MILLER PACIFIC
ENGINEERING GROUP

GEOTECHNICAL INVESTIGATION
GARAGE/OFFICE ADDITION
18876 STATE ROUTE 1
MARSHALL, CALIFORNIA

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Job No. 3206.001

Prepared For:
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CERTIFICATION

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1.0 INTRODUCTION

This report presents the results of our Geotechnical Investigation for the detached garage and office addition to the existing single-family residence at 18876 State Route 1 in Marshall, California. As shown on Figure 1, the project site is located on the east side of Tomales Bay, roughly a quarter mile north of the Marconi Conference Center.

Our work was performed in accordance with our Agreement for Professional Services authorized on April 25, 2021. The purpose of our investigation was to explore subsurface conditions within the proposed project area and to develop geotechnical recommendations and criteria for use in design and construction of the project. The scope of our services includes:

- Reviewing published geologic and geotechnical background information.
- Exploring subsurface conditions with two borings located near the footprint of the proposed structure.
- Laboratory testing to estimate pertinent engineering properties of the soils encountered during our subsurface exploration.
- Evaluating relevant geologic hazards including seismic shaking, fault surface rupture, slope instability and other hazards.
- Engineering analyses to develop geotechnical recommendations and design criteria related to building foundations, site grading, retaining walls, seismic design and other geotechnical-related items.
- Preparation of this Geotechnical Investigation report which summarizes the subsurface exploration and laboratory testing programs, evaluation of relevant geologic hazards, and geotechnical recommendations and design criteria.

Issuance of this report completes our initial phase of services. Subsequent phases of work should include geotechnical plan review and observation and testing of geotechnical-related work items during construction.

2.0 PROJECT DESCRIPTION

Based on our discussions with Ronald Casassa (project Architect), we understand the project will consist of constructing a new detached structure within an approximately 700-square-foot building footprint toward the north side of the property. While detailed plans are not yet available, we understand the new building will be a two-story, wood-framed structure which will serve as a garage on the bottom floor and will provide office and living space above. Site grading is expected to consist of primarily excavation into the hillside to create a level building pad. A retaining wall with heights of up to about ten feet is anticipated to support the planned excavation. Ancillary improvements may include new concrete flatwork, underground utilities, landscaping and other improvements typical of such developments. The approximate location of the proposed building is shown on the Site Plan, Figure 2.

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 project site is located along the eastern shoreline of Tomales Bay. Regional geologic mapping by the California Geological Survey (CGS, 2017) indicates the site is located near a geologic contact between Holocene-age sediments of central Tomales Bay and Jurassic- to Cretaceous-age mélange of the Franciscan Complex. While not shown on the regional mapping, we understand the western portion of the site was likely previously filled to facilitate construction of the North Pacific Coast railroad line that formerly existed along much of the eastern shoreline of Tomales Bay. The mapping further indicates the site is located roughly 800 feet east of an active trace of the San Andreas Fault which trends roughly northwest/southeast. A Regional Geologic Map and descriptions of the mapped geologic units are shown on Figure 3.

3.2 Seismicity

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

Faults are seldom single cracks in the earth's crust but are typically comprised of localized shear zones which link together to form larger fault zones. Within the Bay Area, faults are concentrated along the San Andreas Fault zone. The movement between rock formations along either side of a fault may be horizontal, vertical, or a combination and is radiated outward in the form of energy waves. The amplitude and frequency of earthquake ground motions partially depends on the material through which it is moving. The earthquake force is transmitted through hard rock in short, rapid vibrations, while this energy becomes a long, high-amplitude motion when moving through soft ground materials, such as Bay Mud.

3.2.1 Regional Active Faults

The California Geological Survey (previously known as the California Division of Mines and Geology), defines a “Holocene-active fault” as one that has had surface displacement within Holocene time (the last 11,700 years). CGS has mapped various faults in the region as part of their Fault Activity Map of California (CGS, 2010). Many of these faults are shown in relation to the project site on the attached Active Fault Map, Figure 4. The nearest known Holocene-active fault is the San Andreas Fault which is located roughly 800 feet west of the site[1]. Other nearby Holocene-active faults include the San Gregorio and Rodgers Creek Faults which are located approximately 30.8 kilometers (19.2 miles) to the southeast and 32.1 kilometers (20.0 miles) to the northeast, respectively[2].

