GEOLOGIC AND GEOTECHNICAL INVESTIGATION
75 HORSESHOE HILL ROAD
BOLINAS, CALIFORNIA

September 16, 2022

Job No. 3281.002

Prepared For:
Scott Weiss
260 Horseshoe Hill Road
Bolinas, California 94924

CERTIFICATION

This document is an instrument of service, prepared by or under the direction of the undersigned professionals, in accordance with the current ordinary standard of care. The service specifically excludes the investigation of polychlorinated byphenols, radon, asbestos, or any other hazardous materials. The document is for the sole use of the client and consultants on this project. No other use is authorized. If the project changes, or more than two years have passed since issuance of this report, the findings and recommendations must be updated.

MILLER PACIFIC ENGINEERING GROUP
(a California corporation)

REVIEWED BY:

PROFESSIONAL GEOLOGIST ZOE M. STEPHENS No. 9860 EXP. 11/30/23 STATE OF CALIFORNIA

CERTIFIED ENGINEERING GEOLOGIST MICHAEL JEWETT CEG No. 2610 Exp.1-31-23 STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER SCOTT A. STEPHENS GE 2398 EXP. 06/30/23 GEOTECHNICAL STATE OF CALIFORNIA

Zoe Stephens
Professional Geologist No. 9860
(Expires 11/30/23)

Michael Jewett
Engineering Geologist No. 2610
(Expires 1/31/23)

Scott Stephens
Geotechnical Engineer No. 3031
(Expires 6/30/23)

MILLER PACIFIC ENGINEERING GROUP

GEOLOGIC AND GEOTECHNICAL INVESTIGATION
75 HORSESHOE HILL ROAD
BOLINAS, CALIFORNIA

TABLE OF CONTENTS

GEOLOGIC AND GEOTECHNICAL INVESTIGATION
75 HORSESHOE HILL ROAD
BOLINAS, CALIFORNIA

1.0 INTRODUCTION

This report presents the results of our Phase 1 Geologic and Geotechnical Investigation for your planned new equestrian center development at 75 Horseshoe Hill Road in Bolinas, California. A Site Location Map is shown on Figure 1.

Our work has been performed in accordance with our Agreement for Professional Services dated July 6, 2022. The purpose of our services is to evaluate site geologic conditions and provide geotechnical recommendations and criteria for use in project design and construction. The scope of our services is outlined in our proposal letter dated June 21, 2022 and includes the following:

- Review of readily-available published geologic mapping, geo-hazards mapping, and geotechnical background information from our in-house library;
- A detailed site reconnaissance for geologic mapping and documentation of existing surface conditions;
- One day of subsurface exploration with five soil borings;
- Geotechnical laboratory testing of recovered samples;
- Evaluation of relevant geologic hazards and development of conceptual mitigation measures as warranted;
- Development of geotechnical recommendations and design criteria for the project; and
- Preparation of this report.

Issuance of this report completes the scope of our Phase 1 services. Future phases of work are anticipated to include Geotechnical Consultation and Plan Review (Phase 2) and Geotechnical Observation/Testing during construction (Phase 3).

2.0 PROJECT DESCRIPTION

Based on our discussion with Ms. Lamotte, we understand the project generally includes construction of a new equestrian facility on an approximately 6-acre parcel. Although detailed plans have not yet been developed, preliminary drawings indicate that project components will include a new 18,000 square-foot covered riding arena, an approximately 2,000 square foot barn, and a new mound-type septic system. The new arena will be sited in rolling terrain and, depending on final site design, could require new cuts and fills up to 5- to 10-feet deep, which may locally be supported with new retaining walls. Ancillary improvements will include new access roads/parking areas, underground utilities, landscaping, and other “typical” items.

We understand that a 1,000 square-foot Accessory Dwelling Unit (ADU) is also being considered and could be incorporated in the upper story of the barn or designed as a standalone building. An approximately 3,000 square-foot primary residence may also be considered. A preliminary Site Plan showing the proposed improvements is presented on Figure 2.

3.0 SITE CONDITIONS

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.

3.1 Regional Geology

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.

Regional geologic mapping (Clark and Brabb, 1997) indicates that the proposed development area is underlain by marine siltstone and sandstone bedrock of the Pliocene-Pleistocene age (approximately 3.6 to 1.8 million years old) Merced Formation. Approximately 600-feet west-southwest of the parcel, the northwest-trending San Gregorio Fault juxtaposes the Merced rocks on the northeast against Santa Cruz Mudstone to the southwest, which is of Miocene age (approximately 11.6 to 5.3 million years old). East of Pine Gulch Creek, both formations are indicated to dip gently to the east and northeast at about 25- to 35-degrees. The eastern boundary trace of the San Andreas Fault Zone is shown trending passing about 1,900-feet northeast of the site along the edge of Bolinas Lagoon. A Regional Geologic Map is 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 Division of Mines and Geology (1998) has mapped various active and inactive faults in the region. These faults are shown in relation to the project site on the attached Active Fault Map, Figure 4. The nearest known active faults to the site are the San Andreas and San Gregorio Faults. As discussed in Section 3.1 and shown on Figures 3 and 4, regional mapping indicates that the parcel is situated between the two fault traces, with the San Gregorio Fault passing about 600-feet to the southwest, and the San Andreas Fault passing about 1,900-feet 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 historic earthquake catalogue search indicates that at least 13 earthquakes with a Richter Magnitude of 5.0 or larger have occurred within 100 kilometers (62 miles) of the site between 1900 and 2022. Of these, the most significant was the M=7.7 Great San Francisco Earthquake of 1906, which caused severe shaking and structural damage throughout the Bay Area along with extensive surface rupture in West Marin, with surface offsets on the order of 20-feet observed in Olema, a few miles north of the site (Lawson, 1908). Other significant earthquakes in recent times include the M=6.9 Loma Prieta Earthquake of 1989, which was centered in the Santa Cruz Mountains and produced strong shaking in Bolinas, and an M=4.6 earthquake centered near downtown Bolinas in 1999 which caused moderate shaking and limited local damage.

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" (2003, 2008; Field et al, 2015) to estimate the probabilities of earthquakes on active faults. These studies have been published cooperatively by the USGS, CGS, and Southern California Earthquake Center (SCEC) as the Uniform California Earthquake Rupture Forecast, Versions 1, 2, and 3 (UCERF, UCERF2, and UCERF3, respectively). In these studies, potential seismic sources were analyzed considering fault geometry, geologic slip rates, geodetic strain rates, historic activity, micro-seismicity, and other factors to arrive at estimates of earthquakes of various magnitudes on a variety of faults in California.

The 2003 study (UCERF) specifically analyzed fault sources and earthquake probabilities for the seven major regional fault systems in the Bay Area region of northern California. The 2008 study (UCERF2) applied many of the analyses used in the 2003 study to the entire state of California and updated some of the analytical methods and models. The most recent 2015 study (UCERF3) further expanded the database of faults considered and allowed for consideration of multi-fault ruptures, among other improvements.

Conclusions from the most recent UCERF3 indicate the highest probability of an earthquake with a magnitude greater than 6.7 on any of the active faults in the San Francisco Bay region by 2045 is assigned to the Rodgers Creek Fault. As shown on Figure 4, the Rodgers Creek Fault is located approximately 30 kilometers northeast of the site, at 33%. The nearest known active fault, the San Gregorio Fault, located about 600-feet southwest of the site, is assigned a probability of about 6%. The San Andreas Fault, located approximately 1,900-feet northeast of the site, is assigned a probability of 22%. 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

We performed a site reconnaissance on July 28, 2022 to observe and document existing conditions at the site. The project site consists of a 6.5-acre, roughly rectangular parcel accessed by a narrow “flag” driveway that extends about 400-feet to the southwest from Horseshoe Hill Road. The site is surrounded by existing semi-rural residential development typical of West Marin.

The site is composed primarily of south-facing slopes which converge to drain towards the south-central part of the parcel. Surface elevations ranging from about +120-feet at the southern end of the parcel to +155-feet in the northeast corner of the lot[1].

The site is currently developed with a single-family residence and several smaller outbuildings in the southwest corner of the parcel. Landscaping around the existing residence is limited to vegetable garden beds and potted plants. Vegetation in the remainder of the property is largely native grasses, wild sage, and other low-lying scrub. Mature trees of various species are located along portions of the property lines.

During our reconnaissance, we did not observe any rock outcrops within the site, and surface soils were noted to consist of loose to medium-dense sand and silty sand.

