GEOTECHNICAL INVESTIGATION
NEW BUILDING & BUILDING ADDITIONS
100 HARVARD AVENUE
MARIN COUNTY, CALIFORNIA

THIS REPORT HAS BEEN PREPARED FOR:

MOUNT TAMALPAIS SCHOOL

ATTN: MR. ANDREW DAVIS, HEAD OF SCHOOL

100 HARVARD AVENUE

MARIN COUNTY, CALIFORNIA 94941

OCTOBER 2022

MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

October 18, 2022
Project No. 3516-1R1

Mount Tamalpais School
Attn: Mr. Andrew Davis
100 Harvard Avenue
Marin County, CA 94941

RE: GEOTECHNICAL INVESTIGATION,
NEW BUILDING & BUILDING ADDITIONS,
100 HARVARD AVENUE,
MARIN COUNTY, CALIFORNIA

Ladies & Gentlemen:

We are pleased to present the results of our geotechnical investigation relating to the design and construction of a new building and main building additions at the referenced property. This report summarizes the results of our investigation and presents conclusions and recommendations concerning the geotechnical engineering aspects of the project.

The conclusions and recommendations presented in this report are contingent upon our review and approval of the project plans and our observation and testing of the geotechnical aspects of the construction.

If you have any questions concerning our investigation, please call.

Sincerely,
MURRAY ENGINEERS, INC.

Marie Pellarin
Senior Staff Engineer

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TABLE OF CONTENTS

TABLE OF CONTENTS
(continued)

GEOTECHNICAL INVESTIGATION
MOUNT TAMALPAIS SCHOOL
100 HARVARD AVENUE
MARIN COUNTY, CALIFORNIA

INTRODUCTION

This report presents the results of our geotechnical investigation for the design and construction of a new building and building additions on the property located at 100 Harvard Avenue in Marin County, California. The project location is indicated on Figure A-1, Vicinity Map. The purpose of our investigation was to evaluate the subsurface conditions on the site in the area of the proposed improvements and to provide geotechnical design criteria and recommendations for the project.

Project Description

The existing buildings include two main buildings in the eastern part of the site and a multipurpose building in the western part of the site. The Project will involve the construction of a new 4,300 square-foot building to the north of the existing multipurpose building, to be located on the northwestern side of the existing relatively level grass field. In addition, building additions are planned on the west and south sides of the existing first and second school buildings which are connected by a common roof. Additional site improvements will include exterior concrete flatwork. Our scope of services was limited to the proposed new building and two additions to the existing main buildings. Our scope of work did not include the multipurpose building, and we did not perform detailed evaluations of the performance of the existing main school buildings to which the additions will be constructed. The layout of the existing improvements is shown on the Partial Site Plan, Figure A-2.

Scope of Services

We performed the following services in accordance with our agreement with you dated June 9, 2022 (executed on June 9, 2022):

- Reviewed geologic and seismic conditions in the site vicinity and commented on the geologic hazards that could potentially affect the site and the proposed improvements
- Reviewed select architectural and structural plans pertaining to construction of the existing buildings
- Performed a reconnaissance of the site in the area of the proposed improvements
- Explored the subsurface conditions by excavating, sampling, and logging eight (8) exploratory borings in the vicinity of the proposed building and building additions
- Performed laboratory analysis of select soil samples for soil classification and to evaluate engineering properties of the subsurface materials

- Performed geotechnical engineering analyses to develop geotechnical engineering design criteria for the proposed improvements
- Prepared this report presenting a summary of our investigation and our geotechnical conclusions, recommendations, and design criteria associated with the proposed improvements

GEOLOGIC & SEISMIC CONDITIONS

Geologic Overview

The site is located along an east-west trending ridge in the low foothills to the southeast of Mount Tamalpais within the California Coast Ranges geomorphic province. According to the USGS topographic map (San Rafael quadrangle) of the area, the site is situated at an approximate elevation of 300 feet above mean sea level (see Figure A-1).

According to the geologic map of the area (Rice & Smith, 1976), the property is located in an area underlain by mélange bedrock of the Franciscan Complex, which is described as a tectonic mixture consisting of small to large masses of resistant rock types, principally of sandstone, greenstone, chert, and serpentinite, but including various exotic metamorphic rock types embedded in a matrix of pervasively sheared or pulverized rock material. According to the geologic map, the mélange mapped in the area of the site locally contains sandstone and shale, and colluvium overlies the bedrock within the area of the swale on the west side of the site. The original site grades were modified at the time of original site development by cutting and filling to create the existing relatively level building pads and grass play field. No landslides are mapped on the site; however, a large landslide is mapped on a northeast-facing hillside located approximately 200 feet northeast of the site, on the northeast side of California Avenue. This slide measures approximately 800 feet wide by 730 feet long. The relevant portion of the geologic map is included as Figure A-3, Local Geologic Map.

According to the relative slope stability map of the area (Rice & Smith, 1976) the central portion of the site is within Zone 1 which corresponds to the most stable category, with resistant rock that is either exposed or is covered only by shallow colluvium. A small area of the site on the northern site is designated Zone 2, which includes narrow ridges and spur crests that are underlain by relatively competent bedrock, but are flanked by steep, potentially unstable slopes. The southwestern and eastern portions of the property are designated Zone 3, which are areas where the steepness of the slopes approaches the stability limits of the underlying geologic materials. The relevant portion of the relative slope stability map is included as Figure A-4, Local Relative Slope Stability Map.

Faulting & Seismicity

The San Francisco Bay Area, which is affected by the San Andreas Fault system, is recognized

by geologists and seismologists as one of the most active seismic regions in the United States. In the Bay Area there are several major faults trending in a northwest direction within the San Andreas Fault system, which have generated about 12 earthquakes per century large enough to cause significant structural damage, including the San Andreas, San Gregorio, Hayward, and Calaveras faults. The main trace of the active San Andreas fault is located off-shore approximately 5.6 miles west of the site, and the San Gregorio fault is located off-shore approximately 7.0 miles southwest of the site. The Hayward and Calaveras faults are located approximately 12 and 27 miles northeast and east of the site, respectively.

Seismologic and geologic experts convened by the U. S. Geological Survey, California Geological Survey, and the Southern California Earthquake Center conclude that there is a 72 percent probability for at least one "large" earthquake of magnitude 6.7 or larger in the Bay Area before the year 2043. The northern portion of the San Andreas fault is estimated to have a 6 percent probability of producing a magnitude 6.7 or larger earthquake by the year 2043 and the Hayward and Calaveras faults are estimated to have a 14 percent and 7 percent probability of producing a similar magnitude earthquake during the same time period (Working Group on California Earthquake Probabilities, 2014). Given a large earthquake on the San Andreas fault similar to the 7.8 magnitude earthquake that occurred in 1906, it is anticipated that ground shaking at the site will be violent and approximately equal to a Modified Mercalli Intensity of 8 (Association of Bay Area Governments, 2021).

DOCUMENT REVIEW

We reviewed copies of plans made available for our review to gain insight into how the site was originally developed as well as the nature of the foundations supporting the existing school buildings. Prior geotechnical reports for the property were not available for our review. If prior reports are available, they should be made available for our review.

Two main school buildings are located in the eastern portion of the site and have classrooms, administrative offices, and a library, and a central breezeway. The first school building was constructed in 1956 and is the easternmost of the two main school buildings. A second school building was constructed in 1962 to the west of the original building. In 1986, a library addition was constructed on the east side of the original school building. In 1994, an addition was constructed on the west side of the second school building, and the multipurpose building was constructed on the west side of the school site. In 2011 a classroom addition was constructed on the north side of the second school building, a canopy was added between the first and second school buildings, and a new entry pergola was constructed between the first and second school buildings on the south side of the canopy. These various buildings and additions are shown on Figure A-2.

First School Building (1956): We reviewed architectural and structural drawings prepared by Corlett and Spackman Architects dated October 11, 1956 for the construction of the first school building. Based on our review of new grading contours that are specified on Sheet A-1, engineered fill up to 15 feet thick was to be placed on the west side of the site to raise the grades in the area of a swale that extends near the existing multipurpose building, and fill up to 7 feet thick was to be placed on the east side of the existing main building in the area of the library. Cuts up to 18 feet tall were to be made on the south side of the site for the parking lot. Based on this plan, the area of the proposed new northern building appears to be underlain by a wedge of fill that may be up to 10 feet thick, and some thickness of fill may underlie the area of the currently proposed additions to the main school buildings.

Sheets S1 and S2 indicate the original school building is supported on shallow spread footing foundations with interior slab-on-grade floors.

Second School Building (1962): We reviewed plan sheets P1 and P2 by Corlett and Spackman Architects dated June 26, 1961 which pertain to the construction of the second school building, located to the west of the original school building. Sheet P1 is a site utilization plan, and P2 includes a floor plan, exterior and interior elevations, and a section view. A foundation plan and appurtenant structural detailing for the second school building was not available at the time of our review. We suspect that the 1962 second school building foundations consist of shallow spread footings, similar to those supporting the first school building.

1986 Library Addition: We reviewed architectural and structural drawings prepared by D. Hugh Gregory Coleman Architects dated April 8, 1986 for the construction of the library addition to the east side of the first school building. Based on our review of these plans, the library addition has raised wood floors over a crawlspace. According to Sheet S1, the existing eastern perimeter footing for the first school building at the western edge of the library addition was underpinned with a new spread footing extending one foot below crawlspace grade, and the remainder of the library addition is supported on 18-inch diameter drilled, reinforced concrete piers extending 6 feet into the underlying bedrock or "as directed by soil engineers."

1994 Western Addition and Multipurpose Building: We reviewed architectural drawings prepared by William Turnbull Associates dated June 4, 1994; structural drawings prepared by MKM Associates dated June 4, 1994; and civil drawings from 1994 prepared by Anrig-Doyle that pertain to the construction of an addition to the west side of the second school building and the construction of the multipurpose building located in the western portion of the site. Based on our review of Sheet S4, Foundation Plan and details on Sheet S6, the 1994 addition is supported on shallow spread footing foundations that are 16 inches wide and extend at least 18 inches below grade. Interior floors consist of concrete slabs -on-grade. The multipurpose

building is supported on drilled pier and grade beam foundations. According to Sheet S6, Detail 5, piers are 16 inches in diameter and are to extend 3 feet into bedrock, and final pier depths are anticipated to vary from 4 feet to 27 feet. Foundation Sheet S1 references a soils report by Miller Pacific Engineering Group dated August 14, 1992; this report was not available for our review.

2011 Classroom Addition: We reviewed architectural drawings prepared by Richardson Architects dated 2011 and structural drawings prepared by Allco Engineering, Inc. dated April 14, 2011 that pertain to the construction of a classroom addition to the north side of the second school building. According to Sheet S2, the 2011 addition is supported on shallow spread footing foundations and interior floors are concrete slab-on-grade. Sheet S4 specifies that 16-inch diameter piers for the canopy that are to extend at least 6 feet 10 inches below bottom of grade beam and at least 10 feet below top of slab elevation. Sheet S1 references a geotechnical report by Purcell, Rhoades & Associates dated March 21, 2011; this report was not available for our review.

SITE EXPLORATION & RECONNAISSANCE

Exploration Program

Our field exploration was performed on July 6, 7, and 19, 2022 and included a site reconnaissance and the excavation, logging, and sampling of eight exploratory borings to depths ranging from approximately 13.5 feet to 20.5 feet. The boring locations are presented on Figure A-2. The boring locations were approximately determined by measuring distance from building corners on the supplied site plan using a tape measure and should be considered accurate only to the degree implied by the mapping technique used.

Soil and samples were collected using a self-propelled track rig equipped with hollow stem augers and split spoon samplers that were driven by a 140-pound automatic hammer repeatedly dropped from a height of 30 inches. The hammer has an efficiency of 90 percent. The split-spoon samplers included 3-inch outside diameter (O.D.) samplers and 2-inch O.D. Standard Penetration Test samplers. The sampler types used are indicated on the logs at the appropriate depths. The number of hammer blows required to drive the samplers were recorded in 6-inch increments for the length of the 18-inch long sampler barrels. The associated blow count data, which is the sum of the second and third 6-inch increment, is presented on the boring logs as sampling resistance in blows per foot. The field blow counts for the 3-inch O.D. samplers have been standardized to Standard Penetration Test blow counts for sampler size; however, the blow count data has not been adjusted for other factors such as hammer efficiency or rod length.

The logs of the borings are presented in Appendix B as Figures B-1 through B-8. Also included in Appendix B is Figure B-9, Key to Boring Logs; Figure B-10, Unified Soil Classification System; and Figure B-11, Key to Bedrock Descriptions.

Our staff geologist logged the borings in general accordance with the Unified Soil Classification System and Key to Bedrock Descriptions. The boring logs show our interpretation of the subsurface conditions at the location and on the dates indicated and it is not warranted that these conditions are representative of the subsurface conditions at other locations and times. In addition, the stratification lines shown on the logs represent approximate boundaries between the soil and bedrock materials; however, the transitions may be gradual. Samples recovered from the borings were retained for laboratory classification and testing and for review by our senior staff engineer.

Site Description

The approximately 6-acre private school site is located in a developed rural-residential area of unincorporated Marin County near Mill Valley. The irregular-shaped property is bounded by California Avenue to the northeast and east, Wellesley Avenue to the southeast, neighboring residential properties to the northwest and west and south, and an undeveloped right of way (Colby Avenue) to the southwest. Harvard Avenue transects the site in a north-south orientation, dividing the eastern, upslope undeveloped tree grove portion of the site from the rest of the school site. Site gradients vary across the site from relatively flat to steeply sloping. The site is developed with school buildings, a parking lot paved with asphaltic concrete, basketball courts, recess/play area, barn, outdoor garden, and a lawn playing field on the north side of the site. The buildings are situated on relatively level building pads that were created by historical grading activities associated with original site development, as was discussed in more detail in the Document Review section of this report.

