County of San Mateo - Planning and Building Department

ATTACHMENT K

ENGINEERING GEOLOGIC & GEOTECHNICAL INVESTIGATION
4-LOT RESIDENTIAL DEVELOPMENT
ZMAY PROPERTY
1551 CRYSTAL SPRINGS ROAD
SAN MATEO COUNTY, CALIFORNIA

THIS REPORT HAS BEEN PREPARED FOR:

NICK ZMAY

1551 CRYSTAL SPRINGS ROAD

HILLSBOROUGH, CALIFORNIA 94010

FEBRUARY 2014

MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

February 10, 2014
Project No. 1847-1R1

Nick Zmay
1551 Crystal Springs Road
Hillsborough, California 94010

RE: ENGINEERING GEOLOGIC & GEOTECHNICAL INVESTIGATION, 4-LOT RESIDENTIAL DEVELOPMENT, ZMAY PROPERTY, 1551 CRYSTAL SPRINGS ROAD, SAN MATEO COUNTY, CALIFORNIA

Dear Mr. Zmay:

We are pleased to present the results of our engineering geologic investigation relating to the design and construction of the proposed 4-lot residential subdivision of your property located at 1551 Crystal Springs Road in San Mateo County, California. The purpose of our services was to evaluate the feasibility of the proposed residential development from both engineering geologic and geotechnical engineering perspectives. This report also summarizes the results of our field, laboratory and engineering work, and presents general recommendations for suggested foundation types and grading for the proposed residential subdivision.

While we believe that our opinions and conclusions are reasonable, it should be clearly understood that the geotechnical recommendations provided in this report are based on highly tentative plans and are for general planning purposes. Once the details of the proposed construction have been developed, we should review the design and confirm that the recommendations included in this report are still appropriate. Please note that this could result in modifications of our opinions and conclusions contained in this report.

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

Very truly yours,
MURRAY ENGINEERS, INC.

A. Nicole Roatch

A. Nicole Roatch
Senior Staff Geologist

REGISTERED PROFESSIONAL ENGINEER
JOHN A. STILLMAN
No. GE2523
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John A. Stillman, G.E., C.E.G. 1868
Principal Geotechnical Engineer

ANR:JAS

Copies: Addressee (6)

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ENGINEERING GEOLOGIC &
GEOTECHNICAL INVESTIGATION
4-LOT RESIDENTIAL DEVELOPMENT
ZMAY PROPERTY
1551 CRYSTAL SPRINGS ROAD
SAN MATEO COUNTY, CALIFORNIA

INTRODUCTION

This report presents the results of our engineering geologic and geotechnical investigation relating to the design and construction of a four-lot subdivision of the property located at 1551 Crystal Springs Road in San Mateo County, California. The project location is indicated on the Vicinity Map, Figure A-1. The purpose of our investigation was to evaluate the engineering geologic and geotechnical conditions on the property in the area of the proposed subdivision in order to evaluate the feasibility of the proposed subdivision, the potential impacts of geologic hazards of future site development, and to provide general geotechnical design criteria and recommendations for the project.

Project Description

The subject property is located on a steep west-facing hillside in a rural residential area of San Mateo County. The property is bounded by Parrott Drive along the uphill (east) side and Crystal Springs Road and Polhemus Road along the downhill (west) side. The proposed subdivision will split an existing approximately 60-acre lot into four approximately 2.5-acre lots for single-family residences and a "remainder lot" to be designated as open space. The proposed new residential building envelopes are to be located in the northeastern portion of the property along Parrot Drive. The details of the construction have not been formalized, but we anticipate that the residential development will include one- or two-story residences in the uphill portion of the lots and may include full or partial basements. Driveway access to the new residences will be provided off of Parrott Road. We understand that you are considering shifting the building site on Lot 1 further downslope from Parrott Drive and providing access to the new improvements along a shared access road extending across Lot 2. Site improvements will likely include retaining walls to accommodate grade changes around the new residences and along the potential driveway on Lots 1 and 2. The layout of the proposed improvements is shown on the Partial Site Plan & Engineering Geologic Map, Figure A-2.

SCOPE OF SERVICES

We performed the following services in accordance with our initial agreement dated November 7, 2013 (executed December 10, 2013):

- Reviewed published geologic maps and aerial photographs to evaluate the prevailing geologic and seismic conditions on the site and in the site vicinity
- Reviewed prior geologic and geotechnical reports for the property by Site Characteristics, Inc., dated July 1983, William Cotton and Associates, dated April 20, 1984, and Bay Area Geotechnical Group, dated December 20, 2007
- Performed an engineering geologic reconnaissance and mapping on the proposed lots and in the vicinity of the proposed improvements
- Explored the subsurface conditions by excavating, logging, and sampling six exploratory borings in the vicinity of the planned improvements
- Performed laboratory analyses and testing on selected soil samples for soil classification and to evaluate engineering properties of the subsurface materials
- Performed engineering geologic and geotechnical analyses to evaluate the relative stability of the proposed building sites and to develop general geotechnical engineering design criteria for the proposed improvements
- Prepared this report presenting a summary of our investigation and our conclusions relating to the geologic hazards that could potentially impact the site and the proposed improvements and the feasibility of the proposed improvements

GEOLOGIC & SEISMIC CONDITIONS

Geologic Overview

The property is located on a west-facing hillside in the foothills along the northeast side of the Santa Cruz Mountains, a northwest-trending range within the California Coast Ranges geomorphic province. The local topography is dominated by a series of west-trending spur ridges and intervening seasonal drainage swales. Crystal Springs Road extends along the western property boundary at the base of the hillside and converges with Polhemus Road near the southern corner of the property. San Mateo Creek and Polhemus Creek run parallel to Crystal Springs Road and Polhemus Road, respectively. Elevations across the site range from approximately 500 feet along Parrott Drive in the eastern portion of the site down to approximately 140 feet above mean sea level at the base of the hillside in the northwest corner of the site (see Figure A-1).

According to the Geologic Map of the Montara Mountain and San Mateo 7-½' Quadrangles (Pampeyan, 1994), the site is located in an area underlain by Cretaceous and Jurassic age (approximately 65 to 200 million years old) sheared rock of the Franciscan Complex (fsr).

The sheared rock generally consists of soft, light- to dark-gray, sheared shale, siltstone, and greywacke sandstone containing various-size tectonic inclusions of Franciscan rock types. According to the geologic map, the lower portion of the slope in the northwest corner of the property is blanketed by Quaternary slope wash, ravine fill and colluvium deposits (Qsr). These deposits generally consist of unconsolidated to moderately consolidated sand, silt, clay, and rock fragments accumulated by slow downslope movement of weathered rock debris and soil. A copy of the relevant portion of the geologic map is presented on Figure A-3, Vicinity Geologic Map.

According to the geologic map, the Geotechnical Hazard Synthesis Map for San Mateo County (Leighton and Associates, 1976), and the Preliminary Map of Landslide Deposits in San Mateo County (Brabb & Pampeyan, 1972), three relatively large landslides are mapped in the central portion of the property. According to the geologic map, the largest feature measures approximately 900 feet in length and 600 feet in width. The upper margin of this feature is located approximately 350 feet to the west (downhill) of Parrott Drive and extends down to Crystal Springs Road, crossing the southwest corner of Lot 4. The second mapped landslide is approximately 700 feet long and 500 feet wide and is located immediately south of the first landslide. In addition, smaller landslide features are mapped in the southern portion of the lot and at the northeast corner just off the property. The relevant portions of these maps are included as Figure A-4, San Mateo County Landslide Map and Figure A-5, San Mateo County Geotechnical Hazard Synthesis 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. These earthquakes occur on faults that are part of the San Andreas fault system, which extends for at least 700 miles along the California Coast and includes the San Andreas, San Gregorio, Hayward, and Calaveras faults. The San Andreas and San Gregorio faults are located approximately 1.1 and 8.3 miles southwest of the site, respectively. The Hayward and Calaveras faults are located approximately 17 and 25 miles northeast of the site, respectively.

Seismologic and geologic experts convened by the U. S. Geological Survey, California Geological Survey, and the Southern California Earthquake Center conclude that there is a 63 percent probability for at least one "large" earthquake of magnitude 6.7 or larger in the Bay Area before the year 2038. The northern portion of the San Andreas fault is estimated to have a 21 percent probability of producing a magnitude 6.7 or larger earthquake by the year 2038 (2007 WGCEP, 2008).

AERIAL PHOTOGRAPH REVIEW

Three sets of historical aerial photographs taken between 1943 and 1974 were reviewed at the U.S. Geologic Survey's library in Menlo Park to aid in evaluating the presence of geomorphic features that may be suggestive of landsliding. The site is readily identifiable in all of the photographs, based on the topography and the location of Parrott Drive, Crystal Springs Road, and Polhemus Road. Other than the development of the neighboring residential properties, there is very little change in the vicinity of the property during the period covered by the photographs. In the 1948 photographs, the streets are present but there is no other development in the vicinity of the property. By the time of the 1968 photographs, most of the homes along Parrott Drive are complete and the building pad on the property immediately north of Lot 1 appears to be graded. In addition, it appears that improvements were made to Parrott Drive and that additional fill was placed along the downhill side of the roadway. The residences to the north of Lot 1 and south of Lot 4 are present by the time of the 1974 photographs.

In the 1943 photographs, two large landslides are present in the central portion of the property, similar to mapping by Pampeyan (see Figure A-3). The landslides are characterized by broad arcuate topography extending from the downhill side of Parrott Drive down to Crystal Springs Road. The northernmost feature crosses the southwest portion of Lot 4, more than 150 feet southwest of the proposed residence (see Figure A-2). The ground surface within the limits of the landslides is generally hummocky with irregular medium to dense vegetation. A small debris flow appears to be located within the limits of the northern landslide. In addition, a debris flow is located uphill of the southern landslide and drops into the upper portion of the landslide feature. The landslide masses are confined by drainage swales extending down the margins of the features to Crystal Springs Road. In addition, a large debris flow-type landslide, also mapped by Pampeyan, is located in the southern portion of the property.

In the 1968 photographs, an access road is present on Lots 1 and 2. This road enters Lot 2 from Parrott Drive, extends across the uphill portion of Lot 1 and to the graded pad on the adjacent northern property. It appears that sometime between 1968 and 1974, a small landslide occurred along the downhill side of the access road along the boundary between Lots 1 and 2. A headscarp is present along the uphill margin of this arcuate feature in the 1974 photographs. No evidence of landsliding was observed immediately east of this feature, however, there is a tonal variation in the vegetation and the topography has a very subdued arcuate shape, suggesting that this area may be prone to shallow sliding.

The drainage swale in the lower portion of Lot 2 is densely vegetated. There is no conclusive evidence in the photographs to suggest that debris flows have occurred along this swale.

A deep drainage ravine extends from the southeast to northwest corners of Lot 4. This feature appears to be confined by a relatively resistant ridge to the south, and then by the northern margin of the large landslide on the slope below Lot 4. This feature is present in the 1943 and 1968 photographs and by the time of the 1974 photographs a storm drain culvert appears to have been constructed along the downhill side of Parrott Drive. The head of the swale appears to be larger in the latest photographs, and is presumably related to grading during construction of the storm drain culvert. The drainage ravine is densely vegetated and any evidence of landsliding or debris flows is obscured.

PREVIOUS GEOLOGIC & GEOTECHNICAL INVESTIGATIONS

Site Characteristics, Inc. (SCI) conducted a geotechnical investigation on the property, dated July 1983, to address three proposed single family residences along Crystal Springs Road in the northwest lower portion of the property. SCI performed a site reconnaissance and mapped a small active landslide below the graded access road on Lot 1 and a relatively small active landslide above the storm drain culvert on Lot 4. In addition, they mapped several shallow features on the slope below the proposed lots. As part of the investigation, SCI excavated and logged seven test pits in the area of the proposed improvements. In general, the test pits exposed variable amounts of colluvium ranging from 1 to 12 feet in thickness underlain by bedrock materials associated with the Franciscan Complex. SCI indicated that there was no evidence of recent slope instability or soil creep in the proposed building site areas, with the exception of Building Site 1, located at the base of the drainage swale along the northern margin of the large mapped landslide. SCI recommended supporting the residences on 12-inch diameter piers, extending at least 8 feet into competent materials. In addition, SCI recommended constructing an earth flow deflection wall above Building Site 1.

Subsequently, William Cotton and Associates (WCA) performed a supplemental geotechnical analysis and presented the results in a report dated April 20, 1984. As part of their investigation, WCA performed a site reconnaissance and mapping, aerial photograph review, and shallow and deep slope stability analyses. WCA observed several small earth slumps on the property, including the active landslide on Lots 1 and 2, but indicated that there was no evidence of debris flows on the property. WCA noted two areas on the proposed lots that may be potentially susceptible to shallow translational sliding and debris flows, including the head of the drainage ravine on Lot 4 and the eastern portion of Lot 3, and an area on Lots 1 and 2 extending from Parrott Drive to the drainage swale below and encompassing the active landslide.

Based on a review of SCI's subsurface exploration data, WCA concluded that the landslide material is composed of a relatively large block, or blocks, of intact bedrock materials and is likely a rock slump that occurred several thousands of years ago. WCA performed a slope stability analysis through the large mapped landslide and reported a factor of safety of 2.5 for static conditions and 1.1 for seismic conditions. WCA concluded that the proposed building site is likely situated on top of an ancient landslide, but based on the slope stability analysis the landslide deposit should remain stable. WCA recommended the construction of a deflection wall at the northeast corner of the proposed residence and improvement of the drainage channel in that area.

In 2007, Bay Area Geotechnical Group (BAGG) performed a geotechnical and engineering geologic investigation for a proposed 20-lot residential subdivision of the subject property. The results of the investigation were presented in a report dated December 20, 2007. As part of the investigation, BAGG excavated six relatively deep borings within the landslide areas and nine additional borings on the remaining portions of the property, and performed laboratory testing on samples, including triaxial shear and direct shear testing. Three of the borings were advanced on Lot 1, one of which is located within the limits of the presumably active landslide, and two were located on the slope below Lots 2 and 4. The locations of these borings are shown on Figure A-2 and the boring logs are included in this report as Appendix D.

In general, BAGG's borings encountered approximately 5 feet of colluvial soil underlain by bedrock associated with the Franciscan Complex. Boring EB-1, located immediately above the head scarp of the active landslide on Lot 1, encountered approximately 4 feet of colluvial soil consisting of sandy lean clay and clayey sand. Mélange bedrock was encountered below the colluvium and persisted to the bottom of the boring at a depth of 24 feet, where effective drilling refusal was encountered. Boring EB-10, located downslope of EB-1 and within the limits of the active landslide, encountered approximately 6.5 feet of sandy clay colluvium underlain by mélange bedrock. Sandstone was encountered at a depth of 10 feet and persisted to the bottom of the boring at a depth of 15.5 feet. Borings EB-2 and EB-3, located below Lot 2 and in the western (downhill) portion of Lot 1, respectively, encountered approximately 7 to 8 feet of sandy lean clay. In Boring EB-2 the clay is underlain by a 10-foot thick layer of clayey sand with fine gravel and in Boring EB-3 the lean clay is underlain by an approximately 4-foot thick layer of fat clay with gravel. Mélange was encountered below the colluvium at a depth of 17.5 and 12 feet, respectively, and the borings were terminated at depths of approximately 21.5 and 19 feet. Boring ERWB-2, located downhill from Lot 4, encountered approximately 5.5 feet of sandy lean clay underlain by mélange that persisted to the bottom of the boring at a depth of 97.5 feet. The mélange generally consisted of Franciscan shale and sandstone fragments in a clayey matrix.

BAGG performed slope stability analyses and Newmark analyses through the two large landslide areas in the central portion of the property. The stability analyses utilized Bishop's simplified method to evaluate a circular failure surface. Strength values used in the analysis were obtained by laboratory testing of samples from the exploratory borings. In general, the softer materials were chosen to perform shear strength testing due to the impracticality of obtaining undisturbed samples of the harder bedrock material. BAGG indicated that bedrock samples obtained across the site varied from minimal to up to 60 percent hard rock in a clayey matrix and borings within the large landslide areas encountered bedrock consisting of 22 to 31 percent hard rock. BAGG assumed that a higher percentage of blocks would add strength to the matrix since the failure surface would have to distort around the blocks and increased the friction angle by up to 7½ degrees, based on the percentage of hard rock, to more realistically represent the strength of the bedrock. Without increasing the friction of the matrix, the slope stability analysis yielded factors of safety against sliding in excess of 1.68 under dry conditions and 1.01 under saturated conditions. In general, factors of safety greater than 1.0 indicate a stable condition, while factors of safety less than 1.0 indicate an unstable condition. The critical failure surface extends up to 80 to 100 feet below the hillside. BAGG concluded that it was unlikely that rain could saturate the slope to this depth, but indicated that there is a potential for shallow soil slumps to occur. Based on their Newmark analyses, BAGG concluded that the two mapped slide areas could move from 6 to 18 inches. Based on their assessment, BAGG concluded that there was a significant risk of seismic slope instability within the two mapped slide areas; however, development of the remaining portions of the site where there is no evidence of deep-seated slope movement is feasible from a geotechnical engineering standpoint.

Based on their investigation, BAGG recommended supporting the proposed residences on drilled piers at least 15 feet in depth and extend a minimum of 10 feet into firm native soils and/or bedrock.

SITE EXPLORATION AND RECONNAISSANCE

Exploration Program

An initial site visit was performed by our principal geotechnical engineer on October 23, 2013. Subsequently, on December 17 and 20, 2013 our senior staff geologist visited the site to perform a site reconnaissance and engineering geologic mapping. Our subsurface field investigation was performed on December 20 and 23, 2013 and included the excavation and logging of six exploratory borings to depths ranging from 18 to 40 feet at the locations shown on Figure A-2. Two borings were located above and within the active landslide on Lot 1, and one boring was advanced on each of the four lots in the vicinity of the proposed building sites. The boring locations were approximately determined by measuring distance and bearing from known points on the supplied site plan using a tape measure and compass, and should be considered accurate only to the degree implied by the mapping technique used.

The borings were advanced using a track-mounted CME-55 drill rig equipped with 6-inch diameter continuous flight augers. Intermittent soil samples were collected with split-spoon samplers that were driven with a 140-pound hammer repeatedly dropped from a height of 30 inches using a pneumatic hammer. The number of hammer blows required to drive the samplers were recorded in 6-inch increments for the length of the 18-inch long sampler barrels. The associated blow count data, which is the sum of the second and third 6-inch increment, is presented on the boring logs as sampling resistance in blows per foot. The blow counts for the 3-inch and 2.5-inch samplers have been standardized to Standard Penetration Test blow counts for sampler size; however, they have not been adjusted for other factors, such as hammer efficiency. Logs of the borings are presented in Appendix B as Figures B-1 through B-6. Also included in Appendix B are Figure B-7, Key to Boring Logs; Figure B-8, Unified Soil Classification System; and Figure B-9, Key to Bedrock Descriptions.

Our staff geologist logged the borings in general accordance with the Unified Soil Classification System and Key to Bedrock Descriptions. The boring logs show our interpretation of the subsurface conditions at the location and on the 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 materials; however, the transitions may be gradual. Samples recovered from the borings were reviewed by our senior staff geologist and principal geotechnical engineer.

Site Description

The irregular-shaped, approximately 60.3-acre property measures approximately 3,500 feet wide along Crystal Springs Road and Polhemus Road, and up to 1,300 feet deep. The site is bounded to the west by Crystal Springs Road and Polhemus Road, to the east by Parrott Drive, and developed and undeveloped residential properties on all other sides. The property is situated on the western flank of a south- to southeast-trending ridgeline. San Mateo Creek and Polhemus Creek run along the base of the ridgeline and converge near the southern corner of the property. The site topography is dominated by a series of westerly-trending spur ridges and intervening drainage swales. The natural ground surface across the property is generally steep with gradients varying from 2:1 to 3:1 (horizontal to vertical) and moderately sloping across portions of the mapped slides with gradients ranging from approximately 4:1 to 5:1. Locally steeper than 2:1 slopes are present, however. Maximum vertical relief across the property is approximately 400 feet from the base of the hillside near the northwest corner of the property up to the upper, eastern property line (see Figures A-1 & A-2).

The proposed 2.5-acre lots are located in the northeast corner of the property, along Parrott Drive. Lot 1 is located on the southern flank of a west-trending spur ridge. The ground surface in the upper portion of the property slopes moderately toward the southwest with gradients of approximately 3:1 to 4:1 and slopes steeply toward the west in the downhill portion of the property with gradients of approximately 2:1 to 3:1. A wedge of fill up to approximately 25 feet tall is located along the downhill side of Parrott Drive and slopes steeply with a gradient of approximately 2:1 (see Figure A-2 and Figure A-6, Geologic Cross-Section A-A'). In addition, it appears that a minor amount fill was placed along the northern property boundary during grading for the adjacent property to the north.

An active landslide is located along the property boundary between Lots 1 and 2. This feature measures up to approximately 160 feet in width and 200 feet in length. An approximately 4- to 5-foot tall headscarp exposing sandy silt is located along the uphill margin of the feature and the ground surface within the slide is very hummocky and saturated. The ground surface within the limits of the active landslide range from approximately 4:1 across the uphill portion of the feature to approximately 2:1 across the downhill portion (see Figure A-2 and Figure A-7, Geologic Cross-Section B-B'). Additional discussion of the landsliding on the proposed lots is included in the Landsliding section below. The vegetation within the landslide generally consists of pompous grass and poison oak. The remaining portions of Lot 1 are vegetated with native grasses, shrubs, and some scattered trees.

Lot 2 is situated across a subdued west-trending spur ridge and a drainage swale. The active landslide discussed above is located within the drainage swale along the northern property boundary. The ground surface across the ridgeline slopes steeply toward the west with gradients of approximately 2.5:1 (see Figure A-2 and Figure A-8, Geologic Cross-Section C-C'). A wedge of fill up to approximately 12 feet tall is located along the downhill side of Parrott Drive and slopes steeply with a gradient of approximately 2:1. An access road extends from Parrott Drive at the southeast corner of the property to the head of the landslide near the northern property boundary. It appears that a thin wedge of fill was placed along the downhill side of the access road during grading. In general, the ridgeline is vegetated with tall grasses and scattered trees and shrubs. In addition, the head of a debris flow is located in the drainage swale at the westernmost downslope end of the property (see figure A-2). The drainage swale and adjacent slopes are densely vegetated with poison oak, trees, and tall pompous grass.

Lot 3 is located across the crest and southern flank of a west-trending spur ridge. The ground surface across the ridgeline slopes steeply toward the west with gradients of approximately 2:1 to 3:1. Along the southern flank, the ground surface is irregular and suggestive of shallow soil creep with very steep slopes ranging from 1.5:1 to 2:1 (see Figure A-2 and Figure A-9, Geologic Cross-Section D-D'). A thin wedge of fill is located along the downhill side of Parrott Drive. In general, the ridgeline is vegetated with grasses, scattered trees and shrubs. The southern flank is densely vegetated with trees and associated underbrush.

Lot 4 is situated across a drainage ravine confined between two west-trending spur ridges. A storm drain culvert is located at the southeast corner of the property and the drainage ravine extends to the northwest corner of the lot. The slopes around the culvert are very steep to precipitous and the culvert is obscured by an abundant growth of poison oak. The drainage ravine is approximately 5 to 8 feet deep and sandstone and sheared rock exposures were observed along sections of the drainage ravine. The ravine was dry at the time of our site reconnaissance. The ridgeline to the south of the ravine appears to be relatively resistant to erosion and is a prominent feature compared to the spur ridges on Lots 1 through 3. The ground surface across the ridgeline slopes steeply to the west with gradients of approximately 2:1 to 1.5:1 (see Figure A-2 and Figure A-10, Geologic Cross-Section E-E'). In the southwest corner of the property, the ridgeline is truncated by a large presumably ancient landslide. The ground surface within the slide area is irregular and the slopes range from 3:1 to 10:1. The slopes across the southern flank in the northeast portion of the property slope steeply toward the drainage ravine with slopes ranging from 1.5:1 to 2:1. In general, the topography along either side of the drainage ravine is suggestive of shallow landsliding and/or debris flows.

Landsliding

As discussed above, a large presumably ancient landslide appears to extend from the downhill side of Parrott Drive across the southwest corner of Lot 4 and to Crystal Springs Road. This feature is approximately 500 feet in width and 1,200 feet in length and, based on our aerial photograph review, appears to have occurred prior to development of the area. This feature crosses the southwest corner of Lot 4; however, it is located on the opposite site of a resistant ridgeline more than 150 feet downslope from the proposed building site. Further discussion of the slope stability analysis performed by BAGG is included in the Previous Geologic & Geotechnical Investigations section above.

As noted above, BAGG mapped an older landslide in the upper portion of Lot 1. One exploratory boring, Boring B-4, was advanced in the center of this feature and encountered bedrock at a depth of 18 inches. Based on our review of aerial photographs, our site reconnaissance, and subsurface exploration, in our opinion there appears to be no strong evidence to support the presence of this feature. We note that this feature was also not identified by SCI or WCA.

An active relatively shallow landslide is located along the property boundary between Lots 1 and 2. This feature was initially mapped by SCI in 1983. Based on our review of aerial photographs and our site reconnaissance, it appears that this feature is larger than initially mapped by SCI. It appears that a 40-foot wide failure appears to have occurred along the downhill side of the graded access road on Lot 2, widening the area of the active landslide. This active landslide was absent from the 1943 and 1968 aerial photographs, but appeared in the latest photographs following construction of the graded access road. In our opinion, grading associated with construction of this road is likely the main probable cause of the landslide. Based on our subsurface exploration, it appears that this active landslide is less than 10 feet thick in depth.

A debris flow was initially mapped by SCI along the drainage swale below Lot 2; however, this was refuted by WCA. This feature was subsequently mapped by BAGG, with the upper limit extending approximately 60 feet onto Lot 2. Based on our site reconnaissance and aerial photograph review, a significant amount of erosion has occurred at the head of this feature; however, very dense vegetation obscures the topography. In our opinion, if this feature were to move, it is located sufficiently away from the proposed building site that it would have little to no impact on the proposed improvements.

For reference proposes, debris flows, in general, commonly involve upon saturation, the rapid removal of relatively shallow thicknesses of granular soil over a firm contact such as bedrock. The saturated soil is transported, in semi-liquid form, from the upper regions of the debris flow causing a scar to form in this area, and the resulting debris deposited along a relatively narrow band or “pathway” to a termination point below. Depending on many factors including the size, steepness of slope, topography, soil type, etc., structures located immediately below slopes potentially prone to debris flow movement may be in an immediate threat of both structural damage and/or life safety. Mitigation measures such as debris fences, impact walls, or deflection walls are commonly recommended to reduce this potential threat.

