Geo-Engineering Solutions, Inc.

Geotechnical Engineering • Engineering Geology • Materials Testing

GEOTECHNICAL ENGINEERING STUDY

Double T Ranch-Barn, Office Shop Complex
8325 Quail Canyon Road
Vacaville, CA 95688

September 7, 2023

Prepared for:

8325 QCR, LLC
756 El Pintado Road
Danville, California 94526

Prepared by:

Geo-Engineering Solutions, Inc.
2570 San Ramon Valley Blvd, Suite #A102
San Ramon, California 94583
Project No. 109-1519C

September 7, 2023

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Subject: **Geotechnical Engineering Study**
Double T Ranch-Barn, Office Shop Complex
8325 Quail Canyon Road
Vacaville, California. 95688
Geo-Eng Project No. 109-1519C

Dear Mr. Nuti:

Geo-Engineering Solutions, Inc. has prepared a Geotechnical Engineering Study for the proposed multi use equestrian development project at the property located at 8325 Quail Canyon Road, Vacaville California. The site of the planned development was previously occupied by a residence with some auxiliary structures that were lost in a fire in 2020. Subsequently some roadways and stables have been developed on this site, but the new development will be much more significant than the existing development.

Transmitted herewith are the results of our findings, conclusions, and recommendations for the design and construction of proposed foundation support, interior concrete slabs, site development/grading and drainage, and utility trench backfilling. In general, the proposed improvements at the site are considered to be geotechnically feasible provided the recommendations in this report are implemented in the design and construction of the project.

Should you or members of the design team have questions or need additional information, please contact the undersigned at (925) 433-0450 or by e-mail at eswenson@geo-eng.net. We greatly appreciate the opportunity to be of your service and to be involved in the design of this project.

Sincerely,

GEO-ENGINEERING SOLUTIONS, INC.

Nicolas Haddad, PE
Senior Geotechnical Engineer

Eric J. Swenson, GE, CEG
Principal Engineer and Geologist

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Boring Logs

APPENDIX B

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1.0 INTRODUCTION

1.1 Purpose and Scope

The purpose of our work was to prepare a Geotechnical Engineering Study, evaluate the subsurface conditions at the site and prepare geotechnical recommendations for the proposed development. We have provided specific recommendations regarding suitability and geotechnical concerns relative to the proposed structural design.

The scope of this study included field exploration, laboratory testing, engineering analysis of the collected samples and test results, and preparation of this report. The conclusions and recommendations presented in this report are based on the limited samples collected and analyzed during this study, and on prudent engineering judgment and experience. This study did not include an in-depth assessment of potentially toxic or hazardous materials that may be present on or beneath the site.

1.2 Site Description

The proposed improvement project is located at 8325 Quail Canyon Road in Vacaville, California as shown on *Figure 1, Site Vicinity Map*. The subject site is an approximately 53-acres size parcel of land in the rural area of northern Vacaville, California. The area consists of low rolling hills with grasslands and oak trees and shrubs. This parcel of land is part of a larger Double T Ranch development which you are currently developing.

The topography of the site includes low rolling hills, with an approximate elevation of +308 based on Google Earth Elevations, and a central east-west valley with a creek and to the east and a flat valley area to the west an existing 2-acre stock pond in the center of the site. The average geographical coordinates used in our engineering analyses are 38.4734 degrees north latitude and -122.0506 degrees west longitude.

1.3 Proposed Development

The site of the planned development was previously occupied by a residence with some auxiliary structures that were lost in a fire in 2020. Subsequently some roadways and stables have been developed on this site, but the new development will be much more significant than the existing development. The newly developed area will be approximately 12 acres of the existing 52-acre site, as shown on *Figure 2, Site Development Plan*. Grading will include the development of building pads as well as new roadways. In addition to construction of the residence, there will be various associated site improvements such as grading, landscaping, paving, and utilities.

1.4 Validity of Report

This report is valid for three years after publication. If construction begins after this time, Geo-Eng should be contacted to confirm that the site conditions have not changed significantly. If the proposed development differs considerably from that described above, Geo-Eng should be notified to determine if additional recommendations are required. Additionally, if Geo-Eng is not involved during the geotechnical aspects of construction, this report may become wholly or in part invalid; since Geo-Eng's geotechnical personnel need to verify that the subsurface conditions anticipated preparing this report are similar to the subsurface conditions revealed during construction. Geo-Eng's involvement should include foundation and grading plan review; observation of foundation excavations; grading observation and testing; testing of utility trench backfill.

2.0 PROCEDURES AND RESULTS

2.1 Literature Review

Pertinent geologic and geotechnical literature pertaining to the site area, and previous geotechnical studies performed by others for projects in the site vicinity were reviewed. These included United States Geological Survey (USGS), California Geological Survey (CGS), and other online resources, and other applicable government and private publications and maps, as included in the References section.

2.2 Field Exploration

Our field exploration program consisted of drilling 8 test borings as shown on *Figure 3, Site Map and Boring Locations.* The eight borings were drilled at the site on August 14, 2023, by California Geotech Services, using a truck mounted drill rig equipped with 4-inch diameter solid flight auger, to a maximum depth of 30 feet below existing ground surface.

A Geo-Eng Staff Engineer visually classified the materials encountered in the borings according to the Unified Soil Classification System as the borings were advanced. Relatively undisturbed soil samples were recovered at selected intervals using a three-inch outside diameter Modified California split spoon sampler containing six-inch long brass liners. A two-inch outside diameter Standard Penetration Test (SPT) sampler was also used to obtain SPT blow counts and obtain disturbed soil samples. The samplers were driven by using a 140-pound safety hammer with an approximate 30-inch fall utilizing N-rods as necessary. Resistance to penetration was recorded as the number of hammer blows required to drive the sampler to the final foot of an 18-inch drive. All the blow counts recorded using Modified California split spoon samplers in the field were converted to equivalent SPT blow counts using appropriate modification factors suggested by Burmister (1948), i.e., a factor of 0.65 assuming an inner diameter of 2.5 inches. Therefore, all blow counts shown on the final boring logs are either directly measured (SPT sampler) or equivalent SPT (MC sampler) blow counts. Bulk samples were obtained from the upper few feet of the borings or from the auger cuttings as needed.

The boring logs with descriptions of the various materials encountered in each boring, the penetration resistance values, and the laboratory test results are presented in Appendix A. The ground surface elevations indicated on the soil boring logs were determined using Google Earth. Actual surface elevations at the boring locations may differ slightly than indicated. The locations of the borings should only be considered accurate to the degree implied by the means and methods used to define them.

2.3 Laboratory Testing

Laboratory tests were performed on selected samples to determine some of the physical and engineering properties of the subsurface soils. The results of the laboratory testing are presented on the boring logs, and included in Appendix B. The following soil tests were performed for this study:

Dry Density and Moisture Content (ASTM D2216 and ASTM 2937) – In-situ dry density and/or moisture tests were conducted on various samples to measure the in-place dry density and moisture content of the subsurface materials. These properties provide information to assist in evaluating the physical characteristics of the subsurface soils. Test results are shown on the boring logs.

Atterberg Limits (ASTM D4318 and CT204) – Liquid Limit, Plastic Limit, and Plasticity Index are useful in the classification and characterization of the engineering properties of soil, helps evaluate the expansive characteristics of the soil, and for determining the soil type according to the USCS. Test results are presented in Section 4.1, in Appendix B, and on the applicable boring logs.

Particle Size Analysis (Wet and Dry Sieve) and Fines Content (ASTM D422 and D1140) - Sieve analysis or fines content (minus No. 200 sieve) tests were conducted on several selected samples to measure the soil particle size distribution. This information is useful for the evaluation of liquefaction potential and characterizing the soil type according to USCS. Test results are presented on the boring logs or in Appendix B.

Unconfined Compressive Strength (ASTM D2166) – Unconfined compressive strength tests were run on several liner samples to obtain strength parameters for use in foundation and retaining wall design. Test results are presented on the boring logs or in Appendix B.

Soil Corrosivity, Redox (ASTM D1498), pH (ASTM D4972), Resistivity (ASTM G57), Chloride (ASTM D4327), and Sulfate (ASTM D4327) – Soil corrosivity testing was performed to determine the effects of constituents in the soil on buried steel and concrete. Water-soluble sulfate testing is required by the CBC and IBC. Test results are presented in Section 4.3 and in Appendix B.

3.0 GEOLOGY AND SEISMICITY

3.1 Geologic Setting

The site is located within the central portion of the Coast Ranges geomorphic province of California. The Coast Ranges geomorphic province consists of numerous small to moderate linear mountain ranges trending north to south and northwest to southeast. The Coast Ranges lies between the Pacific Ocean to the west and the Great Valley Geomorphic Province to the east. This province is approximately 400 miles long and extends from the Klamath Mountains in the north to the Santa Ynez River within Santa Barbara County in the south. It generally consists of marine sedimentary rocks and volcanic rocks. The province is characterized by northwest-trending faults and folds, as well as erosion and deposition within the broad transform boundary between the North American and Pacific plates. Translational motion along the plate boundary occurs across a distributed zone of right-lateral shear expressed as a nearly 50-mile-wide zone of northwest-trending, near-vertical active strike-slip faults. This motion occurs primarily along the active San Andreas, Hayward, Calaveras and San Gregorio faults.

The site is located northeast of the San Francisco Bay Area and southwest of Sacramento. The site is underlain by Quaternary aged alluvial sediments deposited from historic and recent stream channels. The alluvium is underlain by Eocene aged sedimentary rocks consisting of shale and sandy mudstone (Dibblee and minch, 2007), Figure 4-Site Vicinity Geologic Map.

3.2 Seismic Setting

Regional transpression has caused uplift and folding of the bedrock units within the Coast Ranges. This structural deformation occurred during periods of tectonic activity that began in the Pliocene and continues today. The Bay Area of Northern California is a seismically active region dominated by four major northwest trending right lateral strike slip faults that include the San Andreas Fault, the Rodgers Creek, and the Green Valley Fault.

Major faults near the subject property include the San Andreas Fault located about 53 miles southwest, the Rodgers Creek Fault located about 33 miles southwest, the West Napa Fault located 22 miles west, and the Green Valley Fault located about 11 miles west, Figure 5-Regional Fault Map. Additional notable faults near the subject property include the Great Valley Thrust Fault located about 0.8 miles east of the project site.

The State of California Earthquake Zones of Required Investigation map shows the subject property is not in a liquefaction zone, Figure 6-Seismic Hazard Map.

4.0 FIELD AND LABORATORY FINDINGS

Subsurface conditions below the project site were interpreted based on the results of the test borings performed for this study, as well as the results of our laboratory testing. Detailed descriptions of the various subsurface soil units encountered during subsurface explorations are described in the following paragraphs.

4.1 Subsurface Soil Conditions

Subsurface conditions below the project site were interpreted based on the results of the test borings performed for this study and the results of our laboratory testing.

During our subsurface exploration program, we investigated the subsurface soils and evaluated soil conditions to a maximum depth of 30 feet in the soil borings performed for this study. From the ground surface to the maximum depth explored, the soils underlying the project site consist primarily of layers of stiff to very stiff silty clay and weathered claystone to the maximum depth explored of 15 feet below ground surface.

4.2 Atterberg Limits

- A soil sample of the near surface fine grained material from boring B-1 at 3.5 feet below ground surface was tested for Atterberg Limits, with measured Liquid Limits (LL) of 37, Plastic Limits (PL) of 19, and corresponding Plasticity Index (PI) of 18.
- A soil sample of the near surface fine-grained material from boring B-2 at 1 foot below ground surface was tested for Atterberg Limits, with measured LL of 52, PL of 26, and corresponding PI of 26.
- A soil sample of the near surface fine-grained material from boring B-4 at 1 foot below ground surface was tested for Atterberg Limits, with measured LL of 38, PL of 22, and corresponding PI of 16.
- A soil sample of the near surface fine-grained material from boring B-6 at 3.5 feet below ground surface was tested for Atterberg Limits, with measured LL of 46, PL of 28, and corresponding PI of 18.

