Geo-Engineering Solutions, Inc.
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Geotechnical Engineering • Engineering Geology • Materials Testing

GEOTECHNICAL ENGINEERING STUDY

Benicia Residence

800 1st Street, Benicia, CA 94510

June 20, 2018

Prepared for:

GKW Architects
710 E McGlincy Lane, #109
Campbell, California 95008

By:

Geo-Engineering Solutions, Inc.
2570 San Ramon Valley Blvd, Suite #A102
San Ramon, California 94583
Project No. 11-1065

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June 20, 2018

GKW Architects
710 E McGlincy Lane, #109
Campbell, California 95008

Attention: Ms. Weiran Jia

Subject: **Geotechnical Engineering Study**
Benicia Residence
800 1st Street
Benicia, California 94510
Geo-Eng Project No. 11-1065

Dear Ms. Jia:

Geo-Engineering Solutions, Inc. has prepared a Geotechnical Engineering Study for the proposed commercial/residential improvements located at 800 1st Street in Benicia, California. It is our understanding that the proposed project will consist of the demolition of an existing single-story commercial unit and the construction of a two to three story building with commercial development on the first floor and residential development on the upper floors.

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 of 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 service to GKW Architects, and to be involved in the design of this project.

Sincerely,

GEO-ENGINEERING SOLUTIONS, INC.

Colin Frost, PE
Project Engineer

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

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June 20, 2018

GEOTECHNICAL ENGINEERING STUDY

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800 1st Street
Benicia, California 94510

Client: **GKW Architects**
Campbell, California

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 the 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 800 1st Street in Benicia, California. The project site is bordered by 1st Street to the northwest and East H Street to the southwest. The project site consists of two adjacent rectangular lots that form an L shape, with a total approximate area of 8,250 square feet, and maximum width of 150 feet and maximum depth of 75 feet. Commercial developments are located adjacent to the northeast and southeast of the subject property. The project site is currently occupied by a single-story commercial development with an attached modular square-shaped structure on the southeastern end. The southeastern portion of the property consists of a gravel covered parking area. The areas to the northwest and southwest of the building are covered in concrete and asphalt and are primarily used for parking. The topography of the site is generally flat, with an approximate elevation of about +35 feet based off Google Earth elevations.

1.3 Proposed Development

Based on proposed architectural plans provided by the client and as shown on *Figure 2, Development Site Plan*, we understand that the development will consist of the demolition of the existing single-story commercial structure and the construction of a new two to three-story structure that will consist of commercial space on the first floor and residential space on the upper floors.

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

A total of three borings were drilled at the site on May 22, 2018 at the locations shown on Figure 3*, Site Plan and Site Geology Map*. The borings were drilled in the concrete parking area located in front of the existing structure, and on the southeastern portion of the property in the asphalt and gravel parking areas to a maximum depth of 35 feet bgs. The borings were drilled using a truck mounted CME 55 drill rig equipped with an eight-inch diameter, hollow-stem flight auger.

A Geo-Eng staff geotechnical engineer visually classified the materials encountered in the borings in general accordance with 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 used to obtain SPT blow counts and obtain disturbed soil samples. The samplers were driven by using a 140-pound wireline hammer with an approximate 30-inch fall utilizing N-rods as necessary. Resistance to penetration was recorded in the field as the number of hammer blows required to drive the sampler the final foot of an 18-inch drive. Following the completion of drilling, the boreholes were backfilled using cement grout.

For reporting purposes, all of the blow counts recorded using Modified California (MC) split spoon samplers in the field were subsequently converted to equivalent SPT blow counts using appropriate modification factors suggested by Burmister (1948); i.e., multiplied by a factor of 0.65 assuming a liner sample with 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.

The boring logs with descriptions of the various materials encountered in each boring, the penetration resistance values, and some of the laboratory test results are presented in Appendix A. The ground surface elevations indicated on the soil boring logs are approximate (rounded to the nearest foot) and were estimated using elevations inferred from the Google Earth Pro application.

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. One test was performed, and the test results are presented in Section 4.1, in Appendix B, and on the applicable boring log.

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.

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 Appendix B and discussed in Section 4.3.

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 north of Carquinez Strait, which connects Susuin Bay in the east to San Pablo Bay in the west, near the foothills of the Coast Ranges. The property is located in a flat depositional environment with Upper Cretaceous clay shale and sandstone of the Panoche Formation, a formation within the Great Valley Sequence, to the north of the property and south of the Carquinez Strait (Dibblee and Minch, 2005). In addition, small areas of Paleocene claystone and siltstone shale of the Martinez Formation are located near hillsides adjacent to Panoche Formation. Across the Franklin Fault to the west is younger Miocene marine sandstone and shale of the Monterey and San Pablo groups. Across the Green Valley Fault to the east is Pliocene Sonoma Volcanics consisting of andesite, and sand, silt, and volcaniclastic rocks of the Tehama Formation (Wagner and Bortugno, 1982).

Deposits within the general area of the property consist of intertidal deposits; the sediments underlying the property consist of Pleistocene alluvial gravel and sand deposits adjacent to nearby Holocene alluvial gravel, sand, and clay.

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 Hayward Fault, the Calaveras Fault, and the Greenville Fault.

Major faults near the subject property include the Concord-Green Valley Fault located about four miles to the east, the Hayward Fault located about 13 miles to the west, the Calaveras Fault located about 15 miles to the south, and the San Andreas Fault located about 34 miles to the west. Additional notable faults near the subject property include the Contra Costa Shear Zone located about 1.5 miles to the west and the Franklin Fault located about four miles to the west.

The subject property is not mapped within a State of California Special Studies Zones map. The closest active fault zone mapped in a Special Studies Zones map is the Green Valley-Concord Fault located about four miles east of the subject property.

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 our test borings performed for this study (see Figures 2 or 3 for locations) and the results of our laboratory testing. Detailed descriptions of the various subsurface soil units encountered during subsurface explorations are described in the following paragraphs.

During our subsurface exploration program, we investigated the subsurface soils in three borings and evaluated soil conditions to a maximum depth of 35 feet for this study. From the ground surface to the maximum depth explored, the soils underlying the project site consist primarily of a layer of very stiff to hard medium to high plasticity clay down to an approximate depth of eight feet below existing ground surface, underlain by a layer of very dense silty sand and hard clayey silt down to an approximate depth of 20 to 30 feet below ground surface, which is underlain by hard silty clay to the maximum depth explored of 35 feet below ground surface. Boring B-3, located at the southeast portion of the property, contained clay throughout to the maximum depth explored of 25 feet below ground surface. In addition, a layer of fill, consisting of a very dense poorly graded gravel drain rock with sand and clay, was encountered in the upper 5 feet of Boring B-1. This fill appears to be have been placed after an underground tank was removed.

Two near surface samples of fine grained material located at the southeast portion of the property were tested for Atterberg Limits. A sample taken at 4.5’ below existing ground surface contained a measured Liquid Limit (LL) of 51, Plastic Limit (PL) of 22, and a corresponding Plastic Index (PI) of 29. In addition, a sample taken at 9.5’ below existing ground surface contained a measured LL of 38, PL of 21, and a corresponding PI of 17. Based on these test results, the near surface soils would be considered of high plasticity and have a high expansion potential.

A geological cross section through the proposed development area is presented in *Figure 6, Schematic Geologic Cross Section A-A’*.

