PRELIMINARY GEOTECHNICAL INVESTIGATION
SPIRIT GROUP SENIOR HOUSING
70 NORTH KNOLL ROAD
MILL VALLEY, CALIFORNIA

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Job No. 3011.002

Prepared for:
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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 residential care facility for the elderly in Mill Valley, California. As shown on Figure 1, the project site is located on the hillside to the east of North Knoll Road, just northeast of the Highway 101 overpass at Tiburon Boulevard in Mill Valley.

Our work was performed in accordance with our Agreement for Professional Services authorized October 6, 2021. As the new geotechnical engineer of record, we have reviewed a previous geotechnical report for the project site by Herzog Geotechnical Consulting Services, dated September 10, 2019. Most of the recommendations and design criteria in that report are appropriate for the project. We have updated and provided supplemental recommendations and criteria as needed in this report. If there are any conflicts between the previous report and this report, this report will supersede.

This report completes our Phase 1 services for the project. Subsequent phases of work should include additional exploration, design-level geotechnical report, 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 design team, we understand the project is expected to include developing the site with a residential care facility for the elderly with 72 total units, including 71 dwelling units and one group living facility with 49 memory care bedrooms. The project includes a level of parking and one accessory structure for a horse trellis. The building will be up to five stories in height above basement level parking, with an elevated walkway from the fifth story to walking trails and a horse stable on the hillside. Ancillary improvements will include a main access driveway, a drop off driveway, parking area, underground utilities, site drainage, walking trails, and a trolley line. The approximate building locations are shown on the Site Plan, Figure 2.

Moderate site grading is planned and includes excavation into the hillside to embed the structures. Retaining walls on the order of 30 feet tall will retain cuts and limit the extent of grading. Fill placement supported by retaining wall is planned for roadways. Moderate to high foundation loads are expected for the structure.

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 building is situated on a sloping hillside on the west facing side of a north-trending ridgeline. Regional geologic mapping (Rice, 1976) indicates the site is underlain by colluvial deposits of Quaternary age, mélange bedrock of the Franciscan Complex, sandstone and shale, and greenstone bedrock. Much of the site is mapped as exhibiting continuous or intermittent downslope soil creep. 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.

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 12.2 kilometers and 14.1 kilometers to the southwest[1], respectively. The Hayward Fault is located roughly 16.5 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 indicate that at least 20 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 16.5 kilometers northeast of the site and is assigned a probability of 33 percent. The San Andreas Fault, located approximately 12.2 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 United States Geologic Survey (USGS) Quaternary Fault and Fold Database, accessed 2024.

3.3 Surface Conditions

The project site encompasses an irregularly-shaped, approximately 6.6-acre parcel (APN 034-012-26) located to the northeast of the intersection of North Knoll Road and Thomas Drive. The site is bordered by a residential development to the west, open space to the north, and residences to the east and south. The average slope of the parcel is about 3:1 (horizontal: vertical) but varies locally. Portions of the hillside display evidence of previous slumping and earthflow landslide movement.

The ground surface within the proposed building area varies from an elevation of approximately 120 feet where the project site connects to North Knoll Road to 245 feet at the upslope limit of the site[2]. The property is currently unimproved and is vegetated with native grasses, shrubbery, and a few trees.

3.4 Reference Subsurface Exploration and Laboratory Testing

Previous geotechnical investigations were completed by Herzog Geotechnical Consulting Engineers in 2019 for the originally planned development. These investigations included excavating thirteen exploratory borings near the planned improvements. The report, including subsurface exploration and laboratory testing data, is presented in Appendix B. The exploration locations are also shown on Figure 2.

3.5 Subsurface Conditions and Groundwater

Based on the review of reference data, the project site is generally underlain by between 0.5 to 8 feet clayey and sandy colluvial soils over mélange and variably weathered bedrock of the Franciscan Complex. The clayey soils are generally soft to medium stiff, and the sandy soils are loose to medium dense, and both are likely derived from the underlying mélange. Four exploratory borings encountered from 4 to 5.5 feet of slide debris on top of bedrock.

The bedrock encountered in the borings predominantly consists of sandstone and pervasively sheared shale, as well as some serpentinite, of the Franciscan Melange unit, which is generally highly to completely weathered and exhibit firm to moderate hardness and friable to moderate strength.

Groundwater was not encountered in the subsurface exploration. However, the borings may not have been left open for an extended period of time, so 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, potential debris flow impact and slope instability. Other geologic hazards are judged less than significant regarding

[2] Surface elevations are based on those shown on the Entitlement Plans by Adobe Associates, Inc, Dated July 22, 2024, Sheet C 2.0.

the proposed project. Geologic hazards, potential impacts and mitigation measures are discussed in further detail in the following sections.

4.1 Fault Surface Rupture

Under the Alquist-Priolo Earthquake Fault Zoning Act, the California Division of Mines and Geology (now known as the California Geological Survey) produced 1:24,000 scale maps showing known active and potentially active faults and defining zones within which special fault studies are required. The nearest known active fault to the site is the San Andreas Fault located approximately 12.2 kilometers to the southwest. The site is not located within the Alquist-Priolo Special Studies Zone. We therefore judge the potential for fault surface rupture in the development area to be low.

Evaluation: Less than significant. No mitigation measures are required.

4.2 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	12.2	0.34	0.19
Hayward/Rodgers Creek	7.6	16.5	0.25	0.45
San Gregorio	7.4	14.1	0.26	0.47
West Napa	6.9	38.0	0.09	0.16

Reference: Abrahamson & Silva, Boore & Atkinson, Campbell & Bozorgnia, and Chiou & Youngs (2008) 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.

We calculated the peak ground acceleration for two separate probabilistic conditions; the two percent chance of exceedance in 50 years (2,475-year statistical return period) and the ten percent chance of exceedance in 50 years (475-year statistical return period). The peak ground acceleration values were calculated utilizing the USGS Unified Hazard Tool. The results of the probabilistic analyses are presented below in Table 2.

Table 2 – Probabilistic Peak Ground Accelerations for Active Faults

Probability of Exceedance	Statistical Return Period	Magnitude	Peak Ground Acceleration (g)
2% in 50 years	2,475 years	7.6	0.88
10% in 50 years	475 years	7.5	0.48

Reference: USGS Unified Hazard Tool (Dynamic: Conterminous U.S. 2014 (unknown)) accessed September 18, 2024.

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 (2022 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: *Less than significant with mitigation.*
Minimum recommendations include design of new structures in accordance with the provisions of the 2022 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.

4.3 Liquefaction and Related Effects

Liquefaction refers to the sudden, temporary loss of soil strength during strong ground shaking. The strength loss occurs as a result of the build-up of excess pore water pressures and subsequent reduction of effective stress. While liquefaction most commonly occurs in saturated, loose, granular deposits, recent studies indicate that it can also occur in materials with relatively high fines content provided the fines exhibit lower plasticity. The effects of liquefaction can vary from cyclic softening resulting in limited strain potential to flow failure which cause large settlements and lateral ground movements.

Based on subsurface exploration, the project site is underlain by a relatively thin layer of clayey soils over shallow Franciscan bedrock which are not susceptible to liquefaction. Therefore, we judge the likelihood of damage to the proposed improvements due to liquefaction is low.

Evaluation: Less than significant. No mitigation measures are required.

4.4 Settlement

Significant settlement can occur when new loads are placed over soft, compressible clays (e.g. Bay Mud) or loose soils. The soft to medium stiff clayey soils and loose to medium dense sandy soils and shallow Franciscan bedrock encountered are not highly compressible and new foundations are expected to bear on bedrock, as discussed in Section 5.3. Therefore, we judge the risk of damage due to settlement induced by new structural loads is low.

Evaluation: Less than significant. Foundation should be embedded into bedrock to provide uniform support to the structure.

4.5 Seismic Densification

Seismic ground shaking can induce settlement in unsaturated, loose, 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. The localized, loose, granular soils encountered in Herzog's borings were relatively shallow and will be removed so that foundations bear on bedrock, therefore the risk of seismic densification impacting the new structures is generally low.

Evaluation: Less than significant. No mitigation measures are required.

4.6 Expansive Soils

Soil expansion occurs when clay particles interact with water causing seasonal volume changes in the soil matrix. The clay soil swells when saturated and then contracts when dried. This phenomenon generally decreases in magnitude with increasing confinement pressures at increasing depths. These volume changes may damage lightly loaded foundations, concrete slabs, pavements, retaining walls and other improvements. Expansive soils also cause soil creep on sloping ground. Laboratory testing on the near-surface soils indicate relatively low expansion potential. Plasticity Index (PI) test on a select sample from Boring 1 was 25 (medium plasticity). Thus, there is a low to medium potential for damage due to expansive soils.

Evaluation: Less than significant with mitigation.
Soils with low expansion potential should be used within the upper three feet of new fills. Soils subgrades and fills should be moisture conditioned above the optimum moisture content during site grading and maintained at this moisture content until imported aggregate base and/or surface flatwork is completed. Retaining structures should be designed with a soil creep load where walls retain sloping ground. Concrete flatwork should be designed to account for some minor expansive soil movement.

4.7 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 much of the site will be covered with new buildings, pavements, or concrete flatwork. Significant erosion is generally not anticipated within these areas. Drainage channels within the relatively steeply-sloping terrain show some active erosion, including gullies, localized small sloughs and raveling along the channel banks. Therefore, we judge the risk of erosion impacting the project to be moderate.

Evaluation: *Less than significant with mitigation.*
Planned improvements or structures on shallow foundations should be setback from unimproved drainage channel. The recommended setback distance is a 3:1 inclination from the channel bed or 10 feet from top of bank, whichever is greater. 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. 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.

4.8 Slope Instability

The development will be located on a hillside which is locally inclined as steeply as about 2:1 but has an average slope of 3:1. Based on our review of Herzog's aerial photo, reconnaissance, and exploration, there are areas of probable previous instability at the location shown on Figure 2. The depth of this probable instability is likely less than 10 feet. In other areas of the site, the surface soils are mapped by Rice (1976) as "creeping" and are prone to soil creep, occasional shallow sloughing, and debris flows in drainage channels which could result in debris impact to the rear of the structures. Deep excavations into the hillside can induce slope instability. 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 and upslope of the project site.

Evaluation: *Less than significant with mitigation.*
Geologic site inspection/mapping and possible supplemental exploration with exploratory trenches and further upslope should be performed as part of the design-level report to better evaluate the potential for instability. Most of the suspected areas of instability within the site will be removed as part of the planned excavation and building construction. Undeveloped areas of instability within the project site that could impact the structure should be over-excavated, subsurface drainage installed, and backfilled with engineered fill. Alternatively, debris

catchment structure or deflection wall/berm could be used upslope of the planned buildings if debris flow paths cross planned structures, as discussed in Section 5.4. Retaining walls should be utilized to support hillside excavations to create the building pad for the structure.

4.9 Flooding

Flood Insurance Rate Maps prepared by the Federal Emergency Management Agency (FEMA, 2016) indicate the site is not mapped within a flood area. Based on the FEMA mapping, the risk of damage to future improvements due to flooding is considered low. 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. No mitigation measures are required. The project Civil Engineer is responsible for site drainage.

4.10 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. These conditions do not exist at the site, therefore the risk of lurching and ground cracking at the project site is low.

Evaluation: No significant impact. No mitigation measures are required

4.11 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 increased elevation and not located near a large body of water. Therefore, seiche and tsunami events are not considered significant geologic hazards at the site.

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

4.12 Soil Corrosion

Corrosive soil can damage buried metallic structures, cause concrete spalling, and deteriorate rebar reinforcement. Laboratory testing was performed on representative samples of the near-surface site soils to evaluate pH, electrical resistivity, chloride and sulfate contents. Based on the site location and geology, corrosive soil is not expected to be a significant geologic hazard at the project site.

Evaluation: Not expected to be significant impact. Corrosion testing should be performed as part of the design level investigation.

4.13 Radon-222 Gas

Radon-222 is a product of the radioactive decay of uranium-238 and raduim-226, which occur naturally in a variety of rock types, mainly phosphatic shales, but also in other igneous, metamorphic, and sedimentary rocks. While low levels of radon gas are common, very high levels, which are typically caused by a combination of poor ventilation and high concentrations of uranium and radium in the underlying geologic materials can be hazardous to human health.

The project site is located in Marin County, California, which is mapped in radon gas Zone 3 by the United States Environmental Protection Agency (USEPA, 2018). Zone 3 is classified by the EPA as exhibiting a “low” potential for Radon-222 gas with average predicted indoor screening levels less than 2 pCi/L. Therefore, the potential for hazardous levels of radon at the project site is low.

Evaluation: No significant impact.

4.14 Volcanic Eruption

Several active volcanoes with the potential for future eruptions exist within northern California, including Mount Shasta, Lassen Peak, and Medicine Lake in extreme northern California, the Mono Lake-Long Valley Caldera complex in east-central California, and the Clear Lake Volcanic Field, located in Lake County approximately 60-miles northeast of the project site. The most recent volcanic eruption in northern California was at Lassen Peak in 1917, while the most recent eruption at the nearest volcanic center to the project site, the Clear Lake Volcanic Field, was about 10,000 years ago. All of northern California’s volcanic centers are currently listed under “normal” volcanic alert levels by the USGS California Volcano Observatory (USGS, 2018). While the aforementioned volcanic centers are considered “active” by the USGS, the likelihood of damage to the proposed improvements due to volcanic eruption is generally low.

