GEOTECHNICAL INVESTIGATION
WATERS RESIDENCE
APN 351-42-004 PEACOCK COURT
SANTA CLARA COUNTY, CALIFORNIA

THIS REPORT HAS BEEN PREPARED FOR:

MELISSA & JEFF WATERS

1063 CHERRY AVENUE

SAN JOSE, CALIFORNIA 95125

APRIL 2020

MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

April 23, 2020
Project No. 3262-1R1

Melissa & Jeff Waters
1063 Cherry Avenue
San Jose, California 95125

RE: GEOLOGIC & GEOTECHNICAL INVESTIGATION, WATERS RESIDENCE, APN 351-42-004 PEACOCK COURT, SANTA CLARA COUNTY, CALIFORNIA

Dear Ms. and Mr. Waters:

We are pleased to present the results of our geologic and geotechnical investigation relating to the design and construction of the proposed residence and associated improvements on your property, APN 351-42-004, on the Peacock Court in an unincorporated area of Santa Clara County, California. This report summarizes the results of our field, laboratory and engineering work, 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.

CERTIFIED ENGINEERING GEOLOGIST MARK F. BAUMANN 1787 STATE OF CALIFORNIA

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INTRODUCTION

This report presents the results of our geologic and geotechnical investigation relating to the design and construction of the proposed residential development on the Waters property, APN 351-42-004, on Peacock Court in unincorporated Santa Clara County, California. The location of the property is shown on the Vicinity Map, Figure A-1. In 2017, we performed a limited geologic and geotechnical investigation to evaluate site development feasibility. The results of our limited investigation were presented in our report dated July 11, 2017. The purpose of this 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. Pertinent information developed as part of our 2017 investigation is included in this report.

Project Description

The project will include construction of a two-story residence with an attached two-car garage, a detached one-car garage, and a detached, single-story chapel along the crest of a spur ridge in the north central (uphill) portion of the hillside property. A two-story accessory dwelling unit (A.D.U.) comprised of upper level living space and a lower level, daylighting basement with a basketball court is planned on the hillside in the western portion of the property. Additional site improvements will include an entry courtyard along the front (west side) of the residence, a covered loggia at the rear (east side) of the residence, and a covered carport along the east side of the A.D.U. In addition, we understand that a swimming pool is planned at the rear of the residence; however, the location of the pool has not been finalized. The site will be accessed by a graded driveway that will extend along the northern property line and lead to a parking area in front of the residence. Retaining walls will be required as part of the foundation systems for the residence, the chapel, and the A.D.U. and will be used to accommodate proposed grade changes along the driveway, parking areas, and court yard. We anticipate that structural loads will be relatively light and typical of wood- and/or steel-framed, single-family residential construction. The approximate layout of the proposed improvements is shown on Figure A-2, Partial Site Plan & Engineering Geologic Map.

Scope of Services

We performed the following services in accordance with our agreement dated October 14, 2019 (executed on October 16, 2019):

- Reviewed geologic and seismic conditions in the area of the site
- Reviewed our limited geologic and geotechnical investigation dated July 11, 2017 and incorporated the exploratory borings, laboratory testing, analyses, and conclusions from that investigation into the current investigation
- Performed a reconnaissance and geologic mapping in the area of the proposed improvements to evaluate site-specific geologic hazards and geotechnical conditions
- Explored the subsurface conditions by excavating, logging, and sampling seven exploratory borings on site in the area of the proposed improvements and on the slope below the proposed residence
- Performed laboratory analyses and testing on selected soil and bedrock samples for soil classification and to evaluate engineering properties of the subsurface materials
- Evaluated geologic hazards that could potentially impact the site and the proposed residence and associated improvements
- Performed geotechnical analyses to evaluate slope stability and the soil and foundation engineering conditions on the site in the area of the proposed residence and associated improvements
- Prepared this report presenting a summary of our investigation and our engineering geologic and geotechnical conclusions and recommendations

GEOLOGIC & SEISMIC CONDITIONS

Geologic Overview

The property is located along the northeast side of the Santa Cruz Mountains, a northwest-trending range within the California Coast Ranges geomorphic province. The area is characterized by gently to moderately sloping ridge lines with steep to very steep flanks. According to the U.S. Geological Survey topographic map for the area, the property is situated at an elevation of approximately 1,000 feet above mean sea level (see Figure A-1). According to the geologic map of the Cupertino and San Jose West quadrangles (Dibblee, Jr., 2007), the property is located in an area underlain by Cretaceous and Jurassic age (approximately 65 to 206 million years old) greywacke sandstone bedrock of the Franciscan Complex (fs). The sandstone is generally described as greenish gray to buff, fine- to coarse-grained, weathered, hard sandstone with interbeds of siltstone and shale (see Figure A-3, Vicinity Geologic Map).

No landslides are mapped on the site by Dibblee, Jr.; however, Dibblee, Jr. has mapped a relatively large landslide deposit (Qls) immediately northeast and downhill from the property (see Figure A-3). More detailed landslide mapping by Sorg and McLaughlin (1975), suggests that the site is located within the northwestern portion of a large landslide complex that is composed of Franciscan Complex bedrock (Qls(fs)). The landslide complex is approximately 4,000 feet wide and 4,200 feet long and it extends from a ridgeline to the south of the site at an elevation of approximately 1,600 feet down into a tributary to Stevens Creek to the east of the site at an elevation of approximately 600 feet. This feature exhibits a general sense of movement to the northeast into the seasonal tributary. The western half of the property is located within the upper margins of this large landside complex and a scarp is mapped immediately west of the site on the uphill side of Peacock Court. A secondary scarp is located at the top of a very steep slope in the central portion of the property to the east of the proposed building site. According to Sorg and McLaughlin, the landslide identified by Dibblee, Jr. immediately northeast of the site was active in 1973. This landslide is approximately 850 feet wide and 630 feet long with movement to the northeast (see Figure A-4, Vicinity Landslide Map).

According to the State of California seismic hazard zones map of the Cupertino quadrangle (California Geologic Survey, 2002a), the property and most of the surrounding properties are located in an area identified as having a potential for earthquake-induced landsliding (see Figure A-5, State Seismic Hazard Zones Map).

Faulting & Seismicity

Geologists and seismologists recognize the San Francisco Bay Area as one of the most active seismic regions in the United States. There are three major faults that trend in a northwest direction through the Bay Area, which have generated about 12 earthquakes per century large enough to cause significant structural damage. The faults along which these earthquakes occur are part of the San Andreas fault system that extends for at least 700 miles along the California Coast, and includes the San Andreas, Hayward, and Calaveras faults. The San Andreas fault is located approximately 1.8 miles southwest of the site. The Hayward and Calaveras faults are located approximately 16 and 19 miles northeast of the site, respectively. In addition, a trace of the potentially active Berrocal fault is located immediately east of the site (see Figures A-3 & A-4).

Seismologic and geologic experts convened by the United States 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. The Hayward and Calaveras faults have a 14 and 7 percent probability of producing a similar magnitude earthquake during the same time period, respectively (Working Group on California Earthquake Probabilities, 2014).

SITE EXPLORATION AND RECONNAISSANCE

Exploration Program

An initial site visit was performed by our principal engineering geologist on December 19, 2016 as part of our prior investigation. In addition, our prior investigation included engineering geologic reconnaissance and mapping on January 24, 2017 and April 21, 2017; and five exploratory borings were advanced on March 28, 2017. At that time, detailed site development plans were not available. Subsequently, once detailed site development plans were prepared, a site reconnaissance was performed by our principal engineering geologist on June 25, 2019 and additional site mapping was performed by our principal engineering geologist and senior staff engineer on November 15, 2019.

Our subsurface investigation, which was performed on March 28, 2017 and November 27, 2019, included the excavation, sampling, and logging of seven exploratory borings with a track-mounted drill rig equipped with continuous flight augers to depths ranging from 15 to 45 feet at the locations shown on the site plan (see Figure A-2). The boring locations were approximately determined by measuring distance and bearing from known points on the site using a laser range finder, tape measure, and compass and should be considered accurate only to the degree implied by the mapping techniques used.

Soil and bedrock samples were collected with split-spoon samplers that were driven with a 140-pound hydraulic automatic hammer repeatedly dropped from a height of 30 inches. Samplers included 2.5- and 3-inch outside diameter (O.D.) split-spoon samplers and a 2-inch (O.D.) standard penetration test sampler. The sampler types used are indicated on the logs at the appropriate depth. The number of hammer blows required to drive the 18-inch long samplers were recorded in 6-inch increments. 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 blow count data has been adjusted to standard penetration blow counts based on sampler diameter; however, the blow count data has not been adjusted for other factors such as hammer efficiency. The logs of our borings are presented in Appendix B as Figures B-1 through B-7 and a key to the logs is presented on Figure B-8, Key to Boring Logs.

Our staff geologists logged the borings in general accordance with the Unified Soil Classification System presented on Figure B-9 and the Key to Bedrock Descriptions presented on Figure B-10. The boring logs show our interpretation of the subsurface conditions at the locations and on the date 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 and the transitions may be gradual. Soil and bedrock samples recovered from the borings were retained for laboratory testing and for review by our senior staff engineer and principal engineering geologist.

Site Description

The undeveloped, 5.64-acre property is triangular in shape and is located along the east (downhill) side of Peacock Court. The property is 80 feet wide at the road, 544 feet wide at the rear, and is up to 865 feet deep. The property is bounded by Peacock Court to the west and by developed properties to the north, south, and east. The ground surface across the eastern portion of the property slopes down steeply to very steeply to the east and northeast. The ground surface across the western portion of the property slopes down gently to moderately to the south into a seasonal drainage ravine that flows through the southern-most portion of the property. Gradients vary from approximately 6:1 (horizontal to vertical) in the uphill (northern) portion of the west end of the site and gradually steepen to 4:1 and 3:1 down to the top of the drainage ravine. The western-most portion of the drainage ravine is a broad swale. To the east of the swale, the banks of the drainage ravine are very steep with a gradient of approximately 0.8:1 and are approximately 40 feet high (see Figure A-2 and Figures A-6 and A-7, Geologic Cross-Sections A-A' and B-B'). The banks of the taller, steeper portion of the ravine have experienced local, shallow sloughing.

Where Peacock Court crosses the head of the drainage ravine, the road is constructed over fill. The fill slope has a gradient of up to approximately 2:1 (horizontal to vertical) and extends down onto the western portion of the property. The approximate limits of the fill are shown on the site plan (see Figure A-2).

Minor grading has occurred in the uphill portion of the western half of the property in the area of the proposed residence. The topsoil has been scraped off exposing sandstone. In addition, a rough graded dirt road starts in this area and continues to the east along a subdued spur ridge. The dirt road curves to the north, cutting obliquely across the hillside and leads onto the adjacent property to the north.

The western portion of the property is vegetated with seasonal grasses except along the drainage ravine, which is vegetated with dense brush. The northeastern portion of the property is covered with dense trees and associated undergrowth. Drainage across the western half of the site is characterized as uncontrolled sheet flow to the south into the seasonal drainage ravine and drainage in the eastern half is characterized as uncontrolled sheet flow to the east and northeast.

Subsurface Conditions

Seven exploratory borings were excavated on the site to evaluate the subsurface conditions on the slope below the proposed residence and in the area of the proposed residence and associated improvements (see Figure A-2). The borings were all terminated in Franciscan Complex material that we interpret as either intact bedrock or bedrock with an ancient landslide deposit. A general description of the subsurface conditions encountered in each boring is presented below. Detailed descriptions are present on the boring logs in Appendix B.

Boring B-1, located on the hillside to the southeast of the proposed residence, encountered approximately 13.5 feet of stiff to very stiff clayey silt with gravel colluvium overlying 15 feet of stiff clayey silt, and medium dense clayey sand to gravelly sand, which we interpret as old landslide debris. Franciscan Complex shale was encountered below the old landslide debris at a depth of 28.5 feet and persisted to the bottom of the boring at a depth of 45 feet (see Figure B-1).

Boring B-2, located on the hillside to the southwest of the proposed residence, encountered approximately 15 feet of stiff to very stiff clayey silt with gravel colluvium overlying 10 feet of stiff to hard clayey silt, which we interpret as old landslide debris. Franciscan Complex shale was encountered below the old landslide debris at a depth of 25 feet and persisted to the bottom of the boring at a depth of 35 feet (see Figure B-2).

Boring B-3, located along the south side of the proposed residence, encountered Franciscan Complex shale at ground surface. The shale persisted to the bottom of the boring at a depth of 15 feet (see Figure B-3).

Boring B-4, located in the proposed driveway between the proposed residence and the proposed A.D.U., encountered approximately 3.5 feet of colluvium consisting of medium stiff clayey silt with gravel. Franciscan Complex sandstone was encountered below the colluvium at a depth of 3.5 feet and persisted to the bottom of the boring at a depth of 20 feet (see Figure B-4).

Boring B-5, located west of the proposed A.D.U., encountered approximately 2 feet of very soft clayey silt fill overlying 6.5 feet of stiff lean clay colluvium. Franciscan Complex sandstone was encountered below the colluvium at a depth of 8.5 feet and persisted at the bottom of the boring to a depth of 20 feet (see Figure B-5).

Boring B-6, located north of the proposed A.D.U., encountered approximately 13 feet of stiff clayey silt colluvium overlying Franciscan Complex mélange. The mélange persisted to a depth of 36 feet where it is underlain by shale. The shale persisted to the bottom of the boring at a depth of 39 feet where effective sampling refusal was encountered (see Figure B-6).

Boring B-7, located south of the proposed A.D.U., encountered approximately 8.5 feet of medium stiff to very stiff clayey silt colluvium overlying Franciscan mélange. The mélange persisted to a depth of 28.5 feet where it is underlain by sandstone, which persisted to the bottom of the boring at a depth of 29.4 feet where effective sampling refusal was encountered (see Figure B-7).

Based on laboratory testing on a sample of lean clay colluvial soil from Boring B-5, this material is characterized as moderately expansive with a liquid limit of 41 percent and a plasticity index of 27 percent. The results of the test are presented on Figure C-1, Liquid & Plastic Limits Test Report.

Groundwater

Groundwater was encountered in Boring B-1 at a depth of 38 feet below grade at the time of drilling on March 28, 2017. Approximately 2 hours after drilling the groundwater level rose to 36 feet. No free groundwater was encountered in any of the other exploratory borings. We note that fluctuations in the level of groundwater can occur due to variations in rainfall, temperature, landscaping, and other factors that may not have been evident at the time our observations were made.

SLOPE STABILITY ANALYSIS

A seismic slope stability screening analysis was performed in general accordance with the guidelines outlined in the following publications:

- Guidelines for Evaluating and Mitigating Seismic Hazards in California (California Geological Survey, 2008)
- Recommended Procedures for Implementation of DMG Special Publication 117 - Guidelines for Analyzing and Mitigating Landslide Hazards in California (Blake and others, 2002)

The screening analysis included static and pseudo-static evaluations of the stability of the site along Cross-Section A-A' (see Figure A-6), which was deemed the most critical slope condition. The analysis was performed using the computer program Slide 6.0, utilizing the Modified Bishop method to search for the critical circular failure surface and calculate the factor of safety. The critical failure surface is defined as the surface with the lowest calculated factor of safety. In general, factors of safety less than 1.0 indicate a potentially unstable condition, while factors of safety greater than 1.0 indicate a stable condition.

