County of San Mateo - Planning and Building Department

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GEOTECHNICAL INVESTIGATION
SULLIVAN RESIDENCE
APN 082-160-130
SAN MATEO COUNTY, CALIFORNIA

THIS REPORT HAS BEEN PREPARED FOR:

TIM SULLIVAN

6175 LA HONDA ROAD

LA HONDA, CALIFORNIA 94102

JANUARY 2015

MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

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SULLIVAN RESIDENCE
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INTRODUCTION

This report presents the results of our geotechnical investigation relating to the design and construction of a new residence and associated improvements on your property, APN 082-160-130 in unincorporated San Mateo County. The project location is indicated on the Vicinity Map, Figure A-1. The purpose of our investigation was to evaluate the subsurface conditions on the site in the area of the proposed improvements and to provide geotechnical design criteria and recommendations for the project.

Project Description

The project consists of constructing a new, 2,200 square foot, prefabricated Blu Homes residence ("Balance Vista" model) in the central northwestern portion of the property. The residence will include a 2,135 square-foot daylighting, custom, site-built full basement and a 960 square foot detached, custom, site-built garage. Raised wood decking is planned along the northwest and east sides of the house, totaling 1,015 square feet. A new gravel-surfaced driveway with a fire truck turn-around is planned from the house site to a shared driveway which extends to La Honda Road. We anticipate that structural loads for the improvements will be relatively light and typical of residential construction. The layout of the existing and proposed improvements is shown on the Site Plan, Figure A-2

Scope of Services

We performed the following services in accordance with our agreement with you dated November 21, 2014 (executed on December 3, 2014):

- Reviewed geologic and seismic conditions in the site vicinity and commented on the geologic hazards that could potentially impact the site and the proposed improvements
- Performed a reconnaissance of the site in the area of the proposed improvements
- Explored the subsurface by advancing and logging two exploratory borings in the vicinity of the proposed improvements
- Performed laboratory analysis of select soil samples for soil classification and to evaluate engineering properties of the subsurface materials
- Performed geotechnical engineering analyses to develop geotechnical engineering design criteria for the proposed improvements
- Prepared this report containing a summary of our investigation and our geotechnical conclusions and recommendations

GEOLOGIC & SEISMIC CONDITIONS

Geologic Overview

The subject property is located along the western side of the central Santa Cruz Mountains, a northwest-trending range within the California Coast Ranges geomorphic province. The site is situated on a gently to moderately sloping, south-facing hillside. Elevations across the property vary from a high of approximately 290 feet above mean sea level along the northwestern side of the property down to a low of approximately 210 feet along the south portion of the property (see Figure A-1).

According to the Preliminary Geologic Map of the La Honda and San Gregorio Quadrangle (Brabb, 1980), the site is located in an area underlain at depth by Pliocene age (approximately 1.8 to 5.3 million years old) Pomponio Mudstone bedrock of the Purisma Formation (Tpp). This bedrock material consists of gray to white porcelaneous shale and mudstone in places rhythmically bedded with alternating layers of non-siliceous mudstone, and resembles Santa Cruz Mudstone and Lambert Shale units. The relevant portion of this preliminary geologic map is included on Figure A-3, Vicinity Geologic Map. This older mapping is consistent with the more recent Geologic Map of the Onshore Part of San Mateo County (Brabb and others, 1998).

According to the Preliminary Map of Landslide Deposits in San Mateo County (Brabb and Pampeyan, 1972), no landslides are mapped on the property. The map does indicate that a landslide scarp is located approximately 1,000 feet northeast of the proposed house site; however, in our opinion this feature appears to be more of an erosional feature than a deep-seated landslide. The relevant portion of the landslide deposit map is included as Figure A-4, Vicinity Landslide Map.

Faulting & Seismicity

The San Francisco Bay Area, which is affected by the San Andreas Fault system, is recognized by geologists and seismologists as one of the most active seismic regions in the United States. In the Bay Area there are three major faults trending in a northwest direction within the San Andreas Fault system, which have generated about 12 earthquakes per century large enough to cause significant structural damage. These faults include the San Andreas, Hayward, and Calaveras faults. The San Gregorio fault is located approximately 3.9 miles southwest of the site and the San Andreas fault is located approximately 7.2 miles northeast of the site. The Hayward and Calaveras faults are located approximately 26 and 30 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).

SITE EXPLORATION & RECONNAISSANCE

Exploration Program

Our field investigation was performed on December 19, 2014 and included a site reconnaissance and the excavation and logging of two exploratory borings to depths ranging from approximately 5 feet to 8.1 feet at the locations shown on Figure A-2. The boring locations were approximately determined by measuring distance from known points on the supplied site plan and should be considered accurate only to the degree implied by the mapping technique used.

The borings were advanced using portable sampling equipment. Soil samples were collected with split-spoon samplers that were driven with a 140-pound hammer repeatedly dropped from a height of 30 inches with a rope and cathead attached to a sampling tripod. The split-spoon samplers included 3-inch and 2.5-inch outside diameter (OD) samplers, and a 2-inch OD Standard Penetration Test sampler. The sampler types used are indicated on the logs at the appropriate depths. The number of hammer blows required to drive the samplers were recorded in 6-inch increments for the length of the 24-inch long sampler barrels. The associated blow count data, which is the sum of the second and third 6-inch increment, is presented on the boring logs as sampling resistance in blows per foot. The field blow counts for the 2.5-inch and 3-inch OD samplers have been standardized to Standard Penetration Test blow counts for sampler size; however, the blow count data has not been adjusted for other factors such as hammer efficiency. The logs of our borings are presented in Appendix B as Figures B-1 and B-2. Also included in Appendix B is Figure B-3, Key to Boring Logs; Figure B-4, Unified Soil Classification System; and Figure B-5, Key to Bedrock Descriptions.

Our staff geologist logged the borings in general accordance with the Unified Soil Classification System. 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.

MURRAY ENGINEERS, INC
GEOTECHNICAL SERVICES

January 28, 2015
Project No. 2150-1R1

Tim Sullivan
6175 La Honda Road
La Honda, California 94102

RE: GEOTECHNICAL INVESTIGATION, SULLIVAN RESIDENCE, APN 082-160-130, SAN MATEO COUNTY, CALIFORNIA

Dear Mr. Sullivan:

We are pleased to present the results of our geotechnical investigation relating to the design and construction of a new residence and associated improvements on your property, APN 082-160-130 in unincorporated San Mateo County, California. This report summarizes the results of our field, laboratory, and engineering work, and presents conclusions and recommendations concerning the geotechnical engineering aspects of the project.

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

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

Sincerely,
MURRAY ENGINEERS, INC.

Carrie Thomas
Staff Geologist

Kristofer T. Korth, P.E.
Project Engineer

REGISTERED PROFESSIONAL ENGINEER
KRISTOFER T. KORTH
82836
Exp. 09/30/16
CIVIL
STATE OF CALIFORNIA

Andrew D. Murray, P.E.
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
ANDREW D. MURRAY
C44562
Exp. 3/31/16
CIVIL
STATE OF CALIFORNIA

CET:KTK:ADM

Copies: Addressee (3)
Blu Homes (3)
Attn: Mark Westlake

Site Description

The undeveloped, irregular-shaped, gently to moderately sloping hillside property is located north of La Honda Road (State Route 84) in a rural area of unincorporated San Mateo County. The site is bounded by developed rural residential properties to the north and south, an unnamed shared driveway to the east, and by undeveloped lands to the west. The site is vegetated with grasses, bushes, and shrubs. The southern, eastern, and western property boundaries are linear and measure approximately 837 feet, 320 feet, and 553 feet, respectively. The northern property boundary is marked by four changes in orientation. Overall site grades generally slope gently to moderately from the northern property boundary down to the southern property boundary.

The site is accessed by a shared driveway that extends northeast from La Honda Road and is surfaced with baserock. A cleared travel way on the site is located along the northern boundary of the property and extends west to a relatively flat, cleared area (location of proposed detached garage) in the central northwest portion of the site. To the north of the cleared area and uphill of the proposed building pad for the residence and garage, the ground surface slopes down at an average gradient of approximately 4:1 (horizontal to vertical). To the south of the cleared area, the ground surface gently slopes at an average gradient of approximately 5:1 (horizontal to vertical). The ground surface in the eastern portion of the site slopes down to the south at an average gradient of approximately 8:1 (horizontal to vertical). A sharply-incised drainage is located immediately west of the western property boundary.

