MPEG **MILLER PACIFIC ENGINEERING GROUP**

GEOTECHNICAL INVESTIGATION
HILLCREST MULTI-USE DEVELOPMENT
1005, 1020 & 1025 NORTHGATE DRIVE
SAN RAFAEL, CALIFORNIA

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Project 2243.001

Prepared For:
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CERTIFICATION

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1.0 INTRODUCTION

This report summarizes Miller Pacific Engineering Group's (MPEG) Geotechnical Investigation for the planned Hillcrest Multi-use project located at 1005, 1020 & 1025 Northgate Drive in San Rafael, California. A Site Location Map is presented on Figure 1. The purpose of our Geotechnical Investigation is to aid in the design and construction of the project. In accordance with our proposal dated December 15, 2015, we are providing our geotechnical engineering services in three phases: 1) geotechnical investigation, 2) supplemental consultation, and, 3) construction observation and testing. This report completes our Phase 1 services and includes the following:

- Review of readily available published geologic and geotechnical reference data;
- Exploration of subsurface conditions with 8 exploratory soil borings;
- Laboratory testing of selected samples to determine the pertinent engineering properties of the soil layers;
- Evaluation of geologic hazards relevant to site development;
- Development of geotechnical recommendations for the project; and,
- Preparation of this report summarizing our findings.

2.0 PROJECT DESCRIPTION

The final project features have not been determined at this time, however we understand, preliminarily, the proposed improvements include constructing three, multi-story, multi-use structures. The proposed structures will consist commercial use on the first-floor and residential units on the upper two to three stories. Each structure will have one to two-stories of subterranean parking. As shown on the Site Plan, Figure 2, the structure located at 1005 Northgate drive will be situated at the northwest corner of Freitas Parkway and Northgate Drive and will encompass an approximate 40,000-square foot footprint. The proposed structure at 1020 Northgate Drive will be situated at the peak of a hill adjacent to the existing Four Points Sheraton Hotel. This structure will require demolishing the existing conference room portion of the hotel and encompass an approximate 50,000 square-foot footprint. The proposed structure located at 1025 Northgate Drive will be situated to the immediate north of the existing hotel and will encompass an approximate 40,000 square foot footprint.

3.0 SITE CONDITIONS

3.1 Regional Geology

The project site lies within the Coast Ranges geomorphic province of California. Regional topography within the Coast Ranges province is characterized by northwest-southeast trending mountain ridges and intervening valleys that parallel the major geologic structures, including the San Andreas Fault System. The province is also generally characterized by abundant landsliding and erosion, owing in part to its typically high levels of precipitation and seismic activity.

The oldest rocks in the region are the sedimentary, igneous, and metamorphic rocks of the Jurassic-Cretaceous age (190- to 65-million years old) Franciscan Assemblage. Within Marin County, a variety of sedimentary and volcanic rocks of Tertiary (1.8- to 65-million years old) and Quaternary (less than 1.8-million years old) age locally overlie the basement rocks of the Franciscan Assemblage. Tectonic deformation and erosion during late Tertiary and Quaternary time (the last several million years) formed the prominent coastal ridges and intervening valleys typical of the Coast Ranges province. The youngest geologic units in the region are Quaternary-age (last 1.8 million years) sedimentary deposits, including alluvial deposits which partially fill most of the valleys and colluvial deposits which typically blanket the lower portions of surrounding slopes.

As shown on the regional Geologic Map, Figure 3, geologic mapping (Rice, Strand & Smith, 1976), indicates the site is located on a contact between Franciscan Mélange (map symbol, fm) and alluvial deposits (map symbol, Qa). Franciscan Mélange consist of a tectonic mixture of small to large masses of resistant rock types including sandstone, greenstone, chert and serpentinite. Alluvial deposits typically consist of unconsolidated gravel, sand, silt, and clay deposited by streams and rivers.

3.2 Site History

We reviewed various historic aerial photographs to develop a site history of the project area. Aerial photographs from 1946 to the present were reviewed. A summary of the pertinent aerial photographs we reviewed is outlined below:

> January 08, 1950: As shown on Figure 4, the site and surrounding area is undeveloped. Gallinas Creek North Fork is visible meandering on the northwest portion of the site. The un-named hill that occupied a majority of the site was "rounder" in shape than it is today.
>
> July 09, 1963: As shown on Figure 5, the site is still relatively undeveloped. It appears some grading and/or vegetation removal has occurred on the hill top. Highway 101 has now been constructed on the east of the project site, requiring minor cuts in to the southeastern flank of the hillside. It appears a significant amount of fill was placed on the northern end of the hillside, giving the hill an elongated appearance. Additionally, Gallinas Creek North Fork has been re-routed along the western side of the fill and lined with concrete. A large residential development has been constructed to the west of the project site.
>
> July 02, 1970: As shown on Figure 6, the site appears to have been recently developed with the existing multi-story hotel and the convenience store and gas station has also been constructed. The office building located to the south and east of the project site (1050 Northgate Drive) is shown under construction. The surrounding area has been highly developed.
>
> 1970 to Present: The site remains relatively unchanged from 1970 until the present.

3.3 Surface Conditions

The project site is located in northern San Rafael on a small hill to the immediate west of Highway 101. The hill has an elongated shape with shallow northeast and steeper slopes, inclined approximately 2:1 (horizontal:vertical) on the southeastern and northwestern flanks.

This elongated appearance is due to fill placement on the northern end of the hill between the 1950's and 1960's. Highway 101 is located at the base of the southeastern flank and Gallinas Creek North Fork is located on the northwestern. The project site can be further separated into three separate areas, as shown on the Site Plan, Figure 2, and described below:

> *1005 Northgate Drive:* 1005 Northgate Drive is located on the northwestern corner of Northgate Drive and Freitas Parkway and is currently occupied with convenience store and gas station. The southwestern end of the property is covered in a grass lawn area while the northern end is undeveloped. The site slopes from 30-feet at the northeast down to 15-feet to the southwest.
>
> *1020 Northgate Drive:* 1020 Northgate Drive is located atop a hill that has been graded relatively flat to construct the Four Points Sheraton Hotel with elevations ranging from 55- to 60-feet. The structure will be constructed to the west of the existing hotel that is currently occupied by asphalt parking areas and the hotel's conference/ball room structure.
>
> *1025 Northgate Drive:* 1025 Northgate Drive will be constructed to the immediate north of the existing hotel in the existing asphalt parking lot. This area of the site slopes gently to the north with elevations ranging from approximately 45- to 35-feet.

3.3 <u>Field Exploration and Laboratory Testing</u>

We explored subsurface conditions in the general vicinity of the planned improvements on December 28th, 29th, 2015 and January 15th, 2016 with eight borings drilled with truck and track mounted equipment to depths between 10.5 and 60.0-feet below the ground surface. The approximate locations of our borings are shown on Figure 2. Our geologist logged the borings in the field and collected select soil samples for laboratory testing. Soil and Rock Classification Charts are presented along with the boring logs on Figures A-1 through A-15.

Laboratory testing of soil samples from the exploratory borings included determination of moisture content, dry density, unconfined compressive strength, plasticity index, and amount material passing the #200 sieve. The results of the moisture content, dry density, material passing the #200 sieve, and unconfined compressive strength tests are presented on the boring logs. The results of the laboratory plasticity index tests are presented on Figure A-16. The laboratory testing program also is discussed in more detail in Appendix A.

3.4 <u>Subsurface and Groundwater Conditions</u>

Our subsurface exploration somewhat confirms the mapped geologic conditions at the site (Rice, Smith & Strand, 1976). The interpreted subsurface conditions are shown on described for each structure location is described below:

1005 Northgate Drive: As shown on the Geologic Cross Section, Figure 7, the subsurface conditions vary significantly across the site. The southwestern end of the site, we encountered approximately 5-feet of medium stiff, medium plasticity, sandy clay fill overlying approximately 17-feet of loose to medium dense clayey sand alluvium. Hard weathered sandstone was encountered below the alluvial soils. The northeastern portion of the site encountered only 1-foot of low to medium plasticity, medium stiff, sandy clay fill overlying weathered sandstone bedrock. Groundwater was observed at approximately 9.0-feet below the ground surface in Boring 8.

1020 Northgate Drive: As shown on the Geologic Cross Section, Figure 8, this site is underlain by 1 to 15-feet of medium dense clayey sand fill. Weathered bedrock was observed below the fill.

1025 Northgate Drive: As shown on the Geologic Cross Section, Figure 8, the southern portion of this site is underlain by 21 to 26-feet medium dense clayey sand fill overlying weathered sandstone bedrock. However, Boring 2, located at the northern portion of the site, encountered loose to dense clayey sand alluvium underlying the fill. This boring extended to a maximum depth of 60-feet below the ground surface without encountering weathered bedrock. Groundwater was observed approximately 22-feet below the ground surface in Boring 2 only.

The borings were not left open for an extended period of time to determine a stabilized depth groundwater depth. Therefore, the groundwater levels observed in the field may not represent actual water levels. Typically, groundwater levels vary seasonally with high groundwater levels expected during the wetter winter months. For design purposes we estimate the groundwater levels approach 5-feet and 15-feet below the ground surface at 1005 and 1025 Northgate Drive, respectively, and we do not anticipate encountering groundwater at 1020 Northgate Drive.

3.5 Seismicity

3.5.1 Active Faults in the Region – The project site is located within a seismically active region that includes the Central and Northern Coast Mountain Ranges. Several active faults are present in the area both east and west of the site including the West Napa, San Andreas, and Rodgers Creek. An "active" fault is defined as one that shows displacement within the last 11,000 years and, therefore, is considered more likely to generate a future earthquake than a fault that shows no sign of recent rupture. The California Department of Conservation, Division of Mines and Geology has mapped various active and inactive faults in the region (CGS 2007). These faults are shown in relation to the project site on the attached Active Fault Map, Figure 9.

The movement between rock formations along either side of a fault may be horizontal, vertical, or a combination and is radiated outward in the form of energy waves. The amplitude and frequency of earthquake ground motions partially depends on the material through which it is moving. The earthquake force is transmitted through hard rock in short, rapid vibrations, while this energy movement becomes a long, high-amplitude motion when moving through soft ground materials, such as bay mud.

3.5.2. Historic Fault Activity – Numerous earthquakes have occurred in the region within historic times. The results of our computer database search indicate that at least 6 earthquakes (Richter Magnitude 5.0 or larger) have occurred within 100 kilometers (62 miles) of the site area between 1900 and 2016. The five most significant historic earthquakes to affect the project site are summarized in Table A.

TABLE A
SIGNIFICANT EARTHQUAKE ACTIVITY
Hillcrest Multi-Use Development
San Rafael, California

Epicenter (Latitude, Longitude)	Historic Richter Magnitude	Year
38.22, -122.31	6.0	2014
37.85, -121.82	5.8	1980
37.56, -121.72	5.7	1957
37.19, -122.15	5.8	1955
37.75, -122.55	7.7	1906

Reference: USGS (2016)

3.5.3 Probability of Future Earthquakes – The site will likely experience moderate to strong ground shaking from future earthquakes originating on any of several active faults in the San Francisco Bay region. The historical records do not directly indicate either the maximum credible earthquake or the probability of such a future event. To evaluate earthquake probabilities in California, the USGS has assembled a group of researchers into the "Working Group on California Earthquake Probabilities" (USGS 2003, 2008; Field, et al, 2015) to estimate the probabilities of earthquakes on active faults. These studies have been published cooperatively by the USGS, CGS, and Southern California Earthquake Center (SCEC) as the Uniform California Earthquake Rupture Forecast, Versions 1, 2, and 3 (aka UCERF, UCERF2, and UCERF3, respectively). In these studies, potential seismic sources were analyzed considering fault geometry, geologic slip rates, geodetic strain rates, historic activity, micro-seismicity, and other factors to arrive at estimates of earthquakes of various magnitudes on a variety of faults in California.

The 2003 study (UCERF) specifically analyzed fault sources and earthquake probabilities for the seven major regional fault systems in the Bay Area region of northern California. The 2008 study (UCERF2) applied many of the analyses used in the 2003 study to the entire state of California and updated some of the analytical methods and models. The most recent 2013 study (UCERF3) further expanded the database of faults considered, and allowed for consideration of multi-fault ruptures, among other improvements.

Conclusions from the most recent UCERF3 indicate the highest probability of an M>6.7 earthquake on any of the active faults in the San Francisco Bay region by 2045 is assigned to the San Andreas Fault at 33%. The nearest known active fault is the San Andreas Fault, located approximately 13.3-kilometers southwest of the project site, assigned a probability of % for a M>6.7 earthquake by 2045. Additional studies by the USGS regarding the probability of large earthquakes in the Bay Area are ongoing. These current evaluations include data from additional active faults and updated geological data.

4.0 GEOLOGIC HAZARDS EVALUATION

4.1 General

The principal geologic hazards which could potentially affect the project site are strong seismic shaking, liquefaction, and differential settlement. Other hazards, such as fault surface rupture, slope instability, and others are not considered significant with regard to the current project. Geologic hazards, their impacts, and recommended mitigation measures are discussed below.

4.2 Fault Surface Rupture

Under the Alquist-Priolo Earthquake Fault Zoning Act, the California Geological Survey (CDMG)/California Geologic Survey (CGS) (1972, 2000) produced 1:24,000 scale maps showing all known active faults and defining zones within which special fault studies are required. Based on currently available published geologic information, the project site is not located within an Alquist-Priolo Earthquake Fault Zone (CGS, 2007). Therefore, the potential for fault surface rupture on the campus is considered to be low.

Evaluation: *No significant impact.*
Mitigation: *No mitigation measures are required.*

4.3 Seismic Shaking

The site will likely experience seismic ground shaking from future earthquakes in the San Francisco Bay Area. Earthquakes along several active faults in the region, as shown on Figure 9, could cause moderate to strong ground shaking at the site.

4.3.1 Deterministic Seismic Hazard Analysis – Deterministic Seismic Hazard Analysis (DSHA) predicts the intensity of earthquake ground motions by analyzing the characteristics of nearby faults, distance to the faults and rupture zones, earthquake magnitudes, earthquake durations, and site-specific geologic conditions. Empirical relations (Campbell and Borzognia, Chiou and Youngs, (2008)) for the stiff soil subsurface conditions were utilized to provide approximate estimates of median site peak ground accelerations (PGA). A summary of the principal active faults affecting the site, their closest distance, maximum moment magnitude, and probable median PGAs which an earthquake on the fault could generate at the site are shown in Table B.

