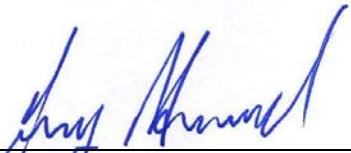


Type of Services	Updated Geotechnical Investigation
Project Name	Highland Estates Lots 5 through 11
Location	Ticonderoga Drive/Cobblehill Place/Cowpens Way San Mateo, California
Client	Ticonderoga Partners, LLC.
Client Address	655 Skyway, Suite 230 San Carlos, California
Project Number	230-1-5
Date	October 30, 2015



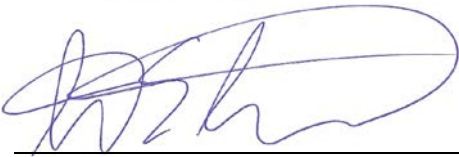
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Type of Services	Updated Geotechnical Investigation
Project Name	Highland Estates Lots 5 through 11
Location	Ticonderoga Drive/Cobblehill Place/Cowpens Way San Mateo, California

SECTION 1: INTRODUCTION

This updated geotechnical report was prepared for the sole use of Ticonderoga Partners LLC for the Highland Estates Lots 5 through 11 project in San Mateo, California. The approximate location of the project sites are shown on the Vicinity Map, Figure 1. For our use, we were provided with the following documents:

- A set of plans for Lots 1 through 11 titled “Highland Estates,” prepared by BKF Engineers, Inc., dated January 20, 2010.
- A report titled “Revised Geologic Evaluation, Environmental Impact Report, Highland Estates Residential Development Project, San Mateo County, California,” prepared by Treadwell & Rollo, dated August 27, 2009.
- A report titled “Geologic Evaluation, Environmental Impact Report, Highlands Estates Residential Development Project, San Mateo County, California,” prepared by Treadwell & Rollo, dated September 23, 2008.
- A report titled “Geotechnical Investigation and Geologic Hazards Review, Four Single-Family Homes, Ticonderoga Drive, San Mateo, California,” prepared by TRC Lowney, dated February 7, 2006.
- A report titled “Supplemental Geotechnical Report, Responding to Geotechnical Review Comments for Highland Estates, San Mateo County, California,” prepared by Soil Foundation Systems, Inc., dated November 1994.
- A report titled “Geotechnical Investigation Report for Highland Estates, San Mateo, California,” prepared by Soil Foundation Systems, Inc., dated July 1993.

1.1 PROJECT BACKGROUND AND PURPOSE

Lots 5 through 11 were once part of a much larger parcel of land known as the “Highland Estates Parcel” located west of Polhemus Road. The vacant, irregularly-shaped parcel consisted of approximately 99-acres of land bounded by existing residential and commercial development in San Mateo County, California. During the past two to three decades, there have been many previous land development proposals and geotechnical/geologic reports prepared for the Highland Estates project site. The current approved land development plan, which consists of 11 lots, is a scaled back version of previous land planning proposals and consists of construction of homes to “infill” undeveloped portions around the perimeter of the large parcel which will remain undeveloped.

Numerous geotechnical and geologic reports have been prepared for the Highland Estates site. The first investigations were performed by Soil Foundation Systems, Inc. (SFSI) in 1990, 1993, and 1994, then more recently by TRC/Lowney Associates in 2006. Mr. K.C. Sohn, G.E., the geotechnical engineer for SFSI is deceased. Mr. Scott Fitinghoff, G.E., principal engineer at Cornerstone Earth Group became the geotechnical engineer for the project after Mr. Sohn’s death in 1999 while employed by Lowney Associates and which was acquired by TRC in 2000. In 2008 and 2009, Treadwell and Rollo, Inc. performed a geologic evaluation for the Environmental Impact Report for the project. To maintain continuity of geotechnical engineers for the Highland Estates project, Cornerstone Earth Group accepted the role of geotechnical engineer-of-record for the project. In 2011, Cornerstone Earth Group performed a design-level geotechnical investigation for Lots 1 through 4. The residences on Lots 1 through 4 have been recently constructed.

The purpose of this report is to provide a summary of the previous reports, the results of our supplemental exploration and engineering analysis, and to prepare an updated geotechnical investigation report for Lots 5 through 11 based on grading for the project shown on the plans by BKF Engineers.

1.2 PROJECT DESCRIPTION

Lots 5 through 8 will be constructed on the northern side of Ticonderoga Drive which slopes upward from Ticonderoga Drive with slopes as steep as approximately 2:1 to 2½:1 (H:V). Lots 9 and 10 will be constructed at the end of Cobblehill Place along the approximate crest of a ridge that slopes gently to steeply downward to the east, northeast away from the end of Cobblehill Place. Lot 11 will be constructed at the end of Cowpens Way and generally slopes downward away from the end of Cowpens Way.

Construction at each lot will consist of a multi-level, single-family, wood-framed house designed to step up the hill (Lots 5 through 8) or down the hill (Lots 9 through 11) and follow the natural contours. Driveways and garages are anticipated to be located adjacent to the fronting road. The structures will be supported on drilled pier and grade beam foundations with raised wood or structural concrete slab floors. Significant grading is anticipated for Lots 5 through 8 to mitigate the mapped landsliding. Grading for Lots 9 through 11 is anticipated to potentially include cuts and fills of up to 10 feet. We assume that retaining walls will be built to retain fill adjacent to

garage and lower house walls. Appurtenant utilities, landscaping, driveways, and other improvements necessary for lot development is also planned.

Structural loads are not available at this time, however loads for the structures are anticipated to be typical of these buildings with interior column loads on the order of 5 to 15 kips. The proposed layout of the residences is shown on the Site Plan and Geologic Maps, Figure 2A to 2C.

1.3 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated April 20, 2015 and consisted of a site reconnaissance, field and laboratory program for Lot 11 to further evaluate physical and engineering properties of the subsurface soils and bedrock, landslide mitigation plans, engineering analysis to prepare recommendations for site work and grading, building foundations, flatwork, retaining walls, and pavements, and preparation of this report. Brief descriptions of our exploration and laboratory programs for Lot 11 are presented below.

1.4 PREVIOUS INVESTIGATIONS BY OTHERS

Soil Foundation Systems (1993 and 1994), TRC Lowney (2006), and Treadwell & Rollo (2009) performed geotechnical investigations and geologic feasibility reviews for Lots 5 through 11. This previous work was reviewed and data obtained from the previous investigations was incorporated into our investigation. Data and logs from these prior investigations are included in Appendix C.

1.5 EXPLORATION PROGRAM

To supplement the previous investigations by others at Lots 5 through 11, our field exploration consisted of one boring drilled on July 28, 2015 with portable Minuteman solid-stem auger drilling equipment. The boring was drilled to a depth of 15 feet. The boring was backfilled with cement grout in accordance with local requirements. The approximate location of our exploratory boring is shown on the Site Plan and Geologic Map, Figure 2C. Details regarding our field program are included in Appendix A.

1.6 LABORATORY TESTING PROGRAM

In addition to visual classification of samples, the laboratory program focused on obtaining data for foundation design and seismic ground deformation estimates. Testing included moisture contents, dry densities, and a Plasticity Index test. Details regarding our laboratory program are included in Appendix B.

1.7 NATURALLY OCCURRING ASBESTOS TESTING

We performed testing for naturally occurring asbestos (NOA) on one sample from our Boring EB-1 drilled at Lot 11 close to the previously identified serpentinite found in Soil Foundations Systems nearby borings. The sample from our boring was tested for naturally occurring asbestos (NOA) using Polarized Light Microscopy in accordance with the California Air Resources Board (CARB) Method 435. NOA was not detected. The analytical report is included in Appendix D.

1.8 ENVIRONMENTAL SERVICES

Environmental services were not requested for this project. If environmental concerns are determined to be present during future evaluations, the project environmental consultant should review our geotechnical recommendations for compatibility with the environmental concerns.

SECTION 2: REGIONAL SETTING

2.1 GEOLOGICAL SETTING

The San Francisco peninsula is a relatively narrow band of rock at the north end of the Santa Cruz Mountains separating the Pacific Ocean from the San Francisco Bay. It represents one mountain range in a series of northwesterly-aligned mountains forming the Coast Ranges geomorphic province of California that stretches from the Oregon border nearly to Point Conception. In the San Francisco Bay area, most of the Coast Ranges have developed on a basement of tectonically mixed Cretaceous- and Jurassic-age (70 to 200 million years old) rocks of the Franciscan Complex. Locally, these basement rocks are capped by younger sedimentary and volcanic rocks. Most of the Coast Ranges are covered by younger surficial deposits that reflect geologic conditions for approximately the last million years.

Lateral and vertical movement on the many splays of the San Andreas Fault system and other secondary faults has produced the dominant northwest-oriented structural and topographic trend seen throughout the Coast Ranges today. This trend reflects the boundary between two of the Earth's major tectonic plates: the North American plate to the east and the Pacific plate to the west.

The San Andreas Fault is the dominant structure in the system, nearly spanning the length of California, and capable of producing the highest magnitude earthquakes. Many other sub-parallel or branch faults within the San Andreas system are equally active and nearly as capable of generating large earthquakes. Right-lateral movement dominates these faults, but an increasingly large amount of thrust faulting resulting from compression across the system is now being identified as well.

The San Andreas Fault is located approximately 4,700 feet west of the lots, where it trends northwesterly through Crystal Springs Reservoir. Distances for other nearby active faults are shown in Tables 1a to 1c.

More locally, the site is in an area dominated by bedrock units of the Cretaceous and/or Jurassic Franciscan Complex. Several regional scale geologic maps covering the area have been published of the area including those by Lajole et al. (1974), Leighton (1976), Brabb and Pampeyan (1983), Wentworth et al. (1985), Pampeyan (1994), Brabb et al. (1998) and Brabb et al. (2000) depict similar geologic units underlying the site. Of these published maps Pampeyan's depiction of the bedrock units is consistent with our site observations (see below). The Pampeyan mapping depicts the area of the Highland Estates as underlain by "Sheared rock" ("Fsr") of the Franciscan Complex.

The sheared rock forms an extensive outcrop across the immediate area. No structural trends within the sheared rock are shown on the Pampeyan map. Pampeyan also shows Quaternary surficial deposits ("slope wash, ravine fill and colluvium," "Qsr") overlying the sheared rock on northeast to southeast facing hillsides located about 150 feet to the southeast of the site. Small, isolated outcrops of greenstone occur in the general area but not adjacent to the site. One area of serpentinite was encountered in some of the exploratory borings conducted on Lot 11. This unit is extensive to the south and this occurrence may represent a local interfingering of the two units in the immediate area of Lot 11 and to the south of the Lot.

The following geologic unit descriptions come from Pampeyan (1994). The Holocene deposits (Qsr) are described as "interfingering deposits of colluvium and ravine fill which is unconsolidated to moderately consolidated deposits of sand, silt, clay and rock fragments." The sheared rock is described as "small to large fragments of hard rock in a matrix of sheared rock that is derived mostly from shale and sandstone of the Franciscan Complex." The sheared rock is generally "coherent and firm, but soft in places, especially where weathered." Serpentinite is described as; "soft, sheared serpentinite enclosing blocks of hard gray to greenish gray, unsheared serpentinite and ultramafic rocks."

2.2 REGIONAL SEISMICITY

The San Francisco Bay area is recognized by geologists and seismologists as one of the most seismically active regions in the United States. Significant earthquakes occurring in the Bay area are generally associated with crustal movement along well-defined, active, fault zones of the San Andreas Fault system (see Figure 3). The San Andreas Fault generated the great San Francisco earthquake of 1906 and the Loma Prieta earthquake of 1989.

The faults considered capable of generating significant earthquakes are generally associated with the well-defined areas of crustal movement, which trend northwesterly. Tables 1a to 1c below present the State-considered active faults in order of increasing distance within 25 kilometers (16.5 miles) of the lot locations.

Table 1a: Approximate Fault Distances for Lots 5 through 8

Fault Name	Distance	
	(miles)	(kilometers)
San Andreas (1906)	0.8	1.3
Monte Vista-Shannon	7.1	11.5
San Gregorio	8.3	13.4

Table 1b: Approximate Fault Distances for Lots 9 and 10

Fault Name	Distance	
	(miles)	(kilometers)
San Andreas (1906)	0.9	1.4
Monte Vista-Shannon	7.2	11.6
San Gregorio	8.4	13.5

Table 1c: Approximate Fault Distances for Lot 11

Fault Name	Distance	
	(miles)	(kilometers)
San Andreas (1906)	0.8	1.3
Monte Vista-Shannon	7.3	11.8
San Gregorio	8.3	13.3

A regional fault map is presented as Figure 3, illustrating the relative distances of the lots to significant fault zones.

2.3 FUTURE EARTHQUAKE PROBABILITIES

The San Francisco Bay area region is one of the most seismically active areas in the Country. While seismologists cannot predict earthquake events, the U.S. Geological Survey's Working Group on California Earthquake Probabilities 2015 revises earlier estimates from their 2008 (2008, UCERF2) publication. Compared to the previous assessment issued in 2008, the estimated rate of earthquakes around magnitude 6.7 (the size of the destructive 1994 Northridge earthquake) has gone down by about 30 percent. The expected frequency of such events statewide has dropped from an average of one per 4.8 years to about one per 6.3 years. However, in the new study, the estimate for the likelihood that California will experience a magnitude 8 or larger earthquake in the next 30 years has increased from about 4.7% for UCERF2 to about 7.0% for UCERF3.

UCERF3 estimates that each region of California will experience a magnitude 6.7 or larger earthquake in the next 30 years. Additionally, there is a 63 percent chance of at least one magnitude 6.7 or greater earthquake occurring in the Bay Area region between 2007 and 2036. During such an earthquake the danger of fault surface rupture at the site is slight, but very strong ground shaking would occur. A similar level of ground shaking was demonstrated when the 1989 Loma Prieta earthquake caused severe damage in Oakland and San Francisco, more than 50 miles from the fault rupture. Although earthquakes can cause damage at a considerable distance, shaking will be very intense near the fault rupture. Therefore, earthquakes located in urbanized areas of the region have the potential to cause much more damage than the 1989 Loma Prieta earthquake.

SECTION 3: SITE CONDITIONS

3.1 RECENT HISTORY

The larger Highland Estates development is located on the northwest side of Ticonderoga Drive within the western boundary of the City of San Mateo, on unincorporated land in San Mateo County, California. The 1943 and 1946 photographs reveal there was no residential development at or near the site and the eastern slope of the site was covered with shrubs and trees, similar to the present condition. The photographs reveal an apparent old landslide located southeast of the smallest water tower within the property. By the time of the 1956 photographs, the Highland Estates development area had been completely cleared and graded but no homes had been constructed yet. By the fall of 1956, roughly one-third of the homes within the Highlands Estates development had been completed and all the streets had been graded. By 1961, most of the Highland Estates development had been completed. The area proposed for Lots 5 through 11 appears as it does presently, with grasses and scattered oak trees. The 1981 photographs show the site appears as it does today. The photos taken between 1983 and 2005, revealed no changes at the site. An area of shallow groundwater seepage or springing was apparent in the area of the currently proposed Lots 5 through 8, near the mapped contact between sandstone and serpentinite.

3.2 SURFACE DESCRIPTION AND TOPOGRAPHY

The proposed 7-lot development is located on the northeast flank of Pulgas Ridge, a knob of resistant bedrock that rises a few hundred feet above the surrounding hilly terrain. The topography of the specific lots is shown on Figures 2A to 2C. The general area is characterized with rolling terrain and northwest trending ridges and drainages on the peninsula segment of the Santa Cruz Mountains. The Highland Estate area is generally bound to the northwest and northeast by Bunker Hill Drive and Polhemus Road, to the southeast by Ticonderoga Drive and a natural drainage course and undeveloped slope, and to the southwest by developed residential parcels. The lots generally slope moderately steep to very steep, with gradients between approximately 2:1 to 3:1.

The current evaluation applies specifically to Lots 5 through 11. Lots 5 through 8 are currently vacant land located along the north side of Ticonderoga Drive. The lots are bound by residential development to the west and north, undeveloped land to the east, and Ticonderoga

Drive to the south. The lots slope upward fairly steeply from Ticonderoga Drive. Lots 9 and 10 are currently vacant land as well. The lots are bounded by residential developments and Cobblehill Place on the southwest and undeveloped land on the remaining boundaries. The lots are located along the crest of a ridge and generally slope gently to steeply toward the east-northeast away from the end of Cobblehill Place. Lot 11 is also currently vacant land located at the end of Cowpens Way. This lot is bounded by residential development and Cowpens Way to the southwest and undeveloped land on the other sides. The lot generally slopes downward away from the end of Cowpens Way. Slopes on the subject lots are generally steep to very steep, with gradients of approximately 2:1 to 3:1 (horizontal to vertical). The subject residential lots have varied topography and contain a very thick growth of oak and other trees as well as a thick understory growth of shrubs. Site drainage is characterized by uncontrolled sheet-flow down to the southeast. Sheet flow coming off the ridges and hillsides have deposited slope debris and colluvium over the older Franciscan rocks.

3.3 SITE GEOLOGY AND SUBSURFACE CONDITIONS

Prior Investigations of the overall 99 acre Highland Estates development:

Several prior investigations were performed for the development of the larger Highland Estates site. A previous investigation by Soil Foundation Systems, Inc (“SFS”; 1993) and a supplemental investigation (SFS, 1994) of the overall Highland Estates were conducted. They had also included within their report previous subsurface data collected at the site (Test Pit logs) by Berlogar Long and Associates (“BLA”) in 1980. The SFS studies included the logging of numerous borings and test pits, laboratory testing and slope stability analyses. Blocks of Graywacke sandstone of up to 2 acres in size were identified in their mapping, which they broke as distinct mapping units. They characterized the 99 acre larger Highland Estates parcel as consisting of Franciscan mélange which contains “isolated monument-like blocks of competent rock (mainly graywacke sandstone) projecting out of the brushy slope.” They reportedly encountered serpentinite in three of their borings on Lot 11 but which apparently is mantled at the ground surface by colluvial soils and is not exposed at the ground surface. The bedrock across the development area is generally mantled by colluvium, alluvium, artificial fill and landslides. The landslides were determined to be typically shallow (less than 5 feet thick). Follow-on investigations of Highland Estates were conducted in 2005-06 by TRC Lowney (“Lowney”) and in 2009 by Treadwell and Rollo (“T&R”; see below).

Subject Lots 5 through 8:

The geotechnical report of SFS (1993, 1994) included (within Lots 5 through 8) the test pit logs of 8 test pits excavated and logged in proximity of the subject lots by BLA (1980). They encountered Franciscan mélange, slide debris and fill on the lots. Lowney in 2005 conducted three test borings on the subject lots. They focused their field investigation in areas underlain by Franciscan mélange. In 2009, Treadwell and Rollo (“T&R”) logged three test pits on lots 5 through 8 (TP-1, 2 and 3). The test pits ranged in depth from about 12 and 30 feet beneath the existing ground surface and were excavated to characterize two mapped landslides on these lots. They also compiled all previous consultant’s exploratory excavations on these lots and reviewed a series of aerial photos covering the site. They concluded the landslides could be

mitigated through conventional engineering measures and provided recommendations to achieve that end, as well as standard site development guidance.

Lots 9 through 11:

BLA in 1980 had performed 9 test pits in proximity of Lots 9, 10 and 11 (TP-1, TP-20, TP-27, TP-30, TP-31, TP-32, TP-33, TP-34, and TP-39; and included the field data reported by SFS; 1993). Additionally they presented boring logs from the earlier investigation of SFS (1993). They encountered sheared rock as well as local accumulations of artificial fill previously placed during grading of the adjacent subdivision. As previously mentioned, SFS in 1993 encountered serpentinite within three of their borings on Lot 11. In 2009 T&R compiled all previous consultant's exploratory excavations on these lots and reviewed a series of aerial photos covering the site. They encountered no evidence of landsliding on these lots.

On July 28, 2015 we conducted an exploratory boring within the upper portion of Lot 11. Our boring extended to a depth of 15 feet where it was met with practical sampling refusal. We encountered up to 6 feet of undocumented fill overlying colluvium and Franciscan sheared rock. The bedrock consisted of interbedded shale and sandstone. We did not encounter any groundwater. The fill appears to be an accumulation of surplus fill placed as part of the grading for Cowpens Way.

Current Site Reconnaissance:

A reconnaissance of the site and immediate vicinity was performed by our Certified Engineering Geologist on July 28, 2015, for the purpose of observing and recording any changes apparent across the site that might have occurred since the most recent site investigation of 2009. We noted no appreciable changes to the site conditions since the most recent investigations. We noted no evidence of severe erosion or sedimentation at the site, nor did we note any evidence of further slope movements (reference our Site Plan and Geologic Map, Figure 2A to 2C).

3.3.1 Plasticity/Expansion Potential

We performed one Plasticity Index (PI) tests on a representative sample from our boring performed at Lot 11. This test result along with PI tests and boring log and test pit logs from previous investigations were used to evaluate the expansion potential of the onsite materials. The result of our PI test indicated a PI of 22 while PI tests performed by others indicated PIs of 6 to 13. Based on the above, soil materials encountered at the lot locations are anticipated to potentially exhibit moderate expansion potential to wetting and drying cycles.

3.4 GROUND WATER

Ground water was not encountered in our current boring within Lot 11 during drilling; however, the boring was not left open but was immediately backfilled when the boring was completed. Previous borings by SFS (B-14, B-16, and B-17) within the general proximity of Lot 11 that extended to a maximum depth of 42 feet encountered groundwater at depths ranging from about 1 to 10 feet below the surface at the time. SFS installed standpipe piezometers and

concluded the ground water was likely runoff from higher up the ridge that percolated through fractures in the bedrock until encountering impermeable serpentinite, which caused the water to surface. Free ground water was not encountered within TRC Lowney's borings within proximity of Lots 5 through 8 that extended to a maximum depth of 20 feet, however they noted observing seepage of ground water along the cut-slope for Ticonderoga Drive. Treadwell & Rollo noted portions of the landslide material within their test pits at Lots 5 through 8 were saturated with perched water above the landslide gouge. They also mentioned no free ground water was observed within the bedrock below the landslide masses. No free ground water was noted within any explorations in the proximity of Lots 9 and 10.

Ground water is not mapped in the area by the State of California, but is anticipated to be generally deep. However, perched ground water may be encountered in fractured bedrock and overlying soils. Fluctuations in ground water levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors.

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT RUPTURE

Although there are significant faults located within 25 kilometers of the site, no active or potentially active faults are mapped transecting the site. The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone (known formerly as a Special Studies Zone) (CDMG, 1982). A regional fault map illustrating known active faults relative to the site is presented in Figure 3. We encountered no evidence suggesting active fault surface traces at the site. This is also consistent with the findings of previous consultants in their studies of the Highland Estates subdivision. It is our conclusion that there is a low potential for the occurrence of fault surface rupture (primary or coseismic) to occur at the subject site.

4.2 ESTIMATED GROUND SHAKING

Moderate to severe (design-level) earthquakes can cause strong ground shaking, which is the case for most sites within the Bay Area. A peak ground acceleration (PGA) of 0.983g, 0.976g, and 0.984g for Lots 5 to 8, Lots 9 and 10, and Lot 11, respectively, was estimated for analysis using $F_{PGA} \times PGA$ (Equation 11.8-1) as allowed in the 2013 California Building Code. Seismic design criteria values are presented in Section 7.2 of this report. This hazard can be mitigated by designing the buildings in accordance with the current building code.

4.3 LIQUEFACTION POTENTIAL

Liquefaction hazard mapping of the site by the California Geologic Survey has not been completed for the site area. Mapping by the Association of Bay Area Governments (ABAG) indicates that the site is located in an area of very low liquefaction potential.

During strong seismic shaking, cyclically induced stresses can cause increased pore pressures within the soil matrix that can result in liquefaction triggering, soil softening due to shear stress loss, potentially significant ground deformation due to settlement within sandy liquefiable layers

as pore pressures dissipate, and/or flow failures in sloping ground or where open faces are present (lateral spreading) (NCEER 1998). Limited field and laboratory data is available regarding ground deformation due to settlement; however, in clean sand layers settlement on the order of 2 to 4 percent of the liquefied layer thickness can occur. Soils most susceptible to liquefaction are loose, non-cohesive soils that are saturated and are bedded with poor drainage, such as sand and silt layers bedded with a cohesive cap. Our analyses indicate that based on the fairly shallow depth to bedrock and ground water depths, the lots have a low potential for liquefaction which is consistent with the mapping in the area by ABAG.

4.4 LATERAL SPREADING

Lateral spreading is horizontal/lateral ground movement of relatively flat-lying soil deposits towards a free face such as an excavation, channel, or open body of water; typically lateral spreading is associated with liquefaction of one or more subsurface layers near the bottom of the exposed slope. As failure tends to propagate as block failures, it is difficult to analyze and estimate where the first tension crack will form.

