MILLER PACIFIC ENGINEERING GROUP

GEOLOGIC AND GEOTECHNICAL FEASIBILITY EVALUATION
NEW RESIDENTIAL SUBDIVISION
11798 HIGHWAY ONE
APN #119-050-04, 119-050-09, 119-140-03 and 119-140-09
POINT REYES STATION, CALIFORNIA

November 6, 2024

Job No. 3614.001

Prepared for:
Mr. Yan Cui
15906 Via Pato
Rancho Santa Fe, California 92067

CERTIFICATION

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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
NATHAN G. KLEMIN
GE 3168
EXP. 03/31/25
GEOTECHNICAL
STATE OF CALIFORNIA

Michael F. Jewett
Engineering Geologist No. 2610
(Expires 1/31/25)

Nathan Klemin
Geotechnical Engineer No. 3168
(Expires 3/31/25)

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1.0 INTRODUCTION

This report summarizes our Geologic and Geotechnical Feasibility Evaluation for the proposed 37-lot residential subdivision at 11798 Highway One in Point Reyes Station, California. As shown on Figure 1, the project site is located near the intersection of Highway One and Point Reyes-Petaluma Road in northeastern Point Reyes Station. Our work has been performed in accordance with our Agreement for Professional Services dated September 6, 2024.

The purpose of our services is to evaluate site surface conditions, evaluate geologic hazards which could impact the site, comment on the geotechnical feasibility of new residential development, and develop preliminary geotechnical recommendations and development guidelines for use in project planning and design development. The scope of our services includes:

- Review of readily available, published public regional geologic mapping and geotechnical background information from our files;
- Site reconnaissance and geologic mapping;
- Evaluation of relevant geologic hazards including seismic shaking, liquefaction, flooding, settlement, landsliding, and other hazards;
- Development of conceptual hazard mitigation measures as warranted.
- Development of preliminary recommendations and guidelines for geotechnical project components; and
- Preparation of this report.

Issuance of this report completes our Phase 1 services for the project. Subsequent phases of work may include subsurface exploration and laboratory testing as part of preliminary or design-level geotechnical investigations; geotechnical consultation and plan review; and observation and testing of geotechnical-related work items during construction.

2.0 PROJECT DESCRIPTION

The project generally consists of subdividing an existing, approximately 82-acre property into a total of 37 lots for single-family residential development, five of which will be specified for affordable housing. Preliminary plans (NexGen Engineering, 2024) indicate that individual lots will range between about 1.02- and 8.2-acres and that building envelopes will typically be sited in the upland, ridgeline parts of the property. Access to the development will be provided via an eastward extension of Water Tank Road, via a new access road from Highway One a few hundred yards north of Water Tank Road, and a new road extending from Point Reyes-Petaluma Road in the southwest corner of the site. Preliminary plans indicate that new cut and fill slopes up to 30 feet high or more will locally be required for development of interior roads, as shown in the Site Map provided in Appendix A.

We anticipate that other subdivision improvements, in addition to new access roads and driveways, will include new underground utilities, new exterior hardscape, landscaping, and other “typical” items. Although preliminary plans have not yet been developed, we anticipate new residential structures will typically be 3 stories or less, be of light wood or steel framing, and impose light to moderate foundation loads. A Site Plan showing preliminary access road alignments, roadway grading, individual lot boundaries, and preliminary building enveloped is provided on Figure 2.

3.0 SITE CONDITIONS

The project site consists of an irregularly shaped, approximately 82-acre parcel bounded to the southwest by Highway One and existing semi-rural residential and agricultural development, to the southeast by Point Reyes-Petaluma Road, and to the northwest and northeast by undeveloped ranchlands.

The property generally consists of two west-trending ridgelines and intervening incised drainages. Site elevations range from a minimum of about +30 feet along Point Reyes-Petaluma Road in the southwest corner of the site to a maximum of about +400 feet near the northern corner of the property. Slopes at the site generally range from about 5:1 (horizontal:vertical) in ridgeline areas and around the base of flanking slopes to about 1.5:1 on the steeper flanking slopes.

3.1 Regional Geology

Marin County lies within the Coast Ranges geomorphic province of California, a region characterized by active seismicity, steep, young topography, and abundant landsliding and erosion owing partly to its relatively high annual rainfall. The regional basement rock consists of sedimentary, igneous, and metamorphic rock of the Jurassic-Cretaceous age (65-190 million years ago) Franciscan Complex and marine sedimentary strata of the Great Valley Sequence, which is of similar age. Within central and northern California, the Franciscan and Great Valley rocks are locally overlain by a variety of late Cretaceous and Tertiary age sedimentary and volcanic rocks which have been deformed by episodes of folding and faulting. The youngest geologic units in the region are Quaternary age (last 1.8 million years) sedimentary deposits. These unconsolidated deposits partially fill many of the valleys of the region.

3.1.1 Regional Geologic Mapping

Regional geologic mapping (Galloway, 1976; Wagner and Smith, 1977; Clark and Brabb, 1997) indicates that the project site is underlain by Franciscan “mélange”, defined as a tectonic mixture of resistant rocks, including sandstone, greenstone, serpentinite, chert, and others, embedded in a matrix of weak, sheared shale. The eastern surface trace of the San Andreas Fault Zone is mapped passing along a northwest trend about 0.9 miles southwest of the site.

The 1977 map by Wagner and Smith is at the most detailed scale of the regional maps and indicates several debris flows around the property, including a large debris flow complex occupying most of the incised drainage at the north end of the property. Small areas of artificial fill are shown at the locations of exiting pond embankments. A copy of Wanger and Smith’s regional geologic map is shown on Figure 3.

3.1.2 Regional Landslide Mapping

Regional reconnaissance-level mapping was performed by the United States Geological Survey (USGS) in the mid 1970's (Wentworth and Frizell, 1975), which primarily involved compilation of maps by prior workers and interpretation of aerial photographs, along with limited field-checking.

As shown on Figure 4, mapping shows most of the drainage areas and flanking slopes at the site to be underlain by debris-flow landslide deposits, and only the uppermost ridgeline areas are shown to be located outside slide areas.

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 composed of localized shear zones which link together to form larger fault zones. Within the Bay Area, faults are concentrated along the San Andreas Fault and Hayward Fault Zones. 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

An "active" fault is one that shows displacement within the last 11,000 years (i.e., Holocene) and has a reported average slip rate greater than 0.1 mm per year. The California Division of Mines and Geology 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 5. The nearest known active faults are the San Andreas and San Gregorio Faults, which are located roughly 0.9- and 16.8-miles southwest of the site, respectively.

3.2.2 Historic Fault Activity

Numerous earthquakes have occurred in the site vicinity in historic times. Significant events include the M=7.8 1906 Great San Francisco Earthquake, which resulted in well-documented surface ruptures throughout the Olema Valley to the south of the site. Sir Francis Drake Boulevard was offset about 20-feet near the west side of Tomales Bay. Very strong ground shaking (estimated modified Mercalli intensity of VIII) resulted in significant damage in Point Reyes Station and the surrounding vicinity, including, including collapse of masonry structures and chimneys, wood-frame buildings thrown from their foundations, and toppling of a railroad steam engine from its tracks. (Lawson, 1908). The M=6.9 Loma Prieta Earthquake of 1989 caused moderate shaking but limited damage in Point Reyes and greater Marin County. A map illustrating the epicentral locations of earthquakes of M > 4.5 since 1900 is shown on Figure 6.

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" (Aagard et al, 2016; 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. In these studies, potential seismic sources were analyzed considering fault geometry, geologic slip rates, geodetic strain rates, historic activity, micro-seismicity, and other factors to arrive at estimates of earthquakes of various magnitudes on a variety of faults in California.

Conclusions from the most recent UCERF3 and USGS (Aagard et al, 2016) indicate the highest probability of an earthquake with a magnitude greater than 6.7 originating on any of the active faults in the San Francisco Bay region by 2043 is assigned to the Hayward/Rodgers Creek Fault system, located approximately 24.1-miles northeast of the site, with a probability of 33 percent. The San Andreas Fault is located approximately 0.9-miles west of the site and is assigned a probability of 22 percent. 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 Historic Site Conditions

We reviewed select aerial photographs of the site spanning the time period between 1952 and 2024 (Historic Aerials, 2024). As of 1952, the site itself is undeveloped and Water Tank Road is not yet in place. What appears to be a home and associated barn or agricultural outbuilding is in place along Highway One near the present-day intersection with Water Tank Road.

By 1971, the agricultural ponds in the western and eastern parts of the site are in place, along with Water Tank Road and one of the two existing water tanks to the southeast. Sometime between 1983 and 1993 a new residence was constructed southeast of the tanks, and by 2005 another new residence was built just northeast of the tanks. No significant changes in site conditions are apparent between 2005 and 2024.