Based on these test results the near surface soil would be considered to be of moderate to high plasticity and have a moderate to high expansion potential. Additional details of the soils encountered in the exploratory borings are included in the boring log presented in Appendix A.

4.3 Groundwater

Free groundwater was encountered in boring B-7 at 12 feet below ground surface. The borings were backfilled with a neat cement grout shortly after drilling. We note that the borings may not have been left open for a sufficient period of time to establish equilibrium groundwater conditions. Groundwater levels can vary in response to time of year, variations in seasonal rainfall, tidal influence, well pumping, irrigation, and alterations to site drainage. Based on a review of historic high groundwater in the seismic hazard mapping report we would recommend a deign groundwater elevation of 10 feet below the ground surface.

4.4 Corrosion Testing

A bulk sample collected from the upper two feet of Boring B-7 was tested to measure sulfate content, chloride content, redox potential, pH, resistivity, and presence of sulfides. Test results are included in Appendix B and are summarized in the following tables.

Table 1: Summary of Corrosion Test Results

Soil Description	Sample Depth (feet)	Sulfate (mg/kg)	Chloride (mg/kg)	Redox (mV)	Resistivity (ohm-cm)	Sulfide	pH
Brown Silty CLAY	4	N.D.	N.D.	160	2,900	Negative	7.55

Water-soluble sulfate can affect the concrete mix design for concrete in contact with the ground, such as shallow foundations, piles, piers, and concrete slabs. Section 4.3 in American Concrete Institute (ACI) 318, as referenced by the CBC, provides the following evaluation criteria:

Table 2: Sulfate Evaluation Criteria

Sulfate Exposure	Water-Soluble Sulfate in Soil, Percentage by Weight or (mg/kg)	Sulfate in Water, ppm	Cement Type	Max. Water Cementitious Ratio by Weight	Min. Unconfined Compressive Strength, psi
Negligible	0.00-0.10 (0-1,000)	0-150	NA	NA	NA
Moderate	0.10-0.20 (1,000-2,000)	150-1,500	II, IP (MS), IS (MS)	0.50	4,000
Severe	0.20-2.00 (2,000-20,000)	1,500-10,000	V	0.45	4,500
Very Severe	Over 2.00 (20,000)	Over 10,000	V plus pozzolan	0.45	4,500

The water-soluble sulfate content was not detected in the soil sample, suggesting the site soil should have negligible impact on buried concrete structures at the site. However, it should be pointed out that the water-soluble sulfate concentrations can vary due to the addition of fertilizer, irrigation, and other possible development activities.

Table 4.4.1 in ACI 318 suggests use of mitigation measures to protect reinforcing steel from corrosion where chloride ion contents are above 0.06% by dry weight. The chloride content was not detected in the soil sample. Therefore, the test result for chloride content does not suggest a corrosion hazard for mortar-coated steel and reinforced concrete structures due to high concentration of chloride.

In addition to sulfate and chloride contents described above, pH, oxidation reduction potential (Redox), and resistivity values were measured in the soil sample. For cast and ductile iron pipes, an evaluation was based on the 10-Point scaling method developed by the Cast Iron Pipe Research Association (CIPRA) and as detailed in Appendix A of the American Water Works Association (AWWA) publication C-105 and shown on Table 3.

Table 3: Soil Test Evaluation Criteria (AWWA C-105)

Soil Characteristics	Points		Soil Characteristics	Points
Resistivity, ohm-cm, based on single probe or water-saturated soil box.			**Redox Potential, mV**	
<700	10		>+100	0
700-1,000	8		+50 to +100	3.5
1,000-1,200	5		0 to 50	4
1,200-1,500	2		Negative	5
1,500-2,000	1		**Sulfides**	
>2,000	0		Positive	3.5
PH			Trace	2
0-2	5		Negative	0
2-4	3		**Moisture**	
4-6.5	0		Poor drainage, continuously wet	2
6.5-7.5	0		Fair drainage, generally moist	1
7.5-8.5	0		Good drainage, generally dry	0
>8.5	5			

Assuming fair site drainage, the tested soil sample had a total score of 1 point, indicating a low corrosive rating. When total points on the AWWA corrosivity scale are at least 10, the soil is classified as corrosive to cast and ductile iron pipe and use of cathodic corrosion protection is often recommended.

These results are preliminary and provide information only on the specific soil sampled and tested. Other soil at the site may be more or less corrosive. Providing a complete assessment of the corrosion potential of the site soil is not within our scope of work. For specific long-term corrosion control design recommendations, we recommend that a California-registered professional corrosion engineer evaluate the corrosion potential of the soil environment on buried concrete structures, steel pipe coated with cement-mortar, and ferrous metals.

5.0 GEOLOGIC HAZARDS

5.1 Seismic Induced Hazards

Seismic hazards resulting from the effects of an earthquake generally include ground shaking, liquefaction and dynamic settlement (densification), lateral spreading, fault ground rupture and fault creep, and tsunamis and seiches. The site is not necessarily impacted by these potential seismic hazards. Applicable potential seismic hazards are discussed and evaluated in the following sections in relation to the planned construction.

5.1.1 Ground Shaking

The site will likely experience severe ground shaking from a major earthquake originating from many significant faults in the San Francisco Bay Area, including the San Andreas, Rodgers Creek and the West Napa Faults. Earthquake intensities vary throughout the Bay Area depending upon the magnitude of the earthquake, the distance of the site from the causative fault, the type of materials underlying the site and other factors.

In addition to shaking of the structure, strong ground shaking can induce other related phenomena that may influence structures, such as liquefaction or dynamic densification settlement; adjacent seismic slope failure, lurching or lateral spreading, or seismically induced waves (tsunamis and seiches).

5.1.2 Liquefaction Induced Phenomena

Research and historical data indicate that soil liquefaction generally occurs in saturated, loose granular soil (primarily fine to medium-grained, clean, poorly-graded sand deposits) during or after strong seismic ground shaking and is typified by a loss of shear strength in the affected soil layer, thereby causing the soil to flow as a liquid. Typically, liquefaction potential increases with increased duration and magnitude of cyclic loading. However, because of the higher intergranular pressure of the soil at greater depths, the potential for liquefaction is generally limited to the upper 40 feet of the soil. Potential hazards associated with soil liquefaction below or near a structure include loss of foundation support, lateral spreading, sand boils, and areal and differential settlement.

Lateral spreading is lateral ground movement, with some vertical component, as a result of liquefaction. The soil literally rides on top of the liquefied layer. Lateral spreading can occur on relatively flat sites with slopes less than two percent under certain circumstances, generally when the liquefied layer is in relatively close proximity to an open, free slope face such as the bank of a creek channel. Lateral spreading can cause surficial ground tension cracking (i.e., lurch cracking) and settlement.

The Seismic Hazard Zone map prepared by CGS, indicates the project site is not within a zone of required investigation for liquefaction, as shown in *Figure 6, Seismic Hazard Map*. However, based on the predominantly fine-grained soil under the site, we judge the potential for liquefaction settlement and resulting impact to the proposed development to be low.

5.1.3 Dynamic Densification (Settlement)

Dynamic compaction is a phenomenon where loose, relatively clean, near-surface sandy soil located above the water table is densified from vibratory loading, typically from strong seismic shaking or vibratory equipment. The site soil generally consists of hard sandy clay and silty clay. Therefore, in our opinion, dynamic settlement and/or any potential effect of dynamic settlement on the proposed construction is not expected to be significant.

5.1.4 Fault Ground Rupture and Fault Creep

The State of California adopted the Alquist-Priolo Earthquake Fault Zone Act of 1972 (Chapter 7.5, Division 2, Sections 2621 – 2630, California Public Resources Code), which regulates development near active faults for the purpose of preventing surface fault rupture hazards to structures for human occupancy. In accordance with the Alquist-Priolo (A-P) Act, the California Geological Survey established boundary zones or *Earthquake Fault Zones* surrounding faults or fault segments judged to be sufficiently active, well-defined and mapped for some distance. Structures for human occupancy within designated Earthquake Fault Zone boundaries are not permitted unless surface fault rupture and fault creep hazards are adequately addressed in a site-specific evaluation of the development site.

The site is not currently within a designated Earthquake Fault Zone as defined by the State (Hart and Bryant, 1997) or any local zone, Figure 6-Seismic Hazard Map. Based on our evaluation, the potential for fault ground rupture or creep at the site is very low to nil.

5.2 Consolidated Settlement

Consolidation occurs as a result of water being squeezed out from a saturated soil as internal pore water pressures induced by an external load are dissipated over time. As the water moves out from the soil, the solid particles re-align into a more-dense configuration with settlement resulting. Consolidation typically occurs as a result of new buildings or fills being placed over them, but consolidation can also occur from groundwater withdrawal. Consolidation of clayey soils is usually a long-term process, whereby the water is squeezed out of the soil matrix with time. Sandy soils consolidate relatively rapidly with an introduction of a load.

5.3 Expansive Soils

Moderately to high expansive fine-grained soils were encountered in the upper five feet during our subsurface exploration. The results of the laboratory testing performed on representative samples of the most expansive near-surface soils indicated a moderate to high plasticity and moderate to high expansion potential. Hence, mitigation for moderately to high expansive soil conditions consisting of combinations of moisture conditioning of the subgrade and use of a non-expansive fill layer below interior floor slabs is recommended for this site. Specific measures to mitigate the potential effects of moderate to high expansive soils on foundations and concrete slabs-on-grade are presented herein.

6.0 CONCLUSIONS AND ENGINEERING RECOMMENDATIONS

The following conclusions and engineering recommendations are based upon the analysis of the information gathered during the course of this study and our understanding of the proposed improvements.

The site is considered suitable from a geotechnical and geologic perspective for the proposed improvements provided the recommendations of this report are incorporated into the design and implemented during construction. The predominant geotechnical and geological issues affecting design or construction that will need to be addressed at this site are summarized below and addressed in the following sections.

Seismic Considerations - The site is located within a seismically active region and the structures should be designed to account for earthquake ground motions, using the applicable building codes, as described in Section 6.1 of this report.

Expansive Soils – Moderately to high expansive clay surficial soils were identified within the project site. As a result, footings should be extended to greater depth than normal, and interior slabs-on-grade should be steel reinforced to resist expansion pressures as well as be supported on a nominal layer of select, non-expansive fill. Moisture conditioning of the fill and upper processed cut surfaces should also be performed and import fill should be non-expansive.

Undocumented Fill Soils – No surficial undocumented fill soils and debris were encountered in our borings during our subsurface investigation. However, due to the presence of existing buildings at the site of the proposed new building, undocumented fills associated with the demolition of the building and removal of associated foundations and utilities may be present. Undocumented onsite fill soils if encountered in the new building pad and loose or debris laden soils if encountered in other areas, should be completely removed and replaced by engineered compacted fill. The portion of over-excavated material not consisting of debris or organic topsoil may be reused as fill material upon approval of the geotechnical engineer.

Winter Construction - If grading occurs in the winter rainy season, appropriate erosion control measures may be required, and weatherproofing of the building pad and/or hardscape areas may need to be considered. Winter rains may also impact foundation excavations and underground utilities.

6.1 Seismic Coefficients

The subject site is located within a seismically active region and should be designed to account for earthquake ground motions as described in this report. Based on the subsurface conditions encountered and our evaluation of the geology of the site, Site Class "D", representative of stiff soil averaged over the uppermost 100 feet of the subsurface profile would be appropriate for this site.

