January 3, 2025
File: 378.169rpt.doc

Marin County Open Space District
3501 Civic Center Drive, Suite 260
San Rafael, CA 94903

Attn: Jason Hoorn

Re: Geotechnical Investigation
Buck Gulch Pedestrian Bridge Abutment
Marin County Open Space District
Novato, California

Introduction

This letter summarizes the results of our Geotechnical Investigation for the Buck Gulch Pedestrian Bridge feasibility analysis on the Buck Gulch Trail at 611 Fairway Drive in Novato, California. A site location map is shown on Figure 1. The purpose of our geotechnical investigation is to evaluate geologic and geotechnical conditions and prepare geotechnical recommendations for use in planning, design and analysis of construction feasibility of a pedestrian bridge.

The scope of our Phase 1 (geotechnical investigation) services is outlined in our agreement authorized on August 5, 2024, and includes reviews of readily-available geotechnical and geologic reference material, subsurface exploration with 2 soil borings, laboratory testing of recovered samples, engineering analysis, and preparation of this report. Issuance of this report completes our Phase 1 scope of services. Future phases or work are anticipated to include bridge abutment design, plan review and consultation, and construction observation and testing.

Description

The geotechnical investigation considers the construction of a new +/- 60 foot by 5-foot-wide steel pedestrian bridge spanning the Arroyo San Jose channel. The bridge would be installed downstream of an existing ford and would connect the existing trail to an old tractor road that reconnects to the Buck Gulch Trail. Given this scenario, we anticipate excavations roughly 3 to 5 feet deep would be required to facilitate abutment construction and placement of compacted fill would be needed to backfill the abutments and build the trail extensions. A site plan is presented on Figure 2.

Regional Geology

Marin County lies within the Coast Ranges geomorphic province of California, a region characterized by active seismicity and abundant landsliding and erosion. The regional basement bedrock geology 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 various 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.

Regional geologic mapping[1] indicates that the site is underlain by unconsolidated, alluvial deposits of clay, silt, sand, and gravel underlying valley bottoms and consisting of materials transported and deposited by Arroyo San Jose. The hills surrounding the site are mapped as being underlain by Cretaceous age sandstone. A section of the hill located just north of the site is mapped as being underlain by Franciscan Melange bedrock. A regional geologic map is shown on Figure 3.

Subsurface Exploration and Laboratory Testing

Subsurface exploration at the site was performed on October 29, 2024, with 2 soil borings excavated at the approximate locations shown on Figure 2. Borings were excavated to maximum explored depths of 28-feet 8.5-inches below the ground surface by use of a track-mounted drill rig equipped with 6-inch hollow stem augers and 4-inch solid augers. Materials encountered were examined and logged by our geologist, and select samples were retained for laboratory testing. A brief explanation of the terms and methodology used in logging earth materials is shown on the Soil Classification Chart and Rock Classification Chart, Figures A-1 and A-2, respectively. Exploratory boring logs are shown on Figures A-3 through A-5.

Laboratory testing included determination of moisture content, dry density, unconfined compressive strength, percentage of particles passing the No. 200 (75-µm) sieve, and gradation (sieve) analysis in general accordance with applicable ASTM procedures. Laboratory test results are presented on the boring logs, excepting the gradation (sieve) analysis results, which are shown on Figure A-6. The subsurface exploration and laboratory testing programs are discussed in further detail in Appendix A.

Subsurface Conditions

Boring 1 was drilled north of the creek channel near where the conceptual trail would connect to the old tractor road and Boring 2 was drilled south of the creek channel, in the approximate location of the conceptual bridge abutment, as shown on Figure 2. Boring 1 encountered approximately 7 feet of unconsolidated, very loose fill composed of sand, gravel, and silt. Underling the fill, we encountered 11 feet of medium dense to dense, sandy and clayey gravel, alluvial deposits over completely weathered metasandstone bedrock at 18 feet. The bedrock became hard to very hard and slightly weathered at 23.5 feet below ground surface. Boring 1 was terminated at a maximum explored depth of 28 feet and 8.5 inches below ground surface.

Boring 2 encountered about 2 feet of very stiff/ dense, sandy and gravelly silt fill overlying 15-feet of medium dense to very dense, sand and gravel alluvial deposits. Hard, slightly weathered metasandstone bedrock was encountered at 17.5 feet below ground surface. Boring 2 was terminated at a maximum depth just beyond 19 feet.

[1] Rice, Salem J., 1974, "Geology of the Western Part of Novato Area, Marin County, California", California Division of Mines and Geology, Scale 1:12,000.

Groundwater was encountered during our exploration in Boring 1 at 28 feet and 8.5 inches and was not encountered in Boring 2. Since the borings were drilled at the end of the dry season and were not left open for an extended period of time, a stabilized depth to groundwater may not have been observed. It is anticipated that the groundwater level will generally correspond to water levels in the creek channel.

Geologic Hazards Evaluation

Based on our site reconnaissance, subsurface exploration, and literature review, we have evaluated commonly-considered geologic hazards that may affect the proposed project. Based on our review, the primary hazards which may affect the proposed improvements are strong seismic ground shaking, liquefaction and related hazards (lateral spreading, lurching, and ground cracking), slope/bank instability, flooding, settlement, and erosion/scour. Other hazards, such as fault surface rupture, tsunami, and others are judged relatively insignificant and are not discussed in detail. Our evaluations and conceptual mitigation measures for geologic hazards judged significant at the site are discussed in greater detail in the following sections.

Seismic Ground Shaking

The site will likely experience seismic ground shaking similar to other areas in the seismically active San Francisco Bay Area. Earthquakes along several active faults in the region, as shown on Figure 4, could cause moderate to strong ground shaking at the site. Estimates of peak ground accelerations are based on either deterministic or probabilistic methods.

Deterministic methods use empirical relations developed from data collected during previous earthquakes to provide estimates of median peak ground accelerations. A summary of the active faults that could most significantly affect the site, their maximum credible magnitude, closest distance to the project area, and probable peak accelerations is provided in Table A.

TABLE A - ESTIMATED SEISMIC GROUND MOTIONS
Buck Qulch Pedestrian Bridge
Novato, California

Fault	Moment Magnitude(1)	Closest Estimated Distance (km)(2)	Median Peak Ground Acceleration (g)(3)	Median PGA +1 Std Dev (g)(3)
San Andreas	8.0	16.3	0.29	0.52
Hayward / Rodgers Creek	7.6	17.0	0.25	0.44
San Gregorio	7.4	19.9	0.20	0.37
Green Valley	6.3	40.6	0.06	0.10

1) USGS Earthquake Scenario Map (BSSC 2014), accessed December 6, 2024.
2) Values estimated using Google Earth¬ KML Files showing Quaternary Faults & Folds in the US obtained from USGS website (Accessed December 6, 2024)
3) Values determined using Vs30 = 560 m/s for very dense soil and soft rock conditions (Site Class “C”) in accordance with 2022 California Building Code.

The potential for strong seismic shaking at the project site is high. The most significant adverse impact associated with strong seismic shaking is potential damage to structures and improvements.

Evaluation: *Less than significant with mitigation.*
Recommendation: *New foundations should be designed in accordance with the latest edition of the California Building Code (2022 CBC) and abutments/retaining structures should be designed with a seismic surcharge load. Seismic design criteria for new foundations and retaining walls are presented in the Conclusions and Recommendations section of this report.*

Liquefaction Potential and Related Impacts
Liquefaction refers to the sudden, temporary loss of soil strength during strong ground shaking. The strength loss occurs as a result of the build-up of excess pore water pressures and subsequent reduction of effective stress. While liquefaction most commonly occurs in saturated, loose, granular deposits, recent studies indicate that it can also occur in materials with relatively high fines content provided the fines exhibit lower plasticity.

The effects of liquefaction can vary from cyclic softening resulting in limited strain potential to flow failure which cause large settlements and lateral ground movements. Lateral spreading refers to a specific type of liquefaction-induced ground failure characterized primarily by horizontal displacement of surficial soil layers as a consequence of liquefaction of a subsurface granular layer (Youd, 1995). Lateral spreads generally move down gentle slopes or slip toward a free face such as an incised river channel.