The first and second main school buildings are separated by a central breezeway and have classrooms, administrative offices, and a library. The main building was originally constructed in 1956, the library was added to the east side in 1986, an addition was constructed to the west side in 1994, and in 2011 a classroom was added to the north side, a canopy was added on the north side, and a new entry pergola was constructed on the south side. The multipurpose building is located on the west side of the site and was constructed in 1994. Temporary classrooms are currently located to the north of the multipurpose buildings.

Site grades are relatively flat in the area of the proposed building additions to the main school buildings. Site grades in the area of the proposed new building are also relatively flat, although the west side of the building is located approximately 12 to 20 feet away from the crest of a steep descending slope that is on the order of approximately 2.2:1 (horizontal to vertical). This is depicted on the geologic cross sections A-A' and B-B', see Figures A-5 and A-6. From the

geologic cross sections, it can be seen that the area of the proposed building is underlain by a wedge of fill that we interpret to be approximately 10 feet thick beneath the west end of the building (Figure A-6).

Drainage in the area of the proposed northern bulding is characterized as uncontrolled sheet flow to the west and north. Drainage in the area of the proposed building additions and main school buildings is characterized as uncontrolled sheet flow to the east into the curb and gutter along Harvard Avenue. Natural sheet flow is interrupted by the existing buildings.

Subsurface

We advanced eight (8) exploratory borings on the site in order to evaluate the subsurface conditions in the area of the proposed improvements (see Figure A-2). The locations of the borings are shown on Figure A-2 and detailed logs are presented in Appendix B. In general, our borings encountered bedrock at relatively shallow depths, overlain in localized areas by variable quantities of fill and colluvium. A description of the subsurface conditions and the approximate locations of each exploratory boring follow below.

Boring B-1, located near the south side of the proposed building, encountered approximately 8.5 feet of fill consisting of stiff silty clay, underlain by approximately 3.5 feet of colluvium consisting of medium stiff lean clay. At a depth of approximately 12 feet, the colluvium is underlain by bedrock of the Franciscan Complex that persists to the bottom of the boring at a depth of approximately 20.2 feet, where effective sampling refusal was encountered.

Boring B-2, located on the east side of the proposed building, encountered approximately 1.5 feet of fill consisting of soft sandy clay, underlain by approximately 1 foot of colluvium consisting of stiff rock fragments in a clayey matrix. At a depth of approximately 2.5 feet, the colluvium is underlain by bedrock of the Franciscan Complex that persists to the bottom of the boring at a depth of approximately 13.5 feet, where effective sampling refusal was encountered.

Boring B-3, located at the center of the proposed building, encountered approximately 3.5 feet of fill consisting of stiff sandy clay, underlain by bedrock of the Franciscan Complex that persists to the bottom of the boring at a depth of approximately 20.5 feet.

Boring B-4, located on the northeast side of the proposed building, encountered approximately 1 foot of fill consisting of soft sandy clay, underlain by bedrock of the Franciscan Complex that persists to the bottom of the boring at a depth of approximately 19.5 feet.

Boring B-5, located at the northwest corner of the proposed building, encountered

approximately 5 feet of fill consisting of soft sandy clay and soft to stiff silty clay, underlain by bedrock of the Franciscan Complex that persists to the bottom of the boring at a depth of approximately 19 feet, where effective sampling refusal was encountered.

Boring B-6, located near the western building addition, encountered approximately 5.5 feet of fill consisting of loose sand and medium stiff to stiff silty clay, underlain by approximately 2 feet of colluvium consisting of stiff rock fragments in a clayey matrix. At a depth of approximately 7.5 feet, the colluvium is underlain by bedrock of the Franciscan Complex that persists to the bottom of the boring at a depth of approximately 17 feet.

Boring B-7, located near the southern building addition, encountered Franciscan bedrock immediately beneath the existing slab and slab underlayment. The bedrock persists to the bottom of the boring at a depth of approximately 16.8 feet, where effective sampling refusal was encountered.

Boring B-8, near the southern building addition, encountered approximately 1 foot of fill consisting of soft lean clay, underlain by bedrock of the Franciscan Complex that persists to the bottom of the boring at a depth of approximately 15.9 feet, where effective sampling refusal was encountered.

Groundwater

Free groundwater was not encountered in the borings. We note that fluctuations in the level of groundwater can occur due to variations in rainfall, landscaping, and other factors that may not have been evident at the time our measurements were made.

LABORATORY TEST RESULTS

Atterberg Limits Testing

Atterberg limits established on a sample of silty clay fill recovered from Boring B-1 at a depth of approximately 8 to 8.5 feet yielded a plasticity index of 10 percent and a liquid limit of 30 percent, which indicates a low potential for expansion and contraction with changes in moisture content. The results of our Atterberg Limits testing are included as Figure C-1, Liquid & Plastic Limits Test Report.

CONCLUSIONS

Five borings were advanced in the area of the new building (Borings B-1 through B-5) that is planned in the northern portion of the site. These borings encountered approximately 1 to 8.5 feet of undocumented fill in some cases further underlain by native colluvium, further underlain by bedrock. The variable thickness of fill is presumably associated with original site

development to create the existing level grass field. The borings encountered bedrock at variable depths ranging from approximately 1 to 12 feet below existing grade, with the greatest depth to bedrock and thickest fill encountered near the southwest corner of the proposed building. The fill appears to have been placed directly over medium stiff to stiff existing native soil in Boring B-1.

One boring was advanced near the northwest corner of the proposed addition for the office of the Director of Equity and Inclusion, which is planned on the west side of the second school building. This boring encountered approximately 5.5 feet of fill underlain by approximately 2 feet of native colluvium, further underlain by bedrock starting at a depth of approximately 7.5 feet.

Two borings were advanced in the area of the planned addition to the south side of the first school building, near the existing parking lot. These borings encountered bedrock at very shallow depths on the order of approximately one foot below existing grade.

From a geotechnical perspective, the primary geotechnical constraints to the proposed improvements are: (1) the potential for differential settlement of surficial fill and colluvium; (2) the potential for lateral creep of surficial fill and colluvium where located in close proximity to slopes steeper than approximately 5:1 (Horizontal to Vertical); (3) the potential for static and/or seismic instability of undocumented fill where located on or in close proximity to the edge of a steep slope; (4) the potential for differential performance to occur where the proposed building additions meet the existing foundations supporting the adjoining existing portions of the main school buildings, due to differences in steel reinforcing, age, and embedment depth; and (5) the potential for very strong ground shaking during a moderate to large earthquake on one of the nearby active faults. In our opinion, these constraints should not have a significant impact on the proposed improvements, provided that the improvements are designed and constructed in accordance with the recommendations presented below.

In our opinion, building foundations for the proposed new building and southern building addition should be embedded in the underlying bedrock, and the surficial fill and colluvium should not be relied upon for foundation support. Foundations for the new western addition may be embedded in the underlying fill for continuity with the existing building foundations, or they may be embedded in the underlying bedrock..

Geologic Hazards

As part of our investigation, we evaluated the potential for geologic hazards to affect the site. The results of our review are presented below:

- Soil Creep – The clayey fill and colluvial soils encountered in Borings B-1 through B-5

may be susceptible to soil creep where located on or in close proximity to the steep descending slope that is located to the west of the proposed new building. Soil creep is defined as the slow downhill movement of soil in response to cyclic changes in soil moisture and temperature. Such a process may occur when water is introduced into the clay portions of the soil, causing them to behave in a plastic state of deformation. As a result, soil compositions of this type become susceptible to slope movement under the forces of gravity. The zone of movement affected by soil creep is generally confined to the upper few feet of soil material, but the depths can vary depending on the zone of moisture fluctuation, local ground water conditions, soil composition, slope steepness, and depth of underlying bedrock or competent soil strata. In general, the steeper the slope condition, the higher potential for creep to occur and at higher rates. In our opinion the potential impact of creep can be mitigated by supporting the new building on drilled pier and grade beam foundations designed in accordance with the recommendations presented in this report, and also by utilizing surface drainage controls to prevent concentrated runoff from flowing down the western slope.

- Differential Compaction - During moderate and large earthquakes, soft or loose, natural or fill soils can settle, often unevenly across a site. Although the existing clayey fill encountered in Borings B-1 through B-6 and B-8 does not appear to be well-compacted and has relatively low plasticity, we judge the fill to have a sufficient degree of cohesion to rule out the potential for differential compaction. In our opinion, the the surficial foot of loose sand encountered in Boring B-6 is susceptible to differential compaction; however, differential compaction of this material should not constitute a hazard to the western addition provided that the addition is supported on a foundation that is designed in accordance with the recommendations presented in this report.

- Landsliding – Based on our investigation, we did not observe any evidence of landsliding on or adjacent to the areas of the proposed improvements, and no landslides are mapped on or immediately adjacent to the site. The existing fill along the western edge of the grass lawn area was placed by others and documentation regarding the placement and compaction of this fill was not available for our review. Therefore, this fill is undocumented, and thereby should not be relied upon for foundation support. The blow counts recorded in the fill across the site indicate a medium stiff to stiff consistency, which suggests the fill may not have been compacted to a high degree. It is our opinion that the potential for a significant, deep-seated, bedrock landslide is low. However, because of the steep slope and wedge of fill in the area of the proposed new building, the occurrence of a new shallow landslide within the fill and/or underlying colluvial soil cannot be excluded. A new landslide within the fill and/or colluvial soil could be triggered by excessive precipitation and/or strong ground shaking associated with an earthquake. In our opinion, a new landslide within the fill and/or colluvial soil should not have a significant impact on the structural

integrity of the proposed building provided that it is supported on a drilled pier and grade beam foundation system that is designed and constructed in accordance with the recommendations presented in this report. In our opinion, the potential for landsliding along the fill slope can be greatly reduced by reconstructing the slope as a properly engineered fill slope.

It should be noted that although our knowledge of the causes and mechanisms of landslides has greatly increased in recent years, it is not yet possible to predict with certainty exactly when and where all landslides will occur. At some time over the span of thousands of years, most hillsides will experience landslide movement as mountains are reduced to plains. Therefore, an unknown level of risk is always present to structures located in hilly terrain. Owners of property located in these areas must be aware of and be willing to accept this risk.

- Fault Rupture – Based on our review of published maps, it is our opinion that no known active or potentially active faults cross the subject property. Therefore, in our opinion the potential for fault rupture to occur at the site is very low.

- Ground Shaking - As noted in the Seismicity section above, moderate to large earthquakes are probable along several active faults in the greater Bay Area. Therefore, strong ground shaking should be expected at some time during the design life of the proposed development. The new buildings should be designed in accordance with current earthquake resistant standards, including the 2019 CBC guidelines and design parameters presented in this report. It should be clearly understood that these guidelines and parameters will not prevent damage to structures; rather they are intended to prevent catastrophic collapse. The magnitude and extent of earthquake-related damage can be mitigated to a degree by utilizing an upgraded structural design. The project structural engineer should be consulted for additional details relating to an upgraded seismic design.

RECOMMENDATIONS

NEW BUILDING: We recommend that the new building to be supported on drilled, cast in place, reinforced concrete piers bearing in the underlying native competent bedrock. We recommend the piers be interconnected with grade beams, and that interior floor slabs be designed as structural slabs spanning between drilled pier and grade beam foundations. Grade beams may be embedded within the structural slab.

WESTERN BUILDING ADDITION: To maintain foundation compatibility with the existing adjacent foundations for the second school building to which the western addition

will be attached, we recommend the new western addition be supported on continuous spread footing foundations with strong structural connections to existing foundations. At a minimum, we suggest that any portions of the existing foundation system that will receive more than an approximately 25 percent increase in load be re-supported on new foundations. As a more conservative alternative, in our opinion the western addition could be supported on drilled piers; however, we anticipate that this could result in a potential for differential foundation movement to occur where new and existing foundations connect due to hybridization of different foundation types. If desired, we can discuss the approach of utilizing drilled piers in more detail and provide specific drilled pier recommendations upon request.

Interior concrete slabs for the western addition should be designed as structural slabs supported on and spanning between foundations.

We recommend that the project structural engineer review Sheet S4, Foundation Plan prepared by MKM Associates dated June 4, 1994. Based on this plan, it appears that some existing spread footings may be located in the area of the planned western addition. From a geotechnical perspective, it is reasonable to utilize existing foundations as part of the planned addition, provided the structural engineer agrees and that the existing foundation layout can be field verified prior to the start of construction. Due to the presence of approximately 5.5 feet of surficial fill in the area of the planned western addition, in our opinion there is a potential for new spread footings to experience some degree of differential settlement that would be similar to that which has potentially occurred within the adjoining existing portions of the structure. We note that we did not perform a detailed evaluation of the performance of the existing second school building, such as a floor level survey. We further note that if a new building rather than an addition was planned in this area of the site, drilled pier and grade beam foundations would likely be recommended in order to mitigate the potential for differential foundation settlement. However, given the relatively small size of the planned addition and presumed shallow nature of the existing spread footing foundations to which the addition will be attached, in our opinion it is reasonable to utilize spread footings to support the addition. In our opinion, drilled piers would provide the greatest assurance against the potential for differential settlement to occur at the western addition; however, if the new addition is supported on drilled piers and attached to an existing spread footing foundation, in our opinion there will be a potential for differential foundation movement to occur due to hybridization of different foundation types.

SOUTHERN BUILDING ADDITION: The southern building addition may be supported on spread footings gaining support in the underlying bedrock. Based on our review of the 1956 plans, the existing building adjacent to the southern addition is supported on spread footings. We recommend strong continuity be provided where new and existing foundations connect. We suggest that existing foundations be underpinned where they connect with

planned new foundations to provide for more robust structural connections. At a minimum, we suggest that any portions of the existing foundation system that will receive more than an approximately 25 percent increase in load be re-supported on new foundations. Concrete slabs for interior floors at the southern addition may be constructed as slabs-on-grade or as structural slabs.