For seismic analysis of the proposed site in accordance with the seismic provisions of the 2022 California Building Code (CBC), we recommend the following seismic ground motion values be used for design shown in Table 4, which are based on procedures outlined in ASCE 7-16 Section 11.4 and Table 11.4-2 of Supplement 1. ASCE 7-16 Section 11.4.8 states that a site-specific ground motion hazard analysis should be performed for all structures on Site Class D soils with S_1 greater than or equal to 0.2, unless the exceptions outlined in Section 11.4.8 are followed and the seismic response coefficient is properly modified during design. A site-specific ground motion hazard analysis was not performed for this site and is outside the scope of this report. If a site-specific ground motion hazard analysis is required for this project or if the project is designed under a different building code than CBC 2022, we should be notified so that we may provide the appropriate seismic design parameters.

Table 4: Seismic Parameters Based on 2022 CBC (per ASCE 7-16)

Item	Value	2019 CBC Source[R1]	ASCE 7-16 Table/Figure[R2]
Site Class	D	Table 1613A.3.2.	Table 20.3-1
Mapped Spectral Response Accelerations			
Short Period, S_s	1.44		Figure 22-1
1-second Period, S_1	0.513		Figure 22-2
Site Coefficient, F_a	1.2	Table 1613A.3.3(1)	Table 11.4-1
Site Coefficient, F_{v*}	1.7	Table 1613A.3.3(2)	Table 11.4-2
MCE (S_{MS})	1.728	Equation 16A-37	Equation 11.4-1
MCE (S_{M1})	0.872	Equation 16A-38	Equation 11.4-2
Design Spectral Response Acceleration			
Short Period, S_{DS}	1.152	Equation 16A-39	Equation 11.4-3
1-second Period, S_{D1**}	0.581	Equation 16A-40	Equation 11.4-4

R1: California Building Standards Commission (CBSC), "California Building Code," 2019 Edition.
R2: U.S. Seismic "Design Maps" Web Application, https://seismicmaps.org/
*F_v are based off Table 11.4-2 from the ASCE 7-16 Supplement 1
**The above design spectral response acceleration parameters may only be used provided that the exception outlined in section 11.4.8 of ASCE 7-16 is met.

6.2 Site Grading

6.2.1 General Grading and Material Requirements

Site grading is generally anticipated to consist of finish grading to establish site grades, or additional mass grading for improved foundation bearing capacities if desired; utility trench excavation and backfills, preparation of supporting subgrades for site pavements and hardscape; and placement of aggregate base (baserock) sections for hardscape and pavements.

On-site soils having an organic content of less than three percent by weight and Plasticity Index of less than 15 can be reused as fill as approved by the Geotechnical Engineer. Imported soil should be non-expansive, having a Plasticity Index of 15 or less, an R-Value greater than 40, and contain sufficient fines so the soil can bind together. Imported materials should be free of environmental contaminants, organic materials and debris, and should not contain rocks or lumps greater than three inches in maximum size. Imported fill materials should be approved by the Geotechnical Engineer prior to use on site.

6.2.2 Project Compaction Recommendations

Table 5 provides the recommended compaction requirements for this project. Some items listed below may not apply to this project. Specific moisture conditioning and relative compaction recommendations will be discussed individually within applicable sections of this report.

Table 5: Project Compaction Recommendations

Description	Percent Relative Compaction	Minimum Percent Above Optimum Moisture Content
Building Pad, Onsite Soil	90	3 to 5
Building Pad, Subgrade Soil	90	3 to 5
Building Pad, Imported Select Fill	90	2
Building Pad, Treated Soil	90	2
AC or Concrete Pavement, Subgrade, Upper 6”	95	3 to 5
AC or Concrete Pavement, Onsite Soil or Fill	90	3 to 5
AC or Concrete Pavement, Class 2 Baserock	95	2
AC or Concrete Pavement, Treated Soil, Subgrade	93	2
Concrete Flatwork, Class 2 Baserock	90	2
Concrete Flatwork, Subgrade Soil	90	3 to 5
Underground Utility Trench Backfill	90	2
Underground Utility Trench Backfill - Landscape Areas (not including areas below flatwork)	85	2
Underground Utility Trench Backfill, Clean Sand	95	4
Underground Utility Trench Backfill, Upper 3’ Feet below Existing Pavement Sections or 6” below New Pavement Sections	95	2

Fill materials should be properly moisture conditioned in accordance with Table 5 as determined using ASTM D-1557 and placed in uniform loose lifts not to exceed eight inches. Smaller lifts may be necessary to achieve the minimum required compaction using lighter weight compaction equipment. It should be noted that the use of on-site soil for fill will require moisture conditioning (drying or wetting). Moisture conditioning may be difficult to achieve during cold, wet periods of the year, or during extreme temperatures and after precipitation events.

6.2.3 Site Preparation and Demolition

Site grading should be performed in accordance with these recommendations. A pre-construction conference should be held at the jobsite with representatives from the owner, general contractor, grading contractor, and Geo-Eng prior to starting the stripping and demolition operations at the site.

The site should be cleared of existing pavements (if any), vegetation, organic topsoil, debris, existing undocumented loose or soft fill, and other deleterious materials within the proposed development area. Removed fill soil may be evaluated by the Geotechnical Engineer for possible reuse and placement as engineered fill. The grading contractor should be aware of the possibility of buried objects and underground utilities at the site which are to be removed or abandoned appropriately. Holes resulting from the removal of underground obstructions extending below the proposed finish grade should be cleared and backfilled with properly compacted engineered fill or other material approved by the Geotechnical Engineer. We recommend backfilling operations for any excavations to remove deleterious material be carried out under the observation of the Geotechnical Engineer.

It is possible that existing underground utilities exist and if so, may impact the project construction. If encountered, the utilities will need to be properly abandoned and/or entirely removed from the proposed building area. In general, utility pipelines less than four inches in diameter to be abandoned may be left in place provided they will not be in close proximity to new foundation elements or interfere with new utilities. Such pipes should be plugged at the ends with concrete or sand-cement slurry. Larger utility pipelines or pipelines that underlie new foundations should be removed and replaced with engineered fill or left in place and completely grouted with flowable sand-cement slurry or other approved Controlled Density Fill (CDF; also, known as Controlled Low Strength Material, or CLSM).

6.2.4 Building Subgrade Preparation

Imported soil should be non-expansive, having a Plasticity Index of 15 or less, an R-Value greater than 40, and contain sufficient fines so the soil can bind together. Imported materials should be free of organic materials and debris and should not contain rocks or lumps greater than three inches in maximum size. Imported fill materials should be approved by the Geotechnical Engineer prior to use onsite.

Following excavation to the required grades, subgrades in areas to receive engineered fill, slabs-on-grade or hardscape should be scarified to a depth of at least six inches; moisture conditioned and compacted to the requirements for engineered fill presented in Table 5. The compacted surface should be firm and unyielding and should be protected from damage caused by traffic or weather. Soil subgrades should be kept moist during construction. To achieve satisfactory compaction of the subgrade and fill materials, it may be necessary to adjust the water content at the time of construction. This may require that water be added to soils that are too dry, or that scarification and aeration be performed in any soils that are too wet. Fill material should be evenly spread and compacted in lifts not exceeding eight inches in pre-compacted thickness.

Newly exposed near-surface soils under existing site pavement once removed are typically saturated to near-saturated. Therefore, it is anticipated that after the underlying soils are over-excavated to construct the non-expansive fill layer, unstable subgrade conditions unworkable for compaction by construction equipment are locally possible, and compaction of the exposed soil subgrade to engineered fill requirements immediately after exposure may not be feasible. Possible options for subgrade stabilization include ripping, air-drying and re-compacting exposed subgrade material; admixtures such as cement; or use of reinforcing stabilization geotextile or geogrid, as discussed below. More detailed recommendations can be provided during construction should unstable subgrades be encountered by the contractor.

Unstable subgrades in smaller, isolated areas can be stabilized by over excavating to a minimum of 18-inch depth below finished subgrade elevation where competent, stable soils are not encountered. The bottom of the excavation should then be completely covered with a ground stabilization geotextile fabric such as Mirafi 500X or equivalent, and typically backfilled with Class 2 aggregate base. Alternatively, with the approval of the Geotechnical Engineer, such areas can be stabilized by over-excavating at least one foot, placing Tensar TriAx TX-140 or equivalent geogrid on the soil, and then placing 12 inches of Class 2 baserock on the geogrid. The upper six inches of the baserock in either case should be compacted to at least 90 percent relative compaction.

Larger unstable areas if encountered may be remedied using soil admixtures, such as cement. A four percent mixture of cement based on a dry soil unit weight of 110 pcf may be assumed if needed. Treatment should vary between 12 to 18 inches, depending on the anticipated construction equipment loads. More detailed and final recommendations can be provided during construction.

Final grading should be designed to provide positive drainage away from the building. We suggest exposed soil/landscape areas, if any, within 10 feet of the proposed building be sloped at a minimum of three percent away from the building. Roof leaders and downspouts should discharge onto paved surfaces sloping away from the building or into a closed pipe system channeled away from the building to an approved collector or outfall.

6.2.5 Flatwork Areas

The existing soil in flatwork areas should be scarified to a depth of at least six inches, moisture conditioned and compacted. Once the compacted subgrade has been reached, it is recommended that baserock in paved areas be placed immediately after grading to protect the subgrade soil from drying. Alternatively, the subgrade should be kept moist by watering until the baserock is placed. Rubber-tired heavy equipment, such as a full water truck, should be used to proof roll exposed pavement subgrade areas where pumping is suspected. Proof rolling will determine if the subgrade soil is capable of supporting construction paving equipment without excessive pumping or rutting.

6.2.6 Site Winterization and Unstable Subgrade Conditions

If grading occurs in the winter rainy season, unstable and unworkable subgrade conditions may be present, and compaction of on-site soils may not be feasible. These conditions may be remedied using appropriate soil admixtures, such as lime or other admixtures. More detailed recommendations can be provided during construction. Stabilizing subgrade in small, isolated areas can be accomplished with the approval of the Geotechnical Engineer by over-excavating one foot, placing Tensar BX1100 or TriAx TX-140 geogrid or equivalent geogrid on the soil, and then placing 12 inches of Class 2 baserock on the geogrid. The upper six inches of the baserock should be compacted to at least 90 percent relative compaction. Alternatively, a non-woven stabilization geotextile such as Mirafi 500X overlain by a minimum 18 inches of baserock may be substituted for geogrid and baserock.

6.3 Utility Trench Construction

6.3.1 Trench Backfilling

Utility trenches may be backfilled with onsite soil or import soil pre-approved by the Geotechnical Engineer above the utility bedding and shading materials. If cobbles, rocks or concrete larger than four inches in maximum size are encountered, they should be removed from the fill material prior to placement in the utility trenches.

Pipeline trenches should be backfilled with fill placed in lifts of approximately eight inches in pre-compacted thickness and compacted to the requirements presented in Section 6.2.2. However, thicker lifts can be used, provided the method of compaction is approved by the Geotechnical Engineer, and the required minimum degree of compaction is achieved.

6.3.2 Utility Penetrations at Building Perimeter

Flexible connections at building perimeters should be considered for utility lines going through perimeter foundations. This would provide flexibility during a seismic event. This could be provided by special flexible connections, pipe sleeving with appropriate waterproofing, or other methods.

6.4 Temporary Excavation Slopes

Below-grade construction, if any is ultimately proposed for the project, may require temporary excavation slopes if more than a few feet below existing grade. The Contractor should incorporate all appropriate requirements of OSHA/Cal OSHA into the design of the temporary construction slopes and shoring system, whichever is used. Excavation safety regulations are provided in the OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, Subpart P, and apply to excavations greater than five feet in depth.

