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- **Geologic & Geotechnical Investigation**
- **Site Improvement 'Set-back' Clarifications**

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GEOLOGIC & GEOTECHNICAL INVESTIGATION
FERNWOOD CEMETERY EXPANSION
301 TENNESSEE VALLEY ROAD
MILL VALLEY, CALIFORNIA 94941

October 29, 2025

Job No. 1734.023

Prepared for:
Mr. Tyler Cassity
Fernwood Cemetery

CERTIFICATION

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1.0 INTRODUCTION

This report summarizes our Geologic and Geotechnical Investigation for the proposed Fernwood Cemetery expansion located at 301 Tennessee Valley Road in Mill Valley, California. As shown on Figure 1, the project site is located along Altra Trail, just south of the Donahue Street cul-de-sac in Marin City, California.

Our work has been performed in accordance with our Agreement for Professional Services dated July 8, 2025. The purpose of our services is to provide geotechnical recommendations and design criteria to aid in the design and construction of the planned improvements The scope of our services includes:

- Review of readily available, published public regional geologic mapping and geotechnical background information;
- Subsurface exploration consisting of five soil borings to a maximum depth of 10-ft and two seismic refraction lines with accompanying 2D shear wave profile;
- Summary of existing subsurface conditions in the project area;
- Evaluation of relevant geologic hazards (including seismic shaking, liquefaction, flooding, settlement, landsliding, and other hazards) and development of hazard mitigation measures as warranted.
- Development of recommendations and design criteria for geotechnical project components, including seismic design criteria and recommendations for site grading and excavations; and
- Preparation of this report.

Issuance of this report completes our Phase 2 services for the project. Subsequent phases of work will include geo-civil design of site retaining walls; geotechnical consultation during the bidding process; and observation and testing of geotechnical-related work items during construction.

2.0 PROJECT DESCRIPTION

Based on review of preliminary plans (Adobe Associates, 2025), shown on Figure 2, the project generally consists of constructing new burial plots, access roads, and related improvements in currently undeveloped areas southeast of the exiting cemetery. Access will be provided via a southward extension of Donahue Street in Marin City, which will connect to the existing cemetery infrastructure via new interior driveways. Cut and fill slopes on the order of 6- to 8-feet high will be locally required to grade the site for the driveway development. We anticipate additional improvements may include new underground utilities, concrete and/or asphalt pavement, retaining walls, landscaping, and others.

3.0 SITE CONDITIONS

The proposed development area comprises roughly 60 acres of undeveloped land straddling the ridgeline between Marin City and the unincorporated Tennessee Valley area of southeastern Mill Valley. The project area is bounded to the north, south, and east by undeveloped open space areas and to the west/northwest by the existing cemetery grounds. Site access is provided via a fire road extending from the southern terminus of Donahue Street at the northeast corner of the work area and by the existing cemetery access road at the northwest corner. Surface elevations within the work area range from a maximum of about +425-feet near an existing water tank and along the ridgeline in the southeastern corner of the proposed work area to a minimum of about +175-feet near the existing cemetery road. Slopes throughout the site are typically inclined between about 2:1 (horizontal: vertical) and 3:1, with slightly flatter slopes along the ridgeline.

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

As shown on Figure 3, regional geologic mapping (Rice and Smith, 1976) indicates that the central and southern parts of the project site are underlain by Franciscan Mélange, a tectonic mixture of resistant rocks (including sandstone, greenstone, greywacke, chert, and others) embedded in a matrix of weak, sheared shale. Outcrops of chert, glaucophane schist, and sandstone are mapped in various locations. Drainages in the central part of the site are shown to be underlain by colluvium, and two small landslides are shown within the drainage downslope of the western roadway alignment. A prominent outcrop of chert and larger landslide are mapped near the southwestern edge of the project area.

Descending slopes east of Alta Trail (above Marin City) are also shown to be underlain by mélange. One landslide is mapped in an incised drainage channel on the eastern side, and zones of soil creep are shown downslope of the entrance at Donahue Street. The northern portion of the site is shown to be underlain by a mass of sandstone within the larger Franciscan Mélange zone.

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-Rodgers Creek 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 4. The nearest known active faults are the San Andreas and Hayward Rodgers Creek Faults, which are located roughly 9.9 kilometers southwest and 19.0 kilometers northeast 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 1906 Great San Francisco Earthquake (M=7.8), which resulted in well-documented surface ruptures throughout the Olema Valley northwest 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 Mill Valley and the surrounding vicinity, 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 1989 Loma Prieta Earthquake (M=6.9) caused moderate shaking but limited damage in Mill Valley and greater Marin County. A map illustrating the epicentral locations of earthquakes of M > 4.5 since 1900 is 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. 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 19.0 kilometers northeast of the site, with a probability of 33 percent. The San Andreas Fault is located approximately 9.9 kilometers southwest 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 Aerial Photograph Review

We reviewed select aerial photographs of the site spanning between 1946 and 2022 (Historic Aerials, 2025). As of 1946, the area of Donahue Street is developed with residential structures, supporting the nearby Marinship shipyards, and the fire road along the ridgeline at the eastern boundary of the work area is visible.

Several apparent landslides are visible occupying ravine areas in the central part of the site, which were not as densely vegetated as current conditions. An additional, relatively small, landslide is visible on the west side of the ridgeline fire road in the southeast corner of the site, generally near the planned hairpin turn at the south end.

By 1968, the Marinship structures have been demolished, Donahue Street has been re-aligned to its current location, and the water tank at the peak in the northeast corner of the site has been constructed, along with the access road extending from the tank to the existing cemetery.

A small landslide appears to have occurred sometime prior to 1982 on the northern flank of the central ravine, downslope of the water tank access road. Over time, residential development filled in along Donahue Drive and along Tennessee Valley Road and some grading work appears to have been performed in the southern part of the existing cemetery between the late 1980's and early 2010's. We also note that formerly grassy areas have become significantly more thickly vegetated with coastal scrub, manzanita, poison oak and similar shrubs over time, but no other significant changes in site conditions were observed.

3.4 Geotechnical Reference Material Review

We reviewed the results of previous work performed by Miller Pacific at the existing cemetery facility, including a 2022 slope evaluation addressing areas near the southwest corner of the currently proposed development area and a landslide repair at the existing access road northwest of the planned new work. Materials we reviewed included the following:

- Miller Pacific Engineering Group (2022), "Slope Evaluation, Fernwood Cemetery, Mill Valley, California", Project No. 1734.021, dated May 26, 2022.

- Miller Pacific Engineering Group (2023), "Fernwood Cemetery, Landslide Repair & New Retaining Wall, 301 Tennessee Valley Road, Mill Valley, California 94941", Project No. 1734.022, dated August 11, 2023.

- Miller Pacific Engineering Group (2022), "Geotechnical Site Safety Evaluation, Fernwood Cemetery, Mill Valley, California", Project No. 1734.022, dated August 16, 2023.

3.5 Site Reconnaissance and Present Surface Conditions

We performed a site reconnaissance on March 25, 2025 to observe and document existing conditions at the site and to prepare a preliminary site geologic map. We observed that existing improvements within the work area are generally limited to graded unpaved fire roads along Alta Trail and the MMWD water tank located just southwest of the Donahue Street cul-de-sac. Vegetation in the ridgeline and upper slope areas consists of dense coastal scrub, brush and sparse trees with some native grasses and ground cover, while the lower slopes and drainages are vegetated with eucalyptus, mature oak, bay and laurel trees along with dense poison oak, broom, and other shrubs.

As shown on Figure 2, geologic mapping performed during our reconnaissance generally verifies the regional mapping and indicates the site is underlain by Franciscan Melange, which is composed primarily of moderately hard fine-grained sandstone. These rocks, observed in outcrops at many locations around the property, are typically moderately hard, moderately strong, and closely to moderately fractured. Local exposures of very hard, very strong meta-graywacke and dark gray shale were noted along the fire road near the eastern margin of the work area, and a prominent outcrop of very hard, locally sheared meta-greenstone and meta-chert was observed just beyond the southern limit of the planned development.

We observed the two landslides previously mapped by Miller Pacific in the northern portion of the site, on a southwest facing hillside during our 2022 work. We noted that scarps remain generally overgrown and as described in that report, and did not observe any significant evidence indicative of recent movement. We were unable to confirm other landslides mapped previously by Miller Pacific and observed in the aerial photographs, as those areas are typically obscured by very thick brush. The observed landslides are indicated on the Preliminary Geologic Map, Figure 6.

3.6 Field Exploration and Laboratory Testing

We explored subsurface conditions near the proposed improvements on September 9, 2025 with five borings. We supplemented our borings with two seismic refraction lines performed on September 25. The approximate locations of our soil borings and seismic refraction lines are shown on Figure 2.

3.6.1 Soil Borings

Our soil borings were excavated using track mounted drilling equipment which were advanced to a maximum depth of 10-feet below ground surface. The borings were logged in the field by our Geologist and soil samples were obtained at select intervals for laboratory testing. Brief descriptions of the terms and methodology used in classifying earth materials are shown on the Soil and Rock Classification Charts and are presented along with the boring logs in Appendix A.

Laboratory testing of soil samples from the exploratory boring included determination of moisture content, dry density, unconfined compressive strength, and Atterberg Limits (Plasticity Index). The results of our laboratory tests are presented on the boring logs. Our laboratory testing program is discussed in greater detail in Appendix A.

3.6.2 Seismic Refraction

Two seismic refraction lines were performed in the field by our geologists to supplement our soil boring data. The data obtained in the field was processed by licensed geophysicists at Terean, Inc. The results of the seismic refraction surveying is presented in Appendix B.

Seismic refraction surveys provide shear wave velocities of the subsurface soil and rock materials to depths exceeding 100-feet. Shear wave velocities are required to determine the Site Classification per the current ASCE 7 and CBC codes. Additionally, shear wave velocities can be correlated to the 'rippability', or ease of excavation, of the subsurface soil and rock materials.

The results of seismic refraction line #1 (SR-1), performed from north to south, revealed 4 distinct layers. The first layer is roughly 25-feet thick and exhibits shear wave velocities ranging from 1,200- to 1,500-feet per second (fps). This shear wave velocity range is considered 'rippable'; however, some shear wave velocities exceeding 1,500-fps were observed indicating some localized zones of marginally 'rippable' material. The second layer is roughly 15- to 20-feet thick and exhibits shear wave velocities of 2,400-fps indicating the weathered bedrock is marginally- to non-rippable. The shear wave velocities drop back down to 1,200- to 1,500-fps in the third layer, again indicating rippable material. Non-rippable/competent bedrock was observed 60- to 70-feet below the ground surface.