We did not observe any evidence of active landsliding on the site during our investigation; however, we did note evidence of shallow erosion at the ground surface in the central northwest portion of the site. Drainage across the property is generally characterized as uncontrolled sheet flow to the south-southwest.

Subsurface Conditions

Boring B-1, located within the southwest portion of the proposed residence footprint, encountered approximately 1 foot of colluvium consisting of medium stiff silty clay underlain by mudstone bedrock which persisted to the bottom of the boring at a depth of approximately 8.1 feet.

Boring B-2, located in the area of the proposed garage, encountered approximately 1 foot of colluvium consisting of medium stiff silty clay underlain by approximately 2.5 feet of colluvium consisting of hard silty clay. At a depth of approximately 3.5 feet, the colluvium is underlain by sandstone bedrock which persisted to the bottom of the boring at a depth of approximately 5 feet.

Groundwater

Groundwater was encountered at a depth of approximately 2 feet below existing site grades while drilling Boring B-2. No groundwater was encountered in Boring B-1. We note that the weather was rainy on the day of drilling. Both borings were backfilled prior to leaving the site on the drill date. We note that fluctuations in the level of groundwater can occur due to variations in temperature, rainfall, and other factors that may not have been evident at the time our observations were made.

CONCLUSIONS

Based on our investigation, it is our opinion that the site is suitable for the proposed residential development provided that the recommendations presented in this report are incorporated in the design and construction of the project. In our opinion, the primary geotechnical constraints to the project are the potential for downhill creep of the surficial colluvial soil on the moderately sloping portions of the site and the potential for very strong ground shaking during a moderate to large earthquake on one of the nearby faults.

Based on our investigation, it appears that the area of the proposed residence and garage is blanketed by roughly one to 3.5 feet of colluvial soil overlying bedrock. Based on our investigation, the surficial colluvial soil is relatively weak and may be subject to future consolidation and downhill creep under the force of gravity. In addition, based on clay content, the colluvial soil material appears to be moderately expansive. In our opinion, the colluvial soil should not be relied on for support of the proposed residence and garage. The colluvial soil is underlain by fractured bedrock. In our opinion, the underlying competent bedrock should provide adequate support for foundations associated with the proposed residence and garage.

Geologic Hazards

As part of this investigation, we evaluated the potential for geologic hazards to impact the proposed development. The results of our evaluation are presented below:

- Expansive Soils – Based on our laboratory testing, portions of the near-surface material is moderately expansive. In general, expansive soil can undergo volume changes with changes in moisture content. Specifically, when wetted as during the rainy season, expansive soil tends to swell and when dried as during the summer months, this material shrinks. Structures and flatwork supported on expansive soil tend to experience cyclic, seasonal heave and settlement. In our opinion, shrink and swell of the surficial soil should not have a significant impact on the structural integrity of the proposed improvements, provided that they are designed and constructed in accordance with the recommendations presented in this report. In our opinion, these recommendations should mitigate the potential for significant heave, but will not eliminate this potential.

- Landsliding – Based on our investigation, we did not observe any evidence of active landsliding in the site improvement area but we did note evidence of shallow erosion at the ground surface in the central northwestern portion of the site. Because of the presence of colluvium blanketing the site and the moderate slopes across portions of the site, the occurrence of a new shallow landslide or shallow sloughing involving these materials cannot be excluded. A new shallow landslide could be triggered by excessive precipitation, erosion, and/or strong ground shaking associated with an earthquake. In our opinion, a new shallow landslide should not pose a significant hazard to the proposed improvements, provided that the improvements are designed and constructed in accordance with the recommendations of this report.

  It should be noted that although our knowledge of the causes and mechanisms of landslides has greatly increased in recent years, it is not yet possible to predict with certainty 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 maps, it is our opinion that no active or potentially active faults cross the property. Therefore, in our opinion, the potential for fault rupture to occur at the site is very low.

- Ground Shaking - As noted in the Seismicity section above, moderate to large earthquakes are probable along several active faults in the greater Bay Area. Therefore, strong ground shaking should be expected at some time during the design life of the proposed development. The improvements should be designed in accordance with current earthquake resistant standards, including the 2013 California Building Code (CBC) guidelines and design parameters presented in this report. It should be clearly understood that these guidelines and parameters will not prevent damage to structures; rather they are intended to prevent catastrophic collapse.

- Differential Compaction – During moderate and large earthquakes, soft or loose, natural or fill soils can settle, often unevenly across a site. In general, we encountered competent bedrock at relatively shallow depths within the area of the proposed residence and garage during our investigation. However, some of the colluvial soil materials encountered above the bedrock were medium stiff and may be susceptible to a moderate degree of differential compaction. Therefore, the colluvial soil should not be relied upon for support of the residence or garage and thus, in our opinion differential compaction should not pose a significant risk to the structural integrity of the proposed residence or garage as long as they are designed and constructed in accordance with the recommendations contained herein.

- Liquefaction – Liquefaction is a soil softening response, by which an increase in the excess pore water pressure results in partial to full loss of soil shear strength. In order for liquefaction to occur, the following four factors are required: 1) saturated soil or soil situated below the groundwater table; 2) undrained loading (strong ground shaking), such as by earthquake; 3) contractive soil response during shear loading, which is often the case for a soil which is initially in a loose or uncompacted state; and 4) susceptible soil type; such as clean, uniformly graded sands, non-plastic silts, or gravels. Structures situated above temporarily liquefied soils may sink or tilt, potentially resulting in significant structural damage. Due to the relatively cohesive nature of the surfical soil materials and because we encountered competent bedrock at relatively shallow depths during our investigation, in our opinion the likelihood of liquefaction occurring and affecting the proposed improvements is very low.

RECOMMENDATIONS

We recommend that the proposed daylighting basement level beneath the proposed residence, its retaining walls, and all loads overlying the daylighting basement be supported on a reinforced concrete mat foundation bearing in the underlying competent bedrock. If required for sliding resistance by the structural engineer, the mat slab may include a down-turned edge along the downhill edge of the basement mat slab that extends at least 24 inches into competent bedrock. In addition, if colluvium is exposed at the bottom of the new basement excavation, the colluvium should be removed and replaced with well compacted select granular fill, such as Class 2 aggregate baserock.

We anticipate that zones of perched subsurface water, not necessarily representative of a regional groundwater level, may be present on the site. Due to the daylighting nature of the proposed basement, in our opinion, groundwater should not significantly impact the basement design but the potential for some perched ground water entering into the basement excavation should be taken into account by the building contractor. Basement retaining walls and the basement mat slab should be provided with subdrainage to alleviate the potential for buildup of hydrostatic pressures against the walls or beneath the mat slab. The building contractor should take the 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. In addition, we strongly encourage the use of a waterproofing consultant and/or waterproofing subcontractor to assure adequate protection from surface water that will accumulate adjacent to the basement walls and bottom of mat slab.

We recommend that any at-grade portions of the residence, including any accessory features such as entrance steps, porches, and overhangs structurally tied to the residence, be either supported on drilled, cast-in-place, reinforced, concrete friction piers or else cantilevered off the retaining walls associated with the daylighting basement level for the residence to limit the potential for differential movement between the daylighting basement and the at-grade portions of the residence. Wood decks that are structurally connected to the residence should preferably be supported on drilled piers; however, given the nature of the proposed deck improvements, in our opinion it is reasonable to support structurally connected wood decks on spread footings provided that the owner is aware of and willing to accept the potential for differential foundation movement between attached decks and the residence that may result in slight shifting of the deck supports and structure over time.

The proposed detached garage may be supported either on drilled piers or on spread footings bearing in the underlying bedrock. Although, in our opinion, piers will perform better than footings in terms of limiting differential foundation movement, spread footings can be expected to perform reasonably well at this site provided that spread footings are founded in competent bedrock.

In general, we recommend that proposed site retaining walls, such as will be required along portions of the driveway perimeter, be supported on drilled piers gaining support in the competent bedrock underlying the site. However, site retaining walls supporting cuts into bedrock may be supported on either spread footings or drilled piers. Although in our opinion piers will perform slightly better than footings in terms of limiting differential foundation movement, spread footings can be expected to perform reasonably well at this site.

