MILLER PACIFIC
ENGINEERING GROUP

GEOTECHNICAL INVESTIGATION
NEW OFFICE DEVELOPMENT
270 ELM ROAD
BOLINAS, CALIFORNIA

October 20, 2023

Project 3528.001

Prepared for:
Bolinas Community Public Utility District
270 Elm Road
Bolinas, California 94924

Attn: Jennifer Blackman

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 radon, asbestos, toxic mold and other biological pollutants, and other hazardous materials. The document is for the sole use of the client and consultants on this project. Use by third parties or others is expressly prohibited without written permission. If the project changes, or more than two years have passed since the issuance of this report, the findings and recommendations must be reviewed by the undersigned.

MILLER PACIFIC ENGINEERING GROUP (a California corporation)	REVIEWED BY
CERTIFIED ENGINEERING GEOLOGIST MICHAEL JEWETT No. 2610 EXP. 01/31/25 STATE OF CALIFORNIA	REGISTERED PROFESSIONAL ENGINEER SCOTT A. STEPHENS GE 2398 EXP. 06/30/25 GEOTECHNICAL STATE OF CALIFORNIA
Mike Jewett Engineering Geologist No. 2610 (Expires 1/31/25)	Scott A. Stephens Geotechnical Engineer No. 2398 (Expires 6/30/25)

1360 Redwood Way, Suite B ■ Petaluma, California 94954 ■ T (707) 765-6140

MILLER PACIFIC
ENGINEERING GROUP

GEOTECHNICAL INVESTIGATION
NEW OFFICE DEVELOPMENT
270 ELM ROAD
BOLINAS, CALIFORNIA

<u>TABLE OF CONTENTS</u>

1.0 INTRODUCTION 1
2.0 PROJECT DESCRIPTION 1
3.0 SITE CONDITIONS 1
3.1 Regional Geology 2
3.2 Seismicity 2
3.2.1 Active Faults in the Region 2
3.2.2 Historic Fault Activity 2
3.2.3 Probability of Future Earthquakes 2
3.3 Surface Conditions 3
3.4 Field Exploration and Laboratory Testing 3
3.5 Subsurface Conditions and Groundwater 4
4.0 GEOLOGIC HAZARDS EVALUATION 5
4.1 Fault Surface Rupture 5
4.2 Seismic Shaking 5
4.3 Liquefaction Potential and Related Impacts 7
4.4 Seismically Induced Ground Settlement 8
4.5 Lurching and Ground Cracking 8
4.6 Erosion 8
4.7 Seiche and Tsunami 9
4.8 Flooding 9
4.9 Expansive Soil 9
4.10 Consolidation Settlement/Subsidence 9
4.11 Slope Instability/Landsliding 10
4.12 Radon-222 Gas 10
4.13 Naturally Occurring Asbestos (NOA) 11
5.0 CONCLUSIONS AND RECOMMENDATIONS 12
5.1 Seismic Design 12
5.2 Site Preparation and Grading 13
5.2.1 Surface Preparation 13
5.2.2 Excavations 13
5.2.3 Fill Materials 14
5.2.4 Fill Placement and Compaction 14
5.3 Foundation Design 14
5.4 Underpinning Considerations 16
5.5 Site and Foundation Drainage 16
5.6 Concrete Slabs-On-Grade 17
5.7 Utility Trench Excavations and Backfills 17
5.8 Asphalt-Concrete Pavements 18
6.0 SUPPLEMENTAL GEOTECHNICAL SERVICES 19
7.0 LIMITATIONS 19
8.0 LIST OF REFERENCES 20

MILLER PACIFIC
ENGINEERING GROUP

FIGURES

Site Location Map ....Figure 1
Site Plan 2
Regional Geologic Map...3
Active Fault Map 4
Historic Fault Map..5
Liquefaction Susceptibility Map 6
Zone of Influence 7

APPENDIX A – SUBSURFACE EXPLORATION AND LABORATORY TESTING

1.0 INTRODUCTION

This report summarizes our Geotechnical Investigation for the planned new office development at 270 Elm Road in Bolinas, California. A Site Location Map is shown on Figure 1. Our services have been provided in accordance with our Agreement dated April 4, 2023. The purpose of our Phase 1 services is to evaluate site geologic conditions and provide geotechnical recommendations and criteria for use in project design and construction.

The scope of our Phase 1 services is described in our proposal letter dated April 4, 2023, and includes the following:

- Summary of regional and local geologic conditions.
- Summary of existing conditions and reconnaissance observations.
- Summary of subsurface exploration and laboratory testing.
- Evaluation of relevant geologic hazards and development of conceptual mitigation measures, including seismic shaking, settlement, expansive soils, flooding, and other hazards.
- Development of 2022 CBC seismic design criteria.
- Criteria for site grading, including excavation, new fill quality, and compaction.
- Geotechnical design criteria for new foundations.
- Criteria and recommendations for interior and exterior concrete slabs-on-grade and moisture vapor barriers.
- Recommendations for geotechnical site drainage.
- Utility trenches backfill criteria.
- Recommendations for new asphalt pavements; and
- Preparation of this report summarizing our findings.

2.0 PROJECT DESCRIPTION

We understand the project generally includes replacing the current office structure at the site with a multi-use development. The existing Bolinas Community Public Utility District (BCPUD) maintenance yard located in the southeast corner of the property will remain. We understand that many project details, including the number of structures, number of stories, framing type, etc. have yet to be determined. From our discussion and review of preliminary site concepts, it is understood that the new development likely consist of up to two, one- or two-story structures occupying a relatively level building envelope in the southern part of the property that overlaps the existing building footprint. Ancillary improvements will likely include a new septic system northwest of the structures, new underground utilities, new exterior flatwork and paving, and other “typical” items.

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. The regional basement rock consists of 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.

3.1 Regional Geology

Regional geologic mapping (Clark and Brabb, 1997) indicates that the project area is underlain by Upper Miocene-age (5.3- to 11.6-million years old) Santa Cruz mudstone (map symbol Tsc) consisting of thin- to thick-bedded and faintly laminated olive-gray to pale-yellowish-brown siliceous mudstone. Quaternary-age terrace deposits and alluvium are mapped along the west-facing bluffs northwest of the site. Terrace deposits (map symbol Qt) are typically composed of weakly consolidated and variably sorted sand, silt, and gravel deposited on stream- and wave-cut surfaces while alluvium (map symbol Qal) is described as poorly consolidated and poorly sorted clay, silt, sand, and gravel deposited within stream and valley floors. A Regional Geologic Map is shown on Figure 3.

3.2 Seismicity

3.2.1 Active Faults in the Region

The project site is located within a seismically active region that includes the Central and Northern Coast Mountain Ranges. Several active faults are present in the area including the San Andreas, San Gregorio, Hayward/Rodgers Creek, among others. An “active” fault is defined as one that shows displacement within the last 11,000 years and, therefore, is considered more likely to generate a future earthquake than a fault that shows no evidence of recent rupture. The California Department of Conservation, California Geologic Survey, formerly the Division of Mines and Geology, has mapped various active and inactive faults throughout California. The faults located near the project site are shown on the Active Fault Map, Figure 4. The San Andreas and San Gregorio Faults are the nearest known active faults to the site, located approximately 1.0-mile and 1.1-miles northeast of the site, respectively.

3.2.2 Historic Fault Activity

Numerous earthquakes have occurred in the region within historic times. Earthquakes (magnitude 2.0 and greater) that have occurred in the San Francisco Bay Area since 1985 have been plotted on a map shown on 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. Historical records do not directly indicate either the maximum credible earthquake or the probability of such a future event. To evaluate earthquake probabilities in California, the USGS has assembled a group of researchers into the “Working Group on California Earthquake Probabilities” (USGS 2003, 2008; Field et al 2015) to estimate the probabilities of earthquakes on active faults. These studies have been published cooperatively by the USGS, CGS, and Southern California Earthquake Center (SCEC) as the Uniform California Earthquake Rupture Forecast, Versions 1, 2, and 3 (aka UCERF, UCERF2, and UCERF3, respectively). In these studies, potential seismic sources were analyzed considering fault geometry, geologic slip rates, geodetic strain rates, historic activity, micro-seismicity, and other factors to arrive at estimates of earthquakes of various magnitudes on a variety of faults in California.

Conclusions from the most recent UCERF3 and the USGS (Aagard, et al 2016) indicate the highest probability of an M>6.7 earthquake on any of the active faults in the San Francisco Bay region by 2043 is assigned to the Hayward-Rodgers Creek Fault system, located about 31.3-kilometers northeast of the site, at 33%. The San Andreas Fault, located about 1.6-kilometers 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

As shown on Figure 2, the project site consists of an approximately 2.2-acre parcel located on the northwest side of Elm Road. The parcel is bounded by Elm Road to the southeast, an unnamed creek/drainage to the northwest, and rural single-family residences to the northeast and southwest. The project site is currently developed with an existing two-story office/community building within the southeast half of the parcel as well as an existing maintenance yard in the southeast corner of the property. A large gravel parking lot is located adjacent to Elm Road, along the southeast side of the existing office building. The site is gently sloping with elevations within the development area ranging from +/- 160 feet above sea level at the top of the slope (northwest of the existing office building) to about +/- 170 feet above sea level along the southeastern property line. Descending slopes north of the proposed development area are inclined at roughly 4:1 (horizontal: vertical)

During our site reconnaissance, we observed that the parcel generally exposes silty to sandy soils at the surface. We noted that the existing building appears to have performed well despite its age and we did not observe any significant geotechnical issues during our reconnaissance. The northern portion of the project site is currently heavily vegetated with native ground cover and bushes.