The results of seismic refraction line #2 (SR-2), performed north to south, revealed 4 distinct material layers. The results of the upper 5-feet exhibit shear wave velocities between 600- and 800-fps, indicating soil material which is easily rippable. The second layer is roughly 10- to 15-feet thick and exhibits shear wave velocities between 1,000- and 1,500-fps, indicating rippable soil and weathered bedrock. A roughly 10- to 30-foot layer of material exhibiting a shear wave velocity of approximately 1,500-fps underlies the second layer and indicates rippable weathered bedrock. An inclusion of harder and more competent bedrock was observed on the southern end of SR-2, with shear wave velocities greater than 2,200-fps, indicating non-rippable material. Competent bedrock, with shear wave velocities exceeding 2,200-fps, was observed roughly 45- to 50-feet below the ground surface.

3.7 Subsurface Conditions and Groundwater

Based on our subsurface exploration, the site is underlain by thin surface colluvial soils over shallow bedrock. Our borings generally encountered between 1.0 and 5.0 feet of sandy clay and clayey silt overlying sandstone and shale mélange bedrock, which extended from the base of the colluvial soils to the maximum explored depth below ground surface.

Groundwater was not encountered in any of our borings during exploration. However, because the borings were not left open for an extended period, a stabilized depth to groundwater may not have been observed. Groundwater elevations fluctuate seasonally, and higher groundwater levels may be present during or following periods of intense rainfall. Perched water tables may also exist within the soil layers or at the soil-rock interface.

4.0 GEOLOGIC HAZARDS EVALUATION

This section summarizes our 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 at the site. 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 noted previously, the site lies about 9.9 kilometers east of San Andreas Fault.

We did not observe any evidence indicative of active or historic faulting during our review of LiDAR topographic data or during 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 engineered mitigation is necessary.*

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 84th percentile ($+1\sigma$) 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]	*+1σ PGA*[2,4]
San Andreas	8.0	9.9 km	0.37g	0.67 g
Rodgers Creek–Hayward	7.6	15.6 km	0.26 g	0.47 g
San Gregorio	7.4	21.9 km	0.19 g	0.34 g
West Napa	7.0	45.7 km	0.08 g	0.14 g
Maacama	7.6	59.5 km	0.09 g	0.16 g

Notes:

1. Values determined using USGS Earthquake Scenario Map (BSSC 2014), accessed 2025.
2. USGS Quaternary Fault and Fold Database, https://www.usgs.gov/natural-hazards/earthquake-hazards/faults.
3. Values determined using V_{S30} = 1,800 ft/s (550 m/s) based on Site Class "C" for measured "soil and soft 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	0.92 g
10% in 50 years	475 years	7.7	0.50 g

Reference: USGS Unified Hazard Tool, https://earthquake.usgs.gov/hazards/interactive/ (accessed 2025)

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 (https://www.usgs.gov/programs/earthquake-hazards/modified-mercalli-intensity-scale) 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 Incorporation of engineered recommendations.*
Recommendations:	*At a 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 are presented in Section 5 of this report. We understand that these recommendations will be incorporated into the final plan set.*

4.3 Slope Instability/Landsliding

Slope instability generally occurs on relatively steep slopes and/or on slopes underlain by weak materials. During our previous site work and this feasibility study, we mapped several landslides around the site. Most landslides appear to consist of relatively shallow earth flows occupying the upper reaches and flanking slopes of the incised drainages around the site, as shown on Figure 2. The margins of these landslides are typically marked by near-vertical head scarps ranging from about 3- to 5 feet high, and slide depths are estimated at between about 5 and 10 feet. Given the history of instability at the project site, erodible surface soils, and steep natural grades, we judge that the risk of landsliding is high.

Evaluation:	*Less than Significant with Incorporation of Engineered Recommendations.*
Recommendations:	*Mitigation measures may include a combination of site grading, new retaining structures, minimum setbacks from slide scarps, and/or new surface and subsurface drainage improvements. Additional discussion regarding slope-stability mitigation measures and site drainage considerations is provided in Section 5 of this report. We understand that these recommendations have been incorporated into the final plan set.*

4.4 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 subsurface exploration, we observed that surface soils typically consist of medium-stiff, sandy clay and clayey silt. The silty soils typically exhibit lower cohesion and will be prone to erosion. Minor fill and gully erosion was noted in steeper portions of the graded dirt access roads. Drainages in between the ridgeline areas exhibit significant downcutting/incision, and the landslides which locally occupy channel banks and flanking slopes are indicative of erosion and scour within the channels 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 Incorporation of Engineered Recommendations.*
Recommendations:	*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 unless other mitigation is provided. 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 widely dispersed via dissipater packs. At minimum, the project Civil Engineer should design site drainage systems to accommodate runoff from a 100-year storm event. Additional discussion regarding site erosion control and drainage considerations is provided in Section 5 of this report. We understand that these recommendations will be included into the final planset.*

5.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our investigation, it is our opinion that development of the site is feasible from a geotechnical perspective. The primary geotechnical issues to be addressed during project planning and design include; providing minimum setbacks or effective mitigation for landsliding risks where roadway alignments are in close proximity to landslide and drainage channel banks; and providing effective site drainage to further reduce the risk of slope instability, adverse roadway performance and impacts to downslope properties. Geotechnical recommendations and guidelines to address these and other planning-level considerations are presented in the following sections.

5.1 Seismic Design

Minimum mitigation of ground shaking includes seismic design of new structures in conformance with the provisions of the most recent edition (2022) of the California Building Code. The magnitude and character of these ground motions will depend on the earthquake and the site response characteristics. We developed seismic design criteria in general accordance with ASCE 7-22 as summarized in Table 3.

Table 3 – 2022 California Building Code Seismic Design Criteria

Parameter	***Design Value***
Site Class	C
Risk Category	II
Spectral Acceleration at Short Periods, S_S	1.74 g
Spectral Acceleration at Period of 1 sec, S_1	0.66 g
Adjusted Spectral Acceleration at Short Periods, S_{MS}	1.84 g
Adjusted Spectral Acceleration at Period of 1 sec, S_{M1}	0.94 g
Design Spectral Acceleration at Short Periods, SDS	1.23 g
Design Spectral Acceleration at Period of 1 sec, SD1	0.63 g
Adjusted MCE_G Peak Ground Acceleration, PGA_M	0.66 g

Reference: OSHPD Seismic Design Maps, accessed on October 10, 2025.

5.2 Site Grading Considerations

Plans indicate that site grading is expected to include cuts and fills a maximum of 10-feet high will be required to develop level driveways/access roads. All earthwork should be performed in general accordance with the recommendations and criteria outlined in the following sections.

5.2.1 Site Preparation

Clear pavements, old foundations, over-sized debris, and organic material from areas to be graded. Debris, rocks larger than six inches, and vegetation are not suitable for structural fill and should be removed from the site. 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.

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.

5.2.2 Excavations

Excavations for new roadways, utilities and other improvements in areas mapped as Franciscan Complex bedrock (Map Unit fm as shown on Figure 2) are anticipated to encounter 1- to 5 feet of silty to clayey residual soils over hard and strong sandstone and shale bedrock. In areas mapped as landslides, similar bedrock is likely to be overlain by thicker deposits of unconsolidated silty to sandy landslide debris. Temporary cut slopes may be required during construction. For planning purposes, cut slopes in soil may be designed for OSHA Type “C” soils while weathered bedrock may be designed for OSHA Type “A”.

Considering the site is steeply sloping, we generally recommend that permanent cut slopes be avoided, and excavations be supported by new retaining structures. If cut slopes are planned, we should evaluate these slopes on a case-by-case basis and provide supplemental recommendations for their construction. Any areas disturbed during grading should be planted following construction reduce the potential for sloughing and erosion.

Based on our seismic refraction survey the upper roughly 25-feet is considered ‘rippable’ to ‘marginally-rippable’; therefore we anticipate that shallow excavations can likely be accomplished with “traditional” equipment, such as medium to large excavators and dozers. Zones of hard competent rock may be encountered within the upper 25-feet, especially in ridgeline areas and where chert, greenstone, or other resistant rock bodies exist. Specialized hard rock excavation techniques and equipment, such as jackhammers and hydraulic breakers may be required to excavate the more competent weathered bedrock. Therefore, we recommend inclusion of a line item and clear definition for “hard rock excavation” in the project bid documents. If hard rock is encountered during construction which prohibits excavation to the required depths, we should be consulted to observe conditions and revise our recommendations and/or design criteria as appropriate. Reducing planned excavation depths will also reduce the potential for hard rock excavation and resulting costs.

5.2.3 Fill Materials

We generally recommend that fills be avoided to the extent possible, and any new fills be supported by retaining walls. Fill slopes are not considered feasible due to the steeply sloping conditions unless the base of the fill is supported by a retaining structure. If used, fill slopes should be inclined at 2:1 or flatter and the fills should be benched and constructed with subsurface drains. Fill slope inclinations of up to 1.5:1 may be achieved if geosynthetic reinforcement are incorporated into the fill.

Fill materials should generally consist of low-expansive materials that are free of organic matter, have a Liquid Limit of less than 45 (ASTM D 4318), a Plasticity Index of less than 20 (ASTM D 4318), and a minimum R-value of 20 (California Test 301). The fill material should have a maximum particle size of four inches. Onsite soils are likely suitable for reuse as fill provided, they are processed to meet the criteria noted above. Any imported fill material needs to be tested to determine its suitability.

Fill materials should be moisture conditioned to just above the optimum moisture content prior to compaction. Properly moisture conditioned fill materials should subsequently be placed in loose, horizontal lifts of eight-inches-thick or less and uniformly compacted to at least 90 percent relative compaction. Where fill thicknesses are greater than five feet, fill materials should be compacted to at least 95 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 Foundation Design

New retaining walls along the downhill side of the driveways or elsewhere in sloping ground 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. If retaining walls are constructed in “cut” areas underlain by weathered bedrock along the inboard side of the driveways, then conventional shallow foundations could be appropriate provided that adequate site drainage and other improvements are provided to maintain acceptable slope-stability factors of safety. The spread footing and drilled pier foundations should be designed using the criteria summarized in Tables.

Table 4 – Spread Footing Design Criteria

Shallow Foundations	
Minimum Embedment[1]	18 inches
Minimum Width	15 inches
Allowable Bearing Pressure, Bedrock[2, 3]	3,500 psf
Base Friction Coefficient	0.40
Lateral Passive Resistance[4,5]	450 pcf
Drilled Piers	
Minimum Bedrock Embedment	5 feet
Minimum Diameter	18 inches
Allowable Skin Friction[3,5]	
Soil	Ignore
Weathered Bedrock	2,000 psf
Lateral Passive Resistance[1,6]	
Soil	Ignore
Weathered Bedrock	450 pcf

Notes:

1. Maintain minimum of seven feet of horizontal distance between the outer edge of footing and face of nearest adjacent slope.
2. Design shallow foundations to similar bearing pressures (i.e., size footing widths to maintain relatively uniform bearing loads).
3. Increase design values by 30% for total design loads including seismic.
4. Equivalent fluid pressure, not to exceed 4,5000 psf. Neglect upper 12 inches unless confined by concrete or asphalt.
5. Use 80 percent of allowable skin friction in evaluating uplift capacity.
6. Equivalent fluid pressure. Apply passive pressure over two pier diameters.