Shallow debris flows also appear to have occurred along the drainage ravine on Lot 4, as evidenced by evacuated head scarps along the northern side of the channel. It appears that these features are related to very steep to precipitous slopes along either side of the ravine in addition to heavy precipitation during past rainfall events. The deeply incised drainage ravine suggests that a large volume of water flows through the culvert during the rainy season. A relatively small active landslide was mapped above the culvert by CSI; however, while evidence of erosion was observed around the culvert, we did not observe any evidence of an active landslide.

We note that due to the dense vegetation and steep slope conditions, only portions of the site was accessed by our firm during our site reconnaissance and mapping phase. Therefore, there could be other shallow slope failures on the property that were not documented by our firm.

Subsurface

In general, the exploratory borings encountered variable amounts of fill and colluvium underlain by sandstone and sheared rock from the surface to the full depth explored of 40 feet. The boring locations are presented on Figure A-2, Partial Site Plan & Engineering Geologic Map and detailed logs of each boring are presented in Appendix B. A general description of the subsurface conditions and the approximate location of each exploratory boring are described hereunder.

Borings B-1 and B-2, located along the uphill side of the proposed building sites on Lot 3 and 4, respectively, encountered approximately 4 to 6.5 feet of stiff to hard sandy silt fill underlain by approximately 2.5 to 4.5 feet of colluvial soil consisting of very stiff to hard sandy silt. Sandstone bedrock was encountered below the colluvium at a depth of 6.5 and 11 feet, respectively, and persisted to a depth of 33 and 28.5 feet. The sandstone bedrock is underlain by sheared rock that persisted to the bottom of the borings at a depth of 40 feet.

Boring B-3, located along the uphill side of the proposed building site on Lot 2, encountered approximately 5 feet of colluvium consisting of stiff to very stiff sandy silt and silty clay. Sandstone bedrock was encountered at a depth of 5 feet and persisted to the bottom of the boring at a depth of 35 feet.

Boring B-4, located along the downhill side of the proposed building site on Lot 1, encountered approximately 18 inches of colluvial soil consisting of stiff sandy silt underlain by sandstone bedrock. The sandstone bedrock persisted to a depth of 30 feet and was, in turn, underlain by sheared rock. The sheared rock persisted to the bottom of the boring at a depth of 38.6 feet.

Boring B-5, located immediately upslope of the active landslide on Lot 1, encountered approximately 5 feet of very stiff sandy silt colluvium underlain by sandstone bedrock. Sheared rock was encountered at a depth of 13.5 feet and persisted to the bottom of the boring at a depth of 22.7 feet.

Boring B-6, located within the limits of the active landslide on Lot 1, encountered approximately 9.5 feet of active landslide deposits consisting of medium stiff to very stiff sandy silt. Sheared rock was encountered below the landslide deposits and persisted to a depth of 18.1 feet.

Laboratory Testing

Atterberg Limits testing was performed on two samples of the surficial soil from Boring B-3 at a depth of 1.5 to 3 feet and Boring B-6 at a depth of 3 to 4.5 feet to evaluate the expansion potential of this material. The testing yielded a liquid limit of 41 and 29 percent, respectively, and a plasticity index of 22 and 11 indicating that this material has a low to moderate potential for expansion (see Figure C-1, Liquid & Plastic Limits Test Report).

Groundwater

Groundwater was encountered in Borings B-4, B-5 and B-6 at the time of drilling at a depth of approximately 28, 18 and 6.5 feet, respectively. Free groundwater was not encountered in any of the other 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:

- Special Publication 117A: 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 a pseudo-static analysis to evaluate the overall seismic deep-seated stability of Lots 1 through 4 in the vicinity of the proposed building sites along Cross-Sections A-A', C-C', D-D', and E-E' (see Figures A-6, A-8, A-9 and A-10) and of the active landslide on Lots 1 and 2 along Cross Section B-B' (see Figure A-7). The analyses were performed using the computer program Slide 5.0, 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 greater than 1.0 indicate a stable condition, while factors of safety less than

1.0 indicate an unstable condition. The pseudo-static analyses utilized a seismic coefficient (k) of approximately 0.27, determined in general accordance with Special Publication 117A for a threshold displacement of 15 centimeters using a peak ground acceleration of 0.57 obtained from the interactive U.S. Geological Survey Earthquake Hazards Program web site (U.S. Geological Survey, 2008). As a state seismic hazard zones report is not available for the San Mateo quadrangle, we utilized a magnitude of 7.9 taken from the Seismic Hazard Zone Report for the adjacent Palo Alto Quadrangle (California Geological Survey, 2006).

Subsurface conditions were approximated based on local geologic maps, our site reconnaissance, and subsurface data collected by our firm and by BAGG. Specifically, the proposed lots are blanketed by colluvial soil underlain by Franciscan Complex bedrock. Strength values used in the analysis were obtained from Table 2.1 of the Seismic Hazard Zone Report for the Palo Alto Quadrangle in conjunction with laboratory testing on samples from our subsurface borings and strength values reported by BAGG.

For the native soils and shallow, softer bedrock materials, BAGG estimated cohesion values of 2,540 and 935 pounds per square foot (psf) and a phi value of 36 and 40 degrees. The Seismic Hazard Zone Report indicates that strength values for Holocene aged deposits range from 500 to 700 psf with a phi value of 21 to 26 degrees. Our analysis utilized much more conservative values, including a phi value of 29 degrees and a cohesion value of 350 psf for the surficial soils. We also utilized the same strength parameters for the active landslide present on Lots 1 & 2 with the assumption that the upper portions of this feature will be stabilized (see recommendations below).

According to the referenced Seismic Hazard Zone Report, the Franciscan Complex bedrock materials have a cohesion value of 650 psf and a phi value of 29 degrees. Direct shear testing on a sample of the bedrock obtained from Boring B-6 between the depths of 9 to 10.5 feet yielded a phi value of 15.6 degrees and a cohesion value of 700 psf (see Figure C-2, Direct Shear Test Chart for Boring B-7 9-10.5 Feet BGS). In our opinion, the results of the direct shear testing are likely low due to disturbance of the samples during drilling. In addition, we note that strength testing by BAGG yielded much higher values which accounts for increase in strength from having to shear or distort around hard bedrock blocks. Therefore, our analysis utilized a cohesion value of 850 and a phi value of 29 degrees to more conservatively represent the strength of the bedrock. Based on our subsurface exploration and because of the elevated topographic position of the site, it is our opinion that the potential for high groundwater at the site is low. Therefore, we did not include a high groundwater level as part of the analysis.

The stability analyses yielded critical failure surfaces extending through the bedrock with a calculated factor of safety ranging from 1.00 to 1.39. The results of the slope stability analyses are included as Figures A-11 through A-15. 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 less than 1.00 will not necessarily fail but the probability of failure will be greater than 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.

CONCLUSIONS & RECOMMENDATIONS

Based on the results of our investigation, it is our opinion that the proposed residential subdivision is feasible from an engineering geologic and geotechnical perspective. In our opinion, the primary constraints to the project include the potential for shallow landsliding and/or debris flows developing along the steeper portions of the property, consolidation, creep, and/or shallow landsliding of the undocumented fill along the downhill side of Parrott Drive, and the potential for strong to very strong ground shaking during a moderate to large earthquake on the nearby San Andreas fault or one of the other nearby active faults.

In general, the proposed residences will be located in the uphill portion of the lots, adjacent to Parrott Drive. We understand that the residence on Lot 1 may alternatively be shifted downhill and accessed by a shared driveway extending from Lot 2. In our opinion, the proposed building pads are feasible; however, due to the logistics of building a structure over a storm drain culvert, we recommend that the residence building site on Lot 4 be shifted to the north, away from the storm drain culvert.

Based on our investigation, the proposed improvement areas are blanketed by variable amounts of fill and colluvium underlain by sandstone and sheared rock bedrock. In particular, a substantial wedge of fill is located along the downhill side of Parrott Drive. We assume that this fill slope was not placed as a properly engineered fill with keyway, benches and possibly subdrainage. Therefore, in our opinion, this material will be subject to future consolidation, downhill creep, and possible shallow landsliding and should not be relied on for support of the proposed improvements. In our opinion, the proposed residences and associated retaining walls should be supported on drilled pier foundations extending through the fill and colluvium and gaining support in the underlying bedrock.

We briefly reviewed the potential for geologic hazards to impact the site, considering the geologic setting and our observations during our site reconnaissance. The results of our review are presented below:

- Landsliding – Based on our investigation, we did not observe any evidence of active landsliding in the immediate area of the proposed residence on Lot 3. However, as noted above, an active landslide is located along the boundary between Lots 1 and 2, approximately 50 feet from the currently proposed residence on Lot 1 and 10 feet from the residence on Lot 2. This feature appears to be directly related to cuts and fills associated with past grading of the access road. Based on our field reconnaissance, this feature also appears to be relatively shallow and does not extend up into the footprint of the building site. Given the location of this feature with respect to the locations of the proposed structures, in our opinion, reactivation of this feature could impact the proposed improvements. Therefore, we recommend mitigating this landslide as discussed in the recommendations section below.

  In addition, a relatively shallow debris flow is located in the drainage swale at the lower end of Lot 2. This feature is located more than 200 feet from the proposed building site and appears to be confined to the drainage swale. In our opinion, if this feature were to reactivate it would have little to no impact on the proposed improvements.

  The evacuated headscarp of a debris flow is located along the downhill side of the proposed residence on Lot 4. This feature appears to be the result of very steep slopes in combination with granular soil type and heavy precipitation during past rainfall events. In our opinion, it is likely that new debris flows and shallow earth slumps will occur along the drainage ravine; however, given that the proposed building site is located upslope of the drainage ravine, in our opinion, future movement of these features should not have a direct impact on the proposed improvements provided that they are design in accordance with the recommendations of this report. As noted above, we also recommend shifting the building site on Lot 4 to the north and away from the drainage culvert.

  In our opinion, given the presence of similar shallow landslide features on the property and steep slope conditions, future movement of these active landslides/debris flows as well as generation of new shallow earth slumps and/or debris flows is likely. In our opinion, future movement of these features should not have a direct impact on the proposed improvements provided that they are designed in accordance with the recommendations of this report.

  Based on our investigation, the slopes on the proposed lots generally appear to be underlain by resistant bedrock and it is our opinion that the potential for a deep-seated landslide emanating from these slopes is low. As noted above, a mapped presumably ancient landslide crosses the southwest corner of Lot 4. Based on the slope stability analyses performed by BAGG, it appears that there is a potential risk of seismic slope instability within the mapped slide areas. However, BAGG indicated that the bedrock strength is higher in areas of the property where there is no evidence of deep-seated slope movement and concluded that development outside of the mapped slide areas is feasible. In our opinion in the unlikely event this landslide feature were to move during a large seismic event, given its proximity from the proposed house site coupled with the reasoning (based on review of past performance of large landslide complexes after large earthquake events such as what occurred after the Loma Prieta Earthquake in the Santa Cruz Mountains) that the feature would likely not fully mobilize but may shift downslope to some degree along its boundaries, in our opinion such anticipated movement would not significantly impact the global stability of the proposed house site on Lot 4.

We note that based on our investigation, it is our opinion that there is a moderate risk for continued erosion and slight retrograde of the active landslide and debris flows on the proposed lots. However, the potential for landsliding significantly impacting the present locations of the proposed building sites is relatively low provided the recommendations in this report are carefully followed and incorporated into the design of the structures. In addition, given the steep slopes across the proposed building sites and the presence of relatively thick surficial colluvial soil, the occurrence of a new shallow landslide in this area cannot be excluded. A new, relatively shallow landslide in the colluvium could be triggered by excessive precipitation and/or strong ground shaking associated with an earthquake. In our opinion, a landslide of this nature should not constitute a significant hazard to the proposed improvements provided that they are designed and constructed in accordance with the recommendations presented in this report. However, there is a potential risk for debris flow activity that could impact property and structures in the lower portions of the site. Evaluation of this potential hazard was beyond the scope of our investigation. However, as discussed in the previous consultants' reports, typical mitigation involves installation of debris impact/deflection walls to impede direct impact on structures.

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

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

- Ground Shaking – As noted in the Seismicity section above, moderate to large earthquakes are probable along several active faults in the greater Bay Area. Therefore, strong to violent ground shaking should be expected in the area during the design-life of the proposed improvements. In our opinion, the improvements should be designed in accordance with the current earthquake resistant standards, including the 2013 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 of structures.

- Differential Compaction – During moderate and large earthquakes, soft or loose, natural or fill soils can become densified and settle, often unevenly across a site. In our opinion that there is a moderate potential for differential compaction of the fill material located in the upper portion of Lots 1 through 4. However, if the proposed improvements are constructed in accordance with the recommendations of this report on foundations sufficiently embedded in competent materials below the fill, in our opinion the potential for damage from differential compaction can be significantly reduced.

PRELIMINARY GEOTECHNICAL RECOMMENDATIONS

The geotechnical recommendations provided below are based on highly tentative plans and are for general planning purposes. Please note that our preliminary opinions and conclusions may change once the details of the proposed construction have been developed and may require supplemental investigative work.

Due to the steep slopes and the presence of undocumented fill and colluvial soil, we recommend that the proposed residences be supported on drilled piers extending adequately into bedrock. If basements will be included in the design, the basement floors should be designed as a structural slab supported on piers. The building contractor should take appropriate precautions to shore the proposed basement excavations. The design and construction of any temporary shoring or dewatering is the responsibility of the building contractor.

It is anticipated that retaining walls will be utilized along the driveway and to accommodate grade changes across the building sites. Retaining walls should be supported on drilled piers embedded into bedrock. If desired, grouted tieback anchors may be utilized to help resist active loads on the piers. Although plans are highly tentative, we anticipate that a driveway may be constructed along the uphill side of the active landslide on Lots 1 and 2. In addition, the proposed building sites on Lots 1 and 2 are approximately 50 and 10 feet, respectively, from the landslide feature. To mitigate the potential for reactivation of the active landslide on Lots 1 and 2 to impact the proposed improvements, we recommend that a retaining wall be constructed along the uphill margin of the slide feature. The retaining wall should be installed prior to grading for the driveway and residence improvements to reduce the potential for the grading to trigger a slope failure. As an alternative, the landslide mitigation may include removing portions of the landslide debris and replacing it as a keyed and benched engineered fill supported on the underlying bedrock in addition to constructing retaining walls to stabilize the slope above. We note that the lower portion of the shallow landslide will remain and therefore subject to continued slope movement. Such slope movement will in our opinion not significantly impact the planned improvements (being located uphill) provided the recommendations in this report are carefully followed.

Slabs-on-grade may be used for driveways, patios, walkways, and garage floors; however, it should be anticipated that some degree of differential movement could occur between these slabs and adjacent pier-supported structures. To significantly minimize the movement potential of concrete slabs, the more critical exterior slabs can alternatively be constructed as structural slabs supported on piers. Alternatively, to minimize repair costs associated with heave and/or settlement cracking of exterior concrete slabs-on-grade, we suggest the use of sand-set pavers, which can be constructed with a thinner section of underlayment and relatively low costs associated with re-leveling of heave-related movement. Detailed recommendations are presented in the following sections of this report.

2013 CBC EARTHQUAKE DESIGN PARAMETERS

Site-specific earthquake design parameters have been developed based on the procedures described in Chapter 16, Section 1613 of the 2013 California Building Code (California Building Standards Commission, 2013). These procedures utilize State standardized spectral acceleration values for maximum considered earthquake ground motion taking into account historical seismicity, available paleoseismic data, and activity rates along known fault traces, as well as site-specified soil and bedrock response characteristics. Contour maps of Class B bedrock horizontal spectral acceleration values for the State of California are included as figures in Chapter 16 of the 2013 CBC, representing both short (0.2 seconds) and long (1.0 second) periods of spectral response and taking into account 5 percent of critical damping. The U.S. Geological Survey (2013) has prepared an online seismic design value application tool, based on the 2010 ASCE with a July 2013 CBC errata, for public use, that allows for site-specific adjustments of these acceleration values for different subsurface conditions, which are defined by site classes. Given representative latitude of 37.539 and longitude of -122.347 in accordance with guidelines presented in the 2013 CBC, the following seismic design parameters will apply for this site:

- 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= 2.166 (Site Class B)
- Mapped Spectral Accelerations for a 1-second Period: S_1=1.216 (Site Class B)
- Design Spectral Accelerations for 0.2 second Period: S_{DS}=1.444 (Site Class C)
- Design Spectral Accelerations for a 1-second Period: S_{D1}=1.054 (Site Class C)

FOUNDATIONS

Drilled Piers

Given the anticipated steep slope conditions and presence of existing undocumented fill, we recommend drilled piers for the residences be at least 16 inches in diameter, at least 20 feet in depth from bottom of grade beam or slab elevation, and should be embedded a minimum of 12 feet into the bedrock. Piers for site retaining walls (those walls that are not structurally tied to any structures) should extend at least 8 feet into the bedrock or to a depth equal to the height of the retaining wall plus the thickness of non-supportive soil in the upper portion of the pier column. In addition, if piers are used for structural supported patio slabs, the piers should extend at least 8 feet into bedrock or to a depth equal to the thickness of non-supportive soil overlying the bedrock. 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 the pier design depths to be greater. In general, drilled piers should be spaced no closer than approximately three pier diameters, center-to-center.

Drilled piers 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 the bedrock with a one-third increase allowed for transient loads, including wind and seismic forces. Any portion of the piers in the fill, colluvium, or landslide deposits, and any point-bearing resistance should be neglected for support of vertical loads. For piers adjacent steep slopes, supportive material (bedrock) should start a minimum horizontal distance of 10-feet from the daylight of slope. The depth however, may be modified by our representative during construction, especially if very dense bedrock areas are encountered.

To resist lateral creep of near surface soils, we recommend that piers be designed to resist an active soil pressure equal to an equivalent fluid weight of 85 pounds per cubic foot (pcf), acting over 2-pier diameters in the downhill direction over the depth of the piers embedded in the non-supportive soil. The depth of the active loads will vary slightly at individual pier locations. Based on our subsurface investigation, we anticipate active soil depths up to approximately 11 feet and less than a foot where grading removes the existing surficial colluvium. To avoid over-design and to facilitate pier construction, we suggest that the project structural engineer develop a pier table that provides required pier embedment depth into supportive bedrock based on the depth of overlying non-supportive material from 0 to 14 feet.

The active loads from soil creep and other lateral loads may be resisted by passive earth pressure based upon an equivalent fluid pressure of 425 pounds per cubic foot (pcf), acting on 2 times the projected area for the depth of the pier in the bedrock. Any passive resistance corresponding to the creep zone described above should be neglected.

To create a relatively rigid structure, we recommend that piers for the residences be interconnected with grade beams. Grade beams for the site retaining walls should be provided based on structural requirements. Perimeter grade beams for the residences 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 structures.

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

If grading for the building pads exposes highly expansive soil, the tops of piers should be prevented from "mushrooming" to minimize the potential for uplift on the piers. This may be accomplished by placing Sonotubes within the upper 2 feet of the pier excavations prior to placement of concrete or by other construction methods. In addition, grade beams embedded in highly expansive soil should be formed over 2-inch thick cardboard void forms, such as manufactured by SureVoid, to minimize the potential for uplift on the grade beams.

Based on our engineering judgment, thirty-year differential foundation movement due to static loads is not expected to exceed approximately ¾ -inch across any 20-foot span of the pier-supported residence and garage.

Grouted Tieback Anchors

If desired, grouted tieback anchors may be incorporated into the planned building and site retaining wall foundation design to help resist lateral loads on the new piers. The current design practice considers tieback capacity as developed in friction along a portion of the tieback length embedded into competent bedrock (bonded length). On a preliminary basis, tieback anchors should be designed to resist dead plus live loads using an allowable skin friction value of 850 psf (bond between bedrock and grout) for the bonded length of the anchor embedded in competent bedrock with a one-third increase allowed for transient loads, including wind and seismic forces. This bond strength should be confirmed in the field during the initial stages of construction with proof and performance load testing.

To maintain good grout retention, the tiebacks should be drilled with an angle of declination of at least roughly 15 degrees from horizontal. The bonded (anchor) length of the tiebacks should be established by the structural engineer based on the recommended allowable bond strength provided above. The unbonded length should be corrosion-protected with a grease-filled tube, a heat-shrinkage sleeve or with other approved methods.

The tiebacks should be proof-tested, performance-tested and creep-tested in accordance with general guidelines of the industry. The drilling and testing of tiebacks should be observed by a representative of Murray Engineers, Inc., to establish that the minimum depths and recommended bond strengths are achieved.

The anchor depths recommended above may require adjustment if differing conditions are encountered during drilling. While we expect that moderate sized drilling equipment can obtain the required depths, the tieback contractor should carefully review the boring logs in our report and should consider the potential for caving of some of the more granular soils encountered at the site.

BASEMENT & SITE RETAINING WALLS

It is anticipated that retaining walls will be used for the new residence basements (if constructed), to stabilize the slope above the active landslide, and to accommodate site grade changes. Basement and site retaining walls should be supported on foundations designed in accordance with the recommendations provided above. Damp proofing or waterproofing of walls should be included in areas where wall moisture would be undesirable, such as where wall finishes could be impacted by concrete moisture. The project architect or a waterproofing consultant should provide detailed recommendations for waterproofing or damp proofing, as necessary.

Lateral Earth Pressures

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.

In accordance with the 2013 CBC, where applicable, retaining walls should also be designed to resist lateral earth pressure from seismic loading, as deemed necessary by the project structural engineer. 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. The allowable passive pressures provided for retaining wall foundations may be increased by one-third for short-term seismic forces.

Where backfill behind the wall will be sloping upward from the wall, we recommend that the equivalent fluid pressures given above be increased by 3 pcf for each 4-degree increase in slope inclination. For sloping conditions steeper than 2:1, we should review the proposed design when it is available and provide specific lateral pressure recommendations upon completion of our review.

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. 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 a thin layer of crushed rock at the base of the 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 12 inches of the finished grade and laterally at least 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 12 inches in loose thickness. The upper approximate 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 basement drainage recommendations are presented schematically on Figure A-16, Basement Subdrain System Alternative A.

The subdrain pipes should be sloped at a minimum of 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.

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 18 inches below finish grade to the base of the retaining wall. A 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-17, Basement Subdrain System Alternative B.

Backfill

Backfill placed behind site retaining walls should be compacted in accordance with the Compaction specifications given in this report, using light compaction equipment. If heavy compaction equipment is used, the walls should be temporarily braced.

CONCRETE SLABS

Because of the steep slopes, we recommend that the basement floors of the residences (if constructed) and preferably the at-grade garage floors be designed and constructed as structural slabs supported on drilled pier foundations designed in accordance with the previous sections. Slabs-on-grade may be used for the driveway, patios, walkways, and possibly the garage floor; however, it should be anticipated that some degree of differential movement could occur between these slabs and adjacent pier-supported improvements. To significantly minimize the movement potential of concrete slabs, the more critical exterior slabs can alternatively be constructed as structural slabs supported on drilled piers. Alternatively, to minimize repair costs associated with heaving and cracking of exterior concrete slabs-on-grade, we suggest the use of sand-set pavers, which can be constructed with a thinner section of underlayment and relatively low costs associated with re-leveling of heave-related movement. The project structural designer should determine structural slab and slab-on-grade reinforcement based on anticipated use and loading.

Structural Slabs

The basement and preferably at-grade garage structural slabs should be supported on drilled piers designed in accordance with the recommendations for the residence provided above. In addition, the basement slab should be provided with a damp-proofing system that is integral with the basement retaining wall waterproofing or damp-proofing systems. We recommend that the slab be underlain by a minimum of 8 inches of ½- to ¾-inch clean crushed rock underlain by filter fabric. Where expansive materials are exposed at the basement subgrade level, the slab should be underlain by 2-inch thick void forms to mitigate excessive uplift forces from expansive soil and/or bedrock against the bottom of the slab and to serve as a capillary break between the underlying subgrade and the slabs. If void forms are utilized, the crushed rock section below may be reduced to 6 inches. The subgrade soil beneath the basement slab should be sloped at an inclination of not less than about 1.5 percent to the perimeter trench where the retaining wall drainage pipe will be located. Please refer to the retaining wall drainage section of this report for additional details.

To minimize the potential for cracking and heaving of the more critical exterior slabs, we suggest that these slabs be designed as structural slabs supported on drilled pier foundations designed in accordance with the recommendations above. Where expansive soils are exposed at the subgrade level, a 2-inch thick void form should underlie these slabs to mitigate excessive uplift forces against the bottom of the slab. The void formers may be placed directly on the uniformly graded subgrade soils.

If it is desired to limit slab dampness from soil moisture vapors, we recommend that a heavy-duty impermeable 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 vapor retarder such as Florprufe™ (Grade Construction Products), which will remain in direct contact with the slab when the cardboard void-former deteriorates. Please 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.

Slabs-on-Grade

We anticipate that concrete slabs-on-grade will be used for driveways, garage floors, and exterior patios and walkways. The driveway and garage slabs should be underlain by a minimum of 12 inches of Class 2 aggregate baserock and slabs for exterior patios and walkways may be underlain by 8 inches of Class 2 aggregate baserock. We note that the placement of the above thickness of baserock beneath proposed slabs will in our opinion substantially mitigate but not completely eliminate the potential for differential performance of these slabs. In general, slabs-on-grade should be designed as "free-floating" slabs, structurally isolated from adjacent foundations. If the garage slab will be structurally tied to the foundation, we recommend increasing the aggregate baserock section to 18 inches.

Slab-on-grade sections adjacent the basement walls should be designed to span the area underlain by the planned basement retaining wall back-fill (approximately 10-feet) to mitigate the concerns for back-fill settlement. In addition, where existing fill is present within areas of new hardscape, the fill should generally be removed and replaced as engineered fill. Prior to the placement of the baserock, the subgrade soils should be scarified and moisture conditioned, as necessary, to a depth of approximately 6 inches and re-compacted in accordance with the Compaction section of this report.

To reduce the potential for slab surface moisture, we recommend that interior slabs, including the garage slabs, be underlain by a vapor retarder consisting of a highly durable membrane not less than 10 mils thick (such as Stego Wrap Vapor Retarder by Stego Industries, LLC or equivalent). The vapor retarder should be 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 Class 2 aggregate 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, especially at interior living spaces, we recommend that consideration be given to utilizing a waterproof membrane in place of the vapor retarder. A qualified waterproofing consultant should provide specific waterproofing products and details.