As shown on Figure 6, regional liquefaction hazard maps indicate the project area is mapped within a zone of “moderate” susceptibility to liquefaction (ABAG, 2024). During exploration, we encountered loose granular soils in the upper 8 feet and medium dense to very dense granular soils below. Since the loose granular soils were above the groundwater level during our exploration, the risk of liquefaction impacting the potential development area is considered low during the dry season. During the wet season, there is a moderate potential for liquefactions of the soils in the upper 10 feet.

Evaluation: *Less than significant with mitigation.*
Recommendation: *Minimum mitigation measures should include designing the foundations to account for the potential for some settlement due to liquefaction of localized sand and gravel layers. We recommend that deep foundations bearing on the metasandstone bedrock beneath any liquefiable horizons be utilized. In addition, a lateral seismic load should be included in the design of the bridge abutments to resist lateral spreading. Recommendations for foundation design are presented in the Conclusions and Recommendations section of this report.*

Seismically-Induced Ground Settlement
Seismic ground shaking can induce settlement of unsaturated (above groundwater level), loose, clean, 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. Our borings encountered medium dense to very dense granular soils below 10-feet and loose, granular soils in the upper 10-feet; therefore, the risk of seismically induced ground settlement is moderate.

Evaluation: *Less than significant with mitigation.*
Recommendation: *Minimum mitigation measures should include designing the bridge foundations to account for the potential for some settlement due to seismic densification of localized sand and gravel layers. We recommend that deep foundations bearing on the metasandstone bedrock beneath granular soils be utilized. Recommendations for foundation design are presented in the Conclusions and Recommendations section of this report.*

Slope Instability
Weak soils and bedrock on moderate to steep slopes can move downslope due to gravity. Slope instability is often initiated application of new loads or loss of toe support. The primary adverse effect of slope instability is damage to structures and improvements.

Creek banks along the Arroyo San Jose channel upstream and downstream of the site are locally inclined at about 3.5:1 (horizontal: vertical) or less, and there is metasandstone bedrock exposed in the creek bank just upstream. The creek banks near the ~~project~~ site are susceptible to sloughing and raveling. The addition of heavy shallow foundation loads and/or removal of toe support by channel scour and erosion may accelerate or exacerbate creek bank instability. Therefore, the risk of damage due to slope instability is moderate.

Evaluation: *Less than significant with mitigation.*
Recommendation: *Foundations for bridge abutments should be deepened to gain support on bedrock below any potentially unstable soils. Additionally, erosion control measures should be considered (i.e. rip-rap or erosion control mats) upstream and downstream of the bridge crossing to reduce the potential for scour and erosion to undermine the bridge abutments. Recommendations and design criteria for new foundations and retaining walls are presented in the Conclusions and Recommendations section of this report.*

Flooding
The most significant potential flood-related impact is water damage to structures. The site is mapped as lying within a FEMA 100 and 500-year flood. Therefore, the risk of damage due to large-scale flooding is moderate.

Evaluation: *Significant to less than significant with mitigation.*
Recommendation: *Mitigation measures should include either designing the bridge to withstand flood waters impacting the bridge or accepting the risk for flooding to damage the bridge which would have to be repaired or replaced after a large flood event. The cost to design the bridge resist flooding will depend on the flood elevation level and water velocity.*

Erosion and Scour
Sandy soils on moderate 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. Slopes at the channel banks near the planned bridge are inclined at about 3.5:1 (horizontal: vertical). The generally sandy near-surface soils will likely be susceptible to erosion during high flow events. Additionally, scour adjacent to the new bridge may also result in erosion of soils or undermining of the bridge's foundation. Therefore, the risk of erosion and scour at the site is moderate to high.

Evaluation: Less than significant with mitigation.
Recommendation: The new bridge abutment should be deepened so that the bottom of the footing is at or near the bottom of the creek channel. Wing walls should also be provided on the upstream and downstream side to keep scour and erosion from undermining the bridge approach. The potential depth of erosion and scour should be determined by the project Civil Engineer. Erosion-control measures should conform to the current guidelines of the local California Stormwater Quality Association's Best Management Practice Handbook. After construction, vegetation should be re-established and erosion-control measures implemented in disturbed areas.

Conclusions and Recommendations

Based on the results of our geotechnical investigation, we conclude that a pedestrian bridge crossing of Arroyo San Jose is feasible at the site from a geotechnical perspective. The primary geotechnical considerations are providing uniform foundation support for the new bridge abutments and appropriate mitigation for potential liquefaction, seismic densification, flood impacts, channel scour, and erosion. Recommendations and design criteria to address these and other geotechnical items are presented in the following sections.

Site Preparation and Grading
Site grading would consist primarily of excavation to facilitate construction of the new bridge abutments. Some additional grading would be required to prepare the new trail and bridge approaches. Site preparation, excavation, and backfill should be performed in accordance with the following recommendations and criteria.

1. Surface Preparation – Clear all trees, brush, roots, over-sized debris, and organic material from areas to be graded. Where new trails and bridge approaches will be constructed, the exposed subgrade surface should be moisture conditioned to near the optimum moisture content and compacted to at least 90% relative compaction (ASTM D-1557) to produce a firm and unyielding surface.

2. Excavations - Based on the results of our subsurface exploration, excavations would generally extend into loose to medium dense sandy and gravelly soils. We anticipate that the majority of the excavations can likely be accomplished with

"conventional" equipment, such as excavators or backhoes. Excavations are anticipated to yield sandy to gravelly mixtures that should be suitable for re-use as fill provided they can be processed to meet the gradation requirements discussed below. Where excavations extend below the groundwater table, significant moisture-conditioning (drying) will likely be required as spoils are to be re-used as fill. Excavation spoils deemed unsuitable for re-use as fill should be removed from the site and legally disposed of.

Excavations having a depth of 5 feet or more must be sloped and/or benched in accordance with OSHA regulations. Pursuant to OSHA classifications, the onsite alluvial soils would be classified as Type "C". For Type "C" soils, excavations up to 20-feet deep must be sloped no steeper than 1½:1 (horizontal: vertical). If vertical slopes are required, they must be shored or braced to a minimum of 18-inches above the top of the vertical slope. The Contractor should be responsible for site safety and should select and maintain and appropriate shoring system for the site conditions and in accordance with OSHA requirements.

Performance of temporary cut slopes will be heavily dependent on the amount of time the cut is unsupported, seepage and surface runoff over the face, soil materials, and other factors. Temporary cut slopes may exhibit some sloughing, especially during wet weather conditions, and cleanup of soil and rock debris at the base of slopes may be required. We recommend the grading contractor be responsible for the performance of temporary cut slopes, and the above recommendations should be confirmed during construction.

3. <u>Fill Compaction</u> – Fill placed as retaining wall backfill (as applicable) should be conditioned to a moisture content within 3 percent of the optimum moisture content. Properly moisture-conditioned soils should be placed in loose horizontal lifts of 8 inches thick or less and uniformly compacted to at least 92 percent relative compaction for fills greater than 5-feet in height. Relative compaction may be reduced to 90-percent provided some settlements are acceptable. In pavement areas, the upper 12-inches should be compacted to a minimum of 95 percent relative compaction.

   The fill material should consist of soil and rock mixtures that: (1) are free of organic material, (2) have a Liquid Limit less than 40, (3) have a Plasticity Index less than 20, (4) have a maximum particle size of 4 inches, and (5) have a minimum R-Value of 20.

4. <u>Fill Slope Construction</u> – Any permanent cut and fill slopes (if planned) should ideally be inclined no steeper than 2:1. If steeper slopes are required, they should incorporate synthetic geogrid reinforcement to improve stability. New fill slopes should be founded on keyways and benches excavated into stable soil. Keyway depths will need to be determined during construction, but we anticipate keyways will extend a minimum of 3-feet into firm soil.

Seismic Design
Minimum mitigation of seismic ground shaking includes design of new structures in conformance to 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 particular earthquake and the site response characteristics. Based on the interpreted subsurface conditions and proximity of the San Andreas, Hayward, and San Gregorio Faults, we recommend the CBC coefficients and site values shown in Table B below to calculate the design base shear of the new construction. To determine site seismic coefficients, we used the USGS Design Maps Tool, using the latitude and longitude shown on Figure 4.