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

4.15 Naturally Occurring Asbestos (NOA)

Naturally occurring asbestos is commonly found in association with serpentinite and associated ultramafic rock types. These rocks are a major constituent of the Franciscan Complex, which underlies vast portions of the greater San Francisco Bay Area. The site is underlain predominantly by sandstone and shale, though some serpentinite was found during exploration. Therefore, the likelihood that naturally-occurring asbestos will be encountered at the site is low.

Evaluation: No significant impact expected. Asbestos testing should be performed during construction if significant outcrops of serpentinite is encountered during excavation. Mitigation is needed and should follow Bay Area Air Quality Management District procedures .

4.16 Hazardous Materials

Hazardous materials were not observed during subsurface exploration. While environmental testing for hazardous materials was beyond the scope of our services, the site is currently undeveloped. Therefore, we judge the potential for hazardous materials being present on the project site, currently or in the future, is very low.

Evaluation: Not expected to be significant impact.

5.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our preliminary 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, support of planned excavations to maintain stability, and mitigation or protection from potential instability of the upslope areas above the proposed development. 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 (2022) 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 – 2022 California Building Code Seismic Design Criteria

Parameter	Design Value
Site Class	C
Site Latitude	37.9060°N
Site Longitude	-122.5112°W
Spectral Response (short), S_S	1.5 g
Spectral Response (1-sec), S_1	0.6 g
Site Coefficient, F_a	1.2
Site Coefficient, F_v	1.4
Spectral Response (Short), S_{MS}	1.8 g
Spectral Response (1 sec), S_{M1}	0.84 g
Design Spectral Response (short), S_{DS}	1.2 g
Design Spectral Response (1 sec), S_{D1}	0.56 g
MCE_G PGA Adjusted, PGA_M	0.686 g

Reference: OSHPD Seismic Design Maps, accessed on September 4, 2024.

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 0.5 to 8 feet of soft to medium stiff clayey soils or loose to medium dense sandy soils over Franciscan bedrock of variable weathering, strength and hardness. The bedrock encountered in reference 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.

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.

Addition exploration should be performed to the depth of the planned excavation during the design level investigation to better evaluate soil and bedrock conditions.

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. Most onsite soils are expected to be suitable for use as fill. On-site materials and 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. The design level investigation should include laboratory testing that includes compaction curves 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.

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 1 to 8 feet of clayey and sandy soils overlying Franciscan Melange. Shallow foundations can be utilized provided they maintain uniform support on competent bedrock. Since the planned grading involves cutting into the hillside for building pads, the downslope sides of the building pads may expose soil, therefore, footings 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. Drilled piers or rock anchors can be utilized for overturning resistance. Preliminary geotechnical foundation design criteria are presented in Table 4. These should be confirmed as part of a design level report.

Table 4 – Foundation Design Criteria

Shallow Spread Footings	
Minimum depth:[1]	18 inches
Allowable bearing capacity:[2]	
Weathered Bedrock	3,000 psf
Base friction coefficient:	0.40
Lateral passive resistance:[3,4]	
Sandy Clay Soils	300 pcf
Weathered Bedrock	400 pcf
Drilled Piers or Rock Anchors	
Min. Diameter:	
Drilled Pier	18 inches
Rock Anchor	6 inches
Minimum Pier Embedment into Bedrock:	5 feet
Allowable skin friction [2,5,6]:	
Sandy Clay Soils	1,000 psf
Weathered Bedrock	2,000 psf
Lateral passive resistance[7]:	
Sandy Clay Soils	300 pcf
Weathered Bedrock	400 pcf

Notes:

(1) Foundations to bear on weathered bedrock. Maintain at least 7 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) For level ground. Ignore uppermost foot of resistance. Shall be reduced for downward sloping conditions.

(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 roadway fill and stabilize cuts made to create level building pads. Site retaining walls can be constructed by laying back slopes, construction walls and backfilling, or by making top-down vertical cuts supported with shotcrete-faced soil nail walls. 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. Preliminary design criteria is presented in Table 5. To be confirmed as part of the design level investigation.

Retaining walls that can deflect at the top such as site walls can be designed using the unrestrained criteria. 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

Active Earth Pressure	Unrestrained[,2]	Restrained[,3]
Level Ground	45 pcf	30 X H psf
2:1 Slope	60 pcf	40 X H psf
Seismic Surcharge[3,4]	15 x H psf	
Vehicular Surcharge[3]	50 psf upper 5 feet	
Soil Creep Surcharge[3]	100 psf upper 3 feet	
Tiebacks or Soil Nails[5]:		
Minimum Diameter:	6 inches	
Design Skin Friction:	2,500 psf	
Unbonded Zone:	0.7 x Wall Height, 5 Feet Min	

Notes:
1. (1) Interpolate earth pressures for intermediate slopes.
2. (2) Equivalent fluid pressure.
3. (3) Rectangular distribution. H = Wall Height = top of soil backfill to bottom of wall.
4. (4) The factor of safety for short-term seismic conditions can be reduced to 1.1 or greater.
5. (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. (6) Angle of Internal Friction, effective stress.
7. (7) Apparent (effective) Cohesion, for seismic conditions 250 psf of additional cohesion may be included.
8. (8) Unit Weight of Soil
9. (9) Ignore skin friction within active wedge of wall (approximately equal to wall height).

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.

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. Debris barriers should be positioned to deflect material away from the development area. Several methods to mitigate debris impact are available.

An earth berm could be constructed behind the proposed development area that could redirect any debris into the existing channels on to the north and south ends of the building area. The earth berm should be at least 8-feet high as measured from the existing ground surface. Side slopes should be no steeper than 2:1 (horizontal: vertical). The berm should be constructed of select fill, placed on a prepared subgrade, and compacted in lifts to 92% relative compaction as described in Section 5.2.3. A minimum 10-foot wide access route should be maintained to facilitate access for trucks and equipment to remove slide debris, maintain the berm, and maintain associated drainage facilities as needed. The access road should extend to form a spillway “notched” into the crest of the catchment berm. The access route should be graded with a minimum 5% cross-slope to force surface flow runoff into an adjacent infiltration trench or stormwater detention basin.

Another 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 soil nail 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 (i.e. garage slabs), we recommend at least four inches of Class 2 aggregate base (Caltrans) 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 three-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 non-perforated 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 under slab 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 building. 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).

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 Pavement Sections

New pavements are expected to include both rigid concrete pavements and flexible asphalt pavements. We have calculated thicknesses for asphalt pavements in accordance with Caltrans procedures for flexible pavement design. For preliminary 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 – Preliminary 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 proposed development plan. As the development plan is refined and accepted, 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 Marin County 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 reference report subsurface data and our experience with soils in this geographic area.

8.0 LIST OF REFERENCES

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SITE COORDINATES
LAT. 37.90583°
LON. -122.51130°

SITE LOCATION

N.T.S.

N

REFERENCE: Google Earth, 2024

MPEG MILLER PACIFIC ENGINEERING GROUP

FILENAME: 3011.002 70 N Knoll Rd.dwg

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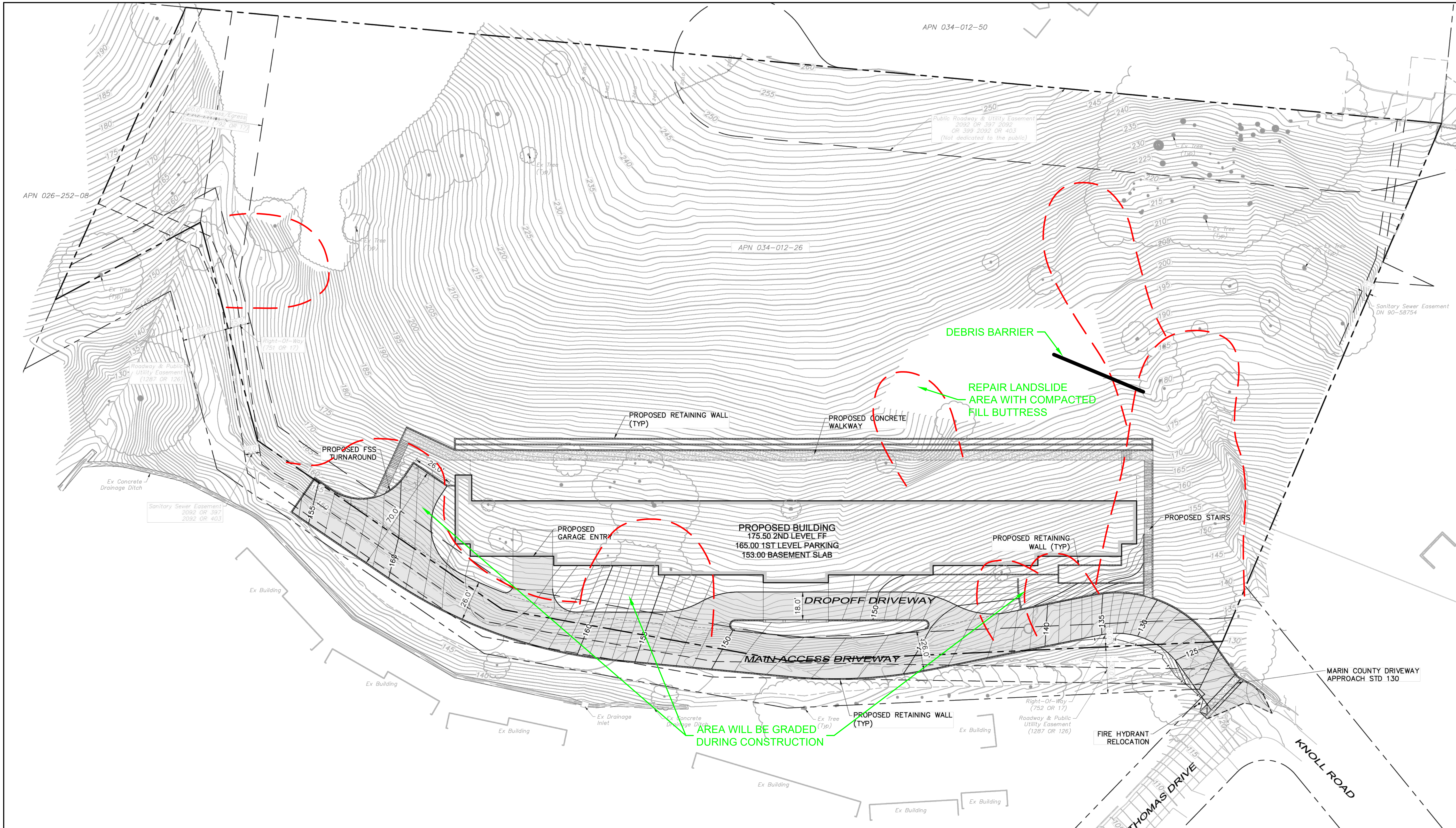
SITE LOCATION MAP

Spirit Living Group, LLC
70 North Knoll Road
Mill Valley, California

Project No. 3011.002 Date: 9/18/2024

Drawn RJA
Checked

1
FIGURE



Approximate slide limits from Herzog Geotechnical Consulting report from 2019

SCALE

0 30 60 120 FEET

N

REFERENCE: Plans by Adobe Associates, inc., "Spirit Living Group Senior Housing", Sheet C2.0, July 22, 2024.

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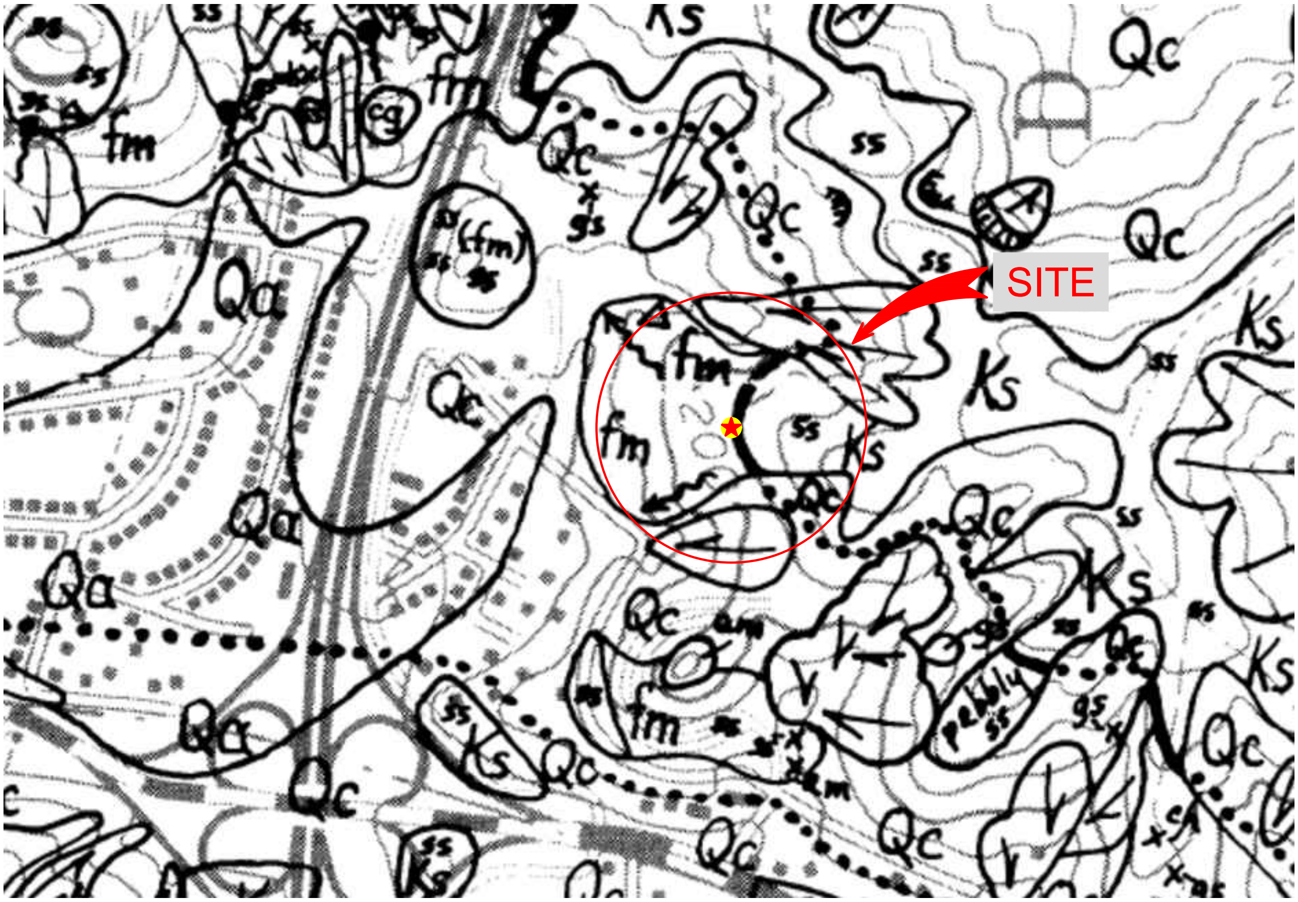
PROPOSED OVERALL SITE PLAN

Spirit Living Group, LLC
70 North Knoll Road
Mill Valley, California

Project No. 3011.002 Date: 9/18/2024

Designed
Drawn RJA
Checked SAS

2
FIGURE



LEGEND

REGIONAL GEOLOGIC MAP

NORTH

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.