EXTERIOR CONCRETE SLABS: In our opinion, structural slabs would best serve to reduce the potential for differential slab movement and slab cracking where new slabs are planned in areas of existing fill. If exterior concrete walkways are be planned along the west side of the new building, we anticipate they will be situated over significant thicknesses of fill and weak colluvium that may be subject to downhill creep. As a result, we strongly suggest that slabs planned in this area be designed as structural slabs supported on drilled piers. If slabs-on-grade are utilized on the west side of the proposed northern building, it should be anticipated that some degree of long-term maintenance and repair will be required in the future. Alternatively, we suggest the use of sand-set pavers, which can be repaired relatively easily compared to slabs.

Exterior hardscapes for patios and walkways in other areas maybe designed as slabs-on-grade be underlain by a section of compacted Class 2 aggregate baserock. However, we should review the specific slab locations and slab detailing as part of the geotechnical plan review to confirm that slabs-on-grade are appropriate at specific locations. The subgrade soils beneath new exterior hardscape should be re-worked in place prior to placement of baserock underlayment. We note that conventional slabs-on-grade may be susceptible to minor differential slab movement and cracking site due to the presence of variable thicknesses of undocumented fill across the site. To significantly reduce the potential for minor slab movement and cracking, more critical slabs can be designed and constructed as structural slabs.

GROUNDWATER CONSIDERATIONS: Based on our subsurface exploration, in our opinion, groundwater should not affect proposed foundation construction. However, the potential for some perched subsurface water entering the excavations, and potential need for localized temporary dewatering, should be taken into account by the building contractor. In addition, we strongly encourage the use of a waterproofing consultant and/or waterproofing subcontractor to specify adequate protection against slab surface moisture and associated damage to flooring materials and finishes.

GEOTECHNICAL PLAN REVIEW: Detailed foundation, grading, and drainage recommendations and geotechnical design criteria are presented below. We should review the proposed layout and design, prior to completion of the final plans, to verify that the following

recommendations are appropriate and have been properly interpreted and incorporated into the plans.

2019 CBC EARTHQUAKE DESIGN PARAMETERS

Based on the location of the site at latitude 37.8880065 and longitude -122.5336943, our investigation and engineering judgment, and the site class definitions presented in Chapter 20 of Minimum Design Loads and Associated Criteria for Buildings and other Structures (ASCE 7-16) (American Society of Civil Engineers, 2017), in accordance with Chapter 16, Section 1613 of the 2019 California Building Code (California Building Standards Commission, 2019), the following seismic design parameters should be utilized for the project:

- Site Class C – "Very Dense Soil and Soft Rock"
- Mapped Spectral Accelerations for 0.2 second Period: S_S= 1.500 g (Site Class B)
- Mapped Spectral Accelerations for a 1-second Period: S_1= 0.601 g (Site Class B)
- Design Spectral Accelerations for 0.2 second Period: S_{DS}= 1.200 g (Site Class C)
- Design Spectral Accelerations for a 1-second Period: S_{D1}= 0.561 g (Site Class C)

The preceding seismic design criteria was developed using the Structural Engineers Association of California (SEAOC) and California's Office of Statewide Health Planning and Development (OSHPD) online seismic design value application tool (SEAOC/OSHPD, 2019) using ASCE 7-16 as the design code reference document.

FOUNDATIONS

Drilled Cast-in-Place Concrete Piers

We recommend that the new building be supported on drilled, reinforced, cast-in-place, concrete friction piers gaining support in the underlying bedrock. In general, drilled piers should be spaced no closer than about three pier-diameters, center-to-center. Drilled piers for the new building should be at least 18 inches in diameter and should extend at least 8 feet into the underlying bedrock. We anticipate that piers will vary from approximately 9 to 20 feet deep relative to existing grades. Please note, that these are recommended minimum pier dimensions and that other structural criterion, such as the need to resist lateral forces, may force the pier design depths to be greater.

Drilled piers supporting structural slabs should be at least 16 inches in diameter and should extend at least 4 feet into the underlying bedrock.

Piers should be designed to resist dead plus live loads using an allowable skin friction value of 500 pounds per square foot (psf) for the depth of the pier within the bedrock zone, and with

a one-third increase allowed for transient loads, including wind and seismic forces. Any point-bearing resistance as well as any frictional resistance within the upper portion of piers surrounded by fill or colluvium should be neglected for support of vertical loads.
Based on our subsurface exploration, we anticipate that piers located along the western edge of the new building will encounter surficial fill that varies in thickness from approximately 5 to 12 feet deep. We recommend that the upper portion of piers located along the western edge of the new building be designed to resist active loads from downhill creep of any fill that may be present at the top of the pier. Active loads from downhill soil creep can be calculated on the basis of an equivalent fluid weight of 75 pounds per cubic foot (pcf) taken over 2 pier diameters for the depth of the pier embedded in the fill. The depth of the active loads will likely vary at individual pier locations. Based on our subsurface investigation, we anticipate piers will need to be designed for active soil depths on the order of approximately 5 to 12 feet along the downhill (western) side of the proposed new building, approximately 1 to 5 feet along the northern edge, and 3 to 12 feet along the southern edge. To avoid over-design and facilitate pier construction, we suggest that the project structural engineer develop a pier table that provides required pier embedment depth into supportive bedrock based on depth of overlying non-supportive material in 2-foot increments from 0 to 12 feet.

Active loads from soil creep and other lateral loads may be resisted by passive earth pressure based upon an equivalent fluid pressure of 375 pounds per cubic foot, acting on two times the projected area for the depth of the in the supportive bedrock. Any passive resistance corresponding to the creep zone described above should be neglected.

The structural engineer should determine pier reinforcing, based on the preceding design criteria and structural requirements.

The contractor should be advised that moderately hard bedrock may be encountered while excavating the foundation piers. Drilling refusal using light-weight equipment (e.g. augers mounted on a backhoe) should be evaluated by our field representative and may not be considered acceptable, necessitating heavier equipment being brought to the site to demonstrate refusal.

The bottoms of the pier excavations should be substantially free of all loose cuttings and soil slough prior to the installation of reinforcing steel and the placement of concrete. In addition, any significant amounts of accumulated water in the pier excavations should be pumped out prior to placing concrete or displaced using the tremie method when placing concrete. A representative of Murray Engineers, Inc. should observe the pier excavations to evaluate whether the piers are founded in the supportive material and whether the pier excavations are properly prepared. The pier depths recommended above may require adjustment, if differing

conditions are encountered during drilling. Pier excavations should be filled with concrete as soon as practical after drilling to minimize the potential for caving.

Grade beams should be incorporated between piers as required by the structural engineer. Perimeter grade beams or thickened slab edges should extend at least 6 inches below the elevation of the interior slab subgrade to reduce the potential for infiltration of surface runoff under the structure. Grade beam size and reinforcing should be determined by the project structural engineer based on the preceding design criteria and structural requirements.

Based on our engineering judgment, thirty-year differential foundation movement due to static loads is not expected to exceed approximately ½-inch across any 20-foot span of the pier-supported building.

Spread Footings for Western Addition

New spread footings for the western building addition may consist of continuous spread footings bearing in the underlying fill to match the existing foundation system to which they will be attached, as discussed previously. Isolated spread footings should not be used. Spread footings should have a minimum width of 16 inches, and should extend at least 24 inches below adjacent exterior grade and 18 inches below bottom of interior slab, whichever is greater. All footings located adjacent to utility lines should bear below a 1:1 plane extended upward from the bottom edge of the utility trench.

We recommend that the base of the footing excavations be compacted prior to placement of reinforcing steel and concrete. Footings bearing in compacted fill may be designed using an allowable bearing pressure of 1,500 psf for dead plus live loads, with a one-third increase for total loads including wind and seismic forces. The weight of the footings can be neglected for design purposes.

Lateral loads may be resisted by friction between the footings and the supporting subgrade using a friction coefficient of 0.30. In addition to the above, lateral resistance may be provided by passive pressures acting against foundations poured neat in the footing excavations below a depth of one foot using an equivalent fluid pressure of 200 pounds per cubic foot.

Actual footing size, depth, and reinforcing should be determined by the project structural engineer based on the preceding design criteria and structural requirements.

The footing excavations should be substantially free of loose soil prior to placing reinforcing steel and concrete. Our representative should observe the footing excavations prior to placing concrete forms and reinforcing steel to evaluate whether they are founded in competent bearing materials and have been properly prepared. Any loose soil in the footing excavations

resulting from the placement of forms and reinforcing steel should be removed prior to placing concrete.

Although difficult to predict, based on our engineering judgment, thirty-year differential foundation movement due to static loads is not expected to exceed approximately 1.5 inches across any 20-foot span of the footing-supported western building addition.

Spread Footings for Southern Addition

New spread footings for the southern building addition may consist of spread footings bearing in the underlying bedrock to match the existing foundation system to which they will be attached, as discussed previously. Continuous spread footings should have a minimum width of 16 inches, and isolated spread footings should be at least 18 inches square. Footings should extend at least 24 inches below adjacent exterior grade, at least 18 inches below bottom of interior slab, and at least 6 inches into competent bedrock, whichever is greater. All footings located adjacent to utility lines should bear below a 1:1 plane extended upward from the bottom edge of the utility trench.

Footings bearing in bedrock may be designed using an allowable bearing pressure of 3,000 psf for dead plus live loads, with a one-third increase for total loads including wind and seismic forces. The weight of the footings can be neglected for design purposes.

Lateral loads may be resisted by friction between the footings and the supporting subgrade using a friction coefficient of 0.30. In addition to the above, lateral resistance may be provided by passive pressures acting against foundations poured neat in the footing excavations below a depth of one foot using an equivalent fluid pressure of 350 pounds per cubic foot.

Actual footing size, depth, and reinforcing should be determined by the project structural engineer based on the preceding design criteria and structural requirements.

The footing excavations should be substantially free of loose soil prior to placing reinforcing steel and concrete. Our representative should observe the footing excavations prior to placing concrete forms and reinforcing steel to evaluate whether they are founded in competent bearing materials and have been properly prepared. Any loose soil in the footing excavations resulting from the placement of forms and reinforcing steel should be removed prior to placing concrete.

Based on our engineering judgment, thirty-year differential foundation movement due to static loads is not expected to exceed approximately ¾- inch across any 20-foot span of the footing-supported improvements founded in bedrock.

CONCRETE SLABS

Interior slabs for the new building and for the proposed western building addition should be designed as structural slabs supported on and spanning between foundations recommended above. Interior floor slabs for the southern addition may be constructed as slabs-on-grade or as structural slabs. If exterior slabs are planned on the west side of the new building near the crest of the fill slope, we recommend they be designed as structural slabs supported on drilled piers. Conventional slabs-on-grade may be used for exterior slabs in other areas of the site.

Structural Slabs

Structural slabs should be supported on foundations designed in accordance with the recommendations provided above. Structural slabs should be underlain by at least 4 inches of ½- to ¾-inch crushed rock to act as a capillary break between the underlying subgrade and the slabs.

To limit interior slab dampness from soil moisture vapors, we recommend that a heavy-duty impermeable membrane be placed directly beneath structural slabs for the new building and western building addition. In particular, we suggest the use of an integrally bonded vapor retarder, such as Preprufe™ (Grace Construction Products), which will remain in direct contact with the slab. Please refer to the Vapor Retarder Considerations section below for additional information relating to slab underlayment. Please note that these recommendations do not comprise a specification for "waterproofing." For greater protection against concrete slab dampness, a concrete slab waterproofing system should be considered. The project architect or a waterproofing consultant should provide project-specific waterproofing design and details.

Slabs-on-Grade

Interior floor slabs for the southern building addition and exterior slabs for patios and walkways may be designed as slabs-on-grade. Slabs-on-grade for the southern building addition should be underlain by a 12-inch section of imported granular fill consisting of a 4-inch capillary break of ½- to ¾-inch crushed rock underlain by at least 8 inches of Class 2 aggregate baserock. The baserock section may be reduced if bedrock is exposed prior to reaching subgrade elevation, provided the capillary break is incorporated.

Exterior slabs-on-grade should be designed as "free-floating" slabs, structurally isolated from adjacent foundations. Slabs for exterior patios and walkways should be underlain by at least 8 inches of Class 2 aggregate baserock. The baserock section may be reduced if bedrock is exposed prior to reaching subgrade elevation.

Where existing fill is present within areas of new slabs-on-grade, portions or all of the fill should be removed and replaced as well-compacted engineered fill as deemed necessary by

our field representative during construction. Alternatively, the thickness of the slab underlayment baserock section may be increased. We suggest that the structural designer specify a thicker slab and increased steel reinforcing density for slabs located in fill areas. The preceding recommendations are intended to mitigate significant slab movement and cracking. We note that minor slab movement or localized cracking of slabs may still occur.

Prior to placement of the select granular fill, the subgrade soils should be scarified to a depth of approximately 6 to 12 inches, moisture conditioned, as necessary, and re-compacted in accordance with the Compaction section of this report.

Slabs should be provided with control joints at spacing of not more than about 10 feet. The project structural engineer should determine slab reinforcing based on anticipated use and loading.

Select granular fill should be compacted in accordance with the Compaction section of this report. Where slab surface moisture would be a significant concern, such as for interior floor slabs-on-grade at the southern addition, we recommend that the slabs be underlain by a vapor retarder consisting of a highly durable membrane not less than 15 mils thick (such as Stego Wrap Vapor Barrier by Stego Industries, LLC or equivalent), underlain by the referenced capillary break of 4 inches of ½- to ¾-inch crushed rock. Please also refer to the Vapor Retarder Considerations section below for additional information. Please note that these recommendations do not comprise a specification for "waterproofing." For greater protection against concrete dampness, we recommend that a waterproofing consultant be retained.

Vapor Retarder Considerations

Based on our understanding, two opposing schools of thought currently prevail concerning protection of the vapor retarder during construction. Some believe that 2 inches of sand should be placed above the vapor retarder to protect it from damage during construction and also to provide a small reservoir of moisture (when slightly wetted just prior to concrete placement) to benefit the concrete curing process. Still others believe that protection of the vapor retarder and/or curing of concrete are not as critical design considerations when compared to the possibility of entrapment of moisture in the sand above the vapor retarder and below the slab. The presence of moisture in the sand could lead to post-construction absorption of the trapped moisture through the slab and result in mold or mildew forming at the upper surface of the slab.

