GEOTECHNICAL ENGINEERING
AND
GEOLOGIC HAZARDS STUDY

Parking Garage and Roadway Widening Project
1650 Los Gamos Drive
San Rafael, California 94903
Kaiser Project # 839-951-01

Prepared for:

National Facilities Services - Kaiser Permanente
Kaiser Foundation Health Plan, Inc.
1950 Franklin Street - 12th Floor
Oakland, California 94612

Prepared by:

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GEO Project No. 91-03695-A

Geosphere Consultants, Inc.
AN ETS COMPANY
Geotechnical Engineering • Engineering Geology
Environmental Management • Water Resources

October 19th, 2016
Revised February 16, 2017

National Facilities Services - Kaiser Permanente
Kaiser Foundation Health Plan, Inc.
1950 Franklin Street - 12th Floor
Oakland, California 94612

Attention: Ms. Catherine Reilly, LEED AP, Senior Land Use Manager

Subject: **Geotechnical Engineering and Geologic Hazards Study Report**
Parking Garage and Roadway Widening Project
1650 Los Gamos Drive, San Rafael, California 94903
Kaiser Project No. 839-951-01; GEO Project No. 91-03695-A

Dear Ms. Reilly:

Geosphere Consultants, Inc. (GEOSPHERE) has completed a Geotechnical Engineering and Geologic Hazards Study for the proposed 1650 Los Gamos Drive Project, in San Rafael, Marin County, California which includes allowing the existing building to be used for medical office uses, the construction of a parking garage, as well as the use of 42 existing parking spaces on the adjacent property at 1600 Los Gamos Drive ("Project"). In addition, this study analyzes the potential impacts related to a related potential roadway improvement project that may be required as mitigation for the Project. This report has been prepared based on our discussion with you, the project planning team, and review of the provided project plans and focuses on the parking structure and off-site mitigation improvements since the reuse of the building and 42 parking spaces would only result in limited construction external to the existing structures, such as landscaping upgrades. Transmitted herewith are the results of our findings, conclusions, and recommendations for foundation, lateral earth pressures, seismic design parameters, interior and exterior concrete slabs, site preparation, grading, foundation excavation, drainage, utility trench backfilling, and pavement design. In general, the proposed improvements at the site are considered to be geotechnically as well as geologically 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 either of the undersigned at (925) 314-7180, or by e-mail at rshrestha@geosphereinc.net or cdare@geosphereinc.net. We appreciate the opportunity to be of service to Kaiser Permanente and to be involved in the design of this project.

Sincerely,
GEOSPHERE CONSULTANTS, INC.

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GEOTECHNICAL ENGINEERING AND GEOLOGIC HAZARDS STUDY

Project: 1650 Los Gamos Drive Project
San Rafael, California

Client: National Facilities Services - Kaiser Permanente
Oakland, California

1.0 INTRODUCTION

1.1 Purpose and Scope

The purpose of this study was to evaluate the subsurface conditions at the site and prepare geotechnical recommendations for the proposed new parking garage and roadway widening associated with potential mitigation that may be required for the 1650 Los Gamos Project. This study provides recommendations for foundations, lateral earth pressures, seismic design parameters, interior and exterior concrete slabs, site preparation, grading, foundation excavation, drainage, utility trench backfilling, and pavement design related to the parking structure and roadway widening and limited surface improvements around the existing building. This study was performed in accordance with the scope of work outlined in our revised proposal dated April 22, 2016 and Change Order Request dated August 22, 2016.

The scope of this study included the review of available geotechnical and geologic literature for the site, the drilling of several subsurface borings within the project site, laboratory testing of selected samples retrieved from the borings, engineering analysis of the accumulated data, and preparation of this report. The conclusions and recommendations presented in this report are based on the data acquired and analyzed during this study, and on prudent engineering judgment and experience. This study did not include an assessment of potentially toxic or hazardous materials that may be present on or beneath the site.

1.2 Site Description

The proposed developments are located at or near 1650 Los Gamos Drive in San Rafael, Marin County, California, as shown on *Figure 1, Site Vicinity Map*. The proposed new parking garage will be located at the southwest corner of the intersection of Los Gamos Drive and Lucas Valley Road and widening of the roadway will be along the Lucas Valley Road, the section from Los Gamos to On/Off Ramp to Highway 101, as shown on *Figure 2, Site Plan and Site Geology Map*. The parking garage is located on relatively sloped topography sloping from west to east, the site average elevations range from approximately 54 feet to 36 feet above mean sea level (amsl) based on topographic map provided by Watry Design Inc. The project area is bounded on the south, west, and north by vacant land/hills and east by Los Gamos Drive. The project site is located at approximately 38.0216° north latitude and 122.5428° west longitude.

Note: Review of historical topographic maps of California, Novato 7.5-minute Quadrangle, from 1960 and 1980 have indicated the existence of an access road passing along the northwest side of the proposed parking structure connecting Los Gamos Drive and Salvador Way. However, that access road had disappeared from the current topographic map and the ALTA/ACSM Land Title Survey Map.

1.3 Proposed Development

It is our understanding that the proposed development will consist of the conversion of an existing building into a Kaiser Medical Office (MOB) Building for medical use; construction of an up to 511-space parking garage, and reuse of 42 existing parking spaces at 1600 Los Gamos Drive as well as potential mitigation which, if required, may result in the need for roadway improvements. We understand that the existing building consists of a little less than 148,000 square feet of space on three floors. The entire site, including the parking lot across Los Gamos Drive, is 11.1 acres in size and has 455 parking spaces. Kaiser Permanente (KP) will use the building for medical offices, which will be phased. Phase 1 will convert floors 2 & 3, and most of floor 1 into medical offices and reserve a portion of floor 1 for the two existing office tenants to finish out their leases. The property is subject to a Planned Development (PD) District, which includes the County building at 1600 Los Gamos Drive. The PD District allows office uses, but does not allow medical office uses. As a result, the PD District will need to be amended to allow for medical office use at 1650 Los Gamos.

The objective of our site investigation is to provide engineering recommendations for the new parking garage, roadway widening mitigation, and minor site improvements. Final detailed plans are not available at the time of this report preparation however we believe that the parking structure can be supported on isolated footing or mat foundation. The location of the proposed development is shown on Figure 2. This report does not provide foundation design recommendations for modifications to the existing office building, as none are anticipated since the building would not be expanded.

2.0 PROCEDURES AND RESULTS

2.1 Literature Review

Pertinent geologic and geotechnical literature pertaining to the site area was reviewed. These included various publications and maps issued by the United States Geological Survey (USGS), water agencies, and other government agencies, as listed in the References section.

2.2 Field Exploration

In order to characterize the subsurface conditions beneath the proposed improvement areas, two separate field exploration programs were conducted which consisted of the drilling of five borings within or near the proposed building footprint area and three borings in the proposed street widening areas as indicated by the Project Civil Engineer on July 21, 2016 and August 28, 2016 respectively, by a Staff Engineer under the supervision of a California-Certified Professional Engineer. The borings were sited to satisfy the project requirements and to facilitate development of soil cross section profiles across the area of the subject project. The locations of the borings relative to the proposed improvements are shown on Figure 2. The borings were drilled using a MCE-75 drill rig equipped with 4" solid flight augers.

A GEOSPHERE representative 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, and a two-inch outside diameter Standard Penetration Test (SPT) sampler. The samplers were driven by means of a 140-pound and 70-pound safety hammers with an approximate 30-inch fall. Resistance to penetration was recorded as the number of hammer blows required to drive the sampler the final foot of an 18-inch drive. All of the field blow counts recorded using Modified California (MC) split spoon sampler were converted in the final logs to equivalent SPT blow counts using appropriate modification factors suggested by Burmister (1948), i.e., a factor of 0.65 with 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, a key to the boring symbols, and select laboratory test results are included in Appendix A. Ground surface elevations indicated on the soil boring logs were estimated to the nearest foot using Google Earth.

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 either presented on the boring logs, and/or are 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 16 samples to measure the in-place dry density and moisture content of the subsurface materials. These properties provide information for evaluating the physical characteristics of the subsurface soils. Test results are shown on the boring logs.

Atterberg Limits (ASTM D4318 and CT204) - Atterberg Limits tests were performed on two samples of cohesive soils encountered at the site. Liquid Limit, Plastic Limit, and Plasticity Index are useful in the classification and characterization of the engineering properties of soil, and help to evaluate the expansive characteristics of the soil and determine the USCS soil classification. Test results are presented in Appendix B, and on the boring logs.

Particle Size Analysis (Wet and Dry Sieve) and Hydrometer (ASTM D422, D1140, and CT202) - Sieve analysis tests were conducted on three selected samples to determine 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 in Appendix B.

R-Value Test (ASTM D2844 and CT301) – One R-value test was performed on a sample collected from Boring B-8 to evaluate the subgrade soil strength for pavement designs. The R-value test was conducted on bulk sample of near-surface materials collected from cuttings generated from Boring B-8 at depth from two to seven feet. Test result is presented in Appendix B, and on the pertinent boring log.

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 UBC, CBC, and IBC.

Natural Occurring Asbestos Test (EPA Method 600/R-93-116, Visual Area Estimation) – A sample of soil from Boring B-5 at depth was visually analyzed to determine a presence of the Natural Occurring Asbestos (NOA) in the soil at the project site. No NOA was detected.

3.0 GEOLOGIC AND SEISMIC OVERVIEW

3.1 Geologic Evolution of the Northern Coast Ranges

The subject site is located within the tectonically active and geologically complex northern Coast Ranges, which have been shaped by continuous deformation resulting from tectonic plate convergence (subduction) beginning in the Jurassic period (about 145 million years ago). Eastward thrusting of the oceanic plate beneath the continental plate resulted in the accretion of materials onto the continental plate. These accreted materials now largely comprise the Coast Ranges. The dominant tectonic structures formed during this time include generally east-dipping thrust and reverse faults.

Beginning in the Cenozoic time period (about 25 to 30 million years ago), the tectonics along the California coast changed to a transpressional regime and right-lateral strike-slip displacements as well as thrusting were superimposed on the earlier structures resulting in the formation of northwest-trending, near-vertical faults comprising the San Andreas Fault System. The northern Coast Ranges were segmented into a series of tectonic blocks separated by major faults including the San Andreas, Rodgers Creek, Hayward, and Calaveras. The project site is situated between the active Rodgers Creek and San Andreas faults, but no known active faults with Holocene movement (last 11,000 years) lie within the limits of the site. The site is not mapped within an Alquist-Priolo Earthquake Fault Zone.

3.2 Regional Geologic Setting

The site is located in the central portion of the northern Coast Ranges geomorphic province of California. The Coast Ranges extend from the Transverse Ranges in southern California to the Oregon border and are comprised of a northwest-trending series of mountain ranges and intervening valleys that reflect the overall structural grain of the province. The ranges consist of a variably thick veneer of Cenozoic volcanic and sedimentary deposits overlying a Mesozoic basement of sedimentary, metamorphic, and basic igneous Franciscan Formation and primarily marine sedimentary rocks of the Great Valley Sequence. East-dipping sedimentary rocks of the Coast Ranges are flanked on the east by sedimentary rocks of the Great Valley geomorphic province (Page, 1966).

The Franciscan complex is composed of weakly to strongly metamorphosed greywacke (sandstone), argillite, limestone, basalt, serpentinite, chert and other rocks. This rock was accreted onto the edge of the North American continent during the long period of active subduction of the Pacific Plate beneath the North American Plate. The formation is derived from Jurassic oceanic crust and pelagic deposits that are overlain by Late Jurassic to Late

Cretaceous sedimentary deposits. Metamorphic grade in this rock is highly variable which reflects the complicated history of the Franciscan.

Since the late Cenozoic era, subduction has been replaced by transform faulting along faults of the San Andreas System. There has also been major climate change and dramatic rising and lowering of sea level. Due to the complex geologic history of the area there is a wide variety of volcanic rocks and sedimentary rocks of varying metamorphic grade to be found in the region. These units are often juxtaposed along ancient fault contacts and the structure is complicated by not only ancient deformation, but by active fault deformation. Imprinted on this geology is the drainage pattern of the Santa Rosa Creek Watershed.

More specifically, the site is located in the north of San Francisco Bay and shown on the map by Dibblee (2005) as being underlain by Holocene alluvial deposits (Qha). The mapped geologic units in the site vicinity per USGS are shown on *Figure 3, Site Vicinity Geologic Map*.

3.3 Local Geologic Setting

The project site is situated near the Gallinas Valley within the foothills of Marin County, between San Pablo Bay and the Pacific Ocean. The highland areas of the county are chiefly comprised of rocks of the Franciscan Formation, which underlie roughly half of southeastern Marin County. The Graymer et al. (2006) geologic map, as presented on Figure 3, shows the site to be located at contact boundary line between Holocene-age alluvium and Cretaceous-age Franciscan Complex Sedimentary rocks. Underlying bedrock, exposed in the hills to the south and northwest of the site, consist of late-Cretaceous to Paleocene-age Franciscan Complex mélange, a mixture of small to large masses of various rock types, principally greywacke sandstone, greenstone (basalt), chert and serpentine in a matrix of sheared or pulverized rock material. Rice et al. (1976) shows similar mapping as Graymer et al. On the southeast side of the project site, alluvium deposits were mapped by Graymer et al.

The mapped geologic units at the site are shown on Figure 2 and Figure 3. The site is shown to be underlain partly by Cretaceous-age Franciscan Complex Sedimentary rocks (Kfs) and partly by quaternary Holocene alluvium (Qha) deposits overlying the bedrock at depth.

3.4 Regional Faulting and Tectonics

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 site is located in a seismically active region that has experienced periodic, large magnitude earthquakes during historic times. This seismic activity appears to be largely controlled by displacement between the Pacific and North American crustal plates, separated by the San Andreas Fault zone located approximately 10.5 miles west of the site and the Hayward Fault zone located approximately 8.5 miles east of the site. The fault location map is presented in *Figure 4, Regional Fault Map*. This plate displacement produced regional strain that is concentrated along major faults of the San Andreas Fault System including the San Andreas, Hayward, and Rodgers Creek faults in this area. The proposed buildings should be designed to resist deformation produced by such tectonic activity applying the relevant seismic design parameters as recommended in Section 6.2, Table 6.2.1 based on the current California Building Code.

3.5 Historic Seismicity

As discussed above, the San Francisco Bay Area is subject to a high level of seismic activity. Within the period of 1800 to 2016 there were an estimated 20 earthquakes equal or exceeding an earthquake magnitude of 6.0 within a 100 mile radius of the site, seven exceeding 6.5, four exceeding 7.0, and one is exceeding 7.5.