3.4 Site Reconnaissance and Present Surface Conditions

We performed a site reconnaissance on September 19, 2024 to observe and document existing conditions at the site and to prepare a preliminary map of geologic conditions. The project site is located in northeastern Point Reyes Station near the intersection of Highway One and Point Reyes-Petaluma Road.

The property consists of four individual assessor's parcels, which encompass about 82 acres in total and form an irregular property boundary. The site is bounded to the north and west by semi-rural residential and agricultural developments along Highway One, Campolindo Road, and Water Tank Road, to the southeast by Point Reyes-Petaluma Road, and to the northwest by undeveloped ranchlands.

Site elevations range from a maximum of about +405 feet along the northeast property line to a minimum of about +30 feet near Point Reyes-Petaluma Road. The site generally consists of narrow ridgelines, typically inclined between about 5:1 (horizontal:vertical), flanked by steep slopes. Incised drainage channels at the base of the slopes are typically bounded by channel banks inclined between about 2:1 and near vertical.

Vegetation in the ridgeline and upper slopes is typically limited to native grasses and ground cover, while the lower slopes and many of the channel areas are vegetated with mature oak, bay, and laurel trees along with dense poison oak, broom, and other shrubs.

The property appears to have been utilized primarily for livestock grazing, and existing improvements are limited to three ponds (one sited in the southeastern part of the site and two in the northern part), each of which is impounded by an earthen embankment about 10- to 15-feet high, and a small stock tank in the central part of the site.

As shown on Figure 2, geologic mapping performed during our reconnaissance indicates the site is underlain by Franciscan "melange", which at the site consists primarily of very hard, very strong meta-graywacke. These rocks are typically dark gray with variable quartz and calcite veins and form prominent outcrops around the property as indicated on Figure 2. Rarer outcrops of moderately hard tan sandstone and dark gray shale were noted in the northeastern part of the site, and one outcrop of very hard meta-chert was observed in the south-central portion of the property.

Franciscan bedrock is typically overlain by a few inches to a few feet of medium dense silty sand to sandy silt which is of limited cohesiveness and is prone to erosion and debris flow landsliding where exposed to concentrated surface runoff.

As shown on Figure 2, we also mapped a number of landslides throughout the site, which typically occupy the heads and steep banks of the incised drainages. Most of the slides appear to be relatively shallow slumps and earth flows that appear to be the result of scour and erosion in the drainage channels during periods of heavy flow.

4.0 PRELIMINARY GEOLOGIC HAZARDS EVALUATION

This section summarizes our preliminary review of commonly-considered geologic hazards and discusses their potential impacts on the site. Based on our evaluation, significant hazards that could impact the project include strong seismic ground shaking, lurching and ground cracking, slope instability/landsliding, and erosion. Other hazards, including fault surface rupture, liquefaction, expansive soils, and others, are considered less than significant. More detailed discussion of individual hazards and respective conceptual mitigation measures are provided in the following sections.

4.1 Fault Surface Rupture

Under the Alquist-Priolo Earthquake Fault Zoning Act, the California Division of Mines and Geology (now known as the California Geological Survey) produced 1:24,000 scale maps showing known active and potentially active faults and defining zones within which special fault studies are required. As shown on Figure 7, the site lies about 0.7 miles from the eastern margin of the Alquist-Priolo Zone associated with the active San Andreas Fault.

We did not observe any evidence indicative of active or historic faulting during our review of LiDAR topographic data or during the course of our site reconnaissance. Therefore, we judge that the risk of fault surface rupture at the site is generally low.

Evaluation: *Less than significant.*
Recommendations: *No special engineering measures are anticipated.*

4.2 Seismic Shaking

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

4.2.1 Deterministic Seismic Hazard Analysis

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, probable peak ground accelerations, and 84^{th} percentile peak ground accelerations are summarized in Table 1. The calculated accelerations should only be considered as reasonable estimates. Many factors (e.g., soil conditions, orientation to the fault, etc.) can influence the actual ground surface accelerations.

Table 1 – Deterministic Peak Ground Accelerations (PGA) for Active Faults

Fault	*Magnitude[1]*	*Distance[2]*	*Median PGA[3,4]*	*84% PGA[2,4]*
San Andreas Fault:	8.0	0.9 km	0.54g	0.98 g
Rodgers Creek – Hayward Fault	7.6	15.6 km	0.23 g	0.43 g
San Gregorio	7.4	21.9 km	0.17g	0.31 g
West Napa	7.0	45.7 km	0.07 g	0.13 g
Maacama	7.6	59.5 km	0.08 g	0.15 g

Notes:

1. Values determined using USGS Earthquake Scenario Map (BSSC 2014), accessed 2024.
2. USGS Quaternary Fault and Fold Database, https://www.usgs.gov/natural-hazards/earthquake-hazards/faults.
3. Values determined using V_{S30} = 2,500 ft/s (760 m/s) based on Site Class “B/estimated” for anticipated “rock” subsurface conditions.
4. Abrahamson, Silva & Kamai (2014), Boore, Stewart, Seyhan & Atkinson (2014), Campbell & Bozorgnia (2014), and Chiou & Youngs (2014) NGA models.

4.2.2 Probabilistic Seismic Hazard Analysis

Probabilistic Seismic Hazard Analysis analyzes all possible earthquake scenarios while incorporating the probability of each individual event to occur. The probability is determined in the form of the recurrence interval, which is the average time for a specific earthquake acceleration to be exceeded. The design earthquake is not solely dependent on the fault with the closest distance to the site and/or the largest magnitude, but rather the probability of given seismic events occurring on both known and unknown faults.

We calculated the peak ground acceleration for two separate probabilistic conditions; the 2 percent chance of exceedance in 50 years (2,475-year statistical return period) and the 10 percent chance of exceedance in 50 years (475-year statistical return period). The peak ground acceleration values were calculated utilizing the USGS Unified Hazard Tool (USGS, 2023). The results of the probabilistic analyses are presented below in Table 2.

Table 2 – Probabilistic Peak Ground Accelerations (PGA) for Active Faults[1]

Probability of Exceedance	*Return Period*	*Magnitude*	*PGA (g)*
2% in 50 years	2,475 years	7.8	1.17 g
10% in 50 years	475 years	7.7	0.54 g

Reference: USGS Unified Hazard Tool, https://earthquake.usgs.gov/hazards/interactive/, accessed 2024.

Ground shaking can result in structural failure and collapse of structures or cause non-structural building elements (such as light fixtures, shelves, cornices, etc.) to fall, presenting a hazard to building occupants and contents. Compliance with provisions of the current edition of the California Building Code in effect at the time of final design 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. While there are many factors impacting the shaking intensity for a given earthquake or peak ground acceleration, the Modified Mercalli Intensity Scale (The Modified Mercalli Intensity Scale | U.S. Geological Survey) helps to correlate earthquake shaking intensity to an estimated level of damage, with higher intensity shaking resulting in increased damage potential.

The potential for strong seismic shaking at the project site is high. Due to its proximity and historic rate of activity, the San Andreas Fault presents the highest potential for severe ground shaking. The most significant adverse impact associated with strong seismic shaking is potential damage to structures and improvements.

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

Recommendations: *At minimum, new improvements should be designed in accordance with the provisions of the 2022 California Building Code or subsequent codes in effect when final design occurs. Seismic design criteria for new improvements should be developed/confirmed on the basis of subsurface exploration and laboratory testing performed as part of a design-level Geotechnical Investigation.*

4.3 Liquefaction and Related Effects

Liquefaction refers to the sudden, temporary loss of soil strength during strong ground shaking. The strength loss occurs as a result of the build-up of excess pore water pressures and subsequent reduction of effective stress. While liquefaction most commonly occurs in saturated, loose, granular deposits, recent studies indicate that it can also occur in materials with relatively high fines content provided the fines exhibit lower plasticity. The effects of liquefaction can vary from cyclic softening of clays, resulting in limited strain potential, to post-liquefaction settlements and lateral ground movements as a result of lurching or lateral spreading. Regional mapping (USGS, 2006) indicates the site lies in a zone of “very low” liquefaction potential, as shown on Figure 8.

In general, subsurface conditions throughout the site are anticipated to consist of variable silty to sandy soils over relatively shallow weathered bedrock. Although minor alluvial deposits may exist at silted-in agricultural ponds or the mouths of the incised channels, they are probably thin and we judge the likelihood of liquefaction there is relatively low. We judge that, were it to occur, the risk of damage to improvements due to liquefaction is low.

Evaluation: *Less than significant.*

Recommendations: *No special engineering measures are anticipated. Evaluation should be confirmed on the basis of subsurface exploration and laboratory testing performed as part of a future Geotechnical Investigation.*

4.4 Lurching and Ground Cracking

Lurching and associated ground cracking can occur during strong ground shaking. Ground cracking generally occurs parallel to and along the crests of slopes where stiff soils are underlain by soft deposits or along steep slopes or channel banks. Lurching refers to the lateral movement of cohesive soils or rock soils toward a "free face", such as a steep slope.