5.4 Retaining Walls

Retaining walls will be needed to retain new cuts and fills where accommodation of recommended setbacks or permanent slope inclinations are infeasible or undesirable. 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, while soil nail and shotcrete retaining walls Mahy be more efficient where heights approach 8 feet or more.

Retaining walls should be designed using the foundation design criteria presented in Section 5.3 and the lateral earth pressures shown in Table 5. Retaining walls that can slightly deflect at the top can be designed using the unrestrained criteria shown below. Walls that are structurally connected and not allowed to deflect (e.g., tied-back walls) are restrained and are commonly designed using a uniform active earth pressure distribution rather than an equivalent fluid pressure.

Table 5 – Lateral Earth Pressures for Retaining Wall Design

Active Pressure	
Level Backfill	40 pcf
3:1 Backfill	50 pcf
2:1 Backfill	65 pcf
At-Rest Pressure	
Level Backfill	60 pcf
3:1 Backfill	70 pcf
2:1 Backfill	85 pcf
Tieback Design Criteria	
Minimum Diameter	6 inches
Minimum Bar Diameter	0.5 inches
Allowable Bedrock Skin Friction	2,000 psf

Notes:
1. Interpolate earth pressures for intermediate slopes.
2. Equivalent fluid pressure.
3. Uniform pressure, H is wall height in feet.

In addition to the pressures noted above, we also recommend the walls be designed to resist a uniform seismic surcharge equal to 12 times the retained height (in pounds per square foot). The factor of safety used in the retaining wall design should be reduced under seismic conditions as permitted by the governing code that is used for design.

5.5 Mitigation of Existing Landslides

In general, we anticipate that landslide mitigation for the purpose of the proposed development may be accomplished either by any combination of **avoidance** or by **landslide repair**.

> **Avoidance** would consist of imposing minimum setbacks for new structures and associated improvements from landslide areas. We recommend that the new driveways be provided with minimum setbacks of 25 feet from mapped landslides (map unit Qls), as shown on Figure 2. If and where such setbacks cannot be accommodated or are undesirable, then landslides 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 relatively hard and strong bedrock, we judge that landslide repairs could include 1) excavation of unstable material, installation of subsurface drainage and construction of compacted earth fill buttresses; 2) design and construction of retaining structures; or (3) a combination of the above.

5.6 Site Drainage Considerations

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. Surface runoff should be collected into a closed-pipe system and conveyed to a location unlikely to cause significant erosion.

Special care should be taken with respect to discharge and dispersal of site runoff. Infiltration-type systems are not recommended given the potential for slope instability at the site. As an alternative, we recommend considering a stormwater detention system, where excess runoff is stored in structures with impermeable bottoms and fitted with appropriately sized outfalls as to achieve the required peak flow rate.

Drainages on both flanks of the north-trending ridgeline are deeply incised, and several small debris flows are apparent, based on aerial photograph review and regional geologic mapping. Residential developments at the base of the slopes in Marin City and along Tennessee Valley Road generally do not have substantial setbacks from the base of the drainages, and are potentially exposed to impact by debris flow landslides, were they to occur. Special care should be taken in these areas to avoid concentrating runoff in any one location.

In general, it is recommended that surface grades be designed so as to mimic “natural” sheet flow drainage conditions. Ideally, any runoff flowing to the east would be conveyed via solid pipes and connected to existing municipal storm drain system in Marin City. If such connection is determined to be infeasible, then runoff should be dispersed via level spreader-type dissipators. To the extent possible it is recommended that dissipators be sited on shallow slopes and avoiding areas overlying incised channels and/or known landslides.

5.7 Underground Utilities

Excavations for utilities will likely encounter relatively silty and clayey residual soil deposits over sandstone and shale 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.

---

[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.

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.

5.8 Asphalt Pavements

Typically, asphalt pavement sections are designed utilizing two variables, the R-Value (a measure of the subgrade resistance) and the Traffic Index (a measure of the amount of daily traffic). Based on the subsurface conditions we judge an R-Value of 10 is appropriate for the site. The project Civil Engineer should select an appropriate Traffic Index for the driveway, or the existing section could be "matched". Based on this assumed R-value, we have calculated thicknesses for asphalt pavements for the new driveway in accordance with Caltrans procedures for flexible pavement design. Our calculations assume an R-value of 10 for subgrade soils and a range of traffic indices from 4 to 7 depending on the expected traffic loads for a twenty-year design life. The Civil Engineer should be responsible for determining the appropriate traffic index for use in design. In general, areas expected to experience loading from heavy vehicles should be designed using the higher traffic index, while parking areas and other lightly loaded areas can utilize a thinner pavement section based on the lower traffic index. The recommended pavement sections are presented in Table7.

Table 7 – Asphalt-Concrete Pavement Sections

Traffic Index	Asphalt Concrete	Aggregate Base
4.0	2.5 inches	7.0 inches
5.0l	3.0 inches	9.0 inches
6.0	3.5 inches	12.0 inches
7.0	4.0 inches	15.0 inches

In pavement areas, the upper 12 inches of subgrade should be compacted to at least 95 percent relative compaction. The aggregate base and asphalt-concrete should conform to the most recent version of Caltrans Standard Specifications and should be compacted to at least 95 percent relative compaction. Additionally, the subgrade and aggregate base should be firm and unyielding under heavy, rubber-tired construction equipment. 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.

6.0 SUPPLEMENTAL GEOTECHNICAL SERVICES

As project plans are nearing completion, we should review them to confirm that the intent of our geotechnical recommendations has been incorporated. We can also consult with project team to supplement or clarify geotechnical recommendations, if needed. During construction, we should be present intermittently to observe foundation excavations, retaining wall drainage and backfill, the installation of debris barriers or other slope improvement measures, subgrade preparation and compaction, proper moisture conditioning of soils, fill placement and compaction and other geotechnical-related work items. The purpose of our observation and testing is to confirm that site conditions are as anticipated, to adjust our recommendations and design criteria if needed, and to confirm that the Contractor’s work is performed in accordance with the project plans and specifications.

7.0 LIMITATIONS

We believe 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 Fernwood Cemetery 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 a detailed environmental assessment of the site.

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

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

8.0 LIST OF REFERENCES

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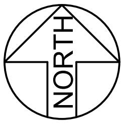
SITE COORDINATES
LAT. 37.874900°
LON. -122.520612°

SITE LOCATION

NOT TO SCALE



REFERENCE: Google Earth, 2025

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SITE LOCATION MAP

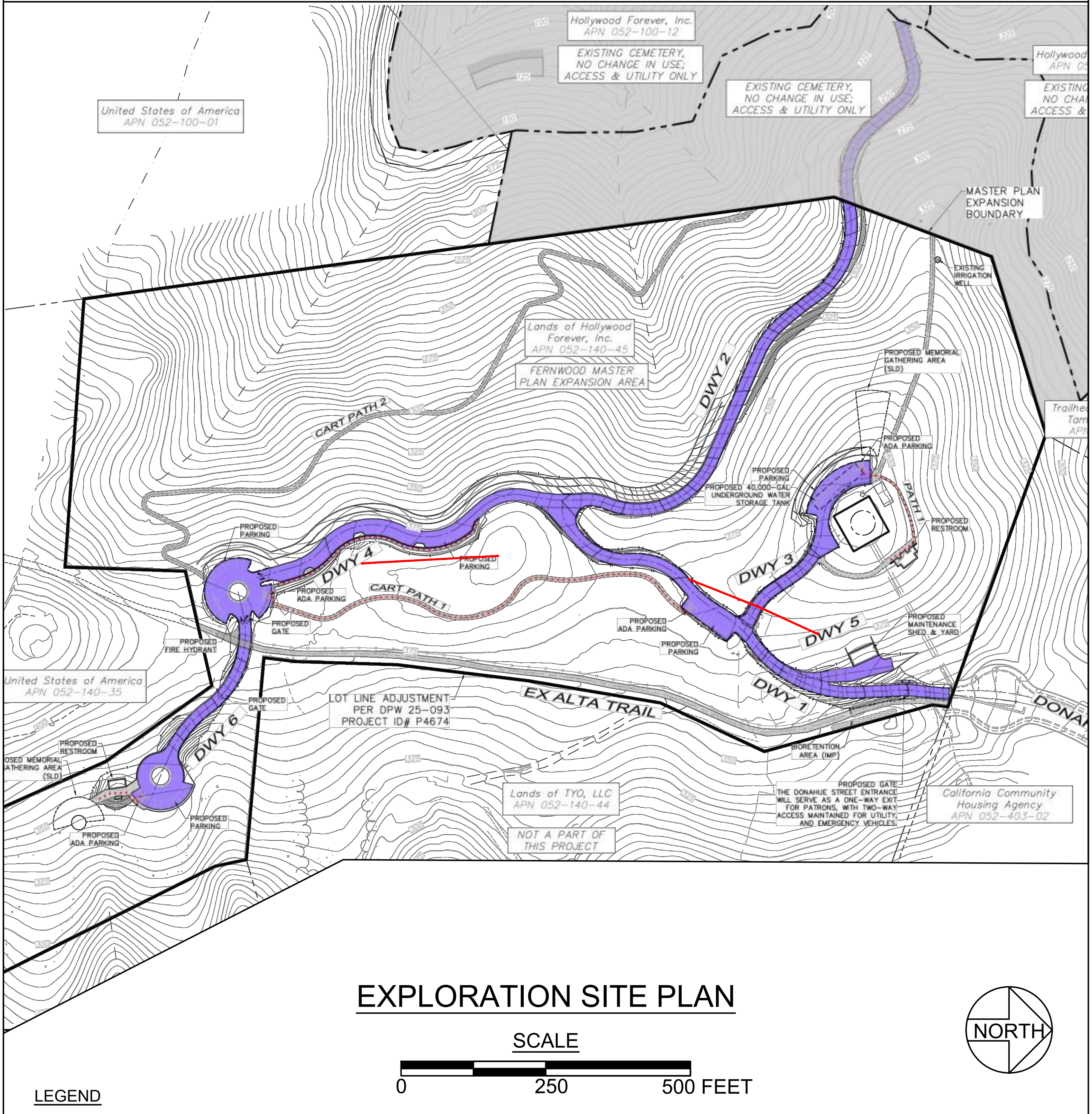
Fernwood Cemetery Expansion
Mill Valley, California