Stratigraphic boundaries utilized for the analysis were derived from our subsurface investigation. Strength data used in the analyses were derived from published mean data for old landslide debris and Franciscan mélange bedrock from the seismic hazard zones report for the Cupertino quadrangle (California Geological Survey, 2002b). The strength values included a phi value of 13.8 degrees and a cohesion value of 532 pounds per square foot (psf) for the colluvium and landslide debris and a phi value of 24 degrees and a cohesion value of 820 psf for the mélange. Based on the subsurface conditions at the site and our experience with similar materials, it is our opinion that these strength values are appropriately conservative. The analyses assumed a groundwater level at a depth of 36 feet below grade based on the exploratory drilling that was performed on March 28, 2017. The exploratory drilling was performed following an above average winter rainy season and we do not anticipate a significantly higher groundwater level.

The pseudo-static analyses utilized a seismic coefficient (*k*) of 0.38, which was determined in accordance with Special Publication 117A for a threshold displacement of 15 centimeters using a site-modified peak ground acceleration with a 2 percent chance of exceedance in 50 years of 1.273g obtained from the U.S. Geological Survey's online seismic design value application tool (U.S. Geological Survey, 2017). In accordance with California Geological Survey Note 48 (California Geological Survey, 2013), the site-modified peak ground acceleration was reduced by a third to remove the risk coefficient.

It should be noted that computer-aided slope stability analyses are mathematical models of slopes and subsurface materials, and they contain many assumptions. Slope stability analyses and the generated factors of safety should only be used to indicate general slope stability trends. In general, factors of safety below 1.00 indicate a potential failure. However, a slope with a factor of safety of less than 1.00 will not necessarily fail but the probability of failure will be greater than that in a slope with a higher factor of safety. Conversely, a slope with a factor of safety greater than 1.00 may fail but the probability of stability is higher than that in a slope with a lower factor of safety.

The static slope stability analysis yielded a critical failure surface up to approximately 35 feet deep extending through the old landslide debris from the base of the seasonal drainage uphill for a distance of approximately 150 feet with a calculated factor of safety of 1.207, suggesting a relatively stable condition. The results of the static slope stability analysis are presented on Figure A-8, Static Slope Stability Analysis along Cross-Section A-A'. The pseudo-static analysis yielded a similar critical failure with a calculated factor of safety of 0.605, suggesting relatively unstable conditions during a design-level earthquake. The results of the pseudo-static slope stability analysis are presented on Figure A-9, Pseudo-Static Slope Stability Analysis Along Cross-Section A-A'.

CONCLUSIONS

Based on our investigation, it is our opinion that the proposed site development is feasible from a geotechnical perspective provided that the recommendations presented in this report are implemented in the design and construction of the project. The primary geotechnical constraints to the proposed improvements are the potential for creep of the colluvial soil blanketing portions of the site, the potential for expansion and contraction of surficial soil blanketing portions of the site, the potential for landsliding, and the potential for very strong to violent ground shaking during a moderate to large earthquake on the San Andreas fault or one of the other nearby active faults.

Based on our subsurface investigation, the residence building site appears to be blanketed by up to approximately 4 feet of colluvial soil and the area of the A.D.U. appears to be blanketed by up to approximately 13 feet of colluvial soil. Where located on or adjacent to moderately steep to steep slopes, the colluvial soil may be prone to creep, the downslope movement of soil under the force of gravity. In addition, based on our laboratory testing, portions of the colluvial soil are moderately expansive and may be prone to expansion and contraction with changes in moisture content. Specifically, when wetted, as during the rainy season, these materials can expand; and when dried, as during the summer months, these materials can contract or shrink. Structures supported on shallow foundations bearing in expansive materials tend to undergo seasonal uplift and settlement. Because of the potential for downhill creep and expansion and contraction, the colluvial soil should not be relied on for the support of proposed structures. In our opinion, creep and expansion of the colluvial soil should not have a significant impact on the structural integrity of proposed improvements provided that the improvements are designed and constructed in accordance with the recommendations presented in this report.

Based on our subsurface investigation, the area of the proposed residence is underlain by sandstone and shale bedrock at relatively shallow depth and the area of the proposed A.D.U. is underlain by mélange bedrock at depths of approximately 8 to 13 feet. These bedrock materials may be part of a large block of relatively intact bedrock incorporated into the large landslide complex mapped by Sorg and McLoughlin. As such, we have denoted these materials as bedrock/ancient landslide deposit (?). A detailed discussion of landsliding is presented below. In our opinion, the sandstone, shale, and mélange should provide adequate support for the foundations of the proposed improvements.

Based on our investigation, in our opinion, local groundwater conditions should not impact the basement design; however, there is a potential for perched groundwater to enter drilled pier excavations and the A.D.U. basement excavation, especially if construction takes place during or immediately following the rainy season. The potential that excavations may encounter perched groundwater and the need for temporary construction dewatering should be taken into consideration by the building contractor.

Geologic Hazards

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

- Landsliding – Based on our review of published geologic maps, the entire property appears to be located within the limits of a large, ancient landslide complex. In general, it is impractical to evaluate the stability of large, ancient landslides when evaluating the development of a single residential property. However, based on our experience in the area, ancient landslides of this nature are generally considered relatively stable and their presence does not generally preclude development.

  Based on our investigation, we did not observe any evidence of active landsliding in the uphill portion of the western half of the property in the area of the proposed improvements. However, based on our investigation, it appears that the hillside below the proposed residence is blanketed by an old landslide deposit. There is no surficial evidence of this feature, but our exploratory borings on the hillside below the residence encountered older landslide debris to a depth of approximately 25 to 30 feet. Based on our slope stability analyses, this feature appears to be stable under static conditions with a factor of safety against landsliding of 1.2. However, based on our analyses, this feature could be potentially unstable during a large earthquake on the nearby San Andreas fault. The pseudo-static analysis yielded a critical failure surface up to approximately 30 feet deep with a factor of safety against landsliding of 0.60. Although this low factor of safety presents a potentially unstable condition, given the relatively gentle slopes across the proposed residence building area and the relatively shallow depth to bedrock/ancient landslide deposit(?), it is our opinion that the risk of significant, deep-seated landsliding through the building site is low. In our opinion, it is unlikely that a significant failure along the seasonal drainage ravine would have a significant impact on future improvements located in the uphill portion of the western half of the property, provided that they are located at least 130 feet from the centerline of the seasonal drainage ravine and are supported on foundations that are designed and constructed in accordance with the recommendations presented in this report.

  It should be clearly understood that landsliding above the ravine is a potential hazard. However, given the location of the proposed residence beyond the anticipated failure envelope, it is our opinion that this presents a reasonable risk. If this risk is deem unacceptable, the potential for future deep-seated landsliding along the seasonal drainage ravine can be substantially mitigated by filling in the ravine to buttress the potentially unstable slope.

Although we did not observe evidence of active landsliding in the proposed building areas, because of the presence of the colluvial soil and the moderate to steep slopes across portions of the property, the occurrence of a new shallow landslide cannot be excluded. A new shallow landslide could be triggered by excessive precipitation or strong ground shaking associated with an earthquake. In our opinion, a new shallow landslide should not pose a significant risk to the structural integrity of the proposed improvements, provided that they are designed and constructed in accordance with the recommendations of this report.

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 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 in the area of the proposed improvements. Therefore, it is our opinion that the potential for surface fault rupture to occur at the building site is 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, very strong to violent ground shaking should be expected at some time during the design-life of the proposed improvements. The improvements should be designed in accordance with current earthquake resistant standards, including the 2019 California Building Code (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.

- Seismic Densification - During moderate and large earthquakes, soft or loose, natural or fill soils can densify and settle, often unevenly across a site. Based on our subsurface exploration, the surficial soil deposits are cohesive in nature and stiff to very stiff. In our opinion, the potential for differential compaction of these materials is low. As discussed above, an area fill is located below Peacock Court and extending onto the western portion of the property. We did not evaluate the condition of this fill and, in our opinion, there is a potential for seismic densification of this material. In our opinion, seismic densification of this material would not constitute a significant hazard to the proposed structures; however, it could temporarily impact access to the site along Peacock Court.

- Liquefaction – Liquefaction is a soil softening response, by which an increase in the excess pore water pressure results in partial to full loss of soil shear strength. In order for liquefaction to occur, the following four factors are required: 1) saturated soil or soil situated below the groundwater table; 2) undrained loading (strong ground shaking), such as by earthquake; 3) contractive soil response during shear loading, which is often the case for a soil which is initially in a loose or uncompacted state; and 4) susceptible soil type; such as clean, uniformly graded sands, non-plastic silts, or gravels. Structures situated above temporarily liquefied soils may sink or tilt, potentially resulting in significant structural damage. Since we did not encounter high groundwater during our subsurface investigation and the area of the proposed improvements is underlain by stiff soil deposits and bedrock/ancient landslide deposits(?), the likelihood of liquefaction occurring and affecting the proposed improvements is negligible.

RECOMMENDATIONS

We recommend that the proposed residence and attached garage, the rear loggia, the chapel, the detached garage, the A.D.U. (including the lower level), and the carport be supported on drilled piers gaining support in the underlying bedrock/ancient landslide deposit(?). Slab floors at interior, habitable spaces should be designed as structural slabs supported on drilled piers and slab floors for the garages may be designed as slabs-on-grade. Site retaining walls should be supported on drilled piers gaining support in the underlying bedrock bedrock/ancient landslide deposit(?); however, site retaining walls supporting cuts into bedrock along the uphill side of the driveway and not structurally tied to building foundations may be supported on either drilled piers or spread footings.

If the proposed swimming pool is located in an area where bedrock/ancient landslide deposits(?) are at or near the surface, the pool may be designed and constructed with a conventional shell. If the pool will be located in an area where bedrock/ancient landslide deposits(?) are not at or near the ground surface, the pool should be supported on drilled piers. The rear loggia slab will be constructed over approximately 5 feet of fill. To mitigate the potential for settlement of the loggia slab, the slab should be designed and constructed as a structural slab supported on drilled piers. Other exterior slabs for patios and walkways may be constructed as slabs-on-grade; however, in areas where minor to moderate slab movement is unacceptable, slabs should be designed and constructed as pier-supported structural slabs.

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.

2019 CBC SEISMIC DESIGN PARAMETERS

We recommend that the design of the project be based on the following updated seismic design parameters. Based on the location of the site at latitude 377.298 and longitude -122.093, 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 – Soil Profile Name: Very Dense Soil and Soft Rock (Table 1613.5.2)
- Mapped Spectral Accelerations for 0.2 second Period: S_S= 2.585 g (Site Class B)
- Mapped Spectral Accelerations for a 1-second Period: S_1= 0.916 g (Site Class B)
- Design Spectral Accelerations for 0.2 second Period: S_{DS}= 2.068 g (Site Class C)
- Design Spectral Accelerations for a 1-second Period: S_{D1}= 0.855 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 (https://seismicmaps.org) using ASCE 7-16 as the design code reference document.

FOUNDATIONS

Drilled Piers

We recommend that all structures be supported on drilled, cast-in-place, reinforced concrete friction piers gaining support in the bedrock/ancient landside deposit(?). Drilled piers should be at least 16 inches in diameter and spaced no closer than approximately three pier-diameters center-to-center.

Drilled piers for the main residence and attached garage, chapel, detached garage, loggia, and adjacent retaining walls should extend at least 10 feet into bedrock/ancient landside deposit (?) or to a depth into the bedrock/ancient landside deposit (?) at least equal to the thickness of non-supportive soil encountered in the upper portion of the pier. If drilled piers are used for retaining walls along the uphill side of the driveway, they should extend at least 6 feet into the bedrock/ancient landside deposit (?). Drilled piers for these improvements should be designed to resist dead plus live loads using an allowable skin friction value of 500 pounds per square foot for the depth of the pier in bedrock with a one-third increase allowed for transient loads, including wind and seismic forces. Any portion of the piers in fill or colluvium, and any point-bearing resistance should be neglected for support of vertical loads.

Drilled piers for the A.D.U. should extend at least 15 feet into the bedrock/ancient landside deposit (?). Drilled piers for the carport and adjacent retaining walls should extend at least 10 feet into the bedrock/ancient landside deposit (?). Drilled piers for the A.D.U., carport and adjacent site retaining walls should be designed to resist dead plus live loads using an allowable skin friction value of 300 pounds per square foot for the depth of the pier in the old landslide debris with a one-third increase allowed for transient loads, including wind and seismic forces. Any portion of the piers in fill or colluvium, and any point-bearing resistance should be neglected for support of vertical loads.

Please note that these are recommended minimum pier dimensions and that other structural criterion, such as the need to resist lateral creep forces may force pier design depths to be greater.

Piers located on or within approximately 10 feet of a slope that is steeper than approximately 5:1 (horizontal to vertical) should be designed to resist active loads from downhill creep of any soil 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 soil. The depth of the active loads will likely vary at individual pier locations. Based on our subsurface investigation, we anticipate active soil depths of between 0 to 5 feet in the area of the residence and up to approximately 9 feet along the downhill side of the A.D.U. To avoid over-design and to facilitate pier construction, we suggest that the project structural engineer develop pier tables for various structures that provide required pier embedment depth into supportive material based on depth of overlying non-supportive material in 2-foot increments from 0 to 6 for the main residence and associated improvements and 0 to 10 for the A.D.U. and associated improvements.

Active loads from soil creep and other lateral loads may be resisted by passive earth pressure based upon an equivalent fluid pressure of 350 pounds per cubic foot, acting on 1.5 times the projected area for the depth of the pier in the supportive material to a maximum value of 4,000 psf. Any passive resistance corresponding to the creep zone described above should be neglected.

Pier reinforcing should be determined by the project structural engineer based on the preceding design criteria and structural requirements.

The bottoms of the pier excavations should be substantially free of 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. Murray Engineers, Inc. should observe the pier excavations to evaluate depth to supportive material and whether the pier excavations are properly prepared. The pier depths recommended above may require adjustment, if differing conditions are encountered during excavation. Pier excavations should be filled with concrete as soon as practical after drilling to minimize the potential for caving.

Where expansive soil is exposed at pad grade, we recommend that the upper 2 to 3 feet of the piers be formed with Sonotubes to prevent "mushrooming" of the concrete. Sonotubes should fit snugly within the pier excavations and should extend 4 inches above bottom of grade beam excavations to account for the placement of a void form at the bottom of the grade beam (see below).

Grade beams should be incorporated between piers as required by the structural engineer. Perimeter grade beams for the proposed structures should extend at least 6 inches below the crawlspace grade or bottom of slab subgrade to reduce the potential for infiltration of surface runoff under the slabs. To mitigate uplift from the moderately to highly expansive surficial soil, we recommend that grade beams for the proposed structures that are excavated into expansive soil be formed over 4-inch thick cardboard void forms, such as manufactured by SureVoid.

Grade beam 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 residence.

Spread Footings

If spread footings are used for the retaining wall along the uphill side of the driveway or the loggia, they should have a minimum width of 15 inches and they should extend at least 18 inches below lowest final adjacent exterior grade and be embedded at least 12 inches into the bedrock/ancient landside deposit (?), whichever is deeper. In addition, footings located adjacent to utility trenches should bear below a 1:1 plane extended upward from the bottom edge of the utility trench.

Spread footings supported in the bedrock/ancient landside deposit (?) may be designed for an allowable bearing pressure of 2,000 pounds per square foot for dead plus live loads with a one-third increase allowed for total loads including wind and seismic forces. The weight of the footings may be neglected for design purposes.