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

Vapor Retarder Considerations

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

We understand that recent trends are to use a highly durable vapor retarder membrane (at least 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 you consult with other members of your design team, such as your structural engineer, architect, and waterproofing consultant for further guidance on this matter.

FLEXIBLE PAVEMENTS

It is anticipated that flexible hardscape may be utilized as part of the proposed construction. Specifically, we anticipate that the driveway may be surfaced with either asphaltic concrete or pavers and that pavers or flagstone may be used for patios and walkways. We note that due to the fill that likely underlies portions of the proposed hardscape, there exists a moderate potential for differential performance of new hardscape. One advantage of using sand-set pavers for exterior hardscape areas at this site would be that the pavers could accommodate slight differential movement and could be relatively easily repaired if differential movement occurred.

Asphalt Driveway

We recommend that the asphalt driveway surface(s), if utilized, be at least 2.5 inches thick and that it be underlain by at least 12 inches of Class 2 aggregate baserock (R-value 78). 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 select granular fill. Prior to placement of the select granular fill, the subgrade soils should be scarified to a depth of approximately 6 inches, moisture conditioned (as necessary), and re-compacted in accordance with the Compaction section of this report.

Sand-Set Pavers

If sand-set stone pavers are planned for the driveway(s), because of traffic loads, we recommend that pavers be underlain by at least 12 inches of Class 2 aggregate baserock. If sand-set pavers or flagstone are planned for patios and walkways, we recommend that the pavers or flagstone be underlain by at least 6 inches of Class 2 aggregate baserock. Prior to placement of baserock, the surficial soil should be scarified to a depth of approximately 6 inches and re-compacted in accordance with the Compaction section of this report.

EARTHWORK

At the time this report was prepared, the scope of the proposed grading had not been determined. However, we anticipate that a moderate to significant amount of earthwork will be required to develop the 4-lot subdivision, including the possibly re-grading of portions of the active landslide feature mapped on Lots 1 & 2. We recommend that proposed earthwork be performed in general accordance with the following recommendations.

Clearing & Site Preparation

Initially, the proposed improvement 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. Excavations that extend below finished grade resulting from the removal of underground obstructions, such as utilities and root balls, should be backfilled with engineered fill, compacted in accordance with the recommendations presented below.

Material for Fill

All on –site soils below the stripped layer having an organic content of less than 3 percent organic material by volume (ASTM D 2974) should be suitable for use as engineered fill. 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 have a plasticity index of less than approximately 15 percent and should be sufficiently cohesive to maintain a temporary vertical excavation. 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 plasticity 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.

Keying & Benching

Unretained fill placed on slopes that are flatter than 5:1 should be supported on level benches bearing in supportive materials, 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 material 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-18, 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 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 competent support material, as measured on the downhill side of the excavation. The depth to 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

Fills exceeding approximately 5 feet in depth (or within areas of the active landslide on Lots 1 & 2 to be re-graded) should be provided with subdrainage. Subdrains should consist of a 4-inch diameter, rigid, heavy-duty, perforated pipe (Schedule 40, SDR 35 or equivalent) 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 their up-gradient ends and at all sharp changes in direction. Subdrain systems should be provided with a minimum 1 percent gradient and should discharge at an appropriate downhill location, as discussed in the Site Drainage section below.

Compaction

The scarified surface soils and all structural fill should be compacted in uniform lifts, no thicker than approximately 8-inches in un-compacted thickness, conditioned to the appropriate 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 (for fills up to 4 feet thick)	90 percent	Near optimum
For fills greater than 4 feet thick	93 percent	Near optimum
Upper 12 inches of potentially expansive subgrade beneath slabs-on-grade (PI>20)	88 to 90 percent	~3% Over optimum or greater
Upper 6 inches of non-expansive subgrade beneath slabs-on-grade (PI<20)	90 percent	Near optimum
Aggregate baserock under hardscapes	95 percent	Near optimum
½- to ¾-inch Crushed Rock - Compact with at least 3 passes of a vibratory plate with lift-thickness ≤ 12 inches.	see note at left	Not critical
Backfill of utility trenches using on-site soils	90 percent	~2% Over optimum
Backfill of utility trenches using imported sand	95 percent	Near optimum

*Relative to ASTM D 1557, latest edition.

Final Slopes

In general, any proposed cut slopes in the surficial soil and any proposed fill slopes should have gradients no steeper than 2:1 (horizontal to vertical). In general, all 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 & Shoring

The contractor should be responsible for all temporary slopes and trenches excavated at the site and design and construction of any required shoring. Shoring and bracing 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

In our opinion, careful design of the site surface drainage system is critical to the successful development of hillside properties. In our opinion, a qualified civil engineer should develop site drainage plans. In general, we recommend that structures be provided with roof gutters and downspouts and that these drainage devices be connected to buried pipes to convey collected water to suitable discharge locations. Because of the steep slopes, we strongly suggest discharging any collected water into the existing storm drain system. If necessary, storm water may be discharged on the property at an appropriate downslope location; however, there is a potential for erosion and shallow landsliding to impact the area below. To minimize erosion, we recommend that all collected water be discharged onto adequately designed energy dissipaters.

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 into the existing storm drain system or at an appropriate downslope location.

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.

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 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:

- All earthwork and site drainage, including site grading, pier and tieback excavations, tieback testing, placement and compaction of engineered fill, preparation of subgrade and underlayment beneath any slabs and/or the driveway, retaining wall backfill, and final surface drainage installation should be performed in accordance with the geotechnical report prepared by Murray Engineers, Inc., dated February 10, 2014. Murray Engineers, Inc. should be provided at least 48 hours advance notification of any earthwork operations and should be present to observe and test, as necessary, the earthwork, foundation, and drainage installation phases of the project.

CONSTRUCTION OBSERVATION SERVICES

Murray Engineers, Inc. should observe and test (as necessary) the earthwork and foundation phases of construction in order to a) confirm that subsurface conditions exposed during construction are substantially the same as those interpolated from our limited subsurface exploration, on which the analysis and design were based; b) observe compliance with the geotechnical design concepts, specifications and recommendations; and c) allow design changes in the event that subsurface conditions differ from those anticipated. The recommendations in this report are based on limited subsurface information. The nature and extent of variation across the site may not become evident until construction. If variations are then exposed, it will be necessary to re-evaluate our recommendations.

LIMITATIONS

This report has been prepared for the sole use of Nick Zmay specifically to evaluate the engineering geologic feasibility of the proposed subdivision and future site development and for developing geotechnical design criteria relating to design and construction of the proposed residences and associated improvements on the property at 1551 Crystal Springs Road in San Mateo County, California. The opinions presented in this report are based upon review of prior reports, information obtained from borings at widely separated locations, site reconnaissance, review of field data made available to us, and upon local experience and engineering judgment. Our conclusions and recommendations have been formulated in accordance with generally accepted geotechnical engineering practices that exist in the San Francisco Bay Area at the time this report was prepared. Further, our recommendations are based on the assumption that soil and geologic conditions at or between borings do not deviate substantially from those encountered. It should be clearly understood that geotechnical conditions may become apparent during construction that were not apparent at the time of our investigation. No warranty, either expressed or implied, is made or should be inferred. We are not responsible for data provided by others.

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

The opinions presented in this report are valid as of the present date for the property evaluated. Changes in the condition of a property can occur with the passage of time, whether due to natural processes or the works of man, on this or adjacent properties. In addition, changes in applicable standards of practice can occur, whether from legislation or the broadening of knowledge. Accordingly, the opinions presented in this report may be invalidated, wholly or partially, by changes outside of our control. Therefore, this report is subject to review and should not be relied upon after a period of three years, nor should it be used, or is it applicable, for any property other than that evaluated.

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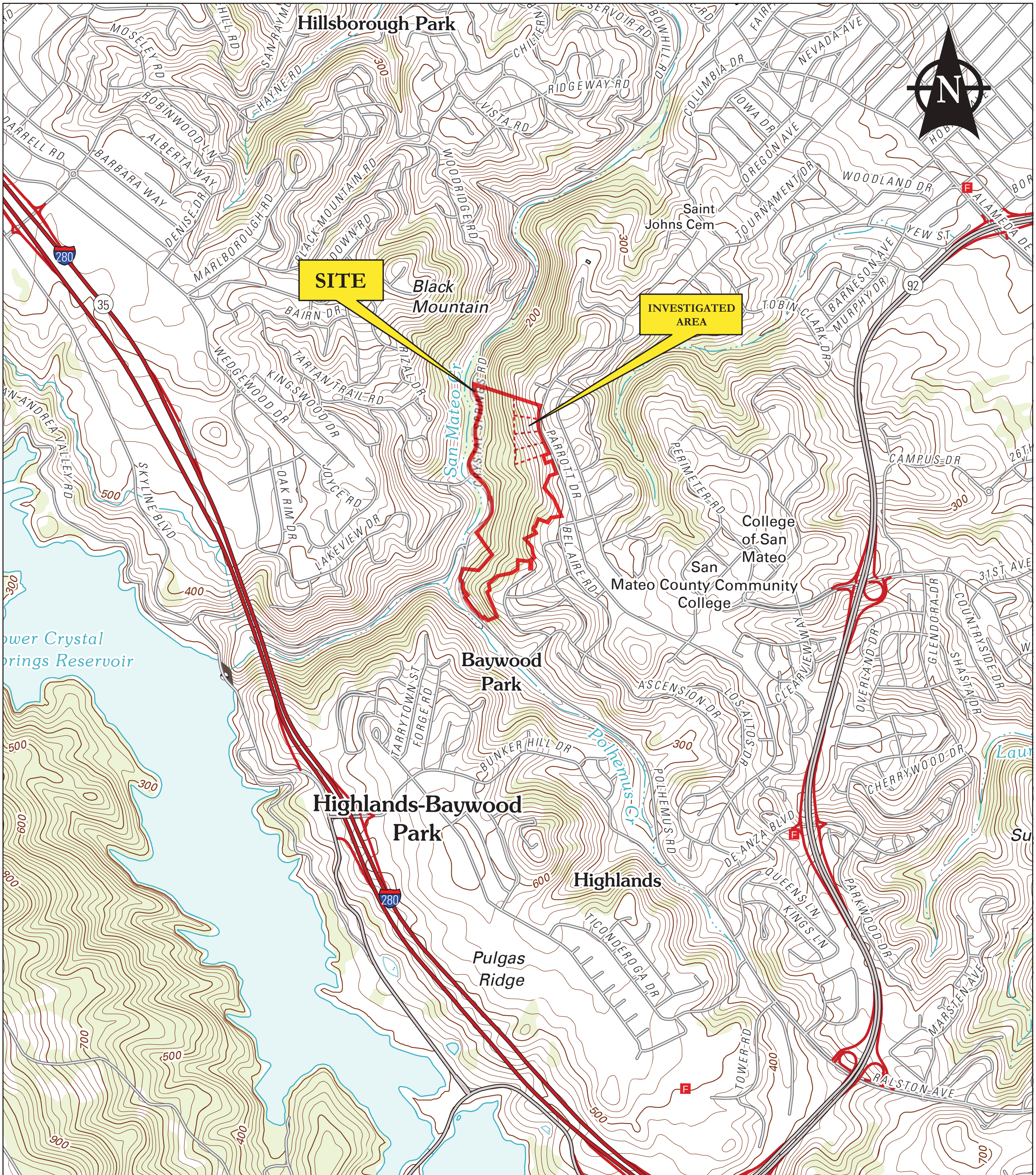
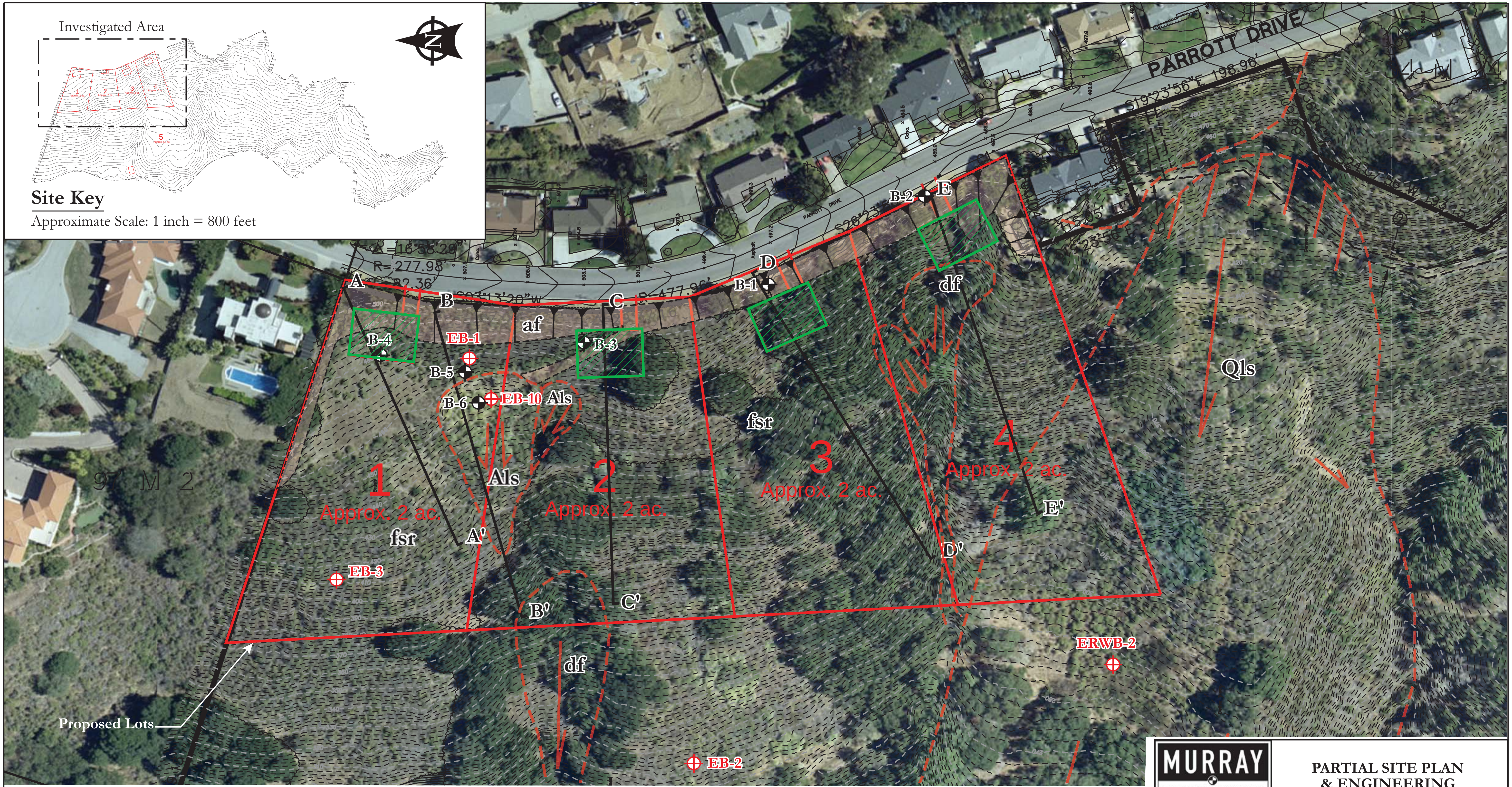


Base: USGS Topographic Map, San Mateo 7.5-Minute Quadrangle, 2012 Scale: 1 inch = 2,000 feet

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	ZMAY RESIDENTIAL SUBDIVISION 1551 CRYSTAL SPRINGS ROAD SAN MATEO COUNTY, CALIFORNIA		VICINITY MAP
	PROJECT NO. 1847-1R1	FEBRUARY 2014	FIGURE A-1



Legend & Selected Map Symbols

B-1 Approximate Location of Soil Boring by Murray Engineers, Inc., drilled December 20-23, 2013
EB-1 Approximate Location of Soil Boring by Bagg drilled July 18-19, 2007 & October 2, 2013
Approximate Limits of Fill and Fill Slope
A A' Location of Cross-Sections (see Figure A-6 to A-10)
Base: Record Boundary & Aerial Topography by BKF Engineers, dated January 2, 2007
Approximate Scale: 1 inch = 100 feet

Als Active Landslide
df Debris Flow
Qls Ancient Landslide
fsr Franciscan Sheared Rock

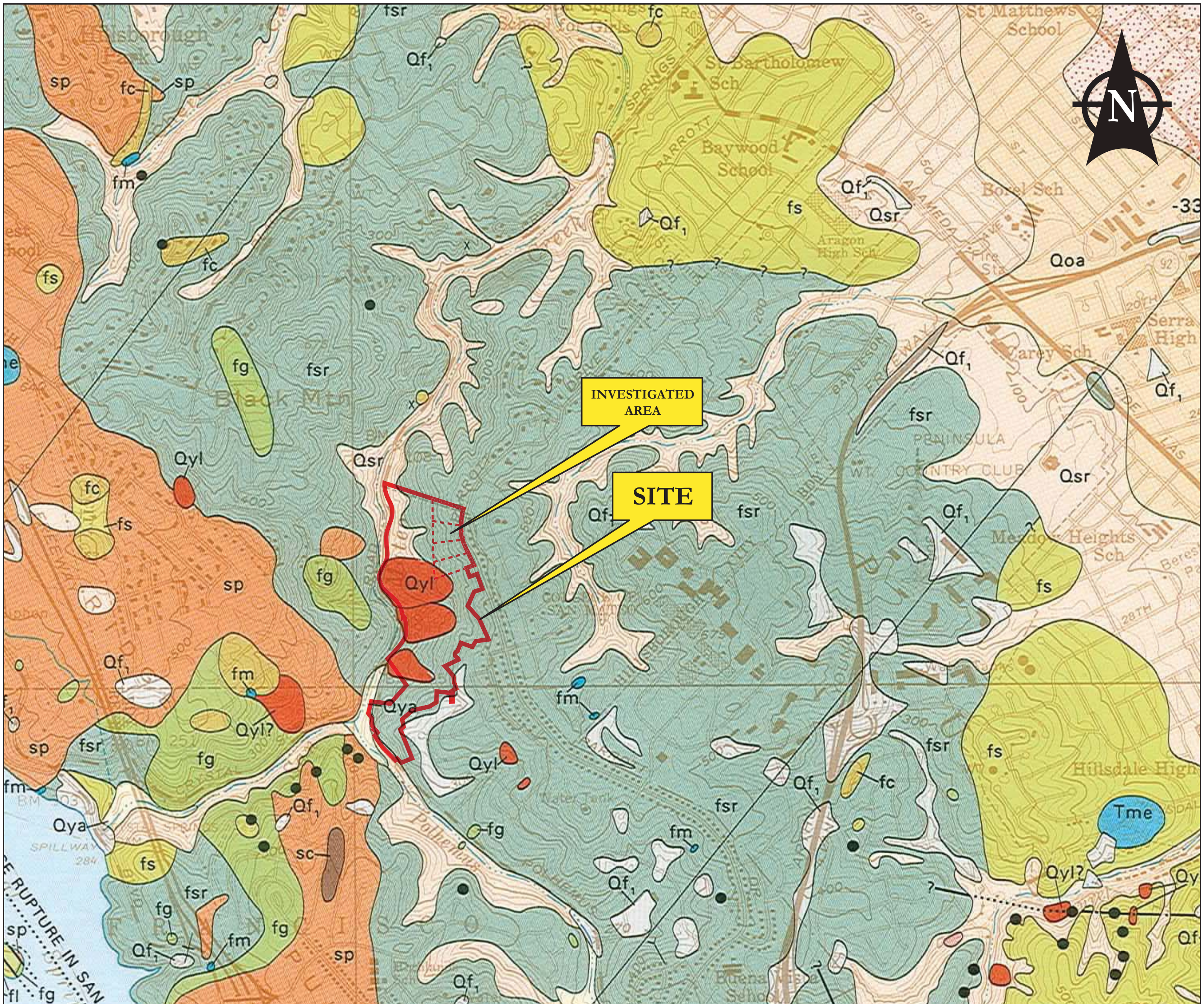
Landslide Limits
Proposed Building Site - Typical

MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

PARTIAL SITE PLAN & ENGINEERING GEOLOGIC MAP

ZMAY RESIDENTIAL SUBDIVISION
1551 CRYSTAL SPRINGS ROAD
SAN MATEO COUNTY, CALIFORNIA

PROJECT NO. 1847-1R1	FEBRUARY 2014	FIGURE A-2



Legend & Selected Map Symbols

Qsr - Slope Wash, Ravine Fill, and Colluvium (Holocene)

Qyl - Younger Landslide Deposits

fg - Franciscan Greenstone

fsr - Franciscan Sheared Rock

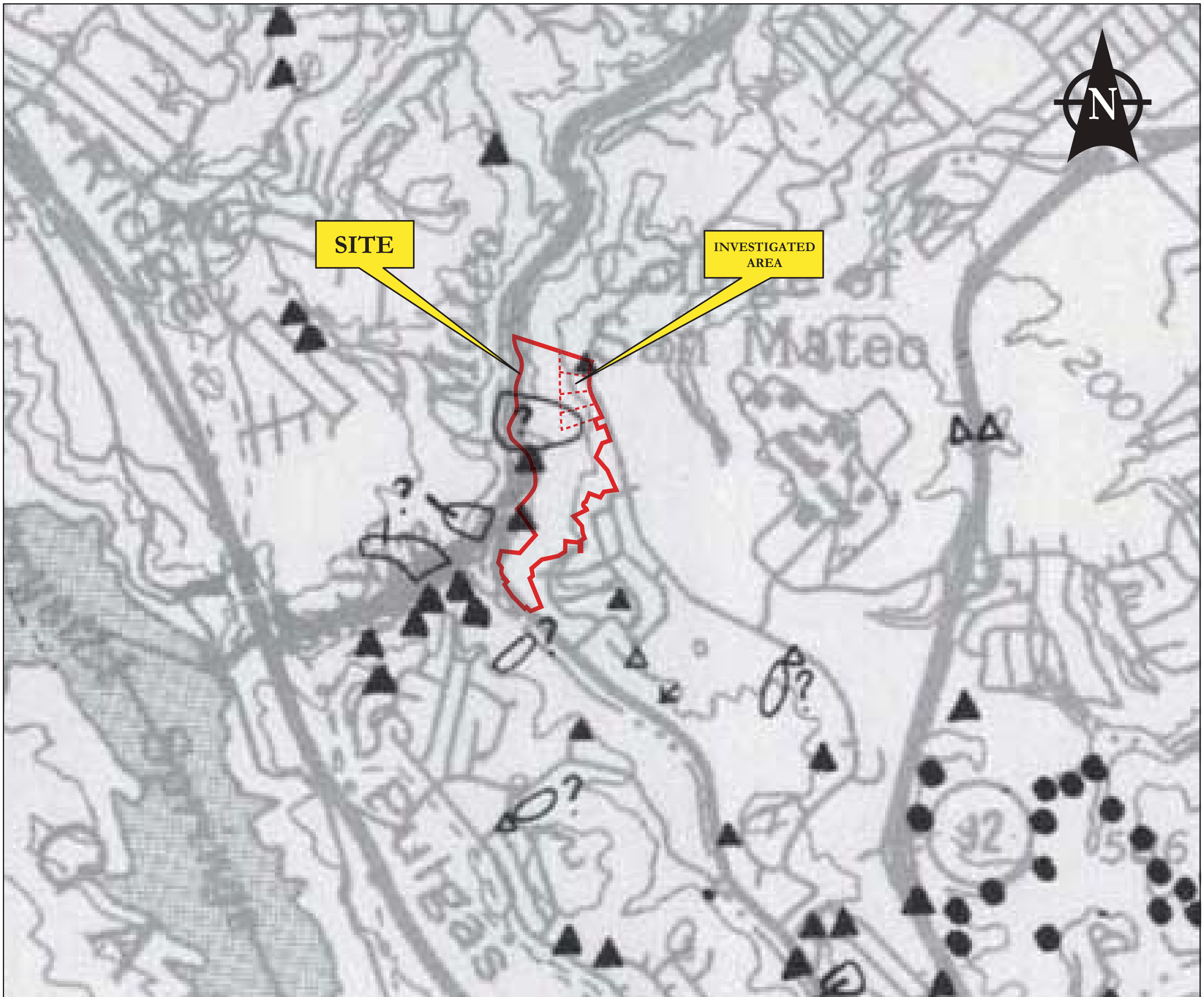
sp - Franciscan Serpentinite

Shallow Landslide

Geologic Contact

Base: Geologic Map of the Montara Mountain and San Mateo 7.5" Quadrangles, San Mateo County, Pampeyan, 1994. Please refer to the Geology section of this report for a description of pertinent geologic symbols noted above. Scale: 1 inch = 2,000 feet

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	ZMAY RESIDENTIAL SUBDIVISION 1551 CRYSTAL SPRINGS ROAD SAN MATEO COUNTY, CALIFORNIA		VICINITY GEOLOGIC MAP
	PROJECT NO. 1847-1R1	FEBRUARY 2014	FIGURE A-3



Legend

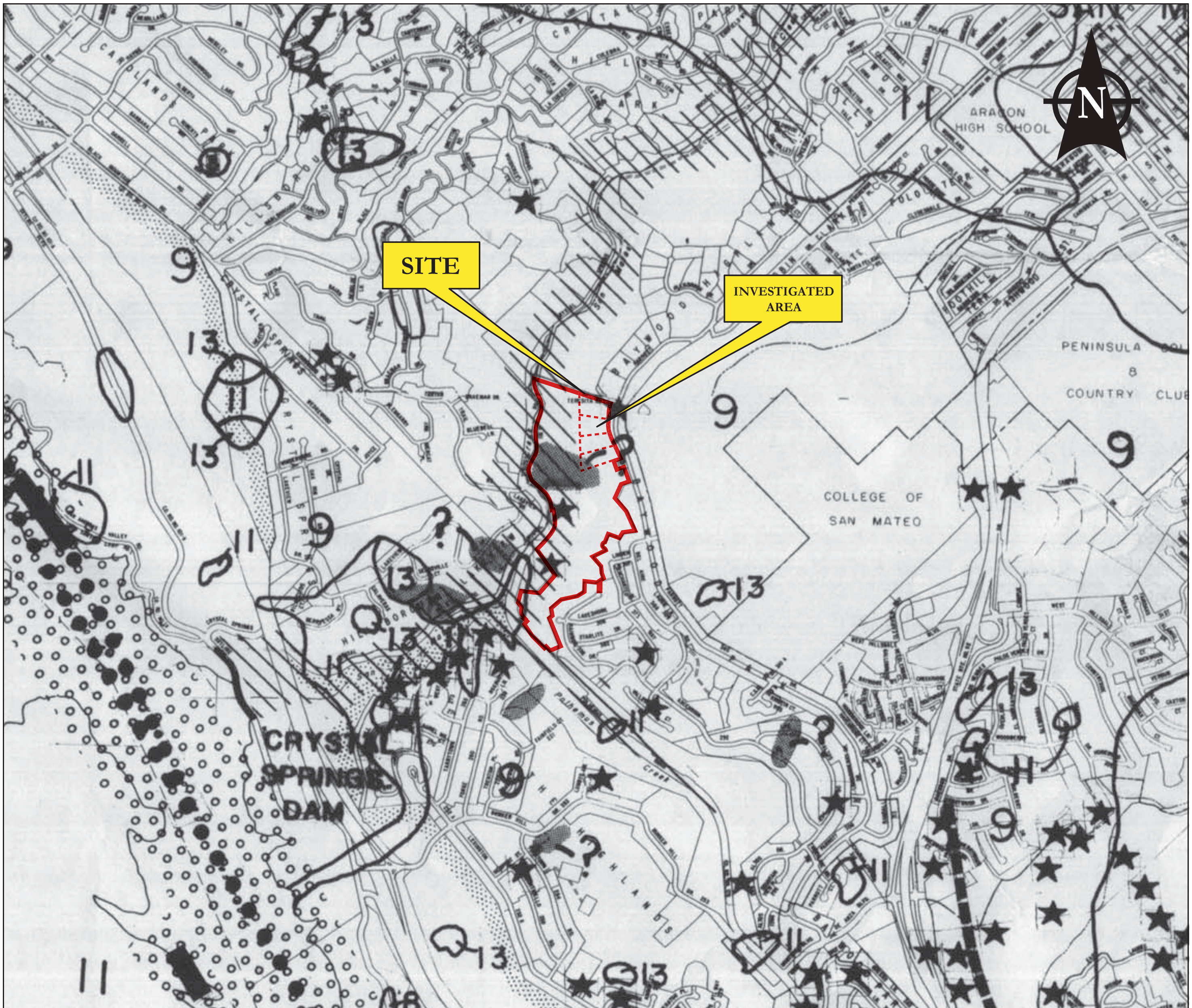
Large landslide deposit, more than 500 feet in maximum dimension. Arrows indicate general direction of downslope movement. D: definite landslide deposit; P: probable landslide deposit; ? questionable landslide deposit; A: active landslide deposit; hatchures indicate approximate location of an inferred main scarp.