We understand that recent trends are to use a highly durable vapor retarder membrane (at least 15 mils thick) without the protective sand covering for interior slabs surfaced with floor coverings including, but not limited to, carpet, wood, or glued tiles and linoleum. However, it is also noted that several special considerations are required to reduce the potential for

concrete edge curling if sand will not be used, including slightly higher placement of reinforcement steel and a water-cement ratio not exceeding 0.5 (Holland and Walker, 1998). We recommend that you consult with other members of your design team, such as your structural engineer, architect, and waterproofing consultant for further guidance on this matter.

FLEXIBLE PAVEMENTS

Sand-Set Pavers or Flagstones

As an alternative to concrete slabs-on-grade for the patios and walkways (if applicable) that would be more capable of accommodating minor differential settlement, sand-set pavers may be constructed to reduce maintenance/repair costs. We generally recommend that sand-set pavers be placed in accordance with the manufacturer's recommendations, except that, at a minimum, we also generally recommend that the pavers be underlain by at least 6 inches of Class 2 aggregate baserock. Prior to placement of the baserock, the subgrade soils should be scarified and moisture conditioned to a depth of at least 6 inches, as necessary, and compacted in accordance with the Compaction section of this report.

EARTHWORK

Earthwork will include site clearing, site grading, drilled pier foundation excavations, spread footing foundation excavations, subgrade preparation beneath slabs-on-grade and exterior hardscape, compaction of engineered fill beneath slabs-on-grade and exterior hardscape, and backfill of utility trenches. Earthwork should be performed in accordance with the following recommendations.

Clearing & Site Preparation

Initially, the proposed building areas should be cleared of obstructions not designated to remain including existing foundations not designated to remain, utilities, large tree roots, and vegetation not designated to remain. A representative from our office should observe the site immediately following demolition to assess the extent of existing excavations and depressions that may not have been evident at the time of this evaluation. Excavations and depressions that extend below finished grade resulting from the removal of underground obstructions, such as foundations, utilities, and root balls, beneath the footprint of the proposed building and associated site improvements should be backfilled with engineered fill placed and compacted in accordance with the recommendations presented below. After clearing, the proposed improvement areas should be adequately stripped to remove surface vegetation and organic-laden topsoil. The stripped material should not be used as engineered fill; however, it may be stockpiled and used for landscaping purposes.

Material for Fill

On-site soils below the stripped layer having an organic content of less than 3 percent organic material by volume (ASTM D 2974) may be suitable for use as engineered fill provided the material of low plasticity and non-expansive. In general, fill material should not contain rocks or pieces larger than 6 inches in greatest dimension, and should contain no more than 15 percent larger than 2.5 inches. Any required imported fill should be predominantly granular material or low plasticity material with a plasticity index of less than approximately 15 percent. Any proposed fill for import should be approved by Murray Engineers, Inc. prior to importing to the site. Our approval process may require index testing to evaluate the expansive potential of the soil; therefore, it is important that we receive samples of any proposed import material at least 3 days prior to importing. Class 2 aggregate baserock should meet the specifications outlined in the Caltrans Standard Specifications, latest edition.

Compaction

Prior to placing engineered fill, the subgrade soil should be scarified and compacted to provide a firm surface to support the fill. Fill material should be spread and compacted in uniform lifts, no thicker than approximately 8-inches in uncompacted thickness. The fill material should be moisture conditioned or dried to approximate the materials optimum moisture content, and compacted to the specifications listed in Table 1 below. The relative compaction and moisture content specified in Table 1 is relative to ASTM D 1557 (latest edition). Compacted fill lifts should be firm and non-yielding under the weight of compaction equipment prior to the placement of successive lifts.

Table 1. Compaction Specifications

Fill Element	Relative Compaction*	Moisture Content*
Fill for raising of site grades	90 percent	Near optimum
Upper 6 to 12 inches of subgrade beneath hardscape	90 percent	Near optimum
Aggregate baserock beneath interior floor slabs and exterior hardscapes	95 percent	Near optimum
½- to ¾-inch Crushed Rock – Compact with at least 3 passes of a vibratory plate with lift-thickness < 12 inches.	See note at left	Not critical
Backfill of utility trenches using on-site low plasticity soil	90 percent	Near Optimum
Backfill of utility trenches using imported sand	90 percent	Near optimum

*Relative to ASTM D 1557, latest edition.

Trench Backfill

In general, bedding and shading materials to be used around underground utilities should be well-graded sand or gravel, conforming to the specifications of the local utility companies or City specifications. Trench backfill placed above utility lines should be placed in approximately 8-inch lifts and compacted in accordance with the recommendations presented

in the following Compaction section of this report or local regulations, if more stringent. On-site soil including soil excavated from the trenches may be used as trench backfill, provided that it meets the recommendations outlined above for fill material. If sand or gravel with less than 10 percent fines (particles passing the No. 200 sieve) is used, it should be compacted to at least 95 percent relative compaction. Drain rock should be mechanically tamped in 12-inch lifts. Trench plugs are recommended where utilities enter into the buildings. The design and construction of trench excavations including any required shoring is the responsibility of the contractor. In addition, care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to the pavement section.

Temporary Slopes & Trench Excavations

The contractor should be responsible for the stability of all temporary cut slopes and trenches excavated at the site, and design and construction of any required shoring and dewatering. Shoring and bracing should be provided in accordance with all applicable local and state safety regulations, including the current OSHA excavation and trench safety standards. Because of the potential for variable soil conditions, field modifications of temporary cut slopes may be required. Unstable materials encountered on the slopes during the excavation should be trimmed off, even if this requires cutting the slope back at flatter inclinations.

SITE DRAINAGE

Control of surface water is critical for projects constructed on hillsides. Roof run-off, rain, and irrigation water should not be allowed to pond near the planned building, additions, exterior hardscape areas, or pavement areas. The new building and building additions should be provided with a roof drainage system. For the new building, we recommend that water collected in the roof drainage system should not be allowed to discharge freely onto the ground surface adjacent to the foundations and should be conveyed away from the building via buried closed conduits and routed to a suitable discharge outlet. For the building additions, water collected in the roof drainage system should not be allowed to discharge freely onto the ground surface adjacent to the foundations and should be conveyed away from the buildings via buried closed conduits or splash blocks and routed to a suitable discharge outlet.

The routing of any existing buried storm drain lines designated to remain should be verified by the project general contractor. The finished grades should be designed to drain surface water away from the proposed buildings, slabs, pavement areas, and landscape areas to suitable discharge points. The ground surface should have positive gradient away from the structures. Where such surface gradients are difficult to achieve, we recommend that area drains or surface drainage swales be installed to collect surface water and convey it away from the buildings. Surface runoff should be prevented from flowing over the top of any artificial

slopes, such as the steep descending slope to the west of the new building. The ground surface at the top of the slope should be graded to slope away from the slope or a berm or lined drainage ditch should be provided at the top of the slope.

We recommend that annual maintenance of the surface drainage systems be performed. This maintenance should include inspection and testing to make sure that the roof drainage systems are in good working order and do not leak; monitoring of site grades near the foundations to see that adequate positive drainage away from the foundations is maintained; inspection and flushing of area drains to make sure that they are free of debris and are in good working order; and inspection of surface drainage outfall locations to verify that introduced water flows freely through the discharge pipes and that no excessive erosion has occurred. If erosion is detected, this office should be contacted to evaluate its extent and to provide mitigation.

REQUIRED FUTURE SERVICES

Plan Review

To better assure conformance of the final design documents with the recommendations contained in this report, and to better comply with the building department's requirements, Murray Engineers, Inc. must review the completed project plans prior to construction. The plans should be made available for our review as soon as possible after completion so that we can better assist in keeping your project schedule on track. We recommend that the following project-specific note be added to the project plans:

- The geotechnical aspects of the construction, including site clearing, site grading, drilled pier foundation excavations, spread footing foundation excavations, subgrade preparation beneath slabs-on-grade and exterior hardscape, compaction of engineered fill beneath slabs-on-grade and exterior hardscape, and backfill of utility trenches, and installation of surface drainage controls should be performed in accordance with the recommendations of the geotechnical report prepared by Murray Engineers, Inc., dated October 18, 2022. Murray Engineers, Inc. should be provided at least 48 hours advance notification (telephone: 650-559-9980; email for Field Services Director: david@murrayengineers.com) of any earthwork operations and should be present to observe and test, as necessary, the earthwork, foundation, and drainage installation phases of the project.

Construction Observation Services

Murray Engineers, Inc. should observe and test (as necessary) the earthwork and foundation phases of construction in order to a) confirm that subsurface conditions exposed during construction are substantially the same as those interpolated from our limited subsurface exploration, on which the analysis and design were based; b) observe compliance with the

geotechnical design concepts, specifications and recommendations; and c) allow design changes in the event that subsurface conditions differ from those anticipated. The recommendations in this report are based on limited subsurface information. The nature and extent of variation across the site may not become evident until construction. If variations are then exposed, it will be necessary to re-evaluate our recommendations.

LIMITATIONS

This report has been prepared for the sole use of the Mount Tamalpais School, specifically for developing geotechnical design criteria relating to design and construction of the proposed new residential building, western and southern addition, and associated improvements on the property located at 100 Harvard Street in Marin County, California. In the event that any changes in the nature or locations of the proposed improvements are planned, the conclusions and recommendations of this report shall not be considered valid unless such changes are reviewed, and the conclusions and recommendations presented in this report are modified or verified in writing by this firm.

The opinions presented in this report are based upon information obtained from borings at widely separated locations, a site reconnaissance, laboratory testing, review of plans and field data made available to us, and upon local experience and engineering judgment, and have been formulated in accordance with generally accepted geotechnical engineering practices that exist in the San Francisco Bay Area at the time this report was prepared. Further, our recommendations are based on the assumption that soil and geologic conditions at or between borings do not deviate substantially from those encountered. It should be clearly understood that geotechnical conditions may become apparent during the course of construction that were not apparent at the time our investigation was performed. No other warranty, expressed or implied, is made or should be inferred. We are not responsible for data provided by others.

The recommendations provided in this report are provided based on the assumption that we will be retained to provide the Future Services described above in order to evaluate compliance with our recommendations. If we are not retained for these services, Murray Engineers, Inc. cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of this report by others. Furthermore, if another geotechnical consultant is retained for follow-up service to this report, Murray Engineers, Inc. will at that time cease to be the Engineer-of-Record.

The opinions presented in this report are valid as of the present date for the property evaluated. Changes in the condition of a property can occur with the passage of time, whether due to natural processes or the works of man, on this or adjacent properties. In addition, changes in applicable standards of practice can occur, whether from legislation or the broadening of knowledge. Accordingly, the opinions presented in this report may be

invalidated, wholly or partially, by changes outside of our control. Therefore, this report is subject to review and should not be relied upon after a period of three years, nor should it be used, or is it applicable, for any property other than that evaluated.