There are no open faces within a distance considered susceptible to lateral spreading; therefore, in our opinion, the potential for lateral spreading to affect the site is low.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose unsaturated sandy soils can settle during strong seismic shaking. In areas of shallow bedrock extending generally to the ground surface, the potential for differential seismic settlement affecting the proposed improvements is low. In landslide repair areas, materials overlying the bedrock will be reengineered and will also have a low potential for differential seismic settlement. In locations of soil or existing fills above the underlying bedrock that will not be reengineered during landslide repair or lot grading activities, there is a potential for differential seismic settlement to occur within the sandier soils. However, as the proposed structures will be supported by drilled pier foundations founded in the underlying bedrock, differential seismic settlement of these soils and fills should not significantly affect the proposed structures.

4.6 LANDSLIDING

The California Geological Survey (CGS) has been producing Seismic Hazard Zone maps for earthquake induced landsliding, however the San Mateo Quadrangle has not been published as of the time of the current study. The site is located within a hilly area with slopes described by Pampeyan (1994) as "unstable, especially when wet," and where small isolated landslides were mapped nearby by Brabb and Pampeyan (1972) and Leighton (1973). The aerial photographs revealed no geomorphic evidence of recent slope movement. We noted the minor slope failures that were previously mapped along Ticonderoga Drive at the site during the site reconnaissance. The interpretive map (landslide susceptibility) published by Brabb et al. (1978) shows the site within an area designated as moderately susceptible to landsliding based on slopes of greater than 30%, but also includes areas with 15% to 30% that are underlain by unstable rock units. Wieczorek et al. (1985) indicates most of the Highlands Estates site is

located in an area mapped as having moderate susceptibility, and the northwest portion of the subdivision is shown as having very low susceptibility to landsliding triggered by a major earthquake. The subject lots are located on the moderate to steep slopes near the crest of Pulgas Ridge, which is underlain at shallow depths by competent sandstone of the Franciscan Complex. We judge the potential for landsliding to be low in the bedrock material and moderate to high in the mapped landslide deposit areas. The existing shallow slope failures are deemed to be the result of slope over steepening associated with the construction of Ticonderoga Drive.

Based on our surface reconnaissance, research of published and unpublished geologic maps and reports, and our review of aerial photographs, no changes in the landslide configurations were noted at or immediately adjacent to the subject lots. Our findings are consistent with the earlier consultant's investigations of the subject Lots 5 through 11. None of the previous consultants' investigations identified landslides at subject Lots 9 through 11. This is consistent with our current findings as well. As determined by T&R, the cutslope failure (landslide) that spans Lot 5 and Lot 6 is 95 feet wide by 55 feet long and was determined to be 7 feet thick and terminates or "toes out" in the slope above Ticonderoga Drive. The landslide that spans Lot 7 and Lot 8 was 160 feet wide by 105 feet long, extends up to about 26 feet deep, and extends beneath Ticonderoga Drive at a depth of about 6 to 7 feet. Detailed descriptions of the landslides were included in the reports by T&R. In 2009 T&R provided landslide mitigation measures for the two landslides. They indicated that the landslide mass that spans Lot 5 and 6 would be removed during the (then) proposed site grading for the building pads and driveways. They indicated the larger landslide that spans Lot 7 and Lot 8 would not be completely mitigated by the (then) proposed grading and therefore recommended it be provided with a fully drained buttress fill. They concluded that a buttress fill embedded into the underlying Franciscan bedrock would provide sufficient stability for the subject lots and for Ticonderoga Drive. Current plans do not appear to fully remove the landslide mass spanning Lot 5 and 6. To address this concern and to supplement T&R's slope stability analysis and landslide mitigation measures for the landslide spanning Lot 7 and 8, we prepared Landslide Mitigation Plans for both landslides (Figures 10 to 13). We summarize T&R slope stability analysis for the landslide spanning Lot 7 and 8 in the section below.

4.7 TREADWELL & ROLLO SLOPE STABILITY ANALYSIS

As discussed in our proposal, since Treadwell & Rollo performed a detailed slope stability evaluation for a fully drained buttress fill landslide repair for the landslide spanning Lot 7 and 8, an additional detailed slope stability evaluation was not included in our scope of work and has not been performed. Additionally, our licensed geotechnical engineer, Scott Fitinghoff, visually observed the test pits performed by Treadwell & Rollo and conferred with their findings and analysis of the slope. We have summarized Treadwell & Rollo's stability analyses in the following sections and provided their model and outputs from their analyses in Appendix C.

4.7.1 Method of Analysis

The stability of a buttress fill repair for the landslide at Lot 7 and 8 was evaluated along the idealized Geologic Cross-Section C-C' (similar to our current Cross-Section B-B'), which was

determined by Treadwell and Rollo’s engineering geologist to be the most critical slope from a topographic standpoint as well as appropriately modeling the apparent landslide movement observed in their test pits. A simplified two-dimensional model of the landslide and bedrock profile and a typical buttress fill repair consisting of benches and a keyway cut into the Franciscan bedrock below the existing landslide was developed. The keyway extended 3 feet below the bottom of landslide and the keyway and bench widths were at least 10 feet.

Slope/W (version 6.22) by Geo-Slope International, Ltd. (2004) was used for the analyses and two-dimensional limit equilibrium methods (Modified Bishop, Janbu, and Spencer’s Method) were used to compute factors of safety. The program determined the most critical failure surface (lowest factor of safety) with the given parameters. Slopes with a static factor of safety of 1.5 or greater and a pseudo static factor of safety of 1.15 with a horizontal seismic coefficient of 0.10 to 0.15 times gravity (g) was considered to be stable (Seed, 1979).

4.7.2 Soil and Bedrock Engineering Properties

Buttress fill material engineering properties were selected based on results from field investigation, laboratory testing, and engineering judgement. Engineering material properties for the existing fill and colluvium at the top of slope and for the landslide materials below Ticonderoga Drive were selected from published CGS strength parameters from the nearby Mindego Hill Quadrangle. Engineering properties for the Franciscan bedrock below the buttress fill repair were determined from published CGS strength parameters from the City and County of San Francisco. A summary of the soil and bedrock parameters used in the analyses are presented in the table below.

Table 2: Engineering Properties used in Treadwell & Rollo’s Slope Stability Analyses

Material Description	Total Unit Weight (pcf)	Effective Cohesion (psf)	Effective Internal Friction Angle (degrees)
Existing Fill	110	500	26.0
Colluvium	120	700	22.0
Buttress Fill	124	60	32.3
Existing Landslide	110	700	11.0
Franciscan Bedrock	135	800	22.0

4.7.3 Ground Water

Ground water was not observed in Treadwell & Rollo’s test pit. The proposed buttress was assumed fully drained and the influence of ground water was not included in the analyses.

4.7.4 Static Stability Results

The static analysis minimum factor of safety for the overall repaired slope was approximately 2.37, which was greater than the generally accepted minimum static factor of safety of 1.5.

4.7.5 Pseudo-Static Stability Results

For the pseudo-static analysis, an earthquake was represented as an equivalent horizontal static force, which was determined by multiplying the mass of potential slide material by a horizontal ground acceleration. For a magnitude 7.9 Earthquake along the San Andreas Fault, a peak seismic coefficient of 0.844g was determined in accordance with the 2006 International Building Code, which corresponded to a repeatable acceleration of 0.563g used in the analysis. With the above acceleration, the minimum factor of safety was determined to be less than 1.0 for the overall repaired slope. A seismic force of 0.378g was determined to correspond to a factor of safety of 1.0 (yield analysis).

To further evaluate earthquake shaking effects, the method developed by Bray and Travasaro (2007) was used to estimate the seismic deformation of the repaired slope. For the analysis, the minimum yield acceleration for the repaired slide mass was determined to be approximately 0.378g, the spectral acceleration was determined to be 1.175g for the site, and the slope's initial Fundamental Period (Ts) was calculated to be 0.10 seconds, with a degraded period equal to 0.15 seconds. This slope displacement analysis indicated permanent slope displacements on the order of 8 to 9 centimeters during the peak earthquake event.

Treadwell & Rollo concluded that the above deformation amount was relatively small and that slope failure hazards should be adequately mitigated for the lots by a buttress fill bearing in the underlying bedrock. They noted that the yield coefficient is dependent on the material strengths of the buttress fill materials and that lower strength materials than what was tested would likely cause greater slope deformations. We concur with Treadwell & Rollo's analysis.

4.8 SOIL CREEP AND LOCALIZED SLOPE INSTABILITY

A thin layer of colluvium and/or undocumented fill on the order of 1 to 11 feet thick was identified in our exploration and explorations performed by others above the underlying bedrock in the areas of the proposed residences. Due to the existing slopes within the lot locations ranging up to 3:1 to 2:1 (H:V), the upper few feet of the soil may be susceptible to creep and localized slope instability and should be expected. As a result, structures and retaining walls should be supported on drilled pier foundations designed to resist creep forces.

4.9 FLOODING

Based on our internet search of the Federal Emergency Management Agency (FEMA) flood map public database, the site is located within Zone X, and area of minimal flood hazard. We recommend the project civil engineer be retained to confirm this information.

4.10 NATURALLY OCCURRING ASBESTOS

Chrysotile and amphibole asbestos occur naturally in certain geologic settings in the San Francisco Bay area, most commonly in serpentinite and other ultramafic rocks. These are igneous and metamorphic rocks with a high content of magnesium and iron minerals. The most

common type of asbestos is chrysotile, which is commonly found in serpentinite rock formations. When disturbed by construction, grading, quarrying, or surface mining operations, asbestos-containing dust can be generated. Exposure to asbestos can result in lung cancer, mesothelioma, and asbestosis. In July 2001, the California Air Resources Board approved an Asbestos Airborne Toxic Control measure for Construction, Grading, Quarrying, and Surface Mining activities in areas where naturally occurring asbestos (NOA) will likely be found and to provide best dust mitigation measures and practices. These are mountainous areas or areas of shallow bedrock that could be encountered during construction. Regional mapping suggests, and the site specific investigations supports the idea that the dominant rock type at the site is sheared rock. The sheared rock that underlies the majority of the site is unlikely to contain NOA bearing material. Localized outcrops of serpentinite have been observed in portions of the canyon area and serpentinite was encountered within three previous exploratory borings conducted at the site. While we did not observe veins of asbestos or bearing minerals, it is not known if rock masses beneath the ground surface could contain veins of asbestos bearing material and the previous samples collected within borings conducted within serpentinite were not analyzed for NOA. We did however obtain a bulk sample of soil and bedrock from our Boring EB-1 at Lot 11 (at a depth range of 8.5 feet to 15 feet depth) which was subsequently analyzed for NOA. The results indicate no NOA detected. Results are shown in Appendix C. However due to the presence of serpentinite locally at the site, we recommend that random samples be collected during grading operations to test for asbestos if serpentinite is observed.

SECTION 5: CONCLUSIONS

5.1 SUMMARY

From a geotechnical viewpoint, the project is feasible provided the concerns listed below are addressed in the project design. Descriptions of each concern with brief outlines of our recommendations follow the listed concerns.

- Potential for landsliding within Lots 5 through 8
- Potential for soil creep and localized slope instability
- Presence of existing undocumented fills
- Presence of moderately expansive soils
- Differential movement at on-grade to on-structure transitions

5.1.1 Potential for Landsliding within Lots 5 through 8

As mentioned above and documented and analyzed by previous investigations and our firm, two landslides are located within the area of proposed Lots 5 through 8. To supplement prior findings and recommendations, we have provided landslide mitigation plans and details on Figures 10 to 13 for mitigating the identified landslides. In addition to restabilizing the landslide areas, to protect the structures and retaining walls from future slope instability (discussed below) at Lots 5 through 11, proposed structures and retaining walls should be supported on drilled piers. Detailed recommendations for the design of drilled pier foundations are presented in the “Foundations” section of this report.

5.1.2 Potential for Soil Creep and Localized Slope Instability

Outside of the landslide areas identified within Lots 5 through 8, our exploration and explorations by others indicate that a thin layer of colluvium and/or undocumented fill is present above the underlying bedrock in the areas of the proposed residences. This colluvium and/or undocumented fill was identified to be on the order of 1 to 11 feet thick. As existing slopes within the lot locations range up to 3:1 to 2:1 (H:V), we judge the upper few feet of the soil to be susceptible to creep. To address this concern, we recommend that the proposed structures, including site retaining walls be supported on drilled piers designed to resist creep forces. Detailed recommendations for the design of drilled pier foundations are presented in the “Foundations” section of this report.

Another geotechnical concern associated with the presence of colluvium is that concentrated water could cause erosion and localized slope instability. To mitigate this condition and satisfy current storm water requirements, we recommend that the storm water be directed to a concrete lined bio-retention basin. Once the water passes through the bio-retention basin, it should be collected in a solid drainage pipe and conveyed to a dissipater/spreader outlet structure which will spread out the flow across the slope without concentrating the water. Detailed recommendations for the design of the dissipater/spreader structure are presented in the “Earthwork” section of this report.

5.1.3 Presence of Existing Undocumented Fills

Undocumented fill was mapped at the lot locations as shown on the Site Plan and Geologic Map, Figures 2A to 2C. If this fill is left in place during driveway and slab-on-grade grading, it should be removed and replaced as properly compacted engineered fill. Detailed recommendations are presented in the “Earthwork”.

5.1.4 Presence of Moderately Expansive Soils

Moderately expansive soils are present at the various lot locations and may be located within the upper portions of the soil profiles following site grading activities. Expansive soils can undergo significant volume change with changes in moisture content. They shrink and harden when dried and expand and soften when wetted. To reduce the potential for damage to the planned structures, slabs-on-grade should have sufficient reinforcement and be supported on a layer of non-expansive fill; footings should extend below the zone of seasonal moisture fluctuation. In addition, it is important to limit moisture changes in the surficial soils by using positive drainage away from buildings as well as limiting landscaping watering. Detailed grading and foundation recommendations addressing this concern are presented in the following sections.

5.1.5 Differential Movement At On-grade to On-Structure Transitions

The proposed structures will be supported by drilled pier foundations and exterior grades and improvements will be supported on-grade. Some of the surficial improvements will transition

from on-grade support to overlying the drilled pier supported structures. Also, some of the surficial improvements will extend above areas of retaining wall backfill for garages and lower levels of the structures. As a result, differential movement will potentially occur between exterior improvements and structures. Concrete flatwork at entrances should be structurally tied to the structure, creating hinged connections, to allow access and limit trip hazards. Additionally, we recommend consideration be given to including subslabs beneath flatwork or pavers that cantilever at least 3 feet beyond retaining walls. If surface improvements are included that are highly sensitive to differential movement, additional measures may be necessary. We also recommend that retaining wall backfill be compacted to 95 percent where surface improvements are planned.

5.2 PLANS AND SPECIFICATIONS REVIEW

We recommend that we be retained to review the geotechnical aspects of the project structural, civil, and landscape plans and specifications, allowing sufficient time to provide the design team with any comments prior to issuing the plans for construction.

5.3 CONSTRUCTION OBSERVATION AND TESTING

As site conditions may vary significantly between the small-diameter borings performed during this investigation, we also recommend that a Cornerstone representative be present to provide geotechnical observation and testing during earthwork and foundation construction. This will allow us to form an opinion and prepare a letter at the end of construction regarding contractor compliance with project plans and specifications, and with the recommendations in our report. We will also be allowed to evaluate any conditions differing from those encountered during our investigation, and provide supplemental recommendations as necessary. For these reasons, the recommendations in this report are contingent of Cornerstone providing observation and testing during construction. Contractors should provide at least a 48-hour notice when scheduling our field personnel.

SECTION 6: EARTHWORK

6.1 SITE DEMOLITION, CLEARING AND PREPARATION

6.1.1 Site Stripping

The lot locations should be stripped of all surface vegetation, and surface and subsurface improvements within the proposed development area. Demolition of existing improvements is discussed in detail below. Surface vegetation and topsoil should be stripped to a sufficient depth to remove all material greater than 3 percent organic content by weight. Based on our site observations, surficial stripping should extend about 3 to 12 inches below existing grade.

6.1.2 Tree and Shrub Removal

Trees and shrubs designated for removal should have the root balls and any roots greater than ½-inch diameter removed completely. Mature trees are estimated to have root balls extending

to depths of 2 to 4 feet, depending on the tree size. Significant root zones are anticipated to extend to the diameter of the tree canopy. Grade depressions resulting from root ball removal should be cleaned of loose material and backfilled in accordance with the recommendations in the “Compaction” section of this report.

6.1.3 Abandonment of Existing Utilities

No utility lines are known to exist within the proposed lots. However, if encountered, all utilities should be completely removed from within planned building areas. For any utility line to be considered acceptable to remain within building areas, the utility line must be completely backfilled with grout or sand-cement slurry (sand slurry is not acceptable), the ends outside the building area capped with concrete, and the trench fills either removed and replaced as engineered fill with the trench side slopes flattened to at least 1:1, or the trench fills are determined not to be a risk to the structure. The assessment of the level of risk posed by the particular utility line will determine whether the utility may be abandoned in place or needs to be completely removed. The contractor should assume that all utilities will be removed from within building areas unless provided written confirmation from both the owner and the geotechnical engineer.

Utilities extending beyond the building area may be abandoned in place provided the ends are plugged with concrete, they do not conflict with planned improvements, and that the trench fills do not pose significant risk to the planned surface improvements.

The risks associated with abandoning utilities in place include the potential for future differential settlement of existing trench fills, and/or partial collapse and potential ground loss into utility lines that are not completely filled with grout. In general, the risk is relatively low for single utility lines less than 4 inches in diameter, and increases with increasing pipe diameter.

6.2 REMOVAL OF EXISTING FILLS

All existing fills should be completely removed from within proposed garage slabs-on-grade, interior slabs-on-grade, and driveway areas and to a lateral distance of at least 2 feet beyond the edge of the improvements or to a lateral distance equal to fill depth below the slab or driveway, whichever is greater. The approximate limits of undocumented fill are shown on Figures 2A to 2C. Existing fills within the location of improvements for Lots 5 to 8 will be removed during site grading operations and landslide repair. The approximate limits of existing fill removal and a corresponding typical keying and benching plan for Lots 9 and 10 are shown on Figures 14 and 15. Typical keying and benching recommendations are provided in Section 6.9. Existing fills should be removed from the driveway and any slab-on-grade locations within Lot 11.

Provided the fills meet the “Material for Fill” requirements below, the fills may be reused when backfilling excavations. If materials are encountered that don’t meet the requirements, such as debris, wood, trash, those materials should be screened out of the remaining material and not be reused. Backfill of excavations should be placed in lifts and compacted in accordance with the “Compaction” section below.

6.3 TEMPORARY CUT AND FILL SLOPES

The contractor is responsible for maintaining all temporary slopes and providing temporary shoring where required. Temporary shoring, bracing, and cuts/fills should be performed in accordance with the strictest government safety standards. On a preliminary basis, the upper 10 feet at the site may be classified as OSHA Soil Type B materials. A Cornerstone representative should be retained to confirm the preliminary site classification.

Excavations performed during site demolition and fill removal should be sloped at 3:1 (horizontal:vertical) within the upper 5 feet below building subgrade. Excavations extending more than 5 feet below building subgrade and excavations in pavement and flatwork areas should be slope at a 1:1 inclination unless the OSHA soil classification indicates otherwise.

6.4 SUBGRADE PREPARATION

After site clearing and demolition is complete, and prior to backfilling any excavations resulting from fill removal or demolition, the excavation subgrade and subgrade within areas to receive additional site fills, slabs-on-grade and/or pavements should be scarified to a depth of 6 inches, moisture conditioned, and compacted in accordance with the "Compaction" section below.

6.5 SUBGRADE STABILIZATION MEASURES

Soil subgrade and fill materials, especially soils with high fines contents such as clays and silty soils, can become unstable due to high moisture content, whether from high in-situ moisture contents or from winter rains. As the moisture content increases over the laboratory optimum, it becomes more likely the materials will be subject to softening and yielding (pumping) from construction loading or become unworkable during placement and compaction.

There are several methods to address potentially unstable soil conditions and facilitate fill placement and trench backfill. Some of the methods are briefly discussed below. Implementation of the appropriate stabilization measures should be evaluated on a case-by-case basis according to the project construction goals and the particular site conditions.

6.5.1 Scarification and Drying

The subgrade may be scarified to a depth of 6 to 12 inches and allowed to dry to near optimum conditions, if sufficient dry weather is anticipated to allow sufficient drying. More than one round of scarification may be needed to break up the soil clods.

6.5.2 Removal and Replacement

As an alternative to scarification, the contractor may choose to over-excavate the unstable soils and replace them with dry on-site or import materials. A Cornerstone representative should be present to provide recommendations regarding the appropriate depth of over-excavation,

whether a geosynthetic (stabilization fabric or geogrid) is recommended, and what materials are recommended for backfill.

6.6 MATERIAL FOR FILL

6.6.1 Re-Use of On-site Soils

On-site soils with an organic content less than 3 percent by weight may be reused as general fill. General fill should not have lumps, clods or cobble pieces larger than 6 inches in diameter; 85 percent of the fill should be smaller than 2½ inches in diameter. Minor amounts of oversized material (smaller than 12 inches in diameter) may be allowed provided, the oversized pieces are not allowed to nest together, and the compaction method will allow for loosely placed lifts not exceeding 12 inches. It is noted that excavation of piers and retaining wall cut, and grade beams may result in large rock fragments that require special handling and disposal. The contractor should anticipate handling of this material during construction.

6.6.2 Potential Import Sources

Imported and non-expansive material should be inorganic with a Plasticity Index (PI) of 15 or less. To prevent significant caving during trenching or foundation construction, imported material should have sufficient fines. Samples of potential import sources should be delivered to our office at least 10 days prior to the desired import start date. Information regarding the import source should be provided, such as any site geotechnical reports. If the material will be derived from an excavation rather than a stockpile, potholes will likely be required to collect samples from throughout the depth of the planned cut that will be imported. At a minimum, laboratory testing will include PI tests. Material data sheets for select fill materials (Class 2 aggregate base, ¾-inch crushed rock, quarry fines, etc.) listing current laboratory testing data (not older than 6 months from the import date) may be provided for our review without providing a sample. If current data is not available, specification testing will need to be completed prior to approval.

Environmental and soil corrosion characterization should also be considered by the project team prior to acceptance. Suitable environmental laboratory data to the planned import quantity should be provided to the project environmental consultant; additional laboratory testing may be required based on the project environmental consultant's review. The potential import source should also not be more corrosive than the on-site soils, based on pH, saturated resistivity, and soluble sulfate and chloride testing.

6.7 COMPACTION REQUIREMENTS

All fills, and subgrade areas where fill, slabs-on-grade, and pavements are planned, should be placed in loose lifts 8 inches thick or less and compacted in accordance with ASTM D1557 (latest version) requirements as shown in the table below. In general, clayey soils should be compacted with sheepsfoot equipment and sandy/gravelly soils with vibratory equipment; open-graded materials such as crushed rock should be placed in lifts no thicker than 18 inches and consolidated in place with vibratory equipment. Each lift of fill and all subgrade should be firm

and unyielding under construction equipment loading in addition to meeting the compaction requirements to be approved. The contractor (with input from a Cornerstone representative) should evaluate the in-situ moisture conditions, as the use of vibratory equipment on soils with high moistures can cause unstable conditions. General recommendations for soil stabilization are provided in the “Subgrade Stabilization Measures” section of this report. Where the soil’s PI is 20 or greater, the expansive soil criteria should be used.

Table 3: Compaction Requirements

Description	Material Description	Minimum Relative ¹ Compaction (percent)	Moisture ² Content (percent)
General Fill (within upper 5 feet)	On-Site Expansive Soils	87 – 92	>3
	On-Site Low Expansion Soils	90	>1
General Fill (below a depth of 5 feet)	On-Site Expansive Soils	93	>3
	On-Site Low Expansion Soils	95	>1
Basement Wall Backfill	Without Surface Improvements	90	>1
Basement Wall Backfill	With Surface Improvements	93 ⁴	>1
Trench Backfill	On-Site Expansive Soils	87 – 92	>3
Trench Backfill	On-Site Low Expansion Soils	90	>1
Trench Backfill (upper 6 inches of subgrade)	On-Site Low Expansion Soils	95	>1
Crushed Rock Fill	¾-inch Clean Crushed Rock	Consolidate In-Place	NA
Non-Expansive Fill	Imported Non-Expansive Fill	90	Optimum
Flatwork Subgrade	On-Site Expansive Soils	87 - 92	>3
Flatwork Subgrade	On-Site Low Expansion Soils	90	>1
Flatwork Aggregate Base	Class 2 Aggregate Base ³	90	Optimum
Pavement Subgrade	On-Site Expansive Soils	87 - 92	>3
Pavement Subgrade	On-Site Low Expansion Soils	90	>1
Pavement Aggregate Base	Class 2 Aggregate Base ³	95	Optimum
Asphalt Concrete	Asphalt Concrete	95 (Marshall)	NA

1 – Relative compaction based on maximum density determined by ASTM D1557 (latest version)

2 – Moisture content based on optimum moisture content determined by ASTM D1557 (latest version)

3 – Class 2 aggregate base shall conform to Caltrans Standard Specifications, latest edition, except that the relative compaction should be determined by ASTM D1557 (latest version)

4 – Using light-weight compaction or walls should be braced

6.7.1 Construction Moisture Conditioning

Expansive soils can undergo significant volume change when dried then wetted. The contractor should keep all exposed expansive soil subgrade (and also trench excavation side walls) moist until protected by overlying improvements (or trenches are backfilled). If expansive soils are

allowed to dry out significantly, re-moisture conditioning may require several days of re-wetting (flooding is not recommended), or deep scarification, moisture conditioning, and re-compaction.