4.2 Groundwater

Free groundwater was not encountered in any of the three borings during drilling, which were drilled to a maximum depth of 35 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 data could not be found near the vicinity of the property; however, it is expected to be at most about 30 feet below ground surface, which is sea level.

4.3 Corrosion Testing

A bulk sample collected from the upper one to three feet of Boring B-3 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 on 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
Dark Olive Brown Sandy CLAY with Gravel	0-2	121	2	380	779	Negative	7.4

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 measured to be about 121 mg/kg (ppm) or 0.0121% by dry weight 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 measured to be 2 mg/kg (ppm) or 0.0002% by dry weight 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 9 points, indicating a moderate 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 soils are 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 Hayward, Calaveras, San Andreas and Concord-Green Valley 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 soils encountered in the subsurface investigation included layers of very stiff to hard silty clay, very stiff to hard clayey silt, and very dense silty sand. These soils are expected to be generally less susceptible to liquefaction due to their relatively high density and fine-grain content. Additionally, groundwater was not encountered at the time of our investigation. Therefore, the potential for liquefaction of the site subsurface soils is judged to be low.

The site is not considered to be susceptible to lateral spreading due to the lack of a nearby free slope face. Therefore, the potential for future seismic settlement due to lateral spreading is judged to be very 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 soils generally consist of very stiff to hard silty clay, very stiff to hard clayey silt, and very dense silty sand at depths of about 15 to 30 feet bgs. 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). Based on our evaluation, the potential for fault ground rupture or creep at the site is very low to nil.

5.2 Expansive Soils

Highly expansive fine-grained soils were encountered in the upper five feet during our subsurface exploration. The results of the laboratory testing performed on a representative sample of the most expansive near-surface soils indicated a measured Plasticity Index of 29, indicative of a high plasticity and high expansion potential. Hence, mitigation for highly 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.

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 – Highly 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 – Undocumented fill soils consisting of very dense poorly graded drain rock with sand and clay were encountered in the upper 5 feet of Boring B-1. The fill appears to have been placed after the removal of an underground storage tank on site. Based on the high density and quality of the material we judge that it does need to be removed from the site during construction. No surficial undocumented fill soils and debris were encountered in other borings during our subsurface investigation on site. However, due to the presence of an existing building 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 proposed building should be designed in accordance with local design practice to resist the lateral forces generated by ground shaking associated with a major earthquake occurring within the greater Bay Area. Based on the subsurface conditions encountered in our borings 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 2016 California Building Code (CBC), we recommend the following seismic ground motion values be used for design shown in table 1, which are based on procedures outlined in ASCE 7-10 section 11.4.

Table 4: Seismic Design Parameters Based on ASCE 7-10

Item	Value	2016 CBC Source[R1]	ASCE 7-10 Table/Figure[R2]
Site Class	D	Table 1613A.3.2	Table 20.3-1
Mapped Spectral Response Accelerations Short Period, S_S 1-second Period, S_1	 1.505 g 0.600 g		 Figure 22-1 Figure 22-2
Site Coefficient, F_a	1.0	Table 1613A.3.3(1)	Table 11.4-1
Site Coefficient, F_v	1.5	Table 1613A.3.3(2)	Table 11.4-2
MCE (S_{MS})	1.505 g	Equation 16A-37	Equation 11.4-1
MCE (S_{M1})	0.900 g	Equation 16A-38	Equation 11.4-2
Design Spectral Response Acceleration Short Period, S_{DS} 1-second Period, S_{D1}	 1.003 g 0.600 g	 Equation 16A-39 Equation 16A-40	 Equation 11.4-3 Equation 11.4-4
Peak Ground Acceleration (PGA_M)	0.553 g	-	Equation 11.8-1

R1: California Building Standards Commission (CBSC), "California Building Code," 2016 Edition.
R2: U.S. Seismic "Design Maps" Web Application, https://geohazards.usgs.gov/secure/designmaps/us/application.php

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. Import fill materials should be approved by the Geotechnical Engineer prior to use on site.

6.2.2 Project Compaction Recommendations

Table 2 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

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 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 near 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

Following excavation to the required grades, subgrades in areas to receive engineered fill, slabs-on-grade, flatwork or pavements should be scarified to a depth of at least eight inches; moisture conditioned and compacted to the requirements for engineered fill presented in Section 6.2.2.

The compacted building pad surfaces should be firm and unyielding and should be protected from damage caused by traffic or weather. Soil subgrades should be kept moist during construction. In order 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.

In the event unstable subgrade conditions are encountered during construction and are unworkable for construction equipment, compaction of exposed on-site soil subgrades may not be feasible after exposure. These conditions may be remedied using soil admixtures, such as cement or a lime-cement mixture such as “Quicklime Plus”. More detailed recommendations can be provided during construction should unstable subgrades be encountered, or winterization measures be chosen by the contractor.

Unstable subgrades in smaller, isolated areas can be stabilized by over excavating to a minimum of 18 inches in 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% relative compaction.

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 eight 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.

6.3 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 B with temporary slope inclination of no steeper than 1:1 (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 Spread Footing Foundations

The proposed building can be supported on conventional continuous and/or isolated spread footings bearing on undisturbed medium dense to dense, onsite native 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 (typically the top of exterior grade) for exterior, perimeter footings, and a minimum of 24 inches below building pad subgrade for interior footings. 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 trench. Footing reinforcement should be determined by the project Structural Engineer.

For the design of the footings bearing within tested and approved new fill or on stiff/very stiff native soil, we recommend the allowable bearing pressures presented in Table 3, 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. The allowable pressures provided are net values, as the weight of the footing itself has already been accounted for and can be neglected as a load for design purposes.

Table 6: Allowable Bearing Pressures for Spread Footings

Load Condition	Allowable Bearing Pressure (psf)
Dead Load	3,000
Dead plus Live Loads	4,500
Total Loads (including wind or seismic)	6,000

6.5.2 Lateral Resistance

For resistance to lateral loads, an allowable coefficient of friction of 0.40 between the base of the foundation elements and underlying material is recommended. In addition, an ultimate passive resistance equal to an equivalent fluid weighing 400 pounds per cubic foot (pcf) acting against the foundation may be used to resist lateral forces. The top 12 inches 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.

6.5.3 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 is present, the excavation should be deepened until suitable supporting material is encountered.

The over excavation should be backfilled using structural or lean concrete 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 soil 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 at-grade interior slab-on-grade floors should be a minimum of five inches in thickness. The concrete floor slab should be underlain by a minimum 18-inch thickness of non-expansive fill (e.g., Class 2 aggregate base). Slab reinforcing should be provided in accordance with the anticipated use and loading of the slab, but as a minimum should consist of No. 4 bars spaced at 18-inch centers each way. Slab-on-grade subgrade surfaces should be proof-rolled to provide a smooth, unyielding surface for slab support.

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 has 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.

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

Retaining and below-grade walls may be founded on spread footing foundations following the recommendations outlined in section 6.5. Assuming a minimum 24-inch footing embedment below lowest adjacent grade, retaining wall footings may be designed using an allowable bearing capacity based off Table 4, in section 6.5.1.

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 persons. 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 upon 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 on the attached figures, logs or records regarding odors noted or other items or conditions observed are for the information of our client only.

8.0 REFERENCES

American Concrete Institute, 2008, Guide for Design and Construction of Concrete Parking Lots: ACI Publication 330R-08.

American Society of Civil Engineers, 2010, Minimum Design Loads for Buildings and Other Structures; ASCE/SEI Standard 7-10.