TABLE B
DETERMINISTIC PEAK GROUND ACCELERATION
Hillcrest Multi-Use Development
San Rafael, California

Fault	Approx. Fault Distance[1]	Max. Moment Magnitude[1]	Median PGA[2,3,4,]
San Andreas	13.7 km	8.0	0.30 g
San Gregorio	14.2 km	7.4	0.26 g
Hayward	15.0 km	7.3	0.24 g
Rodgers Creek	23.9 km	7.3	0.18 g
Mt. Diablo Thrust	0.08 km	6.6	0.10 g

Notes:
1. Caltrans ARS V2.3.06 (2016)
2. Campbell and Borzognia (2008)
3. Chiou and Youngs (2008)
4. Values determined using Vs^{30} = 270 m/s for Site Class "D"

4.3.2 Probabilistic Seismic Hazard Analysis – Probabilistic Seismic Hazard Analysis (PSHA) analyzes all possible earthquake scenarios while incorporating the probability of each individual event to occur. The probability is determined in the form of the recurrence interval, which is the average time for a specific earthquake acceleration to be exceeded. The design earthquake is not solely dependent on the fault with the closest distance to the site and/or the largest magnitude, but rather the probability of given seismic events occurring on both known and unknown faults.

We calculated the PGA for two separate probabilistic conditions, the 2% chance of exceedance in 50 years (2,475 year statistical return period) and the 10% chance of exceedance in 50 years (475 year statistical return period), utilizing the 2008 Interactive Deaggregation (USGS, 2008). The results of the probabilistic analyses are presented below in Table C.

TABLE C
PROBABILISTIC SEISMIC HAZARD ANALYSES
Harvest Middle School Solar Array
San Rafael, California

	Statistical Return Period	Magnitude	PGA
2% in 50 years	2,475 years	7.0	0.74 g
10% in 50 years	475 years	7.0	0.47 g

Reference: USGS 2008 Interactive Deaggregation (2016)

The potential for strong seismic shaking at the project site is high. Due to their close proximity and/or historic seismic activity, the San Andreas, Hayward and Rodgers Creek Faults present the highest potential for strong ground shaking. The most significant adverse impact associated with strong seismic shaking is potential damage to structures and improvements.

Evaluation: *Less than significant with mitigation.*
Mitigation: *Minimum mitigation measures should include designing the structures and foundations in accordance with the most recent version (2013) of the California Building Code. Recommended seismic coefficients are provided in Section 5.2 of this report.*

4.4 Liquefaction Potential and Related Impacts

Liquefaction refers to the sudden, temporary loss of soil shear strength during strong ground shaking. Liquefaction-related phenomena include liquefaction-induced settlement, flow failure, and lateral spreading. These phenomena can occur where there are saturated, loose, granular deposits. Recent advances in liquefaction studies indicate that liquefaction can occur in granular materials with a high, 35 to 50%, fines content (soil particles that pass the #200 sieve), provided the fines exhibit a plasticity less than 7. As shown on Figure 10 the northern portion of the site is mapped by the Association of Bay Area Governments (ABAG, 2016) as lying in a zone of "moderate" to "very High" liquefaction susceptibility.

To evaluate soil liquefaction, the seismic energy from an earthquake is compared with the ability of the soil to resist pore pressure generation. The earthquake energy is termed the cyclic stress ratio (CSR) and is a function of the maximum credible earthquake peak ground acceleration (PGA) and depth. The soil resistance to liquefaction is based on the relative density, and the amount and plasticity of the fines (silts and clays). The relative density of cohesionless soil is correlated with the CPT and SPT blow count data measured in the field and corrected for overburden, sampler type, hammer efficiency and percent fines to determine the Cyclic Resistance Ratio value.

We analyzed the potential for liquefaction utilizing the procedures outlined by Idriss and Boulanger (2008 & 2010), the data collected during our subsurface exploration, and a groundwater level of 15-feet below the ground surface in Borings 1 through 3 and 5-feet below the ground surface in Boring 8. The seismic event utilized for our analyses consisted of a magnitude 7.0 earthquake producing a PGA of 0.47 g, which corresponds to a rupture along the Hayward Fault (475 year statistical return period). The results of our analyses are presented on Figure 11 and indicate the following layers are liquefiable, resulting in a factor of safety (F.S.) less than 1.5:

- Boring 1: 15.0 to 25.5-feet below the ground surface;
- Boring 2: 20 to 23.5-feet and 50.0 to 60.0 (plus) feet below the ground surface;
- Boring 3: 15.0 to 21.0-feet below the ground surface, and;
- Boring 8: 5.0 to 21.5-feet below the ground surface.

Based on current post liquefaction settlement analyses procedures, settlement can occur in soils that exhibit a factor of safety against liquefaction of 2.0 or less. Settlement of the aforementioned layers occurring at greater depths may not result in settlements at the surface provided there is a sufficient layer of non-liquefiable soil above. Although there is currently an approximate 50-foot cap of soil overlying one of the liquefiable layers in Boring 2. However, we have to assume 20-feet of this cap will be removed, resulting in an insufficient cap of soil. Therefore, post liquefaction settlement occurring in the deep liquefiable layer will manifest to the surface. Based on the procedures outlined by Idriss and Boulanger (2008 & 2010) we estimate the following post liquefaction settlement can occur during the design seismic event:

- Boring 1: 0.5 to 1.0-inch;
- Boring 2: 2.0 to 3.0-inches;
- Boring 3: 0.5 to 1.0-inch, and;
- Boring 8: 2.0 to 3.0 inches.

Evaluation: *Less than significant with mitigation.*
Mitigation: *Mitigation measures may include removing the liquefiable soils from the site during excavation for the parking structure or supporting the structure on a deep foundation system in areas underlain with liquefiable soil. The Site Grading and Foundation recommendations given later in this report should be followed.*

4.5 Seismically Induced Ground Settlement

Seismic ground shaking can induce settlement of unsaturated, loose, and "clean" granular soils. Settlement occurs as the loose soil particles rearrange into a denser configuration when subjected to seismic ground shaking. Varying degrees of settlement can occur throughout a deposit, resulting in differential settlement of structures founded on such deposits. We did not encounter loose, "clean" granular deposits above the groundwater level. Therefore, seismically induced ground settlement is not considered a geologic hazard on the project site.

Evaluation: *No significant impact.*
Mitigation: *No mitigation measures are required.*

4.6 Lurching and Ground Cracking

Lurching and associated ground cracking can occur during strong ground shaking. The ground cracking generally occurs along the tops of slopes where stiff soils are underlain by soft deposits or along steep slopes or channel banks. Although Gallinas Creek North Fork flows along the northwestern flank of the project site it is lined with concrete, therefore the risk of lurching and ground cracking is low.

Evaluation: *No significant impact.*
Mitigation: *No mitigation measures are required.*

4.7 Erosion

Sandy soils on moderate slopes or clayey soils on steep slopes are susceptible to erosion when exposed to concentrated water runoff. The work areas are relatively level or slope gently, however, the flanks of the project site are moderately sloped and covered in vegetation. Therefore, excess erosion is considered to be a moderate long-term geologic hazard.

Evaluation: *Less than significant with mitigation.*
Mitigation: *Mitigation measures include maintaining the vegetation on the slopes of the hillside to prevent excess erosion. Additionally, the project Civil Engineer should design a site drainage system to collect surface water and discharging it into an established storm drainage system. An erosion control plan should also be developed prior to construction per the current guidelines of the California Stormwater Quality Association's Best Management Practice Handbook (2009).*

4.8 Seiche and Tsunami

Seiche and tsunamis are short duration, earthquake-generated water waves in large enclosed bodies of water and the open ocean, respectively. The extent and severity of a seiche or tsunami would be dependent upon ground motions and fault offset from nearby active faults. The project site is at an elevation of +15 to +60-feet above mean sea level. Additionally, the project site is not mapped within a tsunami flood zone by the Association of Bay Area Governments (ABAG, 2016), therefore the risk of seiche and tsunami is low.

Evaluation: *No significant impact.*
Mitigation: *No mitigation measures are required.*

4.9 Flooding

As shown on Figure 13, the northern project site is mapped within a FEMA 500-year flood zone (ABAG, 2016). However, the northern project site elevations are between 35 and 45-feet above sea-level, therefore, it is our opinion large scale flooding is not considered a geologic hazard at the project site. The project Civil Engineer or Architect is responsible for site drainage and should evaluate localized flooding potential and provide appropriate mitigation.

Evaluation: *Less than significant with mitigation.*
Mitigation: *The project Civil Engineer or Architect should evaluate the risk localized flooding and provide appropriate storm drain design.*

4.11 Expansive Soil

Expansive soils will shrink and swell with fluctuations in moisture content and are capable of exerting significant expansion pressures on building foundations, interior floor slabs and exterior flatwork. Distress from expansive soil movement can include cracking of brittle wall coverings (stucco, plaster, drywall, etc.), racked door and/or window frames, and uneven floors and cracked slabs. Flatwork, pavements, and concrete slabs-on-grade are particularly vulnerable to distress due to their low bearing pressures. While high plasticity clays were not observed during our subsurface exploration. Therefore, the risk of expansive soil affecting the proposed improvements is low.

Evaluation: *No significant impact.*
Mitigation: *No mitigation measures are required.*

4.12 Settlement/Subsidence

Significant settlement can occur when new loads are placed at sites due to consolidation of soft compressible clays (i.e. Bay Mud) or compression of loose granular soils. Large deposits of soft compressible materials were not observed during our subsurface exploration. However, due to the varying subsurface conditions (i.e. shallow rock to deep fill and alluvial soils) that underlie the 1005 and 1025 Northgate Drive structures. This soil profile can provide varying foundation support materials that will react differently under load (i.e. the soils will settle while the bedrock will not) creating a potential differential settlement condition. Therefore, differential settlement is considered a geologic hazard.

Evaluation: *Less than significant with mitigation.*
Mitigation: *As previously discussed, subterranean garages are currently planned as part of the project. If the weight of the soil removed (100 pcf times the excavation depth) is at least 1.5 times the weight of the structure, then significant differential settlement is not a concern, some minor differential settlements may occur. If these minor settlements are not acceptable, then the deep foundation recommendations outlined in the Foundation section of this report should be followed.*

4.13 Slope Instability/Landsliding

Slope instability generally occurs on relatively steep slopes and/or on slopes underlain by weak materials. The project site is located on a on a hill with approximate 2:1 slopes along its flanks. We did not observe signs of instability (i.e. ground cracking, headscarps, etc.), therefore the risk of ladsliding at the project site is low.

Evaluation: *No significant impact.*
Mitigation: *No mitigation measures are required.*

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 General

Based on the results of our subsurface exploration, laboratory testing, and experience with similar projects, we conclude that the site is geotechnical suitable for the planned improvements. The primary geotechnical issues to address in design of the project will be potential differential settlement, providing uniform foundation support and structural design to resist strong seismic ground shaking and excavation conditions.

5.2 Seismic Design

The project site is located in a seismically active area, therefore, the structure should be designed in conformance with the seismic provisions of the California Building Code (CBC) to mitigate the potential effects of strong seismic ground shaking to the proposed structures. Per ASCE 7-10, the project sites at 1005 and 1025 Northgate Drive should be classified as a Site Class F (S_F)"liquifyable soil site". However per the code, structures that have a fundamental period less than 0.5 seconds within a Site Class F site may be designed utilizing Site Class D, "stiff soil" site (S_D). Therefore, the building sites where soils are planned to be at least 10-feet thick may be designed utilizing Site Class D parameters, provided the fundamental period of the structure is less than 0.5-seconds. If the fundamental period is greater than or equal to 0.5-seconds then we should perform a site specific seismic analyses. Building sites where 10-feet or less of soil is planned underlying the structure may be designed utilizing Site Class C (S_C), "very dense soil and soft rock", parameters. At a minimum, we recommend the project Structural Engineer utilize the 2013 CBC coefficients shown in Table D below to determine the base shear values.

TABLE D
2013 CBC FACTORS
Hillcrest Multi-Use Development
San Rafael, California

Factor Name	Coefficient	2013 CBC Site Specific Value	2013 CBC Site Specific Value
Site Class[1,2]	$S_{A,B,C,D,E,F}$	S_D	S_C
Site Coefficient	F_a	1.00	1.00
Site Coefficient	F_v	1.50	1.30
Spectral Acc. (short)	S_S	1.50 g	1.50 g
Spectral Acc. (1-sec)	S_1	0.60 g	0.60 g
Spectral Response (short)	SM_S	1.50 g	1.50 g
Spectral Response (1-sec)	SM_1	0.90 g	0.78 g
Design Spectral Response (short)	SD_S	1.00 g	1.00 g
Design Spectral Response (1-sec)	SD_1	0.60 g	0.52 g
MCE_G[3] PGA adjusted for Site Class	PGA_M	0.50 g	0.50 g
Seismic Design Category	A,B,C,D, E	D	D

Notes:

1. Site Class D Description: Stiff soil profile with shear wave velocities between 600 and 1,200 ft/sec, standard blow counts between 15 and 50 blows per foot, and undrained shear strength between 1,000 and 2,000 psf.
2. Site Class C Description: Very dense soil and soft rock profile with shear wave velocities between 1,200 and 2,500 ft/sec, standard blow counts greater than 50 blows per foot, and undrained shear strength greater than 2,000 psf.
3. Maximum Considered Earthquake Geometric Mean

5.3 Site Preparation and Grading

Based on our understanding of the preliminary project details, one to two-stories of subterranean parking will be constructed under each structure. These parking structures will require significant excavation and site grading to complete. Excavations and site grading should be performed in accordance with the following recommendations.

5.3.1 Site Preparation – Clear pavements, foundations, trees, brush, roots, over-sized debris, and organic material from areas to be graded. Trees that will be removed (in structural areas) must also include removal of stumps and roots larger than two inches in diameter. Excavated areas (i.e., excavations for stump removal) should be restored with properly moisture conditioned and compacted fill as described in the following sections. Any loose soil or rock at subgrade will need to be excavated to expose firm natural soils or bedrock. Debris, rocks larger than six inches and vegetation are not suitable for structural fill and should be removed from the site. Alternatively, vegetation strippings may be used in landscape areas.

Where fills or other structural improvements are planned on level ground, the subgrade surface should be scarified to a depth of about eight inches, moisture conditioned to above the optimum moisture content, and compacted to a minimum of 90% relative compaction (ASTM D-1557) and should be increased to a minimum of 95% where new asphalt pavements are planned. Relative compaction refers to the ratio, in percent, of the in-situ dry density to the maximum laboratory density of the soil. If soft, wet or otherwise unsuitable materials are encountered at the subgrade elevation during construction, we will provide supplemental recommendations/field directives to address the specific condition.

5.3.2 Materials – Based on our laboratory testing, onsite soils are suitable for use as fill. If imported fill is required, the material shall consist of soil and rock mixtures that: (1) are free of organic material, (2) have a Liquid Limit less than 40 and a Plasticity Index of less than 20, (3) consist of at least 50% material retained on the #200 sieve, and (4) have a maximum particle size of 6 inches. Any imported fill material needs to be tested to determine its suitability for use as fill material.