3.2.2 Historic Fault Activity

Numerous earthquakes have occurred in the region within historic times. The results of our USGS earthquake search catalogue indicates that at least eight earthquakes with a Richter Magnitude of 5.0 or larger have occurred within 100 kilometers (62 miles) of the site between 1900 and 2021. The approximate locations of earthquakes which occurred between 1985 and 2014 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

[1] Distance to San Andreas Fault zone estimated based on the California Division of Mines and Geology Special Studies Zone for the Tomales Quadrangle (CDMG, 1974).

[2] Distances to other faults estimated using USGS Fault Overlays on Google Earth, accessed June 4, 2021.

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 Rodgers Creek Fault is located approximately 30.8 kilometers (19.2 miles) northeast of the site and is assigned a probability of 33 percent. The San Andreas Fault, located approximately 800 feet west 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.

3.3 Surface Conditions

The project site encompasses an approximately half-acre, irregularly-shaped parcel (APN 106-050-14) that is situated near the base of a westerly-facing hillside along the eastern shoreline of Tomales Bay. The site has been developed with a two-story single-family residence and separate single-story studio along the central and southern portions of the property, respectively. The proposed building will occupy an approximately 24-foot by 32-foot building footprint just north of the existing residence. Near the new building, the ground surface is inclined at about 2:1 or flatter and previous site grading appears to have included fill placement for the former railroad line and subsequent site improvements.

We understand a few feet of fill was recently removed from the proposed building footprint and the ground surface is partially denuded from the recent grading. The surface surrounding the building area is covered with decorative stone surfacing, natural grasses, shrubbery and a few mature trees. Surface elevations range from about 50 feet within the upper/eastern portions of the property to about 12 feet along the western portion adjacent to Highway 1[3]. The existing surface conditions are shown on Figure 2.

3.4 Field Exploration and Laboratory Testing

We explored subsurface conditions near the proposed improvements on May 27, 2020 with two borings at the approximate locations shown on Figure 2. The borings were excavated using track-mounted drilling equipment to approximate depths of 14.5 to 20 feet below ground surface. The borings were logged by our Engineer and samples were obtained for classification and laboratory testing. We prepared boring logs based on soil descriptions in the field, as well as visual examination and testing of the soil and rock samples in our laboratory. The boring logs are presented in Appendix A.

[3] Surface elevations referenced herein are based on elevation contours presented on MarinMap GIS which are based on NAVD 88.

Laboratory testing of soil samples from the exploratory borings included determination of moisture content, dry density, unconfined compressive strength and expansion index. The results of our laboratory tests are presented on the boring logs with the exception of the expansion index testing which is shown on Figure A-5 and our laboratory testing program is discussed in greater detail in Appendix A.

3.5 Subsurface Conditions and Groundwater

Based on our subsurface exploration, the site is generally underlain by fill over mélange bedrock of the Franciscan Complex. The mélange bedrock was encountered at the ground surface in Boring B-1 which is near the east side of the proposed building footprint, and at about 14 feet in Boring B-2 which was excavated near the west end of the building footprint. The fill encountered in Boring B-2 consists of about four feet of medium dense to dense clayey sand over roughly ten feet of medium stiff to stiff clayey soils of high plasticity. The underlying mélange is generally characterized as highly to completely weathered, weak and exhibits low hardness.

Groundwater was not encountered in our borings which were excavated in late spring following a relatively dry winter. Because the borings were not left open for an extended period of time, a stabilized depth to groundwater may not have been observed. Groundwater elevations fluctuate seasonally and higher groundwater levels may be present during high tide or following periods of heavy rainfall. Perched water tables may also exist within the soil and bedrock materials.

4.0 GEOLOGIC HAZARDS

This section summarizes our review of commonly considered geologic hazards and discusses their potential impacts. The primary geologic hazards which could affect the existing structures and new building include strong seismic ground shaking and fault surface rupture. Other geologic hazards are judged less than significant with regard to the existing structure. Geologic hazards, potential impacts and mitigation measures are discussed in further detail in the following sections.