Large landslide deposit, more than 500 feet in maximum dimension. F, mapped in field; A, active

▲ ← Small landslide deposit, 50 to 500 feet in maximum dimension. Arrow indicates general direction of downslope movement. Solid triangle indicates mapped in field.

Base: Preliminary Map of Landslide Deposits in San Mateo County, California, by E.E. Brabb & E.H. Pampeyan, 1972 Approximate Scale: 1 inch = 2,000 feet

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	ZMAY RESIDENTIAL SUBDIVISION 1551 CRYSTAL SPRINGS ROAD SAN MATEO COUNTY, CALIFORNIA		SAN MATEO COUNTY LANDSLIDE DEPOSIT MAP
	PROJECT NO. 1847-1R1	FEBRUARY 2014	FIGURE A-4

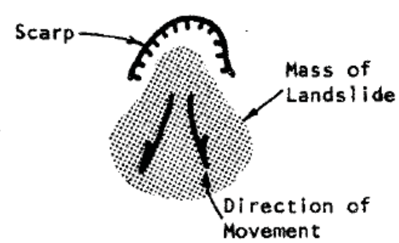


Legend & Selected Map Symbols

9	Serpentine Sheared Rocks of the Franciscan Assemblage
11	Basalt and Other Volcanic Rocks, Franciscan Greenstone, Portions of Butano Sandstone of Point San Pedro, Undivided Franciscan Sandstone
11	Chert, Limestone, Granitic Rocks Metamorphic Rocks

Geological Contact, dashed where approximate.

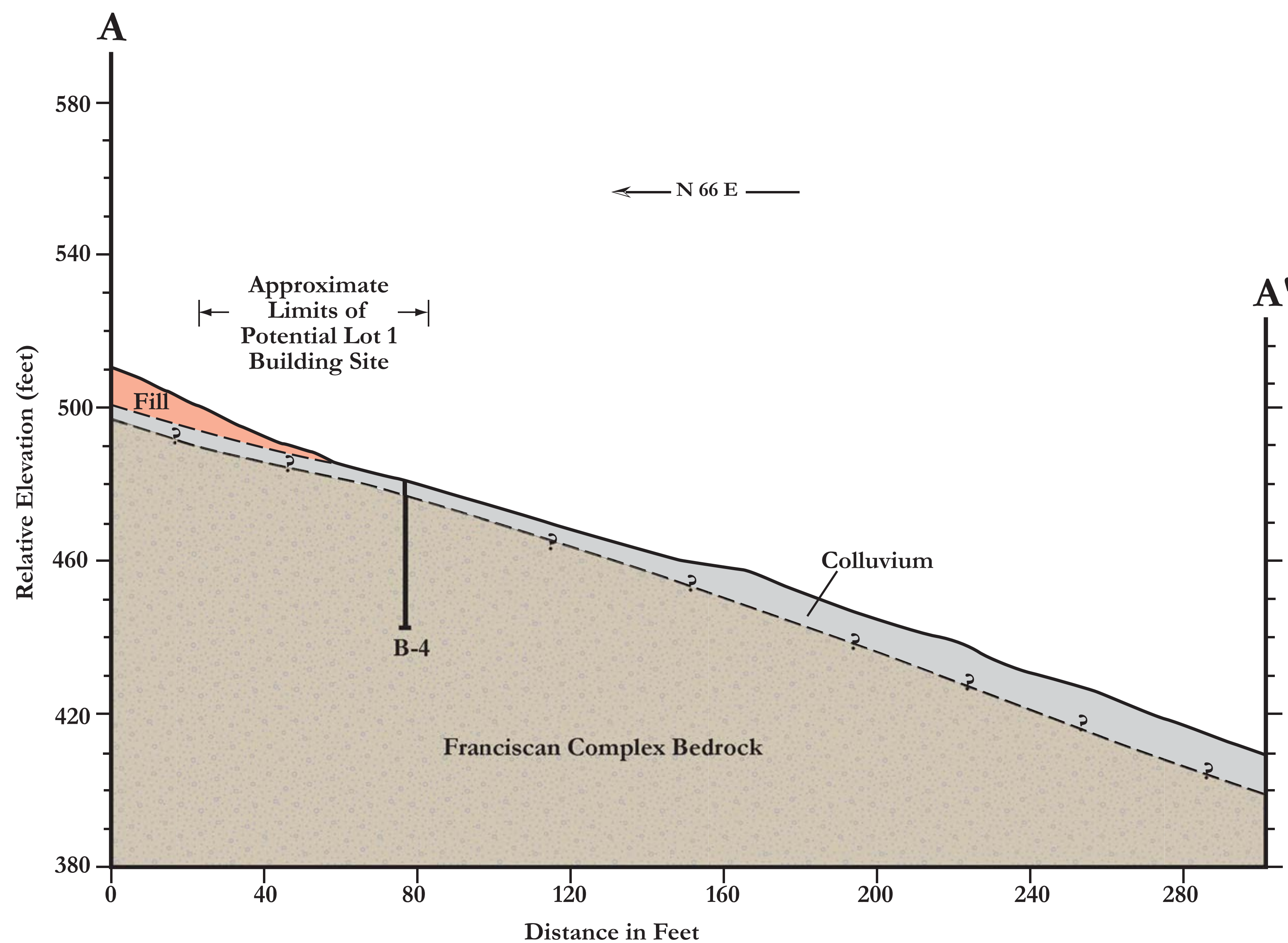
★ APP



? Suspected or conjectured landslide.

Base: Geotechnical Hazard Synthesis Map (Sheet 2) by S.H. Cantwell Jr. (1973).
Approximate Scale 1 inch = 2,000 feet

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	ZMAY RESIDENTIAL SUBDIVISION 1551 CRYSTAL SPRINGS ROAD SAN MATEO COUNTY, CALIFORNIA	SMC GEOTECHNICAL SYNTHESIS MAP
PROJECT NO. 1847-1R1	FEBRUARY 2014	FIGURE A-5



LEGEND

B-4 Approximate Location of Soil Boring by Murray Engineers, Inc., drilled December 23, 2013

– – – – Approximate Geologic Contact, (?) Queried, Where Uncertain

Base: Record Boundary & Aerial Topography by BKF Engineers, dated January 2, 2007

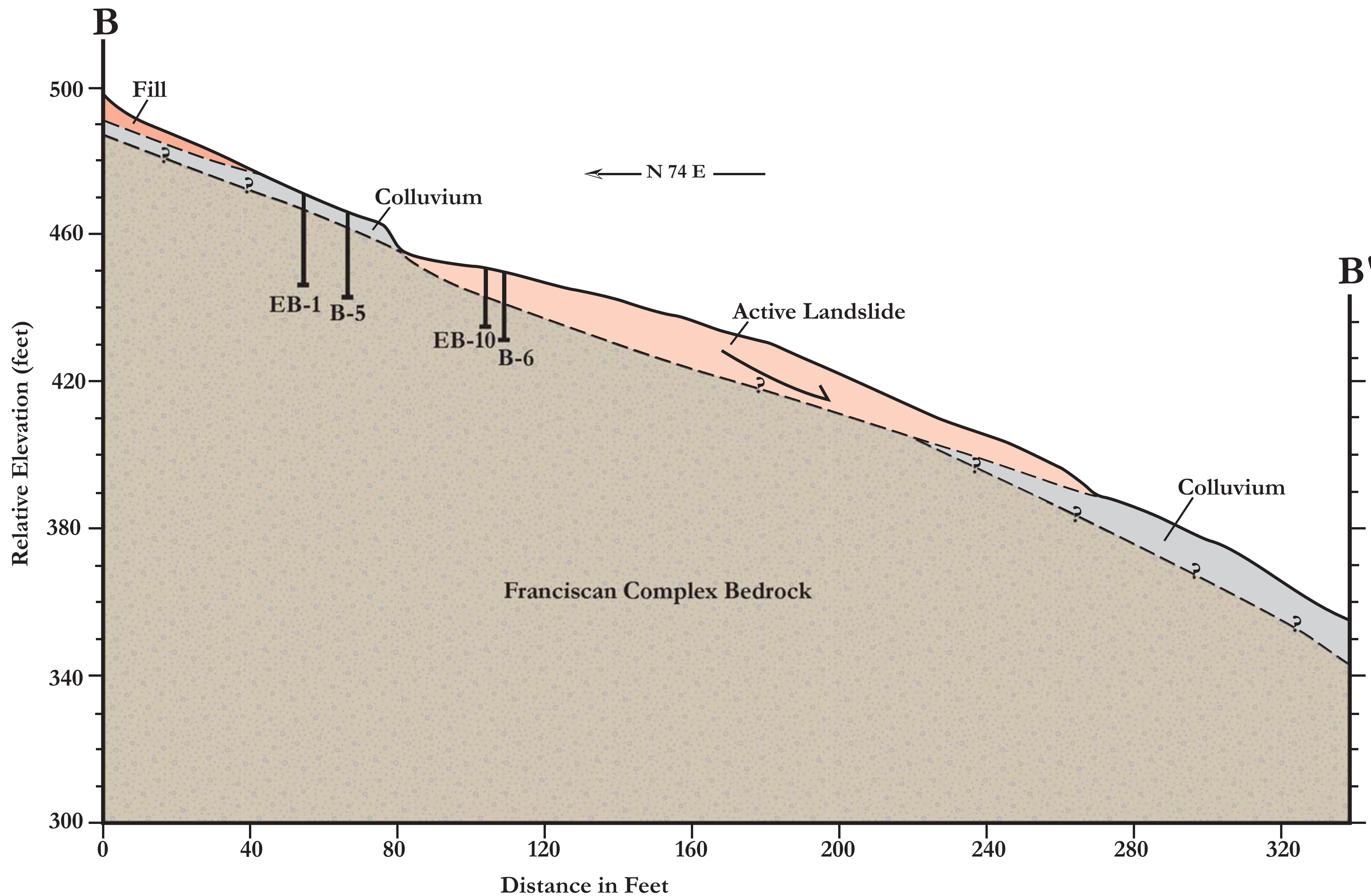
Approximate Scale: 1 inch = 40 feet (horizontal = vertical)

MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

GEOLOGIC CROSS-SECTION A-A'

ZMAY RESIDENTIAL SUBDIVISION
1551 CRYSTAL SPRINGS ROAD
SAN MATEO COUNTY, CALIFORNIA

PROJECT NO. 1847-1R1	FEBRUARY 2014	FIGURE A-6



LEGEND

B-4 Approximate Location of Soil Boring by Murray Engineers, Inc., drilled December 23, 2013

EB-1 Approximate Location of Soil Boring by BAGG drilled October 1 & 2, 2007

– – – Approximate Geologic Contact, (?) Queried, Where Uncertain

Base: Record Boundary & Aerial Topography by BKF Engineers, dated January 2, 2007

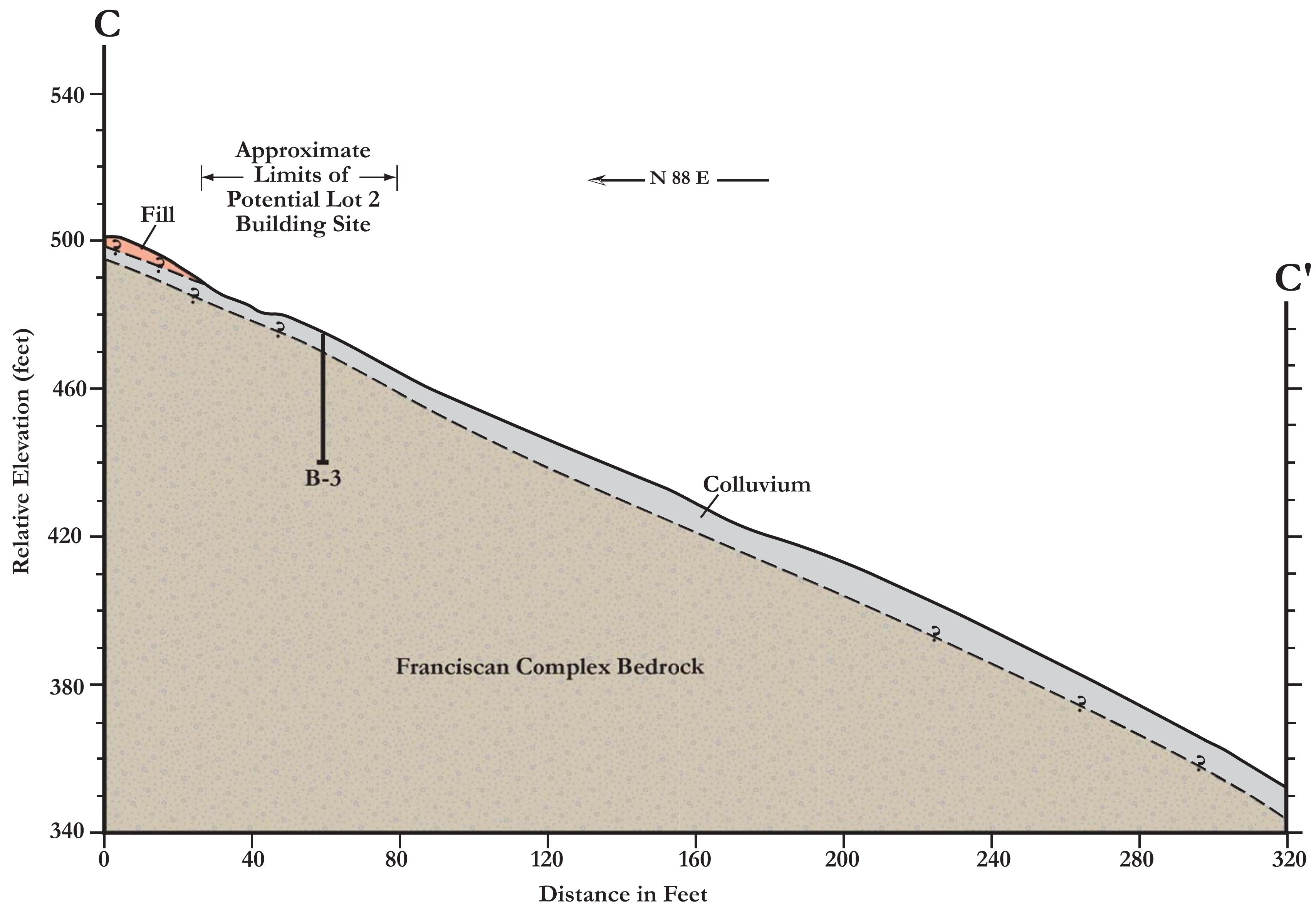
Approximate Scale: 1 inch = 40 feet (horizontal = vertical)

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES

GEOLOGIC CROSS-SECTION B-B'

ZMAY RESIDENTIAL SUBDIVISION
1551 CRYSTAL SPRINGS ROAD
SAN MATEO COUNTY, CALIFORNIA

PROJECT NO. 1847-1R1	FEBRUARY 2014	FIGURE A-7



LEGEND

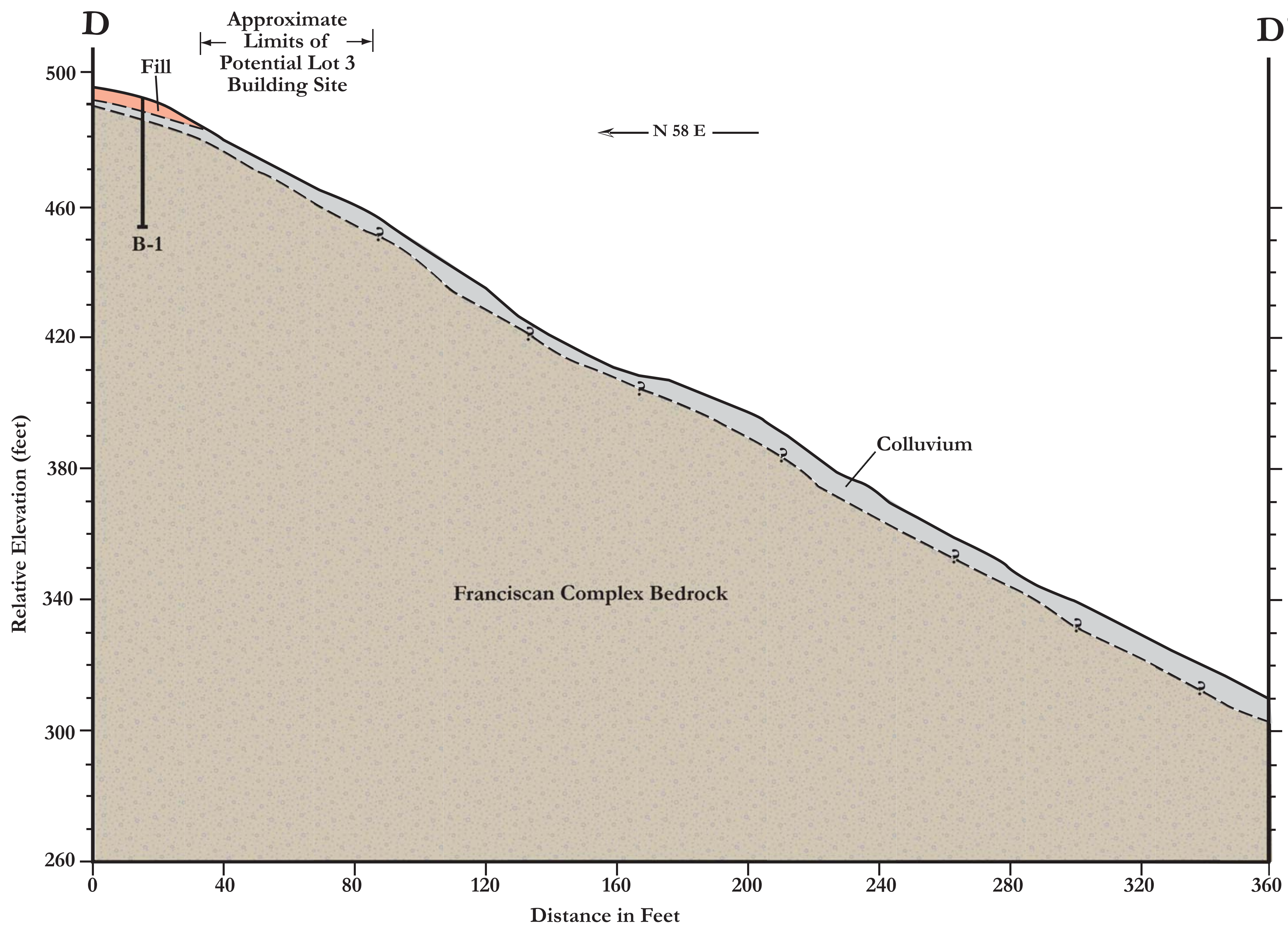
B-3 Approximate Location of Soil Boring by Murray Engineers, Inc., drilled December 20, 2013

– – – – Approximate Geologic Contact, (?) Queried, Where Uncertain

Base: Record Boundary & Aerial Topography by BKF Engineers, dated January 2, 2007

Approximate Scale: 1 inch = 40 feet (horizontal = vertical)

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	GEOLOGIC CROSS-SECTION C-C'	
ZMAY RESIDENTIAL SUBDIVISION 1551 CRYSTAL SPRINGS ROAD SAN MATEO COUNTY, CALIFORNIA		
PROJECT NO. 1847-1R1	FEBRUARY 2014	FIGURE A-8



LEGEND

B-1 Approximate Location of Soil Boring by Murray Engineers, Inc., drilled December 20, 2013

– – – – Approximate Geologic Contact, (?) Queried, Where Uncertain

Base: Record Boundary & Aerial Topography by BKF Engineers, dated January 2, 2007

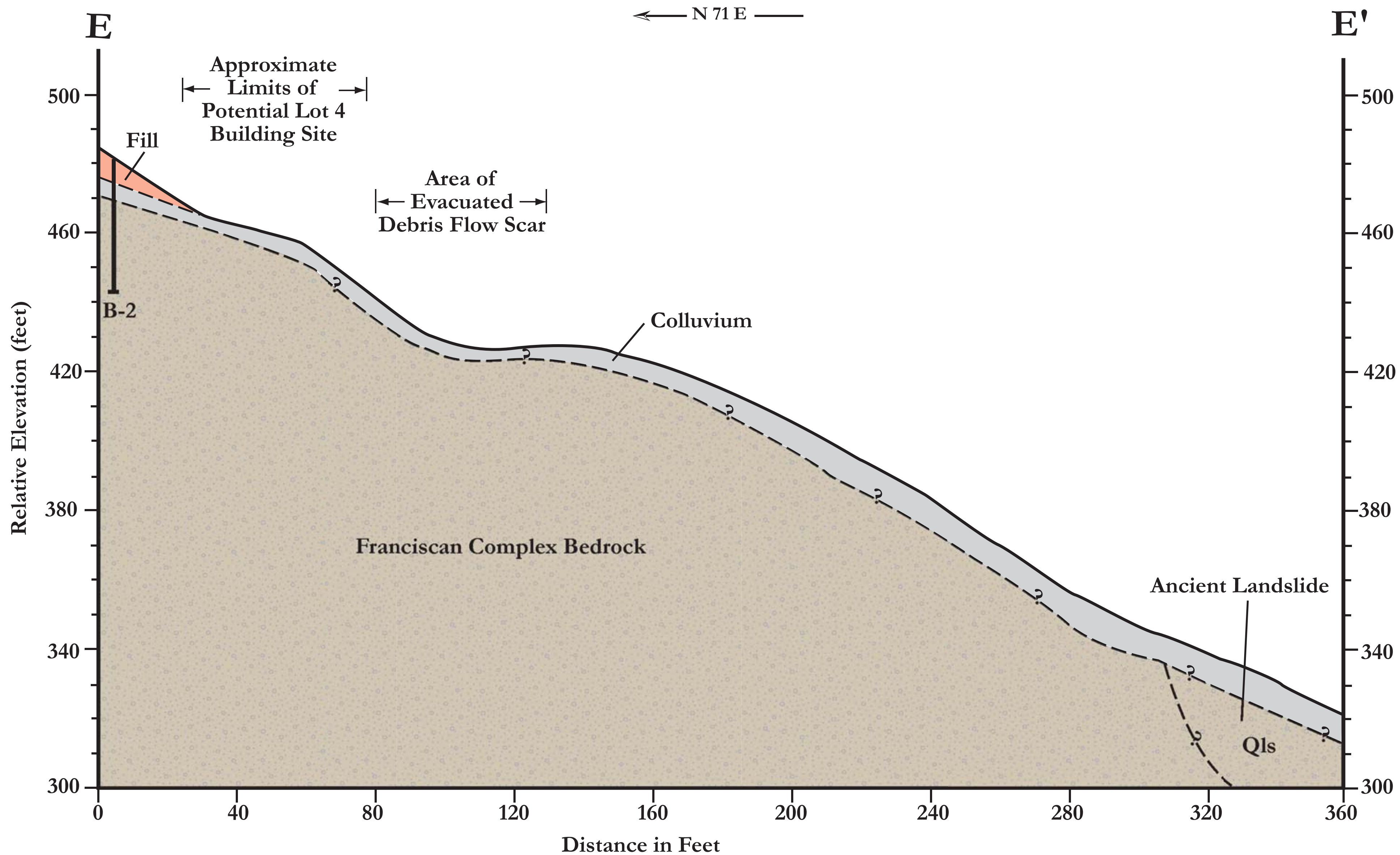
Approximate Scale: 1 inch = 40 feet (horizontal = vertical)

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES

GEOLOGIC CROSS-SECTION D-D'