Association of Bay Area Governments (ABAG), September 2003, Liquefaction Susceptibility maps, based on William Lettis and Associates and USGS mapping, website: http://www.abag.ca.gov

Blake, Thomas F., EQSearch version 3.00, EQFault1 version 3.00, FriskSP version 4.00, software and manuals with 2004 CGS fault model updates.

California Building Code, 2016, Chapter 18.

California Department of Transportation (Caltrans), Memorandum: Foundation Report for Laurel Curve Soil Nail Wall

California Department of Transportation (Caltrans), ARS Online Website: http://dap3.dot.ca.gov/ARS_Online/ (Accessed May 2018)

California Division of Mines and Geology (CDMG), Digital Images of Official Maps of Alquist-Priolo Earthquake Fault Zones of California, Central Coast Region, DMG CD 2000-004, 2000.

California Division of Mines and Geology (CDMG), 1996, Probabilistic Seismic Hazard Assessment for the State of California: CDMG Open-File Report 96-08.

California Geological Survey, 2008, Guidelines for evaluating and mitigating seismic hazards in California: California Geological Survey Special Publication 117A, 98 p.

California Geological Survey, 2003, Seismic Hazard Zone Report for the Cupertino 7.5-Minute Quadrangle, Santa Clara County, California: Seismic Hazard Zone Report 090, 55 p.

California Geological Survey, 1996, Probabilistic seismic hazard assessment for the state of California, DMG Open-File report 96-08 (USGS Open-File Report 96-706), and updated Appendix A, Appendix A – 2002 California Fault Parameters.

Cao, T., Bryant, W.A., Rowshandel, B., Branum, D., and Wills, C.J., The Revised 2002 California Probabilities Seismic Hazard Maps, June 2003.

Dibblee, T.W., and Minch, J.A., 2005, Geologic map of the Benicia quadrangle, Contra Costa & Solano Counties, California: Dibblee Geological Foundation, Dibblee Foundation Map DF-146, scale 1:24,000, Image provided by Dibblee Geological Foundation.

National Earthquake Hazards Reduction Program (NEHRP), 2015, Recommended Seismic Provisions for New Buildings and Other Structures; FEMA P-1050-1/2015 Edition.

Graymer, R.W., Moring, B.C., Saucedo, G.J., Wentworth, C.M., Brabb, E.E., and Knudsen, K.L., 2006, Geologic Map of the San Francisco Bay Region, California: U.S. Geological Survey Scientific Investigations Map 2918, Scale 1:275,000.

Graymer, R.W., Jones, D.L., and Brabb, E.E., 2002, Geologic map and map database of northeastern San Francisco Bay region, California -- most of Solano County and parts of Napa, Marin, Contra Costa, San Joaquin, Sacramento, Yolo, and Sonoma Counties: U.S. Geological Survey, Miscellaneous Field Studies Map MF-2403, scale 1:100,000.

Gregg Drilling, Northern California Groundwater Depth Chart, http://www.greggdrilling.com/Resources/water_table_n.html

Hart, E. W., 1983, Summary Report: Fault Evaluation Program, 1981 – 1982, Area-Northern Coast Ranges region, California, California Department of Conservation, Division of Mines and Geology, Open-File Report 83-10.

Hart, E.W., and Bryant, W.A., 1997, Fault-rupture hazard zones in California: California Geological Survey Special Publication 42, revised 1997 with Supplements 1 and 2 added 1999, 38 p.

Helley, E.J. and Graymer, R.W., 1997, Quaternary Geology of Alameda County and Surrounding Areas, California: Derived from the Digital Database Open-File 97-97; U.S. Geological Survey.

Jennings, C.W., and Bryant, W.A., compilers, 2010: 2010 Fault activity map of California: California Geological Survey, Geologic Data Map No. 6, scale 1:750,000, with 94-page Explanatory Text booklet.

Page, B.M., 1966, Geology of the Coast Ranges of California: in Bailey, E.H., Jr., editor, Geology of Northern California: California Geological Survey Bulletin 190, p. 255-276.

Sims, J.D., Fox, K.F., Bartow, J.A., and Helley, E.J., 1973, Preliminary geologic map of Solano County and parts of Napa, Contra Costa, Marin, and Yolo Counties, California: U.S. Geological Survey, Miscellaneous Field Studies Map MF-484, scale 1:62,500.

Sowers, J.M., 1999, Creek & Watershed Map of Fremont & Vicinity: Oakland Museum of California, Oakland, CA, 1:25,800 scale.

Tokimatsu, K. and Seed, H.B., 1987, Evaluation of Settlements in Sands due to Earthquake Shaking: ASCE Journal of Geotechnical Engineering, Vol. 113, No. 8, pp. 861-878.

University of California, Berkeley, Bancroft Library: Historic Maps of California – San Francisco Bay Area: http://sunsite.berkeley.edu/histopo/

U.S. Department of Agriculture, Natural Resources Conservation Service, Web Soil Survey; http://websoilsurvey.nrcs.usda.gov/app/WebSoilSurvey.aspx

U.S. Geological Survey, 1999, Earthquake probabilities in the San Francisco Bay region: 2000 to 2030 – A summary of findings, by Working Group on California Earthquake Probabilities, Open File Report 99-517, Online Version 1.0.

U. S. Geological Survey Earthquake Information Center, 2003, website earthquake.usgs.gov.

U. S. Geological Survey, 2012, Newark Quadrangle, California – Alameda Co. 7.5 Minute Series (Topographic).

U. S. Geological Survey, Earthquake Hazards Program website eqhazmaps.usgs.gov.

2007 Working Group on California Earthquake Probabilities (WGCEP), 2008, The Uniform California Earthquake Rupture Forecast, Version 2 (UCERF 2): U.S. Geological Survey Open-File Report 2007-1437.

Wagner, D. L., and others, compilers, 1991, Geologic map of the San Francisco – San Jose quadrangle: California Geological Survey, Regional Geologic Map 5A; scale 1:250,000.