The Contractor, or his specialty subcontractor, should design temporary construction slopes to conform to the OSHA regulations and should determine actual temporary slope inclinations based on the subsurface conditions exposed at the time of construction. For pre-construction planning purposes, the on-site near-surface materials may be assumed to be granular or weak cohesive materials and categorized as OSHA Type C with temporary slope inclination of no steeper than 1.5:1 (horizontal: vertical) for excavations less than 20 feet deep.

If temporary slopes are left open for extended periods of time, exposure to weather and rain could have detrimental effects such as sloughing and erosion on surficial soils exposed in the excavations. We recommend that all vehicles and other surcharge loads be kept at least 10 feet away from the top of temporary slopes, and that such temporary slopes are protected from excessive drying or saturation during construction. In addition, adequate provisions should be made to prevent water from ponding on top of the slope and from flowing over the slope face. Desiccation or excessive moisture in the excavation could reduce stability and require shoring or laying back side slopes.

6.5 Foundations

6.5.1 General

The elevations for the planned buildings were not available during this phase of design and they will have an impact on the optimal foundation design. Once planning is further along, we should be consulted regarding the final selection of the foundation type. For the design of the new structures at the site, we anticipate that the buildings can be supported on continuous and /or isolated spread footings, or drilled pier, bearing on undisturbed stiff to very stiff, onsite native soil.

6.5.2 Shallow Foundations

The proposed buildings can be supported on continuous and/or isolated spread footings bearing on undisturbed stiff to very stiff native soil or engineered fill. Where over excavations below design footing depth is required, the over excavated portion of footing excavation should be backfilled with structural or lean concrete or a Controlled Low Strength Material (CLSM). Footings should be founded a minimum of 24 inches below lowest adjacent finished grade. Continuous footings should have a minimum width of at least 18 inches, and isolated column footings should have a minimum width of at least 24 inches. In addition, footings located adjacent to other footings or utility trenches should bear below an imaginary 1.5:1 (horizontal to vertical) plane projected upward from the bottom edge of the adjacent footings or utility trenches. Footing reinforcement should be determined by the project Structural Engineer.

Footing should be designed for the following allowable bearing pressures, assuming design Factors-of-Safety of 3.0, 2.0 and 1.5 for dead loads, dead plus live loads and total loads, respectively, from the calculated ultimate bearing pressure.

Table 6: Allowable Bearing Pressures for Spread Footings

Load Condition	Allowable Bearing Pressure (psf)
Dead Load	2,000
Dead plus Live Loads	3,000
Total Loads (including wind or seismic)	5,000

These allowable bearing pressures are net values; therefore, the weight of the footing can be neglected for design purposes. Footings should be designed with sufficient reinforcing to provide structural continuity and permit spanning of local irregularities. These pressures assume a uniform embedment into stiff native soil or engineered fill. Footings may need to be over-excavated during construction to achieve this requirement and all footings shall be observed by a Geo-Eng Engineer to confirm this.

If site preparation and foundation observation services are conducted as outlined in the Geotechnical Study report, vertical static settlement is expected to be less than one inch for footings bearing within the materials described in the report and designed to the allowable bearing pressures. Differential settlement across the structure is not expected to exceed about ½ this value within a 30-foot span.

6.5.3 Lateral Resistance

Shallow foundations can resist lateral loads with a combination of bottom friction and passive resistance. An allowable coefficient of friction of 0.35 between the base of the foundation elements and underlying material is recommended. In addition, an *ultimate* passive resistance equal to an equivalent fluid weighing 350 pounds per cubic foot (pcf) acting against the foundation may be used for lateral load resistance against the sides of footings perpendicular to the direction of loading where the footing is poured neat against undisturbed material. The top foot of passive resistance at foundations not adjacent to pavement or hardscape should be neglected. To fully mobilize this passive resistance, a lateral footing deflection on the order of one to two percent of the embedment of the footing is required. If it is desired to limit the amount of lateral deflection to mobilize the passive resistance, a proportional safety factor should be applied. The friction between the bottom of a slab-on-grade floor and the underlying soil should not be utilized to resist lateral forces.

6.5.4 Drilled Pier Foundations

The foundation for the structures may consist of drilled pier foundations, deriving their vertical supporting capacity through skin friction between the side surfaces of the foundations and the adjacent soil. We recommend that the drilled 24" diameter piers be at a minimum of 10-feet deep. Type II Cement with 3,000 PSI concrete is applicable for the pier foundations. For design purposes, the allowable skin friction for gravity loads may be assumed to be 425 psf for the portion of pier embedded in native soils, with the upper two foot of the pier to be neglected. If a steel casing is installed and left in place, a 20% reduction in allowable skin friction can be used. These values should not be increased for seismic loads, but they can be increased by 1/3 for transient wind loads. A 1/3 increase in lateral bearing pressure is applicable when alternate ASD load combinations are used for pier design using wind/seismic loading. Uplift loads should be limited to 0.8 times these values. For piers situated adjacent to or on slopes, the portion of pier with horizontal cover less than 10 feet, measured from outside perimeter of the pier to the slope surface should be neglected in computing vertical capacity. These values assume that there is a minimum spacing between piers of 3 pier diameters measured center to center.

Lateral resistance for drilled pier foundations may be determined for onsite soils using an allowable passive resistance equal to an equivalent fluid weighing 400 pounds per cubic foot (pcf) acting against the foundation for lateral load resistance against the sides of foundations perpendicular to the direction of loading where the foundation is poured neat against undisturbed material (i.e., native soils, engineered fills or existing fills). For pier foundations, passive pressure can be assumed to act across two times the pier diameter. For piers situated adjacent to or on slopes, the portion of pier with horizontal cover less than 10 feet, measured from outside perimeter of the pier to the slope surface should be neglected in computing lateral capacity. Geo-Eng personnel should be retained to observe and confirm that soil or bedrock encountered during footing excavations, prior to formwork and reinforcing steel placement, is consistent with the assumptions of this report. If unsuitable soil or bedrock is present, the excavation should be deepened until suitable supporting material is encountered. The over excavation should be backfilled using engineered soil or lean concrete (or a sand-cement slurry mix acceptable to the Geotechnical Engineer) up to the bottom of the footing concrete.

Although the near surface soils were generally cohesive, isolated sand layers can be encountered requiring casing or use of drilling mud if encountered. Any pier constructed below groundwater should either be pumped dry or use the tremie method of concrete placement as part of the pier construction. If groundwater is encountered during drilling, piers should be poured within a few days with the supervision of the geotechnical engineer of record.

6.5.5 Construction Considerations

Geo-Eng personnel should be retained to observe and confirm that footing excavations prior to formwork and reinforcing steel placement bear in soils suitable for the recommended maximum design bearing pressure. If unsuitable soil or bedrock is present, the excavation should be deepened until suitable supporting material is encountered. The over excavation should be backfilled using engineered soil or lean concrete (or a sand-cement slurry mix acceptable to the Geotechnical Engineer) up to the bottom of the footing concrete.

Footing excavations should have firm bottoms and be free from excessive slough prior to concrete or reinforcing steel placement. Care should also be taken to prevent excessive wetting or drying of the bearing materials during construction. Extremely wet or dry or any loose or disturbed material in the bottom of the footing excavations should be removed prior to placing concrete. If construction occurs during the winter months, a thin layer of concrete (sometimes referred to as a rat slab) could be placed at the bottom of the footing excavations. This will protect the bearing material and facilitate removal of water and slough if rainwater fills the excavations.

6.6 Concrete Slabs-on-Grade

6.6.1 General Recommendations

Non-structural concrete interior slab-on-grade floors should be a minimum of five inches in thickness. As a minimum, slab reinforcing should consist of No. 4 steel reinforcement spaced at 18-inch centers each way, and in any case, be sufficient to satisfy the anticipated use and loading of the slab. Slab-on-grade subgrade surfaces should be proof-rolled to provide a smooth, unyielding surface for slab support. Due to the presence of moderately expansive soils, we recommend that the upper 6-inches of the building pad consist of a non-expansive fill layer. This fill should be compacted in accordance with the recommendations presented in the grading section of this report, Section 6.2. The non-expansive layer should extend a minimum of 5-feet outside of the building envelope.

Care should be taken to maintain the minimum recommended moisture content in the subgrade until floor slabs and/or engineered fills are constructed. Positive drainage should also be developed away from the building to prevent water from ponding along the perimeter and affecting future floor slab performance. We recommend a positive cutoff in utility trenches at the structure/building lines to reduce the potential for water migrating through the utility trench backfill to areas under the building.

Slab-on-grade concrete floors with moisture sensitive floor coverings should be underlain by a moisture retarder system constructed between the slab and subgrade. Such a system could consist of four inches of free-draining gravel, such as 3/4-inch, clean, crushed, uniformly graded gravel with less than three percent passing No. 200 sieve, or equivalent, overlain by a relatively impermeable vapor retarder placed between the subgrade soil and the slab. The vapor retarder should be at least 10-mil thick and should conform to the requirements for ASTM E 1745 Class A, B, or C Underslab Vapor Retarders (e.g., Griffolyn Type 65, Griffolyn Vapor Guard, Moistop Ultra C, or equivalent). If additional protection is desired by the owner, a higher quality vapor barrier conforming to the requirements of ASTM E 1745 Class A, with a water vapor transmission rate less than or equal to 0.006 gr/ft^2/hr (i.e., 0.012 perms) per ASTM E 96 (e.g., 15-mil thick "Stego Wrap Class A") may be used in place of the retarder.

The vapor retarder or barrier should be placed directly under the slab. A capillary rock layer or rock cushion is not required if Class A barriers have been used beneath the floor slab and a sand layer is not required over the vapor retarder from a geotechnical standpoint. If sand on top of the vapor retarder is required by the design structural engineer, we suggest the thickness be minimized to less than one inch. If construction occurs in the winter months, water may pond within the sand layer since the vapor retarder may prevent the vertical percolation of rainwater.

ASTM E1643 should be utilized as a guideline for the installation of the vapor retarder. During construction, all penetrations (e.g., pipes and conduits,) overlap seams, and punctures should be completely sealed using a waterproof tape or mastic applied in accordance with the vapor retarder manufacturer's specifications. The vapor retarder or barrier should extend to the perimeter cutoff beam or footing.

6.6.2 Exterior Concrete Flatwork

Exterior concrete flatwork with pedestrian traffic should be at least four inches thick and should be underlain by at least six inches of aggregate baserock. The subgrade beneath the flatwork should be moisture conditioned and compacted as specified in the grading section of this report.

Control joints should be constructed in accordance with ACI 224 "Control of Cracking in Concrete Structures". In general, for typical flatwork, joints would be required every 24 to 36 times the concrete thickness.

6.7 Retaining/Basement Walls

6.7.1 Lateral Earth Pressures

The following recommended lateral earth design pressures are based on the assumption that on-site soils will be used as wall backfill. For a level backfill condition, unrestrained walls (i.e., walls that are free to deflect or rotate) should be designed to resist an equivalent fluid pressure of 40 pounds per cubic foot. Restrained walls for a level backfill condition should be designed to resist an equivalent fluid pressure of 40 pounds per cubic foot, plus an additional uniform lateral pressure of 5H pounds per square foot, where H = height of backfill above the top of the wall footing, in feet. For seismic design of walls greater than six feet in retained height, unrestrained and restrained walls with level backfill should be designed to resist an additional uniform load equal to 15H psf, added to the *unrestrained* condition in either case. A seismic increment is not required for site walls retaining less than six feet.

Walls with inclined backfill should be designed for an additional equivalent fluid pressure of one pound per cubic foot for every two degrees of slope inclination from horizontal. Walls subjected to surcharge loads should be designed for an additional uniform lateral pressure equal to 0.33 times the anticipated surcharge load for unrestrained walls, and 0.50 times the anticipated surcharge load for restrained walls.