4.1 Fault Surface Rupture

The provisions of the Alquist-Priolo Earthquake Fault Zoning Act (“A-P Act”, 1972) and the current edition of the California Building Code (2016) dictate that structures intended for human occupancy (i.e. more than 2,000 hours per year) may not be developed within 50 feet of an active fault trace unless a smaller setback can be justified by appropriate geologic evidence. For the purpose of the A-P Act, an “active” fault is defined as one that has ruptured within Holocene time (the last 11,700 years).

In addition to restricting development near active faults, the A-P Act requires the California Geological Survey to publish maps delineating Earthquake Fault Zones. Based on review of the applicable Alquist-Priolo map for the Tomales and Point Reyes NE Quadrangles (CDMG, 1974), the project site lies approximately 800 feet east of the Earthquake Fault Zone associated with an active trace of the San Andreas Fault, as shown on Figure 6. Given the proximity of the site to the San Andreas Fault, we judge there is a low to moderate risk of fault surface rupture impacting the addition and existing residence during a future seismic event.

Evaluation: *Potentially significant.*

Recommendation: *Design new structure in accordance with the provisions of the 2019 California Building Code or subsequent codes in effect when final design occurs. No specific mitigation measures for fault surface rupture are required since the project site is not mapped within an Alquist-Priolo Special Studies Zone.*

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. For residential developments, deterministic methods are typically used.

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 (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)(1)	Median Peak Ground Acceleration (g)(2)	Median PGA +1 Std Dev (g)(2)
San Andreas	8.0	0.25(3)	0.55	0.99
San Gregorio	7.4	30.8	0.13	0.23
Rodgers Creek	7.3	32.1	0.12	0.21
Hayward	7.3	44.3	0.09	0.17
Maacama	7.4	46.9	0.09	0.16

(1) Estimated using USGS Fault Overlays on Google Earth.
(2) Estimated using Abrahamson & Silva, Boore & Atkinson, Campbell & Bozorgnia, and Chiou & Youngs 2014 NGA models using V_{s30} = 760 m/s.
(3) Estimated based upon AP Map for Tomales and Point Reyes NE Quadrangle

The calculated bedrock accelerations should only be considered as reasonable estimates. Many factors (soil conditions, orientation to the fault, etc.) can influence the actual ground surface accelerations. Ground shaking can result in structural failure and collapse of structures or cause non-structural building elements (such as light fixtures, shelves, cornices, etc.) to fall, presenting a hazard to building occupants and contents. Compliance with provisions of the most recent version of the California Building Code (2019 CBC) should result in structures that do not collapse in an earthquake. Damage may still occur and hazards associated with falling objects or non-structural building elements will remain.

The potential for strong seismic shaking at the project site is high. Due to their proximity and historic rates of activity, the San Andreas Fault presents 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.*
Recommendation: *Design new structures in accordance with the provisions of the 2019 California Building Code or subsequent codes in effect when final design occurs. Recommended seismic design coefficients and spectral accelerations are presented in Section 5.1 of this report.*

4.3 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.

The results of our subsurface exploration indicate the project site is underlain by predominantly clayey soils over shallow Franciscan bedrock. While a few feet of granular soils were encountered in Boring B-2, the soils are relatively dense, contain relatively high percentages of clay and are located above groundwater. 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 Seismic Densification

Seismic ground shaking can induce settlement of 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. Loose granular soils were not encountered in our borings. Therefore, we judge the likelihood of seismically induced settlement is low.

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

4.5 Expansive Soil

Expansive soils will shrink and swell with fluctuations in moisture content and are capable of exerting significant expansion pressures on building foundations, interior floor slabs and exterior flatwork. Distress from expansive soil movement can include cracking of brittle wall coverings (stucco, plaster, drywall, etc.), racked door and/or window frames, uneven floors, and cracked slabs. Flatwork, pavements, and concrete slabs-on-grade are particularly vulnerable to distress due to their low bearing pressures. Expansive soils also cause soil creep on sloping ground.

The near-surface materials encountered in our borings consist of sandy soils and mélange bedrock which generally exhibit low expansion potential. However, the planned excavation for the building pad may also result in the medium stiff to stiff clayey soils being located at or near the ground surface. The results of our laboratory testing indicate the clayey soils exhibit an expansion index of 90 which correlates to a "medium to high" expansive potential.