ZMAY RESIDENTIAL SUBDIVISION
1551 CRYSTAL SPRINGS ROAD
SAN MATEO COUNTY, CALIFORNIA

PROJECT NO. 1847-1R1	FEBRUARY 2014	FIGURE A-9



LEGEND

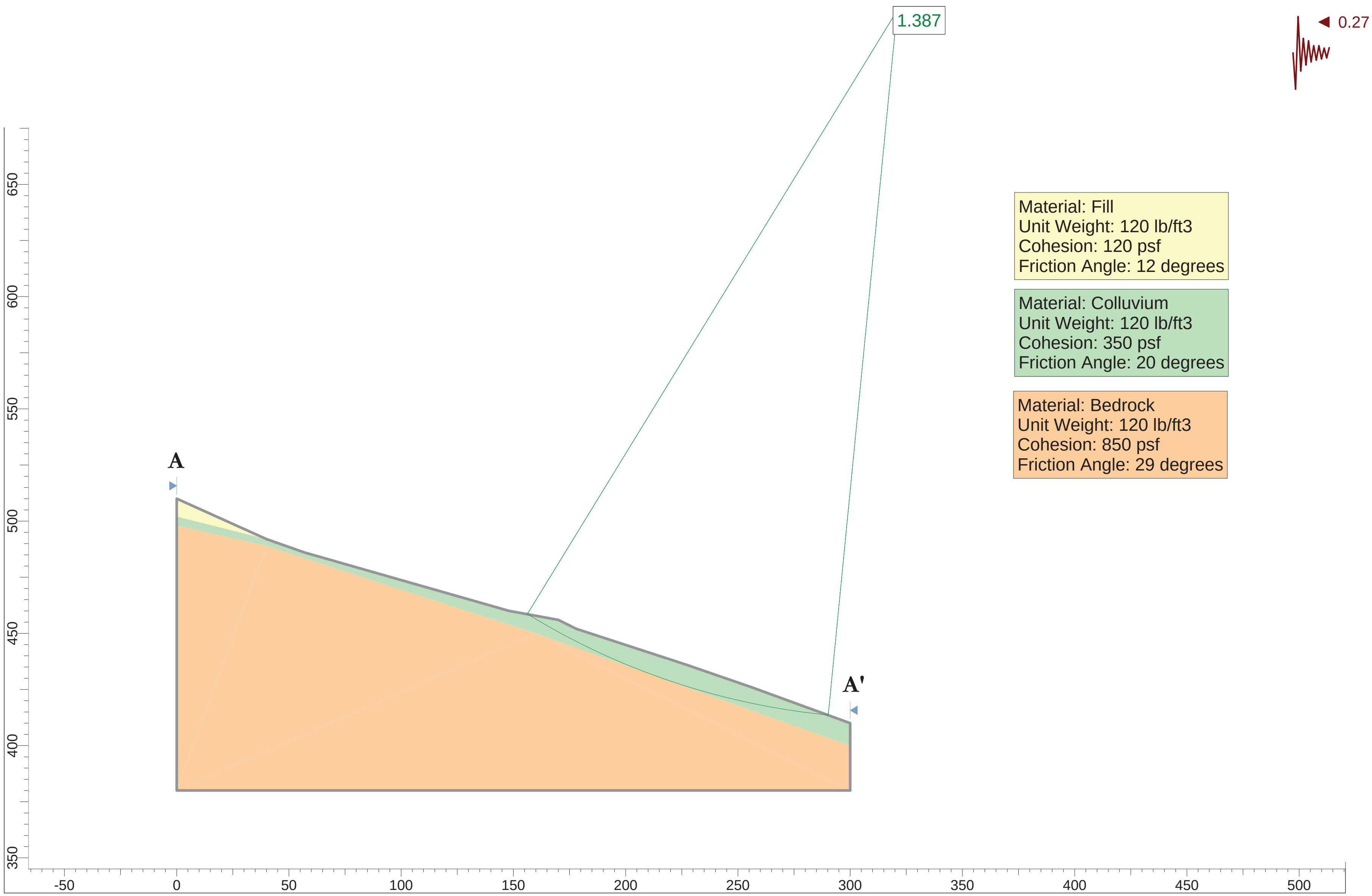
B-1 Approximate Location of Soil Boring by Murray Engineers, Inc., drilled December 20, 2013

– – – – Approximate Geologic Contact, (?) Queried, Where Uncertain

Base: Record Boundary & Aerial Topography by BKF Engineers, dated January 2, 2007

Approximate Scale: 1 inch = 40 feet (horizontal = vertical)

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	GEOLOGIC CROSS-SECTION E-E'
ZMAY RESIDENTIAL SUBDIVISION 1551 CRYSTAL SPRINGS ROAD SAN MATEO COUNTY, CALIFORNIA	
PROJECT NO. 1847-1R1	FEBRUARY 2014 — FIGURE A-10



Base: Figure A-6, Geologic Cross-Section A-A'

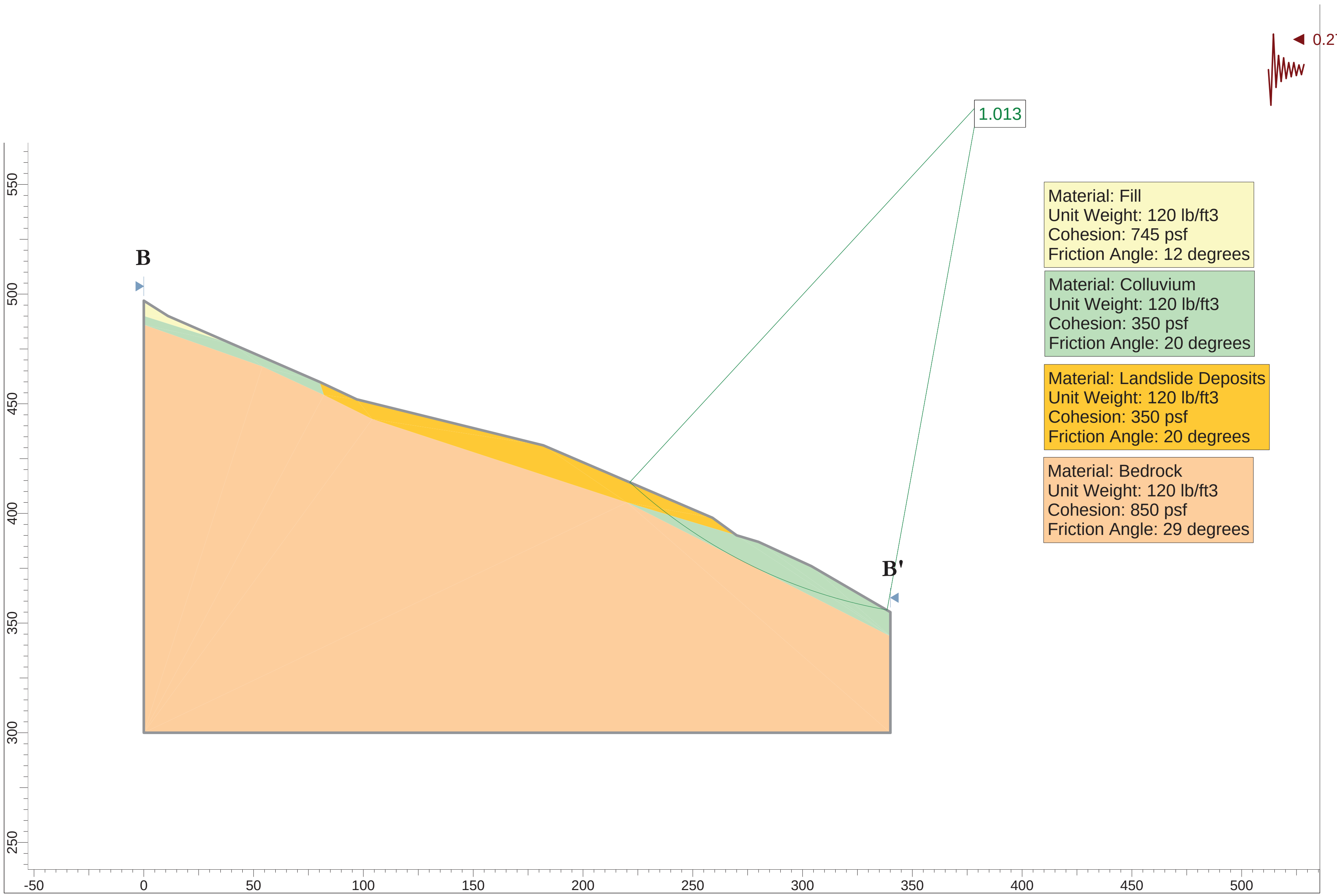
Approximate Scale: 1 inch = 50 feet

MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

ZMAY RESIDENTIAL SUBDIVISION
1551 CRYSTAL SPRINGS ROAD
SAN MATEO COUNTY, CALIFORNIA

CROSS-SECTION A-A'
SLOPE STABILITY ANALYSIS

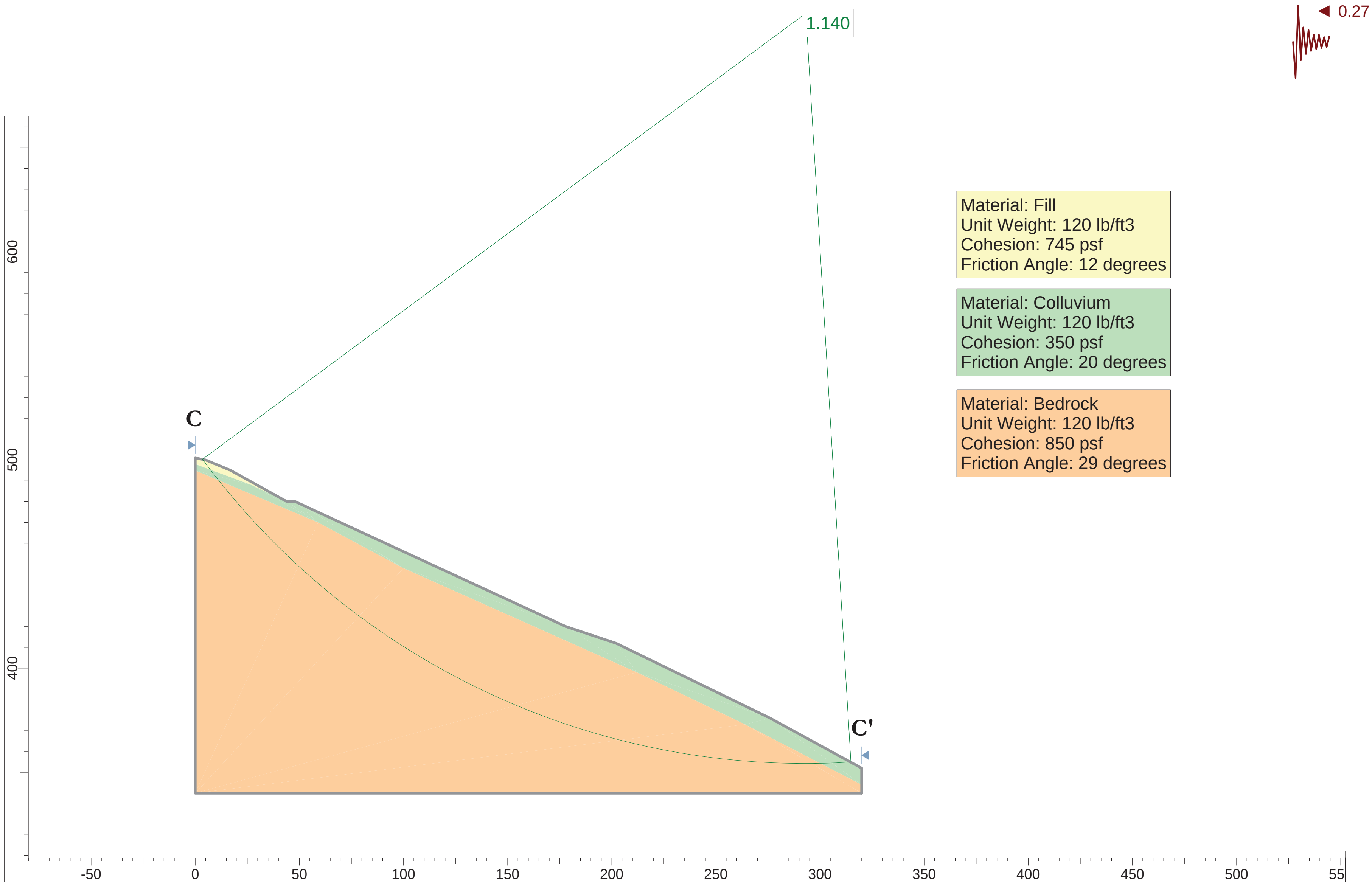
PROJECT NO. 1847-1R1	FEBRUARY 2014	FIGURE A-11



Base: Figure A-7, Geologic Cross-Section B-B'

Approximate Scale: 1 inch = 50 feet

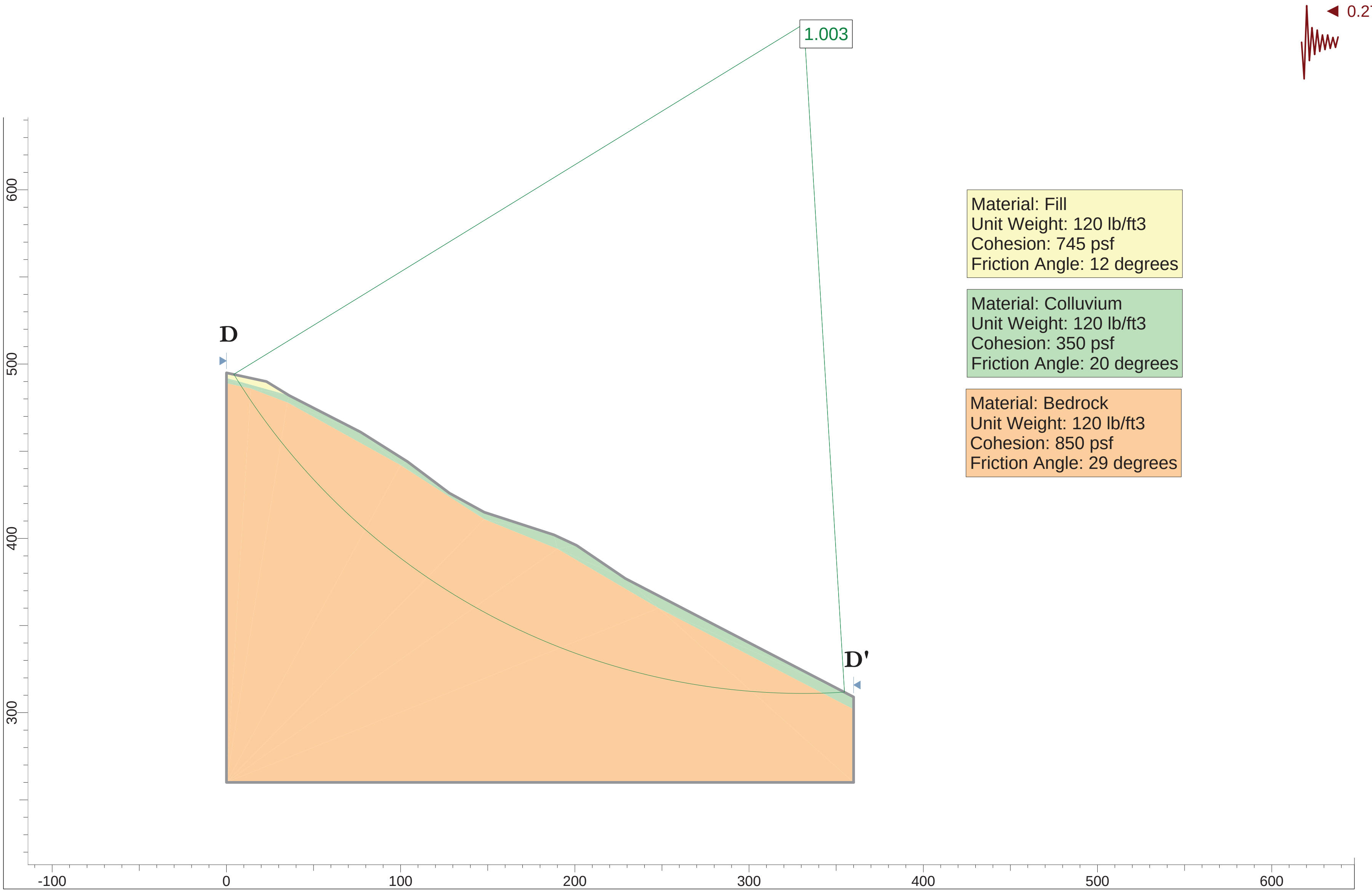
MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	ZMAY RESIDENTIAL SUBDIVISION 1551 CRYSTAL SPRINGS ROAD SAN MATEO COUNTY, CALIFORNIA		CROSS-SECTION B-B' SLOPE STABILITY ANALYSIS
	PROJECT NO. 1847-1R1	FEBRUARY 2014	FIGURE A-12



Base: Figure A-8, Geologic Cross-Section CA-C'

Approximate Scale: 1 inch = 50 feet

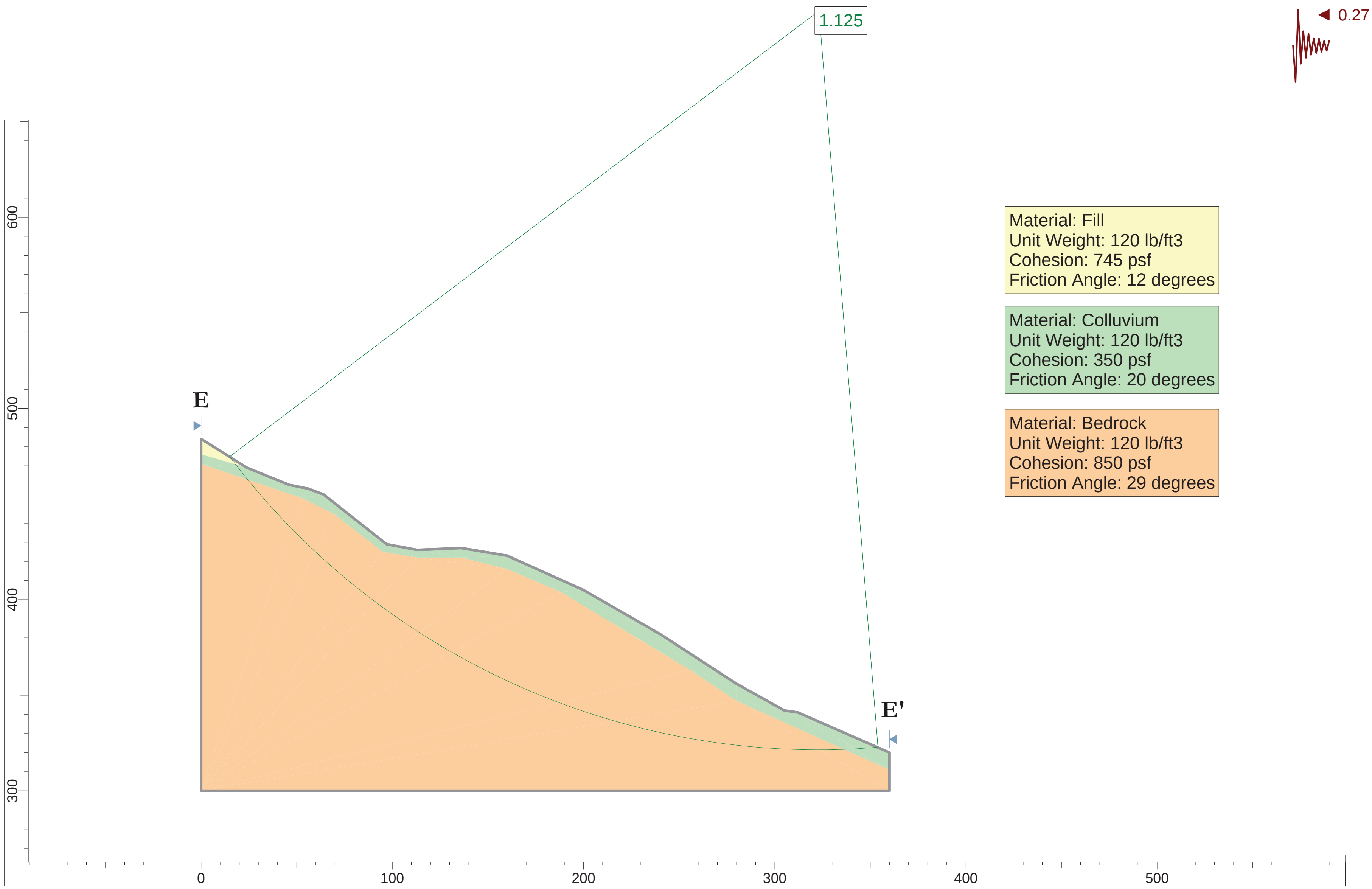
MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	ZMAY RESIDENTIAL SUBDIVISION 1551 CRYSTAL SPRINGS ROAD SAN MATEO COUNTY, CALIFORNIA		CROSS-SECTION C-C' SLOPE STABILITY ANALYSIS
	PROJECT NO. 1847-1R1	FEBRUARY 2014	FIGURE A-13



Base: Figure A-9, Geologic Cross-Section D-D'

Approximate Scale: 1 inch = 50 feet

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	ZMAY RESIDENTIAL SUBDIVISION 1551 CRYSTAL SPRINGS ROAD SAN MATEO COUNTY, CALIFORNIA		CROSS-SECTION D-D' SLOPE STABILITY ANALYSIS
	PROJECT NO. 1847-1R1	FEBRUARY 2014	FIGURE A-14



Base: Figure A-10, Geologic Cross-Section E-E'

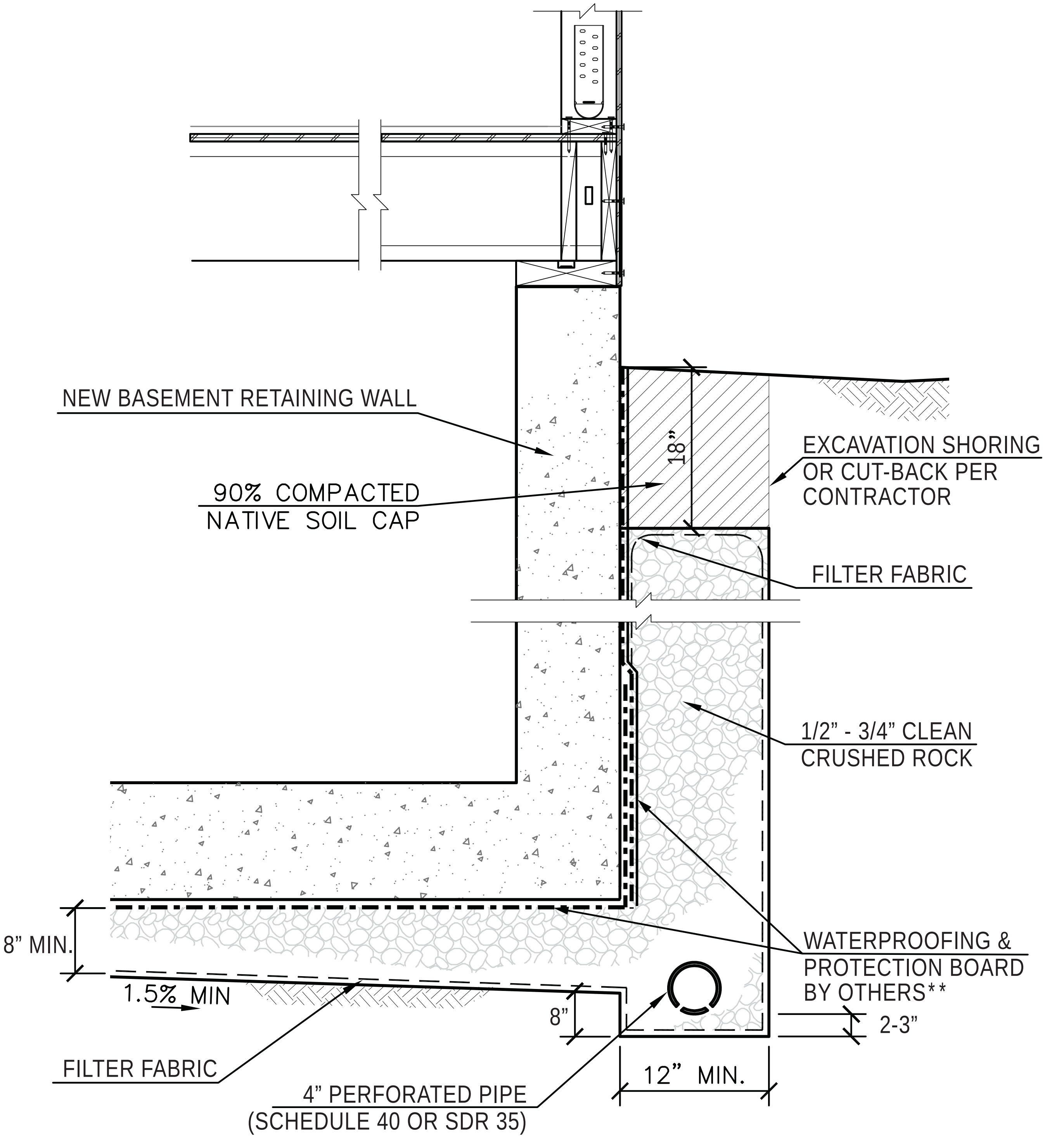
Approximate Scale: 1 inch = 50 feet

MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

ZMAY RESIDENTIAL SUBDIVISION
1551 CRYSTAL SPRINGS ROAD
SAN MATEO COUNTY, CALIFORNIA

CROSS-SECTION E-E'
SLOPE STABILITY ANALYSIS

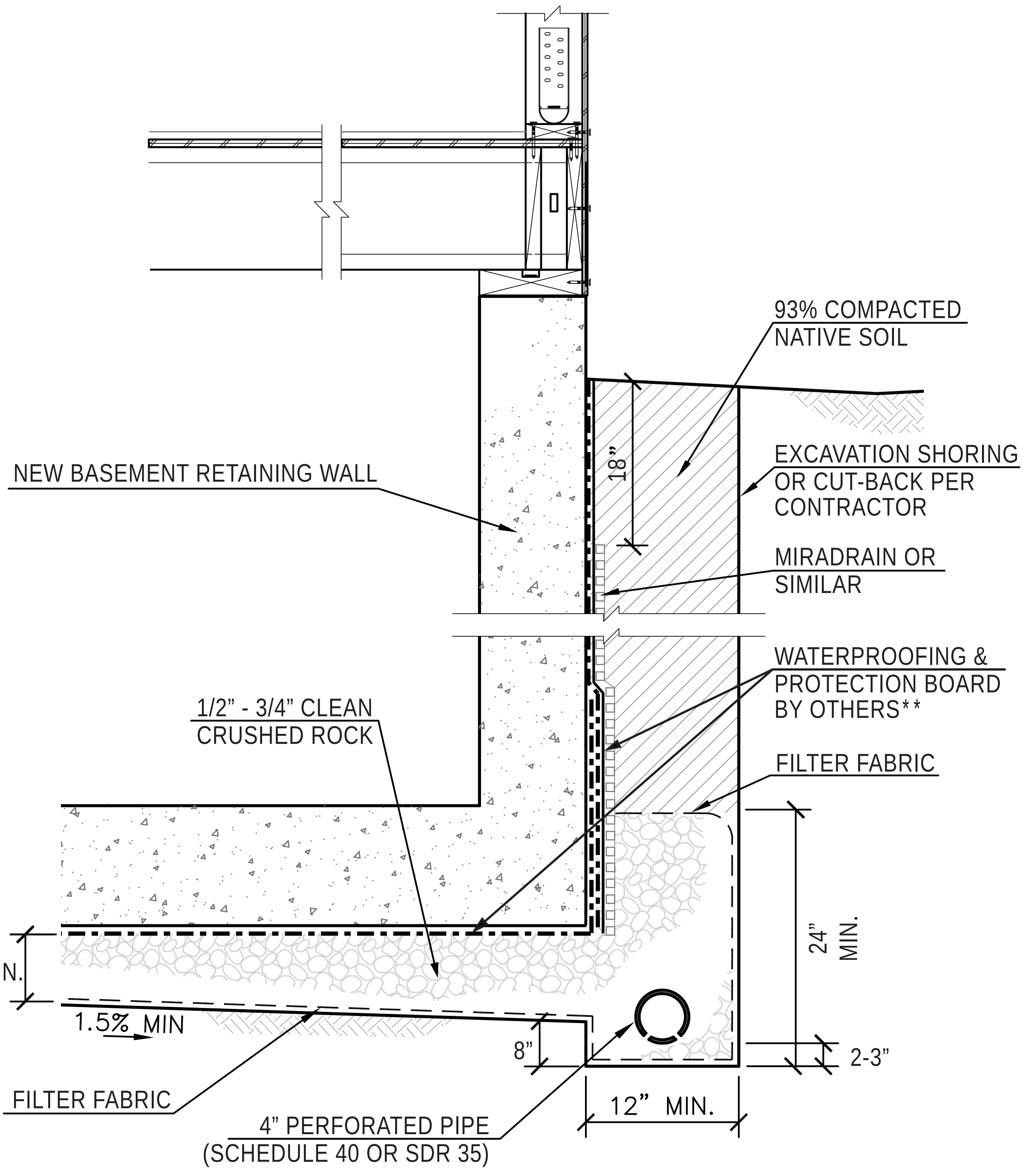
PROJECT NO. 1847-1R1 | **FEBRUARY 2014** | **FIGURE A-15**



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.