5.3.3 Excavations – As previously discussed excavations up to 20-feet deep are anticipated to create subterranean parking structures. The subsurface conditions vary across the project site ranging from shallow weathered sandstone bedrock to deep fill soils overlying alluvial deposits. We anticipate the soil will be easily excavated with standard equipment (i.e. excavators, dozers, scrapers, etc.) while weathered bedrock may require some hard rock excavation (i.e. large dozers with a single ripping tooth, hoe rams, etc.).

Excavations for will vary from medium dense clayey sands and weathered bedrock. Soils and weathered bedrock encountered in excavations appear to be Type C and B, respectively. Excavations having a depth of five feet or more, and will be entered by workers must be sloped, braced, or shored in accordance with current Cal/OSHA regulations. All excavations can result in collapse of sidewalls, slopes and/or bottom that could result in injury or death of workers. Therefore, excavations should be evaluated by the Contractor's safety officer and designated competent person prior to workers entering in accordance with current Cal/OSHA regulations.

Care should be taken when excavating adjacent to existing structures. The existing structures may need to be underpinned to provide additional support and the excavations should be shored to prevent impacting existing improvements. We recommend a shoring consultant should be contacted to aid in the design of the required shoring and/or underpinning.

5.4 Foundation Design

Based on our experience with similar projects, each of the proposed structures should gain foundation support from similar materials to prevent differential settlements. As previously discussed, one to two-story subterranean garage structures will be constructed under the proposed buildings. The location and depth of these excavations will dictate the recommended foundation system to support the structures. A discussion of each building location is described below:

1005 Northgate Drive: The subsurface conditions at this site vary from weathered sandstone bedrock near the ground surface at the northeastern end to approximately 20-feet of potentially liquefiable alluvial soils overlying weathered sandstone bedrock at the southwest. If a 20-foot basement excavation were to occur in this area then weathered bedrock would most likely be exposed and the structure could be supported on a shallow foundation system. However, if an excavation less than 20-feet is planned, then shallow foundations should be utilized to support the structure where bedrock is exposed and drilled piers (interconnected with gradebeams) that extend through the potentially liquefiable alluvial soils, and socket into the weathered bedrock, should be utilized to support the structure where soils was exposed.

1020 Northgate Drive: The subsurface conditions in this area consist of 1- to 15-feet of fill soil overlying weathered sandstone and shale bedrock. Once the excavation for the subsurface parking garage occurs (two-stories), weathered bedrock should be exposed. Therefore, a shallow foundation system may be utilized to support the structure. Localized deepening of the foundations or shallow drilled piers may be required to expose the bedrock.

1025 Northgate Drive: The subsurface conditions in the southwestern portion of the planned building pad site consist of approximately 20- to 25-feet of fill soils overlying weathered sandstone and serpentinite bedrock. The northeastern portion of the planned building is underlain by approximately 25-feet of fill overlying deeper alluvial deposits. In this areas, we drilled to a maximum depth of 60-feet below the ground surface without encountering bedrock.

If a 20-foot plus excavation (2-story parking garage) is planned, bedrock may be exposed on the southwestern side of the proposed structure and alluvial soils will be exposed on the northeastern. Therefore, shallow foundations may be utilized where bedrock is at or near the ground surface and a deep foundation system, interconnected with gradebeams, should be utilized where deeper alluvial soils are exposed. These deep foundations should extend through the liquefiable soils and socket into the underlying weathered bedrock. At this time we do not have an estimate on the depth of the deep foundations, therefore an additional boring should be performed to determine the depth to bedrock.

If a 10-foot or less excavation were to occur (1-story parking garage) then fill soils will likely be exposed. These fill soils are potentially liquefiable, as are the deeper alluvial soils located to the north, therefore this structure should be supported entirely on a deep foundation system, interconnected with gradebeams, that extend and socket into the underlying bedrock.

As previously described each structure will contain a one- to two-story subterranean parking garage. If subsurface drainage is not provided for the garage, hydrostatic uplift pressures may occur on retaining walls and floors if the water table rises. This condition may occur in the 1005 and 1025 Northgate Drive structures only. Therefore, the foundations should be designed to withstand uplift pressure.

Both shallow foundation (spread footings and mat slabs) and drilled pier design criteria are presented in Table E below. We can provide additional foundation design criteria for alternative foundation types (i.e. driven piles, micropiles, helical anchors, torque down piles, etc.) upon request. Additionally, we can provide vertical capacity versus depth and lateral pile analyses, once the foundation parameters are known, upon request.

TABLE E
DRILLED PIER DESIGN CRITERIA
Hillcrest Multi-Use Development
San Rafael, California

Shallow Spread Footings	
Minimum footing width[1]:	
One-story structure	12 inches
Two-story structure:	15 inches
Three-story, or more, structure	18 inches
Minimum footing embedment depth (below lowest adjacent grade):	18 inches
Allowable soil bearing pressure (dead plus live loads)[2]:	
Soil/fill:	2,000 psf
Weathered bedrock:	4,000 psf
Base friction coefficient:	0.35
Lateral passive resistance[2, 3, 4]:	
Soil/fill:	300 pcf
Weathered bedrock:	600 pcf
Mat Slabs	
Minimum thickness:	6-inches
Modulus of subgrade reaction:	
Soil/fill:	150 pci
Weathered bedrock:	300 pci
Drilled Piers	
Minimum diameter:	18 inches
Minimum embedment into bedrock:	5 feet
Skin friction[2,5]:	
Soil/fill:	500 psf
Weathered Bedrock:	1,500 psf
Lateral Passive Resistance[2,4,6]:	
Soil/fill:	300 pcf
Weathered Bedrock:	600 psf
Hydrostatic Uplift Pressure[7]	62.4 x H_W psf

1. Size footing widths to avoid significantly different foundation pressures.
2. May increase design values by 1/3 for total design loads including seismic.
3. Equivalent Fluid Pressure, not to exceed 3,000 psf.
4. Ignore uppermost 6-inches unless concrete or asphalt surfacing exists adjacent to foundation.
5. Uplift capacity is equal to 80% of the downward skin resistance.
6. Apply passive resistance over two pier diameters; neglect upper two feet of soil where there is less than ten feet of horizontal confinement.
7. H_W = Height of groundwater. Apply as uphill pressure on floor if subsurface drainage is not provided.

5.5 Retaining Wall Design

We anticipate retaining walls up to 20-feet in height will be required to construct the subsurface parking garage. These walls should be constructed utilizing tie-backs and built in 5-foot vertical increments to shore the excavation as it is created. Smaller site retaining walls may be constructed with conventional supported retaining systems (i.e. shallow foundation or drilled pier supported). Walls free to rotate at the top, "unrestrained", and tie-back walls or walls structurally connected at the top should be designed using the "restrained" design criteria shown in Table F.

TABLE E
RETAINING WALL DESIGN CRITERIA
Hillcrest Multi-Use Development
San Rafael, California

Foundations			
See Table E			
Tie-back Design Parameters	Unit Wt., γ	Cohesion, c	Friction, ϕ
Fill/soil:	120 pcf	500 psf	30°
Weathered bedrock:	140 pcf	1,000 psf	38°
Unrestrained Earth Pressure[1,2]			
Level Ground:			40 pcf
2:1 Slope:			60 pcf
Restrained Earth Pressure[1,3]			
Level Ground:			60 pcf
2:1 Slope:			80 pcf
Seismic Surcharge[3]			10 x H psf
Hydrostatic Surcharge[4]			62.4 pcf

Notes:

1. Interpolate earth pressures for intermediate slopes.
2. Equivalent fluid pressure.
3. Rectangular distribution. The factor of safety for short-term seismic conditions can be reduced to 1.1 or greater.
4. Apply if subsurface drainage is not provided.

Drainage shall be provided for all retaining walls taller than 3 feet. Either Caltrans Class 1B permeable material within filter fabric or Caltrans Class 2 permeable material can be used. The seepage should be collected in a 4-inch perforated PVC drain line at the base of the wall. The permeable material shall extend at least 12 inches from the back of the wall and be continuous from the bottom of the wall to within 12 inches of the ground surface. Alternatively, drainage panels, such as Mirifi 100N, may be utilized. A typical detail of retaining wall drainage is shown on Figure 14.

Seepage collected in the drain line should be conveyed off-site by gravity in closed pipe to the storm drainage system. The pipe shall have a minimum slope of 1 percent to drain. To maintain the wall drainage system, clean outs shall be installed at the upstream end and at all major changes in direction. Water proofing of any below grade residential walls should be designed by the Architect to prevent moisture infiltration through the wall into living spaces.

5.6 Concrete Slabs-on-Grade

Where concrete slabs are needed, we recommend they be at least 5-inches thick and reinforced with steel bars (not wire mesh). Contraction joints should be incorporated in the concrete slab in both directions, no greater than 10 feet on center. Additionally, the reinforcing bars shall extend through the control joints. For improved performance, concrete slabs on grade may be increased to 6-inches thick. The project Structural Engineer should design the concrete slab floors.

To improve interior moisture conditions, a 4-inch minimum layer of clean, free draining, 3/4-inch angular gravel or crushed base rock should be placed beneath the interior concrete slabs to form a capillary moisture break. The base rock must be placed on a properly moisture conditioned and compacted subgrade that has been approved by the Geotechnical Engineer. A plastic membrane vapor barrier, 15-mils or thicker, should be placed over the drain rock. The vapor barrier shall meet the Class A requirements outlined in ASTM E 1745 and be installed per ASTM 1643. Eliminating the capillary moisture break and/or plastic vapor barrier may result in excess moisture intrusion through the floor slabs resulting in poor performance of floor coverings, mold growth or other adverse conditions.

Exterior concrete slabs should be at least 4-inches thick and reinforced as described above for interior slabs. For improved performance exterior concrete slabs shall be underlain with at least 4-inches or more of Caltrans Class 2 Aggregate Base compacted to at least 92 percent relative compaction. Some movement should be expected for exterior concrete slabs as the underlying soils react to seasonal moisture changes.

5.7 Pavement Design

Typically, asphalt pavement sections are designed utilizing two variables, the R-Value (a measure of the subgrade resistance) and the Traffic Index (TI – a measure of the amount of daily traffic). Based on the results of our s R-Values of 10 and 40 were utilized in our calculations for soil/fill and weathered bedrock, respectively. We have calculated pavement sections for the project site and anticipated soil conditions in accordance with Caltrans procedures for flexible pavement design utilizing variable TI values as shown in Table F.

TABLE F
ASPHALT PAVEMENT SECTIONS
Hillcrest Multi-Use Development
San Rafael, California

T.I.	Asphalt Concrete	Aggregate Baserock Soil/Fill Subgrade	Aggregate Baserock Weathered Rock Subgrade
4.0	3.0-inches	6.0-inches	4.0-inches
5.0	3.0-inches	9.0-inches	5.0-inches
6.0	3.5-inches	11.5-inches	6.0-inches
7.0	4.0-inches	14.5-inches	7.0-inches

Note: To reduce the overall section thickness the "2 to 1" rule of thumb may be applied, where 2-inches of AB is equivalent to 1-inch of AC. For example a section consisting of 4.0-inches of AC overlying 15.5-inches of AB (19.5-inches total) may be reduced to 6.0-inches of AC overlying 11.5-inches of AB (17.5-inches total).

Prior to construction of the new pavement section, the existing subgrade should be scarified to a minimum depth of 8-inches, moisture-conditioned to near-optimum moisture content. The subgrade should then be compacted to a minimum of 95 percent relative compaction per ASTM D-1557 and to produce a firm and unyielding surface when proof rolled with heavy construction equipment.

5.8 Site and Foundation Drainage

Portions of the site are relatively flat in the area of the proposed improvements and there is a possibility that new grading could result in adverse drainage patterns and water ponding around buildings. Careful consideration should therefore be given to design of finished grades at the site. We recommend that landscaped areas adjoining new structures be sloped downward at least 0.25 feet for 5 feet (5 percent) from the perimeter of building foundations. Where hard surfaces, such as concrete or asphalt adjoin foundations, slope these surfaces at least 0.10 feet in the first 5 feet (2 percent). Roof gutter downspouts may discharge onto the pavements, but should not discharge onto any landscaped areas. Provide area drains for landscape planters adjacent to buildings and parking areas and collect downspout discharges into a tight pipe collection system. Site drainage improvements should be connected into the existing campus storm drainage system.

5.6 Utility Trench Backfills

The excavation conditions outlined above should be consulted for utility trench excavations. Bedding materials for utility pipes should be well graded sand with 90 to 100 percent of particles passing the No. 4 sieve and no more than 5 percent finer than the No. 200 sieve. Provide the minimum bedding beneath the pipe in accordance with the manufacturer's recommendation, typically 3 to 6 inches. Trench backfill may consist of on-site soils moisture conditioned to at least

2 percent over the optimum moisture content, placed in thin lifts and compacted to at least 90 percent R.C. Backfill for trenches within pavement areas should consist of non-expansive granular fill. Use equipment and methods that are suitable for work in confined areas without damaging utility conduits. Where utility lines cross under or through perimeter footings, they should be sealed to reduce moisture intrusion into the areas under the slabs and/or footings.

6.0 SUPPLEMENTAL GEOTECHNICAL SERVICES

We must review the plans and specifications for the project when they are nearing completion to confirm that the intent of our geotechnical recommendations has been incorporated and provide supplemental recommendations, if needed. During construction, we must observe and test site grading, foundation excavations for the structures and associated improvements to confirm that the soils encountered during construction are consistent with the design criteria.

7.0 LIMITATIONS

This report has been prepared in accordance with generally accepted geotechnical engineering practices in the San Francisco Bay Area at the time the report was prepared. This report has been prepared for the exclusive use of San Rafael Hillcrest Residential, LLC and San Rafael Hillcrest Commercial, LLC and/or their assignees specifically for this project. No other warranty, expressed or implied, is made. Our evaluations and recommendations are based on the data obtained during our subsurface exploration program and our experience with soils in this geographic area.

Our approved scope of work did not include an environmental assessment of the site. Consequently, this report does not contain information regarding the presence or absence of toxic or hazardous wastes.

The evaluations and recommendations do not reflect variations in subsurface conditions that may exist between boring locations or in unexplored portions of the site. Should such variations become apparent during construction, the general recommendations contained within this report will not be considered valid unless MPEG is given the opportunity to review such variations and revise or modify our recommendations accordingly. No changes may be made to the general recommendations contained herein without the written consent of MPEG.

We recommend that this report, in its entirety, be made available to project team members, contractors, and subcontractors for informational purposes and discussion. We intend that the information presented within this report be interpreted only within the context of the report as a whole. No portion of this report should be separated from the rest of the information presented herein. No single portion of this report shall be considered valid unless it is presented with and as an integral part of the entire report.