Evaluation: *Less than significant with mitigation.*

Recommendation: *Expansive soils exposed during site grading should be removed to a depth of 36 inches below the finished building pad grade and replaced with non-expansive fill materials which conform to the recommendations outlined in Section 5.2.3. Soils should be moisture conditioned to slightly above the optimum moisture content during site grading and maintained at this moisture content until imported aggregate base and/or surface flatwork is completed. Alternatively, the grade beams and slab floor could be designed to resist uplift pressures associated with potentially expansive soils. Void boxes could also be considered beneath grade beams to reduce potential uplift pressures.*

4.6 Settlement

Significant settlement can occur when new loads are placed over soft, compressible clays (e.g. Bay Mud) or loose soils. While loose sands and soft clays were not encountered, our borings indicate the depth to bedrock varies significantly over short horizontal distances throughout the building footprint. New foundation loads supported on soils of variable thickness can result in differential settlements across the footprint of the new structure. Considering these variable subsurface conditions, we judge the risk of damage due to settlement is low to moderate.

Evaluation: *Less than significant with mitigation.*

Recommendation: *The new structure should be supported on a foundation system that uniformly bears on firm bedrock, as discussed in Section 5.3.*

4.7 Slope Instability/Landslides

Slope instability generally occurs on relatively steep slopes and/or on slopes underlain by weak materials. The project site is located on sloping terrain inclined at about 2:1 or flatter and is underlain by fill and mélange bedrock. Previous geologic mapping does not indicate the presence of landslides or slope instability within the project area, and we did not observe scarps, cracking, or other evidence that would suggest active or recent slope movement at the site. Based on the results of our site reconnaissance and subsurface exploration, the risk of damage to the planned improvements due to slope instability is generally low provided that site grading, retaining walls and other improvements are constructed in accordance with the recommendations outlined in subsequent sections of this report.

Evaluation: *Less than significant with mitigation.*
Recommendation: *Mitigation measures should include minimizing the thickness of new fills and completing site grading in accordance with the recommendations outlined in Section 5.2.*

4.8 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. Construction of the proposed improvements will require grading and changes to existing surface drainage patterns which, if not properly addressed during design and construction, could lead to concentrated surface water flows and increased erosion. Considering the sloping terrain that surrounds the project site, and the disturbance to existing vegetation and drainage patterns that may result from site grading, we judge the risk of damage to improvements due to erosion is moderate.

Evaluation: *Less than significant with mitigation.*
Recommendation: *Mitigation measures include designing a site drainage system to collect surface water 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 should be developed prior to construction per the current Marin County guidelines.*

4.9 Flooding

The project site is located at elevations ranging from about 12 to 50 feet above sea level and is not mapped within a FEMA-designated special flood hazard area (Federal Emergency Management Agency, 2016). Therefore, large scale flooding is not considered a significant hazard at the project site.

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

4.10 Tsunami/Seiche

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 site is mapped just east of a tsunami inundation zone, as shown on Figure 7

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

4.11 Sea Level Rise

The State of California Sea-Level Rise Guidance, 2018 Update provides predictions for sea-level rise at various representative tidal gauge stations throughout California (California Natural Resources Agency, 2018). Based upon the probabilistic predictions for a "low risk aversion" scenario at the Point Reyes tidal gauge, the sea level is expected to rise between 1.6 and 1.9 feet by the year 2070.

The nearest National Oceanic and Atmospheric Administration tidal gage station in Point Reyes indicates a mean highwater elevation of 5.1 feet and a finished floor elevation of about 15 feet or higher is anticipated for the new structure. Based upon the sea level rise predictions, the mean highwater elevation under still water conditions is estimated to be 6.7 to 7.0 feet by the year 2070 which is well below the anticipated finished floor elevation for the new structure. Therefore, the likelihood of sea level rise impacting the new structure is generally considered low.

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

5.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our subsurface exploration, we judge that construction of the proposed structure and associated improvements is feasible from a geotechnical standpoint. Primary geotechnical considerations for the project will include ensuring that site grading is completed in a manner that is appropriate for the hillside setting, providing uniform foundation support for the new structure, designing the structure to resist strong seismic ground shaking and potentially expansive soils. Additional discussion and recommendations addressing these and other considerations are presented in the following sections.