3.4 Subsurface Exploration

Subsurface exploration for the project included excavation of five soil borings, drilled at the approximate locations shown on Figure 2 on July 28, 2022. Our borings were excavated to maximum explored depths between about 11.0- and 21.5-feet below the ground surface by use of a track-mounted drill rig equipped with 4-inch solid-stem augers. Borings were logged by our Geologist and samples were collected at select intervals for further examination. Brief descriptions of the terms and methodology used in classifying earth materials are provided on the Soil and Rock Classification Charts, Figures A-1 and A-2, respectively, and the Boring Logs are shown on Figures A-3 through A-7.

3.5 Subsurface Conditions and Groundwater

Our subsurface exploration generally confirms the mapped geologic conditions at the site. The site is typically underlain by approximately 5.0- to 10.0-feet of stiff/dense sandy clay, silty sand and to sandy silt surface soils over relatively shallow siltstone and sandstone bedrock of the Merced Formation. Boring 1 was drilled on the west side of the parcel between the approximate

---

[1] Surface elevations taken from Marin County GIS (www.marinmap.org) and are based on NAVD88.

locations of the guest cottage and the arena. Boring 1 encountered silty sand in the upper 10.5-feet and was underlain by dark gray siltstone from 10.5-feet to the maximum explored depth of 21.5-feet. Boring 2 was drilled on the east side of the parcel at the southwest corner of the proposed barn. Boring 2 encountered silty sand and sandy silt in the upper 7.5-feet. Sandstone cobbles and gravel were encountered from 7.5-feet to approximately 12.5-feet and was underlain by bright orange red sandstone from 12.5 to 15.5-feet. A layer of dense silty sand with shell fragments throughout was encountered below the sandstone and extended to the maximum explored depth of 17.3-feet.

Borings 3 and 4 were drilled at the northeast corner of the barn and the southwest corner of the arena, respectively. These borings encountered similar subsurface conditions. Silty sand to clayey sand surface soils were encountered from the ground surface to approximately 5.0-feet and were underlain by dark gray to blue gray siltstone to the maximum explored depth of 11.0-feet. Boring 5, drilled in the southwest corner of the property near the planned bioretention pond, encountered about 6.0-inches of gravel fill placed for the driveway over sandy silt with clay to a depth of 8.0-feet. Dense clayey sand with gravel was encountered at 8.0-feet and extended to the maximum explored depth of 9.5-feet. No weathered bedrock was encountered in the subsurface of Boring 5.

Groundwater was encountered only in Boring 2 at about 10.0-feet below ground surface. Because the borings and fault trench were not left open for an extended period of time, a stabilized depth to groundwater may not have been observed. Given site topography which generally drains internally to the south-central part of the site and the site’s location in close proximity to active fault zones, we anticipate groundwater may exist within 10-feet of the ground surface throughout the year, and could be significantly higher during the winter months, particularly in the lower-lying parts of the site.

4.0 GEOLOGIC HAZARDS

This section summarizes our review of commonly considered geologic hazards, discusses their potential impacts on the planned improvements, and identifies proposed remedial options. The primary geologic hazard which could affect the proposed development is strong seismic ground shaking. Other geologic hazards are judged relatively insignificant with regard to the proposed project. Each geologic hazard considered is discussed in further detail in the following paragraphs.

4.1 Fault Surface Rupture

The provisions of the Alquist-Priolo Earthquake Fault Zoning Act (“A-P Act”, 1972; revised 1988) and the current edition of the California Building Code (2019) dictate that structures intended for human occupancy (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 11,000 years). Typically, direct observation via logging trench excavations is the most effective means through which to determine whether or not, A) a fault trace is present, B) if any identified fault trace should be considered “active”, and C) whether the proposed project is therefore subject to restriction under the auspices of the A-P Act. It should be noted that the Act specifically exempts single-family homes unless they are part of a 3+ unit subdivision, but grants the local “Lead Agency” (in this case, the Marin County Planning Department) discretion to apply the full act to single-unit developments.

In addition to restricting development near active faults, the A-P Act requires the California Geological Survey to publish maps delineating Earthquake Fault Zones (“APEFZs”) within which Fault Trench Investigations are required prior to review of development applications. Based on review of the applicable Alquist-Priolo map for the Inverness Quadrangle (California Division of Mines and Geology, 1974), the project site lies entirely within the APEFZ associated with the San Andreas and San Gregorio Faults, as shown on Figure 5. Although the primary active traces of the San Gregorio and San Andreas Faults are shown in the same location as on regional geologic maps, the A-P map additionally indicates that the surface trace of an apparent secondary, en-echelon type fault passes through the site along a trend of approximately N40°W. Although the en-echelon trace is not indicated specifically to have ruptured in 1906, it appears to be associated with the San Andreas Fault, where surface rupture just east of the site was documented firsthand shortly following the earthquake (Lawson et al, 1908).

Although no surface evidence indicative of recent or historic surface rupture was observed at site, we did note that stratigraphy encountered in Boring 2, located nearest to the mapped secondary fault trace, was markedly different from other borings, including more significant rounded gravels, cobbles, and shell fragments. Based on our findings and lack of evidence to refute the fault shown on the A-P map, we judge that, barring further evidence to the contrary, a moderate to high risk of surface rupture will exist at the site in association with future earthquakes on either the San Gregorio or San Andreas Faults.

Evaluation: *Less than significant with remedial measures.*

Recommendations: *We recommend that a Fault Trench Investigation be performed to evaluate the existence and location of active fault traces at the site and develop adequate building setbacks, particularly for proposed habitable structures or for structures where collapse due to surface rupture would be unacceptable.*

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. Given the proximity of the site to the San Andreas and San Gregorio Faults, high levels of ground shaking should be anticipated. 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 2.

Table 1 – Deterministic Peak Ground Accelerations for Active Faults

75 Horseshoe Hill Road

Bolinas, California

Fault	Moment Magnitude for Characteristic Earthquake[1]	Closest Estimated Distance[2] (km)	Median Peak Ground Acceleration[3] (g)	Median PGA +1 Std Dev[3] (g)
San Andreas	8.0	0.6	0.58	1.05
San Gregorio	7.4	0.2	0.57	1.02
Hayward/Rodgers Creek	7.6	29.7	0.16	0.29
West Napa	7.0	50.4	0.07	0.13
Calaveras	7.5	58.3	0.08	0.16

Reference:

1) Values obtained from USGS Earthquake Scenario Map (BSSE 2014) using the maximum moment magnitude for rupture along all sections of each fault (Accessed August 25 2022).

2) Values estimated using Google Earth KML Files showing Quaternary Faults & Folds in the US obtained from USGS website (Accessed August 25, 2022).

3) Values calculated using Vs_{30} = 560 m/s for Site Class "C" in accordance with the 2019 CBC and 2016 ASCE-7. See additional discussion regarding Site Class determination and recommended seismic design criteria in Section 5 of this report.

4) Values determined using Pacific Earthquake Engineering Research Center (PEER) NGS-West2 Excel Spreadsheet, http://peer.berkeley.edu/ngawest2/databases/.

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 rate of activity, the San Andreas and San Gregorio 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 remedial measures.*

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

4.3 Liquefaction and Related Effects

Liquefaction refers to the sudden, temporary loss of soil strength during strong ground shaking. This phenomenon can occur in saturated, loose, granular deposits subjected to seismic shaking. Liquefaction can result in flow failure, lateral spreading, settlement, and other related effects. Recent advances in liquefaction studies indicate that liquefaction can occur in granular materials with relatively high fines content (i.e., containing 35 to 50 percent clay and silt particles that pass the #200 sieve) provided the fines exhibit a plasticity index less than 7.

The results of our subsurface exploration indicate that the planned building envelope is underlain by medium-dense to dense silty sand and sandy silt soils which are not likely to liquefy. We did encounter a 3-foot zone of potentially-liquefiable saturated gravel in Boring 2, which may be an isolated gravel bed within the Merced Formation, or perhaps equally likely, transported materials within a fault zone as shown on the Alquist-Priolo Map. Regardless, given the anticipation that groundwater will result in at least a partially-saturated soil column throughout the year, and given that isolated seams and stringers of looser granular soils may exist within the residual soils overlying the Merced Formation, we judge there is a low to moderate risk of liquefaction at the site. Liquefaction would most likely be manifested at the ground surface in the form of local sand boils, ground cracking, and minor differential settlements.