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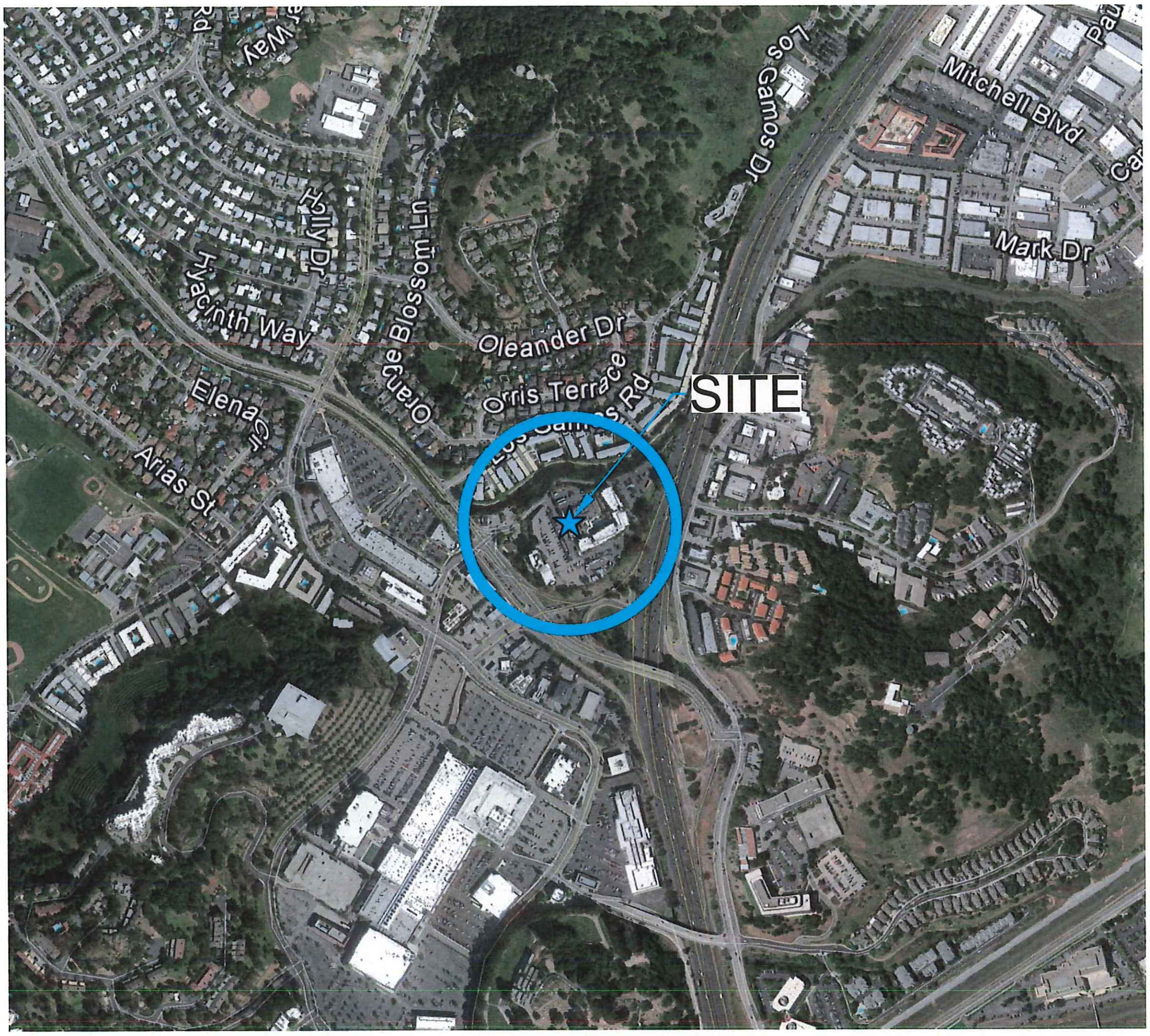


SITE: LATITUDE, 38.0088°
LONGITUDE, -122.5435°

SITE LOCATION
N.T.S.

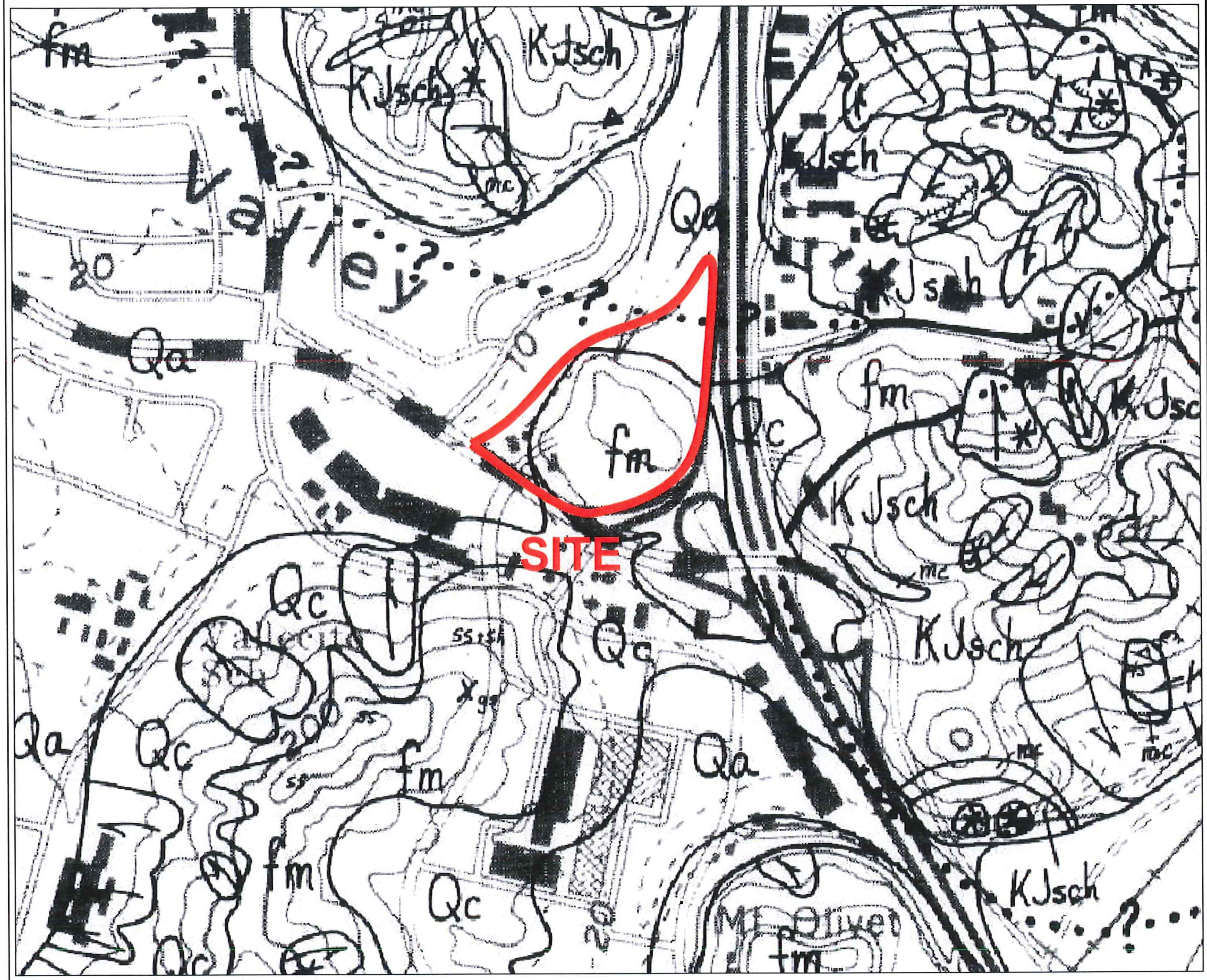
NORTH

REFERENCE: Google Earth, 2016

Miller Pacific ENGINEERING GROUP	504 Redwood Blvd. Suite 220 Novato, CA 94947 T 415 / 382-3444 F 415 / 382-3450 www.millerpac.com	SITE LOCATION MAP		
		San Rafael Hillcrest Multi-Use Development San Rafael, California	Drawn BSP Checked	1 FIGURE
A CALIFORNIA CORPORATION, © 2012, ALL RIGHTS RESERVED		Project No. 2243.001 Date: 1/21/16		
FILE: 936.185 SLM.dwg				

SITE PLAN

504 Redwood Blvd.



GEOLOGIC MAP

(Not to Scale)

Qa	**ALLUVIUM - Quaternary** Unconsolidated deposits of clay, silt, sand, and gravel underlying the bottom lands of the main stream valleys, consisting of materials transported and deposited by the streams.
fm	**FRANCISCAN MELANGE - Jurassic** A tectonic mixture consisting of small to large masses of resistant rock types, principally of sandstone, greenstone, chert and serpentine, but including various exotic metamorphic rock types, embedded in a matrix of pervasively sheared or pulverized rock material.
Kjsch	**SEMI-SCHIST, PHYLLITE and SCHIST - Createaceous-Jurassic** Predominantly slightly to well foliated or liniated metamorphosed sedimentary and volcanic rocks.

Reference: Rice, Salem J., Strand, Rudolph G, and Smith, Theodore C., Geology of the Eastern Part of the San Rafael Area, Geology for Planning in Central and Southeastern Marin County, California (1976). Map Scale 1:12,000

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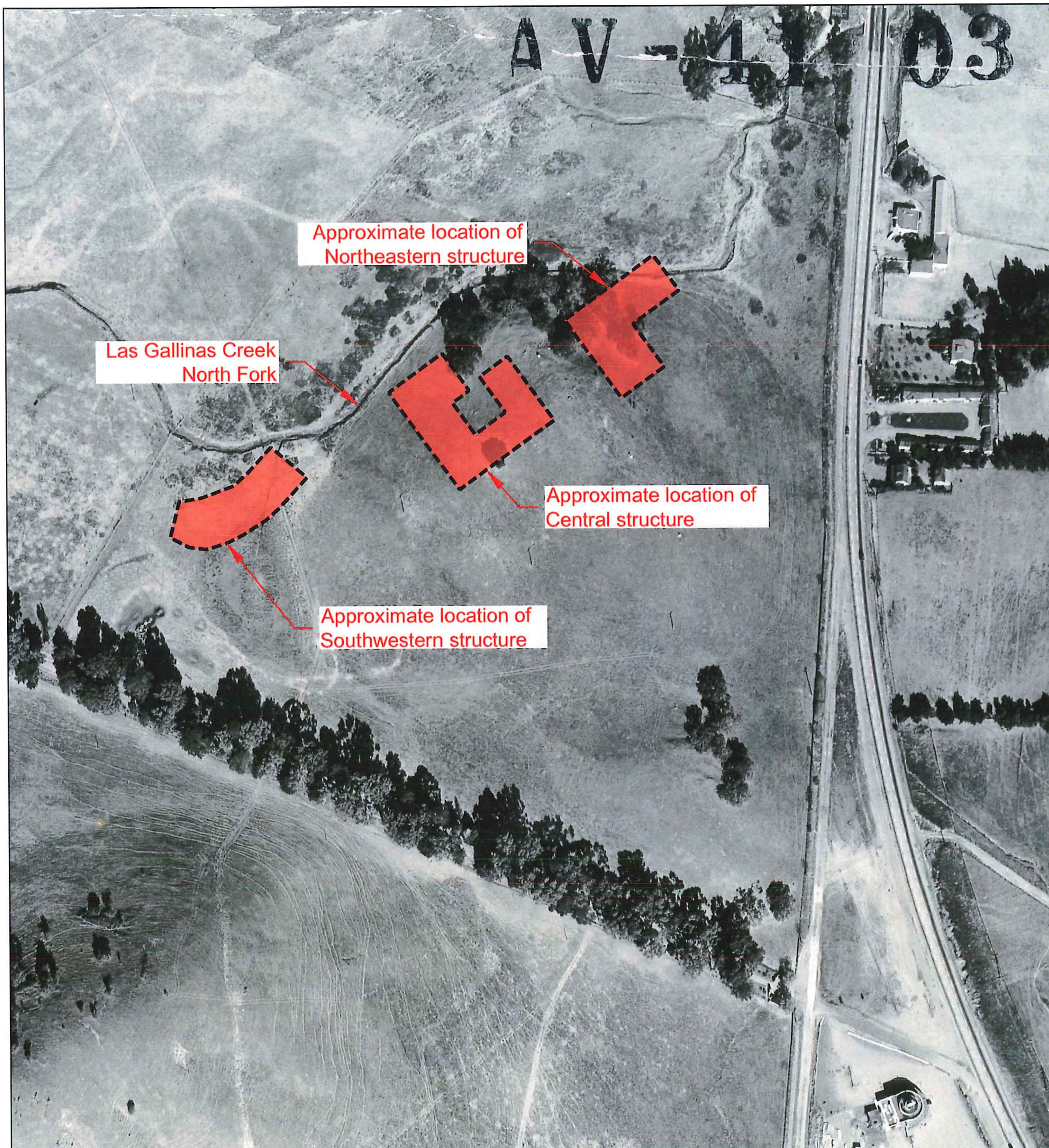
FILE: 2243.001 RGM.dwg



AERIAL PHOTOGRAPH - 1/08/1950

1" = 1,000'