Such conditions are present locally around the site, primarily along steeper ravine banks and around the upper scarps of mapped soil slumps and debris flow slides. Although these areas are typically underlain by shallow hard bedrock we judge there is a moderate to high risk of lurching and ground cracking during future seismic events.

Evaluation: *Less than significant with special engineering measures.*
Recommendations: *For planning purposes, we recommend that minimum 50-foot setbacks be provided from mapped landslides and channel banks for all structures and "critical" improvements. Additional discussion regarding recommended building envelopes and landslide mitigation/avoidance is provided in Section 5 of this report.*

4.5 Consolidation Settlement

Significant settlement can occur when new loads are placed over soft, compressible clays or loose granular soils, both of which are generally found in marsh, lake, and alluvial environments, or where significant structural loads are transferred to the underlying soils.

Based on our geologic mapping and reconnaissance observations, we do not anticipate soft compressible soils will underlie the project site, and therefore judge the risk of damage due to consolidation settlement is low.

Evaluation: *Less than significant.*
Recommendations: *No special engineering measures are anticipated. Evaluation should be confirmed on the basis of subsurface exploration and laboratory testing performed as part a design-level Geotechnical Investigation.*

4.6 Seismic Densification

Seismic ground shaking can induce settlement in unsaturated, loose granular soils. Settlement occurs as the loose soil particles rearrange into a denser configuration when subjected to seismic ground shaking. Varying degrees of settlement can occur throughout a deposit, resulting in differential settlement of structures founded on such deposits.

Based on our geologic mapping and reconnaissance observations, we do not anticipate that significant deposits of loose granular soils underlie the project site, and the likelihood of seismic densification is low.

Evaluation: *Less than significant.*
Recommendations: *No special engineering measures are anticipated. Evaluation should be confirmed on the basis of subsurface exploration and laboratory testing performed as part a design-level Geotechnical Investigation.*

4.7 Expansive Soils

Soil expansion occurs when clay particles interact with water causing seasonal volume changes in the soil matrix. The clay soil swells when saturated and then contracts when dried. This phenomenon generally decreases in magnitude with increasing confinement pressures at increasing depths. These volume changes may damage lightly loaded foundations, concrete slabs, pavements, retaining walls and other improvements. Expansive soils also cause soil creep on sloping ground.

We did not observe significant evidence (ground cracking, cracked pavements, etc.) of expansive soil during our site reconnaissance, and we anticipate that onsite soils are likely of low expansive potential.

Evaluation: *Less than significant.*
Recommendations: *No special engineering measures are anticipated. Evaluation should be confirmed on the basis of subsurface exploration and laboratory testing performed as part a design-level Geotechnical Investigation.*

4.8 Slope Instability/Landsliding

Slope instability generally occurs on relatively steep slopes and/or on slopes underlain by weak materials. As discussed previously and shown on Figures 3 and 4, regional geologic mapping and reconnaissance-level landslide mapping shows most of the drainage areas and flanking slopes at the site to be underlain by debris-flow landslide deposits, and only the uppermost ridgeline areas are shown to be located outside slide areas. Notably, this mapping was performed in the early to mid 1970's and was prepared primarily on the basis of aerial photograph review and apparently limited field-checking.

During our reconnaissance, we mapped several landslides around the site. Most slides appear to consist of relatively shallow earth flows occupying the upper reaches of the incised drainages around the site, as shown on Figure 2. The margins of these slides are typically marked by near-vertical scarps ranging from about 1- to 3-feet high.

Many of these slides are located downslope and well away from planned development, and thus are unlikely to significantly impact the project. However, some slides are located in areas proximal to proposed building enveloped and access road alignments. Most significantly, proposed building envelopes on Lots 15 and 29 are sited very close to the upper headscarps of mapped slides, while Lot 32 may be underlain entirely by a shallow slide. Several proposed access road fill slopes are also shown within mapped slide limits, including the areas between Lots 3 and 10, 12 and 14, and just downslope of Lot 15.

Given the relatively steep slopes and generally erodible soils, we judge the likelihood of landsliding at the site is high.

Evaluation: *Less than significant with special engineering measures.*
Recommendations: *Depending on actual subsurface conditions at the site, special engineering measures may include a combination of site grading, special foundation design, new retaining structures, minimum setbacks from slide scarps, and/or new surface and subsurface drainage improvements. Additional discussion regarding slope-stability mitigation measures, probable foundation types, and site drainage considerations is provided in Section 5 of this report.*

4.9 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.

During our reconnaissance, we noted that surface soils typically consist of medium-dense, porous silty sand and sandy silt which typically exhibit little cohesion and will be prone to erosion. Drainages in between the ridgeline areas exhibit significant downcutting/incision, and flanking slopes and ravine banks exhibit local evidence of undermining, which may result from scour at the toe of the slope during periods of high flow. Based on our observations and given the largely cohesionless near-surface soils, we judge the risk of erosion at the site is high.

Evaluation: *Less than significant with special engineering measures.*
Recommendations: *For planning purposes, we recommend that minimum 50-foot setbacks be considered from the crest of the swale/creek banks to reduce the risk of damage due to erosion and scour. Surface runoff should be collected in area drains, catch basins, ditches, swales, or similar and conveyed to an appropriate location unlikely to result in new erosion, ideally an established municipal storm drain system or directly into the established swale/creek channel(s). At minimum, the project Civil Engineer should design site drainage systems to accommodate runoff from a 100-year storm event. An erosion control plan should be developed prior to construction per the current guidelines of the California Stormwater Quality Association's Best Management Practice Handbook or superseding local regulations.*

4.10 Flooding

The project site is located at elevations above +30 feet and is not located within a FEMA 100- or 500-year flood zone. Therefore, we judge the risk of large-scale flooding at the site is generally remote. The project Civil Engineer or Architect is responsible for site drainage and should evaluate localized flooding potential and provide appropriate mitigation.

Evaluation: *Less than significant.*
Recommendations: *No special engineering measures are anticipated.*

4.11 Tsunami and Seiche

The project site is located at elevations above +30 feet and is located about 4,300 feet northeast of Tomales Bay. The site is not located in a tsunami inundation zone (CalEMA 2009). Therefore, the risk of tsunami/seiche inundation is low.

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

4.12 Corrosive Soil

Corrosive soil can damage buried metallic structures, cause concrete spalling and deteriorate rebar reinforcement. Per Caltrans Corrosion Guidelines (2003) a soil is considered corrosive if the pH level is less than 5.5, the chloride concentration is greater than 500 ppm, and/or the sulfate concentration is 2,000 ppm or greater. In our previous experience, Franciscan Complex bedrock and derivative soils are not typically highly corrosive, and the likelihood of corrosive soils existing at the site is judged low to moderate.

Evaluation: *Less than significant.*
Recommendations: *No mitigation measures are anticipated. Evaluation should be confirmed on the basis of laboratory corrosivity testing performed as part of a future Geotechnical Investigation.*

4.13 Radon

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, 2024). Zone 3 is classified by the EPA as exhibiting a "low potential for Radon-222 gas.

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

4.14 Volcanic Eruption

Several active volcanoes with the potential for future eruptions exist within northern California, including Mount Shasta, Lassen Peak, and Medicine Lake in extreme northern California, the Mono Lake-Long Valley Caldera complex in east-central California, and the Clear Lake Volcanic Field, located in Lake County approximately 60 miles north of the project site. The most recent volcanic eruption in northern California was at Lassen Peak in 1917, while the most recent eruption at the nearest volcanic center to the project site, the Clear Lake Volcanic Field, was about 10,000 years ago. All of northern California's volcanic centers are currently listed under "normal" volcanic alert levels by the USGS California Volcano Observatory (USGS, 2024). While the aforementioned volcanic centers are considered "active" by the USGS, the likelihood of damage to the proposed improvements due to volcanic eruption is generally low.

Evaluation: *Less than significant.*
Recommendations: *No mitigation measures are anticipated.*

4.15 Naturally-Occurring Asbestos

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 primarily by sandstone bedrock of the Franciscan Complex, with lesser amounts of shale and chert also present. We did not observe any outcrops of serpentinite or related ultramafic rocks during our reconnaissance. Although it is possible such materials exist but are not readily exposed at the ground surface, we judge that the likelihood that significant amounts of naturally-occurring asbestos exist at the site is low.

Evaluation: *Less than significant.*
Recommendations: *No mitigation measures are anticipated.*

5.0 PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our research, reconnaissance, and evaluation, it is our opinion that development of the site is feasible from a geotechnical perspective. In addition to providing uniform foundation support and adequate seismic design for new structures, primary geologic and geotechnical issues to be addressed during project planning and preliminary design will include the potential for new or renewed slope instability, the potential for significant erosion, and the potential for lurching and ground cracking during strong seismic ground shaking.