Of the major earthquakes known to have affected the site, the 1906 San Francisco earthquake caused the strongest shaking resulting in enormous destruction and loss of life in San Francisco and portions of the Bay Area. The South San Francisco area was sparsely developed at that time and no records of fatalities or building collapse were noted. The 1957 Daly City earthquake and the 1989 Loma Prieta earthquake also caused strong shaking but relatively minor property damage and no fatalities or injuries.

4.0 SUBSURFACE CONDITIONS

4.1 Subsurface Soil Conditions

Multi-Story Garage Site - During our subsurface exploration program, we investigated the subsurface soils and evaluated soil conditions to maximum depths of about 27 feet, the maximum depth possible without utilizing rock coring machine. From our collected data, we conclude that where explored, the areas of the proposed parking garage is generally underlain by shallow surficial stiff to very stiff Sandy Clay/ Clay underneath that is a soft to medium, weathered bedrock. The depths of rock vary from near surface in the northwest corner of the proposed site to a maximum depth of about 20 feet in the southwest corner. The upper 9.5 feet in Boring B-2 appear to be soft to medium dense bedrock however, below that depth appeared to be hard and auger refusal was encountered. The upper five feet of soils in Boring B-1 appear to be undocumented fill.

Our interpretations of the subsurface geologic and soil conditions are presented in *Figure 5a, Schematic Geologic Cross-Sections A-A'*, and *Figure 5b, Schematic Geologic Cross-Section B-B'*. Additional details of materials encountered in the exploratory borings including laboratory test results, are included in the boring logs in Appendix A, and laboratory test summaries are presented in Appendix B.

Roadway Site (Lucas Valley Road) - We performed three soil borings along the Lucas Valley Road to determine existing pavement thicknesses and subsurface soil strength (R-Value test) as requested by the Project Civil Engineer. We investigated the subsurface soils and evaluated soil conditions to maximum depths of about 10 feet. Based on the exploratory borings, the thickness of the asphalt concrete (AC) was measured to be two to four-inches, and aggregate bases were measured to be 22" to 23", however 50" of aggregate base or similar material was encountered in Boring B-7. The soils underneath these layers were moist, stiff to very stiff sandy clay or medium dense clayey sand. Bedrock was encountered in Borings B-6 & B7 at about seven feet below the existing pavement surface. More specific details about the surface soils/bedrock are presented in Appendix A.

4.2 Groundwater Conditions

No groundwater was encountered in any of the borings during our field exploration at this location. Due to the topographical and geological formation of the project site, it is very unlikely that we would encounter groundwater at this location. Review of the Department of Water Resources "Groundwater Levels Data Library" shows no any groundwater information in and around the project site. Based on this information we anticipated the historical groundwater level should be very deep at the site.

Groundwater levels can vary in response to time of year, variations in seasonal rainfall, well pumping, irrigation, and alterations to site drainage. A detailed investigation of local groundwater conditions was not performed and is beyond the scope of this study.

4.3 Corrosion Testing

A sample collected from the upper two to four feet of the soil profile at Boring B-4 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 table.

Table 4.3.1: Summary of Corrosion Test Results

Soil Description	Sample Depth (feet)	Sulfate (mg/kg)	Chloride (mg/kg)	Redox (mV)	Resistivity (ohm-cm)	Sulfide	pH
Yellowish Brown Sandy Clay	2 - 4	20	6	408	5652	Negative	6.4

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 4.3.1 in American Concrete Institute (ACI) 318, as referenced by the CBC, provides the following evaluation criteria:

Table 4.3.2: 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 20 mg/kg or 0.002% 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 6 mg/kg or 0.0006% 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 4.3.3.

Table 4.3.3: 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 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, lateral spreading, dynamic settlement, fault ground rupture and fault creep, dam inundation, and tsunamis and seiches. The site is not necessarily impacted by all of these potential seismic hazards. Nonetheless, 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 the major active Bay Area faults, particularly the nearby San Andreas Fault (approximately 10.5 miles from the site) or Hayward-Rodgers Creek Fault (approximately 8.5 miles from the site).

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 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. However, because of the higher inter-granular 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. Lateral spreading can cause ground cracking and settlement.

The site is not currently within the State of California Special Study Zones. However, per the liquefaction hazard susceptible map regenerated by Association of Bay Area Governments (ABAG) based on the USGS and William Lettis & Associates, the site is located at the boundary between a zone of very low to moderate liquefaction potential (see attached *Figure 6, Liquefaction Susceptibility Map*). The site is predominantly underlain by a layer of shallow very stiff to hard sandy clay to highly weathered, fractured bedrock. Groundwater was not encountered in any of the borings (up to a depth of 27 feet below the existing ground surface). Based on the information collected during the field investigation, laboratory test results, dense nature of the soils encountered in the borings within the project site, and great depth to groundwater, we do not anticipate potential for liquefaction at the project site.

5.1.3 Dynamic Compaction (Settlement)

Dynamic compaction is a phenomenon where loose, sandy soil located above the water table densified from vibratory loading, typically from seismic shaking or vibratory equipment. The site is generally underlain by upper few layers of very stiff sandy clay/clayey sand and weathered bedrock down below. Dynamic compaction at this site should not be an issue.

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 Act, the California Geological Survey established boundary zones or Earthquake Fault Zone 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). The closest Earthquake Fault Zone is associated with Hayward Fault, located about 8.5 miles from the site (see Regional Fault Map, Figure 4). Since the site is not within an Earthquake Fault Zone, the potential for fault ground rupture and fault creep hazards are judged to be very low.

5.2 Other Hazards

Potential geologic hazards other than those caused by a seismic event generally include ground failure and subsidence, landslides, expansive and collapsible soils, flooding, and soil erosion. These are discussed and evaluated in the following sections.

5.2.1 Ground Cracking and Subsidence

Withdrawal of groundwater and other fluids (i.e. petroleum and the extraction of natural gas) from beneath the surface has been linked to large-scale land subsidence and associated cracking on the ground surface. Other causes for ground cracking and subsidence include the oxidation and resultant compaction of peat beds, the decline of groundwater levels and consequent compaction of aquifers, hydrocompaction and subsequent settlement of alluvial deposits above the water table from irrigation, or a combination of any of these causes. Due to the absence of any of these factors, the potential for subsidence or related ground cracking is considered low.

5.2.2 Consolidation Settlement

Consolidation is the densification of soil into a more dense arrangement from additional loading, such as new fills or foundations. 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. Consolidation of soft and loose soil layers and lenses can cause settlement of the ground surface or buildings. Based on testing in the field, laboratory testing, and type of soils and depth of groundwater level, as well as due to the proposed structure being located at cut site, potential for consolidation settlement is low. However, due to the structure being partly on bedrock and partly on native soils it could cause differential settlement which should be remediated by providing a layer of an engineered fill as a cushion. Details about a remediation will be discussed more in building pad preparation section.

5.2.3 Expansive and Collapsible Soils

The result of the laboratory testing performed on representative sample of the near-surface soils indicated low plasticity soils. Hence, there should not be an issue of expansive soil at this site.

The subsurface deposits encountered during the drilling program generally consisted of stiff to hard or medium dense to very dense clay, clayey sand, and bedrock. Collapsible soils are loose chemically bonded fine sandy and silty soils that have been laid down by the action of flowing water, usually in alluvial fan deposits. Terrace deposits and fluvial deposits can also contain collapsible soil deposits. The soil particles are usually bound together with a mineral precipitate. The loose structure is maintained in the soil until a load is imposed on the soil and water is introduced. The water breaks down the inter-particle bonds and the newly imposed loading densifies the soil. These types of soils are not present at this site. Therefore, the potential for collapsible soils underlying the site is considered to be low for this project site.

5.2.4 Flooding

The site is located in an area of minimal flooding hazard. FIRM (2009) has mapped the site vicinity as Zone X, areas determined to be outside the 0.2% annual chance floodplain, see attached FEMA figure regenerated by ABAG, *Figure*

7, FEMA Flood Hazard Map. Based on the site's proximity to this Flooding Zone, the topography of the proposed construction site can be considered to have a low hazard potential for seasonal flooding. Determining the flood hazard of the site is beyond the scope of this study or our expertise, and a flood specialist should be contacted if a more in-depth flooding analysis is desired.

5.2.5 Landsliding

The site is not yet evaluated by CGS sources as being located within an existing landslide or potential landslide area. Based on the USGS rainfall induced landslide map and surrounding topography, the site is not considered prone to potential landslide. See attached USGS Landslide Map regenerated by ABAG, *Figure 8, Existing Landslide Map*.

5.2.6 Soil Erosion

Present construction techniques and agency requirements have provisions to limit soil erosion and resultant siltation during construction. These measures will reduce the potential for soil erosion at the site during the various construction phases. Long-term erosion at the site will be reduced by landscaping and hardscape areas, such as parking lots and walkways, designed with appropriate surface drainage facilities.

5.2.7 Naturally Occurring Asbestos (NOA)

The borings did not encounter any soils which are a concern for potential asbestos hazard. Due to the site being considered in a Franciscan complex formation area which is considered potential for a high NOA occurring site, a soil sample collected from Boring B-5@2 was tested for the Asbestos content. The test result at the location indicated no asbestos detected. However, test results may vary from location to location and at various depths. The test report is attached in Appendix B.

5.2.8 Other Geologic Hazards

Due to the site's location, subsurface soil conditions, groundwater levels and land use factors, the site is not subject to the potential geologic hazards of loss of mineral resources, volcanism, tsunamis, seiches, dam failure inundation, cyclic softening of soils or loss of unique geologic features.

6.0 CONCLUSIONS AND RECOMMENDATIONS

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

6.1 Conclusions

The site is considered geotechnically suitable 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 that need to be addressed at this site are summarized below.

Hydrological Soil Group – The majority of the upper three to five feet of the surficial soils appeared to be Lean Clay (CL) to Sandy Clay (CS). Therefore, based on Hydrological Soil Groups, Appendix A, the soils at the site are more appropriate to classify as Group D.

Seismic Ground Shaking – The site is located within a seismically active region. As a minimum, the building design for should consider the effects of seismic activity in accordance with the latest edition of the California Building Code (CBC-2013).

Cut and Fill – The proposed site for the parking structure is partly in a Cut-and-Fill area. Significant amount of cut will be required on the west side and gradually decrease to the east. Cutting could range from 25 feet in the west to a minimum of four feet in the east. We anticipated the required engineered fill of about two to four feet.

Rock Cutting – Due to presence of very shallow bedrock at the parking structure site, larger quantities of rock excavation will be required during construction. The amount of rock excavation decreases from northwest to southeast. We anticipate approximately 25 feet of rock excavation will be required in the northwest corner of the proposed site and gradually decrease to the southwest corner. We believe about a depth of 10 feet of the bedrock in the northwest can be excavated with regular excavating tools however deeper than 10 feet might need special tools or technique to excavate the bedrock. The contractor should review the boring logs carefully to determine the suitable tools and technique for proper excavation.

Undocumented Fill – The upper approximately five feet of soils at the current proposed multi-story garage structure site appeared to be undocumented fill especially near Boring B-1. However, the anticipated structure is being planned to be placed at a lower elevation than the existing surface, therefore these undocumented fills are anticipated to be removed from the site. Therefore, the existing fill at this site should not impact proposed project development.

Differential Settlement – The parking structure site could experience low to moderate settlements due to the cut-and-fill. Maximum settlement could be up to one inch (1”). The consolidation settlement should be considered during the structure design.

Corrosive Soil – The preliminary corrosion evaluation indicated the test sample was very low corrosive to buried cast and ductile iron pipe. There is not potential for corrosion at the project site.

Winter Construction – If grading occurs in the winter rainy season, appropriate erosion control measures will be required and weatherproofing of the building pads, foundation excavations, and/or pavement areas should be considered. Winter rains may also impact foundation excavations and underground utilities.

Groundwater – Historical groundwater level on the proposed project site is not available or very deep. Groundwater should not be problematic with placement of anticipated shallow foundation. Construction of the shallow conventional foundation may be problematic if foundation work was done during winter rain.

Utility Connections – As a general suggestion, where utility damage during a design seismic event may be an issue, the Structural Engineer may wish to consider utility connections at building perimeters designed for at least one inch (1”) of potential movement in any direction where the utility enters the buildings. This flexibility would help accommodate potential differential movement during a seismic event.

R-Value Test Result – As requested, we collected one soil sample from Boring B-8 at depth from two to seven feet below the existing surface to run R-value test. The R-value was determined to be 12 at 300 psi exudation pressure.

6.2 Seismic Design Parameters

The proposed structures 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 San Francisco Bay region. Based on the subsurface conditions encountered in our borings we judge Site Class “C”, representative of very dense and soft rocks averaged over the uppermost 100 feet of the subsurface profile to be appropriate for this site. For design of the proposed site structures in accordance with the seismic provisions of the CBC 2016 and American Society of Civil Engineers (ASCE) 7-10, the following seismic ground motion values should be used for design.

Table 6.2.1: Seismic Coefficients Based on 2016 CBC (per ASCE 7-10)

Item	Value	2016 CBC Source[R1]	ASCE 7-10 Table/Figure[R2]
Site Class	C	Table 1613A.3.2.	Table 20.3-1
Mapped Spectral Response Accelerations			
Short Period, S_s	1.500 g		Figure 22-1
1-second Period, S_1	0.600 g		Figure 22-2
Site Coefficient, F_a	1.0	Table 1613A.3.3(1)	Table 11.4-1
Site Coefficient, F_v	1.3	Table 1613A.3.3(2)	Table 11.4-2
MCE (S_{MS})	1.500 g	Equation 16A-37	Equation 11.4-1
MCE (S_{M1})	0.780 g	Equation 16A-38	Equation 11.4-2
Design Spectral Response Acceleration			
Short Period, S_{DS}	1.000 g	Equation 16A-39	Equation 11.4-3
1-second Period, S_{D1}	0.520 g	Equation 16A-40	Equation 11.4-4
Peak Ground Acceleration, PGA_M	0.500 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
ASCE 7-15 § 11.6-1 and 11.6-2 indicate that the Seismic Design Category for all Occupancy Categories is “D”.

6.3 Site Grading and Site Preparations

6.3.1 General Grading, Demolition, Preparation, and Drainage

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 Geosphere/CEL prior to starting the clearing and demolition operations at the site.

Site grading for the proposal parking structure is generally anticipated to consist of major cuts and fills required to construct the new proposed structure and to establish new site grades as required. We understand that the floor finished level of the proposed parking structure has been planned at an elevation close to 38 feet (amsl) however floor finished elevation could vary if the proposed structure/parking garage is sloped at 2% which could potentially reduce the amount of bedrock cuttings. We anticipate about 18 feet of cutting/excavation will be required for this project especially in the northwest corner, however the actual amount will depend on the final plan and elevations. The upper approximately 9.5 feet of the existing bedrock is anticipated to be highly weathered however, with increasing depth the excavation will become difficult. The construction contractor should evaluate the subsurface condition carefully in determining the correct technique and tools for excavation of the bedrock during project bidding.