6.8 TRENCH BACKFILL

Utility lines constructed within public right-of-way should be trenched, bedded and shaded, and backfilled in accordance with the local or governing jurisdictional requirements. Utility lines in private improvement areas should be constructed in accordance with the following requirements unless superseded by other governing requirements.

All utility lines should be bedded and shaded to at least 6 inches over the top of the lines with crushed rock ($\frac{3}{8}$ -inch-diameter or greater) or well-graded sand and gravel materials conforming to the pipe manufacturer's requirements. Open-graded shading materials should be consolidated in place with vibratory equipment and well-graded materials should be compacted to at least 90 percent relative compaction with vibratory equipment prior to placing subsequent backfill materials.

General backfill over shading materials may consist of on-site native materials provided they meet the requirements in the "Material for Fill" section, and are moisture conditioned and compacted in accordance with the requirements in the "Compaction" section.

On hillside sites it is desirable to reduce the potential for water migration into building areas through the granular shading materials. We recommend that a plug of low-permeability clay soil, sand-cement slurry, or lean concrete be placed within trenches just outside where the trenches pass into building areas.

6.8.1 Flexible Utility Connections

The new structures will be supported on pier and grade beam systems. As some utilities will extend from on-grade support to the pier and grade beam supported structures, due to the presence of moderately expansive soils that will expand/heave and contract and the potential for some soil creep due to the sloping grades at the lot locations, consideration should be given to including flexible utility connections that will accommodate 1 to 3 inches of ground movement relative to the buildings.

6.9 PERMANENT CUT AND FILL SLOPES

All permanent cut slopes in soil should have a maximum inclination of 2:1 (horizontal:vertical) for slopes up to 10 feet high; slopes greater than 10 feet should be inclined at no greater than 2.5:1. Un-retained fill slopes constructed on existing slopes steeper than 4:1 should not be allowed on this project unless our office is contacted for review of the proposed slope. Fill slopes constructed on natural slopes 4:1 or flatter should have a maximum inclination of 2:1. Refer to the "Erosion Control" section of this report for a discussion regarding protection of sloped surfaces.

6.9.1 Keyways and Benches

Fill placed on existing ground inclined at 6:1 or greater should be benched into the existing slope and a keyway constructed at the toe of the fill. Benches should be angled slightly into the slope, be spaced vertically at no greater than 4 feet between benches, and be at least 6 feet wide. Depending on the thickness of any existing fill and/or colluvium soil that blankets the bedrock, the benches may need to be widened beyond the minimum width to extend into competent bedrock. The keyway should also be angled slightly into the slope (minimum 2 percent inclination), extend at least 2 feet into suitable bedrock or soil as determined by our staff during construction, and be at least 10 feet wide. Keyway and benching plans and recommendations for the two landslide repair areas of Lots 5 to 8 are shown on Figures 10 to 13. A typical key and benching plan for Lot's 9 and 10 existing fill removal and fill slope placement process is depicted in Figures 14 and 15.

6.9.2 Fill Drainage

A permanent subsurface drainage system consisting of a series of perforated gravity pipes or drainage strips should be constructed between engineered fill placed against a bedrock slope and within all keyways. This system is intended to intercept perched water flowing through the bedrock and transmit it to suitable outlet structures and reduce the potential for hydrostatic pressures building up behind the fills, and causing slope instability. The drain lines should be placed at the back of the keyways and benches. Bench drains should be spaced vertically at no greater than 10 feet. For Lots 9 and 10, bench drains are not anticipated based on the soil conditions disclosed by previous investigations. However, field conditions should be observed at the time of construction and bench drains installed if needed.

Drainage systems should be constructed in small trenches or v-ditches, and consist of a minimum 4-inch-diameter perforated SDR 35 (perforations placed downward), bedded and shaded in Caltrans Class 2 Permeable Material (latest version) or ½- to ¾-inch crushed rock; if crushed rock is used, the rock should be encapsulated in filter fabric. The bedding should be at least 2 inches. Alternatively, geocomposite strip drains may be used. All drainage lines should slope towards suitable outlet structures at an inclination of at least 1 percent. Suitable outlet structures may consist of connecting the drainage lines to a storm drain system, with a sump if required; if the drain lines will outlet overland at the toe of the slope, an appropriate rock spill pad should be provided; the drain lines should not outlet onto the slope. Vertical cleanouts should be provided at all upslope ends of the drainage lines and at all 90-degree bends. Drainage material descriptions and additional details are provided on the Figure 13.

6.9.3 Plan Review and Construction Monitoring

We should be retained to review the grading and sub-drainage plans and we can provide more specific input regarding the location of keyways and fill drainage for the final plans. A Cornerstone representative should be on site during grading and foundation construction. Field modifications to the planned construction may be required based on encountered field conditions. In addition, it has been our experience that cut slopes in the Franciscan Formation bedrock are prone to localized weak zones and sloughing along bedding planes. We

recommend that a Cornerstone engineering geologist observe the condition of all cut slopes and evaluate the potential for localized adverse materials or bedding orientation.

We recommend that the project civil engineer or land surveyor be retained to survey in place all keyways, sub-drainage lines, solid pipes, and cleanouts, and create an as-built plan. This plan will be of use for any future maintenance or repair work.

6.10 SITE DRAINAGE

6.10.1 General Surface Drainage

Surface runoff should not be allowed to flow over the top of or pond at the top or toe of engineered slopes or retaining walls. Ponding should also not be allowed on or adjacent to pavements or concrete flatwork. Surface drainage should be directed towards suitable drainage facilities such as lined v-ditches or drain inlets. Lined v-ditches should be included at the top of slopes and intermediate benches, and at the toe of open space adjacent to planned development. All v-ditches and drain inlets should be sized to accommodate the design storm events for the upslope tributary area. Concrete-lined v-ditches should be reinforced as required and have adequate control and construction joints, and should be constructed neat in excavations; backfill around formed ditches should not be allowed.

Upslope sources of water should be evaluated. If upslope irrigation is present or planned, additional surface and subsurface drainage, or construction of subdrains may be needed to protect site improvements. We should be consulted if this issue will affect the project.

6.10.2 Lot Surface Drainage

Ponding should not be allowed adjacent to building foundations, slabs-on-grade, or pavements. Hardscape surfaces should slope at least 1 percent towards suitable discharge facilities; landscape areas should slope at least 2 percent. Roof runoff should be directed away from building areas. Where minimal side yards are planned (10 feet or less), we recommend that area drains collect surface runoff and transmit the runoff to other suitable landscape drainage facilities to prevent ponding adjacent to building foundations. Landscape drainage such as drain inlets and storm water filtration and/or infiltration trenches should be provided to collect and transmit storm water runoff to project storm drains discharge facilities.

Rainfall runoff from the residences will be piped to a dissipation structure below the residences and spread out across the existing hillslope. The proposed layout of the proposed dissipation structures are shown on Figures 2A to 2C, Site Plan and Geologic Map. As discussed previously, a geotechnical concern associated with the presence of undocumented fill and colluvium is that concentrated water could cause erosion and localized slope instability. To mitigate this condition and satisfy current storm water requirements, we recommend that the storm water be directed to a concrete lined bio-retention basin. Once the water passes through the bio-retention basin, it should be collected in a solid drainage pipe and conveyed to dissipater/spreader outlet structure which will spread out the flow across the slope without concentrating the water. The dissipater/spreader should be at least 10 feet wide and discharge

the water uniformly along the hillside. The outfall should be protected by Rip-Rap rock on Mirafi 700x or equal geotextile fabric.

6.11 PERMANENT EROSION CONTROL MEASURES

Hillside grading will require periodic maintenance after construction to reduce the potential for erosion and sloughing. At a minimum all slopes should be vegetated by hydroseeding or other landscape ground cover. The establishment of vegetation will help reduce runoff velocities, allow some infiltration and transpiration, trap sediment within runoff, and protect the soil from raindrop impact. Depending on the exposed material type and the slope inclination, more aggressive erosion control measures may be needed to protect slopes for one or more winter seasons while vegetation is establishing. For slopes with inclinations of 2:1 (horizontal:vertical) or greater, erosion control may consist of jute netting, straw matting, or erosion control blankets used in combination with hydroseeding.

Both construction and post-construction Storm Water Pollution Prevention Plans (SWPPPs) should be prepared for the project-specific requirements. We recommend that final grading plans be provided for our review.

6.12 CRAWL SPACE SEEPAGE MITIGATION

For structures with raised floor foundation systems, crawl spaces are typically lower than adjacent exterior grades and grade beams are generally poured neat in shallow trenches or constructed at-grade. For this type of foundation system, in our opinion, water accumulation in the crawl space is possible even if adequate surface drainage is provided adjacent to the structure. Although water seepage into the crawl space does not generally affect the foundation from a geotechnical viewpoint, it may have undesirable impacts to the floor system.

To mitigate water seepage into crawl space areas, we recommend either minimizing water from getting into the crawl space, or collecting and discharging the water if it does migrate beneath the house. Listed below are several methods for mitigating crawl space seepage, in order of decreasing effectiveness, in our opinion.

1. Grade crawl spaces to drain to common low points; install area drains or sump pumps at low points to collect and discharge water.
2. Construct a series of shallow drainage channels (4 to 6 inches deep and 6 to 12 inches wide) around the perimeter of the crawl space. These channels should also drain toward a common low point; install area drains or sump pumps at low points to collect and discharge water.
3. Install adequate crawl space ventilation to help drying of wet or moist soil.

Due to the complex geologic conditions and unpredictable landscape watering patterns, some minor seepage may still occur, especially if exterior grades are adversely modified by homeowners. Therefore, if desired to further reduce the risk of crawl seepage, Items 2 or 3 may

be used in conjunction with Item 1. We recommend that we review the finished grading and landscaping plans to check for conformance with the above recommendations.

SECTION 7: FOUNDATIONS

7.1 SUMMARY OF RECOMMENDATIONS

In our opinion, the proposed structures may be supported on drilled pier foundations provided the recommendations in the “Earthwork” section and the sections below are followed.

7.2 SEISMIC DESIGN CRITERIA

The project structural design should be based on the 2013 California Building Code (CBC), which provides criteria for the seismic design of buildings in Chapter 16. The “Seismic Coefficients” used to design buildings are established based on a series of tables and figures addressing different site factors, including the soil profile in the upper 100 feet below grade and mapped spectral acceleration parameters based on distance to the controlling seismic source/fault system. Based on previous test pits and borings performed by others, our boring, and review of local geology, the various lot locations are underlain by shallow bedrock and/or soils with an anticipated average SPT “N” value within the upper 100 feet of the surface above 50 blows per foot. Therefore, we have classified the lot locations as Soil Classification C. The mapped spectral acceleration parameters S_S and S_1 were calculated using the USGS computer program *Design Maps*, located at <http://geohazards.usgs.gov/designmaps/us/application.php>, based on the site coordinates presented below and the site classification. The tables below lists the various factors used to determine the seismic coefficients and other parameters for the various lot locations.

Table 4a: 2013 CBC Site Categorization and Site Coefficients for Lots 5 through 8

Classification/Coefficient	Design Value
Site Class	C
Site Latitude	37.51551°
Site Longitude	-122.33826°
0.2-second Period Mapped Spectral Acceleration ¹ , S _s	2.561g
1-second Period Mapped Spectral Acceleration ¹ , S ₁	1.231g
Short-Period Site Coefficient – Fa	1.0
Long-Period Site Coefficient – Fv	1.3
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S _{MS}	2.561g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S _{M1}	1.600g
0.2-second Period, Design Earthquake Spectral Response Acceleration – S _{DS}	1.708g
1-second Period, Design Earthquake Spectral Response Acceleration – S _{D1}	1.066g
Mapped MCE Geometric Mean Peak Ground Acceleration - PGA	0.983g
Site Coefficient Based on PGA and Site Class - F _{PGA}	1.0

¹For Site Class B, 5 percent damped.

Table 4b: 2013 CBC Site Categorization and Site Coefficients for Lots 9 and 10

Classification/Coefficient	Design Value
Site Class	C
Site Latitude	37.51662°
Site Longitude	-122.33734°
0.2-second Period Mapped Spectral Acceleration ¹ , S _s	2.543g
1-second Period Mapped Spectral Acceleration ¹ , S ₁	1.222g
Short-Period Site Coefficient – Fa	1.0
Long-Period Site Coefficient – Fv	1.3
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S _{MS}	2.543g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S _{M1}	1.588g
0.2-second Period, Design Earthquake Spectral Response Acceleration – S _{DS}	1.695g
1-second Period, Design Earthquake Spectral Response Acceleration – S _{D1}	1.059g
Mapped MCE Geometric Mean Peak Ground Acceleration - PGA	0.976g
Site Coefficient Based on PGA and Site Class - F _{PGA}	1.0

¹For Site Class B, 5 percent damped.

Table 4c: 2013 CBC Site Categorization and Site Coefficients for Lot 11

Classification/Coefficient	Design Value
Site Class	C
Site Latitude	37.51683°
Site Longitude	-122.33938°
0.2-second Period Mapped Spectral Acceleration ¹ , S_s	2.563g
1-second Period Mapped Spectral Acceleration ¹ , S_1	1.231g
Short-Period Site Coefficient – F_a	1.0
Long-Period Site Coefficient – F_v	1.3
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S_{MS}	2.563g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	1.601g
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.709g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	1.067g
Mapped MCE Geometric Mean Peak Ground Acceleration - PGA	0.984g
Site Coefficient Based on PGA and Site Class - F_{PGA}	1.0

¹For Site Class B, 5 percent damped.

7.3 DEEP FOUNDATIONS

Because the residential structures will be located on the existing sloping ground, we recommend all buildings and retaining walls be founded on drilled piers and designed with the parameters recommended below.

7.3.1 Drilled Piers Lots 5 to 8

The proposed structural loads may be supported on drilled, cast-in-place, straight-shaft friction piers with minimum diameters of 16 inches. In pier locations where the existing hillside will be reworked as part of the landslide repair process, the piers should extend to a minimum depth of at least 10 feet below the adjacent grade and at least 5 feet below bottom of the re-compacted fill for the landslide area into undisturbed soil or bedrock. Based on our review of cross-sections in the landslide areas, we estimate the minimum depth these piers will be is on the order of 11 feet for Lot 5 and 27 feet for Lot 8. In pier locations where the existing hillside material will likely not be reworked as part of the landslide repair process (generally Lots 6 and 7), the piers should extend to a depth of at least 10 feet below adjacent grade or at least 5 feet into bedrock, whichever is greater. Adjacent pier centers should be spaced at least three diameters apart, otherwise, a reduction for group effects may be required. Grade beams should span between piers and/or pier caps in accordance with structural requirements. Conventional slabs-on-grade for the garages may be used provided the subgrade soils are restrained laterally with retaining

walls of grade beams and subgrade is prepared in accordance with the “Earthwork” section of this report.

In pier locations for Lots 5 to 8, the vertical capacity of the piers may be designed based on an allowable skin friction of 500 psf for combined dead plus live loads based on a factor of safety of 2.0; dead loads should not exceed two-thirds of the allowable capacities. The upper 24 inches of soil should be neglected. The allowable skin friction may be increased by one-third for wind and seismic loads. Frictional resistance to uplift loads may be developed along the pier shafts based on an ultimate frictional resistance of 400 psf; the structural engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate uplift capacity.

Total settlement of individual piers or pier groups of four or less should not exceed $\frac{2}{3}$ -inch to mobilize static capacities and post-construction differential settlement over a horizontal distance of 30 feet should not exceed $\frac{1}{4}$ -inch due to static loads.

7.3.2 Drilled Piers Lots 9 to 11

The proposed structural loads may be supported on drilled, cast-in-place, straight-shaft friction piers with minimum diameters of 16 inches. The piers should extend to a depth of at least 10 feet below adjacent grade or at least 5 feet into bedrock, whichever is greater. Adjacent pier centers should be spaced at least three diameters apart, otherwise, a reduction for group effects may be required. Grade beams should span between piers and/or pier caps in accordance with structural requirements. Conventional slabs-on-grade for the garages may be used provided the subgrade soils are restrained laterally with retaining walls of grade beams and subgrade is prepared in accordance with the “Earthwork” section of this report.

In pier locations for Lots 9 to 11, the vertical capacity of the piers may be designed based on an allowable skin friction of 500 psf for combined dead plus live loads based on a factor of safety of 2.0; dead loads should not exceed two-thirds of the allowable capacities. The upper 24 inches of soil should be neglected. The allowable skin friction may be increased by one-third for wind and seismic loads. Frictional resistance to uplift loads may be developed along the pier shafts based on an ultimate frictional resistance of 400 psf; the structural engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate uplift capacity.

Total settlement of individual piers or pier groups of four or less should not exceed $\frac{2}{3}$ -inch to mobilize static capacities and post-construction differential settlement over a horizontal distance of 30 feet should not exceed $\frac{1}{4}$ -inch due to static loads.

7.3.3 Lateral Capacity

Lateral loads exerted on the piers may be resisted by a passive resistance based on an ultimate equivalent fluid pressure of 450 pcf acting against twice the projected area of piers below the pier cap or grade beam within pier groups of two or more and over two pier diameters for single piers. The lateral pressure may increase up to a maximum uniform pressure of 3,000 psf at depth in locations where piers are positioned outside of landslide repair areas. The upper 24 inches of soil should be neglected when determining lateral capacity due to sloping ground

conditions. The piles should also be designed for an equivalent lateral earth pressure of 60 pcf acting over two pier diameters to simulate soil creep on the piers. The structural engineer should apply an appropriate factor of safety to the ultimate passive pressures.

7.3.4 Construction Considerations

The excavation of all drilled shafts should be observed by a Cornerstone representative to confirm the soil profile, verify that the piers extend the minimum depth into suitable materials and that the piers are constructed in accordance with our recommendations and project requirements. The drilled shafts should be straight, dry, and relatively free of loose material before reinforcing steel is installed and concrete is placed. If ground water is encountered and cannot be removed from the excavations prior to concrete placement, drilling slurry or casing may be required to stabilize the shaft and the concrete should be placed using a tremie pipe, keeping the tremie pipe below the surface of the concrete to avoid entrapment of water or drilling slurry in the concrete.

SECTION 8: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

8.1 INTERIOR SLABS-ON-GRADE

As the Plasticity Index (PI) of the surficial soils ranges up to 22, to reduce the potential for slab damage due to soil heave, the any proposed garage and interior slabs-on-grade should be supported on at least 8 inches of non-expansive fill (NEF) consisting of Class 2 aggregate base. The NEF layer should be constructed over subgrade prepared in accordance with the recommendations in the “Earthwork” section of this report. If moisture-sensitive floor coverings are planned, the recommendations in the “Interior Slabs Moisture Protection Considerations” section below may be incorporated in the project design if desired. If significant time elapses between initial subgrade preparation and slab-on-grade (NEF) construction, the subgrade should be proof-rolled to confirm subgrade stability, and if the soil has been allowed to dry out, the subgrade should be re-moisture conditioned to at least 3 percent over the optimum moisture content.

The structural engineer should determine the appropriate slab reinforcement for the loading requirements and considering the expansion potential of the underlying soils. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness.

8.2 INTERIOR SLABS MOISTURE PROTECTION CONSIDERATIONS

The following general guidelines for concrete slab-on-grade construction where floor coverings are planned are presented for the consideration by the developer, design team, and contractor. These guidelines are based on information obtained from a variety of sources, including the American Concrete Institute (ACI) and are intended to reduce the potential for moisture-related problems causing floor covering failures, and may be supplemented as necessary based on project-specific requirements. The application of these guidelines or not will not affect the geotechnical aspects of the slab-on-grade performance.

- Place a minimum 10-mil-thick vapor retarder conforming to ASTM E 1745, Class C requirements or better directly below the concrete slab. The vapor retarder should extend to the slab edges and be sealed at all seams and penetrations in accordance with manufacturer's recommendations and ASTM E 1643 requirements.
- A 4-inch-thick capillary break, consisting of ½- to ¾-inch crushed rock with less than 5 percent passing the No. 200 sieve, should be placed below the vapor retarder and consolidated in place with vibratory equipment. The capillary break rock may be considered as the upper 4 inches of the non-expansive fill previously recommended.
- The concrete water:cement ratio should be 0.45 or less. Mid-range plasticizers may be used to increase concrete workability and facilitate pumping and placement.
- Water should not be added after initial batching unless the slump is less than specified and/or the resulting water:cement ratio will not exceed 0.45.
- Polishing the concrete surface with metal trowels is not recommended.
- Where floor coverings are planned, all concrete surfaces should be properly cured.
- Water vapor emission levels and concrete pH should be determined in accordance with ASTM F1869 and F710 requirements and evaluated against the floor covering manufacturer's requirements prior to installation.

8.3 PEDESTRIAN EXTERIOR CONCRETE FLATWORK

Exterior concrete flatwork subject to pedestrian traffic only should be at least 4 inches thick and supported on at least 6 inches of non-expansive fill (NEF) overlying subgrade prepared in accordance with the "Earthwork" recommendations of this report. In addition, the upper 4 inches of the NEF should also meet Class 2 aggregate base requirements. As an alternative, the Class 2 aggregate base can also be increased to the full depth of NEF as recommended above. Flatwork that will be subject to heavier or frequent vehicular loading should be designed in accordance with the recommendations in the "Vehicular Pavements" section below.

To help reduce the potential for uncontrolled shrinkage cracking, adequate expansion and control joints should be included. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness. Flatwork should be isolated from adjacent foundations or retaining walls except where limited sections of structural slabs are included to help span irregularities in retaining wall backfill at the transitions between at-grade and on-structure flatwork.

SECTION 9: VEHICULAR PAVEMENTS

9.1 ASPHALT CONCRETE

The following asphalt concrete pavement recommendations tabulated below are based on the Procedure 608 of the Caltrans Highway Design Manual, estimated traffic indices for various pavement-loading conditions, and on a design R-value of 5. The design R-value was chosen based on engineering judgment considering the variable surface conditions.

Table 5: Asphalt Concrete Pavement Recommendations, Design R-value = 5

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base* (inches)	Total Pavement Section Thickness (inches)
4.0	2.5	7.5	10.0
4.5	2.5	9.0	11.5
5.0	3.0	10.0	13.0
5.5	3.0	11.5	14.5
6.0	3.5	12.0	15.5
6.5	4.0	12.0	17.0

*Caltrans Class 2 aggregate base; minimum R-value of 78

Frequently, the full asphalt concrete section is not constructed prior to construction traffic loading. This can result in significant loss of asphalt concrete layer life, rutting, or other pavement failures. To improve the pavement life and reduce the potential for pavement distress through construction, we recommend the full design asphalt concrete section be constructed prior to construction traffic loading. Alternatively, a higher traffic index may be chosen for the areas where construction traffic will be using the pavements.

Asphalt concrete pavements constructed on expansive subgrade where the adjacent areas will not be irrigated for several months after the pavements are constructed may experience longitudinal cracking parallel to the pavement edge. These cracks typically form within a few feet of the pavement edge and are due to seasonal wetting and drying of the adjacent soil. The cracking may also occur during construction where the adjacent grade is allowed to significantly dry during the summer, pulling moisture out of the pavement subgrade. Any cracks that form should be sealed with bituminous sealant prior to the start of winter rains. One alternative to reduce the potential for this type of cracking is to install a moisture barrier at least 24 inches deep behind the pavement curb.

9.2 PORTLAND CEMENT CONCRETE

The exterior Portland Cement Concrete (PCC) pavement recommendations tabulated below are based on methods presented in the Portland Cement Association (PCA) design manual (PCA, 1984). Recommendations for garage slabs-on-grade were provided in the "Concrete Slabs and

Pedestrian Pavements” section above. We have provided a few pavement alternatives as an anticipated Average Daily Truck Traffic (ADTT) was not provided. An allowable ADTT should be chosen that is greater than what is expected for the development.

Table 6: PCC Pavement Recommendations, Design R-value = 5

Allowable ADTT	Minimum PCC Thickness (inches)
0.8	5.0
13	5.5
130	6.0

The PCC thicknesses above are based on a concrete compressive strength of at least 3,500 psi, supporting the PCC on at least 6 inches of Class 2 aggregate base compacted as recommended in the “Earthwork” section, and laterally restraining the PCC with curbs or concrete shoulders. Adequate expansion and control joints should be included. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness. Due to the expansive soils present, we recommend that the construction and expansion joints be dowelled.

9.3 PAVEMENT CUTOFF

Surface water penetration into the pavement section can significantly reduce the pavement life, due to the expansive clays. While quantifying the life reduction is difficult, a normal 20-year pavement design could be reduced to less than 10 years; therefore, increased long-term maintenance may be required.