Preliminary geotechnical recommendations and guidelines to address these and other planning-level considerations are presented in the following sections. Note that (at minimum) subsurface exploration, laboratory testing, and slope stability analyses will need to be performed in order to develop design-level geotechnical recommendations and criteria for the project.

5.1 Mitigation of Existing Landslides

In general, we anticipate that landslide mitigation for the purpose of residential development may be accomplished either by **avoidance** or by **landslide repair,** or perhaps via a combination of strategies.

Avoidance would consist of imposing minimum setbacks for new structures and associated improvements (such as pools, septic systems, etc.) from landslide areas. On a preliminary basis, we recommend that minimum setbacks of 50-feet be imposed from mapped landslides (map unit Qls), as shown on Figure 2. Note that most building envelopes shown on the preliminary plans incorporate such setbacks, and that this recommendation primarily affects Lots 15 and 29, as well as access road fill slopes between Lots 3 and 10, opposite Lot 12, and below Lot 15. If and where such setbacks cannot be accommodated or are undesirable, then slides must be repaired as described below.

Landslide repairs should improve the stability of the landslide area such that the calculated factor of safety[1] is at least 1.5 for static conditions and greater than 1.0 for pseudo-static[2] (seismic) conditions.

Based on our interpretation of subsurface conditions, wherein landslide activity is probably limited to the loose surface soils overlying hard and strong bedrock, we judge that landslide repairs could include 1) excavation of unstable material, installation of subsurface drainage and construction of a compacted earth fill buttress; (2) design and construction of retaining structures; (3) de-watering with subsurface drainage; or (4) a combination of the above. Subsurface exploration, laboratory testing, and slope stability analyses will be required to confirm the applicability of these options and develop design-level geotechnical recommendations and criteria. Note that, if and where existing slope inclinations exceed 1.5:1, earth buttresses will not be feasible and alternative mitigation strategies will be required.

[1] The factor of safety is defined as the ratio of the resisting forces to the driving forces. Slopes with a factor of safety less than 1.0 are unstable and a landslide will commence. Slopes with a factor of 1.0 are marginally stable. The higher the factor of safety, the more stable the slope.

[2] The seismic acceleration used in the pseudo-static analyses shall be the maximum ground acceleration determined from deterministic methods, or the probabilistic ground acceleration that corresponds with a 10 percent chance of being exceeded in 50 years. See Section 4.2 for preliminary estimated deterministic and probabilistic ground accelerations.

Additional discussion regarding site excavation/grading, probable foundation types, optional retaining wall configurations, and site drainage considerations are provided in subsequent sections of this report.

5.2 Building Envelope Development

We understand that many factors, including aesthetics, engineering/construction costs, permitting considerations, parcel line setbacks, well/septic requirements, and others, may affect ultimate siting of new structures and improvements.

In general, most of the preliminary building envelopes appear generally suitable for the proposed residential development. Specific lots which may require re-siting of the building envelope and/or landslide mitigation could include Lots 3, 10, and 15, depending on actual structural setbacks from landslide areas downslope. The building envelope at Lot 29 is also sited very close to the upper limit of and existing landslide, and an avoidance strategy for landslide mitigation may not be feasible given the current extents of the building envelope. This building envelope should be relocated farther north and west if possible; otherwise an alternative approach such as special foundation design or new retaining structures will be required given that the slide area appears too steep for a compacted buttress. Lot 32 is mapped (on a preliminary basis) as being underlain by landslide debris; field observations suggest that debris is likely relatively thin, and this lot may potentially be developed via earth buttress construction and/or special foundation design, depending on actual thickness.

In addition, several access road cut and fill slopes are shown encroaching on mapped slide areas, including near Lots 3, 10, 4, and 15. In these areas, landslide repairs, such as compacted buttresses or new retaining walls, are likely to be required. In general, the most economical design for the planned project would site new building envelopes, access roads, and other improvements as far from mapped slides and incised drainages as practical.

5.3 Site Grading Considerations

Preliminary plans indicate that extensive grading, including cuts and fills locally on the order of 30 feet high, will be required to develop level building pads and access roads. For planning purposes, all earthwork should be performed in general accordance with the preliminary recommendations and criteria outlined in the following sections.

5.3.1 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. Existing foundations and utilities which are to be abandoned as part of the work, if and where they exist, should be removed from structural areas. In non-structural areas, utilities could be abandoned in place provided cement grout completely fills any void in the utility.

Where fills or other structural improvements are planned, the subgrade surface should be scarified to a depth of 8 inches, moisture conditioned to within two percent of the optimum moisture content and compacted to at least 90 percent relative compaction. Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density, as determined by ASTM D1557. Areas exposing weathered bedrock at subgrade need not be scarified and recompacted.

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 should provide supplemental recommendations to address the specific condition.

5.3.2 Excavations and Temporary Cuts

Excavations for new foundations, utilities and other improvements in areas mapped as Franciscan Complex bedrock (Map Unit Fm as shown on Figure 2) are likely to encounter 1- to 3-feet of medium-dense silty sandy residual soils over hard and strong sandstone bedrock. In areas mapped as landslides, similar bedrock is likely to be overlain by thicker deposits of unconsolidated silty to sandy landslide debris. It is anticipated that shallow excavations can likely be accomplished with “traditional” equipment, such as medium to large excavators and dozers. Where zones of especially hard/strong rock are encountered, or where excavations deeper than 3- to 4-feet are planned, specialized hard rock excavation techniques and equipment, such as jackhammers and hydraulic breakers, may be required.

All excavations deeper than 5-feet must be braced, shored, or sloped in accordance with OSHA requirements. We recommend a maximum allowable temporary slope inclination of 1.5:1 in onsite soil materials, based on an OSHA “Type B” soil profile. Bedrock can probably be considered Type “A” and sloped at 1:1. These classifications should be confirmed on the basis of subsurface exploration and laboratory testing performed as part of a future design-level Geotechnical Investigation.

The Contractor should be solely responsible for site safety and should provide adequate shoring as needed. The selected shoring system should be capable of providing immediate support to the sides of the excavation so as to minimize the amount of time that cuts are unsupported.

5.3.3 Fill Materials, Placement and Compaction

Fill materials should consist of non-expansive materials that are free of organic matter, have a Liquid Limit of less than 40 (ASTM D 4318), a Plasticity Index of less than 15 (ASTM D 4318), and a minimum R-value of 20 (California Test 301). The fill material should contain less than 50 percent of particles passing a No. 200 sieve and should have a maximum particle size of 4 inches.

Based on our mapping and experience with similar projects, we anticipate that onsite excavations will yield silty to gravelly/cobbly material that could be suitable for re-use but may require substantial processing to meet the criteria specified above, especially for materials generated via deep excavations into bedrock. Any imported fill material needs to be tested to determine its suitability.

Fill materials should be moisture conditioned to within two percent of the optimum moisture content prior to compaction. Properly moisture conditioned fill materials should subsequently be placed in loose, horizontal lifts of 8 inches-thick or less and uniformly compacted to at least 90 percent relative compaction. Where fill thicknesses are greater than 5 feet, fill materials should be compacted to at least 92 percent relative compaction. In pavement areas, the upper 12 inches of fill should be compacted to at least 95 percent relative compaction. The maximum dry density and optimum moisture content of fill materials should be determined in accordance with ASTM D1557.

5.3.4 Permanent Cut and Fill Slopes

For planning purposes, permanent cut slopes should generally be no steeper than 2:1 in soil and 1:1 in rock materials, based on OSHA Type "C" and Type "A" classifications, respectively. Steeper slopes may be feasible where cuts are planned in hard bedrock, depending on the results of design-level subsurface exploration and laboratory testing.

New fill slopes should not be inclined steeper than 2:1. All new fill slopes will need to bear on keyways and benches excavated into firm native bedrock below any slide-prone soils, and keyways and benches will need to be provided with subdrains to reduce the risk of future instability. A typical detail for hillside fill construction is included on Figure 10.

Fill slopes higher than 25-feet will require intermediate terraces and v-ditch for positive surface drainage. Fill slopes steeper than 2:1 may locally be feasible; however, they will require synthetic geotextile reinforcement and will need to be specifically designed.

5.4 Preliminary Seismic Design

Minimum mitigation of ground shaking includes seismic design of new structures in conformance with the provisions of the most recent edition (2022) of the California Building Code. The magnitude and character of these ground motions will depend on the earthquake and the site response characteristics. Based on the anticipated site grading, subsurface conditions, and proximity of active faults, we recommend the CBC coefficients and site values shown in Table 3 be used to preliminarily calculate the design base shear of new structures as applicable. The values presented in Table 3 should be confirmed based on supplemental subsurface exploration and laboratory testing.