The back slope of the bedrock cutting could provide a maximum of 1:1.5 (H:V) if verified and confirmed by the Geotechnical Engineer during construction phase. Buildings adjacent to slopes shall be set back a sufficient distance from the slope to provide protection from slope drainage, erosion and shallow failures. The building should be setback a minimum of 1/2H from the base of slope or 15-feet whichever is greater where H is the slope height. However where basement or semi basements are constructed, the setback is considered from the top of the ground level adjacent to the building.

The structure site will also require at least two feet of granular non expansive engineered fill approved by the Geotechnical Engineer or his/her representative, as a mitigation to reduce differential settlement due to material transition from bedrock to alluvium soil. Undocumented fill near Boring B-1 on the south side and alluvium deposits near Borings B-4 and B-3 on the southeast and east sides of the project site are anticipated to be high plastic clay/ clayey sand and therefore are not suitable for a backfill. If import fill is required for a backfill for this project it should be non-expansive, having a Plasticity Index of 12 or less, an R-Value greater than 40, and enough fines so the soil can bind together but not more than 20 percent. Imported soils should be free of organic materials and debris, and should not contain rocks or lumps greater than three inches in maximum size. The Geotechnical Engineer should approve imported fill prior to delivery onsite.

We note that the final garage location has not been determined at this time. When the final location has been determined these grading recommendations should be reviewed and updated. We anticipate that differential bearing conditions may occur and either additional remedial grading will be required or deepened footings to provide uniform bearing support to the structure.

There will be minor demolition of existing structures at the project site. Prior to commencement of grading activities, all the existing pavements, foundation remnants, utilities, trees and roots, surface vegetation, organic-laden soils, building materials, existing loose soil, concrete, debris and other deleterious materials should be cleared. Debris resulting from site stripping operations should be removed from the site, unless otherwise permitted by the Geotechnical Engineer.

Excavations resulting from the removal of abandoned underground utilities, or deleterious materials should be cleaned down to firm soil, processed as necessary, and backfilled with engineered fill in accordance with the grading sections of this report. The Geotechnical Engineer's representative should verify the adequacy of site clearing operations during construction, prior to placement of engineered fill.

Existing underground utilities proposed to be abandoned, if present, should be properly grouted, closed, or removed as needed. The extent of removal/abandonment depends on the diameter of the pipe, depth of the pipe, and proximity to buildings and pavement.

Final grading should be designed to provide drainage away from structures and the top of slopes. Soil areas within 10 feet of proposed structures or as applicable from the site condition should slope at a minimum of five percent away from the building. Adjacent concrete hardscape should slope a minimum two percent away from the building. Roof leaders and downspouts should not discharge into landscape areas adjacent to buildings, and should discharge onto paved surfaces sloping away from the structure or into a closed pipe system channeled away from the structure to an approved collector or outfall.

6.3.2 Project Compaction Recommendations

The following table provides the recommended compaction requirements for this project. Not all soils, aggregates and scenarios listed below may be applicable for this project. Specific grading recommendations are discussed individually within applicable sections of this report.

Table 6.3.2.1: Project Compaction Requirements

Description	Min. Percent Relative Compaction (per ASTM D1557)	Percent Above/below Optimum Moisture Content
Fill Areas, Engineered Fill, Onsite Soil	90	+ 3
Fill Areas, Engineered Fill, Select Fill	95	± 3
Building Pads, Onsite Soil – Scarified Subgrade or used as Fill	90	+ 3
Building Pads, Baserock or Select (non-expansive) Engineered Fill	95	± 3
Building Pads – Treated Soil	95	± 3
Concrete Flatwork, Subgrade Soil	90	+ 3
Concrete Flatwork, Baserock	95	± 3
Underground Utility Backfill - Below 3 feet	90	+ 3
Underground Utility Backfill - Upper 3 feet	95	+ 3
AC Pavement – Onsite Subgrades (upper 12 inches)	95	+ 3
AC Pavement – Non-Expansive Subgrades in Traffic Areas (upper 12 inches)	95	± 3
Pavement – Class 2 Aggregate Base Section	95	± 3

6.3.3 Building Pad Grading

After cutting the existing alluvium and/or bedrock at required depths, upper 24 inches of the garage building pad should be a granular non-expansive engineered fill compacted to the project compaction requirements. Before placement of engineered fill, the building pad subgrade soil should be scarified to a depth of at least eight-inches below the existing alluvium soils or placed directly over the bedrock acceptable to the Geotechnical Engineer, moisture conditioned to at least ± 3% within optimum moisture, and compacted to 95% relative compaction determined by ASTM D1557 (Modified Proctor) as required by the project compaction Table 6.3.2.1. If loose or soft soil is encountered, these soils should be removed to expose firm soil and backfilled with engineered fill. Engineered fill should be placed in maximum eight-inch thick, un-compacted lifts. The fill should be moisture conditioned and thoroughly mixed during placement to provide uniformity in each layer. Requirements for a non-expansive select fill layer underlying the building pad are presented in Section 6.3.1.

6.3.4 Grading Flatwork Areas

Areas to receive pavements, if any, should be scarified to a depth of eight inches below existing grade or final subgrade whichever is lower. Scarified areas should be moisture conditioned and compacted. Where required, engineered fill should be placed and compacted to reach design subgrade elevation. Once the compacted pavement subgrade has been reached, it is recommended that baserock in paved and on-grade concrete slab areas be placed immediately after grading to protect the subgrade soil from drying. Alternatively, the subgrade should be kept moist by watering until baserock is placed.

Rubber-tired heavy equipment, such as a full water truck, should be used to proofload exposed subgrade areas where pumping is suspected. Proof loading will determine if the subgrade soil is capable of supporting construction equipment without excessive pumping or rutting.

6.3.5 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 onsite soils may not be feasible. These conditions may be remedied using soil admixtures, such as lime/cement. A 4% mixture of lime based on a soil unit weight of 120 pcf is recommended for planning purposes. 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. 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 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 95 percent relative compaction.

6.4 Utility Trench Construction

6.4.1 Trench Backfilling

Utility trenches may be backfilled with onsite selected soil above the utility bedding and shading materials. If 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. Utility bedding and shading compaction requirements should be in conformance with the requirements of the local agencies having jurisdiction and as recommended by the pipe manufacturers. Jetting of trench backfill is not recommended. Compaction recommendations are presented in Table 6.3.2.1, Project Compaction Recommendations.

Pea gravel, rod mill, or other similar self-compacting material should not be utilized for trench backfill since this material will transmit the shallow perched/groundwater to other locations within the site and potentially beneath the buildings. Additionally, fines may migrate into the voids in the pea gravel or rod mill, which could cause settlement of the ground surface above the trench.

If rain is expected and the trench will remain open, the bottom of the trench may be lined with one to two inches of gravel. This would provide a working surface in the trench bottom. The trench bottom may have to be sloped to a low point to pump the water out of the trench.

6.4.2 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.

Utility trenches should be sealed with concrete, clayey soil, sand-cement slurry, or controlled density fill (CDF) where the utility enters the building under the perimeter foundation. This would reduce the potential for migration of water beneath the building through the shading material in the utility trench.

6.4.3 Pipe Bedding and Shading

Pipe bedding material is placed in the utility trench bottom to provide a uniform surface, a cushion, and protection for the utility pipe. Shading material is placed around the utility pipe after installation and testing to protect the pipe. Bedding and shading material and placement are typically specified by the pipe manufacturer, agency, or project designer. Agency and pipe manufacturer recommendations may supersede our suggestions. These suggestions are intended as guidelines and our opinions based on our experience to provide the most cost-effective method for protecting the utility pipe and surrounding structures. Other geotechnical engineers, agency personnel, contractors, and civil engineers may have different opinions regarding this matter.

Bedding and Shading Material - The bedding and shading material should be the same material to simplify construction. The material should be clean, uniformly graded, fine to medium grained sand. It is suggested that bedding and shading material contain less than three percent fines with 100 percent passing the No. 8 sieve. Coarse sand, angular gravel or baserock should be avoided since this type of shading material may bridge when backfilling around the pipe, possibly creating voids, and may be too stiff as bedding material. Open graded gravel should be avoided for shading since this material contains voids, and the surrounding soil could wash into the voids, potentially causing future ground settlement. However, open graded gravel may be required for bedding material when water is entering the trench. This would provide a stable working surface and a drainage path to a sump pit in the trench for water in the trench. The maximum size for bedding material should be limited to about ¾ -inch.

Bedding Material Placement - The thickness of the bedding material should be minimized to reduce the amount of trench excavation, soil export, and imported bedding material. Two to three inches for pipes less than eight-inches in diameter and about four to six inches for larger pipes are suggested. Bedding for very large diameter pipes are typically controlled by the pipe manufacturer. Compaction is not required for thin layers of bedding material. The pipe needs to be able to set into the bedding, and walking on a thin layer of bedding material should sufficiently compact the sand. Rounded gravel may be unstable during construction, but once the pipe and shading material is in place, the rounded gravel will be confined and stable.

Shading Material Placement – Jetting is not typically recommended since the type of shading material is unknown when preparing the geotechnical report and agencies typically do not permit jetting. If the sand contains fines or if the sand is well graded, jetting will not work. Additionally, if too much water is used during jetting, this could create a wet and unstable condition. However, clean, uniformly graded and fine to medium sand can be placed by jetting. The shading material should be able to flow around and under the utility pipe during placement. Some compactive effort along the sides of the pipe should be made by the contractor to consolidate the shading material around the pipe. A minimum thickness of about six-inches of shading material should be placed over the pipe to protect the pipe from compaction of the soil above the shading material. The contractor should provide some compactive effort to densify the shading material above the pipe. Relative compaction testing is not usually performed on the shading material. However, the contractor is ultimately responsible for the integrity of the utility pipe.

6.5 Temporary Excavation Slopes and Shoring

Construction of the below grade parking structure will require either temporary excavation slopes or shoring to construct the building foundations. The Contractor should incorporate all appropriate requirements of OSHA/Cal OSHA into the design of any temporary construction slopes or 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 subsurface materials in the areas of the site where excavation may take place may be assumed to consist of stiff Sandy Clay/Lean Clay categorized as OSHA Type A with temporary slope inclination of no steeper than 1:1 (horizontal: vertical). However in some areas subsurface soils are soft rock which should be able to maintain a temporary slope inclination of no steeper than 2:1 (horizontal: vertical) or vice versa in case of hard rock, 1:2 or vertical. The type of slope material and temporary construction slopes should be confirmed during construction by a competent engineering geologist responsible to the grading contractor.

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.

If use of temporary slopes is not feasible, a temporary shoring system may be required to protect adjacent properties/structures against undesirable movement or deflection. The Contractor, or his specialty shoring subcontractor, should design and install temporary shoring. Possible shoring systems include soldier beam and lagging walls with or without tiebacks or sheet piles.

We recommend that the geotechnical and structural engineers review any temporary shoring plan to confirm compliance with the anticipated soil conditions encountered at the site. In addition, we recommend that the geotechnical engineer's representative observe the installation of the temporary shoring systems. The Contractor should incorporate all appropriate requirements of OSHA into the design of the temporary shoring system.

6.6 Building Foundations Recommendations

6.6.1 Shallow Spread Foundations

We believe that the proposed parking garage structure can be supported on conventional isolated and/or continuous spread footings bearing on bedrock or improved subgrade soils/engineered fill. The improved subgrade soils or engineered fill should extend a minimum of three feet laterally from the edge of the footings. Footings should be founded a minimum of 36 inches below lowest adjacent finished grade in case of improved subgrade soils/engineered fills or 24 inches below at the bedrock areas. Continuous footings should have a minimum width of at least 24 inches, and isolated column footings should have a minimum width of three feet. 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. Deepening of the footings may be required due to differential bearing conditions. Field conditions may dictate the over-excavation of the footings to a uniform bearing material. Lean concrete can be used as a suitable fill material to fill over-excavated footings in the event that deepening is required. All footings must be observed by the geotechnical engineer prior to reinforcing steel and concrete placement.

For the design of footings bearing within tested and approved improved subgrade soil or engineered fill we recommend 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 including transient, respectively, from the estimated ultimate bearing pressure.

Table 6.6.1.1: Allowable Bearing Pressures for Spread Footings

Load Condition	Allowable Bearing Pressure (psf)
Dead Load	2,300
Dead plus Live Loads	3,500
Total Loads (including wind or seismic)	4,600

Note: *Allowable bearing pressures can be increased by 1.5 times if all the footings rest on firm bedrock.*

Geosphere personnel should be retained to observe, test, and confirm that foundations are prepared as recommended in the Geotechnical Report. Geotechnical Engineer should approve the foundation prior to placement of formwork and reinforcing steel. 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 (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 soil and facilitate removal of water and slough if rainwater fills the excavations.

If site preparation and foundation observation services are conducted as outlined in the Geotechnical Study report, vertical settlement is not expected to exceed more than one inch for footings bearing within the materials described in the report and designed to the aforementioned allowable bearing pressures. Differential settlement across the structure is not expected to exceed more than a 1/2 inch with columns spaced at 30 feet.

6.6.2 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 allowable 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. 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 friction between the bottom of a slab-on-grade floor and the underlying soil should not be utilized to resist lateral forces.

6.6.3 Interior Slabs-on-Grade

We understand that the structure is being planned to construct with the interior slab-on-grade floor slabs. These slabs are subject to moisture variation, treatment of the slab subgrade will be required due to shallow historic ground water and surrounding expansive soils. For non-structural concrete slab-on-grade floors we recommend a minimum of five-inch thick slab. However, actual thickness of the slab should be determined by the Structure Engineer. Additionally, on-grade concrete floor slabs should be underlain by a minimum of four inches of Class II AB over a minimum 12 inch thickness of select, non-expansive fill or native soil.

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 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”), or to Class B (Griffolyn Type 85, Moistop Ultra B, or equivalent) may be used in place of a Class C retarder.

The vapor retarder or barrier should be placed directly under the slab. A capillary rock layer or rock cushion is not required if a Class A barrier is 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. The thickness of the capillary rock layer and sand, if either or both are used, may be considered to comprise part of the recommended non-expansive fill layer underlying the floor slab.

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.

6.7 Retaining or Below Grade Walls

We believe that the proposed garage may require soil retaining structures. We anticipated that there will not be any other loads that will influence the wall other than lateral soil pressure, and the backfill soils behind the wall will be either level/flat or sloped. We provide the following pressures on the wall for the design of the retaining structures.

The active pressure for soils given below assumes the backfill behind the retaining wall is granular soils or sands with proper sub-drainage behind the walls. If the walls are not provided with a drainage system then hydrostatic pressure should be added, which significantly increases the lateral pressure on the wall. An allowable bearing capacity of 3,500 pounds/square-foot can be used for designing the wall foundation. Any surcharges should be considered if imposed behind the top of the wall within a zone established by a 45 degree projection upward from the bottom of the wall footing.