Evaluation:	*Less than significant with remedial measures.*
Recommendations:	*We recommend that structures be founded on rigid foundations designed to withstand differential settlements of up to 1-inch over a span of 30-feet. Foundation design criteria and recommendations are provided in Section 5.3 of this report.*

4.4 Seismically-Induced Ground Settlement

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 above the expected water table, and the risk of damage due to seismically-induced settlement is therefore judged to be low.

Evaluation:	*No significant impact.*
Recommendations:	*No remedial measures 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.

Based on our exploration, the majority of the development area is underlain by surface soils that are of low to medium plasticity, and no significant evidence indicative of significant expansion potential was observed during our reconnaissance. We did encounter high-plasticity clays in Boring 5 near the bioretention pond, but no structural development is planned in this area. Therefore, the risk of expansive soil affecting the proposed structures is generally low.

Evaluation:	*No significant impact.*
Recommendations:	*We should be consulted once the Site Plan has been finalized and structure footprints are known to provide supplemental recommendations as needed. If new structures are planned in the area of Boring 5, possible remedial measures could include lime-treatment, over-excavation, and replacement of expansive soils with select fill, or special design of new improvements to resist expansive effects. If expansive soils are otherwise encountered within structural footprints, remediation would likely consist of over-excavation and replacement with 3-feet of non-expansive materials within the structural footprint.*

4.6 Settlement

Significant settlement can occur when new loads are placed over soft, compressible clays or loose soils. Based on our exploration, surface soils are generally stiff, and bedrock is relatively shallow throughout the site, and the risk of significant settlement is therefore judged to be low.

Evaluation:	*No significant impact.*
Recommendations:	*No remedial measures required.*

4.7 Slope Instability/Landslides

The project site is located in a small swale at the crest of a steeply-sloping ridgeline. Elevations at the site range from about +120-feet at the central, southern end of the parcel to +155-feet in the northeast corner of the lot. However, descending west-facing slopes just west of the property exhibit average inclinations of about 2:1 (Horizontal:Vertical).

During our site reconnaissance, we did not observe topography suggestive of previous slope instability. No significant evidence of recent or imminent instability was observed, such as tension cracks, fresh scarps, or debris piles. Therefore, we judge that there is a low risk of damage due to slope instability.

Evaluation:	*No significant impact.*
Recommendations:	*No remedial measures required.*

4.8 Erosion

Sandy soils on moderately steep 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.

Gently sloping to level terrain and relatively dense/stiff surface soils make the site unlikely to be prone to erosion. We judge the risk of damage to improvements due to erosion is low.

Evaluation:	*Less than significant with remedial measures.*
Recommendations:	*For new improvements at the site, careful attention should be paid to finished grades and the project Civil Engineer should design the site drainage system to collect surface water into a storm drain system and discharge water at appropriate locations. Re-establishment of vegetation on disturbed areas will minimize erosion. Erosion control measures during and after construction should be in accordance with a prepared Storm Water Pollution Prevention Plan and should conform to the most recent version of the California Stormwater Quality Association Best Management Practice Handbook (CASQA, 2003) or similar standards.*

4.9 Tsunami and 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 not mapped within a tsunami inundation zone (California Emergency Management Agency, 2009), and tsunami inundation is therefore not considered a significant hazard at the site.

Evaluation:	*No significant impact.*
Recommendations:	*No remedial measures required.*

4.10 Flooding

The proposed improvements are located at elevations ranging from about 120 to 150 feet above sea level and are not mapped within a FEMA 100- or 500-year flood zone (Federal Emergency Management Agency, 2009). Therefore, large scale flooding is not considered a significant hazard at the project site. The project Civil Engineer or Architect is responsible for site drainage and should evaluate localized flooding potential and provide appropriate mitigation.

Evaluation:	*No significant impact.*
Recommendations:	*The project Civil Engineer or Architect should evaluate the risk localized flooding and provide appropriate storm drain design.*

5.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our subsurface exploration, we judge that construction of the proposed cottage, arena, barn, and associated improvements is feasible from a geotechnical standpoint. Primary geotechnical considerations for the project will include providing adequate foundation support and seismic design for the new structures. 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 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 3 be used to calculate the design base shear of the new construction.

Table 2 – 2019 California Building Code Seismic Design Criteria
75 Horseshoe Hill Road
Bolinas, California

Parameter	Design Value
Site Class	C
Site Latitude	37.921°N
Site Longitude	-122.698°W
Spectral Response (short), S_S	2.434 g
Spectral Response (1-sec), S_1	1.021 g
Site Coefficient, F_a	1.2
Site Coefficient, F_V	1.4

Reference: SEAOC/OSHPD Seismic Design Maps (web-based seismic response calculator tool), https://seismicmaps.org/, accessed August 31, 2022.

5.2 Site Grading

Moderate site grading, including cuts and fills up to a few feet high, is anticipated for the project. 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.

Where fills or structural improvements are planned, the subgrade surface should be scarified to a depth of 8 inches, moisture conditioned to slightly above the optimum moisture content, and compacted to at least 90 percent relative compaction. Areas exposing bedrock at subgrade need not be scarified and recompacted. 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 5-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 <u>Excavations</u>

Site excavations for new foundations, utilities, and other improvements will generally encounter a silty sand and sandy silt in the upper 5.0- to 10.0-feet over highly weathered siltstone and sandstone. Based on our subsurface exploration, we judge that excavations within the upper 10 feet of the ground surface can likely be performed with “traditional” equipment, such as medium-size dozers and excavators.

5.2.3 <u>Fill Materials, Placement and Compaction</u>

New fill up to 4.0-ft thick is planned to construct a level footprint for the proposed arena. As a minimum, fill materials should be non-expansive and 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 have 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 4 inches. Some of the onsite soils may be suitable for use as fill provided, they meet the criteria specified above. Any imported fill material needs to be tested to determine its suitability.

Fill materials should be moisture conditioned to near the optimum moisture content prior to compaction. Properly moisture conditioned fill materials should subsequently be placed in loose, horizontal lifts of 8 inches-thick or less and uniformly compacted to at least 90 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 <u>Permanent and Temporary Cut and Fill Slopes</u>

Permanent cut slopes in onsite soils and weathered bedrock should be inclined no steeper than 2:1 (horizontal:vertical). New fill slopes steeper than 10:1 should be keyed, benched, fully drained, and should not exceed 2:1. Permanent fills steeper than 2:1 are generally not recommended at the site.

Onsite soils should be considered “Type C” materials per Cal/OSHA categorization. As such, temporary cuts should be inclined no steeper than 1.5:1. Steeper cuts in more competent bedrock may be possible, but geologic inspection during construction will be required in that event.

5.3 Foundation Design

Bedrock is relatively shallow throughout the site, with about 5 to 9 feet of silty and sandy soils overlying Merced Formation siltstone bedrock. Based on the preliminary Site Plan shown on Figure 2, grading for the site may expose a variety of soil and bedrock conditions. We judge that shallow foundations are appropriate provided they bear entirely and uniformly on either stiff soil or weathered bedrock. Suitable shallow foundations could include continuous spread footings or concrete mat slabs; isolated footings should be avoided. If needed, drilled micropiles may be utilized to resist seismic uplift loads.

Where the soil layer is deeper than a few feet and deepening footings to bedrock is judged impractical, then drilled, cast-in-place, reinforced concrete piers and grade beams should be used. Drilled piers are also likely appropriate for the planned covered arena structure. Drilled piers should be embedded into bedrock beneath the surface soils in order to provide uniform support. The project structural engineer should utilize the foundation design criteria given below in Table 3.