TABLE B - 2022 CBC SEISMIC DESIGN FACTORS
Buck Gulch Pedestrian Bridge
Novato, California

Factor Name	Coefficient	CBC Table/ Figure	Site Specific Value[1]
Site Class[2]	$S_{A,B,C,D,E,\text{ or }F}$	1613.5.2	S_C
Spectral Acc. (short)	S_s	1613.5(3)	1.500 g
Spectral Acc. (1-sec)	S_1	1613.5(4)	0.600 g
Site Coefficient	F_a	1613.5.3(1)	1.2
Site Coefficient	F_v	1613.5.3(2)	1.4

1) Values determined in accordance with the 2016 ASCE-7 standard.
2) Soil Profile Type S_C Description: Very dense soil and soft rock, Shear Wave Velocity between 1,200 and 2,500 feet per second, Standard Penetration blow counts greater than 50, and undrained shear strength greater than 2,000 psf over upper 100 feet.

The effects of earthquake shaking (i.e. protection of life safety) can be mitigated by close adherence to the seismic provisions of the current edition of the CBC. However, some structural damage may still occur during strong ground shaking.

Foundation Design
We judge that a rigid foundation system that incorporates deeper seismic support elements in the underlying competent soil/bedrock can provide adequate strength to support the proposed bridge and abutment structure. Various deep foundation options are discussed below. Shallow foundation could be considered under static condition, but would be susceptible to moderate seismic induces settlement and lateral displacement.

Deep Foundations
Given the site's subsurface conditions, proximity to the creek and the desire for the bridge to withstand floodwater forces, torque-down piles or helical piers are the likely foundations system for the bridge abutments to provide additional vertical support and overturning resistance during a seismic or flood event. Driven piles were also considered but site conditions restrict access for the large construction equipment needed to drive piles.

Torque-Down Piles or Helical Piers - Torque-down piles (TDP) are a displacement pile system consisting of steel pipe with helical tips and cutting teeth to assist in pile installation. The piles are installed with a specialized drill rig using a combination of torque and downward pressure. This system also allows additional sections of pile to be welded on in the field as necessary to reach suitable bearing strata. Once the torque down pile reaches the design depth, the steel shell is filled with reinforced concrete. Based on our exploration and laboratory testing, TDP could be expected to develop capacities on the order of 60-kips at depths of about 25 feet.

Similar to torque-down piles, helical piles could also be used to support the abutments. Helical piles are hollow, steel piles (typically 3" to 4" diameter) with helical fins at the base and are "screwed" into place gaining their vertical (compressive and tensile) capacities primarily from the helices. The piles should be installed and corrosion-protected in accordance with the manufacturer's specifications. The piles should derive support from competent native soils and weathered bedrock. Helical piles can be installed with small construction equipment and battered to provide resistance to lateral loads.

Helical piers extending into approved competent soils/bedrock and extending below a depth of 15 feet may be designed using an allowable bearing capacity of 30 kips for dead plus live loads. The actual depth and bearing capacity of the anchors should be evaluated based on measured torque values obtained during installation. Helical piers should be interconnected with a foundation footing / grade beams to support structural loads.

Torque-down piles and helical piles minimize site disturbance, and the quantity of spoils brought to the surface. The torque-down piles would need to be structurally designed to resist the vertical and lateral loads imposed by the bridge and abutment structure. Elements which are not supported on the piles may experience minor future settlement, especially during a strong seismic event. Therefore, some minor differential settlements should be anticipated between the bridge and trail.

Drilled Piers - Drilled piers are likely not preferred at this site due to the potential for squeezing and caving of an unsupported drilled pier excavations. Therefore, casing or slurry support of the drilled excavation will be needed for cast in drilled hole (CIDH) piers. Concrete should be placed via tremie method to displace groundwater and mud/slurry out of the top of the hole. The displaced soil may be suitable for on-site use. Any excess or unstable materials should be off-hauled and legally disposed. Drilled pier foundation design criteria is presented in Table C.

Table C: Drilled Pier Design Criteria
Buck Gulch Pedestrian Bridge
Novato, California

Minimum Diameter:		18 inches
Minimum Embedment:		3’ into bedrock
Skin Friction[1,2]:	Soil	Bedrock
Static :	250 psf	1500 psf
Lateral Passive Resistance[3,4,5,6]:	Soil	Bedrock
Level Ground	250 pcf	500 pcf
2:1 Downslope:	150 pcf	300 pcf

Notes:
1. Dead plus live loads. May increase by 1/3 for total design loads (including wind and seismic).
2. Uplift resistance is equal to 80% of the total skin friction. Ignore upper 3-feet for uplift.
3. Equivalent fluid pressure not to exceed 10 times value in psf
4. Ignore lateral soil support where there is less than 7 feet horizontal distance to face of slope.
5. Apply values over an effective width of 2 pier diameters.
6. Use linear interpolation between values for intermediate slopes flatter than 2:1.

Shallow Foundation

As previously discussed, shallow foundations alone are susceptible to vertical and lateral displacement (estimate up to 6 inches) during a seismic event. If used, shallow foundations should be rigid and designed to span over areas of non-uniform support without significant structural distress. Shallow foundation design criteria are presented below.

Table D: Shallow Foundation Design Criteria
Buck Gulch Pedestrian Bridge
Novato, California

Parameter	Design Value
Minimum Embedment	36 inches
Minimum Footing Width[1]	24 inches
Allowable Bearing Pressure - (Dead+Live)[1,2]	1,000 psf
Unsupported Center Span	10 feet
Unsupported Edge Span	5 feet
Base Friction Coefficient	0.30
Lateral Passive Resistance[3]	250 pcf
Modulus of Subgrade Reaction	75 psi per inch

Notes:
(1) Design shallow foundations to similar bearing pressures (i.e., size footing widths to maintain relatively uniform bearing loads).
(2) Increase design values by 1/3 for total design loads including seismic.
(3) Equivalent fluid pressure, not to exceed 3,000 psf. Neglect upper 12 inches unless confined by concrete or rip-rap.

Retaining Structures

Since the new bridge abutments would be tied together through the bridge and wing walls, the abutment walls are essentially “restrained” retaining walls and should be designed using the “restrained” earth pressures shown in Table E.

New walls may be required to mitigate for potential instability of adjacent creekbanks caused by channel scour and erosion or to support cuts and/or fills for the trails. These walls may be unrestrained (free to rotate at the top of wall), or restrained, in the case tied-back walls. If incorporated, site retaining walls should be designed utilizing the unrestrained design criteria.

TABLE E
ABUTMENT / RETAINING WALL DESIGN CRITERIA
Buck Gulch Pedestrian Bridge
Novato, California

Foundation		
See Above		
Lateral Active Earth Pressure	Unrestrained[1,2]	Restrained[1,3]
Level Ground	40 pcf	25 X H psf
3.5:1 Slope	60 pcf	40 X H psf
Seismic Surcharge[3, 4]	15 x H psf	

Notes:
(1) Interpolate earth pressures for intermediate slopes.
(2) Equivalent fluid pressure.
(3) Rectangular distribution.
(4) The factor of safety for short-term seismic conditions can be reduced to 1.1 or greater.

All walls higher than 3-feet require drainage to prevent the build-up of hydrostatic pressure. Either Caltrans Class 1B permeable material within filter fabric, drainage panels, or Caltrans Class 2 permeable material can be used. The drainage should be collected in 4-inch, perforated, Schedule 40 PVC drain line placed at the base of the wall. Seepage collected in the drains should be discharged through weep holes in the wall. To maintain the wall drainage system, clean-outs must be provided for perforated pipes at the upstream end. Sweep fittings should be used at all major changes in direction. A typical retaining wall drain detail is shown on Figure 7. Retaining wall backfill should be compacted in accordance with the recommendations presented previously in the Site Grading section of this report.

Trench Backfill

Although the need for new underground utilities do not appear to be required for the current project, if needed, we recommend minimum of 6 inches of non-corrosive sand (or other approved pipe bedding material) be placed in the bottom of trench excavations. The bedding material should be continuous around the utility pipe and extend at least 6 inches above the top of pipe. The bedding material over the pipe should be compacted prior to placement of intermediate backfill.

