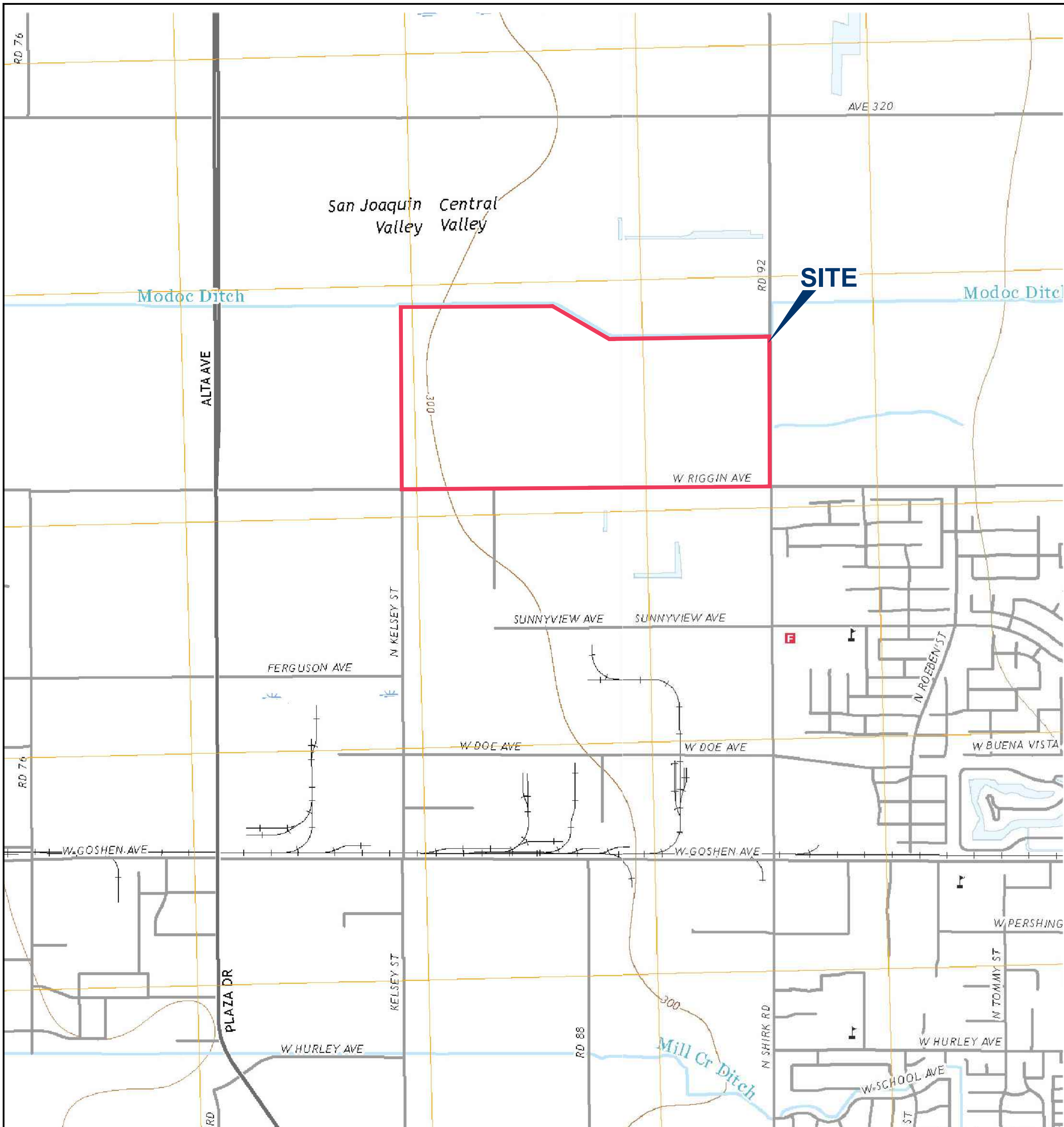
FIGURES



NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: USGS, 2021a, b.

N

FEET
0 2,000 4,000

1_211987001_SL.dwg 07/29/2022 GK, JDP

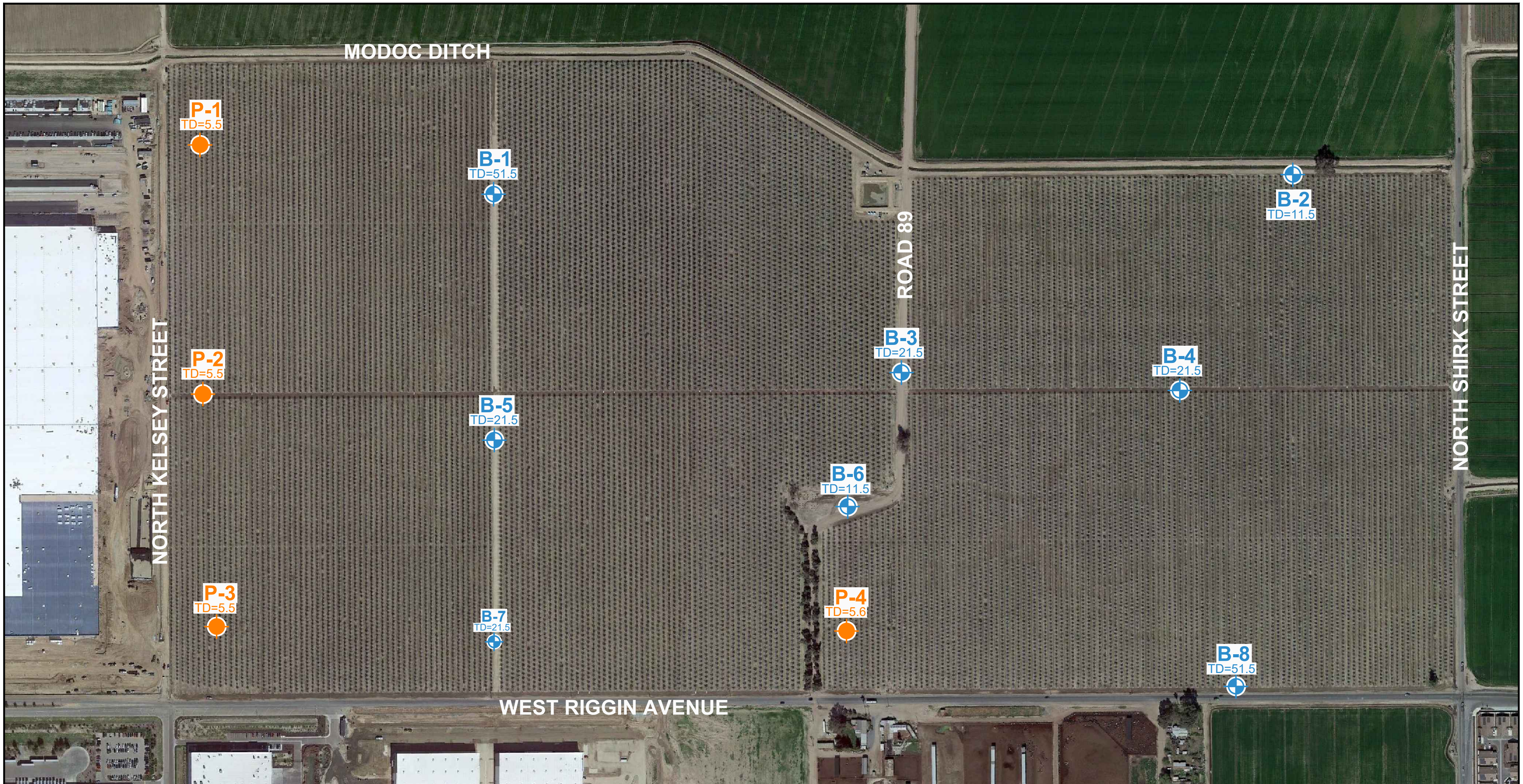
FIGURE 1

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

SITE LOCATION

SHIRK & RIGGIN INDUSTRIAL PARK
VISALIA, CALIFORNIA

211987001 I 8/22



2_211987001_SAP.dwg 07/25/2022 GK

LEGEND

P-4 TD=5.6 PERCOLATION TEST; TD=TOTAL DEPTH IN FEET

B-8 TD=51.5 BORING; TD=TOTAL DEPTH IN FEET

N

FEET

0 600 1,200

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: GOOGLE EARTH, 2022.