6.7.1 Lateral Earth Pressures

For granular soils or sand above any free water surface, recommended equivalent fluid pressures for foundation elements are presented on the Table below.

Table 6.7.1.1: Recommended Equivalent Fluid Pressures

Lateral Load Condition	Backfill Soil	Slope Behind the Wall			
		Flat/Level	3:1 (H:V)	2:1 (H:V)	1.5:1 (H:V)
Active Or Unrestrained Wall	Sand	30	37	42	52
Active + Seismic	Sand	90	-	-	-
At Rest Or Restrained Wall	Sand	50	50	50	50
Coefficient of Friction between Concrete/Native Soil	0.35				
Coefficient of Friction between Concrete/ Firm Bedrock	0.45				

Walls subjected to surcharge loads should be designed for an additional uniform lateral pressure equal to 0.3 times the anticipated surcharge load for unrestrained walls, and 0.4 times the anticipated surcharge load for restrained walls. A seismic increment is not required for site walls retaining less than six feet.

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. Below-grade walls such as elevator pit walls can be designed to accommodate an additional hydrostatic pressure increment.

Retaining or below-grade 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. Compaction of each lift adjacent to walls should be accomplished with hand-operated tampers or other lightweight compactors. Over compaction may cause excessive lateral earth pressures which could result in wall movement. If heavy compaction equipment is used, the walls should be appropriately designed to withstand loads exerted by the heavy equipment, and/or temporarily braced.

6.7.2 Retaining Wall Drainage

To reduce hydrostatic loading on retaining walls, a subsurface drain system should be placed behind the wall. 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 either weep holes or 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. 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.

Geosphere personnel should be retained to observe and evaluate that foundation excavations terminate in soils suitable for the design bearing pressure. If unsuitable soil is present, the excavation should be extended until suitable material is encountered. Unsuitable soil or fill removal should also extend at least eight inches beyond the foundation edge for each 12-inch thickness of unsuitable soil being removed. The material removed should be replaced with an approved engineered fill/granular soil, placed and compacted.

6.8 Building Clearance from Slopes (if applicable)

6.8.1 Building Clearance from Ascending Slope

In general, any building located below an ascending slope shall be set a sufficient distance from the slope to provide protection from slope drainage, erosion and shallow failures. For slopes of 1:1 or flatter, the outer face of the structure should be at least one half the height of the slope or 15 feet away from the toe of slope, whichever is less.

6.8.2 Building Clearance from Descending Slopes

Foundations adjacent to descending slope surfaces shall be founded in firm materials with an embedment and setback from the slope surface sufficient to provide vertical and lateral support for the foundation without detrimental settlement. For slopes flatter than 1:1, the outer face of the exterior building footing should be at least the smaller of one third of the height of the slope, or 40 feet lateral distance from the top of adjacent slope.

6.9 Plan Review

We recommend that GEOSPHERE be provided the opportunity to review the final project plans prior to construction. The purpose of this review is to assess the general compliance of the plans with the recommendations provided in this report and confirm the incorporation of these recommendations into the project plans and specifications.

6.10 Observation and Testing During Construction

We recommend that GEOSPHERE be retained to provide observation and testing services during site preparation, mass grading, underground 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 in the event that subsurface conditions differ from those anticipated prior to the start of construction.

7.0 VALIDITY OF REPORT

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

8.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

The recommendations of this report are based upon the soil and conditions encountered in the borings. If variations or undesirable conditions are encountered during construction, GEOSPHERE 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 GEOSPHERE 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 GEOSPHERE 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 GEOSPHERE 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 GEOSPHERE 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.

9.0 REFERENCES

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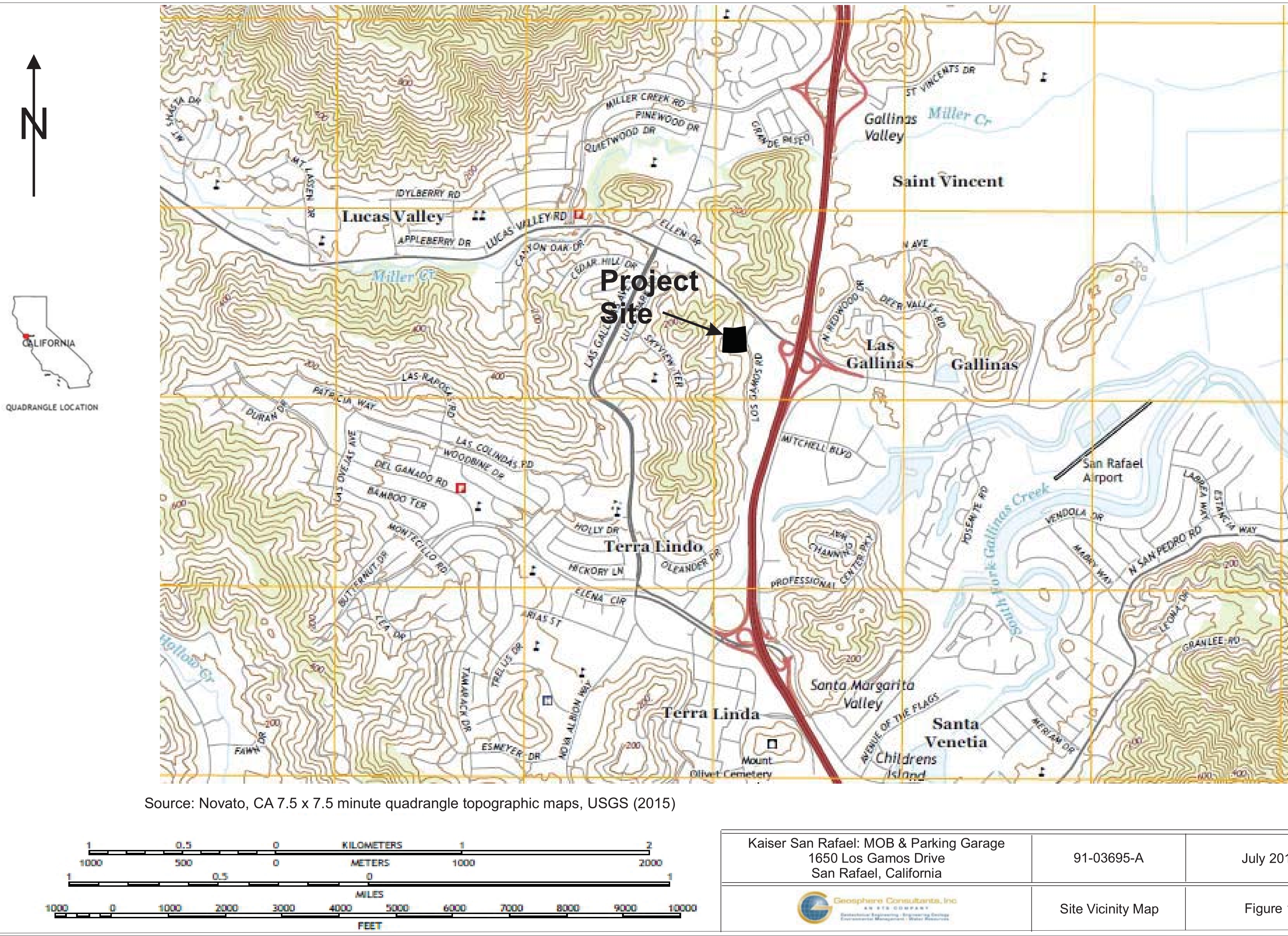
FIGURES



Source: Novato, CA 7.5 x 7.5 minute quadrangle topographic maps, USGS (2015)

Kaiser San Rafael: MOB & Parking Garage 1650 Los Gamos Drive San Rafael, California	91-03695-A	July 2016
Geosphere Consultants, Inc	Site Vicinity Map	Figure 1

0 90 ft - Approximate Scale

- Approximate Boring Location

Qha - Alluvium, Holocene

Kfs - Fransican Complex sedimentary rocks, Cretaceous

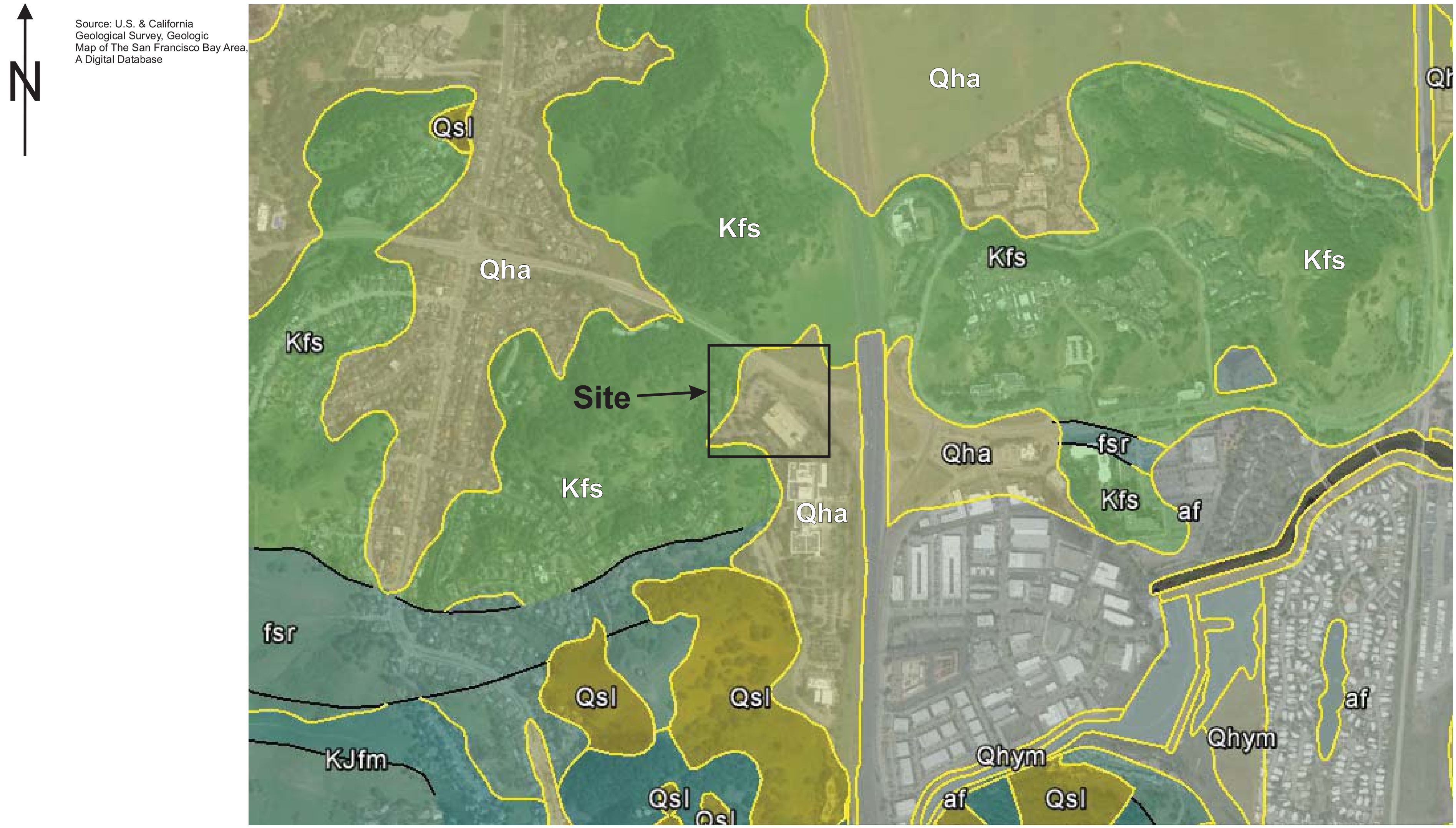
- Project Site

- Proposed Mitigation Measure

- Geologic Cross-Section

Base Map Reference: Google Earth

Kaiser San Rafael: MOB & Parking Garage 1650 Los Gamos Drive San Rafael, California	91-03695-A	February 2017
Geosphere Consultants, Inc.	Site Plan and Site Geology Map	Figure 2



Qha - Alluvium, Holocene
Qsl - Hillslope Deposits, Quaternary
Qhym - Mud Deposits, late Holocene
Kfs - Fransican Complex sedimentary rocks, Cretaceous
fsr - Fransican Complex melange, Eocene, Paleocent, &/or Late Cretaceous
af - artificial fill

Geologic faults
Geologic contacts

Kaiser San Rafael: MOB & Parking Garage 1650 Los Gamos Drive San Rafael, California	91-03695-A	February 2017
Geosphere Consultants, Inc	Site Vicinity Geologic Map	Figure 3

2010 FAULT ACTIVITY MAP OF CALIFORNIA

California Geological Survey, Geologic Data Map No. 6

Compilation and Interpretation by: Charles W. Jennings and William A. Bryant

Graphics by: Milind Patel, Ellen Sander, Jim Thompson, Barbara Wanish and Milton Fonseca

Explanation

Fault traces on land are indicated by solid lines where well located, by dashed lines where approximately located or inferred, and by dotted lines where concealed by younger rocks or by lakes or bays. Fault traces are queried where continuation or existence is uncertain.

FAULT CLASSIFICATION COLOR CODE
(Indicating Recency of Movement)

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 (>1.6Million Y/O) or fault without recognized Quaternary displacement.