Lateral loads may be resisted by friction between the footings and the supporting subgrade using a coefficient of friction of 0.3. In addition to the above, lateral resistance may be provided by passive pressures acting against foundations poured neat in the footing excavations within the bedrock/ancient landside deposit (?) using an equivalent fluid pressure of 350 pounds per cubic foot.

Footing 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 all 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 see that 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 1-inch across any 20-foot span of the footing-supported improvements.

BASEMENT & SITE RETAINING WALLS

Basement and site retaining walls should be supported on foundations designed in accordance with the recommendations provided above. Waterproofing or damp-proofing of retaining walls should be included in areas where wall moisture would be undesirable, such as at living space or where wall finishes could be impacted by moisture. The project architect or a waterproofing consultant should provide detailed recommendations for waterproofing or damp proofing, as necessary. The A.D.U. basement wall waterproofing system should be integral with the A.D.U. basement structural slab waterproofing.

Lateral Earth Pressures

Basement and site retaining walls should be designed to resist lateral earth pressure from the adjoining natural soils, backfill, and any anticipated surcharge loads. Assuming that the backfill behind the wall will be level (e.g., not sloping upward) and that adequate drainage will be incorporated as recommended below, we recommend that unrestrained retaining walls be designed to resist an equivalent fluid pressure of 45 pounds per cubic foot (pcf) plus one-third of any anticipated surcharge loads. Walls restrained from movement at the top should be designed to resist an equivalent fluid pressure of 45 pcf plus a uniform pressure of 8H pounds per square foot (psf), where H is the height in feet of the retained soil. Restrained walls should also be designed to resist an additional uniform pressure equal to one-half of any surcharge loads applied at the surface.

Where backfill behind the wall will be sloping upward from the wall, we recommend that the equivalent fluid pressures provided above be increased by 3 pcf for each 4-degree increase in slope inclination.

In accordance with the 2019 CBC, where applicable, retaining walls should also be designed to resist lateral earth pressure from seismic loading. We recommend that the seismic loading be based on a uniform pressure of 10H pounds per square foot (psf)/foot of wall height, where H is the height in feet of the retained soil. In our opinion, site retaining walls less than 6 feet high do not need to be designed for seismic loading. The allowable passive pressures provided for retaining wall foundations may be increased by one-third for short-term seismic forces.

Retaining Wall Drainage

We recommend that retaining walls include a subsurface drainage system to mitigate the buildup of water pressure from surface water infiltration and other possible sources of water. As noted above, the A.D.U. basement wall drainage system should be integral with the A.D.U. basement structural slab drainage system, as discussed below.

Retaining wall backdrains should consist of a minimum 4-inch diameter, perforated rigid pipe, Schedule 40 or SDR 35 (or equivalent) with the perforations facing down, resting on about a 2- to 3-inch thick layer of crushed rock. The perforated pipe should be placed within a minimum 8-inch deep by 12-inch wide trench excavated below basement subgrade elevation at the perimeter of the basement walls. Subdrain pipes should be bedded and backfilled with ½- to ¾-inch clean crushed rock separated from the native soil with a geotextile filter fabric, such as TC Mirafi 140N or equivalent. The crushed rock backfill should extend vertically to within approximately 18 inches of the finished grade and laterally at least approximately 12 inches from the rear face of the wall. The crushed rock should be compacted with a jumping jack or vibratory plate compactor in lifts not exceeding roughly

12 inches in loose thickness. The upper roughly 18 inches of backfill should consist of native soil, which should be compacted in accordance with the Compaction section of this report to mitigate infiltration of surface water into the subdrain systems. The preceding recommendations are presented schematically on Figure A-10, Basement Subdrain System Alternative A.

As an alternative to crushed rock, Miradrain, Enkadrain, or other geosynthetic drainage panels approved by this office may be used for retaining wall drainage. If used, the drainage panels should extend from a depth of approximately 18 inches below finish grade to the base of the retaining wall. An approximate 2-foot section of crushed rock wrapped in filter fabric should be placed around the drainpipe, as discussed previously. Geosynthetic drainage panels should be installed in strict compliance with manufacturer's recommendations with filter fabric against the crushed rock and soil backfill. The preceding recommendations are presented schematically on Figure A-11, Basement Subdrain System Alternative B.

Subdrain pipes should be sloped at a minimum approximately 1.5 percent and should be connected to rigid, solid (non-perforated) discharge pipes to convey any collected water to a suitable discharge location downslope from walls. The subdrain pipes should be provided with cleanout risers at their up-gradient ends and at most sharp directional changes to facilitate maintenance. All surface drainage pipes, including those connected to downspouts and area drains should be kept completely separate from the retaining wall drainage systems. Clean-out risers should be terminated below grade in a Christy box and should be clearly marked as subdrains to reduce the risk that cleanout pipes might be inadvertently used as discharge pipes for surface drains or downspout.

Retaining Wall Backfill

Backfill placed behind the walls should be compacted in accordance with the specifications outlined in Table 1 of the Compaction section of this report using light compaction equipment. If heavy compaction equipment is used, the walls should be temporarily braced. Please refer also to the Earthwork section of this report for important recommendations regarding wall backfill.

SWIMMING POOL

If the proposed swimming pool is located in an area where bedrock/ancient landslide deposits (?) are at or near the surface, the pool may be designed and constructed with a conventional shell. If the pool will be located in an area where bedrock/ancient landslide deposits (?) are not at or near the ground surface, the pool should be supported on drilled piers. We should review the proposed swimming pool location prior to construction and verify in writing the appropriate recommendations.

Conventional Pool Shell

If the proposed swimming pool is located in an area where bedrock/ancient landslide deposits (?) are at or near the surface, the swimming pool may be designed and constructed as a conventional shell bearing on the underlying competent bedrock/ancient landslide deposits (?).

The swimming pool walls should be designed to resist a lateral earth equivalent fluid pressure of 65 pounds per cubic foot plus an additional uniform pressure equivalent to one-half of any surcharge loads applied at the surface. Any portion of the pool walls above ground should be designed as free-standing walls.

We recommend that one or more pressure relief valves be placed in the bottom of the deepest portion of the pool to limit potential damage from hydrostatic (buoyant) pressure, a condition that could result if the pool were empty and the water level outside of the pool were temporarily high. At least four inches of clean ½- to ¾-inch crushed rock should be placed beneath the pool shell to allow water to flow to the pressure relief valve(s). Filter fabric, such as Mirafi 140N, should be placed on the pool subgrade prior to placement of the crushed rock.

Our representative should observe the pool excavation prior to placing reinforcing steel assess whether it is founded in competent bearing material. Any loose soil that falls into the pool excavation during placement of reinforcing steel should be removed prior to placing concrete. In addition, we should observe the installation of the crushed rock prior to placement of reinforcing steel and gunite.

Pier-Supported Swimming Pool

If the pool will be located in an area where bedrock/ancient landslide deposits (?) are not at or near the ground surface, the pool should be supported on drilled piers designed in accordance with the recommendations provided above for support of the residence. If the pool location is shifted closer to the A.D.U., the pier design should be based on the pier recommendations provided for support of the A.D.U. The bottom of the pool shell should be constructed as a structural slab spanning between drilled piers. If expansive soil is exposed at the bottom of the pool excavation, we recommend that such structural slab and any associated grade beams be underlain by 4-inch thick void forms to limit uplift on the slabs however, if non-expansive material is exposed across the base of the entire pool excavation, void forms will not be needed.

The pool side walls should be designed to resist a lateral earth equivalent fluid pressure of 75 pounds per cubic foot plus an additional uniform pressure equivalent to one-half of any surcharge loads applied at the surface. Any portion of the pool walls above ground should be designed as free-standing walls.

We should observe the pier excavations to evaluate depth to supportive material and whether the pier excavations are properly prepared. The pier depths recommended above may require adjustment, if differing conditions are encountered during excavation. Pier excavations should be filled with concrete as soon as practical after drilling to minimize the potential for caving.

CONCRETE SLABS

We anticipate that concrete slabs may be used for the interior floors of the proposed residence and attached garage, chapel, detached garage, and A.D.U. In addition, we anticipate that concrete slabs may be used for exterior hardscapes, such as the loggia, driveway, and patios and walkways. We recommend that slab floors at interior, habitable spaces and the loggia be designed as structural slabs supported on drilled piers. Slab floors for the garages may be designed as structural slabs or slabs-on-grade. In addition, exterior slabs may be constructed as structural slabs or slabs-on-grade. As noted above, in areas where minor slab movement is unacceptable, we recommend utilizing structural slabs.

It should be anticipated that some degree of differential slab-on-grade movement and cracking could occur due differential movement related to heave, settlement and/or soil creep. If slight slab movement and cracking is unacceptable, then we suggest that critical hardscape features be designed and constructed as structural slabs supported on drilled piers. In our opinion, the use of structural slabs would best serve to reduce cracking of slab surfaces. Detailed recommendations are presented in the following sections of the report.

Structural Slabs

Structural slabs should be supported on drilled piers designed in accordance with the recommendations provided above for support of the proposed structures. If expansive soils are encountered at subgrade level, the slabs should be underlain by 4-inch thick cardboard void forms to mitigate excessive uplift forces from expansive soil against the bottom of the slab. If a damp proofing system is used beneath interior structural slabs, the void form may be used to serve as a capillary break between the underlying subgrade and the slabs.

The project structural engineer should determine structural slab thickness and reinforcement based on anticipated loading and the pier design criteria presented above.

In areas where dampness from soil moisture vapors is a concern, we recommend that a heavy-duty damp-proofing or waterproofing membrane be placed over the void form to limit slab dampness from soil moisture vapors. In particular, we suggest the use of an integrally bonded membrane, such as Preprufe or Florprufe™ (Grace Construction Products), which will remain in direct contact with the slab when the cardboard void deteriorates. For the below-grade slab floor of the A.D.U., we recommend using a waterproofing membrane and we recommend that the void form be underlain by 4 inches of free-draining gravel. The project architect or a waterproofing consultant should provide project-specific waterproofing or damp-proofing design and details.

Slabs-on-Grade

If concrete slabs-on-grade are used for the proposed garages, driveway, and parking areas, they should be underlain by at least 12 inches of Class 2 aggregate baserock. Exterior slabs-on-grade for the loggia, patios, and walkways should be underlain by at least 8 inches of Class 2 aggregate baserock. Prior to placement of the baserock, the subgrade soils should be scarified to a depth of approximately 6 inches, moisture conditioned to near the materials optimum moisture content, and compacted in accordance with the Compaction section of this report. Where highly expansive soil is exposed at subgrade, the subgrade should be scarified to a depth of approximately 6 to 10 inches, moisture conditioned to approximately 3 to 5 percent over optimum moisture content, and compacted to approximately 90 percent relative compaction. Over-compaction of highly expansive subgrade soil should be avoided. Where existing fill is present within areas of new pavement, portions or all of the fill should be removed and replaced as a properly engineered fill as deemed necessary by our field representative during construction. These recommendations are intended to mitigate the potential for significant slab movement and distress; however, they will not eliminate the potential for minor slab movement and distress.

In general, slabs-on-grade for the garages, patios, and walkways should be designed as "free-floating" slabs, structurally isolated from adjacent foundations. We recommend that exterior slabs be provided with control joints at spacing of not more than about 10 feet. The project civil or structural engineer should determine slab-on-grade thickness and reinforcement based on anticipated use and loading.

Where slab surface moisture would be a significant concern, such as for the garage floors, we recommend that the slabs be underlain by a vapor retarder consisting of a highly durable membrane not less than 10 mils thick (such as Stego Wrap Vapor Barrier by Stego Industries, LLC or equivalent), underlain by a capillary break consisting of 4 inches of ½- to ¾-inch crushed rock. The capillary break may be considered the equivalent thickness as the upper 4 inches of baserock recommended above. 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.

Waterproofing Membrane & Vapor Retarder Considerations

Based on our understanding, two opposing schools of thought currently prevail concerning protection of the waterproofing membrane or vapor retarder during construction. Some believe that 2 inches of sand should be placed above the membrane or 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 membrane or vapor retarder and curing of the concrete are not as critical design considerations when compared to the possibility of entrapment of moisture in the sand above the membrane or 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 membrane or vapor retarder membrane (at least 10 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 the project structural engineer, architect, and/or waterproofing consultant be consulted for further guidance on this matter.

FLEXIBLE PAVEMENTS

Asphaltic Concrete

We anticipate that asphaltic concrete pavement may possibly be used for the driveway. At a minimum, we recommend that the proposed asphalt surface be at least 2.5 inches thick and that it be underlain by at least 12 inches of imported Class 2 aggregate baserock (R-value of 78). The baserock should extend at least 2 feet beyond the edge of the pavement to reduce the potential of cracking along the edge of the pavement. We note that the placement of the above thickness of baserock beneath proposed AC pavements will in our opinion mitigate but not eliminate the potential for differential movement and cracking of these pavements. Where existing fill is present within areas of new pavement, portions or all of the fill should be removed and replaced as a properly engineered fill as deemed necessary by our field representative during construction. Prior to placement of the baserock, the subgrade soils should be scarified to a depth of approximately 6 inches, moisture conditioned to near the materials optimum moisture content, and compacted in accordance with the Compaction section of this report. If highly expansive soil or soft subgrade conditions are encountered at subgrade elevation along the driveway, it may be advisable to increase the thickness of the baserock. In addition, if highly expansive subgrade soils are encountered, the subgrade should be scarified to a depth of approximately 6 to 10 inches, moisture conditioned to approximately 3 to 5 percent over optimum moisture content, and compacted to approximately 90 percent relative compaction. Over-compaction of expansive subgrade soil should be avoided. In our opinion, these recommendations should mitigate the potential for significant pavement distress, but will not eliminate the potential for minor pavement distress.

Sand-Set Pavers

We anticipate that sand-set pavers or flagstones may be used for exterior flatwork. We recommend that pavers for the driveway and parking areas be underlain by at least 12 inches of compacted Class 2 aggregate baserock and pavers for patios and walkways should be underlain by at least 8 inches of compacted Class 2 aggregate baserock. Prior to placement of the baserock, the subgrade soils should be scarified to a depth of approximately 4 to 6 inches, moisture conditioned to near the material's optimum moisture content, and compacted in accordance with the Compaction section of this report. In addition, if highly expansive subgrade soils are encountered, the subgrade should be scarified to a depth of approximately 6 to 12 inches, moisture conditioned to approximately 3 to 5 percent over optimum moisture content, and compacted to approximately 90 percent relative compaction. In our opinion, these recommendations should mitigate the potential for significant heave and settlement of pavers, but will not eliminate the potential for minor movement of the pavers.

EARTHWORK

A moderate amount of earthwork is anticipated as part of the proposed construction, including grading to construct the driveway and building pads, the A.D.U. basement excavation, backfill behind retaining walls, subgrade preparation and baserock compaction beneath flatwork, and backfill of utility trenches. The earthwork should be performed in accordance with the following recommendations.

Clearing & Site Preparation

Initially, the proposed improvement areas should be cleared of obstructions. Holes or depressions resulting from the removal of underground obstructions below proposed subgrade levels should be backfilled with engineered fill, placed and compacted in accordance with the recommendations provided 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 contingent upon review by our firm. 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 material with a plasticity index of less than 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 planned 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 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*
General fill for raising of site grades, driveway, patio areas, and retaining wall backfill (fills up to 4 feet thick)	90 percent	Near optimum
For fills greater than 4 feet thick (i.e. basement retaining wall backfill or fill slope grading)	93 percent	Near optimum
Upper 6 inches of relatively non-expansive subgrade beneath flatwork	90 percent	Near optimum
Upper 6 to 12 inches of expansive subgrade beneath hardscape	87 to 90 percent	>3% over optimum
Class 2 aggregate baserock beneath flatwork	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 soil	90 percent	Near optimum

*Relative to ASTM D 1557, latest edition.