Table 3: Foundation Design Criteria
75 Horseshoe Hill Road
Bolinas, California

	Soil	Bedrock
Shallow Footings		
Minimum Width:		
One-Story:		12 inches
Two-Story:		15 inches
Minimum Embedment below Adjacent Grade:		12 inches
Allowable Bearing Pressure:		
	Soil	Bedrock
Dead Plus Live Loads:	2,000 psf	4,000 psf
Total Design Loads (includes wind or seismic):	3,000 psf	5,500 psf
Base Friction Coefficient:	0.30	0.35
Lateral Passive Resistance [2,3]:	300 pcf	450 pcf
Drilled Piers		
Minimum Diameter:		18 inches
Minimum Embedment:		5 feet into bedrock
	Soil	Bedrock
Skin Friction[4]:	500 psf	2,000 psf
Lateral Passive Resistance[3,5]:	300 pcf	450 pcf
Mat Slabs		
Minimum Embedment[6]:		12 inches
Minimum Slab Thickness[7]:		8 inches
Maximum Unsupported Interior Span:		6 feet
Maximum Unsupported Edge Span:		3 feet
Allowable Bearing Pressure:		
	Soil	Bedrock
Dead Plus Live Loads:	2,000 psf	4,000 psf
Total Design Loads (includes wind or seismic):	3,000 psf	5,500 psf
Base Friction Coefficient:	0.30	0.35
Lateral Passive Resistance [2,3]:	300 pcf	450 pcf

Notes:
(1) Size footing widths to avoid significantly different foundation pressures, maintain 7-foot minimum horizontal distance from bottom of footing to slope grade. Slabs or footing
(2) Can combine sliding resistance with passive resistance. Equivalent Fluid Pressure, not to exceed 4,000 psf.
(3) Ignore upper 6-inches on level ground and upper 18-inches on sloping terrain unless concrete or asphalt surfacing exists adjacent to foundation.
(4) May increase design values by 1/3 for total design loads including seismic. Uplift use 80% friction.
(5) Apply passive over two pier diameters. Maintain pier spacing of at least three pier diameters.
(6) Deepen mat slab as required to bear uniformly on firm soil.
(7) Actual thickness, load distribution and unsupported spans must be determined by the Structural Engineer to reduce deformations to acceptable levels.

5.4 Retaining Walls

Retaining walls up to 9.0-feet tall are currently planned to support cuts for the new covered arena. Foundations for new retaining walls should be designed per Section 5.3 of this report. Retaining walls that can deflect a small amount at the top, such as site or landscape walls, can be designed using the unrestrained criteria shown in Table 4. Walls that are structurally connected at the top and not allowed to deflect (such as basement walls or tieback 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 seismic loads as well as any anticipated traffic surcharge loads (such as for walls supporting driveway areas, if planned).

Table 4 – Active Earth Pressure for Retaining Wall Design
75 Horseshoe Hill Road
Bolinas, California

Backfill Inclination[1]	Unrestrained[2,3]	Restrained[3,4]
Level	40 pcf	30 x H psf
3:1	50 pcf	35 x H psf
2:1	60 pcf	40 x H psf

Notes:
(1) Interpolate earth pressures for intermediate slopes
(2) Equivalent fluid pressure.
(3) Wall design should account for a seismic surcharge of 15 x H (in psf) in addition to active pressure and factor of safety should be >1.0
(4) Rectangular distribution, H is wall height in feet.

Wall drainage is required for all retaining walls taller than 3 feet. Either Caltrans Class 1B permeable material within filter fabric or Caltrans Class 2 permeable material can be used for wall drainage. The drainage should be collected in a 4-inch, SDR 35, 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 schematic retaining wall drainage detail is shown on Figure 7.

5.5 Concrete Slabs-On-Grade

Reinforced concrete slab-on-grade floors are judged to be appropriate for the site conditions. The concrete slabs-on-grade 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 5 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 4-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 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 4 inches of Class 2 Aggregate Base (Caltrans, 2015) compacted to at least 95 percent relative compaction.

5.6 Exterior Flatwork

Exterior concrete walkway slabs not subjected to vehicle loads should be a minimum of 4-inches-thick and underlain with 4 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 8 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 settlements), exterior slabs can be thickened to 5 inches and reinforced with steel reinforcing bars (not welded wire mesh). We recommend crack control joints no farther than 6 feet apart in both directions and that the reinforcing bars, if used, extend through the control joints. Some movement should be expected due to seasonal shrink/swell of soils.

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 5 feet (5 percent) from the perimeter of building foundations. Where hard surfaces, such as concrete or asphalt adjoin foundations, slope these surfaces at least 0.10 feet in the first 5 feet (2 percent).

Roof gutter downspouts may discharge onto the pavements but should not discharge onto landscaped areas immediately adjacent to the home. Provide area drains for landscape planters adjacent to buildings and parking areas 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.

Given the slopes that exist throughout the project site, we recommend including foundation drains around the upslope side of all building foundations which are not provided with a retaining wall and associated backdrain. A schematic foundation drain detail is included as Figure 8.

5.8 New Utilities

Excavations for utilities will likely be encounter silty sand and sandy silt surface soils and may encounter groundwater at shallow depths if wintertime or early spring work is performed. Trench excavations having a depth of 5 feet or more must be excavated and shored in accordance with OSHA regulations. Pursuant to OSHA classifications, on-site soils should be considered Type C.

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 5 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 3 to 6 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

We must review the plans and specifications for site development and foundation design when they are nearing completion to confirm that the intent of our recommendations has been incorporated and to provide supplemental recommendations as needed. During construction, we must inspect geotechnical items relating to site preparation and grading, retaining walls and foundation construction. We should observe foundation excavations, subgrade preparation and compaction, proper moisture conditioning of soils, fill placement and compaction, retaining wall drainage and backfilling and other geotechnical-related work items.

7.0 LIMITATIONS

We believe 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 data obtained during our subsurface exploration program and our experience with soils in this geographic area. The exploratory test pits and description of soils encountered reflect conditions only at the location of the test pit 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.

8.0 LIST OF REFERENCES

American Society of Civil Engineers (ASCE) (2016), “Minimum Design Loads for Buildings and Other Structures” (2016 ASCE-7), Structural Engineering Institute of the American Society of Civil Engineers.

American Society for Testing and Materials, (2021) “2021 Annual Book of ASTM Standards, Section 4, Construction, Volume 4.08, Soil and Rock; Dimension Stone; Geosynthetics,” ASTM, Philadelphia.

Association of Bay Area Governments (ABAG), Geographic Information System, http://quake.abag.ca.gov/mitigation/, 2016. Date Accessed August 30, 2022

California Building Code, 2019 Edition, California Building Standards Commission/International Conference of Building Officials, Whittier, California.

California Department of Conservation, Division of Mines and Geology (1972), Special Publication 42, “Alquist-Priolo Special Studies Zone Act,” (Revised 1988).

California Department of Conservation, Division of Mines and Geology (2000), “Digital Images of Official Maps of Alquist-Priolo Earthquake Fault Zones of California, Central Coast Region”, DMG CD 2000-004.

California Department of Conservation, Division of Mines and Geology, “Geology of the Tomales Bay Study Area”, Marin County, California (1:12,000)”, Open-File Report 77-15 S.F., Plate 2.

California Department of Transportation (Caltrans) (2018), 2018 Standard Specifications.

California Department of Transportation (Caltrans) (2018), “Caltrans ARS Online, V 2.3.09”, http://dap3.dot.ca.gov/ARS_Online/, accessed September 6, 2018.

California Division of Mines and Geology, “State of California, Special Studies Zone, Bolinas Quadrangle”, July 1, 1974. Map Scale 1:24,000.

California Emergency Management Agency, “Tsunami Inundation Map for Emergency Planning, Inverness Quadrangle”, 2009.

California Stormwater Quality Association, “Stormwater Management Best Practices Handbook, New Development and Redevelopment”, January, 2003.

Clark, Joseph C and Brabb, Earl E. 1997. “Geology of the Point Reyes National Seashore and Vicinity.” United States Geological Survey. Open File Report 97-456. Map Scale 1:48,000.

Federal Emergency Management Agency, “National Flood Insurance Program, Flood Insurance Rate Map, Marin County, California and Incorporated Areas”, Panel 230 of 531, Map Number 06041C0230D, dated May 4, 2009.

Field, E.H. et al (2015), "Long-Term Time-Dependent Probabilities for the Third Uniform California Earthquake Rupture Forecast (UCERF3)", Bulletin of the Seismological Society of America, Volume 105, No. 2A, 33pp., April 2015, doi: 10.1785/0120140093.

Idriss, I.M. & Boulanger, R.W. “SPT-Based Liquefaction Triggering Procedures” Department of Civil and Environmental Engineering, College of Engineering, University of California at Davis, UCD/GCM-10/02, December 2010.

Occupational Safety and Health Administration (OSHA)(2005), Title 29 Code of Federal Regulations, Part 1926, 2005.

SEAOC/OSHPD Seismic Design Maps (2021), www.seismicmaps.org, accessed August 30, 2022.

United States Geological Survey, “Database of Potential Sources for Earthquakes Larger than Magnitude 6 in Northern California,” The Working Group on Northern California Earthquake Potential, Open File Report 96-705, 1996.

United States Geological Survey (2003), “Summary of Earthquake Probabilities in the San Francisco Bay Region, 2002 to 2032,” The 2003 Working Group on California Earthquake Probabilities, 2003.