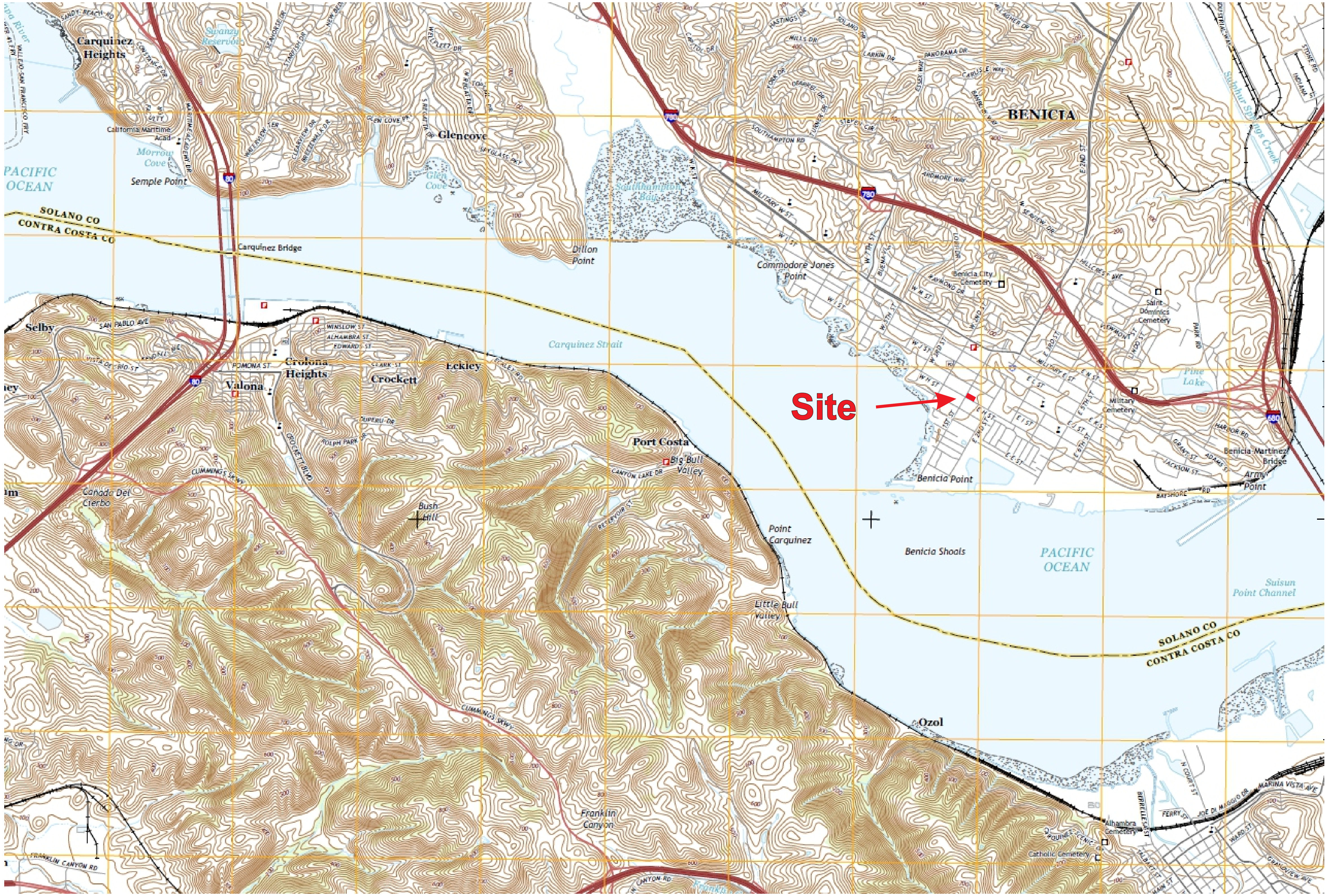
Wagner, D.L., and Bortugno, E.J., 1982, Geologic map of the Santa Rosa quadrangle, California, 1:250,000: California Division of Mines and Geology, Regional Geologic Map 2A, scale 1:250,000.

Youd, T.L., and Hoose, S.N., 1978, Historic ground failures in northern California triggered by earthquakes: U.S. Geological Survey Professional Paper 993, 177 p., 5 pls. in pocket.

Youd, T.L., Idriss, I.M., Andrus, R.D., Arango, I., Castro, G., Christian, J.T., Dobry, R., Finn, W.D.L., Harder, L.F. Jr., Hynes, M.E., Ishihara, K., Koester, J.P., Liao, S.S.C., Marcuson, W.F. III, Martin, G.R., Mitchell, J.K., Moriwaki, Y., Power, M.S., Robertson, P.K., Seed, R.B., and Stokoe, K.H. II, 2001, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils: ASCE Journal of Geotechnical and Environmental Engineering, Vol. 127, No. 10, October 2001, p. 817-833.

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

FIGURES



Source: Benicia Quadrangle, California, US Topographic Map
7.5-Minute Series, United States Geological Survey (2015)

Map Scale

Benicia Residence 800 1st Street Benicia, CA	11-1065	June 2018
	Site Vicinity Map	Figure 1

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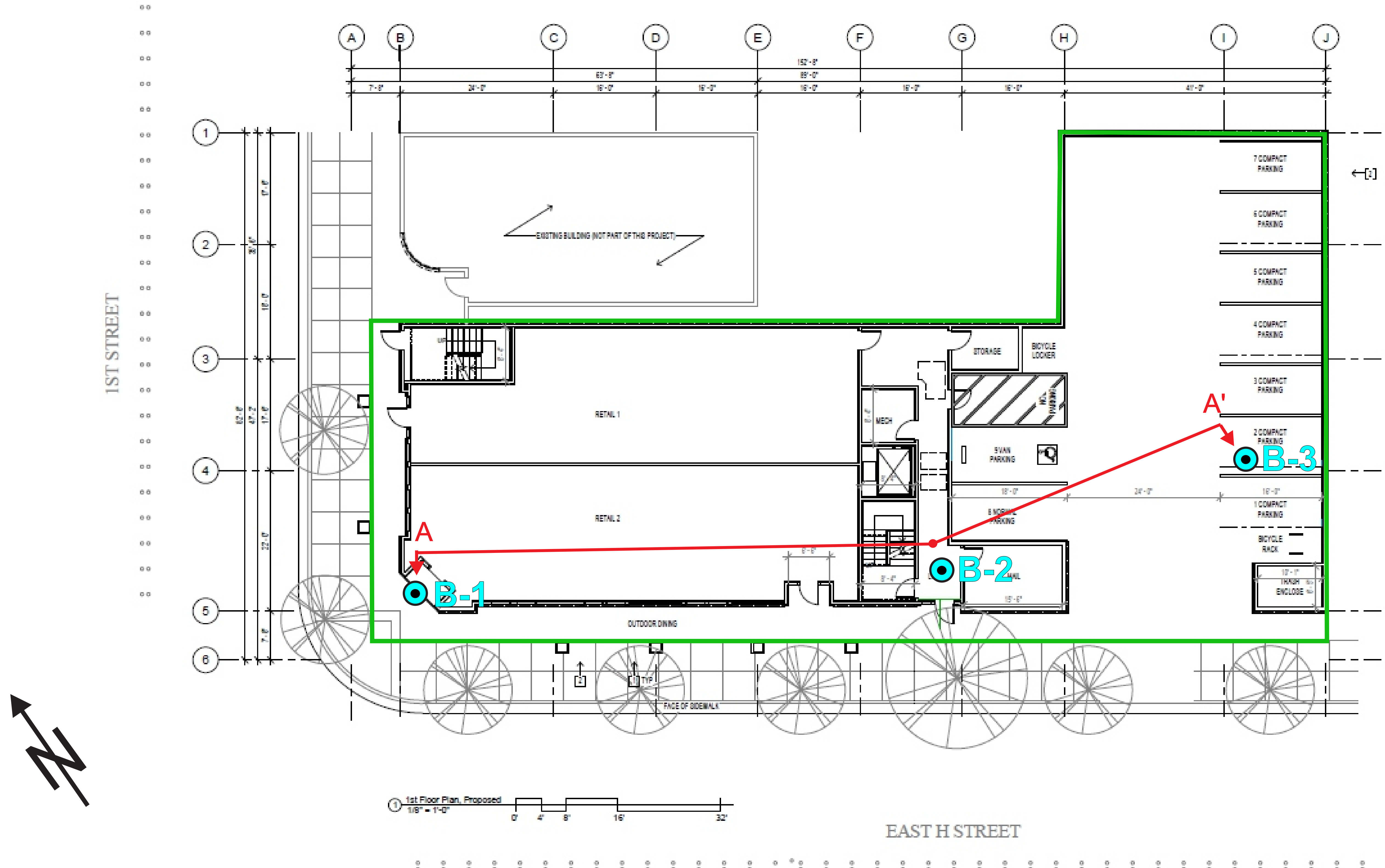