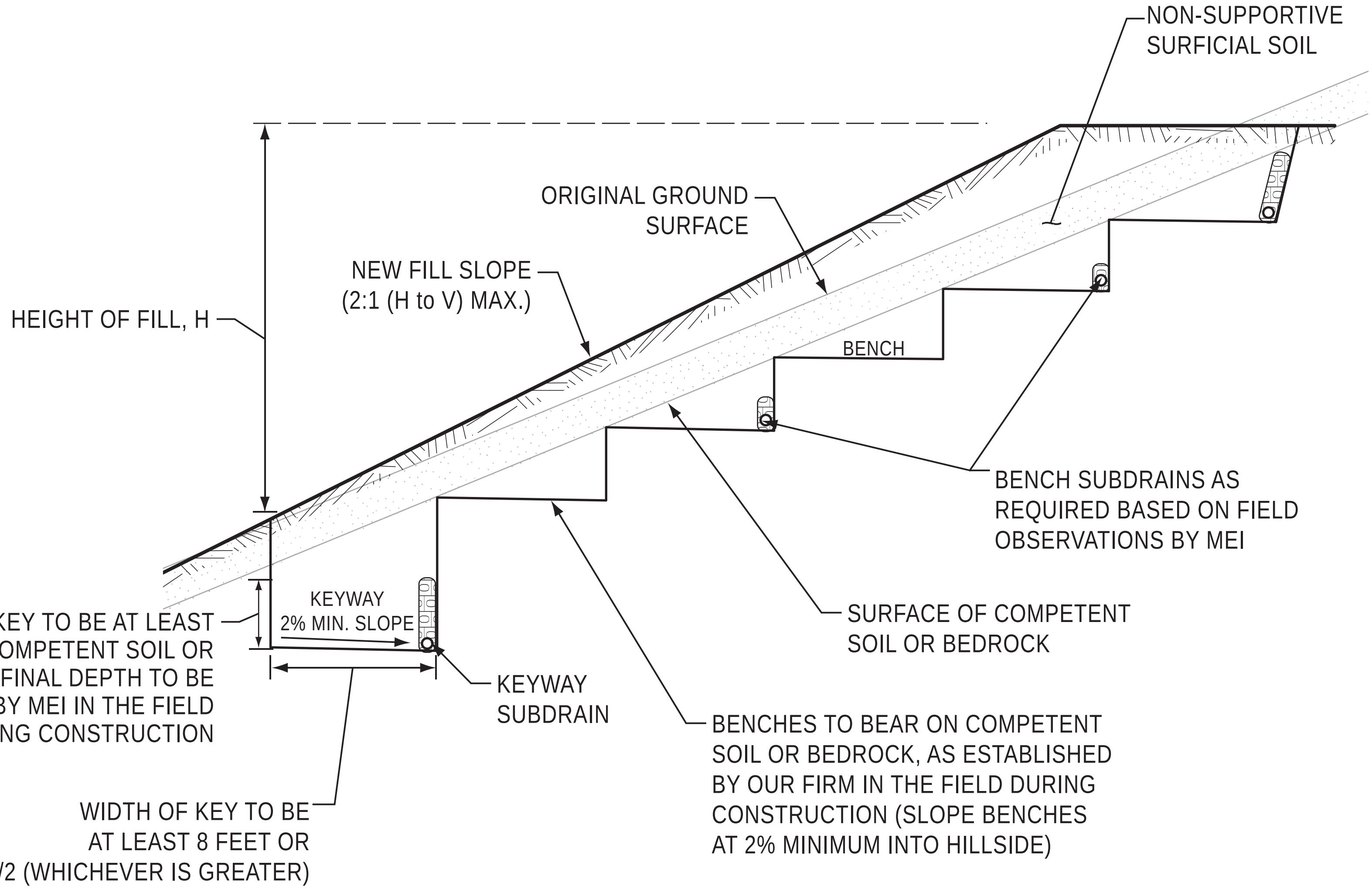
MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	**ZMAY RESIDENTIAL SUBDIVISION 1551 CRYSTAL SPRINGS ROAD SAN MATEO COUNTY, CALIFORNIA**	**BASEMENT SUBDRAIN SYSTEM ALTERNATIVE A**
	PROJECT NO. 1847-1R1 **FEBRUARY 2014**	**FIGURE A-16**



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	ZMAY RESIDENTIAL SUBDIVISION 1551 CRYSTAL SPRINGS ROAD SAN MATEO COUNTY, CALIFORNIA		BASEMENT SUBDRAIN SYSTEM ALTERNATIVE B
	PROJECT NO. 1847-1R1	FEBRUARY 2014	FIGURE A-17



MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	ZMAY RESIDENTIAL SUBDIVISION 1551 CRYSTAL SPRINGS ROAD SAN MATEO COUNTY, CALIFORNIA	SCHEMATIC FILL SLOPE DETAIL
PROJECT NO. 1847-1R1	FEBRUARY 2014	FIGURE A-18

Date(s) Drilled **December 20, 2013**	Logged By **KP**	Checked By **NR**
Drilling Method **Continuous Flight Auger**	Drill Bit Size/Type **6 inch rock bit**	Total Depth of Borehole **40 feet bgs**
Drill Rig Type **Truck Mounted CME 55**	Drilling Contractor **Britton Exploration, Inc.**	Approximate Surface Elevation **490 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, pneumatic**
Borehole Backfill **Cuttings**	Location **East of proposed residence (Lot 3)**	

Elevation, feet	Depth, feet	Sample Type	Sampling Resistance, blows/foot	Relative Consistency	USCS Symbol	MATERIAL DESCRIPTION	Water Content, %	Dry Density (PCF)
	0		12	Stiff to Hard	ML	FILL: SANDY SILT, yellowish brown, heterogeneous, low plasticity, fine to very fine sand, fine to coarse, subrounded to subangular gravels, irregular pockets of sandy clay in upper 2 feet, trace rootlets in upper 2 feet, slightly moist	9	
488			16				8	
			13				7	
	5		34	Hard	ML	SANDY SILT with GRAVEL, yellowish brown, 30-60% sandstone fragments in a sandy silt matrix, fine to coarse, subrounded to subangular gravels, slightly moist (Colluvium)	13	
483			34	Soft*	BR		7	
			27			SANDSTONE, light yellowish brown, severely weathered, friable, fine to medium grained, slight iron and manganese oxide staining, moist to slightly moist (Franciscan Complex)	9	119
478	10							
			61				10	
473	15							
			61				15	108
468	20							
			32				6	
463	25							
			87/9"				6	
458	30							
			83/11"	Soft*	BR	SHEARED ROCK, dark yellowish brown, moderately to severely weathered, highly fractured, fragments of less weathered dark gray shale in a sandy matrix, some iron oxide staining, moist (Franciscan Complex)	7	
453	35							
			34			*designates hardness of bedrock (see Figure B-9)	9	
	40					Bottom of Boring at 40 feet bgs		
448								

Date(s) Drilled	**December 20, 2013**	Logged By	**KP**	Checked By	**NR**
Drilling Method	**Continuous Flight Auger**	Drill Bit Size/Type	**6 inch rock bit**	Total Depth of Borehole	**40 feet bgs**
Drill Rig Type	**Truck Mounted CME 55**	Drilling Contractor	**Britton Exploration, Inc.**	Approximate Surface Elevation	**485 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, pneumatic**
Borehole Backfill	**Cuttings**	Location	**East of proposed residence (Lot 4)**		

Elevation, feet	Depth, feet	Sample Type	Sampling Resistance, blows/foot	Relative Consistency	USCS Symbol	MATERIAL DESCRIPTION	Water Content, %	Dry Density (PCF)
	0		25	Very Stiff to Hard	ML	FILL: SANDY SILT with GRAVEL, yellowish brown, slightly heterogeneous, low plasticity, fine to very fine sand, subangular fragments of greenstone and sandstone, slightly moist	9	
483			55				9	
			24				10	110
	5		62				14	
478			37	Very Stiff to Hard	ML	SANDY SILT, yellowish brown, homogeneous, low plasticity, fine to very fine sand, ~5% sandstone fragments, slightly moist (Colluvium)	8	
			20				8	
	10			Soft	BR	SANDSTONE, light yellowish brown, severely weathered, friable, fine to medium grained, slight iron and manganese oxide staining, interbeds of greenstone at 13.5 to 15 feet, moist to slightly moist (Franciscan Complex)		
473			73				6	
	15							
468			50/2"				5	
	20							
463			50/5"				8	
	25							
458			50/3"	Soft*	BR	SHEARED ROCK, dark yellowish brown, moderately to severely weathered, highly fractured, fragments of less weathered dark gray shale in a sandy matrix, some iron oxide staining, moist (Franciscan Complex)	8	127
	30							
453			50/2"			*designates hardness of bedrock (see Figure B-9)	7	
	35							
448			50/1"					
	40					Bottom of Boring at 40 feet bgs		
443								

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MURRAY ENGINEERS INC GEOTECHNICAL SERVICES

ZMAY RESIDENTIAL SUBDIVISION
1551 CRYSTAL SPRINGS ROAD
SAN MATEO COUNTY, CALIFORNIA

LOG OF BORING B-2

PROJECT NO. 1847-1R1 | **FEBRUARY 2014** | **FIGURE B-2**

Date(s) Drilled **December 20, 2013**	Logged By **KP**	Checked By **NR**
Drilling Method **Continuous Flight Auger**	Drill Bit Size/Type **6 inch rock bit**	Total Depth of Borehole **35 feet bgs**
Drill Rig Type **Truck Mounted CME 55**	Drilling Contractor **Britton Exploration, Inc.**	Approximate Surface Elevation **475 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, pneumatic**
Borehole Backfill **Cuttings**	Location **Northeast corner of proposed residence (Lot 2)**	

Elevation, feet	Depth, feet	Sample Type	Sampling Resistance, blows/foot	Relative Consistency	USCS Symbol	MATERIAL DESCRIPTION	Water Content, %	Dry Density (PCF)
	0		11	Stiff	ML	SANDY SILT, yellowish brown, homogeneous, low plasticity, fine to very fine sand, ~5% sandstone fragments, slightly moist (Colluvium)	12	105
473			14	Stiff to Very Stiff	CL		16	116
			19			SILTY CLAY, yellowish brown, homogeneous, low to medium plasticity, minor fine sand, slight iron oxide staining, moist (Colluvium)	13	
	5		85	Soft*	BR	- PI=11%; LL=41% (sample from 1.5 to 3 feet)	8	
468			85				7	
			48			SANDSTONE, light yellowish brown, severely weathered, friable, fine to medium grained, slight iron and manganese oxide staining, moist to slightly moist (Franciscan Complex)	8	
	10							
463						*designates hardness of bedrock (see Figure B-9)		
	15		39				6	
458								
	20		65				9	
453								
	25		41			SANDSTONE, consists of fractured, intact sandstone fragments in a very severely to completely weathered matrix below ~23.5 feet (Franciscan Complex)	12	
448								
	30		46				11	
443								
	35		72				8	
						Bottom of Boring at 35 feet bgs		
438								
	40							
433								

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MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

ZMAY RESIDENTIAL SUBDIVISION 1551 CRYSTAL SPRINGS ROAD SAN MATEO COUNTY, CALIFORNIA	LOG OF BORING B-3
PROJECT NO. 1847-1R1 — FEBRUARY 2014	FIGURE B-3

Date(s) Drilled **December 23, 2013**	Logged By **KP**	Checked By **NR**
Drilling Method **Continuous Flight Auger**	Drill Bit Size/Type **6 inch rock bit**	Total Depth of Borehole **38.6 feet bgs**
Drill Rig Type **Truck Mounted CME 55**	Drilling Contractor **Britton Exploration, Inc.**	Approximate Surface Elevation **480 feet (relative)**
Groundwater Level and Date Measured **28 feet ATD**	Sampling Method(s) **3" OD, 2.5" OD, & 2" OD SPT Split Spoon Samplers**	Hammer Data **140 lb, 30 in drop, pneumatic**
Borehole Backfill **Cuttings**	Location **West side of proposed residence (Lot 1)**	

Elevation, feet	Depth, feet	Sample Type	Sampling Resistance, blows/foot	Relative Consistency	USCS Symbol	MATERIAL DESCRIPTION	Water Content, %	Dry Density (PCF)
	0		11	Stiff	ML	SANDY SILT, yellowish brown, heterogeneous, low plasticity, fine to very fine sand, fine to coarse, subrounded to subangular gravels, irregular pockets of sandy clay in upper 2 feet, trace rootlets in upper 2 feet, slightly moist (Colluvium)	14	
478			28	Soft*	BR		12	117
			53				9	
	5		9			SANDSTONE, light yellowish brown, severely weathered, friable, fine to medium grained, slight iron and manganese oxide staining, moist to slightly moist (Franciscan Complex)	6	
473			89				7	
			53				7	
	10					*designates hardness of bedrock (see Figure B-9)		
468								
			44				9	
	15							
463								
			18				10	
	20							
458								
			17				14	
	25					Transition to fractured sandstone fragments in a saturated weathered shale matrix ~24 to 30 feet		
453						(ATD) ▽		
			50/3"				16	
	30			Soft*	BR	SHEARED ROCK, dark yellowish brown, moderately to severely weathered, highly fractured, fragments of less weathered shale in a sandy matrix, some iron oxide staining, moist (Franciscan Complex)		
448								
			50/2"					
	35							
443								
			50/1"					
						Bottom of Boring at 38.6 feet bgs		
	40							
438								

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MURRAY ENGINEERS INC GEOTECHNICAL SERVICES

ZMAY RESIDENTIAL SUBDIVISION 1551 CRYSTAL SPRINGS ROAD SAN MATEO COUNTY, CALIFORNIA		LOG OF BORING B-4
PROJECT NO. 1847-1R1	FEBRUARY 2014	FIGURE B-4

Date(s) Drilled **December 23, 2013**	Logged By **KP**	Checked By **NR**
Drilling Method **Continuous Flight Auger**	Drill Bit Size/Type **6 inch rock bit**	Total Depth of Borehole **22.7 feet bgs**
Drill Rig Type **Track Mounted CME**	Drilling Contractor **Britton Exploration, Inc.**	Approximate Surface Elevation **465 feet (relative)**
Groundwater Level and Date Measured **18 feet ATD**	Sampling Method(s) **3" OD, 2.5" OD, & 2" OD SPT Split Spoon Samplers**	Hammer Data **140 lb, 30 in drop, pneumatic**
Borehole Backfill **Cuttings**	Location **Top of active landslide (Lot 1)**	

Elevation, feet	Depth, feet	Sample Type	Sampling Resistance, blows/foot	Relative Consistency	USCS Symbol	MATERIAL DESCRIPTION	Water Content, %	Dry Density (PCF)
463	0		22 29 19	Very Stiff	ML	SANDY SILT, yellowish brown, heterogeneous, low plasticity, fine to very fine sand, fine to coarse, subrounded to subangular gravels, irregular pockets of sandy clay in upper 2 feet, trace rootlets in upper 2 feet, slightly moist (Colluvium)	10 7 13	115 120
458	5		50/6" 89 44	Soft*	BR	SANDSTONE, light yellowish brown, severely weathered, friable, fine to medium grained, slight iron and manganese oxide staining, moist to slightly moist (Franciscan Complex) *designates hardness of bedrock (see Figure B-9)	10 6 9	 151
453 448 443	10 15 20		20 72 50/2"	Soft*	BR	SHEARED ROCK, dark grayish brown, 80-90% subangular shale fragments in a saturated silty sand matrix, intensely fractured, saturated (Franciscan Complex) (ATD) ▽	10 7 10	
438 433 428 423	25 30 35 40					Refusal at 22.7 feet bgs		

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MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	ZMAY RESIDENTIAL SUBDIVISION 1551 CRYSTAL SPRINGS ROAD SAN MATEO COUNTY, CALIFORNIA	LOG OF BORING B-5
PROJECT NO. 1847-1R1	FEBRUARY 2014	FIGURE B-5

Date(s) Drilled **December 23, 2013**	Logged By **KP**	Checked By **NR**
Drilling Method **Continuous Flight Auger**	Drill Bit Size/Type **6 inch rock bit**	Total Depth of Borehole **18.1 feet bgs**
Drill Rig Type **Truck Mounted CME 55**	Drilling Contractor **Britton Exploration, Inc.**	Approximate Surface Elevation **450 feet (relative)**
Groundwater Level and Date Measured **6.5 feet ATD**	Sampling Method(s) **3" OD, 2.5" OD, & 2" OD SPT Split Spoon Samplers**	Hammer Data **140 lb, 30 in drop, pneumatic**
Borehole Backfill **Cuttings**	Location **Upper center of active landlside (Lot 1)**	

Elevation, feet	Depth, feet	Sample Type	Sampling Resistance, blows/foot	Relative Consistency	USCS Symbol	MATERIAL DESCRIPTION	Water Content, %	Dry Density (PCF)
450	0		16	Stiff	ML	SANDY SILT, yellowish brown, heterogeneous, low plasticity, fine to very fine sand, fine to coarse, subrounded to subangular gravels, irregular pockets of sandy clay in upper 2 feet, trace rootlets in upper 2 feet, slightly moist (Landslide Deposits)	11	
			16	Medium Stiff to Very Stiff	ML		15	114
			26			SANDY SILT, light grayish brown, slightly heterogeneous, low to medium plasticity, very fine to medium sand, subangular rock fragments, moderate iron oxide staining, abundant rootlets, moist (Landslide Deposits)	12	
445	5		12	Medium Stiff to Stiff	ML	- PI=11%; LL=29% (sample from 3 to 4.5 feet)	17	92
			5			SANDY SILT with GRAVEL, dark grayish brown, 40-70% rock fragments in a sandy silt matrix, saturated (Landslide Deposits) (ATD)	15	
			13			Soft saturated zone ~6 to 8 feet base of landslide	12	
440	10		33	Soft*	BR	SHEARED ROCK, dark yellowish brown, moderately to severely weathered, highly fractured, fragments of less weathered shale in a sandy matrix, some iron oxide staining, moist (Franciscan Complex)	11	
			39				6	
			43			*designates hardness of bedrock (see Figure B-9)	13	
435	15							
			50/1"			Refusal at 18.1 feet bgs		
430	20							

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MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

ZMAY RESIDENTIAL SUBDIVISION 1551 CRYSTAL SPRINGS ROAD SAN MATEO COUNTY, CALIFORNIA	LOG OF BORING B-6
PROJECT NO. 1847-1R1 — FEBRUARY 2014	FIGURE B-6

Elevation, feet	Depth, feet	Sample Type	Sampling Resistance, blows/foot	Relative Consistency	USCS Symbol	MATERIAL DESCRIPTION	Water Content, %	Dry Density (PCF)
1	2	3	4	5	6	7	8	9

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 to advance driven sampler per foot (or distance shown) beyond seating interval. Blow counts for coarse-grained soils have been standardized to Standard Penetration Test (SPT) counts by factors of 0.8 and 0.7 for the 2.5-inch OD and 3.0-inch OD samplers, respectively. These factors were derived using the Geology Field Manual (2001), published by the U.S. Bureau of Reclamation.

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.

9 **Dry Density (PCF):** Dry weight per unit volume of soil sample measured in laboratory in pounds per cubic foot.

FIELD AND LABORATORY TEST ABBREVIATIONS

CHEM: Chemical tests to assess corrosivity
COMP: Compaction test
CONS: One-dimensional consolidation test
LL: Liquid Limit, percent
PI: Plasticity Index, percent

SA: Sieve analysis (percent passing No. 200 Sieve)
UC: Unconfined compressive strength test, Qu, in ksf
WA: Wash sieve (percent passing No. 200 Sieve)

TYPICAL MATERIAL GRAPHIC SYMBOLS

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

TYPICAL SAMPLER GRAPHIC SYMBOLS

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

OTHER GRAPHIC SYMBOLS

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

GENERAL NOTES

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

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MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

ZMAY RESIDENTIAL SUBDIVISION 1551 CRYSTAL SPRINGS ROAD SAN MATEO COUNTY, CALIFORNIA		KEY TO BORING LOGS
PROJECT NO. 1847-1R1	FEBRUARY 2014	FIGURE B-7

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 OD split spoon sampler; Blow counts for coarse-grained soils have been standardized to SPT counts by factors of 0.8 and 0.7 for the 2.5-inch OD and 3.0-inch OD samplers, respectively.

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

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 except 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 a 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 brocken 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 AND FOLIATION SPACING

Spacing	Joints	Bedding and 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

LABORATORY TESTS

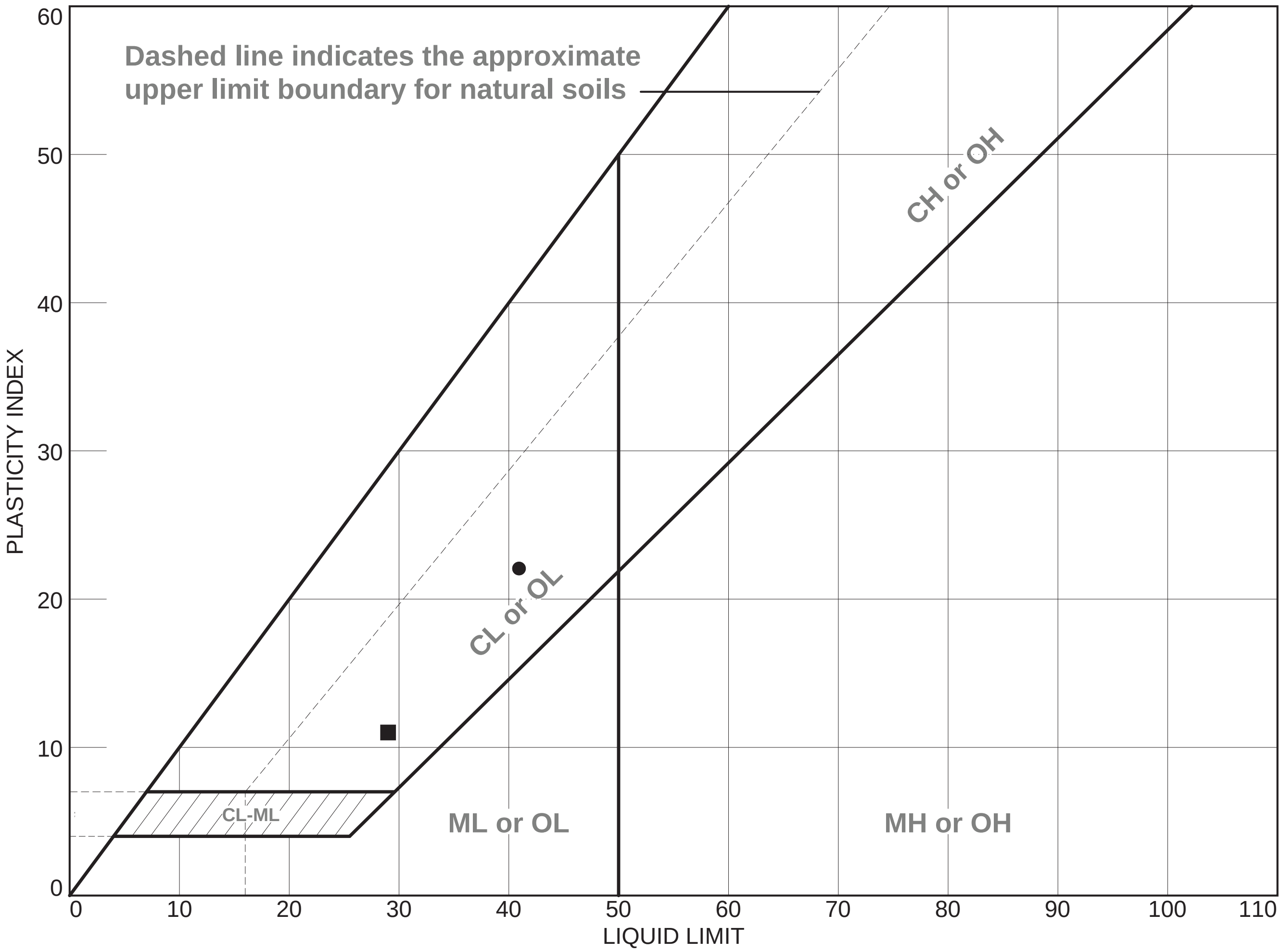
Samples from the subsurface exploration were selected for tests to establish the physical and engineering properties of the soils. The tests performed are briefly described below.