Keying & Benching

Unretained fill placed on slopes that are flatter than 5:1 should be supported on level benches bearing in supportive soil, as determined by this office in the field during construction. Unretained fill placed on slopes that are steeper than 5:1 should be keyed and benched into supportive soil to provide a firm, stable surface on which to support the fill. Keying and benching should be performed in general accordance with the attached Figure A-12, Schematic Fill Slope Detail.

Prior to fill placement on slopes steeper than 5:1, a construction keyway should be excavated at the toe of the fill. The keyway should be a minimum of 8 feet wide or of a width equal to half the height of the fill slope, whichever is greater. The keyway should be excavated a minimum of 2 feet into bedrock or competent supportive material, as measured on the downhill side of the excavation. The depth to competent supportive material should be determined by this office in the field during construction. The base of the keyway excavation should have a nominal slope of approximately 2 percent dipping toward the back (uphill side) of the key. Subsequent construction benches should be excavated to remove any non-supportive surficial soil and should also have a nominal slope of approximately 2 percent dipping in the uphill direction. Our representative should observe the completed keyway and bench excavations to confirm that they are founded in materials with sufficient supporting capacity.

Fill Subdrainage

In general, fills exceeding approximately 5 feet in depth may need the placement of subdrainage as deemed necessary in the field by our firm's representative. Subdrains should consist of a 4-inch diameter, rigid, heavy-duty, perforated pipe (Schedule 40, SDR 35 or equivalent), approved by the soil engineer, embedded in ½- to ¾-inch clean crushed rock placed along the upslope side of keyways and benches for the full height of the keyway or bench cut. The crushed rock should be separated from the fill and the native material by a geotextile filter fabric. The perforated subdrain pipe should be placed with the perforations down on a 2- to 3-inch bed of drain rock. Subdrain pipes should be provided with clean-out risers at suitable locations. Subdrain systems should be provided with a minimum 1 percent gradient and should discharge onto an energy dissipater at an appropriate downhill location.

Final Slopes

In general, any proposed cut slopes in the surficial soil and any proposed fill slopes should have gradients no steeper than approximately 2:1 (horizontal to vertical). In general, new fill slopes should be over-filled and then cut back to proposed final slope gradients. All graded surfaces or areas disturbed by construction should be revegetated prior to the onset of the rainy season following construction to mitigate excessive soil erosion. If vegetation is not established, other erosion control provision should be employed. Ground cover, once established should be properly maintained to provide long-term erosion control.

Temporary Slopes & Trench Excavations

The contractor should be responsible for all temporary excavations, slopes and trenches excavated at the site, including dewatering and design and construction of any required safety cuts or shoring. Safety cuts and shoring should be provided in accordance with all applicable local, state, and federal 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 drainage is critical for the development of hillside properties. Roof run-off, rain, and irrigation water should not be allowed to pond near the residence, accessory structures, or on exterior hardscape. The proposed buildings should be provided with roof gutters and downspouts. Downspout drainage should preferably be collected in closed pipe systems and routed to a suitable discharge outlet, although splash blocks are also acceptable from a geotechnical perspective provided that the discharge will not create ponding or excessive erosion. The finished grades around the structures should be designed to drain surface water away from the proposed buildings, slabs, and yard areas to suitable discharge points. 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 residence.

Surface runoff should be prevented from flowing over the top of any artificial slope. 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. In addition, retaining walls at the bases of descending slopes should be provided with lined drainage swales along their uphill side to collect surface water from above. All collected water should be conveyed away from the development area by buried closed conduit and discharged onto an energy dissipater at an appropriate downslope location, approved by this office. Drainage systems that saturate the surficial soil or discharge water onto the steeper slopes in the southern portion of the property should be avoided.

We recommend that annual maintenance of the surface drainage systems be performed. This maintenance should include inspection and testing to make sure that roof gutters and downspouts are in good working order and do not leak; 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 recommendations, if needed.

REQUIRED FUTURE SERVICES

Plan Review

To better note conformance of the final design documents with the recommendations contained in this report, and to better comply with the County's 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 architectural, structural, and civil plans:

- The geotechnical aspects of the construction, including basement excavation, pier drilling, footing excavations, retaining wall backdrains and backfill, subgrade preparation beneath hardscape, placement and compaction of engineered fill, and site drainage, should be performed in accordance with the recommendations April 23, 2020. Murray Engineers, Inc. should be provided at least 48 hours advance notification (650-559-9980) of any geotechnical aspects of the construction and should be present to observe and test the earthwork, foundation, and drainage installation phases of the project.

Construction Observation Services

Murray Engineers, Inc. should observe and test 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 encountered during the course of the construction, it may be necessary to re-evaluate the preceding recommendations.

LIMITATIONS

This report has been prepared for the sole use of Melissa and Jeff Waters, specifically for developing geotechnical design criteria and recommendations for the new residence and associated improvements, as discussed above, on the property, APN 351-42-004, on Peacock Court in unincorporated Santa Clara County, California. The opinions presented in this report are based upon our site reconnaissance, information obtained from borings at widely separated locations, review of field data made available to us, and upon local experience and engineering judgment. We are not responsible for the accuracy of data provided by others. The conclusions and recommendations presented in this report 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. The 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 understood that geotechnical conditions may become apparent during the course of construction that were not apparent at the time our investigation was performed. No warranty, expressed or implied, is made or should be inferred.

The recommendations presented in this report are based on the assumption that we will be retained to provide the Future Services described above 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 geotechnical consultant 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. In addition, this report should not be used and is not applicable for any property other than that evaluated.

REFERENCES

ASTM International, 2012, *Annual Book of ASTM Standards*, 2012, Section Four, Construction, Volume 04.08, Soil and Rock (I): D420-D5876: ASTM International, West Conshohocken, PA, 1809 p.

Blake, T.F., R.A. Hollingsworth, and J.P. Stewart, 2002, *Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Landslide Hazards in California*. Los Angeles, Calif.: Southern California Earthquake Center, University of Southern California.

California Geological Survey, 2013, *Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Building*: California Geological Survey - Note 48: California Geological Survey.

California Geological Survey, 2008, *Guidelines for evaluating and mitigating seismic hazards in California*: Special Publication 117A, California Geological Survey.

California Geological Survey, 2002a, *Seismic hazard zone report for the Cupertino 7.5-minute quadrangle, Santa Clara County*: Seismic Hazard Zone Report 068, California Geological Survey.

California Geological Survey, 2002b, *State of California, Seismic Hazard Zones, Cupertino Quadrangle*, Official Map, Released: September 23, 2002: California Geological Survey.

Dibble, T.W., Jr., 2007, *Geologic Map of the Cupertino and San Jose West Quadrangle, Santa Clara and Santa Cruz Counties*: J.A. Minch, ed.., California, Dibblee Geology Center Map DF-351, Santa Barbara Museum of Natural History, Santa Barbara, California, used with permission.

Murray Engineers, Inc., 2017, *Limited Geologic & Geotechnical Investigation, Site Development Feasibility, APN 351-42-004 – Peacock Court, Santa Clara County, California*: consultant's report prepared for Christine and Alan Loudermilk.

Sorg, D.H. & McLaughlin, R.J , 1975, *Geologic Map of Sargent-Berrocal Fault Zone Between Los Gatos & Los Altos Hills, Santa Clara County, California*: U.S. Geological Survey Miscellaneous Field Studies Map MF-643.

Structural Engineers Association of California and California's Office of Statewide Health Planning and Development, 2019, *Seismic Design Maps*, https://seismicmaps.org/, accessed April 2, 2020.

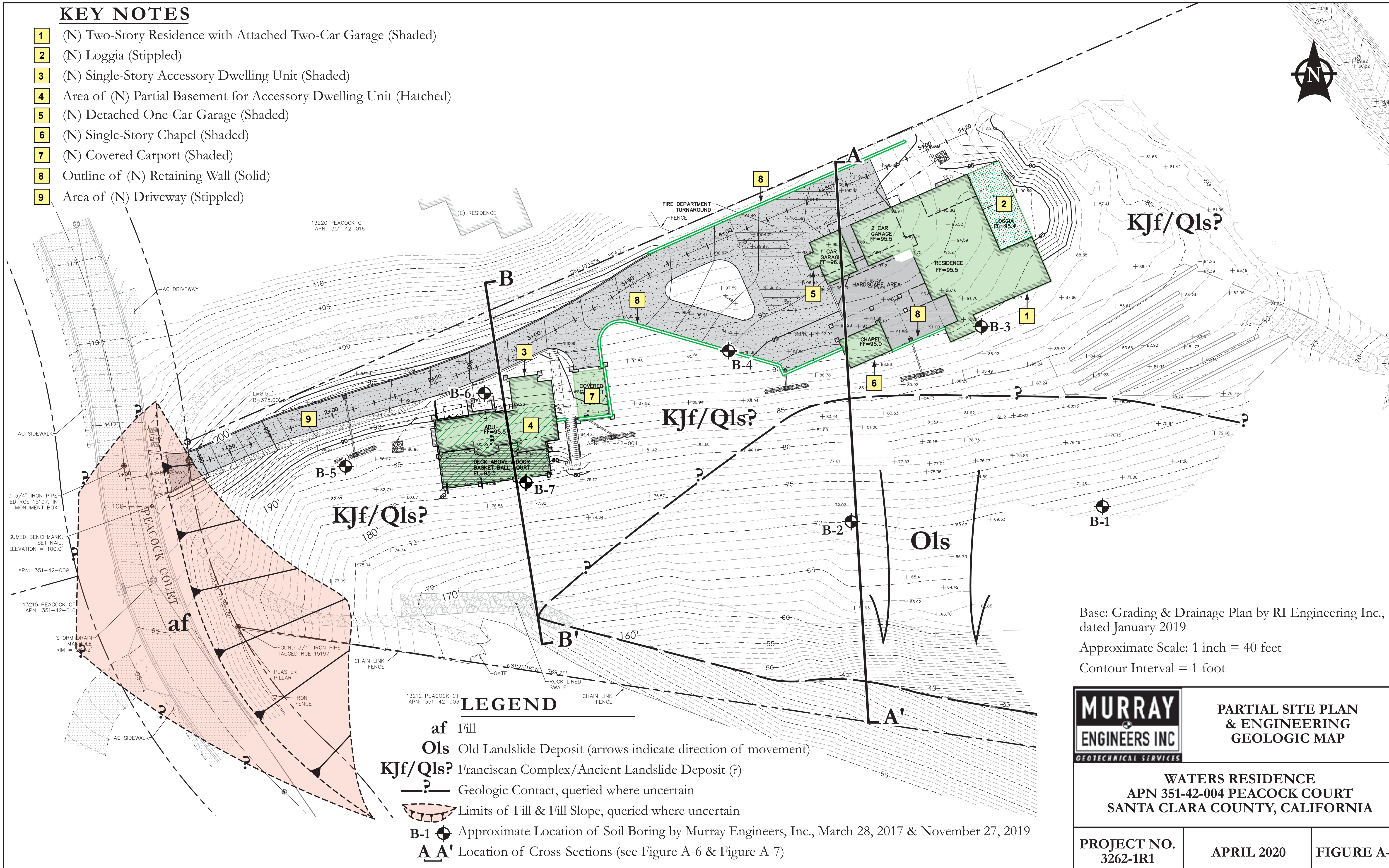
Working Group on California Earthquake Probabilities 2014, *The Uniform California Earthquake Rupture Forecast, Version 3 (UCERF 3)*: U.S. Geological Survey Open-File Report 2013-1165; California Geological Survey Special Report 228; Southern California Earthquake Center Contribution 1792.

Base: USGS Topographic Map, Cupertino Quadrangle, 7.5-Minute Series, 2015 Scale: 1 inch = 2,000 feet

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	WATERS RESIDENCE APN 351-42-004 PEACOCK COURT SANTA CLARA COUNTY, CALIFORNIA		VICINITY MAP
	PROJECT NO. 3262-1R1	APRIL 2020	FIGURE A-1

KEY NOTES

1. (N) Two-Story Residence with Attached Two-Car Garage (Shaded)
2. (N) Loggia (Stippled)
3. (N) Single-Story Accessory Dwelling Unit (Shaded)
4. Area of (N) Partial Basement for Accessory Dwelling Unit (Hatched)
5. (N) Detached One-Car Garage (Shaded)
6. (N) Single-Story Chapel (Shaded)
7. (N) Covered Carport (Shaded)
8. Outline of (N) Retaining Wall (Solid)
9. Area of (N) Driveway (Stippled)



Base: Grading & Drainage Plan by RI Engineering Inc., dated January 2019

Approximate Scale: 1 inch = 40 feet

Contour Interval = 1 foot

LEGEND

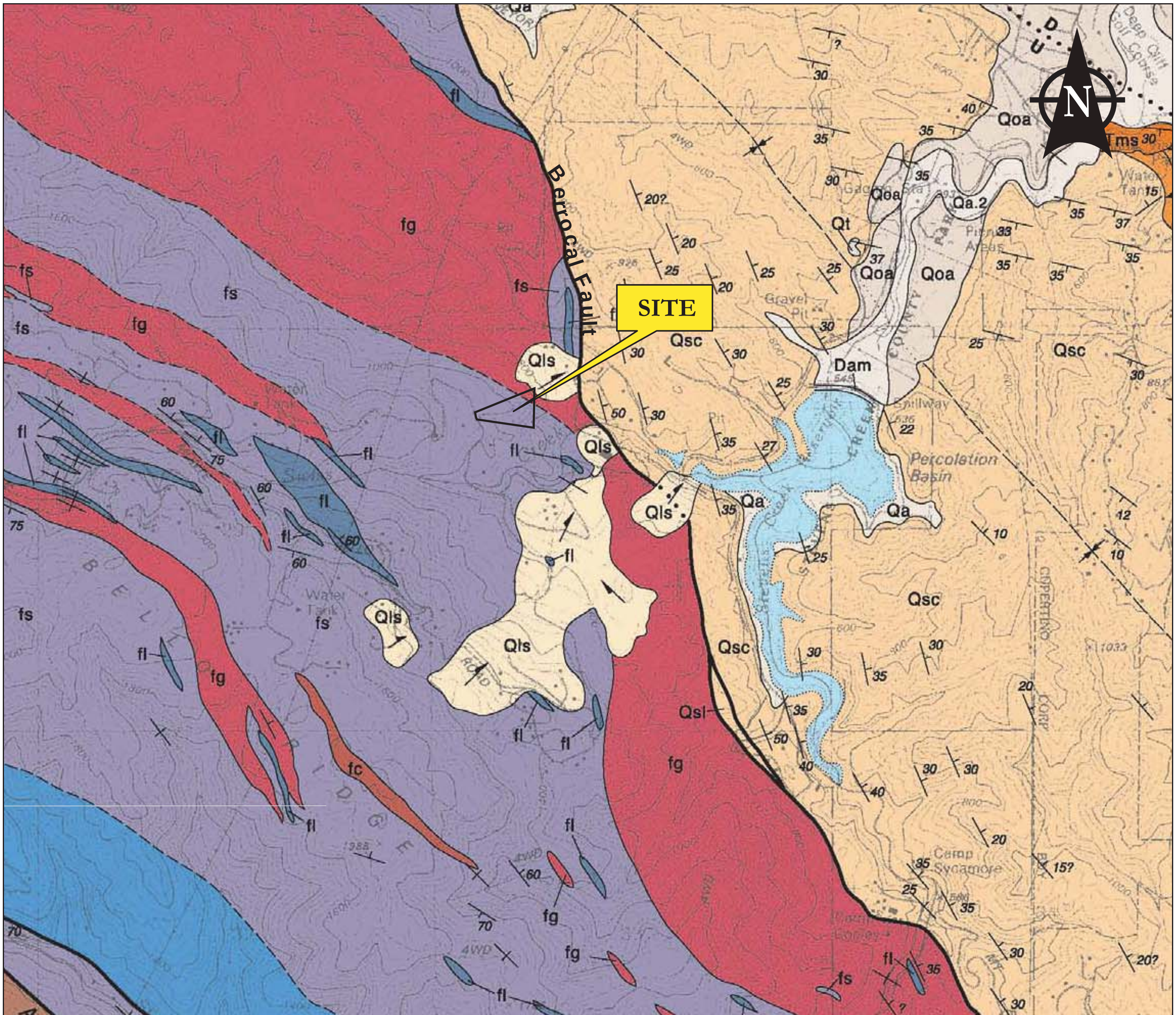
- **af** Fill
- **Qls** Old Landslide Deposit (arrows indicate direction of movement)
- **KJf/Qls?** Franciscan Complex/Ancient Landslide Deposit (?)
- Geologic Contact, queried where uncertain
- Limits of Fill & Fill Slope, queried where uncertain
- **B-1** Approximate Location of Soil Boring by Murray Engineers, Inc., March 28, 2017 & November 27, 2019
- **A A'** Location of Cross-Sections (see Figure A-6 & Figure A-7)

MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

PARTIAL SITE PLAN & ENGINEERING GEOLOGIC MAP

WATERS RESIDENCE
APN 351-42-004 PEACOCK COURT
SANTA CLARA COUNTY, CALIFORNIA

PROJECT NO. 3262-1R1	APRIL 2020	FIGURE A-2



Legend & Selected Map Symbols

Qls	Landslide Debris
fs	Franciscan Graywacke Sandstone
fg	Franciscan Greenstone

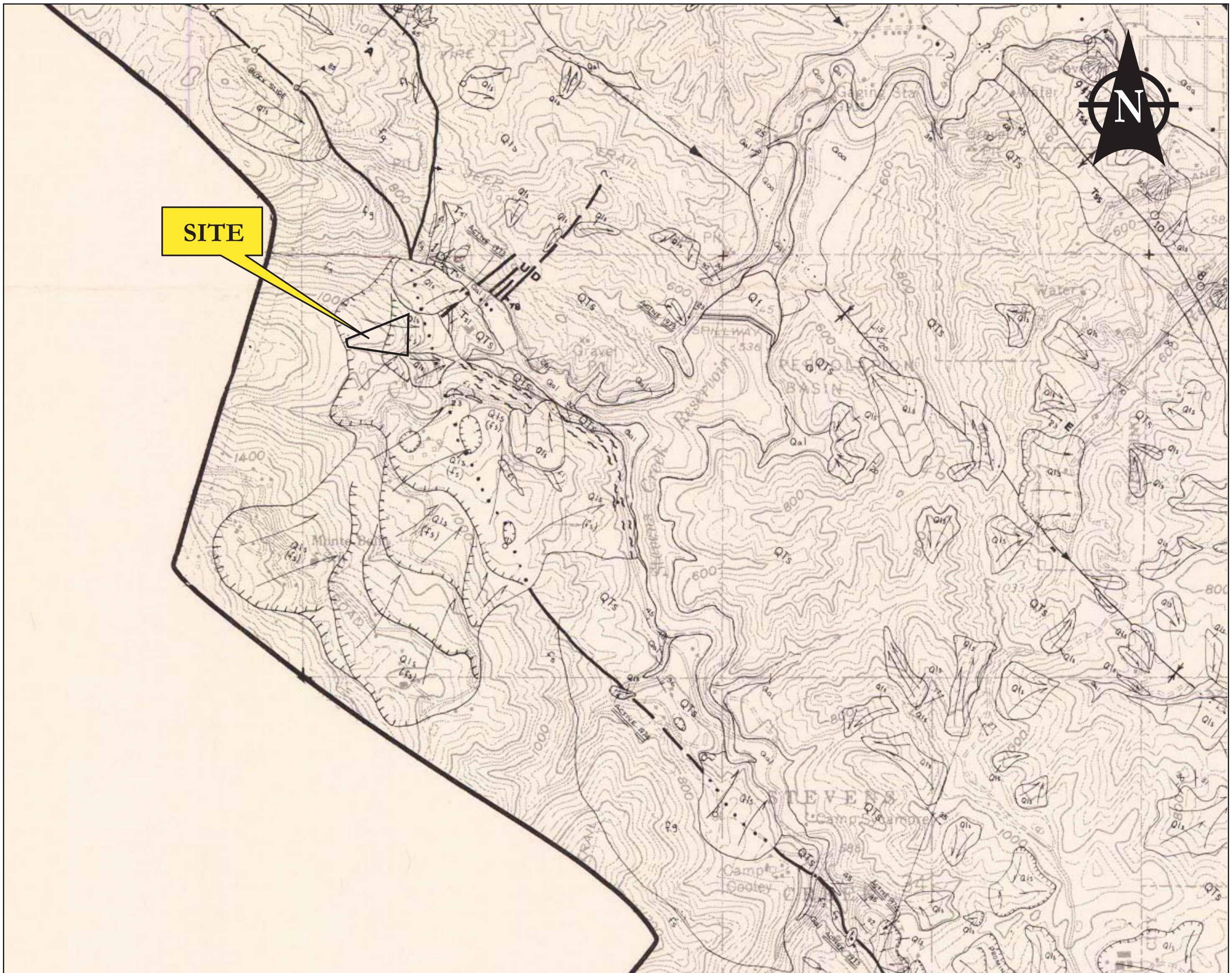
Geologic contact, dashed where approximate

? Fault, dashed where approximately located, dotted where concealed, queried where uncertain

35 Strike and dip of bedding

Base: Geologic Map of the Cupertino and San Jose West Quadrangles, Santa Clara and Santa Cruz County, California, by Thomas W. Dibblee, Jr., 2007, used with permission Scale: 1 inch = 2,000 feet

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	WATERS RESIDENCE APN 351-42-004 PEACOCK COURT SANTA CLARA COUNTY, CALIFORNIA	VICINITY GEOLOGIC MAP
	PROJECT NO. 3262-1R1 — APRIL 2020	FIGURE A-3



Legend

Qls	Landslide Deposit
fs	Franciscan Sandstone
fg	Franciscan Greenstone
	Closed natural depression
	Gouge or shear zone

60 D U

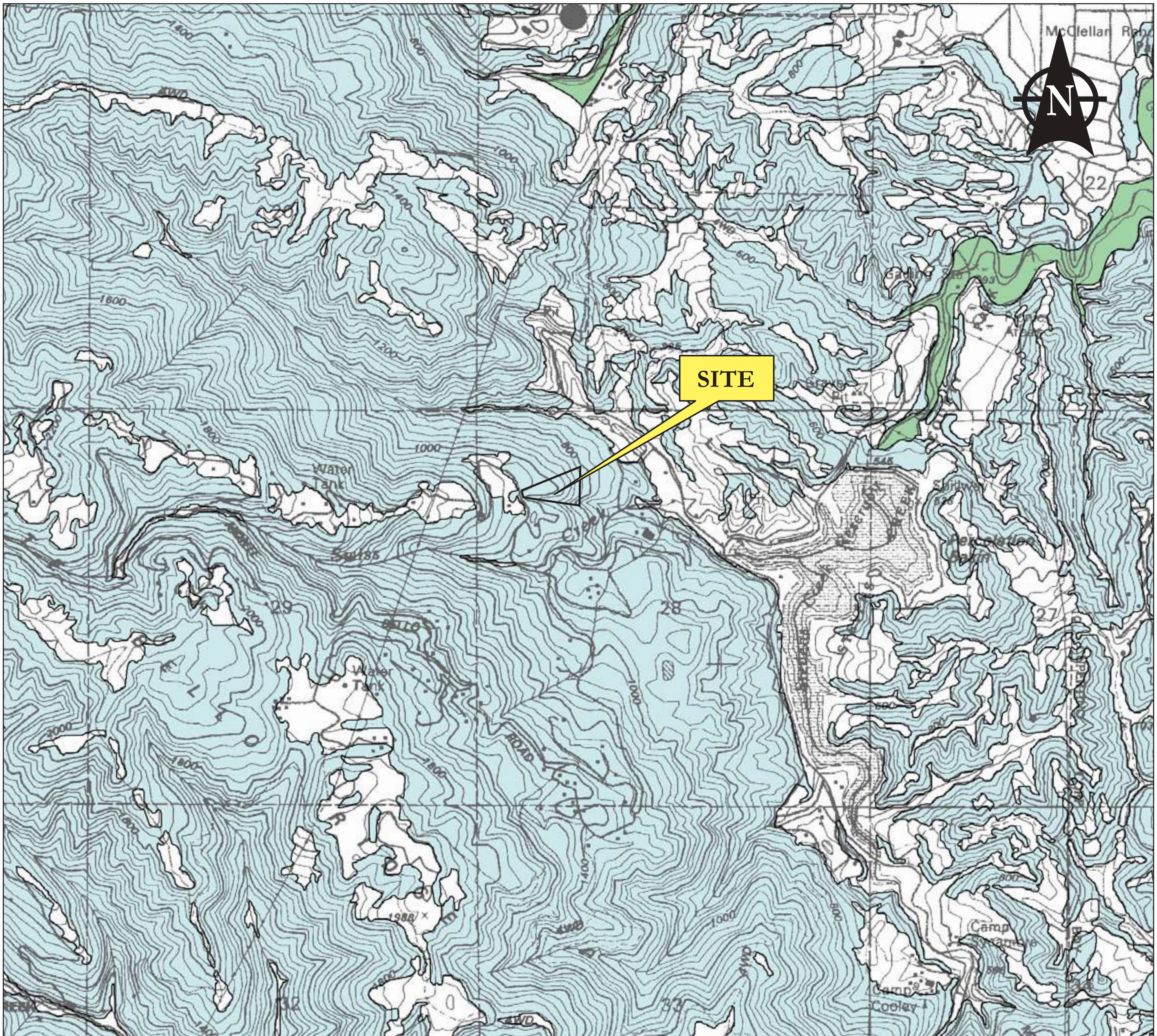
Fault, showing dip or direction of dip
Dashed where approximately located; dotted where concealed.

(fs) Qls

Landslide deposits, boundaries of landslide deposit known. Arrows indicated general direction of movement. Symbols in parentheses indicated map unit involved in landsliding. Hachures indicate presence of scarp at head of landslide deposit

Base: Geologic Map of Sargent-Berrocal Fault Zone Between Los Gatos & Los Altos Hills, Santa Clara County, California by Sorg & McLaughlin, 1975 Approximate Scale: 1 inch = 2,000 feet

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	WATERS RESIDENCE APN 351-42-004 PEACOCK COURT SANTA CLARA COUNTY, CALIFORNIA		VICINITY LANDSLIDE MAP
	PROJECT NO. 3262-1R1	APRIL 2020	FIGURE A-4



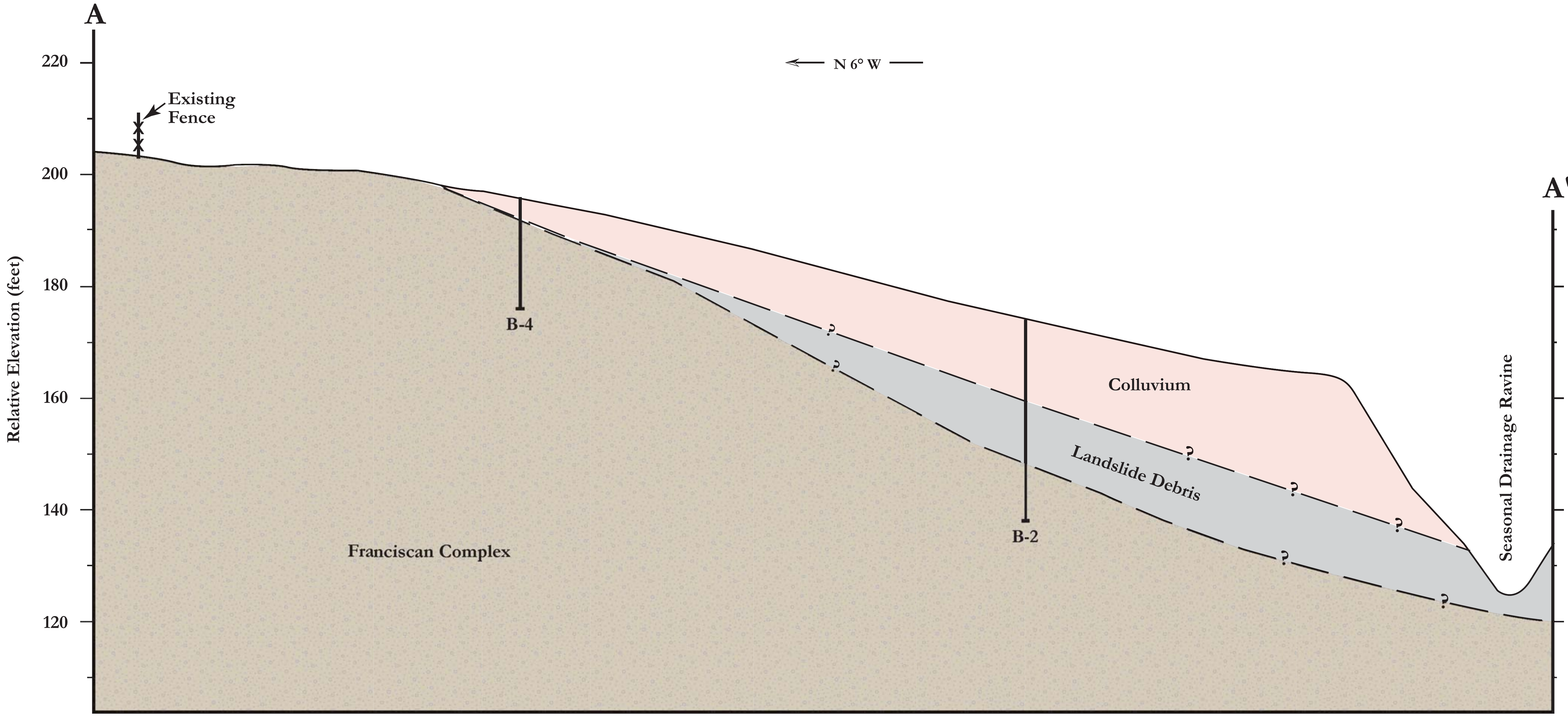
Legend

Areas where historic occurrence of liquefaction, or local, geological, geotechnical and groundwater conditions indicate a potential for earthquake-induced liquefaction.

Areas where previous occurence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for earthquake-induced landslide.