NORTH

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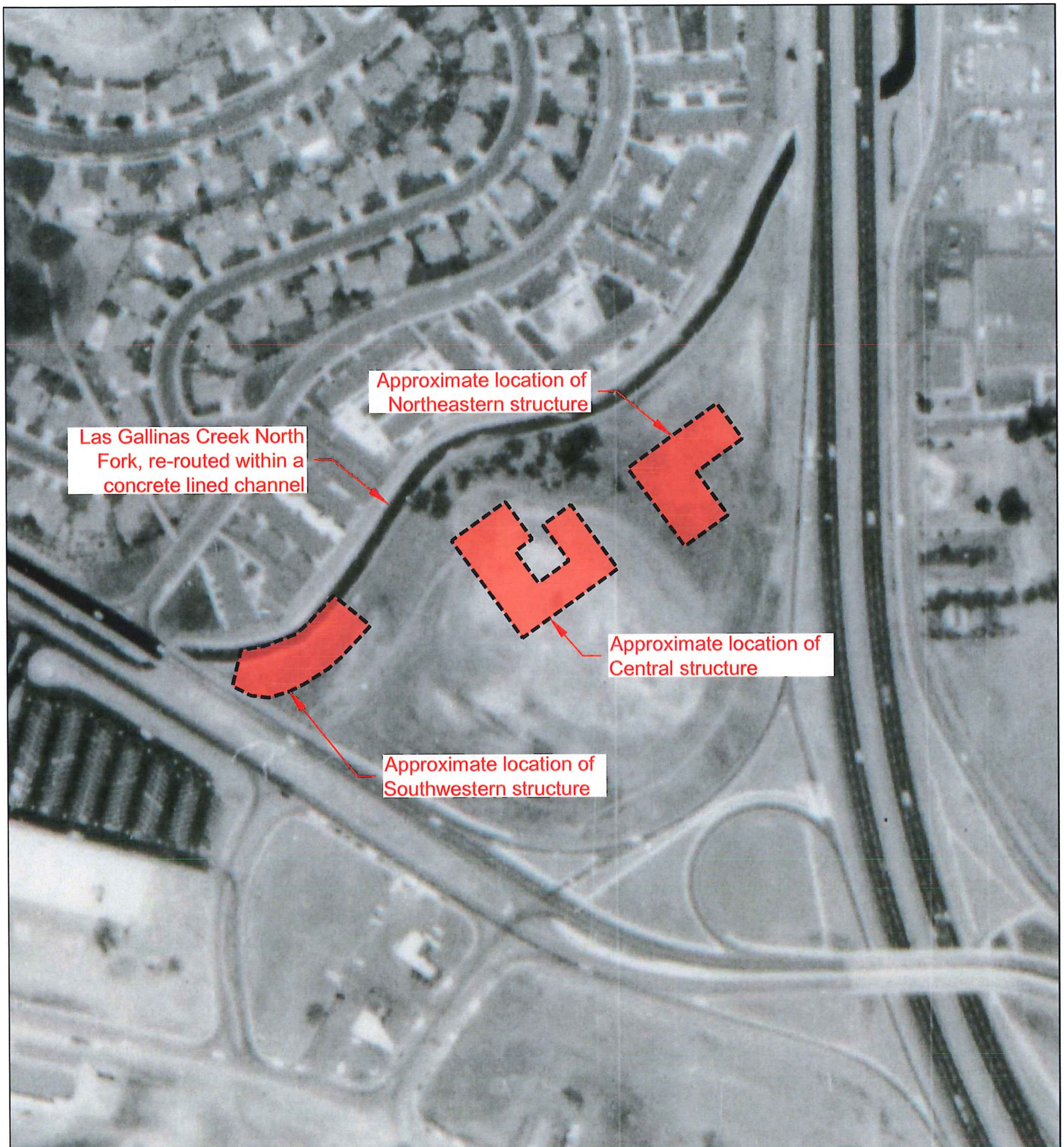

AERIAL PHOTOGRAPH - 7/09/1963

1" = 1,000'

NORTH

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FILE:2243.001AirPhoto.dwg

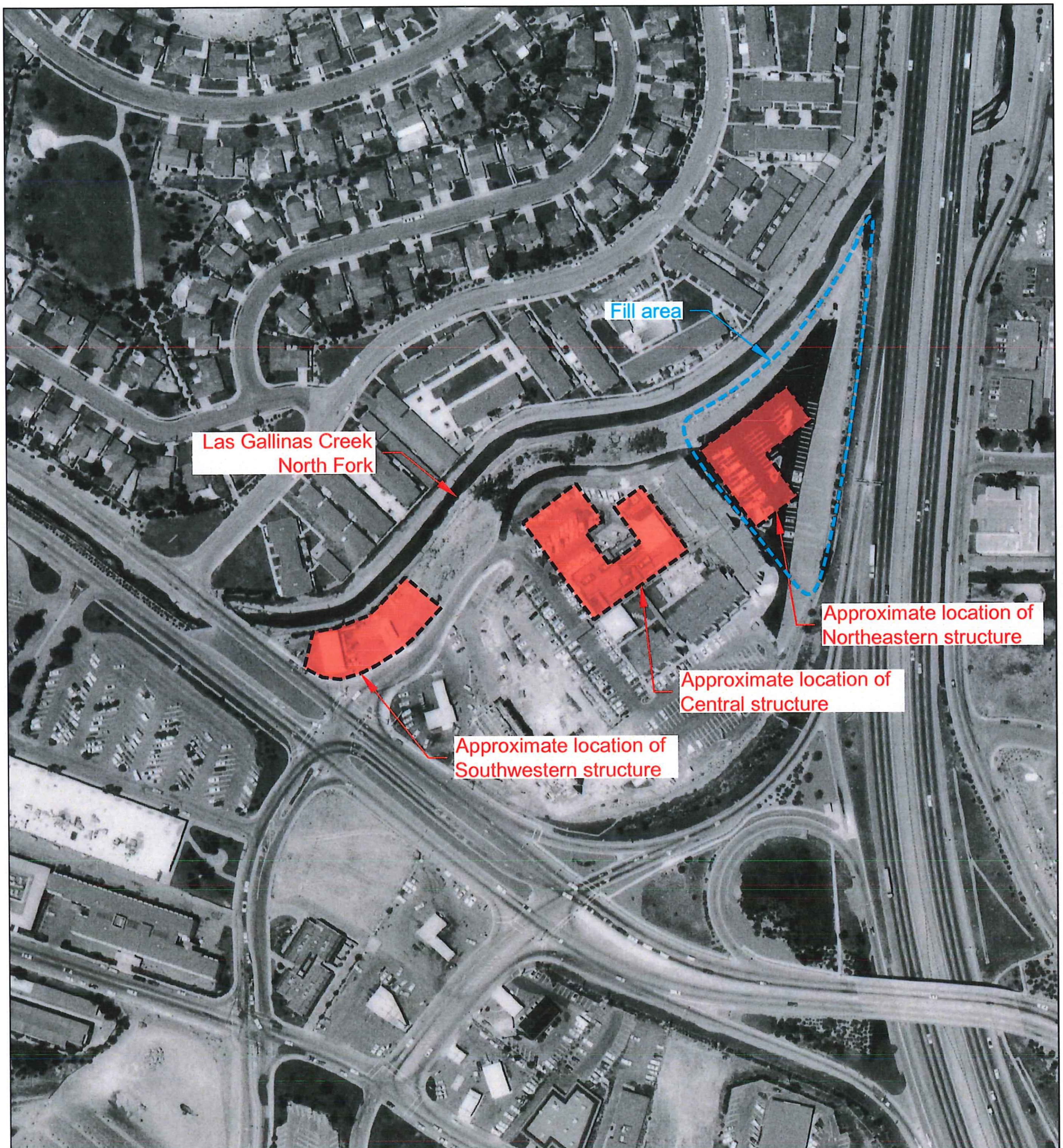
Aerial Photograph - 7/09/1963

Hillcrest San Rafael
Multi-Use Development
San Rafael, California

Project No. 2243.001 Date: 1/20/16

Drawn
Checked BSP

5
FIGURE



AERIAL PHOTOGRAPH - 7/02/1970

1" = 1,000'

NORTH

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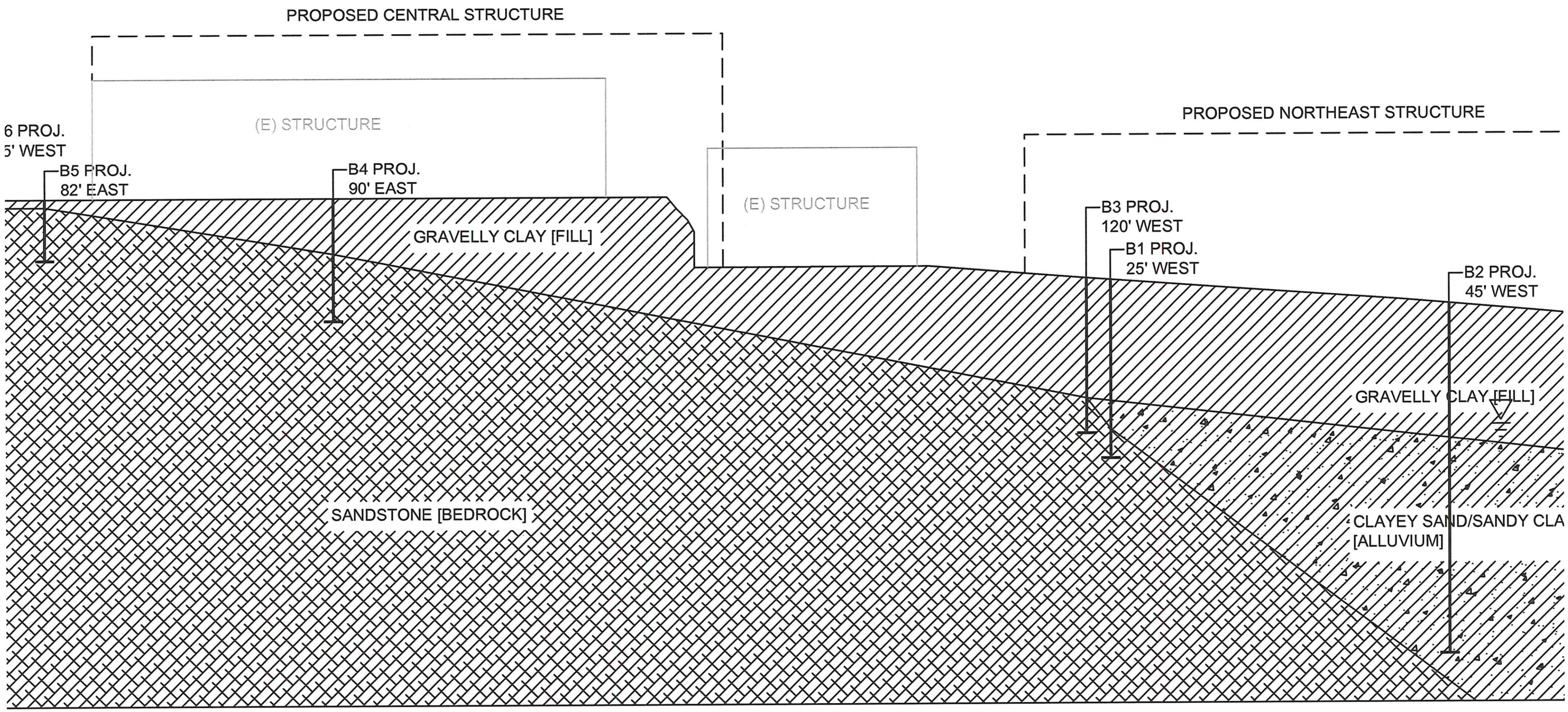
FILE:2243.001AirPhoto.dwg

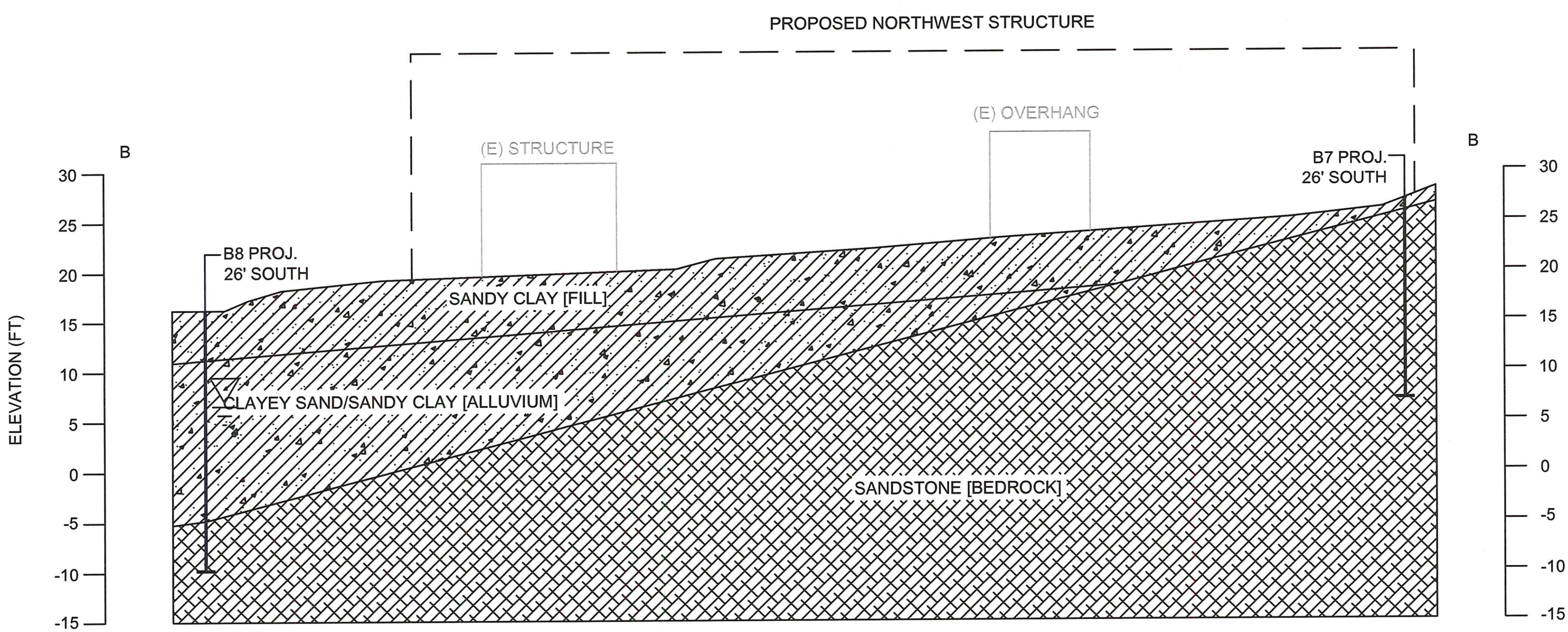


SECTION A-A'

Horiz Scale: 1"=50'
Vert. Scale: 1"=25'