For resistance to lateral loads, an allowable coefficient of friction of 0.35 between the base of the foundation elements and underlying material is recommended. In addition, an *ultimate* passive resistance equal to an equivalent fluid weighing 350 pounds per cubic foot (pcf) acting against the foundation may be used for lateral load resistance against the sides of the footing perpendicular to the direction of loading where the footing is poured neat against undisturbed material (i.e., native soils or engineered fills). The top foot of passive resistance at foundations not adjacent to and confined by pavement, interior floor slab, or hardscape should be neglected. In order to fully mobilize this passive resistance, a lateral footing deflection on the order of one to two percent of the embedment of the footing is required. If it is desired to limit the amount of lateral deflection to mobilize the passive resistance, a proportional safety factor should be applied.

The lateral earth pressures herein do not include any factor-of-safety and are not applicable for submerged soils/hydrostatic loading. Additional recommendations may be necessary if submerged conditions are to be included in the design.

6.7.2 Retaining Wall Foundations

Site retaining wall may be founded on spread footing foundations bearing on undisturbed, onsite native clay soil. Where over excavations below design footing depth is required, the over excavated portion of footing excavation should be backfilled with structural or lean concrete or a Controlled Low Strength Material (CLSM). Footings should be founded a minimum of 24 inches below lowest adjacent finished grade. Continuous footings should have a minimum width of at least 18 inches. In addition, footings located adjacent to other footings or utility trenches should bear below an imaginary 1.5:1 (horizontal to vertical) plane projected upward from the bottom edge of the adjacent footings or utility trenches. Footing reinforcement should be determined by the project Structural Engineer.

Footing should be designed for the following allowable bearing pressures, assuming design Factors-of-Safety of 3.0, 2.0 and 1.5 for dead loads, dead plus live loads and total loads, respectively, from the calculated ultimate bearing pressure.

Table 7: Allowable Bearing Pressures for Spread Footings

Load Condition	Allowable Bearing Pressure (psf)
Dead Load	1,500
Dead plus Live Loads	2,250
Total Loads (including wind or seismic)	3,000

These allowable bearing pressures are net values; therefore, the weight of the footing can be neglected for design purposes. Footings should be designed with sufficient reinforcing to provide structural continuity and permit spanning of local irregularities. These pressures assume a uniform embedment into very stiff native soil or engineered fill. Footings may need to be over-excavated during construction to achieve this requirement and all footings shall be observed by a Geo-Eng Engineer to confirm this.

6.7.3 Retaining Wall Drainage

The aforementioned recommended lateral pressures assume that walls are fully back drained to prevent the build-up of hydrostatic pressures. To reduce the potential for hydrostatic loading on retaining and below-grade walls due to possible seasonal subsurface groundwater seepage, a subsurface drain system may be considered for construction behind below-grade walls. Alternatively, below-grade walls can be designed to accommodate an additional hydrostatic pressure increment.

The drain system should consist of free-draining granular soils containing less than five percent fines passing a No. 200 sieve, placed adjacent to the wall. The free-draining granular material should be graded to prevent the intrusion of fines, or else should be encapsulated in a suitable filter fabric. A drainage system consisting of perforated drain lines (minimum 4” diameter placed near the base of the wall) should be used to intercept and discharge water which would tend to saturate the backfill. Sub drains constructed to protect interior spaces should have the invert elevation of the sub drain a minimum of six inches below the interior finished floor elevation. Where used, drain lines should be embedded in a uniformly graded filter material and provided with adequate clean-outs for periodic maintenance. An impervious soil should be used in the upper one-foot layer of backfill to reduce the potential for water infiltration. As an alternative, a prefabricated drainage structure, such as geo-composite, may be used as a substitute for the granular backfill adjacent to the wall.

The retaining wall drainage system should be sloped to outfall to the storm drain system or other appropriate facility. The foundation of the retaining wall should be protected and prevented from any erosion of the surroundings.

6.7.4 Retaining Wall Backfill Compaction

Retaining wall backfill less than five feet deep should be compacted to at least 90 percent relative compaction using light compaction equipment. Backfill greater than a depth of five feet should be compacted to at least 95 percent relative compaction. If heavy compaction equipment is used, the walls should be appropriately designed to withstand loads exerted by the heavy equipment, and/or temporarily braced. Over compaction or surcharge from heavy equipment too close to the wall may cause excessive lateral earth pressures which could result in excessive outward wall movement.

6.8 Observation and Testing During Construction

We recommend that Geo-Eng be retained to provide observation and testing services during site preparation, site grading, pavement section preparation, utility construction, foundation excavation, and to observe final site drainage. This is to observe compliance with the design concepts, specifications and recommendations, and to allow for possible changes if subsurface conditions differ from those anticipated prior to the start of construction.

7.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

The recommendations of this report are based upon the soil and conditions encountered in the field explorations (i.e., borings). If variations or undesirable conditions are encountered during construction, Geo-Eng should be contacted so that supplemental recommendations may be provided.

This report is issued with the understanding that it is the responsibility of the owner or his representatives to see that the information and recommendations contained herein are called to the attention of the other members of the design team and incorporated into the plans and specifications, and that the necessary steps are taken to see that the recommendations are implemented during construction.

The findings and recommendations presented in this report are valid as of the present time for the development as currently proposed. However, changes in the conditions of the property or adjacent properties may occur with the passage of time, whether by natural processes or the acts of other people. In addition, changes in applicable or appropriate standards may occur through legislation or the broadening of knowledge. Accordingly, the findings and recommendations presented in this report may be invalidated, wholly or in part, by changes outside our control. Therefore, this report is subject to review by Geo-Eng after a period of three (3) years has elapsed from the date of issuance of this report. In addition, if the currently proposed design scheme as noted in this report is altered, Geo-Eng should be provided the opportunity to review the changed design and provide supplemental recommendations as needed.

Recommendations are presented in this report which specifically request that Geo-Eng be provided the opportunity to review the project plans prior to construction and that we be retained to provide observation and testing services during construction. The validity of the recommendations of this report assumes that Geo-Eng will be retained to provide these services.

This report was prepared at your request for our services, and in accordance with currently accepted geotechnical engineering practice. No warranty based on the contents of this report is intended, and none shall be inferred from the statements or opinions expressed herein. The scope of our services for this report did not include an environmental assessment or investigation for the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater or air, on, below or around this site. Any statements within this report or in the attached figures, logs or records regarding odors noted or other items or conditions observed are for the information of our client only.

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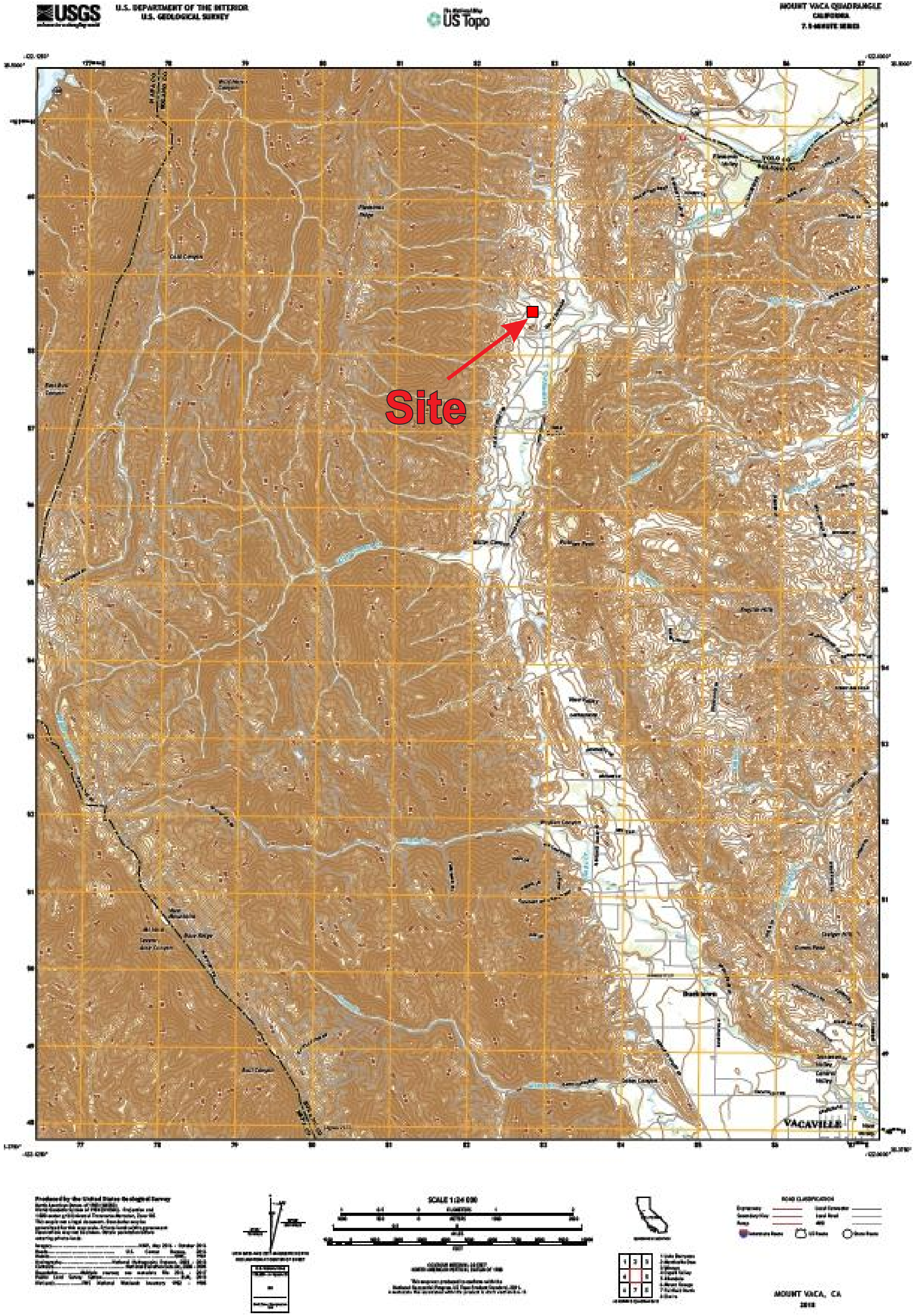
Publications may have been used as general reference and not specifically cited in the report text.