Base: State of California Seismic Hazard Zone Map, Cupertino Quadrangle, California Geological Survey, 2002
Approximate Scale: 1 inch = 2,000 feet

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	**WATERS RESIDENCE** **APN 351-42-004 PEACOCK COURT** **SANTA CLARA COUNTY, CALIFORNIA**	**STATE SEISMIC HAZARD ZONES MAP**
PROJECT NO. 3262-1R1	**APRIL 2020**	**FIGURE A-5**



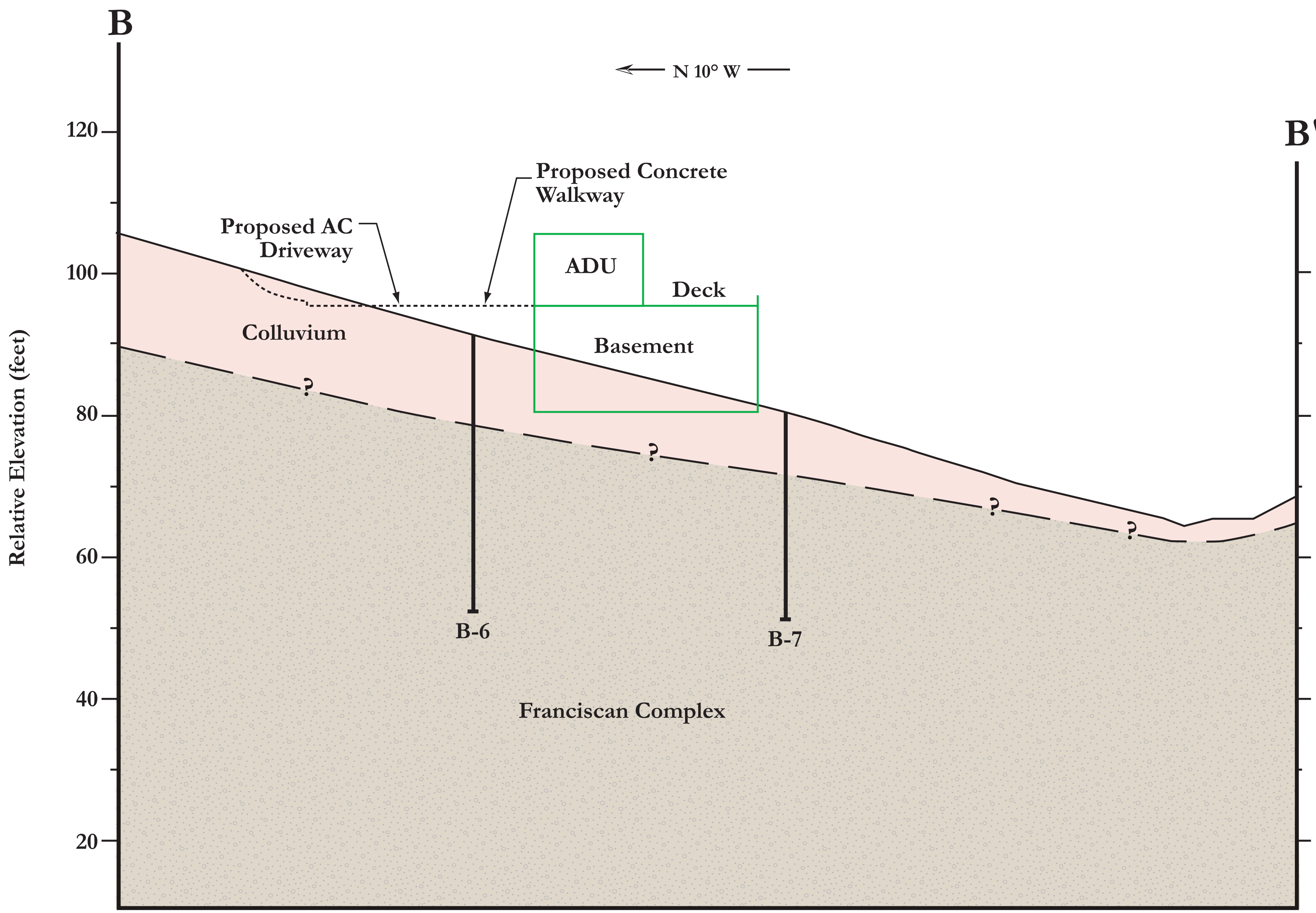
LEGEND

B-1 Approximate Location of Soil Boring by Murray Engineers, Inc., drilled March 28, 2017

Base: Laser Range Finder and Zip Level Survey by Murray Engineers, Inc., April 21, 2017

Approximate Scale: 1 inch = 20 feet (horizontal = vertical)

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	GEOLOGIC CROSS-SECTION A-A'	
WATERS RESIDENCE APN 351-42-004 PEACOCK COURT SANTA CLARA COUNTY, CALIFORNIA		
PROJECT NO. 3262-1R1	APRIL 2020	FIGURE A-6



LEGEND

B-7 Approximate Location of Soil Boring by Murray Engineers, Inc., drilled November 27, 2019

Base: Grading & Drainage Plan by RI Engineering Inc., dated January 2019

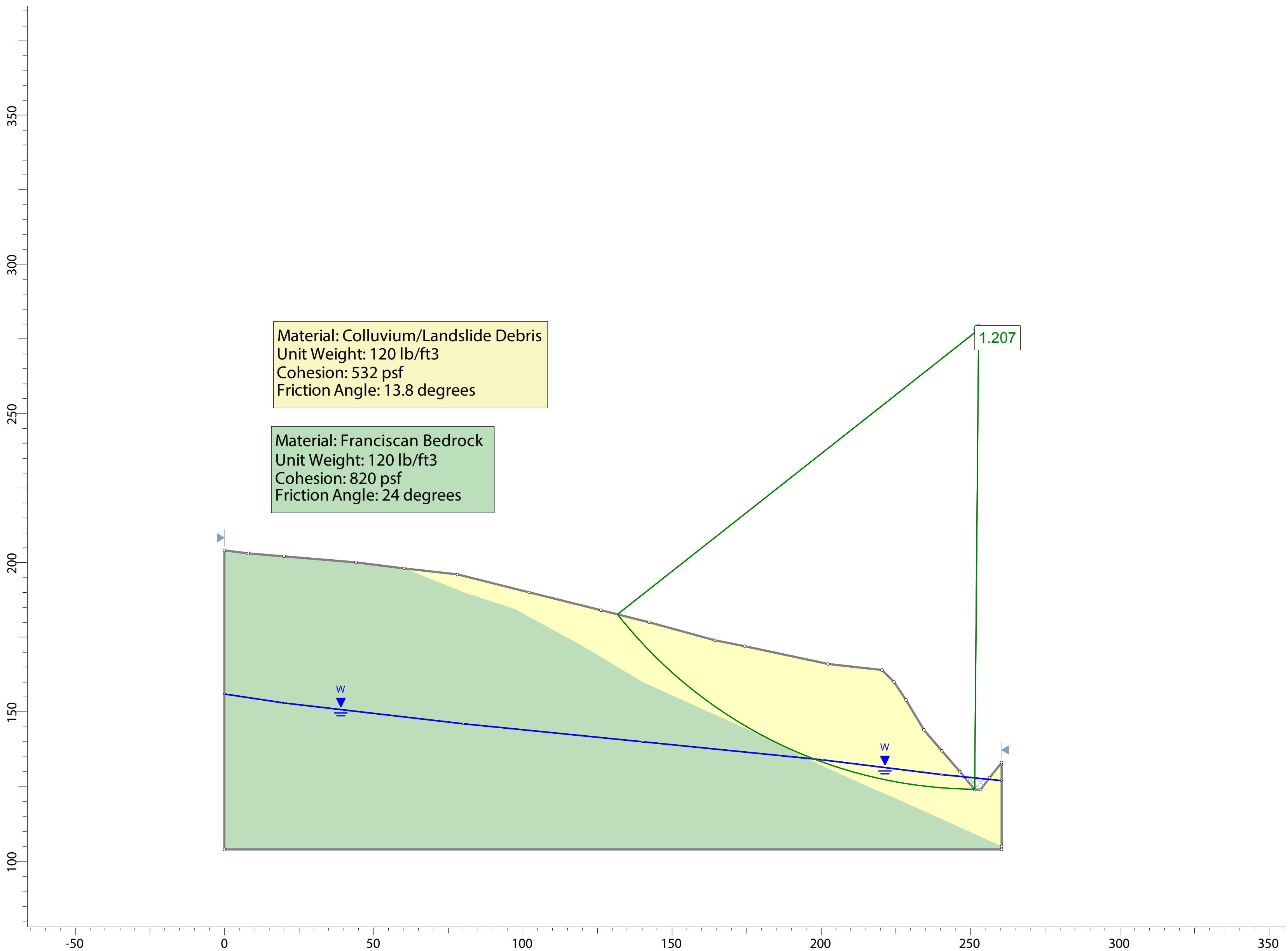
Approximate Scale: 1 inch = 20 feet (horizontal = vertical)

MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

GEOLOGIC CROSS-SECTION B-B'

WATERS RESIDENCE
APN 351-42-004 PEACOCK COURT
SANTA CLARA COUNTY, CALIFORNIA

PROJECT NO. 3262-1R1	APRIL 2020	FIGURE A-7



MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

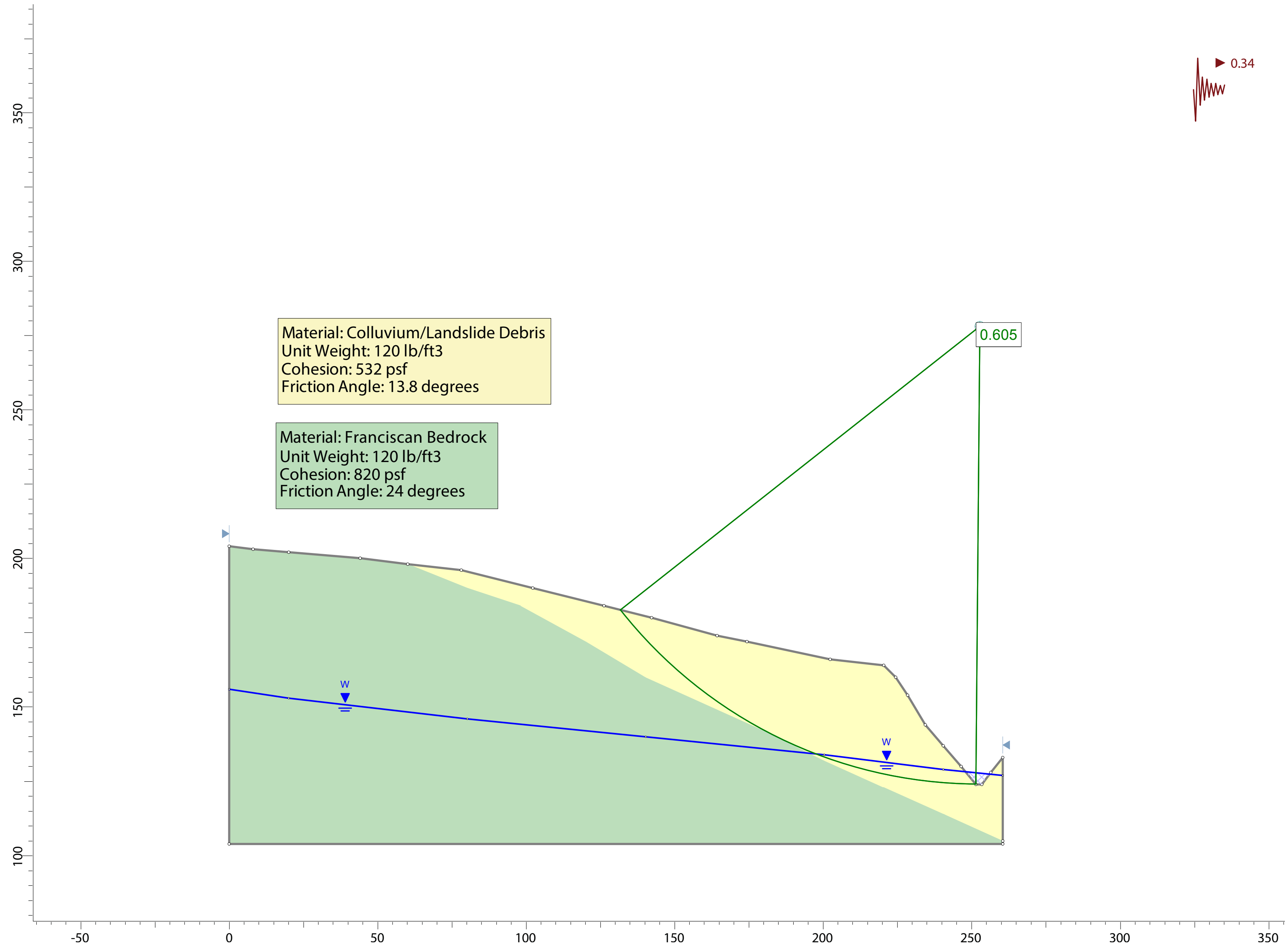
WATERS RESIDENCE
APN 351-42-004 PEACOCK COURT
SANTA CLARA COUNTY, CALIFORNIA

STATIC SLOPE STABILITY
ANALYSIS ALONG
CROSS-SECTION A-A'