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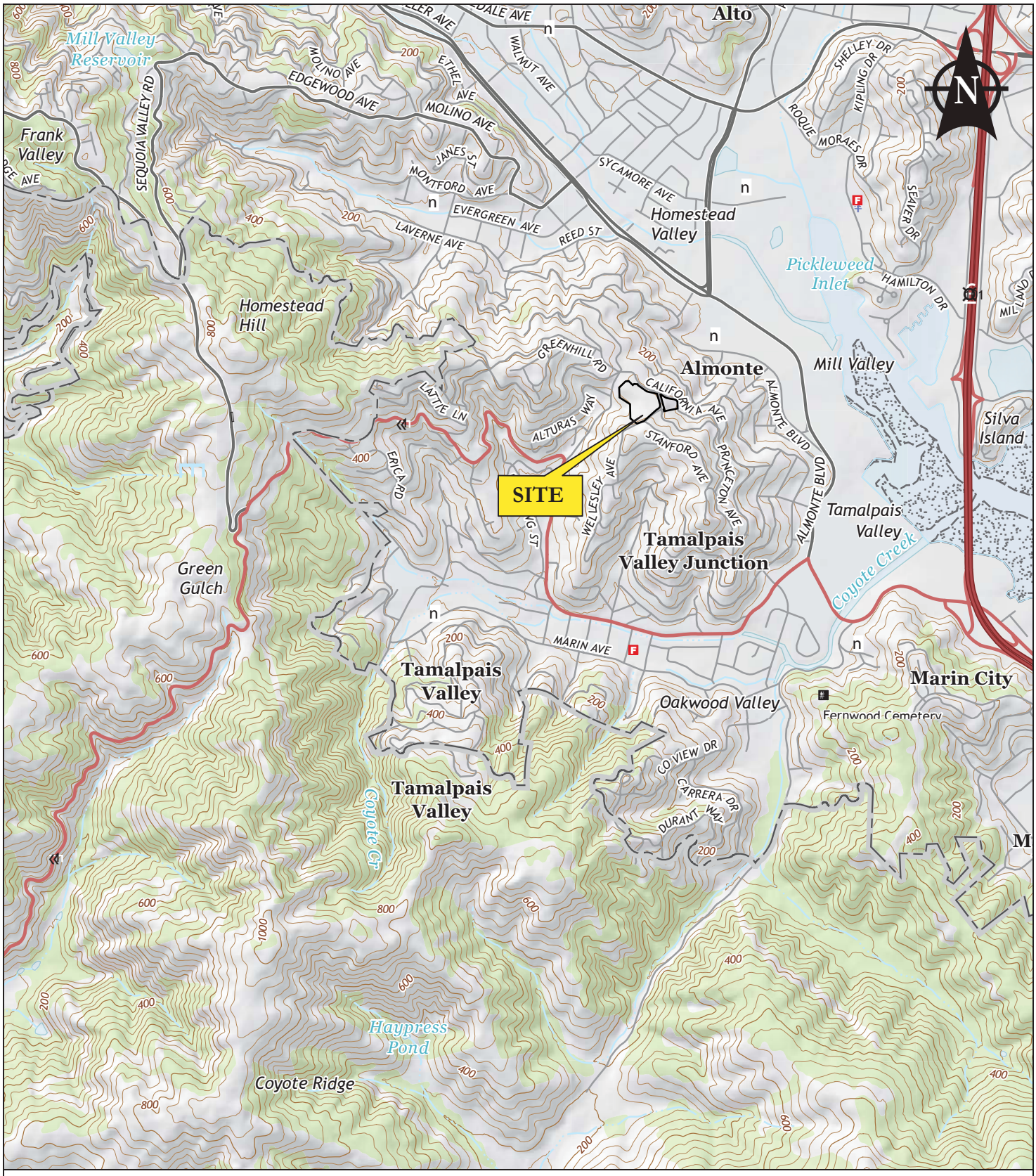
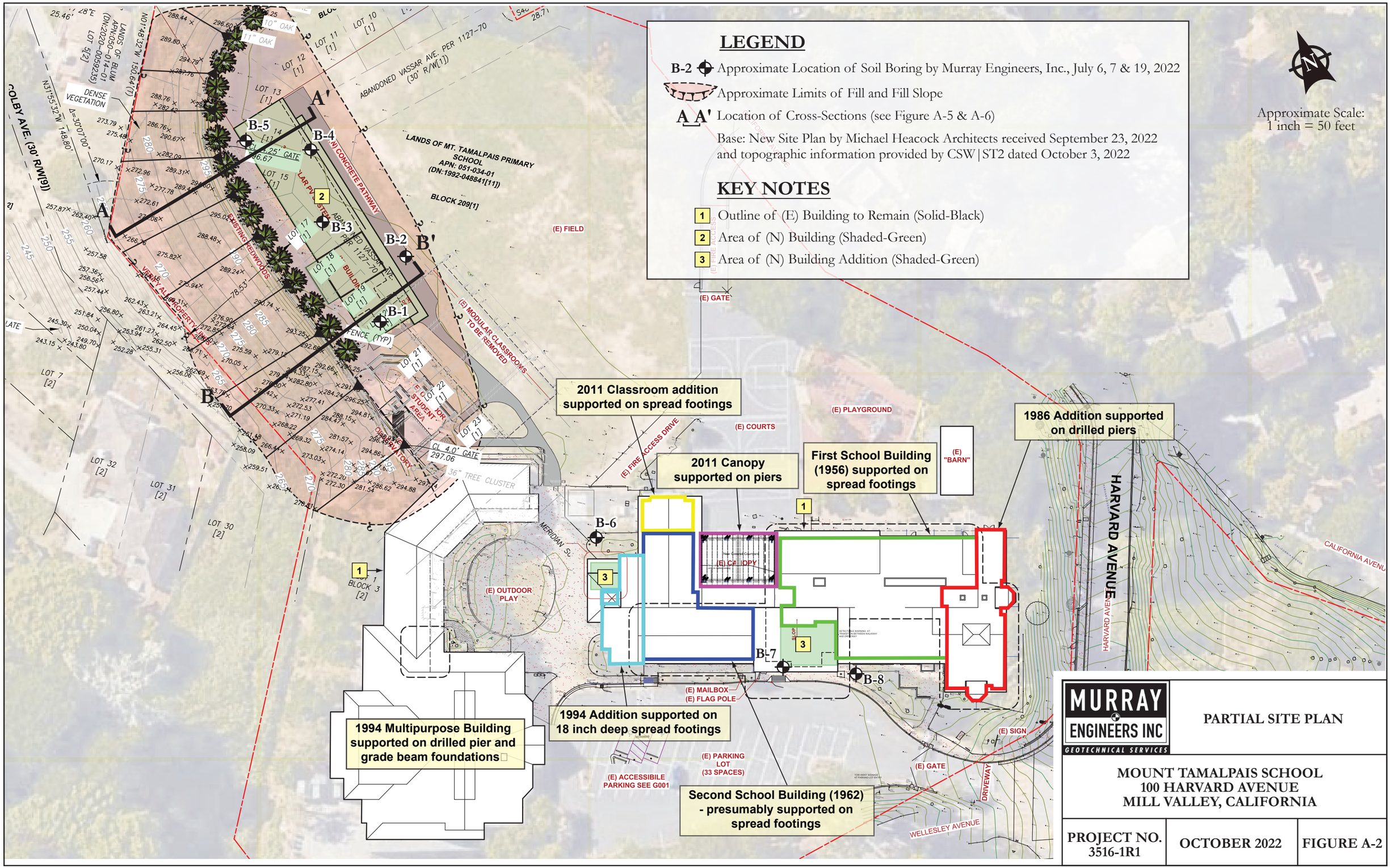
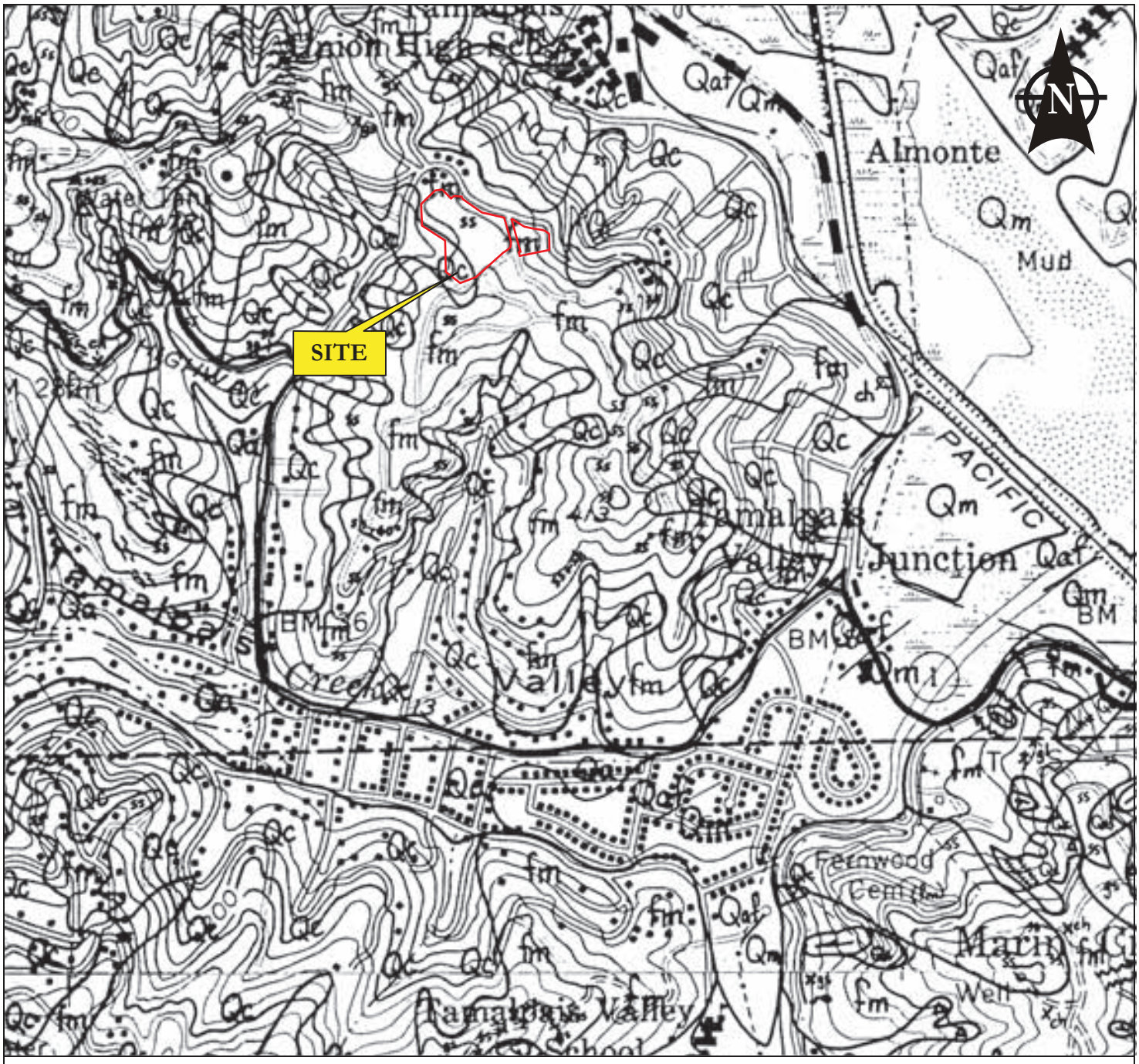


Base: USGS Topographic Maps, San Rafael & Point Bonita Quadrangles, 7.5 Minute Series, 2015
Scale: 1 inch = 2,000 feet

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	MOUNT TAMALPAIS SCHOOL 100 HARVARD AVENUE MILL VALLEY, CALIFORNIA	VICINITY MAP
	PROJECT NO. 3516-1R1 / OCTOBER 2022	FIGURE A-1


LEGEND
B-2 Approximate Location of Soil Boring by Murray Engineers, Inc., July 6, 7 & 19, 2022
Approximate Limits of Fill and Fill Slope
A A' Location of Cross-Sections (see Figure A-5 & A-6)
Base: New Site Plan by Michael Heacock Architects received September 23, 2022 and topographic information provided by CSW|ST2 dated October 3, 2022
KEY NOTES
1 Outline of (E) Building to Remain (Solid-Black)
2 Area of (N) Building (Shaded-Green)
3 Area of (N) Building Addition (Shaded-Green)
Approximate Scale: 1 inch = 50 feet
N
2011 Classroom addition supported on spread footings
2011 Canopy supported on piers
First School Building (1956) supported on spread footings
1986 Addition supported on drilled piers
1994 Multipurpose Building supported on drilled pier and grade beam foundations
1994 Addition supported on 18 inch deep spread footings
Second School Building (1962) - presumably supported on spread footings
(E) "BARN"
(E) FIELD
(E) PLAYGROUND
(E) COURTS
(E) GATE
(E) OUTDOOR PLAY
(E) CANOPY
(E) MAILBOX
(E) FLAG POLE
(E) PARKING LOT (33 SPACES)
(E) ACCESSIBILE PARKING SEE G001
(E) SIGN
(E) GATE
DRIVEWAY
WELLESLEY AVENUE
HARVARD AVENUE
CALIFORNIA AVENUE
COLBY AVE. (30' R/W[9])
ABANDONED VASSAR AVE. PER 1127-70 (30' R/W[1])
LANDS OF MT. TAMALPAIS PRIMARY SCHOOL APN: 051-034-01 (DN:1992-048841[11])
BLOCK 209[1]
(E) MODULAR CLASSROOMS TO BE REMOVED
(E) FIRE ACCESS DRIVE
MERIDIAN ST.
36" TREE CLUSTER
EXISTING REDWOODS
DENSE VEGETATION
A
A'
B
B'
B-1
B-2
B-3
B-4
B-5
B-6
B-7
B-8
MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES
PARTIAL SITE PLAN
MOUNT TAMALPAIS SCHOOL
100 HARVARD AVENUE
MILL VALLEY, CALIFORNIA
PROJECT NO. 3516-1R1
OCTOBER 2022
FIGURE A-2

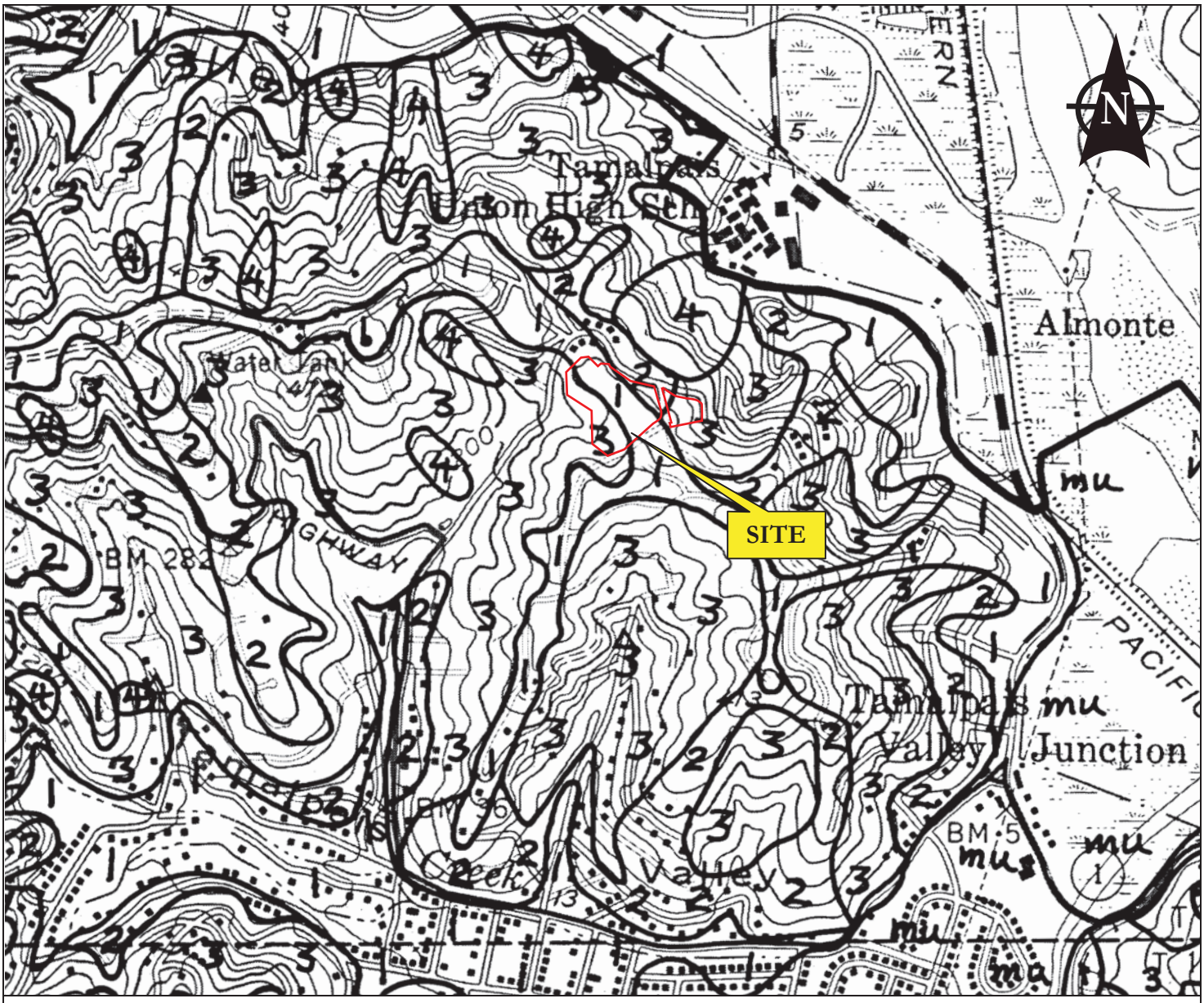


LEGEND

Qaf	Artifical Fill	Qc	Colluvium		Debris Flow Landslide
Qm	Bay Mud	fm	Franciscan Melange (ss = sandstone & shale)		Block Slump Landslides
Qa	Alluvium				

Base: Geology of the Lower Ross Valley, Corte Madera, Homestead Valley, Tamalpais Valley, Tennessee Valley, and Adjacent Areas by Rice & Smith, 1976 | Scale: 1 inch = 1,000 feet

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	MOUNT TAMALPAIS SCHOOL 100 HARVARD AVENUE MILL VALLEY, CALIFORNIA		LOCAL GEOLOGIC MAP
	PROJECT NO. 3516-1R1	OCTOBER 2022	FIGURE A-3



Legend & Selected Map Symbols

Decreasing stability →

1	2	3	4

Localities where there is an apparent threat of boulders or other loose rock masses tumbling down steep slopes.