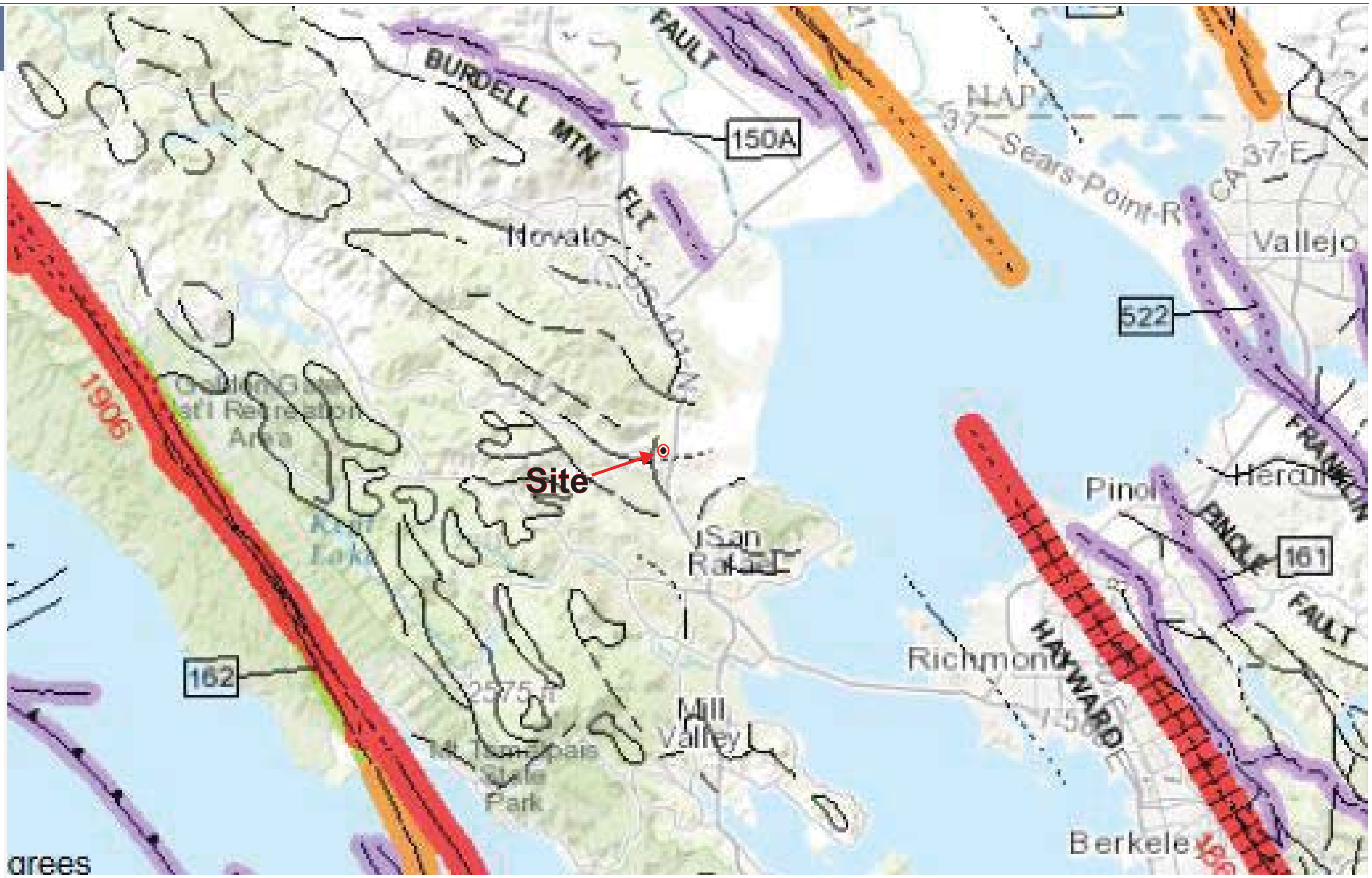
ADDITIONAL FAULT SYMBOLS

Bar and ball on downthrown side (relative or apparent).

Arrow along fault indicates relative or apparent direction of lateral movement.

Arrow on fault indicates direction of dip.

Low angle fault (barbs on upper plate).



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

Kaiser San Rafael: MOB & Parking Garage 1650 Los Gamos Drive San Rafael, California	91-03695-A	July 2016
Geosphere Consultants, Inc	Regional Fault Map	Figure 4

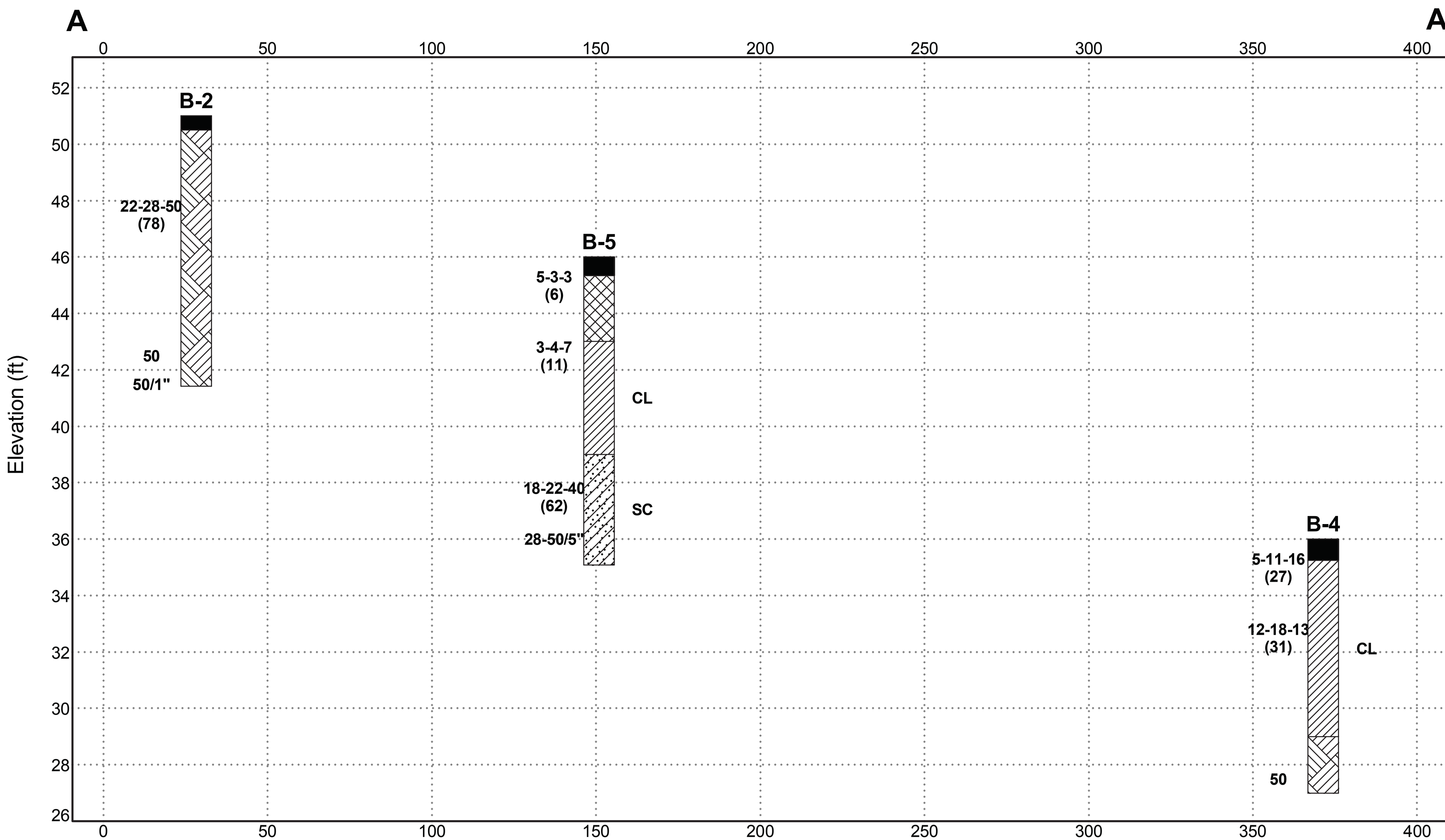
SUBSURFACE DIAGRAM

CLIENT National Facilities Services - Kaiser Permanente

PROJECT NUMBER 91-03695-A

PROJECT NAME Multi-Story Parking Garage Structure Addition

PROJECT LOCATION 1650 Los Gamos Drive, San Rafael, CA 94903



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

Schematic Geologic Cross-Section A-A' | Figure 5a

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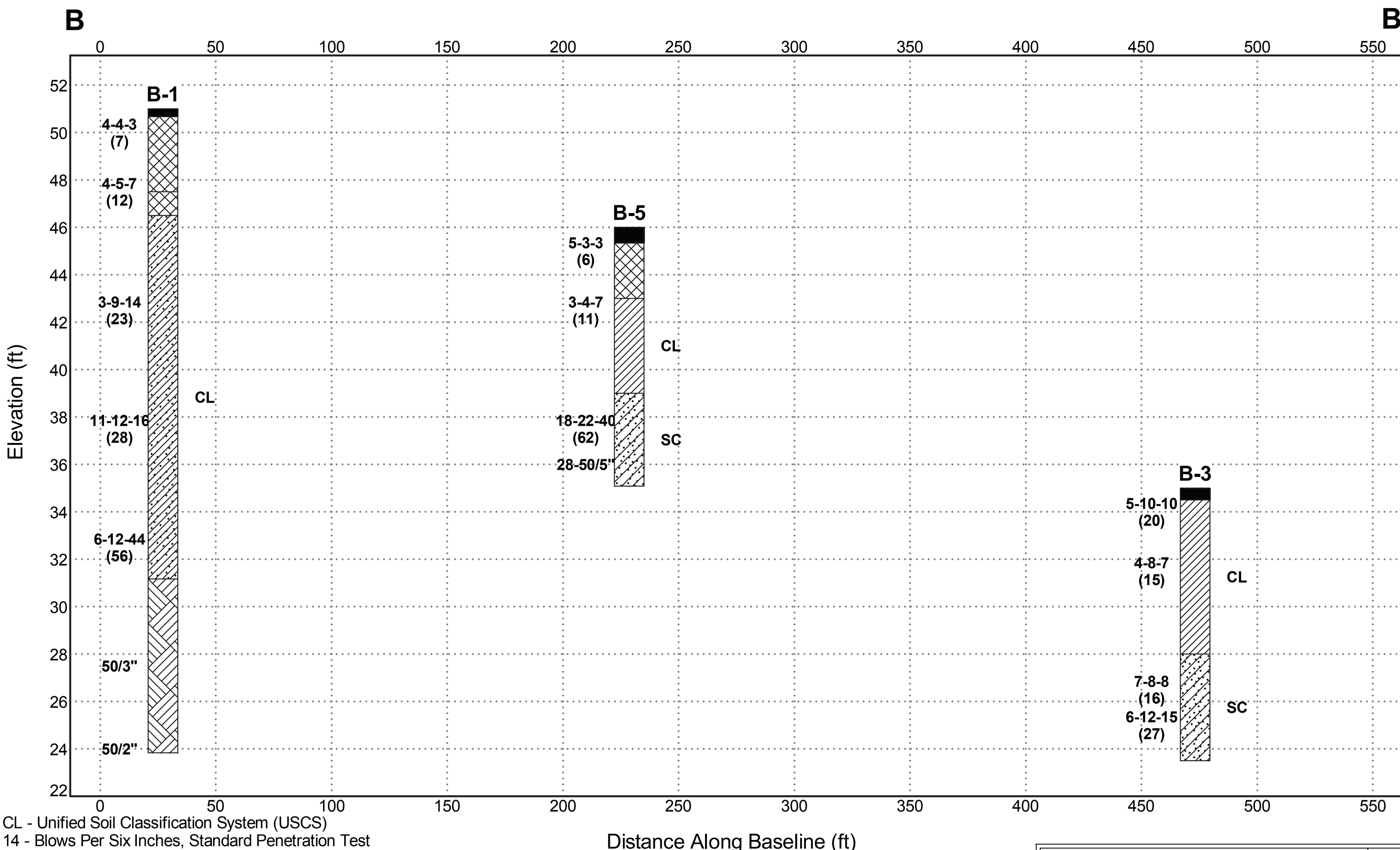
SUBSURFACE DIAGRAM

CLIENT National Facilities Services - Kaiser Permanente

PROJECT NUMBER 91-03695-A

PROJECT NAME Multi-Story Parking Garage Structure Addition

PROJECT LOCATION 1650 Los Gamos Drive, San Rafael, CA 94903



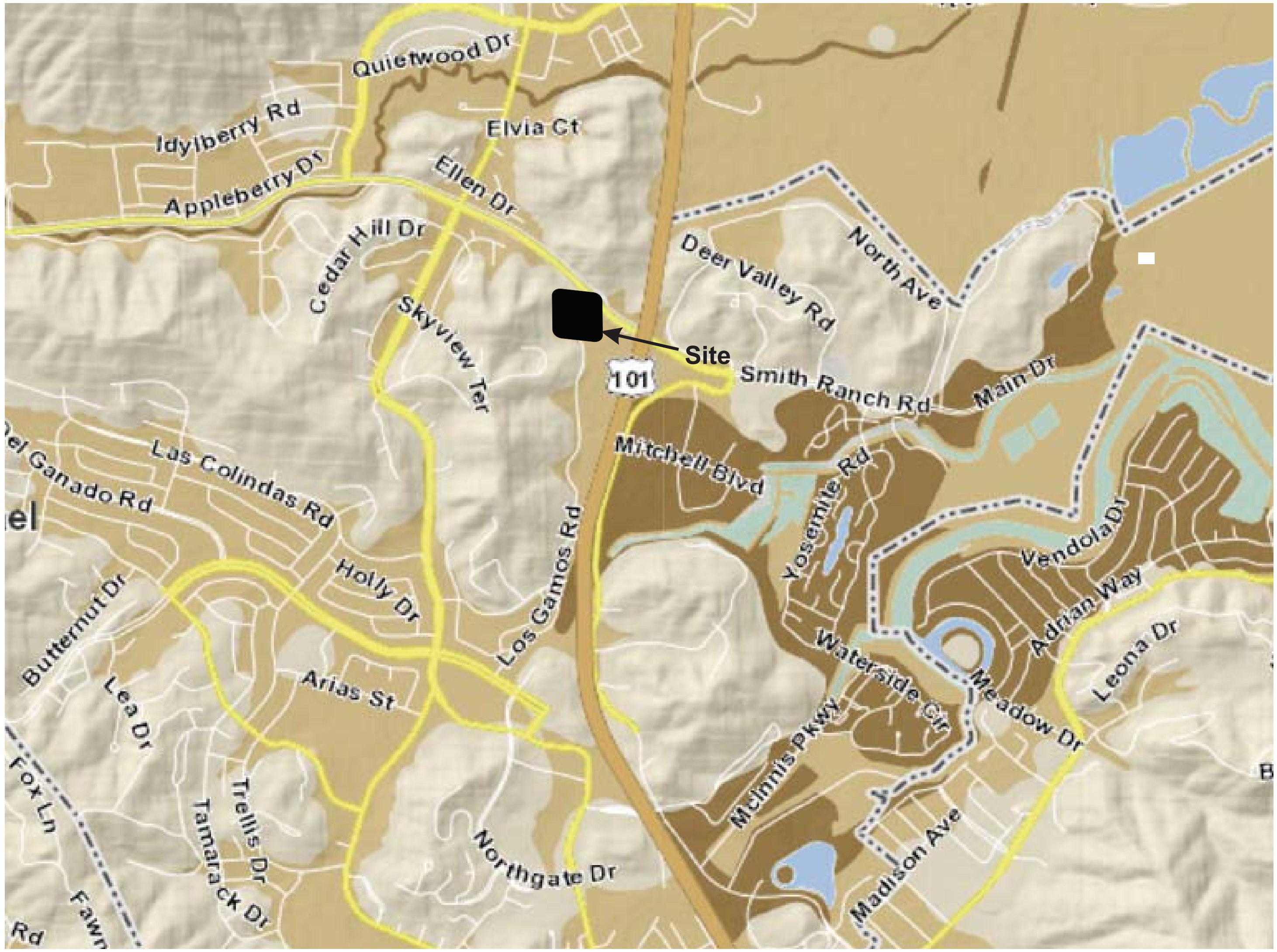
CL - Unified Soil Classification System (USCS)
14 - Blows Per Six Inches, Standard Penetration Test
(21) - N Value