5.1 Seismic Design

Minimum mitigation of ground shaking includes seismic design of new structures in conformance with the provisions of the most recent edition (2019) of the California Building Code. The magnitude and character of these ground motions will depend on the particular earthquake and the site response characteristics. Based on the interpreted subsurface conditions and close proximity of several nearby faults, we recommend the CBC coefficients and site values shown in Table 2 be used to calculate the design base shear of the new construction.

Table 2 – 2019 California Building Code Seismic Design Criteria

Parameter	Design Value
Site Class	B
Site Latitude	38.1480°N
Site Longitude	-122.8829°W
Spectral Response (short), S_S	2.42 g
Spectral Response (1-sec), S_1	1.01 g
Site Coefficient, F_a	1.0
Site Coefficient, F_v	1.0
Spectral Response (Short), S_{MS}	2.42 g
Spectral Response (1 sec), S_{M1}	1.01 g
Design Spectral Response (short), S_{DS}	1.61 g
Design Spectral Response (1 sec), S_{D1}	0.68 g
MCE_G PGA Adjusted, PGA_M	1.03 g
Seismic Design Category	E

Reference: ATC Hazard by Location, accessed on June 4, 2021.

5.2 Site Grading

Site grading is expected to include cuts of up to about ten-feet-high and minor fill placement as required to create a level building pad for the new building. Site grading should be performed in accordance with the recommendations and criteria outlined in the following sections.

5.2.1 Site Preparation

Clear pavements, old foundations, over-sized debris, and organic material from areas to be graded. Debris, rocks larger than six inches, and vegetation are not suitable for structural fill and should be removed from the site. Trees that are located within the building areas should be removed and the root systems excavated. Existing foundations and utilities which are to be abandoned as part of the work should be removed from structural areas. In non-structural areas, utilities could be abandoned in place in many cases provided cement grout completely fills any void in the utility.

Potentially expansive soils exposed during grading for the building pad should be removed to a depth of 36 inches below the finished building pad grade and replaced with non-expansive fill materials which conform to the recommendations outlined in Section 5.2.3. Where fills or other structural improvements are planned on level ground, the subgrade surface should be scarified to a depth of eight inches, moisture conditioned to slightly 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 three feet beyond the planned building envelope 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 foundations, retaining walls, utilities, and other improvements will generally encounter medium dense clayey sand and medium stiff to stiff clayey soils over weathered Franciscan bedrock. Temporary (steeper) cut slopes may be required during construction. For planning purposes, cut slopes into weathered rock may be designed for an OSHA Type “B” soil whereas cut slopes into soil should be designed for an OSHA Type “C” soil. Permanent cut slopes should be inclined no steeper than 2:1.

Based on our subsurface exploration, we judge the majority of site excavation can be performed with conventional equipment, such as medium-size dozers and excavators. However, Franciscan bedrock often contains inclusions and zones of harder, more resistant rock which may require specialized techniques or equipment to excavate (e.g., jackhammers or hydraulic breakers). 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 potential for hard rock excavation and resulting costs.

5.2.3 Fill Materials, Placement and Compaction

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

Fill materials should be uniformly moisture conditioned to within two percent of 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. In pavement areas subjected to vehicle loads, the upper 12 inches of fill or natural soil should be compacted to at least 95 percent relative compaction and a firm and unyielding condition. The maximum dry density and optimum moisture content of fill materials should be determined in accordance with ASTM D1557.

5.3 Foundation Design

Due to the variable depth to bedrock and anticipated cut/fill transitions under the new building, we generally recommend that the structure be supported on a drilled pier and grade beam foundation system embedded into bedrock. However, shallow spread footings could be utilized in areas where site grading exposes firm bedrock. Shallow foundations should be designed using the criteria provided in Table 3.