PROJECT NO. 3262-1R1

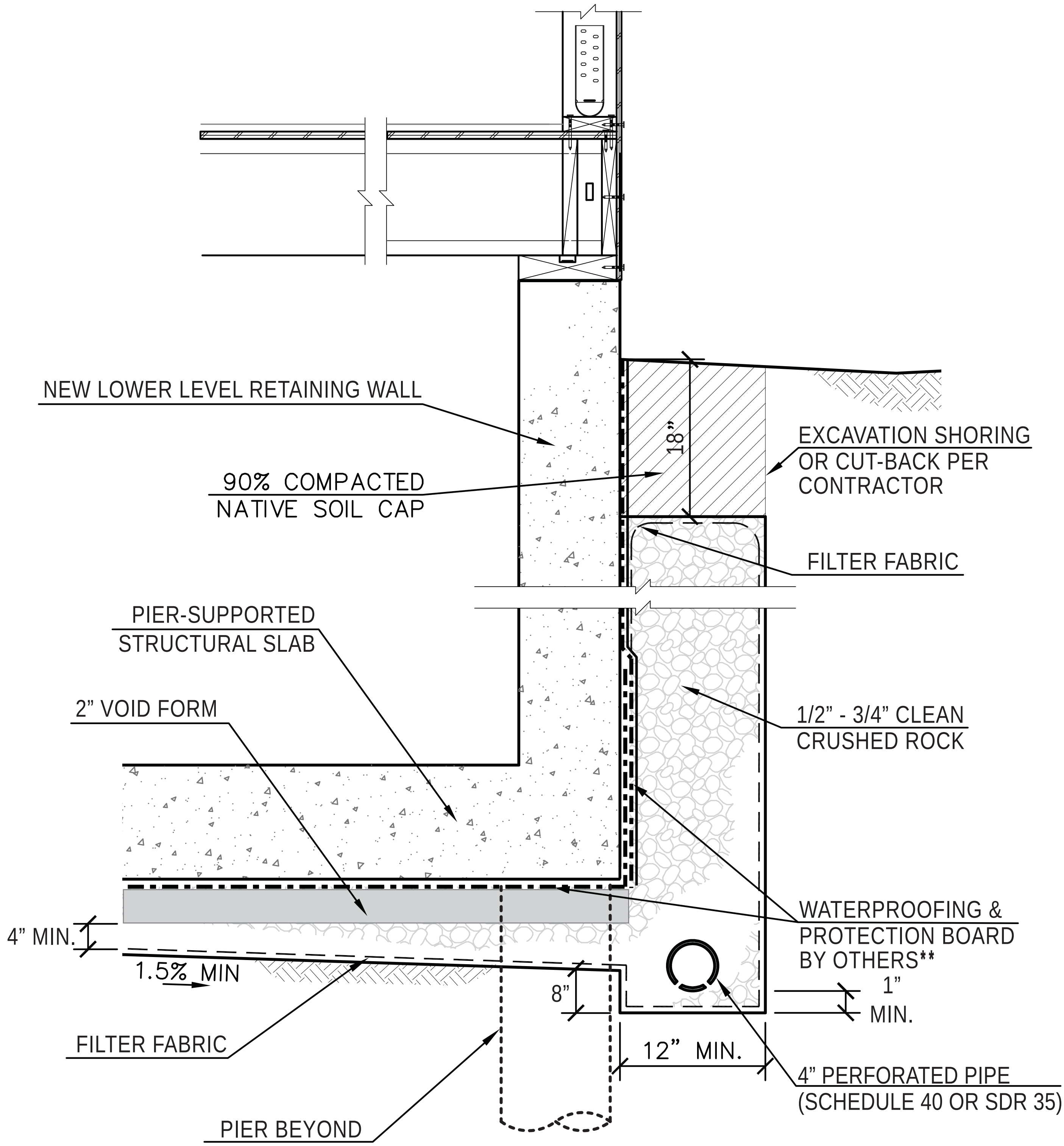
APRIL 2020

FIGURE A-8



MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

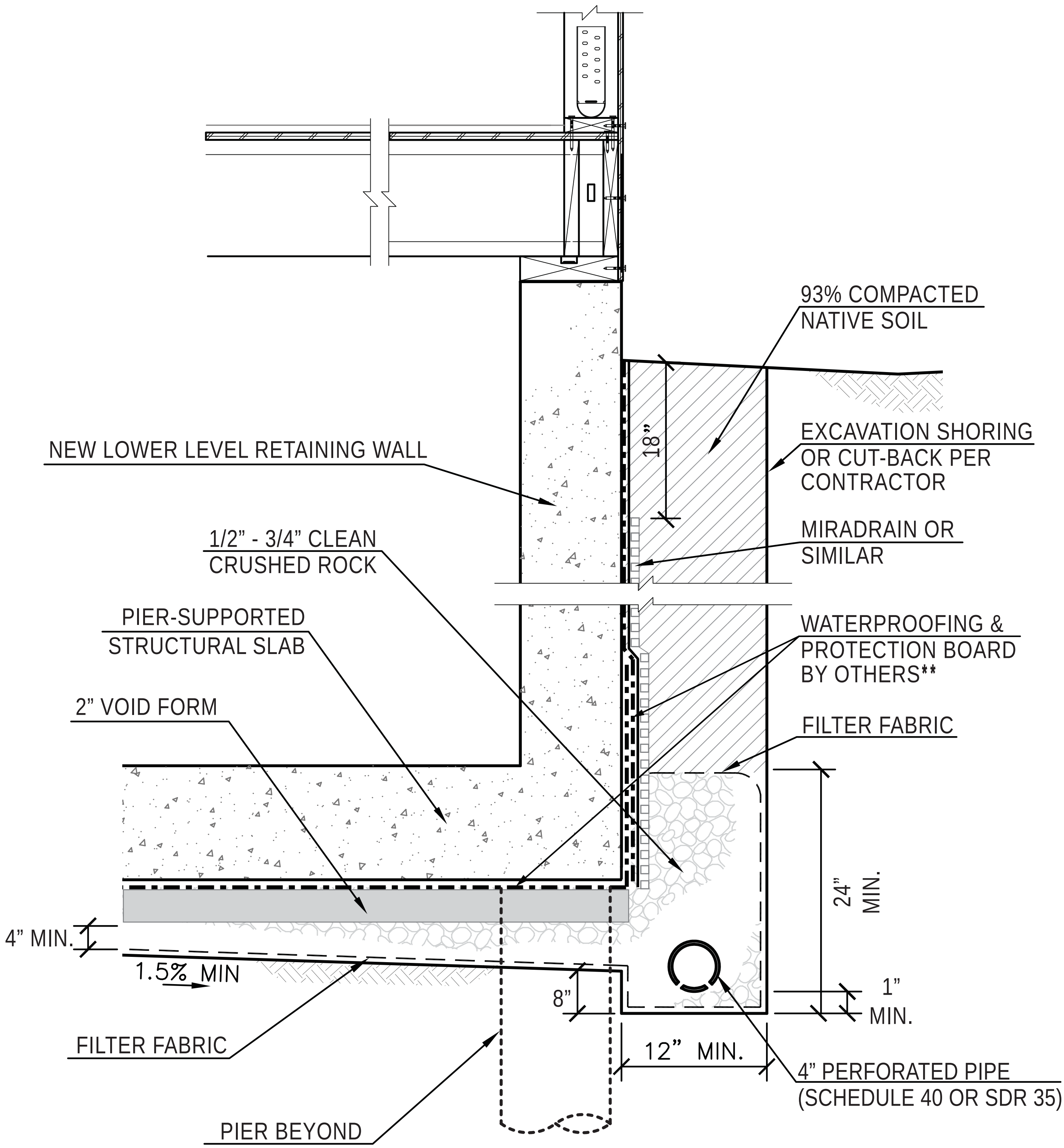
WATERS RESIDENCE APN 351-42-004 PEACOCK COURT SANTA CLARA COUNTY, CALIFORNIA		PSEUDO-STATIC SLOPE STABILITY ANALYSIS ALONG CROSS-SECTION A-A'
PROJECT NO. 3262-1R1	APRIL 2020	FIGURE A-9



NOT TO SCALE **** MiraDRAIN is not recommended as protection board for waterproofing**

Note: This diagram is provided solely to schematically depict the recommendations presented in this report for the basement and basement retaining wall subdrainage. Reference to the basement slab and wall, framing, waterproofing, and extent of excavation are only shown for clarity.

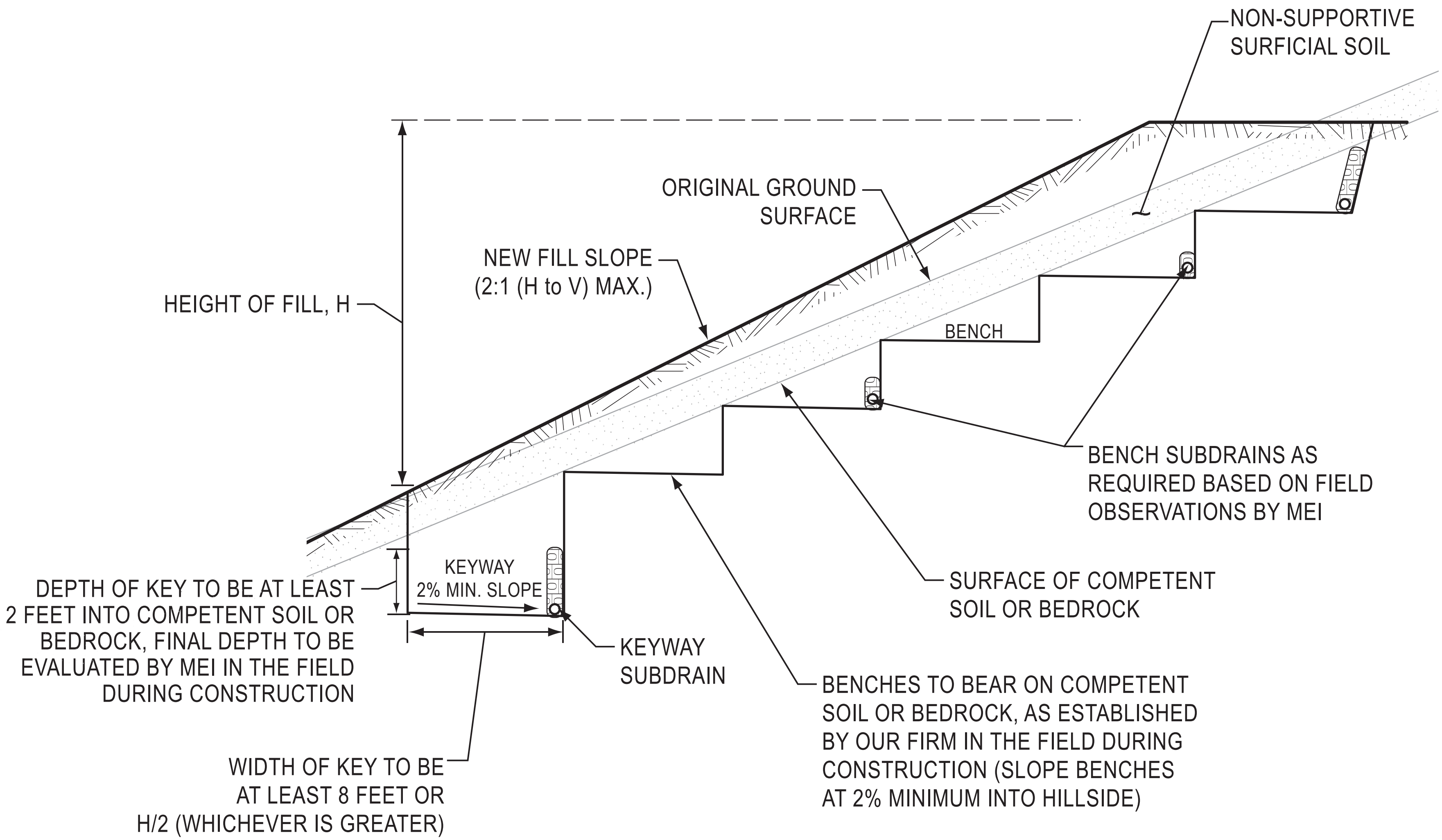
MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	**WATERS RESIDENCE APN 351-42-004 PEACOCK COURT SANTA CLARA COUNTY, CALIFORNIA**	**BASEMENT SUBDRAIN SYSTEM ALTERNATIVE A**
	PROJECT NO. 3262-1R1 **APRIL 2020**	**FIGURE A-10**



NOT TO SCALE **** MiraDRAIN is not recommended as protection board for waterproofing**

Note: This diagram is provided solely to schematically depict the recommendations presented in this report for the basement and basement retaining wall subdrainage. Reference to the basement slab and wall, framing, waterproofing, and extent of excavation are only shown for clarity.

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	WATERS RESIDENCE APN 351-42-004 PEACOCK COURT SANTA CLARA COUNTY, CALIFORNIA		BASEMENT SUBDRAIN SYSTEM ALTERNATIVE B
	PROJECT NO. 3262-1R1	APRIL 2020	FIGURE A-11



MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	WATERS RESIDENCE APN 351-42-004 PEACOCK COURT SANTA CLARA COUNTY, CALIFORNIA	SCHEMATIC FILL SLOPE DETAIL
PROJECT NO. 3262-1R1	APRIL 2020	FIGURE A-12

Date(s) Drilled	**March 28, 2017**	Logged By	**AK**	Checked By	**KP/MB**
Drilling Method	**Continuous Flight Auger**	Drill Bit Size/Type	**4 inch drill bit**	Total Depth of Borehole	**45 feet bgs**
Drill Rig Type	**Track-Mounted CME 55**	Drilling Contractor	**Britton Excavation**	Approximate Surface Elevation	**171 feet (relative)**
Groundwater Level and Date Measured	**38 feet ATD, 36 feet after 2 hours**	Sampling Method(s)	**3" OD, 2.5" OD, & 2" OD SPT Split Spoon Samplers**	Hammer Data	**140 lb, 30 in drop, hydraulic**
Borehole Backfill	**Cuttings**	Location	**Southeast of proposed residence**		

Elevation, feet	Depth, feet	Sampling Resistance, blows/foot	Relative Consistency	USCS Symbol	MATERIAL DESCRIPTION	Water Content, %
170	0		Stiff to Very Stiff	ML	CLAYEY SILT with GRAVEL, yellowish brown, homogeneous, low plasticity fines, trace to minor shale, chert and sandstone fragments, moist to slightly moist (Colluvium)	
		11				12
165	5					
		12				9
160	10					
		14	Stiff	ML	CLAYEY SILT, yellowish brown, homogeneous, low plasticity fines, fine- to coarse-grained sand, trace to minor subrounded to subangular sandstone and shale fragments, slightly wet (Old Landslide Debris)	10
155	15	16				8
		19				8
150	20	10	Medium Dense	SC	CLAYEY SAND, olive brown, homogeneous, low plasticity fines, fine- to coarse-grained sand, moist (Old Landslide Debris)	8
		20	Medium Dense	SW	GRAVELLY SAND, yellowish brown, heterogeneous, low plasticity fines, angular to subangular sandstone and shale fragments, slightly moist (Old Landslide Debris)	2
145	25	11				3
		41	Soft*	BR	SHALE, dark gray to very dark gray, moderately weathered, variably weathered, highly fractured, slightly moist to moist (Franciscan Complex/Ancient Landslide Deposit (?))	5
140	30	30				10
		47				7
135	35				(after 2 hours)	
					(ATD)	
		32				10
130	40					
		20		BR	*designates hardness of bedrock (see Figure B-10)	6
125	45				Bottom of Boring at 45 feet bgs	

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MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

WATERS RESIDENCE
APN 351-42-004 PEACOCK COURT
SANTA CLARA COUNTY, CALIFORNIA

LOG OF BORING B-1

PROJECT NO. 3262-1R1 | APRIL 2020 | FIGURE B-1

Date(s) Drilled **March 28, 2017**	Logged By **AK**	Checked By **KP/MB**
Drilling Method **Continuous Flight Auger**	Drill Bit Size/Type **4 inch drill bit**	Total Depth of Borehole **35 feet bgs**
Drill Rig Type **Track-Mounted CME 55**	Drilling Contractor **Britton Excavation**	Approximate Surface Elevation **173 feet (relative)**
Groundwater Level and Date Measured **Not Encountered ATD**	Sampling Method(s) **3" OD, 2.5" OD, & 2" OD SPT Split Spoon Samplers**	Hammer Data **140 lb, 30 in drop, hydraulic**
Borehole Backfill **Cuttings**	Location **Southwest of proposed residence**	

Elevation, feet	Depth, feet	Sample Type	Sampling Resistance, blows/foot	Relative Consistency	USCS Symbol	MATERIAL DESCRIPTION	Water Content, %
172	0			Stiff to Very Stiff	ML	CLAYEY SILT with GRAVEL, yellowish brown to olive brown, low plasticity fines, fine- to medium-grained sand, subrounded to subangular sandstone and shale fragments, slightly moist (Colluvium)	
			8				7
167	5						
			9				7
162	10						
			11				8
157	15		12	Stiff to Hard	ML	CLAYEY SILT, yellowish brown, homogeneous, low plasticity fines, fine- to coarse-grained sand, trace to minor subrounded to subangular sandstone and shale fragments, moist (Old Landslide Debris)	7
			26				10
152	20		20				10
			34				7
147	25		29	Soft*	BR	SHALE, dark gray to very dark gray, moderately weathered, variably weathered, highly fractured, slightly moist (Franciscan Complex/Ancient Landslide Deposit (?))	7
			30				8
142	30						
			36			*designates hardness of bedrock (see Figure B-10)	5
137	35					Bottom of Boring at 35 feet bgs	
132	40						
127	45						

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MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