Project No. 1734.023

Date: 10/29/2025

Drawn QHC
Checked BSP

1
FIGURE



EXPLORATION SITE PLAN

LEGEND

— Approximate location of seismic refraction line, completed by MPEG, 2025

⊕ Approximate location of boring, completed by MPEG, 2025

REFERENCE: Adobe Associates, Inc. (2025), "Fernwood Cemetery, Master Plan", APN 052-140-45, dated November 14, 2025

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EXPLORATION SITE PLAN

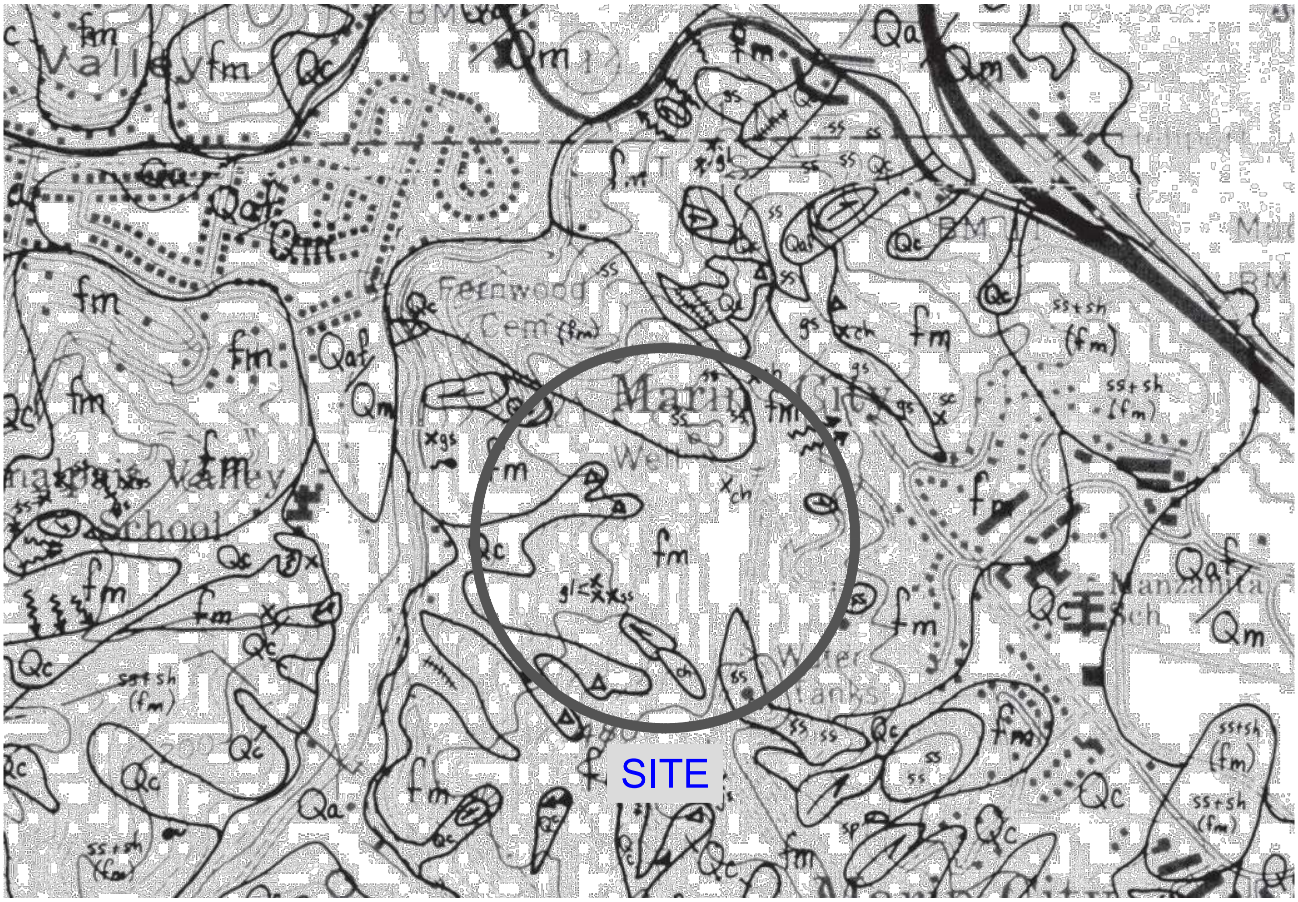
Fernwood Cemetery Expansion
Mill Valley, California

Project No. 1734.023

Date: 4/20/2026

Drawn ZMS
Checked BSP

2
FIGURE



REGIONAL GEOLOGIC MAP

NORTH

LEGEND

Qaf/Qm	Artificial fill over marine and marsh (Bay Mud) deposits (Quaternary): Deposits of rock, soil, garbage, and trash placed by man upon natural Bay Mud, mostly for engineering purposes.
Qal	Alluvium (Quaternary): Unconsolidated deposits of clay, silt, sand, and gravel underlying valley bottoms, consisting of materials transported and deposited by streams.
fm	Franciscan Melange: A tectonic mixture consisting of small to large masses of resistant rock types, principally sandstone, greenstone, chert, and serpentine, but including various exotic metamorphic rock types, embedded in a matrix of pervasively sheared or pulverized rock material. Exposures of rock masses within the melange matrix are indicated by the following lithologic symbols:

ss + sh	Greywacke (Cretaceous): Mainly graywacke type sandstone, with or without alternating beds of dark gray shale.
gs	Greenstone (Jurasic): Altered or metamorphosed basaltic igneous rocks
ch	Chert (Cretaceous and Jurassic): and allied silicious rocks. Mainly isolated prominent outcrops of reddish brown, greenish, or light gray, thinly bedded chert, but includes prominent exposures of red and yellow jasper.

REFERENCE: Rice, S.J. and Smith, T.C. , "Geology of the Lower Ross Valley, Corte Madera, Homestead Valley, Tamalpais Valley, Tennessee Valley, and Adjacent Areas Marin County, California," California Division of Mines and Geology (now California Geologic Survey), Scale 1:12,000, 1976

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REGIONAL GEOLOGIC MAP

Fernwood Cemetery Expansion
Mill Valley, California

Project No. 1734.023
Date: 10/29/2025

Drawn QHC
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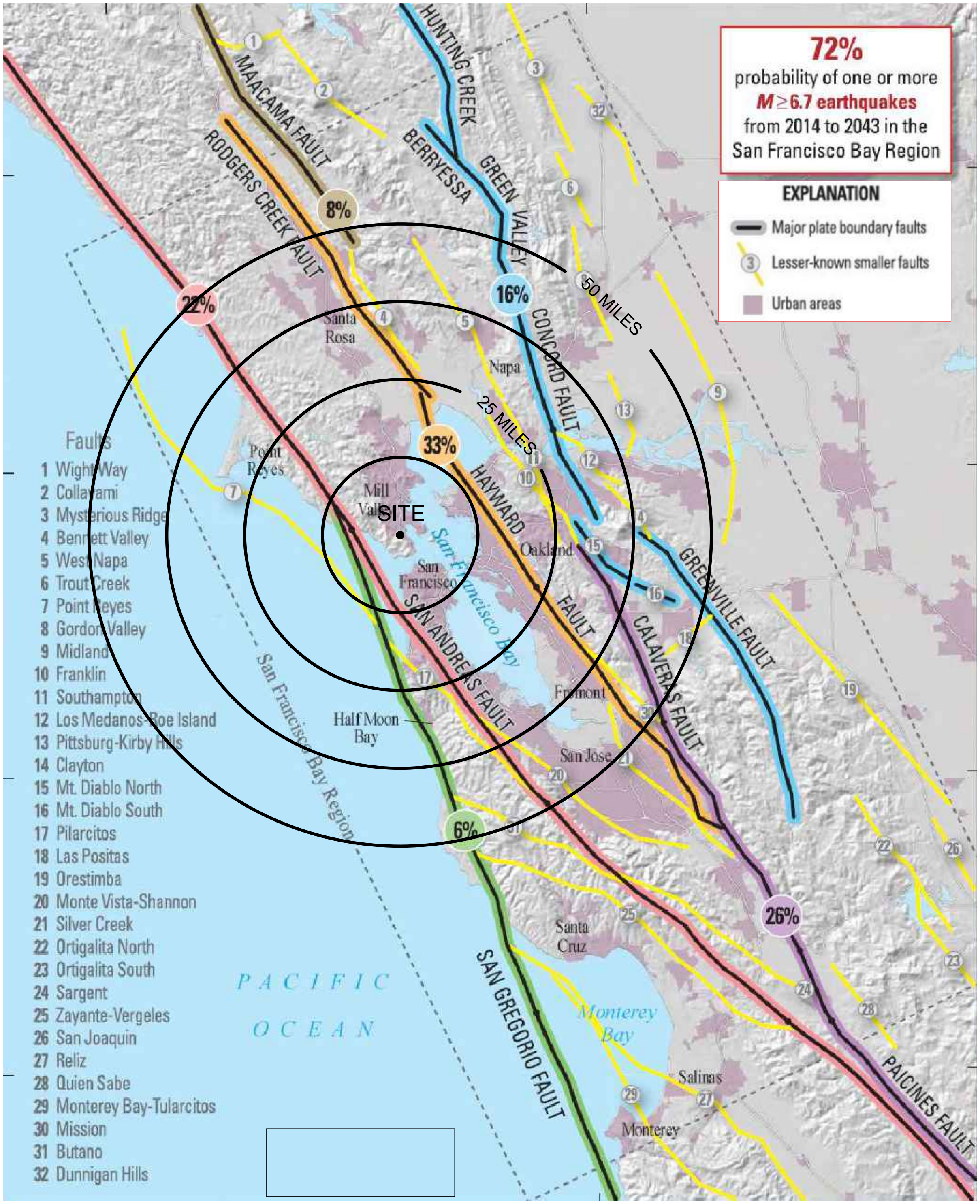
3
FIGURE



SITE COORDINATES
LAT. 37.874900°
LON. -122.520612°

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).