It would be beneficial to include a pavement cut-off, such as deepened curbs, redwood-headers, or “Deep-Root Moisture Barriers” that are keyed at least 4 inches into the pavement subgrade. This will help limit the additional long-term maintenance.

SECTION 10: RETAINING WALLS

10.1 STATIC LATERAL EARTH PRESSURES

The structural design of any site retaining wall should include resistance to lateral earth pressures that develop from the soil behind the wall, any undrained water pressure, and surcharge loads acting behind the wall. Provided a drainage system is constructed behind the wall to prevent the build-up of hydrostatic pressures as discussed in the section below, we recommend that the walls be designed for the following pressures:

Table 7: Recommended Lateral Earth Pressures

Sloping Backfill Inclination (horizontal:vertical)	Lateral Earth Pressure*	
	Unrestrained – Cantilever Wall	Restrained – Braced Wall
Level	45 pcf	45 pcf + 8H
3:1	55 pcf	55 pcf + 8H
2½:1	60 pcf	60 pcf + 8H
2:1	65 pcf	65 pcf + 8H
Additional Surcharge Loads	1/3 of vertical loads at top of wall	1/2 of vertical loads at top of wall

* Lateral earth pressures are based on an equivalent fluid pressure

** H is the distance in feet between the bottom of footing and top of retained soil

In our opinion, garage and basement walls should be designed as restrained walls. If adequate drainage cannot be provided behind the wall, an additional equivalent fluid pressure of 40 pcf should be added to the values above for both restrained and unrestrained walls for the portion of the wall that will not have drainage. Damp proofing or waterproofing of the walls may be considered where moisture penetration and/or efflorescence are not desired.

10.2 SEISMIC LATERAL EARTH PRESSURES

The 2013 CBC states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. We reviewed the seismic earth pressures for the proposed basement walls using procedures generally based on the Mononobe-Okabe method. Because the walls will likely be in the 10 to 12 feet in height, and peak ground accelerations are greater than 0.40g, we checked the result of the seismic increment when added to the recommended active earth pressure against the recommended fixed wall earth pressures. Because the basement walls are restrained, or will act as restrained walls, and will be designed for 45 pcf (equivalent fluid pressure) plus a uniform earth pressure of 8H psf, based on current recommendations for seismic earth pressures (Lew et al., SEAOC 2010), it appears that active earth pressures plus a seismic increment do not exceed the fixed wall earth pressures. Therefore, in our opinion, an additional seismic increment above the design earth pressures is not required as long as the basement walls are designed for the restrained wall earth pressures recommended above.

We also checked the result of the seismic increment for cantilevered (unrestrained) walls. The seismic increment again does not exceed the unrestrained wall earth pressures. Therefore, in our opinion, an additional seismic increment above the design earth pressures is not required as long as the cantilever walls are designed for the unrestrained wall earth pressures recommended above.

10.3 WALL DRAINAGE

Adequate drainage should be provided by a subdrain system behind all walls. This system should consist of a 4-inch minimum diameter perforated pipe placed near the base of the wall

(perforations placed downward). For walls adjacent to habitable living areas, we recommend that the wall subdrain be placed at least 12 inches below the bottom of the adjacent interior floor slab. The pipe should be bedded and backfilled with Class 2 Permeable Material per Caltrans Standard Specifications, latest edition. The permeable backfill should extend at least 12 inches out from the wall and to within 2 feet of outside finished grade. Alternatively, ½-inch to ¾-inch crushed rock may be used in place of the Class 2 Permeable Material provided the crushed rock and pipe are enclosed in filter fabric, such as Mirafi 140N or approved equivalent. The upper 2 feet of wall backfill should consist of compacted on-site soil. The subdrain outlet should be connected to a free-draining outlet or sump.

Miradrain, Geotech Drainage Panels, or equivalent drainage matting can be used for wall drainage as an alternative to the Class 2 Permeable Material or drain rock backfill. Horizontal strip drains connecting to the vertical drainage matting may be used in lieu of the perforated pipe and crushed rock section. The vertical drainage panel should be connected to the perforated pipe or horizontal drainage strip at the base of the wall, or to some other closed or through-wall system such as the TotalDrain system from AmerDrain. Sections of horizontal drainage strips should be connected with either the manufacturer's connector pieces or by pulling back the filter fabric, overlapping the panel dimples, and replacing the filter fabric over the connection. At corners, a corner guard, corner connection insert, or a section of crushed rock covered with filter fabric must be used to maintain the drainage path.

Drainage panels should terminate 18 to 24 inches from final exterior grade. The Miradrain panel filter fabric should be extended over the top of and behind the panel to protect it from intrusion of the adjacent soil.

10.4 BACKFILL

Where surface improvements will be located over the retaining wall backfill, backfill placed behind the walls should be compacted to at least 93 percent relative compaction using light compaction equipment. Where no surface improvements are planned, backfill should be compacted to at least 90 percent. If heavy compaction equipment is used, the walls should be temporarily braced. Based on the current plans, we understand that v-ditches are planned behind the retaining walls, which we highly recommend.

10.5 FOUNDATIONS

Retaining walls may be supported on drilled piers designed in accordance with the recommendations presented in the "Foundations" section of this report.

SECTION 11: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of Ticonderoga Partners, LLC specifically to support the design of the Highland Estates Lots 5 through 11 project in San Mateo, California. The opinions, conclusions, and recommendations presented in this report have been formulated in accordance with accepted geotechnical

engineering practices that exist in Northern California at the time this report was prepared. No warranty, expressed or implied, is made or should be inferred.

Recommendations in this report are based upon the soil and ground water conditions encountered during our subsurface exploration and information provided in previous investigations by others at the proposed lot locations. If variations or unsuitable conditions are encountered during construction, Cornerstone must be contacted to provide supplemental recommendations, as needed.

Ticonderoga Partners, LLC may have provided Cornerstone with plans, reports and other documents prepared by others. Ticonderoga Partners, LLC understands that Cornerstone reviewed and relied on the information presented in these documents and cannot be responsible for their accuracy.

Cornerstone prepared this report with the understanding that it is the responsibility of the owner or his representatives to see that the recommendations contained in this report are presented to other members of the design team and incorporated into the project plans and specifications, and that appropriate actions are taken to implement the geotechnical recommendations during construction.

Conclusions and recommendations presented in this report are valid as of the present time for the development as currently planned. Changes in the condition of the property or adjacent properties may occur with the passage of time, whether by natural processes or the acts of other persons. In addition, changes in applicable or appropriate standards may occur through legislation or the broadening of knowledge. Therefore, the conclusions and recommendations presented in this report may be invalidated, wholly or in part, by changes beyond Cornerstone's control. This report should be reviewed by Cornerstone after a period of three (3) years has elapsed from the date of this report. In addition, if the current project design is changed, then Cornerstone must review the proposed changes and provide supplemental recommendations, as needed.

An electronic transmission of this report may also have been issued. While Cornerstone has taken precautions to produce a complete and secure electronic transmission, please check the electronic transmission against the hard copy version for conformity.

Recommendations provided in this report are based on the assumption that Cornerstone will be retained to provide observation and testing services during construction to confirm that conditions are similar to that assumed for design, and to form an opinion as to whether the work has been performed in accordance with the project plans and specifications. If we are not retained for these services, Cornerstone cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of Cornerstone's report by others. Furthermore, Cornerstone will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services.

SECTION 12: REFERENCES

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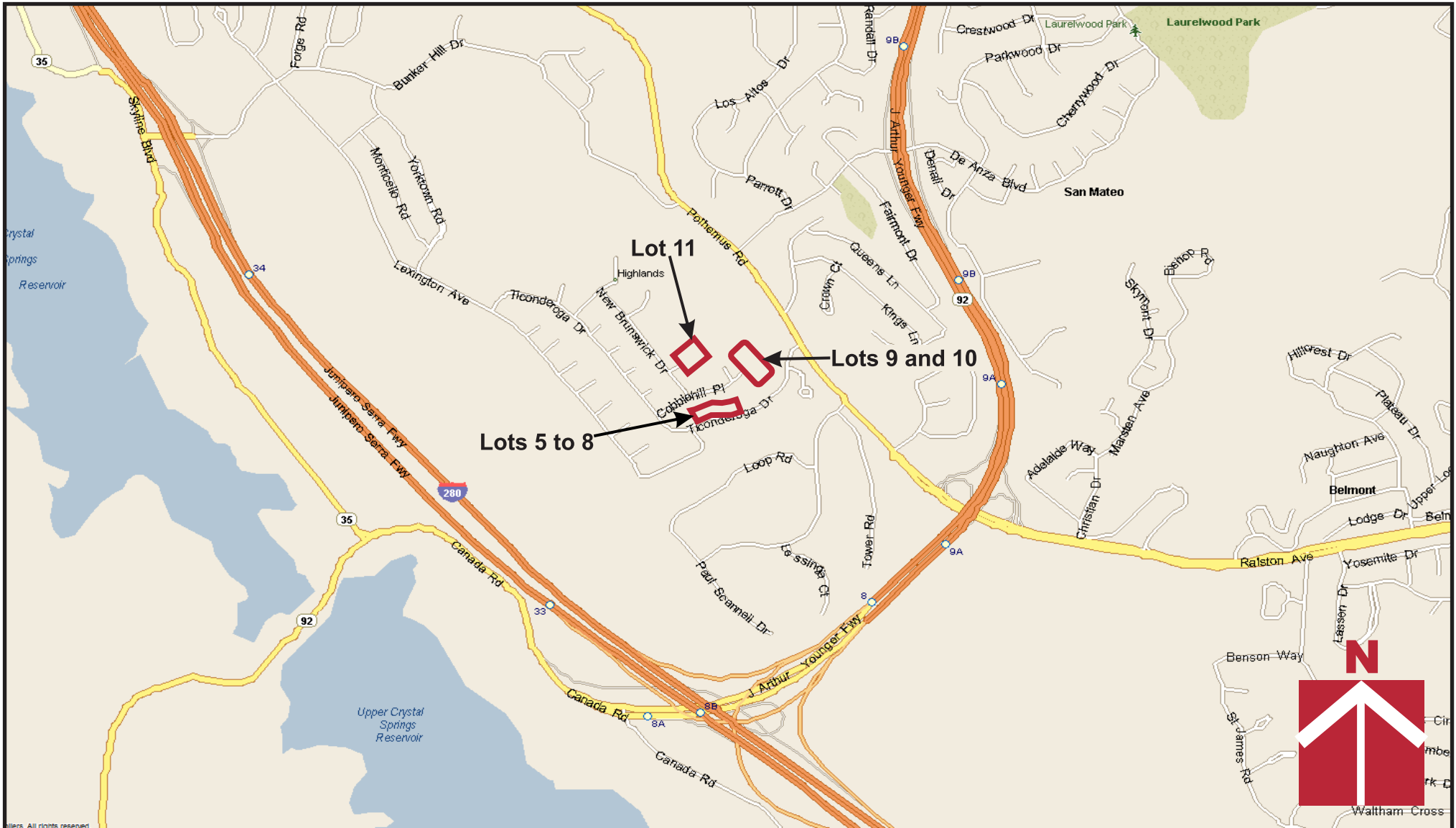
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Aerial Photographs

Geomorphic features on the following aerial photographs were interpreted at the U.S. Geological Survey in Menlo Park as part of this investigation:

<u>Date</u>	<u>Flight</u>	<u>Frames</u>	<u>Scale</u>	<u>Type</u>
October 11, 1943	DDB	2B-111, 112, 56, 57	1:20,000	Black & White
July 29, 1946	GS-CP	2-136, 137	1:20,000	Black & White
May 27, 1956	DDB	1R-89, 90	1:20,000	Black & White
April 18, 1968	GS-VBZJ	1-204, 205	1:30,000	Black & White
May 8, 1973	3567	3-117, 118, 119	1:12,000	Black & White
June 25, 1974	Area 9	9-20	1:20,000	Natural color

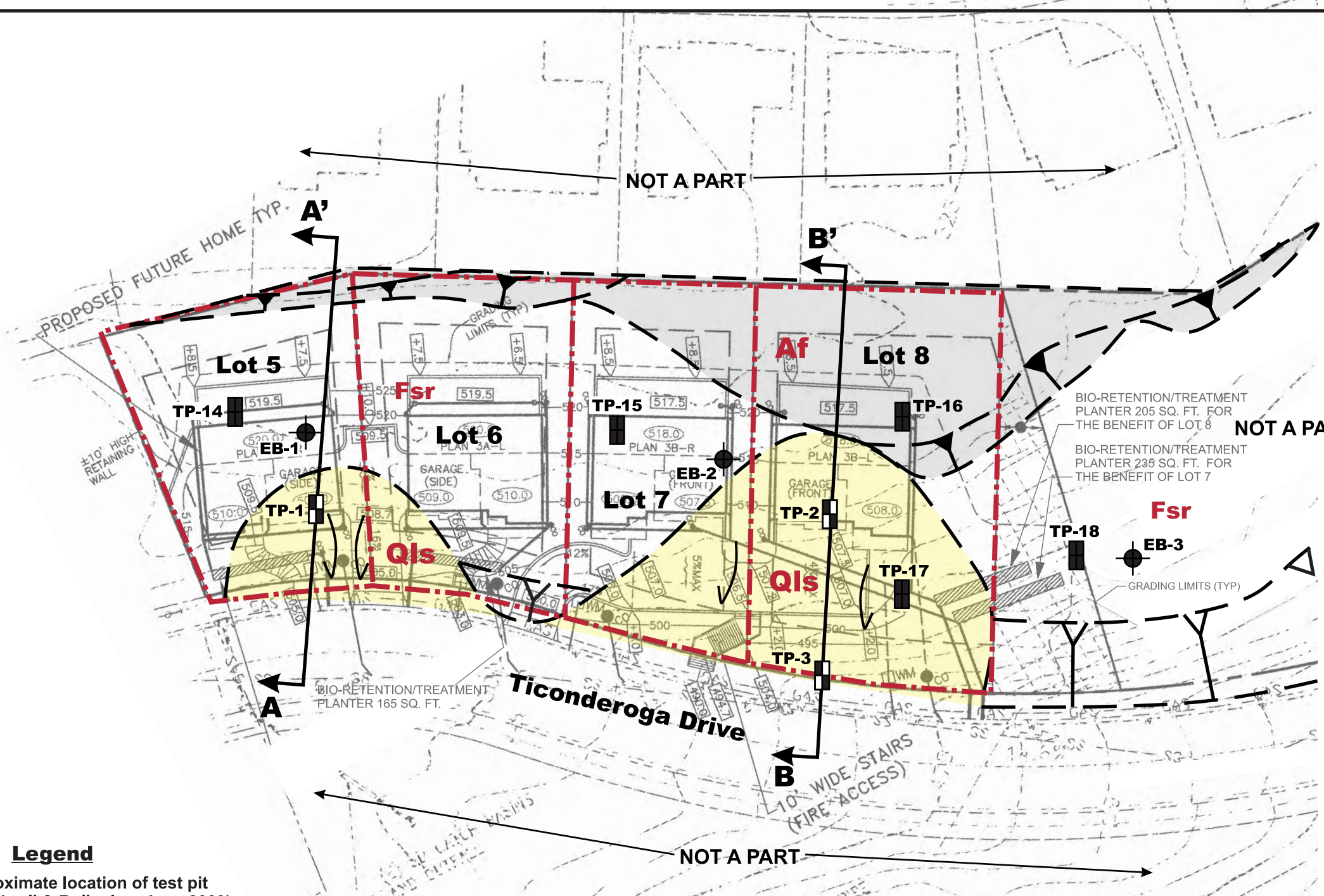



CORNERSTONE
EARTH GROUP

Vicinity Map

Highland Estates - Lots 5 to 11
San Mateo County, CA

Project Number	230-1-5
Figure Number	Figure 1
Date	September 2015
Drawn By	RRN

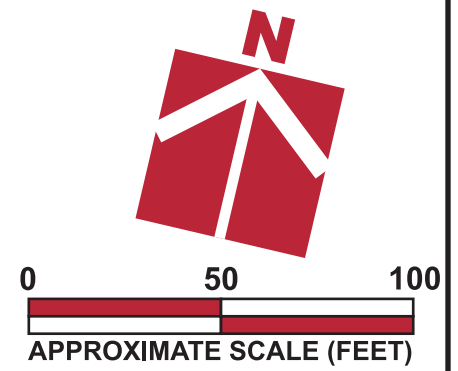


Legend

- TP-1 Approximate location of test pit (Treadwell & Rollo, Inc, June 2009)
- EB-1 Approximate location of boring (Lowney, 2006)
- TP-14 Approximate location of test pit (Berloger, Long and Associates, 1980)
- Geologic section (see Figures 5 and 6)
- Geologic contact (dashed where approximate)
- Cut slope
- Fill slope
- Fill
- Landslide

- Geologic Units**
- Af** Artificial fill
 - Qls** Landslide deposits (Holocene and Pleistocene)
 - Fsr** Franciscan sheared rock
- Note: Colluvium not shown

Base by BKF Engineering, "Preliminary Grading & Utility Plan - Sheet 3", dated 2/2/2010

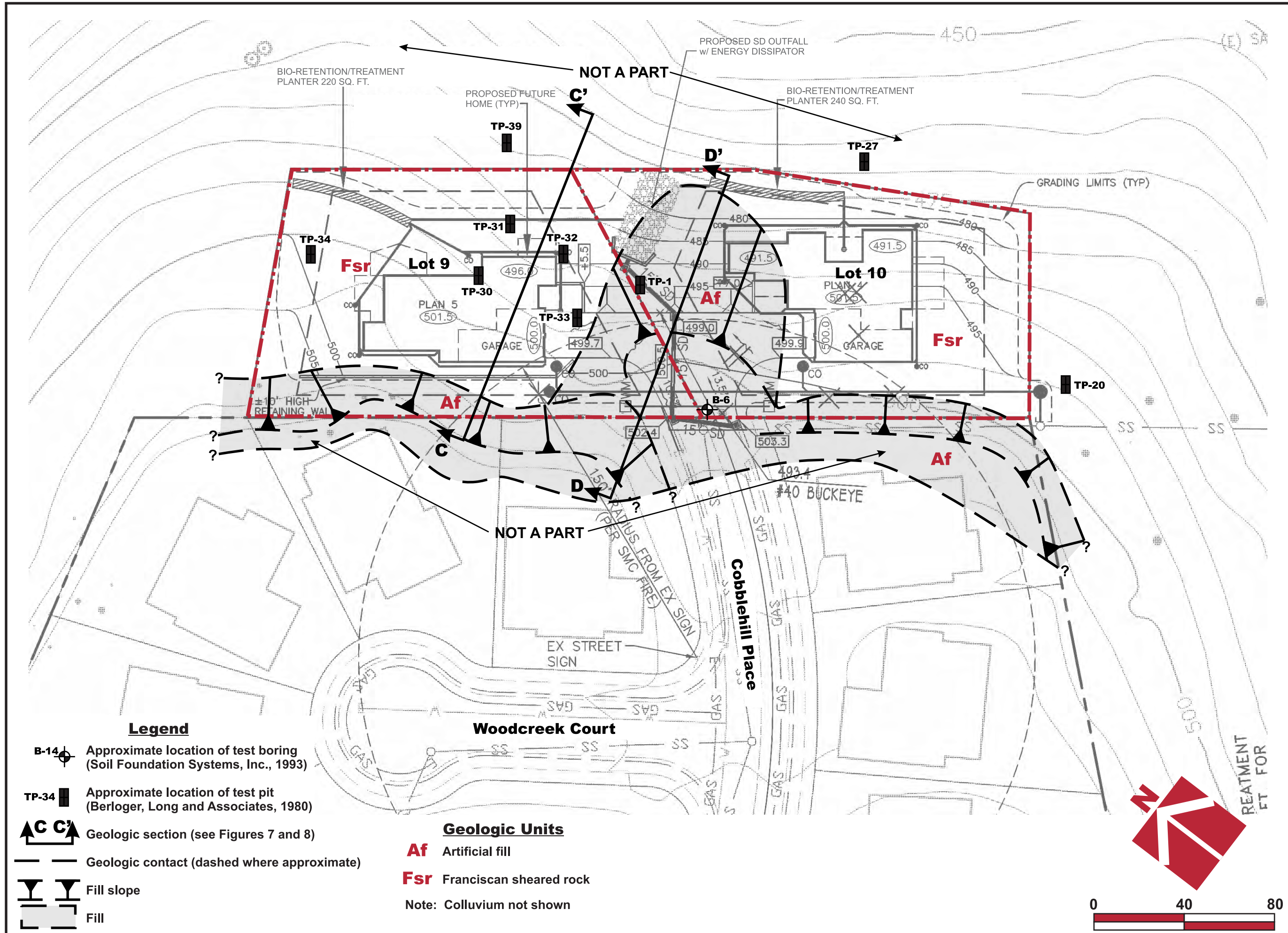


Project Number	230-1-5
Figure Number	Figure 2A
Date	September 2015
Drawn By	RRN

Site Plan and Geologic Map (Lots 5 to 8)

Highland Estates - Lots 5 to 11
San Mateo County, CA

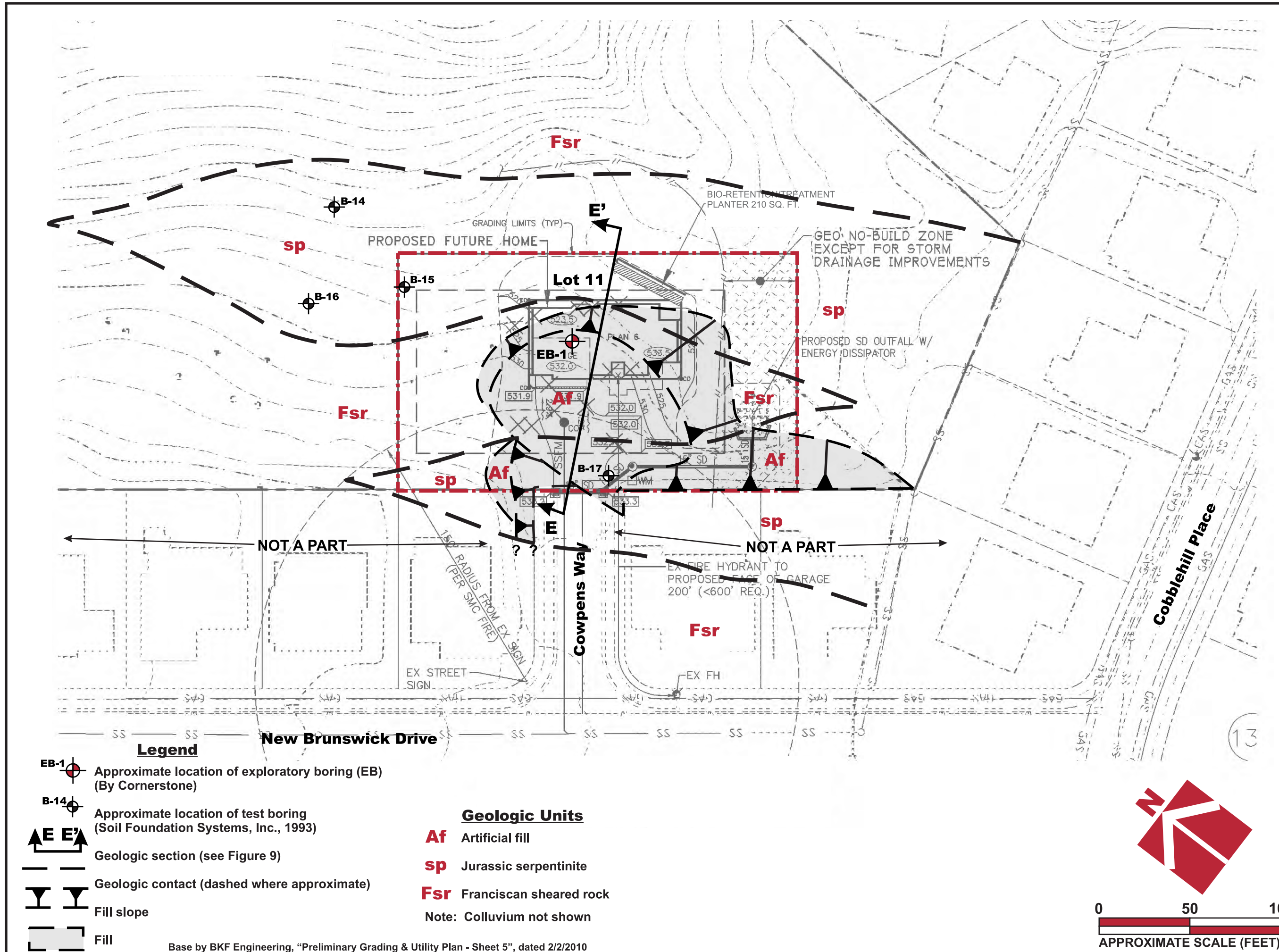




Base by BKF Engineering, "Preliminary Grading & Utility Plan - Sheet 4", dated 2/2/2010

Site Plan and Geologic Map (Lot 11)