504 Redwood Blvd.

SIMPLIFIED GE



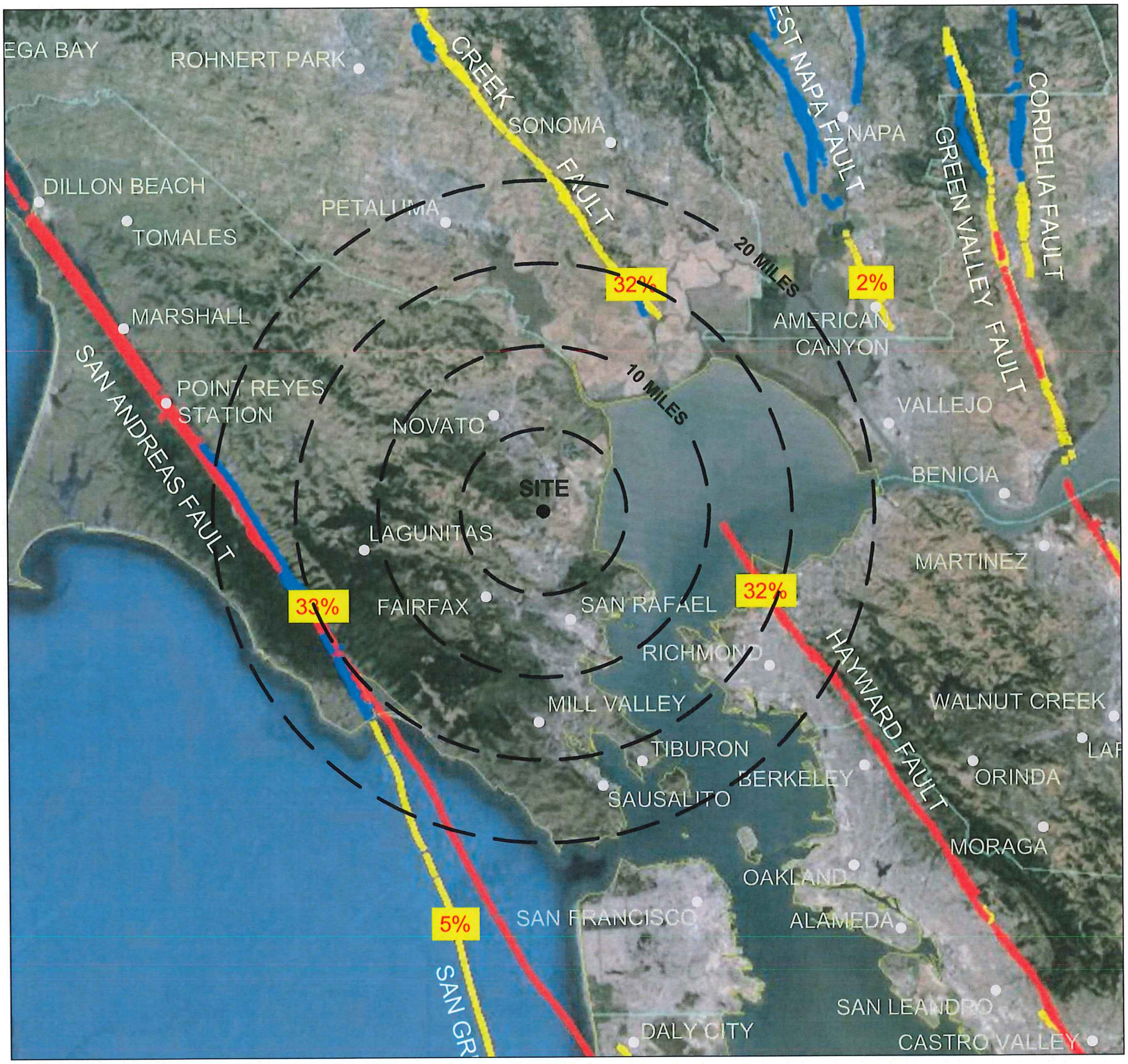
SECTION B-B'

Horiz Scale: 1"=30'

Vert. Scale: 1"=15'

504 Redwood Blvd.

SIMPLIFIED GE



LAT: 38.0088; LONG: -122.5435

LEGEND

NORTH

(COLOR INDICATES AGE OF MOST RECENT KNOWN MOVEMENT)

- HISTORIC (<150 YEARS)
- HOLOCENE (<11,000 YEARS)
- LATE QUATERNARY (<1.0M YEARS)
- 32% PROBABILITY OF AT LEAST ONE M>6.7 EARTHQUAKE BETWEEN 2015 AND 2045 FOR FAULTS SHOWN

DATA SOURCE:
1) Working Group on California Earthquake Probabilities (WGCEP)(2014), "Long-Term Time-Dependent Probabililtites for the Third Uniform California Earthquake Rupture Forecast (UCERF3), Bulletin of the Seismological Society of America (BSSA), Volume 105, No. 2A, 33pp, April 2015.

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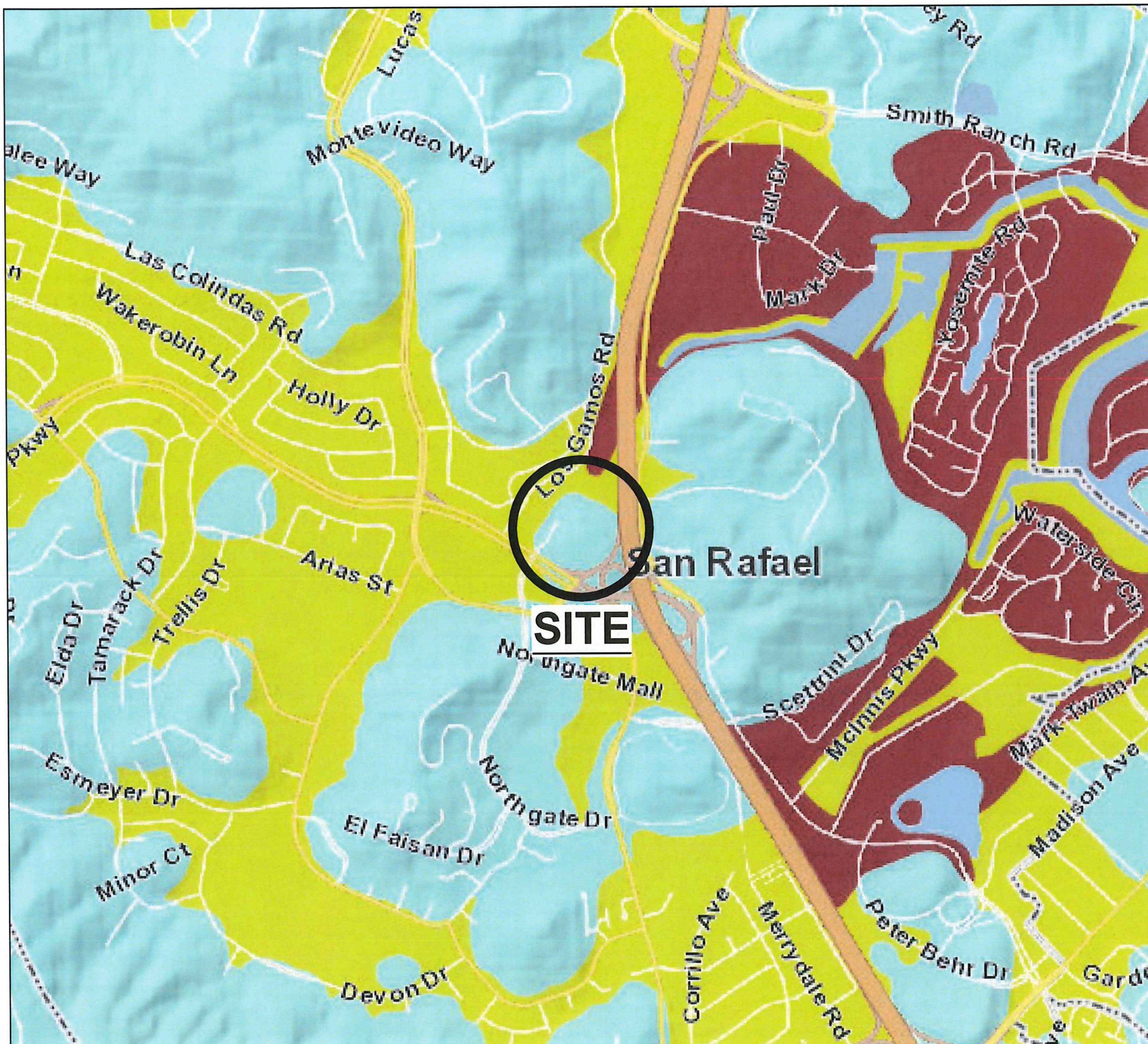
		San Rafael Hillcrest Multi-Use Development San Rafael, California
		Project No. 2243.001 Date: 1/21/16
		Drawn ENE Checked
		9 FIGURE

FILE: 2243.001 AFM.dwg



Legend

- Very High
- High
- Moderate
- Low
- Very Low
- Water Bodies

LIQUEFACTION SUSCEPTIBILITY MAP

(Not to Scale)

NORTH

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

January 20, 2016

Miller Pacific
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FILE: 2243.001LiqPlots.dwg

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LIQUEFACTION SUSCEPTIBILITY MAP

San Rafael Hillcrest
Multi-Use Development
San Rafael, California

Project No. 2243.001 Date: 1/21/16

Drawn BSP
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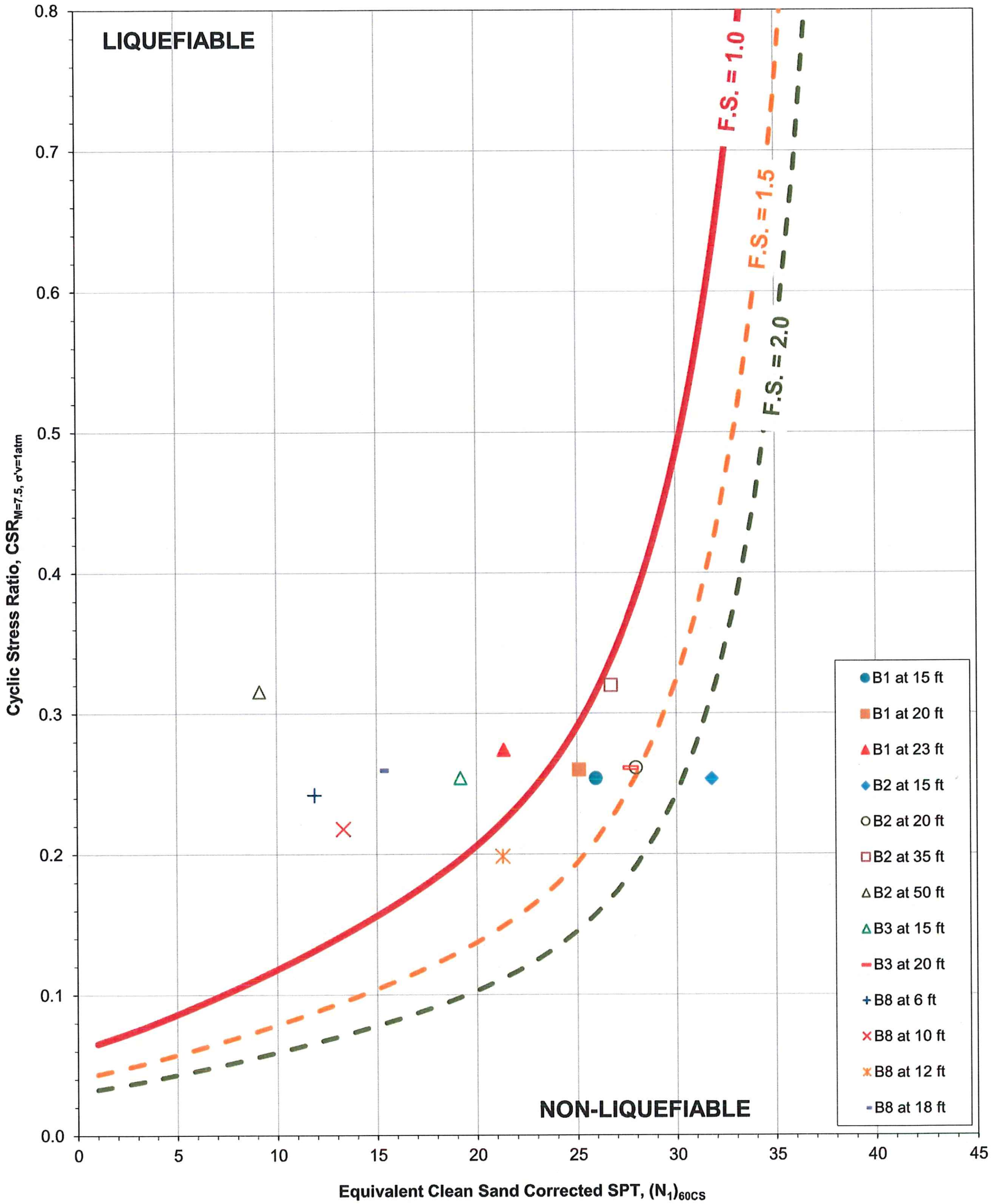
10
FIGURE

Liquefaction Analysis
(Idriss, I.M.& Boulanger, R.W., 2008 & 2010)



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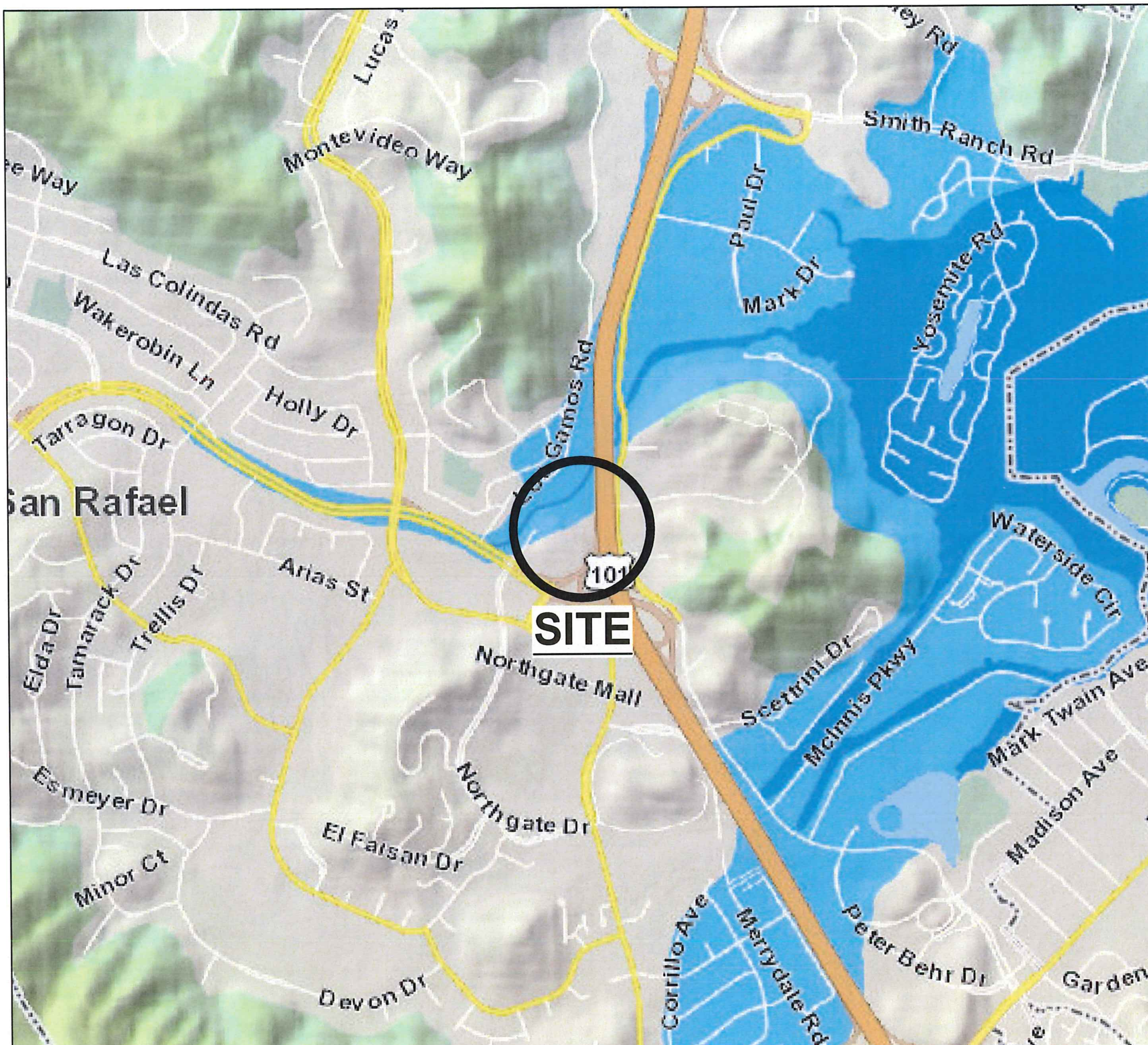
		San Rafael Hillcrest Multi-Use Development San Rafael, California	Drawn BSP Checked	11 FIGURE
		Project No. 2243.001 Date: 1/21/16		