Slope Creep: Slopes exhibiting evidence of continuous or intermittent downslope creep of surface zone.

Qa Alluvium: Unconsolidated deposits of clay, silt, sand, and gravel underlying valley bottoms, consisting of materials transported and deposited by streams.

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: with minor amounts of conglomerate. Occurrences of principal rock types in this unit are indicated by the following lithologic symbols:

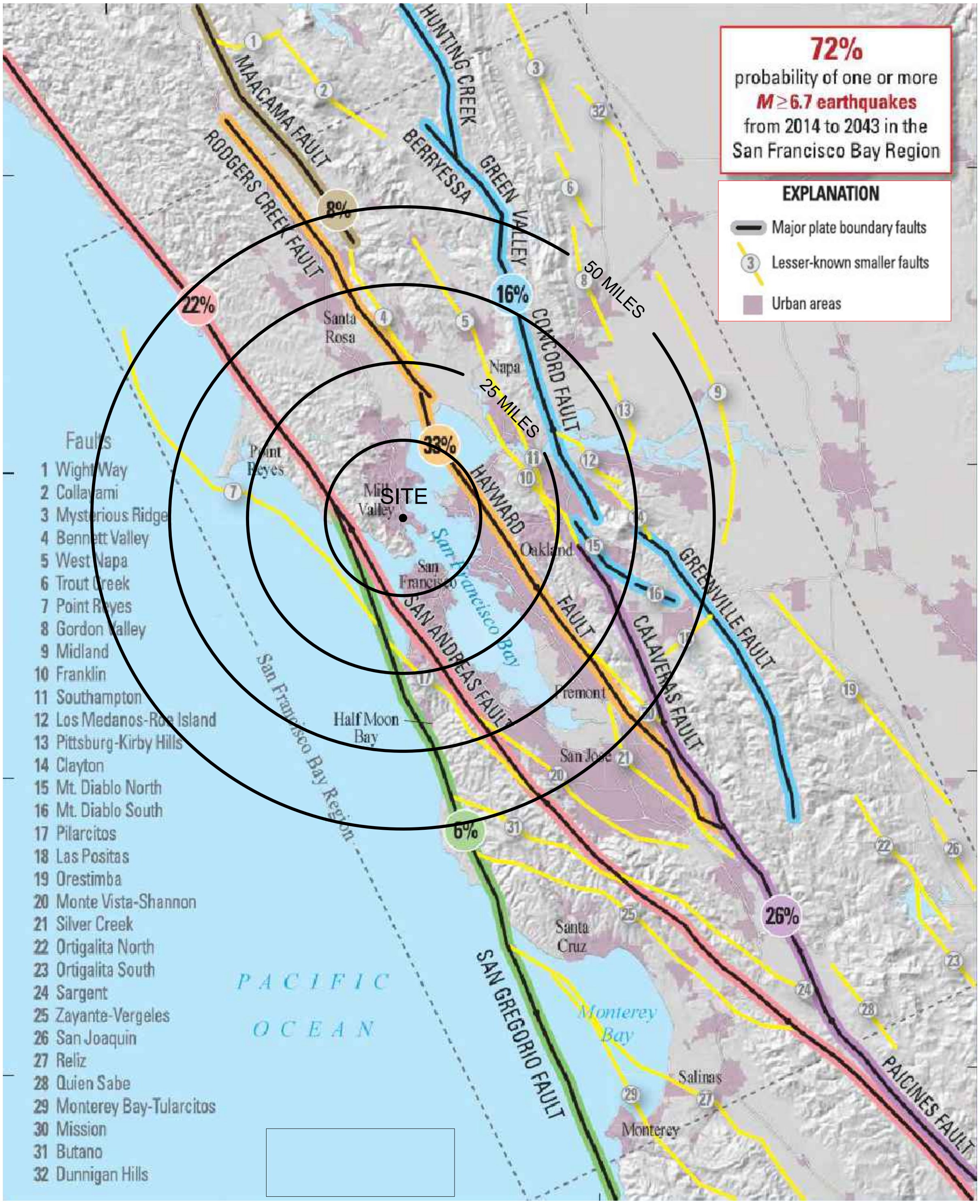
- ss Sandstone: mainly thick bedded to massive, medium to coarse grained, fairly well sorted, angular to sub-rounded grains. Light gray where fresh, buff where weathered.

fm Franciscan Melange: A tectonic mixture consisting of small to large masses of resistant rock types, principally sandstone, greenstone, chert, and serpentine, but including various exotic metamorphic rock types, embedded in a matrix of pervasively sheared or pulverized rock material. Exposures of rock masses within the melange matrix are indicated by the following lithologic symbols:

- ss Sandstone and Shale: Mainly graywacke type sandstone, with or without alternating beds of dark gray shale.
- gs Greenstone: Altered or metamorphosed basaltic igneous rocks
- sp Serpentinite: Pale green to dark green, fine grained, metamorphic rocks composed almost entirely of the magnesium silicate minerals lizardite and chysotile.

REFERENCE: Rice, Salem J., Smith, Theodore C., 1976, "Geology of the Tiburon Peninsula, Sausalito, and Adjacent Areas, Marin County, California", California Division of Mines and Geology, Scale 1:12,000.

MPEG MILLER PACIFIC ENGINEERING GROUP	504 Redwood Blvd. / Suite 220 / Novato, CA 94947 / T 415 / 382-3444 / F 415 / 382-3450 / www.millerpac.com	REGIONAL GEOLOGIC MAP		
A CALIFORNIA CORPORATION, © 2024, ALL RIGHTS RESERVED		Spirit Living Group, LLC 70 North Knoll Road Mill Valley, California	Drawn RJA Checked	3 FIGURE
FILENAME: 3011.002 70 N Knoll Rd.dwg		Project No. 3011.002 Date: 9/18/2024		



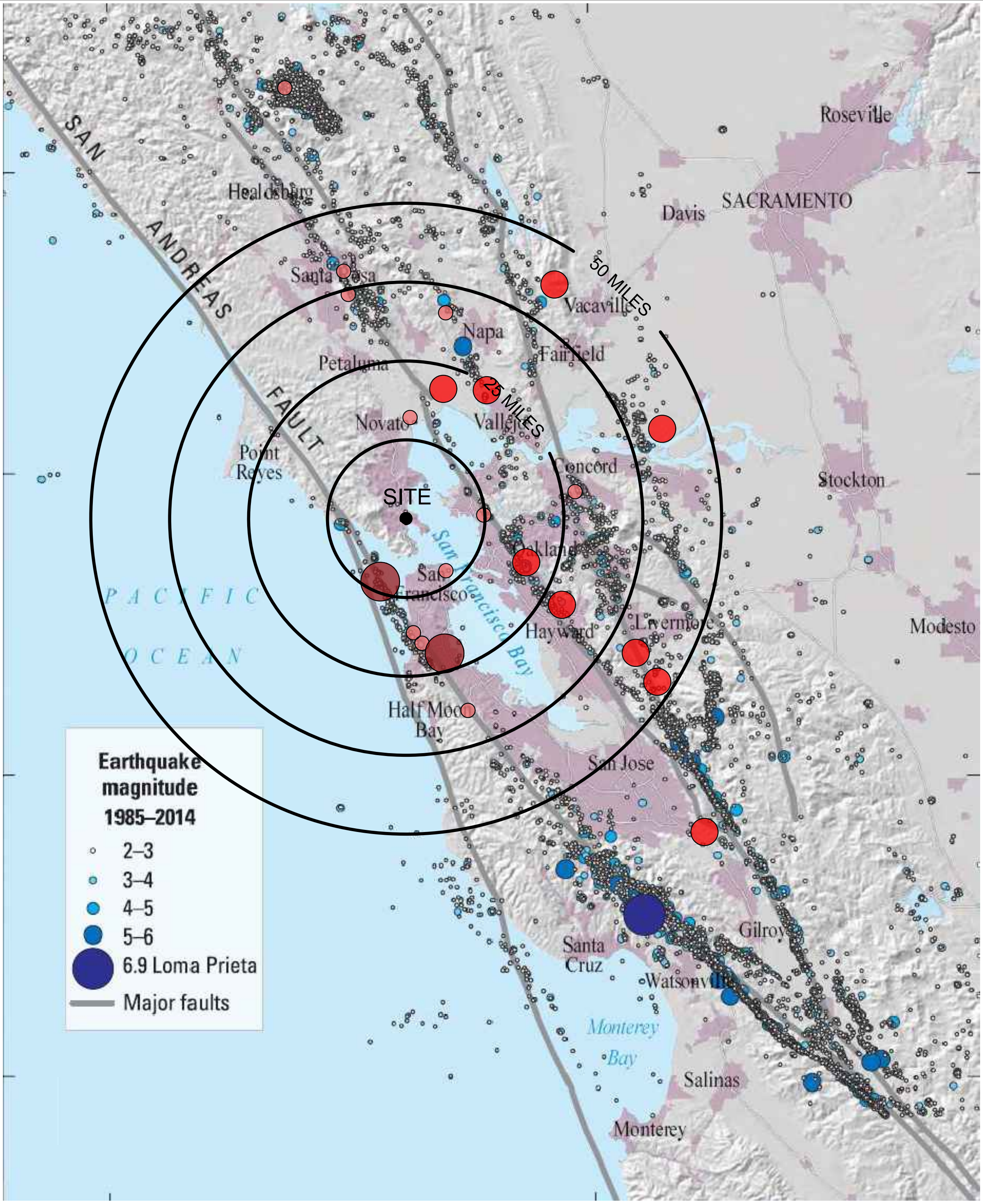
SITE COORDINATES
LAT. 37.90583°
LON. -122.51130°

SCALE
0 12.5 25 50 MILES

NORTH

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).

MPEG MILLER PACIFIC ENGINEERING GROUP	504 Redwood Blvd. Suite 220 Novato, CA 94947 T 415 / 382-3444 F 415 / 382-3450 www.millerpac.com	ACTIVE FAULT MAP		
A CALIFORNIA CORPORATION, © 2024, ALL RIGHTS RESERVED FILENAME: 3011.002 70 N Knoll Rd.dwg		Spirit Living Group, LLC 70 North Knoll Road Mill Valley, California Project No. 3011.002 Date: 9/18/2024	Drawn RJA Checked	4 FIGURE



SITE COORDINATES
LAT. 37.90583°
LON. -122.51130°

SCALE
0 12.5 25 50 MILES

NORTH

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

Spirit Living Group, LLC
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Project No. 3011.002 Date: 9/18/2024

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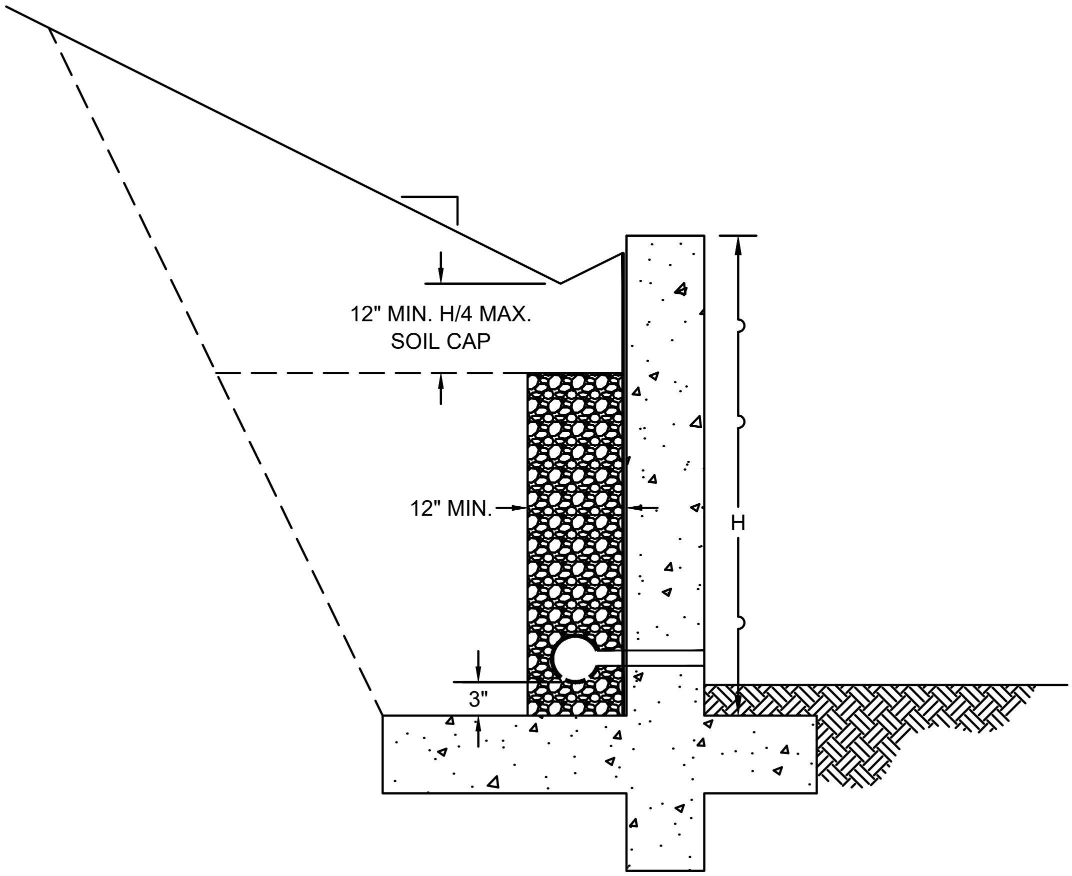
5
FIGURE



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.