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ACTIVE FAULT MAP

Fernwood Cemetery Expansion
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Date: 10/29/2025

Drawn QHC
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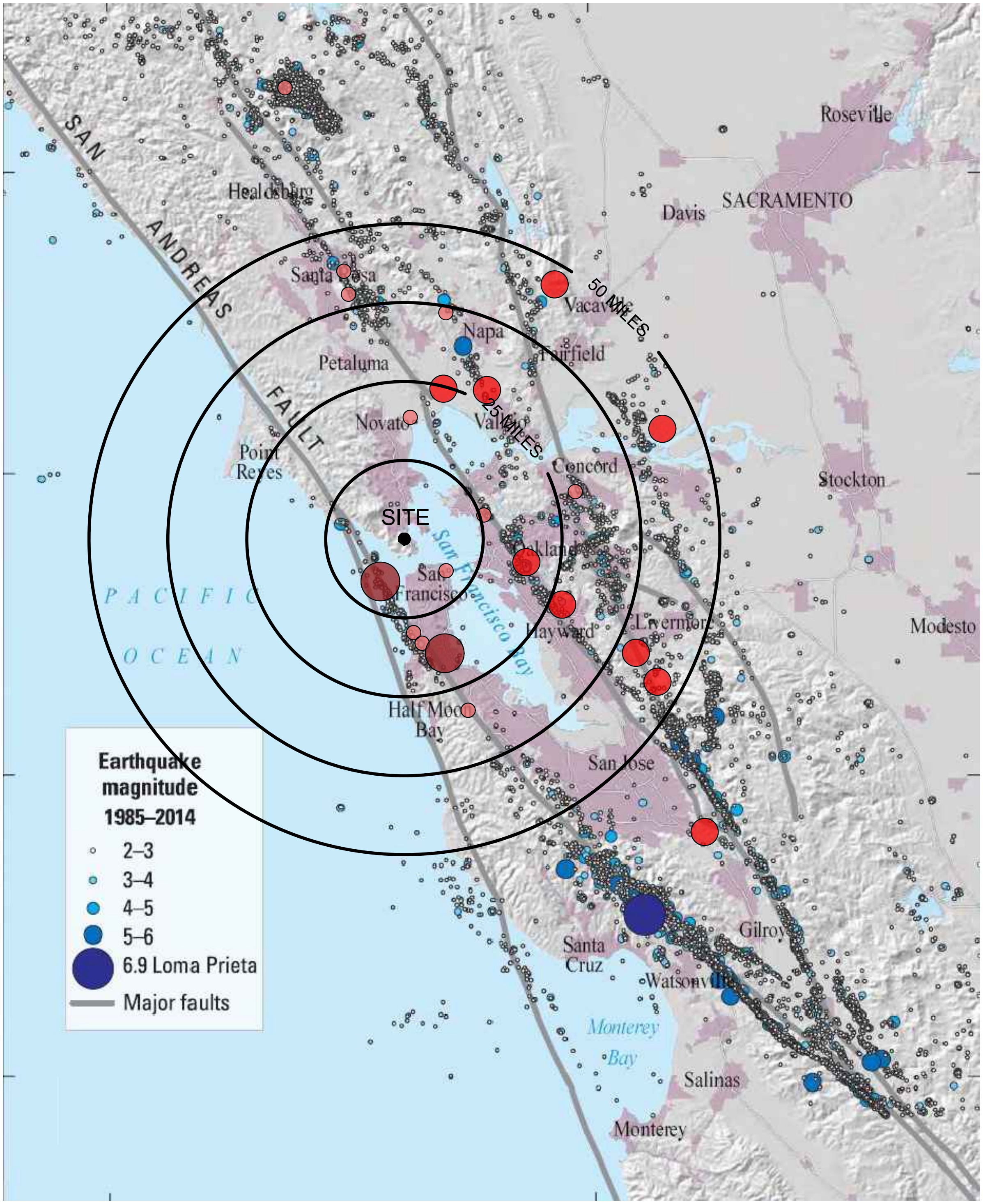
4
FIGURE



SITE COORDINATES

LAT. 37.874900°

LON. -122.520612°

SCALE

0 12.5 25 50 MILES

NORTH

LEGEND & DATA SOURCE:

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

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

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HISTORIC EARTHQUAKE MAP

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Date: 10/29/2025

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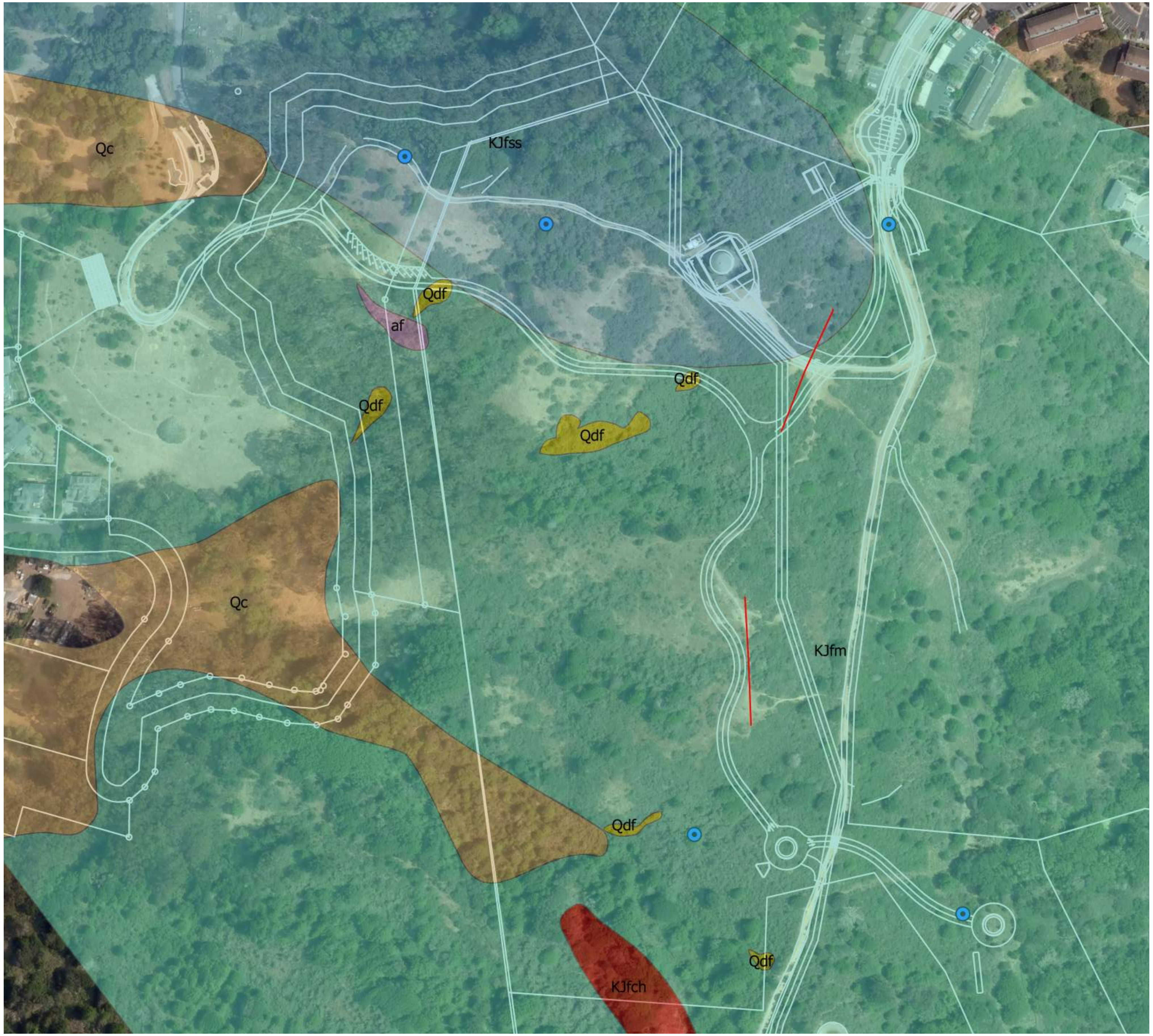
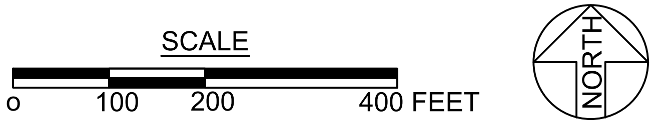
5

FIGURE



LEGEND

af	Artificial fill: Deposits of rock, soil, garbage, and trash placed by man upon natural surfaces, mostly for engineering purposes.
Qdf	Landslide or Debris Flow Scars: Unit composed of deposits of unconsolitdated and unsorted soil and rock debris that have moved downslope en masse or in increments due to flow or creep processes.
KJfm	Franciscan Melange: A tectonic mixture consisting of small to large masses of resistant rock types, principally sandstone, greenstone, chert, and serpentine, but including various exotic metamorphic rock types, embedded in a matrix of pervasively sheared or pulverized rock material.
KJfss	Franciscan Sandstone and Shale: Unit composed of sandstone and shale bedrock with very minor amounts of conglomerate. The sandstone is generally thick bedded, medium to coarse grained, light gray where fresh, light yellow brown where weathered. The shale is commonly dark gray, fine grained, thinly bedded, and exhibits iron oxide staining along joints.
KJFch	Franciscan Chert: Unit composed of generally isolated prominant outcrops of green, gray, or red chert and metachert.



REFERENCE: Adobe Associates, Inc. (2025), "Fernwood Cemetery Grading and Drainage Plan", APN 052-140-45, Project No. 23275, dated February 4, 2025, Revised September 24, 2025.
Marin Map Orthophoto Download, accessed October 8, 2025

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Drawn ZMS
Checked ZMS

PRELIMINARY GEOLOGIC MAP

Fernwood Cemetery Expansion
Mill Valley, California

Project No. 1734.023
Date: 10/29/25

FIGURE
6

APPENDIX A
SUBSURFACE EXPLORATION AND LABORATORY TESTING

A. SUBSURFACE EXPLORATION

We explored subsurface conditions on September 9, 2025 with five borings, and on September 25 with two seismic refraction lines at the approximate locations shown on Figure 2. The exploration was conducted under the technical supervision of our Field Geologist who examined and logged the soil materials encountered and obtained samples. The subsurface conditions encountered in the test borings are summarized and presented on the Boring Logs, Figures A-1 through A-7.

Disturbed samples were obtained using a 3-inch diameter, split-barrel Modified California Sampler with 2.5 by 6-inch tube liners. 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. Borings were grouted and capped with tamped cold-patch asphalt.

B. LABORATORY TESTING

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

- Laboratory Determination of Water (Moisture Content) of Soil, Rock, and Soil-Aggregate Mixtures, ASTM D 2216
- Density of Soil in Place by the Drive-Cylinder Method, ASTM D 2937
- Unconfined Compressive Strength of Cohesive Soil, ASTM D2166
- Atterberg Limits, ASTM D 4318

The moisture content, dry density, unconfined compressive strength and Atterberg Limits results are shown on the exploratory boring logs. The Atterberg Limits charts are presented on figures A-8 and A-9.