Intermediate trench backfill above the bedding material and up to the subgrade elevation may be select fill material or aggregate base, unless otherwise specified. Native soil and rock materials derived from excavations at the site are likely suitable for re-use as select fill, provided they are properly processed to conform to the fill criteria discussed in the Site Preparation and Grading/Fill Compaction section above.

Intermediate backfill should be moisture-conditioned to near the optimum-moisture content and compacted to at least 90 percent relative compaction. Within pavement or other structural areas, the uppermost 12-inches should be compacted to at least 95 percent relative compaction, in general accordance with ASTM D-1557. The compacted surface must also be non-yielding when proof-rolled with heavy construction equipment. Refer to the applicable utility district Standard Specifications for additional utility trench backfill requirements.

Supplemental Services

After review of the report, we should consult with the County to select abutment and foundation systems and preferred layout. We will perform engineering analyses, calculation and prepare draft and final plans and technical specifications. We should conduct geotechnical consultation with the design team for preparation of contract documents for bidding and construction. Additionally, we should be present during construction to verify that actual conditions encountered are consistent with our recommendations and design criteria and provide required inspection and testing.

We trust that this letter includes the information you require at this time. Please do not hesitate to contact us should there be any questions or concerns.

Yours very truly,
MILLER PACIFIC ENGINEERING GROUP

REVIEWED BY

REGISTERED PROFESSIONAL ENGINEER
SCOTT A. STEPHENS
GE 2398
EXP. 06/30/25
GEOTECHNICAL
STATE OF CALIFORNIA

Rachel Anderson
Staff Geologist

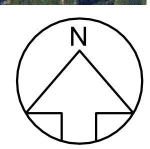
Scott Stephens
Geotechnical Engineer No. 2398
(Expires 6/30/25)

Attachments: Figures 1 through 7; Appendix A

SITE COORDINATES
LAT. 38.06242°
LON. -122.58185°

SITE LOCATION

N.T.S.



REFERENCE: Google Earth, 2024

MILLER PACIFIC ENGINEERING GROUP

FILENAME: 378.169 Standard Figures.dwg

504 Redwood Blvd.
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Novato, CA 94947
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SITE LOCATION MAP

MCOSD
Buck Gulch Pedestrian Bridge
Novato, California

Project No. 378.169 Date: 1/3/2025

Drawn RJA
Checked

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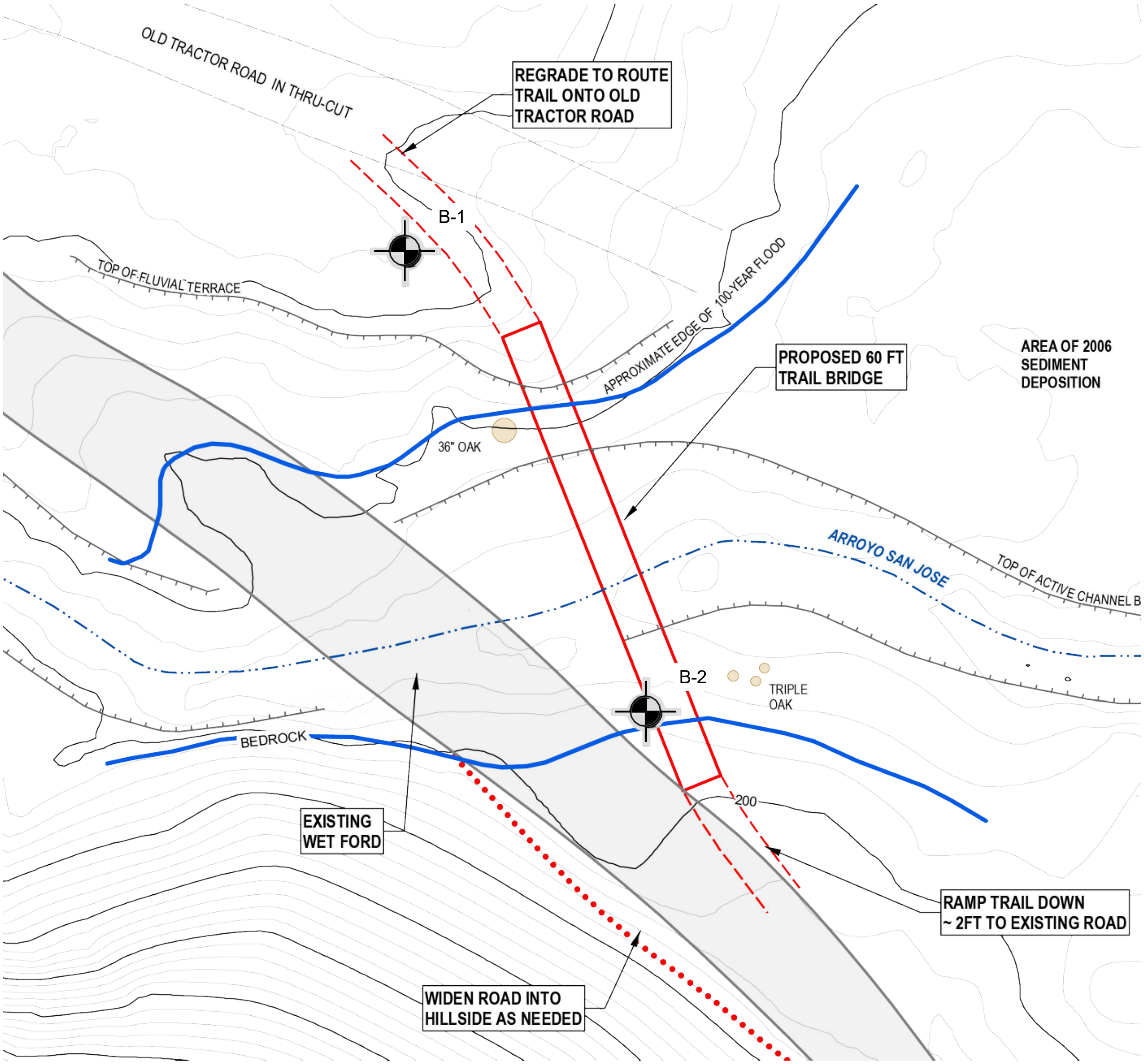
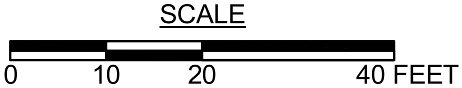
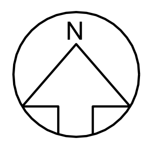
FIGURE

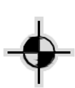


SITE PLAN

SCALE






Approximate location of boring completed by MPEG, 2024

REFERENCE: Timothy C. Best, CEG, "Buck Gulch Falls Trail Bridge Geologic Hazard Assessment and Feasibility Study, *Preliminary Site Map and Section*", 8/30/2021.