FIGURE 2

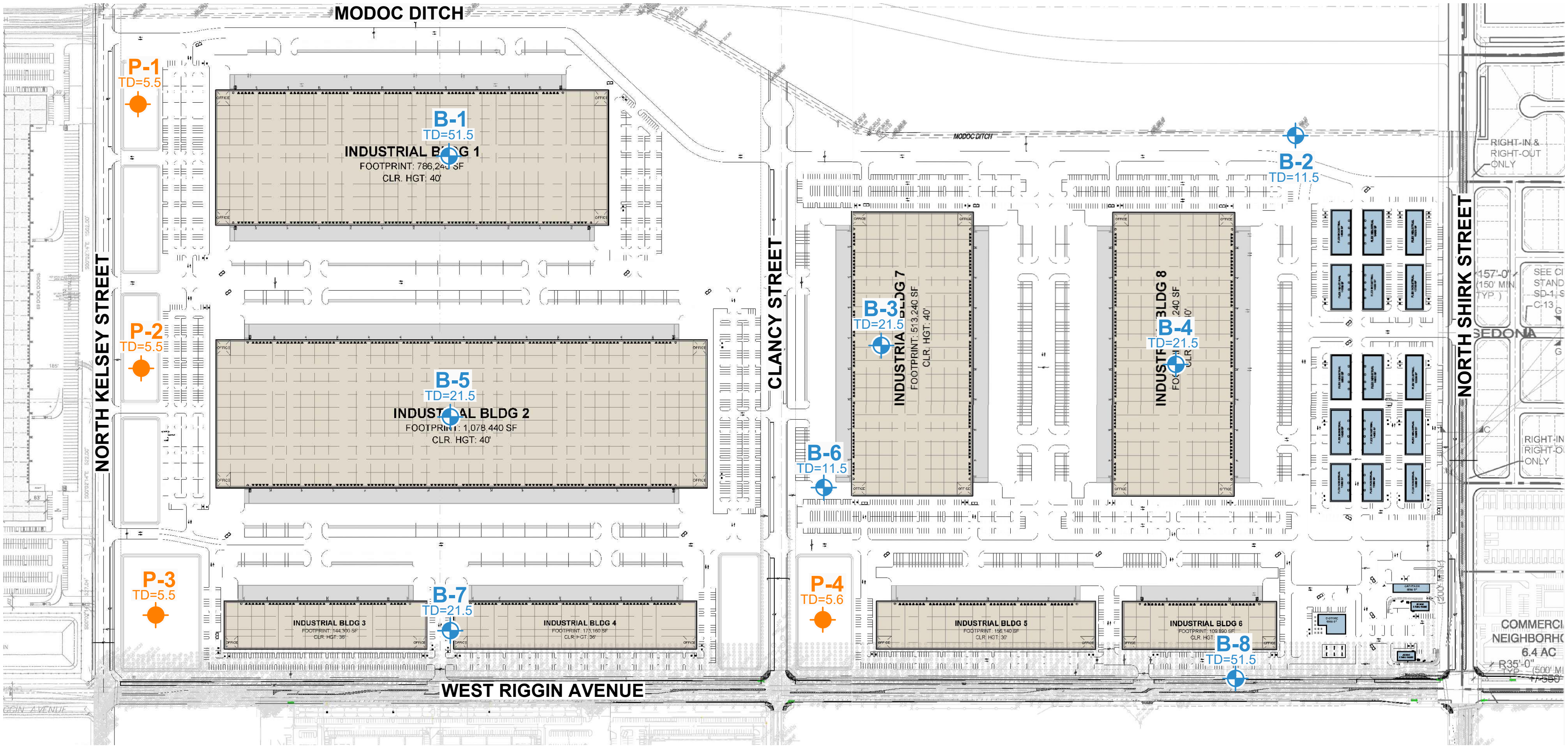
Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

SITE AERIAL PHOTOGRAPH

SHIRK & RIGGIN INDUSTRIAL PARK
VISALIA, CALIFORNIA

211987001 I 8/22



LEGEND

P-4 TD=5.6 — PERCOLATION TEST; TD=TOTAL DEPTH IN FEET

B-8 TD=51.5 — BORING; TD=TOTAL DEPTH IN FEET

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: 4CREEKS, 2022.

N

FEET

0 600 1,200

3_211987001_SP.dwg 07/25/2022 GK

FIGURE 3

Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

SITE PLAN

SHIRK & RIGGIN INDUSTRIAL PARK
VISALIA, CALIFORNIA

211987001 I 8/22

LEGEND

Qf	FAN DEPOSITS
Qb	BASIN DEPOSITS
	GEOLOGIC CONTACT

N

FEET

0 4,000 8,000

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: MATHEWS & BURNETT, 1965

4_211987001_RG.dwg 07/25/2022 GK

FIGURE 4

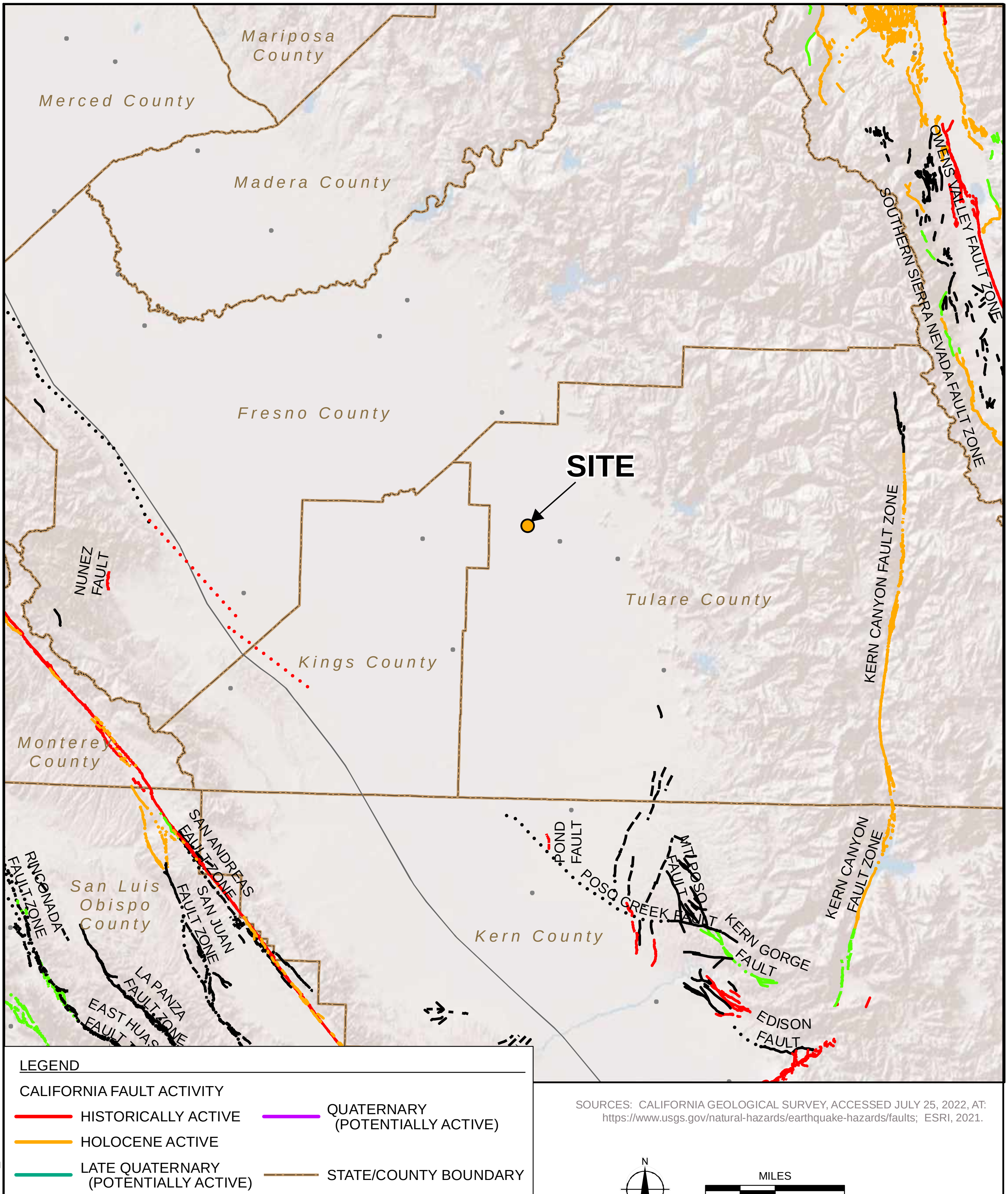
Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

REGIONAL GEOLOGY

SHIRK & RIGGIN INDUSTRIAL PARK
VISALIA, CALIFORNIA

211987001 I 8/22



211987001_FL.mxd 7/27/2022

SOURCES: CALIFORNIA GEOLOGICAL SURVEY, ACCESSED JULY 25, 2022, AT: https://www.usgs.gov/natural-hazards/earthquake-hazards/faults; ESRI, 2021.

