

**GEOTECHNICAL INVESTIGATION
KELTY RESIDENCE
380 MARGARITA DRIVE
SAN RAFAEL, CALIFORNIA**

October 14, 2021

Job No. 3270.001

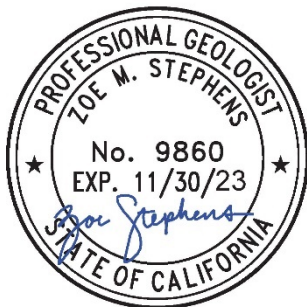
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CERTIFICATION

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1.0 INTRODUCTION

This report presents the results of our preliminary geotechnical investigation for the proposed residence located at 380 Margarita Drive in San Rafael, California. As shown on Figure 1, the project site is located on the north side Margarita Drive on a hillside south of China Camp State Park.

Our work was performed in accordance with our Agreement for Professional Services authorized August 19, 2021. The purpose of our investigation was to explore subsurface conditions and to develop preliminary geotechnical criteria for design and construction of the proposed residence. The scope of our services includes:

- Reviewing published geologic mapping and geotechnical background information from our files, including existing geotechnical data from previous site investigations.
- Performing supplemental subsurface exploration with three borings located within the general vicinity of the planned improvements.
- Evaluating relevant geologic hazards including seismic shaking, slope instability and other hazards.
- Preparing geotechnical recommendations and design criteria related to building foundations, site grading, retaining walls, seismic design, and other geotechnical-related items.
- Preparing this preliminary geotechnical report which summarizes subsurface exploration, evaluation of relevant geologic hazards, and preliminary geotechnical recommendations and design criteria.

This report completes our Phase 1 services for the project. Subsequent phases of work should include geotechnical plan review and observation and testing of geotechnical-related work items during construction.

2.0 PROJECT DESCRIPTION

Based on our review of preliminary plans and discussions with the owner, we understand the project is expected to include developing the site with a single-family residence on the existing cut fill pad. A pool is planned along the east side of the building pad, and a garage will be located roughly in the southwest corner. Several new retaining walls will be required as part of the project. Ancillary improvements will include a new driveway, firetruck hammerhead, parking area, underground utilities, site drainage, and landscaping. The approximate location of the new improvements is shown on the Site Plan, Figure 2.

Minor to moderate site grading is anticipated for the new driveway leading up the hill to the residence. Retaining walls up to 16 feet tall will support the downslope edge of the planned new pool. Fill placement is planned for the driveway and to backfill retaining walls. Light to moderate foundation loads are expected for the structures.

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 Complex. 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 Complex. 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.

The proposed buildings are situated on a previously excavated building pad located on sloping hillside on the south facing side of a north-trending ridgeline. Regional geologic mapping (Rice, 1976) indicates the site is underlain by mélangé bedrock of the Franciscan Complex. Most of the northern portion of the parcel is mapped as debris flow landslide deposits. The building area is not mapped within this slide, but rather is located on a topographic nose mapped as Franciscan bedrock with chert outcrops above it. A Regional Geologic Map and descriptions of the mapped geologic units are shown on Figure 3.

3.2 Seismicity

The project site is located within the seismically active San Francisco Bay Area and will therefore experience the effects of future earthquakes. Earthquakes are the product of the build-up and sudden release of strain along a “fault” or zone of weakness in the earth's crust. Stored energy may be released as soon as it is generated, or it may be accumulated and stored for long periods of time. Individual releases may be so small that they are detected only by sensitive instruments, or they may be violent enough to cause destruction over vast areas.

Faults are seldom single cracks in the earth's crust but are typically comprised of localized shear zones which link together to form larger fault zones. Within the Bay Area, faults are concentrated along the San Andreas Fault zone. 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 becomes a long, high-amplitude motion when moving through soft ground materials, such as Bay Mud.

3.2.1 Regional Active Faults

The California Geological Survey (previously known as the California Division of Mines and Geology), defines a “Holocene-active fault” as one that has had surface displacement within Holocene time (the last 11,700 years). CGS has mapped various faults in the region as part of their Fault Activity Map of California (CGS, 2010). Many of these faults are shown in relation to the project site on the attached Active Fault Map, Figure 4. The nearest known Holocene-active faults are the San Andreas, Hayward, and San Gregorio Faults. The San Andreas and San Gregorio Faults are located approximately 16.6 kilometers and 17.8 kilometers to the southwest¹, respectively. The Hayward Fault is located roughly 11.8 kilometers northeast of the site.

3.2.2 Historic Fault Activity

Numerous earthquakes have occurred in the region within historic times. The results of our USGS earthquake search catalogue indicates that at least 13 earthquakes with a Richter Magnitude of 5.0 or larger have occurred within 100 kilometers (62 miles) of the site between 1900 and 2019. The approximate locations of these earthquakes are shown on the Historic Earthquake Map, Figure 5.

3.2.3 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, 2013) 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. 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.

Conclusions from the most recent UCERF3 and USGS indicate the highest probability of an earthquake with a magnitude greater than 6.7 originating on any of the active faults in the San Francisco Bay region by 2043 is assigned to the Hayward/Rodgers Creek Fault system. The Hayward Fault is located approximately 11.8 kilometers northeast of the site and is assigned a probability of 33 percent. The San Andreas Fault, located approximately 16.6 kilometers southwest of the site, is assigned a 22 percent probability of an earthquake with a magnitude greater than 6.7 by 2043. 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.

¹ Distances to faults estimated using Caltrans ARS Online (v2.3.09), accessed October 18, 2021.

3.3 Surface Conditions

The project site encompasses an irregularly-shaped, approximately 11.77-acre parcel (APN 015-320-03) located between Margarita Drive and Harry Barbier Memorial Park in northeastern San Rafael. The site is bordered by open space to the east and north, single family residences to the south and west. The average slope of the parcel is very steep, about 60% or 1.6:1 (horizontal:vertical) but varies locally. Access to the site is provided from a shared driveway for 374, 376, and 378 off of Margarita Drive. Only the southern portion of the parcel is being developed with a single-family residence and new driveway, the remainder of the parcel is to remain unimproved.

The ground surface within the planned building area for the home, pool, and garage is generally level, as a cut-fill pad and access road were previously graded at this site. A new driveway and fire access turn around is also planned as part of the project. Elevations from the bottom of the new road to the existing building pad range from 552 feet to 594 feet. The property is currently unimproved and is vegetated with native grasses, shrubbery, and a few trees. Hillside drainage swales or ravines cross the north and south ends of the property.

3.4 Subsurface Exploration and Laboratory Testing

We explored subsurface conditions on August 20, 2021, with three borings drilled with a track mounted drill to depths between about 13.5- and 16.5-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-5.

Laboratory testing of soil samples from the exploratory borings include determination of moisture content, dry density, and percent passing the No 200 sieve. Lab test results are presented on the boring logs. The laboratory testing program also is discussed in more detail in Appendix A.

3.5 Reference Geotechnical Data and Aerial Photo Review

A previous geotechnical investigation was completed by John C. Hom and Associates Inc. for the previous owner (JCH, 2015). These investigations included excavating six exploratory borings at the approximate locations shown on Figure 2. The boring logs from that report are presented in Appendix B, Reference Geotechnical Data.

Review of digital aerial photos of the site shows that the cut fill pad and access road appear to have been constructed around 1948. Since then, the site has remained largely unchanged.

3.6 Subsurface Conditions and Groundwater

Based on our recent exploration and review of reference data, the project site is generally underlain by between 0 to 7 feet of sandy and clayey fill soils over sandstone bedrock of the Franciscan Complex. The surface soils are generally medium dense clayey sand and silty sand with varying amounts of gravel. Within the building area, soils are likely a mix of native colluvial soils (towards the north) and old fill (towards the south) from the grading and creation of the building pad.

The bedrock encountered in the borings and test pits predominantly consists of sandstone and shale of the Franciscan Complex unit, which is generally moderately to highly weathered, exhibits moderate hardness, and yields weak to moderate strength.

Groundwater was not encountered in the subsurface exploration. However, because the borings were not left open for an extended period of time, a stabilized depth to groundwater may not have been observed. Groundwater elevations fluctuate seasonally with higher groundwater levels during periods of intense rainfall. Groundwater seepage will likely flow downslope along the soil to bedrock contact during winter and early spring.

4.0 GEOLOGIC HAZARDS

This section summarizes our review of commonly considered geologic hazards and discusses their potential impacts on the planned improvements. The primary geologic hazards which could affect the proposed development include strong seismic ground shaking, erosion, and slope instability. Other geologic hazards are judged less than significant regarding the proposed project. Geologic hazards, potential impacts and mitigation measures are discussed in further detail in the following sections.

4.1 Seismic Shaking

The site will likely experience seismic ground shaking similar to other areas in the seismically active Bay Area. The intensity of ground shaking will depend on the characteristics of the causative fault, distance from the fault, the earthquake magnitude and duration, and site-specific geologic conditions. Estimates of peak ground accelerations are based on either deterministic or probabilistic methods.

Deterministic methods use empirical attenuation relations that provide approximate estimates of median peak ground accelerations. A summary of the active faults that could most significantly affect the planning area, their maximum credible magnitude, closest distance to the center of the planning area, and probable peak ground accelerations are summarized in Table 1. The calculated accelerations should only be considered as reasonable estimates. Many factors (e.g., soil conditions, orientation to the fault, etc.) can influence the actual ground surface accelerations.

Table 1 – Deterministic Peak Ground Accelerations for Active Faults

Fault	Moment Magnitude for Characteristic Earthquake	Closest Estimated Distance (km)	Median Peak Ground Acceleration (g)	Median PGA +1 Std Dev (g)
San Andreas	8.0	16.6	0.28	0.51
Hayward	7.3	11.8	0.28	0.51
San Gregorio	7.4	17.8	0.22	0.40
Rodgers Creek	7.3	21.3	0.19	0.34
West Napa	6.6	33.1	0.09	0.16

Reference: Abrahamson & Silva, Boore & Atkinson, Campbell & Bozorgnia, and Chiou & Youngs (2014) NGA models using $V_{s30} = 560$ m/s.

Probabilistic Seismic Hazard Analysis 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.