3.4 Field Exploration and Laboratory Testing

We explored subsurface conditions on July 28th, 2023 with four soil borings at the locations shown on Figure 2. The borings were excavated to depths between 9.8- and 16.5-feet below the ground surface utilizing a truck-mounted, hydraulic-powered drill rig equipped with 6.0-inch solid-stem, continuous flight augers. Soil and rock materials encountered were examined and logged by our Geologist, and select samples were obtained for laboratory testing of pertinent engineering properties. Brief descriptions of the terms and methodology used in classifying earth materials are shown on the attached Soil and Rock Classification Charts, Figures A-1 and A-2, respectively. Our exploratory boring logs are presented on Figures A-3 through A-6.

Laboratory testing of relatively undisturbed samples from our exploratory borings included determination of moisture content, dry density, unconfined compressive strength, and percent passing the #200 sieve in general accordance with applicable ASTM standards. The results of our laboratory testing are presented on the Boring Logs, Figures A-3 through A-6. The laboratory testing program is also described in greater detail in Appendix A.

3.5 Subsurface Conditions and Groundwater

The results of our subsurface exploration generally confirm the regionally mapped conditions. Boring 1, located at the west end of the gravel parking lot, encountered 16-inces of gravel road base over loose to medium dense, silty sand and medium stiff to stiff, sandy clay terrace deposits to a depth of 8.5-feet below ground surface. The terrace deposits were underlain by completely weathered mudstone bedrock to the maximum explored depth of 9.8-feet.

Boring 2, located at the northwestern corner of the existing office building, encountered 8-feet of medium dense, silty sand terrace deposits over completely weathered mudstone bedrock to the maximum explored depth of 10-feet below ground surface.

Boring 3, located at the northwest corner of the existing maintenance yard, encountered 16-inches of gravel road base over about 5-feet of loose, silty sand and gravel fill soils which were underlain by 3-feet of loose, silty sand terrace deposits to a depth of 8-feet below ground surface. The surficial soils were underlain by completely weathered mudstone bedrock to the maximum explored depth of 16.5-feet below ground surface.

Boring 4, located at the east end of the gravel parking lot, encountered 12-inches of gravel road base over loose to medium dense, silty sand and medium stiff, sandy clay terrace deposits to a depth of about 9-feet below ground surface. The terrace deposits were underlain by completely weathered mudstone bedrock to the maximum explored depth of 13.5-feet below ground surface.

Groundwater was encountered in Boring 1 at 7-feet below ground surface. However, since the boring was not left open for an extended period, a stabilized depth to groundwater may not have been observed. Based on our experience with nearby sites underlain by similar geologic conditions, groundwater should generally be expected to exist near the soil-rock interface. However, groundwater will fluctuate seasonally, and seepage may be near the ground surface during the winter and springtime or after periods of heavy rainfall. For the purposes of liquefaction analysis and project design, we estimate that the highest historic groundwater elevation is about 5 feet below the ground surface.

4.0 GEOLOGIC HAZARDS EVALUATION

The principal geologic hazards which could potentially affect the project site are strong seismic shaking from future earthquakes in the San Francisco Bay Region, liquefaction, and settlement. Other hazards, such as fault rupture, erosion, expansive soils, and others, are not considered significant at the site. A more detailed discussion of each geologic hazard considered, their anticipated impacts, and recommended mitigation measures are discussed below.

4.1 Fault Surface Rupture

Under the Alquist-Priolo Earthquake Fault Zoning Act, the California Geological Survey (CDMG)/California Geologic Survey (CGS) (1972, 2000) produced 1:24,000 scale maps showing all known active faults and defining zones within which special fault studies are required. Based on currently available published geologic information, the project site is not located within the Alquist-Priolo Earthquake Fault Zone (CGS, 2000) nor is within the City's General Plan Fault Rupture Hazard Zone. Therefore, we judge the potential for fault surface rupture at the project site is low.

Evaluation: *No significant impact.*
Recommendations: *No special engineering measures are required.*

4.2 Seismic Shaking

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

Deterministic methods use empirical attenuation relations to 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 A.

TABLE A
DETERMINISTIC PEAK GROUND ACCELERATION
New Office Development
270 Elm Road
Bolinas, California

Fault Rupture Scenario	Fault Distance[1]	Moment Magnitude[2]	Median PGA[3,4]	+1σ PGA[3,4]
San Andreas Fault	1.6 km	8.0	0.57 g	1.01 g
San Gregorio	1.7 km	7.4	0.54 g	0.97 g
Hayward/Rodgers Creek	31.3 km	7.6	0.16 g	0.28 g
West Napa	49.7 km	7.0	0.07 g	0.13 g

Notes:

1. Values derived from USGS Quaternary Fault and Fold Database, https://www.usgs.gov/programs/earthquake-hazards/faults, accessed 2023.
2. Values determined using USGS Earthquake Scenario Map (BSSC 2014), accessed 2023.
3. Values determined using V_{S30} = 560 m/s for Site Class "C".
4. Abrahamson, Silva and Kamai (2014); Boore, Stewart, Seyhan and Atkinson (2014); Campbell and Borzognia (2014); and Chiou and Youngs (2014).

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 California Building Code (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 close proximity, 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 special engineering measures.*

Recommendations: *Mitigation measures should include designing the structure and foundations in accordance with the most recent version of the California Building Code. Recommended seismic coefficients are provided in Section 5.1 of this report. Flexible utility connections should be considered to reduce the risk of damage or breakage during very strong seismic ground shaking.*

4.3 Liquefaction Potential and Related Impacts

Liquefaction refers to the sudden, temporary loss of soil shear strength during strong ground shaking. Liquefaction-related phenomena include liquefaction-induced settlement, flow failure, and lateral spreading. These phenomena can occur where there are saturated, loose, granular deposits. Recent advances in liquefaction studies indicate that liquefaction can occur in granular materials with a high, 30% or greater, fines content (soil particles that pass the #200 sieve), provided the fines exhibit a plasticity less than 7.

Regional mapping (ABAG, 2023) indicates the site lies in a zone of "very low" liquefaction susceptibility, as shown on Figure 6. However, deposits of loose to medium-dense granular soils, including thin surficial fills and native terrace deposits, were observed during our exploration within the upper 8.5- to 9-feet of the subsurface.

To evaluate soil liquefaction, the seismic energy from an earthquake is compared with the ability of the soil to resist pore pressure generation, known as the Cyclic Resistance Ratio (CRR). Earthquake energy is termed the cyclic stress ratio (CSR) and is a function of the maximum considered earthquake peak ground acceleration (PGA) and depth. Soil resistance to liquefaction is based on its relative density, and the amount and plasticity of the fines (silts and clays). The relative density of cohesionless soil is correlated with the Standard Penetration Test (SPT) blow count data measured in the field and corrected for hammer efficiency, overburden, and percent fines to determine the $(N_1)_{60,CS}$ value.

We analyzed the potential for liquefaction utilizing our laboratory test data, normalized SPT blow count data, and the procedures outlined by Idriss and Boulanger (2008, 2010 & 2014). Our analyses assumed a groundwater elevation of 5-feet below the ground surface and considered a magnitude 8.0 earthquake producing a PGA of 1.12-g, which corresponds to the PGA_M value as defined by ASCE 7-16.

The results of our analyses indicate that the silty sand terrace deposits encountered at depths between about 5- and 9-feet below the ground surface are liquefiable during strong ground shaking. Analyses further indicate that up to about an inch of liquefaction-induced settlement may be possible.

Based on the results of our subsurface exploration, laboratory testing, and engineering analyses, it is our professional opinion that there is a moderate potential for liquefaction damage at the project site.

Evaluation: *Less than significant with special engineering measures.*

Recommendations: *Foundation systems should be designed to withstand up to 1.0-inches of total and 0.5-inch of differential settlement over a 30-foot span. Further discussion of foundation systems and design criteria to mitigate the potential effects of liquefaction are provided in Section 5 of this report. Additionally, flexible utility connections should be required to allow for movement without rupturing if liquefaction does occur.*

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.

Granular soils were observed within the upper 5-feet of our borings, above the historic high groundwater level. We utilized the procedure outlined by Tokimatsu and Seed (1984) to predict the magnitude of potential seismic settlements. Our analysis considered an M=8.0 earthquake generating a ground acceleration of 1.12g, which corresponds to the PGA_m value defined by ASCE 7-16. The results of analyses indicate a few tenths of an inch of seismically induced settlement may be expected. Therefore, we judge the risk of seismically induced ground settlement at the project site is generally low.

Evaluation: *Less than significant with special engineering measures.*

Recommendations: *Provided that foundations are designed to accommodate estimated liquefaction settlements as discussed above and that subgrade soils are prepared and compacted as recommended in Section 5, we judge no special engineering measures are required to mitigate the potential for seismic densification.*

4.5 Lurching and Ground Cracking

Lurching and associated ground cracking can occur during strong ground shaking. The ground cracking generally occurs along the tops of slopes where stiff soils are underlain by soft deposits or along steep slopes or channel banks. The site consists of gently sloping terrain and subsurface soils and rock generally grade denser with depth; however, near surface soils are relatively weak and will be prone to lurching when saturated. Therefore, the risk of lurching and ground cracking impacting the proposed improvements is low to moderate.

Evaluation: *Less than significant with special engineering measures.*

Recommendations: *Structures supported on shallow foundations should be set back a minimum of 15-feet from the crest of the slope. Where structures are sited within 15-feet of descending slopes, they should be supported on deep foundations embedded into firm bedrock.*

4.6 Erosion

Sandy soils on moderate slopes or clayey soils on steep slopes are susceptible to erosion when exposed to concentrated water runoff. The project site is gently sloping and surficial soils consist of clayey sand terrace deposits and gravel fill soils. We judge there is a moderate risk of erosion where soils are disturbed and exposed during construction.