Table 3 – Preliminary 2022 California Building Code Seismic Design Criteria

Parameter	Design Value
Site Class	B (est.)
Site Latitude, Longitude	38.0781°, -122.7990°
Spectral Response (short), S_S	2.29 g
Spectral Response (1-sec), S_1	0.958 g
Site Coefficient, F_a	1.0
Site Coefficient, F_V	1.0

Reference: SEAOC/OSHPD Seismic Design Maps, https://www.seismicmaps.org/, accessed October 2024.

5.5 Probable Foundation Types

New structures in sloping ground or that will span cut/fill transitions should be supported on deep foundations consisting of drilled, cast-in-place concrete piers. Piers should extend through any soil materials to gain strength from underlying bedrock and should be connected at the tops through a series of upslope/downslope and transverse grade beams.

If and where structures are sited on level ground underlain entirely by engineered/compacted fill of uniform thickness, or in "cut" areas underlain by weathered bedrock, then shallow foundations, consisting of conventional spread or continuous footings, could be appropriate provided that adequate site drainage and other improvements are provided to maintain acceptable slope-stability factors of safety. Subsurface exploration, laboratory testing, and slope stability analyses will be required to confirm the applicability of these options and develop design-level geotechnical recommendations and criteria.

5.6 Optional Retaining Wall Configurations

Retaining walls may be needed to retain new cuts and fills for development of level building pads and access roads. Many retaining wall options are available, including soldier-pile (steel H-beam) and timber or concrete lagging; reinforced, cast-in-place concrete; mechanically-stabilized earth (MSE); and soil-nail and shotcrete. Where new cuts will be retained, soldier-pile and timber lagging or soil nail and shotcrete walls are typically the most cost-efficient. For new fill areas, mechanically stabilized earth (MSE) walls may be more cost-effective. All walls taller than 3-feet will require back drainage to prevent the buildup of hydrostatic pressure as shown on Figure 11. Any retaining walls adjacent to interior living space (such as conditioned basement or "daylight" lower floor walls) will need to be waterproofed.

5.7 Site and Foundation Drainage Considerations

Although the site currently contains slopes that will drain, new grading could result in adverse drainage patterns causing water to pond around the development. 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 hardscape areas but should not discharge onto landscaped areas immediately adjacent to the buildings. Provide area drains for landscape planters adjacent to buildings and collect downspout discharges into a tight pipe collection system that discharges well away from the building foundations. Where new structures will not incorporate new retaining walls along the upslope side, then perimeter foundation drains should be considered as shown schematically on Figure 12.

Site drainage should be discharged away from the building area and outlets should be designed to reduce erosion. Site drainage improvements should ideally be connected into an established storm drainage system that discharges at a suitable location and in a manner unlikely to result in new erosion.

It is recommended that all site drainage systems be designed such that the post-project peak flow rate is equal to or less than pre-project conditions, and that all systems be designed (at minimum) in consideration of a 100-year storm event. If reduction in post-project flow rates is required, it should be noted that infiltration-type systems are NOT recommended in the sloping portions of this site. In those areas, we recommend consideration of a detention-type approach, wherein excess runoff is stored in structures with impermeable bottoms and fitted with appropriately sized outfalls as to achieve the required peak flow rate. Note that infiltration systems are judged appropriate from a geotechnical perspective if sited in the lower-lying, flatter parts of the site provided adequate field capacity exists to accommodate infiltration.

5.8 Underground Utilities

Excavations for utilities will likely encounter relatively weak silty to sandy residual soil deposits over hard, strong sandstone bedrock. Groundwater could seasonally be encountered at shallow depths, typically near the soil/rock interface. Trench excavations having a depth of 5 feet or more must be excavated and shored in accordance with OSHA regulations, as discussed in Section 5.2.2.

Unless otherwise recommended by the pipe manufacturer, pipe bedding and embedment materials should consist of well-graded sand with 90 to 100 percent of particles passing the No. 4 sieve and no more than 5 percent finer than the No. 200 sieve. Crushed rock or pea gravel may also be considered for pipe bedding. Provide the minimum bedding thickness beneath the pipe in accordance with the manufacturer's recommendations (typically 3 to 6 inches). Trench backfill may consist of on-site soils, provided that the soil meets the fill criteria outlined in Section 5.2.3 or imported aggregate baserock. Trench backfill should be 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.

5.9 Asphalt Pavements

For new asphalt-paved areas, the subgrade should be prepared as described in Section 5.2.1 and compacted to at least 95% relative compaction. For typical residential driveway loads, we recommend a pavement section consisting of 2-inches of asphalt concrete over 6-inches of Caltrans Class 2 aggregate baserock. For improved performance and a longer lifespan, or if "heavier than normal" traffic loads are anticipated (such as for garbage collection, emergency vehicles, etc.), the baserock could be thickened to 8-inches and the asphalt surface increased to 3-inches. Aggregate baserock and asphalt concrete materials should conform to the latest edition of the Caltrans Standard Specifications (2022) and be compacted to a minimum of 95% relative compaction per ASTM D 1557.

New access roads will likely require specific design in accordance with Marin County Fire Department requirements. We recommend that the project Civil Engineer design new pavement sections in consideration of local requirements, and on the basis of subgrade R-Value testing performed during the course of a future Geotechnical Investigation.

6.0 SUPPLEMENTAL GEOTECHNICAL SERVICES

Following review of this report and any subsequent plan revisions, we recommend that the next phase of supplemental services consists of a Design-Level Geotechnical Investigation for the subdivision. The purpose of such work would be to confirm actual subsurface conditions, confirm the depths/extents of any actual landslide deposits, and refine specific recommendations for landslide mitigation where applicable.

Such an Investigation should generally include a minimum of one soil boring or two to three test pits per parcel to confirm subsurface stratigraphy and preliminary foundation recommendations. Additional borings should also be drilled where significant cut and fill slopes are planned. The results of such data collection would then be used to perform objective slope stability analyses and allow for refinement of landslide mitigation recommendations specific to each lot.

Once development plans are better developed on the basis of identifying the "preferred" slide mitigation approaches, Design-Level Geotechnical Investigations (including supplemental subsurface exploration, laboratory testing, and/or analyses as needed) should be undertaken to provide design-level recommendations and criteria for the project.

As project plans near completion, we should review them to ensure that the intent of our recommendations has been sufficiently incorporated. During construction, we should be present intermittently to observe and test the geotechnical portions of the work. The purpose of our observation and testing is to confirm that site conditions are as anticipated, to adjust our recommendations and design criteria if needed, and to confirm that the Contractor's work is performed in accordance with the project plans and specifications.

7.0 LIMITATIONS

We believe 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 (Mr. Yan Cui) 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 detailed information regarding the presence or absence of toxic or hazardous waste.

We recommend that this report, in its entirety, be made available to project team members, designers, 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.

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SITE COORDINATES
38.0781400°N
122.7990044°W

MAP SCALE 1:24,000

0 500 1,000 2,000 3,000 4,000 Feet

N W E S

MILLER PACIFIC ENGINEERING GROUP

NOVATO 415-382-3444
PETALUMA 707-765-6140
NAPA 707-265-7982
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SITE LOCATION MAP

Proposed Residential Subdivision
11798 Highway One
Point Reyes Station, California

Project No. 3614.001 Date: 10/21/2024 3:41 PM

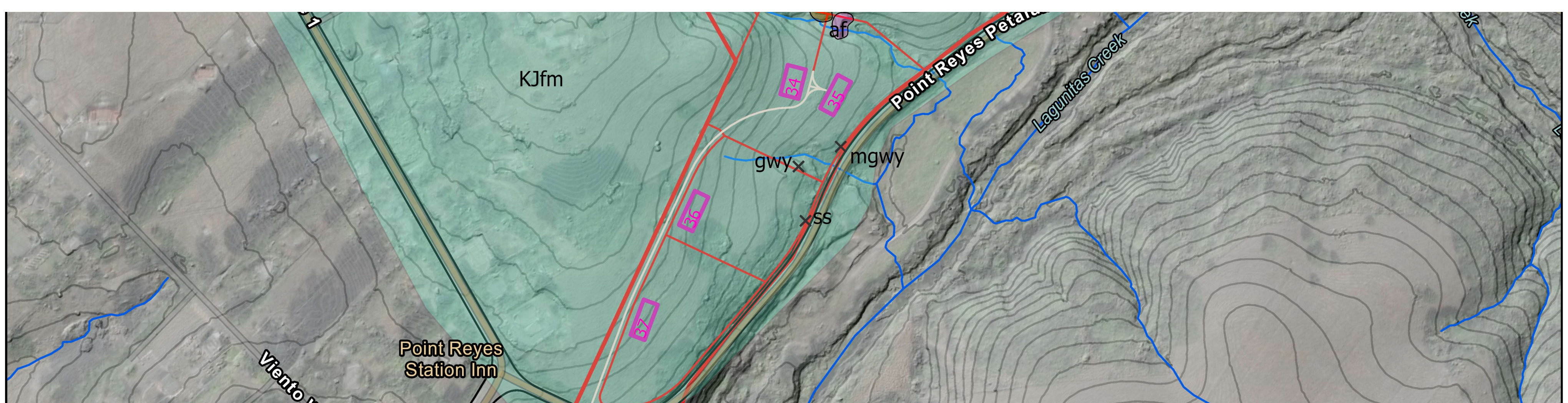
FIGURE 1

ey to Map Symbols

(Holocene)
s (Holocene)