MPEG MILLER PACIFIC ENGINEERING GROUP

FILENAME: 378.169 Standard Figures.dwg

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SITE PLAN

MCOSD
Buck Gulch Pedestrian Bridge
Novato, California

Project No. 378.169 Date: 1/3/2025

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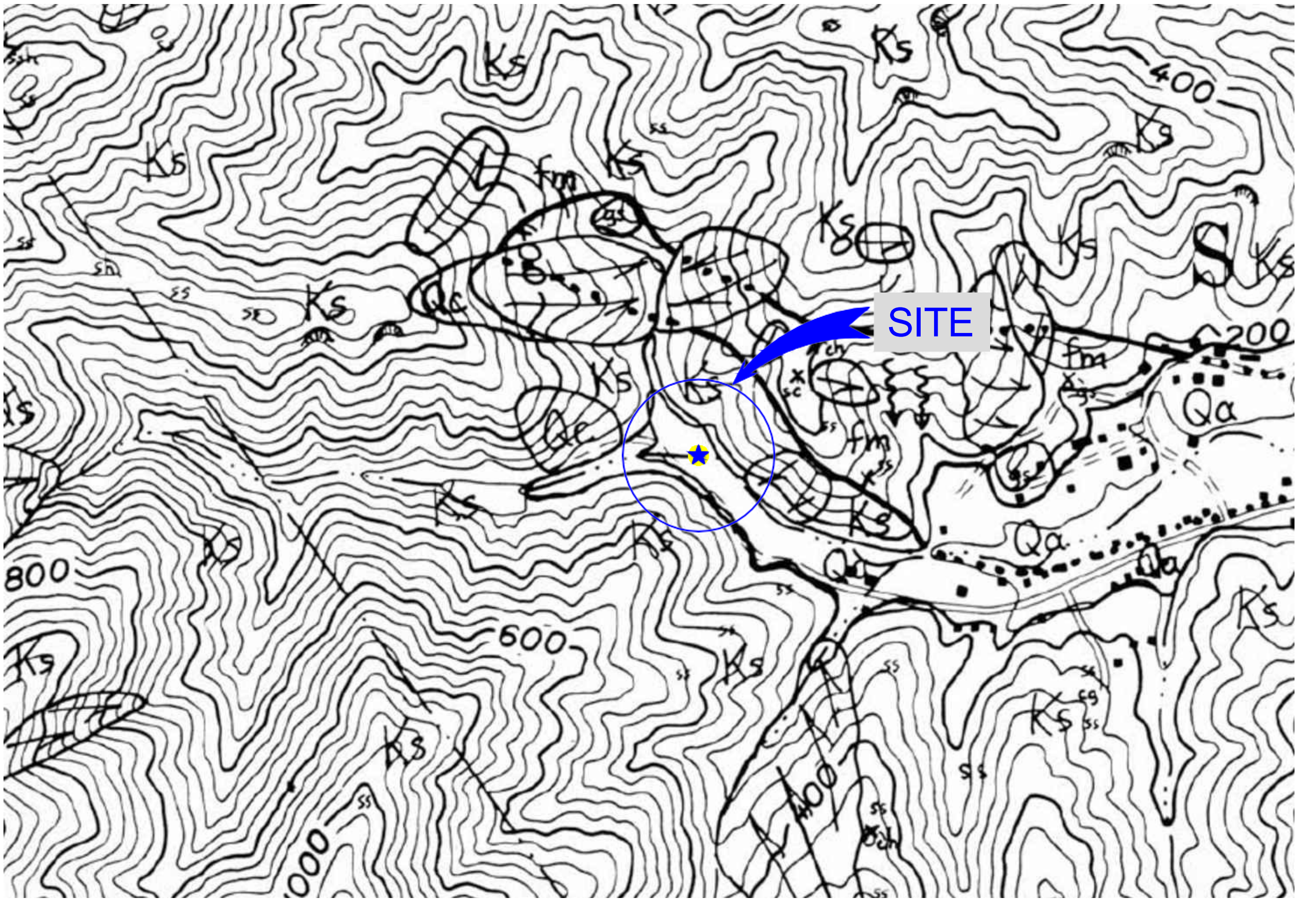
2
FIGURE



REGIONAL GEOLOGIC MAP

LEGEND

Debris Flow Landslides: Deposits of unconsolidated and unsorted soil and rock debris that have moved down slope en masse or in increments by flow or creep processes.

Qa Alluvium: Unconsolidated deposits of clay, silt, sand, and gravel underlying valley bottoms, consisting of materials transported and deposited by streams.

Qc Colluvium: Unconsolidated and unsorted soil material and weathered rock fragments accumulated at or on the bases of slopes by natural gravitational or slope wash processes. Derived by weathering and decomposition of bedrock materials underlying slopes.

Ks Sandstone and shale: with minor amounts of conglomerate. Occurrences of principal rock types in this unit are indicated by the following lithologic symbols:
- ss Sandstone: mainly thick bedded to massive, medium to coarse grained, fairly well sorted, angular to sub-rounded grains. Light gray where fresh, buff where weathered.
- sh Shale: well bedded siltstone,dark gray where fresh, light gray and stained by iron oxide were weathered along joints.

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 Sandstone and Shale: Mainly graywacke type sandstone, with or without alternating beds of dark gray shale.
- ch Chert: 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.
- sp Serpentinite: Pale green to dark green, fine grained, metamorphic rocks composed almost entirely of the magnesium silicate minerals lizardite and chysotile.

REFERENCE: Rice, Salem J., 1974, "Geology of the Western Part of the Novato Area, Marin County, California", California Division of Mines and Geology, Scale 1:12,000.

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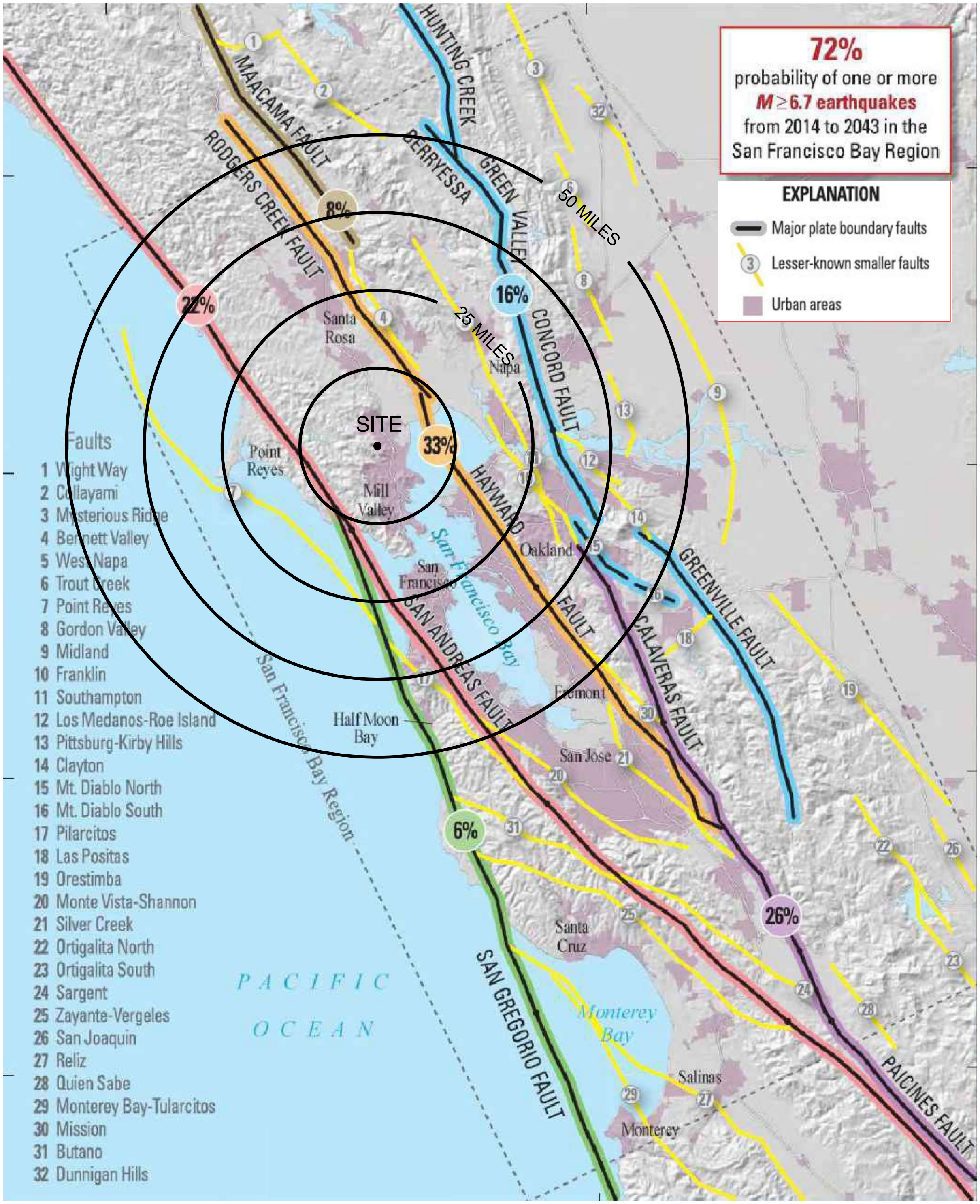
REGIONAL GEOLOGIC MAP