United States Geological Survey (2008), “The Uniform California Earthquake Rupture Forecast, Version 2,” The 2007 Working Group on California Earthquake Probabilities, Open File Report 2007-1437, 2008.

United States Geological Survey (2022), Earthquake Hazards Program, Earthquake Catalogue Search https://earthquake.usgs.gov/earthquakes/search/, accessed September 8, 2022.

SITE COORDINATES
LAT. 37.921°
LON. -122.698°

SITE LOCATION

N.T.S.

NORTH

REFERENCE: Google Earth, 2022

MPEG MILLER PACIFIC ENGINEERING GROUP	504 Redwood Blvd. Suite 220 Novato, CA 94947 T 415 / 382-3444 F 415 / 382-3450 www.millerpac.com	SITE LOCATION MAP		
A CALIFORNIA CORPORATION, © 2022, ALL RIGHTS RESERVED FILENAME: 3281.001 Figures.dwg		75 Horseshoe Hill Road Bolinas, California Project No. 3281.002 Date: 9/13/2022	Drawn ZMS Checked	1 FIGURE

Approximate location of boring completed by MPEG, 2022

SCALE

0 30 60 120 FEET

MPEG MILLER PACIFIC ENGINEERING GROUP

A CALIFORNIA CORPORATION, © 2022, ALL RIGHTS RESERVED

FILE: 3281.001 Figures.dwg

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

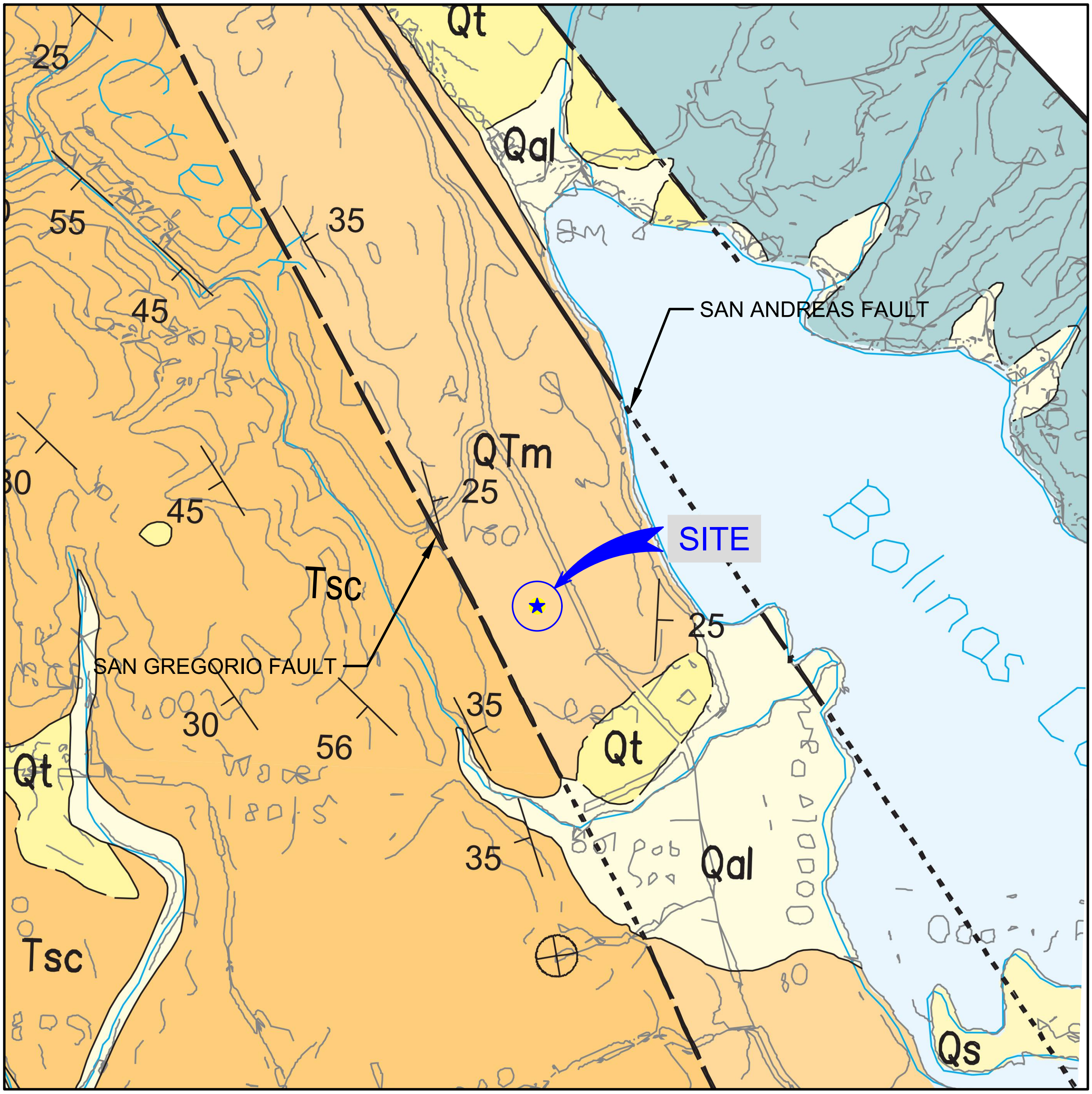
SITE PLAN

75 Horseshoe Hill Road
Bolinas, California

Project No. 3281.002 Date: 9/13/2022

Designed ZMS
Drawn ZMS
Checked ZMS

2
FIGURE



REGIONAL GEOLOGIC MAP

LEGEND

Qal	Alluvium (Holocene)
Qt	Terrace deposits (Pleistocene)
QTm	Merced Formation (upper Pliocene to Pleistocene) marine sandstone and siltstone
Tsc	Santa Cruz Mudstone (upper Miocene) marine siliceous mudstone

Contact - Dashed where approximately located or gradational, dotted where concealed.

Fault - Dashed where inferred, dotted where concealed. Half arrows indicate relative horizontal movement.

NORTH

Reference: Clark, Joseph C and Brabb, Earl E. 1997. "Geology of the Point Reyes National Seashore and Vicinity." United States Geological Survey. Open File Report 97-456. Map Scale 1:48,000