ge (Jurassic to Cretaceous)

e
rt
ywacke
e and shale

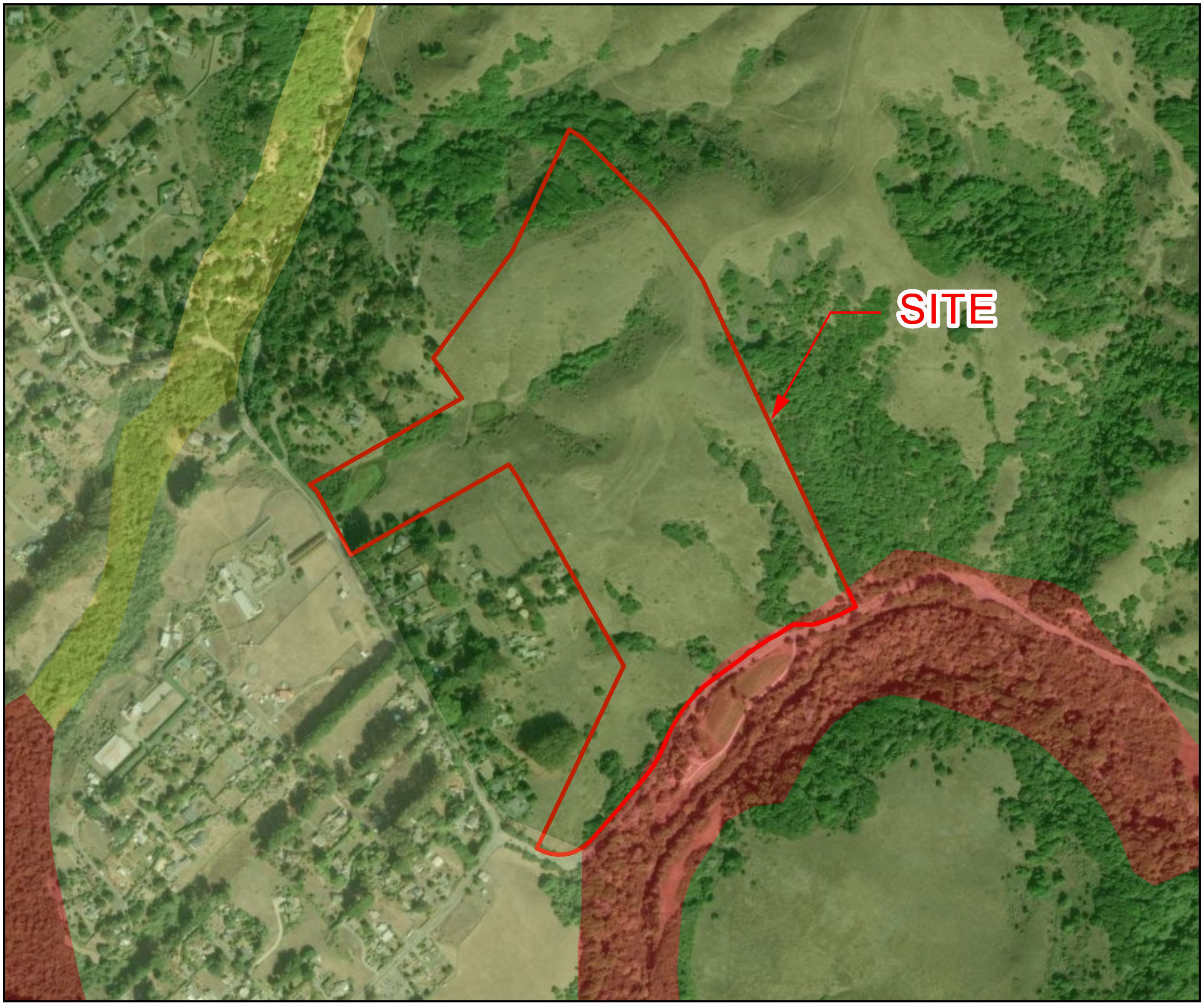


N
W E
S

MAP SCALE 1:6,000

0 250 500 1,000 1,500 Feet

MPEG MILLER PACIFIC ENGINEERING GROUP	NOVATO 415-382-3444	SITE PLAN AND PRELIMINARY GEOLOGIC MAP	FIGURE 2
	PETALUMA 707-765-6140	Proposed Residential Subdivision 11798 Highway One Point Reyes Station, California	
	NAPA 707-265-7982		
A CALIFORNIA CORPORATION © 2024, ALL RIGHTS RESERVED	www.millerpac.com	Project No. 3614.001 Date: 10/29/2024 3:30 PM	



SITE COORDINATES
38.0781400°N
122.7990044°W

MAP SCALE 1:12,000

0 500 1,000 2,000 Feet

N W E S

LEGEND AND KEY TO MAP SYMBOLS

Liquefaction Susceptibility

- Very High
- High
- Medium
- Low
- Very Low

REFERENCE: Witter, R.C., et al (2006), "Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, California", United States Geological Survey Open File Report 2006-1037, Version 1.1. http://pubs.usgs.gov/of/2006/1037/

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LIQUEFACTION SUSCEPTBILITY MAP

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11798 Highway One
Point Reyes Station, California

Project No. 3614.001 Date: 10/21/2024 1:26 PM

FIGURE 8

SITE COORDINATES
38.0781400°N
122.7990044°W

MAP SCALE 1:12,000

0 500 1,000 2,000 Feet

N W E S

FEMA FLOOD ZONES

A	AO	X500C
A99	V	XL
AE	VE	X
AH	X500	D

REFERENCE: Federal Emergency Management Agency (FEMA) National Flood Hazard Layer (NFHL), https://www.arcgis.com/apps/webappviewer/index.html?id=8b0adb51996444d4879338b5529aa9cd, accessed 2024.

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FEMA FLOOD ZONE MAP

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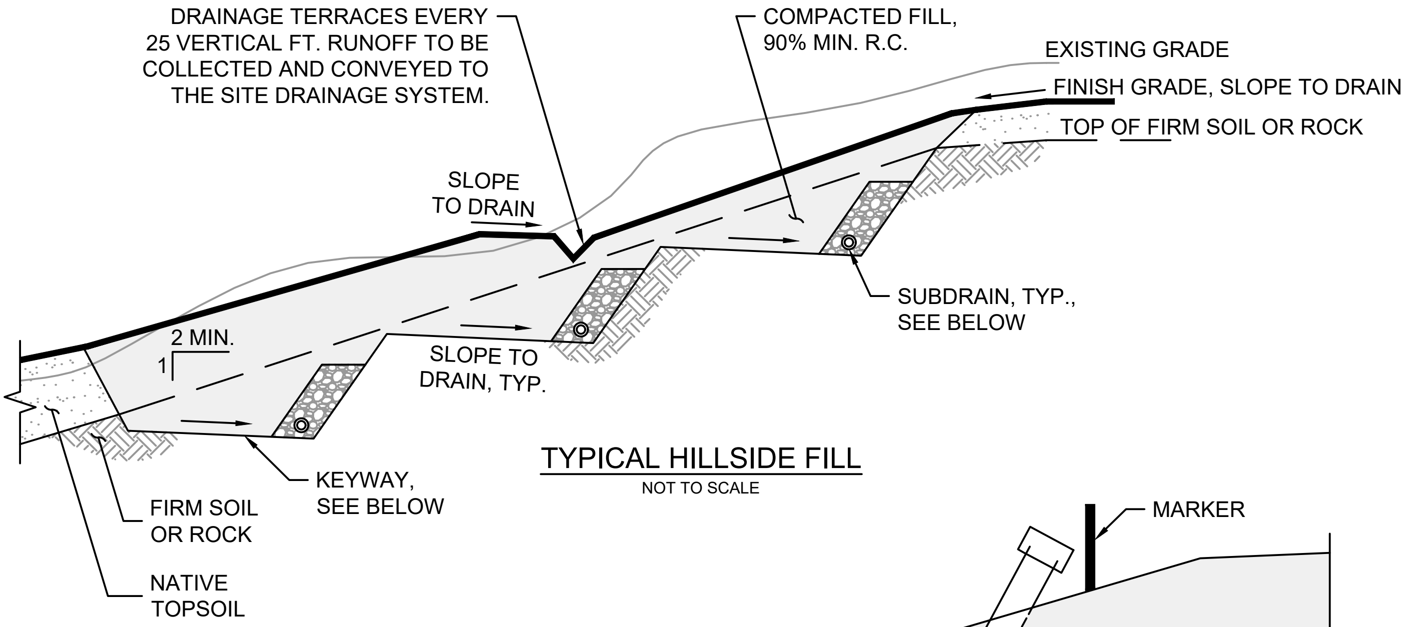
Point Reyes Station, California