Evaluation: *Less than significant with special engineering measures.*

Recommendations: *The project Civil Engineer of Architect is typically responsible for designing the site drainage system. Erosion control during and after construction could conform to a site-specific Stormwater Pollution Prevention Plan (SWPPP) prepared by the project Civil Engineer, the guidelines of the most recent edition of the Marin County Stormwater Pollution Prevention Program (2015), or superseding local requirements, as appropriate.*

4.7 Seiche and Tsunami

Seiche and tsunamis are short duration, earthquake-generated water waves in large, enclosed bodies of water and the open ocean, respectively. The extent and severity of a seiche or tsunami would be dependent upon ground motions and fault offset from nearby active faults. The project site is not mapped within a tsunami inundation zone (ABAG, 2023) and lies at elevations above +170-feet. Therefore, we judge the risk of inundation by seiche and tsunami waves is remote.

Evaluation: *No significant impact.*
Recommendations: *No special engineering measures are required.*

4.8 Flooding

The project site is not mapped within an ABAG (Association of Bay Area Governments) 100- or 500-year flood zone and lies at elevations above +170-feet. Therefore, we judge the risk of large-scale flooding at the site is low.

Evaluation: *Less than significant.*
Recommendations: *The project Civil Engineer should consider the potential for ponding of water and small-scale flooding during the design of site grades and drainage systems. Additional recommendations regarding geotechnical site drainage are provided in Section 5 of this report.*

4.9 Expansive Soil

Expansive soils will shrink and swell with fluctuations in moisture content and are capable of exerting significant expansion pressures on building foundations, interior floor slabs, and exterior flatwork. Distress from expansive soil movement can include cracking of brittle wall coverings (stucco, plaster, drywall, etc.), racked door and/or window frames, and uneven floors and cracked slabs. Flatwork, pavements, and concrete slabs-on-grade are particularly vulnerable to distress due to their low bearing pressures. Based on our subsurface exploration, the surficial soils consist of loose to medium-dense silty sands and gravels which do not exhibit significant expansive potential. Therefore, the risk of expansive soil affecting the proposed improvements is low.

Evaluation: *No significant impact.*
Recommendations: *No special engineering measures are required.*

4.10 Consolidation Settlement/Subsidence

Significant settlement can occur when new loads are placed at sites due to consolidation of soft compressible clays (i.e., Bay Mud) or compression of loose granular soils. The rate and magnitude of potential settlements are dependent on the new loads that are applied, the stress history of subsurface soils, the presence of drainage layers, the thickness and compressibility of subsurface materials, and other factors. Differential settlement may occur where structures span cut/fill transitions or other variable support conditions.

Based on our subsurface exploration, the site is generally underlain by about 8.5- to 9-feet of loose to medium dense silty sand and medium stiff sandy clay terrace deposits over mudstone bedrock. While significant total settlements are generally not expected at the site, we judge there is low to moderate potential for minor differential settlements (likely less than a half-inch) where moderately- to heavily loaded structures span transitions from weak silty sand terrace deposits and fill soils to locally stiffer soils.

Evaluation:	*Less than significant with special engineering measures.*
Recommendations:	*Provided that foundations are designed to accommodate estimated liquefaction settlements as discussed above and that subgrade soils are prepared and compacted as recommended in Section 5, we judge no special engineering measures are required to mitigate the potential for seismic densification.*

4.11 Slope Instability/Landsliding

Slope instability generally occurs on relatively steep slopes and/or on slopes underlain by weak materials. The southern, developed part of the project site is gently sloping and the risk of slope instability impacting the present building envelope is generally low. The undeveloped, sloping northern portion of the property has moderate potential for localized slope instability under static conditions and a moderate to high potential during a strong seismic event.

Evaluation:	*Less than significant with special engineering measures.*
Recommendations:	*Structures supported on shallow foundations should be set back a minimum of 15-feet from the crest of the slope. Where structures are sited within 15-feet of descending slopes, they should be supported on deep foundations embedded into firm bedrock.*

4.12 Radon-222 Gas

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

The project site is located in Marin County, California, which is mapped in radon gas Zone 3 by the United States Environmental Protection Agency (USEPA, 2023). Zone 3 is classified by the EPA as exhibiting a “low” potential for Radon-222 gas with average predicted indoor screening levels less than 2 pCi/L. Therefore, the potential for hazardous levels of radon at the project site is generally low.

Evaluation:	*No significant impact.*
Recommendations:	*No special engineering measures are required.*

4.13 Naturally Occurring Asbestos (NOA)

Naturally occurring asbestos is commonly found in association with serpentinite and associated ultramafic rock types. These rocks are a major constituent of the Franciscan Complex, which underlies vast portions of the greater San Francisco Bay Area. The site is underlain by terrace deposits and fill soils over mudstone bedrock, and while it lies in a region dominated in part by Franciscan Complex bedrock, no such bedrock is mapped west of the San Andreas Fault at Bolinas. As such, the risk of naturally occurring asbestos being encountered at the site is low.

Evaluation: *No significant impact.*
Recommendations: *No special engineering measures are required.*

5.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our subsurface exploration and experience with similar projects in the Bolinas area, it is our opinion that the planned improvements are feasible from a geologic and geotechnical standpoint. The primary geotechnical concerns for the project, aside from providing uniform foundation support and adequate seismic design for the new structures, will include designing foundations to accommodate minor static and post-seismic (liquefaction-induced) settlements. Additional discussion and recommendations addressing these, and other considerations, are presented in the following sections.

5.1 Seismic Design

The project site is located in a seismically active area. Therefore, the structure should be designed in conformance with the seismic provisions of the California Building Code (CBC, 2022) and ASCE 7-16 to mitigate the potential effects of strong seismic ground shaking to the proposed structures. The goal of the building code is protection of life and safety, some structural damage may still occur during strong ground shaking. Based on the interpreted subsurface conditions and closest fault type and distance, we recommend the seismic coefficients and site values shown in Table B below for use to calculate the design base shear of the new construction.

TABLE B
ASCE 7-16 / 2022 CBC FACTORS
New Office Development
270 Elm Road
Bolinas, California

Factor Name	Coefficient	2022 CBC[1] Site Specific Value
Site Class[2]	$S_{A,B,C,D,E,\text{ or }F}$	S_C
Site Coefficient	F_a	1.2
Site Coefficient	F_v	1.4
Spectral Acc. (short)	S_S	2.231 g
Spectral Acc. (1-sec)	S_1	0.929 g
Spectral Response (short)	S_{MS}	2.677 g
Spectral Response (1-sec)	S_{M1}	1.301 g
Design Spectral Response (short)	S_{DS}	1.785 g
Design Spectral Response (1-sec)	S_{D1}	0.868 g
Seismic Design Category	A – F	D

Notes:

1. 2022 CBC Parameters based on ASCE 7-16
2. Site Class C Description: Very Dense Soil and Soft Rock with shear wave velocities between 1,200 & 2,500 feet per second, Standard Penetration Test N values greater than 50, and undrained shear strength greater than 2,000 psf within the upper 30-meters.

Reference: SEAOC/OSHPD Seismic Design Maps Web Tool (2023), www.seismicmaps.org

5.2 Site Preparation and Grading

Minor to moderate grading may be required to construct the proposed improvements. We anticipate site grading will be limited to foundation excavations and minor “cuts and fills” to develop level building pads. Any site preparation and grading should be performed in accordance with the following recommendations.

5.2.1 Surface Preparation

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

Where fills or other structural improvements are planned on level ground, the subgrade soils should be scarified to a depth of about eight inches, moisture conditioned above the optimum moisture content, and compacted to at least 90% relative compaction and to a firm and unyielding surface. Relative compaction should be increased to a minimum of 95% where new asphalt pavements are planned. If soft, wet, or otherwise unsuitable materials are encountered at the subgrade elevation during construction, we will provide supplemental recommendations/field directives to address the specific condition. Relative compaction, maximum dry density, and optimum moisture content of fill materials should be determined in accordance with ASTM Test Method D 1557, "Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using a 10-lb. Rammer and 18-in. Drop."

5.2.2 Excavations

Site excavations for new foundations, utilities, and other improvements will generally encounter 8.5- to 9-feet of loose to medium dense silty sand and medium stiff sandy clay over mudstone bedrock. Based on our exploration and laboratory testing, we judge that most onsite excavations can be reasonably accomplished with conventional equipment, such as small excavators and limited-access drilling equipment.

Per the California Occupational Safety and Health Administration (Cal/OSHA), trench excavations having a depth of five feet or more which will be entered by workers must be sloped, braced, or shored to protect workers from potential collapse. Cal/OSHA dictates allowable slope configurations and minimum shoring requirements based on categorized soil types. Based on our subsurface exploration and laboratory testing, onsite fill, and terrace deposit soils should be considered Type “C” while underlying bedrock may be considered “Type B”.

The Contractor should be responsible for site safety and should select an appropriate shoring system(s) for the anticipated site conditions. The chosen system(s) should be capable of providing immediate support to the sides of the excavation in order to minimize the amount of time the excavation is unsupported.

5.2.3 Fill Materials

Onsite excavations are expected to yield low to moderately-plastic silty sand mixtures, which should be suitable for use as structural fill. All native or imported fill material should consist of soil and rock mixtures that: (1) are free of organic material, (2) have a Liquid Limit less than 40 and a Plasticity Index of less than 20, and (3), have a maximum particle size of 6 inches. Any imported fill material needs to be tested to determine its suitability for use as fill material.