Schematic Geologic Cross-Section B-B'	Figure 5b

Liquefaction Susceptibility

Liquefaction Susceptibility Hazard

- Very High Susceptibility
- High Susceptibility
- Moderate Susceptibility
- Low Susceptibility
- Very Low Susceptibility



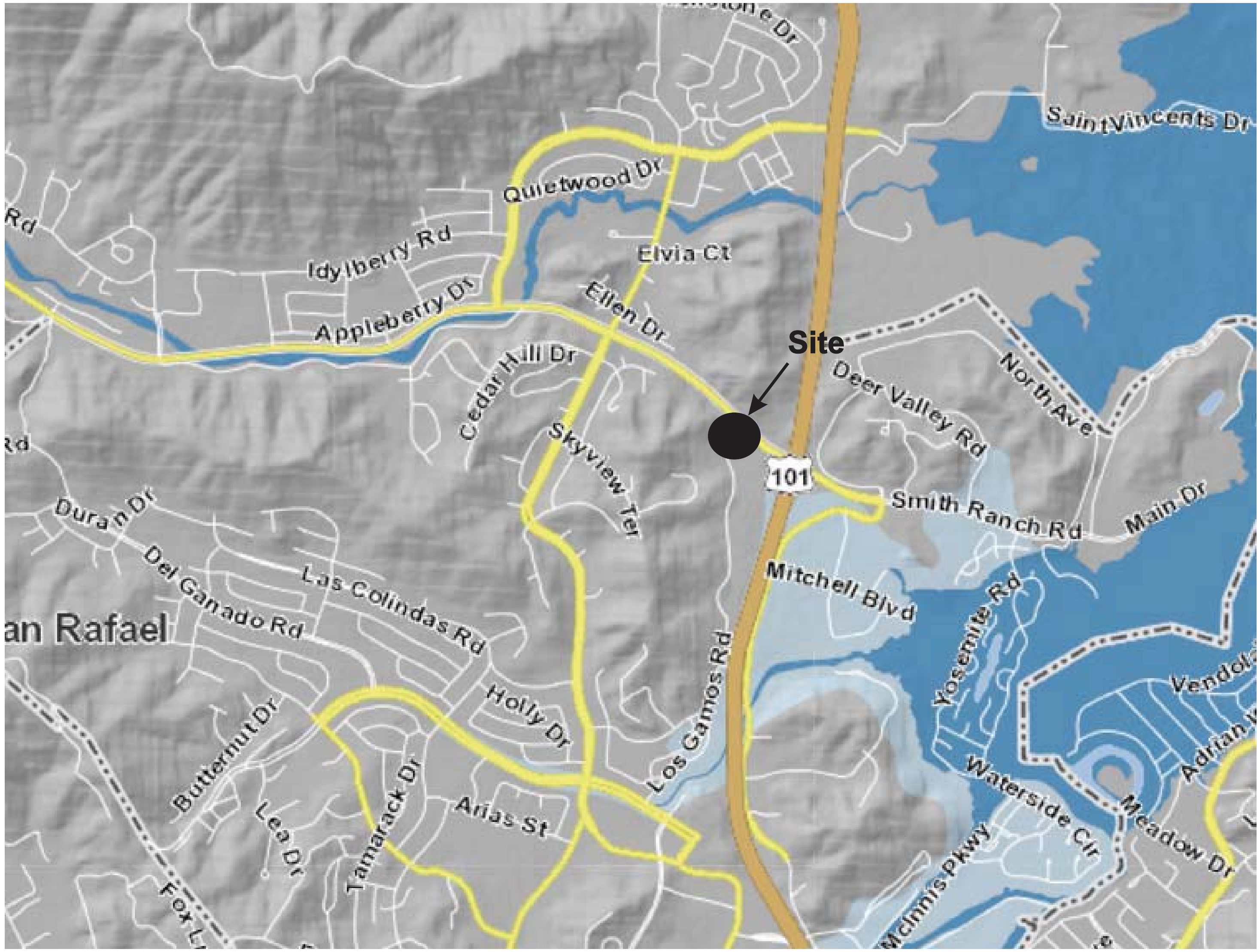
Base Map Reference: ABAG Liquefaction Susceptibility Map

Kaiser San Rafael: MOB & Parking Garage 1650 Los Gamos Drive San Rafael, California	91-03695-A	February 2017
Geosphere Consultants, Inc.	Liquefaction Susceptibility Map	Figure 6

FEMA Flood Zones - 100 & 500 Year Floods

FEMA Flood Zone

- A
- A99
- AE
- AH
- AO
- V
- VE
- X500
- X500C
- XL
- X
- D



Base Map Reference: ABAG Liquefaction Susceptibility Map

Kaiser San Rafael: MOB & Parking Garage 1650 Los Gamos Drive San Rafael, California	91-03695-A	February 2016
Geosphere Consultants, Inc	Flood Hazard Map	Figure 7

San Francisco Bay Area Hazards

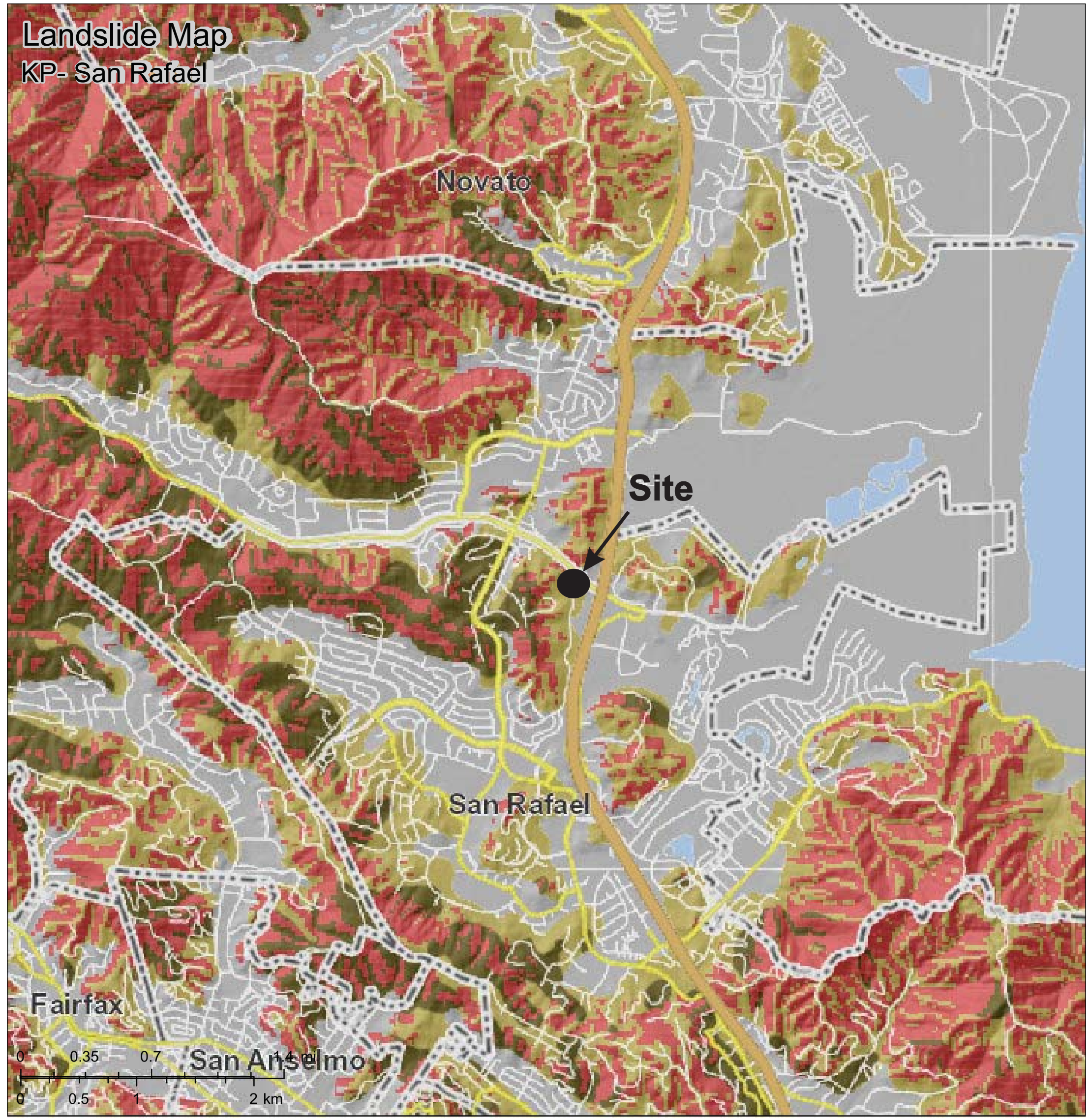
Legend

- Potential Debris Flow Source
- Mostly Landslide
- Many Landslides
- Few Landslides
- Very Few Landslides
- Surficial Deposits
- Area Not Yet Evaluated
- Earthquake-Induced Landslide Hazard Zone
- Mapping Planned
- Mapping in Progress

This map is intended for planning only and is not intended to be site specific. Rather, it depicts the general risk within neighborhoods and the relative risk from community to community.

ASSOCIATION OF BAY AREA GOVERNMENTS RESILIENCE PROGRAM

July 22, 2016 ABAG GIS



Kaiser San Rafael: MOB & Parking Garage 1650 Los Gamos Drive San Rafael, California	91-03695-A	February 2017
Geosphere Consultants, Inc	Existing Landslide Map	Figure 8

APPENDIX A

Key to Boring Log Symbols
Boring Logs

UNIFIED SOIL CLASSIFICATION (ASTM D-2487)						Legend
Material Types	Criteria for Assigning Soil Group Names			Group Symbol	Soil Group Names	
Coarse Grained Soils	Gravels >50% of Coarse Fraction Passes on No. 4 Sieve	Clean Gravels <5% Fines	Cu≥4 and 1≤Cc≤3	GW	Well-Graded Gravel	
			Cu<4 and/or [Cc<1 or Cc>3]	GP	Poorly-Graded Gravel	
		Gravels with Fines >12% Fines	Fines Classify as ML or MH	GM	Silty Gravel	
>50% Retained on No. 200 Sieve			Fines Classify as CL or CH	GC	Clayey Gravel	
	Sands ≥50% of Coarse Fraction Passes on No. 4 Sieve	Clean Sands <5% Fines	Cu≥6 and 1≤Cc≤3	SW	Well-Graded Sand	
			Cu<6 and/or [Cc<1 or Cc>3]	SP	Poorly-Graded Sand	
		Sands and Fines >12% Fines	Fines Classify as ML or MH	SM	Silty Sand	
			Fines Classify as CL or CH	SC	Clayey Sand	
Fine Grained Soils	Silts and Clays Liquid Limits<50	Inorganic	PI>7 and Plots≥"A" Line	CL	Lean Clay	
			PI<4 and Plots<"A" Line	ML	Silt	
		Organic	LL (Oven Dried)/LL(Not Dried <0.75)	OL	Organic Silt	
≥50% Passes No. 200 Sieve	Silts and Clays Liquid Limits≥50	Inorganic	PI Plots≥"A" Line	CH	Fat Clay	
			PI Plots<"A" Line	MH	Elastic Silt	
		Organic	LL (Oven Dried)/LL(Not Dried <0.75)	OH	Organic Clay	
Highly Organic Soils		Primarily Organic Matter, Dark in Color and Organic Odor		PT	Peat	

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

SOIL MOISTURE	
DESCRIPTOR	DESCRIPTION
Dry	Dry of Standard Proctor Optimum
Damp	Sand Dry
Moist	Near Standard Proctor Optimum
Wet	Wet of Satandard Proctor Optimum
Saturated	Free Water in Sample

PARTICLES 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

Grab Bulk Sample

Standard Penetration Test

2.5 Inch Modified California

Shelby Tube

No Recovery

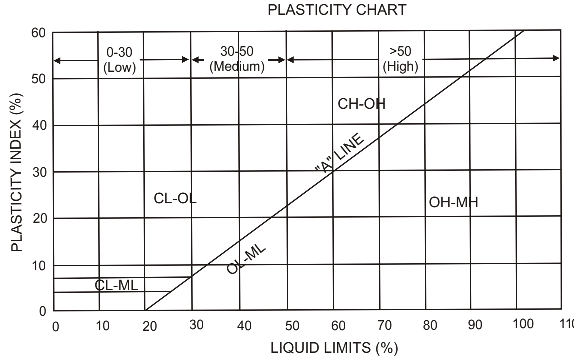
Initial Water Level Reading

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.

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

PLASTICITY CHART



CU -Consolidated Undrained triaxial test completed. Refer to laboratory results
DS - Results of Direct Shear test in terms of total cohesion (C, KSF) or effective cohesion and friction angles (C', KSF and degrees)
LL - Liquid Limit
PI - Plasticity Index
PP - Pocket Penetrometer test
TV - Torvane Shear Test results in terms of undrained shear strength (KSF)
UC - Unconfined Compression test results in terms of undrained shear strength (KSF)
#200 - Percent passing number 200 sieve
Cu - Coefficient of Uniformity
Cc - Coefficient of Concavity

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 that will be identified in the report or on the project site plan. The location and elevation of borings should be considered accurate only to the degree implied by the method used.

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 time and under conditions stated on the boring logs. This data has been reviewed and interpretations have been made in the text of this report. However, it must be noted that fluctuations in the level of the groundwater may occur due to variations in rainfall, tides, temperature and other factors at the time measurements were made.

4. The boring logs and attached data should only be used in accordance with the report.

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KEY TO EXPLORATORY BORING LOGS

BORING NUMBER B-1

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CLIENT National Facilities Services - Kaiser Permanente
PROJECT NAME Multi-Story Parking Garage Structure Addition
PROJECT NUMBER 91-03695-A
PROJECT LOCATION 1650 Los Gamos Drive, San Rafael, CA 94903
DATE STARTED 8/26/16 **COMPLETED** 8/26/16
GROUND ELEVATION 51 ft **HOLE SIZE** 4 inches
DRILLING CONTRACTOR Geo-Ex Subsurface Exploration
GROUND WATER LEVELS:
DRILLING METHOD Solid Flight CME-75
AT TIME OF DRILLING ---
LOGGED BY AL **CHECKED BY** RS
AT END OF DRILLING ---
NOTES
AFTER DRILLING --- No groundwater encountered.