Boundary of study area

Zone 1: The most stable category. This zone includes resistant rock that is either exposed or is covered only by shallow colluvium or soil.

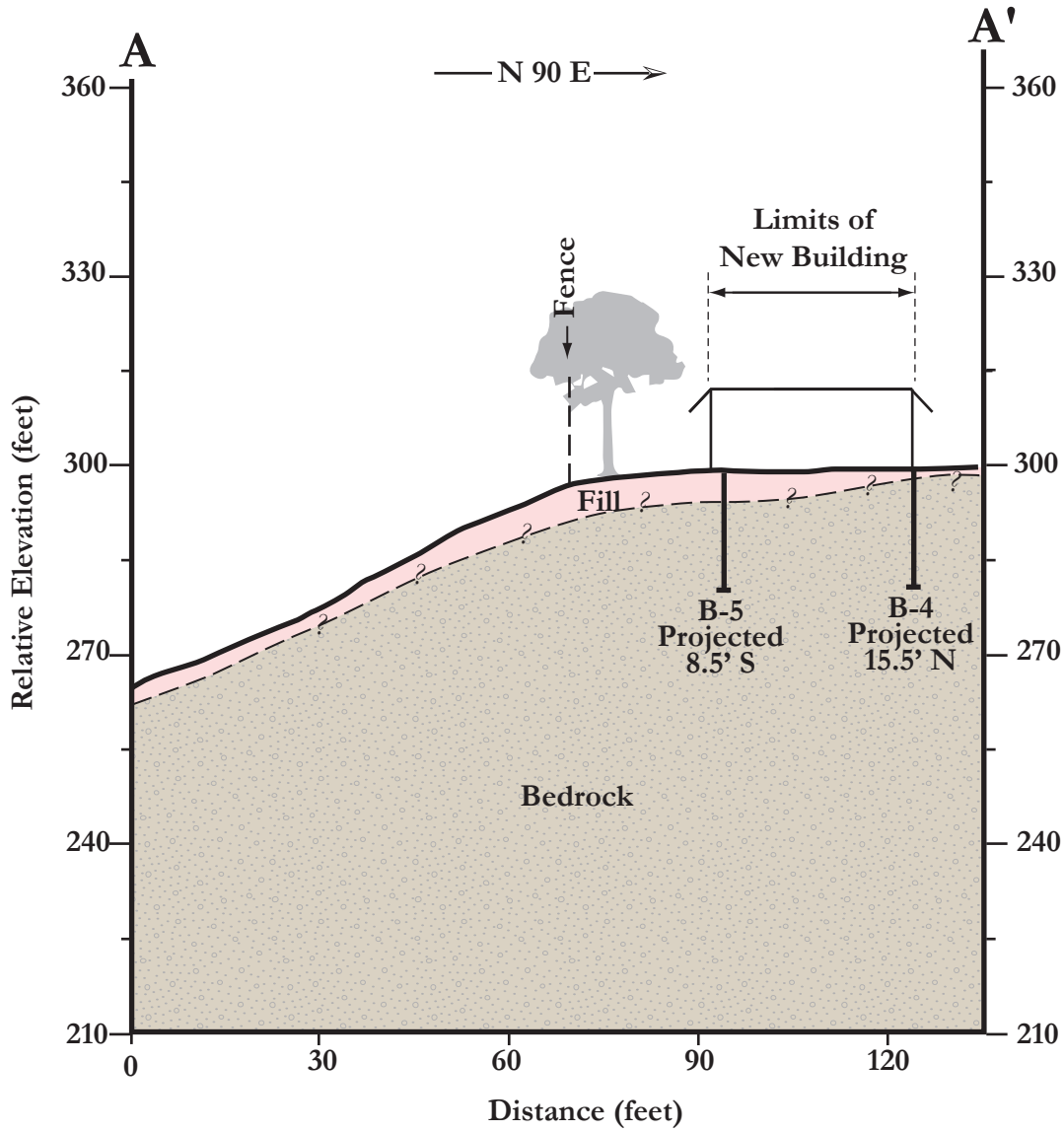
Zone 2: Includes narrow ridge and spur crests that are underlain by relatively competent bedrock, but are flanked by steep, potentially unstable slopes.

Zone 3: Areas where the steepness of the slopes approaches the stability limits of the underlying geological materials.

Zone 4: The least stable category. This includes most landslide deposits in upslope areas, whether presently active or not, and slopes on which there is substantial evidence of downlope creep of the surface materials.

Base: Interpretation of the Relative Stability of Upland Slopes in the Lower Ross Valley, Corte Madera, Homestead Valley, Tamalpais Valley, and Tennessee Valley Areas Marin County, California, Rice & Smith, 1976
Approximate Scale: 1 inch = 1,000 feet

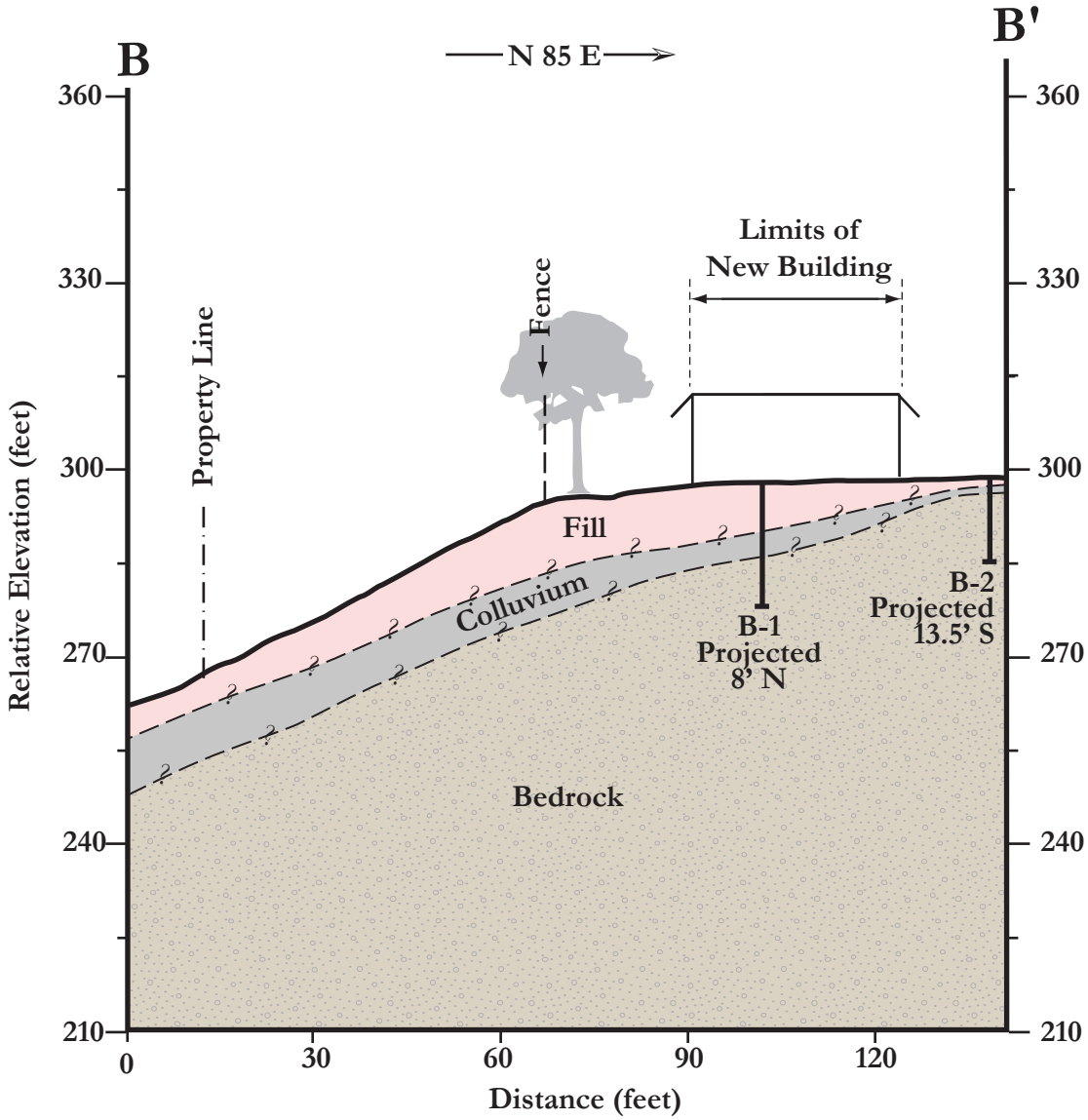
MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	MOUNT TAMALPAIS SCHOOL 100 HARVARD AVENUE MILL VALLEY, CALIFORNIA		LOCAL RELATIVE SLOPE STABILITY MAP
	PROJECT NO. 3516-1R1	OCTOBER 2022	FIGURE A-4



Base: Topographic Survey by CSW | ST2, dated October 3, 2022

Approximate Scale: 1 inch = 30 feet (horizontal=vertical)

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	MOUNT TAMALPAIS SCHOOL 100 HARVARD AVENUE MILL VALLEY, CALIFORNIA	GEOLOGIC CROSS-SECTION A-A'
	PROJECT NO. 3516-1R1 / OCTOBER 2022	FIGURE A-5



Base: Topographic Survey by CSW | ST2, dated October 3, 2022

Approximate Scale: 1 inch = 30 feet (horizontal=vertical)

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	MOUNT TAMALPAIS SCHOOL 100 HARVARD AVENUE MILL VALLEY, CALIFORNIA	GEOLOGIC CROSS-SECTION B-B'
	PROJECT NO. 3516-1R1 / OCTOBER 2022	FIGURE A-6

Date(s) Drilled **July 6, 2022**	Logged By **NB**	Checked By **MP**
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **7 inch diameter**	Total Depth of Borehole **20.1 feet bgs**
Drill Rig Type **Track Mounted Rig**	Drilling Contractor **Cuesta Geo, Inc.**	Approximate Surface Elevation **~297 feet (relative)**
Groundwater Level and Date Measured **Not Encountered ATD**	Sampling Method(s) **3" OD & 2" OD SPT Split Spoon Samplers**	Hammer Data **130 lb, 30 in drop,**
Borehole Backfill **Grout**	Location **South side of proposed building**	

Elevation, feet	Depth, feet	Sample Type	Sampling Resistance, blows/foot	Relative Consistency	USCS Symbol	MATERIAL DESCRIPTION	Water Content, %	Dry Density (PCF)
	0			Stiff	CL	6" GRASS over FILL: SILTY CLAY, dark brown, heterogeneous, low plasticity, trace fine-grained sand, trace rock fragments, trace rootlets, moist		
			10				16	
			14	Stiff	CL	FILL: SILTY CLAY, yellowish brown to grayish brown, low plasticity, trace fine-grained sand, trace iron oxide staining, moist	26	
	5		10	Stiff	CL	FILL: SILTY CLAY, dark brown, homogeneous, low plasticity, trace fine-grained sand, trace organics, moist	21	
					CL	PI=10%; LL=30% (sample from 8 to 8.5 feet)	20	
			7	Medium Stiff	CL	LEAN CLAY, dark bluish gray, homogeneous, moderate plasticity, trace fine- to medium-grained sand, moist (Colluvium)	22	
	10		33	Hard			26	
				Soft*	BR	SANDSTONE, yellowish brown, moderately weathered, slightly moist (Franciscan Complex) *designates hardness of bedrock (see Figure B-11)	13	
	15		61/10"				7	
	20		50/2"				4	
						Refusal at 20.1 feet bgs		

MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

MOUNT TAMALPAIS SCHOOL
100 HARVARD AVENUE
MILL VALLEY, CALIFORNIA

LOG OF BORING B-1

PROJECT NO. 3516-1R1 | **OCTOBER 2022** | **FIGURE B-1**

Date(s) Drilled **July 7, 2022**	Logged By **NB**	Checked By **MP**
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **7 inch diameter**	Total Depth of Borehole **13.5 feet bgs**
Drill Rig Type **Track Mounted Rig**	Drilling Contractor **Cuesta Geo, Inc.**	Approximate Surface Elevation **~297 feet (relative)**
Groundwater Level and Date Measured **Not Encountered ATD**	Sampling Method(s) **3" OD & 2" OD SPT Split Spoon Samplers**	Hammer Data **140 lb, 30 in drop, rope & cathead**
Borehole Backfill **Grout**	Location **East side of proposed building**	