MPEG MILLER PACIFIC ENGINEERING GROUP

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

A CALIFORNIA CORPORATION, © 2022, ALL RIGHTS RESERVED

FILENAME: 3281.001 Figures.dwg

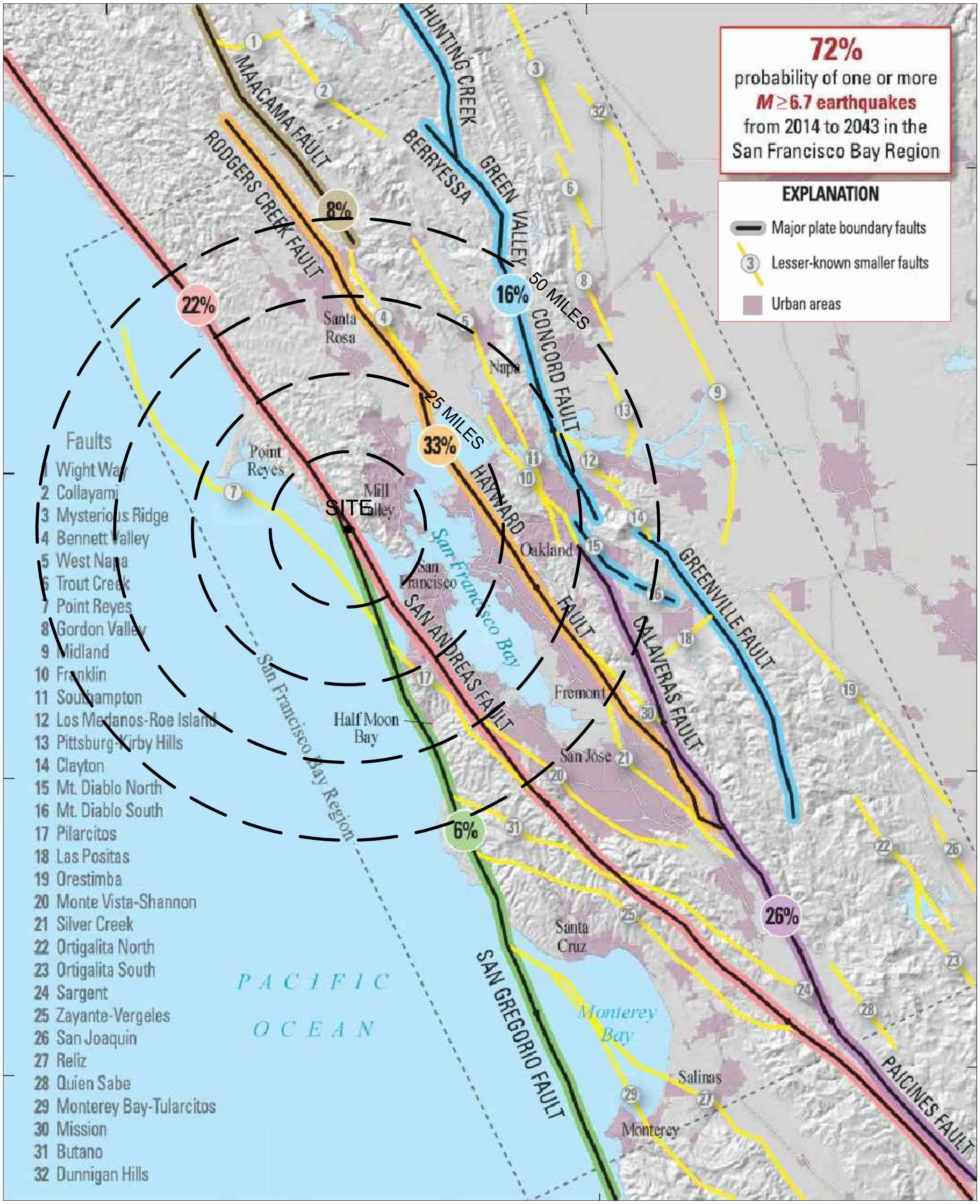
REGIONAL GEOLOGIC MAP

75 Horseshoe Hill Road
Bolinas, California

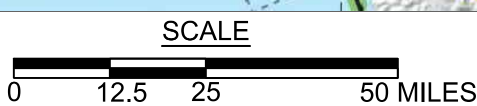
Drawn ZMS
Checked

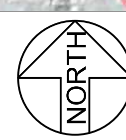
Project No. 3281.002 Date: 9/13/2022

3
FIGURE



SITE COORDINATES
LAT. 37.921°
LON. -122.698°





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

A CALIFORNIA CORPORATION, © 2022, ALL RIGHTS RESERVED

FILENAME: 3281.001 Figures.dwg

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

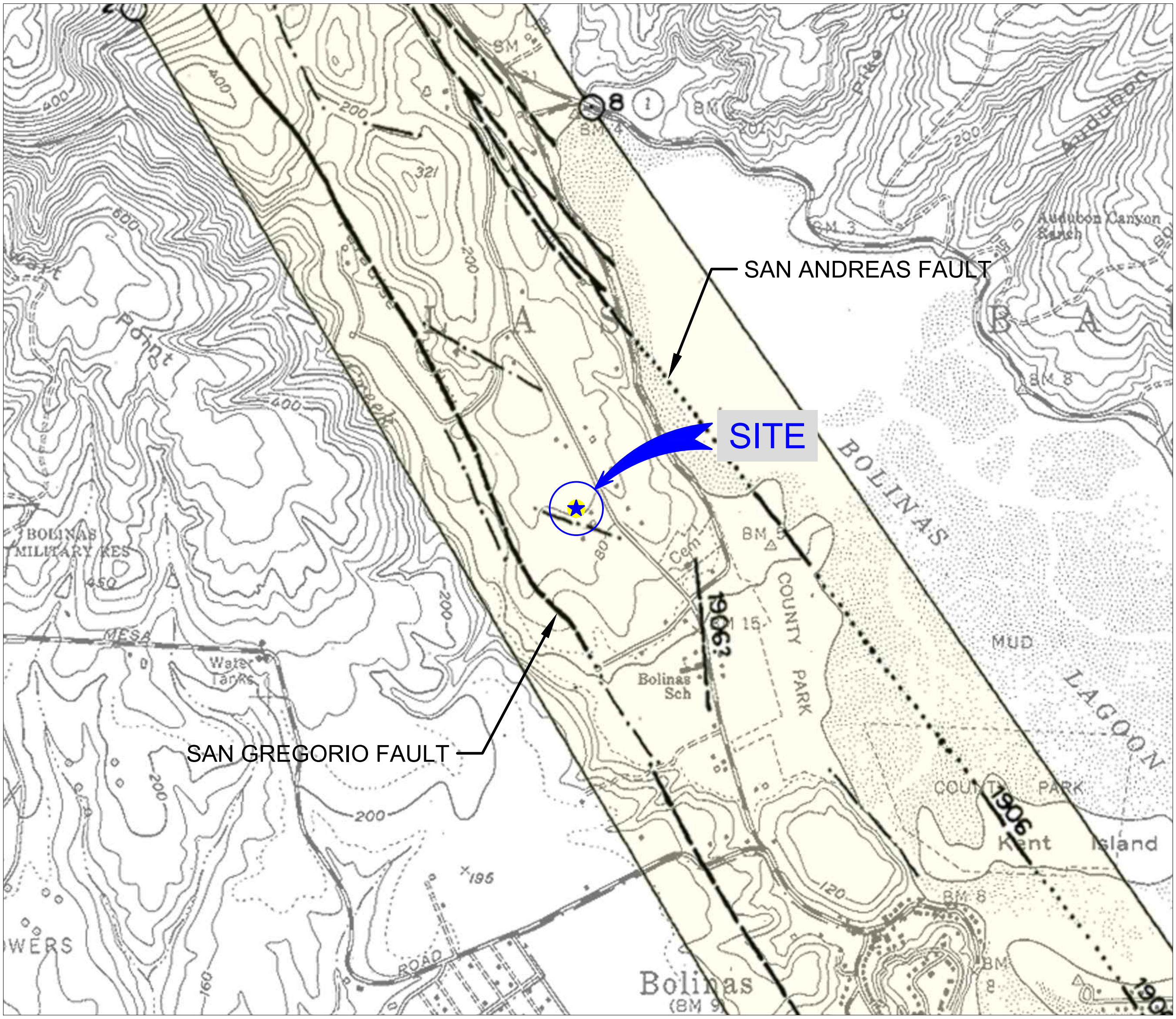
ACTIVE FAULT MAP

75 Horseshoe Hill Road
Bolinas, California

Project No. 3281.002 Date: 9/13/2022

Drawn ZMS
Checked

4
FIGURE



ALQUIST PRIOLO MAP

NORTH

MAP EXPLANATION

Potentially Active Faults

Faults considered to have been active during Quaternary time; solid line where accurately located, long dash where approximately located, short dash where inferred, dotted where concealed; query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by creep or possible creep.

Aerial photo lineaments (not field checked); based on youthful geomorphic and other features believed to be the results of Quaternary faulting.

Special Studies Zone Boundaries

These are delineated as straight-line segments that connect consecutively numbered turning points so as to define one or more special studies zone segments.

Seaward projection of zone boundary.

Reference: California Division of Mines and Geology, "State of California, Special Studies Zone, Bolinas Quadrangle", July 1, 1974. Map Scale 1:24,000.