Property boundary

Approximate Boring Location

Geologic Cross Section

Base Map Reference: Google Earth

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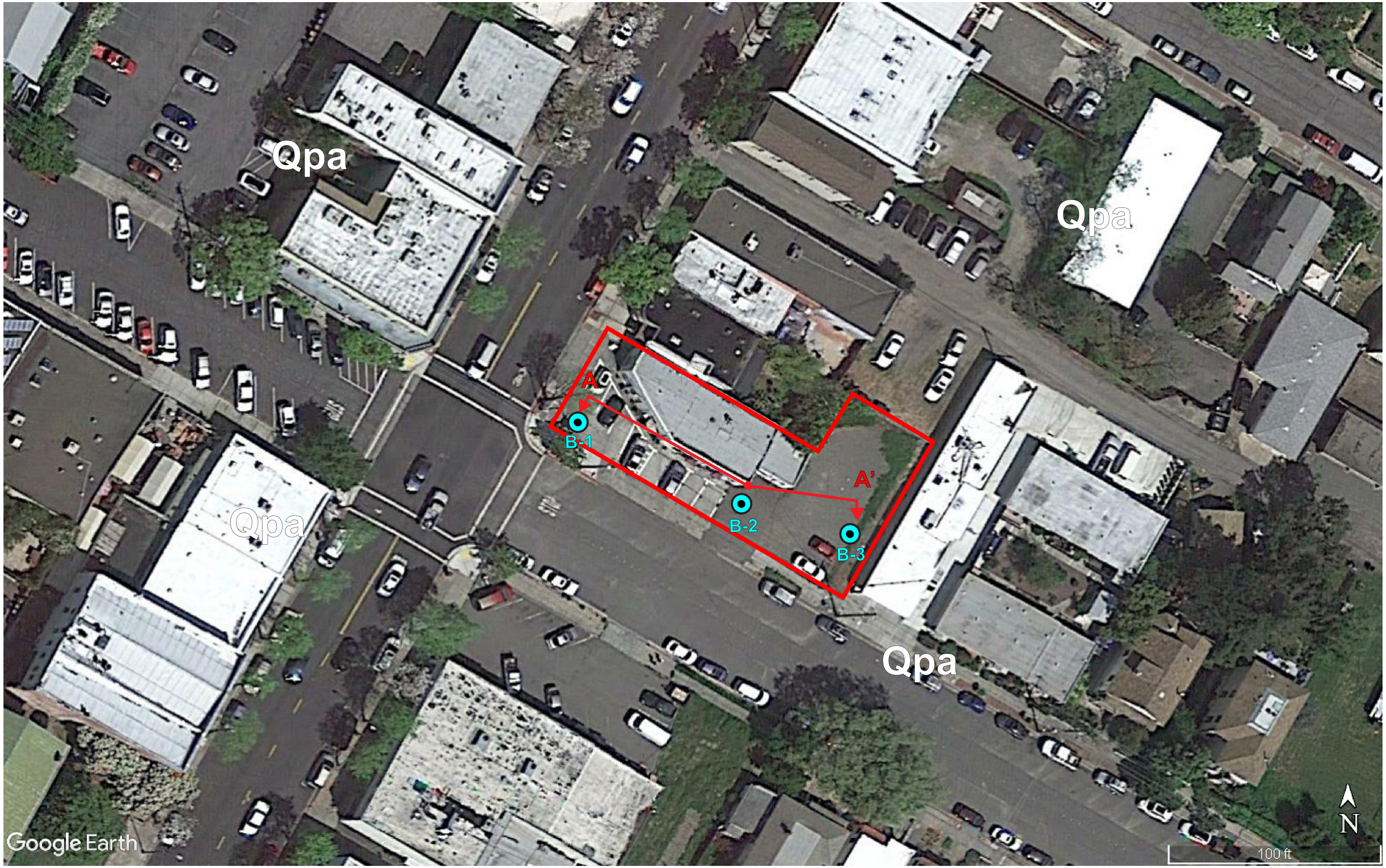
Benicia Residence 800 1st Street Benicia, CA	11-1065	June 2018
	Site Development Plan	Figure 2



Source: USGS w/ California Geologic Survey Scientific Investigations Map 2918, Geologic Map of the San Francisco Bay Region

Property Boundaries

Qpa - Alluvium, Pleistocene

- Approximate Boring Location

- Geologic Cross Section

Base Map Reference: Google Earth

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Benicia Residence 800 1st Street Benicia, CA	11-1065	June 2018
	Site Plan and Site Geologic Map	Figure 3

Legend

af	Artificial Fill
Qhym	Mud deposits (late Holocene)
Qha	Alluvium (Holocene)
Qpa	Alluvium (Pleistocene)
Qoa	Alluvium (early Pleistocene)
Tpas	Sedimentary rocks (Paleocene)
Ks	Great Valley complex sedimentary rocks (Cretaceous)

Geologic Faults
Geologic Contacts

Source: USGS w/ California Geologic Survey Scientific Investigations Map 2918, Geologic Map of the San Francisco Bay Region

Base Map Reference: Google Earth

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Benicia Residence 800 1st Street Benicia, CA	11-1065	June 2018
	Site Vicinity Geology Map	Figure 4

Legend

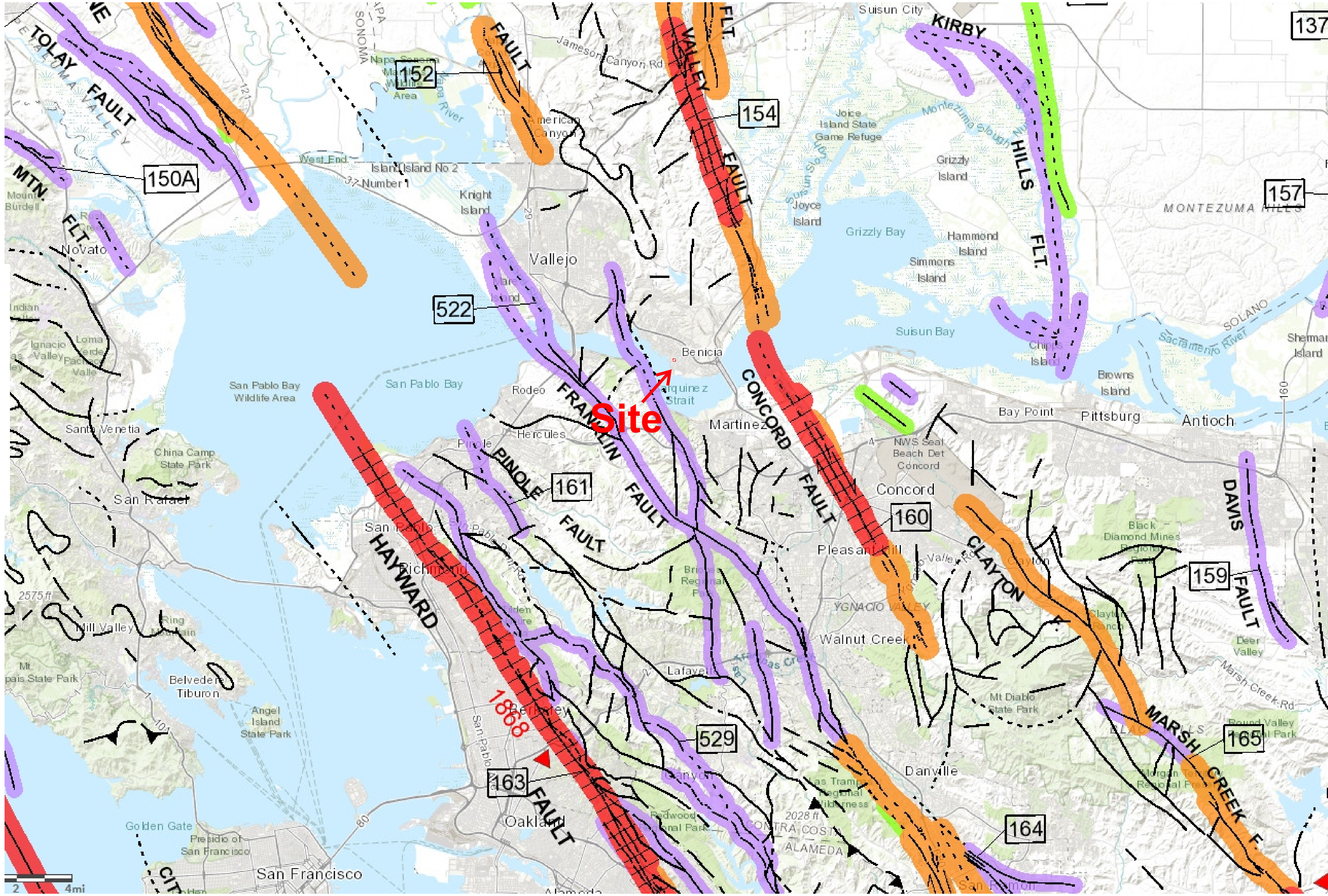
Fault along which historic (last 200 years) displacement has occurred.