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

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

Direct shear strength testing was performed by Cooper Testing Laboratory on one sample in accordance with ASTM D3080m. This test measures the angle of internal friction (phi) and cohesion (C) of the soil. The results of this test are presented as Figure C-2.

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	Boring 3	1	1.5' to 3'	15.2	19	41	22	CL
■	Boring 6	2	3' to 4.5'	11.8	18	29	11	CL

MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

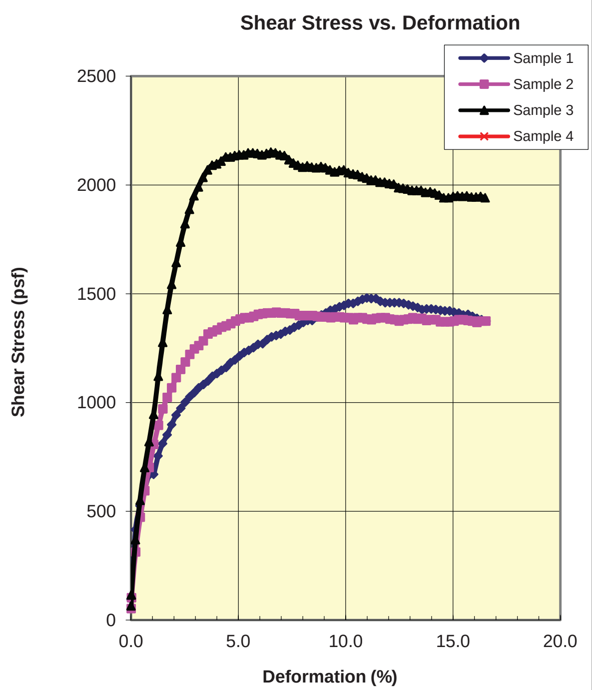
ZMAY RESIDENTIAL SUBDIVISION 1551 CRYSTAL SPRINGS ROAD SAN MATEO COUNTY, CALIFORNIA	LIQUID & PLASTIC LIMITS TEST REPORT
PROJECT NO. 1847-1R1 — FEBRUARY 2014	FIGURE C-1

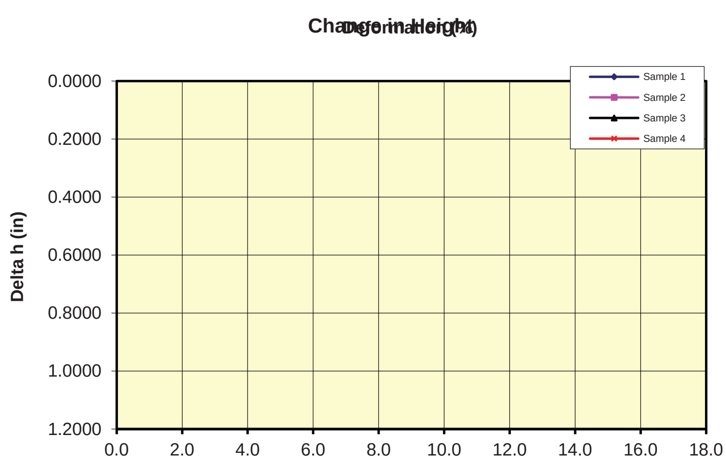
Consolidated Undrained Direct Shear
(ASTM D3080M)

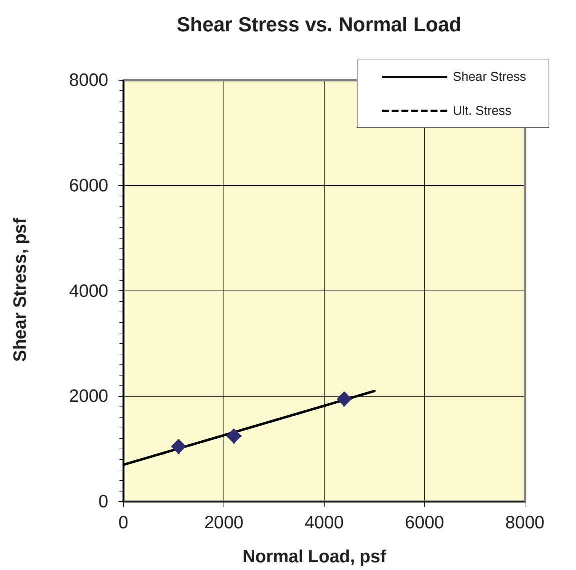
CTL Job #:	560-131	Project #:	1847-1	By:	MD
Client:	Murray Engineers	Date:	1/21/2014	Checked:	PJ
Project Name:	Zmay	Remolding Info:			

Specimen Data	1	2	3	4
Boring:	B-6	B-6	B-6	
Sample:	1	1	1	
Depth (ft):	9-10.5	9-10.5	9-10.5	
Visual Description:	Dark Olive Brown Sandy CLAY w/ Gravel	Dark Olive Brown Sandy CLAY w/ Gravel	Dark Olive Brown Sandy CLAY w/ Gravel	
Normal Load (psf)	1100	2200	4400	
Dry Mass of Specimen (g)	138.0	139.0	141.9	
Initial Height (in)	1.01	1.00	1.00	
Initial Diameter (in)	2.42	2.42	2.42	
Initial Void Ratio	0.509	0.492	0.464	
Initial Moisture (%)	15.4	13.4	12.5	
Initial Wet Density (pcf)	131.2	130.5	131.9	
Initial Dry Density (pcf)	113.7	115.1	117.3	
Initial Saturation (%)	82.9	75.0	74.1	
ΔHeight Consol (in)	0.0053	0.0139	0.0270	
At Test Void Ratio	0.501	0.471	0.425	
At Test Moisture (%)	18.1	16.6	15.0	
At Test Wet Density (pcf)	135.1	136.2	138.7	
At Test Dry Density (pcf)	114.4	116.8	120.6	
At Test Saturation (%)	99.3	96.8	97.0	
Strain Rate (%/min)	1.1	1.0	1.1	
Strengths Picked at	3%	3%	3%	
Shear Stress (psf)	1046	1246	1950	
ΔHeight (in) at 3%				
Ultimate Stress (psf)				

Phi (deg)	15.6	Ult. Phi (deg)	
Cohesion (psf)	700	Ult. Cohesion (psf)	







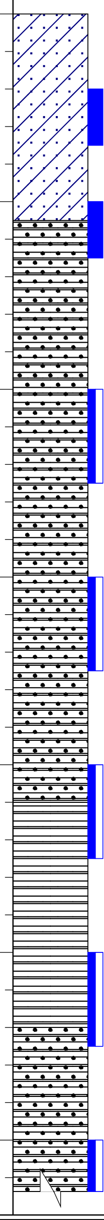
MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	ZMAY RESIDENTIAL SUBDIVISION 1551 CRYSTAL SPRINGS ROAD SAN MATEO COUNTY, CALIFORNIA	DIRECT SHEAR TEST CHART FOR BORING B-6 9-10.5 FEET BGS
	PROJECT NO. 1847-1R1 / FEBRUARY 2014	FIGURE C-2

BvGG ENGINEERS

BORING LOG

JOB NAME: Proposed Residential Subdivision
CLIENT: S.W. Syme Properties, Inc.
LOCATION: 1551 Crystal Springs Road Hillsborough, California
DRILLER: Pitcher Drilling, Inc.
DRILL METHOD: 5" Rotary Wash, Fraste Multidrill XL, Track Mounted

JOB NO.: SWSYM-01-00
DATE DRILLED: 7/18-19/07
ELEVATION: 282± feet
LOGGED BY: MRB
CHECKED BY:

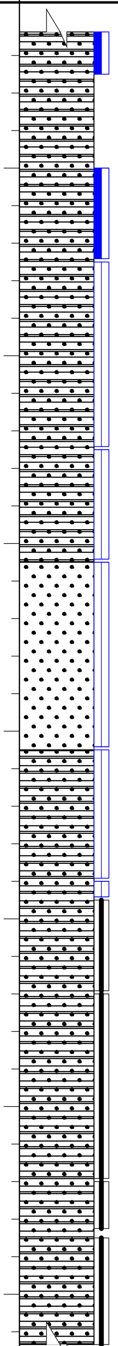


Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						0		CL	SANDY LEAN CLAY, light yellowish brown, dry, very stiff, medium plasticity, little silt	Disturbed soil
							13 26 29	CL	SANDY LEAN CLAY, light yellowish brown, moist, hard, medium plasticity, little silt	Residual soil
						5	24 40 48	ROCK	MELANGE, shale, sandstone, and clay matrix, medium grayish orange and dark gray, intensely weathered, medium soft, extremely fractured	Franciscan Formation
						10				5 min/ft @1200 psi 100% recover
						15				8 min/ft @1200 psi 80% recovery
						20				4 min/ft @800 psi 47% recovery
TXCU	1600	16.7	1300	19.1	114				...shale and clay matrix with little sandstone, dark brownish gray, moist, soft, decomposed to intensely weathered	
						25			...zone of fractured rock	9 min/ft @500 psi 83% recovery
									...shale, sandstone and clay matrix	
						30				8 min/ft @800 psi 67% recovery

BORING LOG

Boring No. ERWB-2

JOB NAME: Proposed Residential Subdivision *JOB NO.:* SWSYM-01-00



Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						35				9 min/ft @900 psi 30% recovery
										2.4 min/ft @900 psi RQD = 0% 28% recovery
						40				
										5.3 min/ft @ 500 psi RQD = 0% 25% recovery
						45			...dominately sandstone with seams of shale and clay matrix, moderately weathered, hard, very closely fractured	9.2 min/ft @ 500 psi RQD = 0% 87% recovery
						50			...melange (shale, sandstone and clay matrix)	20 min/ft @ 500 psi RQD = 0% 0% recovery
										4 min/ft @ 500 psi RQD = 0% 0% recovery
						55				5 min/ft @ 1000 psi 77% recovery
										3 min/ft @ 1000 psi 37% recovery
						60			...grayish yellow	4 min/ft @ 1000 psi 88% recovery
						65				3 min/ft @ 1000 psi

BORING LOG

Boring No. ERWB-2

JOB NAME: Proposed Residential Subdivision *JOB NO.:* SWSYM-01-00

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
										0% recovery
									...sandy clay matrix	3 min/ft @ 1000 psi 0% recovery
						70				2 min/ft @ 800 psi 75% recovery
										2 min/ft @ 500 psi 33% recovery
						75			...dark brownish gray	3 min/ft @ 400 psi 100% recovery
										3 min/ft @ 700 psi 58% recovery
						80				2 min/ft @ 1000 psi 67% recovery
										3 min/ft @ 1000 psi 97% recovery
						85				2 min/ft @ 1000 psi 63% recovery
										4 min/ft @ 1000 psi 0% recovery
						90			...12" zone of sandstone	3 min/ft @ 1800 psi 75% recovery
										4 min/ft @ 1500 psi 31% recovery
						95				4 min/ft @ 2000 psi 83% recovery
									Boring terminated at 97.5 feet. Backfilled with neat cement.	3 min/ft @ 800 psi 100% Recovery
						100				

BORING LOG

Boring No. EB-1

BAGG ENGINEERS

JOB NAME: Proposed Residential Subdivision
CLIENT: S.W. Syme Properties, Inc.
LOCATION: 1551 Crystal Springs Road Hillsborough, California
DRILLER: Clear Heart Drilling
DRILL METHOD: Tracked Morooka drill rig w/ 4½" diameter flight augers

JOB NO.: SWSYM-01-00
DATE DRILLED: 10/02/07
ELEVATION: 472± feet
LOGGED BY: DCL
CHECKED BY:

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						0		CL	SANDY LEAN CLAY, light brown, dry, hard, medium plasticity	Colluvium
DS	320	Nat.	3400	13.9	110		15 26 36	SC	...silty, increasing fine sand content	Residual Soil
DS	1700	Nat.	4000	13.1	113					
								ROCK	CLAYEY SAND, light tan brown with orange brown, dry, very dense	Franciscan Formation
DSX	2100	19.4	1850	14.1	111	5	35 50-5"			
							50-2"		MELANGE, sandstone and claystone, buff and light orange brown, dry, decomposed, very soft	uniform drilling to 8' depth, then softer drilling to 12' depth
				5.4	111	10	50		no recovery	Bedrock
									sandstone, buff and light orange brown, dry, soft, decomposed to intensely weathered	
DS	1700	Nat.	2350	7.7	130		23 44 50		...light brown and orange brown	
						15				
DS	2000	Nat.	2400	11.0	120		36 50-4"			
						20	50-3"		...light orange brown and gray, with clayey infilling	
									...dark gray, intensely to moderately weathered	harder drilling
				12.8	107		50-4" 50-5"		...fractured sandstone, increasing moisture content	softer drilling at 23' depth
						25			Boring terminated at 24 feet in drilling refusal. Tremie grouted with neat cement.	
						30				

BVGG ENGINEERS

BORING LOG

Boring No. EB-2

JOB NAME: Proposed Residential Subdivision
CLIENT: S.W. Syme Properties, Inc.
LOCATION: 1551 Crystal Springs Road Hillsborough, California
DRILLER: Clear Heart Drilling
DRILL METHOD: Tracked Morooka drill rig w/ 4½" diameter flight augers

JOB NO.: SWSYM-01-00
DATE DRILLED: 10/01/07
ELEVATION: 281± feet
LOGGED BY: DCL
CHECKED BY:

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						0		CL	SANDY LEAN CLAY, with silt, brown, dry, hard, low to medium plasticity	Colluvium
DS DS	320 1700	Nat. Nat.	1950 6000	11.7 11.6	111 121		24 31 32			
						5			moist, with trace coarse-grained sand	firm drilling
				12.7	117		25 39 38			
									increasing fine sand content	
								SC	CLAYEY SAND WITH FINE GRAVEL, light orange brown, moist, very dense	Residual Soil
DS DS	1100 2600	Nat. Nat.	3700 3800	11.9 11.9	119 118	10	26 37 50			
									with gravels up to 1½" in size	very firm drilling
									olive brown with orange brown, increasing fines content	
				10.2	118		50-5"			
						15				very firm drilling
								ROCK	MELANGE, shale, gray, decomposed, dry, soft, with thin laminations	Francscican Formation very difficult drilling
				7.7	153	20	16 23 37			
									Boring terminated at 21.5 feet. Backfilled with neat cement.	No groundwater encountered.
						25				
						30				

BORING LOG

Boring No. EB-3

JOB NAME: Proposed Residential Subdivision
CLIENT: S.W. Syme Properties, Inc.
LOCATION: 1551 Crystal Springs Road Hillsborough, California
DRILLER: Clear Heart Drilling
DRILL METHOD: Tracked Morooka drill rig w/ 4½" diameter flight augers

JOB NO.: SWSYM-01-00
DATE DRILLED: 10/02/07
ELEVATION: 407± feet
LOGGED BY: DCL
CHECKED BY:

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						0		CL	SANDY LEAN CLAY, light orange brown, dry, very stiff, medium plasticity	Colluvium
DS	320	Nat.	3200	12.4	11		12 20 26			
DS	1700	Nat.	4000	12.3	112					
									...decreasing sand content	
				3.7	113	5	21 30 23			
								CH	FAT CLAY WITH GRAVEL, olive gray, moist, very stiff, high plasticity	Residual Soil
DS	1200	Nat.	3000	13.6	124	10	16 25 36			
DS	2700	Nat.	2450	18.6	111					
								ROCK	MELANGE, sandstone, shale, and claystone, light brown, light tan brown, and brown, dry, decomposed, very soft	Franciscan Formation
DS	1600	Nat.	1350	8.7	113	15	16 20 23			
DS	3000	Nat.	2700	9.6	122				...with intensely weathered to moderately weathered fragments	
									sandstone, light tan brown, dry, intensely weathered, soft to moderately soft	
						20			Boring terminated at 19.0 feet in drilling refusal. Backfilled with neat cement.	No groundwater encountered.
						25				
						30				

BORING LOG

Boring No. EB-10

JOB NAME: Proposed Residential Subdivision
CLIENT: S.W. Syme Properties, Inc.
LOCATION: 1551 Crystal Springs Road Hillsborough, California
DRILLER: Clear Heart Drilling
DRILL METHOD: Hydraulic portable w/ 4½" diameter flight augers

JOB NO.: SWSYM-01-00
DATE DRILLED: 10/02/07
ELEVATION: 446± feet
LOGGED BY: DCL
CHECKED BY:

Type of Strength Test	Test Surcharge Pressure, psf	Test Water Content, %	Shear Strength, psf	In-Situ Water Content, %	In-Situ Dry Unit Weight, pcf	Depth, ft.	Soil Symbols, Samplers and Blow Counts	USCS	Description	Remarks
						0		CL	SANDY LEAN CLAY WITH GRAVEL, light orange brown and light tan, dry, very stiff, medium plasticity	Colluvium
				12.1	111		11 10 15		...light brown ...brown with olive brown	
				12.7	120	5	11 11 13		...decreasing sand content ...brown and dark olive, moist, with slickensides, decomposed rock fragments	
								ROCK	MELANGE, shale, dark gray, intensely weathered, wet, soft, with clayey, saturated infilling	Franciscan Formation
DS	1000	Nat.	1400	12.3	120		26 50			
DS	2500	Nat.	3150	7.3	124					
DS	1200	Nat.	1370	13.7	120	10	28 16 17		...with sandstone fragments	
DS	2700	Nat.	2200	12.6	124					
									sandstone, dark gray, wet, intensely weathered, soft to moderately soft ...firmer, moist	very stiff drilling
				8.4	140	15	17 23 50			
									Boring terminated at 15.5 feet. Backfilled with neat cement.	
						20				
						25				
						30				

County of San Mateo - Planning and Building Department

ATTACHMENT L

March 18, 2015
Project No. 1847-1L2

Nick Zmay
1551 Crystal Springs Road
Hillsborough, California 94010

RE: SUPPLEMENTAL EVALUATION & RESPONSE TO REVIEW COMMENTS, ZMAY PROPERTY, 1551 CRYSTAL SPRINGS ROAD, SAN MATEO COUNTY, CALIFORNIA

Dear Mr. Zmay:

As requested, we have prepared this letter in response to the County of San Mateo's geotechnical review sheet dated December 4, 2014. We have previously conducted an engineering geologic and geotechnical investigation for the development of a four-lot residential subdivision (each containing 2 acres) on the property located at 1551 Crystal Springs Road in an unincorporated area of San Mateo County, near Hillsborough. Our original report was dated February 10, 2014, and summarized the results of our investigation and presented geotechnical design recommendations for the proposed residential subdivision. We prepared a supplemental letter regarding the updated subdivision building envelopes, dated August 26, 2014. In the review sheet, the County presented two review comments. Comment No. 1 requests a supplemental geologic and geotechnical investigation addressing the five sub-comments contained within Comment No. 1. As a part of their comments, they have requested we perform a limited evaluation of the remaining 48 acres of the property. The results of our additional evaluations are presented below, followed by our responses to the County comments. Our responses to the review comments are presented in the same order in which they appear on the geotechnical review sheet.

PROJECT DISCUSSION

Geologic Review

The entire approximate 60 acre property is located on a west-facing hillside in the foothills along the northeast side of the Santa Cruz Mountains, a northwest-trending range within the California Coast Ranges geomorphic province. The local topography is dominated by a series of west-trending spur ridges and intervening seasonal drainage swales. Crystal Springs Road extends along the western property boundary at the base of the hillside and converges with Polhemus Road near the southern corner of the property. San Mateo Creek and Polhemus Creek run parallel to Crystal Springs Road and Polhemus Road, respectively. Elevations across the site range from approximately 500 feet along Parrott Drive in the eastern portion of the site down to approximately 140 feet above mean sea level at the base of the hillside in the northwest corner of the site (see Figure A-1 of Murray Engineers Inc. (MEI's) 2014 report).

According to the Geologic Map of the Montara Mountain and San Mateo 7-½' Quadrangles (Pampeyan, 1994), the site is located in an area underlain by Cretaceous and Jurassic age (approximately 65 to 200 million years old) sheared rock of the Franciscan Complex (fsr).

The sheared rock generally consists of soft, light- to dark-gray, sheared shale, siltstone, and greywacke sandstone containing various-size tectonic inclusions of Franciscan rock types. According to the geologic map, the lower portion of the slope in the northwest corner of the property is blanketed by Quaternary slope wash, ravine fill, and colluvium deposits (Qsr). These deposits generally consist of unconsolidated to moderately consolidated sand, silt, clay, and rock fragments accumulated by slow downslope movement of weathered rock debris and soil. A copy of the relevant portion of the geologic map is presented on Figure A-3, Vicinity Geologic Map, of MEI's 2014 report.

According to the geologic map, the Geotechnical Hazard Synthesis Map for San Mateo County (Leighton and Associates, 1976), and the Preliminary Map of Landslide Deposits in San Mateo County (Brabb & Pampeyan, 1972), three relatively large landslides are mapped in the central portion of the property. According to the geologic map, the largest feature measures approximately 900 feet in length and 600 feet in width. The upper margin of this feature is located approximately 350 feet to the west (downhill) of Parrott Drive and extends down to Crystal Springs Road. The second mapped landslide is approximately 700 feet long and 500 feet wide and is located immediately south of the first landslide. In addition, smaller landslide features are mapped in the southern portion of the lot and at the northeast corner just off the property. The relevant portions of these maps are included as Figure A-4, San Mateo County Landslide Map and Figure A-5, San Mateo County Geotechnical Hazard Synthesis Map, of MEI's 2014 report.

Previous Relevant Geologic & Geotechnical Investigations

A full discussion of prior geologic and geotechnical investigations was provided in Murray Engineers Inc. (MEI's) 2014 engineering geologic and geotechnical report. However, because the report focused on the subdivision of 8 acres in the upper northeast portion of the property, portions of previous investigations were not discussed in the report. Therefore, we will summarize the relevant information contained in prior reports as it pertains to the County's review comments, listed below; specifically, with respect to the property as a whole and not solely focused on the northeastern portion proposed to be subdivided. For additional information not discussed below, please refer to MEI's 2014 report.

Site Characteristics, Inc. (SCI) conducted a geotechnical investigation on the property, dated July 1983, to address three proposed single family residences along Crystal Springs Road in the northwest lower portion of the property. Subsequently, William Cotton and Associates (WCA) performed a supplemental geotechnical analysis and presented the results in a report dated April 20, 1984. Based on site reconnaissance, subsurface investigations, and slope stability analyses, both consultants indicated that although there were several shallow landslide and slump features on the property, there was no evidence of recent slope instability or of debris flows on the property.

In 2007, Bay Area Geotechnical Group (BAGG) performed a geotechnical and engineering geologic investigation for a proposed 20-lot residential subdivision of the subject property. The results of the investigation were presented in a report dated December 20, 2007. As part of the investigation, BAGG excavated six relatively deep borings within the landslide areas and nine additional borings on the remaining portions of the property, and performed laboratory testing on samples, including triaxial shear and direct shear testing. The locations of these borings are shown on Figure 1. The results of BAGG's slope stability analyses are discussed in MEI's 2014 report.

In general, BAGG's borings encountered approximately 5 feet of colluvial soil underlain by bedrock associated with the Franciscan Complex. However, Borings B-2 and B-3, located in the northern portion of the property, encountered approximately 17.5 and 12 feet of colluvial soil, respectively, and Borings B-7 and B-8, located in the southern portion of the property, encountered approximately 14.5 and 12 feet of colluvial soil, respectively. According to BAGG, the colluvial soil consists of stiff to very stiff, low to medium plasticity, lean clay, sandy clay, gravelly clay, and silty gravel. The sixteen borings advanced by BAGG all encountered bedrock at depths of approximately 2 to 17.5 feet, consisting of Franciscan materials with varying degrees of weathering and shearing in a clayey matrix. Based on the subsurface investigation, BAGG formed the opinion that although numerous landslide and slump features were found on the property, site development was feasible outside the mapped slide areas.

Aerial Photography Review

Four sets of historical aerial photographs taken between 1943 and 1974 were reviewed at the U.S. Geologic Survey's library in Menlo Park to aid in evaluating the presence of geomorphic features that may be suggestive of landsliding on the entire 60 acre property. The site is readily identifiable in all of the photographs, based on the topography and the location of Parrott Drive, Crystal Springs Road, and Polhemus Road. Other than the development of the neighboring residential properties, there is very little change in the vicinity of the property during the period covered by the photographs. In the 1943 and 1946 photographs, the streets are present but there is no other development in the vicinity of the property. By the time of the 1968 photographs, most of the homes along Parrott Drive are complete and the building pad on the property immediately northeast of the property appears to be graded. In addition, it appears that improvements were made to Parrott Drive and that additional fill was placed along the downhill side of the roadway. The residences that currently exist along Parrott Drive are present by the time of the 1974 photographs.

In the 1943 and 1946 photographs, two large landslides are present in the central portion of the property, similar to mapping by Pampeyan. The landslides are characterized by broad arcuate topography extending from the downhill side of Parrott Drive down to Crystal Springs Road. The ground surface within the limits of the landslides is generally hummocky with irregular medium to dense vegetation. A small debris flow appears to be located within the limits of the northern landslide. In addition, a debris flow (No. 24-see attached site plan) is located uphill of the southern landslide and drops into the upper portion of the landslide feature. The landslide masses are confined by drainage swales extending down the margins of the features to Crystal Springs Road. In addition, a large debris flow-type landslide complex, also mapped by Pampeyan, is located in the southern portion of the property. There are no signs of quarrying near the mapped quarry in either set of photographs.

It appears that sometime between 1946 and 1968, grading activities were conducted near the southeast property corner in the vicinity of Bel Air Road, Linden Lane, and Enchanted Way, presumably associated with the development of properties in this area. The 1968 photographs show a series of graded terraces, with residences built above, that appear to be relatively cleared of vegetation. The 1974 photographs show the same configuration of what appears to be artificial fill terraces constructed below the residences; however, the ground surface appears to be more vegetated and the terracing is less obvious. Although there is no conclusive evidence to suggest that this grading was conducted as part of a landslide repair, the grading appears to be coincident with the neighborhood located near the southeast property corner and is likely a result of neighborhood development.