Elevation, feet	Depth, feet	Sample Type	Sampling Resistance, blows/foot	Relative Consistency	USCS Symbol	MATERIAL DESCRIPTION	Water Content, %	Dry Density (PCF)
	0			Soft	CL	~3" GRASS over FILL: SANDY CLAY, dark to olive brown, heterogeneous, low plasticity, medium- to fine-grained sand, moist		
				Soft	CL		25	
				Stiff	CL	FILL: SANDY CLAY, dark to olive brown, heterogeneous, low plasticity, trace medium- to fine-grained sand, moist	18	
			50/11"	Soft*	BR		10	
			57/10"			ROCK FRAGMENTS in a CLAYEY MATRIX, dark brown, homogeneous, low plasticity, subangular rock fragments, moist (Colluvium)	7	125
			57/10"				7	109
	5					MELANGE (SANDSTONE & SHALE), dark yellowish brown to reddish brown, severely weathered, moist (Franciscan Complex)	10	111
						*designates hardness of bedrock (see Figure B-11)		
			47/11"				12	
							10	120
	10						6	
			50/1"				3	
						Refusal at 13.5 feet bgs		
	15							
	20							

MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

MOUNT TAMALPAIS SCHOOL
100 HARVARD AVENUE
MILL VALLEY, CALIFORNIA

LOG OF BORING B-2

PROJECT NO. 3516-1R1 | **OCTOBER 2022** | **FIGURE B-2**

Date(s) Drilled **July 6, 2022**	Logged By **NB**	Checked By **MP**
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **7 inch diameter**	Total Depth of Borehole **20.5 feet bgs**
Drill Rig Type **Track Mounted Rig**	Drilling Contractor **Cuesta Geo, Inc.**	Approximate Surface Elevation **~298 feet (relative)**
Groundwater Level and Date Measured **Not Encountered ATD**	Sampling Method(s) **3" OD & 2" OD SPT Split Spoon Samplers**	Hammer Data **140 lb, 30 in drop, rope & cathead**
Borehole Backfill **Grout**	Location **Center of proposed building**	

Elevation, feet	Depth, feet	Sample Type	Sampling Resistance, blows/foot	Relative Consistency	USCS Symbol	MATERIAL DESCRIPTION	Water Content, %	Dry Density (PCF)
	0			Soft	CL	GRASS over FILL: SANDY CLAY, dark to olive brown, heterogeneous, low plasticity, medium- to fine-grained sand, moist		
			11	Stiff	CL	FILL: SANDY CLAY, dark to olive brown, heterogeneous, low plasticity, trace medium- to fine-grained sand, moist	28 16	111
			16				22	
				Soft*	BR	SHALE, dark brown, severely weathered, slightly moist (Franciscan Complex)	14	113
	5		52				10 8	123
			51/11"				8 8	113
			50/5"	Soft*	BR	MELANGE (SANDSTONE & SHALE), dark yellowish brown to reddish brown, severely weathered, moist (Franciscan Complex) *designates hardness of bedrock (see Figure B-11)	9	
	10							
			50/4"	Soft*	BR	SANDSTONE, yellowish brown, moderately weathered, slightly moist (Franciscan Complex)	6	
	15							
			82	Soft*	BR	MELANGE (SANDSTONE & SHALE), dark yellowish brown to reddish brown, severely weathered, moist (Franciscan Complex) *designates hardness of bedrock (see Figure B-11)	11	
	20							

Bottom of Boring at 20.5 feet bgs

MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

MOUNT TAMALPAIS SCHOOL
100 HARVARD AVENUE
MILL VALLEY, CALIFORNIA

LOG OF BORING B-3

PROJECT NO. 3516-1R1 | **OCTOBER 2022** | **FIGURE B-3**

C:\Users\odessa\Desktop\Borings\Mt Tam School 3616-1.bgs [123 Murray 22-WC, DD.tpl]

Date(s) Drilled **July 6, 2022**	Logged By **NB**	Checked By **MP**
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **7 inch diameter**	Total Depth of Borehole **19.5 feet bgs**
Drill Rig Type **Track Mounted Rig**	Drilling Contractor **Cuesta Geo, Inc.**	Approximate Surface Elevation **~298 feet (relative)**
Groundwater Level and Date Measured **Not Encountered ATD**	Sampling Method(s) **3" OD & 2" OD SPT Split Spoon Samplers**	Hammer Data **140 lb, 30 in drop, rope & cathead**
Borehole Backfill **Grout**	Location **Northeast side of proposed building**	

Elevation, feet	Depth, feet	Sample Type	Sampling Resistance, blows/foot	Relative Consistency	USCS Symbol	MATERIAL DESCRIPTION	Water Content, %	Dry Density (PCF)
	0		10	Soft	CL	GRASS over FILL: SANDY CLAY, dark to olive brown, heterogeneous, low plasticity, medium- to fine-grained sand, moist	46	
				Soft*	BR		25	92
			19			SHALE, dark brown, severely weathered, slightly moist (Franciscan Complex)	11	
							14	112
							12	
	5		36	Soft*	BR	MELANGE (SANDSTONE & SHALE), dark yellowish brown to reddish brown, severely weathered, moist (Franciscan Complex)	13	
							6	120
			54/11"				10	
						*designates hardness of bedrock (see Figure B-11)	4	130
	10		30				13	
							11	
			57/8"				11	104
	15						8	
			45				8	
	20					Bottom of Boring at 19.5 feet bgs		

MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

MOUNT TAMALPAIS SCHOOL
100 HARVARD AVENUE
MILL VALLEY, CALIFORNIA

LOG OF BORING B-4

PROJECT NO. 3516-1R1 | **OCTOBER 2022** | **FIGURE B-4**

Date(s) Drilled **July 6, 2022**	Logged By **NB**	Checked By **MP**
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **7 inch diameter**	Total Depth of Borehole **18.8 feet bgs**
Drill Rig Type **Track Mounted Rig**	Drilling Contractor **Cuesta Geo, Inc.**	Approximate Surface Elevation **~298 feet (relative)**
Groundwater Level and Date Measured **Not Encountered ATD**	Sampling Method(s) **3" OD & 2" OD SPT Split Spoon Samplers**	Hammer Data **140 lb, 30 in drop, rope & cathead**
Borehole Backfill **Grout**	Location **Northwest corner of proposed building**	

Elevation, feet	Depth, feet	Sample Type	Sampling Resistance, blows/foot	Relative Consistency	USCS Symbol	MATERIAL DESCRIPTION	Water Content, %	Dry Density (PCF)
	0			Soft	CL	GRASS over FILL: SANDY CLAY, dark to olive brown, heterogeneous, low plasticity, medium- to fine-grained sand, moist		
			10	Stiff	CL	FILL: SILTY CLAY, dark brown, heterogeneous, low plasticity, trace fine-grained sand, trace rock fragments, trace rootlets, moist	19 17	103
			8				20 24	95
			3	Soft				
	5			Soft*	BR	MELANGE (SANDSTONE & SHALE), dark yellowish brown to reddish brown, severely weathered, moist (Franciscan Complex)	14	110
			36				10	123
			53/11"	Soft*	BR	SANDSTONE, yellowish brown, moderately weathered, slightly moist (Franciscan Complex)	9 14	119
			60/9"				11	
	10			Soft*	BR	MELANGE (SANDSTONE & SHALE), dark yellowish brown to reddish brown, severely weathered, moist (Franciscan Complex) *designates hardness of bedrock (see Figure B-11)	12	116
							9	
	15		96/11"				6	
			50/4"	Soft*	BR	SANDSTONE, yellowish brown, moderately weathered, slightly moist (Franciscan Complex) *designates hardness of bedrock (see Figure B-11)		
	20					Refusal at 18.8 feet bgs		

MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

MOUNT TAMALPAIS SCHOOL
100 HARVARD AVENUE
MILL VALLEY, CALIFORNIA

LOG OF BORING B-5

PROJECT NO. 3516-1R1 | **OCTOBER 2022** | **FIGURE B-5**

Date(s) Drilled **July 7, 2022**	Logged By **NB**	Checked By **MP**
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **7 inch diameter**	Total Depth of Borehole **17 feet bgs**
Drill Rig Type **Track Mounted Rig**	Drilling Contractor **Cuesta Geo, Inc.**	Approximate Surface Elevation **~298 feet (relative)**
Groundwater Level and Date Measured **Not Encountered ATD**	Sampling Method(s) **3" OD & 2" OD SPT Split Spoon Samplers**	Hammer Data **140 lb, 30 in drop, rope & cathead**
Borehole Backfill **Grout**	Location **Near western building addition**	

Elevation, feet	Depth, feet	Sample Type	Sampling Resistance, blows/foot	Relative Consistency	USCS Symbol	MATERIAL DESCRIPTION	Water Content, %	Dry Density (PCF)
	0			Loose	SP	FILL: SAND, dark brown, heterogeneous, coarse- to medium-grained, poorly cemented, trace roots, moist	9	
				Medium Stiff	CL	FILL: SILTY CLAY, dark brown, heterogeneous, low plasticity, trace fine-grained sand, trace rootlets, moist	13	
			7				20	
							21	100
			14	Stiff			23	
	5						17	108
			10	Stiff	CL	ROCK FRAGMENTS in a CLAYEY MATRIX, dark brown, homogeneous, low plasticity, subangular rock fragments, moist (Colluvium)	19	
							21	101
			12				18	
				Soft*	BR	MELANGE (SANDSTONE & SHALE), dark yellowish brown to reddish brown, severely weathered, moist (Franciscan Complex)	34	90
	10		31			*designates hardness of bedrock (see Figure B-11)	14	
	15		46				11	
						Bottom of Boring at 17 feet bgs		
	20							

MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

MOUNT TAMALPAIS SCHOOL
100 HARVARD AVENUE
MILL VALLEY, CALIFORNIA

LOG OF BORING B-6

PROJECT NO. 3516-1R1 | **OCTOBER 2022** | **FIGURE B-6**

Date(s) Drilled **July 19, 2022**	Logged By **NB**	Checked By **MP**
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **7 inch diameter**	Total Depth of Borehole **16.8 feet bgs**
Drill Rig Type **Track Mounted Rig**	Drilling Contractor **Cuesta Geo, Inc.**	Approximate Surface Elevation **~298 feet (relative)**
Groundwater Level and Date Measured **Not Encountered ATD**	Sampling Method(s) **3" OD & 2" OD SPT Split Spoon Samplers**	Hammer Data **140 lb, 30 in drop, rope & cathead**
Borehole Backfill **Grout**	Location **Near southern building addition**	

Elevation, feet	Depth, feet	Sample Type	Sampling Resistance, blows/foot	Relative Consistency	USCS Symbol	MATERIAL DESCRIPTION	Water Content, %	Dry Density (PCF)
	0					5" Concrete Slab over 4" Aggregate Baserock		
			49/11"	Soft*	BR	MELANGE (SANDSTONE & SHALE), dark yellowish brown to reddish brown, severely weathered, moist (Franciscan Complex)	8 8	118
			35				8 12	121
			56/11.5"				15	
	5		33/4"				7	
			50/2"				3	
	10			Soft*	BR	SANDSTONE, yellowish brown, moderately weathered, slightly moist (Franciscan Complex) *designates hardness of bedrock (see Figure B-11)		
			50/4"				5	
	15						6	
			50/4"				6	
						Refusal at 16.8 feet bgs		
	20							

MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

MOUNT TAMALPAIS SCHOOL
100 HARVARD AVENUE
MILL VALLEY, CALIFORNIA

LOG OF BORING B-7

PROJECT NO. 3516-1R1 | **OCTOBER 2022** | **FIGURE B-7**

Date(s) Drilled **July 19, 2022**	Logged By **NB**	Checked By **MP**
Drilling Method **Hollow Stem Auger**	Drill Bit Size/Type **7 inch diameter**	Total Depth of Borehole **15.9 feet bgs**
Drill Rig Type **Track Mounted Rig**	Drilling Contractor **Cuesta Geo, Inc.**	Approximate Surface Elevation **~298 feet (relative)**
Groundwater Level and Date Measured **Not Encountered ATD**	Sampling Method(s) **3" OD & 2" OD SPT Split Spoon Samplers**	Hammer Data **140 lb, 30 in drop, rope & cathead**
Borehole Backfill **Grout**	Location **Near southern building addition**	

Elevation, feet	Depth, feet	Sample Type	Sampling Resistance, blows/foot	Relative Consistency	USCS Symbol	MATERIAL DESCRIPTION	Water Content, %	Dry Density (PCF)
	0			Soft	CL	FILL: LEAN CLAY, olive brown to brown, heterogeneous, low plasticity, trace fine-grained sand, trace rootlets, slightly moist		
				Soft*	BR	MELANGE (SANDSTONE & SHALE), dark yellowish brown to reddish brown, severely weathered, moist (Franciscan Complex)		
			14				16 15	105
			23				15 13	114
	5		32				15 10	114
			40/11"				14 15	110
	10		80/11"	Soft*	BR	SANDSTONE, yellowish brown, moderately weathered, slightly moist (Franciscan Complex) *designates hardness of bedrock (see Figure B-11)	6	
	15		69/11"				12	
						Refusal at 15.9 feet bgs		
	20							

MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

MOUNT TAMALPAIS SCHOOL
100 HARVARD AVENUE
MILL VALLEY, CALIFORNIA

LOG OF BORING B-8

PROJECT NO. 3516-1R1 | **OCTOBER 2022** | **FIGURE B-8**

Elevation, feet	Depth, feet	Sample Type	Sampling Resistance, blows/foot	Relative Consistency	USCS Symbol	MATERIAL DESCRIPTION	Water Content, %
1	2	3	4	5	6	7	8

COLUMN DESCRIPTIONS

1 **Elevation, feet:** Elevation (MSL, feet)

2 **Depth, feet:** Depth in feet below the ground surface.

3 **Sample Type:** Type of soil sample collected at the depth interval shown.

4 **Sampling Resistance, blows/foot:** Number of blows required to advance the sampler 12 inches or the distance shown. Blow counts for the 3.0-inch O.D. and 2.5-inch O.D. samplers have been corrected for sampler size to SPT values using conversion factors of 0.65 and 0.77, respectively.