Ground shaking can result in structural failure and collapse of structures or cause non-structural building elements (such as light fixtures, shelves, cornices, etc.) to fall, presenting a hazard to building occupants and contents. Compliance with provisions of the most recent version of the California Building Code (2019 CBC) should result in structures that do not collapse in an earthquake. Damage may still occur, and hazards associated with falling objects or non-structural building elements will remain.

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

Evaluation: Minimum recommendations include design of new structures in accordance with the provisions of the 2019 California Building Code or subsequent codes in effect when final design occurs. Recommended seismic design coefficients and spectral accelerations are presented in Section 5.1 of this report.

4.2 Erosion

Sandy soils on most slopes or clayey soils on steep slopes are susceptible to erosion when exposed to concentrated surface water flow. The potential for erosion is increased when established vegetation is disturbed or removed during normal construction activity.

The proposed improvements indicate that although much of the building pad and driveway will be covered with new buildings, pavements, or concrete flatwork, the remainder of the site will remain unimproved. Although significant erosion is generally not anticipated within the project area, the hillside both above and below the planned improvements contain a thin layer of sandy soil on steeply sloping terrain. Therefore, we judge the risk of erosion impacting the project is moderate.

Evaluation: The site drainage system should be designed to collect surface water from the maximum credible rainfall event and discharging it into an established storm drainage system. V-ditch drainage systems should be utilized upslope of the proposed improvements to direct surface water from the hillside away from the residence. The project Civil Engineer is responsible for designing the site drainage system.

An erosion control plan could be developed prior to construction per the current guidelines of the California Stormwater Quality Association's Best Management Practice Handbook. Additionally, regular monitoring of the upslope areas should be performed, particularly during and following periods of heavy rainfall. Regular maintenance of upslope areas should also be performed and should include maintaining vegetative cover on slopes, clearing debris from the v-ditches and drain inlets, and promptly repairing any erosion or shallow instabilities that occur.

4.3 Slope Instability

The development will be located on a hillside which is locally inclined as steeply as about 1.6:1. Based on our review of published geologic mapping, several portions of the parcel are mapped as debris flow deposits as shown on Figure 3. Unimproved areas on the north side of the parcel and upslope of the planned residence could experience debris flow type landslides similar to the observed slide scars to the east of the site. Deep excavations into the hillside can further increase the likelihood of slope instability. However, shallow bedrock is exposed at the surface in several outcrops throughout the proposed building area as well as directly upslope from the cut fill pad. Since there does not appear to be evidence of instability at the building site, we judge there will be a low risk of instability within the developed area of the site and a moderate risk of slope instability in the undeveloped areas within the parcel.

Evaluation: Supplemental exploration with exploratory trenches and geology site inspection/mapping further upslope could be performed to better evaluate the potential for instability in other areas of the parcel. Debris catchment structure or deflection wall may be utilized upslope of the planned buildings if additional protection is desired, as discussed in Section 5.5. Retaining walls upslope of the building area should have at least 1-ft of freeboard at the top of the wall to catch soil or rock that may ravel off the hillside.

5.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our investigation, we conclude the geologic and geotechnical site conditions are suitable for the proposed improvements. The primary geotechnical considerations will include designing the improvements to resist strong seismic ground shaking and potential instability of the upslope areas above the proposed development that could impact the new

residence and other improvements. Additional discussion and preliminary conclusions and recommendations addressing these, and other considerations are presented in the following sections.

5.1 Seismic Design

Minimum mitigation of ground shaking includes seismic design of new structures in conformance with the provisions of the most recent edition (2019) of the California Building Code. The magnitude and character of these ground motions will depend on the particular earthquake and the site response characteristics. Based on the interpreted subsurface conditions and proximity of several nearby faults, we recommend the CBC coefficients and site values shown in Table 3, be used to calculate the design base shear of new improvements as applicable.

Table 3 – 2019 California Building Code Seismic Design Criteria

Parameter	Design Value
Site Class	B (estimated)
Site Latitude	37.9845°N
Site Longitude	-122.5017°W
Spectral Response (short), S_s	1.5 g
Spectral Response (1-sec), S_1	0.6 g
Site Coefficient, F_a	1.0
Site Coefficient, F_v	1.0
Spectral Response (Short), S_{MS}	1.5 g
Spectral Response (1 sec), S_{M1}	0.6 g
Design Spectral Response (short), S_{DS}	1.0 g
Design Spectral Response (1 sec), S_{D1}	0.4 g
MCE_G PGA Adjusted, PGA_M	0.529 g

Reference: ATC Hazard by Location, accessed on October 20, 2021.

5.2 Site Grading

Site grading and earthwork should be performed in accordance with the recommendations and criteria outlined in the following sections.

5.2.1 Site Preparation

Clear over-sized debris and organic material from areas are to be graded. Debris, rocks larger than six inches, and vegetation are not suitable for structural fill and should be removed from the site.

Where fills or other structural improvements are planned, the subgrade surface should be scarified to a depth of eight inches, moisture conditioned to above the optimum moisture content, and compacted to at least 90 percent relative compaction. Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry

density, as determined by ASTM D1557. Subgrade preparation should extend a minimum of five feet beyond the planned building envelopes in all directions. The subgrade should be firm and unyielding when proof-rolled with heavy, rubber-tired construction equipment. If soft, wet, or otherwise unsuitable materials are encountered at subgrade elevation during construction, we will provide supplemental recommendations to address the specific condition.

5.2.2 Excavations

Site excavations for new underground utilities, retaining walls, building foundations and other improvements will encounter from 0 to 7 feet of clayey and sandy soils over sandstone and shale bedrock of variable weathering, strength, and hardness. The bedrock encountered in our borings generally exhibited low to moderate hardness and strength and is highly to completely weathered. Temporary (steeper) cut slopes may be required during construction. For planning purposes, the soil layer may be designed for a Cal-OSHA Type "C" soil profile, and the underlying weathered bedrock as Cal-OSHA Type "A" soil profile.

Temporary, short (6-ft typical), vertical cuts are possible during dry conditions and for short term excavations, such as cuts for retaining wall construction. However, adversely bedded rock or seepage/weak soils near the ground surface may require lower cut heights, and/or temporary vertical supports for soil nails above the cut.

Based on our subsurface exploration, we judge that most of the site excavation can be performed with typical equipment, such as medium-size dozers and excavators. However, Franciscan bedrock contains inclusions and zones of harder, more resistant rock which cannot be efficiently excavated with typical equipment and requires specialized techniques or equipment to excavate (e.g., jackhammers or hoe-rams). Therefore, we recommend inclusion of a line item and clear definition for "hard rock excavation" in the project bid documents. If hard rock is encountered during construction which prohibits excavation to the required depths, we should be consulted to observe conditions and revise our recommendations and/or design criteria as appropriate. Reducing planned excavation depths will also reduce the volume of rock excavation and resulting costs.

5.2.3 Fill Materials, Placement and Compaction

Fill materials should consist of non-expansive materials that are free of organic matter, have a Liquid Limit of less than 40 (ASTM D 4318), a Plasticity Index of less than 20 (ASTM D 4318), and a minimum R-value of 20 (California Test 301). The fill material should contain no more than 50 percent of particles passing a No. 200 sieve and should have a maximum particle size of four inches. Onsite soils may be suitable for use as fill, provided they meet the criteria specified above. Any imported fill material needs to be tested to determine its suitability.

Fill materials should be moisture conditioned to above the optimum moisture content prior to compaction. Properly moisture conditioned fill materials should subsequently be placed in loose, horizontal lifts of eight inches-thick or less and uniformly compacted to at least 90 percent relative compaction. Where fill thicknesses are greater than five feet, fill materials should be compacted to at least 92 percent relative compaction. In pavement areas, the

upper 12 inches of fill should be compacted to at least 95 percent relative compaction. The maximum dry density and optimum moisture content of fill materials should be determined in accordance with ASTM D1557.

5.2.4 Bulking and Shrinkage

During site grading, bulking or shrinkage can occur as the soil and bedrock is excavated and replaced as compacted fill. Bulking and shrinkage estimates are variable based on soil type, loading (thickness of fill) and degree of compaction. Some rough estimates are presented below. A laboratory testing program that includes compaction curves is recommended to refine estimates of grading quantities.

For excavation of on-site soil for use as fill placed and compacted 90% relative compaction, we estimated net volume change of 10% shrinkage. For on-site bedrock excavated for use as fill placed and compacted 90% relative compaction, we estimated net volume change of 5 to 10% bulking.

Due to significant bulking of materials placed in trucks for off-haul and disposal, we recommend excavated soil for removal and disposal be bid on a per ton basis.

For deep fills, some compression of the deep soil will occur from the overburden load and will likely cause some settlement at the ground surface. Long term compression settlement is estimated at 0.5 to 1% of the fill height and will typically occur within 5 to 10 years after construction.

5.3 Foundation Design

Bedrock is relatively shallow throughout the site, with about 0 to 7 feet of clayey and sandy soils overlying sandstone and shale of the Franciscan Complex. Shallow foundations can be utilized provided they maintain uniform support on competent bedrock and are deepened so they have adequate horizontal confinement. We anticipate that the north (cut) side of the pad will expose shallow bedrock, while the south (fill) side of the pad will expose thicker fill soils. Shallow footings on the north side should be deepened to provide uniform bearing support on the weathered bedrock to minimize potential for differential settlement. Drilled, cast-in-place piers could also be utilized for the building foundation to extend through soils and into the underlying bedrock on the south side of the pad. Drilled piers or rock anchors can be utilized for overturning resistance. Geotechnical foundation design criteria are presented in Table 4.