MPEG MILLER PACIFIC ENGINEERING GROUP

A CALIFORNIA CORPORATION, © 2022, ALL RIGHTS RESERVED

FILENAME: 3281.001 Figures.dwg

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

ALQUIST PRIOLO SPECIAL STUDIES ZONES MAP

75 Horseshoe Hill Road
Bolinas, California

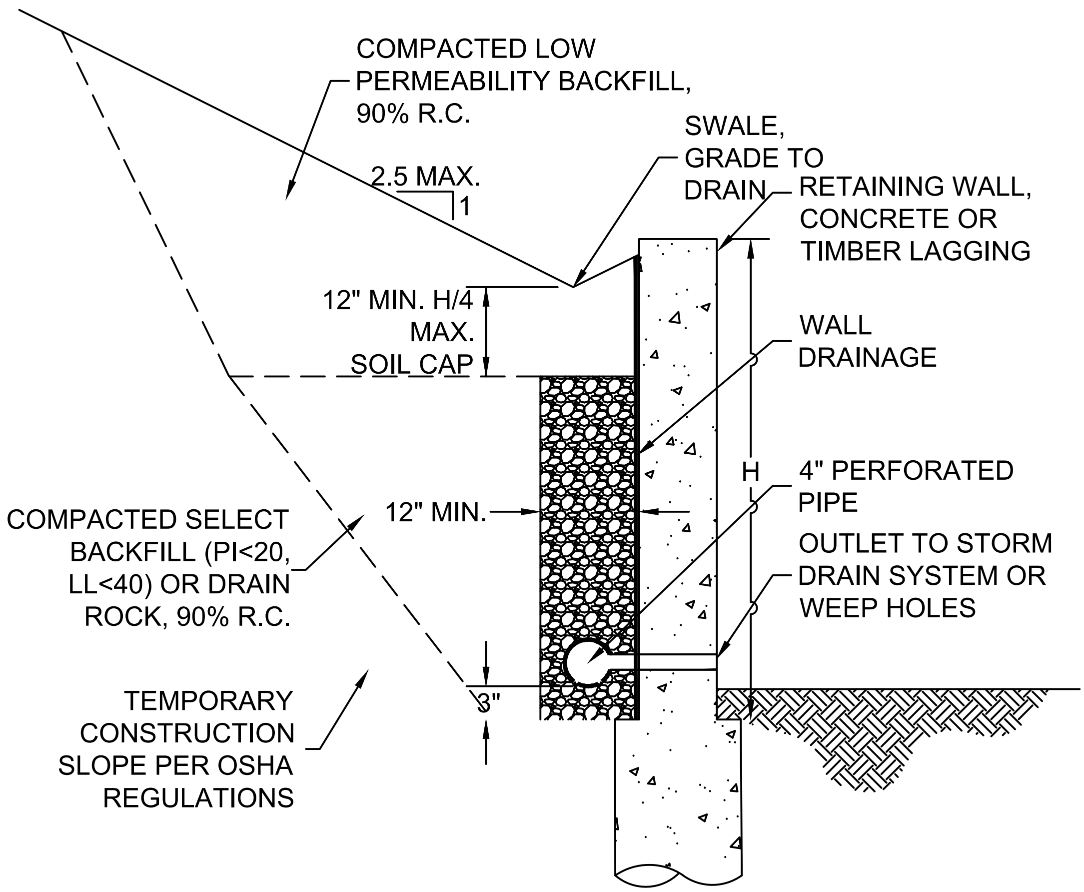
Drawn ZMS
Checked

Project No. 3281.002

Date: 9/13/2022

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.

MPEG
MILLER PACIFIC ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com
A CALIFORNIA CORPORATION, © 2022, ALL RIGHTS RESERVED
FILENAME: 3281.001 Figures.dwg

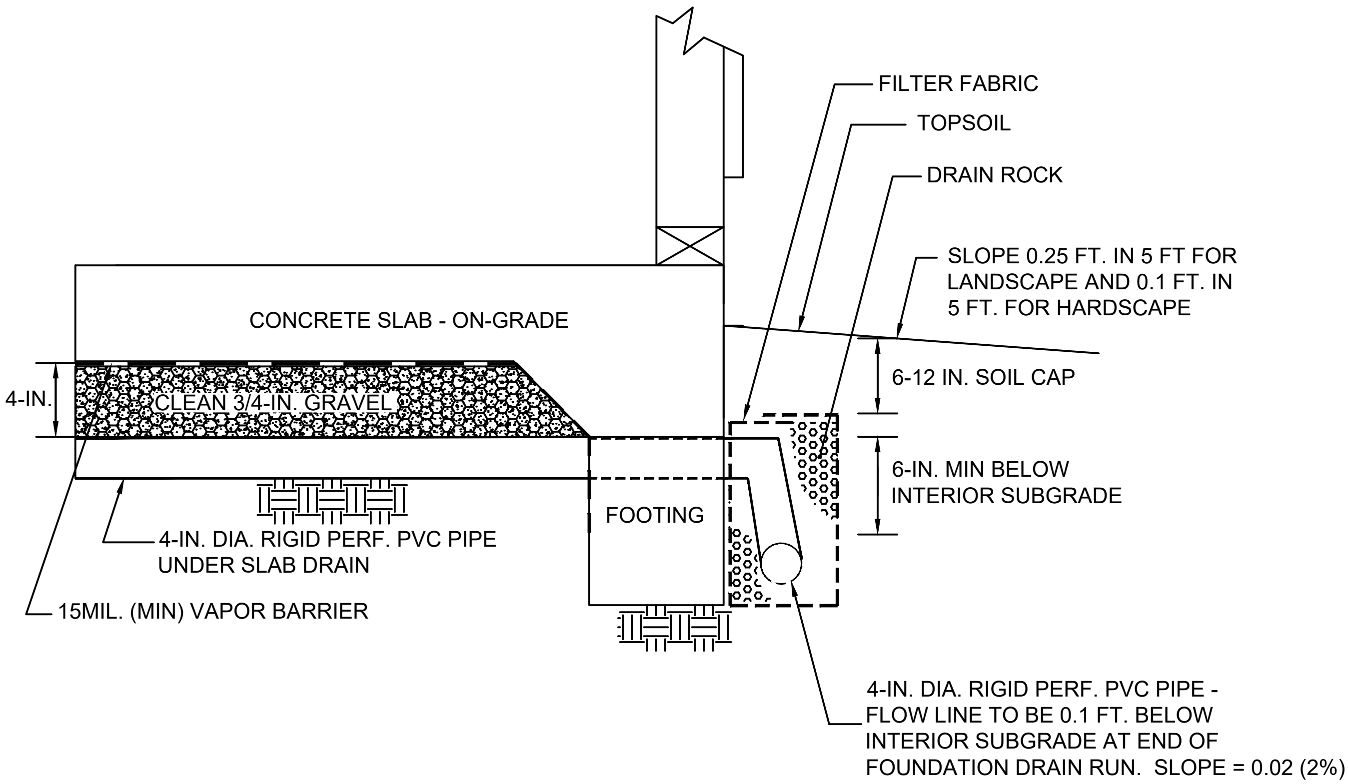
TYPICAL RETAINING WALL BACKDRAIN

75 Horseshoe Hill Road
Bolinas, California

Drawn ZMS
Checked

Project No. 3281.002 Date: 9/13/2022

6
FIGURE



NOTES:

1. Do not connect downspouts or area drains to foundation drain.

2. Discharge through rigid, non-perforated PVC pipe, slope 0.02 (2%) to storm drain system or outfall per civil engineer.

3. Provide backflow prevention so that exterior water does not flow into interior.

SCHEMATIC FOUNDATION DRAINAGE DETAIL

75 Horseshoe Hill Road
Bolinas, California

Project No. 3281.002 Date: 9/13/2022

Drawn ZMS
Checked

7
FIGURE

APPENDIX A
SUBSURFACE EXPLORATION AND LABORATORY TESTING

A. SUBSURFACE EXPLORATION

We explored subsurface conditions with 5 exploratory borings drilled with track-mounted drilling equipment on July 28, 2022, at the approximate locations shown on the Site Plan, Figure 2. The exploration was conducted under the technical supervision of our Field Geologist who examined and logged the soil materials encountered and obtained samples. Brief descriptions of the terms and methodology used in classifying earth materials are provided on the Soil and Rock Classification Charts, Figures A-1 and A-2, respectively, and the Boring Logs are shown on Figures A-3 through A-7.

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:

- Determining the Amount of Material Finer than 75-µm (No. 200) Sieve in Soils by Washing, ASTM D1140
- 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
- Particle Size Distribution of Soils using Sieve Analysis, ASTM D6914
- Atterberg Limits Plasticity Index, ASTM D4318

The laboratory test results are presented on the boring logs in Appendix A. The exploratory boring logs, description of soils encountered, and the laboratory test data reflect conditions only at the location of the boring at the time they were excavated or retrieved. Conditions may differ at other locations and may change with the passage of time due to a variety of causes including natural weathering, climate, and changes in surface and subsurface drainage.