Holocene fault displacement (during past 11,700 years) without historic record.

Late Quaternary fault displacement (during past 700,000 years).

Quaternary fault (age undifferentiated).

Pre-Quaternary fault (older that 1.6 million years) or fault without recognized Quaternary displacement.



Base Map Reference: California Geological Survey - 2010 Fault Activity Map of California

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		Regional Fault Map	Figure 5

Schematic Geologic Cross Section

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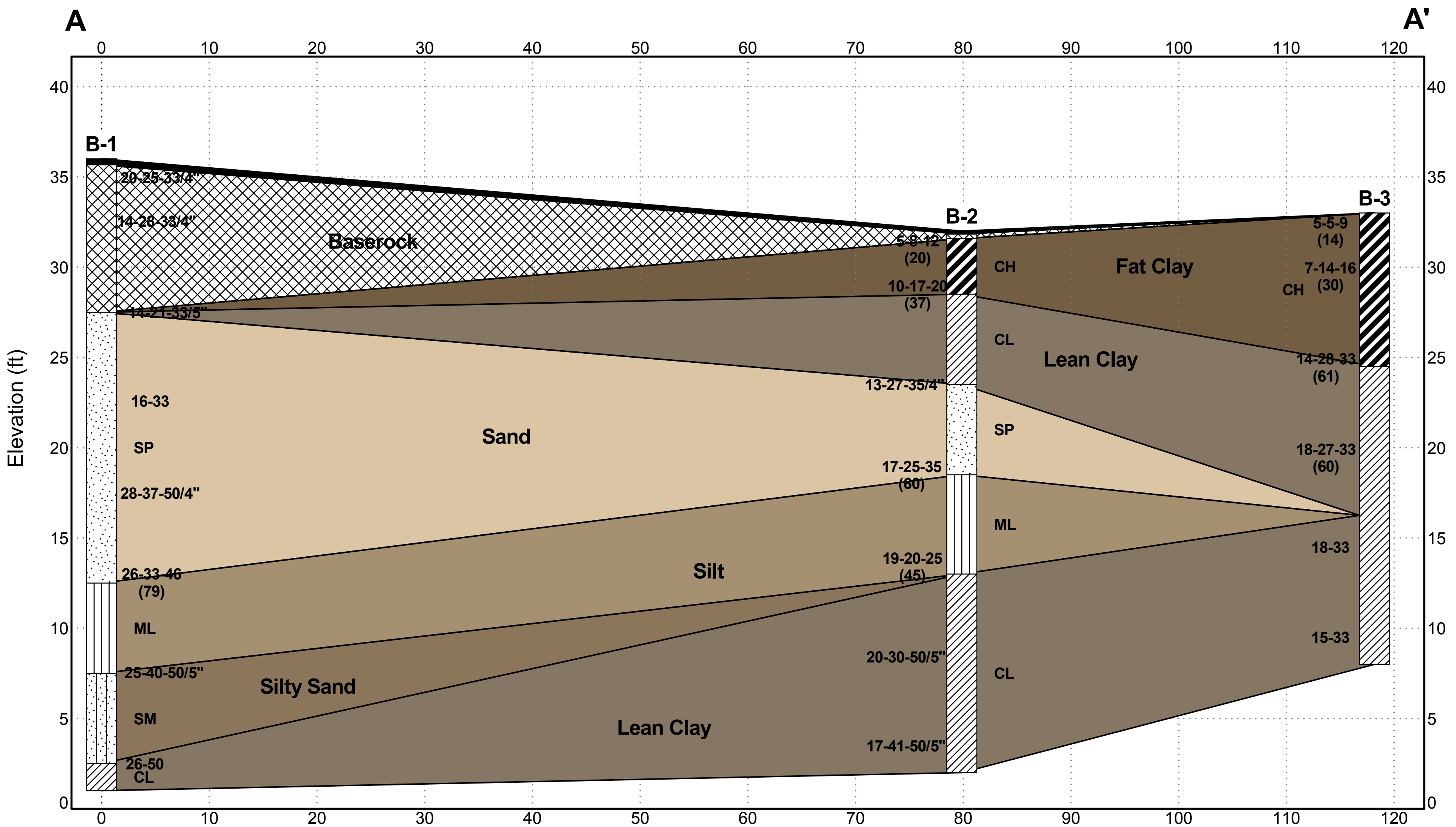
CLIENT GKW Architects

PROJECT NUMBER 11-1065

PROJECT NAME 800 1st Street in Benicia

PROJECT LOCATION 800 1st Street, Benicia



CL - Unified Soil Classification System (USCS)
14 - Blows Per Six Inches, Standard Penetration Test
(21) - N Value

Schematic Geologic Cross-Section A-A'	Figure 6

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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

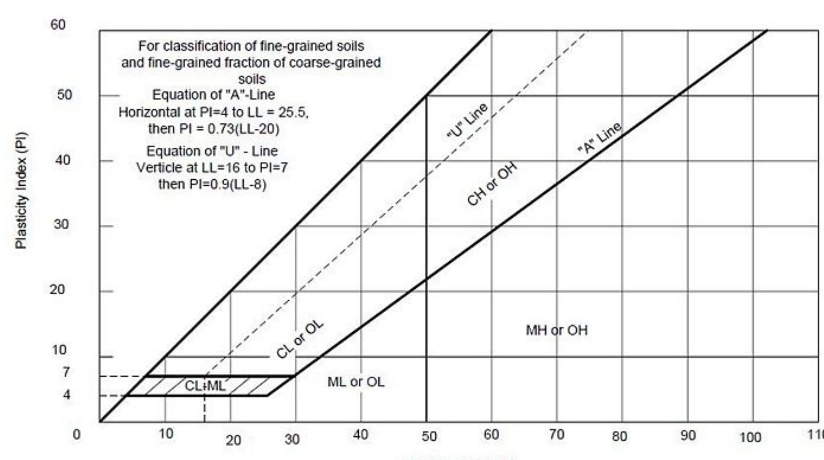
Blow Count
The number of blows of the sampling hammer required to drive the sampler through each of three 6-inch increments. Less than three increments may be reported if more than 50 blows are counted for any increment. The notation 50/5” indicates 50 blows recorded for 5 inches of penetration. Note 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