5.2.4 Fill Placement and Compaction

Following subgrade preparation in accordance with Section 5.2.1, fill materials should be conditioned to near the optimum moisture content, placed in loose horizontal lifts not exceeding 8-inches in thickness, and be compacted to a minimum of 90% relative compaction. Where asphalt pavements or other vehicle-loaded areas are planned, compaction should be increased to 95% minimum. Compaction may be reduced to 85% minimum in landscape areas where no new structures are planned.

5.3 Foundation Design

As previously discussed, careful consideration should be given to the high expected ground accelerations and potential static and post-seismic settlements during the design of new foundations. Provided that the site is prepared in conformance with the recommendations in Section 5.2 and, and that potential post-seismic settlements described above are acceptable, we judge a rigid shallow foundation, such as a thick, heavily reinforced mat slab, a “waffle” slab (consisting of continuous, interconnected footings), or a post-tensioned slab, is suitable for new structures at the site. The foundation should be designed to accommodate up to 1-inch of total and 0.5-inches of different settlement over a 30 foot zone. In the event significant post-seismic settlements were to occur, the rigid shallow foundation could be re-leveled via jet- or compaction-grouting, by underpinning with helical or pipe piles, or by other means. Shallow foundation design criteria are presented in Table C below.

TABLE C
RIGID SHALLOW FOUNDATION DESIGN CRITERIA
New Office Development
270 Elm Road
Bolinas, California

Shallow Interconnected "Waffle-Grid" Footings

Minimum width:[1]	18 inches
Minimum depth:[2]	18 inches
Allowable bearing capacity:[3,4]	
Native Soils:	2,000 psf
Base friction coefficient :	0.30
Lateral passive resistance:[5]	
Native Soils:	300 pcf

Rigid Mat or Post-Tensioned Slab:

Modulus of subgrade reaction, k_s	100 pci
Minimum thickness at edge of slab:[6]	12 inches
Maximum unsupported interior span:[7]	15 feet
Maximum unsupported edge (corner) cantilever:[7]	7 feet
Edge moisture variation (e_m) – Center Lift	15 feet
Edge moisture variation (e_m) – Edge Lift	7 feet
Differential soil movement (y_m) – Center Lift	1.0 inch
Differential soil movement (y_m) – Edge Lift	1.0 inch

Notes:
(1) Size foundations to maintain uniform bearing pressures, i.e., size footing widths to design loads instead of uniform foundation widths.
(2) Footings may need to be deeper if the Structural Engineer determines additional rigidity is required to evenly spread column loads.
(3) Dead plus live loads. May increase by 1/3 for total design loads, including wind and seismic.
(4) Foundation to bear on prepared subgrade as described in Section 5 of this report.
(5) Equivalent fluid pressure. Ignore upper 6-inches unless confined by asphalt or concrete.
(6) Actual thickness, load distribution, and unsupported spans must be determined by Structural Engineer to reduce deformations to acceptable levels.
(7) Assumes rigid slab behavior with idealized fixed end conditions.

If the anticipated building settlements are not acceptable, a deep foundation system which derives its support from weathered bedrock at depths below 9 feet will be required. Deep foundation options could include "traditional" cast-in-drilled-hole (CIDH) concrete piers or drilled micro-piles. Helical piles are not ideal for this site due to their relative lack of lateral stiffness and the very high horizontal ground accelerations expected during future earthquakes. Micro-piles would entail drilling small-diameter (typically 6- to 8-inch diameter) shafts utilizing steel casing with a "cutting" tip to a sufficient depth in firm mudstone bedrock. The casing is left in place and the annular space between the soil and steel and within the micro-pile is backfilled with grout/concrete. A steel thread bar or rebar cage may be inserted into the steel casing prior to placing grout to increase the lateral capacity. We anticipate micro-piles would develop capacities on the order of about 20-kips at depths of 20- to 30-feet depending on the diameter utilized. Recommended foundation design criteria for deep foundations are presented in Table D.

TABLE D
DEEP FOUNDATION DESIGN CRITERIA
New Office Development
270 Elm Road
Bolinas, California

Minimum Diameter:		18 inches
Minimum Embedment into Bedrock:		5 feet
Skin Friction[1]:	Static	Seismic
Native Soils:	500 psf	250 psf
Bedrock:	1,500 psf	1,500 psf
Lateral Passive Resistance [2,3,4]:	Level Ground	2:1 Slope
Native Soils:	250 pcf	Neglect
Weathered Bedrock:	450 pcf	350 pcf

Notes:

(1) Uplift resistance is equal to 80% of the total skin friction. Ignore upper 3-feet for uplift.
(2) Equivalent Fluid Pressure, not to exceed 10 times value in psf.
(3) Apply values over effective width of 2 pier diameters.
(4) Values may be interpolated for intermediate slopes flatter than 2:1.
(5) Micropiles and rock anchors should be designed for load-testing up to 150% of the design load. Load testing to be performed in general accordance with the procedures recommended by the Post-Tensioning Institute.

5.4 Underpinning Considerations

If and where excavations for new foundations are performed within the “zone of influence” of existing foundation elements which will remain, those elements will need to be underpinned to avoid undermining and potential damage. The “zone of influence” is described as the region below a 1.5:1 (horizontal: vertical) line projected downward from the existing foundation elements, as illustrated in Figure 7. Underpinning may be accomplished by any of several means, including hand- or machine-dug footings, drilled piers, helical piles, or other systems. All underpinning elements should be designed in accordance with the applicable recommendations in Section 5.3.

5.5 Site and Foundation Drainage

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

5.6 Concrete Slabs-On-Grade

Concrete slab-on-grade floors for the proposed structures, if utilized, may be poured monolithically or independently from the foundation system, at the Structural Engineer's discretion. We generally recommend a 5-inch-thick interior slab section that is reinforced with bars (not mesh) for improved performance. Contraction joints should be incorporated in the concrete slab in both directions, no greater than 10 feet on center, and reinforcing bars should extend continuously through the control joints. The upper 8-inches of subgrade beneath any concrete slabs should be scarified and compacted to a minimum of 92 percent relative compaction per ASTM D-1557.

If the interior floor coverings are sensitive to water vapor, a 4-inch layer of clean, free draining, 3/4-inch angular gravel or crushed rock should be placed beneath the interior concrete slabs to form a capillary moisture break. This rock must be placed on a properly moisture conditioned and compacted subgrade that has been approved by the Geotechnical Engineer. A plastic membrane vapor barrier, 10 miles or thicker, should be placed over the compacted base rock. The vapor barrier shall meet the ASTM E 1745 Class A requirements and be installed per ASTM 1643. Eliminating the capillary moisture break and/or plastic vapor barrier may result in excess moisture intrusion through the floor slabs resulting in poor performance of floor coverings, mold growth or other adverse conditions.

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

5.7 Utility Trench Excavations and Backfills

Excavations for utilities will most likely extend into loose to medium dense silty sand and gravel soils. Trench excavations having a depth of five feet or more that will be entered by workers must be sloped, braced, or shored in accordance with current Cal/OSHA regulations. The loose to medium dense granular soils appear to be Type C. All excavations where collapse of excavation sidewall, slope or bottom could result in injury or death of workers, should be evaluated by the contractor's safety officer, and designated competent person prior to entering in accordance with current Cal/OSHA regulations.

Bedding materials for utility pipes should be well graded sand with 90 to 100% of particles passing the No. 4 sieve and no more than 5% finer than the No. 200 sieve. Provide the minimum bedding beneath the pipe in accordance with the manufacturer's recommendation, typically 3 to 6 inches. Trench backfill may consist of on-site soils, moisture conditioned to within 2% of the optimum moisture content, placed in thin lifts and compacted to a minimum of 90% relative compaction. Backfill for trenches within pavement areas should consist of non-expansive granular fill. Use equipment and methods that are suitable for work in confined areas without damaging utility conduits. Where utility lines cross under or through perimeter footings, they should be sealed to reduce moisture intrusion into the areas under the slabs and/or footings.

5.8 Asphalt-Concrete Pavements

New pavements will likely be needed for drive aisles and parking areas. We have calculated preliminary pavement sections in accordance with Caltrans procedures for flexible pavement design using an assumed R-value of 15. We have provided a range of Traffic Indices (TI) from 3 to 5 depending on the expected traffic loads for a twenty-year design life. In general, areas expected to experience loading from heavy vehicles (such as fire lanes, loading dock access roads, trash enclosures, etc.) should be designed using the higher Traffic Index, while parking areas and other lightly loaded areas can utilize a thinner pavement section based on the lower Traffic Index. Preliminary recommended pavement sections are shown in Table E; these should be verified on the basis of supplemental laboratory testing.

TABLE E
PAVEMENT DESIGN CRITERIA
New Office Development
270 Elm Road
Bolinas, California

T.I.	Asphalt Concrete	Aggregate Baserock
3.0	2.0-inches	4.5-inches
4.0	2.5-inches	7.0-inches
5.0	3.0-inches	9.0-inches

Subgrade preparation for asphalt-paved areas should be performed in accordance with the grading recommendations of this report. The base rock should consist of compacted Class 2 Aggregate Base (Caltrans, 2018), be conditioned to near optimum moisture content, placed in lifts no more than six inches thick, and compacted to achieve at least 95 percent relative compaction and a non-yielding surface when proof-rolled with heavy construction equipment. The subgrade should also be maintained near or slightly above optimum moisture content prior to placement of aggregate base rock. Areas of soft or saturated soils encountered during construction should be excavated and replaced with properly moisture conditioned fill or aggregate base.