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TYPICAL RETAINING WALL BACKDRAIN

Spirit Living Group, LLC
70 North Knoll Road
Mill Valley, California

Project No. 3011.002 Date: 9/18/2024

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6
FIGURE

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APPENDIX A – GEOTECHNICAL REFERENCE DATA

HERZOG
GEOTECHNICAL
CONSULTING ENGINEERS

September 10, 2019
Project Number 3623-01-18

Vikrum and Gagan Nijjar
134 Chesapeake Lane
Novato, California 94949

RE: Supplemental Geotechnical Investigation
Proposed Residence
Assessor's Parcel #034-012-26
Mill Valley, California

Dear Mr. and Mrs. Nijjar:

This presents the results of our supplemental geotechnical investigation for the proposed residence at Assessor's Parcel #034-012-26 in Mill Valley, California. We previously performed a geotechnical investigation in connection with a different project layout and presented results in our report dated January 22, 2018. The scope of our current investigation was to review selected geologic references, observe exposed site conditions, drill three test borings in the revised project area, perform laboratory testing, conduct engineering analyses, and develop geotechnical recommendations for the design and construction of the project. Our scope of work was outlined in our professional services agreement dated December 18, 2017.

PROJECT DESCRIPTION

The project will consist of constructing a new single family residence on the existing vacant lot above the northeast corner of the intersection of Thomas Drive and North Knoll Road . The project will be accessed by an approximately 300 foot long driveway. The project is shown on the plans by Ziegler Civil Engineering transmitted August 18, 2019.

WORK PERFORMED

Prior to performing our investigation, we reviewed historical aerial photos and selected geologic references. A listing of reference materials reviewed is attached to this report.

On January 11, 2018, we explored the subsurface conditions at the site to the extent of ten test borings (B-1 through B10) ranging between approximately 2-1/2 and 8-1/2 feet deep, and extending into bedrock. On August 1, 2019, we explored the subsurface conditions in the area of the currently proposed residence to the extent of three supplemental borings (B-11 through B-13)

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ranging between approximately 6 and 8-1/2 feet deep, and extending into bedrock. The test borings were drilled with portable drilling equipment at the approximate locations shown on the attached *Site Plan*, Plate 1.

Our personnel observed the drilling, logged the subsurface conditions encountered, and collected soil samples for visual examination and laboratory testing. Samples were retrieved using Sprague and Henwood and Standard Penetration Test samplers driven with a 70-pound hammer. Penetration resistance blow counts were obtained by dropping the hammer through a 30-inch free fall. The number of blows was recorded for each 6 inches of sampler penetration. These blow counts were then correlated to equivalent standard penetration resistance blow counts. The blows per foot recorded on the boring logs represent the accumulated number of correlated standard penetration blows that were required to drive the sampler the last 12 inches or fraction thereof.

Logs of our recent and previous test borings are presented on Plates 2 through 14. The soils encountered are described in accordance with the criteria presented on Plate 15. Bedrock is described in accordance with the *Engineering Geology Rock Terms* presented on Plate 16. The logs depict our interpretation of subsurface conditions on the date and at the depths indicated. The stratification lines on the logs represent the approximate boundaries between soil types; the actual transitions may be gradational.

Selected samples were laboratory tested to determine their moisture content, dry density and plasticity. Laboratory test results are posted on the boring logs in the manner described on the *Key to Test Data*, Plate 15. Results of the Atterberg Limits Plasticity testing are presented on Plate 17.

FINDINGS

Site Conditions

The project area is an undeveloped west-facing hillside located northeast of the intersection of North Knoll Road and Thomas Drive in Mill Valley, California. A dirt road extending across the base of the site was created by excavating along the upslope side and placing fill beneath the downslope portion. This fill bank ranges to about 10 feet high and extends down at between about 2:1 to 2-1/2:1 (horizontal:vertical) to a neighboring residential development. The cut bank along the upslope side of the road ranges to about 8 feet high and is inclined between approximately 1-1/2:1 and 2:1. This bank generally exposes colluvial soils (slopewash), and has experienced erosion and localized shallow slumping. A grass-covered hillside above the cut bank extends up towards the east at inclinations ranging between approximately 3:1 and 2:1 to a southwest-trending spur ridge. The hillside contains trees and areas of dense low vegetation. Portions of the hillside display lobate and hummocky topography indicative of previous slumping and earthflow landslide movement. Areas of landsliding judged to pose a potential impact to the project or to off-site improvements are delineated on Plate 1.

Subsurface Conditions

The site is within the Coast Range Geomorphic Province which includes San Francisco Bay and the northwest-trending mountains that parallel the coast of California. These features were formed by tectonic forces resulting in extensive folding and faulting of the area. Previous geologic mapping by Rice (1976) indicates that the site is underlain by bedrock of the Franciscan Melange. This unit is Jurassic to Cretaceous in age, and typically consists of a heterogeneous mixture of sandstone, sheared shale, metavolcanic rock, serpentinite and chert.

Our test borings encountered topsoil, colluvium (slopewash), slide debris, and residual soils overlying bedrock. The topsoil encountered generally consists of soft and organic sandy silt and clay and of loose and organic silty sand. The colluvial soils encountered consist of soft sandy clay and of loose to medium dense silty and clayey sand and gravel. The slide debris encountered consists of soft to medium stiff sandy and gravelly clay. The residual soils encountered consist of loose to medium dense clayey sand derived from the in-place weathering of the underlying parent bedrock. The soils encountered are relatively weak and compressible, and are subject to downslope creep. In addition, portions of the soils at the site are moderately expansive. Expansive soils undergo changes in volume with changes in moisture content, and can cause slabs, pavements and lightly loaded foundations to heave and crack and can increase pressures on retaining walls. Bedrock encountered in the borings generally consists of firm to moderately hard shale and firm to hard sandstone.

The approximate test boring locations are shown on the *Site Plan* (Plate 1). The test borings encountered the following profiles:

	Depth (feet)			
Boring	**Topsoil/Colluvium**	**Slide Debris**	**Residual Soil**	**Bedrock**
B-1	---	0-5.5	---	5.5-6.0+
B-2	0-8.0	---	---	8.0-8.5+
B-3	0-4.0	---	---	4.0-5.0+
B-4	0-1.5	---	---	1.5-2.5+
B-5	0-0.5	---	0.5-2.2	2.2-3.0+
B-6	0-2.5	---	2.5-4.0	4.0-5.0+
B-7	0-1.0	---	---	1.0-3.0+
B-8	0-0.5	---	0.5-2.0	2.0-3.0+
B-9	---	0-4.5	---	4.5-5.0+
B-10	---	0-4.0	---	4.0-5.0+
B-11	---	0-5.0	5.0-7.5	7.5-8.5+
B-12	0-4.5	---	4.5-5.5	5.5-6.0+
B-13	0-4.0	---	4.0-5.0	5.0-6.0+

Descriptions of the subsurface conditions encountered are presented on the boring logs.

Groundwater

Free groundwater did not develop in the borings prior to backfilling. Groundwater levels at the site are expected to fluctuate over time due to variations in rainfall and other factors. Rainwater percolates through the relatively porous surface soils. On hillsides, the water typically migrates downslope in the form of seepage within the porous soils, at the interface of the soil/bedrock contact, and within the upper portions of the weathered and fractured bedrock.

GEOLOGIC AND SEISMIC HAZARDS

Fault Rupture

The property is not within a current Alquist-Priolo Earthquake Fault Zone (EFZ), and we did not observe geomorphic features that would suggest the presence of active faulting at the site. As such, we judge that the risk of ground rupture along a fault trace is low at this site.

Ground Shaking

The San Francisco Bay Region has experienced several historic earthquakes from the San Andreas and associated active faults. Mapped active faults (those experiencing surface rupture within the past 11,000 years) nearest the site are summarized in the following table.

Fault	Distance		Moment Magnitude[1]	Acceleration (g)[2]	
	Miles	Kilometers		M[3]	M+1 [3]
San Andreas (Northern)	7.3	11.8	8.0	0.39	0.70
Seal Cove/San Gregorio	8.8	14.2	7.4	0.29	0.53
Hayward	9.4	15.2	7.3	0.26	0.48
Healdsburg/Rodgers Creek	18.4	29.6	7.3	0.17	0.31

(1) Estimated maximum magnitudes from Caltrans Fault Database (Version 2A).
(2) Peak ground acceleration averaged from New Generation Attenuation (NGA-West 2) relationships by Abrahamson, Silva & Kamai (2104), Boore, Stewart, Seyhan and Atkinson (2014), Campbell and Bozorgnia (2014), Chiou and Youngs (2014), and Idriss (2014). Estimated shear wave velocity (V_{S30}) = 525 m/s.
(3) M = mean value; M+1 = mean+1 standard deviation value.

Deterministic information generated for the site considering the proximity of active faults and estimated ground accelerations are presented in the table above. The estimated ground accelerations were derived from the above-referenced mean attenuation relationships, and are based on the published estimated maximum earthquake moment magnitudes for each fault, the shortest distance between the site and the respective fault, the type of faulting, and the estimated shear wave velocities of the on-site geologic materials. The deterministic evaluation of the potential for ground shaking assumes that the anticipated maximum magnitude earthquake produces fault rupture at the closest proximity to the site, and does not take recurrence intervals or other probabilistic effects into consideration. This evaluation also does not consider directivity effects, topographic amplification, or other phenomena which may act to amplify ground motions.

Data presented by the U.S. Geological Survey (2016) estimates the chance of one or more large earthquakes (Magnitude 6.7 or greater) in the San Francisco Bay region before the year 2043 to be 72 percent. Consequently, we judge that the site will likely be subject to strong earthquake shaking during the life of the improvements.

Liquefaction

During ground shaking from earthquakes, liquefaction can occur in saturated, loose, cohesionless sands. The occurrence of this phenomenon is dependent on many factors, including the intensity and duration of ground shaking, soil density, particle size distribution, and position of the ground water table (Idriss and Boulanger, 2008). The soils encountered in our test borings contain a high percentage of fine grained materials (silt and clay). Thus, we judge that the likelihood of liquefaction during ground shaking is low.

Densification

Densification can occur in low density, uniformly-graded sandy soils above the groundwater table. We judge that significant densification is unlikely to occur in the areas explored because of the high silt and clay content of the soils encountered in the test borings.

Landsliding

Regional mapping by Rice (1976) does not indicate the presence of previous landsliding in the area of the proposed residence, and maps of slope failures resulting from the severe 1982 storms (Davenport, 1984) and of slope failures resulting from the heavy 1997/1998 storms (USGS, 1999) do not indicate that sliding was reported at the site at either of those times. However, the Rice mapping indicates that the site is subject to severe downslope creep, and depicts previous landsliding within the swales extending across the northern and southern portion of the site. The slide within the northern portions of the site is mapped as an approximately 800 horizontal foot long earthflow complex about 500 feet wide at the top, and funneling down to less than 100 feet wide at the base. This slide and an additional small failure noted at the slide toe are situated within a confined swale area offset well away from proposed improvements, and do not pose a risk to the proposed project.

The mapped slide located south of the project is an approximately 400 horizontal foot long earthflow which is about 250 feet wide at the top and narrows to less than 50 feet wide where it terminates at the intersection of North Knoll Road and Thomas Drive. The northern margin of this mapped slide appears to roughly correspond with the southern slide feature depicted on Plate 1. We noted additional small slump failures north of this area which extend across the proposed driveway alignment. We judge that the slides in this area could be subject to renewed instability, particularly as a result of earthquake shaking, heavy rainfall and/or time-dependent material strength loss. In order to reduce the risk to the portions of the driveway extending across slide areas, it will be necessary to repair these slide areas as properly compacted and subdrained fill buttresses which are benched and keyed into bedrock below the depth of sliding. We recommend that slide repairs extend at least 30 feet upslope of the driveway to reduce the reduce the risk of instability of the alignment. Catchment facilities will need to be provided to reduce the risk of debris impact originating from unrepaired slopes above the buttresses. If the risk of reactivated movement of other portions of the slides will not be acceptable, it will be necessary to extend slope repairs to encompass these areas. Because of the slope above the residence, it would be prudent to provide slough catchment along the upslope side of the house.