FIGURES

Figure 1 – Site Vicinity Map
Figure 2 – Site Development Plan
Figure 3 – Site Map and Boring Locations
Figure 4 – Site Vicinity Geologic Map
Figure 5 – Regional Fault Map
Figure 6 – Seismic Hazard Map



Geo-Engineering Solutions, Inc.
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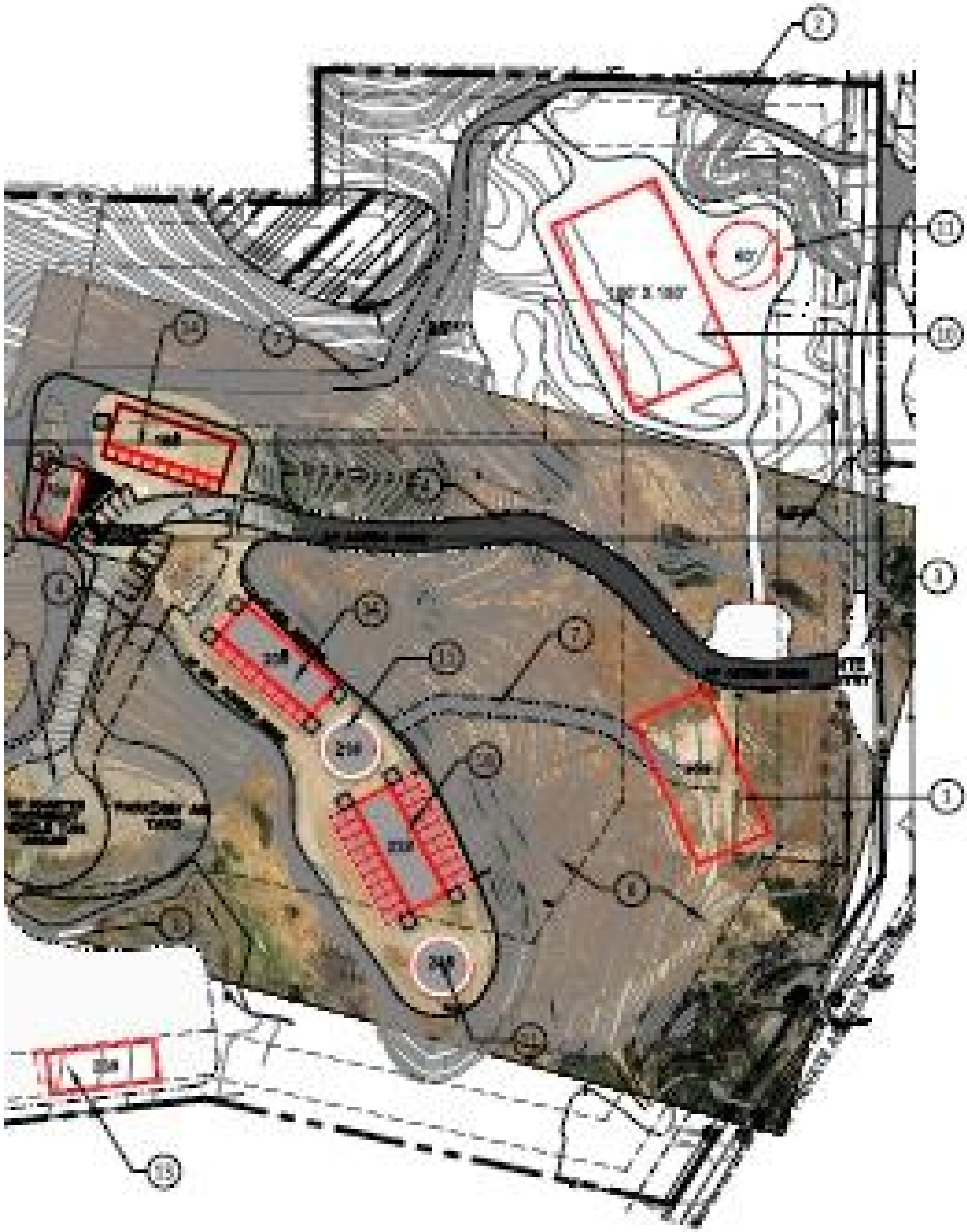
Double T Ranch
Barn, Office Shop Complex
8325 Quail Canyon Rd
Vacaville, CA

109-1519C

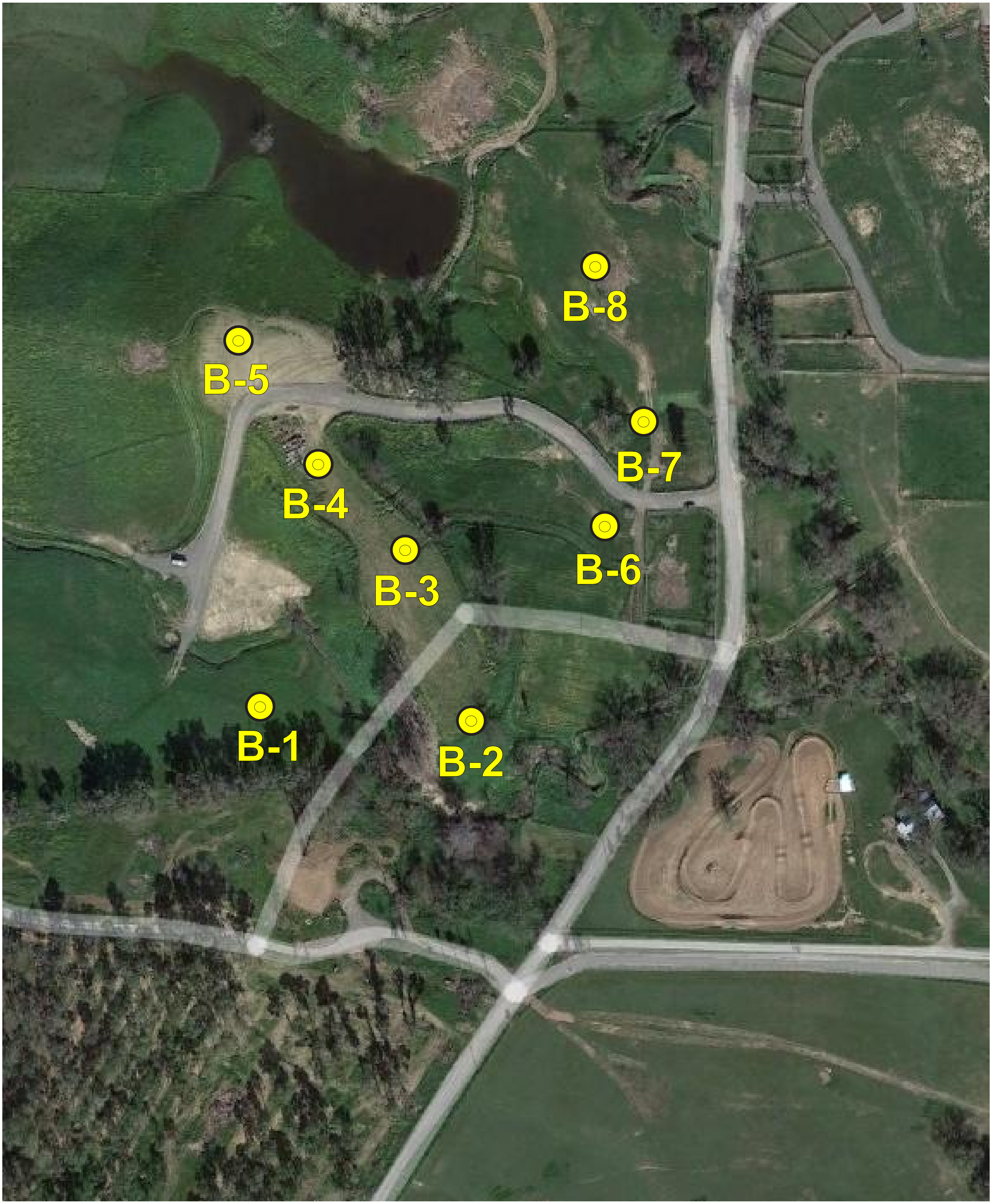
August 2023

Site Vicinity Map

Figure 1



Geo-Engineering Solutions, Inc. Geotechnical Engineering • Engineering Geology • Materials Testing	Double T Ranch Barn, Office Shop Complex 8325 Quail Canyon Rd Vacaville, CA	109-1519C	August 2023
		Site Development Plan	Figure 2



Approximate Boring Location

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Double T Ranch Barn, Office Shop Complex 8325 Quail Canyon Rd Vacaville, CA	109-1519C	August 2023
	Site Map and Boring Locations	Figure 3

Ted		DOMENGINE SANDSTONE -- light-brownish-gray to yellowish-brown, coarse-grained, quartz sandstone, commonly crossbedded; locally contains interbedded clay and silty shales; pebbly layers near base.
Tec		CAPAY FORMATION -- brown and gray shale and sandy mudstone; glauconitic horizon at top of unit in Vacaville area.
Tpu		UNNAMED FORMATION -- poorly exposed brown to gray-brown silty and sandy shale and interbedded thin well-sorted white to gray friable sandstone.
Tpus		Upper sandstone member. Massive, locally crossbedded, white to light-gray, medium-grained, quartz sandstone in Pleasants Valley.
Tpm		MARTINEZ FORMATION -- drab brown to greenish-brown quartzose sandstone with thin shale interbeds and thick beds of well-cemented pebble conglomerate. Two members differentiated south of Sacramento River and Carquinez Strait.

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Double T Ranch Barn, Office Shop Complex 8325 Quail Canyon Rd Vacaville, CA	109-1519C	August 2023
	Site Vicinity Geologic Map	Figure 4

Double T Ranch Barn, Office Shop Complex 8325 Quail Canyon Rd Vacaville, CA	109-1519C	August 2023
	Regional Fault Map	Figure 5

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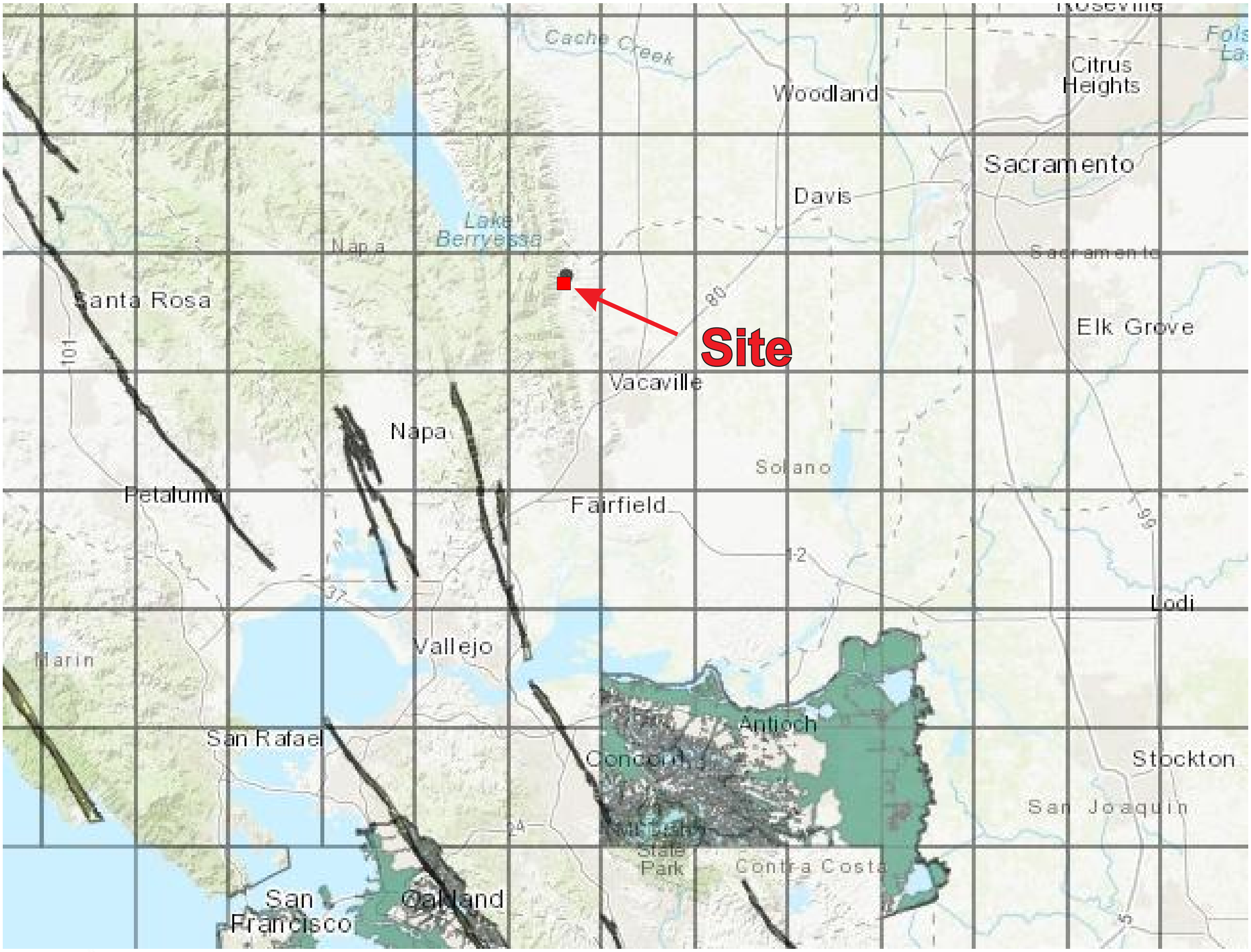

Liquefaction Zones
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

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

Earthquake Fault Zones
Zone boundaries are delineated by straight-line segments; the boundaries define the zone encompassing active faults that constitute a potential hazard to structures from surface faulting or fault creep such that avoidance as described in Public Resources Code Section 2621.5(a) would be required.