DEPTH (ft)	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	PLASTIC LIMIT	PLASTICITY INDEX	FINES CONTENT (%)
0	**2" AC and 2" AB**										
	SANDY CLAY: Med stiff, black, moist, organics, trace sand & fine gravel. [Fill]	MC 1-8		4-4-3 (7)		107	19				
5	**LEAN CLAY**: Stiff, dark brown, moist, organics, trace sand & fine gravel. [Fill]	MC 1-1		4-5-7 (12)	1.0						
	(CL) **SANDY CLAY**: Stiff, redish brown, trace fine gravel. med to high plasticity.										
10	becomes very stiff, increased sand content.	MC 1-2		3-9-14 (23)		115	17				
15	up to 1" gravel, seams of grey clay.	MC 1-3		11-12-16 (28)							
20		SPT 1-4		6-12-44 (56)							
	CLAYSTONE: Gray-brown, highly weathered, weak, firm.										
25		SPT 1-5		50/3"							
	Bottom of borehole at 27.2 feet.	SPT 1-6		50/2"							

BORING NUMBER B-2

Geosphere Consultants, Inc.
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CLIENT National Facilities Services - Kaiser Permanente
PROJECT NAME Multi-Story Parking Garage Structure Addition
PROJECT NUMBER 91-03695-A
PROJECT LOCATION 1650 Los Gamos Drive, San Rafael, CA 94903
DATE STARTED 8/26/16 **COMPLETED** 8/26/16
GROUND ELEVATION 51 ft **HOLE SIZE** 4 inches
DRILLING CONTRACTOR Geo-Ex Subsurface Exploration
GROUND WATER LEVELS:
DRILLING METHOD Solid Flight CME-75
AT TIME OF DRILLING ---
LOGGED BY AL **CHECKED BY** RS
AT END OF DRILLING ---
NOTES
AFTER DRILLING --- No groundwater encountered.

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		**3" AC and 3" AB**										
		CLAYSTONE : Gray-brown, highly weathered, extremely fractured, soft, friable to weak.										
5			SPT 2-1		22-28-50 (78)							
			SPT 2-2		50							
		becomes weak and firm.	SPT 2-3		50/1"							
		Bottom of borehole at 9.6 feet.										

BORING NUMBER B-3

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CLIENT National Facilities Services - Kaiser Permanente

PROJECT NAME Multi-Story Parking Garage Structure Addition

PROJECT NUMBER 91-03695-A

PROJECT LOCATION 1650 Los Gamos Drive, San Rafael, CA 94903

DATE STARTED 7/21/16 **COMPLETED** 7/21/16

GROUND ELEVATION 35 ft **HOLE SIZE** 4 inches

DRILLING CONTRACTOR Clear Heart Drilling Inc.

GROUND WATER LEVELS:

DRILLING METHOD Solid Flight CME-75

AT TIME OF DRILLING ---

LOGGED BY AL **CHECKED BY** RS

AT END OF DRILLING ---

NOTES

AFTER DRILLING --- No groundwater encountered.

DEPTH (ft)	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	PLASTIC LIMIT	PLASTICITY INDEX	FINES CONTENT (%)
0	**2" AC and 4" AB**										
	(CL) **LEAN CLAY** : Very stiff, brown, dry, mottled. *modcal has no liner*	MC 3-1		5-10-10 (20)	>4.5						
5	contains organics, trace gravel	MC 3-2		4-8-7 (15)		101	12				
	(SC) **CLAYEY SAND** : Med dense, orange brown, dry.										
10		MC 3-3		7-8-8 (16)		114	14				48
	increased sand content, large piece of organic/root?, bedrock fragments at the end.	SPT 3-4		6-12-15 (27)							

Bottom of borehole at 11.5 feet.

BORING NUMBER B-4

Geosphere Consultants, Inc.
AN ETS COMPANY
Geotechnical Engineering · Engineering Geology
Environmental Management · Water Resources

CLIENT National Facilities Services - Kaiser Permanente
PROJECT NAME Multi-Story Parking Garage Structure Addition
PROJECT NUMBER 91-03695-A
PROJECT LOCATION 1650 Los Gamos Drive, San Rafael, CA 94903
DATE STARTED 7/21/16 **COMPLETED** 7/21/16
GROUND ELEVATION 36 ft **HOLE SIZE** 4 inches
DRILLING CONTRACTOR Clear Heart Drilling Inc.
GROUND WATER LEVELS:
DRILLING METHOD Solid Flight CME-75
AT TIME OF DRILLING ---
LOGGED BY AL **CHECKED BY** RS
AT END OF DRILLING ---
NOTES
AFTER DRILLING --- No groundwater encountered.

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		**3" AC and 6" AB**										
		(CL) **LEAN CLAY** : Very stiff, redish brown, dry, mottled.	MC 4-1		5-11-16 (27)							
5		becomes redish gray with up to 1.5" of gravel.	MC 4-2		12-18-13 (31)		110	16	43	20	23	66
		CLAYSTONE : Black/gray, highly weathered, mod strong, firm to mod hard.	SPT 4-3		50							

Bottom of borehole at 9.0 feet.

BORING NUMBER B-5

Geosphere Consultants, Inc.
AN ETS COMPANY
Geotechnical Engineering · Engineering Geology
Environmental Management · Water Resources

CLIENT National Facilities Services - Kaiser Permanente
PROJECT NAME Multi-Story Parking Garage Structure Addition
PROJECT NUMBER 91-03695-A
PROJECT LOCATION 1650 Los Gamos Drive, San Rafael, CA 94903
DATE STARTED 7/21/16 **COMPLETED** 7/21/16
GROUND ELEVATION 46 ft **HOLE SIZE** 4 inches
DRILLING CONTRACTOR Clear Heart Drilling Inc.
GROUND WATER LEVELS:
DRILLING METHOD Solid Flight CME-75
AT TIME OF DRILLING ---
LOGGED BY AL **CHECKED BY** RS
AT END OF DRILLING ---
NOTES
AFTER DRILLING --- No groundwater encountered.

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		**2" AC and 6" AB**										
		LEAN CLAY: Med stiff, black, moist.	MC 5-1		5-3-3 (6)	1.0						
		(CL) **LEAN CLAY**: Stiff, redish brown, moist, mottled, seams of grey clay.	MC 5-2		3-4-7 (11)	1.8						
5												
		(SC) **CLAYEY SAND**: Very dense, dry, redish brown, caliche. (50/5" in MC)	MC 5-3		18-22-40 (62)							
10			SPT 5-4		28-50/5"							

Bottom of borehole at 10.9 feet.

BORING NUMBER B-6

Geosphere Consultants, Inc.
AN ETS COMPANY
Geotechnical Engineering · Engineering Geology
Environmental Management · Water Resources

CLIENT National Facilities Services - Kaiser Permanente

PROJECT NAME Multi-Story Parking Garage Structure Addition

PROJECT NUMBER 91-03695-A

PROJECT LOCATION 1650 Los Gamos Drive, San Rafael, CA 94903

DATE STARTED 8/26/16 **COMPLETED** 8/26/16

GROUND ELEVATION 35 ft **HOLE SIZE** 4 inches

DRILLING CONTRACTOR Geo-Ex Subsurface Exploration

GROUND WATER LEVELS:

DRILLING METHOD Solid Flight CME-75

AT TIME OF DRILLING ---

LOGGED BY AL **CHECKED BY** RS

AT END OF DRILLING ---

NOTES

AFTER DRILLING --- No groundwater encountered.

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		**3" AC**										
		23" AB/Claystone fragments	MC 6-1		13-7-5 (12)	1.8			23	15	8	54
		(CLS) **SANDY CLAY**: Stiff, dark brown, moist. [Fill]										
5		(CL) **LEAN CLAY**: Very stiff, reddish brown, moist.	MC 6-2		4-7-11 (18)							
		CLAYSTONE: Highly weathered, friable, soft. 50/3" in MC.	MC 6-3		25-67							

Bottom of borehole at 9.3 feet.

BORING NUMBER B-7

Geosphere Consultants, Inc.
AN ETS COMPANY
Geotechnical Engineering · Engineering Geology
Environmental Management · Water Resources

CLIENT National Facilities Services - Kaiser Permanente

PROJECT NAME Multi-Story Parking Garage Structure Addition

PROJECT NUMBER 91-03695-A

PROJECT LOCATION 1650 Los Gamos Drive, San Rafael, CA 94903

DATE STARTED 8/26/16 **COMPLETED** 8/26/16

GROUND ELEVATION 36 ft **HOLE SIZE** 4 inches

DRILLING CONTRACTOR Geo-Ex Subsurface Exploration

GROUND WATER LEVELS:

DRILLING METHOD Solid Flight CME-75

AT TIME OF DRILLING ---

LOGGED BY AL **CHECKED BY** RS

AT END OF DRILLING ---

NOTES

AFTER DRILLING --- No groundwater encountered.

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		**4" AC** : Med dense, brown, moist, up to 1.5" gravel.										
		50" of AB/Claystone fragments : Med dense, brown, moist, up to 1.5" gravel.	MC 7-1		12-9-16 (25)							
			MC 7-2		8-8-10 (18)							
5		(SPG) **POORLY GRADED SAND WITH GRAVEL** : Med dense, brown, moist, drains rocks, next to water line.										
		CLAYSTONE : Highly weathered, friable, soft. 50/4" in MC.										
			MC 7-3		50							

Bottom of borehole at 8.8 feet.

BORING NUMBER B-8

Geosphere Consultants, Inc.
AN ETS COMPANY
Geotechnical Engineering · Engineering Geology
Environmental Management · Water Resources

CLIENT National Facilities Services - Kaiser Permanente
PROJECT NAME Multi-Story Parking Garage Structure Addition
PROJECT NUMBER 91-03695-A
PROJECT LOCATION 1650 Los Gamos Drive, San Rafael, CA 94903
DATE STARTED 8/26/16 **COMPLETED** 8/26/16
GROUND ELEVATION 32 ft **HOLE SIZE** 4 inches
DRILLING CONTRACTOR Geo-Ex Subsurface Exploration
GROUND WATER LEVELS:
DRILLING METHOD Solid Flight CME-75
AT TIME OF DRILLING ---
LOGGED BY AL **CHECKED BY** RS
AT END OF DRILLING ---
NOTES
AFTER DRILLING --- No groundwater encountered.

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		**2" AC and 22" AB**										
		22" AB	MC 8-1		12-10-9 (19)							
		CLAYEY SAND : Very stiff, grey/dark brown, moist, up to 1.5" gravel.	GB 8-4									
			MC 8-2		3-4-8 (12)	1.0			36	15	21	44
5		(CL) **LEAN CLAY :** Stiff, redish brown, moist.										
10		residual soil, clay stone fragments.	MC 8-3		10-18-20 (38)							

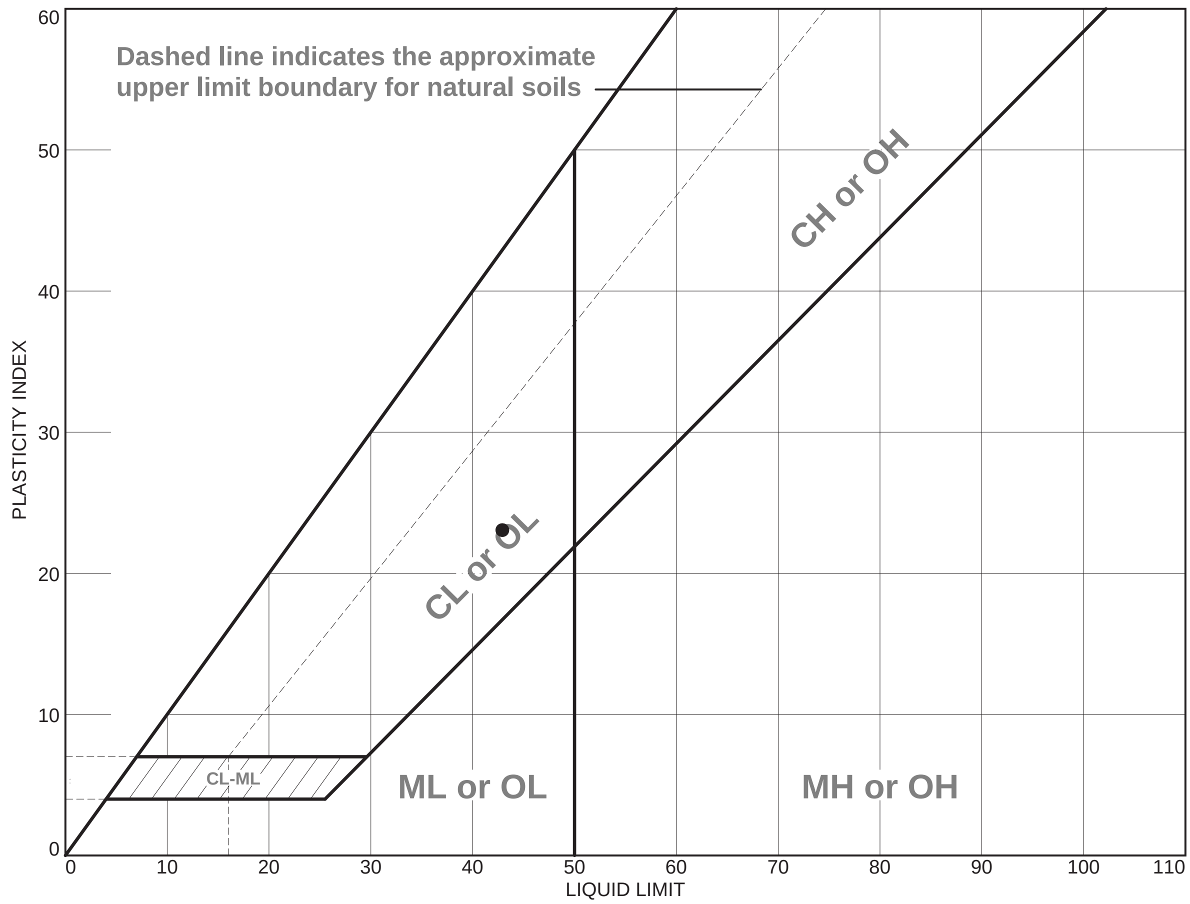
Bottom of borehole at 10.0 feet.

APPENDIX B

LABORATORY TEST RESULTS

Liquid and Plastic Limits Test Report
Sieve Analysis Result
R-Value Test Result
Corrosivity Test Results
Natural Occurring Asbestos Test Result

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Gray & brown sandy LEAN CLAY(CLAYSTONE).Trace fine gravel inclusions.	43	20	23	81.5	66.3	CL

Project No. 91-03695-A **Client:** Geosphere Consultants,Inc.

Project: Kaiser,San Rafael.