Table 4 – Foundation Design Criteria

<u>Shallow Spread Footings</u>	
Minimum depth: ¹	18 inches
Allowable bearing capacity: ²	
Weathered Bedrock	3,000 psf
Base friction coefficient:	0.35
Lateral passive resistance: ^{3,4}	
Sandy Clay Soils	300 pcf
Weathered Bedrock	450 pcf
<u>Drilled Piers or Rock Anchors</u>	
Min. Diameter:	
Drilled Pier	18 inches
Rock Anchor	6 inches
Minimum Pier Embedment into Bedrock:	10 feet
Allowable skin friction ^{2,5,6} :	
Sandy Clay Soils	800 psf
Weathered Bedrock	1,500 psf
Lateral passive resistance ⁷ :	
Sandy Clay Soils	250 pcf
Weathered Bedrock	400 pcf

Notes:

- (1) Foundations to bear on weathered bedrock. Maintain at least 10 feet horizontal distance from base of footing to slope.
- (2) May increase design values by 1/3 for total design loads including wind or seismic.
- (3) Equivalent fluid pressure. Not to exceed 4000 psf.
- (4) Ignore uppermost foot of resistance.
- (5) Anchors should be specified with a minimum bonded length and minimum capacity. All rock anchors shall be double corrosion-protected anchors and should be tested to at least 1.33 times the design load per the “Recommendations for Prestressed Rock and Soil Anchors” by the Post-Tensioning Institute, Phoenix, Arizona.
- (6) Use 80 percent of skin friction for uplift design.
- (7) Apply lateral passive resistance over width of two pier diameters.

5.4 Retaining Walls

We understand retaining walls will be utilized to support the new driveway, stabilize cuts, and to support the pool on the east side of the residence. Taller site retaining walls can be constructed by laying back slopes, construction walls and backfilling, or by making vertical cuts supported with shotcrete-faced and soil walls. The soil nail walls can be designed as a temporary shoring wall or could be part of a permanent building wall. Reinforced earth walls may be a good choice for site walls that support fills.

Retaining walls that can deflect at the top such as site walls can be designed using the unrestrained criteria shown in Table 5. Walls that are structurally connected at the top and not allowed to deflect, such as basement or tied-back walls are considered restrained. Restrained conditions are commonly designed using a uniform earth pressure distribution rather than an equivalent fluid pressure. Lateral support can be obtained from either passive soil resistance (i.e., keyways) or frictional sliding resistance of footings or from tiebacks. In addition to the soil loads, the retaining walls should be designed to resist temporary vehicular or seismic loads.

Table 5 - Retaining Wall Design Criteria

Foundations: See Table 4

<u>Active Earth Pressure</u>	<u>Unrestrained</u> ²	<u>Restrained</u> ³	
Level Ground	40 pcf	30 X H psf	
2:1 Slope	60 pcf	40 X H psf	
<u>Seismic Surcharge</u> ^{3,4}	15 x H psf		
<u>Vehicular Surcharge</u> ^{3,4}	50 psf upper 10 feet		
<u>Tiebacks or Soil Nails</u> ⁵ :			
Minimum Diameter:	5 inches		
Design Skin Friction:	1,500 psf		
Unbonded Zone:	0.7 x Wall Height, 6 Feet Min		
	<u>Phi</u> ⁶	<u>C (psf)</u> ⁷	<u>Gamma (pcf)</u> ⁸
Sandy Clay Soils (upper 5')	30°	750	125
Weathered Bedrock	32°	1,500	130

Notes:

- (1) Interpolate earth pressures for intermediate slopes.
- (2) Equivalent fluid pressure.
- (3) Rectangular distribution. H = Wall Height = top of soil backfill to bottom of wall.
- (4) The factor of safety for short-term seismic conditions can be reduced to 1.1 or greater.
- (5) Tiebacks should be specified with a minimum bonded length and minimum capacity. All tiebacks shall be double corrosion protected anchors that are installed and tested to at least 1.33 times the design load per the "Recommendations for Pre-stressed Rock and Soil Anchors" by the Post-Tensioning Institute, Phoenix, Arizona.
- (6) Angle of Internal Friction, effective stress.
- (7) Apparent (effective) Cohesion, for seismic conditions 250 psf of additional cohesion may be included.
- (8) Unit Weight of Soil

All walls higher than 3-feet require drainage to prevent the build-up of hydrostatic pressure. Either Caltrans Class 1B permeable material within filter fabric, drainage panels, or Caltrans Class 2 permeable material can be used. The project Architect should design a water-proofing system for walls adjacent to living space. The drainage should be collected in 4-inch, perforated, Schedule 40 PVC drain line placed at the base of the wall or discharged through weep-holes in the case of soil nail or cast-in-place concrete walls. Seepage collected in the drains should be conveyed in a closed pipe system to a suitable discharge outlet well away from the structures.

To maintain the wall drainage system, clean-outs must be provided for perforated pipes at the upstream end. Sweep fittings should be used at all major changes in direction. A typical retaining wall drain detail is shown on Figure 6. Retaining wall backfill should be compacted in accordance with the recommendations presented in site grading.

Retaining walls on the upslope side of the project site should be designed to include 1-ft of freeboard at the top to catch soil or debris generated from natural gravitational erosion of the hillside.

5.5 Debris Barriers

As discussed above, debris impact with the planned structures could occur if instability upslope of the project results in the release of a sufficient volume of debris. A possible mitigation option could be a new debris-catchment structure with a minimum height of six feet and sited about 10 to 20 feet upslope from the planned buildings. While various structure types are feasible, a debris fence consisting of a combination of mesh, posts, and anchored cables would likely be relatively cost-effective and would allow for entrapment of debris upslope of the concrete v-ditch above the retaining wall. Regular maintenance, including visual inspections and as-needed removal of debris would need to be performed to confirm the catchment structure is performing as intended.

5.6 Interior Concrete Slabs-On-Grade

Reinforced concrete slab-on-grade floors are judged to be appropriate for the proposed structures. The concrete slabs-on-grade may be poured monolithically or separated with a cold joint. We recommend that interior concrete slabs have a minimum thickness of five inches and be reinforced with steel reinforcing bars (not mesh) with rebar extending through crack control joints. Slabs should be placed on a moist subgrade to reduce potential for future shrink/swell behavior. The project Structural Engineer should specifically design the concrete slabs, including locations of crack control joints.

To reduce the potential for moisture to move upward through the slab, a four-inch layer of clean, free draining, ¾-inch angular gravel should be placed beneath interior concrete slabs to form a capillary moisture break. The gravel 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 compacted base rock. The vapor barrier shall meet the ASTM E1745 Class A requirements and be installed per ASTM E1643. 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.

We note that over time, placing sand between the vapor barrier and concrete is becoming less common because of elevated interior moisture contents. If sand is used, it should be dry, and if it is not used, the slab should be carefully designed with a lower water-cement ratio (generally less than 0.45) since eliminating the sand can cause cracking or “curling” of the new concrete. For slabs that are not sensitive to moisture vapor, we recommend at least four inches of Class 2 aggregate base (Caltrans, 2015) compacted to 95 percent relative compaction.

Where the gravel capillary break layer is placed beneath slabs, there is a possibility that water will tend to collect in the gravel layer and become trapped. If this condition occurs, the potential for moisture issues at the surface of the slab will be increased. One method of minimizing the potential for this to occur would be to construct a subdrain trench through and just below the gravel layer so that water collected in this area can escape. The subdrain should extend at least 12 inches below the base of the slab and 6 inches below the bottom of the gravel layer and would consist of a four-inch-diameter, perforated pipe (Schedule 40 PVC) surrounded by gravel with non-woven filter fabric (Mirafi 140N or approved equal) lining the trench. The subdrain would connect to the gravel layer beneath the slab, and the pipe should lead (at a minimum 0.5 percent slope) to a storm drain or another suitable outlet point. The perforated pipe should transition to nonperforated pipe at a point three feet inside the perimeter footing of the structure. A compacted clayey soil plug should be used at the point where the outlet pipe penetrates the perimeter footing to prevent seepage from back-flowing into the underslab gravel layer.

5.7 Exterior Concrete Slabs

Exterior concrete walkway slabs and other concrete slabs that are not subjected to vehicle loads should be a minimum of four inches thick and underlain with four inches or more of Class 2 aggregate base. The aggregate base should be moisture conditioned to near optimum and compacted to at least 95 percent relative compaction. The upper eight inches of subgrade on which aggregate base is placed should be prepared as previously discussed under Section 5.2.

Where improved performance is desired (i.e., reduced risks of cracking or small movements), exterior slabs can be thickened to five inches and reinforced with steel reinforcing bars (not welded wire mesh). We recommend crack control joints no farther than six feet apart in both directions, and that the reinforcing bars extend through the control joints. Some movement or offset at sidewalk joints should be expected as the underlying soils expand and shrink from seasonal moisture changes.

5.8 Site and Foundation Drainage

New grading could result in adverse drainage patterns causing water to pond around the new buildings. Careful consideration should be given to design of finished grades at the site. We recommend that the building areas be raised slightly and that the adjoining landscaped areas be sloped downward at least 0.25 feet for five feet (five 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 five feet (two percent).

Concrete v-ditches should be constructed on the upslope side of retaining wall per the civil engineer to redirect surface water from the hillside away from the proposed residence. Roof gutter downspouts may discharge onto pavements but should not discharge onto landscaped

areas immediately adjacent to the buildings. Provide area drains for landscape planters adjacent to buildings and collect downspout discharges into a tight pipe collection system that discharges well away from the building foundations. Site drainage should be discharged away from the building area and outlets should be designed to reduce erosion. Site drainage improvements should be connected into an established storm drainage system.

5.9 Underground Utilities

Site excavations for new underground utilities and other improvements will encounter up to about four to nine feet of medium stiff to stiff clayey soils over Franciscan bedrock of variable weathering, strength, and hardness. Trench excavations having a depth of five feet or more must be excavated and shored in accordance with OSHA regulations, as discussed in Section 5.2.

Unless otherwise recommended by the pipe manufacturer, pipe bedding and embedment materials should consist of well-graded sand with 90 to 100 percent of particles passing the No. 4 sieve and no more than five percent finer than the No. 200 sieve. Crushed rock or pea gravel may also be considered for pipe bedding. Provide the minimum bedding thickness beneath the pipe in accordance with the manufacturer’s recommendations (typically three to six inches). Trench backfill may consist of on-site soils, provided that the soil meets the fill criteria outlined in Section 5.2. Trench backfill should be moisture conditioned and placed in thin lifts and compacted to at least 90 percent. The upper 12 inches of backfill should be compacted to at least 95 percent in new pavement areas. The Contractor should use equipment and methods that are suitable for work in confined areas without damaging utility conduits.