We judge that the cut banks for the existing dirt road extending across the base of the site are subject to erosion and slumping. We judge that the risk to the improvements will be mitigated by supporting walls and foundations in competent bedrock as outlined in this report. However, if the risk of bank erosion and slumping will not be acceptable, we should be contacted to provide geotechnical recommendations for stabilizing these areas.

CONCLUSIONS

Foundation Support

Our test borings indicate that the project area is underlain by varying thicknesses of relatively weak soils which are subject to settlement under new foundation loads and to gradual downslope creep. In addition, portions of the soils at the site are moderately expansive, and can cause slabs and lightly loaded foundations to heave and crack. Expansive soils can increase pressures on retaining walls. We therefore conclude that proposed improvements should be supported on spread footings and/or drilled, cast-in-place, reinforced concrete piers which extend into undisturbed bedrock. It will be necessary to design foundations to resist lateral forces imposed by creeping soils above the bedrock. Spread footings will only be feasible in areas where level cuts expose bedrock well away from downslopes, while drilled piers can be used everywhere. Hard drilling or coring in resistant bedrock will be required to achieve required pier embedments. It will be necessary to interconnect piers with grade beams, and to separate grade beams from the expansive soils with an approved void forming product. We estimate that differential settlements of foundations designed in accordance with the recommendations contained in this report will be on the order of half an inch.

Slab and Pavement Support

To reduce differential settlements, pavements, interior slabs-on-grade, and other settlement sensitive slabs should be either founded on bedrock, or on properly compacted fill founded on bedrock. In order to reduce expansive soil heave, at least the upper 30 inches of fill beneath slab subgrade and the upper 18 inches of fill below pavement subgrade should consist of approved non-expansive material. Alternatively, slabs should be designed to structurally span between foundations supported on bedrock and separated from the expansive soils with an approved void forming product.

Grading and Retaining Walls

Due to the presence of relatively weak soils, it will be necessary to fully retain all cuts with engineered retaining walls. Retaining walls should be supported on foundations which extend into undisturbed bedrock, and which are designed to resist creep forces imposed by the soils above the rock. Walls should be provided with adequate backdrainage to prevent hydrostatic buildup. In order to reduce expansive soil heave against retaining walls, wall backfill should consist of approved non-expansive fill.

Unretained fills should be keyed and benched into bedrock, and provided with subdrainage as outlined in this report. In areas where the driveway will extend across the mapped slide areas depicted on Plate 1, it will be necessary to reconstruct the slopes within and upslope of the proposed improvements as properly compacted and subdrained earth fill buttresses which are keyed and benched into bedrock. Side repairs should extend at least 30 feet upslope of the driveway to reduce the reduce the risk of instability of the alignment. In areas where required fill slopes will be steeper than 2:1 (horizontal:vertical), it will be necessary to utilize geogrid reinforcement, retaining walls or rip-rap.

Excavation and Shoring

We anticipate that planned cuts will expose relatively weak soils and bedrock with bedding, fracture and shear surfaces which may slope adversely into the excavations. Excavations must therefore be shored to laterally support the banks and to maintain stability of adjacent areas. Shoring should be designed by the Contractor's engineer to resist the lateral earth pressures presented in this report. Adequate drainage facilities should be provided to prevent hydrostatic buildup behind the shoring.

During construction, cuts should be examined by our personnel for the presence of adverse bedding, fracturing conditions, or lithologic contacts that could promote slope instability. As excavation proceeds, conditions may be exposed which require design modifications.

Our investigation indicates that excavations will likely encounter areas of hard bedrock which will necessitate the use of heavy-duty, hydraulically-driven excavation equipment. Resistant blocks of hard rock may require hoe-ramming. Hard drilling or coring may be required to achieve the required penetrations for drilled piers.

Geotechnical Drainage

It is important that surface and subsurface water be controlled to reduce future moisture variations in the weak and expansive on-site soils. Perimeter subdrains and slab underdrains should be provided to reduce water infiltration beneath the structures, and all roofs should be provided with gutters and downspouts. It will be necessary to extend drains to approved storm drain or at approved erosion resistant outlets well away from improvements or potentially unstable slopes.

RECOMMENDATIONS

Site Grading/Slope Stabilization

Clearing

Areas to be developed or graded should be cleared of trees, brush and deleterious material, and then stripped of the upper soils containing root growth and organic matter. The cleared materials and strippings should be removed from the site.

Overexcavation

Areas of planned fills, pavements, interior slabs, and/or other settlement sensitive slabs-on-grade (except where void forms will be provided beneath structural slabs) should be overexcavated as necessary to create level benches in bedrock. Overexcavation should extend at least 3 horizontal feet beyond the edges of planned slabs and pavements. Along the downslope edges of unretained slabs and pavements, overexcavation should extend laterally as necessary to encompass an imaginary 2:1 plane projected down from the slab/pavement edge to the overexcavated bedrock surface. The depth and extent of required overexcavations should be approved in the field by the geotechnical engineer prior to placement of fill or improvements. Expansive soils should be segregated during excavation, and not used in Select Fill zones.

Where unretained fills will be located on slopes steeper than 5:1, keyways should be excavated into bedrock at the downslope edges of planned fills. The keyways should slope inward, should be at least 12 feet wide, and should extend at least 18 inches into competent bedrock along the downslope edge. The downslope edge of the keyway should be located beyond a 1:1 line projected down from the planned toe of the fill slope. Additional benches should be excavated into bedrock upslope of the keyway. The final dimensions and depths of keyways and benches should be evaluated by our representative in the field during construction.

In order to reduce the risk to the portions of the driveway extending across slide areas, overexcavation and keyway/bench excavation should extend at least 30 horizontal feet upslope of the driveway.

Temporary Slopes

Temporary slopes should be laid back or shored in conformance with OSHA standards. The Contractor should slope temporary excavations no steeper than 1-1/2:1, or should install shoring as excavations proceed in order to maintain lateral support. Excavations within slide areas should be performed in narrow slots during the dry season to avoid triggering instability of upslope areas. All temporary slopes and shoring should be contractually established as solely the responsibility of the Contractor, and design and inspection of temporary slopes and shoring are specifically excluded from our scope of work.

Subdrainage

A chimney subdrain should be installed along the rear of the keyways. A heavy-walled rigid perforated pipe should be placed on a 2-inch thick basal layer of drain rock (1/2 to 3/4 inch diameter) or Caltrans Class II Permeable Material. All drain pipes should be at least 4 inches in diameter, should be PVC or ABS, and should be Schedule 40 or have an SDR of 35 or equivalent. The pipe should be covered by a 1.5 foot thick (minimum) chimney of drain rock that extends at least 5 feet up the rear wall of the keyway excavation. If clean drain rock is used, the drain rock should be entirely wrapped with a layer of geotextile filter cloth (Mirafi 140-N or equivalent). If Caltrans Class II Permeable Material is used, the filter cloth may be eliminated. A capped cleanout riser should be provided for all subdrains. The perforated pipe should outlet into a solid line that discharges at an approved protected outlet. Additional subdrains should be installed at least every 15 vertical feet, along the upper boundary of planned fills, and where evidence of seepage is observed, as recommended by our representative in the field during construction. Subdrains should be constructed in a manner similar to that of the keyway chimney drain. Subdrains should be located so as to avoid future foundation construction. As-built drawings of subdrain locations should be developed during construction.

Compaction of Fill

Within 30 inches of slab subgrade, 18 inches of pavement subgrade, and within 30 inches of the surface of fill slopes, only non-expansive Select Fill should be used. The Select Fill should be placed in lifts not exceeding 8 inches in uncompacted thickness, moisture conditioned, and compacted to at least 90 percent relative compaction. Below the Select Fill zone, the excavated fill may be replaced in lifts not exceeding 8 inches in uncompacted thickness, moisture conditioned to at least 3 percent over optimum moisture content, and recompacted to between 90 and 93 percent relative compaction. Relative compaction refers to the in-place dry density of a soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557 test procedure. Optimum moisture content is the water content of the soil (percentage by dry weight) corresponding to the maximum dry density. As the fill continues upslope, it should be continually benched and keyed into bedrock as recommended by our representative in the field during construction.

Material for Fill

All fill material should be free of organic matter. The fill material should not contain rocks or lumps larger than 4 inches in greatest dimension, and no more than 15 percent should be larger than 2 inches. Select Fill should consist of material with little or no potential for expansion. The Select Fill material should have a plasticity index of 15 percent or less, and a maximum liquid limit of 40 percent. Herzog Geotechnical should approve all imported fill prior to it being brought to the site.

Finished Slopes

All cuts in soil within improvements areas should be retained with retaining walls. Finished unreinforced fill banks and cut slopes in bedrock should be retained or constructed at an inclination no steeper than 2:1. The outer 30 inches of fill slopes should consist of compacted non-expansive Select Fill to reduce sloughing. Fill slopes should be overbuilt, and trimmed back as necessary to expose a well-compacted surface. Proposed fill banks steeper than 2:1 should be reinforced with geogrid to mitigate sloughing and instability. We should be contacted to provide specifications for geogrid based on the proposed finished slope geometry and on strength testing of the proposed fill material.

Routine maintenance of slope sloughing and erosion should be anticipated. Fill slopes and areas disturbed during construction should be planted with vegetation to reduce erosion. Surface water runoff should be intercepted and diverted away from banks and retaining walls.

Rip-Rap Buttress Repairs

In areas where slopes will be reconstructed as rip-rap buttresses, all existing slide debris and weak soils should be carefully removed from the slide areas as necessary to provide a keyway and benches into bedrock as described below, and to provide room for an at least 3-foot thick rip-rap buttress. Rip-rap should be founded on at least 8-foot wide level keyways excavated into competent bedrock at the toe of the slides. The downslope edge of the keyways should extend at least 18 inches into undisturbed bedrock. In areas where the toe of rip-rap buttresses will be supported by retaining walls, the keyway should be at least 6 feet wide and should be located at least 3 feet below the top of the wall. Additional benches should be excavated into the slopes as recommended by our representative in the field during construction. A layer of heavy geofabric (Mirafi Filterweave 700 or equivalent) should be placed over the keyway and between the excavated slope and the rip-rap.

Subdrains should be installed at the rear of the keyways. The subdrains should consist of 6-inch diameter, perforated PVC or ABS pipe embedded in at least 1 cubic foot of Caltrans Class 2 Permeable Material per lineal foot. The pipe should be Schedule 40 or have an SDR of 23.5 or equivalent, and should be sloped to drain at least 1 percent to an approved outlet or street. A cleanout riser should be provided for the subdrain. Sweeps or sanitary wyes should be used to allow for future maintenance.

Rip-rap graded in size from 12 to 18 inches should be placed on top of the geofabric to obtain the final slope. The rip-rap section should be at least 3 feet thick measured perpendicular to the slope. The rip-rap should extend to the elevation of the top of the slide scarps, and should be at least 3 horizontal feet thick at the top edge.

Rip-rap inclined between 1:1 and 1-1/2:1 should be grouted in place. Grout should be placed using means that does not result in segregation by gravity, and should penetrate at least 18 inches into the rip-rap. Grout should be spaded and rodded into place by an approved method which results in the interstices being filled with grout.

Seismic Design

Based on the results of our investigation, the following seismic design criteria were developed in accordance with the *California Building Code* (2016) and *ASCE 7-10* (July 2013 errata):

Site Class	C
Site Coefficient F_a	1.0
Site Coefficient F_v	1.3
0.2 sec Spectral Acceleration S_S	1.50
1.0 sec Spectral Acceleration S_1	0.60
0.2 sec Max Spectral Response S_{MS}	1.50
1.0 sec Max Spectral Response S_{M1}	0.79
0.2 sec Design Spectral Response S_{DS}	1.00
1.0 sec Design Spectral Response S_{D1}	0.52
Design Category	D

Foundations

Drilled Piers

Drilled piers should be at least 18 inches in diameter and should extend at least 8 feet into bedrock. Design pier depths and diameters should be calculated by the Project Structural Engineer using the criteria presented below. The materials encountered during pier drilling should be evaluated by our representative in the field. Drill spoils should be removed from the site or placed as properly engineered and retained fill. The sidewalls of pier holes allowed to remain open may be subject desiccation and deterioration, which adversely impacts skin friction capacity. If concrete is not placed in pier holes within 72 hours of drilling, we should be notified to re-evaluate the holes to determine if they need to be reamed out or re-drilled.

Piers should be interconnected with grade beams to support structural loads and to redistribute stresses imposed by the creeping soils. Piers and grade beams should be designed and reinforced to resist creep forces acting from the finished ground surface to the top of the rock, and exerting an active equivalent fluid pressure of 60 pounds per cubic foot (pcf). For piers, this pressure should be assumed to act on 2 pier diameters.

Where expansive soils are encountered, a compressible void form product (Sure Void or equivalent) should be provided beneath the grade beams to prevent uplift due to expansive soils. Grade beams should be formed above the trench to prevent overpours, and care should be taken to prevent overpours (mushrooming) at the tops of piers.

The portion of the piers extending into bedrock can impose a passive equivalent fluid pressure of 400 pounds per cubic foot (pcf) acting over 2 pier diameters, and vertical dead plus real live loads of 1000 pounds per square foot (psf) in skin friction. These values may be increased by 1/3 for seismic and wind loads, but should be decreased by 1/3 for determining uplift resistance. The portion of piers designed to impose passive pressures should have at least 7 feet of horizontal confinement from the face of the nearest slope or wall. End bearing should be neglected due to the uncertainty of mobilizing end bearing and skin friction simultaneously.