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

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

KEY TO BORING AND TEST PIT SYMBOLS

CLASSIFICATION TESTS

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

SAMPLER TYPE

- ■ MODIFIED CALIFORNIA
- ◫ STANDARD PENETRATION TEST
- ▨ THIN-WALLED / FIXED PISTON
- ⊠ HAND SAMPLER
- ⊠ ROCK CORE
- X DISTURBED OR BULK SAMPLE

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

STRENGTH TESTS

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

SAMPLER DRIVING RESISTANCE

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

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

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SOIL CLASSIFICATION CHART

Fernwood Cemetery Expansion
Mill Valley, California

Project No. 1734.023

Date: 10/29/2025

Drawn QHC
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.

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ROCK CLASSIFICATION CHART

Fernwood Cemetery Expansion
Mill Valley, California

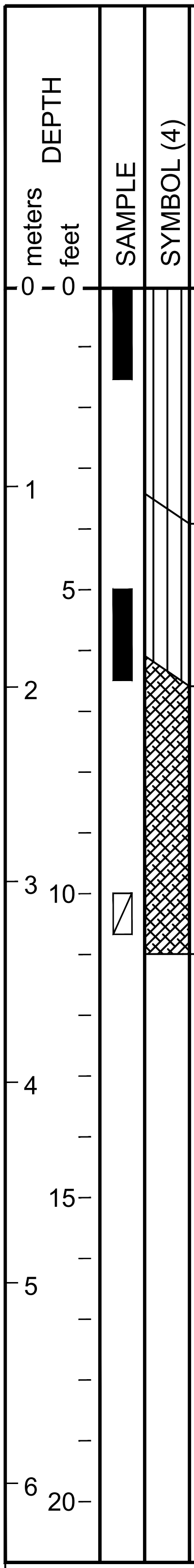
Project No. 1734.023
Date: 10/29/2025

Drawn QHC
Checked

A-2
FIGURE

BORING 1

EQUIPMENT: CAT-Mounted Hydraulic Drill Rig
with 4.0-inch Solid Flight Auger
DATE: 9/9/25
ELEVATION: 346 - feet*
*REFERENCE: Google Earth, 2025



Description	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
SILT with Sand and Clay (ML) Tan to brown, dry, stiff, low plasticity, ~20-30% silt, ~10-20% fine to coarse sand, organics present [Colluvium]	25	93	8.7	UC 850	LL:37 PI:10	
SILT with Sand and Clay (ML) Tan to dark brown with orange, dry, very stiff, low plasticity, ~20-30% silt, ~10-20% fine to coarse sand, organics present, rock fabric present at bottom of sample [Residual Soil]	63	124	8.1	UC 1100	LL:39 PI:16	
Shale Melange (fm) Brown to gray with orange, dry, low to moderately hard, weak, high to complete weathering, intensely to closely fractured [Bedrock]	71/8"		6.2			
Bottom of Boring at 10.5-feet. No groundwater encountered.						

▽ 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

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BORING LOG

Fernwood Cemetery Expansion
Mill Valley, California

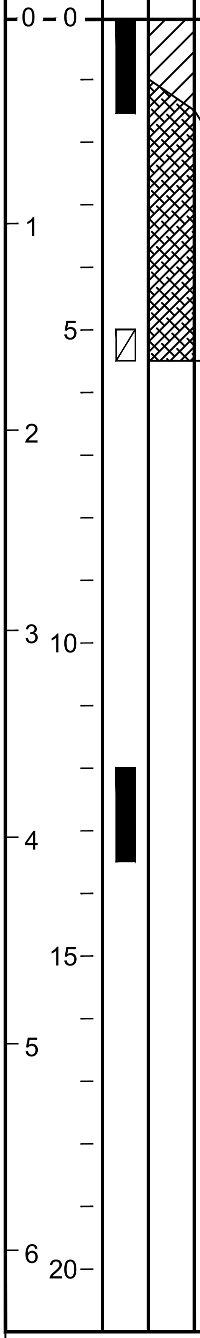
Project No. 1734.023
Date: 10/29/2025

Drawn QHC
Checked

A-3
FIGURE

BORING 2

EQUIPMENT: CAT-Mounted Hydraulic Drill Rig
with 4.0-inch Solid Flight Auger
DATE: 9/9/25
ELEVATION: 295 - feet*
*REFERENCE: Google Earth, 2025



Description	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
Sandy CLAY (CL) Tan to brown, dry, stiff, low plasticity, ~40-50% fine to medium sand, ~10-20% angular sandstone gravel, organics present [Colluvium]	55	105	8.8	UC 225		
Sandstone (fm) Tan to yellow, dry, low to moderately hard, friable, moderate to high weathering, crushed [Bedrock]	50/6"		7.7			
Bottom of Boring at 5.0-feet. No groundwater encountered.						

∇ 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

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BORING LOG

Fernwood Cemetery Expansion
Mill Valley, California

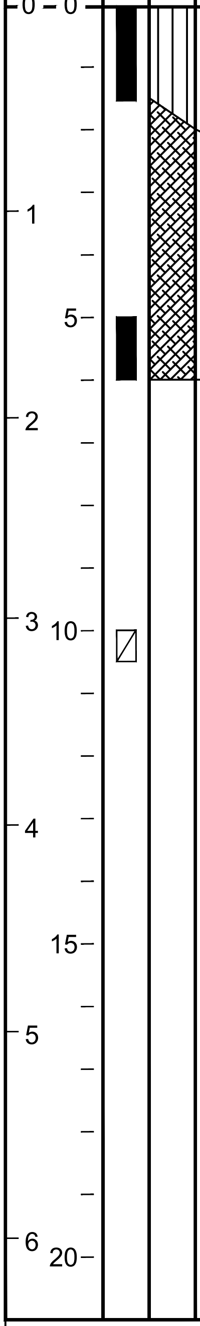
Project No. 1734.023
Date: 10/29/2025

Drawn QHC
Checked

A-4
FIGURE

BORING 3

EQUIPMENT: CAT-Mounted Hydraulic Drill Rig with 4.0-inch Solid Flight Auger
DATE: 9/9/25
ELEVATION: 376 - feet*
*REFERENCE: Google Earth, 2025



Description	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
SILT with Sand and Clay (ML) Medium brown, dry, very stiff, low to medium plasticity, ~10-20% fine to coarse grained sand, trace silt, organics present [Colluvium]	36	108	9.4			
Sandstone (fm) Tan to yellow, dry, low to moderately hard, friable, moderate to high weathering, closely fractured [Bedrock]	88/12"	116	7.3			
Bottom of Boring at 6.0-feet. No groundwater encountered.						

∇ 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

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BORING LOG

Fernwood Cemetery Expansion
Mill Valley, California

Project No. 1734.023
Date: 10/29/2025

Drawn QHC
Checked

A-5
FIGURE

BORING 4

EQUIPMENT: CAT-Mounted Hydraulic Drill Rig
with 4.0-inch Solid Flight Auger
DATE: 9/9/25
ELEVATION: 380 - feet*
*REFERENCE: Google Earth, 2025

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)		BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
0	0	■	▥	**SILT with Sand and Clay (ML)** Medium brown, dry, stiff, low to medium plasticity, ~20-30% clay, ~10-20% fine to coarse grained sand, organics present [Colluvium]	18	83	10.6	UC 375		
1	5	▯	▩	**Sandstone (fm)** Yellow to orange, dry, low to moderately hard, friable, moderate to high weathering, crushed [Bedrock]	95		13.0			
2 3 4 5 6	10 15 20			Bottom of Boring at 6.5-feet. No groundwater encountered.						

▽ 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

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BORING LOG

Fernwood Cemetery Expansion
Mill Valley, California

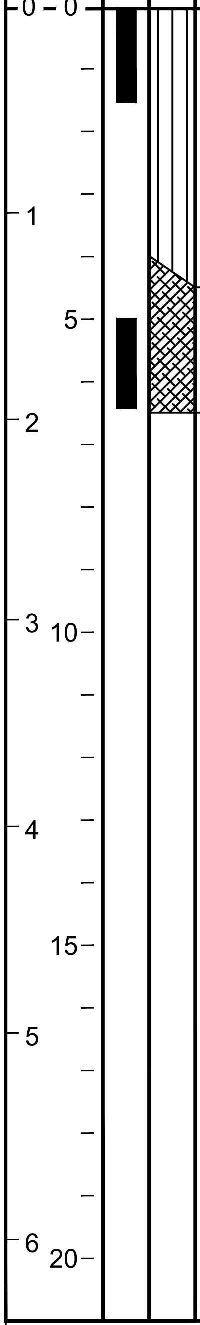
Project No. 1734.023
Date: 10/29/2025

Drawn QHC
Checked

A-6
FIGURE

BORING 5

EQUIPMENT: CAT-Mounted Hydraulic Drill Rig with 4.0-inch Solid Flight Auger
DATE: 9/9/25
ELEVATION: 370 - feet*
*REFERENCE: Google Earth, 2025



Description	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
SILT with Sand and Clay (ML) Medium brown, dry, stiff, low to medium plasticity, ~20-30% clay, ~10-20% fine to coarse grained sand, organics present [Colluvium]	15	91	14.6	UC 625		
Sandstone (fm) Yellow to orange, dry, low to moderately hard, friable, moderate to high weathering, crushed [Bedrock]	88/11"	112	13.8			
Bottom of Boring at 6.5-feet. No groundwater encountered.						