Highland Estates - Lots 5 to 11
San Mateo County, CA



Legend

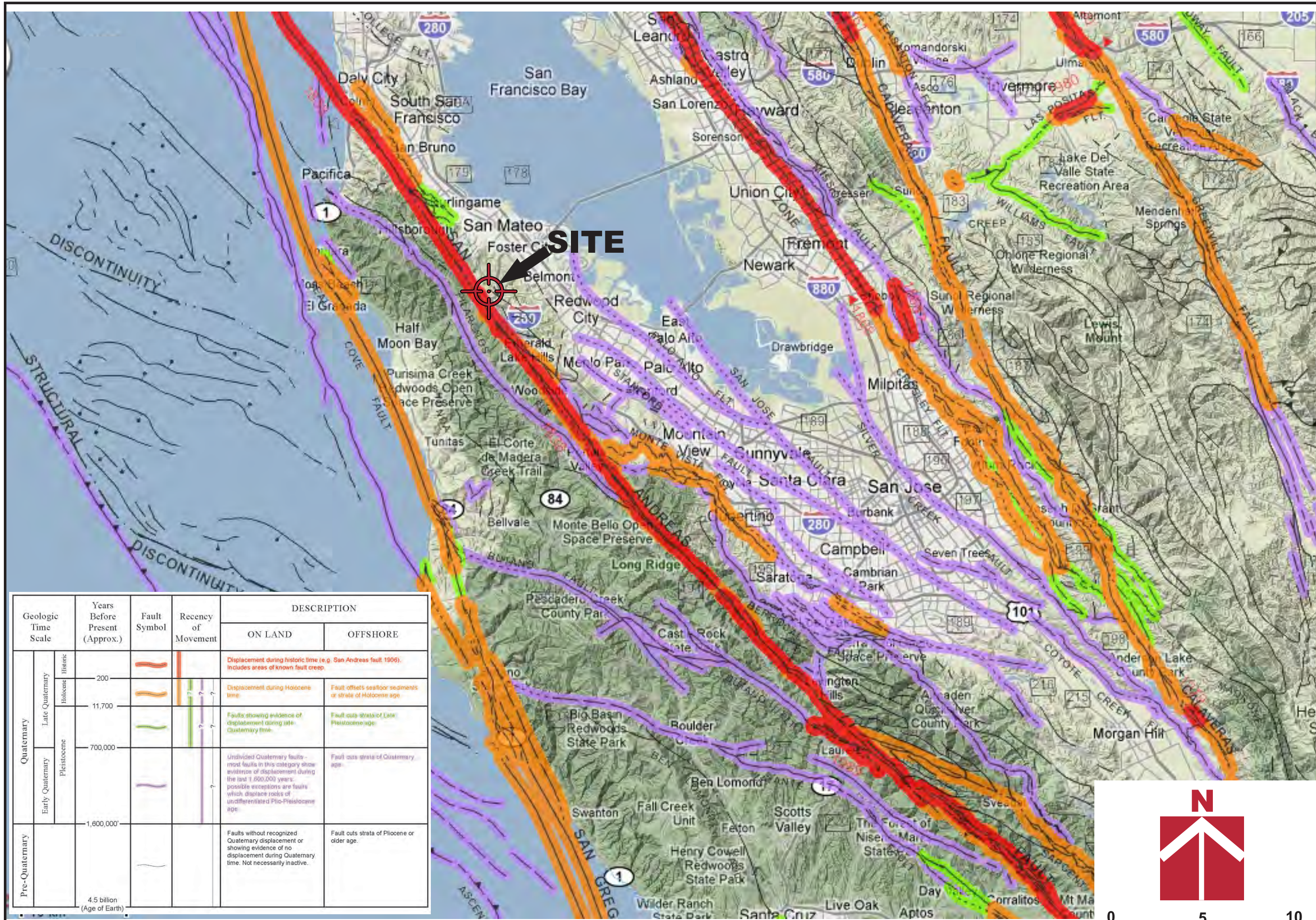
- EB-1 Approximate location of exploratory boring (EB) (By Cornerstone)
- B-14 Approximate location of test boring (Soil Foundation Systems, Inc., 1993)
- Geologic section (see Figure 9)
- Geologic contact (dashed where approximate)
- Fill slope
- Fill

Geologic Units

- Af** Artificial fill
- sp** Jurassic serpentinite
- Fsr** Franciscan sheared rock

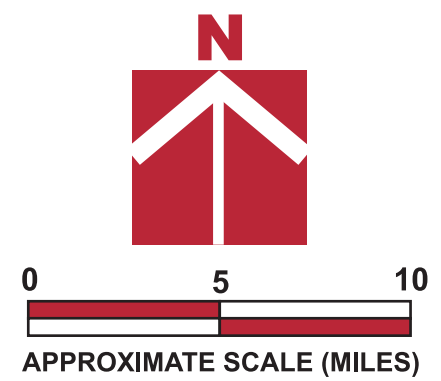
Note: Colluvium not shown

Base by BKF Engineering, "Preliminary Grading & Utility Plan - Sheet 5", dated 2/2/2010



Geologic Time Scale	Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION	
				ON LAND	OFFSHORE
Quaternary	Late Quaternary Holocene / Historic			Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
				Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
Quaternary	Early Quaternary Pleistocene			Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
				Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary	1,600,000 - 4.5 billion (Age of Earth)			Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

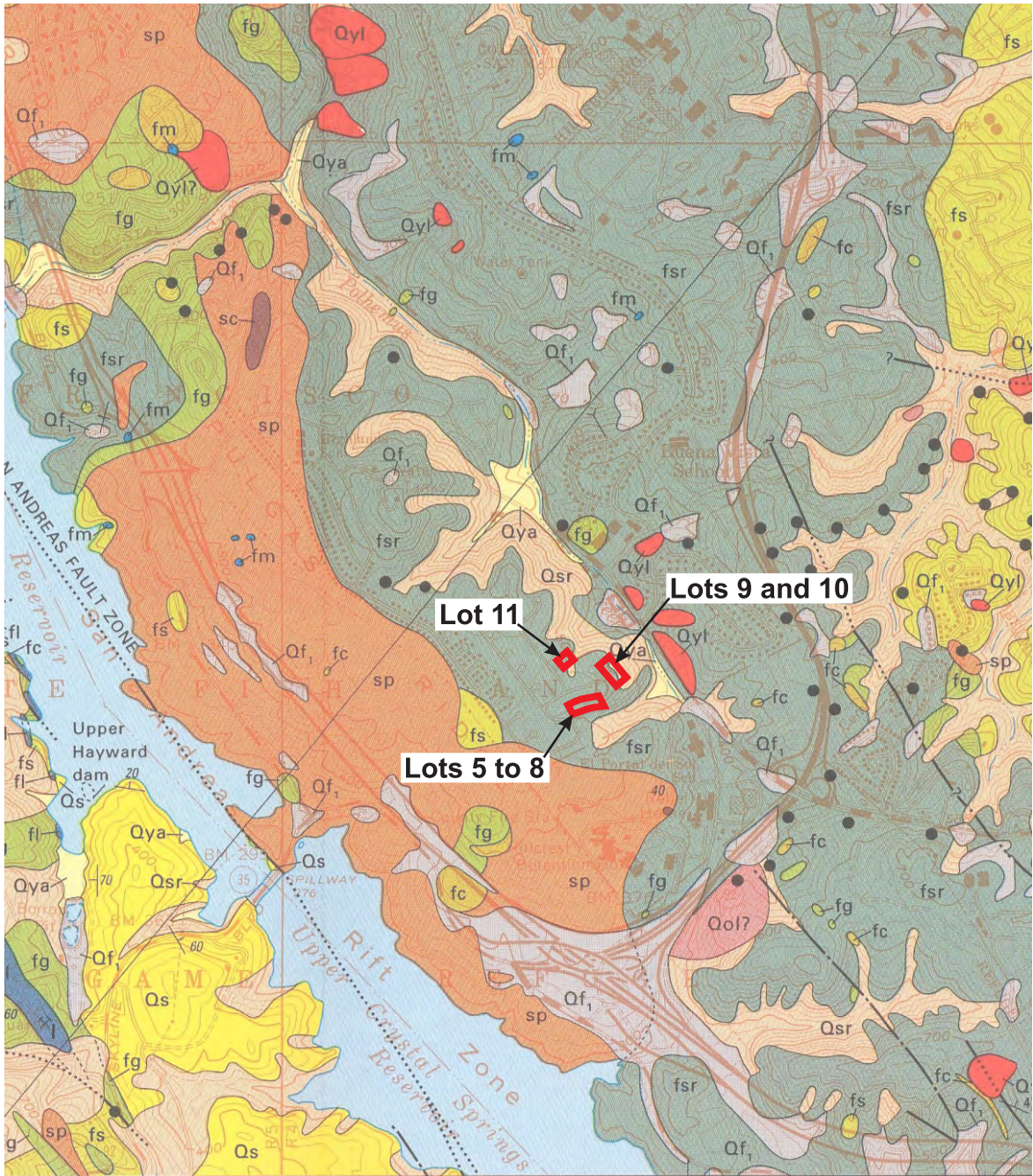
Base by California Geological Survey - 2010 Fault Activity Map of California (Jennings and Bryant, 2010)



Project Number: 230-1-5
Figure Number: Figure 3
Date: September 2015
Drawn By: RRN

Regional Fault Map
Highland Estates - Lots 5 to 11
San Mateo, CA



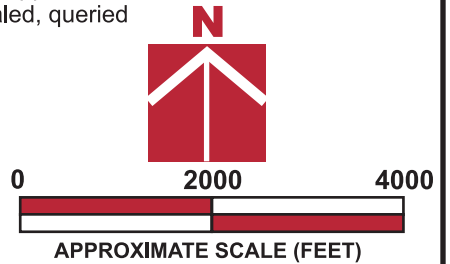


Geologic Units

- Qya** Younger alluvium (Holocene)
- Qsr** Slope wash, ravine fill, and colluvium (Holocene)
- fsr** Sheared rock

Explanation

- Contact- dashed where approximate, dotted where concealed
- Fault- dashed where approximate, dotted where concealed, queried where uncertain



Base: USGS, Geologic Map of the Montara Mountain and San Mateo 7.5' Quadrangle, San Mateo County, California, by Pampeyan, 1994



Vicinity Geologic Map

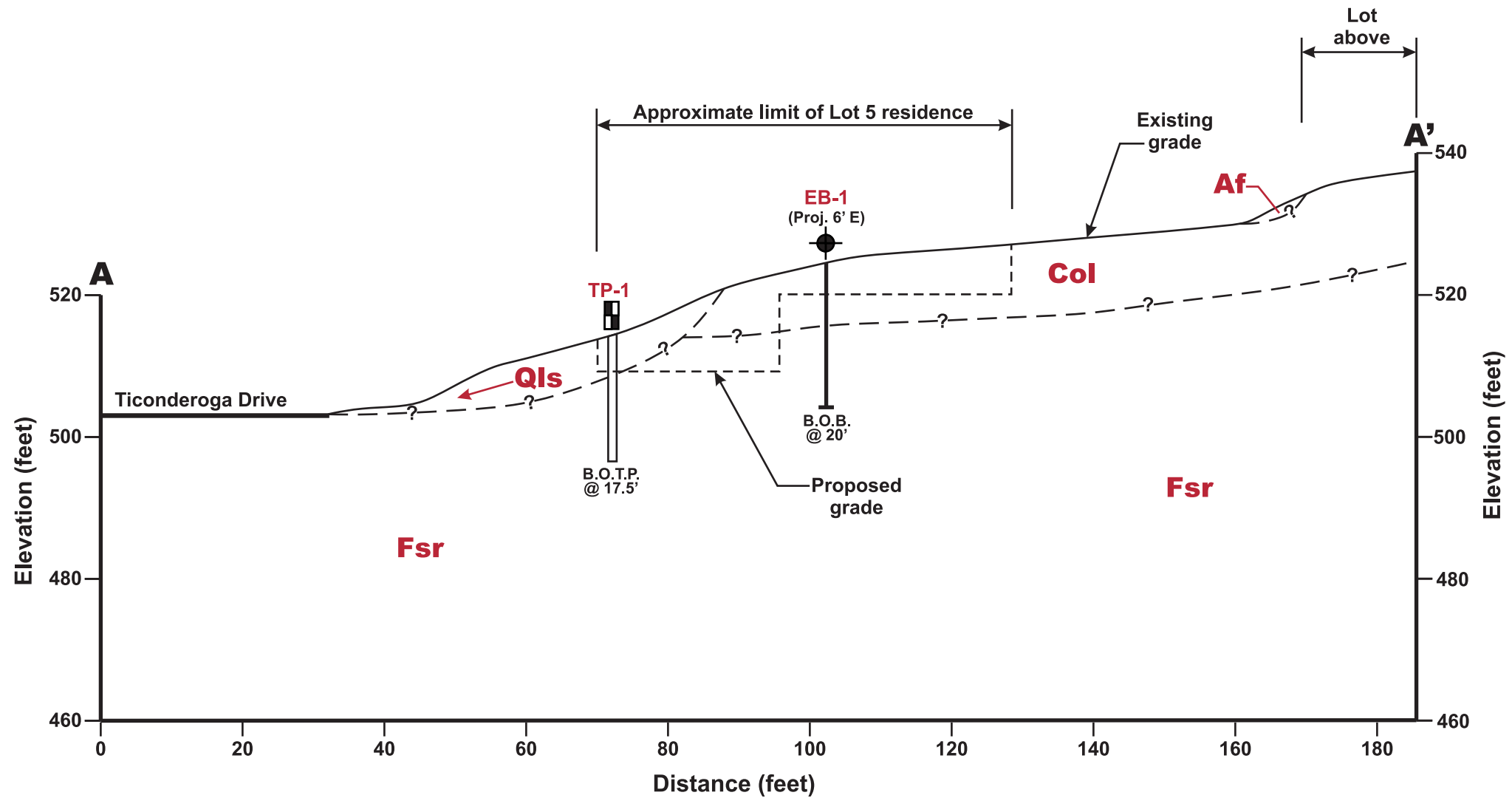
Highland Estates - Lots 5 to 11
San Mateo County, CA

Project Number
230-1-5

Figure Number
Figure 4

Date
September 2015

Drawn By
RRN






Section A-A'
 (View Looking Southwest)
 1"=20' H:V

Explanation

Geologic Units

- Af** Artificial fill
- Col** Colluvium
- QIs** Landslide deposits (Holocene and Pleistocene)
- Fsr** Franciscan sheared rock

Symbols

- TP-1**  Approximate location of test pit (Treadwell & Rollo, Inc, June 2009)
- EB-1**  Approximate location of boring (Lowney, 2006)
-  Geologic contact (Approximate where dashed)

Notes:

- 1) Topographical information provided by BKF Engineering, "Preliminary Grading & Utility Plan - Sheet 3", dated 2/2/2010.
- 2) Surficial fills associated with existing pavements, landscaping or utilities are not shown.
- 3) The subsurface profile is conceptual and is based on limited subsurface data obtained from widely spaced explorations. Actual subsurface conditions may vary significantly between explorations.
- 4) See Figure 2A for location of cross section.

Geologic Cross Section A-A'

Highland Estates - Lots 5 to 11
 San Mateo County, CA

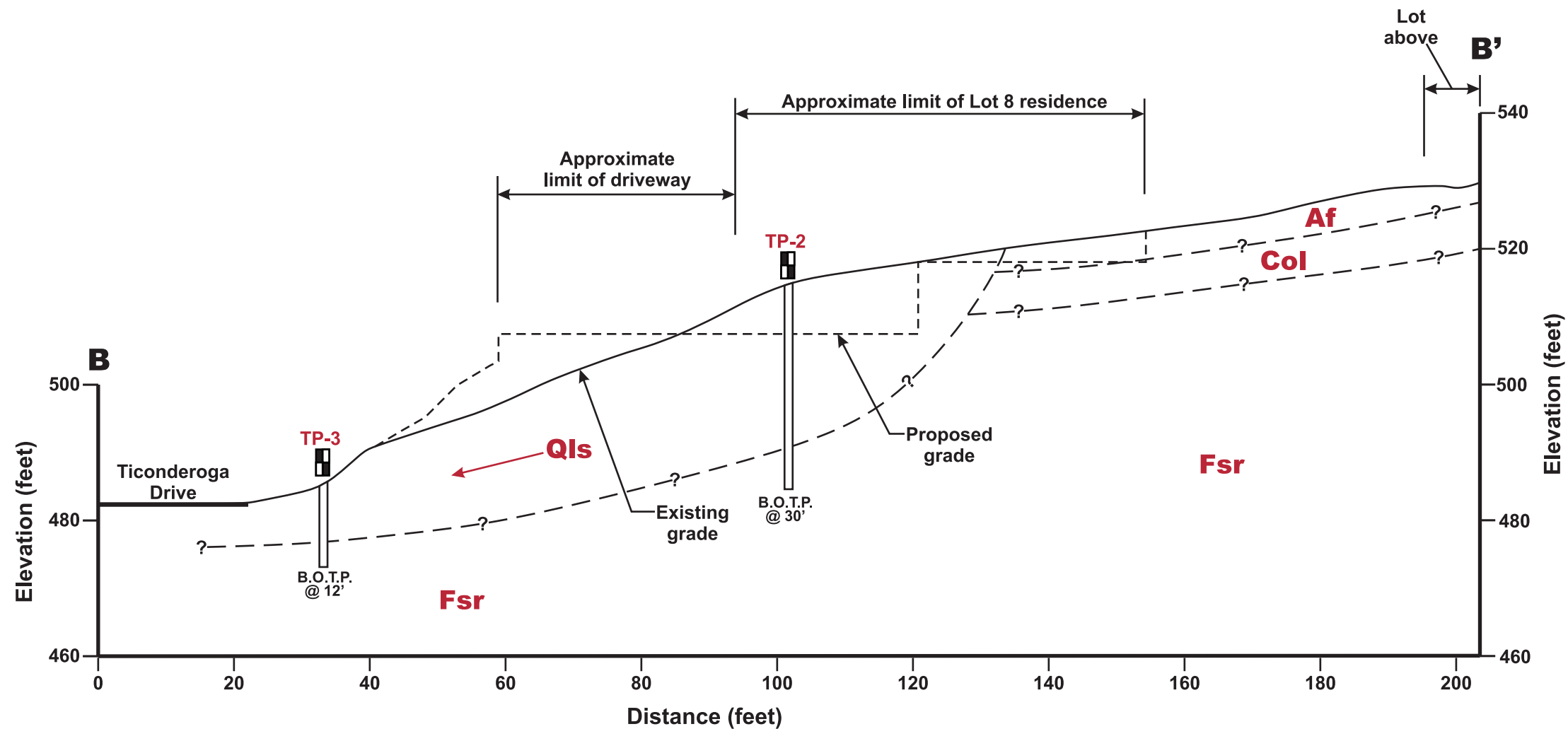


Project Number
 230-1-5

Figure Number
 Figure 5

Date
 September 2015

Drawn By
 RRN



Section B-B'
(View Looking Southwest)
1"=20' H:V

Explanation

Geologic Units

- Af** Artificial fill
- Col** Colluvium
- Qls** Landslide deposits (Holocene and Pleistocene)
- Fsr** Franciscan sheared rock

Symbols

- TP-1** Approximate location of test pit (Treadwell & Rollo, Inc, June 2009)
- ?—** Geologic contact (Approximate where dashed)

Notes:

- 1) Topographical information provided by BKF Engineering, "Preliminary Grading & Utility Plan - Sheet 3", dated 2/2/2010.
- 2) Surficial fills associated with existing pavements, landscaping or utilities are not shown.
- 3) The subsurface profile is conceptual and is based on limited subsurface data obtained from widely spaced explorations. Actual subsurface conditions may vary significantly between explorations.
- 4) See Figure 2A for location of cross section.

Geologic Cross Section B-B'

Highland Estates - Lots 5 to 11
San Mateo County, CA

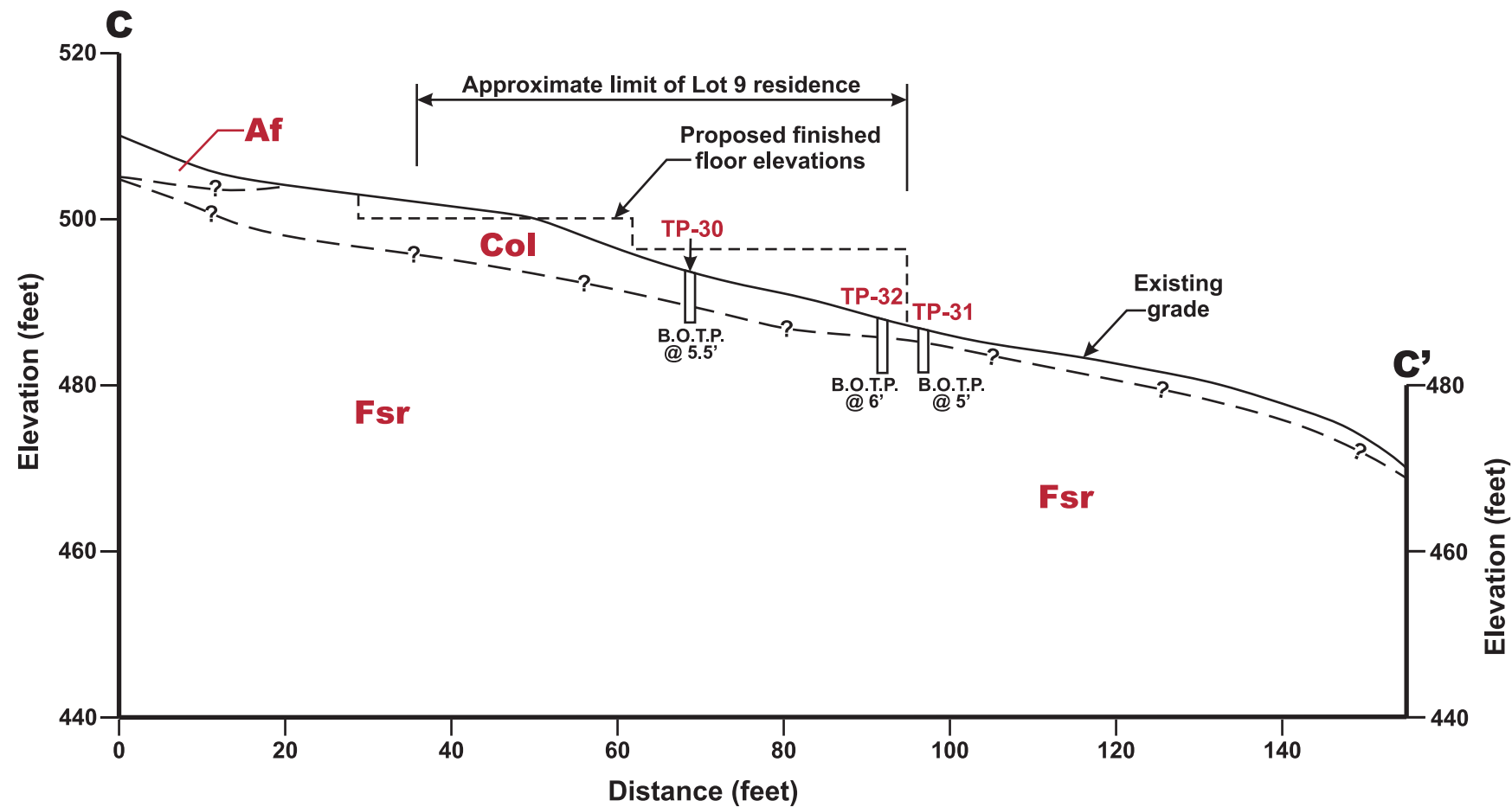


Project Number
230-1-5

Figure Number
Figure 6

Date
September 2015

Drawn By
RRN



Section C-C'
 (View Looking North)
 1"=20' H:V

Explanation

Geologic Units

- Af** Artificial fill
- Col** Colluvium
- Fsr** Franciscan sheared rock

Symbols

- TP-30** Approximate location of test pit (Berloger, Long and Associates, 1980)
- ?— Geologic contact (Approximate where dashed)

Notes:

- 1) Topographical information provided by BKF Engineering, "Preliminary Grading & Utility Plan - Sheet 4", dated 2/2/2010.
- 2) Surficial fills associated with existing pavements, landscaping or utilities are not shown.
- 3) The subsurface profile is conceptual and is based on limited subsurface data obtained from widely spaced explorations. Actual subsurface conditions may vary significantly between explorations.
- 4) See Figure 2B for location of cross section.