5.10 Pavements

5.10.1 Asphalt-Concrete Pavement Sections

New pavements are expected to include both rigid concrete pavements flexible asphalt pavements. We have calculated thicknesses for asphalt pavements in accordance with Caltrans procedures for flexible pavement design. Our calculations assume an R-value of 10 for subgrade soils and a range of Traffic Indices from 4.0 to 7.0 depending on the expected traffic loads for a twenty-year design life. In general, areas expected to experience loading from heavy vehicles should be designed using the higher Traffic Index, while parking areas and other lightly-loaded areas can utilize a thinner pavement section based on the lower Traffic Index. The recommended pavement sections are presented in Tables 6 and 7.

Table 6 – Preliminary Asphalt-Concrete Pavement Sections

Traffic Index ¹	Asphalt Concrete (inches)	Aggregate Base (inches)
4.0	3.0	7.0
5.0	3.5	8.0
6.0	5.0	8.5
7.0	5.0	13.0

(1) Traffic Index for final pavement design to be determined by the project Civil Engineer.

In areas where concrete pavement is planned, the concrete pavement design should conform to recommendations for rigid pavements from the Portland Cement Association (PCA, 1984). Concrete reinforcement should consist of No. 4 rebar (Grade 40 or higher) spaced at a maximum of 18 inches on center in both directions. Recommended design criteria for rigid pavements are summarized in Table 7.

Table 7 – Design Criteria for Concrete Pavements

Parameter	Value
Minimum Concrete Thickness	5 inches
Minimum Aggregate Base Thickness	4 inches
Modulus of Rupture (ASTM C78)	600 psi
Maximum Water-Cement Ratio (by weight)	0.45
Modulus of Subgrade Reaction	100 pci
Joint Spacing	12 to 15 feet

In pavement areas, the upper 12 inches of subgrade should be compacted to at least 95 percent relative compaction. The aggregate base and asphalt-concrete should conform to the most recent version of Caltrans Standard Specifications and should be compacted to at least 95 percent relative compaction. Additionally, the subgrade and aggregate base should be firm and unyielding under heavy, rubber-tired construction equipment.

6.0 SUPPLEMENTAL GEOTECHNICAL SERVICES

This report provides preliminary geotechnical recommendations and design criteria based on the current development plan. As the development plan is refined and to further evaluate geologic conditions as discussed, we should perform supplemental exploration and laboratory testing as needed to update this report for the design level final report.

As project plans are nearing completion, we should review them to confirm that the intent of our geotechnical recommendations has been incorporated. We can also consult with project team to supplement or clarify geotechnical recommendations, if needed. If requested, we can perform analyses and prepare plans, details, technical specifications, and calculation package for soil nail or tied-back retaining structures.

During construction, we should be present intermittently to observe foundation excavations, fill placement, trench backfill, retaining wall drainage and backfill and other geotechnical-related work items. The purpose of our observation and testing is to confirm that site conditions are as anticipated, to adjust our recommendations and design criteria if needed, and to confirm that the Contractor’s work is performed in accordance with the project plans and specifications.

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 the project Owner 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.

8.0 LIST OF REFERENCES

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SITE COORDINATES

LAT. 37.9831°
LON. -122.5023°

SITE LOCATION

N.T.S.



REFERENCE: Google Earth, 2021



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SITE LOCATION MAP

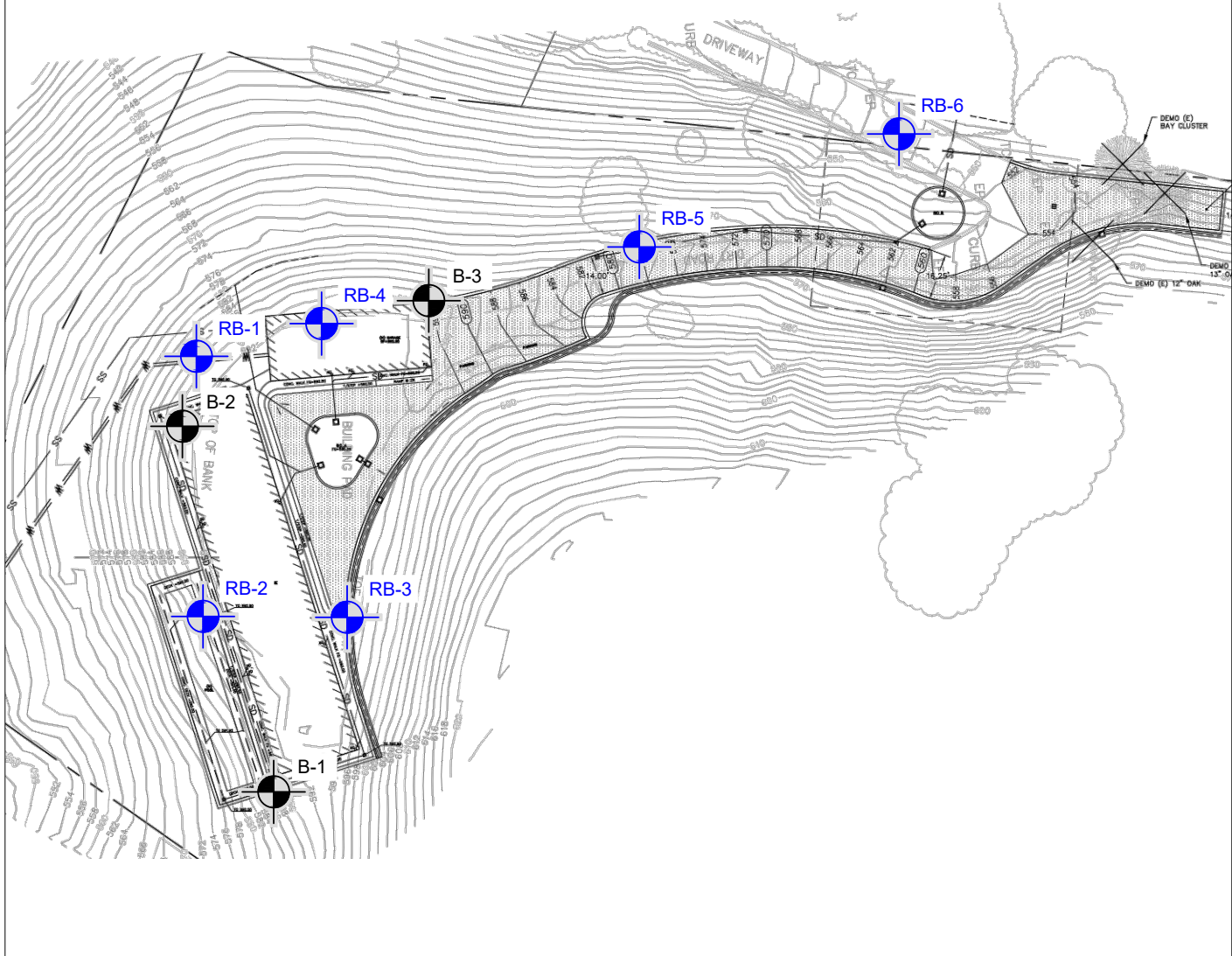
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380 Margarita Drive
San Rafael, California**

Project No. 3270.001

Date: 10/21/2021

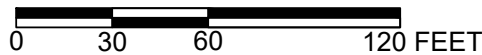
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1
FIGURE



SITE PLAN

SCALE



Approximate boring location completed by MPEG, 2021



Approximate boring location completed by John C. Hom, 2015

REFERENCE: Oberkamper, 2021



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SITE PLAN

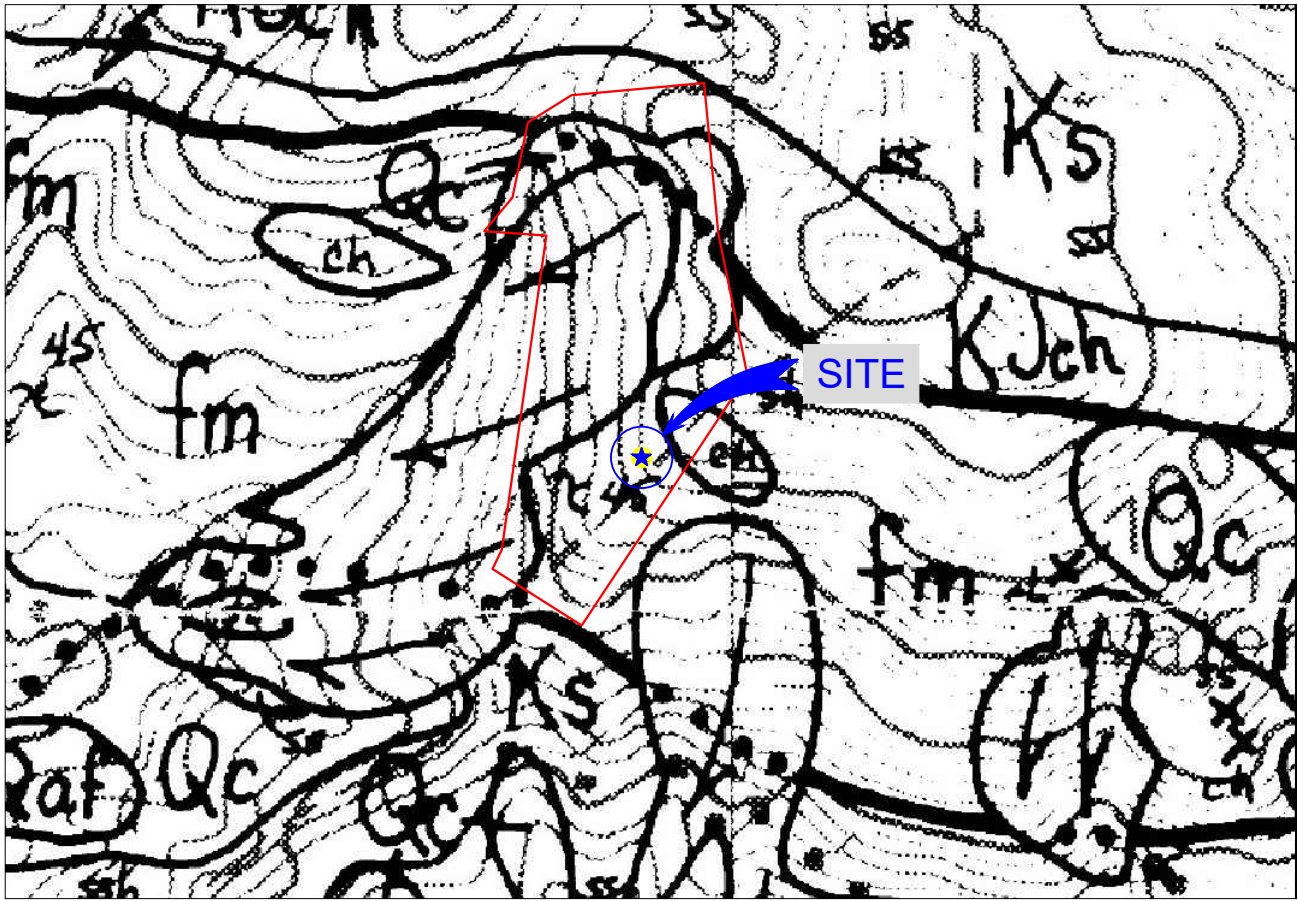
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2
 FIGURE