FILE: 2243.001LiqPlots.dwg



Legend

- 100 year flood zone (Coastal)
- 100 year flood zone
- 500 year flood zone

FLOOD MAP

(Not to Scale)

NORTH

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

January 25, 2016

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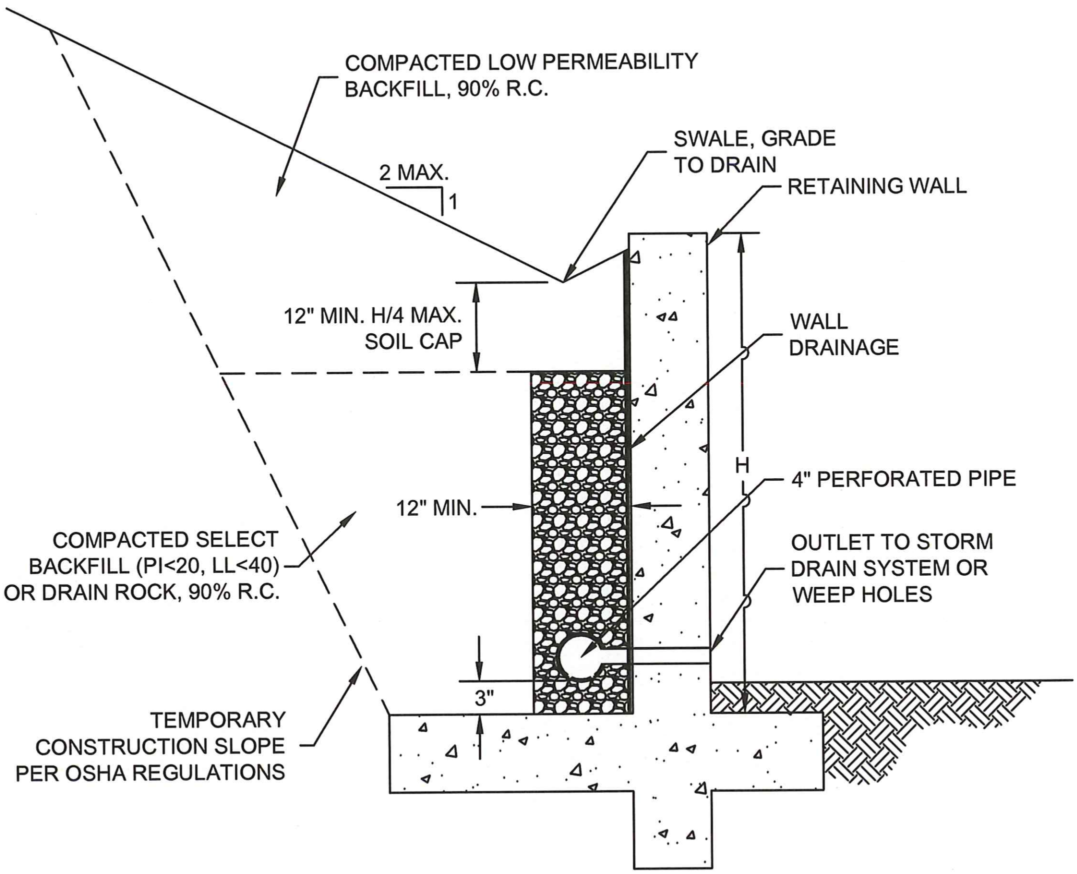
FILE: 2243.001LiqPlots.dwg



NOTES:

1. Wall drainage should consist of clean, free draining 3/4 inch crushed rock (Class 1B Permeable Material) wrapped in filter fabric (Mirafi 140N or equivalent) or Class 2 Permeable Material. Alternatively, pre-fabricated drainage panels (Miradrain G100N or equivalent), installed per the manufacturers recommendations, may be used in lieu of drain rock and fabric.

2. All retaining walls adjacent to interior living spaces shall be water/vapor proofed as specified by the project architect or structural engineer.

3. Perforated pipe shall be SCH 40 or SDR 35 for depths less than 20 feet. Use SCH 80 or SDR 23.5 perforated pipe for depths greater than 20 feet. Place pipe perforations down and slope at 1% to a gravity outlet. Alternatively, drainage can be outlet through 3" diameter weep holes spaced approximately 20' apart.

4. Clean outs should be installed at the upslope end and at significant direction changes of the perforated pipe. Additionally, all angled connectors shall be long bend sweep connections.

5. During compaction, the contractor should use appropriate methods (such as temporary bracing and/or light compaction equipment) to avoid over-stressing the walls. Walls shall be completely backfilled prior to construction in front of or above the retaining wall.

6. Refer to the geotechnical report for lateral soil pressures.

7. All work and materials shall conform with Section 68, of the latest edition of the Caltrans Standard Specifications.

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FILE: 2243.001Wall.dwg

RETAINING WALL BACKDRAIN CRITERIA

San Rafael Hillcrest
Multi-Use Development
San Rafael, California

Project No. 2243.001 Date: 1/21/16

Designed BSP
Drawn BSP
Checked

13
FIGURE

APPENDIX A
SUBSURFACE EXPLORATION (BORINGS) AND LABORATORY TESTING

1.0 Subsurface Exploration

We explored subsurface conditions at the site with eight exploratory borings drilled with truck and track mounted equipment on December 28th 29th, 2015 and on January 15, 2016 at the approximate boring locations shown on Figure 2. The borings were drilled to a maximum depth of 60.0-feet below the ground surface. The samples obtained were examined in the field, sealed to prevent moisture loss, and transported to our laboratory.

The soils encountered were logged and identified in the field in general accordance with ASTM Standard D 2487, "Field Identification and Description of Soils (Visual-Manual Procedure)." This standard is briefly explained on Figures A-1 and A-2, Soil and Rock Classification Charts, respectively. The boring log is presented on Figure A-3.

2.0 Laboratory Testing

We conducted laboratory tests on selected intact samples to verify field identifications and to evaluate engineering properties. The following laboratory tests were conducted in accordance with the ASTM standard test method cited:

- Laboratory Determination of Water (Moisture Content) of Soil, Rock, and Soil-Aggregate Mixtures, ASTM D 2216;
- Density of Soil in Place by the Drive-Cylinder Method, ASTM D 2937;
- Unconfined Compressive Strength of Cohesive Soil, ASTM D 2166;
- Amount of Material in Soils Finer Than the No. 200 (75 μm) Sieve, ASTM D 1140; and,
- Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index, ASTM D 4318.

The moisture content, dry density, unconfined compression test, and amount material passing the #200 sieve test results are shown on the exploratory Boring Logs, Figures A-3 through A-15 and the results of the Plasticity Index tests are shown on Figure A-16. The exploratory boring logs, description of soils encountered and the laboratory test data reflect conditions only at the location of the boring at the time they were excavated or retrieved. Conditions may differ at other locations and may change with the passage of time due to a variety of causes including natural weathering, climate and changes in surface and subsurface drainage.

MAJOR DIVISIONS		SYMBOL	DESCRIPTION
COARSE GRAINED SOILS over 50% sand and gravel	CLEAN GRAVEL	GW	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
	GRAVEL with fines	GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	CLEAN SAND	SW	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
	SAND with fines	SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS over 50% silt and clay	SILT AND CLAY liquid limit <50%	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
		CL	Inorganic clays of low to medium plasticity, gravely clays, sandy clays, silty clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	SILT AND CLAY liquid limit >50%	MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic clays of medium to high plasticity
HIGHLY ORGANIC SOILS		PT	Peat, muck, and other highly organic soils
ROCK			Undifferentiated as to type or composition

KEY TO BORING AND TEST PIT SYMBOLS

CLASSIFICATION TESTS

PI	PLASTICITY INDEX
LL	LIQUID LIMIT
SA	SIEVE ANALYSIS
HYD	HYDROMETER ANALYSIS
P200	PERCENT PASSING NO. 200 SIEVE
P4	PERCENT PASSING NO. 4 SIEVE

SAMPLER TYPE

- MODIFIED CALIFORNIA
- STANDARD PENETRATION TEST
- THIN-WALLED / FIXED PISTON
- HAND SAMPLER
- ROCK CORE
- X DISTURBED OR BULK SAMPLE

NOTE: Test boring and test pit logs are an interpretation of conditions encountered at the excavation location during the time of exploration. Subsurface rock, soil or water conditions may vary in different locations within the project site and with the passage of time. Boundaries between differing soil or rock descriptions are approximate and may indicate a gradual transition.

STRENGTH TESTS

TV	FIELD TORVANE (UNDRAINED SHEAR)
UC	LABORATORY UNCONFINED COMPRESSION
TXCU	CONSOLIDATED UNDRAINED TRIAXIAL
TXUU	UNCONSOLIDATED UNDRAINED TRIAXIAL
	UC, CU, UU = 1/2 Deviator Stress

SAMPLER DRIVING RESISTANCE

Modified California and Standard Penetration Test samplers are driven 18 inches with a 140-pound hammer falling 30 inches per blow. Blows for the initial 6-inch drive seat the sampler. Blows for the final 12-inch drive are recorded onto the logs. Sampler refusal is defined as 50 blows during a 6-inch drive. Examples of blow records are as follows:

25	sampler driven 12 inches with 25 blows after initial 6-inch drive
85/7"	sampler driven 7 inches with 85 blows after initial 6-inch drive
50/3"	sampler driven 3 inches with 50 blows during initial 6-inch drive or beginning of final 12-inch drive

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SOIL CLASSIFICATION CHART

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San Rafael, California

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Drawn ENE
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A-1
FIGURE

FRACTURING AND BEDDING

Fracture Classification	Spacing	Bedding Classification
Crushed	less than 3/4 inch	Laminated
Intensely fractured	3/4 to 2-1/2 inches	Very thinly bedded
Closely fractured	2-1/2 to 8 inches	Thinly bedded
Moderately fractured	8 to 24 inches	Medium bedded
Widely fractured	2 to 6 feet	Thickly bedded
Very widely fractured	greater than 6 feet	Very thickly bedded

HARDNESS

Low	Carved or gouged with a knife
Moderate	Easily scratched with a knife, friable
Hard	Difficult to scratch, knife scratch leaves dust trace
Very hard	Rock scratches metal

STRENGTH

Friable	Crumbles by rubbing with fingers
Weak	Crumbles under light hammer blows
Moderate	Indentations <1/8 inch with moderate blow with pick end of rock hammer
Strong	Withstands few heavy hammer blows, yields large fragments
Very strong	Withstands many heavy hammer blows, yields dust, small fragments

WEATHERING

Complete	Minerals decomposed to soil, but fabric and structure preserved
High	Rock decomposition, thorough discoloration, all fractures are extensively coated with clay, oxides or carbonates
Moderate	Fracture surfaces coated with weathering minerals, moderate or localized discoloration
Slight	A few stained fractures, slight discoloration, no mineral decomposition, no affect on cementation
Fresh	Rock unaffected by weathering, no change with depth, rings under hammer impact

NOTE: Test boring and test pit logs are an interpretation of conditions encountered at the location and time of exploration. Subsurface rock, soil and water conditions may differ in other locations and with the passage of time.

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ROCK CLASSIFICATION CHART

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A-2
FIGURE

BORING 1

EQUIPMENT: Truck-mounted CME 75 Drill Rig with 4-inch Solid Flight Auger

DATE: 12/28/15

ELEVATION: 37-Feet*

*REFERENCE: Google Earth 2015

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)
			26	7.7	120
			53	5.0	121
		1250 UC	19	8.4	120
P200 23.3%	LL:35 PI:16	1000 UC	21	8.4	123
			27	8.5	115
		1300 UC	27	12.0	123
			26	11.8	115

DEPTH: meters 0, 1, 2, 3, 4, 5, 6; feet 0, 5, 10, 15, 20

SAMPLE | SYMBOL (3)

4" ASPHALTIC CONCRETE

Clayey SAND / Sandy CLAY with Gravel (SC/CL)
Gray, medium brown, moist, dense, medium plasticity clay, with ~25% sub angular to angular gravels up to 1.5-inches, with ~25-50% fine-medium grained sand. [Fill]

Chert gravels present in abundance.