In the 1968 photographs, an access road is present near the northeastern property corner. This road enters the subject property from Parrott Drive, extends across the uphill portion (roughly parallel to Parrott Drive) and to the graded pad on the adjacent northern property. It appears that sometime between 1968 and 1974, a small landslide occurred along the downhill side of this access road. A headscarp is present along the uphill margin of this arcuate feature in the 1974 photographs. No evidence of landsliding was observed immediately east of this feature, however, there is a tonal variation in the vegetation and the topography has a very subdued arcuate shape, suggesting that this area may be prone to shallow sliding.

In the 1968 and 1974 photographs, the quarry appears to be active or recently active, evidenced by a bare hillside with little to no vegetation. The mapped landslide immediately north of the quarry (on the eastern side of Crystal Springs Road) appears to have activated sometime between 1946 and 1968, possibly as a result of quarrying activities or due the generation of over-steepened road cuts in this area. A headscarp is present along the uphill margin of this arcuate feature in the 1968 and 1974 photographs and the ground surface within the limits of the landslide is generally hummocky.

The drainage swales located across the property are densely vegetated in the photographs. Any conclusive evidence suggestive of landsliding or debris flows is obscured along these channels.

Supplemental Geologic Mapping

As part of the supplemental evaluation, our project geologist and principal geotechnical engineer conducted additional limited geologic mapping on the property on March 2, 2015. The results of this supplemental geologic mapping and site reconnaissance are included on the Site Plan and Engineering Geologic Map (Figure 1). Due to the scale of the attached site plan and the large area encompassed by the property, we have identified the more significant landslide features on Figure 1 but note that there may be additional shallow features on the property that are not depicted on the map. A brief discussion of the prominent mapped features is included in MEI's 2014 report and the general locations of these features are shown on Figure 1. More detailed discussions of the property are presented in MEI's 2014 report.

As previously discussed, the site topography is dominated by a series of westerly-trending spur ridges and intervening drainage swales. The natural ground surface across the property is generally steep with gradients varying from 2:1 to 3:1 (horizontal to vertical) and moderately sloping across portions of the mapped landslides with gradients ranging from approximately 4:1 to 5:1. Steeper than 2:1 slopes are present, however, particularly along steep ravines associated with the seasonal drainage swales and pre-existing road and quarry cuts.

Below is a discussion of the landslide features mapped on Figure 1, moving north to south across the property. For ease of reference, these features discussed below are also numbered on Figure 1.

An active relatively shallow landslide (1) is located near the northeastern property corner within the proposed Lot 2 of the referenced 4-lot subdivision. Based on our review of aerial photographs, our site reconnaissance, and as previously discussed ion our referenced subdivision report, it appears that a 40-foot wide failure appears to have occurred along the downhill side of the graded access road, widening the area of the active landslide from what was previously mapped. This active landslide was absent from the 1943 and 1968 aerial photographs, but appeared in the latest photographs following construction of the graded access road (as discussed above). In our opinion, grading associated with construction of this road is likely the main probable cause of the landslide. It appears that this active landslide is less than 10 feet thick in depth.

An additional active, relatively shallow landslide (2) is located near the northwest property corner, along the road cut above Crystal Springs Road. Based on our site reconnaissance, this feature appears to be approximately 200 feet wide and approximately 100 feet in length. The slide mass is characterized by generally hummocky topography. In our opinion, grading associated with construction of Crystal Springs Road is likely the main probable cause for activation of the landslide. It appears that this active landslide is relatively shallow, likely less than 10 feet thick in depth. Two similar, smaller features (3 and 4) are located further south along Crystal Springs Road with slide mass dimensions of approximately 75 feet wide and approximately 25 feet in length and approximately 50 feet wide and approximately 60 feet in length, respectively.

A debris flow type feature (5) was initially mapped by SCI along the drainage swale below the active landslide in the northeastern property corner, below the proposed lots 2 and 3; however, this feature was questioned by WCA. This feature was subsequently mapped again by BAGG. Based on our site reconnaissance and aerial photograph review, a significant amount of erosion has occurred at the head of this feature; however, very dense vegetation obscures the topography. Additional small shallow landslide features (6 and 7) are located below the mapped debris flow, further down the subtle seasonal drainage swale.

Shallow debris flows (8) also appear to have occurred along the drainage ravine near the eastern property boundary (south of the proposed subdivision), as evidenced by evacuated headscarps along the northern side of the channel. It appears that these features are related to very steep slopes along either side of the ravine in addition to heavy precipitation during past rainfall events. The deeply incised drainage ravine appears to be acerbated by the presence of an existing culvert which discharges road runoff from Parrott Drive into the upper area of this feature. Several approximately 20- to 40-foot wide rotational landslide features (9, 10, and 11) are located on the north side of this channel, further downslope. A catchment basin is located near the base of this channel, approximately 20 feet east of the existing residence. A culvert routes water from the catchment basin, under the existing driveway, and out to Crystal Springs Road. An existing earth swale is located above the catchment basin designed to divert overflow during heavy storm events to the north and away from the residence.

As discussed above, a large presumably ancient landslide (12) appears to extend from the downhill side of Parrott Drive to Crystal Springs Road in the north-central portion of the property. This Ols feature is approximately 500 feet in width and 1,200 feet in length. Two additional large, dormant landslides (13 & 14) are located immediately south of this feature, in the south-central portion of the property. A smaller dormant landslide feature (15) is mapped in the northwestern corner of the site. The larger of the dormant features (14) is approximately 400 feet in width and 1,100 feet in length. The margins of these two features (13 & 14) coincide with a central deeply incised seasonal drainage channel (located south of the ancient landslide and north of the dormant landslide). The channel bounding these features is flanked by numerous, relatively small active landslides (17 through 23). The landslides appear to flank both margins of the channel and appear to be mostly rotational in nature, with 2- to 5-foot tall headscarps observed during site mapping. The features appear to be approximately 50- to 200-feet wide and are characterized by generally hummocky topography. Their activity was presumably triggered by undercutting along the steeply incised seasonal drainage channel during past heavy storm events.

A graded road/path enters the property near the eastern margin of the mapped ancient landslide (Ols) and continues in a southwesterly direction toward the mapped quarry. This grading is associated with the existing sewer line that services residences along Parrott Drive. Along the uphill side of this access road, Franciscan materials are exposed that range from relatively competent rock outcrops to highly sheared, severely to completely weathered materials. During site mapping, we observed an arcuate break in slope below the road, located uphill from boring RWB-4 (see Figure 1 within Landslide 14). While this feature may be a scarp related to past movement, the surrounding topography and relatively close position to the graded access road appear to suggest that this feature is likely a remnant associated with past grading. We did not see additional features similar in nature to this on the property, but it is possible they were obscured by the dense vegetation.

An active relatively shallow landslide (25) is located near the central western portion of the property, within the road cut above Crystal Springs Road. Based on our site reconnaissance, this feature appears to be approximately 200 feet wide and approximately 100 feet in length. The slide mass is characterized by generally hummocky topography and is bounded to the north, east, and south by an approximate 2- to 3-foot tall headscarp. Based on aerial photographs, this feature appears to have activated sometime between 1946 and 1968. In our opinion, grading associated with construction of this over-steepened cut slope along the uphill side of Crystal Springs Road is likely the main probable cause of the landslide; however, quarrying activity associated with the old quarry located uphill and to the south may have contributed to the failure. It appears that this active landslide is relatively shallow, likely less than 10 feet thick in depth.

A debris flow complex (26) was initially mapped by SCI along the drainage swale located southeast of the old quarry. Based on our site reconnaissance and aerial photograph review, a significant amount of erosion has occurred at the head of this feature; however, very dense vegetation obscures the topography and evidence of past debris flow movement is inconclusive; however, given its geomorphology, in our opinion this area possesses a potential debris source. Additional shallow active landslide features are located within the mapped debris flow.

We note that due to the dense vegetation and steep slope conditions, only portions of the site were accessed by during our site reconnaissance and mapping phase. Therefore, there could be other relatively shallow to moderate slope failures on the property that have not been documented.

RESPONSE TO COUNTY COMMENTS

The comments contained in the County of San Mateo's geotechnical review sheet, dated December 4, 2014, are presented verbatim below in italics. Our responses are presented below each comment in normal-face type.

Comment No. 1:

Supplemental investigation of the site landslide hazards and potential offsite impacts should be completed. This work should include, but not necessarily be limited to, the following:

A) The approximate area for stabilization repair of active landsliding within Parcels 1 and 2 should be depicted in plan view and cross section. Conceptual design measures should be presented that are intended to prevent future reactivation or enlargement of landsliding across the common property line. If a grading repair is selected, approximate grading volume estimates should be prepared.

Based on the reconfiguration of parcel boundaries, the majority of the mapped active landslide is located within Parcel 2. Please refer to Figure 1 for the reconfiguration of the proposed parcel lines and refer to Cross Section B-B' (Figure A-7) of MEI's 2014 report for reference. We understand that the project civil engineer will be providing a cross section depicting the proposed landslide repair, including keying and benching details of the fill, fill subdrainage, and grading volumes.

B) If a fourth residential house site is desired, then consideration should be given to other favorable property slopes that are no steeper than the proposed building areas on Parcels 1, 2, and 3.

The reconfiguration of the proposed parcel boundaries results in four smaller parcels with slopes that are no steeper than the previous locations of parcels 1 through 3. Specifically, the parcels have been shifted away from the debris flow and steep ravine mapped south of the newly proposed parcel 4. Please refer to our attached site plan for further clarification.

C) General geologic mapping should be conducted to identify potential areas of the 60.26 acre property that present a moderate to high risk for initiation of slope failures, and have a significant potential for adverse offsite impacts to existing residential developments or roadways. Mapping should include delineation of probable debris transport paths and deposition areas.

Based on our review of the above information, prior engineering geologic and geotechnical studies, and our recent site mapping activities, it is our opinion that the larger landslide features mapped on the subject property appear relatively stable, as a whole. Specifically, the larger landslide masses mapped in the central portion of the property, extending from Parrott Drive to Crystal Springs Road, appear to consist of relatively resistant central ridges bounded by incised stream channels with their basal toe likely buttressed by deep soil at the base of the slope fronting Crystal Springs Road. In addition, these features are constrained from significant movement due to its location within a narrow valley. Therefore, in our opinion the potential for full reactivation of these features is relatively low; however, continued erosion along the seasonal drainage channels, loss of lateral support along the lower toe margin area from existing over-steepened road cut slopes, and/or strong earthquake ground shaking may cause partial reactivation(s) along the margins of these features. Although there is evidence of active and past landsliding on the subject property, there is no obvious historic evidence that landsliding on the property has caused any substantial impacts to Crystal Springs Road below. Therefore, in our opinion if partial reactivation of these features were to occur, the probability of this type of slope movement significantly impacting the long-term performance of existing off-site improvements is relatively low. Slope movements affecting existing off-site improvements, such as the road below, will likely result in continued maintenance-level issues and may result in damage and temporary closures of the roadway in local areas. However, this slope stability risk can be expected in this general area along Crystal Springs Road adjacent steep hillside terrain and over-steepened road cut slopes. As stated in our referenced report, we note that although our knowledge of the causes and mechanisms of landslides has greatly increased in recent years, it is not yet possible to predict with certainty exactly when and where all landslides will occur, including deep-seated landslides. 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 and government agency infrastructures located in these areas must be aware of and be willing to accept this risk.

As stated above, the margins of the larger, central landslide features have experienced active landsliding in the recent past. Movement along the incised seasonal drainage channels across the properties generally appears to be more rotational in nature, with less evidence of classic debris-flow type movement. The landslides mapped along the channels generally are evidenced by 2- to 5-foot tall headscarps, generally hummocky topography, and, to a lesser extent, slightly deflected channels away from the landslide masses. However, due to the steepness of slopes and the observed erosion/incision, the channels on the property have the potential to become sources and/or pathways for future debris flow movement. Specifically, based on our site reconnaissance, although slope movement in these areas may continue to occur in a more rotational manner, landslide movement into the channel area could impede drainage flow and cause a temporary buildup of water that could trigger debris flow movement. For reference proposes, debris flows, in general, commonly involve upon saturation, the rapid removal of relatively shallow thicknesses of granular soil over a firm contact such as bedrock. The saturated soil is transported, in semi-liquid form, from the upper regions of the debris flow causing a scar to form in this area, and the resulting debris deposited along a relatively narrow band or "pathway" to a termination point below. Depending on many factors including the size, steepness of slope, topography, soil type, etc., structures located immediately below slopes potentially prone to debris flow movement may be in an immediate threat of both structural damage and/or life safety. Mitigation measures such as debris fences, impact walls, or deflection walls are commonly recommended to reduce this potential threat.

Although there remains a risk of future localized landsliding and/or debris flow movement onto Crystal Springs Road, we note that during our supplemental investigation, we observed a series of improvements that appear to be designed to mitigate this concern along portions of this road segment. For example, a concrete retaining wall has been constructed northeast of the intersection of Crystal Springs Road and Tartan Trail Road as well as rock debris fences just south of this area. In addition, various storm drain improvements exist, including several storm drain culverts along the eastern side of Crystal Springs Road. In addition, the headwall areas near the base of the seasonal drainage swales where the storm drains transect beneath the road, did not show significant buildup of debris at the time of our field observations suggesting that they are periodically maintained.

Based on our site observations, we observed that a substantial concrete debris/deflection wall was installed to presumably help protect the school property (Odyssey School) located northeast of the intersection of Crystal Springs Road and Polhemus Road. This wall appears to have ample capacity and a favorable deflection angle to mitigate the concern for potential debris flow impact to the school development initiating from the adjacent seasonal drainage channels located immediately east of this property..

We observed a catchment basin near the base of the seasonal drainage channel above and approximately 20 feet east of the existing residence located approximately 600 feet northeast of the intersection of Crystal Springs Road and Tartan Trail Road. A culvert routes water from the catchment basin, under the existing driveway, and presumably out to Crystal Springs Road. As previously stated, an existing earth swale is located above the catchment basin designed to divert overflow during heavy storm events to the north and away from the residence. These improvements help mitigate the potential concern associated with direct impact from debris flows and significant flooding.

D) Mitigation measure design options should be presented to address unacceptable offsite impacts.

Based on the findings and discussion above, in our opinion new mitigations measures will not be necessary at this time to address offsite impacts primarily because the existing drainage and wall improvements have historically mitigated significant landslide and debris flow hazard concerns. However, there remains a risk that reactivation of the referenced landslide features or activation of new features may result in maintenance-level issues relating to the serviceability of the road below (such as temporary closures due to debris on the roadway). This risk can be expected in any area with over-steepened road cuts below steep hillside terrain. In addition, although very unlikely, there will always remain some life safety risk to drivers or pedestrians associated with slope movement onto the road and for structures built at the base of steep slopes. However, we emphasize that in our opinion this potential risk has been mitigated by the existing improvements mentioned above and is not substantially different than other areas along this same road segment subject to steep slope conditions.

E) Geotechnical design recommendations for the proposed project should be updated as warranted based on identified site conditions.

The geotechnical design recommendations contained in MEI's 2014 report appear to be applicable to the proposed project. If site conditions varying from those described herein and in MEI's 2014 report, we are prepared to update project geotechnical design recommendations as warranted.

Comment No. 2:

Future proposed subdivision plans should be evaluated and approved by the Project Geotechnical Consultant for conformance with recommendations prior to submittal of revised Tentative Map documentation to the County.

MEI is prepared to evaluate future subdivision plans for conformance with geotechnical recommendations.

Limitations

Our supplemental evaluation has been performed and the preceding conclusions have been developed in accordance with engineering geologic and geotechnical engineering principles and practices generally accepted at this time and location. A more detailed investigation that might include detailed site mapping, subsurface exploration and testing, slope stability analyses, and laboratory testing could result in modifications to our limited evaluation. We make no warranty, either expressed or implied.

If you have any questions concerning the content of this letter or other aspects of the project, please call.

Sincerely,

MURRAY ENGINEERS

REGISTERED PROFESSIONAL ENGINEER
JOHN A. STILLMAN
GE2523
Exp: 12/31/16
STATE OF CALIFORNIA

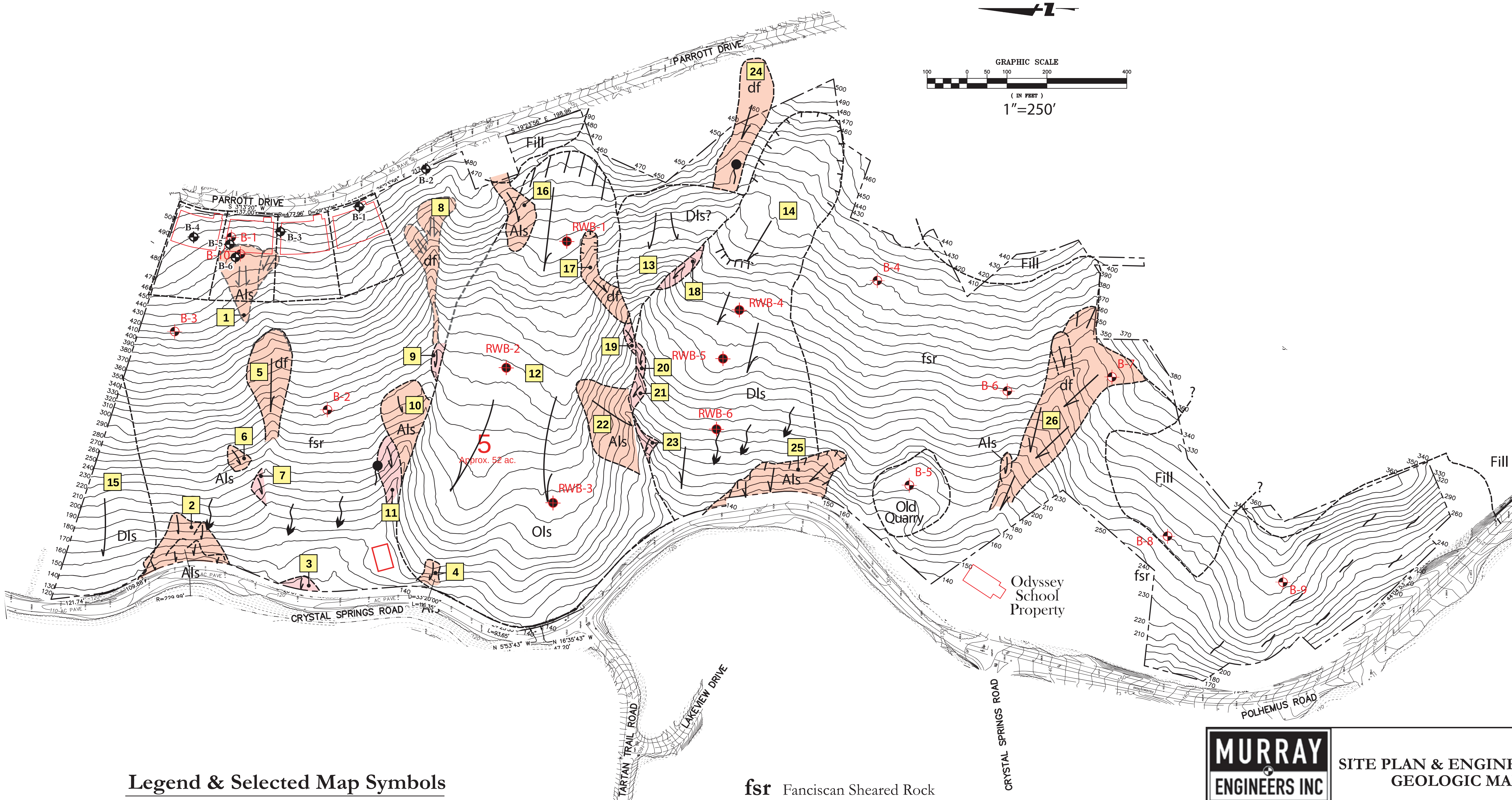
Kaysea A. Porter, P.G. 9269
Project Geologist

John A. Stillman, G.E., C.E.G. 1868
Principal Geotechnical Engineer

KAP:JAS

Copies: Addressee (3)
MacLeod and Associates (1)
Attn: Mr. Daniel MacLeod, P.E.

Attachments: Figure 1, Site Plan & Engineering Geologic Map



Legend & Selected Map Symbols

B-1 Approximate Location of Soil Boring by Murray Engineers, Inc., drilled December 20-23, 2013

RWB-1 / B-4 Approximate Location of Soil Borings by BAGG drilled July 18-19, 2007 & October 2, 2007

— — — ? Geologic Contact, Dashed where Approximate, Queried Where Uncertain

Base: Topographic Survey by BKF Engineers, dated January 2, 2007

Approximate Scale: 1 inch = 100 feet

fsr Franciscan Sheared Rock

Ols Older Landslide

Dls Dormant Landslide

Als Ancient Landslide

df Debris Flow

Landslide Limits

Proposed Building Site - Typical

MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

SITE PLAN & ENGINEERING GEOLOGIC MAP

ZMAY RESIDENTIAL SUBDIVISION
1551 CRYSTAL SPRINGS ROAD
SAN MATEO COUNTY, CALIFORNIA

PROJECT NO. 1847-1L1	MARCH 2015	FIGURE 1

Erica Adams

From: John Stillman <john@murrayengineers.com>
Sent: Thursday, September 24, 2020 2:47 PM
To: Kathy Zmay
Cc: Erica Adams; Steve Zmay; Vergel Galura; Dan Macleod
Subject: RE: 1551 Crystal Springs Dev-Zmay

Hi Erica/Kathy,

Given the sewer main's significant distance away from the planned development and assuming the construction is performed to acceptable standards, it is our geotechnical opinion, the proposed pier drilling for the landslide repair or the new residences should not have a significant impact on the referenced sewer main.

Hope that helps.

Regards,

John A. Stillman, G.E., C.E.G.
Principal Geotechnical Engineer
Murray Engineers, Inc.
www.murrayengineers.com
650-274-1742 (M) 650-559-9980 (O) 650-559-9985 (F)

Bay Area Regional Offices
Peninsula 935 Fremont Avenue, Los Altos, CA 94024 | 650-559-9980
North Bay 409 4th Street, San Rafael, CA 94901 | 415-888-8952

From: Kathy Zmay [mailto:kuz5@sbcglobal.net]
Sent: Wednesday, September 16, 2020 8:32 AM
To: John Stillman
Cc: Erica Adams; Steve Zmay
Subject: 1551 Crystal Springs Dev-Zmay

Hello John,

The County is requesting the following. Can you please provide?

I have cc'd Erica Adams, from the County, our warrior on this project.
Please include her when you send the 'statement'.

Thank you,
Kathy and Steve

Hello again,

Can I get a statement from your Geotech about whether pier drilling for eithr landslide repair or the houses would impact the Billy Goat sewer main. It appears to be about 1,500 ft. away from working areas, but I need a statement from an expert.

Thanks.

Erica

Kathy Zmay
650-430-8220

Erica Adams

From: John Stillman <john@murrayengineers.com>
Sent: Thursday, September 24, 2020 2:38 PM
To: Erica Adams; Steve Zmay
Cc: Nick Zmay; Kathy Zmay; Vergel Galura; Dan Macleod
Subject: RE: Geology questions

CAUTION: This email originated from outside of San Mateo County. Unless you recognize the sender's email address and know the content is safe, do not click links, open attachments or reply.

Hi Erica,

See my comments in red below

Regards,

John A. Stillman, G.E., C.E.G.
Principal Geotechnical Engineer
Murray Engineers, Inc.
www.murrayengineers.com
650-274-1742 (M) 650-559-9980 (O) 650-559-9985 (F)

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North Bay 409 4th Street, San Rafael, CA 94901 | 415-888-8952

From: Erica Adams [mailto:eadams@smcgov.org]
Sent: Thursday, September 24, 2020 12:50 PM
To: John Stillman; Steve Zmay
Cc: Nick Zmay; Kathy Zmay
Subject: RE: Geology questions

Hi Steve (and John),

I need these questions are answered. Can you let me know when that may happen. I am close to having a final document, but I have some holes to fill in.

Let me know if there are issues.

Erica D. Adams
Planner III
San Mateo County
Planning and Building Department
455 County Center
Redwood City, CA 94063
T (650) 599-1559 | F (650) 363-4849
planning.smcgov.org

Due to COVID-19, the Planning and Building Department is closed to the public. Please refer to our website for additional temporary closure information.

From: Erica Adams
Sent: Monday, September 7, 2020 6:45 PM
To: John Stillman <john@murrayengineers.com>
Cc: Steve Zmay <zmaysteve@gmail.com>
Subject: Geology questions

Hello John,

I have a couple of questions and I need some assistance responding to comments regarding the environmental document. Especially to some of the issues raised in one comment letter in particular. (see attached)

Several responses refer to my report as inadequate because it does not discuss sub-surface hydrology, groundwater and even source water for the wetlands. There is question about "source water" but it's runoff right? Am I correct that the wetlands are being formed from runoff on Parrott Drive, or is there a groundwater source? A few people are bringing up hydrology, which I think is has no environmental impacts for this project, but if I can get an expert's statement on that (or point me to where it is in the existing documents), I would appreciate it. We are not experts in hydrology/wetland evaluation but based on my experience, there could be a potential for perched groundwater within bedrock zones along slopes or shallow groundwater may concentrate within swale zones. Past landsliding on the property (including beyond the subdivision limits) can create water barriers that could lead to localized seeps/wetland areas. Other source water could come from storm water runoff or irrigation from upslope properties.

Based on this answer, there seems to be a few separate issues…
1) Whether the resulting water path (after the retaining walls) will sustain the wetlands? No sure exactly what you are asking but I assume it relates to whether installation of walls will impact water sources below
2) How the new drainage path impacts landside susceptibility? This statement is a bit ambiguous-too hard to answer with much accuracy.
3) With respect to the future residential development, will retaining walls needed to support the driveway alter water paths? Difficult to answer with much accuracy

I think I need a statement directly addressing redirecting of water due to the stitch pier walls. Will stich pier have any impact on surface or ground water or wetlands? Difficult to answer with much accuracy

I am going to be in the office Tuesday and Thursday after 10AM and you can call me if email is not enough.

Erica, I think it would be good to have a Zoom call with you, myself, and Dan/Vergel so we can clarify these issues collectively together.

Thanks in advance for your help.

Erica D. Adams, Planner III
Planning and Building Department
455 County Center, Second Floor
Redwood City, CA 94063
Phone: (650) 363-1828
Fax: (650) 363-4849