If groundwater is encountered, it may be necessary to dewater the holes and/or to place concrete by the tremie method. If caving soils are encountered it will be necessary to case the holes. Hard drilling or coring will be required to achieve the required penetrations.

Spread Footings

Spread footings should only be used where level cuts expose bedrock. Spread footings should be at least 16 inches wide, should be bottomed at least 12 inches into bedrock, and should extend at least 18 inches below finished grade. Footings should be stepped as necessary to produce level tops and bottoms, and should be deepened as necessary to provide at least 5 feet of horizontal clearance in rock between the portion of footings designed to impose passive pressures and the face of the nearest slope or wall. Spread footings extending into competent bedrock can be designed to impose dead plus code live load bearing pressures of 4000 pounds per square foot (psf), and total design load bearing pressures of 5300 psf.

Resistance to lateral pressures can be obtained in rock from passive pressures against the sides of footings poured neat against rock and from friction along the base of footings. We recommend the following criteria for design:

Passive Pressures*	=	400 pounds per cubic foot (pcf) equivalent fluid pressure
Friction Factor	=	0.40 times net vertical dead load

* Neglect passive pressure in the top 12 inches where the surface is not confined by slabs or pavements.

Slab Support

In areas where slab subgrade excavations for interior, garage and other settlement sensitive slabs do not expose bedrock, slabs should be structurally supported, or else underlain by compacted fill which is founded on bedrock as outlined previously. Fill within the upper 30 inches of slab subgrade should be non-expansive.

Slab subgrade within living and garage areas should be sloped to drain into a 12 inch deep trench excavated beneath the middle of each slab. The trenches should be lined completely with a filter fabric such as Mirafi 140N, or equivalent. A 4-inch diameter rigid-perforated PVC or ABS (Schedule 40, SDR 35 or equivalent) pipe should be placed on a 1-inch layer of drain rock at the bottom of the trench with perforations down. The trench should be backfilled with drain rock up to slab subgrade elevation. The filter fabric should be wrapped over the top of the drain rock. The pipe should be sloped to drain by gravity to a non-perforated pipe which discharges at an approved outlet. The trench for the non-perforated pipe should be backfilled with properly compacted soil.

Interior and garage slabs should be underlain by a capillary moisture break consisting of at least 4 inches of free-draining, crushed rock or gravel (slab base rock) at least 1/4 inch, and no larger than 3/4 inch, in size. Moisture vapor detrimental to floor coverings or stored items will condense on the undersides of slabs. A moisture vapor barrier should therefore be installed over the capillary break. The barrier should be specified by the slab designer. It should be noted that conventional concrete slab-on-grade construction is not waterproof. The local standard under-slab construction of crushed rock and vapor barrier will not prevent moisture transmission through slab-on-grade. Where moisture sensitive floor coverings are to be installed, a waterproofing expert and/or the flooring manufacturer should be consulted for recommended moisture and vapor protection measures, including moisture barriers, concrete admixtures and/or sealants.

For protection from expansive soils, structural slabs should be underlain by an approved void forming product for protection from expansive soil heave. The void forms should consist of at least a 2-inch thick degradable and compressible paper product (SureVoid®, or equivalent). The capillary moisture break should be installed beneath the void form, and the moisture barrier should be carefully installed over the top of the void form.

Non-structural slabs should be at least 5 inches thick (or at least 6 inches thick for driveways) and should be reinforced at least with #4 reinforcing bars spaced at 12 inches on-center each way to control cracking due to differential movement. Control joints should be provided as determined by the Structural Engineer. Reinforcement should be continuous across joints. Slabs should be structurally separated from pier supported elements to accommodate differential movement. All slabs should be as designed by the project structural engineer.

Retaining Walls

Retaining walls should be supported in rock on foundations designed in accordance with the recommendations presented in this report. Free-standing retaining walls should be designed to resist active lateral earth pressures equivalent to those exerted by a fluid weighing 45 pounds per cubic foot (pcf) where the backslope is level, and 60 pcf for backfill at a 2:1 slope. Retaining walls restrained from movement at the top should be designed to resist an "at-rest" equivalent fluid pressure of 60 pcf for level backfill and 75 pcf for backfill at a 2:1 slope. For intermediate slopes, interpolate between these values. Walls supporting rip-rap buttresses should be designed to resist an equivalent fluid pressure of 65 pcf. Where wall backfill will be subject to vehicular loading, a traffic surcharge equivalent to 2 feet of additional backfill should also be added to walls. A minimum factor of safety against instability of 1.5 should be used to evaluate static stability of retaining walls.

Seismic wall stability should be evaluated based on a uniform lateral earth pressure of 10xH psf (where H is the height of the wall in feet). This pressure is in addition to the active equivalent fluid pressures presented in the report. For restrained walls, seismic pressures may be assumed to act in combination with active rather than at-rest earth pressures. The factor of safety against instability under seismic loading should be at least 1.1.

In addition to lateral earth pressures, retaining walls must be designed to resist horizontal pressures that may be generated by uphill retaining walls. Where a 1-1/2:1 (horizontal:vertical) plane projected downward from the base of an upslope retaining wall intersects the downslope wall, that portion of the downslope wall below the intersection should be designed for an additional horizontal uniform pressure equivalent to the maximum calculated lateral earth pressure at the base of the upslope wall.

Due to the presence of the slope above the house, the upslope wall of the building or walls upslope of the structure should be provided with at least 30 vertical inches of slough catchment. The wall design should include an equivalent fluid pressure of 75 pcf acting on the catchment area.

Retaining walls should be fully backdrained. The backdrains should consist of 4-inch diameter, rigid perforated pipe surrounded by a drainage blanket. The top of the drain pipe should be at least 8 inches below lowest adjacent downslope grade. The pipe should be PVC Schedule 40 or ABS with an SDR of 35 or better, and the pipe should be sloped to drain at least 1 percent by gravity to an approved outlet. Accessible subdrain cleanouts should be provided, and should be maintained on a routine basis. The drainage blanket should consist of clean, free-draining crushed rock or gravel wrapped in a filter fabric such as Mirafi 140N. Alternatively, the drainage blanket could consist of Caltrans Class 2 "Permeable Material", in which case the filter fabric may be omitted. A prefabricated drainage structure such as Mirafi Miradrain may also be used provided that the backdrain pipe is embedded in permeable material or fabric-wrapped crushed rock. The drainage blanket should be continuous, at least 1 horizontal foot thick, and should extend to within 1 foot of the surface. The uppermost 1 foot should be backfilled with compacted soil to exclude surface water. Where migration of moisture through retaining walls would be detrimental or undesirable, retaining walls should be waterproofed as specified by the Project Architect or Structural Engineer.

In order to reduce expansive soil heave against retaining walls, the zone located above a 1:1 plane projected up from the base of the wall should be backfilled with approved non-expansive Select Fill. Wall backfill should be spread in level lifts not exceeding 8 inches in thickness, brought to near the optimum moisture content, and compacted to at least 90 percent relative compaction. Retaining walls will yield slightly during backfilling. Therefore, walls should be backfilled prior to building onto or adjacent to the walls, and should be properly braced during the backfilling operations. Backfilling adjacent to walls should be performed only with hand-operated equipment to avoid over-stressing the walls.

Even well compacted backfill will settle about 1 percent of its thickness. Therefore, slabs and other improvements crossing the backfill should be designed to span or to accommodate this settlement.

Debris Barrier

Debris catchment facilities should be installed along the upslope side of the driveway. The catchment should be at least 5 feet high and should consist of either structural walls or fencing designed for an equivalent fluid impact pressure of 125 pounds per cubic foot (pcf). The catchment wall or fencing should be founded on pier foundations designed in accordance with the criteria presented previously. The passive pressure acting on the piers may be increased by 1/3 for transient impact loads.

Provisions should be provided for drainage from behind the barrier. The barrier should be periodically inspected for damage, and maintained and repaired as necessary. Clear storage space should be provided and maintained upslope of the barrier. The catchment area behind the barrier should be cleaned out following each slide episode, and annually prior to the winter rains. In addition, it may be necessary to remove fines that migrate through the barrier.

Pavements

Pavements should be underlain by bedrock or by properly compacted fill founded on bedrock as outlined in the *Site Preparation and Grading* section of this report. The upper 6 inches of subgrade should be moisture conditioned and compacted to at least 95 percent relative compaction, and should be smooth and unyielding. Aggregate baserock should be compacted to at least 95 percent relative compaction to provide a smooth unyielding surface. Characteristics and placement of asphalt concrete and aggregate base, and preparation for the subgrade should conform to the *California Department of Transportation Standard Specifications*, latest edition, except that the test method for compaction should be determined by ASTM D1557.

We assume that vehicle loading for this project will be light, consisting of automobiles and light trucks. Based on our experience with similar projects, a pavement section of 3 inches of asphalt concrete over 8 inches of Class 2 Aggregate Base may be used for planning purposes. If desired, the final pavement section may be refined during grading based on an R-value of the pavement subgrade and a traffic index (TI) provided by the Project Civil Engineer. Increasing asphalt concrete thickness would increase the life and durability of pavements. If pavements will be subjected to construction or other heavy truck traffic, the pavement thickness should be increased as recommended by the Project Civil Engineer.

Utility Trenches

Trenches should be backfilled with non-expansive Select Fill that is mechanically compacted to at least 90 percent relative compaction. Uncompacted lift thicknesses should not exceed 8 inches. Compaction by jetting should not be permitted. In order to prevent utility trench backfill conducting water into the expansive soils beneath the building or flatwork, granular backfill should not be used beneath the building or flatwork. Governmental or public utility requirements exceeding those listed above should govern where applicable.

Geotechnical Drainage

Positive drainage should be provided away from structures, walls and slabs. All roofs should be provided with gutters and downspouts. Drop inlets should be provided at low points as necessary to prevent ponding of surface water. Provisions should be made to intercept runoff upslope of the structure, and provisions should be made for fail-safe drainage around the structure to prevent flooding in the event that the drains become clogged. All downspouts and surface drains should be connected to non-perforated conduits which discharge at an approved storm drain or at approved erosion resistant outlets well away from improvements and/or potentially unstable slopes. Conduit should consist of rigid PVC or ABS pipe which is Schedule 40, SDR 35 or equivalent. Downspouts, surface drains and subsurface drains should be checked for blockage and cleared and maintained on a regular basis. Surface drains and downspouts should be maintained entirely separate from retaining wall backdrains, slab underdrains and foundation subdrains.

Foundation drains should be installed adjacent to all perimeter foundations. Perimeter retaining wall backdrains may be substituted for foundation drains. The drains should consist of trenches which extend 18 inches deep, or 12 inches below lowest adjacent interior or crawl space grade, whichever is deeper, and which are sloped to drain at least 1 percent by gravity. The trenches should be lined completely with a filter fabric such as Mirafi 140N, or equivalent. A 4-inch diameter rigid perforated PVC or ABS pipe (Schedule 40, SDR 35 or equivalent) should be placed on a 1-inch thick layer of drain rock at the bottom of the trenches with perforations down. The pipes should be sloped to drain at least 1 percent by gravity to a non-perforated pipe (Schedule 40, SDR 35 or equivalent) which discharges at an approved outlet. The trench for the perforated pipe should be backfilled to within 6 inches of the ground surface with drain rock. The filter fabric should be wrapped over the top of the drain rock. The upper 6 inches of the trenches should be backfilled with compacted clayey soil to exclude surface water. The trench for the non-perforated outlet pipe should be completely backfilled with compacted soil.

Water will accumulate in depressed crawl spaces. Depressed crawl spaces should be graded to create a smooth sloping surface, and covered with an approved pre-fabricated drainage material such as Mirafi Miradrain 6000. A 4-inch diameter, perforated Schedule 40 or SDR 35 pipe should be provided in a trench excavated extending across the lowest portion of the crawl space. The trench should extend 12 inches deep, and should be sloped to drain at least 1 percent by gravity. The trench should be completely lined with Mirafi 140N filter fabric, or equivalent. The perforated pipe should slope to drain at least 1 percent to a non-perforated Schedule 40 or SDR 35 pipe which discharges at an approved outlet. The surface and trench should then be covered with reinforced gunite.

Supplemental Services

Our conclusions and recommendations are contingent upon Herzog Geotechnical being retained to review the project plans and specifications to evaluate if they are consistent with our recommendations, and our being retained to provide intermittent observation and appropriate field and laboratory testing during site grading, overexcavation, keyway excavation, buttress subdrain installation, geotextile and geogrid installation, rip-rap placement, fill and backfill placement and compaction, pier drilling, tieback and soil nail drilling and load testing, footing excavation, void form installation, retaining wall backdrain installation, wall backfilling, utility trench backfilling, and subdrainage installation to evaluate if subsurface conditions are as anticipated and to check for conformance with our geotechnical recommendations. If concrete is not placed immediately following pier drilling, we should be contacted to re-inspect pier holes immediately prior to concrete placement. We should also be notified to observe the completed project. Steel, concrete, slab moisture barriers, shoring, surface drainage, and/or waterproofing should be inspected by the appropriate party, and are not part of our scope of work.

If during construction subsurface conditions different from those described in this report are observed, or appear to be present beneath excavations, we should be advised at once so that these conditions may be reviewed and our recommendations reconsidered. The recommendations made in this report are contingent upon our being notified to review changed conditions.

If more than 18 months have elapsed between the submission of this report and the start of work at the site, or if conditions have changed because of natural causes or construction operations at or adjacent to the site, the recommendations of this report may no longer be valid or appropriate. In such case, we recommend that we review this report to determine the applicability of the conclusions and recommendations considering the time elapsed or changed conditions. The recommendations made in this report are contingent upon such a review.