REGIONAL GEOLOGIC MAP

(NOT TO SCALE)



LEGEND



Debris Flow Landslides: Deposits of unconsolidated and unsorted soil and rock debris that have moved down slope en masse or in increments by flow or creep processes.

Qc Colluvium: Unconsolidated and unsorted soil material and weathered rock fragments accumulated at or on the bases of slopes by natural gravitational or slope wash processes. Derived by weathering and decomposition of bedrock materials underlying slopes.

Ks Sandstone and shale: mainly thick bedded to massive, medium to coarse grained, fairly well sorted, buff to tan sandstone (ss) and well bedded siltstone (sh), dark gray where fresh, light gray and stained by iron oxide where weathered along joints with minor amounts of conglomerate (cg).

fm Franciscan Melange: A tectonic mixture consisting of small to large masses of resistant rock types, principally sandstone (ss), greenstone (gs), chert (ch), and serpentine (sp), but including various exotic metamorphic rock types, embedded in a matrix of pervasively sheared or pulverized rock material.

— Property Line (approximate)

Reference: Rice, Salem J. 1976 "Geology of the Eastern Part of the San Rafael Area, Marin County, California." by The California Division of Mines and Geology in *Geology for Planning in Central and Southeastern Marin County, California*. OFR 76-2. Plate 1C. East San Rafael, Geology.



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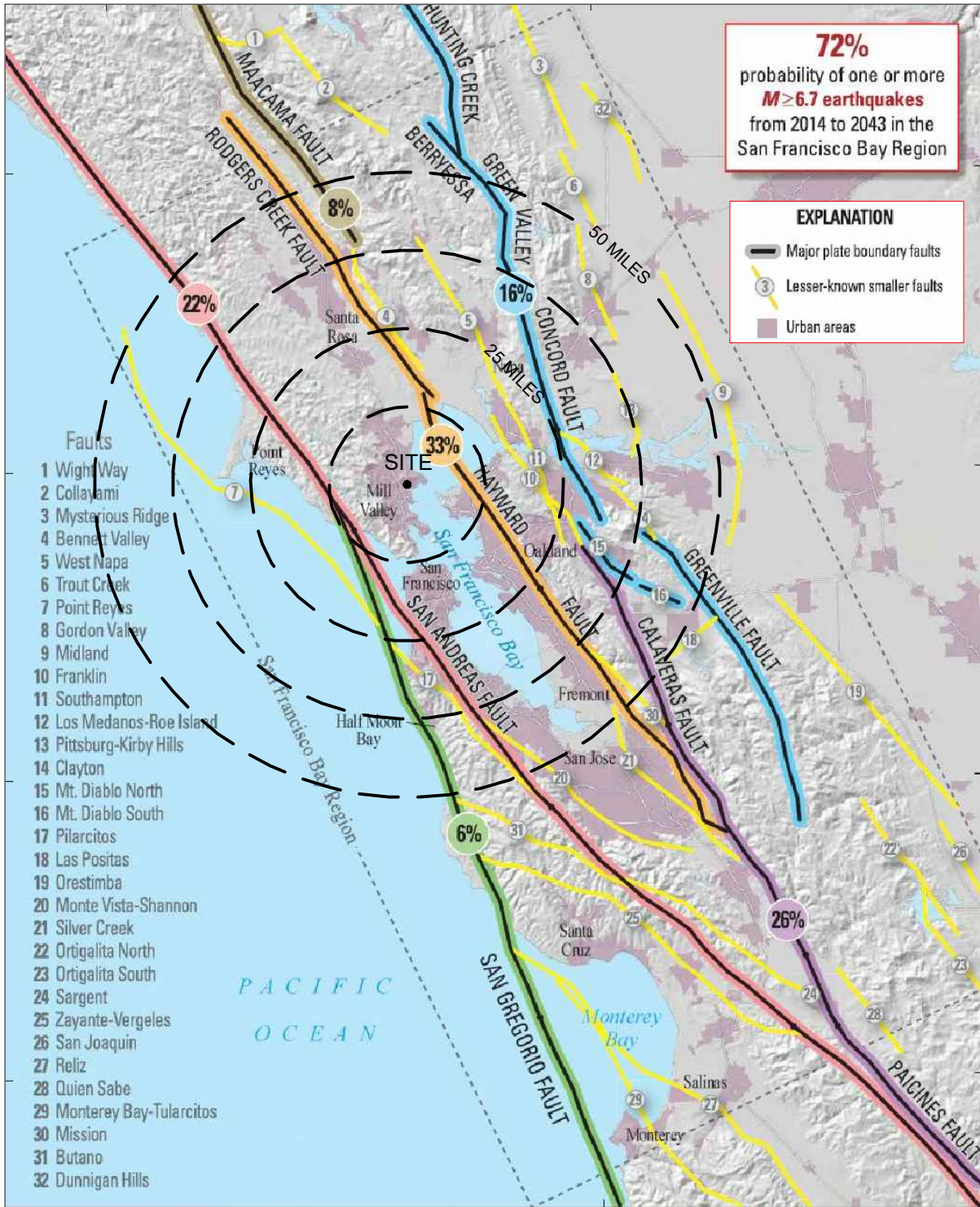
REGIONAL GEOLOGIC MAP

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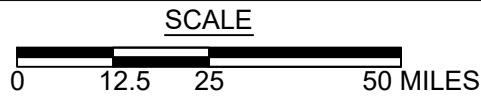
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Checked ZMS

3

FIGURE



SITE COORDINATES
LAT. 37.9831°
LON. -122.5023°



DATA SOURCE:

1) U.S. Geological Survey, U.S. Department of the Interior, "Earthquake Outlook for the San Francisco Bay Region 2014-2043", Map of Known Active Faults in the San Francisco Bay Region, Fact Sheet 2016-3020, Revised August 2016 (ver. 1.1).



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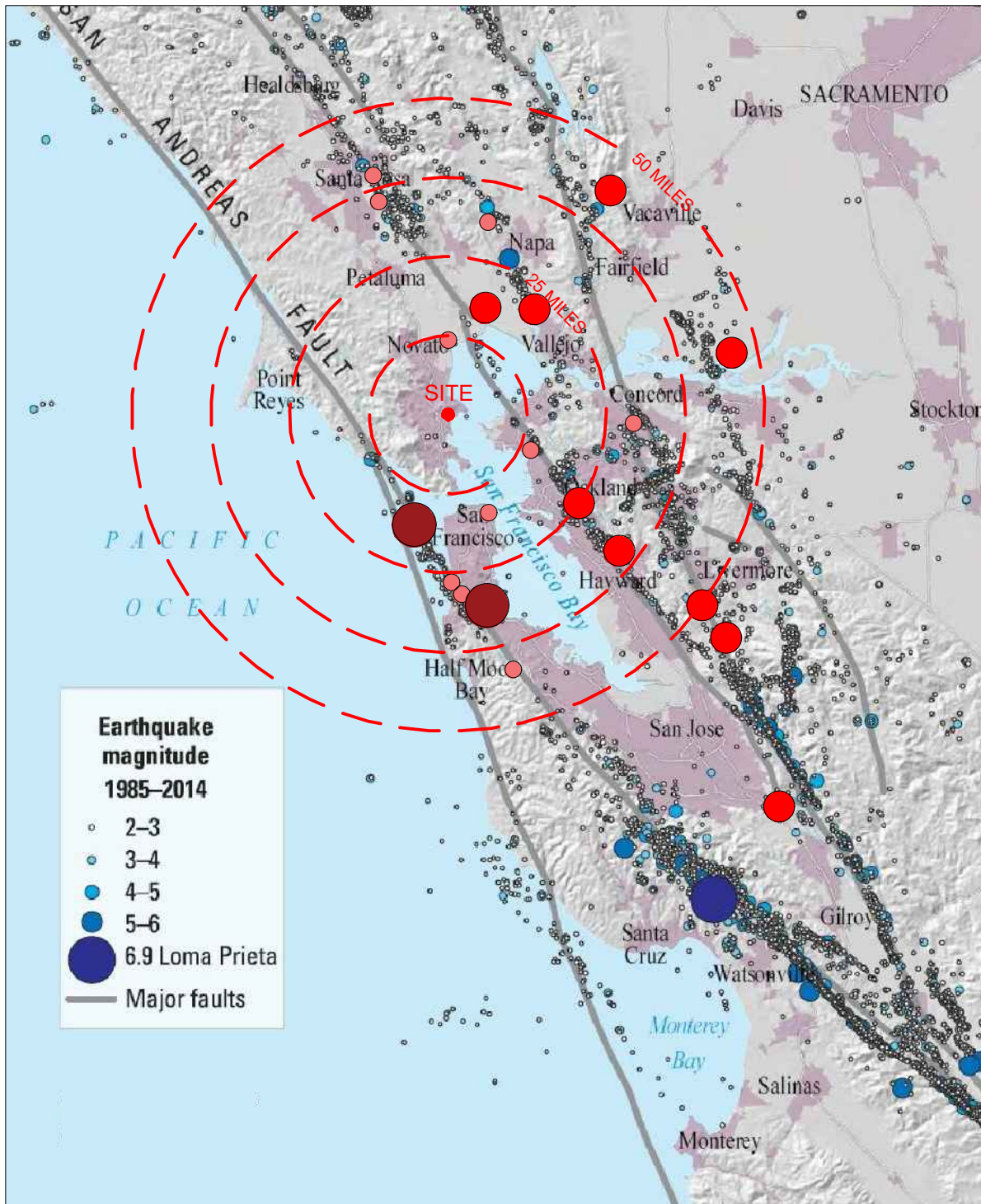
ACTIVE FAULT MAP