Active Fault Traces
1906 C ? ?
Faults considered to have been active during Holocene time and to have potential for surface rupture: Solid Line in Black or Red where Accurately Located; Long Dash in Black or Solid Line in Purple where Approximately Located; Short Dash in Black or Solid Line in Orange where Inferred; Dotted Line in Black or Solid Line in Rose where Concealed; Query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by fault creep.

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		Seismic Hazard Map	Figure 6

APPENDIX A

FIELD EXPLORATION
Key to Exploratory Boring Logs
Boring Logs

Unified Soil Classification (USC) System (from ASTM D 2487)

Major Divisions				Typical Names
Course-Grained Soils More than 50% retained on the 0.075 mm (No. 200) sieve	**Gravels** 50% or more of course fraction retained on the 4.75 mm (No. 4) sieve	Clean Gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		Gravels with Fines	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sands 50% or more of course fraction passes the 4.75 (No. 4) sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines
			SP	Poorly graded sands and gravelly sands, little or no fines
		Sands with Fines	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
Fine-Grained Soils More than 50% passes the 0.075 mm (No. 200) sieve	**Silts and Clays** Liquid Limit 50% or less		ML	Inorganic silts, very fine sands, rock four, silty or clayey fine sands
			CL	Inorganic clays of low to medium plasticity, gravelly/sandy/silty/lean clays
			OL	Organic silts and organic silty clays of low plasticity
	Silts and Clays Liquid Limit greater than 50%		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
			CH	Inorganic clays or high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
Highly Organic Soils			PT	Peat, muck, and other highly organic soils

PENETRATION RESISTANCE (RECORDED AS BLOWS/0.5 FEET)				
SAND AND GRAVEL		SILT AND CLAY		
RELATIVE DENSITY	N-VALUE (BLOWS/FOOT)*	CONSISTENCY	N-VALUE (BLOWS/FOOT)*	COMPRESSIVE STRENGTH
Very Loose	0 - 3	Very Soft	0 - 1	0 - 0.25
Loose	4 - 10	Soft	2 - 4	0.25 - 0.50
Medium Dense	11 - 29	Medium Stiff	5 - 7	0.50 - 1.0
Dense	30 - 49	Stiff	8 - 14	1.0 - 2.0
Very Dense	50 +	Very Stiff	15 - 29	2.0 - 4.0
		Hard	30 +	Over 4.0

Particle Sizes		
Components		*Size or Sieve Number*
Boulders		Over 12 inches
Cobbles		3 to 12 inches
Gravels	Coarse	3/4 to 3 inches
	Fine	Number 4 to 3/4 inch
Sand	Coarse	Number 10 to Number 4
	Medium	Number 40 to Number 10
	Fine	Number 200 to Number 40
Fines (Silt and Clay)		Below Number 200

Soil Moisture	
Descriptor	*Description*
Dry	Dry of Standard Proctor Optimum
Damp	Sand Dry
Moist	Near Standard Proctor Optimum
Wet	Wet of Standard Proctor Optimum
Saturated	Free Water in Sample

Bulk Sample

Standard Penetration Test

2.5 Inch Modified California Sampler

Shelby Tube

First Water Level Reading

Final Water Level Reading

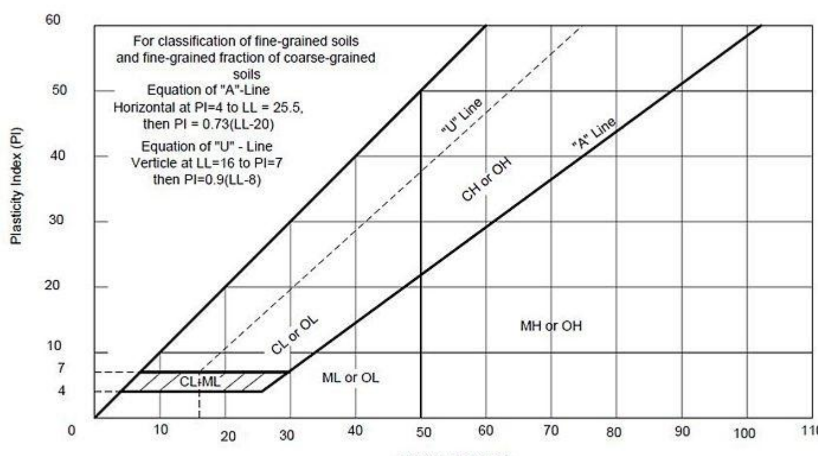
Blow Count
The number of blows of the sampling hammer required to drive the sampler through each of three 6-inch increments. Less than three increments may be reported if more than 50 blows are counted for any increment. The notation 50/5" indicates 50 blows recorded for 5 inches of penetration. Note all of the field blow counts recorded using a Modified California sampler were converted to equivalent SPT blow counts.

N-Value
Number of blows 140 LB hammer falling 30 inches to drive a 2 inch outside diameter (1-3/8 innch I.D.) split barrel sampler the last 12 inches of an 18 inch drive (ASTM-1586 Standard Penetration Test).

General Notes:
1. The boring locations were determined by pacing, sighting and/or measuring from site features. Locations are approximate. Elevations of borings (if included) were determined by interpolation between plan contours or from another source identified in the report. The location and elevation of borings should be considered accurate only to the degree implied by the method.

2.The stratification lines represent the approximate boundary between soil types. The transition may be gradual.

3. Water level readings in the drill holes were recorded at the time and under the conditions stated on the boring logs. It should be noted that flucuations in the level of groundwater may occur due to variations in rainfall, tides and other factors at the time measurements were made



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Key to Exploratory Boring Logs

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BORING NUMBER B-1

CLIENT Jim Nuti
PROJECT NAME Double T Ranch - Main Residence
PROJECT NUMBER 109-1519-C
PROJECT LOCATION 8325 Quail Canyon Rd, Vacaville CA
DATE STARTED 8/14/23 **COMPLETED** 8/14/23
GROUND ELEVATION 257 ft **HOLE SIZE** 2.5"
DRILLING CONTRACTOR California Geotech Services, LLC
GROUND WATER LEVELS:
DRILLING METHOD Solid Flight CME-75
AT TIME OF DRILLING ---
LOGGED BY SS **CHECKED BY**
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	SPT BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		(CL) **Silty CLAY** : Gray-brown to brown, moist, stiff to very stiff, low plasticity										
			MC 1-1		7-7-10 (17)	>4.5	108	13				
5			MC 1-2		5-6-7 (13)	>4.5	102	13	37	19	18	
10			MC 1-3		5-5-6 (11)	>4.5	103	19				
15			SPT 1-4		4-5-6 (11)							

Bottom of borehole at 15.0 feet.

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BORING NUMBER B-2

CLIENT Jim Nuti
PROJECT NAME Double T Ranch - Main Residence
PROJECT NUMBER 109-1519-C
PROJECT LOCATION 8325 Quail Canyon Rd, Vacaville CA
DATE STARTED 8/14/23 **COMPLETED** 8/14/23
GROUND ELEVATION 270 ft **HOLE SIZE** 2.5"
DRILLING CONTRACTOR California Geotech Services, LLC
GROUND WATER LEVELS:
DRILLING METHOD Solid Flight CME-75
AT TIME OF DRILLING ---
LOGGED BY SS **CHECKED BY**
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	SPT BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		(CL) **Silty CLAY** : Brown to yellow-brown, moist, stiff to very stiff, low to medium plasticity										
			MC 2-1		3-8-10 (18)	>4.5	91	20	52	26	26	
5			MC 2-2		5-6-7 (13)	3.5	111	16				
10			MC 2-3		4-5-7 (12)							
15			SPT 2-4		3-4-5 (9)			28				

Bottom of borehole at 15.0 feet.

BORING NUMBER B-3

GEO-ENGINEERING SOLUTIONS, INC.
Geotechnical Engineering • Engineering Geology • Materials Testing

CLIENT Jim Nuti
PROJECT NAME Double T Ranch - Main Residence
PROJECT NUMBER 109-1519-C
PROJECT LOCATION 8325 Quail Canyon Rd, Vacaville CA
DATE STARTED 8/14/23 **COMPLETED** 8/14/23
GROUND ELEVATION 290 ft **HOLE SIZE** 2.5"
DRILLING CONTRACTOR California Geotech Services, LLC
GROUND WATER LEVELS:
DRILLING METHOD Solid Flight CME-75
AT TIME OF DRILLING ---
LOGGED BY SS **CHECKED BY**
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	SPT BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		**Weathered CLAYSTONE** : Gray-brown, moist, very stiff to hard										
			MC 3-1		3-8-9 (17)							
5			MC 3-2		7-14-33 (47)	>4.5	112	16				
10			SPT 3-3		19-22-50 (72)							

Bottom of borehole at 10.0 feet.

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CLIENT Jim Nuti
PROJECT NAME Double T Ranch - Main Residence
PROJECT NUMBER 109-1519-C
PROJECT LOCATION 8325 Quail Canyon Rd, Vacaville CA
DATE STARTED 8/14/23 **COMPLETED** 8/14/23
GROUND ELEVATION 285 ft **HOLE SIZE** 2.5"
DRILLING CONTRACTOR California Geotech Services, LLC
GROUND WATER LEVELS:
DRILLING METHOD Solid Flight CME-75
AT TIME OF DRILLING ---
LOGGED BY SS **CHECKED BY**
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	SPT BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		**Weathered CLAYSTONE** : Gray-brown, moist, stiff										
			MC 4-1		3-3-4 (7)	>4.5	99	21	38	22	16	
		(CL) **Silty CLAY** : Brown, moist, stiff to very stiff, medium plasticity	MC 4-2		5-4-6 (10)							
5												
			MC 4-3		5-7-10 (17)	4.0	99	22				
10												
		Weathered CLAYSTONE : Gray-brown, moist, stiff	SPT 4-4		7-7-5 (12)							
15												

Bottom of borehole at 15.0 feet.

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BORING NUMBER B-5

CLIENT Jim Nuti
PROJECT NAME Double T Ranch - Main Residence
PROJECT NUMBER 109-1519-C
PROJECT LOCATION 8325 Quail Canyon Rd, Vacaville CA
DATE STARTED 8/14/23 **COMPLETED** 8/14/23
GROUND ELEVATION 296 ft **HOLE SIZE** 2.5"
DRILLING CONTRACTOR California Geotech Services, LLC
GROUND WATER LEVELS:
DRILLING METHOD Solid Flight CME-75
AT TIME OF DRILLING ---
LOGGED BY SS **CHECKED BY**
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	SPT BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		**Weathered CLAYSTONE** : Gray-brown, moist, hard										
			MC 5-1		12-14-20 (34)	>4.5	65	94				
			MC 5-2		14-25-33 (58)							
5												
			SPT 5-3		22-50			17				
10												

Bottom of borehole at 10.0 feet.