▽ 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

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BORING LOG

Fernwood Cemetery Expansion
Mill Valley, California

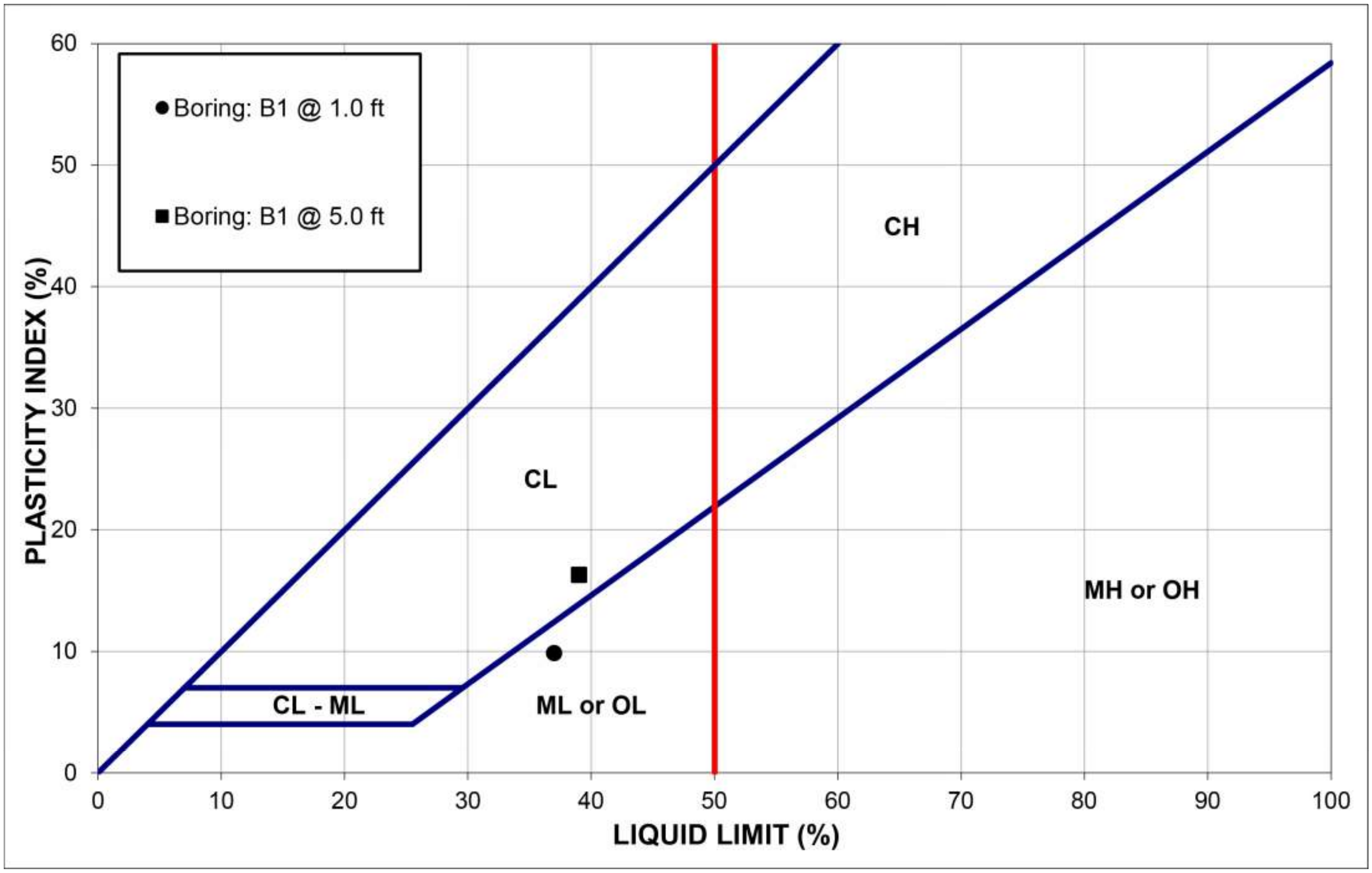
Project No. 1734.023 Date: 10/29/2025

Drawn QHC
Checked

A-7
FIGURE

MILLER PACIFIC ENGINEERING GROUP

ATTERBERG LIMITS TEST (ASTM D 4318)



Sample	Classification	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
Boring: B1 @ 1.0 ft	SILT w/ Sand and Clay Tan-brown	37	27	10
Boring: B1 @ 5.0 ft	CLAY w/ Sand and Silt Tan-brown	39	23	16

PI = 0-3: Non-Plastic
PI = 3-15: Slightly Plastic
PI = 15-30: Medium Plasticity
PI = >30: High Plasticity

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ATTERBERG LIMITS TEST RESULTS

Fernwood Cemetery Expansion
Mill Valley, California

Project No. 1734.023
Date: 10/29/2025

Drawn DMS
Checked

A-8
FIGURE

APPENDIX B
SEISMIC REFRACTION SURVEY RESULTS

Shear-Wave Velocity Report

VsSurf ReMi® 2.1 Software

Results

ASCE 7-16	**Line SR1**	**Line SR2**
Vs100	**1,934 ft/s**	**1,555 ft/s**
Site Class	C	C

Survey Parameters

Geophone Count:	24
Geophone Spacing:	10 ft
Array Length:	230 ft
Survey Date:	Survey Date
Performed By:	Miller Pacific
Analysis By:	Aasha Pancha

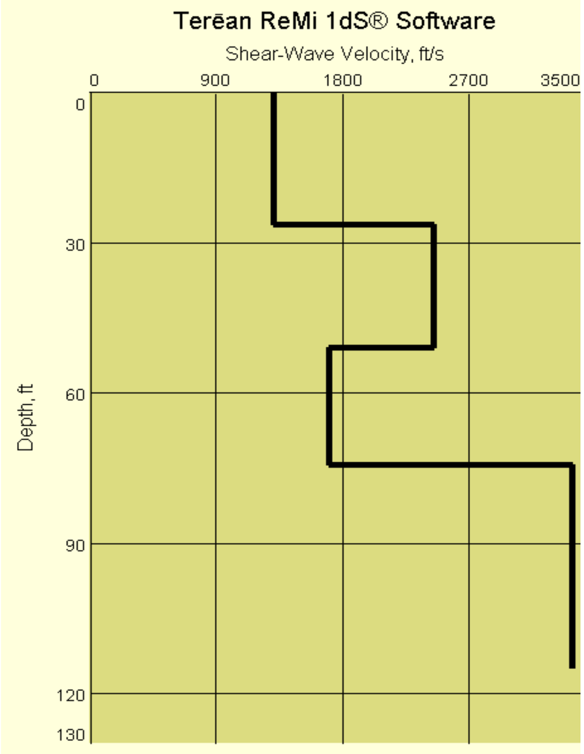
Narrative

SHEAR WAVE VELOCITIES AND SEISMIC SITE CLASS

Miller Pacific’s geophysical site evaluation included subsurface seismic imaging and earthquake ground shaking potential evaluation using Terēan’s VsSurf ReMi® seismic data processing software. Seismic surveys were performed to determine depth to bedrock and the seismic site class per ASCE 7-16 using the weighted-average soil shear wave velocity for the upper 100 feet (Vs100). There were two survey lines, SR1 and SR2. The surveys performed by recording active and ambient (passive) seismic sources. The seismic recording array for these surveys consisted of 24, 4.5 Hz geophones at 10 ft spacing, for a total survey length of 230 feet. Noise generated by hammer strikes during data acquisition while ambient noise was generated from traffic along the nearby roads. The seismic data were acquired using a ReMiDAQ® 5-24 channel seismograph, while data was processed using Terēan's ReMi® software (terean.com/products). Survey results indicate a weighted-average soil shear wave velocity of the upper 100 feet (Vs100) for the SR1 and SR2 locations of 1,934 ft/s and 1,555 ft/s respectively. This results in a designation of a Seismic Site Class C according to ASCE 7-16.

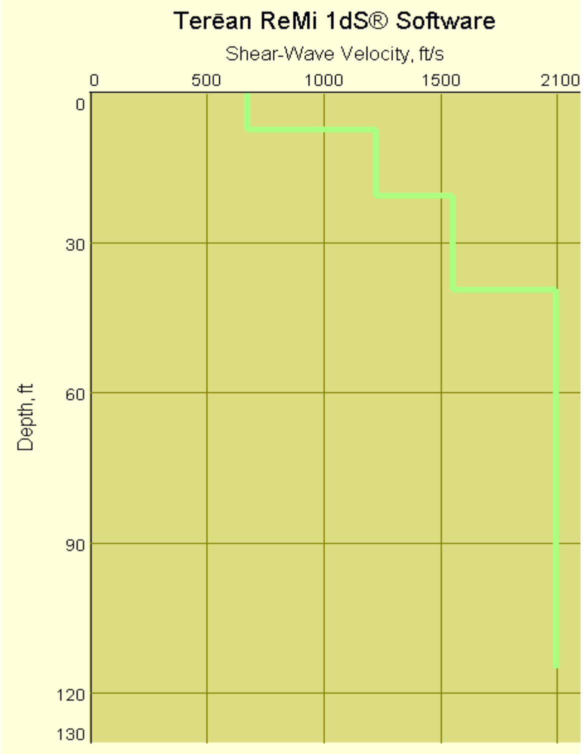
Line SR1

Depth (ft)	Vs (ft/s)
0.0	1,307
26.1	1,307
26.1	2,449
50.8	2,449
50.8	1,702
74.4	1,702
74.4	3,438
114.8	3,438



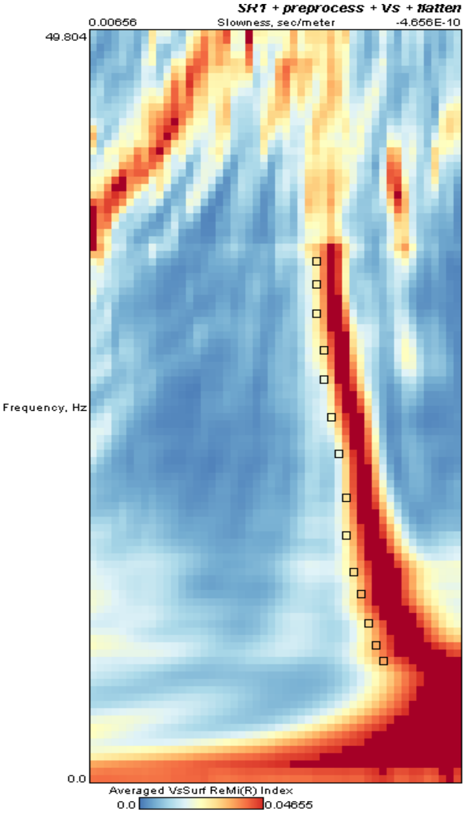
Line SR2

Depth (ft)	Vs (ft/s)
0.0	675
7.2	675
7.2	1,222
20.4	1,222
20.4	1,553
39.3	1,553
39.3	1,992
114.8	1,992



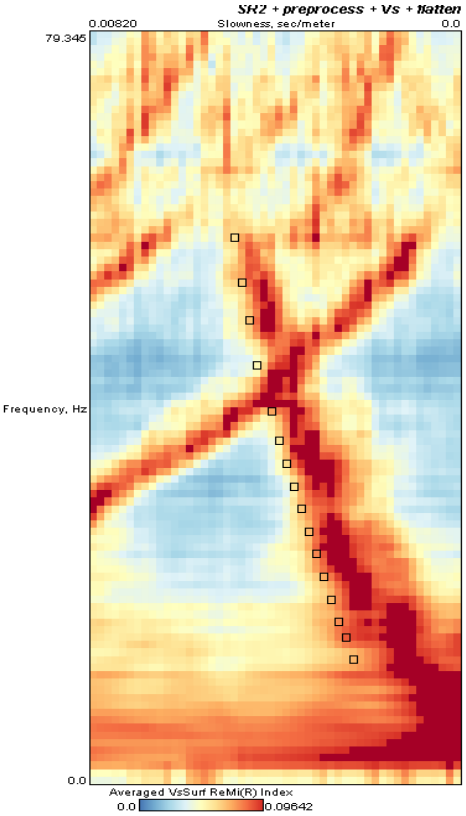
Line SR1

Frequency (Hz)	Slowness (s/m)
8.301	0.00144
9.277	0.00157
10.742	0.00170
12.695	0.00183
14.160	0.00196
s16.601	0.00209
19.042	0.00209
21.972	0.00223
24.414	0.00236
26.855	0.00249
28.808	0.00249
31.249	0.00262
34.667	0.00262
33.203	0.00262



Line SR2

Frequency (Hz)	Slowness (s/m)
57.922	0.00508
53.161	0.00492
49.194	0.00475
44.433	0.00459
39.672	0.00426
36.499	0.00410
34.118	0.00393
31.738	0.00377
29.357	0.00360
26.977	0.00344
24.597	0.00328
22.216	0.00311
19.836	0.00295
17.456	0.00278
15.869	0.00262
13.488	0.00246



Terēan ReMi® 2dS Analysis and Results
Miller Pacific collected two 24-channel seismic lines to image the "1734.023" site. Terēan performed two-dimensional S-wave analysis on all two lines. The 2D modelling results were based on results from the 1D analysis presented above.