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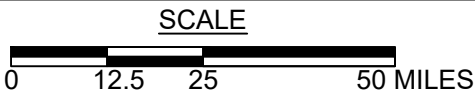
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4

FIGURE



SITE COORDINATES
 LAT. 37.9831°
 LON. -122.5023°



LEGEND & DATA SOURCE:

- ● ● See legend above. U.S. Geological Survey, U.S. Department of the Interior, "Earthquake Outlook for the San Francisco Bay Region 2014-2043", Map of Known Active Faults in the San Francisco Bay Region, Fact Sheet 2016-3020, Revised August 2016 (ver. 1.1).
- ● ● Large circles indicate earthquakes $M > 7.0$, medium circles indicate $6.0 < M < 7.0$ and small circles indicate $5.0 < M < 6.0$. U.S. Geological Survey, Earthquake Catalog Search, <https://earthquake.usgs.gov/earthquakes/search/>. Earthquakes between 1830 and 2021.



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HISTORIC EARTHQUAKE MAP

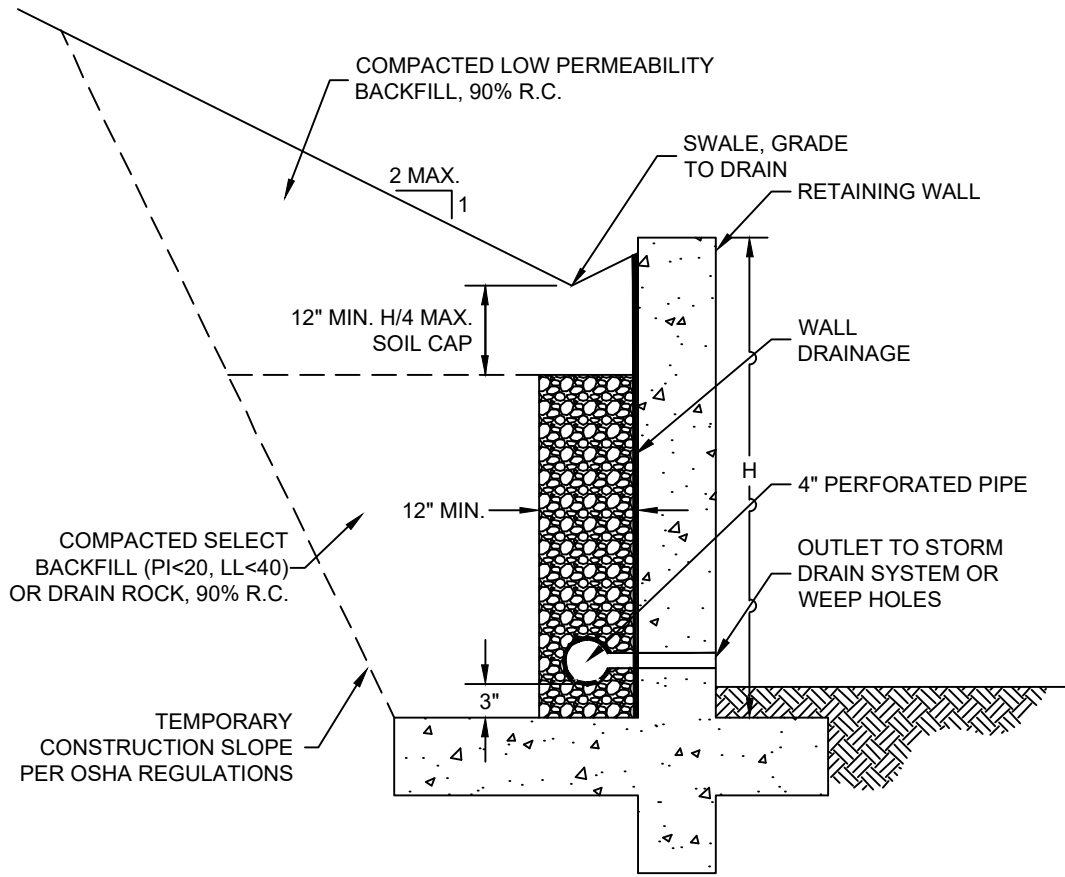
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Date: 10/21/2021

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 Checked _____

5
 FIGURE



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.

 MILLER PACIFIC ENGINEERING GROUP	504 Redwood Blvd. Suite 220 Novato, CA 94947 T 415 / 382-3444 F 415 / 382-3450 www.millerpac.com	TYPICAL RETAINING WALL BACKDRAIN		Drawn _____ ZMS Checked _____	<div style="font-size: 48px; font-weight: bold; margin: 0;">6</div> <div style="font-weight: bold; margin: 0;">FIGURE</div>
	Kelty Residence 380 Margarita Drive San Rafael, California		Project No. 3270.001	Date: 10/21/2021	

APPENDIX A – LABORATORY TESTING

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, gravelly 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

STRENGTH TESTS

UC	LABORATORY UNCONFINED COMPRESSION
TXCU	CONSOLIDATED UNDRAINED TRIAXIAL
TXUU	UNCONSOLIDATED UNDRAINED TRIAXIAL
	UC, CU, UU = 1/2 Deviator Stress
DS (2.0)	DRAINED DIRECT SHEAR (NORMAL PRESSURE, ksf)

SAMPLER TYPE

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

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

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.



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SOIL CLASSIFICATION CHART

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Checked:

A-1
FIGURE

Project No. 3270.001

Date: 10/21/2021

FRACTURING AND BEDDING

Fracture Classification

Crushed
Intensely fractured
Closely fractured
Moderately fractured
Widely fractured
Very widely fractured

Spacing

less than 3/4 inch
3/4 to 2-1/2 inches
2-1/2 to 8 inches
8 to 24 inches
2 to 6 feet
greater than 6 feet

Bedding Classification

Laminated
Very thinly bedded
Thinly bedded
Medium bedded
Thickly bedded
Very thickly bedded

HARDNESS

Low
Moderate
Hard
Very hard

Carved or gouged with a knife
Easily scratched with a knife, friable
Difficult to scratch, knife scratch leaves dust trace
Rock scratches metal

STRENGTH

Friable
Weak
Moderate
Strong
Very strong

Crumbles by rubbing with fingers
Crumbles under light hammer blows
Indentations <1/8 inch with moderate blow with pick end of rock hammer
Withstands few heavy hammer blows, yields large fragments
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

Kelty Residence
San Rafael, California

Drawn ZMS
Checked _____

A-2
FIGURE

DEPTH		BORING 1				BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
meters	feet	SAMPLE	SYMBOL (4)	EQUIPMENT: Track Mounted Hydraulic Drill Rig with 4.0-inch Solid Flight Auger	DATE: 8/22/21						
				ELEVATION: 191 - feet*	*REFERENCE: Google Earth, 2021						
0	0			Clayey SAND with Gravel (SC) Medium brown, dry to slightly moist, fine to coarse grained sand, 42% fines, gravel up to 3-inches in diameter [Fill]		26	103	7.4			
1											
	5					25	94	9.0		42.4% P200	
2											
	10			SANDSTONE and SHALE (fm) Color varies from red brown to gray brown, highly to completely weathered, moderately hard, moderately strong [Bedrock]		63		8.1			
3											
4											
	15			End of boring at 15.0 feet No groundwater encountered		60		8.1			
5											
6	20										

- ▽ Water level encountered during drilling
- ▼ Water level measured after drilling

NOTES: (1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT $\text{kN/m}^3 = 0.1571 \times \text{DRY UNIT WEIGHT (pcf)}$
(3) METRIC EQUIVALENT STRENGTH $(\text{kPa}) = 0.0479 \times \text{STRENGTH (psf)}$
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY



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BORING LOG

Kelty Residence
San Rafael, California

Drawn _____
ZMS
Checked _____

A-3
FIGURE

Project No. 3270.001

Date: 10/21/2021

DEPTH		BORING 2				BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
meters	feet	SAMPLE	SYMBOL (4)	EQUIPMENT: Track Mounted Hydraulic Drill Rig with 4.0-inch Solid Flight Auger	DATE: 8/22/21						
				ELEVATION: 191 - feet*	*REFERENCE: Google Earth, 2021						
0	0			Clayey SAND with Gravel (SC) Medium brown, dry to slightly moist, fine to coarse grained sand, 18% fines, gravel up to 3-inches in diameter [Fill]		26	109	7.9	18.0% P200		
1				SANDSTONE and SHALE (fm) Color varies from red brown to gray brown, highly to completely weathered, moderately hard, moderately strong [Bedrock]		64		5.1			
2											
3	10					77		5.4			
4						50 3/4"		9.5			
5	15			End of boring at 15.5 feet No groundwater encountered							
6	20										

- ▽ Water level encountered during drilling
- ▽ Water level measured after drilling

NOTES: (1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT $\text{kN/m}^3 = 0.1571 \times \text{DRY UNIT WEIGHT (pcf)}$
(3) METRIC EQUIVALENT STRENGTH (kPa) = $0.0479 \times \text{STRENGTH (psf)}$
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY



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BORING LOG

Kelty Residence
San Rafael, California

Drawn: _____
ZMS
Checked: _____

A-4
FIGURE

Project No. 3270.001

Date: 10/21/2021

DEPTH		BORING 3				BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
meters	feet	SAMPLE	SYMBOL (4)	EQUIPMENT: Track Mounted Hydraulic Drill Rig with 4.0-inch Solid Flight Auger	DATE: 8/22/21						
				ELEVATION: 191 - feet*	*REFERENCE: Google Earth, 2021						
0	0			Clayey SAND with Gravel (SC) Medium brown, dry to slightly moist, fine to coarse grained sand, trace fines, gravel up to 3-inches in diameter [Fill]		16	102	4.8			
1						20		4.8			
5				SANDSTONE and SHALE (fm) Color varies from red brown to gray brown, highly to completely weathered, moderately hard, moderately strong [Bedrock]							
2				Grades predominantly shale		45		15.5			
3	10										
4						75		10.2			
15				End of boring at 13.5 feet No groundwater encountered							
5											
6	20										

- ▽ Water level encountered during drilling
- ▼ Water level measured after drilling

NOTES: (1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT $\text{kN/m}^3 = 0.1571 \times \text{DRY UNIT WEIGHT (pcf)}$
(3) METRIC EQUIVALENT STRENGTH (kPa) = $0.0479 \times \text{STRENGTH (psf)}$
(4) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY



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BORING LOG

Kelty Residence
San Rafael, California

Drawn _____
ZMS
Checked _____

A-5
FIGURE

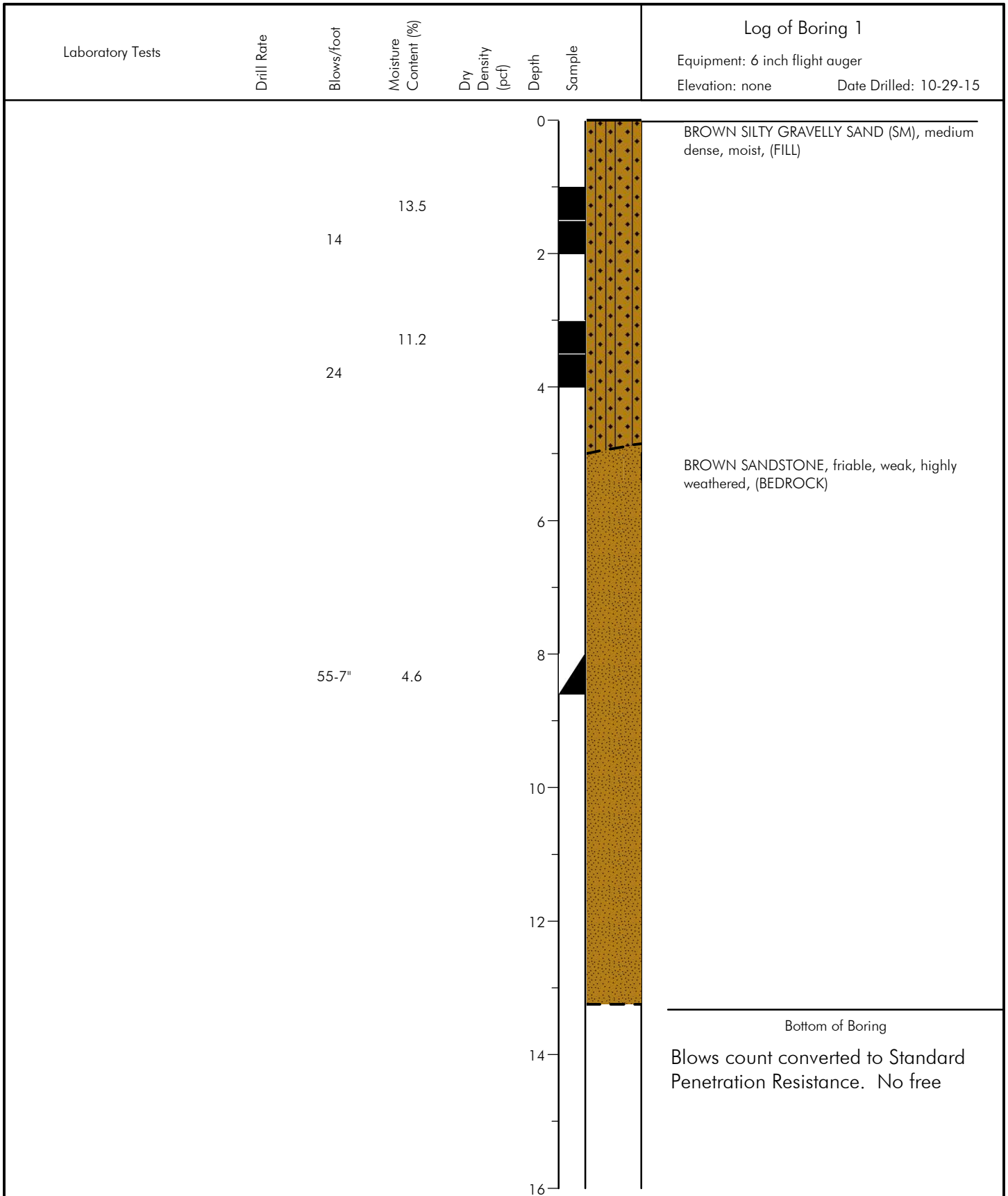
Project No. 3270.001

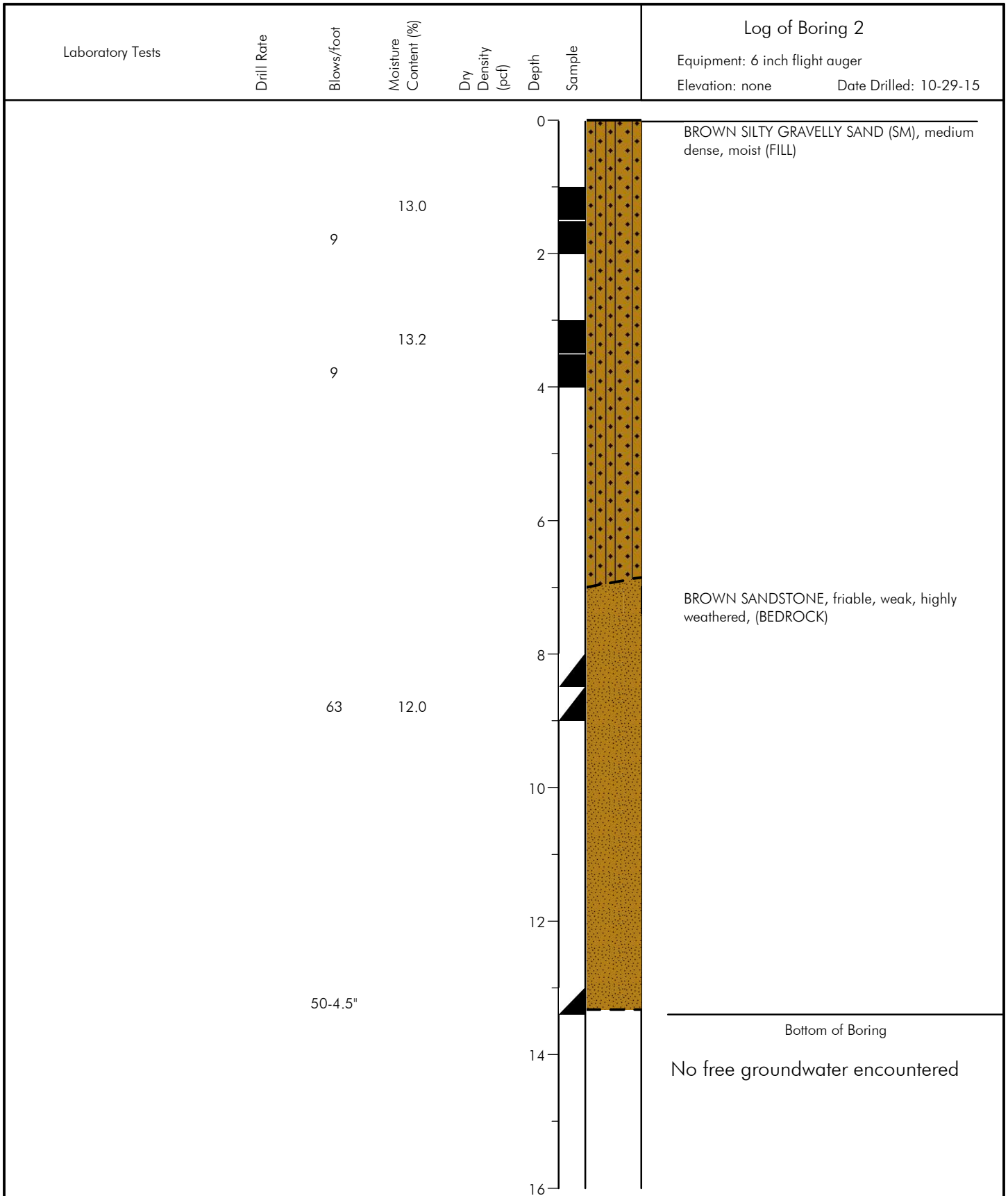
Date: 10/21/2021

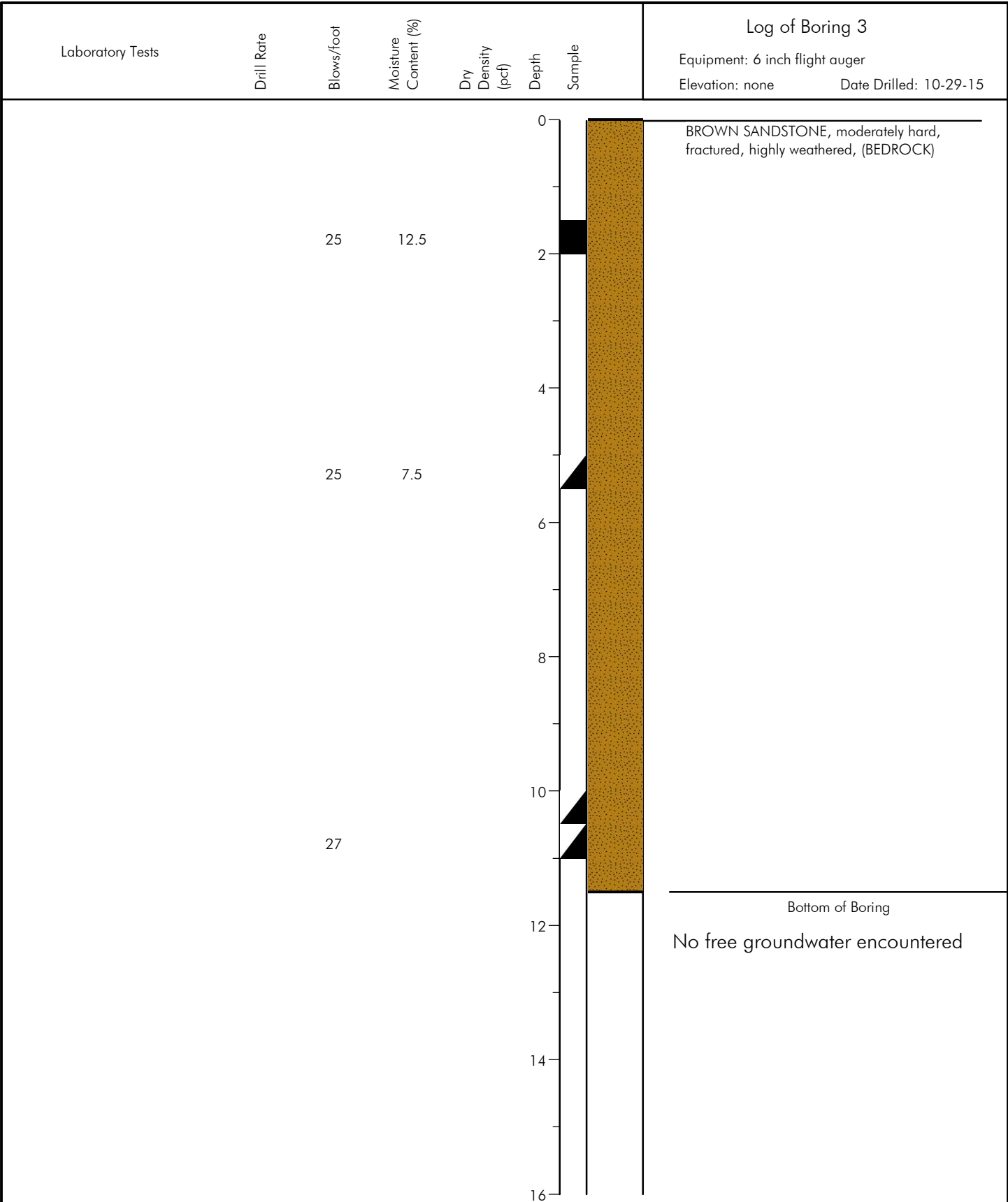
APPENDIX B – REFERENCE DATA

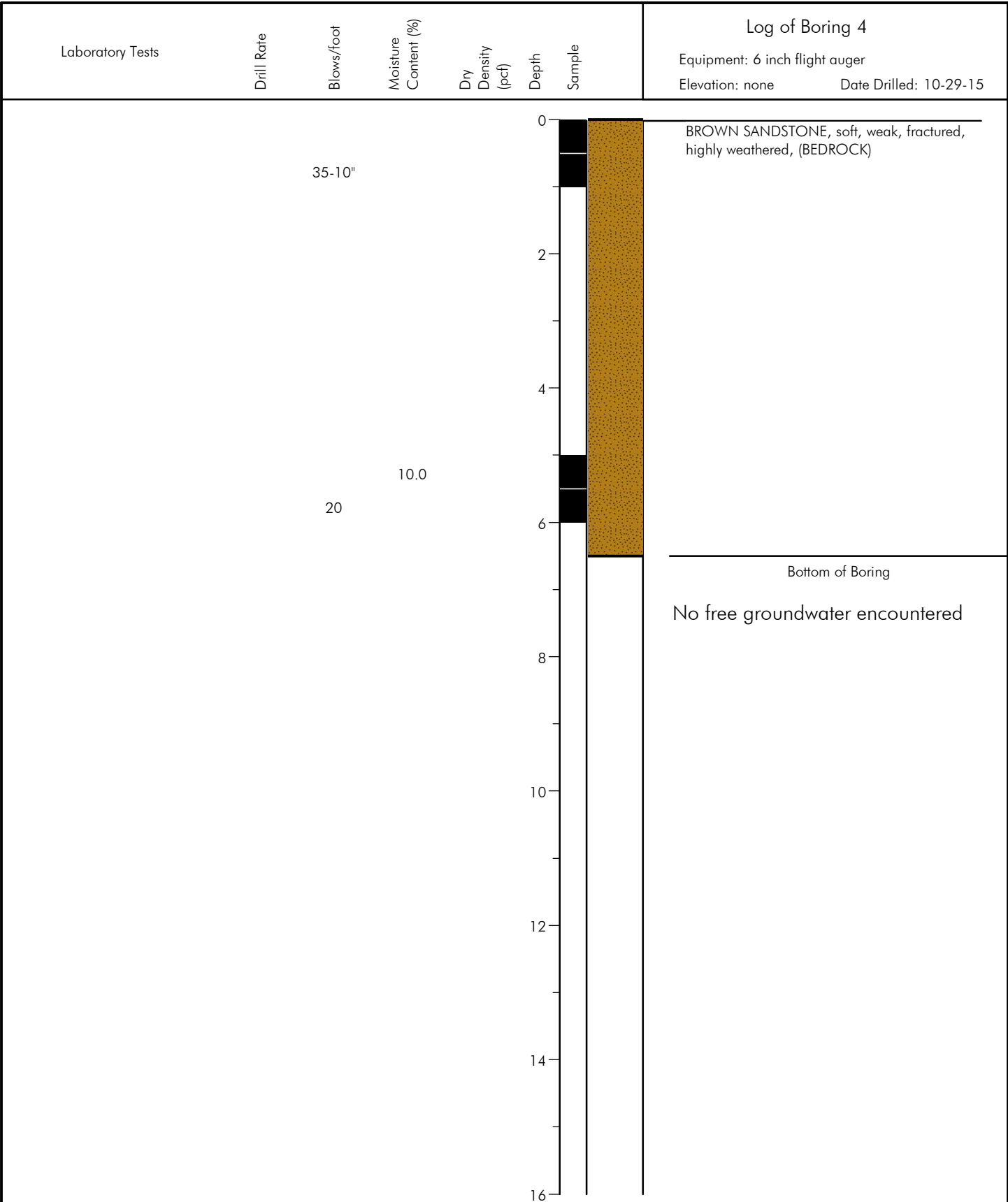



⊕ Test Boring Location









Laboratory Tests	Drill Rate	Blows/foot	Moisture Content (%)	Dry Density (pcf)	Depth	Sample	Log of Boring 5 Equipment: 4 inch auger Elevation: none Date Drilled: 11-8-15	
					0 1 2 3 4 5 6 7 8		<p>BROWN SILTY GRAVELLY SAND (SM) medium dense, moist, (FILL)</p>	<p>BROWN SANDSTONE, friable, weak, highly weathered, (BEDROCK)</p> <hr/> <p>Bottom of Boring</p> <p>No free groundwater encountered</p>

Laboratory Tests	Drill Rate	Blows/foot	Moisture Content (%)	Dry Density (pcf)	Depth	Sample	<p style="text-align: center;">Log of Boring 6</p> <p>Equipment: 4 inch flight auger Elevation: none Date Drilled: 11-10-15</p>
					0	[Hatched pattern]	BROWN SANDY CLAY (CL), medium stiff, moist, (SLIDE DEBRIS)
					1	[Hatched pattern]	
					2	[Dotted pattern]	BROWN SANDSTONE, friable, weak, highly weathered, (BEDROCK)
					3	[Empty]	Bottom of Boring No free groundwater encountered
					4	[Empty]	
					5	[Empty]	
					6	[Empty]	
					7	[Empty]	
					8	[Empty]	

MAJOR DIVISIONS					TYPICAL NAMES	
COARSE GRAINED SOILS MORE THAN HALF IS LARGER THAN #200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW		WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES	
			GP		POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES	
		GRAVELS WITH OVER 12% FINES	GM		SILTY GRAVELS, POORLY GRADED GRAVEL-SAND-SILT MIXTURES	
			GC		CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES	
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LITTLE OR NO FINES	SW		WELL GRADED SANDS, GRAVELLY SANDS	
			SP		POORLY GRADED SANDS, GRAVELLY SANDS	
		SANDS WITH OVER 12% FINES	SM		SILTY SANDS, POORLY GRADED SAND-SILT MIXTURES	
			SC		CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES	
			SILTS AND CLAYS LIQUID LIMIT LESS THAN 50	ML		INORGANIC SILTS AND VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL		INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
OL		ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY				
SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50	MH		INORGANIC SILTS, FINE SANDY OR SILTY SOILS, PLASTIC SILTS			
	CH		INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS			
	OH		ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS			
HIGHLY ORGANIC SOILS			Pt		PEAT AND OTHER HIGHLY ORGANIC SOILS	

UNIFIED SOIL CLASSIFICATION SYSTEM

Consol	Consolidation	Tx	20 (1600)	Unconsolidated Undrained Triaxial
LL	Liquid Limit (in %)	TxCU	320 (2600)	Consolidated Undrained Triaxial
PL	Plastic Limit (in %)	DS	2750 (2000)	Consolidated Drained Direct Shear
PI	Plastic Index (in %)	FVS	470	Field Vane Shear
Gs	Specific Gravity	UC	2000	Unconfined Compression
SA	Sieve Analysis	LVS	700	Laboratory Vane Shear
	Undisturbed Sample	SS	Shrink Swell	
	Auger Sample	EXP	Expansion	
	Standard Penetration Sample	P	Permeability	
	Excavation Sample			
	Sample Attempt No Recovery			

Note: All strength tests on 2.8" or 2.4" diameter samples unless otherwise indicated.

KEY TO TEST DATA

JCH JOHN C. HOM & ASSOCIATES, INC. <i>Geotechnical Consultants</i>	Job No: 1863.1	SOIL CLASSIFICATION CHART AND KEY TO TEST DATA 380Margarita Drive San Rafael, California	PLATE
	Appr: JCH		8
	Date: 11/15		