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

FIGURE 5

FAULT LOCATIONS

SHIRK & RIGGIN INDUSTRIAL PARK
VISALIA, CALIFORNIA

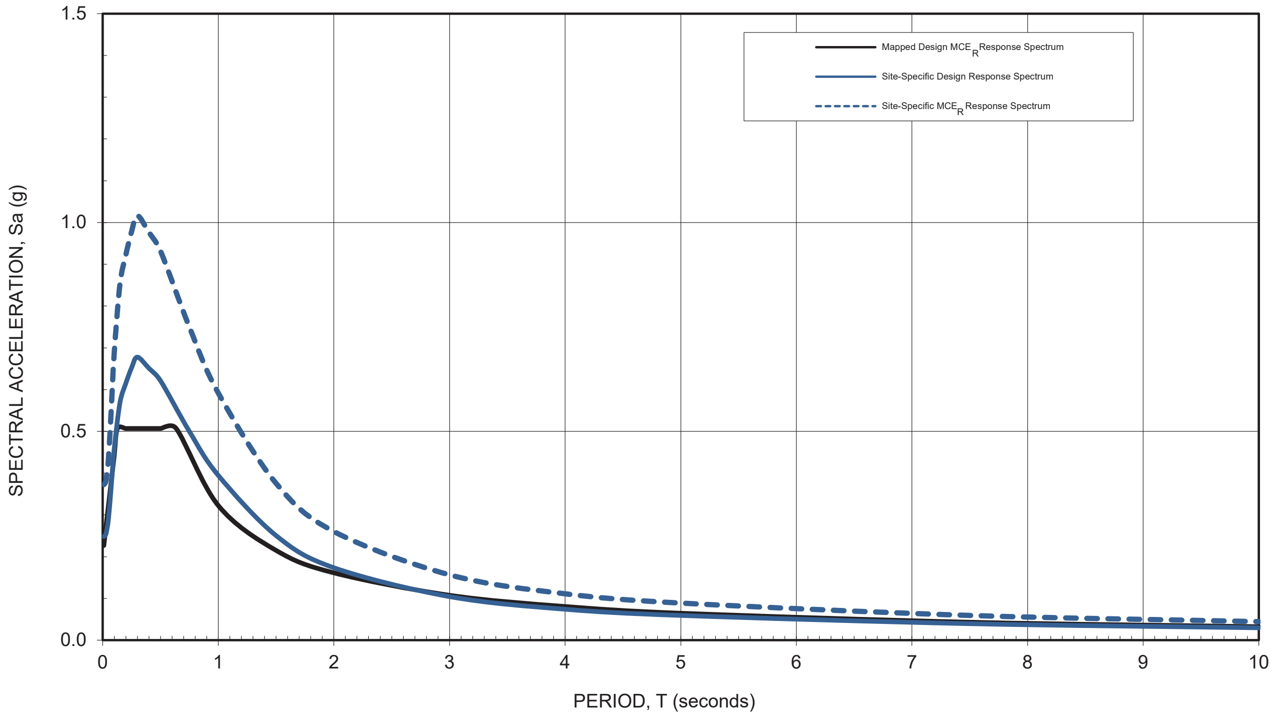
211987001 | 8/22

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

PERIOD (seconds)	SITE-SPECIFIC MCE_R RESPONSE SPECTRUM Sa (g)	SITE-SPECIFIC DESIGN RESPONSE SPECTRUM Sa (g)
0.010	0.372	0.248
0.020	0.376	0.250
0.030	0.384	0.256
0.050	0.434	0.290
0.075	0.557	0.372
0.100	0.688	0.459
0.150	0.854	0.569
0.200	0.922	0.615
0.250	0.978	0.652
0.300	1.017	0.678
0.400	0.977	0.651

PERIOD (seconds)	SITE-SPECIFIC MCE_R RESPONSE SPECTRUM Sa (g)	SITE-SPECIFIC DESIGN RESPONSE SPECTRUM Sa (g)
0.500	0.932	0.621
0.750	0.750	0.500
1.000	0.592	0.395
1.500	0.376	0.250
2.000	0.260	0.173
3.000	0.156	0.104
4.000	0.111	0.074
5.000	0.089	0.059
7.500	0.059	0.039
10.000	0.044	0.030

S_{DS} = 0.610 g	S_{D1} = 0.395 g	S_{MS} = 0.915 g	S_{M1} = 0.592 g	PGA_M = 0.365 g



NOTES:

1 The probabilistic ground motion spectral response accelerations are based on the risk-targeted Maximum Considered Earthquake (MCE_R) having a 2% probability of exceedance in 50 years in the maximum direction using the Chiou & Youngs (2014), Campbell & Bozorgnia (2014), Boore et al. (2014), and Abrahamson et al. (2014) attenuation relationships and the risk coefficients.

2 The deterministic ground motion spectral response accelerations are for the 84th percentile of the geometric mean values in the maximum direction using the Chiou & Youngs (2014), Campbell & Bozorgnia (2014), Boore et al. (2014), and Abrahamson et al. (2014) attenuation relationships for deep soil sites considering a Mw 8.0 event on the San Andreas fault zone located 107.0 kilometers from the site. It conforms with the lower bound limit per ASCE 7-16 Section 21.2.2.

3 The Site-Specific MCE_R Response Spectrum is the lesser of spectral ordinates of deterministic and probabilistic accelerations at each period per ASCE 7-16 Section 21.2.3. The Site-Specific Design Response Spectrum conforms with lower bound limit per ASCE 7-16 Section 21.3.

4 The Mapped Design MCE_R Response Spectrum is computed from mapped spectral ordinates modified for Site Class D (stiff soil profile) per ASCE 7-16 Section 11.4. It is presented for the sake of comparison.

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 falling freely from a height of 30 inches in general accordance with ASTM D 1586. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration. Soil samples were observed and removed from the sampler, bagged, sealed and transported to the laboratory for testing.

Field Procedure for the Collection of Relatively Undisturbed Samples

Relatively undisturbed soil samples were obtained in the field using the following method.

The Modified Split-Barrel Drive Sampler

The sampler, with an external diameter of 3 inches, was lined with 1-inch-long, thin brass rings 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 rings, 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.	BORING LOG EXPLANATION SHEET
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								
								The total depth line is a solid line that is drawn at the bottom of the boring.
20								

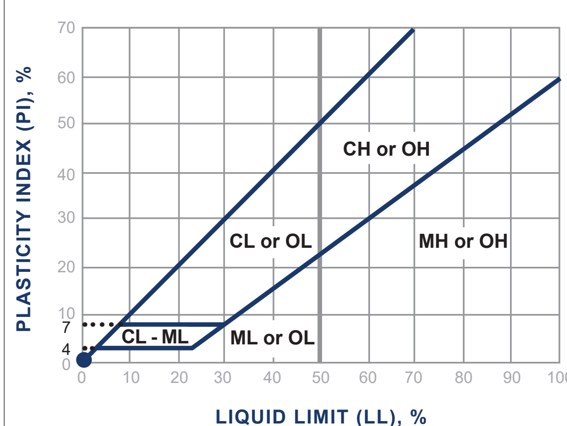
Soil Classification Chart Per ASTM D 2488

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 clay
		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 6/13/22 BORING NO. B-1

GROUND ELEVATION 300' ± (MSL) SHEET 1 OF 2

METHOD OF DRILLING 8" Hollow-Stem Auger (MR Drilling)

DRIVE WEIGHT 140 lbs (Auto. Trip Hammer) DROP 30"

SAMPLED BY SJQ LOGGED BY SJQ REVIEWED BY GMC

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0	■						SM	FILL: Brown, moist, loose, silty SAND.
	■						SM	ALLUVIUM: Brown, moist, loose, silty SAND.
		■	12	5.5	108.6			
10		■	18					Medium dense.
							SP-SM	Light brownish gray, moist, medium dense, poorly graded SAND with silt; fine to medium sand.
		■	11	3.8	102.9			Loose; trace gravel.
							ML	Gray, brown and reddish brown, moist, loose, sandy SILT; trace gravel; mottled.
							SP-SM	Brown to reddish brown, moist, medium dense, poorly graded SAND with silt; trace gravel.
20		◤	14					
							ML	Brown to grayish brown, moist, medium dense, sandy SILT.
		■	25	15.2	110.6			
							SM	Brown to grayish brown, moist, dense, silty SAND; interbedded with very thin beds of poorly graded sand.
30		◤	27					
		■	30				ML	Grayish brown, moist, medium dense, sandy SILT; oxidation staining.
40								

FIGURE A- 1

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DATE DRILLED	6/13/22	BORING NO.	B-1
GROUND ELEVATION	300' ± (MSL)	SHEET 2 OF 2	
METHOD OF DRILLING	8" Hollow-Stem Auger (MR Drilling)		
DRIVE WEIGHT	140 lbs (Auto. Trip Hammer)	DROP	30"
SAMPLED BY	SJQ	LOGGED BY SJQ	REVIEWED BY GMC

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
40			22				ML	ALLUVIUM: (Continued) Grayish brown, moist, dense, sandy SILT; oxidation staining.
			33				SM	Brown to reddish brown, moist, medium dense; silty SAND; interbedded with very thin beds of silt.
50								Very dense.
			38				SP	Light grayish brown, moist, very dense, poorly graded SAND; fine to medium sand; trace gravel.
								Total Depth = 51.5 feet. Groundwater was not encountered during drilling. Backfilled with on-site soil on 6/13/22. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
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80								