In general, slabs-on-grade and flexible pavements should be underlain by a section of compacted Class 2 aggregate baserock over a prepared subgrade. Any slabs-on-grade planned adjacent to the basement walls should be designed to span the area underlain by the planned basement retaining wall backfill (approximately 10-feet) to mitigate the concerns for backfill settlement. Where existing fill is present within areas of new hardscape, portions of the fill should be removed and replaced as a properly engineered fill as deemed necessary by our field representative during construction.

Because of the complexity of the project and the potential for design and layout changes, we should review the proposed layout and design, prior to completion of the final plans, to verify that the following recommendations are appropriate. Detailed foundation, grading, and drainage recommendations and geotechnical design criteria are presented below.

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 landslide deposit 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 (2014) 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. Based on coordinates derived from Google Earth, the approximate location of the proposed residence will be latitude 37.3196 and longitude -122.3310. Given these coordinates and based on our subsurface investigation, 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= 1.652 (Site Class B)
- Mapped Spectral Accelerations for a 1-second Period: S_1= 0.676 (Site Class B)
- Design Spectral Accelerations for 0.2 second Period: S_{DS}= 1.102 (Site Class C)
- Design Spectral Accelerations for a 1-second Period: S_{D1}= 0.586 (Site Class C)

FOUNDATIONS

Basement Mat Foundation

In our opinion, the daylighting basement level beneath the proposed residence and associated retaining walls may be supported on a reinforced concrete mat slab foundation bearing on the underlying competent bedrock. If required for sliding resistance by the structural engineer, the mat slab may include a down-turned edge along the downhill edge of the basement mat slab that extends at least 24 inches into competent bedrock. Because we anticipate the downhill edge of the basement mat slab may overlie a small wedge of non-supportive colluvium, we recommend the mat slab include a down-turned edge extending a minimum of 24-inches into bedrock. In addition, if colluvium is exposed at the bottom of the basement excavation it should be removed and replaced with well compacted select granular fill, such as Class 2 aggregate baserock. We recommend that the bottom of the mat

MURRAY ENGINEERS INC

slab be waterproofed and that the waterproofing be designed and constructed by qualified professionals.

Mat foundations may be designed for allowable bearing pressures of 2,500 pounds per square foot for combined dead plus live loads, with a one-third increase allowed for transient loads, including wind or seismic forces. If the structural engineer will utilize a modulus of subgrade reaction in the mat design, we estimate that the modulus of vertical subgrade reaction for a 1-foot square plate (based on Terzaghi's method - Figure 6 of the Navy Design Manual, Chapter 5, NAVFAC DM 7.1) for the bedrock anticipated at mat slab subgrade elevation to be approximately 75 pounds per cubic inch (pounds per square inch per inch).

Lateral loads may be resisted by friction between the mat slab and the supporting subgrade. A frictional resistance of 0.30 can be used. In addition to the above, lateral resistance may be provided by passive pressures acting against the lower two-thirds of the embedded portions of the basement retaining walls using an equivalent fluid pressure of 350 pounds per cubic foot.

The mat foundation should be reinforced with grids of steel reinforcing bars. The project structural engineer should determine actual mat reinforcing based on anticipated loading and the design criteria presented in this report.

We recommend that the basement mat slab foundation be provided with a subdrain system integrally designed with the basement retaining wall drainage system. Figures A-5 and A-6 present schematic details for alternative subdrain systems for basement retaining walls and mat foundations. We recommend that mat slab be underlain by a minimum of approximately 8 inches of ½- to ¾-inch clean crushed rock, underlain by filter fabric. To facilitate drainage, the subgrade soils beneath the mat should be sloped at an inclination of approximately 1.5 percent to a perimeter trench where the retaining wall drainage pipe will be located. Please also refer to the Retaining Wall Drainage section of this report.

Our representative should observe the basement excavation upon its completion and prior to placement of the slab subdrainage system to evaluate the condition of the subgrade materials and to make sure that the conditions are consistent with those anticipated from our borings. It may be necessary to compact the subgrade material in the excavations if loose or disturbed areas are created or encountered during construction.

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

Drilled Pier & Grade Beam

We recommend that any at-grade portions of the residence, including attached porches, balconies and/or overhangs, be supported on drilled, reinforced, cast-in-place, concrete friction pier and grade beam foundations gaining support in the underlying bedrock. Site retaining walls, decks and the detached garage may also be supported on drilled piers. Drilled piers for at-grade portions of the residence and the garage should be at least 16 inches in diameter, should extend at least 10 feet below bottom of grade beam elevation, and should achieve at least 8 feet of embedment into the underlying competent bedrock. Note that piers which are drilled through basement retaining wall backfill and/or basement access ramp backfill will need to extend at least 8 feet into bedrock beneath any backfill. Drilled piers for site retaining walls should be at least 12 inches in diameter and should extend at least 6 feet into bedrock and to a depth into bedrock equal to the retained height of the wall plus the depth of any non-supportive soil at the top of the pier, whichever is deeper. Drilled piers for exterior decks should be at least 12 inches in diameter and should extend at least 4 feet into bedrock. Please note, that these are recommended minimum pier dimensions and that other structural criterion, such as the need to resist lateral forces, may force the pier design depths to be greater. In general, drilled piers should be spaced no closer than about three pier-diameters, center-to-center.

The 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 bedrock with a one-third increase allowed for transient loads, including wind and seismic forces. Any portion of the piers in non-engineered fill and unsupportive soil, and any point-bearing resistance should be neglected for support of vertical loads.

Piers on or within 10 feet of a slope steeper than 5:1 should be designed to resist active loads from downhill creep of soil. Active loads from soil creep may be calculated based on an equivalent fluid weight of 75 pcf acting over 2-pier diameters for the upper depth of the piers in the colluvium or fill. The depth of the active loads will likely vary between approximately one to three feet at individual pier locations.

Active loads from soil creep and other lateral loads may be resisted by passive earth pressure based upon an equivalent fluid pressure of 400 pounds per cubic foot, acting on 1.5 times the projected area for the depth of the pier in bedrock. Any passive resistance corresponding to the creep zone described above should be neglected. In addition, piers located within 10 feet of the basement walls should neglect passive resistance above a 1:1 plane projected upward from the base of the basement retaining wall. The structural engineer should determine pier reinforcing, based on the preceding design criteria and structural requirements.

To prevent mushrooming of the concrete at the tops of the piers and the potential for uplift from the moderately expansive surficial soil, we recommend that the upper approximately 2 feet of piers be formed with Sonotubes, where located in areas of expansive surficial soil.

The contractor should be advised that hard bedrock may be encountered while excavating the foundation piers. "Refusal" to drilling with lightweight equipment (e.g. augers mounted on a backhoe tractor) should be evaluated by our field representative and may not be considered acceptable, necessitating heavier equipment being brought to the site to demonstrate "refusal".

The bottoms of the pier excavations should be substantially free of loose cuttings and soil slough prior to the installation of reinforcing steel and the placement of concrete. In addition, any significant amounts of accumulated water in the pier excavations should be pumped out prior to placing concrete or displaced using the tremie method when placing concrete. A representative of Murray Engineers, Inc. should observe the pier excavations to evaluate whether the piers are founded in the supportive material and whether the pier excavations are properly prepared. The pier depths recommended above may require adjustment, if differing conditions are encountered during excavation. Pier excavations should be filled with concrete as soon as practical after drilling to minimize the potential for caving.

Grade beams should be incorporated between piers as required by the structural engineer. Perimeter foundations should extend at least 6 inches below the crawlspace grade or bottom of slab subgrade to mitigate the potential for infiltration of surface runoff under the at-grade portions of the structures. Grade beam reinforcing should be determined by the project structural engineer based on the preceding design criteria and structural requirements.

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

Spread Footings

The detached garage, wood decking surrounding the residence, and site retaining walls retaining cuts into bedrock may be supported on spread footings. Continuous spread footings for the garage should have a minimum width of 15 inches and isolated footings should be at least 18 inches square. Spread footings for the garage should extend at least 24 inches below lowest adjacent grade; at least 12 inches below bottom of garage slab, and extend at least 6 inches into competent bedrock, whichever is deeper. Spread footings supporting wood decking should extend at least 18 inches below lowest adjacent grade and at least 6 inches into competent bedrock. Site retaining walls retaining cuts into bedrock and sufficiently away from descending slopes may also be supported on spread footings bearing in the underlying bedrock. New continuous footings for site retaining walls should have a minimum width of 15 inches, should extend at least 18 inches below final adjacent exterior grade, and be embedded at least 6 inches into bedrock, whichever is deeper.