Geologic Cross Section C-C'

Highland Estates - Lots 5 to 11
 San Mateo County, CA

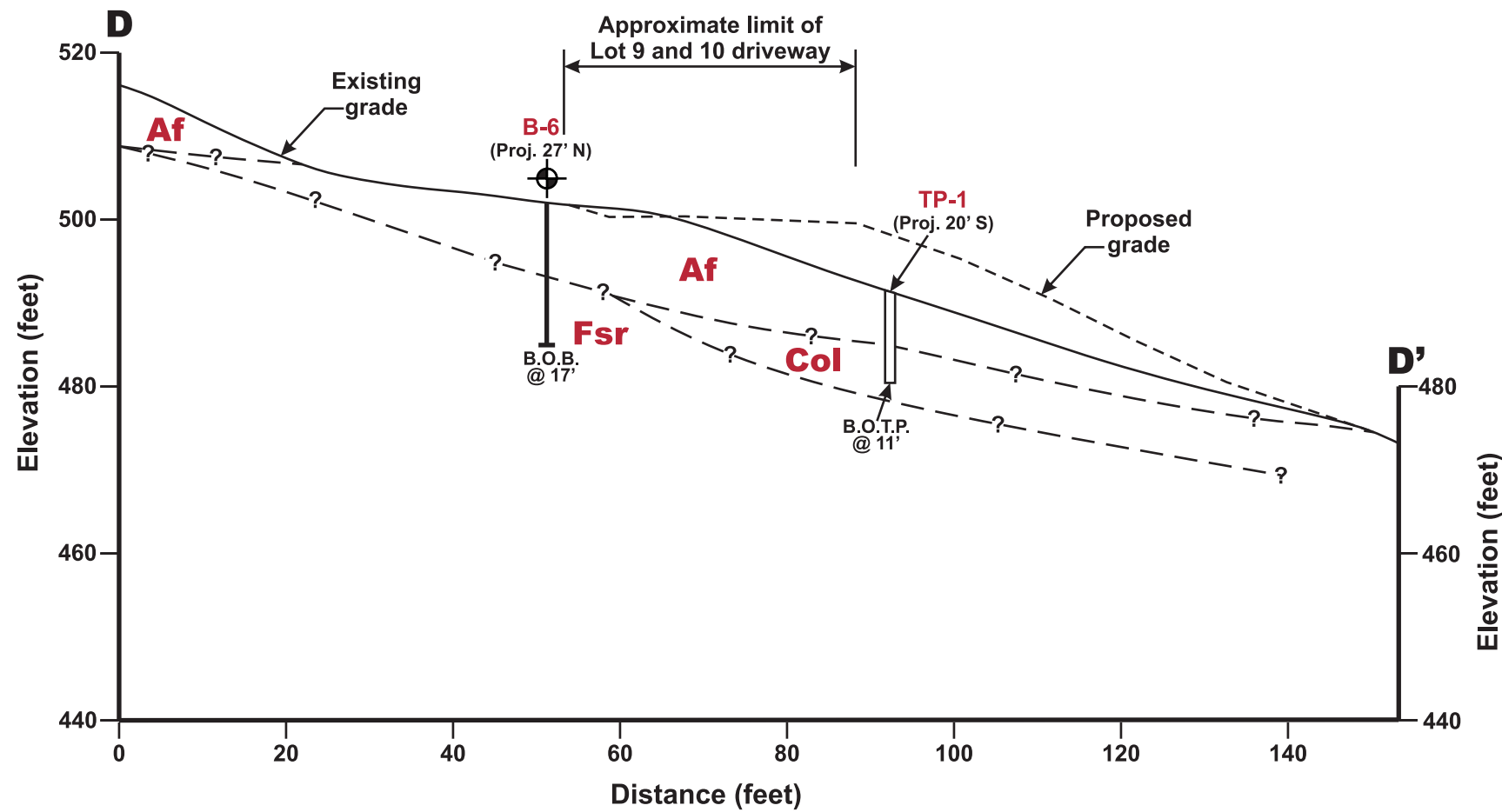


Project Number
 230-1-5

Figure Number
 Figure 7

Date
 September 2015

Drawn By
 RRN



Section D-D'
(View Looking North)
1"=20' H:V

Explanation

Geologic Units

- Af** Artificial fill
- Col** Colluvium
- Fsr** Franciscan sheared rock

Symbols

- TP-1** Approximate location of test pit (Berloger, Long and Associates, 1980)
- B-6** Approximate location of test boring (Soil Foundation Systems, Inc., 1993)
- ?— Geologic contact (Approximate where dashed)

Notes:

- 1) Topographical information provided by BKF Engineering, "Preliminary Grading & Utility Plan - Sheet 4", dated 2/2/2010.
- 2) Surficial fills associated with existing pavements, landscaping or utilities are not shown.
- 3) The subsurface profile is conceptual and is based on limited subsurface data obtained from widely spaced explorations. Actual subsurface conditions may vary significantly between explorations.
- 4) See Figure 2B for location of cross section.

Geologic Cross Section D-D'

Highland Estates - Lots 5 to 11
San Mateo County, CA

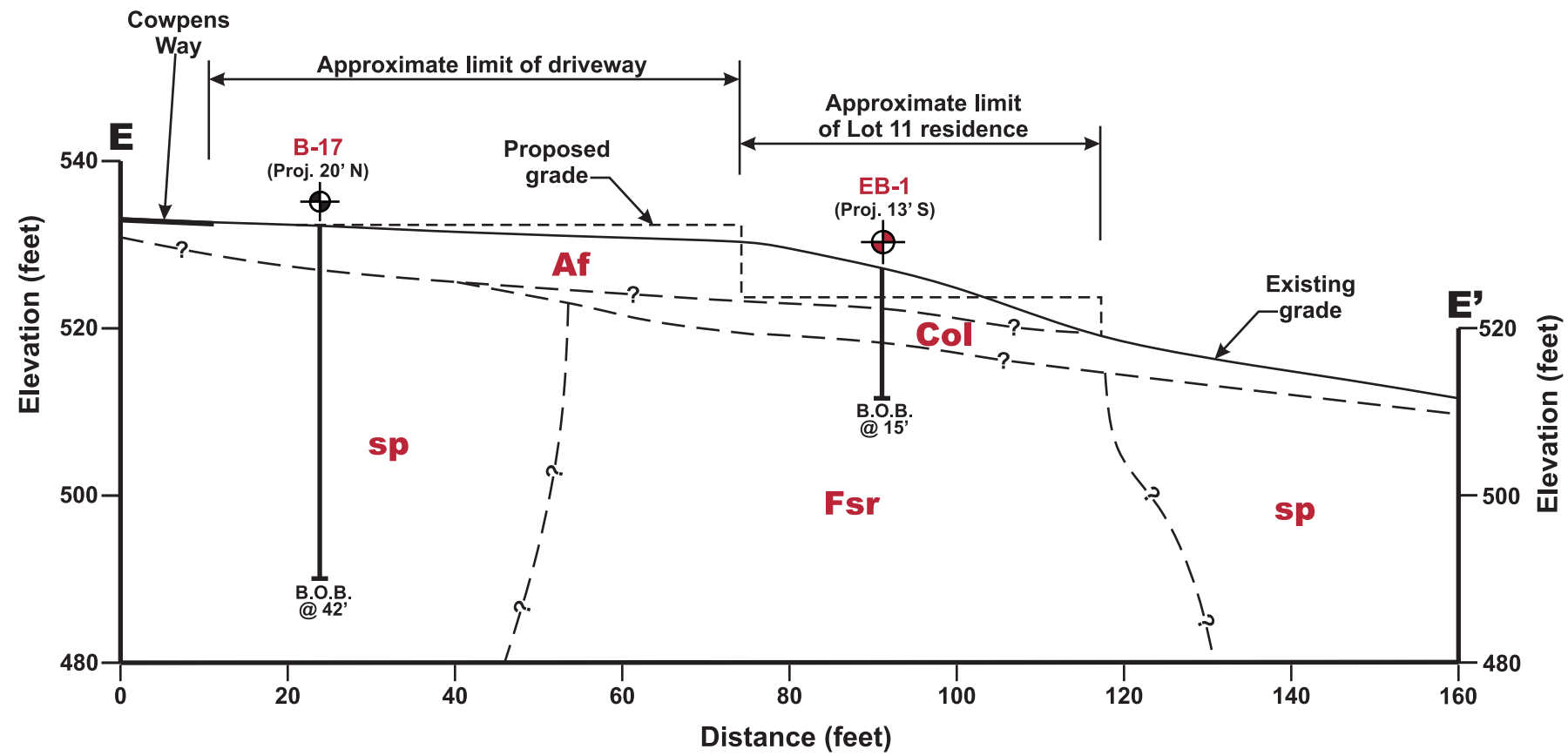


Project Number
230-1-5

Figure Number
Figure 8

Date
September 2015

Drawn By
RRN



Section E-E'
(View Looking North)
1"=20' H:V

Explanation

Geologic Units

- Af** Artificial fill
- Col** Colluvium
- Fsr** Franciscan sheared rock
- sp** Jurassic serpentinite

Symbols

- EB-1** Approximate location of exploratory boring (EB)
- B-14** Approximate location of test boring (Soil Foundation Systems, Inc., 1993)
- Geologic contact (Approximate where dashed)

Notes:

- 1) Topographical information provided by BKF Engineering, "Preliminary Grading & Utility Plan - Sheet 5", dated 2/2/2010.
- 2) Surficial fills associated with existing pavements, landscaping or utilities are not shown.
- 3) The subsurface profile is conceptual and is based on limited subsurface data obtained from widely spaced explorations. Actual subsurface conditions may vary significantly between explorations.
- 4) See Figure 2C for location of cross section.

Geologic Cross Section E-E'

Highland Estates - Lots 5 to 11
San Mateo County, CA

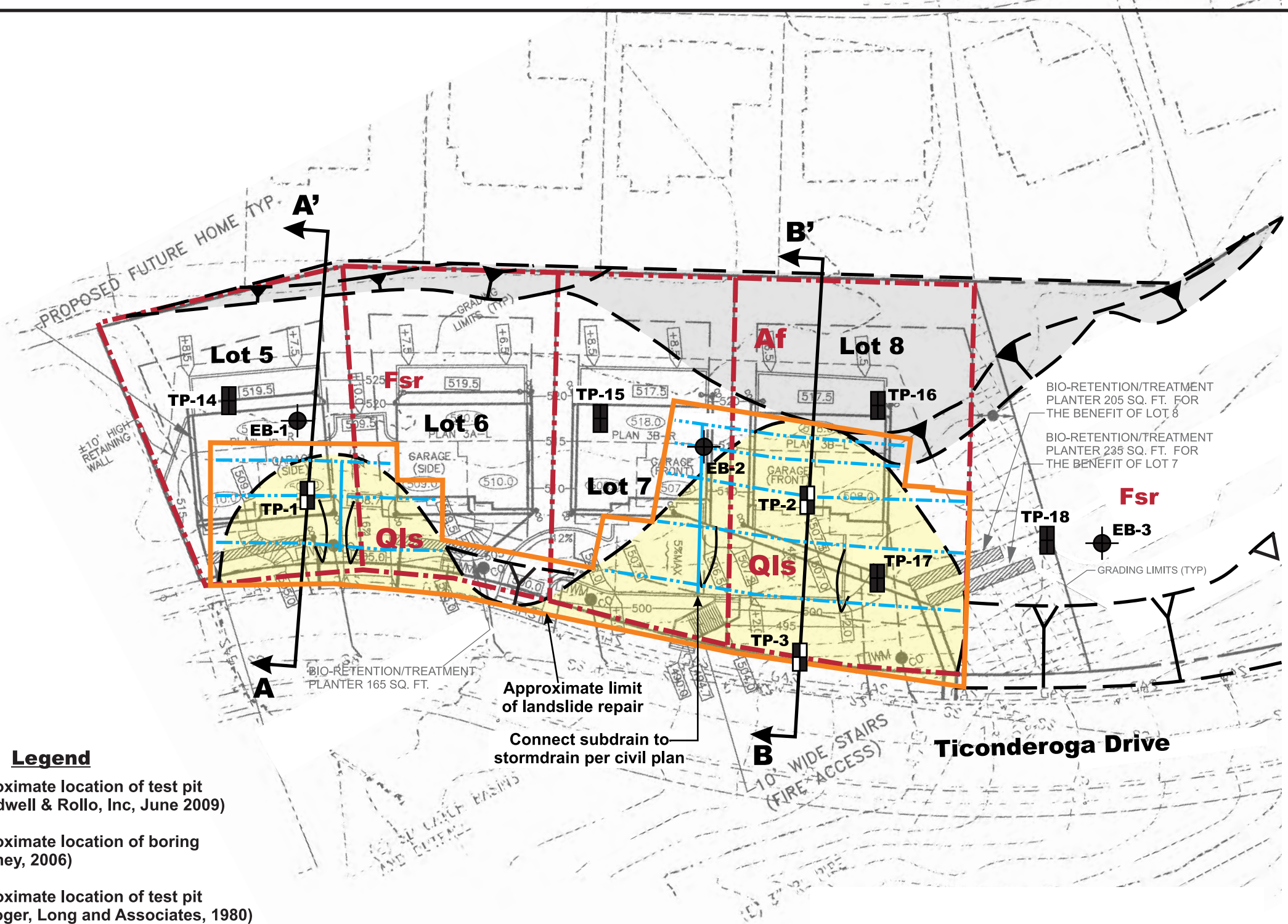


Project Number
230-1-5

Figure Number
Figure 9

Date
September 2015

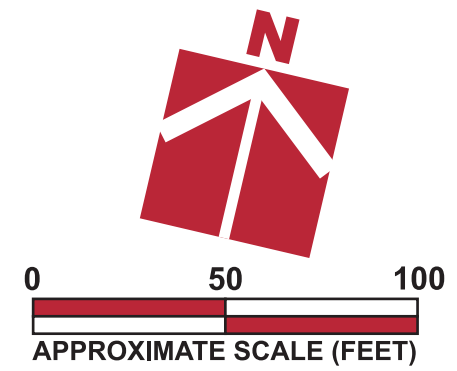
Drawn By
RRN



Legend

- TP-1 Approximate location of test pit (Treadwell & Rollo, Inc, June 2009)
- EB-1 Approximate location of boring (Lowney, 2006)
- TP-14 Approximate location of test pit (Berloger, Long and Associates, 1980)
- Geologic section (see Figures 5 and 6)
- Geologic contact (dashed where approximate)
- Cut slope
- Fill slope
- Fill
- Landslide
- Subdrain
- Solid pipe

- Geologic Units**
- Af** Artificial fill
 - Qls** Landslide deposits (Holocene and Pleistocene)
 - Fsr** Franciscan sheared rock
- Note: Colluvium not shown



The undersigned Geotechnical Engineer has performed a geotechnical investigation at the site including performing field investigation, laboratory testing, engineering analysis, and report preparation as described in the October 30, 2015 report by Cornerstone Earth Group, Inc. for the project. The geotechnical aspects of these plan sheets have been prepared and reviewed by the undersigned Geotechnical Engineer and are based upon Limitations described in the Geotechnical Investigation report. These plans are not a stand-alone document and should be considered as part of the geotechnical investigation report. The geotechnical design aspects in these plans are contingent upon a Geotechnical Engineer and Engineering Geologist observing certain aspects of the project grading. These plans are subject to modification and revision during construction based on the field conditions encountered.



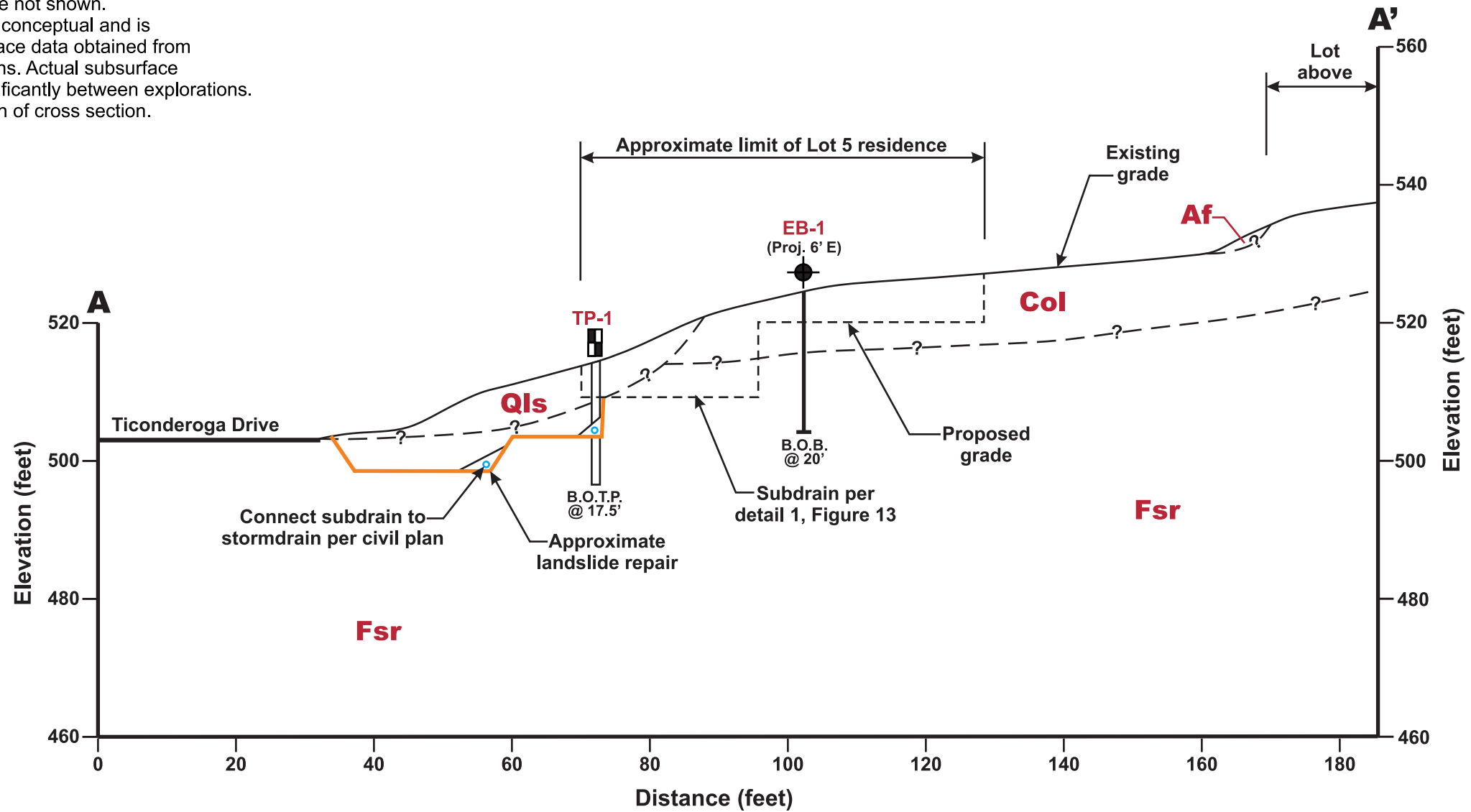
Scott E. Fitinghoff
 Scott E. Fitinghoff, P.E., G.E.

10/30/15

Project Number 230-1-5	Figure Number Figure 10	Date October 2015	Drawn By RRN
Geotechnical Mitigation Plan (Lots 5 to 8) Highland Estates - Lots 5 to 11 San Mateo County, CA			

Notes:

- 1) Topographical information provided by BKF Engineering, "Preliminary Grading & Utility Plan - Sheet 3", dated 2/2/2010.
- 2) Surficial fills associated with existing pavements, landscaping or utilities are not shown.
- 3) The subsurface profile is conceptual and is based on limited subsurface data obtained from widely spaced explorations. Actual subsurface conditions may vary significantly between explorations.
- 4) See Figure 10 for location of cross section.



Section A-A'
(View Looking Southwest)
1"=20' H:V

Explanation

Geologic Units

- Af** Artificial fill
- Col** Colluvium
- Qls** Landslide deposits (Holocene and Pleistocene)
- Fsr** Franciscan sheared rock

Symbols

- TP-1** [Symbol] Approximate location of test pit (Treadwell & Rollo, Inc, June 2009)
- EB-1** [Symbol] Approximate location of boring (Lowney, 2006)
- ?-** Geologic contact (Approximate where dashed)

The undersigned Geotechnical Engineer has performed a geotechnical investigation at the site including performing field investigation, laboratory testing, engineering analysis, and report preparation as described in the October 30, 2015 report by Cornerstone Earth Group, Inc. for the project. The geotechnical aspects of these plan sheets have been prepared and reviewed by the undersigned Geotechnical Engineer and are based upon Limitations described in the Geotechnical Investigation report. These plans are not a stand-alone document and should be considered as part of the geotechnical investigation report. The geotechnical design aspects in these plans are contingent upon a Geotechnical Engineer and Engineering Geologist observing certain aspects of the project grading. These plans are subject to modification and revision during construction based on the field conditions encountered.



[Signature]
Scott E. Fitinghoff, P.E., G.E.

10/30/15

Geotechnical Mitigation Cross Section A-A'

Highland Estates - Lots 5 to 11
San Mateo County, CA



Project Number
230-1-5

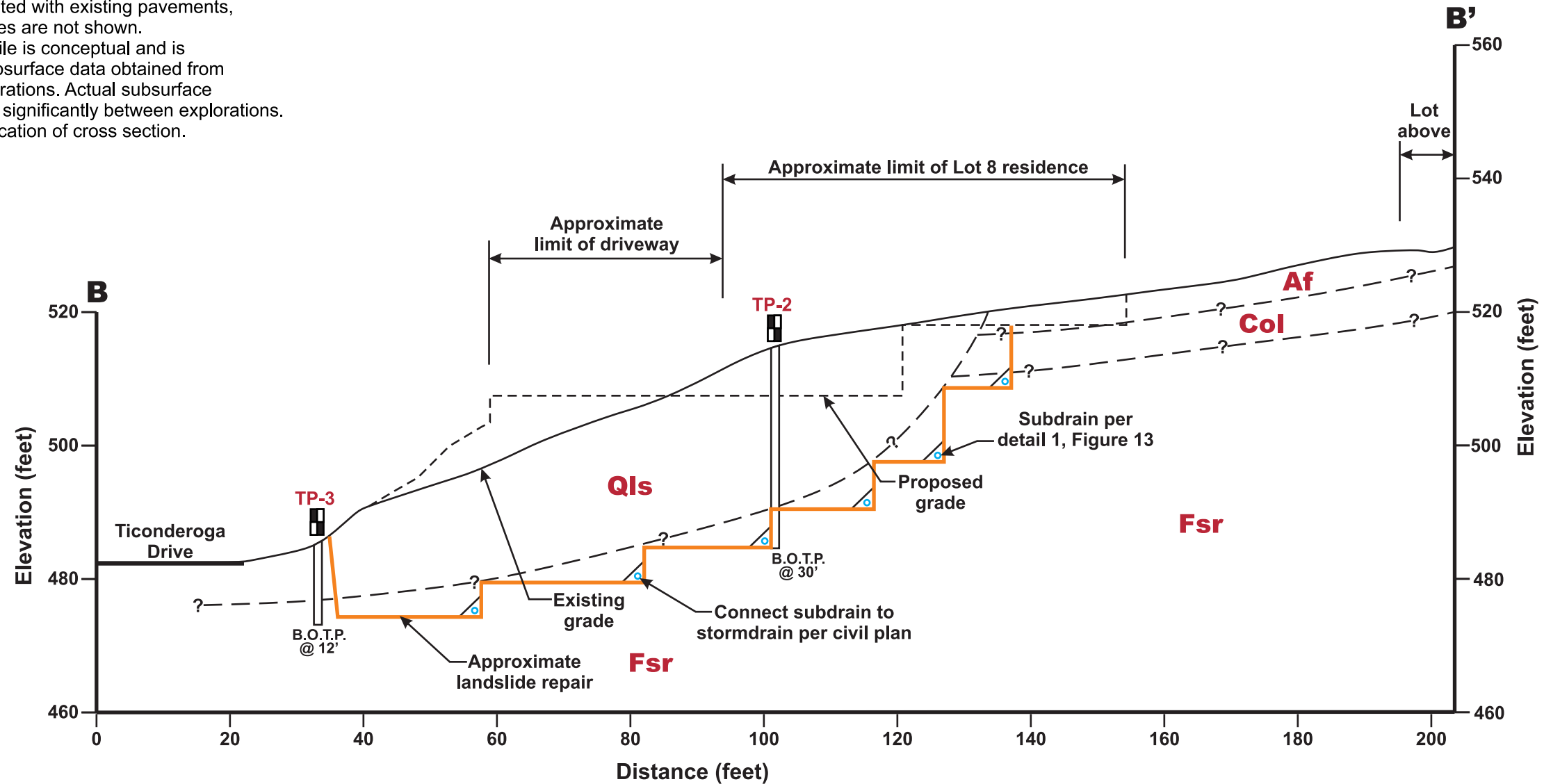
Figure Number
Figure 11

Date
October 2015

Drawn By
RRN

Notes:

- 1) Topographical information provided by BKF Engineering, "Preliminary Grading & Utility Plan - Sheet 3", dated 2/2/2010.
- 2) Surficial fills associated with existing pavements, landscaping or utilities are not shown.
- 3) The subsurface profile is conceptual and is based on limited subsurface data obtained from widely spaced explorations. Actual subsurface conditions may vary significantly between explorations.
- 4) See Figure 10 for location of cross section.