DATE DRILLED	6/13/22	BORING NO.	B-2
GROUND ELEVATION	305' ± (MSL)	SHEET 1 OF 1	
METHOD OF DRILLING	8" Hollow-Stem Auger (MR Drilling)		
DRIVE WEIGHT	140 lbs (Auto. Trip Hammer)	DROP	30"
SAMPLED BY	SJQ	LOGGED BY SJQ	REVIEWED BY GMC

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							ML	FILL: Brown, moist, medium dense; sandy SILT.
							ML	ALLUVIUM: Brown, moist, loose, sandy SILT; trace gravel.
			11	11.9	99.0			Grayish brown.
10			13				SM	Grayish brown, moist, loose, silty SAND.
								Total Depth = 11.5 feet. Groundwater was not encountered during drilling. Backfilled with on-site soil on 6/13/22. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
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DATE DRILLED	6/14/22	BORING NO.	B-3
GROUND ELEVATION	300' ± (MSL)	SHEET 1 OF 1	
METHOD OF DRILLING	8" Hollow-Stem Auger (MR Drilling)		
DRIVE WEIGHT	140 lbs (Auto. Trip Hammer)	DROP	30"
SAMPLED BY	SJQ	LOGGED BY SJQ	REVIEWED BY GMC

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	FILL: Brown, moist, loose to medium dense, silty SAND.
							SM	ALLUVIUM: Brown, moist, loose, silty SAND.
			9					Brown to grayish brown; iron oxidation staining.
							ML	Brown to light brown, moist, medium dense, sandy SILT; iron oxidation staining.
10			17	18.3	97.0			
								Grayish brown; increase in sand content.
			32	16.6	113.4		CL	Brown, moist, hard, lean CLAY; caliche stringers.
20								Very stiff.
			15				ML	Brown to grayish brown, moist, medium dense, sandy SILT.
								Total Depth = 21.5 feet. Groundwater was not encountered during drilling. Backfilled with on-site soil on 6/14/22.
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Notes:
Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

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

DATE DRILLED	6/14/22	BORING NO.	B-4
GROUND ELEVATION	300' ± (MSL)	SHEET 1 OF 1	
METHOD OF DRILLING	8" Hollow-Stem Auger (MR Drilling)		
DRIVE WEIGHT	140 lbs (Auto. Trip Hammer)	DROP	30"
SAMPLED BY SJQ	LOGGED BY SJQ	REVIEWED BY	GMC

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							ML	FILL: Brown to grayish brown, moist, loose, SILT with sand.
							ML	ALLUVIUM: Brown to grayish brown, moist, loose to medium dense, SILT with sand.
			24				SM	Brown to grayish brown, moist, medium dense, silty SAND.
							SP	Light brown to light grayish brown, moist, medium dense, poorly graded SAND.
10			25	10.1	102.5		ML	Light brown to light grayish brown, moist, medium dense, sandy SILT; trace gravel.
							SP	Light brown to light grayish brown, moist, loose, poorly graded SAND.
			10				ML	Grayish brown, moist, loose, sandy SILT.
							SM	Brown to grayish brown, moist, medium dense, silty SAND.
20			20				SP	Light grayish brown, moist, medium dense, poorly graded SAND.
30								
40								

Total Depth = 21.5 feet.
Groundwater was not encountered during drilling.
Backfilled with on-site soil on 6/14/22.

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

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

DATE DRILLED	6/13/22	BORING NO.	B-5
GROUND ELEVATION	300' ± (MSL)	SHEET 1 OF 1	
METHOD OF DRILLING	8" Hollow-Stem Auger (MR Drilling)		
DRIVE WEIGHT	140 lbs (Auto. Trip Hammer)	DROP	30"
SAMPLED BY	SJQ	LOGGED BY SJQ	REVIEWED BY GMC

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							ML	FILL: Brown, moist, medium dense, sandy SILT.
							ML	ALLUVIUM: Brown to grayish brown, moist, medium dense, sandy SILT.
							SP	Light brown to light grayish brown, moist, medium dense, poorly graded SAND; trace silt.
			23				ML	Light brown, moist, medium dense, SILT with sand.
10			23	16.0	96.8			Grayish brown; trace gravel; iron oxidation staining.
			26					
20			8					
								Total Depth = 21.5 feet. Groundwater was not encountered during drilling. Backfilled with on-site soil on 6/14/22. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
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DATE DRILLED	6/14/22	BORING NO.	B-6
GROUND ELEVATION	305' ± (MSL)	SHEET 1 OF 1	
METHOD OF DRILLING	8" Hollow-Stem Auger (MR Drilling)		
DRIVE WEIGHT	140 lbs (Auto. Trip Hammer)	DROP	30"
SAMPLED BY SJQ	LOGGED BY SJQ	REVIEWED BY	GMC

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0	X						ML	FILL: Brown, moist, medium dense, sandy SILT.
	X						SM	ALLUVIUM: Brown to grayish brown, moist, medium dense, silty SAND.
		X	15	6.1	110.3			
10		X	19				ML	Light brown to light grayish brown, moist, medium dense, SILT with sand.
								Total Depth = 11.5 feet. Groundwater was not encountered during drilling. Backfilled with on-site soil on 6/14/22. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
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DATE DRILLED	6/13/22	BORING NO.	B-7				
GROUND ELEVATION	300' ± (MSL)	SHEET	1	OF	1		
METHOD OF DRILLING	8" Hollow-Stem Auger (MR Drilling)						
DRIVE WEIGHT	140 lbs (Auto. Trip Hammer)	DROP	30"				
SAMPLED BY	SJQ	LOGGED BY	SJQ	REVIEWED BY	GMC		

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							ML	FILL: Brown, moist, loose to medium dense, sandy SILT.
							ML	ALLUVIUM: Light brown to grayish brown, moist, loose, sandy SILT.
			14	8.0	101.6			
10			67					Dense; decrease in sand content (becomes SILT with sand); trace gravel.
			24					Medium dense; iron oxidation staining.
20			8					
								Total Depth = 21.5 feet. Groundwater was not encountered during drilling. Backfilled with on-site soil on 6/13/22. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
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DATE DRILLED 6/14/22 BORING NO. B-8

GROUND ELEVATION 305' ± (MSL) SHEET 1 OF 2

METHOD OF DRILLING 8" Hollow-Stem Auger (MR Drilling)

DRIVE WEIGHT 140 lbs (Auto. Trip Hammer) DROP 30"

SAMPLED BY SJQ LOGGED BY SJQ REVIEWED BY GMC

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	FILL: Brown to grayish brown, moist, loose to medium dense, silty SAND.
							ML	ALLUVIUM: Brown to grayish brown, moist, loose to medium dense, sandy SILT.
			20					Light brown.
							SP-SM	Light grayish brown to light gray, moist, loose, poorly graded SAND with silt.
10			10					
							ML	Light brown to grayish brown, moist, loose, sandy SILT.
			13	19.1	106.2			
20			11					Medium dense.
			23					
30			30					Dense; iron oxidation staining.
							SP	Light brown to light grayish brown, moist, dense, poorly graded SAND.
			50				SM	Light brown to light grayish brown, moist, dense, silty SAND; interbedded with very thin beds of silt.
							SP	Light grayish brown, moist, dense, poorly graded SAND.
40								

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DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
40			32				SP	ALLUVIUM: (Continued) Light grayish brown, moist, dense, poorly graded SAND.
							CL	Brown to light brown, moist, hard, lean CLAY; trace silt.
			36				SM	Brown to light grayish brown, moist, medium dense, silty SAND.
							ML	Brown to grayish brown, moist, dense, sandy SILT.
50			23				SP	Light grayish brown, moist, dense, poorly graded SAND.
60								Total Depth = 51.5 feet. Groundwater was not encountered during drilling. Backfilled with on-site soil on 6/14/22. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
70								
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DATE DRILLED 6/14/22 BORING NO. B-8

GROUND ELEVATION 305' ± (MSL) SHEET 2 OF 2

METHOD OF DRILLING 8" Hollow-Stem Auger (MR Drilling)

DRIVE WEIGHT 140 lbs (Auto. Trip Hammer) DROP 30"

SAMPLED BY SJQ LOGGED BY SJQ REVIEWED BY GMC

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
								DATE DRILLED 6/13/22 BORING NO. P-1
								GROUND ELEVATION 300' ± (MSL) SHEET 1 OF 1
								METHOD OF DRILLING 8" Hollow-Stem Auger (MR Drilling)
								DRIVE WEIGHT 140 lbs (Auto. Trip Hammer) DROP 30"
								SAMPLED BY SJQ LOGGED BY SJQ REVIEWED BY GMC
0							SM	FILL: Brown to light brown, moist, medium dense, silty SAND.
							SM	ALLUVIUM: Brown to grayish brown, moist, medium dense, silty SAND; interbedded with silt.
								Total Depth = 5.5 feet. Groundwater was not encountered during drilling. In-situ percolation testing performed on 6/16/22. Backfilled with on-site soil on 6/16/22. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
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20								
30								
40								