GEO-ENGINEERING SOLUTIONS, INC.
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Key to Exploratory Boring Logs

BORING NUMBER B-1

GEO-ENGINEERING SOLUTIONS, INC.
2570 San Ramon Valley Blvd, Suite A102
San Ramon, CA 94583
Telephone: 925433450

CLIENT GKW Architects
PROJECT NAME 800 1st Street in Benicia
PROJECT NUMBER 11-1065
PROJECT LOCATION 800 1st Street, Benicia
DATE STARTED 5/22/18 **COMPLETED** 5/22/18
GROUND ELEVATION 36 ft **HOLE SIZE** 8"
DRILLING CONTRACTOR Exploration Geoservices Inc.
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger
AT TIME OF DRILLING ---
LOGGED BY EP **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		**Concrete**										
		Poorly Graded Gravel : Grey, dry to moist, very dense, with sand and clay										
		(Fill: Drain Rock)	MC 1-1		20-25-33/4"							
			MC 1-2		14-28-33/4"							
5												
		(SM) **Silty Sand :** Brown, moist, very dense, fine to medium grained										
10			MC 1-3		14-21-33/5"		93	6				
		Reddish brown	MC 1-4		16-33		90	9				16
15												
20			SPT 1-5		28-37-50/4"							
		(ML) **Clayey Silt :** Light brown, moist, hard, with two interbedded clay layers each about 1" thick at 29.5'	SPT 1-6		26-33-46 (79)			28				
25												

(Continued Next Page)

Geo-Engineering Solutions, Inc.

2570 San Ramon Valley Blvd, Suite A102
San Ramon, CA 94583
Telephone: 925433450

BORING NUMBER B-1

CLIENT GKW Architects

PROJECT NAME 800 1st Street in Benicia

PROJECT NUMBER 11-1065

PROJECT LOCATION 800 1st Street, Benicia

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 (%)
25		(ML) **Clayey Silt** : Light brown, moist, hard, with two interbedded clay layers each about 1" thick at 29.5' *(continued)*										
30		(SM) **Silty Sand** : Brown, moist, very dense, with some clay	SPT 1-7		25-40-50/5"							
35		(CL) **Silty Clay** : Brown, moist, hard, with fine grained sand	SPT 1-8		26-50			34				

Bottom of borehole at 35.0 feet.

BORING NUMBER B-2

2570 San Ramon Valley Blvd, Suite A102
San Ramon, CA 94583
Telephone: 925433450

CLIENT GKW Architects
PROJECT NAME 800 1st Street in Benicia
PROJECT NUMBER 11-1065
PROJECT LOCATION 800 1st Street, Benicia
DATE STARTED 5/22/18 **COMPLETED** 5/22/18
GROUND ELEVATION 32 ft **HOLE SIZE** 8"
DRILLING CONTRACTOR Exploration Geoservices Inc.
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger
AT TIME OF DRILLING ---
LOGGED BY EP **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		**Asphalt**										
		Baserock										
		(CH) **Fat Clay** : Brown, moist, stiff, with little silt, high plasticity										
		Undraindned shear strength= 1020 psf	MC 2-1		5-8-12 (20)	0.50						
		(CL) **Lean Clay** : Light brown, moist, hard, with some silt and shell fragments, low plasticity	MC 2-2		10-17-20 (37)	>4.5	92	25				
5												
		(SP) **Poorly Graded Sand** : Brown, moist, very dense, fine grained, with few silt	MC 2-3		13-27-35/4"		91	7				7
10												
		(ML) **Clayey Silt** : Light brown, moist, hard	SPT 2-4		17-25-35 (60)							
15												
		(CL) **Clay** : Light grayish brown, moist, hard, with trace to few silt, medium plasticity	SPT 2-5		19-20-25 (45)							
20												
		Brown with few silt	SPT 2-6		20-30-50/5"							
25												

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San Ramon, CA 94583
Telephone: 925433450

BORING NUMBER B-2

CLIENT GKW Architects

PROJECT NAME 800 1st Street in Benicia

PROJECT NUMBER 11-1065

PROJECT LOCATION 800 1st Street, Benicia

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 (%)
25		(CL) **Clay** : Light grayish brown, moist, hard, with trace to few silt, medium plasticity *(continued)*										
30		With little to some silt	SPT 2-7		17-41-50/5"			26				

Bottom of borehole at 30.0 feet.

BORING NUMBER B-3

GEO-ENGINEERING SOLUTIONS, INC.
2570 San Ramon Valley Blvd, Suite A102
San Ramon, CA 94583
Telephone: 925433450

CLIENT GKW Architects
PROJECT NAME 800 1st Street in Benicia
PROJECT NUMBER 11-1065
PROJECT LOCATION 800 1st Street, Benicia
DATE STARTED 5/22/18 **COMPLETED** 5/22/18
GROUND ELEVATION 33 ft **HOLE SIZE** 8"
DRILLING CONTRACTOR Exploration
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger
AT TIME OF DRILLING ---
LOGGED BY EP **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		(CH) **Fat Clay** : Dark brown, moist, very stiff, with trace organics										
			MC 3-1		5-5-9 (14)	2.5	101	19				
5		Hard, with some silt	MC 3-2		7-14-16 (30)	2.0	100	20	51	22	29	
10		(CL) **Lean Clay** : Brown, moist, hard, with few silt	MC 3-3		14-28-33 (61)	>4.5	106	19	38	21	17	
15		Yellow brown, with some silt	MC 3-4		18-27-33 (60)	4.0	99	21				
20		With reddish brown mottling, with few fine sand	MC 3-5		18-33	2.3						
25		Light brown with reddish brown mottling	MC 3-6		15-33	2.5	90	28				

Bottom of borehole at 25.0 feet.