Section B-B'
(View Looking Southwest)
1"=20' H:V

Explanation

Geologic Units

- Af** Artificial fill
- Col** Colluvium
- QIs** Landslide deposits (Holocene and Pleistocene)
- Fsr** Franciscan sheared rock

Symbols

- TP-1** [Symbol] Approximate location of test pit (Treadwell & Rollo, Inc, June 2009)
- ?** Geologic contact (Approximate where dashed)

The undersigned Geotechnical Engineer has performed a geotechnical investigation at the site including performing field investigation, laboratory testing, engineering analysis, and report preparation as described in the October 30, 2015 report by Cornerstone Earth Group, Inc. for the project. The geotechnical aspects of these plan sheets have been prepared and reviewed by the undersigned Geotechnical Engineer and are based upon Limitations described in the Geotechnical Investigation report. These plans are not a stand-alone document and should be considered as part of the geotechnical investigation report. The geotechnical design aspects in these plans are contingent upon a Geotechnical Engineer and Engineering Geologist observing certain aspects of the project grading. These plans are subject to modification and revision during construction based on the field conditions encountered.



[Signature]
Scott E. Fitinghoff, P.E., G.E.

10/30/15

Geotechnical Mitigation Cross Section B-B'

Highland Estates - Lots 5 to 11
San Mateo County, CA



Project Number
230-1-5

Figure Number
Figure 12

Date
October 2015

Drawn By
RRN

DRAINAGE MATERIAL

Alternative 1

Class 2 Permeable Material
(Caltrans Standard Specs, latest edition)

Material shall consist of clean, coarse sand and gravel or crushed stone, conforming to the following gradation requirements:

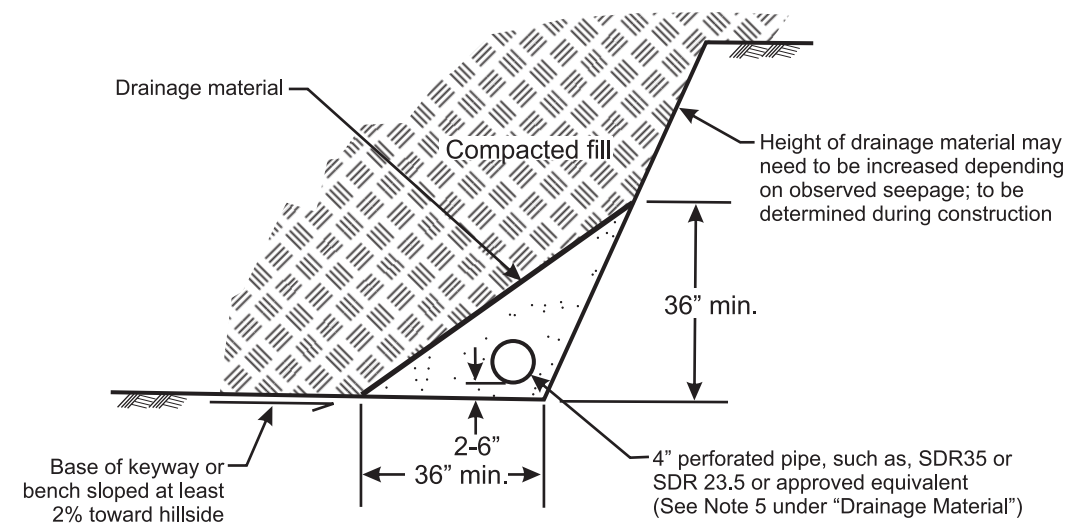
Sieve Size	% Passing Sieve
1"	100
3/4"	90-100
3/8"	40-100
#4	25-40
#8	18-33
#30	5-15
#50	0-7
#200	0-3

Alternative 2

1/2- to 3/4- inch Clean Crushed Rock or Gravel Wrapped in Filter Fabric

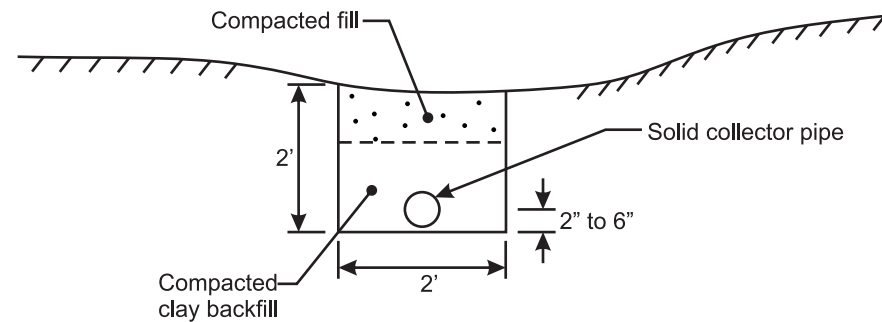
All non-woven filter fabric shall meet the following minimum average roll values unless otherwise specified by Cornerstone Earth Group

Grab Strength (ASTM D-4632):	180 lbs.
Mass Per Unit Area (ASTM D-4751):	5 oz/yd
Apparent Opening Size (ASTM D-4751):	70-100 U.S. std. sieve
Flow Rate (ASTM D-4491):	80 gal/min/ft
Puncture Strength (ASTM D-4833):	80 lbs.



Detail 1 - Typical Bench and Keyway Subdrain

Not to scale



**Detail 2 - Solid Collector Pipe
Detail at Cross Section A-A' and B-B'**

Not to scale

Notes:

- Swale in this area may have active seepage during construction.
- Collector pipe should be 6" perforated pipe, such as SDR-35 or SDR-23.5 or approved equivalent (See Detail 1 Note 5 under "Drainage Material")
- Pipe fittings for clean-outs and other 90° bends in the subdrain system (except the connection between the 4" perforated pipes and 6" collection pipes) should be "Sweep 90's" or other approved equivalent.
- Contractor to provide all incidental fittings in their bid price to construct the subdrain system. Not all incidental fittings are shown on these plans.
- Final subdrain layout and placement to be determined by geotechnical engineer at time of construction.

Notes:

- 1% fall (minimum) along all keyways, benches and subdrain lines.
- All perforated pipe placed perforations down.
- All pipe joints shall be glued.
- All subdrains should be discharged to a free draining outlet approved by the Civil Engineer.
- Subdrain pipe (perforated or solid connector) should consist of SDR-35 PVC pipe when placed in fills less than 30 feet deep. SDR-23.5 PVC pipe should be used when fill is greater than 30 feet deep.
- Use 4" perforated pipe on keyway or benches.
- Use 6" solid pipe for collector pipes or 6" perforated pipe (Detail 2)
- Pipe fittings for clean-outs and other 90° bends in the subdrain system (except the connection between the 4" perforated pipes and 6" collection pipes) should be "Sweep 90's" or other approved equivalent.
- Contractor to provide all incidental fittings in their bid price to construct the subdrain system. Not all incidental fittings are shown on these plans.
- Final subdrain layout and placement to be determined by geotechnical engineer at time of construction.



Scott E. Fitinghoff

10/30/15

Scott E. Fitinghoff, P.E., G.E.

Geotechnical Mitigation Details

Highland Estates - Lots 5 to 11
San Mateo County, CA

**CORNERSTONE
EARTH GROUP**

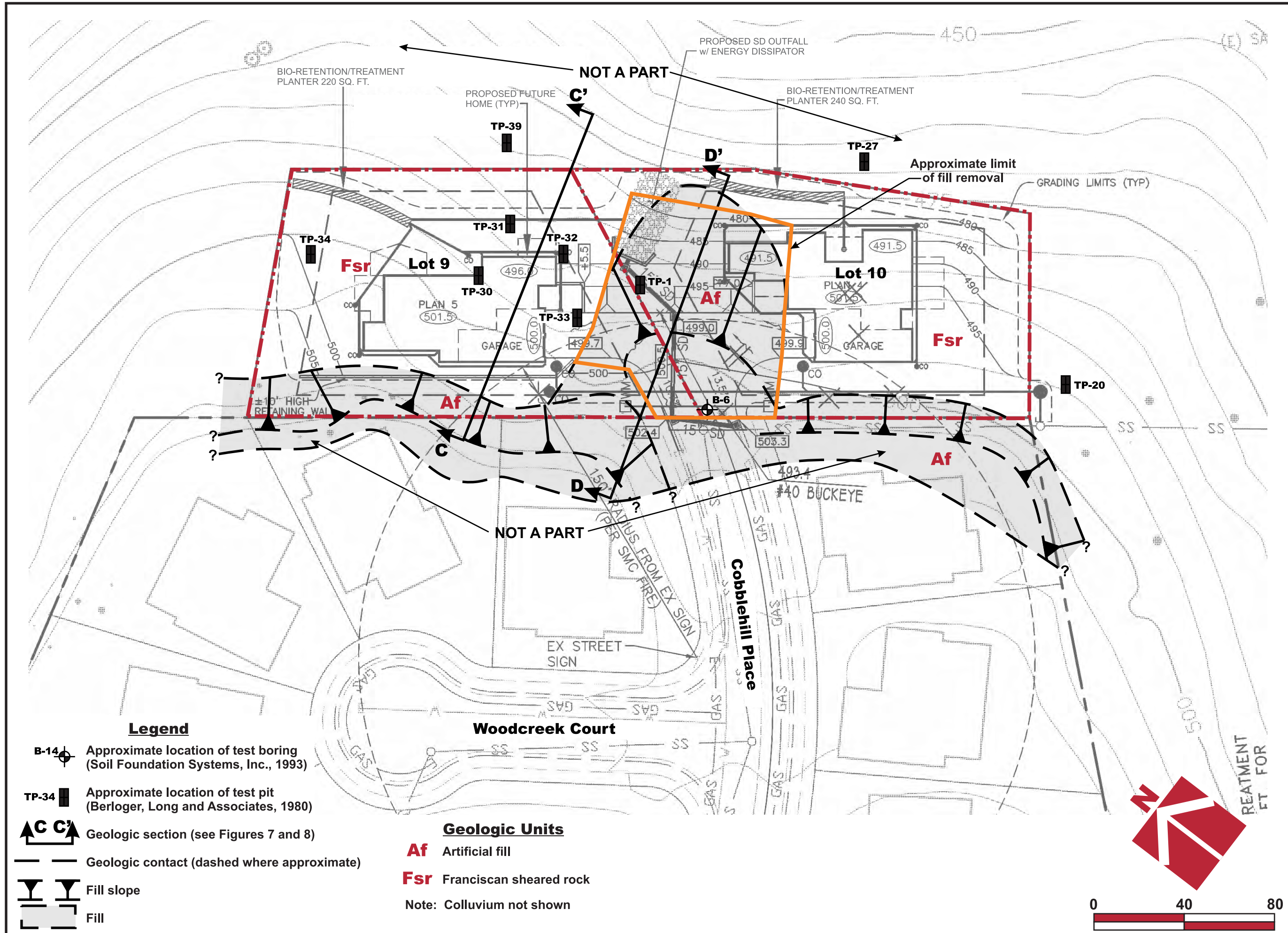


Project Number
230-1-5

Figure Number
Figure 13

Date
October 2015

Drawn By
RRN

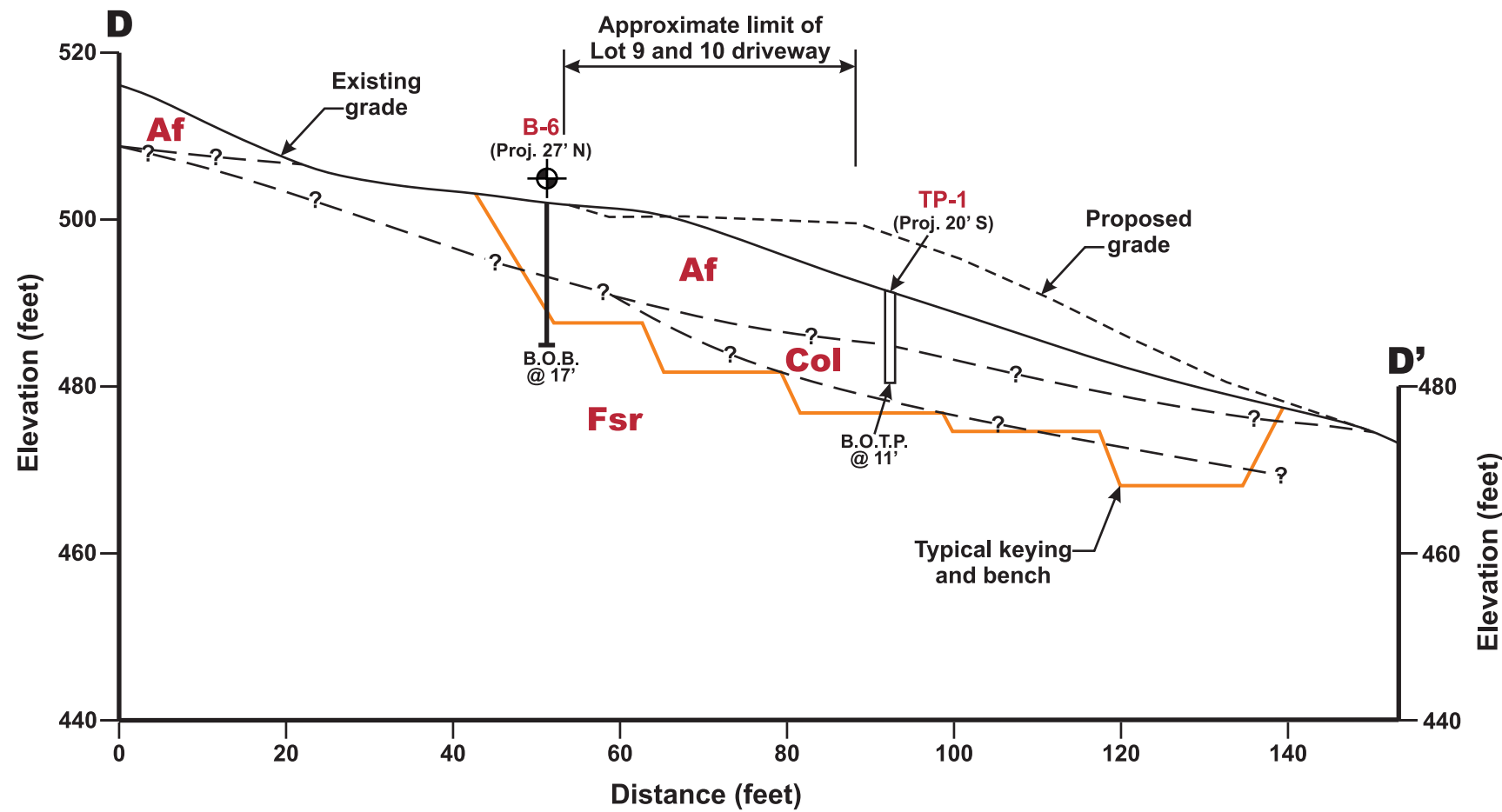


Base by BKF Engineering, "Preliminary Grading & Utility Plan - Sheet 4", dated 2/2/2010

Project Number	230-1-5
Figure Number	Figure 14
Date	October 2015
Drawn By	RRN

Keying and Benching Plan (Lots 9 and 10)

Highland Estates - Lots 5 to 11
San Mateo County, CA



Section D-D'
(View Looking North)
1"=20' H:V

Explanation

Geologic Units

- Af** Artificial fill
- Col** Colluvium
- Fsr** Franciscan sheared rock

Symbols

- TP-1** Approximate location of test pit (Berloger, Long and Associates, 1980)
- B-6** Approximate location of test boring (Soil Foundation Systems, Inc., 1993)
- ?— Geologic contact (Approximate where dashed)

Notes:

- 1) Topographical information provided by BKF Engineering, "Preliminary Grading & Utility Plan - Sheet 4", dated 2/2/2010.
- 2) Surficial fills associated with existing pavements, landscaping or utilities are not shown.
- 3) The subsurface profile is conceptual and is based on limited subsurface data obtained from widely spaced explorations. Actual subsurface conditions may vary significantly between explorations.
- 4) See Figure 14 for location of cross section.

Keying and Benching Cross Section D-D'

Highland Estates - Lots 5 to 11
San Mateo County, CA



Project Number
230-1-5

Figure Number
Figure 15

Date
October 2015

Drawn By
RRN

APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using limited-access, solid-stem auger drilling equipment. One 4-inch-diameter exploratory boring was drilled on July 28, 2015 to a depth of 15 feet. The approximate location of the exploratory boring is shown on Site Plan and Geologic Map, Figure 2C. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Boring logs, as well as a key to the classification of the soil and bedrock, are included as part of this appendix.

The boring location was approximated using existing site boundaries, a tape measure, and other site features as references. The boring elevation was not determined. The location of the boring should be considered accurate only to the degree implied by the method used.

Representative soil and bedrock samples were obtained from the boring at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. The standard penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The 2-inch O.D. split-spoon sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration (ASTM D1586). 2.5-inch I.D. samples were obtained using a Modified California Sampler driven into the soil with the 140-pound hammer previously described. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows required to drive the last 12 inches. The various samplers are denoted at the appropriate depth on the boring log.

Field tests included an evaluation of the unconfined compressive strength of the soil samples using a pocket penetrometer device. The results of these tests are presented on the individual boring logs at the appropriate sample depths.

The attached boring log and related information depict subsurface conditions at the locations indicated and on the date designated on the log. Subsurface conditions at other locations may differ from conditions occurring at this boring location. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the log represent the approximate boundary between soil types and the transition may be gradual.

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-98)

MATERIAL TYPES	CRITERIA FOR ASSIGNING SOIL GROUP NAMES			GROUP SYMBOL	SOIL GROUP NAMES & LEGEND	
COARSE-GRAINED SOILS >50% RETAINED ON NO. 200 SIEVE	GRAVELS >50% OF COARSE FRACTION RETAINED ON NO 4. SIEVE	CLEAN GRAVELS <5% FINES	$Cu > 4$ AND $1 < Cc < 3$	GW	WELL-GRADED GRAVEL	
			$Cu > 4$ AND $1 > Cc > 3$	GP	POORLY-GRADED GRAVEL	
		GRAVELS WITH FINES >12% FINES	FINES CLASSIFY AS ML OR CL	GM	SILTY GRAVEL	
			FINES CLASSIFY AS CL OR CH	GC	CLAYEY GRAVEL	
	SANDS >50% OF COARSE FRACTION PASSES ON NO 4. SIEVE	CLEAN SANDS <5% FINES	$Cu > 6$ AND $1 < Cc < 3$	SW	WELL-GRADED SAND	
			$Cu > 6$ AND $1 > Cc > 3$	SP	POORLY-GRADED SAND	
		SANDS AND FINES >12% FINES	FINES CLASSIFY AS ML OR CL	SM	SILTY SAND	
			FINES CLASSIFY AS CL OR CH	SC	CLAYEY SAND	
FINE-GRAINED SOILS >50% PASSES NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT < 50	INORGANIC	$PI > 7$ AND PLOTS > "A" LINE	CL	LEAN CLAY	
			$PI > 4$ AND PLOTS < "A" LINE	ML	SILT	
		ORGANIC	LL (oven dried)/ LL (not dried) < 0.75	OL	ORGANIC CLAY OR SILT	
	SILTS AND CLAYS LIQUID LIMIT > 50	INORGANIC	PI PLOTS > "A" LINE	CH	FAT CLAY	
			PI PLOTS < "A" LINE	MH	ELASTIC SILT	
		ORGANIC	LL (oven dried)/ LL (not dried) < 0.75	OH	ORGANIC CLAY OR SILT	
HIGHLY ORGANIC SOILS	PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR			PT	PEAT	

OTHER MATERIAL SYMBOLS	
	Poorly-Graded Sand with Clay
	Clayey Sand
	Sandy Silt
	Artificial/Undocumented Fill
	Poorly-Graded Gravelly Sand
	Topsoil
	Well-Graded Gravel with Clay
	Well-Graded Gravel with Silt
	Sand
	Silt
	Well Graded Gravelly Sand
	Gravelly Silt
	Asphalt
	Boulders and Cobble

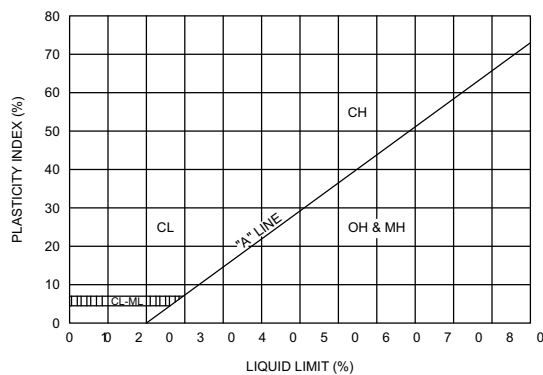
SAMPLER TYPES

	SPT		Shelby Tube
	Modified California (2.5" I.D.)		No Recovery
	Rock Core		Grab Sample

ADDITIONAL TESTS

CA - CHEMICAL ANALYSIS (CORROSIVITY)	PI - PLASTICITY INDEX
CD - CONSOLIDATED DRAINED TRIAXIAL	SW - SWELL TEST
CN - CONSOLIDATION	TC - CYCLIC TRIAXIAL
CU - CONSOLIDATED UNDRAINED TRIAXIAL	TV - TORVANE SHEAR
DS - DIRECT SHEAR	UC - UNCONFINED COMPRESSION
PP - POCKET PENETROMETER (TSF)	(1.5) - (WITH SHEAR STRENGTH IN KSF)
(3.0) - (WITH SHEAR STRENGTH IN KSF)	-
RV - R-VALUE	UU - UNCONSOLIDATED UNDRAINED TRIAXIAL
SA - SIEVE ANALYSIS: % PASSING #200 SIEVE	
	- WATER LEVEL

PLASTICITY CHART



PENETRATION RESISTANCE (RECORDED AS BLOWS / FOOT)

SAND & GRAVEL		SILT & CLAY		
RELATIVE DENSITY	BLOWS/FOOT*	CONSISTENCY	BLOWS/FOOT*	STRENGTH** (KSF)
VERY LOOSE	0 - 4	VERY SOFT	0 - 2	0 - 0.25
LOOSE	4 - 10	SOFT	2 - 4	0.25 - 0.5
MEDIUM DENSE	10 - 30	MEDIUM STIFF	4 - 8	0.5 - 1.0
DENSE	30 - 50	STIFF	8 - 15	1.0 - 2.0
VERY DENSE	OVER 50	VERY STIFF	15 - 30	2.0 - 4.0
		HARD	OVER 30	OVER 4.0

* NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1-3/8 INCH I.D.) SPLIT-BARREL SAMPLER THE LAST 12 INCHES OF AN 18-INCH DRIVE (ASTM-1586 STANDARD PENETRATION TEST).

** UNDRAINED SHEAR STRENGTH IN KIPS/SQ.FT. AS DETERMINED BY LABORATORY TESTING OR APPROXIMATED BY THE STANDARD PENETRATION TEST, POCKET PENETROMETER, TORVANE, OR VISUAL OBSERVATION.

HARDNESS

1. **Soft** – Reserved for plastic material alone.
2. **Low hardness** – Can be gouged deeply or carved easily with a knife blade.
3. **Moderately hard** – Can be readily scratched by a knife blade: scratch leaves a heavy trace of dust and is readily visible after the powder has been blown away.
4. **Hard** – Can be scratched with difficulty: scratch produces little powder and is often faintly visible.
5. **Very hard** – Cannot be scratched with knife blade: leaves a metallic streak.

STRENGTH

1. **Plastic** or very low strength.
2. **Friable** – Crumbles easily by rubbing with fingers.
3. **Weak** – An unfractured specimen of such material will crumble under light hammer blows.
4. **Moderately strong** – Specimen will withstand a few heavy hammer blows before breaking.
5. **Strong** – Specimen will withstand a few heavy ringing blows and will yield with difficulty only dust and small flying fragments.
6. **Very strong** – Specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.

WEATHERING – The physical and chemical disintegration and decomposition of rocks and minerals by natural processes such as oxidation, reduction, hydration, solution, carbonation, and freezing and thawing.

- D. **Deep** – Moderate to complete mineral decomposition: extensive disintegration: deep and thorough discoloration: many fractures, all extensively coated or filled with oxides, carbonates and/or clay or silt.
- M. **Moderate** – Slight change or partial decomposition of minerals: little disintegration: cementation little to unaffected. Moderate to occasionally intense discoloration. Moderately coated fractures.
- L. **Little** – No megascopic decomposition of minerals: little or no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains or fracture surfaces.
- F. **Fresh** – Unaffected by weathering agents. No disintegration or discoloration. Fractures usually less numerous than joints.