Spread footings supported in bedrock may be designed for an allowable bearing pressure of 2,500 pounds per square foot for dead plus live loads, with a one-third increase allowed for total loads including wind and seismic forces. The weight of the footings may be neglected for design purposes.

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

Final footing dimensions and steel reinforcing should be determined by the project structural engineer based on the preceding design criteria and structural requirements. In addition, footings located adjacent to utility trenches should bear below a 1:1 plane extended upward from the bottom edge of the utility trench.

The footing excavations should be substantially free of all loose soil, prior to placing reinforcing steel and concrete. Our representative should observe the footing excavations prior to placing concrete forms and reinforcing steel to see that they are founded in competent bearing materials and have been properly cleaned. In addition, any loose soil in the footing excavations resulting from the placement of forms and reinforcing steel should be removed prior to placing concrete.

Based on our engineering judgment, thirty-year differential foundation movement due to static loads is not expected to exceed approximately 1-inch across any 20-foot span of the footing-supported improvements.

BASEMENT & SITE RETAINING WALLS

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

Lateral Earth Pressures

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

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

In accordance with the 2013 CBC, where applicable, retaining walls should also be designed to resist lateral earth pressure from seismic loading. We recommend that the seismic loading be based on a uniform pressure of 8H pounds per square foot (psf)/foot of wall height, where H is the height in feet of the retained soil. In our opinion, site retaining walls less than 6 feet high do not need to be designed for seismic loading. The allowable passive pressures provided for retaining wall foundations may be increased by one-third for short-term seismic forces.

Retaining Wall Drainage

We recommend that retaining walls include a subsurface drainage system to mitigate the buildup of water pressure from surface water infiltration and other possible sources of water. As noted above, the basement wall drainage system for the proposed residence should be integral with the basement mat slab foundation drainage system.

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

12 inches in loose thickness. The upper roughly 18 inches of backfill should consist of native soil, which should be compacted in accordance with the Compaction section of this report to mitigate infiltration of surface water into the subdrain systems. The preceding recommendations are presented schematically on Figure A-5, Basement Subdrain System Alternative A.

As an alternative to crushed rock, Miradrain, Enkadrain, or other geosynthetic drainage panels approved by this office may be used for retaining wall drainage. If used, the drainage panels should extend from a depth of approximately 18 inches below finish grade to the base of the retaining wall. An approximate 2-foot section of crushed rock wrapped in filter fabric should be placed around the drainpipe, as discussed previously. Geosynthetic drainage panels should be installed in strict compliance with manufacturer's recommendations with filter fabric against the crushed rock and soil backfill. The preceding recommendations are presented schematically on Figure A-6, Basement Subdrain System Alternative B.

Subdrain pipes should be sloped at a minimum of approximately 1.5 percent and should be connected to rigid, solid (non-perforated) discharge pipes to convey any collected water to a suitable discharge location away from the walls. The subdrain pipes for site retaining walls should be provided with cleanout risers at their up-gradient ends and at most sharp directional changes to facilitate maintenance. We recommend against the use of cleanout risers associated with the basement retaining wall subdrain pipes because of the future risk that cleanout pipes might be accidentally connected to a surface drain or roof downspout, thereby risking flooding of the basement light well and subsequently the basement itself. In general, downspouts and surface area drains should be kept completely separate from the retaining wall drainage system.

Retaining Wall Backfill

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

CONCRETE SLABS-ON-GRADE

We anticipate that concrete slabs-on-grade will be utilized for the garage floor and possibly also for miscellaneous concrete patios and walkways. Concrete slabs-on-grade for the garage floor should be underlain by at least 12 inches of Class 2 aggregate baserock. Other exterior hardscape should be underlain by at least 8 inches of Class 2 aggregate baserock. If non-expansive bedrock is exposed at subgrade level, the baserock thickness beneath slabs-on-grade may be reduced to 6 inches. Any slabs-on-grade planned adjacent to the basement walls should be designed to span the area underlain by the planned basement retaining wall backfill (approximately 10-feet) to mitigate the concerns for backfill settlement. Where existing fill is present within areas of new hardscape, portions of the fill should be removed and replaced as a properly engineered fill as deemed necessary by our field representative during construction. The preceding recommendations are intended to mitigate significant slab movement and cracking. We note that minor slab movement or localized cracking of slabs may still occur.

Prior to placement of the baserock, the subgrade soils should be scarified and moisture conditioned, as necessary, to a depth of approximately 6 inches and recompacted in accordance with the Compaction section of this report. In addition, if highly expansive subgrade soils are encountered, the subgrade should be scarified to a depth of approximately 6 to 12 inches, moisture conditioned to at least 3 percent over optimum moisture content, and compacted to between 87 percent to 90 percent relative compaction. Over-compaction of highly expansive material should be avoided. In our opinion, these recommendations should mitigate the potential for significant heave, but will not eliminate this potential.

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

Select granular fill should be compacted in accordance with the Compaction section of this report. Where slab surface moisture would be a significant concern, such as for the garage floor, we recommend that the slabs be underlain by a vapor retarder consisting of a highly durable membrane not less than 15 mils thick (such as Stego Wrap Vapor Barrier by Stego Industries, LLC or equivalent), underlain by 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 select granular fill recommended above. Please also refer to the Vapor Retarder Considerations section below for additional information. Please note that these recommendations do not comprise a specification for "waterproofing." For greater protection against concrete dampness, we recommend that a waterproofing consultant be retained.

Vapor Retarder Considerations

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

We understand that recent trends are to use a highly durable vapor retarder membrane (at least 15 mils thick) without the protective sand covering for interior slabs surfaced with floor coverings including, but not limited to, carpet, wood, or glued tiles and linoleum. However, it is also noted that several special considerations are required to reduce the potential for concrete edge curling if sand will not be used, including slightly higher placement of reinforcement steel and a water-cement ratio not exceeding 0.5 (Holland and Walker, 1998). We recommend that you consult with other members of your design team, such as your structural engineer, architect, and waterproofing consultant for further guidance on this matter.

FLEXIBLE PAVEMENTS

Gravel or Baserock Driveway

We understand that the new driveway extending from the existing shared driveway along the eastern property boundary to the new detached garage, including the fire truck turnaround, will be surfaced with gravel or will have an unfinished baserock surface. We recommend that the driveway be underlain by at least 12 inches of compacted Class 2 aggregate baserock, with or without a landscaping gravel covering. Prior to placement of the baserock, the subgrade soils should be scarified and moisture conditioned to a depth of at least 6 inches, as necessary, and compacted in accordance with the Compaction section of this report. If soft subgrade conditions are encountered during construction, it may be necessary to thicken the baserock section or place a geotextile strength fabric, such as MirafiRS380i or equivalent, on the subgrade soil. A representative from our office should observe the subgrade conditions at the driveway prior to placement of baserock.

While we anticipate that a 12-inch thick section of Class 2 aggregate baserock would be capable of handling occasional fire or garbage truck loading, we note that some localized rutting or yielding may still occur along the driveway as a result of surface water infiltrating into the underlying subgrade soils; however, in our opinion the driveway would remain serviceable. If it is desired to reduce the potential for rutting/yielding, the thickness of the baserock could be increased or a geotextile strength fabric such as MirafiRS380i or equivalent could be incorporated between the subgrade and the overlying Class 2 aggregate baserock.

Sand Set Pavers or Flagstones

We anticipate that sand-set pavers or flagstones may be used for exterior hardscape. We generally recommend that they be placed in accordance with the manufacturer's recommendations. At a minimum, we generally recommend that pavers be underlain by at least 6 inches of compacted Class 2 aggregate baserock for pedestrian loads. A representative from our office should observe the subgrade conditions of the hardscape prior to placement of baserock. Prior to placement of the baserock, the subgrade soils should be scarified and moisture conditioned to a depth of at least 6 inches and compacted in accordance with the Compaction section of this report.