MCOSD
Buck Gulch Pedestrian Bridge
Novato, California

Project No. 378.169 Date: 1/3/2025

Drawn RJA
Checked

3
FIGURE



SITE COORDINATES
LAT. 38.06242°
LON. -122.58185°

SCALE
0 12.5 25 50 MILES

N

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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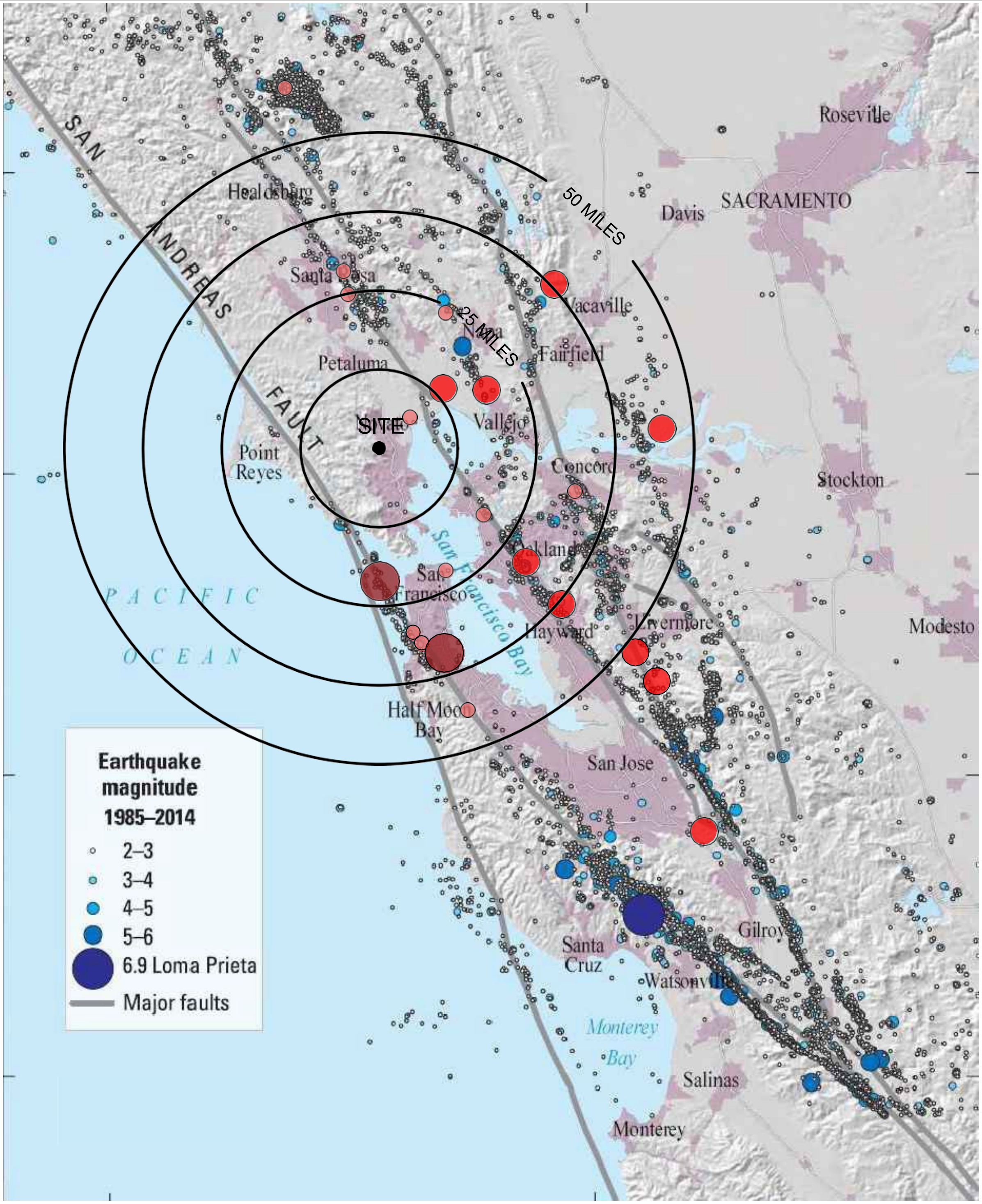
		MCOSD Buck Gulch Pedestrian Bridge Novato, California	Drawn RJA Checked	4 FIGURE
		Project No. 378.169 Date: 1/3/2025		

FILENAME: 378.169 Standard Figures.dwg



SITE COORDINATES
LAT. 38.06242°
LON. -122.58185°

SCALE
0 12.5 25 50 MILES

N

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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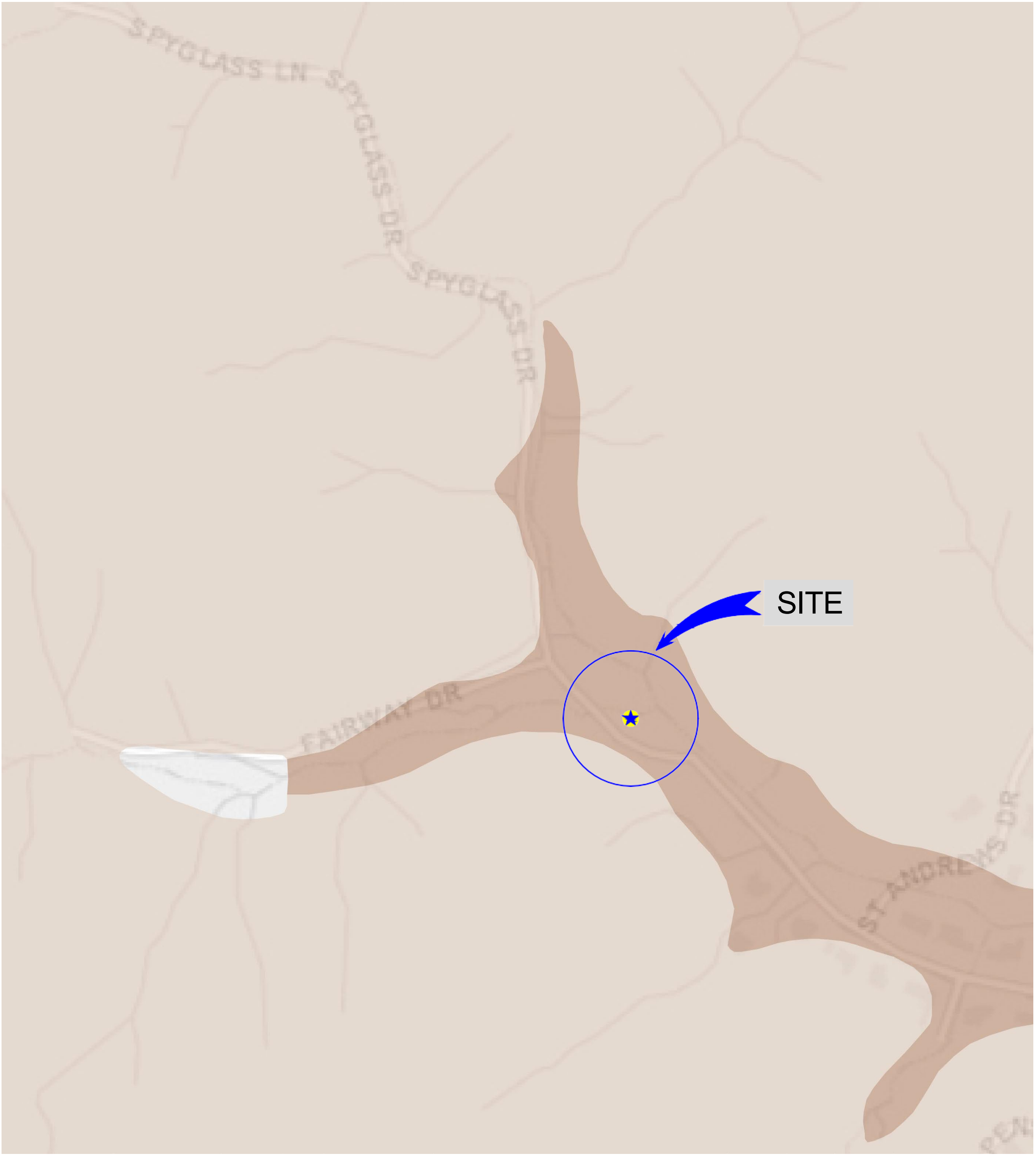
5
FIGURE