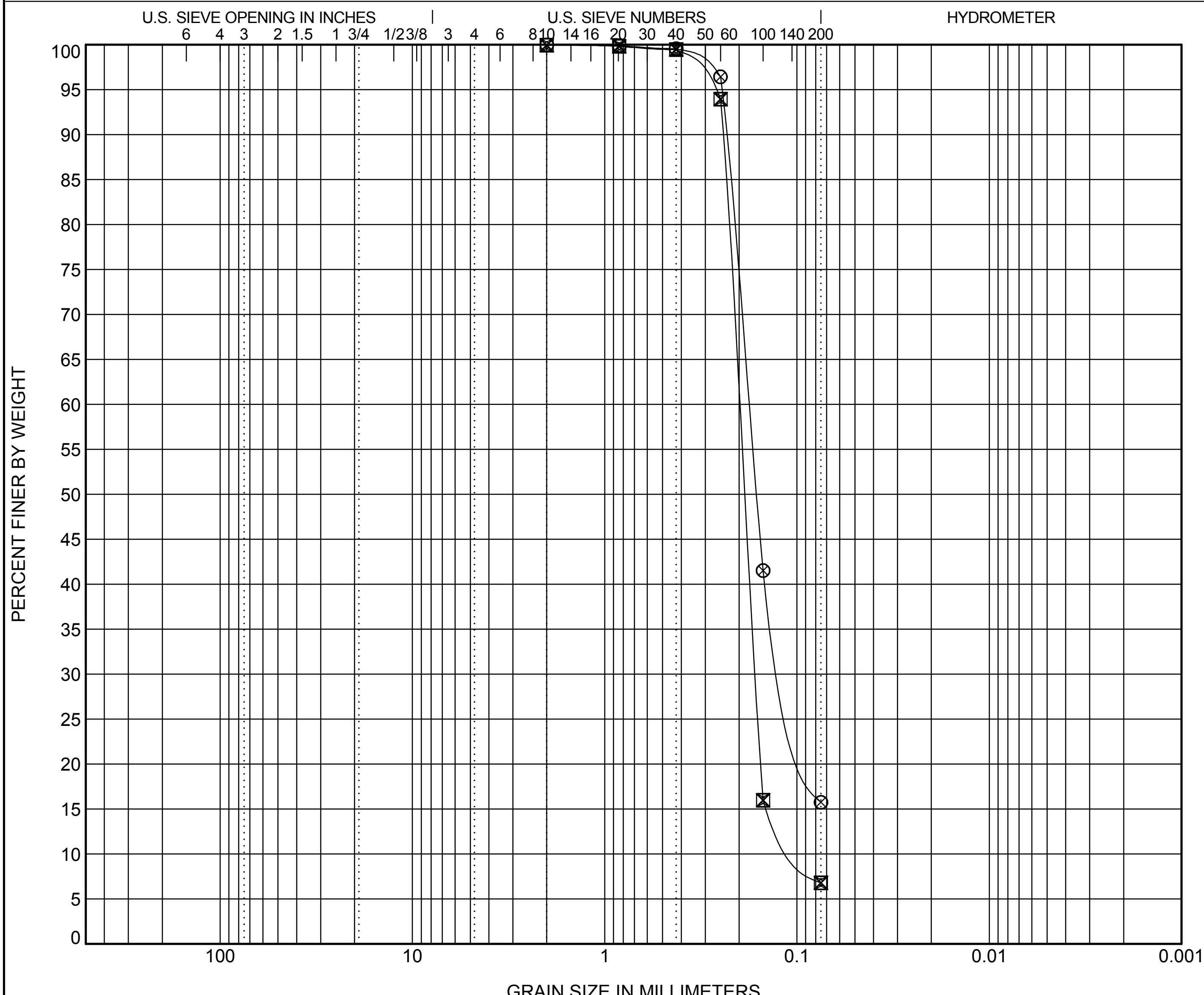
APPENDIX B

LABORATORY TEST RESULTS
Particle Size Distribution Report
Atterberg Limits Test Results

GRAIN SIZE DISTRIBUTION

2570 San Ramon Valley Blvd, Suite A102
San Ramon, CA 94583
Telephone: 9254330450

CLIENT GKW Architects
PROJECT NAME 800 1st Street in Benicia
PROJECT NUMBER 11-1065
PROJECT LOCATION 800 1st Street, Benicia



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	LL	PL	PI	Cc	Cu
⊗	**B-1**	**Depth: 14.5'**	**Silty SAND (SM)**					
⊠	**B-2**	**Depth: 9.5'**	**Poorly Graded SAND (SP)**				**1.41**	**2.09**

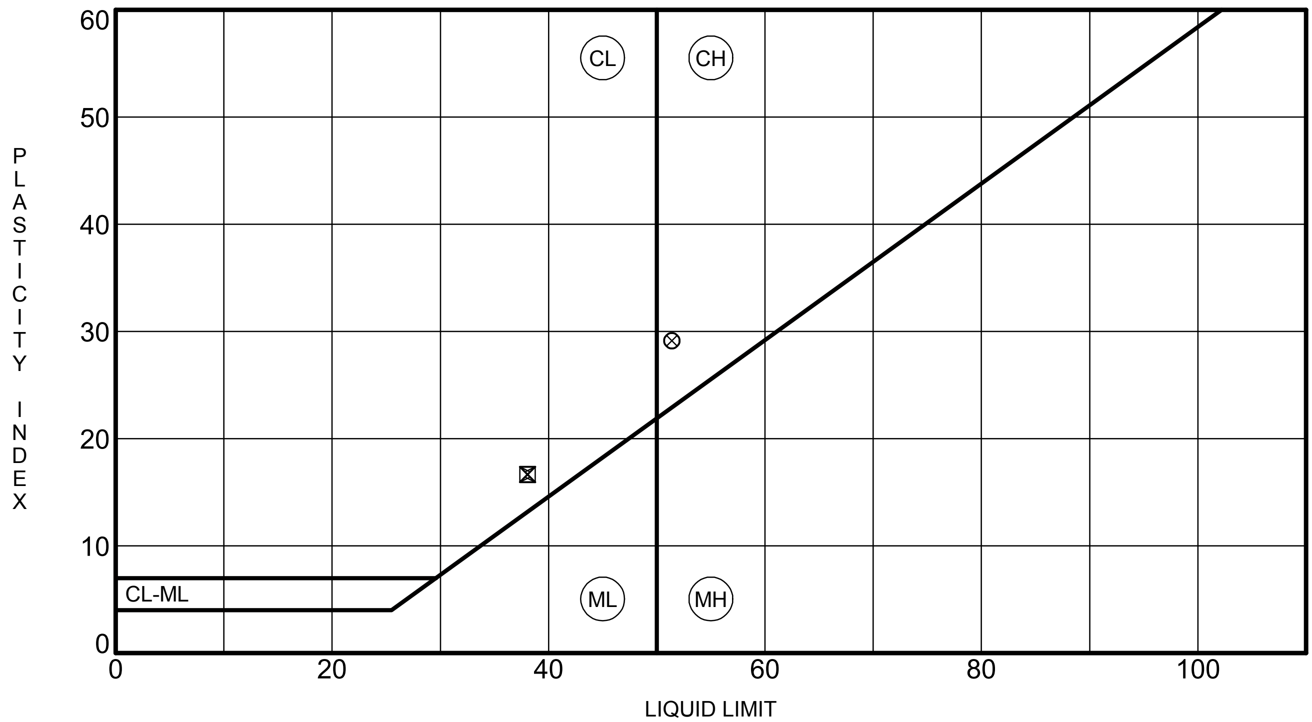
	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
⊗	**B-1**	**Depth: 14.5'**	**2**	**0.178**	**0.11**			**84.2**	**15.7**	
⊠	**B-2**	**Depth: 9.5'**	**2**	**0.2**	**0.164**	**0.096**		**93.2**	**6.8**	

ATTERBERG LIMITS' RESULTS

GEO-ENGINEERING SOLUTIONS, INC.
2570 San Ramon Valley Blvd, Suite A102
San Ramon, CA 94583
Telephone: 925433450

CLIENT GKW Architects **PROJECT NAME** 800 1st Street in Benicia

PROJECT NUMBER 11-1065 **PROJECT LOCATION** 800 1st Street, Benicia



	Specimen Identification		LL	PL	PI	Fines	Classification
⊗	**B-3**	**Depth:4.5'**	**51**	**22**	**29**		**Fat CLAY (CH)**
⊠	**B-3**	**Depth:9.5'**	**38**	**21**	**17**		**Lean CLAY (CL)**