EARTHWORK

A moderate amount of earthwork is anticipated as part of the proposed construction, including site grading, basement excavation, excavation of drilled pier and spread footing foundations, retaining wall drainage and backfill, subgrade preparation beneath hardscape, placement and compaction of engineered fill, backfill in utility trenches, and installation of final surface drainage controls. Earthwork should be performed in accordance with the following recommendations.

Clearing & Site Preparation

Initially, the proposed improvement areas should be cleared of obstructions, including existing flatwork, utilities, and trees not designated to remain. Holes or depressions resulting from the removal of underground obstructions below proposed subgrade levels, such as root balls, should be backfilled with engineered fill, placed and compacted in accordance with the recommendations provided below. After clearing, the proposed improvement areas should be adequately stripped to remove surface vegetation and organic-laden topsoil. The stripped material should be used as engineered fill; however, it may be stockpiled and used for landscaping purposes.

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) may be suitable for use as engineered fill contingent upon review by our firm. In general, fill material should not contain rocks or pieces larger than 6 inches in greatest dimension, and should contain no more than 15 percent larger than 2.5 inches. Any required imported fill should be predominantly granular material or low plasticity material with a plasticity index of less than approximately 15 percent. Any proposed fill for import should be approved by Murray Engineers, Inc. prior to importing to the site. Our approval process may require index testing to establish the expansive potential of the soil; therefore, it is important that we receive samples of any proposed import material at least 3 days prior to planned importing. Class 2 aggregate baserock should meet the specifications outlined in the Caltrans Standard Specifications, latest edition.

Location & Backfill of Temporary Basement

In planning the location for any temporary access ramps for the basement, the contractor should consider the future location of any at-grade structures or hardscape. If possible, we recommend that ramp excavations be kept approximately 5 feet away from proposed at-grade structures and hardscape. If placement of the ramp within this zone is unavoidable, it is imperative that the backfilled soils be compacted in accordance with the specifications outlined in Table 1 of the Compaction section of this report. A representative of Murray Engineers, Inc. should observe and test the compaction of the ramp backfill. In addition, we recommend that a note be included on the structural plans referencing these recommendations.

Compaction

Prior to placing engineered fill, the subgrade soil should be scarified and compacted, as necessary. Material used for fill should be placed in uniform lifts, no more than 8-inches in uncompacted thickness. The fill material should be moisture conditioned, as necessary, and compacted in accordance with the specifications listed in Table 1 below. The relative compaction and moisture content specified in Table 1 are 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 (entire fill)	Near optimum
Upper 6 inches of relatively non-expansive subgrade beneath hardscape	90 percent	Near optimum
Upper 6 to 12 inches of relatively expansive subgrade beneath hardscape	87 to 90 percent	≥3% over optimum
Aggregate baserock under hardscape	95 percent	Near optimum
½- to ¾-inch Crushed Rock - Compact with at least 3 passes of a vibratory plate with lift-thickness ≤ 12 inches.	see note at left	Not critical
Backfill of utility trenches using on-site soil	90 percent	Near optimum
Backfill of utility trenches using imported sand	90 percent	Near optimum

*Relative to ASTM D 1557, latest edition.

Keying & Benching

Unretained fill placed on slopes that are flatter than 5:1 should be supported on level benches bearing in supportive bedrock, 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.

Prior to fill placement on slopes steeper than 5:1, a construction keyway should be excavated at the toe of the fill. The keyway should be a minimum of 8 feet wide or of a width equal to half the height of the fill slope, whichever is greater. The keyway should be excavated a minimum of 2 feet into competent supportive bedrock 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

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

Final Slopes

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

Temporary Slopes & Trench Excavations

The contractor should be responsible for the stability of all temporary cut 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 and state safety regulations, including the current OSHA excavation and trench safety standards. Because of the potential for variable soil conditions, field modifications of temporary cut slopes may be required. Unstable materials encountered on the slopes during the excavation should be trimmed off, even if this requires cutting the slope back at flatter inclinations.

SITE DRAINAGE

Control of surface drainage is critical for projects on hillsides and in expansive soil areas. Roof run-off, rain, and irrigation water should not be allowed to pond near the residence, detached garage or on exterior hardscape. The proposed residence and detached garage should be provided with roof gutters and downspouts. Water collected in the gutters should not be allowed to discharge freely onto the ground surface adjacent to the foundations and should be conveyed away from the structures via buried closed conduits and routed to a suitable discharge outlet. The finished grades around the structures should be designed to drain surface water away from the structures, slabs, and yard areas to suitable discharge points. Where such surface gradients are difficult to achieve, we recommend that area drains or surface drainage swales be installed to collect surface water and convey it away from the residence.

Surface runoff should be prevented from flowing over the top of any artificial slope. The ground surface at the top of any artificial slopes 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 structures by buried closed conduit and discharged onto an energy dissipater 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 recommendations, if needed.

REQUIRED FUTURE SERVICES

Plan Review

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

- All earthwork and site drainage, including site grading, basement excavation, excavation of drilled pier and spread footing foundations, retaining wall drainage and backfill, subgrade preparation beneath hardscape, placement and compaction of engineered fill, backfill in utility trenches, and installation of final surface drainage controls should be performed in accordance with the geotechnical report prepared by Murray Engineers, Inc., dated January 28, 2015. 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) evaluate 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 exposed during construction, it may be necessary to re-evaluate our recommendation.

LIMITATIONS

This report has been prepared for the sole use of Tim Sullivan, specifically for developing geotechnical design criteria relating to design and construction of the proposed residence and associated improvements on the property, APN 082-160-130 in unincorporated San Mateo County, California. The opinions presented in this report are based upon borings at widely separated locations, site reconnaissance, review of field data made available to us, and upon local experience and engineering judgment. Our opinions have been formulated in accordance with generally accepted engineering geologic and geotechnical engineering practices that exist in the San Francisco Bay Area at the time this report was prepared. The recommendations presented in this report are based on the assumption that soil and geologic conditions at or between borings do not deviate substantially from those encountered. It should be understood that geotechnical issues may become apparent during the course of construction that were not apparent at the time this report was prepared. No warranty, expressed or implied, is made or should be inferred. In addition, we are not responsible for data presented by others.