● **Depth:** 4.5' **Sample Number:** B-4 4-2

Soil Mechanics Lab

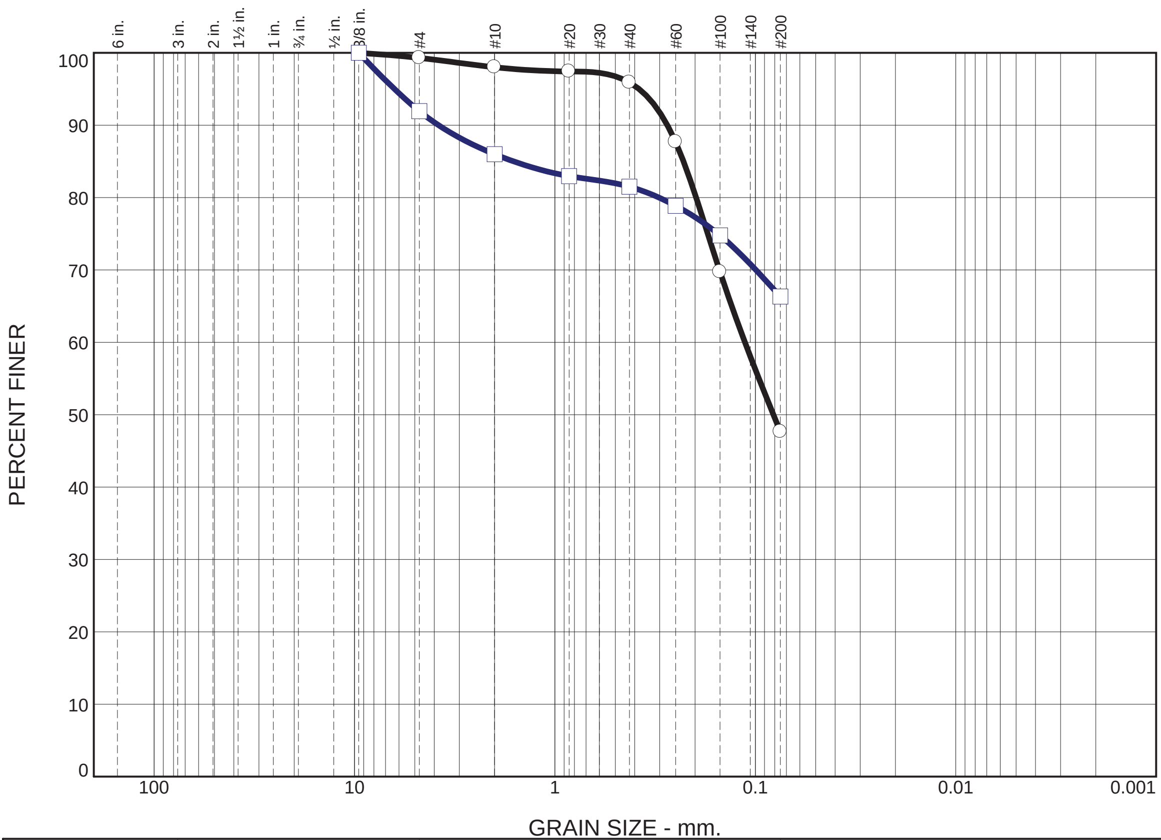
Oakland, California

Remarks:

Plate

Tested By: MA **Checked By:**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay

MATERIAL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○		B-3 3-3	9.5'	Brown clayey fine SAND.	SC
□		B-4 4-2	4.5'	Gray & brown sandy LEAN CLAY(CLAYSTONE).Trace fine	CL
				gravel inclusions.	

Soil Mechanics Lab **Oakland, California**	**Client:** Geosphere Consultants,Inc. **Project:** Kaiser,San Rafael. **Project No.:** 91-03695-A **Plate**

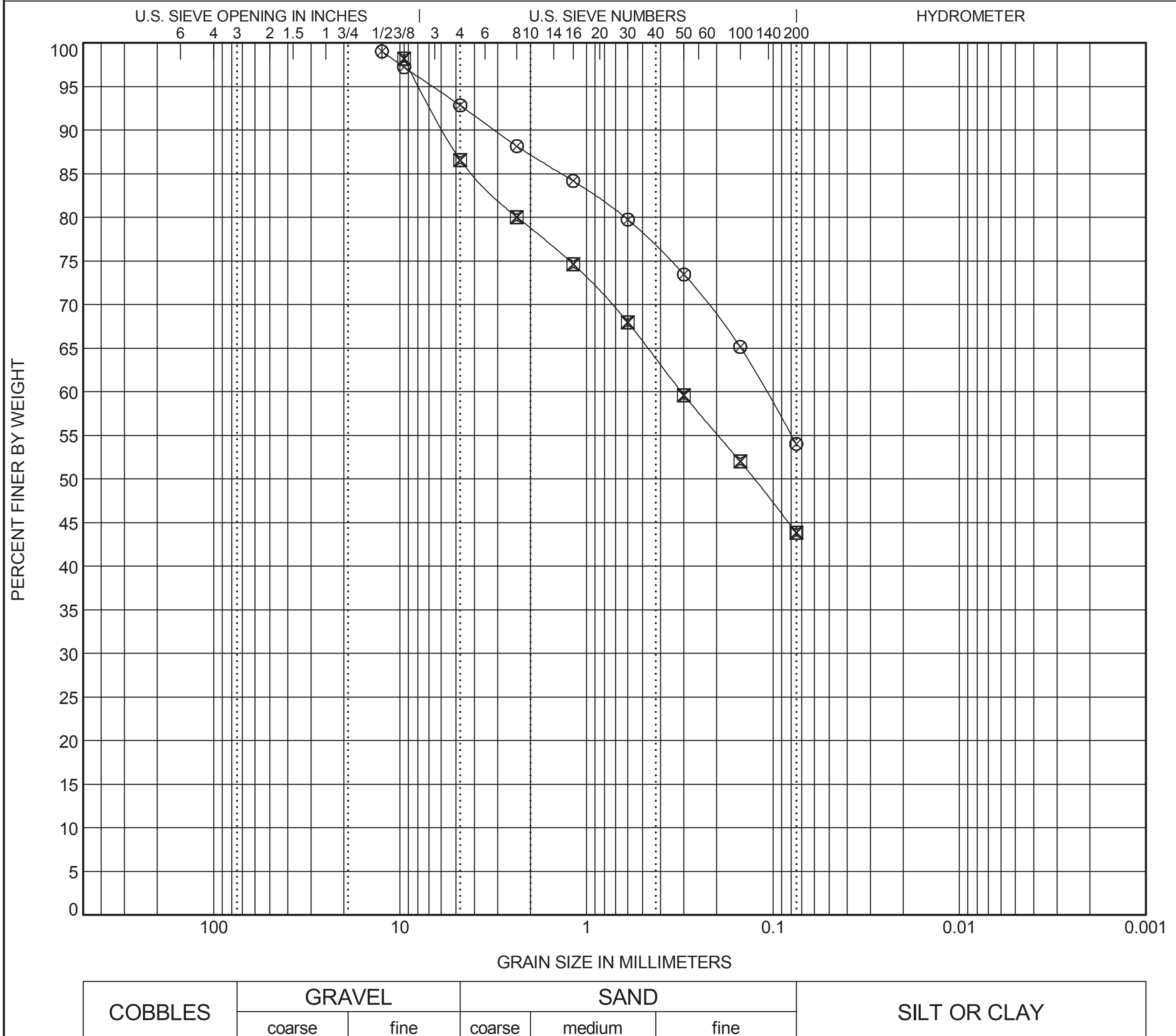
Tested By: MA **Checked By:**

GRAIN SIZE DISTRIBUTION

Geosphere Consultants, Inc.
AN ETS COMPANY
Geotechnical Engineering · Engineering Geology
Environmental Management · Water Resources

2001 Crow Canyon Rd, Ste 210
CA 94583
Telephone: 9253147180
Fax: 9258557140

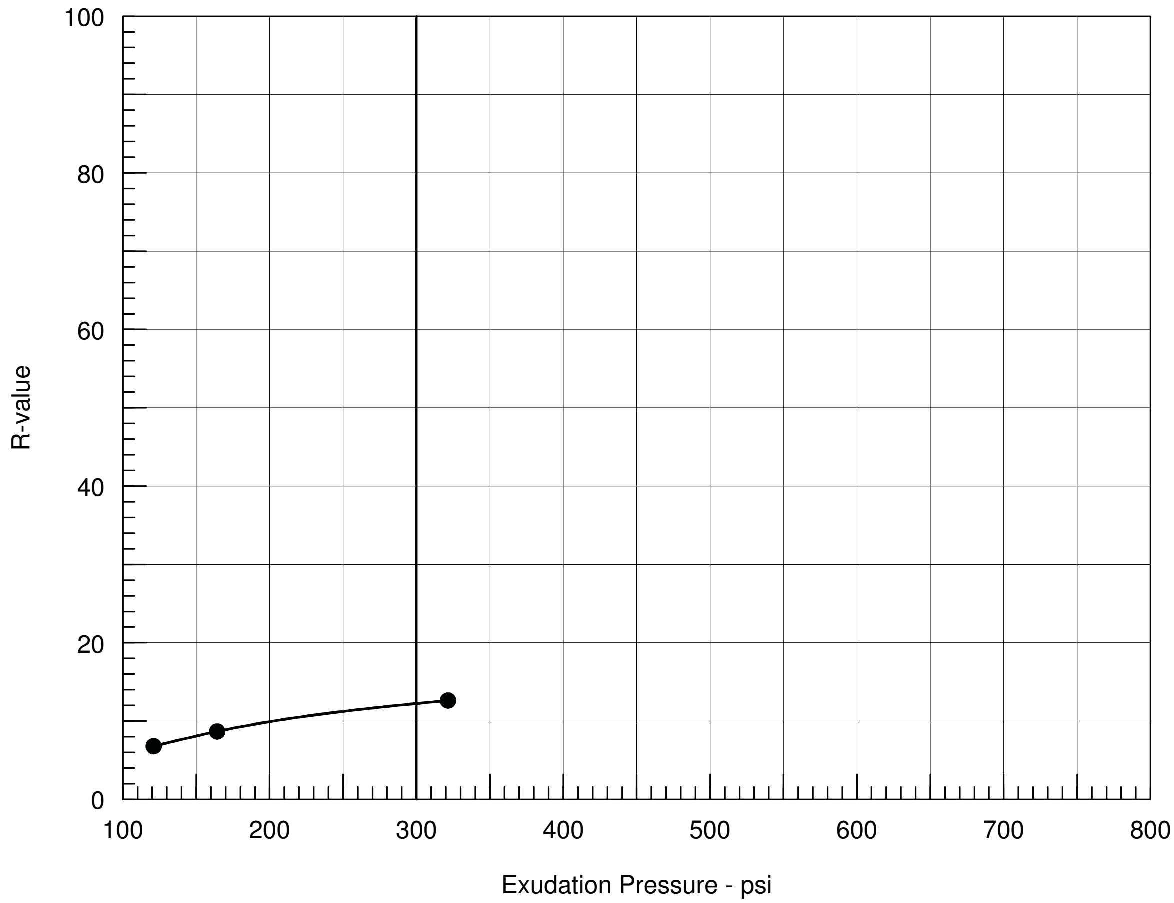
CLIENT National Facilities Services - Kaiser Permanente
PROJECT NAME Multi-Story Parking Garage Structure Addition
PROJECT NUMBER 91-03695-A
PROJECT LOCATION 1650 Los Gamos Drive, San Rafael, CA 94903



Specimen Identification		Classification	LL	PL	PI	Cc	Cu
⊗ **B-6**	**Depth: 2.0'**	**1.0SANDY LEAN CLAY(CL)**	**23**	**15**	**8**		
⊠ **B-8**	**Depth: 4.5'**	**1.0CLAYEY SAND (SC)**	**36**	**15**	**21**		

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
⊗ **B-6**	**Depth: 2.0'**	**12.5**	**0.109**			**6.2**	**38.8**	**54.0**	
⊠ **B-8**	**Depth: 4.5'**	**9.5**	**0.31**			**11.6**	**42.7**	**43.9**	

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - ASTM D 2844

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	80	110.0	17.2	0.00	141	2.54	121	7	7
2	120	113.4	15.3	0.00	137	2.52	164	9	9
3	170	115.0	13.4	0.00	133	2.50	321	13	13

Test Results	Material Description
R-value at 300 psi exudation pressure = 12	Brown Sandy Clay (B-8 2'-7') Sampled on 8/22/16 by A. Lim

Project No.: 9103695A

Project: Kaiser San Rafael - MOB & Future Parking

Location: Kaiser San Rafael/Los Gatos

Sample Number: 10S160901-2

Date: 9/6/2016

Tested by: JLA

Checked by: WY

Remarks:

R-VALUE TEST REPORT

CONSOLIDATED ENGINEERING LABORATORIES -- SAN RAMON, CA

10S160901-2

Corrosivity Tests Summary

CTL # 724-134 **Date:** 8/1/2016 **Tested By:** PJ **Checked:** PJ
Client: Geosphere Consultants **Project:** Kaiser San Rafeal **Proj. No:** 91-03695-A
Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP		Sulfide	Moisture	Soil Visual Description
			As Rec.	Min	Sat.	mg/kg	mg/kg	%		(Redox)		Qualitative	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		E_H (mv)	At Test	by Lead	%	
Boring	Sample, No.	Depth, ft.	ASTM G57	Cal 643	ASTM G57	ASTM D4327	ASTM D4327	ASTM D4327	ASTM G51	ASTM G200	Temp °C	Acetate Paper	ASTM D2216	
B4	-	2-4	-	-	5,652	6	20	0.0020	6.4	408	24	Negative	17.0	Yellowish Brown Sandy CLAY

Forensic Analytical LABORATORIES

Final Report

Bulk Asbestos Analysis

(EPA Method 600/R-93-116, Visual Area Estimation)

McCampbell Analytical, Inc.
Account Payable
1534 Wilow Pass Rd

Pittsburg, CA 94565

Client ID: A31409
Report Number: B225191
Date Received: 07/25/16
Date Analyzed: 08/01/16
Date Printed: 08/01/16
First Reported: 08/01/16

Job ID/Site: 1607A33 - 91-03695-A, Kaiser San Rafael

Date(s) Collected: 07/21/2016

FALI Job ID: A31409
Total Samples Submitted: 1
Total Samples Analyzed: 1

Sample ID	Lab Number	Asbestos Type	Percent in Layer	Asbestos Type	Percent in Layer	Asbestos Type	Percent in Layer
B-5@2	11788406						
Layer: Dark Grey Soil			**ND**				
Total Composite Values of Fibrous Components:		**Asbestos (ND)**					
Cellulose (Trace)							

Tad Thrower

Tad Thrower, Laboratory Supervisor, Hayward Laboratory

Note: Limit of Quantification ('LOQ') = 1%. 'Trace' denotes the presence of asbestos below the LOQ. 'ND' = 'None Detected'.

3777 Depot Road, Suite 409, Hayward, CA 94545 / Telephone: (510) 887-8828 (800) 827-FASI / Fax: (510) 887-4218