WATERS RESIDENCE
APN 351-42-004 PEACOCK COURT
SANTA CLARA COUNTY, CALIFORNIA

LOG OF BORING B-2

PROJECT NO. 3262-1R1 | APRIL 2020 | FIGURE B-2

Date(s) Drilled **March 28, 2017**	Logged By **AK**	Checked By **KP/MB**
Drilling Method **Continuous Flight Auger**	Drill Bit Size/Type **4 inch drill bit**	Total Depth of Borehole **15 feet bgs**
Drill Rig Type **Track-Mounted CME 55**	Drilling Contractor **Britton Excavation**	Approximate Surface Elevation **191 feet (relative)**
Groundwater Level and Date Measured **Not Encountered ATD**	Sampling Method(s) **3" OD, 2.5" OD, & 2" OD SPT Split Spoon Samplers**	Hammer Data **140 lb, 30 in drop, hydraulic**
Borehole Backfill **Cuttings**	Location **Near south corner of residence**	

Elevation, feet	Depth, feet	Sample Type	Sampling Resistance, blows/foot	Relative Consistency	USCS Symbol	MATERIAL DESCRIPTION	Water Content, %
191	0		5	Soft*	BR	SHALE, dark gray to very dark gray, moderately weathered, variably weathered, highly fractured, slightly moist to moist (Franciscan Complex/Ancient Landslide Deposit (?))	4
			17				12
			15				13
			22				7
			21			*designates hardness of bedrock (see Figure B-10)	3
176	15					Bottom of Boring at 15 feet bgs	

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MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

WATERS RESIDENCE
APN 351-42-004 PEACOCK COURT
SANTA CLARA COUNTY, CALIFORNIA

LOG OF BORING B-3

PROJECT NO. 3262-1R1 | APRIL 2020 | FIGURE B-3

Date(s) Drilled **March 28, 2017**	Logged By **AK**	Checked By **KP/MB**
Drilling Method **Continuous Flight Auger**	Drill Bit Size/Type **4 inch drill bit**	Total Depth of Borehole **20 feet bgs**
Drill Rig Type **Track-Mounted CME 55**	Drilling Contractor **Britton Excavation**	Approximate Surface Elevation **197 feet (relative)**
Groundwater Level and Date Measured **Not Encountered ATD**	Sampling Method(s) **3" OD, 2.5" OD, & 2" OD SPT Split Spoon Samplers**	Hammer Data **140 lb, 30 in drop, hydraulic**
Borehole Backfill **Cuttings**	Location **Proposed driveway between A.D.U. and residence**	

Elevation, feet	Depth, feet	Sample Type	Sampling Resistance, blows/foot	Relative Consistency	USCS Symbol	MATERIAL DESCRIPTION	Water Content, %
196	0		5	Medium Stiff	ML	CLAYEY SILT with GRAVEL, yellowish brown, homogeneous, low plasticity fines, subangular to subrounded gravel, trace to minor shale, chert and sandstone fragments, moist to slightly moist (Colluvium)	11
			5				11
			11	Soft*	BR	SILTY SANDSTONE, yellowish brown to olive brown, moderately to very severely weathered, slightly moist (Franciscan Complex/Ancient Landslide Deposit (?))	15
191	5						
			29				15
186	10						
			19				10
181	15						
			22			*designates hardness of bedrock (see Figure B-10)	8
176	20					Bottom of Boring at 20 feet bgs	
171	25						
166	30						
161	35						
156	40						
151	45						

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MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

WATERS RESIDENCE
APN 351-42-004 PEACOCK COURT
SANTA CLARA COUNTY, CALIFORNIA

LOG OF BORING B-4

PROJECT NO. 3262-1R1 | **APRIL 2020** | **FIGURE B-4**

Date(s) Drilled **March 28, 2017**	Logged By **AK**	Checked By **KP/MB**
Drilling Method **Continuous Flight Auger**	Drill Bit Size/Type **4 inch drill bit**	Total Depth of Borehole **20 feet bgs**
Drill Rig Type **Track-Mounted CME 55**	Drilling Contractor **Britton Excavation**	Approximate Surface Elevation **190 feet (relative)**
Groundwater Level and Date Measured **Not Encountered ATD**	Sampling Method(s) **3" OD, 2.5" OD, & 2" OD SPT Split Spoon Samplers**	Hammer Data **140 lb, 30 in drop, hydraulic**
Borehole Backfill **Cuttings**	Location **West of A.D.U.**	

Elevation, feet	Depth, feet	Sample Type	Sampling Resistance, blows/foot	Relative Consistency	USCS Symbol	MATERIAL DESCRIPTION	Water Content, %
189	0		1	Very Soft	ML	FILL: CLAYEY SILT, yellowish brown, homogeneous, low to medium plasticity, trace fine- to medium-grained sand, scarce rootlets, very moist	16
			9	Stiff	CL	LEAN CLAY, olive brown, homogeneous, high plasticity fines, scarce fine-grained sand, moist (Colluvium)	19
			15			PI=27%; LL=41% (sample from 2 to 3.5 feet)	11
184	5						
			47	Soft*	BR	SANDSTONE, yellowish brown to olive brown, moderately to very severely weathered, slightly moist (Franciscan Complex/Ancient Landslide Deposit (?))	5
179	10						
			24				6
174	15						
			28			*designates hardness of bedrock (see Figure B-10)	9
169	20					Bottom of Boring at 20 feet bgs	
164	25						
159	30						
154	35						
149	40						
144	45						

Date(s) Drilled **November 27, 2019**	Logged By **MM**	Checked By **HL**
Drilling Method **Continuous Flight Auger**	Drill Bit Size/Type **4 inch Continuous Flight Auger**	Total Depth of Borehole **39 feet bgs**
Drill Rig Type **Track-Mounted CME 55**	Drilling Contractor **Britton Excavation**	Approximate Surface Elevation **92 feet (relative)**
Groundwater Level and Date Measured **Not Encountered ATD**	Sampling Method(s) **3" OD, 2.5" OD, & 2" OD SPT Split Spoon Samplers**	Hammer Data **140 lb, 30 in drop, rope & cathead**
Borehole Backfill **Cuttings**	Location **North of ADU**	

Elevation, feet	Depth, feet	Sample Type	Sampling Resistance, blows/foot	Relative Consistency	USCS Symbol	MATERIAL DESCRIPTION	Water Content, %
91	0		12	Stiff	ML	CLAYEY SILT, reddish to yellowish brown, homogeneous, low plasticity fines, subangular to subrounded sandstone fragments, slightly moist to moist (Colluvium)	5
							6
			15				7
86	5						7
			14				8
							12
81	10						
76	15		17	Soft*	BR?	MÉLANGE, yellowish to grayish to olive brown, slightly heterogeneous, trace coarse- to medium sand, low plasticity fines, subangular to angular shale and graywacke fragments, moist (Franciscan Complex/Ancient Landslide Deposit (?))	12 9
71	20		24				6
66	25		23				6
61	30		35				5
56	35		28				14 6
				Soft*	BR	SHALE, dark brown to dark gray, moderately weathered, highly fractured, slightly moist Complex/Ancient Landslide Deposit (?))	
			50/6"				3
51	40					*designates hardness of bedrock (see Figure B-10) Bottom of Boring at 39 feet bgs	
46	45						

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MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

WATERS RESIDENCE **APN 351-42-004 PEACOCK COURT** **SANTA CLARA COUNTY, CALIFORNIA**	**LOG OF BORING B-6**
PROJECT NO. 3262-1R1 · **APRIL 2020**	**FIGURE B-6**

Date(s) Drilled	**November 27, 2019**	Logged By	**MM**	Checked By	**HL**
Drilling Method	**Continuous Flight Auger**	Drill Bit Size/Type	**4 inch Continuous Flight Auger**	Total Depth of Borehole	**29.4 feet bgs**
Drill Rig Type	**Track-Mounted CME 55**	Drilling Contractor	**Britton Excavation**	Approximate Surface Elevation	**80 feet (relative)**
Groundwater Level and Date Measured	**Not Encountered ATD**	Sampling Method(s)	**3" OD, 2.5" OD, & 2" OD SPT Split Spoon Samplers**	Hammer Data	**140 lb, 30 in drop, rope & cathead**
Borehole Backfill	**Cuttings**	Location	**South of ADU**		

Elevation, feet	Depth, feet	Sampling Resistance, blows/foot	Relative Consistency	USCS Symbol	MATERIAL DESCRIPTION	Water Content, %
79	0	8	Medium Stiff to Very Stiff	ML	CLAYEY SILT, reddish to yellowish brown, homogeneous, low plasticity fines, subangular to subrounded sandstone fragments, slightly moist to moist (Colluvium)	7
						8
		12				9
74	5					10
		23				14
						20
			Soft*	BR?	MÉLANGE, yellowish to grayish to olive brown, slightly heterogeneous, trace coarse- to medium sand, low plasticity fines, subangular to angular shale and graywacke fragments, moist (Franciscan Complex/Ancient Landslide Deposit (?))	10
69	10					
64	15	17				12
59	20	24				10
54	25	15				12
		60/11"	Medium*	BR	SANDSTONE, gray, very slightly weathered, mildly fractured (Franciscan Complex/Ancient Landslide Deposit (?))	12
						1
49	30				*designates hardness of bedrock (see Figure B-10)	
					Refusal at 29.4 feet bgs	
44	35					
39	40					
34	45					

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 SAND with Silt (SW-SM)	Lean-Fat CLAY, CLAY w/SAND, SANDY CLAY (CL/CH)
Well graded GRAVEL (GW)	Well graded SAND with Clay (SW-SC)	SILTY CLAY (CL-ML)
Poorly graded GRAVEL (GP)	Poorly graded SAND with Silt (SP-SM)	Lean CLAY/PEAT (CL-OL)
Well graded GRAVEL with Silt (GW-GM)	Poorly graded SAND with Clay (SP-SC)	Fat CLAY/SILT (CH-MH)
Well graded GRAVEL with Clay (GW-GC)	Silty SAND (SM)	Fat CLAY/PEAT (CH-OH)
Poorly graded GRAVEL with Silt (GP-GM)	Clayey SAND (SC)	Silty SAND to Sandy SILT (SM-ML)
Poorly graded GRAVEL with Clay (GP-GC)	SILT, SILT w/SAND, SANDY SILT (ML)	Silty SAND to Sandy SILT (SM-MH)
Silty GRAVEL (GM)	Lean CLAY, CLAY w/SAND, SANDY CLAY (CL)	Clayey SAND to Sandy CLAY (SC-CL)
Clayey GRAVEL (GC)	SILT, SILT w/SAND, SANDY SILT (MH)	Clayey SAND to Sandy CLAY (SC-CH)
Well graded SAND (SW)	Fat CLAY, CLAY w/SAND, SANDY CLAY (CH)	SILT to CLAY (CL/ML)
Poorly graded SAND (SP)	SILT, SILT with SAND, SANDY SILT (ML-MH)	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	WATERS RESIDENCE APN 351-42-004 PEACOCK COURT SANTA CLARA COUNTY, CALIFORNIA		UNIFIED SOIL CLASSIFICATION SYSTEM
	PROJECT NO. 3262-1R1	**APRIL 2020**	**FIGURE B-9**

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

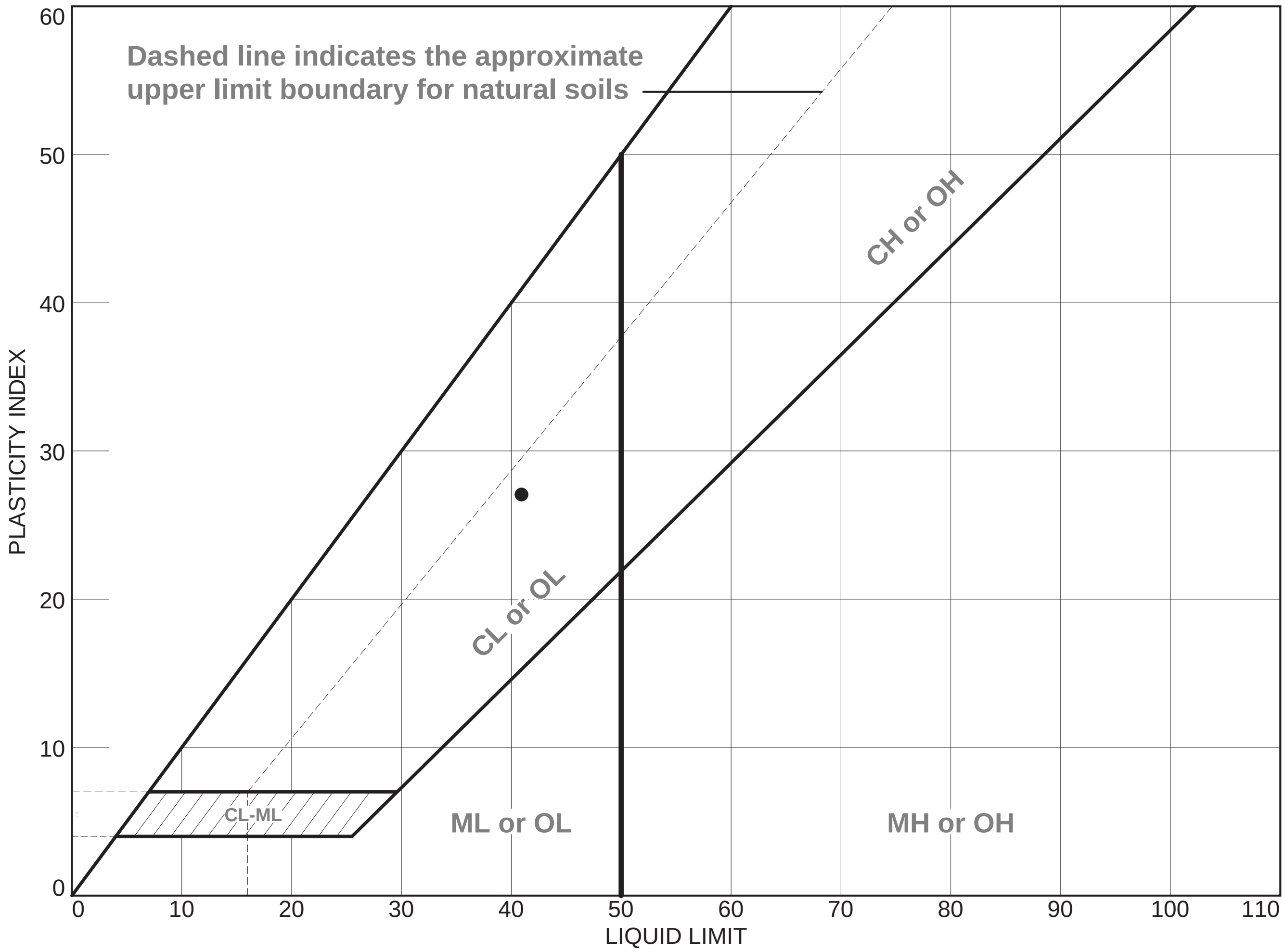
LABORATORY TESTS

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

Natural moisture content was determined for most samples recovered from the borings in accordance with ASTM D2216. This test determines the moisture content representative of field conditions at the time the samples were collected. The results are presented on the boring log at the appropriate sample depths.

The Atterberg limits were evaluated on one sample in accordance with ASTM D 4318. The Atterberg limits are the moisture content within which the soil is workable or plastic. This index test provides an indication of the expansive potential of the soil. The results are presented in Figure C-1 and on the boring logs at the appropriate sample depth.

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	Boring 5	1	2-3.5	19.0	14	41	27	CL

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES

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SANTA CLARA COUNTY, CALIFORNIA

LIQUID & PLASTIC LIMITS TEST REPORT

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