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

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

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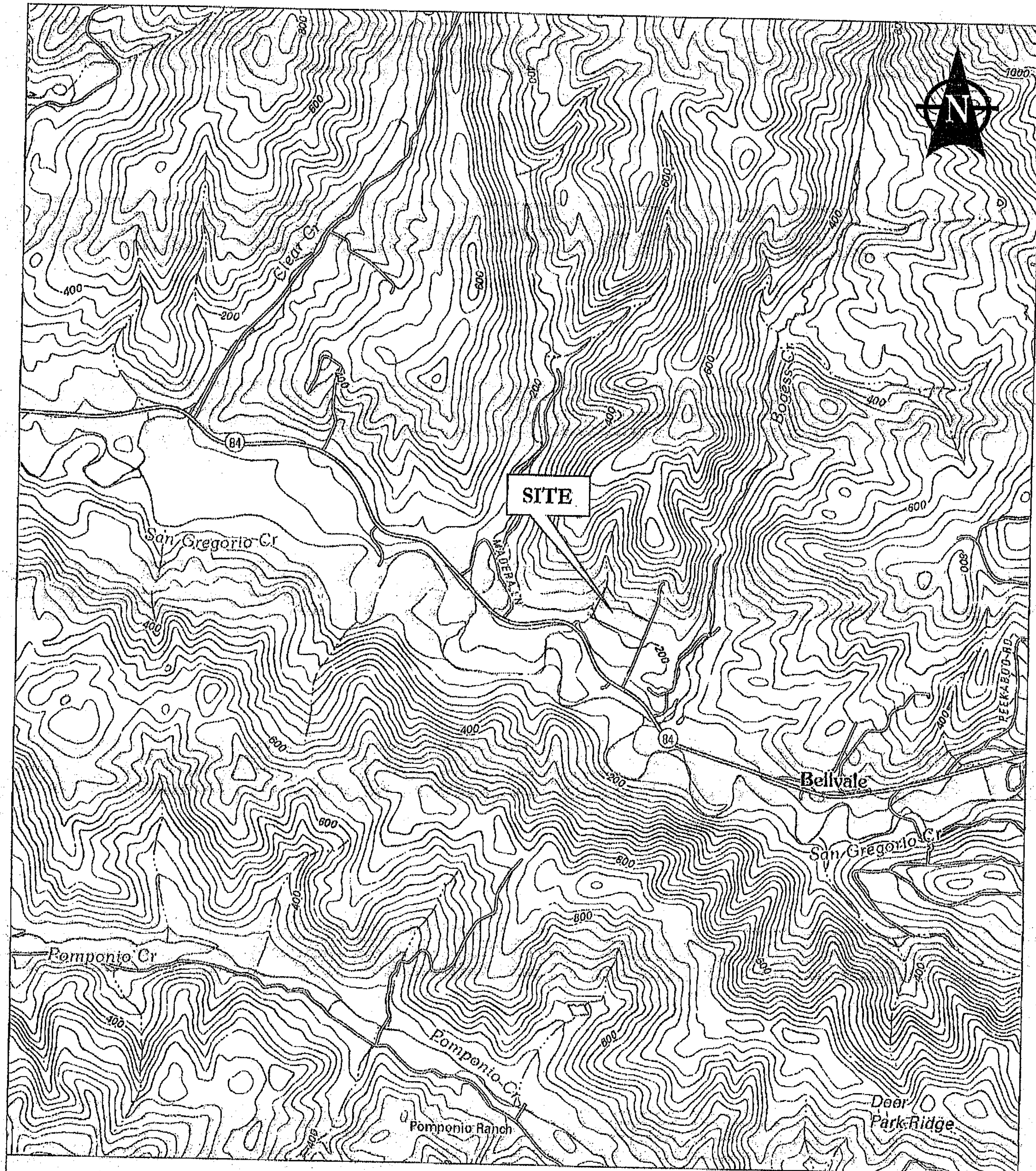
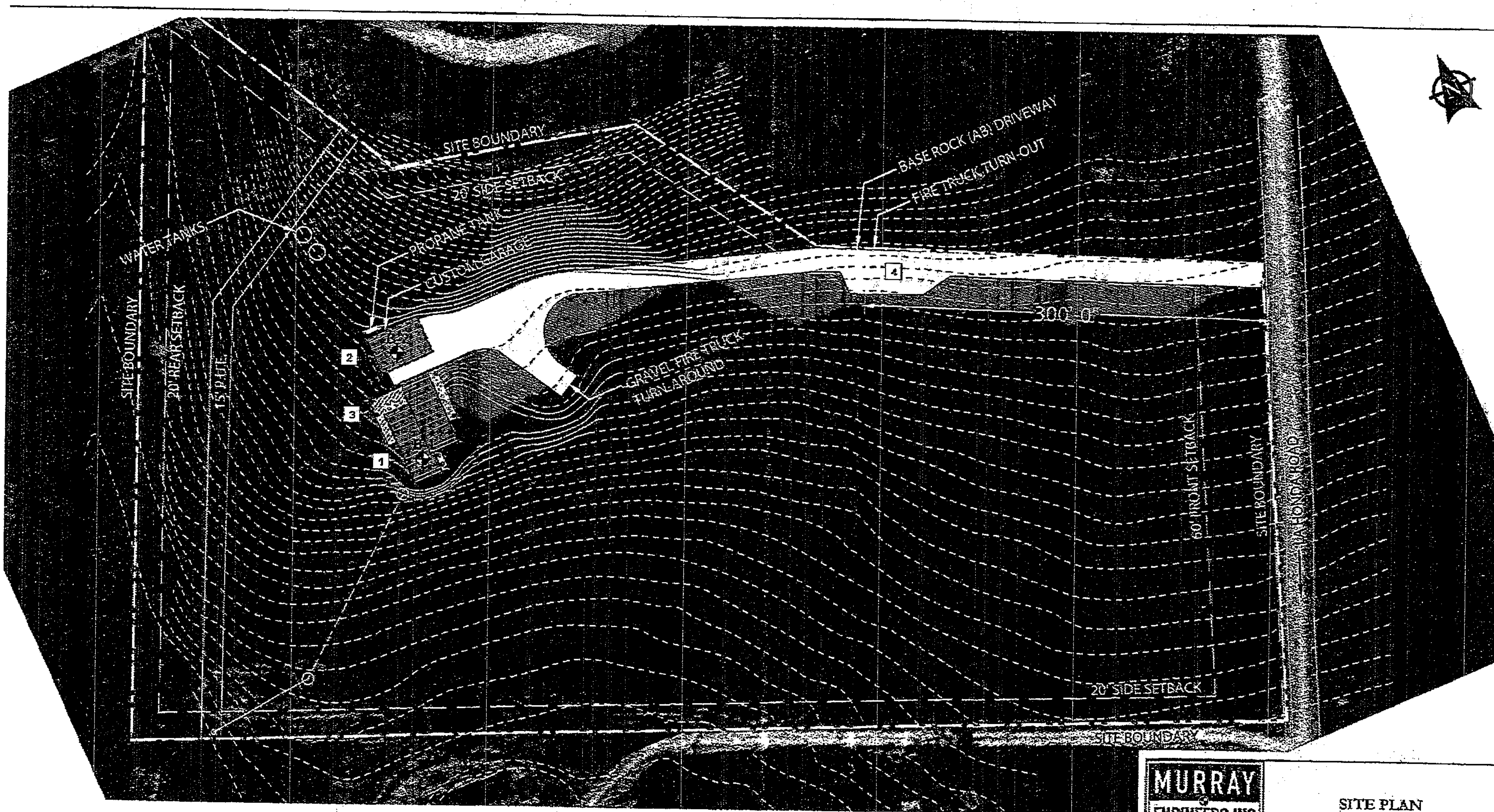


Base: USGS La Honda Quadrangle, 7.5 Minute Series (Topographic), 2012 Scale: 1 inch = 2,000 feet

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	SULLIVAN RESIDENCE APN 082-160-130 SAN MATEO COUNTY, CALIFORNIA		VICINITY MAP
	PROJECT NO. 2150-1R1	JANUARY 2015	FIGURE A-1

25



KEY NOTES

1. Outline of (N) Residence with Daylighting Basement Level Beneath (Dashed)
2. Outline of (N) Garage (Dashed)
3. Area of (N) Exterior Wood Decking (Hatched)
4. Area of (N) Driveway (Shaded)