LEGEND

Very Low
Low
Moderate
High
Very High

REFERENCE: ABAG Hazard Viewer Map, 2024

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ABAG EARTHQUAKE LIQUEFACTION SUSCEPTIBILITY

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Novato, California

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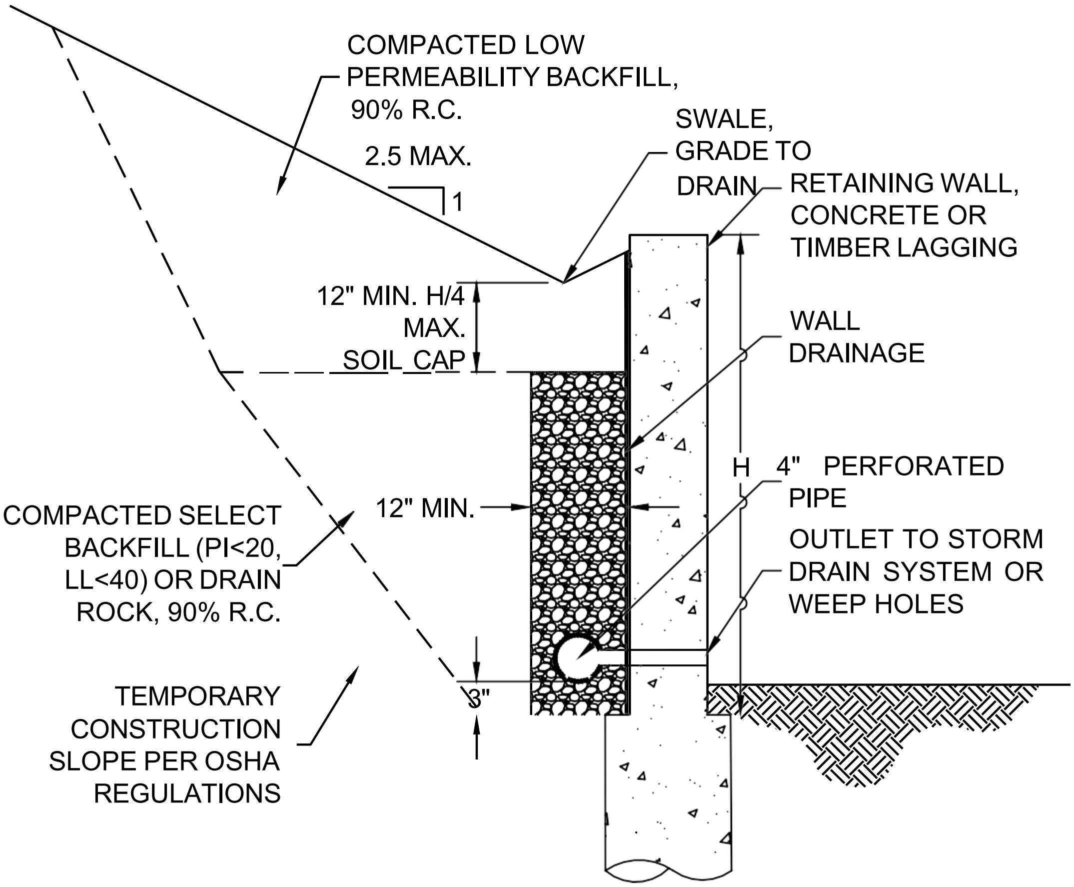
6
FIGURE



NOTES:

1. Wall drainage should consist of clean, free draining 3/4 inch crushed rock (Class 1B Permeable Material) wrapped in filter fabric (Mirafi 140N or equivalent) or Class 2 Permeable Material. Alternatively, pre-fabricated drainage panels (Miradrain G100N or equivalent), installed per the manufacturers recommendations, may be used in lieu of drain rock and fabric.

2. All retaining walls adjacent to interior living spaces shall be water/vapor proofed as specified by the project architect or structural engineer.

3. Perforated pipe shall be SCH 40 or SDR 35 for depths less than 20 feet. Use SCH 80 or SDR 23.5 perforated pipe for depths greater than 20 feet. Place pipe perforations down and slope at 1% to a gravity outlet. Alternatively, drainage can be outlet through 3" diameter weep holes spaced approximately 20' apart.

4. Clean outs should be installed at the upslope end and at significant direction changes of the perforated pipe.Additionally, all angled connectors shall be long bend sweep connections.

5. During compaction, the contractor should use appropriate methods (such as temporary bracing and/or light compaction equipment) to avoid over-stressing the walls. Walls shall be completely backfilled prior to construction in front of or above the retaining wall.

6. Refer to the geotechnical report for lateral soil pressures.

7. All work and materials shall conform with Section 68, of the latest edition of the Caltrans Standard Specifications.

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TYPICAL RETAINING WALL BACKDRAIN

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Buck Gulch Pedestrian Bridge
Novato, California

Project No. 378.169 Date: 1/3/2025

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7
FIGURE

APPENDIX A
SUBSURFACE EXPLORATION AND LABORATORY TESTING

A. Soil and Rock Classification Systems

We explored subsurface conditions at the site with 2 exploratory borings drilled on October 29, 2024. Borings were excavated to a maximum depth of 28-feet and 8.5-inches using a track-mounted drilling equipment with 4-inch solid-stem, continuous flight augers. The soils encountered were logged and identified by our field geologist in general accordance with ASTM Standard D 2487, "Field Identification and Description of Soils (Visual-Manual Procedure)." This standard is briefly explained on Figure A-1, Soil Classification Chart and A-2, Rock Classification Chart. Exploratory boring logs are shown on Figures A-3 through A-5.

B. Laboratory Testing

We conducted laboratory tests on selected intact samples to verify field identifications and to evaluate engineering properties. The following laboratory tests were conducted in 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 D 2166;
- Amount of Material in Soils Finer than No. 200 (75-µm) Sieve, ASTM D 1140; and
- Particle-Size Analysis (Gradation) of Soils, ASTM D 6913.

The moisture content, dry density, unconfined compressive strength and percentage of particles finer than the No. 200 sieve test results are shown on the Boring Logs, Figures A-3 through A-5. Gradation results are shown on Figure A-6.

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

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

KEY TO BORING AND TEST PIT SYMBOLS

CLASSIFICATION TESTS

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

SAMPLER TYPE

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

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

STRENGTH TESTS

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

SAMPLER DRIVING RESISTANCE

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

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

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SOIL CLASSIFICATION CHART

Marin County Open Space District
Buck Gulch Pedestrian Bridge
Novato, California

Project No. 378.169 Date: 1/2/2025

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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

Marin County Open Space District
Buck Gulch Pedestrian Bridge
Novato, California

Project No. 378.169 Date: 1/2/2025

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FIGURE A-2

BORING 1

EQUIPMENT: DR8K Track-Mounted Drill Rig with 4.0-inch Solid and 7.0-inch Hollow Flight Augers
DATE: 10/29/24
ELEVATION: 206 - feet*
*REFERENCE: Timothy Best, CEG, *Buck Gulch Falls Trail Bridge, Preliminary Site Map and Section*, 8/30/2021

DEPTH meters	DEPTH feet	Description	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	OTHER TEST DATA
0	0	2-3" Organics						
		Silty SAND with Gravel (SM) Buff, dry, very loose, very fine to very coarse grained, 30-40% low plasticity fines, 15-25% subangular gravel up to 1.5" Ø [Fill]	2	114	3.7			
1								
	5	Silty SAND with Gravel (SM) Brown, dry, very loose, 25-35% low plasticity fines, very fine to medium grained, 20-30% subangular to subround gravel up to 3" Ø, roots [Fill] Switched to hollow stem augers due to caving loose soils in upper 7'	0		6.7			
2								
		GRAVEL with Sand (GW) Gray, tan, and red, dry, medium dense, 75% subangular to rounded gravel up to 3" Ø, 23% very fine to very coarse mixed lithology sand, 3 fines [Alluvium]	31	108	4.1		SA	
3	10	Grades decrease gravel size to typically 1" Ø and weathered	38	124	5.8	772		
4								
	15	GRAVEL with Clay and Sand (GW-GC) Gray, tan, and red, moist, dense, subangular to rounded gravel up to 1" Ø, 20-30% very fine to very coarse mixed lithology sand, 11.9% fines, gravel is completely weathered to sand [Alluvium]	66	125	5.8		P200 11.9%	
5		Drilling stiffens from 17' to 18'						
6	20	Metasandstone Gray, moist, moderate to hard hardness, moderate to strong strength, locally completely weathered with slight weathering in more resistant nodules, intensely fractured gravel in sand matrix [Bedrock]	50/4"		5.8			
			50/2"		7.9			