5 **Relative Consistency:** Relative consistency of the subsurface material.

6 **USCS Symbol:** USCS symbol of the subsurface material.

7 **MATERIAL DESCRIPTION:** Description of material encountered. May include consistency, moisture, color, and other descriptive text.

8 **Water Content, %:** Water content of the soil sample, expressed as percentage of dry weight of sample.

FIELD AND LABORATORY TEST ABBREVIATIONS

CHEM: Chemical tests to assess corrosivity
COMP: Compaction test
CONS: One-dimensional consolidation test
LL: Liquid Limit, percent
PI: Plasticity Index, percent
SA: Sieve analysis (percent passing No. 200 Sieve)
UC: Unconfined compressive strength test, Qu, in ksf
WA: Wash sieve (percent passing No. 200 Sieve)

TYPICAL MATERIAL GRAPHIC SYMBOLS

- Sandstone
- Well graded GRAVEL (GW)
- Poorly graded GRAVEL (GP)
- Well graded GRAVEL with Silt (GW-GM)
- Well graded GRAVEL with Clay (GW-GC)
- Poorly graded GRAVEL with Silt (GP-GM)
- Poorly graded GRAVEL with Clay (GP-GC)
- Silty GRAVEL (GM)
- Clayey GRAVEL (GC)
- Well graded SAND (SW)
- Poorly graded SAND (SP)
- Well graded SAND with Silt (SW-SM)
- Well graded SAND with Clay (SW-SC)
- Poorly graded SAND with Silt (SP-SM)
- Poorly graded SAND with Clay (SP-SC)
- Silty SAND (SM)
- Clayey SAND (SC)
- SILT, SILT w/SAND, SANDY SILT (ML)
- Lean CLAY, CLAY w/SAND, SANDY CLAY (CL)
- SILT, SILT w/SAND, SANDY SILT (MH)
- Fat CLAY, CLAY w/SAND, SANDY CLAY (CH)
- SILT, SILT with SAND, SANDY SILT (ML-MH)
- Lean-Fat CLAY, CLAY w/SAND, SANDY CLAY (CL/CH)
- SILTY CLAY (CL-ML)
- Lean CLAY/PEAT (CL-OL)
- Fat CLAY/SILT (CH-MH)
- Fat CLAY/PEAT (CH-OH)
- Silty SAND to Sandy SILT (SM-ML)
- Silty SAND to Sandy SILT (SM-MH)
- Clayey SAND to Sandy CLAY (SC-CL)
- Clayey SAND to Sandy CLAY (SC-CH)
- SILT to CLAY (CL/ML)
- Silty to Clayey SAND (SC/SM)

TYPICAL SAMPLER GRAPHIC SYMBOLS

- 2 inch-OD Unlined Split Spoon (SPT)
- 2.5 inch-OD Unlined Split Spoon
- 3 inch-OD Unlined Split Spoon
- Shelby Tube (thin-walled, fixed head)
- Grab Sample
- Bulk Sample
- Pitcher Sample
- Other Sampler

OTHER GRAPHIC SYMBOLS

- Water level (at time of drilling, ATD)
- Water level (after waiting a given time)
- Minor change in material properties within a stratum
- Inferred or gradational contact between strata
- Queried contact between strata

GENERAL NOTES

1. Soil classifications are based on the Unified Soil Classification System. Descriptions and stratum lines are interpretive, and actual lithologic changes may be gradual. Field descriptions may have been modified to reflect results of lab tests.
2. Descriptions on these logs apply only at the specific boring locations and at the time the borings were advanced. They are not warranted to be representative of subsurface conditions at other locations or times.

PRIMARY DIVISIONS			SOIL TYPE	SECONDARY DIVISIONS
COARSE GRAINED SOILS *(<50% Fines)*	GRAVEL	CLEAN GRAVEL *(<5% Fines)*	**GW**	Well graded gravel, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravel or gravel-sand mixtures, little or no fines.
		GRAVEL *with* FINES	**GM**	Silty gravels, gravel-sand-silt mixtures, non-plastic fines.
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines.
	SAND	CLEAN SAND *(<5% Fines)*	**SW**	Well graded sands, 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, non-plastic fines.
			SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS *(>50% Fines)*	SILT AND CLAY *Liquid limit <50%*		**ML**	Inorganic silts and very fine sands, with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, lean clays.
			OL	Organic silts and organic clays of low plasticity.
	SILT AND CLAY *Liquid limit >50%*		**MH**	Inorganic silt, micaceous or diatomaceous fine sandy or silty soil.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			**Pt**	Peat and other highly organic soils.

RELATIVE DENSITY

SAND & GRAVEL	BLOWS/FOOT*
VERY LOOSE	0 to 4
LOOSE	4 to 10
MEDIUM DENSE	10 to 30
DENSE	30 to 50
VERY DENSE	OVER 50

CONSISTENCY

SILT & CLAY	STRENGTH^	BLOWS/FOOT*
VERY SOFT	0 to 0.25	0 to 2
SOFT	0.25 to 0.5	2 to 4
MEDIUM STIFF	0.5 to 1	4 to 8
STIFF	1 to 2	8 to 16
VERY STIFF	2 to 4	16 to 32
HARD	OVER 4	OVER 32

GRAIN SIZES

BOULDERS	COBBLES	GRAVEL		SAND			SILT & CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE	

12" 3" 3/4" 4 10 40 200

SIEVE OPENINGS U.S. STANDARD SERIES SIEVE

Classification is based on the Unified Soil Classification System; fines refer to soil passing a No. 200 sieve.

*Standard penetration test (SPT) resistance using a 140-pound hammer falling 30 inches on a 2-inch outside diameter split spoon sampler; blow counts for the 3.0-inch O.D. and 2.5-inch O.D. samplers have been corrected for sampler size to SPT values using conversion factors of 0.65 and 0.77, respectively.

^ Shear strength in tons/sq. ft. as estimated by SPT resistance, field and laboratory tests, and/or visual observation.

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES

MOUNT TAMALPAIS SCHOOL
100 HARVARD AVENUE
MILL VALLEY, CALIFORNIA

UNIFIED SOIL CLASSIFICATION SYSTEM

PROJECT NO. 3516-1R1 | **OCTOBER 2022** | **FIGURE B-10**

WEATHERING

Fresh
Rock fresh, crystals bright, few joints may show slight staining. Rock rings under hammer if crystalline.

Very Slight
Rock generally fresh, joints stained, some joints may show thin clay coatings, crystals in broken face show bright. Rock rings under hammer if crystalline.

Slight
Rock generally fresh, joints stained, and discoloration extends into rock up to 1 inch. Joints may contain clay. In granitoid rocks some occasional feldspar crystals are dull and discolored. Crystalline rocks ring under hammer.

Moderate
Significant portions of rock show discoloration and weathering effects. In granitoid rocks, most feldspars are dull and discolored; some are clayey. Rock has dull sound under hammer and shows significant loss of strength as compared with fresh rock.

Moderately Severe
All rock excepts quartz discolored or stained. In granitoid rocks, all feldspars dull and discolored and majority show kaolinization. Rock shows severe loss of strength and can be excavated with geologist's pick. Rock goes "clunk" when struck.

Severe
All rock except quartz discolored or stained. Rock "fabric" clear and evident, but reduced in strength to strong soil. In granitoid rocks, all feldspars kaolinized to some extent. Some fragments of strong rock usually left.

Very Severe
All rock except quartz discolored and stained. Rock "fabric" discernible, but mass effectively reduced to "soil" with only fragments of strong rock remaining.

Complete
Rock reduced to "soil". Rock fabric not discernible or discernible only in small scattered locations. Quartz may be present as dikes or stringers.

HARDNESS

Very Hard
Cannot be scratched with knife or sharp pick. Hand specimens requires several hard blows of geologist's hammer.

Hard
Can be scratched with knife or pick only with difficulty. Hard blow of hammer required to detach hand specimen.

Moderately Hard
Can be scratched with knife or pick. Gouges or grooves to 1/4 inch deep can be excavated by hard blow of point of a geologist's pick. Hard specimen can be detached by moderate blow.

Medium
Can be grooved or gouged 1/16 inch deep by firm pressure on knife or pick point. Can be excavated in small chips to pieces about 1 inch maximum size by hard blows of the point of geologist's pick.

Soft
Can be gouged or grooved readily with knife or pick point. Can be excavated in chips to pieces several inches in size by moderate blows of a pick point. Small thin pieces can be broken by finger pressure.

Very Soft
Can be carved with knife. Can be excavated readily with point of pick. Pieces 1 inch or more in thickness can be broken with finger pressure. Can be scratched readily by fingernail.

JOINT BEDDING & FOLIATION SPACING

Spacing	Joints	Bedding & Foliation
Less than 2 in.	Very Close	Very Thin
2 in to 1 ft.	Close	Thin
1 ft. to 3 ft.	Moderately Close	Medium
3 ft. to 10 ft.	Wide	Thick
More than 10 ft.	Very Wide	Very Thick

ROCK QUALITY DESIGNATOR (RQD)

RQD, as a percentage	Descriptor
Exceeding 90	Excellent
90 to 75	Good
75 to 50	Fair
50 to 25	Poor
Less than 25	Very Poor

APPENDIX C

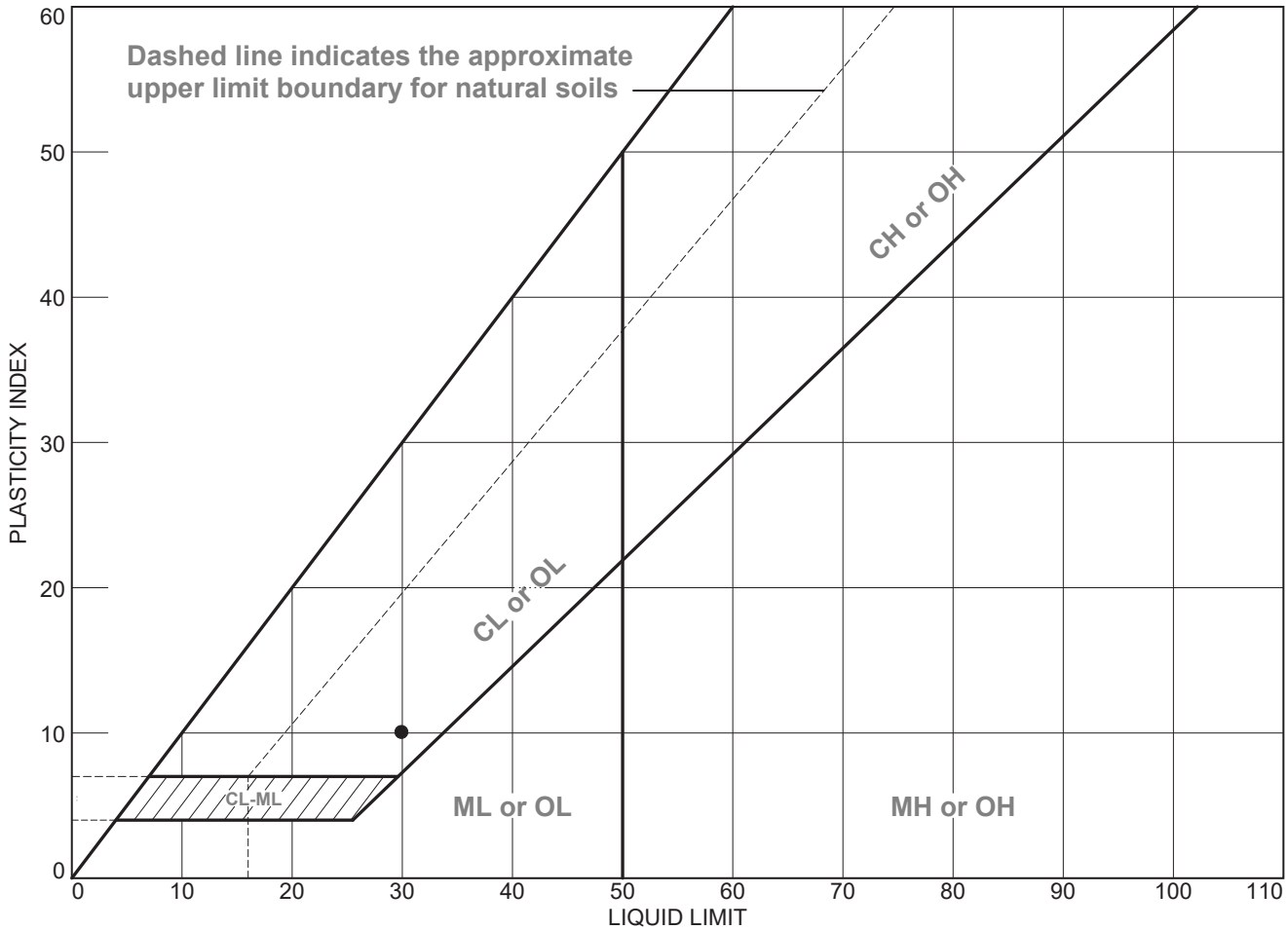
LABORATORY TESTS

Samples from the subsurface exploration were selected for tests to establish the physical and engineering properties of the soils. The tests performed are briefly described below.

The natural moisture content and dry density was established on most samples and dry density on selected samples recovered from the borings. The samples were initially trimmed to obtain volume and wet weight measurements and subsequently dried in accordance with ASTM D2216. After drying, the weight of each sample was obtained to determine the moisture content representative of field conditions and time the samples were collected. The results are presented on the boring logs at the appropriate sample depths.

The Atterberg Limits were determined on one sample in accordance with ASTM D 4318. The Atterberg Limits are the moisture content within which the soil is workable or plastic. The results of this testing are presented on Figure C-1 and on the boring logs at the appropriate sample depths.

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	B-1	1	8-8.5	22.3	20	30	10	CL

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LIQUID & PLASTIC LIMITS TEST REPORT

PROJECT NO. 3516-1R1 | OCTOBER 2022 | FIGURE C-1