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

KEY TO BORING AND TEST PIT SYMBOLS

CLASSIFICATION TESTS

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

SAMPLER TYPE

MODIFIED CALIFORNIA

STANDARD PENETRATION TEST

THIN-WALLED / FIXED PISTON

HAND SAMPLER

ROCK CORE

X DISTURBED OR BULK SAMPLE

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

STRENGTH TESTS

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

SAMPLER DRIVING RESISTANCE

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

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

MPEG MILLER PACIFIC ENGINEERING GROUP

A CALIFORNIA CORPORATION, © 2022, ALL RIGHTS RESERVED

FILENAME: 3281.002 BL.dwg

504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

SOIL CLASSIFICATION CHART

75 Horseshoe Hill Road
Bolinas, California

Project No. 3281.002 Date: 9/7/2022

Drawn ZMS
Checked

A-1
FIGURE

FRACTURING AND BEDDING

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

HARDNESS

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

STRENGTH

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

WEATHERING

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

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

MPEG MILLER PACIFIC ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

A CALIFORNIA CORPORATION, © 2022, ALL RIGHTS RESERVED

FILENAME: 3281.002 BL.dwg

ROCK CLASSIFICATION CHART

75 Horseshoe Hill Road
Bolinas, California

Project No. 3281.002 Date: 9/7/2022

Drawn ZMS
Checked

A-2
FIGURE

BORING 1

EQUIPMENT: Track Mounted Drill Rig with 4.0-inch Solid Flight Auger
DATE: 7/28/22
ELEVATION: 130 - feet*
*REFERENCE: Google Earth, 2022

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)	DESCRIPTION	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
0	0			Organic matter						
				Silty SAND (SM) Buff to orange, lightly mottled, dry, dense, fine grained, ~15% non plastic silt, rootlets present throughout. [Colluvium]	41	106	12.2			
1	5			Silty SAND (SM) Buff to orange, moist, dense, fine grained, approx. 20% low to non-plastic silt, completely weathered siltstone gravel present throughout. [Residual soil]	45	104	19.9			
2				Hard/gravelly drilling at 8.0-ft						
3	10			Grades orange brown, moist,medium dense to dense, approx. 30-40% silt, dark gray siltstone, gravels with diameter typically 2.5-cm, highly weathered.						
				SILTSTONE Dark gray, moderately strong, low hardness, moderately to highly weathered, massive/very faint bedding. [Bedrock]	80	99	23.8			
4	15			As above, bedding/rock structure more visible.						
5					37		21.6			
6	20			Bottom of boring at 21.5-ft. No groundwater encountered.	70		22.1			

Water level encountered during drilling
Water level measured after drilling

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

MPEG MILLER PACIFIC ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

A CALIFORNIA CORPORATION, © 2022, ALL RIGHTS RESERVED
FILENAME: 3281.002 BL.dwg

BORING LOG

75 Horseshoe Hill Road
Bolinas, California

Project No. 3281.002 Date: 9/7/2022

Drawn ZMS
Checked

A-3
FIGURE

BORING 2

EQUIPMENT: Track Mounted Drill Rig with 4.0-inch Solid Flight Auger
DATE: 7/28/22
ELEVATION: 120 - feet*
*REFERENCE: Google Earth, 2022

DEPTH: meters 0–6; feet 0–20

DESCRIPTION	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
Organic matter						
Silty SAND (SM) / Sandy SILT (ML) Buff to light tan, dry, medium stiff/dense, fine grained sand, approx. 40% non plastic silt, trace gravel, rootlets present. [Old Alluvium]	22	92	8.1	750		
Grades dark yellow/yellow gray, moist, increased clay percent, can roll approximately 0.75-inch snake.	20	105	17.2	1200		
As above, sandstone cobble in front of sampler, pushing the rock. Grinding on rock at 8ft, hard and slow drilling (approx. 3min/ft).	50	101	13.5			
Gravel (rounded, ~1in diameter) coming off auger, wet, perched water present, varying amounts of fines.			4.1			
SHALE cobble Dark gray, hard, strong, moderately weathered.						
SANDSTONE Bright orange red, strong, low to medium hardness, fine grained, thinly bedded, distinct color striations, moderately-highly weathered. [Bedrock or Boulder]	75					
Silty SAND w/ Shells Light yellow brown, moist, dense, fine grained, many shell fragments throughout. [Old Alluvium]						
Bottom of boring at 17.3-ft. Ground water encountered during drilling at 9.0-ft. Groundwater at 10.0-ft after drilling.	50/3		12.0			

Water level encountered during drilling
Water level measured after drilling

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

MPEG MILLER PACIFIC ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

A CALIFORNIA CORPORATION, © 2022, ALL RIGHTS RESERVED
FILENAME: 3281.002 BL.dwg

BORING LOG

75 Horseshoe Hill Road
Bolinas, California

Project No. 3281.002 Date: 9/7/2022

Drawn ZMS
Checked

A-4
FIGURE

BORING 3

EQUIPMENT: Track Mounted Drill Rig with 4.0-inch Solid Flight Auger
DATE: 7/28/22
ELEVATION: 120 - feet*
*REFERENCE: Google Earth, 2022

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)	DESCRIPTION	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
0	0			Organic matter						
				Sandy SILT (ML) Gray and light brown, dry, stiff, low plasticity silt, approx. 40% fine grained sand, rootlets present throughout. [Old Alluvium]	22	90	8.2	350	63.3% P200	
1				Grades light tan, increase in moisture with depth, increase fines, medium plasticity.	22	111	10.2	1100		L.L. 29 P.L. 15 P.I. 14
	5									
2				SILTSTONE Orange tan, medium strong to strong, low hardness, highly weathered, iron oxidation on laminations, thinly bedded to laminated, clay seams throughout, cross bedding present. [Bedrock]	50	89	30.8			
3	10			Grades coarser grained.	74	108	19.5			
				Bottom of boring at 11.0 ft. No groundwater encountered.						
4										
	15									
5										
6	20									

Water level encountered during drilling
Water level measured after drilling

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

MPEG MILLER PACIFIC ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

A CALIFORNIA CORPORATION, © 2022, ALL RIGHTS RESERVED
FILENAME: 3281.002 BL.dwg

BORING LOG

75 Horseshoe Hill Road
Bolinas, California

Project No. 3281.002   Date: 9/7/2022

Drawn ZMS
Checked

A-5
FIGURE

BORING 4

EQUIPMENT: Track Mounted Drill Rig with 4.0-inch Solid Flight Auger
DATE: 7/28/22
ELEVATION: 118 - feet*
*REFERENCE: Google Earth, 2022

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)	DESCRIPTION	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
0	0			Organic matter						
				Silty SAND (SM) Medium brown and buff, dry, dense, fine grained sand, approx. 20% low plasticity silt. [Old Alluvium]						
1					20	100	10.7			
	5			Sandy CLAY w/ Gravel (CL) Light tan, moist, stiff, medium plasticity clay, fine grained sand, gravel with typical diameter of 2.0-cm, sub-rounded to rounded, varies from competent to locally weathered. [Old Alluvium]	25	102	20.2		57.6% P200	
2										
				SILTSTONE Light gray and orange, strong, low hardness, thinly bedded, iron oxidation between beds. [Bedrock]	78/11	104	22.7			
3	10			Grades blue gray, moderately strong to strong, moderately weathered, iron oxide lining fractures	46					
				Bottom of boring a 11.0-ft. No groundwater encountered.						
4										
	15									
5										
6	20									

∇ Water level encountered during drilling
▼ Water level measured after drilling

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

MPEG MILLER PACIFIC ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

A CALIFORNIA CORPORATION, © 2022, ALL RIGHTS RESERVED
FILENAME: 3281.002 BL.dwg

BORING LOG

75 Horseshoe Hill Road
Bolinas, California

Project No. 3281.002 Date: 9/7/2022

Drawn ZMS
Checked

A-6
FIGURE

BORING 5

EQUIPMENT: Track Mounted Drill Rig with 4.0-inch Solid Flight Auger
DATE: 7/28/22
ELEVATION: 120 - feet*
*REFERENCE: Google Earth, 2022

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)	DESCRIPTION	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
0	0			Aggregate baserock (driveway fill)						
				Sandy CLAY (CH)						
1				Orange tan, moist, stiff, high plasticity clay, percent clay varies with depth, approximately 25% fine grained sand, rootlets present in upper half. [Old Alluvium]	23	101	21.2	5800	74.0% P200	L.L. 64 P.L. 30 P.I. 34
	5			Grades light tan with orange spots throughout, dry, decreased clay, slight increase in sand grain size.	49	106	13.2	3040		
2				Clayey SAND w/ Gravel (SC)						
				Dark yellow brown, moist, dense, fine-medium grained sand, approx. 15-25% clay (varies with depth) low plasticity, small gravel present throughout sample. [Old Alluvium]	42		12.4			
3	10			Bottom of boring a 9.5-ft. No groundwater encountered.						
4										
	15									
5										
6	20									

∇ Water level encountered during drilling
▼ Water level measured after drilling

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

MPEG MILLER PACIFIC ENGINEERING GROUP
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

A CALIFORNIA CORPORATION, © 2022, ALL RIGHTS RESERVED
FILENAME: 3281.002 BL.dwg

BORING LOG

75 Horseshoe Hill Road
Bolinas, California

Project No. 3281.002 Date: 9/7/2022

Drawn ZMS
Checked

A-7
FIGURE