Table 3 – Foundation Design Criteria

Parameter	Design Value
Minimum Embedment[1]	18 inches
Minimum Width	18 inches
Allowable Bearing Pressure, Bedrock[2, 3]	3,000 psf
Base Friction Coefficient	0.35
Lateral Passive Resistance[4]	400 pcf

Notes:
(1) Maintain minimum of seven feet of horizontal distance between the outer edge of footing and face of nearest adjacent slope.
(2) Design shallow foundations to similar bearing pressures (i.e. size footing widths to maintain relatively uniform bearing loads).
(3) Increase design values by 33 percent for total design loads including seismic.
(4) Equivalent fluid pressure, not to exceed 3,000 psf. Neglect upper 12 inches unless confined by concrete.

Drilled pier foundations should be used in areas where the new structure crosses cut/fill transitions or the thickness of the soil precludes the use of shallow foundations that bear on bedrock. The drilled piers should be designed using a nominal skin friction of 500 pounds per square foot for soil and 2,000 pounds per square foot for bedrock, a minimum diameter of 18 inches, and a minimum embedment of three feet into firm rock. Lateral resistance for drilled piers should be calculated using a passive resistance of 300 pounds per cubic foot (equivalent fluid pressure) for soil and 400 pounds per cubic foot for bedrock. The passive pressure should be applied over two pier diameters. The upper three feet of embedment should be ignored in calculating the lateral and vertical capacity of drilled piers.

5.4 Retaining Walls

We understand a new retaining wall with heights of up to about ten feet may be required to support the excavation for the new structure. The wall should be designed using the foundation design criteria presented in Section 5.3 and the lateral earth pressures shown in Table 4. Retaining walls that can slightly deflect at the top can be designed using the unrestrained criteria shown below. Walls that are structurally connected and not allowed to deflect (e.g. tied-back walls) are restrained and are commonly designed using a uniform active earth pressure distribution rather than an equivalent fluid pressure.

Table 4 – Lateral Earth Pressures for Retaining Wall Design

Backfill Inclination[1]	Unrestrained[2]	Restrained[3]
Level	45 pcf	30 x H psf
3:1	50 pcf	35 x H psf
2:1	60 pcf	40 x H psf

(1) Interpolate earth pressures for intermediate slopes.
(2) Equivalent fluid pressure.
(3) Rectangular distribution, H is wall height in feet.

In addition to the pressures noted above, we also recommend the walls be designed to resist a uniform seismic surcharge equal to 12 times the retained height (in psf). The factor of safety used in the retaining wall design should be reduced under seismic conditions as permitted by the governing code that is used for design. A minimum uniform lateral surcharge of 100 pounds per square foot should also be applied to the upper five feet to account for surcharge loads due to light vehicles, compaction, equipment or other surcharges. The wall designer should adjust the surcharge load at their discretion commensurate with the specific loading condition that is anticipated.

Soil nail and shotcrete retaining walls may be a relatively efficient retention system in areas where new cuts are planned. If used, soil nail and shotcrete retaining walls should be designed using the criteria presented in Table 5.

Table 5 - Soil Nail Wall Design Criteria

Parameter	Franciscan Bedrock
Friction Angle	32 degrees
Cohesion	500 psf
Unit Weight	135 pcf
Nominal Soil Nail Bond Strength	2,000 psf

Wall drainage is required for all retaining walls taller than three feet. Either Caltrans Class 1B permeable material within filter fabric or Caltrans Class 2 permeable material can be used for wall drainage. The drainage should be collected in a four-inch perforated PVC drain line at the base of the wall. The permeable material should extend at least 12 inches from the back of the wall and be continuous from the bottom of the wall to within 12 inches of the ground surface. Alternatively, drainage panels, such as Mirafi 100N, may be utilized. A typical wall backdrain detail is presented on Figure 8. The Architect or waterproofing consultant should also specify a waterproofing membrane on retaining walls where interior seepage or moisture vapor would be problematic.

5.5 Interior Concrete Slabs

Reinforced concrete slab floors are judged to be appropriate for the new structures provided the building pads are prepared in accordance with our recommendations. The concrete slab floors may be poured monolithically or separated with a cold joint at the Structural Engineer's discretion. We recommend that interior concrete slabs have a minimum thickness of five inches and be reinforced with steel reinforcing bars (not mesh). Slabs should be placed on a moist subgrade to reduce potential for future expansive 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-thick layer of clean, free draining, ¾-inch angular gravel should be placed and compacted 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 free draining gravel. The vapor barrier should 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 since eliminating the sand can cause cracking or "curling" of the new concrete. For slabs that are not sensitive to moisture vapor, we recommend at least four inches of Class 2 Aggregate Base (Caltrans, 2015) compacted to at least 95 percent relative compaction.