▽ 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

Marin County Open Space District
Buck Gulch Pedestrian Bridge
Novato, California

Project No. 378.169 Date: 1/2/2025

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A-3
FIGURE

BORING 1 (CONTINUED)

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)	DESCRIPTION	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	DRILL RATE (FT/MIN)
	20			**Metasandstone** Gray, moist, moderate to hard hardness, moderate to strong strength, locally completely weathered with slight weathering in more resistant nodules, intensely fractured gravel in sand matrix [Bedrock]	50/4" 50/2"		5.8 7.9			0.7
7				Drilling hard at 23.5'						
	25			Grades hard to very hard, strong, slight to moderate weathering, fractures under hammer blow, crushed, quartz, lithics, and fine to medium sand present [Bedrock]	50/1" 50/1"		1.4			0.3
8										
				Grades wet	50/2.5"		7.0			
9	30			Boring terminated at 28 feet and 8.5 inches. Groundwater encountered at 28 feet and 8.5 inches.						
10										
	35									
11										
12	40									

∇ 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

Marin County Open Space District
Buck Gulch Pedestrian Bridge
Novato, California

Project No. 378.169 Date: 1/2/2025

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A-4
FIGURE

BORING 2

EQUIPMENT: DR8K Track-Mounted Drill Rig with 4.0-inch Solid Flight Auger
DATE: 10/29/24
ELEVATION: 198 - feet*
*REFERENCE: Timothy Best, CEG, *Buck Gulch Falls Trail Bridge, Preliminary Site Map and Section*, 8/30/2021

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)	DESCRIPTION	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	DRILL RATE (FT/MIN)
0	0			Sandy SILT with Gravel (ML) Brown, dry, very stiff, low to medium plasticity, ~25-35% very fine to very coarse sand, ~15-20% subangular to subround gravel up to 1" Ø [Fill]	34		3.8			
1				Silty SAND with Gravel (SM) Tan, red, orange, and gray, dry, medium dense, very fine to very coarse sand, 20-30% low plasticity fines,15-25% subround sandstone and metasandstone gravel up to 1" Ø [Alluvium]	44		5.5			
2	5			Silty SAND with Gravel (SM) Dark brown with tan, dry to moist, medium dense, very fine to very coarse grained, 35-45% low plasticity fines, 15-25% variably weathered gravel up to 1" Ø [Alluvium] Grades decrease gravel to 15-20%	50	113	7.0		P200 44.2%	
3, 4	10			Clayey SAND (SC) Brown, tan, and mottled orange and red, moist, medium dense, 35-45% low to medium plasticity fines, trace gravel [Alluvium]	29	116	8.7		P200 41.1%	
5	15			Silty GRAVEL with Sand (GM) Gray and tan, wet, very dense, 20-30% very fine to very coarse sand, ~15% low plasticity fines, crushed metasandstone gravel up to 3" Ø with some shale [Alluvium] Drilling hard at 17.5', drilling very hard at 19'	99	123	9.2			
6	20			Metasandstone Gray and tan, hard, strong, slight to moderately weathered, recovery yields rock dust and some chips of rock [Bedrock]	50/1.5"		4.1			0.3 0" for 10mins at 19'
				Boring terminated at 19 feet and 1.5 inches. 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

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Buck Gulch Pedestrian Bridge
Novato, California

Project No. 378.169 Date: 1/2/2025

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A-5
FIGURE

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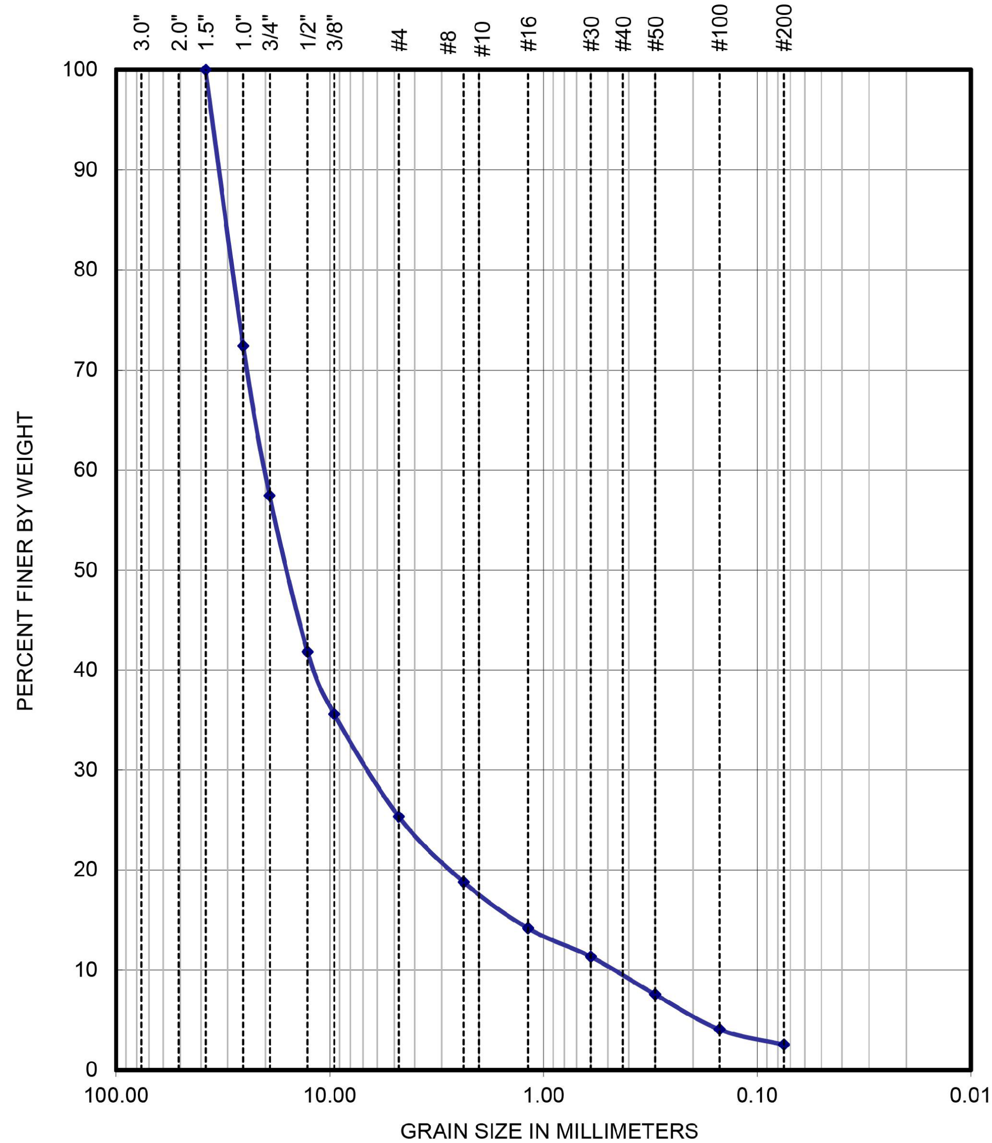
PARTICLE SIZE ANALYSIS - ASTM D 6913 & ASTM D 1140



SYMBOL	SAMPLE SOURCE	CLASSIFICATION
—◆—	B1@7.5'	GRAVEL with Sand (GW)

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SIEVE ANALYSIS

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Buck Gulch Pedestrian Bridge
Novato, California

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A-6
FIGURE