6.0 SUPPLEMENTAL GEOTECHNICAL SERVICES

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

7.0 LIMITATIONS

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

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

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

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

8.0 LIST OF REFERENCES

Aagard, B.T. et al (2016), “Earthquake Outlook for the San Francisco Bay Region 2014-2043”, United States Geological Survey Fact Sheet 2016-3020, Version 1.1, Revised August 2016.

Abrahamson, Silva and Kamai, “Summary of the ASK14 Ground Motions Relation for Active Crustal Regions,” Earthquake Engineering Research Institute, Spectra Vol 30 Issue 3, August 2014.

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

American Society for Testing and Materials, “2020 Annual book of ASTM Standards, Section 4, Construction, Volume 4.08, Soil and Rock; Dimension Stone; Geosynthetics,” ASTM, Philadelphia, 2020.

Association of Bay Area Governments (ABAG), Geographic Information System, http://quake.abag.ca.gov/mitigation/, 2023.

Boore, Stewart, Seyhan and Atkinson, “NGA-West2 Equations for Predicting PGA, PGV and 5% Damped PSA for Shallow Crustal Earthquakes,” Earthquake Engineering Research Institute, Spectra Vol 30 Issue 3, August 2014.

Boulanger, R.W. & Idriss, I.M. “SPT-Based Liquefaction Triggering Procedures” Department of Civil and Environmental Engineering, College of Engineering, University of California at Davis, UCD/GCM-14/01, April 2014.

California Building Code, 2019 Edition, California Building Standards Commission, Sacramento, California.

California Department of Conservation, California Geologic Survey, “Digital Images of the Alquist Priolo Maps” http://www.conservation.ca.gov/cgs/rghm/ap/Pages/Index.aspx, 2023.

California Department of Transportation (Caltrans), “Corrosion Guidelines Version 1.0,” September 2010.

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

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

Campbell and Borzognia, “NGA-West2 Ground Motion Model for the Average Horizontal Components of PGA, GV and 5% Damped Linea Acceleration Response Spectra,” Earthquake Engineering Research Institute, Spectra Vol 30 Issue 3, August 2014.

Chiou and Youngs, “Update of the Chiou and Youngs NGA Model for the Averafe Horizontal Component of Peak Ground Motion and Response Spectra,” Earthquake Engineering Research Institute, Spectra Vol 30 Issue 3, August 2014.

Clark, J.C. and Brabb, E.E., "Geology of the Point Reyes National Seashore and Vicinity," United States Geological Survey, Open-File Report 97-456, 1997.

Field, E.H., Biasi, G.P., Bird, P., Dawson, T.E., Felzer, K.R., Jackson, D.D., Johnson, K.M., Jordan, T.H., Madden, C., Michael, A.J., Milner, K.R., Page, M.T., Parsons, T., Powers, P.M., Shaw, B.E., Thatcher, W.R., Weldon, R.J., II, and Zeng, Y., 2015, Uniform California earthquake rupture forecast, version 3 (UCERF3) – The 2015 time-independent model: U.S. Open-File Report 2013–1165, 97 p., California Geological Survey Special Report 228, and Southern California Earthquake Center Publication 1792, http://pubs.usgs.gov/of/2013/1165/.

Idriss, I.M. & Boulanger, R.W. "Soil Liquefaction during Earthquakes," Earthquake Engineering Research Institute Monograph 12, 2008.

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.

Jennings, C.W. and Bryant, W.A. (2010), "2010 Fault Activity Map of California", California Department of Conservation, California Geological Survey Geologic Data Map No. 6, http://www.quake.ca.gov/gmaps/FAM/faultactivitymap.html#.

National Oceanic and Atmospheric Administration (NOAA) (2023), National Geophysical Data Center/(NGDC/WDS) Global Historical Tsunami Database, National Geophysical Data Center, Boulder, Colorado, USA, http://www.ngdc.noaa.gov/hazard/tsu_db.shtml, doi:10.7289/V5PN93H7, accessed July 2023.

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

Shahi, K.S. & Baker, J.W., "NGA Models for Ground Motion Directionality" May 2013.

Southern California Earthquake Center (SCEC), "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction Hazards in California," University of Southern California, March 1999.
United States Geologic Survey (USGS), "Documentation for the 2008 Update of the United States National Seismic Hazard Maps," Open-File-Report 2008-1128, 2008.

United States Geological Survey (USGS), "Earthquake Archive Search & URL Builder," http://earthquake.usgs.gov/earthquakes/search/, 2023.

United States Geological Survey (USGS), "Earthquake Probabilities in the San Francisco Bay Region, 2002 to 2031 – A Summary of Finding," The Working Group on California Earthquake Probabilities, Open File Report 99-517, 2003.

United States Geological Survey (USGS), Unified Hazard Tool," https://earthquake.usgs.gov/hazards/interactive/, 2023.

United States Geological Survey (USGS), "U.S. Seismic Design Maps," web application http://geohazards.usgs.gov/designmaps/us/application.php, 2023.

United States Geologic Survey (USGS), “Documentation for the 2008 Update of the United States National Seismic Hazard Maps,” Open-File-Report 2008-1128, 2008.

United States Geological Survey & SC/EC, “The Uniform California Earthquake Rupture Forecast, Version 2 (UCERF 2), 2007 Working Group on California Earthquake Probabilities, USGS Open File Report 2007-1437, CGS Special Report 203, SCEC Contribution #1138, 2008

SITE COORDINATES
LAT. 37.89910°
LON. -122.70434°

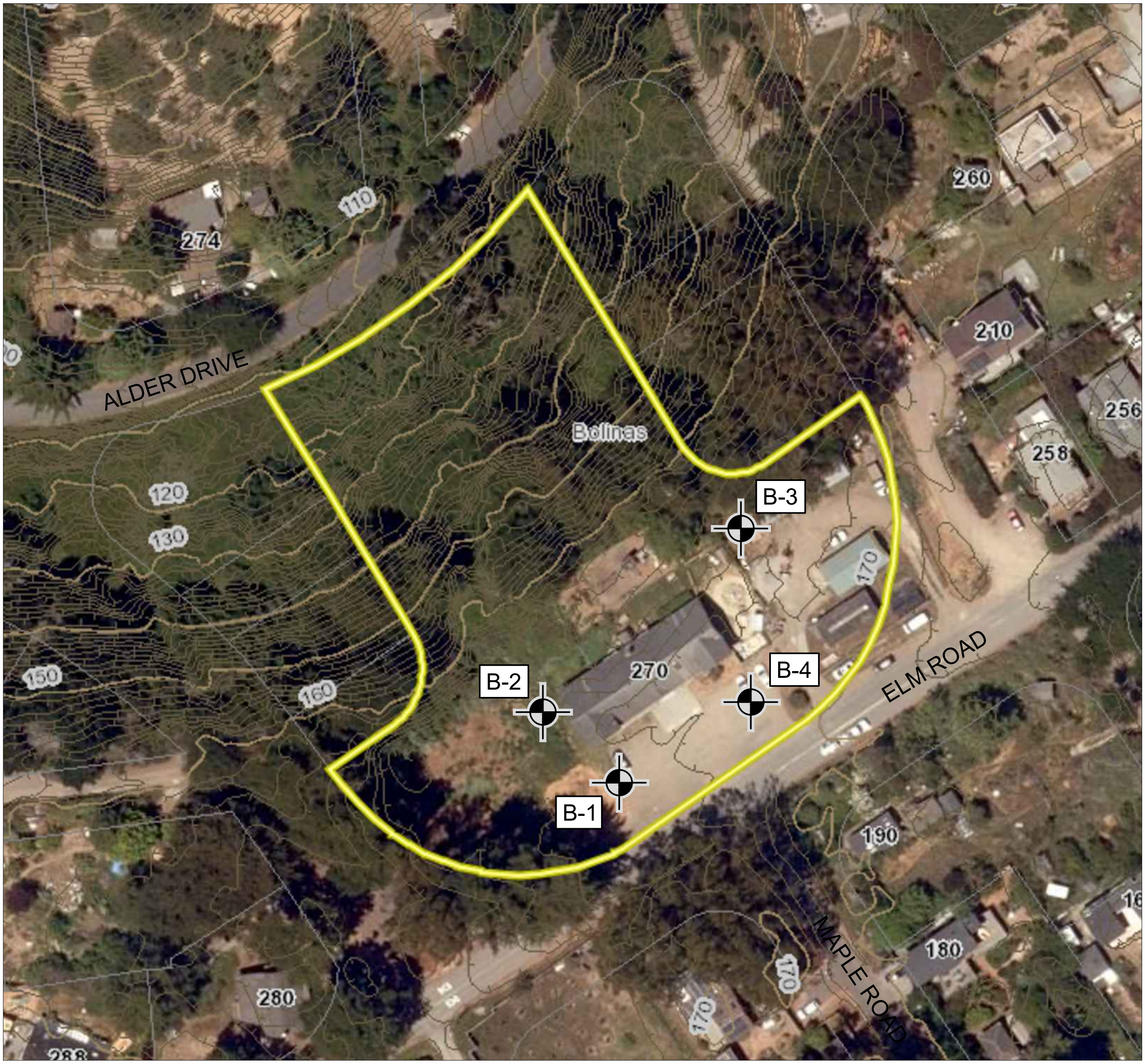
SITE LOCATION

N.T.S.