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	FILL: Brown to light brown, moist, medium dense, silty SAND.
							SM	ALLUVIUM: Brown to grayish brown, moist, medium dense, silty SAND; interbedded with very thin beds of silt.
								Total Depth = 5.5 feet. Groundwater was not encountered during drilling. In-situ percolation testing performed on 6/16/22. Backfilled with on-site soil on 6/16/22. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
10								
20								
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DATE DRILLED 6/13/22 BORING NO. P-2

GROUND ELEVATION 300' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Hollow-Stem Auger (MR Drilling)

DRIVE WEIGHT 140 lbs (Auto. Trip Hammer) DROP 30"

SAMPLED BY SJQ LOGGED BY SJQ REVIEWED BY GMC

DATE DRILLED 6/13/22 BORING NO. P-3

GROUND ELEVATION 300' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Hollow-Stem Auger (MR Drilling)

DRIVE WEIGHT 140 lbs (Auto. Trip Hammer) DROP 30"

SAMPLED BY SJQ LOGGED BY SJQ REVIEWED BY GMC

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	FILL: Brown to light brown, moist, medium dense to dense, silty SAND.
							SM	ALLUVIUM: Brown, moist, medium dense to dense, silty SAND.
								Total Depth = 5.5 feet. Groundwater was not encountered during drilling. In-situ percolation testing performed on 6/16/22. Backfilled with on-site soil on 6/16/22. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
10								
20								
30								
40								

FIGURE A- 13

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VISALIA, CALIFORNIA

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	FILL: Brown, moist, medium dense, silty SAND.
	Bulk						SM	ALLUVIUM: Brown to grayish brown, moist, medium dense, silty SAND.
								Total Depth = 5.6 feet. Groundwater was not encountered during drilling. In-situ percolation testing performed on 6/16/22. Backfilled with on-site soil on 6/16/22. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
10								
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DATE DRILLED 6/13/22 BORING NO. P-4

GROUND ELEVATION 305' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Hollow-Stem Auger (MR Drilling)

DRIVE WEIGHT 140 lbs (Auto. Trip Hammer) DROP 30"

SAMPLED BY SJQ LOGGED BY SJQ REVIEWED BY GMC

FIGURE A- 14

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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 and dry density of relatively undisturbed samples obtained from the exploratory borings were evaluated in general accordance with ASTM D 2937. The test results are presented on the logs of the exploratory borings in Appendix A.

Gradation Analysis
Gradation analysis tests were performed on selected representative soil samples in general accordance with ASTM D 422. 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.

200 Wash
An evaluation of the percentage of particles passing the No. 200 sieve in selected soil samples was performed in general accordance with ASTM D 1140. The results of the tests are summarized on Figures B-4 and B-5.

Atterberg Limits
Tests were performed on selected representative fine-grained soil samples to evaluate the liquid limit, plastic limit, and plasticity index in general 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-6.

Consolidation Tests
Consolidation tests were performed on selected relatively undisturbed soil samples in general accordance with ASTM D 2435. The samples were inundated during testing to represent adverse field conditions. The percent of consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The results of the tests are summarized on Figures B-7 and B-8.

Direct Shear Test
A direct shear test was performed on a remolded sample in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of the selected material. The sample was inundated during shearing to represent adverse field conditions. The results are shown on Figure B-9.

Expansion Index Tests
The expansion index of selected material was evaluated in general accordance with ASTM D 4829. Specimens were molded under a specified compactive energy at approximately 50 percent saturation (plus or minus 1 percent). The prepared 1-inch thick by 4-inch diameter specimens were loaded with a surcharge of 144 pounds per square foot and were inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The results of these tests are presented on Figure B-10.

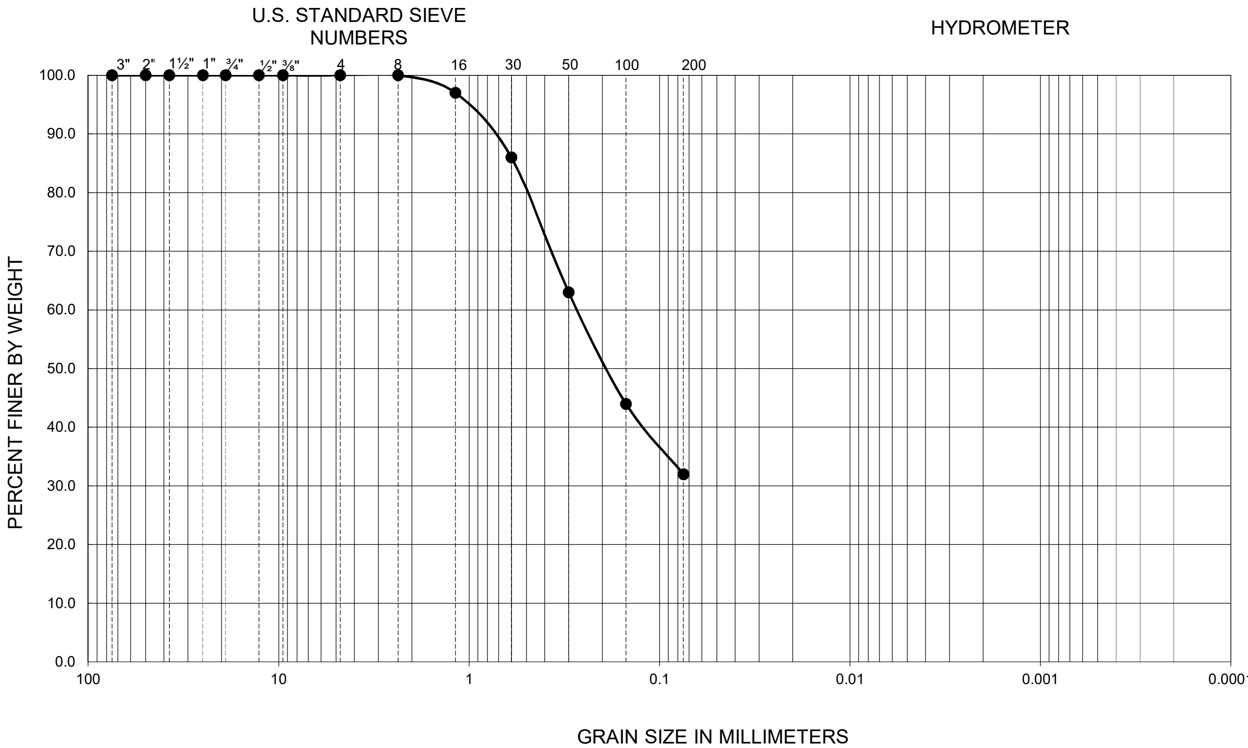
R-Value

The resistance value, or R-value, of near-surface site soils were 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-11.

Soil Corrosivity Tests

Soil pH and resistivity tests were performed on representative samples in general accordance with CT 643. The soluble sulfate and chloride content of the selected samples were evaluated in general accordance with CT 417 and CT 422, respectively. The test results are presented on Figure B-12.

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-1	0.0-5.0	--	--	--	--	--	--	--	--	32	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

FIGURE B-1

GRADATION TEST RESULTS

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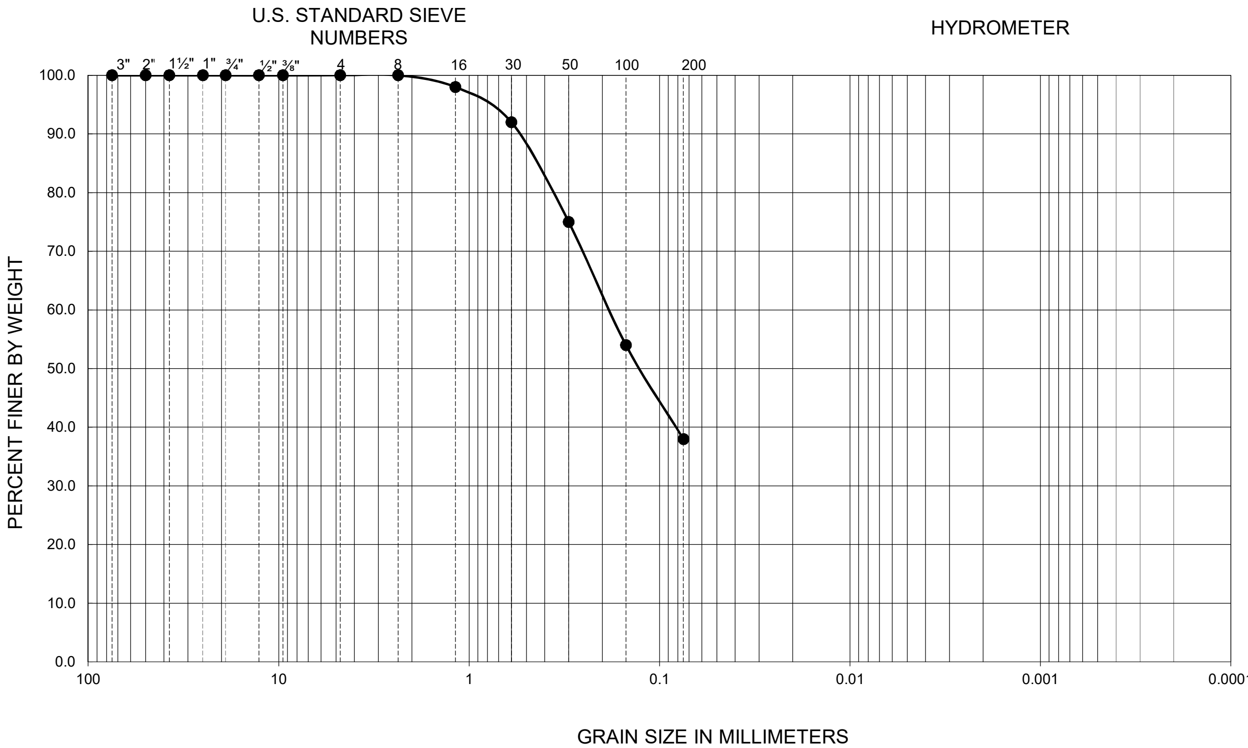
GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