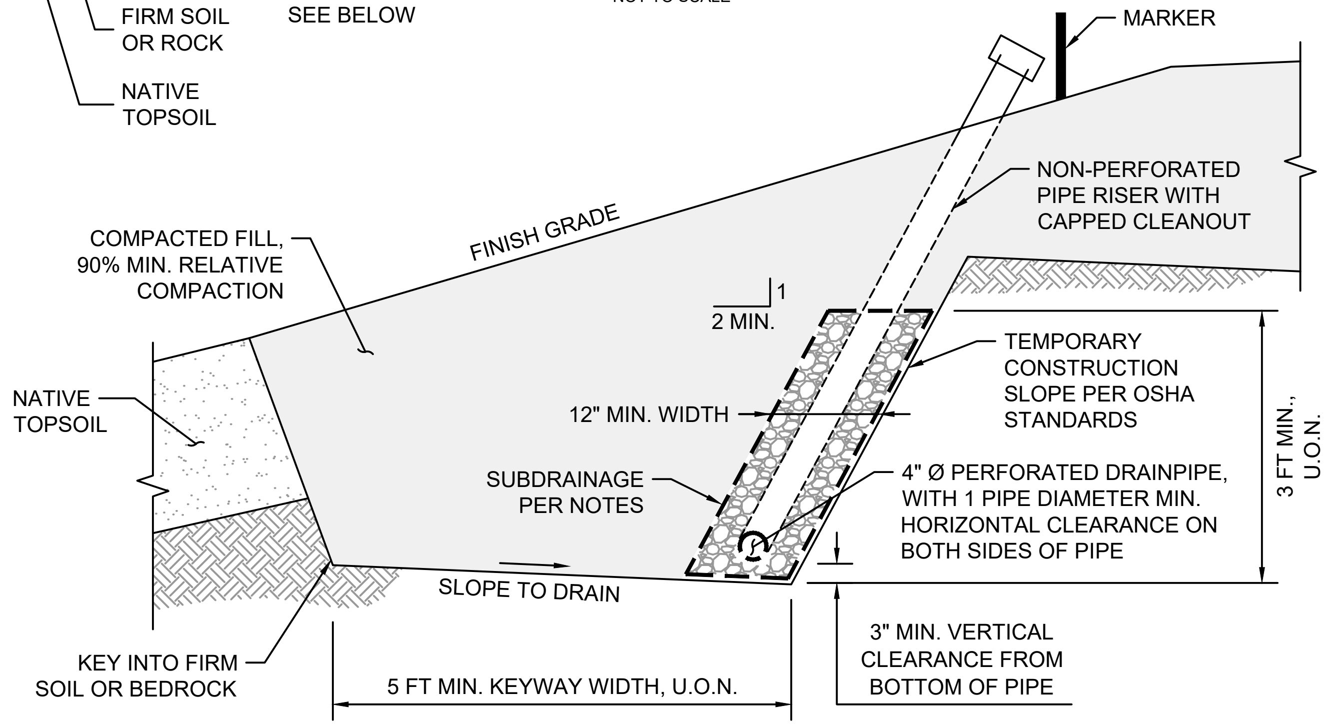
Project No. 3614.001 Date: 10/21/2024 1:26 PM

FIGURE 9



TYPICAL HILLSIDE FILL

NOT TO SCALE



TYPICAL KEYWAY AND SUBDRAIN

NOT TO SCALE

NOTES:

1. Subdrain 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.
2. 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. Pipe perforations shall face down. Pipe sloped at 1% to a gravity outlet, with non-perforated pipe to gravity discharge.
3. 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.
4. 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

FILENAME: 3614.001 Standard Figures.dwg

SCHEMATIC HILLSIDE FILL DETAIL

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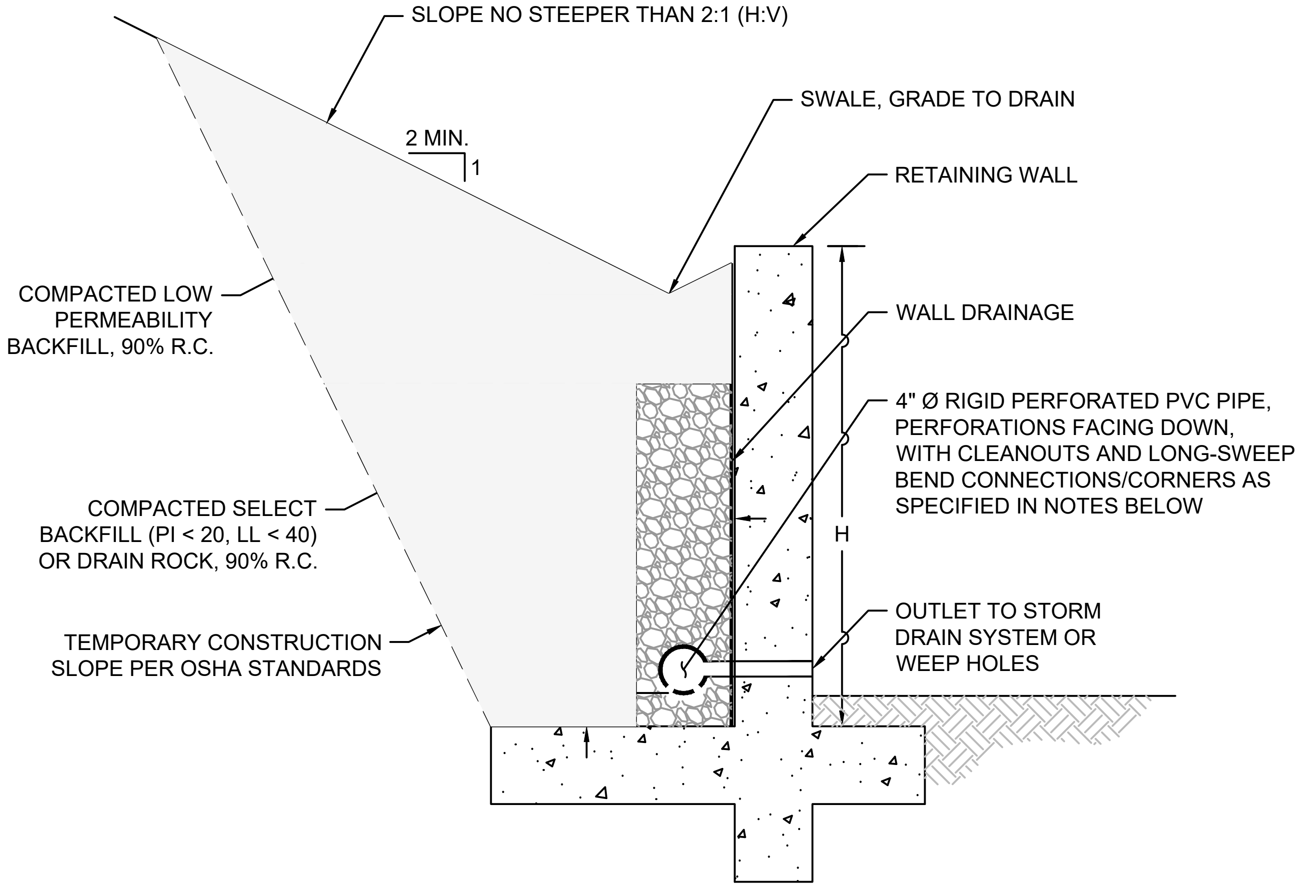
11798 Highway One
Point Reyes Station, California

Project No. 3614.001 Date: 10/24/2024

Drawn PHA
Checked

10
FIGURE



SCHEMATIC RETAINING WALL BACKDRAIN

NOT TO SCALE

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-inch diameter weep holes spaced approximately 20 feet apart.
4. Clean outs should be installed at the upslope end and at significant direction changes of the perforated pipe. Additionally, all angled connectors/corners shall be long-sweep bend 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

FILENAME: 3614.001 Standard Figures.dwg

SCHEMATIC RETAINING WALL BACKDRAIN DETAIL

Proposed Residential Subdivision
11798 Highway One
Point Reyes Station, California