We should be notified at least 48 hours before the beginning of each phase of work requiring our observation, and upon resumption after interruptions. These services are performed on an as-requested basis and are in addition to this geotechnical reconnaissance. We cannot provide comment on conditions, situations or stages of construction that we are not notified to observe.

LIMITATIONS

This report has been prepared for the exclusive use of Vikrum and Gagan Nijjar and their consultants for the proposed project described in this report. Our services consist of professional opinions and conclusions developed in accordance with generally-accepted geotechnical engineering principles and practices. We provide no other warranty, either expressed or implied. Our conclusions and recommendations are based on the information provided us regarding the proposed construction, the results of our field exploration and laboratory testing programs, and professional judgment. Verification of our conclusions and recommendations is subject to our review of the project plans and specifications, and our observation of construction.

The test boring logs represents subsurface conditions at the locations and on the date indicated. It is not warranted that they are representative of such conditions elsewhere or at other times. Site conditions and cultural features described in the text of this report are those existing at the time of our field exploration, and may not necessarily be the same or comparable at other times. The locations of the test borings were established in the field by reference to existing features, and should be considered approximate only.

There is an inherent risk of instability associated with all hillside construction. We therefore recommend that the owner obtains appropriate landslide and earthquake insurance. Unrepaired landslides at the site will continue to be subject to instability.

Our scope of services did not include an environmental assessment or an investigation of the presence or absence of hazardous, toxic or corrosive materials in the soil, surface water, ground water or air, on or below, or around the site, nor did it include an evaluation or investigation of the presence or absence of wetlands. Our work also did not include an evaluation of any potential mold hazard at the site.

We appreciate the opportunity to be of service to you. If you have any questions, please call us at (415) 388-8355.

Sincerely,
HERZOG GEOTECHNICAL

REGISTERED PROFESSIONAL ENGINEER
CRAIG W. HERZOG
No. 002383
GEOTECHNICAL
STATE OF CALIFORNIA

Craig Herzog, G.E.
Principal Engineer

Attachments: References
Plate 1 - 17

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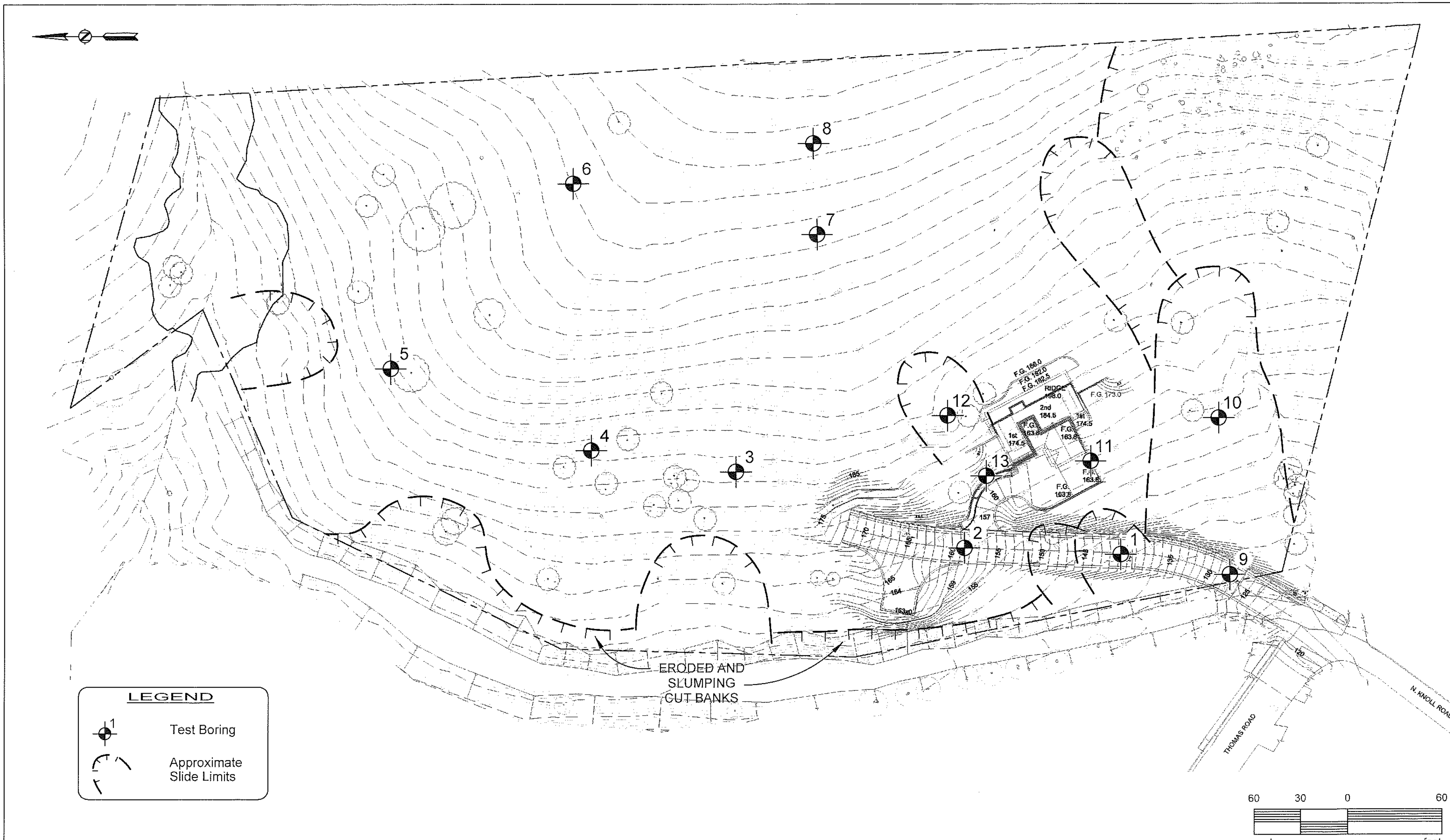

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HERZOG GEOTECHNICAL CONSULTING ENGINEERS	Job. No: 3623-01-19 Appr: Drwn: LPDD Date: SEP 2019	SITE PLAN Assessor's Parcel #034-012-026 Mill Valley, California	PLATE 1

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	Sample	DEPTH (FEET)	EQUIPMENT: **4" Flight Auger** LOGGED BY: **C.H.** ELEVATION: ** START DATE: **1-11-18** FINISH DATE: **1-11-18**
							0	BROWN SANDY CLAY (CL), medium stiff, wet, with roots (Slide Debris)
							1	
LL=44, PI=25; see Plate 17		19.5	101		8			
							2	
								ORANGE-GRAY-BROWN CLAYEY GRAVEL (GC), medium dense, moist (Slide Debris)
							3	
							4	
					25			
							5	
								GRAY SHEARED SHALE, firm, friable, highly weathered
							6	BOTTOM OF BORING 1 @ 6 FEET No Free Water Encountered

* Converted to equivalent standard penetration blow counts.

** Existing ground surface at time of investigation.

HERZOG GEOTECHNICAL CONSULTING ENGINEERS

Job No: 3623-01-18

Appr:

Drwn: LPDD

Date: SEP 2019

LOG OF BORING 1

Assessor's Parcel #034-012-026

Mill Valley, California

PLATE

2

EQUIPMENT: **4" Flight Auger**

ELEVATION: **

LOGGED BY: **C.H.**

START DATE: **1-11-18**

FINISH DATE: **1-11-18**

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	Sample	DEPTH (FEET)	Description
							0	DARK BROWN SANDY SILT (ML), soft, moist, with roots
							1	ORANGE-BROWN SANDY CLAY (CL), soft, moist to wet, with decomposed sandstone fragments
							2	
		19.0	106		7			
							3	
							4	
							5	ORANGE-GRAY-BROWN CLAYEY GRAVEL (GC), loose, moist
							6	
					11			
							7	
					40			
							8	GRAY SHEARED SHALE, moderately hard, weak, highly weathered

BOTTOM OF BORING 2 @ 8.5 FEET
No Free Water Encountered

* Converted to equivalent standard penetration blow counts.

** Existing ground surface at time of investigation.

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Job No: 3623-01-18

Appr:

Drwn: LPDD

Date: SEP 2019

LOG OF BORING 2

Assessor's Parcel #034-012-026

Mill Valley, California

PLATE

3

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	Sample	DEPTH (FEET)	EQUIPMENT: **4" Flight Auger** LOGGED BY: **C.H.** ELEVATION: ** START DATE: **1-11-18** FINISH DATE: **1-11-18**
							0	BROWN SANDY SILT (ML), soft, wet, with roots
							1	BROWN SANDY CLAY (CL), soft to medium stiff, moist
							2	
							3	
							4	LIGHT GRAY SHALE, firm, friable, highly weathered
							5	BOTTOM OF BORING 3 @ 5 FEET No Free Water Encountered

* Converted to equivalent standard penetration blow counts.

** Existing ground surface at time of investigation.

HERZOG GEOTECHNICAL CONSULTING ENGINEERS

Job No: 3623-01-18

Appr:

Drwn: LPDD

Date: SEP 2019

LOG OF BORING 3

Assessor's Parcel #034-012-026

Mill Valley, California

PLATE 4

EQUIPMENT: **4" Flight Auger**

LOGGED BY: **C.H.**

ELEVATION: **

START DATE: **1-11-18**

FINISH DATE: **1-11-18**

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	Sample	DEPTH (FEET)	Description
							0	DARK BROWN SANDY SILT (ML), soft, wet, with roots
								BROWN SILTY SAND (SM), medium dense, moist
							1	
								ORANGE-BROWN SANDSTONE, moderately hard, weak, highly weathered
					40/6"		2	

BOTTOM OF BORING 4 @ 2.5 FEET
No Free Water Encountered

* Converted to equivalent standard penetration blow counts.

** Existing ground surface at time of investigation.

HERZOG GEOTECHNICAL CONSULTING ENGINEERS	Job No: 3623-01-18 Appr: Drwn: LPDD Date: SEP 2019	**LOG OF BORING 4** Assessor's Parcel #034-012-026 Mill Valley, California
		PLATE 5

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	Sample	DEPTH (FEET)	EQUIPMENT: **4" Flight Auger** LOGGED BY: **C.H.** ELEVATION: ** START DATE: **1-11-18** FINISH DATE: **1-11-18**
							0	BROWN SILTY SAND (SM), loose, wet, with roots
								BROWN CLAYEY SAND (SC), loose, moist (Residual Soil)
						X	1	
							2	
								ORANGE-GRAY-BROWN SANDSTONE, firm, friable, highly weathered
						X	3	
								BOTTOM OF BORING 5 @ 3 FEET No Free Water Encountered

* Converted to equivalent standard penetration blow counts.

** Existing ground surface at time of investigation.

HERZOG GEOTECHNICAL CONSULTING ENGINEERS

Job No: 3623-01-18

Appr:

Drwn: LPDD

Date: SEP 2019

LOG OF BORING 5

Assessor's Parcel #034-012-026

Mill Valley, California

PLATE 6

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	Sample	DEPTH (FEET)	EQUIPMENT: **4" Flight Auger** LOGGED BY: **C.H.** ELEVATION: ** START DATE: **1-11-18** FINISH DATE: **1-11-18**
							0	DARK BROWN SILTY SAND (SM), loose, wet, with roots
								BROWN CLAYEY SAND (SC), loose to medium dense, moist
							1	
						X	2	
								ORANGE-GRAY-BROWN CLAYEY SAND (SC), medium dense, moist (Residual Soil)
						X	3	
							4	ORANGE-BROWN SANDSTONE, moderately hard, weak, highly weathered
						X	5	BOTTOM OF BORING 6 @ 5 FEET No Free Water Encountered

* Converted to equivalent standard penetration blow counts.

** Existing ground surface at time of investigation.

HERZOG GEOTECHNICAL CONSULTING ENGINEERS

Job No: 3623-01-18
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Date: SEP 2019

LOG OF BORING 6

Assessor's Parcel #034-012-026

Mill Valley, California

PLATE 7

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	Sample	DEPTH (FEET)	EQUIPMENT: **4" Flight Auger** LOGGED BY: **C.H.**	ELEVATION: ** START DATE: **1-11-18** FINISH DATE: **1-11-18**
							0	BROWN SILTY SAND (SM), loose, wet, with roots	
						X			
							1	ORANGE-BROWN SANDSTONE, moderately hard, weak, highly weathered	
						X			
							2		
						X		becomes moderately hard at 2-1/2'	
							3	BOTTOM OF BORING 7 @ 3 FEET No Free Water Encountered	

* Converted to equivalent standard penetration blow counts.

** Existing ground surface at time of investigation.

HERZOG GEOTECHNICAL CONSULTING ENGINEERS

Job No: 3623-01-18

Appr: [signature]

Drwn: LPDD

Date: SEP 2019

LOG OF BORING 7

Assessor's Parcel #034-012-026

Mill Valley, California

PLATE 8

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	Sample	DEPTH (FEET)	EQUIPMENT: **4" Flight Auger** LOGGED BY: **C.H.**	ELEVATION: ** START DATE: **1-11-18** FINISH DATE: **1-11-18**
							0	BROWN SILTY SAND (SM), loose, wet, with roots	
								ORANGE-GRAY-BROWN CLAYEY SAND (SC), loose to medium dense, moist (Residual Soil)	
						X	1		
							2	ORANGE-BROWN SANDSTONE, firm, friable to weak, highly weathered	
						X	3	BOTTOM OF BORING 8 @ 3 FEET No Free Water Encountered	

* Converted to equivalent standard penetration blow counts.