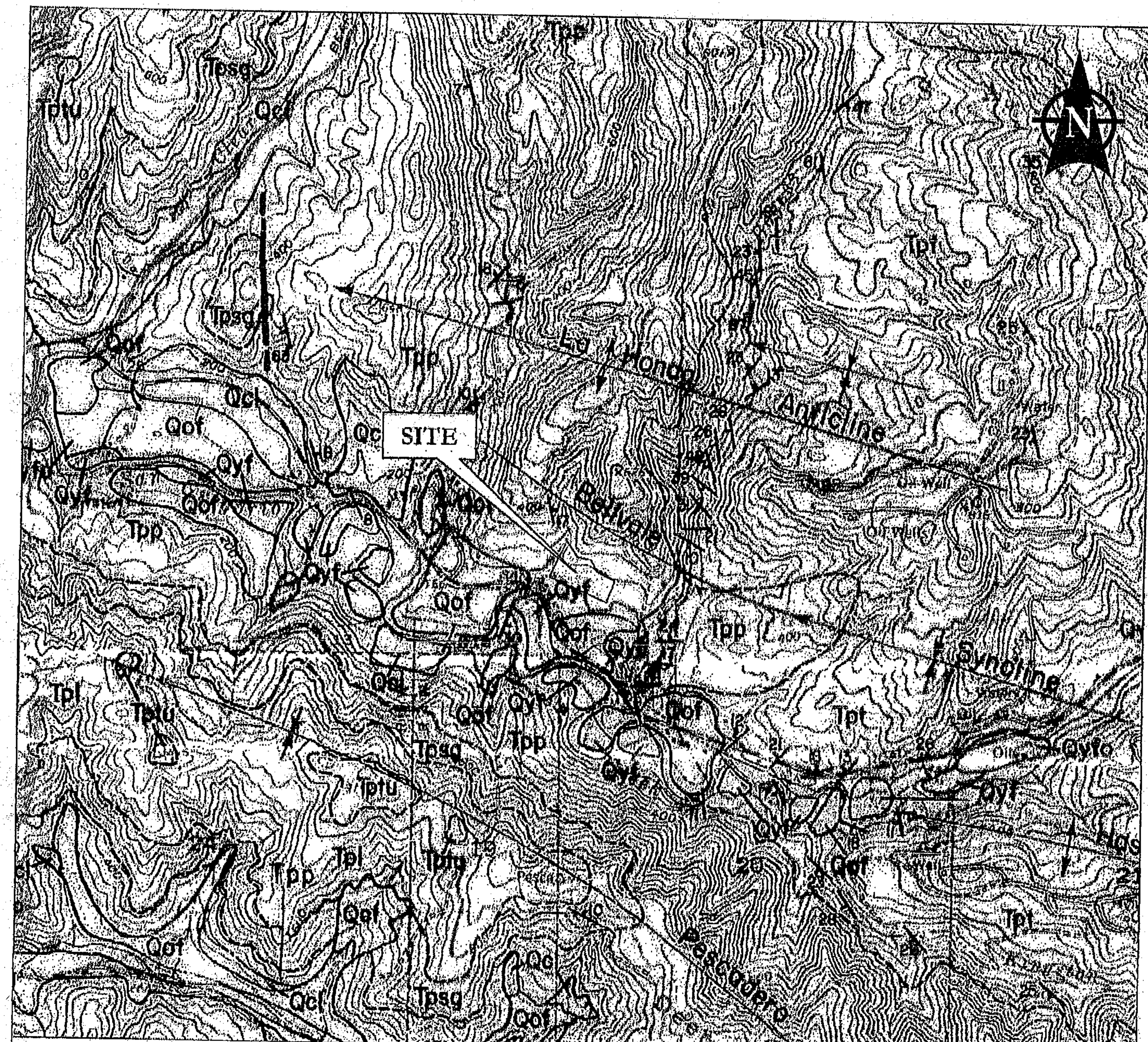
LEGEND

B-1 Approximate Location of Soil Boring by Murray Engineers, Inc., dated December 19, 2014

Base: Conceptual Site Plan by Blu Homes, dated November 18, 2014

Approximate Scale: 1 inch = 70 feet

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	SITE PLAN	
SULLIVAN RESIDENCE APN 082-160-130 SAN MATEO COUNTY, CALIFORNIA		
PROJECT NO. 2150-1R1	JANUARY 2015	FIGURE A-2



Legend & Selected Map Symbols

Qyf Younger Alluvial Deposits

Qof Coarse-Grained Older Alluvial Fan and Stream Terrace Deposits

Tpp Purisma Formation (Upper Miocene and Pliocene)

Contact, dashed where approximately located

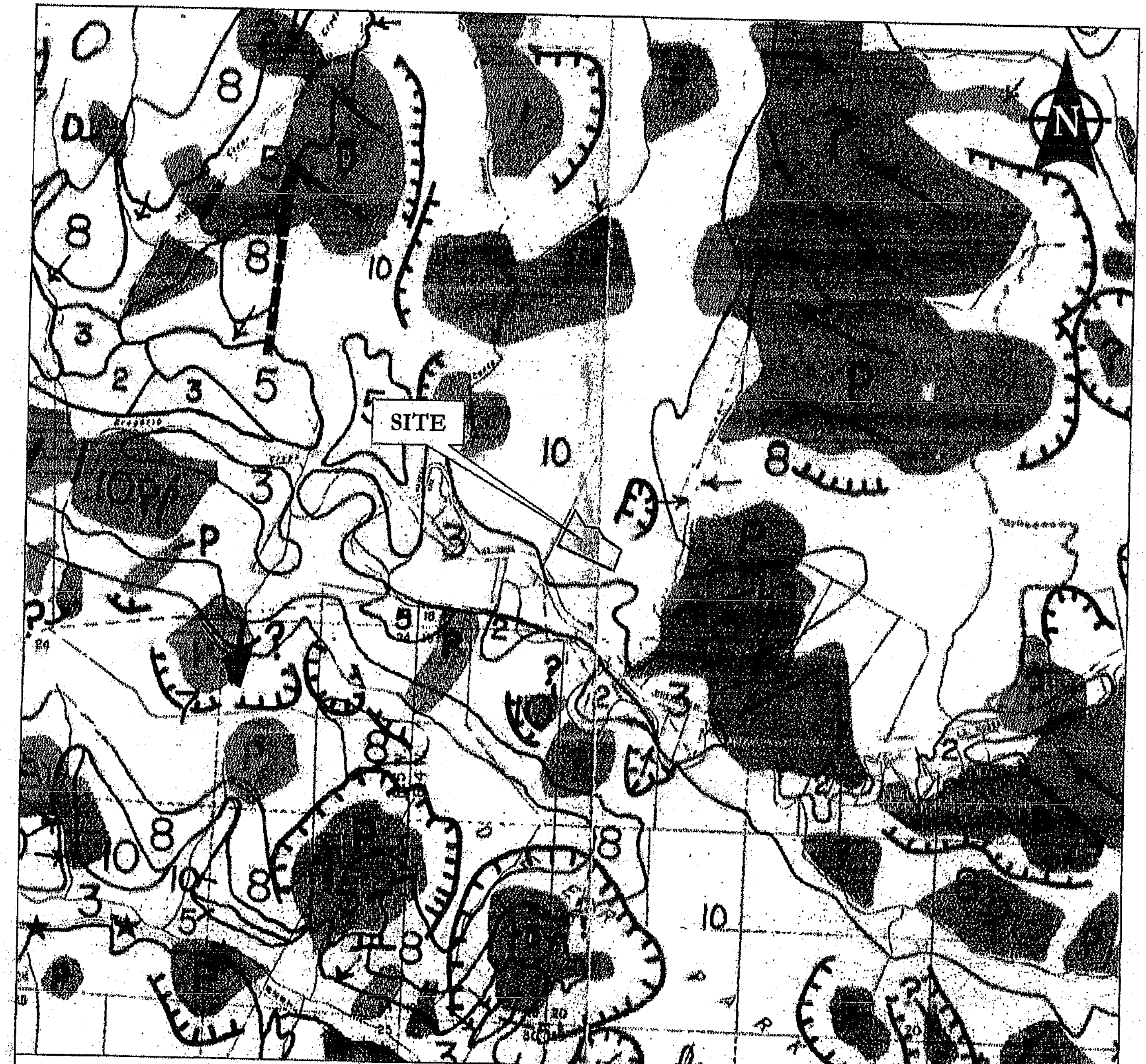
Anticline

Syncline

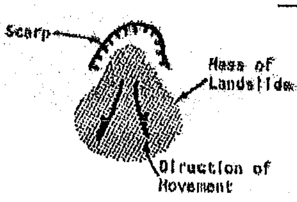
7 Inclined

Base: Preliminary Geologic Map of the La Honda & San Gregorio Quadrangle, Earl E. Brabb, 1980
Approximate Scale: 1 inch = 2,000 feet

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	SULLIVAN RESIDENCE APN 082-160-130 SAN MATEO COUNTY, CALIFORNIA	VICINTY GEOLOGIC MAP
	PROJECT NO. 2150-1R1 / JANUARY 2015	FIGURE A-3



Legend & Selected Map Symbols



D = Definite landslide
A = Active
F = Mapped in the field
P = Probable landslide deposit
Small landslide (50-500 ft. max. dimension mapped by photointerpretation)
Small landslide (50-500 ft.) mapped in the field
Suspected or conjectured landslide.