5.6 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 offsets due to seasonal movements), exterior slabs can be thickened to five inches and reinforced with steel reinforcing bars (not welded wire mesh). Driveways and slabs subject to vehicle loads should be a minimum of five-inches-thick and designed to resist traffic loading. 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.

5.7 Site and Foundation Drainage

New grading could result in adverse drainage patterns causing water to pond around the residence. 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, these surfaces should be sloped at least 0.10 feet in the first five feet (two percent).

Roof gutter downspouts may discharge onto the pavements but should not discharge onto landscaped areas immediately adjacent to the garage. 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.

5.8 Underground Utilities

Excavations for utilities will be in medium dense sandy soils, medium stiff to stiff clayey soils and weathered bedrock and may encounter groundwater at shallow depths if wintertime or early spring work is performed. Trench excavations having a depth of five feet or more must be excavated and shored in accordance with OSHA regulations per Section 5.2.2. Bedding materials for utility pipes should be poorly 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 beneath the pipe in accordance with the manufacturer’s recommendation, typically three to six inches. Trench backfill may consist of on-site soils, moisture conditioned and placed in thin lifts and compacted to at least 90 percent. Use equipment and methods that are suitable for work in confined areas without damaging utility conduits.

6.0 SUPPLEMENTAL GEOTECHNICAL SERVICES

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. During construction, we should be present intermittently to observe foundation excavations, retaining wall drainage and backfill, subgrade preparation and compaction, proper moisture conditioning of soils, fill placement and compaction 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

We believe his report has been prepared in accordance with generally accepted geotechnical engineering practices in the San Francisco Bay Area at the time the report was prepared. This report has been prepared for the exclusive use of Bill and Tracy Manheim and/or their assignees specifically for this project. No other warranty, expressed or implied, is made. Our evaluations and recommendations are based on the data obtained during our subsurface exploration program and our experience with soils in this geographic area. Our approved scope of work did not include an environmental assessment of the site. Consequently, this report does not contain information regarding the presence or absence of toxic or hazardous wastes.

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

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

8.0 LIST OF REFERENCES

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APPENDIX A
SUBSURFACE EXPLORATION AND LABORATORY TESTING

A. SUBSURFACE EXPLORATION

We explored subsurface conditions with two exploratory borings drilled with track-mounted drilling equipment on May 27, 2021, at the approximate locations shown on the Site Plan, Figure 2. The exploration was conducted under the technical supervision of our Engineer who examined and logged the soil materials encountered and obtained samples. The subsurface conditions encountered in the test borings are summarized and presented on the boring logs, Figures A-1 through A-4.

Relatively "undisturbed" samples were obtained using a three-inch diameter, split-barrel Modified California Sampler with 2.5 by six-inch tube liners or a Standard Penetration Test (SPT) Sampler. The samplers were driven by a 140-pound hammer at a 30-inch drop. The number of blows required to drive the samplers 18 inches was recorded and is reported on the boring logs as blows per foot for the last 12 inches of driving. The samples obtained were examined in the field, sealed to prevent moisture loss, and transported to our laboratory.

B. LABORATORY TESTING

We conducted laboratory tests on selected intact samples to classify soils and to estimate engineering properties. The following laboratory tests were conducted in general accordance with the ASTM standard test method cited:

- Laboratory Determination of Water (Moisture Content) of Soil, Rock, and Soil-Aggregate Mixtures, ASTM D 2216
- Density of Soil in Place by the Drive-Cylinder Method, ASTM D2937
- Unconfined Compressive Strength of Cohesive Soil, ASTM D2166
- Expansion Index Testing, ASTM D4829

The moisture content, dry density, and unconfined compression test results are shown on the exploratory boring logs while the results of our expansion index testing is shown on Figure A-5. The exploratory boring logs, description of soils encountered and the laboratory test data reflect conditions only at the location of the boring at the time they were excavated or retrieved. Conditions may differ at other locations and may change with the passage of time due to a variety of causes including natural weathering, climate and changes in surface and subsurface drainage.