** Existing ground surface at time of investigation.

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Date: SEP 2019

LOG OF BORING 8

Assessor's Parcel #034-012-026

Mill Valley, California

PLATE 9

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	Sample	DEPTH (FEET)	EQUIPMENT: **4" Flight Auger** LOGGED BY: **C.H.** ELEVATION: ** START DATE: **1-11-18** FINISH DATE: **1-11-18**
							0	BROWN GRAVELLY CLAY (CL), soft to medium stiff, wet, with angular chert gravels and cobbles (Slide Debris)
							1	
							2	
					19		3	
					49		4	
								GRAY-BROWN SANDSTONE, hard, moderately strong, highly weathered
							5	BOTTOM OF BORING 9 @ 5 FEET No Free Water Encountered

* Converted to equivalent standard penetration blow counts.

** Existing ground surface at time of investigation.

HERZOG GEOTECHNICAL CONSULTING ENGINEERS

Job No: 3623-01-18

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Drwn: LPDD

Date: SEP 2019

LOG OF BORING 9

Assessor's Parcel #034-012-026

Mill Valley, California

PLATE 10

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	Sample	DEPTH (FEET)	EQUIPMENT: 4" Flight Auger LOGGED BY: C.H. ELEVATION: ** START DATE: 1-11-18 FINISH DATE: 1-11-18
							0	BROWN SANDY CLAY (CL), soft, wet, with roots (Slide Debris)
							1	GRAY-BROWN GRAVELLY CLAY (CL), soft, moist (Slide Debris)
							2	
							3	
							4	GRAY-BROWN SANDSTONE, moderately hard, moderately strong, highly weathered
							5	BOTTOM OF BORING 10 @ 5 FEET No Free Water Encountered

* Converted to equivalent standard penetration blow counts.

** Existing ground surface at time of investigation.

HERZOG
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Job No: 3623-01-18

Appr:

Drwn: LPDD

Date: SEP 2019

LOG OF BORING 10

Assessor's Parcel #034-012-026

Mill Valley, California

PLATE

11

EQUIPMENT: **4" Flight Auger**

LOGGED BY: **C.H.**

ELEVATION: **

START DATE: **8-1-19**

FINISH DATE: **8-1-19**

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	DEPTH (FEET)	Description
						0	GRAY-BROWN SANDY CLAY WITH GRAVEL (CL), soft, dry, porous, with roots
						1.5	OLIVE-GRAY SANDY CLAY (CH), soft, moist
LL=58, PI=40, see Plate 17		8.3	116		6	2	
		16.8	115		11	4.5	
						5	DARK GRAY SANDY CLAY (CL/CH), medium stiff, moist, sheared texture (Residual Soil)
					16	6.5	
					44	7.5	GRAY SHEARED SHALE, firm to moderately hard, weak, highly weathered
						8.5	BOTTOM OF BORING 11 @ 8.5 FEET No Free Water Encountered

* Converted to equivalent standard penetration blow counts.

** Existing ground surface at time of investigation.

HERZOG GEOTECHNICAL CONSULTING ENGINEERS

Job No: 3623-01-19

Appr: [signature]

Drwn: LPDD

Date: SEP 2019

LOG OF BORING 11

Assessor's Parcel #034-012-026

Mill Valley, California

PLATE 12

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	Sample	DEPTH (FEET)	EQUIPMENT: **4" Flight Auger** LOGGED BY: **C.H.**	ELEVATION: ** START DATE: **8-1-19** FINISH DATE: **8-1-19**
							0	GRAY-BROWN SILTY SAND WITH GRAVEL (SM), medium dense, dry, with roots	
						X	1		
							2		
						X	3		
							4		
						X		OLIVE-BROWN SANDY CLAY (CL), medium stiff, dry (Residual Soil)	
							5		
						X		OLIVE-GRAY SHEARED SERPENTINITE, firm, friable, highly weathered	
							6	BOTTOM OF BORING 12 @ 6 FEET No Free Water Encountered	

* Converted to equivalent standard penetration blow counts.

** Existing ground surface at time of investigation.

HERZOG GEOTECHNICAL CONSULTING ENGINEERS

Job No: 3623-01-19
Appr: CH
Drwn: LPDD
Date: SEP 2019

LOG OF BORING 12

Assessor's Parcel #034-012-026

Mill Valley, California

PLATE 13

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	Sample	DEPTH (FEET)	EQUIPMENT: **4" Flight Auger** LOGGED BY: **C.H.** ELEVATION: ** START DATE: **8-1-19** FINISH DATE: **8-1-19**
							0	GRAY-BROWN SILTY SAND (SM), loose, dry, with roots
							1	
						X	2	
						X	3	
							4	GRAY SANDY CLAY (CH), medium stiff, dry, sheared texture (Residual Soil)
						X	5	GRAY SHEARED SHALE, firm to moderately hard, friable, highly weathered
						X	6	BOTTOM OF BORING 13 @ 6 FEET No Free Water Encountered

* Converted to equivalent standard penetration blow counts.

** Existing ground surface at time of investigation.

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Job No: 3623-01-19
Appr: [signature]
Drwn: LPDD
Date: SEP 2019

LOG OF BORING 13

Assessor's Parcel #034-012-026

Mill Valley, California

PLATE

14

MAJOR DIVISIONS				TYPICAL NAMES
COARSE GRAINED SOILS More than Half > #200 sieve	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL GRADED GRAVELS, GRAVEL-SAND
			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	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, POOORLY GRADED SAND-SILT MIXTURES
			SC	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES
FINE GRAINED SOILS More than Half < #200 sieve	SILTS AND CLAYS LIQUID LIMIT LESS THAN 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 CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACIOUS FINE SANDY OR SILTY SOILS, ELASTIC 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

Shear Strength, psf
Confining Pressure, psf

Consol	Consolidation	Tx	2630 (240)	Unconsolidated Undrained Triaxial
LL	Liquid Limit (in %)	Tx sat	2100 (575)	Unconsolidated Undrained Triaxial, saturated prior to test
PL	Plastic Limit (in %)	DS	3740 (960)	Unconsolidated Undrained Direct Shear
PI	Plasticity Index	TV	1320	Torvane Shear
Gs	Specific Gravity	UC	4200	Unconfined Compression
SA	Sieve Analysis	LVS	500	Laboratory Vane Shear
■	Undisturbed Sample (2.5-inch ID)	FS	Free Swell	
◪	2-inch-ID Sample	EI	Expansion Index	
◩	Standard Penetration Test	Perm	Permeability	
⊠	Bulk Sample	SE	Sand Equivalent	

KEY TO TEST DATA

HERZOG GEOTECHNICAL CONSULTING ENGINEERS

Job No: 3623-01-19
Appr:
Drwn: LPDD
Date: SEP 2019

SOIL CLASSIFICATION CHART AND KEY TO TEST DATA
Assessor's Parcel #034-012-026
Mill Valley, California

PLATE 15

ROCK SYMBOLS

SHALE OR CLAYSTONE	CHERT	SERPENTINITE
SILTSTONE	PYROCLASTIC	METAMORPHIC ROCKS
SANDSTONE	VOLCANIC	DIATOMITE
CONGLOMERATE	PLUTONIC	SHEARED ROCKS

LAYERING

MASSIVE	Greater than 6 feet
THICKLY BEDDED	2 to 6 feet
MEDIUM BEDDED	8 to 24 inches
THINNLY BEDDED	2-1/2 to 8 inches
VERY THINNLY BEDDED	3/4 to 2-1/2 inches
CLOSELY LAMINATED	1/4 to 3/4 inches
VERY CLOSELY LAMINATED	Less than 1/4 inch

JOINT, FRACTURE, OR SHEAR SPACING

VERY WIDELY SPACED	Greater than 6 feet
WIDELY SPACED	2 to 6 feet
MODERATELY SPACED	8 to 24 inches
CLOSELY SPACED	2-1/2 to 8 inches
VERY CLOSELY SPACED	3/4 to 2-1/2 inches
EXTREMELY CLOSELY SPACED	Less than 3/4 inch

HARDNESS

SOFT - Pliable; can be dug by hand

FIRM - Can be gouged deeply or carved with a pocket knife

MODERATELY HARD - Can be readily scrached by a knife blade; scratch leaves heavy trace of dust and is readily visable after the powder has been blown away

HARD - Can be scratched with difficulty; scratch produces little powder and is often faintly visable

VERY HARD - Cannot be scratched with pocket knife; leaves a metallic streak

STRENGTH

PLASTIC - Capable of being molded by hand

FRIABLE - Crumbles by rubbing with fingers

WEAK - An unfractured specimen of such material will crumble under light hammer blows

MODERATELY STRONG - Specimen will withstand a few heavy hammer blows before breaking

STRONG - Specimem will withstand a few heavy ringing hammer blows and usually yields large fragments

VERY STRONG - Rock will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments

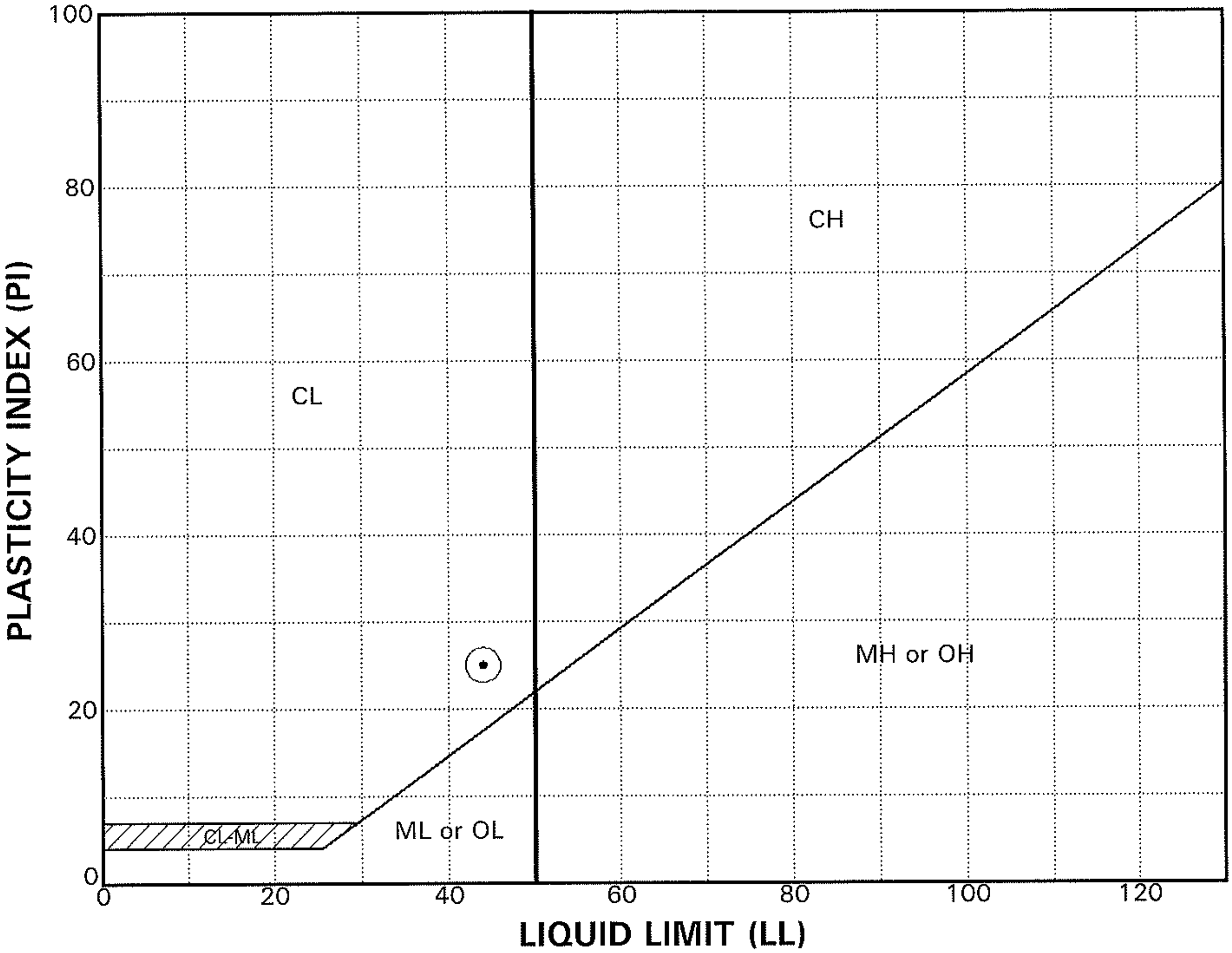
DEGREE OF WEATHERING

HIGHLY WEATHERED - Abundant fractures coated with oxides, carbonates, sulphates, mud, etc., thourough discoloration, rock disintegration, mineral decomposition

MODERATELY WEATHERED - Some fracture coating, moderate or localized discoloration, little to no effect on cementation, slight mineral decomposition

SLIGHTLY WEATHERED - A few stained fractures, slight discoloration, little or no effect on cementation, no mineral decomposition

FRESH - Unaffected by weathering agents, no appreciable change with depth



SAMPLE SOURCE	CLASSIFICATION	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTICITY INDEX (%)	% PASSING #200 SIEVE
⊙ Bor. 1 @ 1.5'	Brown Sandy Clay (CL)	44	19	25	

HERZOG
GEOTECHNICAL
CONSULTING ENGINEERS

Job No: 3623-01-18
Appr:
Drwn: LPDD
Date: SEP 2019

PLASTICITY CHART

Assessor's Parcel #034-012-026

Mill Valley, California

PLATE

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