NORTH

REFERENCE: Google Earth, 2023

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: 3528.001 Figures.dwg		270 Elm Road Bolinas, California Project No. 3528.001 Date: 7/10/2023	Drawn EIC Checked	1 FIGURE



SITE PLAN

SCALE

0 50 100 200 FEET

NORTH

Approximate location of boring completed by MPEG, 2023

REFERENCE: MarinMaps Web Viewer, 2023.

MPEG
MILLER PACIFIC ENGINEERING GROUP

FILENAME: 3528.001 Figures.dwg

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

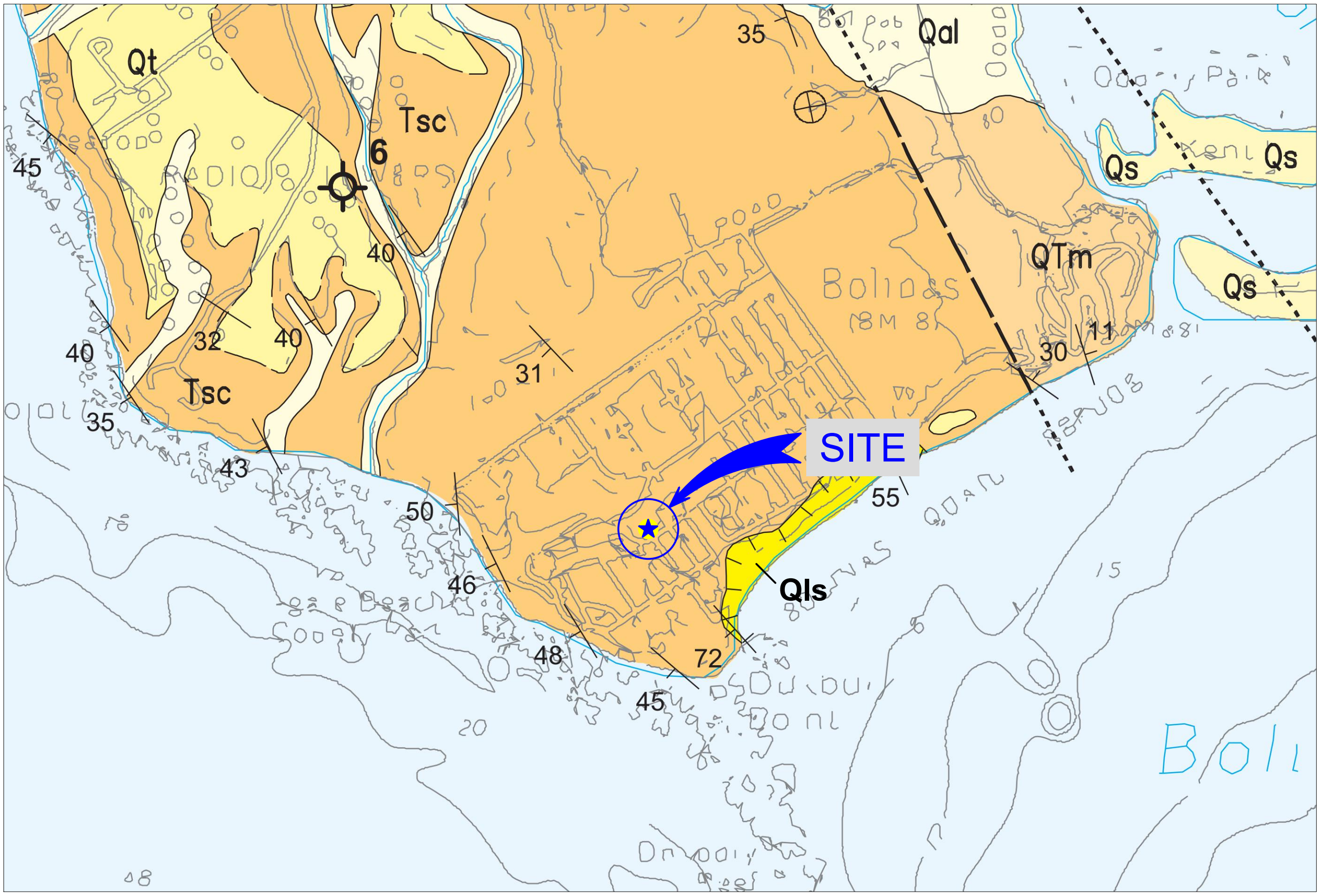
SITE PLAN

270 Elm Road
Bolinas, California

Project No. 3528.001 Date: 7/10/2023

Drawn EIC
Checked

FIGURE 2



REGIONAL GEOLOGIC MAP

NORTH

Qs	**Beach sands** [Holocene] - Discontinuous accumulations of well- to moderately-sorted, fine- to coarse-grained loose sand locally are interspersed with pebble to boulder gravel.
Qal	**Alluvium** [Holocene] - Poorly consolidated, poorly sorted clay, silt, sand, and gravel usually fill stream and valley floors.
Qls	**Landslide deposits** [Pleistocene and Holocene] - Only large slides in the vicinity of Double Point and at Bolinas are shown, where they consist mainly of intact to highly disrupted masses of Santa Cruz Mudstone.
Qt	**Terrace deposits** [Pleistocene] - Discontinuous deposits of weakly consolidated and variably sorted sand, silt, and gravel deposited on stream- and wave-cut surfaces.
Tsc	**Santa Cruz Mudstone** [upper Miocene] - Thin- to thick-bedded and faintly laminated olive-gray to pale-yellowish-brown siliceous mudstone contains thin elongate carbonate concretions.

REFERENCE: Clark, J.C. and Brabb, E.E., (1997) 'Geology of the Point Reyes National Seashore and Vicinity', United States Geological Survey, Open-File Report 97-456, 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