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
●	P-1	1.0-5.5	--	--	--	--	--	--	--	--	38	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

FIGURE B-2

GRADATION TEST RESULTS

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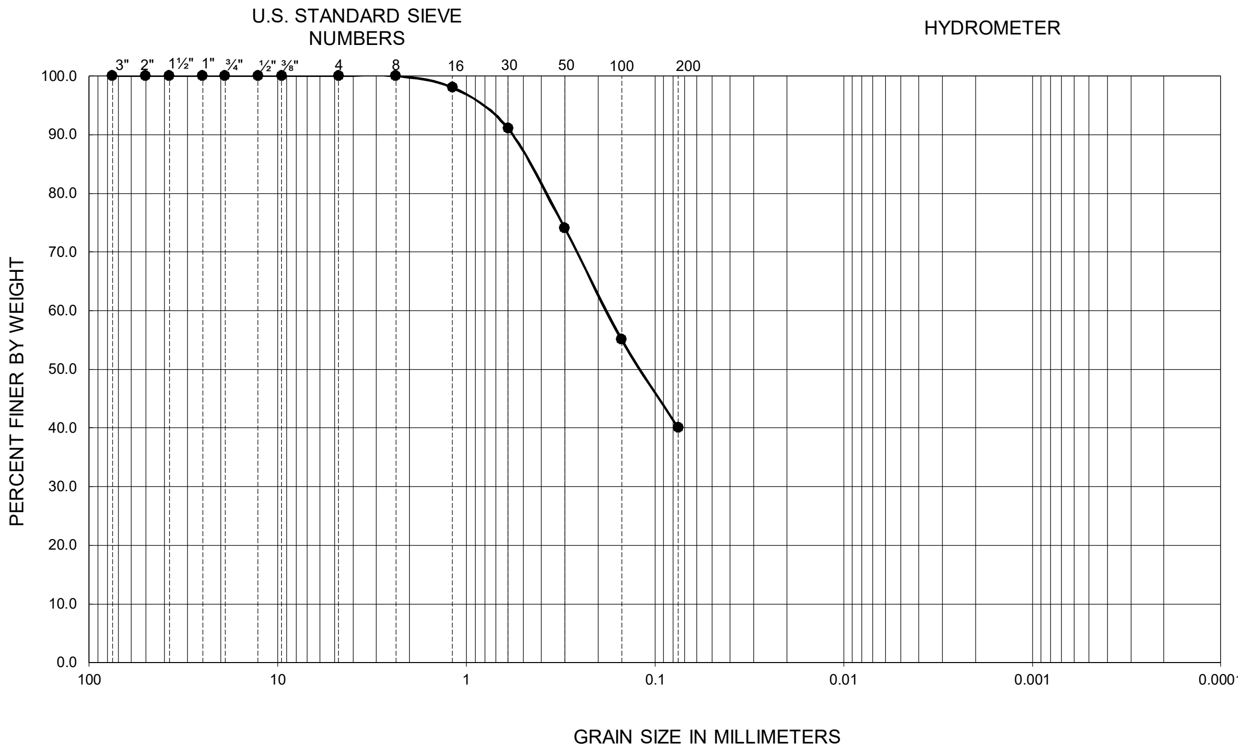
GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



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
●	P-3	1.0-5.0	--	--	--	--	--	--	--	--	40	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

FIGURE B-3

GRADATION TEST RESULTS

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	USCS (TOTAL SAMPLE)
B-1	15.0-16.0	POORLY GRADED SAND WITH SILT	97	6	SP-SM
B-2	0.5-5.0	SANDY SILT	99	50	ML
B-3	1.0-5.0	SILTY SAND	100	41	SM
B-4	0.0-4.0	SILT WITH SAND	100	74	ML
B-5	10.0-11.5	SILT WITH SAND	96	72	ML
B-6	10.0-11.5	SILT WITH SAND	100	83	ML
B-7	10.0-11.5	SILT WITH SAND	98	77	ML
B-8	10.0-11.5	POORLY GRADED SAND WITH SILT	100	7	SP-SM
B-8	30.0-31.5	SANDY SILT	100	51	ML

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1140

FIGURE B-4

NO. 200 SIEVE ANALYSIS TEST RESULTS

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	USCS (TOTAL SAMPLE)
P-2	1.0-5.0	SILTY SAND	100	34	SM
P-4	1.0-5.0	SILTY SAND	100	30	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1140

FIGURE B-5

NO. 200 SIEVE ANALYSIS TEST RESULTS

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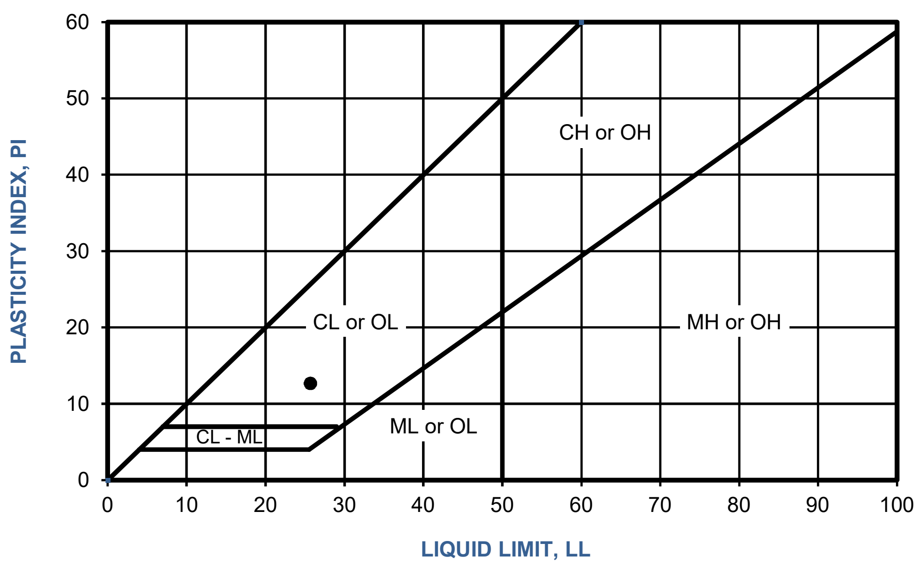
SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS
●	B-3	15.0-16.5	26	13	13	CL	CL
■	B-7	0.0-5.0				NP	ML