BORING NUMBER B-6

GEO-ENGINEERING SOLUTIONS, INC.
Geotechnical Engineering • Engineering Geology • Materials Testing

CLIENT Jim Nuti
PROJECT NAME Double T Ranch - Main Residence
PROJECT NUMBER 109-1519-C
PROJECT LOCATION 8325 Quail Canyon Rd, Vacaville CA
DATE STARTED 8/15/23 **COMPLETED** 8/15/23
GROUND ELEVATION 247 ft **HOLE SIZE** 2.5"
DRILLING CONTRACTOR California Geotech Services, LLC
GROUND WATER LEVELS:
DRILLING METHOD Solid Flight CME-75
AT TIME OF DRILLING ---
LOGGED BY SS **CHECKED BY**
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	SPT BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		(CL) **Silty CLAY** : Light brown to gray-brown, moist, very stiff, low plasticity										
			MC 6-1		5-8-9 (17)			12				
5			MC 6-2		5-8-10 (18)	>4.5	106	16	46	28	18	
10			MC 6-3		5-6-8 (14)							
15			SPT 6-4		4-5-6 (11)							

Bottom of borehole at 15.0 feet.

BORING NUMBER B-7

GEO-ENGINEERING SOLUTIONS, INC.
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CLIENT Jim Nuti
PROJECT NAME Double T Ranch - Main Residence
PROJECT NUMBER 109-1519-C
PROJECT LOCATION 8325 Quail Canyon Rd, Vacaville CA
DATE STARTED 8/15/23 **COMPLETED** 8/15/23
GROUND ELEVATION 246 ft **HOLE SIZE** 2.5"
DRILLING CONTRACTOR California Geotech Services, LLC
GROUND WATER LEVELS:
DRILLING METHOD Solid Flight CME-75
∇ **AT TIME OF DRILLING** 12.00 ft / Elev 234.00 ft
LOGGED BY SS **CHECKED BY**
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	SPT BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		(CL) **Silty CLAY** : Brown, moist, soft to medium stiff, low plasticity										
			MC 7-1		2-1-2 (3)							
5			MC 7-2		2-3-2 (5)							
		Weathered CLAYSTONE : Gray-brown, moist, very stiff	MC 7-3		7-8-13 (21)							
10		∇										
15			SPT 7-4		17-22-25 (47)			25				

Bottom of borehole at 15.0 feet.

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BORING NUMBER B-8

CLIENT Jim Nuti
PROJECT NAME Double T Ranch - Main Residence
PROJECT NUMBER 109-1519-C
PROJECT LOCATION 8325 Quail Canyon Rd, Vacaville CA
DATE STARTED 8/15/23 **COMPLETED** 8/15/23
GROUND ELEVATION 244 ft **HOLE SIZE** 2.5"
DRILLING CONTRACTOR California Geotech Services, LLC
GROUND WATER LEVELS:
DRILLING METHOD Solid Flight CME-75
AT TIME OF DRILLING ---
LOGGED BY SS **CHECKED BY**
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	SPT BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		(CL) **Silty CLAY** : Brown, moist, medium stiff to stiff, low plasticity										
			MC 8-1		3-3-3 (6)	1.5	88	15				
5			MC 8-2		2-3-4 (7)							
		Weathered CLAYSTONE : Red-brown to gray-brown, moist, hard										
10			MC 8-3		8-14-16 (30)	>4.5	88	29				
15			SPT 8-4		13-22-50 (72)							

Bottom of borehole at 15.0 feet.

APPENDIX B

LABORATORY TEST RESULTS
Atterberg Limits Results
Grain Size Distribution Test Results
Corrosion Test Results

ATTERBERG LIMITS' RESULTS

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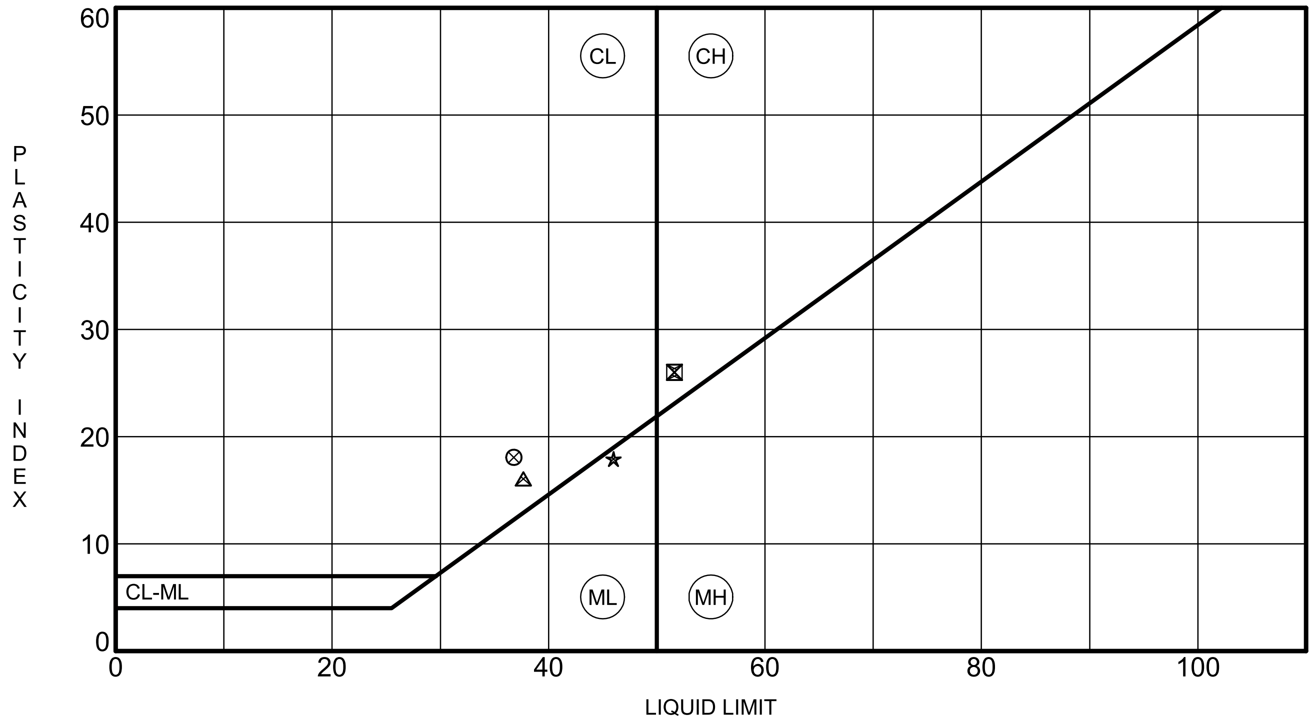
CLIENT Jim Nuti **PROJECT NAME** Double T Ranch - Main Residence

PROJECT NUMBER 109-1519-C **PROJECT LOCATION** 8325 Quail Canyon Rd, Vacaville CA



	Specimen Identification		LL	PL	PI	Fines	Classification
⊗	**B-1**	**Depth:3.5'**	**37**	**19**	**18**		
⊠	**B-2**	**Depth:1.0'**	**52**	**26**	**26**		
◬	**B-4**	**Depth:1.0'**	**38**	**22**	**16**		
★	**B-6**	**Depth:3.5'**	**46**	**28**	**18**		

GRAIN SIZE DISTRIBUTION

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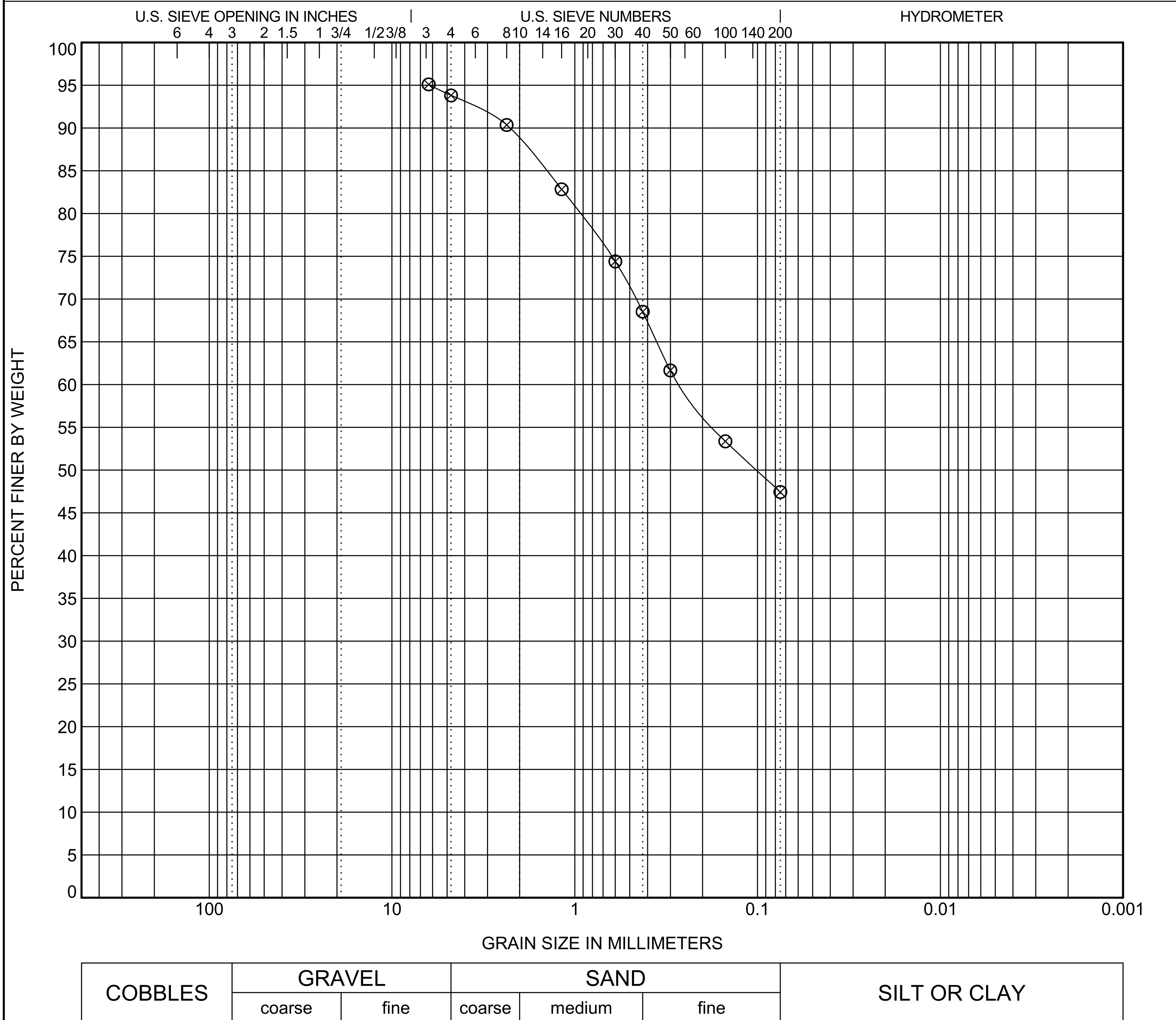
CLIENT Jim Nuti
PROJECT NAME Double T Ranch - Main Residence
PROJECT NUMBER 109-1519-C
PROJECT LOCATION 8325 Quail Canyon Rd, Vacaville CA



Specimen Identification		Classification	LL	PL	PI	Cc	Cu
⊗ B-1	Depth: 13.5'						

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
⊗ B-1	Depth: 13.5'	6.3	0.261			1.3	46.4	47.4	

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

Client: Geo-Engineering Solutions
Client's Project No.: 109-1519-C
Client's Project Name: Double T Ranch - Vacaville, CA
Date Sampled: 14-Aug-2023
Date Received: 17-Aug-2023
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 23-Aug-2023

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Resistivity (As Received) (ohms-cm)	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
2308039-001	B7-2	160	7.55	-	2,900	-	N.D.	N.D.

Method:		ASTM D1498	ASTM D4972	ASTM G57	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:		-	-	-	-	50	15	15
Date Analyzed:		17-Aug-2023	18-Aug-2023	-	21-Aug-2023	-	18-Aug-2023	18-Aug-2023

Julia Clauson
Chemist

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

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