FILENAME: 3528.001 Figures.dwg

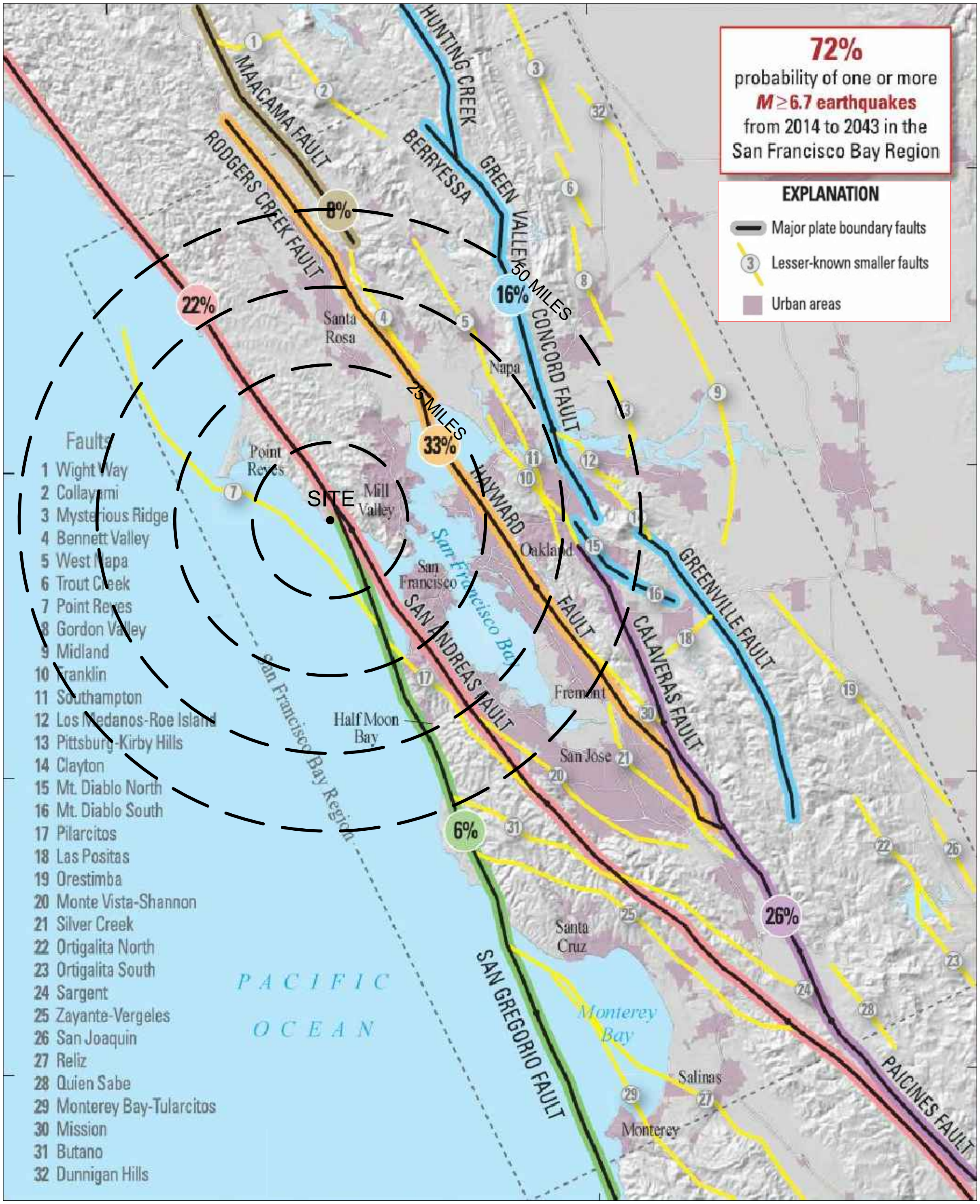
REGIONAL GEOLOGIC MAP

270 Elm Road
Bolinas, California

Project No. 3528.001 Date: 7/10/2023

Drawn EIC
Checked

3
FIGURE



SITE COORDINATES
LAT. 37.89910°
LON. -122.70434°

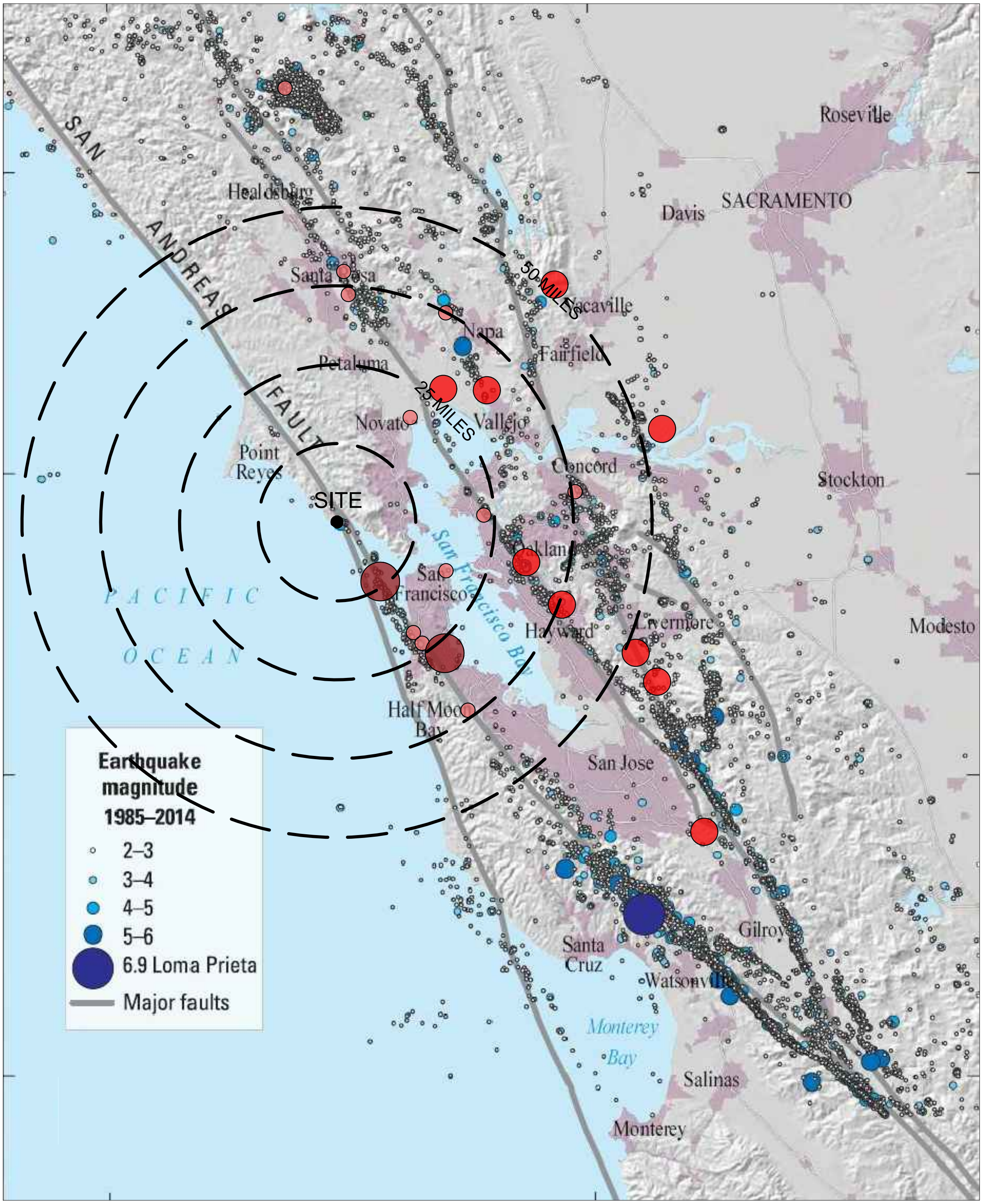
SCALE
0 12.5 25 50 MILES

NORTH

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

MPEG MILLER PACIFIC ENGINEERING GROUP	504 Redwood Blvd. Suite 220 Novato, CA 94947 T 415 / 382-3444 F 415 / 382-3450 www.millerpac.com	ACTIVE FAULT MAP 270 Elm Road Bolinas, California Project No. 3528.001 Date: 7/10/2023	Drawn EIC Checked	4 FIGURE

FILENAME: 3528.001 Figures.dwg



SITE COORDINATES
LAT. 37.89910°
LON. -122.70434°

SCALE
0 12.5 25 50 MILES

NORTH

LEGEND & DATA SOURCE:

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

Large circles indicate earthquakes M>7.0, medium circles indicate 6.0<M<7.0 and small circles indicate 5.0<M<6.0 .U.S. Geological Survey, Earthquake Catalog Search, https://earthquake.usgs.gov/earthquakes/search/. Earthquakes between 1830 and 2021.

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

FILENAME: 3528.001 Figures.dwg

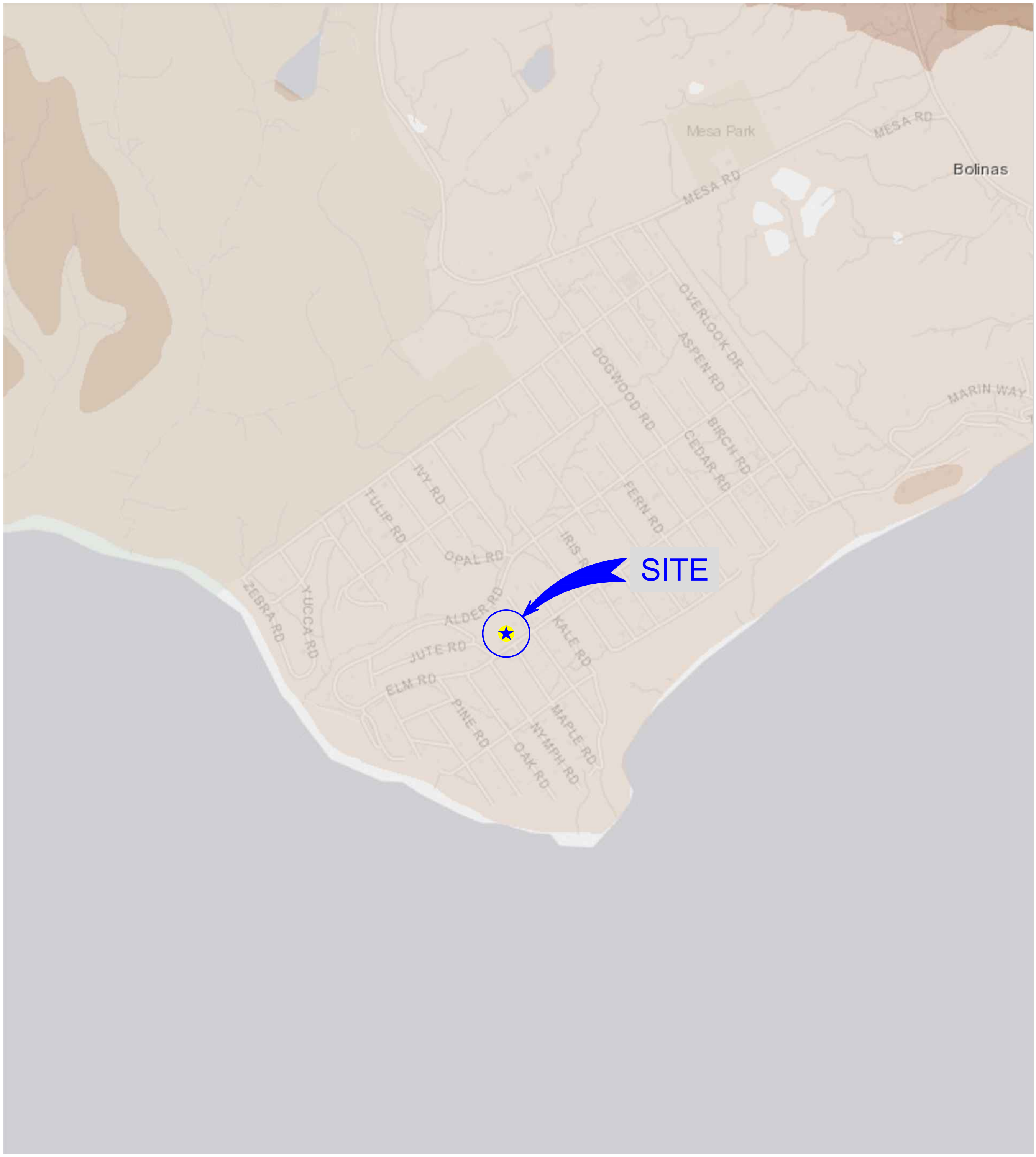
HISTORIC EARTHQUAKE MAP

270 Elm Road
Bolinas, California

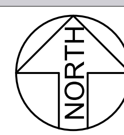
Project No. 3528.001 Date: 7/10/2023

Drawn EIC
Checked

5
FIGURE



Very High	Moderate	Very Low
High	Low	



REFERENCE: ABAG Hazard Viewer, 2023

MILLER PACIFIC ENGINEERING GROUP

FILENAME: 3528.001 Figures.dwg

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

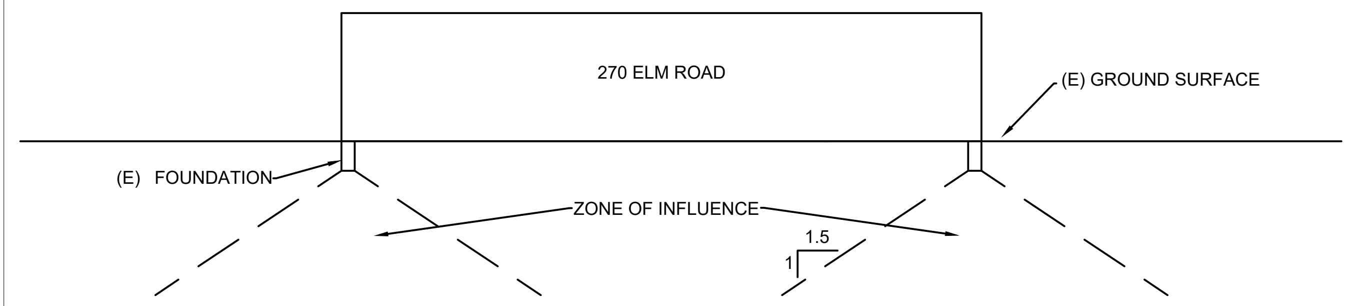
LIQUEFACTION SUSCEPTIBILITY MAP

270 Elm Road
Bolinas, California

Project No. 3528.001 Date: 7/10/2023

Drawn EIC
Checked

6
FIGURE



SCHEMATIC SECTION

(Scale 1"=30')