NP - INDICATES NON-PLASTIC



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

FIGURE B-6

ATTERBERG LIMITS TEST RESULTS

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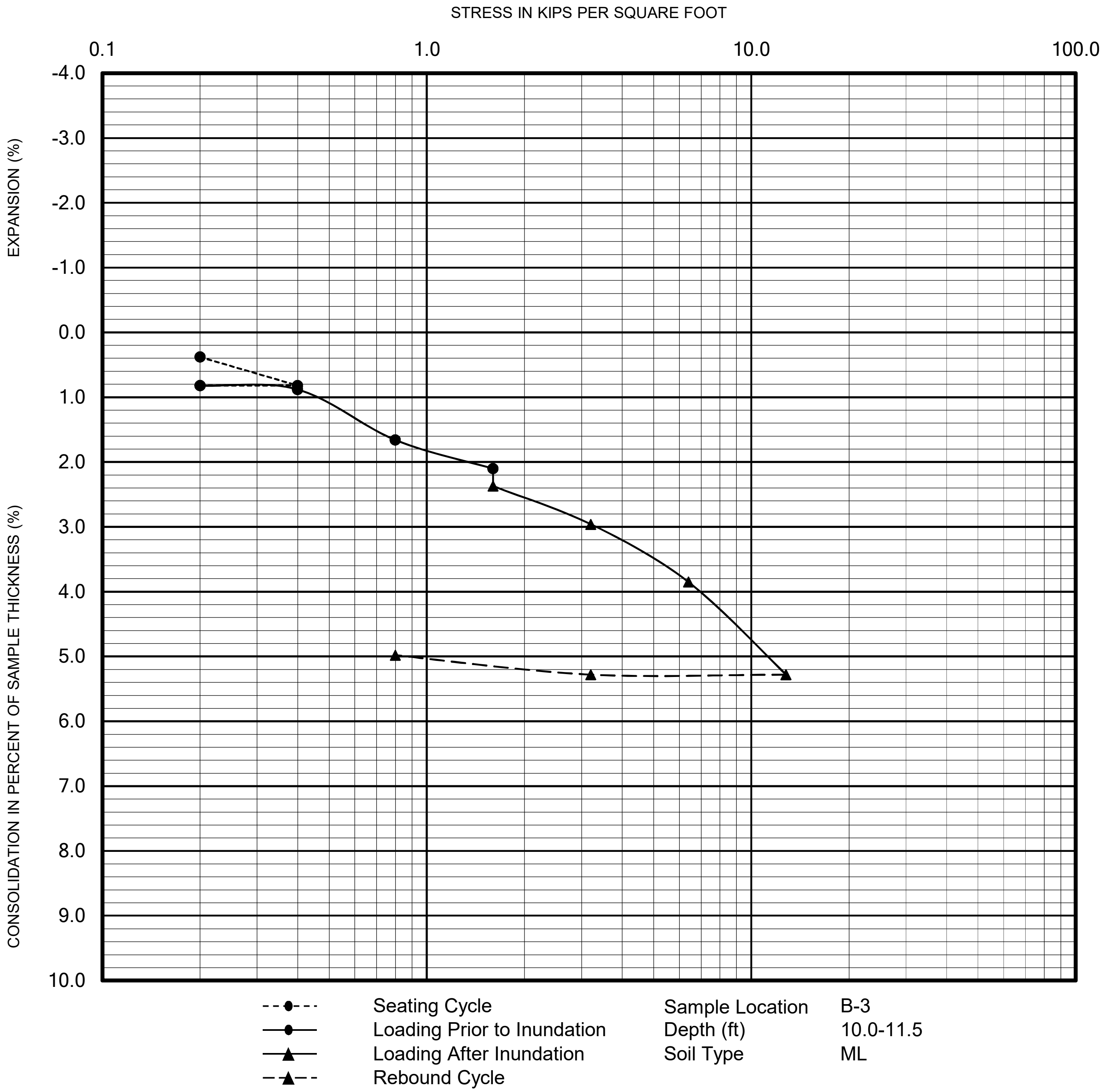

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

FIGURE B-7

CONSOLIDATION TEST RESULTS

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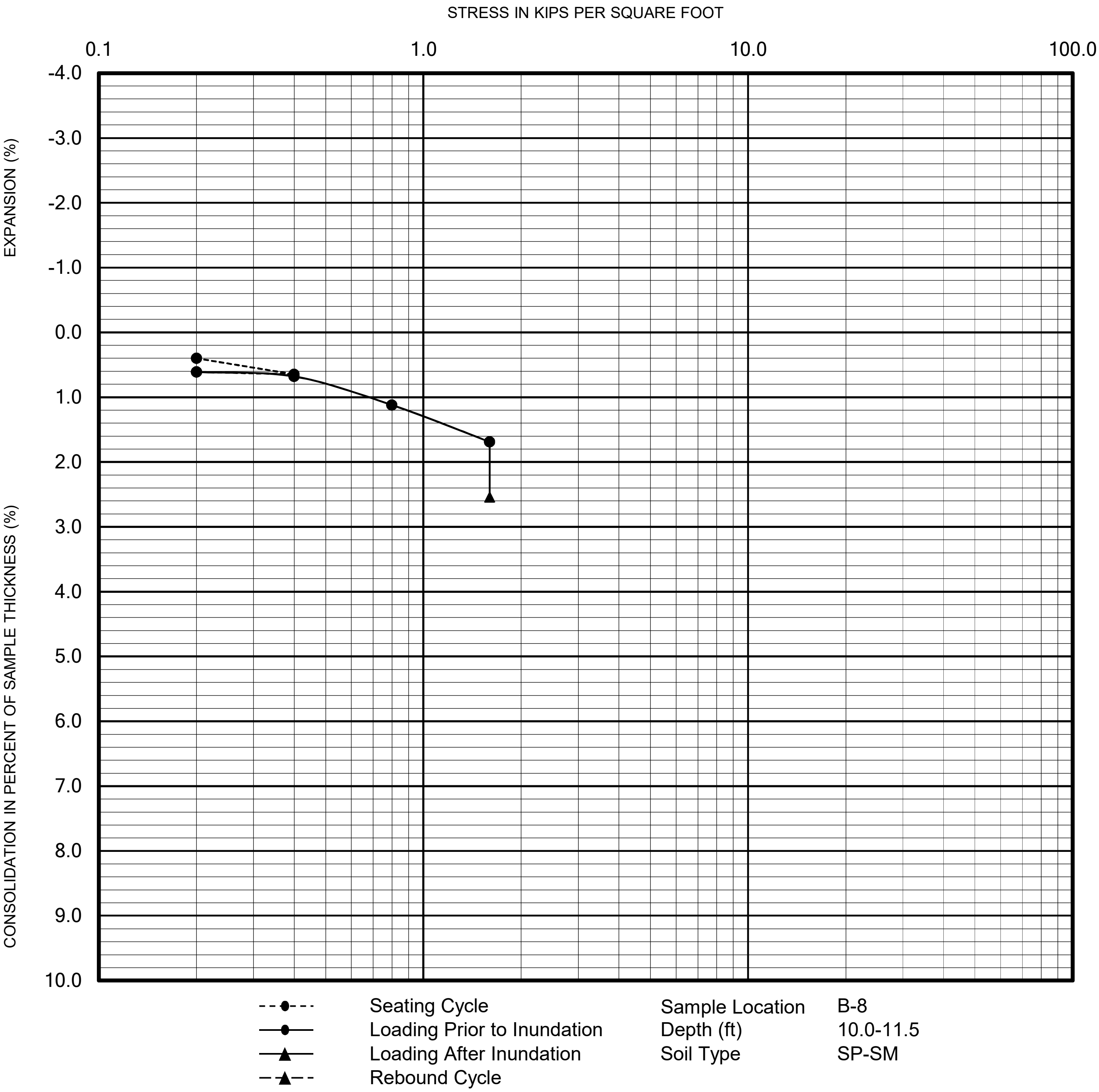

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

FIGURE B-8

CONSOLIDATION TEST RESULTS

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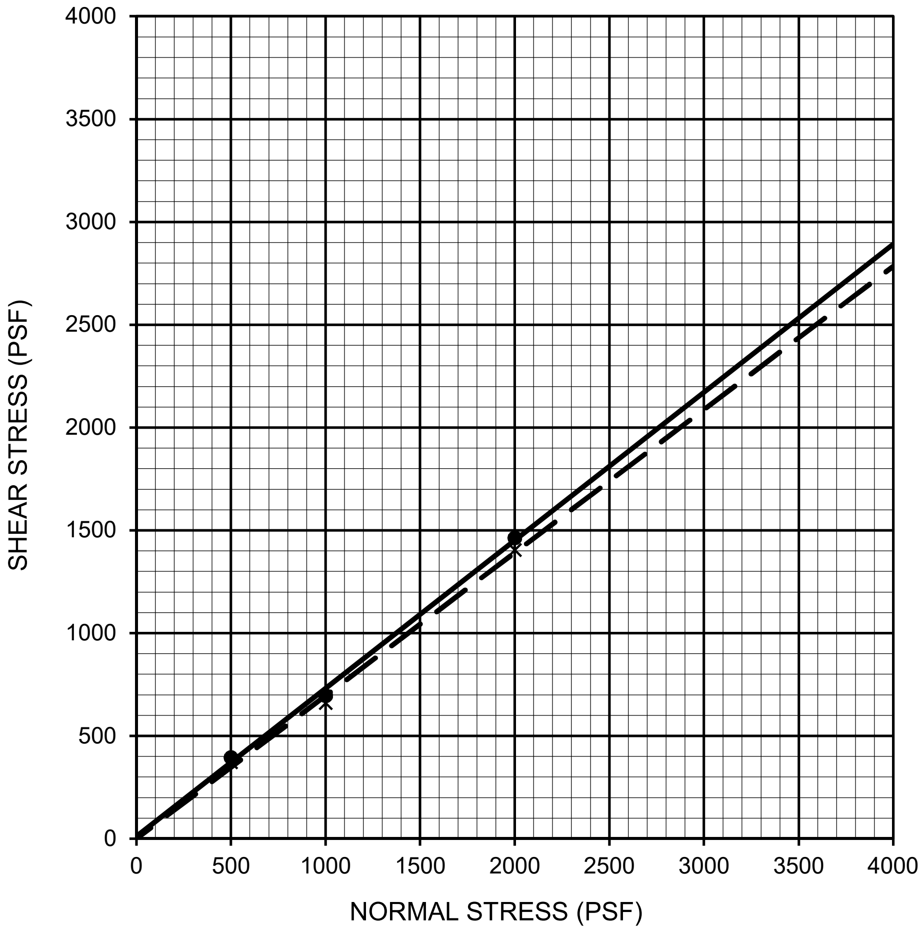

Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
SANDY SILT	——●——	B-7	5.0-6.5	Peak	12	36	ML
SANDY SILT	– – X – –	B-7	5.0-6.5	Ultimate	0	35	ML

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

FIGURE B-9

DIRECT SHEAR TEST RESULTS

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	INITIAL MOISTURE (percent)	COMPACTED DRY DENSITY (pcf)	FINAL MOISTURE (percent)	VOLUMETRIC SWELL (in)	EXPANSION INDEX	POTENTIAL EXPANSION
B-5	0.0-5.0	8.5	116.3	12.8	0.001	1	Very Low

PERFORMED IN GENERAL ACCORDANCE WITH ☐ UBC STANDARD 18-2 ☑ ASTM D 4829

FIGURE B-10

EXPANSION INDEX TEST RESULTS

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	SOIL TYPE	R-VALUE
B-2	0.5-5.0	SANDY SILT	53
B-6	0.5-5.0	SILTY SAND	72
B-8	0.5-5.0	SANDY SILT	59
P-1	1.0-5.0	SILTY SAND	69

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2844/CT 301

FIGURE B-11

R-VALUE TEST RESULTS

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	pH [1]	RESISTIVITY [1] (ohm-cm)	SULFATE CONTENT [2]		CHLORIDE CONTENT [3] (ppm)
				(ppm)	(%)	
B-1	0.0-5.0	6.4	4,990	10	0.001	20
B-4	0.0-5.0	7.4	2,110	10	0.001	30

[1] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 643
[2] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 417
[3] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 422

FIGURE B-12

CORROSIVITY TEST RESULTS

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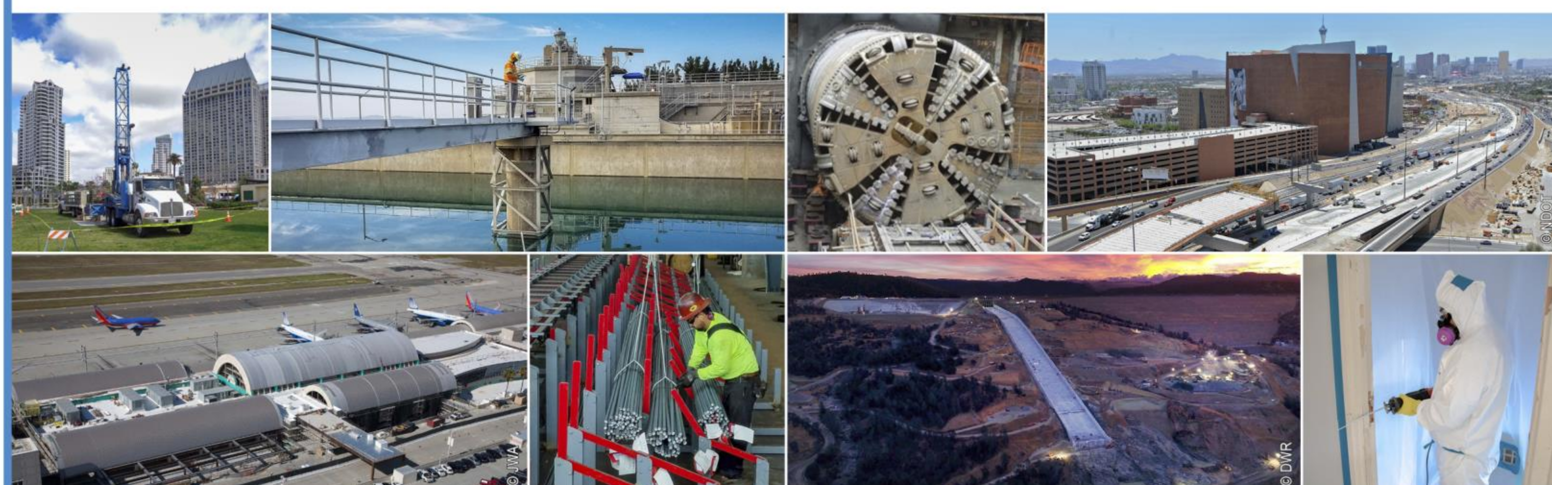

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E.2 - Paleontological Records Search

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Kenneth L. Finger, Ph.D.
Consulting Paleontologist

18208 Judy St., Castro Valley, CA 94546-2306   510.305.1080   klfpaleo@comcast.net

June 20, 2022

Dana DePietro
FirstCarbon Solutions
1350 Treat Boulevard, Suite 380
Walnut Creek, CA 94597

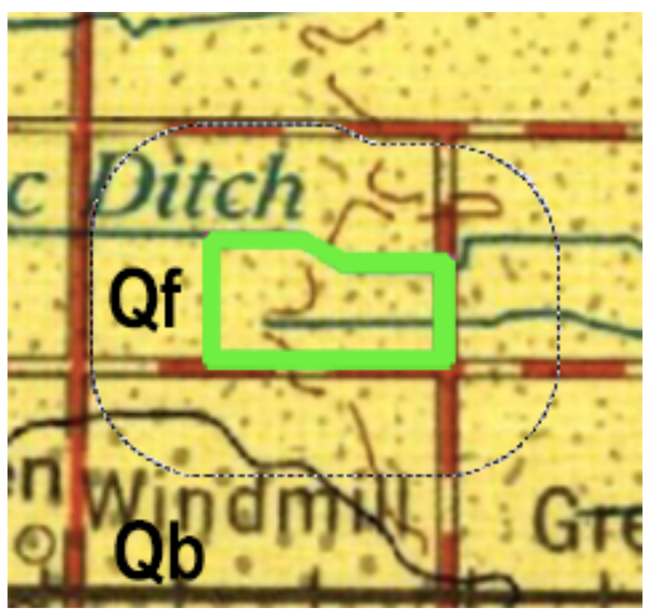
Re: Paleontological Records Search for the Shirk & Riggin Industrial Park Project (4119.0039), near the City of Visalia, Tulare County

Dear Dr. DePietro:

As per the request of Madelyn Dolan, I have performed a paleontological records search on the University of California Museum of Paleontology (UCMP) database for the Shirk & Riggin Industrial Park Project on unincorporated land in the City of Visalia's Sphere of Influence. A farm currently occupies the proposed 280-acre site, which is bounded by Avenue 320 to the north, Road 92 to the east, and North Kelsey Street to the west. Its Public Land Survey (PLS) location is S½, Sec. 16, T18S, R24E, Goshen and Visalia quadrangles (USGS 7.5-series topographic map). The applicant is proposing construction of a logistics center that will be annexed by the City of Visalia. The total building footprint of 3,686,350 square feet will consist of eight industrial buildings, 15 smaller flex industrial buildings, a convenience store, a car wash facility, and two drive-thrus.

<u>Geologic Units</u>

According to the part of the geologic map of Matthews and Burnett (1965) shown here, the project site (green outline at center) is solely upon Holocene Great Valley fan deposits (Qf). The southwest part of the surrounding half-mile search area (dotted black outline) also includes subjacent Holocene Great Valley basin deposits (Qb). The nearest Pleistocene deposits are more than five miles east of the project site.



<u>Records Search</u>

The database search focused on the Pleistocene of Tulare County and adjacent Fresno and Kings counties. Tulare County has 10 vertebrate and no plant localities listed. Nearest to the project site is locality V6540 (Tulare Co General), 5.5 miles to the southeast, which yielded mammoth tooth fragments. Fresno County has six vertebrate and 12 plant localities and Kings County has 10 vertebrate and no plant localities, but none are within 10 miles of the project site. The absence of any potentially fossiliferous localities within five miles suggests that the Pleistocene layer is well below the depth of all excavations that will be made for this project.

Remarks and Recommendations

The Holocene Great Valley deposits are too young to be fossiliferous and it is highly unlikely that Pleistocene deposits are in the shallow subsurface of the project site. Thus, neither a paleontological walkover survey nor construction monitoring is recommended.

In the highly unlikely event that any significant paleontological resources (i.e., bones, teeth, or unusually abundant and well-preserved invertebrates or plants) be unearthed, the crew should not attempt to remove them**,** as they could be extremely fragile and therefore prone to crumbling, and to ensure their occurrence is properly recorded; instead, all work in the immediate vicinity of the discovery should be diverted at least 15 feet until a professional paleontologist assesses the find and, if deemed appropriate, salvages it in a timely manner. All recovered fossils should be deposited in an appropriate repository, such as the UCMP, where they will be properly curated and made accessible for future study.

Sincerely,

Ken Finger

Reference Cited

Matthews, R.A., and Burnett, J.L, 1965, Geologic map of California: Fresno sheet. California Division of Mines and Geology, scale: 1:250,000.