Project No. 3614.001 Date: 10/24/2024

Drawn PHA
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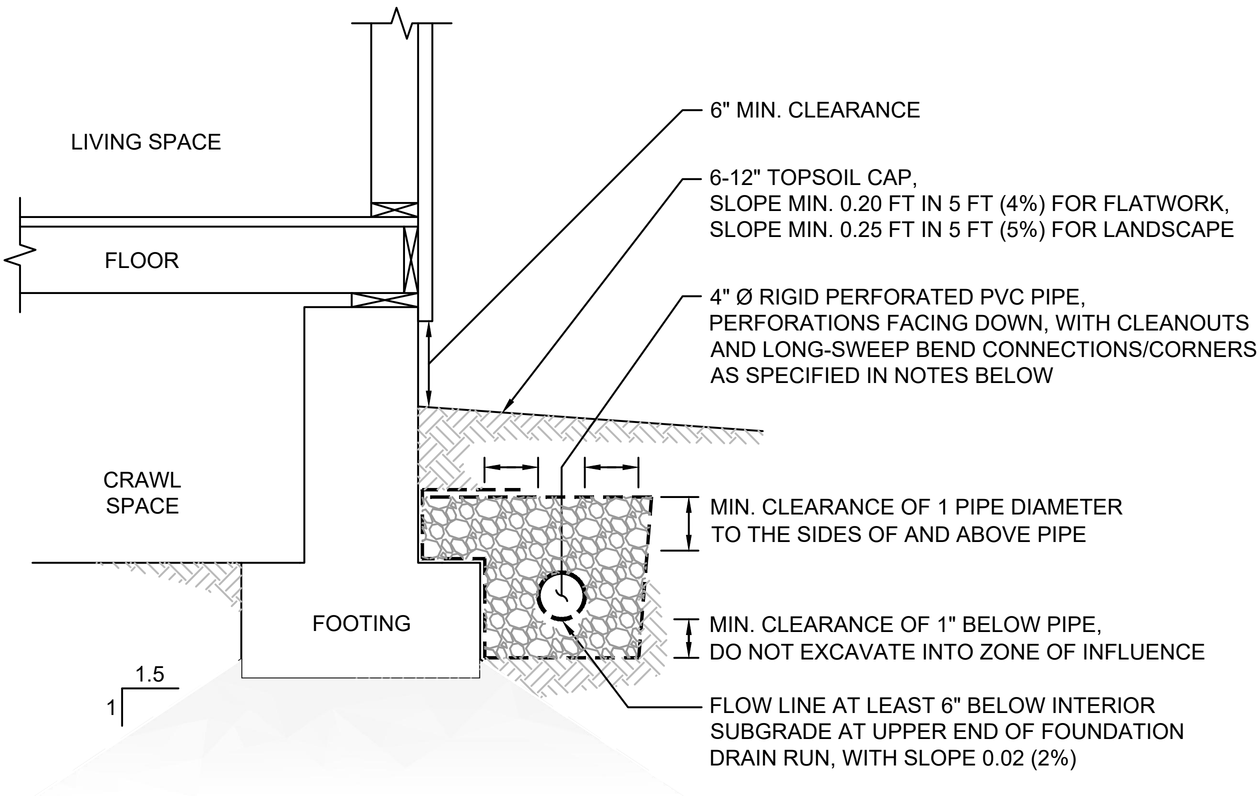
FIGURE 11



SCHEMATIC FOUNDATION SUBDRAIN

NOT TO SCALE

NOTES:

1. Do not connect downspout or area drain to foundation drain system.
2. Clean outs should be installed at the upslope end and at significant direction changes of the perforated pipe, or as specified by designer. Additionally, all angled connectors/corners shall be long-sweep bend connections.
3. Perforated pipe shall be SCH 40, SDR 35, or approved equivalent for depths less than 20 feet. Perforations shall face down and slope 0.02 (2%) to a gravity outlet.
4. Discharge through rigid, non-perforated pipe, slope 0.02 (2%).
5. Foundation drainage around pipe to consist of clean, free draining 3/4" crushed rock (Class 1B Permeable Material) wrapped in filter fabric (Mirafi 140N or equivalent) or Class 2 Permeable Material.
6. All footings and stem walls adjacent to interior living spaces shall be water/vapor proofed as specified by the project architect, structural engineer, or other governing consultant.
7. Do not undermine foundation or excavate into the footing zone of influence.
8. All work and materials shall conform with Section 68, of the latest edition of the Caltrans Standard Specifications.

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Novato, CA 94947
T 415 / 382-3444
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SCHEMATIC FOUNDATION DRAIN DETAIL

Proposed Residential Subdivision
11798 Highway One
Point Reyes Station, California

Project No. 3614.001 Date: 10/24/2024

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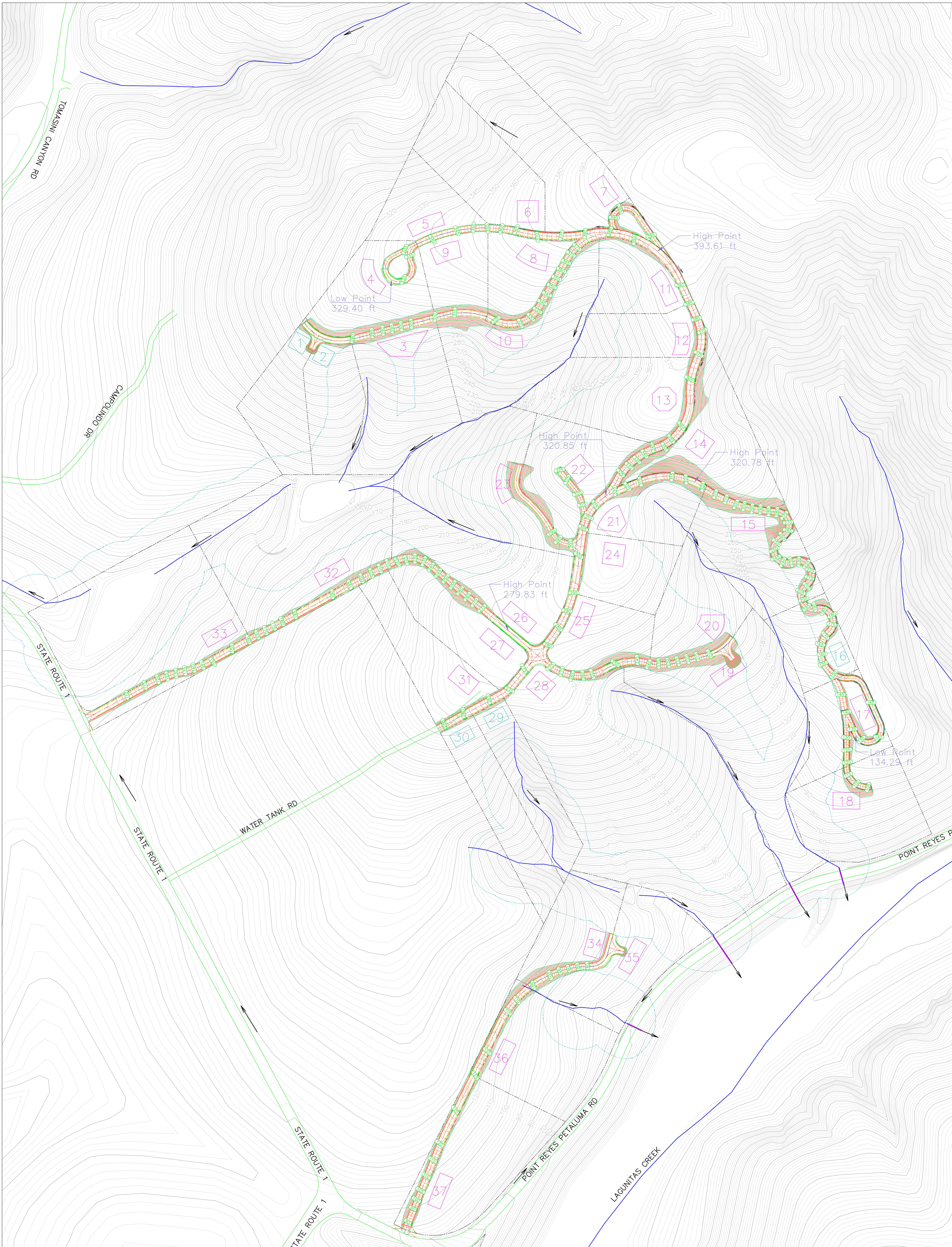
FIGURE

APPENDIX A: SITE MAP


TOMASINI CANYON RD
CAMPOLINDO DR
STATE ROUTE 1
WATER TANK RD
POINT REYES PETALUMA RD
LAGUNITAS CREEK
High Point
393.61 ft
Low Point
329.40 ft
High Point
320.85 ft
High Point
320.78 ft
High Point
279.83 ft
Low Point
134.29 ft