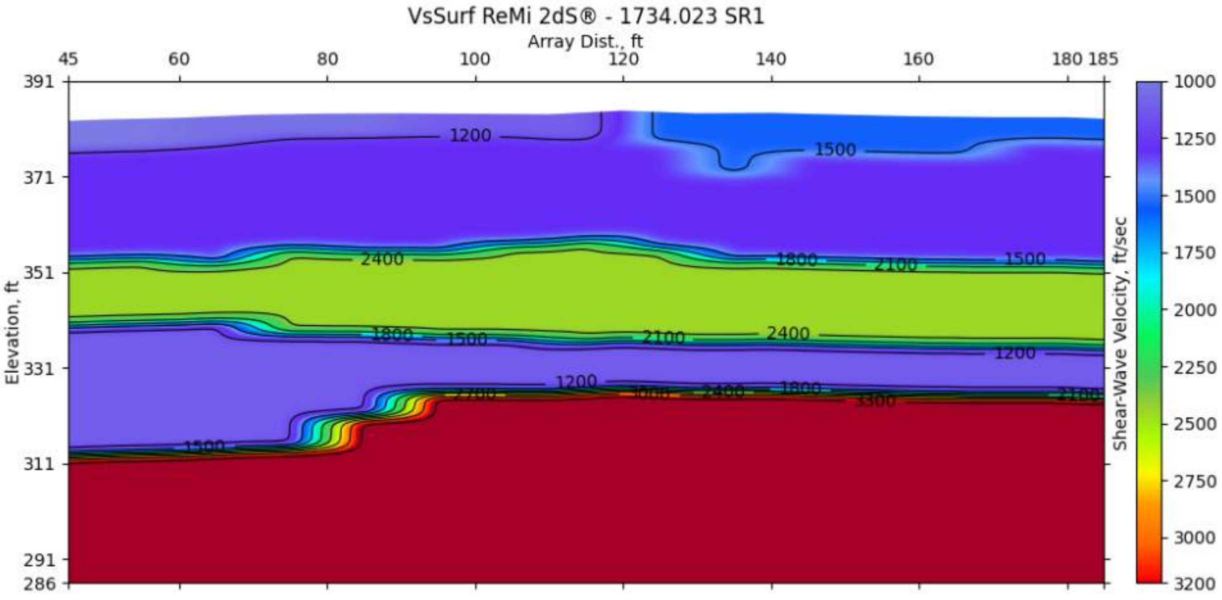
S-wave ReMi® Results

Line SR1:

The 2D shear-wave velocity profile beneath Line SR1 can be characterized by four general layers. The first layer extends from the ground surface to approximately 355 ft elevation and is characterized by shear-wave velocities of 1,000 ft/s ≤ Vs < 1,500 ft/s, indicating rippable materials. An exception occurs between array distances of about 125 ft and 185 ft, where the upper 9–13 ft show Vs > 1,500 ft/s (shown in blue), suggesting localized zones of denser material that may approach the upper bound of the rippable range.

The second layer exhibits velocities of Vs > 2,400 ft/s (shown in green) and has an average thickness of approximately 12–20 ft. This velocity range represents marginally rippable to non-rippable material. Beneath this layer, a reduction in velocity defines the third layer, with 1,100 ft/s ≤ Vs < 1,200 ft/s (shown in purple). This layer varies in thickness, approximately 28 ft near the beginning of the array, narrowing to about 14 ft between array distances of 95 ft and the end of the array, and represents rippable material.

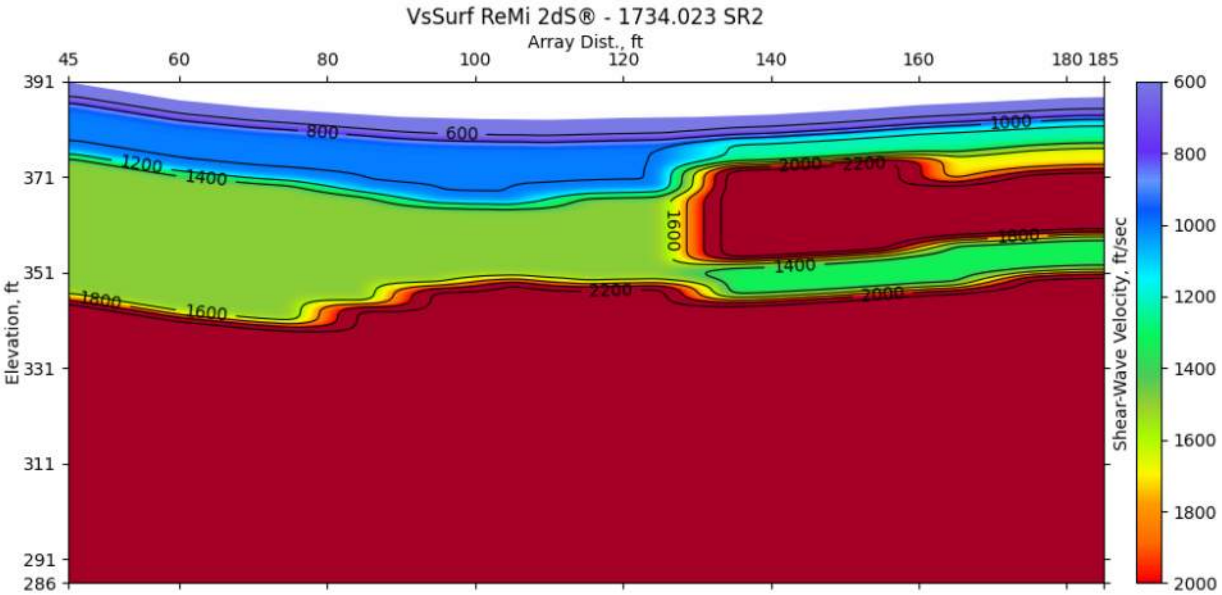
These upper layers are underlain by a high-velocity interface, below which Vs > 3,000 ft/s (shown in red). The elevation of this interface varies from about 316 ft near the beginning of the array, shallowing to approximately 325 ft elevation between array distances of 75 ft and 95 ft. Material at and below this boundary is classified as non-rippable, corresponding to competent rock.



Line SR2:

From an array distance of approximately 45 ft to 135 ft, Line SR2 can be characterized by three layers overlying a high-velocity interface, below which Vs ≥ 2,200 ft/s (shown in red). The upper ~5 ft exhibits Vs < 600 ft/s (shown in purple), indicating soft, loose material that is rippable. Beneath this, the second layer is characterized by Vs ≈ 1,000 ft/s, consistent with loose to medium-dense material also considered rippable. The third layer has shear-wave velocities of 1,400 ft/s < Vs < 1,500 ft/s, suggesting denser material near the upper bound of the rippable range. Across this portion of the array, the high-velocity interface varies in elevation from approximately 357 ft to 346 ft.

Between array distances of 135 ft and 185 ft, the upper ~5 ft continues to show Vs < 600 ft/s (purple), while a distinct high-velocity interval (Vs ≥ 2,300 ft/s, shown in red) occurs between roughly 375 ft and 355 ft elevation. Material with Vs = 1,200–1,400 ft/s is present both immediately above and below this higher-velocity zone. Beneath the profile, the primary high-velocity interface (Vs ≥ 2,200 ft/s) shallows slightly to an elevation of approximately 352 ft, marking the transition to non-rippable, competent rock.



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June 19, 2026
File: 1734.023altr.doc

Fernwood Cemetery
301 Tennessee Valley Road
Mill Valley, California 94941

Attn: Mr. Tyler Cassity

Re: Site Improvement ‘Set-back’ Clarifications
Cemetery Expansion – Fernwood Cemetery
Mill Valley, California

Introduction

This letter presents clarifications to the construction offset distances outlined in our geotechnical investigation report dated October 29, 2025 regarding the expansion of Fernwood Cemetery in Mill Valley, California. Our services were performed in accordance with our Agreement for Professional Engineering and Testing Services dated July 8th, 2025.

Clarifications

Our geotechnical investigation report evaluated geologic hazards that affect the project site and could impact the proposed improvements. These hazards include landsliding, slope erosion, and other site conditions. A key mitigation measure for landsliding and erosion is to maintain minimum set-back distances between the proposed improvements and existing site features.

To mitigate slope instability, proposed improvements should be set back at least 25 feet from mapped landslides so that new loads do not affect unstable hillside areas. To reduce erosion potential, collected surface and subsurface water should not be discharged as concentrated flow above landslide areas, existing slope crests, or creek banks. Although uncontrolled discharge is not recommended, any such discharge should be located at least 50 feet from known landslide areas and from the crests of swales or creek banks to limit erosion and debris accumulation in existing drainageways.

These set-back requirements may be reduced or eliminated if appropriate mitigation measures are implemented. In landslide areas, improvements may be constructed behind retaining walls to reduce loading on unstable areas. Downslope retaining walls may also be supported on deep foundations that transfer loads below the landslide plane and into the underlying weathered bedrock. To reduce erosion along swale and creek-bank crests, collected surface and subsurface drainage may be discharged through dissipater packs, which distribute water over a longer distance rather than from a concentrated outlet.

Additional Services

We should review the final plan set as it nears completion to confirm that our recommendations have been properly incorporated. We should also be retained to provide geotechnical construction observation and testing to verify the contractor’s compliance with the geotechnical portions of the plans and specifications.

We trust this information addresses your current needs. Please contact us with any questions or concerns.

Sincerely,
MILLER PACIFIC ENGINEERING GROUP

REGISTERED PROFESSIONAL ENGINEER
BENJAMIN S. PAPPAS
GE 2786
EXP. 09/30/26
GEOTECHNICAL
STATE OF CALIFORNIA

Benjamin S. Pappas
Geotechnical Engineer No. 2786
(Expires 9/30/2026)