Base: Preliminary Map of Landslide Deposits in San Mateo County, California, 1972
Approximate Scale: 1 inch = 2,000 feet

MURRAY ENGINEERS INC GEOTECHNICAL SERVICES	SULLIVAN RESIDENCE APN 082-160-130 SAN MATEO COUNTY, CALIFORNIA		VICINITY LANDSLIDE MAP
	PROJECT NO. 2150-1R1	JANUARY 2015	FIGURE A-4

Date(s) Drilled: December 19, 2014	Logged By: CT	Checked By: KK
Drilling Method: Continuous Sampling & Flight Auger	Drill Bit Size/Type: 4 inch diameter drill bit	Total Depth of Borehole: 8.1 feet bgs
Drill Rig Type: Sampling Tripod & Minuteman	Drilling Contractor: Exploration Geoservices, Inc.	Approximate Surface Elevation: ""
Groundwater Level and Date Measured: Not Encountered ATD	Sampling Method(s): 3" OD, 2.5" OD, & 2" OD SPT Split Spoon Samplers	Hammer Data: 140 lb, 30 in drop, rope & cathead
Borehole Backfill: Cuttings	Location: Southwest portion of proposed residence	

Elevation, feet	Depth, feet	Sample Type	Sampling Resistance, blows/foot	Relative Consistency	USCS Symbol	MATERIAL DESCRIPTION	Water Content, %
	0			Medium Stiff	CL	SILTY CLAY, dark brown, homogeneous, medium plasticity, trace mudstone gravels, moist (Colluvium)	30
			6	Soft*	BR	MUDSTONE, yellowish brown, severely weathered, moderately fractured, moist (Purisima Formation) *designates hardness of bedrock (see Figure B-5)	29
							21
			39/3"				23
			50/3"				28
	5						
			50/1"			Refusal at 8.1 feet bgs	
	10						

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MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

SULLIVAN RESIDENCE
APN 082-160-130
SAN MATEO COUNTY, CALIFORNIA

PROJECT NO. 2150-1R1 | JANUARY 2015

LOG OF BORING B-1

FIGURE B-1

Date(s) Drilled	December 19, 2014	Logged By	CT	Checked By	KK
Drilling Method	Continuous Sampling & Flight Auger	Drill Bit Size/Type	4 inch diameter drill bit	Total Depth of Borehole	5 feet bgs
Drill Rig Type	Sampling Tripod & Minuteman	Drilling Contractor	Exploration Geoservices, Inc.	Approximate Surface Elevation	--
Groundwater Level and Date Measured	2 feet ATD	Sampling Method(s)	3" OD, 2.5" OD, & 2" OD SPT Split Spoon Samplers	Hammer Data	140 lb, 30 in drop, rope & cathead
Borehole Backfill	Cuttings	Location	Area of proposed garage		

Elevation, feet	Depth, feet	Sample Type	Sampling Resistance, blows/foot	Relative Consistency	USCS Symbol	MATERIAL DESCRIPTION	Water Content, %
	0			Medium Stiff	CL	SILTY CLAY, dark brown, homogeneous, medium plasticity, trace mudstone gravels, moist (Colluvium)	27
			7	Hard	CL	SILTY CLAY, yellowish brown, homogeneous, medium plasticity, minor fine-grained sand, trace mudstone gravels, moist (Colluvium) PI=22%; LL=36% (sample from 1 to 2 feet) (ATD) ⊻	19 19
			65 50/1"	Soft*	BR	SANDSTONE, light yellowish brown, severely weathered, moderately fractured, moist (Purisima Formation) *designates hardness of bedrock (see Figure B-5)	19 32
	5					Refusal at 5 feet bgs	
	10						

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MURRAY ENGINEERS INC
GEOTECHNICAL SERVICES

SULLIVAN RESIDENCE APN 082-160-130 SAN MATEO COUNTY, CALIFORNIA		LOG OF BORING B-2
PROJECT NO. 2150-1R1	JANUARY 2015	FIGURE B-2

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

COLUMN DESCRIPTIONS

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

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

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

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

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

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

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

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

FIELD AND LABORATORY TEST ABBREVIATIONS

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

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

TYPICAL MATERIAL GRAPHIC SYMBOLS

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

TYPICAL SAMPLER GRAPHIC SYMBOLS

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

OTHER GRAPHIC SYMBOLS

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

GENERAL NOTES

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

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

RELATIVE DENSITY

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

CONSISTENCY

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

GRAIN SIZES

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

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

SIEVE OPENINGS U.S. STANDARD SERIES SIEVE

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

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

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

WEATHERING

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

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

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

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

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

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

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

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

HARDNESS

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

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

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

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

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

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

JOINT BEDDING & FOLIATION SPACING

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

ROCK QUALITY DESIGNATOR (RQD)

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

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SULLIVAN RESIDENCE APN 082-160-130 SAN MATEO COUNTY, CALIFORNIA		KEY TO BEDROCK DESCRIPTIONS
PROJECT NO. 2150-1R1	JANUARY 2015	FIGURE B-5

APPENDIX C

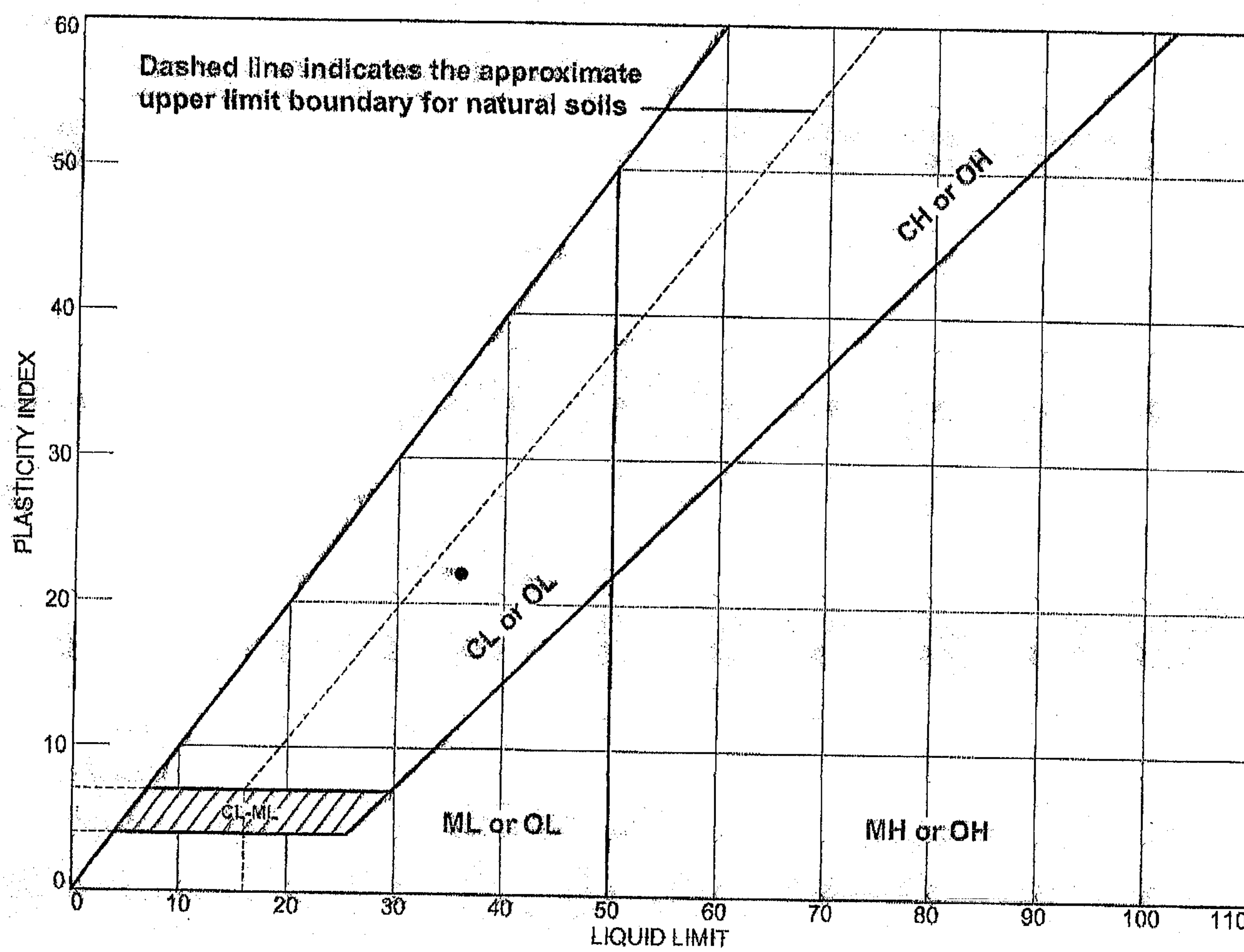
LABORATORY TESTS

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

The natural moisture content was evaluated in general accordance with ASTM D 2216 on most samples recovered from the borings. This test determines the moisture content representative of field conditions at the time the samples were collected. The results are presented on the boring logs, at the appropriate sample depths.

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

LIQUID & PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	Boring 2	1	1' to 2'	19.3	14	36	22	CL

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LIQUID & PLASTIC LIMITS TEST REPORT

PROJECT NO. 2150-1R1 | JANUARY 2015 | FIGURE C-1