SCALE

0 15 30 60 FEET

NOTES

1. This drawing is for illustrative purposes only and is not based on survey data.
2. Any existing foundations lying within the excavation's "zone of influence" (ZOI), defined as the region above a 1.5:1 line projected up from the base of the nearest excavation, need to be underpinned to avoid loss of lateral support and potential damage. New underpinning elements must extend below the ZOI. See report text for additional information.

MPEG MILLER PACIFIC ENGINEERING GROUP

FILENAME: 3528.001 Figures.dwg

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

ZONE OF INFLUENCE

270 Elm Road
Bolinas, California

Project No. 3528.001 Date: 7/10/2023

Drawn EIC
Checked

7
FIGURE

APPENDIX A
SUBSURFACE EXPLORATION AND LABORATORY TESTING

1.0 Subsurface Exploration

We explored subsurface conditions at the site by drilling four test borings on July 28, 2023 at the locations shown on Figure 2. The test borings were excavated with truck-mounted drilling equipment using 6-inch solid augers. The exploration was done under the technical supervision of our Geologist who examined and logged the soil and rock materials encountered and obtained samples. The subsurface conditions encountered in the test borings are summarized and presented on the Boring Logs, Figures A-3 through A-6. The depth to groundwater, if encountered, was noted during the drilling and measured before backfilling the borings. “Undisturbed” samples were obtained using a 3-inch diameter, split-barrel Modified California sampler with 2.5 by 6-inch brass 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.

Brief descriptions of the terms and methodology used in classifying earth materials are shown on the attached Soil and Rock Classification Charts, Figures A-1 and A-2, respectively.

2.0 Laboratory Testing

We re-examined the samples in the laboratory to confirm field classification and suitability for testing. We conducted laboratory tests on selected intact samples to verify field identifications and to evaluate physical engineering properties. The following laboratory tests were conducted in general accordance with ASTM standard test methods modified as appropriate for local conditions and practice to provide the data needed for our engineering judgment:

- Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures, ASTM D 2216;
- Density of Soil in Place by the Drive-Cylinder Method, ASTM D 2937;
- Unconfined Compressive Strength of Cohesive Soil, ASTM D 2166; and
- Amount of Material in Soils Finer than the No. 200 Sieve, ASTM D 1140.

The water content, dry density, unconfined compressive strength, and percent passing the #200 sieve test results are reported on the Boring Logs, Figures A-3 through A-6. The boring logs, description of soils and rock encountered, and the laboratory test data reflect conditions only at the location of the borings or sampling 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
- HAND SAMPLER
- STANDARD PENETRATION TEST
- ROCK CORE
- THIN-WALLED / FIXED PISTON
- 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
504 Redwood Blvd.
Suite 220
Novato, CA 94947
T 415 / 382-3444
F 415 / 382-3450
www.millerpac.com

FILENAME: 3528.001 BL.dwg

SOIL CLASSIFICATION CHART

270 Elm Road
Bolinas, California

Project No. 3528.001 Date: 8/28/2023

Drawn EIC
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

FILENAME: 3528.001 BL.dwg

ROCK CLASSIFICATION CHART

270 Elm Road
Bolinas, California

Project No. 3528.001 Date: 8/28/2023

Drawn EIC
Checked

A-2
FIGURE

BORING 1

EQUIPMENT: Mobile B-53 Hydraulic Drill Rig with 6.0-inch Solid Flight Auger
DATE: 7/28/2023
ELEVATION: 170 - feet*
*REFERENCE: Google Earth, 2023

DEPTH meters	DEPTH feet	DESCRIPTION	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
0	0	16-inches Gravel						
		Silty SAND (SM) Dark brown, moist, loose, fine to medium grained sand, 40-45% low to medium plasticity silt. [Terrace Deposits]	8	108	17.6	UC 675	P200 42.7%	
1		Sandy CLAY (CL) Light tan, moist, medium stiff to stiff, medium plasticity, ~20-30% fine to medium grained sand. [Terrace Deposits]	13	102	22.6	UC 2000		
	5		15	89	22.0			
2		Silty SAND (SM) Light tan with orange mottling, moist to wet, medium dense, fine to medium grained sand, 10-15% low plasticity silt. [Terrace Deposits]	20				P200 11.0%	
		Santa Cruz Mudstone Medium gray-brown, low hardness, friable, highly to completely weathered. [Bedrock]	25/3"	101	23.4			
3	10	Bottom of boring at 9-feet 9-inches. Boring caved to ~7-feet. Groundwater measured at 7-feet upon completion.						
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

FILENAME: 3528.001 BL.dwg

BORING LOG
270 Elm Road
Bolinas, California
Project No. 3528.001 Date: 8/28/2023
Drawn EIC
Checked
A-3
FIGURE

BORING 2

EQUIPMENT: Mobile B-53 Hydraulic Drill Rig with 6.0-inch Solid Flight Auger
DATE: 7/28/2023
ELEVATION: 167 - feet*
*REFERENCE: Google Earth, 2023

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			Silty SAND (SM) Dark brown, slightly moist, medium dense, fine to medium grained sand, ~30-35% low plasticity silt. [Terrace Deposits]						
1					17	113	9.4	UC 2275	P200 35.8%	
				grades medium to light tan-brown, moist to wet						
	5				15	103	23.7	UC 775	P200 30.5%	
2										
				Santa Cruz Mudstone Medium gray-brown, low hardness, friable, highly to completely weathered. [Bedrock]	39	108	21.2			
					21		19.5			
3	10			Bottom of boring at 10-feet. No groundwater observed upon completion.						
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

FILENAME: 3528.001 BL.dwg

BORING LOG

270 Elm Road
Bolinas, California

Project No. 3528.001 Date: 8/28/2023

Drawn EIC
Checked

A-4
FIGURE

BORING 3

EQUIPMENT: Mobile B-53 Hydraulic Drill Rig with 6.0-inch Solid Flight Auger
DATE: 7/28/2023
ELEVATION: 167 - feet*
*REFERENCE: Google Earth, 2023

DEPTH (meters / 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			16-inches Gravel						
1			Silty SAND with Gravel (SM) Medium brown, moist, loose, predominately fine to medium grained sand, ~35% low to medium plasticity silt, ~5-15% angular gravels. [Fill]	7	110	17.3		P200 34.5%	
5				5				P200 22.5%	
2	X X		Silty SAND (SM) Light tan-brown, moist to wet, very loose, fine to medium grained sand, ~20-25% low plasticity silt. [Terrace Deposits]			22.2			
			Santa Cruz Mudstone Medium gray-brown, low hardness, friable, highly to completely weathered, laminated. [Bedrock]	72	103	23.0			
3 / 10									
4			rock is slightly softer/friable	25		16.8			
15 / 5			rock grades slightly harder and stronger	28		14.6			
			Bottom of boring at 16.5-feet. No groundwater observed upon completion.						
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

FILENAME: 3528.001 BL.dwg

BORING LOG

270 Elm Road
Bolinas, California

Project No. 3528.001 Date: 8/28/2023

Drawn EIC
Checked

A-5
FIGURE

BORING 4

EQUIPMENT: Mobile B-53 Hydraulic Drill Rig with 6.0-inch Solid Flight Auger
DATE: 7/28/2023
ELEVATION: 170 - feet*
*REFERENCE: Google Earth, 2023

DEPTH meters / 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			12-inches Gravel						
			Silty SAND (SM) Dark brown, moist, loose, fine to medium grained sand, ~30-40% low plasticity silt. [Terrace Deposits]	13	111	15.4			
1			Sandy CLAY (CL) Light tan, moist, medium stiff, medium plasticity, ~25-35% fine to medium grained sand. [Terrace Deposits]	7		19.5			
5									
2			Silty SAND (SM) Light tan, moist to wet, medium dense, fine to medium grained sand, 10-15% low plasticity silt. [Terrace Deposits]	21		25.8		P200 14.5%	
				20		26.7			
3 / 10			Santa Cruz Mudstone Medium gray-brown, low hardness, friable, highly to completely weathered, thinly bedded to laminated. [Bedrock]						
4			same as above	52		12.6			
			Bottom of boring at 13.5-feet. No groundwater observed upon completion.						
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

FILENAME: 3528.001 BL.dwg

BORING LOG

270 Elm Road
Bolinas, California

Project No. 3528.001 Date: 8/28/2023

Drawn EIC
Checked

A-6
FIGURE