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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BORING LOG

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San Rafael, Caliifornia

Project No. 2243.001 Date: 12/30/15

Drawn
Checked ENE

A-3
FIGURE

BORING 1 (CONTINUED)

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH feet	DEPTH meters	SAMPLE	SYMBOL (3)	DESCRIPTION
						20				Clayey SAND / Sandy CLAY with Gravel (SC/CL) Gray, medium brown, moist, dense, medium plasticity clay, with ~25% sub angular to angular gravels up to 1.5-inches, with ~25-50% fine-medium grained sand. [Fill]
			26	11.8	115					
							7			
		1150 UC	22	16.5	114					
						25	8			SANDSTONE Tan, gray, laminated-medium bedded, hard, strong, moderately-highly weathered, sheared, with localized, green clay mineralization between bedding, sub-vertical bedding.
			75	8.7	123					
							9			Grades to contain shale interbeds.
			76/6"	5.6		30				Boring terminated at 30'6". No groundwater observed during drilling.
							10			
						35	11			
							12			
						40				

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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A-4
FIGURE

BORING 2

EQUIPMENT: Truck-mounted CME 75 Drill Rig with 4-inch Solid Flight Auger

DATE: 12/28/15

ELEVATION: 33-Feet*

*REFERENCE: Google Earth 2015

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters / feet	SAMPLE	SYMBOL (3)	DESCRIPTION
						0 – 0			4" ASPHALTIC CONCRETE
			31	8.7	107				Clayey SAND / Sandy CLAY with Gravel (SC/CL) Gray, medium brown, moist, medium dense to dense, medium plasticity clay, with ~25% sub angular to angular gravels up to 2.5-inches, with ~20-50%, fine-medium grained sand. [Fill]
						1			
						5			
			50						
						2			
			40	8.0	115				Chert gravels present in abundance.
						3 / 10			
			35	11.5	115				Sandstone gravels present in abundance.
						4			
						15			
			34	11.9	119				
						5			
						6 / 20			
P200 34.7%			29	11.1	125				

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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BORING LOG

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Drawn
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A-5
FIGURE

BORING 2
(CONTINUED)

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (3)	
							20			Clayey SAND / Sandy CLAY with Gravel (SC/CL) Gray, medium brown, moist, medium dense to dense, medium plasticity clay, with ~25% sub angular to angular gravels up to 2 1/2 inches, with ~20-50%, fine-medium grained sand. [Fill]
P200 34.7%			29	11.1	125					
						7 ▽				
										Sandy CLAY (CL) Gray, tan, wet, very stiff, medium plasticity clay, ~40-45% fine-coarse grained sand. [Alluvium]
							25			
P200 56.8%			34	21.4	90	8				
						9				Sandy CLAY with Gravel (CL) Gray, tan, mottled orange, blue, wet, stiff, with ~30% fine-coarse grained sand, with ~10% gravels (predominantly sandstone) up to 1 inch. [Alluvium]
							30			
			24	18.3	114					
						10				
										Clayey SAND with Gravel (SC) Tan, medium brown, wet, medium dense, medium-coarse grained sand, with ~45-50% medium plasticity clay, with ~15% gravels, oxidized and weathered, up to ¼ inch. [Alluvium]
						11	35			
P200 49.1%		600 UC	29	20.0	112					
						12				Sandy CLAY (CL) Tan, medium brown, wet, very stiff, medium plasticity clay, ~30-40% medium-coarse grained sand. [Alluvium]
							40			

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m³ = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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BORING LOG

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Project No. 2243.001 Date: 12/30/15

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A-6
FIGURE

BORING 2 (CONTINUED)

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (3)	DESCRIPTION
							40			Sandy CLAY (CL) Tan, medium brown, wet, very stiff, medium plasticity clay, ~20-30% medium-coarse grained sand. [Alluvium]
	P200 77.2%		47	20.3	110					
						13				
							45			Clayey Gravelly SAND (SC) Tan, medium brown, wet, dense, medium-coarse grained sand, ~15-20% medium plasticity clay, ~15-20% gravel. [Alluvium]
	P200 19.6%		72	14.4	122	14				
						15				
							50			
			8							No sample recovered.
						16				
							55			
						17				
						18				No bedrock. No sample recovered.
							60			Boring terminated at 60'0". Groundwater encountered at 21'9"

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
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Project No. 2243.001 Date: 12/30/15

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A-7
FIGURE

BORING 3

EQUIPMENT: Truck-mounted CME 75 Drill Rig with 4-inch Solid Flight Auger

DATE: 12/28/15
ELEVATION: 38-Feet*
*REFERENCE: Google Earth 2015

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (3)	DESCRIPTION
						0	0			4" ASPHALTIC CONCRETE
			23	6.4						Clayey SAND with Gravel (SC) Gray, medium brown, moist, medium dense, medium plasticity clay, with ~10 to 15% sub angular to angular gravels up to 2 1/2 inches, with ~15 to 20% fine-medium grained sand. [Fill]
			50	6.9	126	1				
							5			
P200 28.6%			41	6.4	119	2				
			35	7.6	125	3	10			Grades tan, with angular gravels.
			22	12.7	112	4				Grades gray, mottled orange.
							15			
P200 38.1%			19	11.8	119	5				
		500 UC	30	20.3	110	6	20			

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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BORING LOG

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San Rafael, Caliifornia
Project No. 2243.001 Date: 12/30/15

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A-8
FIGURE

BORING 3 (CONTINUED)

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH feet	DEPTH meters	SAMPLE	SYMBOL (3)	DESCRIPTION
		500 UC	30	20.3	110	20				Clayey SAND with Gravel (SC) Gray, medium brown, moist, medium dense, medium plasticity clay, with ~10 to 15% sub angular to angular gravels up to 2 1/2 inches, with ~15 to 20% fine-medium grained sand. [Fill]
							7			SERPENTINE Green, greasy, closely fractured, moderately hard, weak, highly weathered, sheared, serpentinite melange. [Bedrock]
			65/6" 72/11"	7.9		25	8			SANDSTONE Brown, hard-very hard, closely fractured, thin-medium bedded, moderately weathered, fine-coarse grained sandstone. [Bedrock]
										Boring terminated at 26'5". No groundwater observed during drilling.
							9			
						30				
							10			
						35	11			
							12			
						40				

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
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BORING LOG

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A-9
FIGURE

BORING 4

EQUIPMENT: Truck-mounted CME 75 Drill Rig with 4-inch Solid Flight Auger

DATE: 12/29/15

ELEVATION: 57-Feet*

*REFERENCE: Google Earth 2015

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (3)	DESCRIPTION
						0	0			Sandy CLAY (CL) Medium brown, moist, medium stiff, medium plasticity clay, with ~30% fine-coarse grained sand. [Top Soil]
			22	10.4	106					Clayey SAND with Gravel (SC) Pink, light brown, moist, medium dense, medium-coarse grained sand, with ~30-35% medium plasticity clay, with ~20% sub rounded-sub angular gravels up to 1/2 inch. [Fill]
						1				
							5			
	P200 35.7%		27	8.5	119					
						2				
			55	4.2		3	10			SANDSTONE with SHALE Light brown, thin bedded, hard, moderately strong, moderately weathered, with interbedded, laminated shale, sub vertically bedded. [Bedrock]
						4				
			100/ 11"	4.5			15			
						5				
						6	20			
			80	5.0						Boring terminated at 21'0". No groundwater observed during drilling.

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m³ = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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BORING LOG

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A-10
FIGURE

BORING 5

EQUIPMENT: Truck-mounted CME 75 Drill Rig with 4-inch Solid Flight Auger

DATE: 12/29/15
ELEVATION: 55-Feet*
*REFERENCE: Google Earth 2015

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (3)	DESCRIPTION
						0	0			Sandy CLAY (CL) Medium brown, moist, medium stiff, medium plasticity clay, with ~30% fine-coarse grained sand. [Top Soil]
										SANDSTONE with SHALE Gray, light brown, thin bedded, hard, moderately strong, moderately weathered, sheared, localized clay alteration, with interbedded, laminated shale, sub vertically bedded. [Bedrock]
			63	6.9	138	1				
							5			
			93/ 11"	4.2	140	2				Grades to predominantly shale, dark gray, waxy luster.
			80/ 11"	3.2		3	10			
										Boring terminated at 10'5". No groundwater observed during drilling.
						4				
							15			
						5				
						6	20			

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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BORING LOG

Hillcrest San Rafael
Multi-Use Development
San Rafael, Caliifornia

Project No. 2243.001 Date: 12/30/15

Drawn ENE
Checked

A-11
FIGURE

BORING 6

EQUIPMENT: Truck-mounted CME 75 Drill Rig with 4-inch Solid Flight Auger

DATE: 12/29/15

ELEVATION: 58-Feet*

*REFERENCE: Google Earth 2015

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (3)	DESCRIPTION
						0	0			6" ASPHALTIC CONCRETE
			21	6.9	126					Gravelly SAND with Clay (SC) Gray, moist, medium dense, medium-coarse grained sand with ~30-40% sub angular gravels up to 1 1/2 inch, with ~15% medium plasticity clay. [Fill]
						1				SHALE and SANDSTONE Dark gray, laminated-thinly bedded, low-moderately hard, moderately weathered, sheared shale, with lesser interbedded sandstone. [Bedrock]
			36	4.5	133		5			
						2				Grades moderately-highly weathered, with localized pyrite mineralization.
						3	10			
			65	4.0	145					Grades to 100% shale, no sandstone.
						4				
							15			
			70	4.4	134	5				
						6	20			
			91	4.0	145					Boring terminated at 21'6". No groundwater observed during drilling.

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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BORING LOG

Hillcrest San Rafael
Multi-Use Development
San Rafael, Caliifornia

Project No. 2243.001 Date: 12/30/15

Drawn ENE
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A-12
FIGURE

BORING 7

EQUIPMENT: Truck-mounted CME 75 Drill Rig with 4-inch Solid Flight Auger

DATE: 12/29/15

ELEVATION: 24-Feet*

*REFERENCE: Google Earth 2015

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)
			27	6.7	
			51	6.6	121
			70	11.1	116
			88/ 11"	5.7	
			80	7.7	
			80	6.8	

DEPTH: meters 0, 1, 2, 3, 4, 5, 6; feet 0, 5, 10, 15, 20

SAMPLE | SYMBOL (3)

Sandy CLAY (CL)

SANDSTONE

Tan, medium brown, very thin-medium bedded, moderately hard, moderately weathered, fine-medium grained sandstone, with localized clay alteration. [Bedrock]

Grades light brown, hard.

Grades medium-coarse grained, very hard.

Boring terminated at 20'0".

No groundwater observed during drilling.

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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BORING LOG

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San Rafael, Caliifornia

Project No. 2243.001 Date: 12/30/15

Drawn ENE
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A-13
FIGURE

BORING 8

EQUIPMENT: Portable Track Rig with 4 1/2 inch Solid Fight Auger

DATE: 1/15/16

ELEVATION: 18-Feet*

*REFERENCE: Google Earth 2016

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (3)	DESCRIPTION
						0	0			Sandy CLAY (CL) Medium brown, moist, medium stiff, medium plasticity clay, with ~20% fine-medium grained sand, with ~20% silt. [Fill]
			8	6.6	121	1				Sandy CLAY with Gravel (CL) Medium brown, mottled gray, orange, moist, medium stiff, medium plasticity clay, with ~20% fine-coarse grained sand, with ~20% sub angular-angular gravels up to 2 inches. [Fill]
							5			Clayey SAND with Silt (SC) Medium brown, moist, loose to medium stiff, medium plasticity clay, with ~20-40% fine sand and silt. [Alluvium]
	LL:40 PI: 20		6	11.1	116	2				
										Groundwater at 9'0".
						3	10			No recovery at 10'0"
		1,200 UC	9	5.7						
			20	7.7		4				Grades to contain ~10% gravels up to 1/4 inch.
						5	15			
P200 28.0%			14	6.8		6	20			

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

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BORING 8 (CONTINUED)

OTHER TEST DATA	OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters / feet	SAMPLE	SYMBOL (3)	DESCRIPTION
						20			
			100/6"	20.3	110	7			SANDSTONE Gray, hard-very hard, fresh, closely fractured, thin-medium bedded, lightly weathered, fine-coarse grained sandstone. [Bedrock]
						25 / 8			Boring terminated at 26'0". Groundwater level at 9'0".
						9			
						30 / 10			
						35 / 11			
						12			
						40			

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
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BORING LOG

Hillcrest San Rafael
Multi-Use Development
San Rafael, Caliifornia

Project No. 2243.001 Date: 12/30/15

Drawn ENE
Checked

A-15
FIGURE

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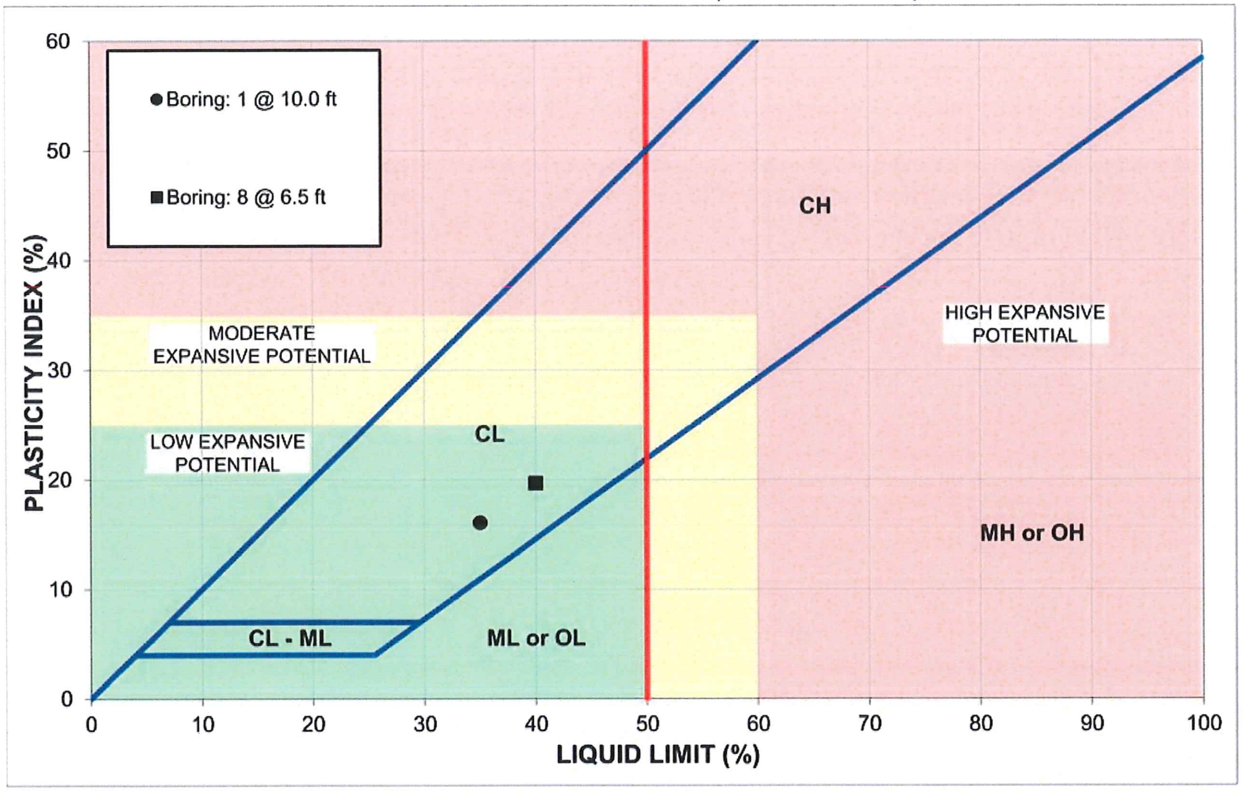
ATTERBERG LIMITS TEST (ASTM D 4318)



Sample	Classification	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
Boring: 1 @ 10.0 ft	Sandy CLAY with Gravel (CL) medium dark gray	35	19	16
Boring: 8 @ 6.5 ft	Sandy CLAY (CL) medium brown	40	20	20

PI = 0-3: Non-Plastic
PI = 3-15: Slightly Plastic
PI = 15-30: Medium Plasticity
PI = >30: High Plasticity

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PLASTICITY INDEX TEST RESULTS

Hillcrest San Rafael
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San Rafael, California
Project No. 2243.001 Date: 2/220/16

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A-16
FIGURE