FRACTURING

Intensity	Size of Pieces in Feet
Very little fractured	Greater than 4.0
Occasionally fractured	1.0 to 4.0
Moderately fractured	0.5 to 1.0
Closely fractured	0.1 to 0.5
Intensely fractured	0.05 to 0.1
Crushed	Less than 0.05

BEDDING OF SEDIMENTARY ROCKS

Splitting Property	Thickness	Stratification
Massive	Greater than 4.0 feet	very thick-bedded
Blocky	2.0 to 4.0 feet	thick-bedded
Slabby	0.2 to 2.0 feet	thin-bedded
Flaggy	0.05 to 0.2 feet	very thin-bedded
Shaly or Platy	0.01 to 0.05 feet	laminated
Papery	less than 0.01 feet	thinly laminated



CORNERSTONE EARTH GROUP

BORING NUMBER EB-1

PAGE 1 OF 1

DATE STARTED 7/28/15 DATE COMPLETED 7/28/15
 DRILLING CONTRACTOR Cenozoic Drilling Inc.
 DRILLING METHOD Minuteman, 4 inch Solid Flight Auger
 LOGGED BY CSH
 NOTES _____

PROJECT NAME Highland Estates Lot 11
 PROJECT NUMBER 230-1-5
 PROJECT LOCATION San Mateo County, CA
 GROUND ELEVATION _____ BORING DEPTH 15 ft.
 LATITUDE _____ LONGITUDE _____
 GROUND WATER LEVELS:
 ▽ AT TIME OF DRILLING Not Encountered
 ▼ AT END OF DRILLING Not Encountered

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf			
										○ HAND PENETROMETER △ TORVANE ● UNCONFINED COMPRESSION ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL			
										1.0	2.0	3.0	4.0
	0		Clayey Sand (SC) [Fill] medium dense, moist, brown, fine sand, some fine to coarse subangular to subrounded gravel Liquid Limit = 40, Plastic Limit = 18	26	MC-1	109	10	22					
				23	MC-2	83	18						
				30	MC-3	87	12						
	5			35	MC								
			Sandy Lean Clay (CL) [Colluvium] very stiff, moist, dark gray brown, fine sand, some fine subangular to subrounded gravel, moderate plasticity	29	SPT-5		16						>4.5
			Sandstone - Franciscan Complex [Fsr] low hardness, weak, deep weathering, yellowish gray, fine to medium sand	46	MC-6	119	10						
	10		Shale - Franciscan Complex [Fsr] low hardness, weak, deep weathering, dark gray to brown, some interbedded sandstone	45	SPT-7		8						
			Sandstone - Franciscan Complex [Fsr] low hardness, weak, deep weathering, yellowish gray, fine to medium sand	60	SPT-8		10						
				67	SPT-9		9						
	15		Bottom of Boring at 15.0 feet.										
	20												
	25												

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 8/12/15 13:23 - P:\DRAFTING\GINT FILES\230-1-5 HIGHLAND EST LOT 11.GPJ

APPENDIX B: LABORATORY TEST PROGRAM

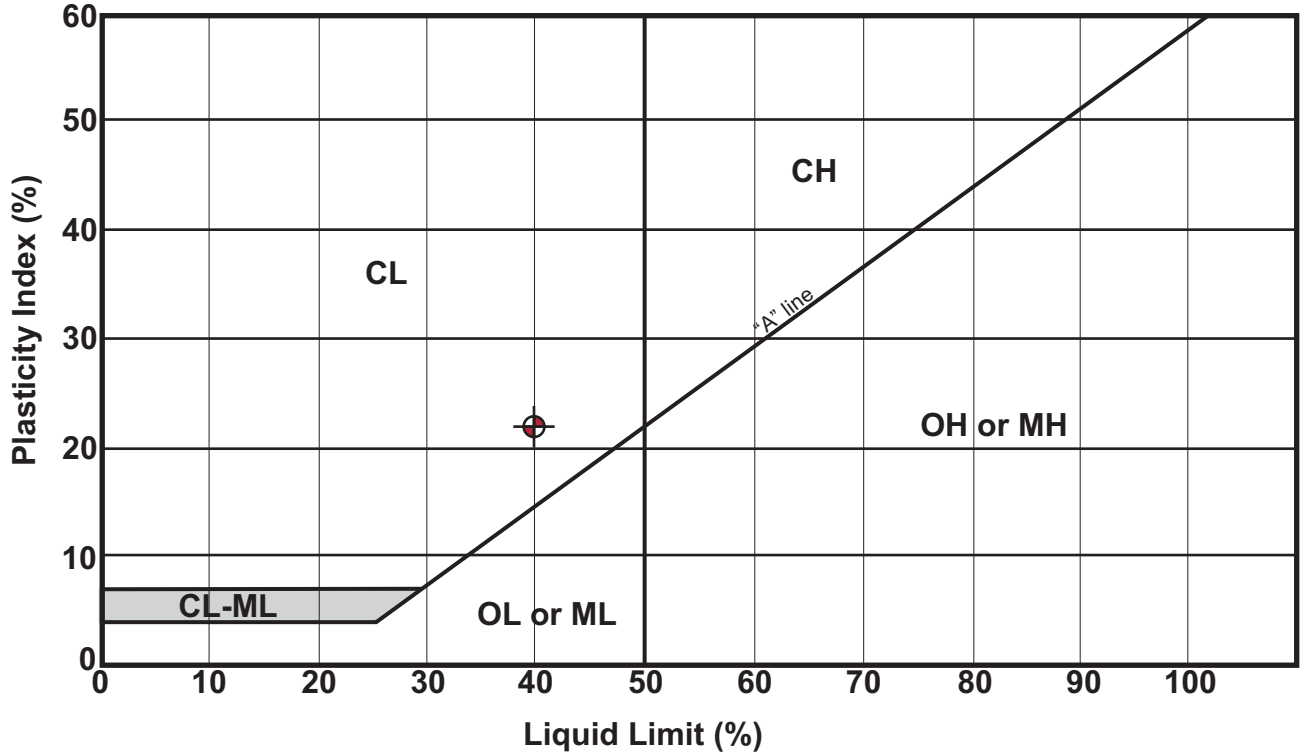
The laboratory testing program was performed to evaluate the physical and mechanical properties of the soils retrieved from the site to aid in verifying soil classification.

Moisture Content: The natural water content was determined (ASTM D2216) on 8 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities: In place dry density determinations (ASTM D2937) were performed on 4 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Plasticity Index: One Plasticity Index determination (ASTM D4318) was performed on a sample of the subsurface soil to measure the range of water contents over which this material exhibits plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential. Results of this test are shown on the boring log at the appropriate sample depth.

Plasticity Index (ASTM D4318) Testing Summary



Symbol	Boring No.	Depth (ft)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index	Passing No. 200 (%)	Group Name (USCS - ASTM D2487)
⊕	EB-1	1.0	10	40	18	22	—	Clayey Sand (SC) (CL fines) [Fill]



Plasticity Index Testing Summary

Highland Estates Lot 11
San Mateo, CA

Project Number
230-1-5

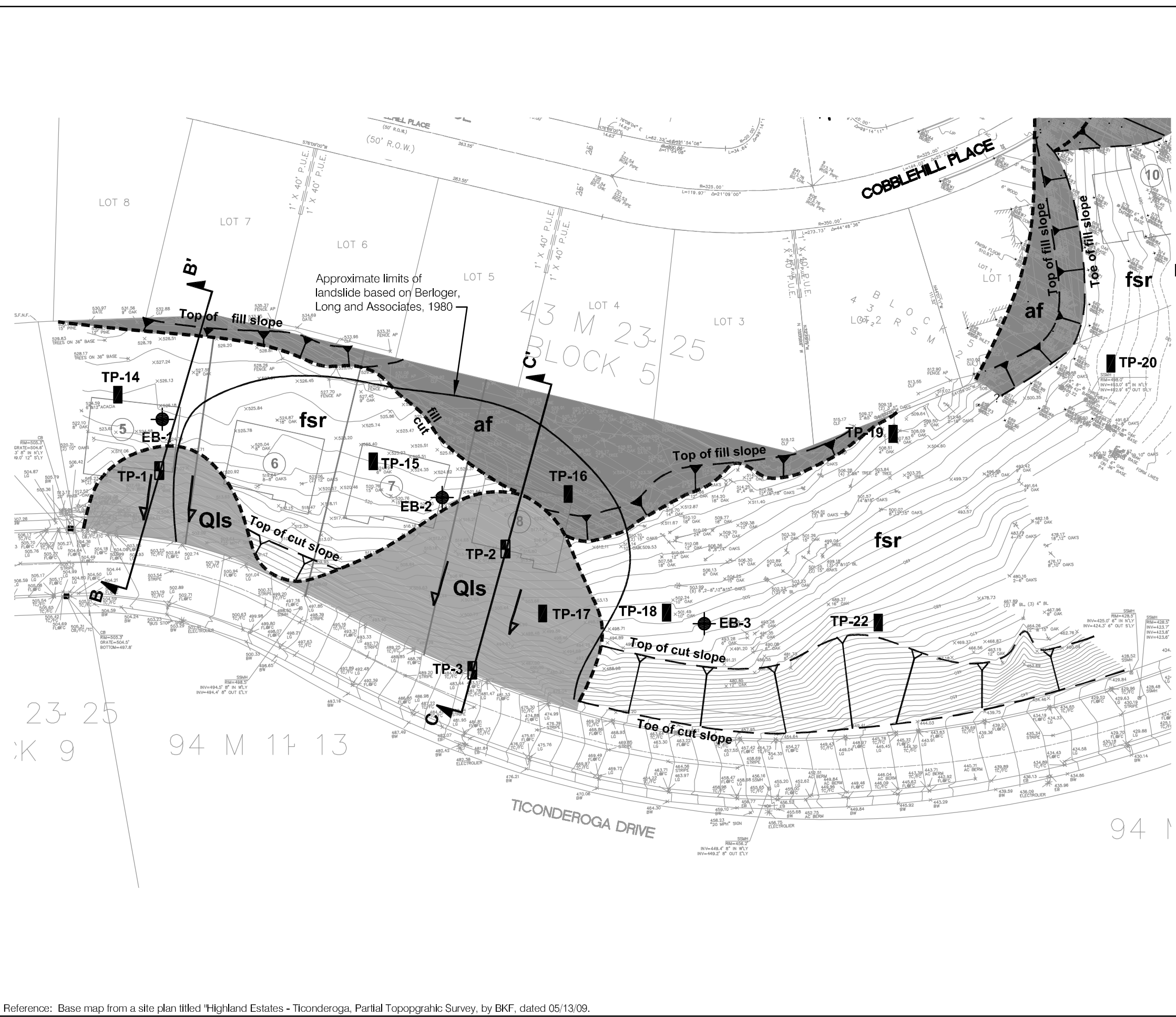
Figure Number
Figure B1

Date
August 2015

Drawn By
FLL

APPENDIX C: TREADWELL & ROLLO STABILITY ANALYSIS OUTPUT

R:\Trgraphics\4800\s\4872.02\4872.02_Site Plan and Engineering_Geo Map.dwg 8/27/09



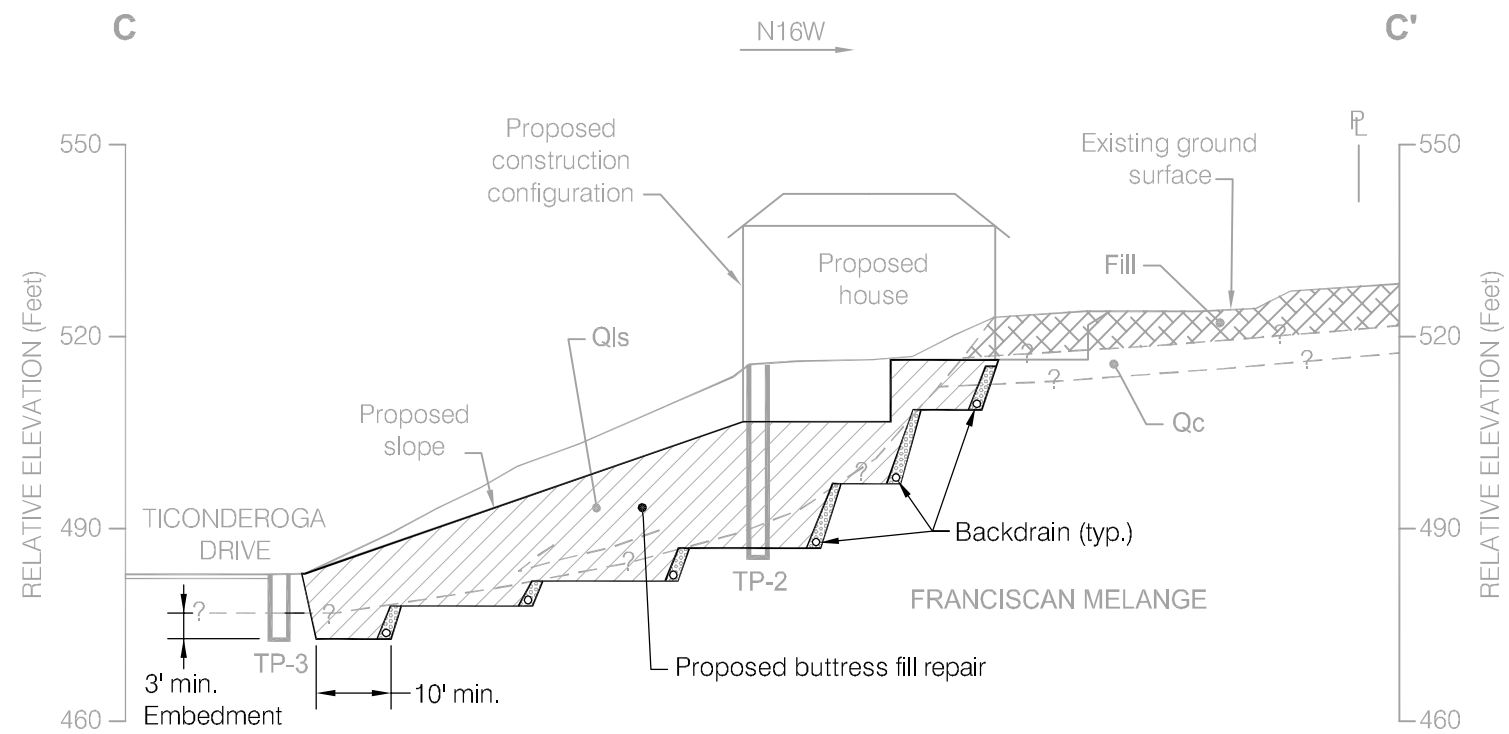
EXPLANATION

- TP-1 [Symbol] Approximate location of test pit by Treadwell & Rollo, Inc., June 2009
- EB-1 [Symbol] Approximate location of boring by Lowney, 2005
- TP-14 [Symbol] Approximate location of test pit by Berloger, Long and Associates, 1980
- [Symbol] Idealized subsurface profile
- af Artificial fill
- fsr Franciscan Melange
- Qls Landslide
- [Symbol] Cut slope
- [Symbol] Fill slope
- [Symbol] Direction of landslide
- 5 Proposed lot number

<p>HIGHLAND ESTATES San Mateo County, California</p>		
<p>SITE PLAN AND ENGINEERING GEOLOGIC MAP 2</p>		
Date 07/06/09	Project No. 4872.02	Figure 2b
<p>Treadwell & Rollo</p>		

Reference: Base map from a site plan titled "Highland Estates - Ticonderoga, Partial Topographic Survey, by BKF, dated 05/13/09.

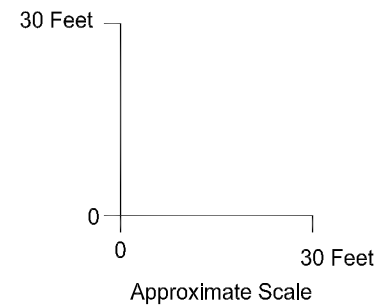
R:\Trgraphics\4800's\4872.02\4872.02 Proposed Buttress Fill Repair Subsurface Profile.dwg 8/27/09



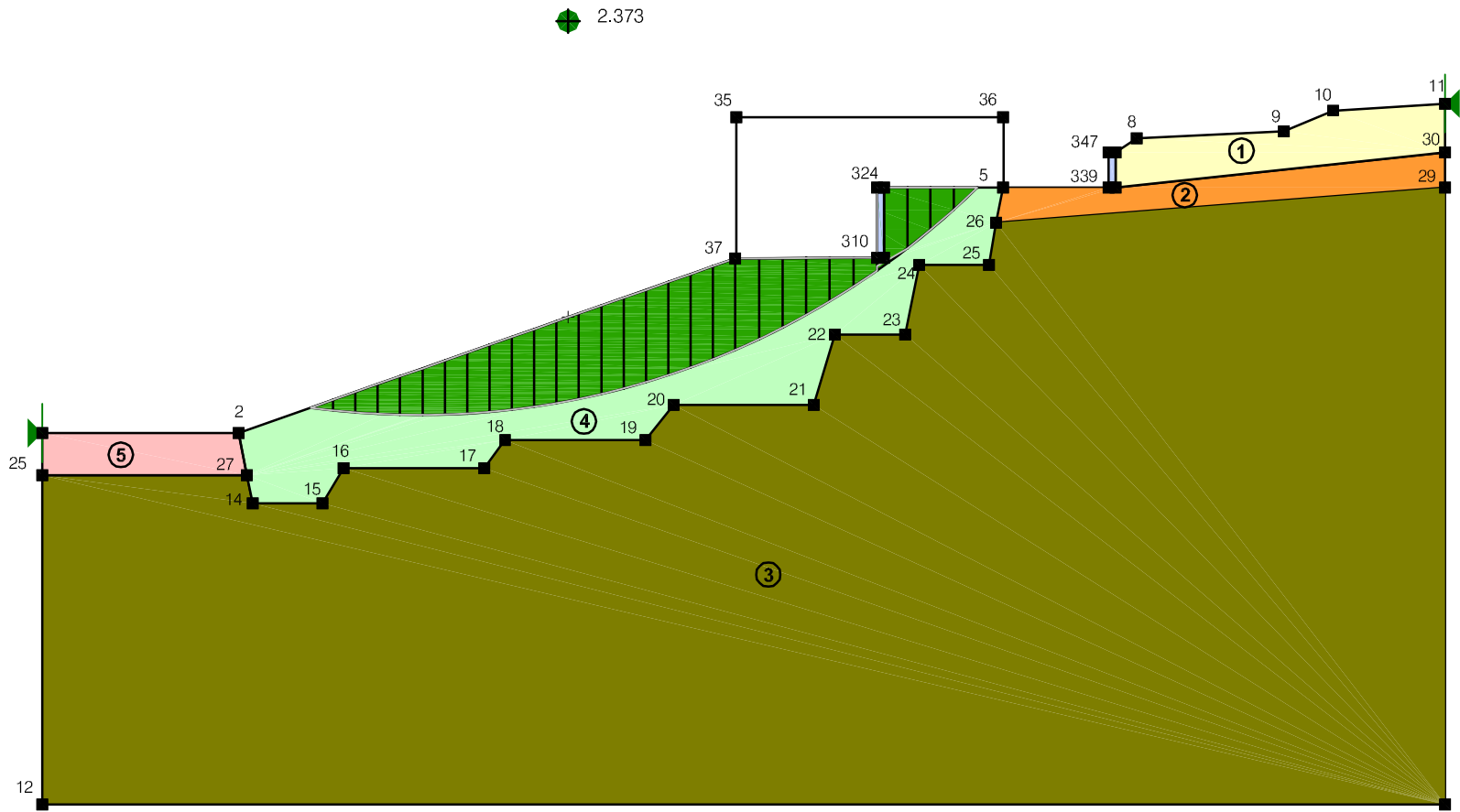
Notes:

1. The above profile represents a generalized soil cross section interpreted from widely spaced test pits and borings. Soil deposits may vary in type, strength, and other important properties between points of exploration.

2. Cross-sections based on our site reconnaissance, topographic surveys by BKF Engineers, Inc., others.



HIGHLAND ESTATES San Mateo County, California		
PROPOSED BUTTRESS FILL REPAIR SUBSURFACE PROFILE		
Date 08/18/09	Project No. 4872.02	Figure C-1
Treadwell&Rollo		



Material # 1

Description: Existing Fill
 Model: MohrCoulomb
 Wt: 110
 Cohesion: 500
 Phi: 26

Material # 2

Description: Colluvium
 Model: MohrCoulomb
 Wt: 120
 Cohesion: 700
 Phi: 22

Material # 3

Description: Franciscan Melange
 Model: MohrCoulomb
 Wt: 135
 Cohesion: 800
 Phi: 22

Material # 4

Description: Proposed Buttress Fill
 Model: MohrCoulomb
 Wt: 124
 Cohesion: 60
 Phi: 32.3

Material # 5

Description: Landslide Deposits
 Model: MohrCoulomb
 Wt: 110
 Cohesion: 700
 Phi: 11

HIGHLAND ESTATES
 San Mateo County, California

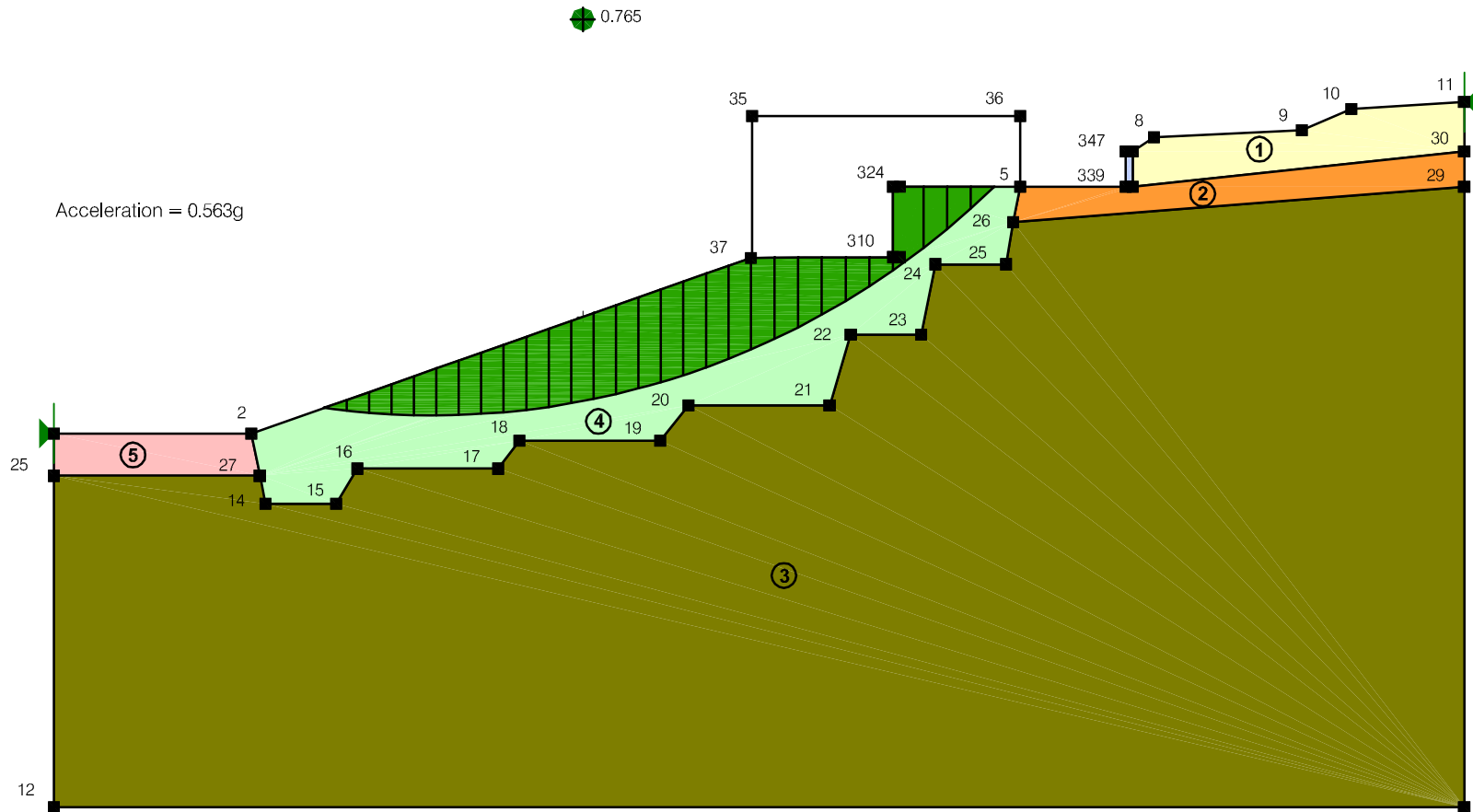
Treadwell & Rollo

STATIC SLOPE STABILITY ANALYSIS

Date 08/21/09

Project No. 4872.02

Figure C-2



Material # 1

Description: Existing Fill
 Model: MohrCoulomb
 Wt: 110
 Cohesion: 500
 Phi: 26

Material # 2

Description: Colluvium
 Model: MohrCoulomb
 Wt: 120
 Cohesion: 700
 Phi: 22

Material # 3

Description: Franciscan Melange
 Model: MohrCoulomb
 Wt: 135
 Cohesion: 800
 Phi: 22

Material # 4

Description: Proposed Buttress Fill
 Model: MohrCoulomb
 Wt: 124
 Cohesion: 60
 Phi: 32.3

Material # 5

Description: Landslide Deposits
 Model: MohrCoulomb
 Wt: 110
 Cohesion: 700
 Phi: 11

HIGHLAND ESTATES
 San Mateo County, California

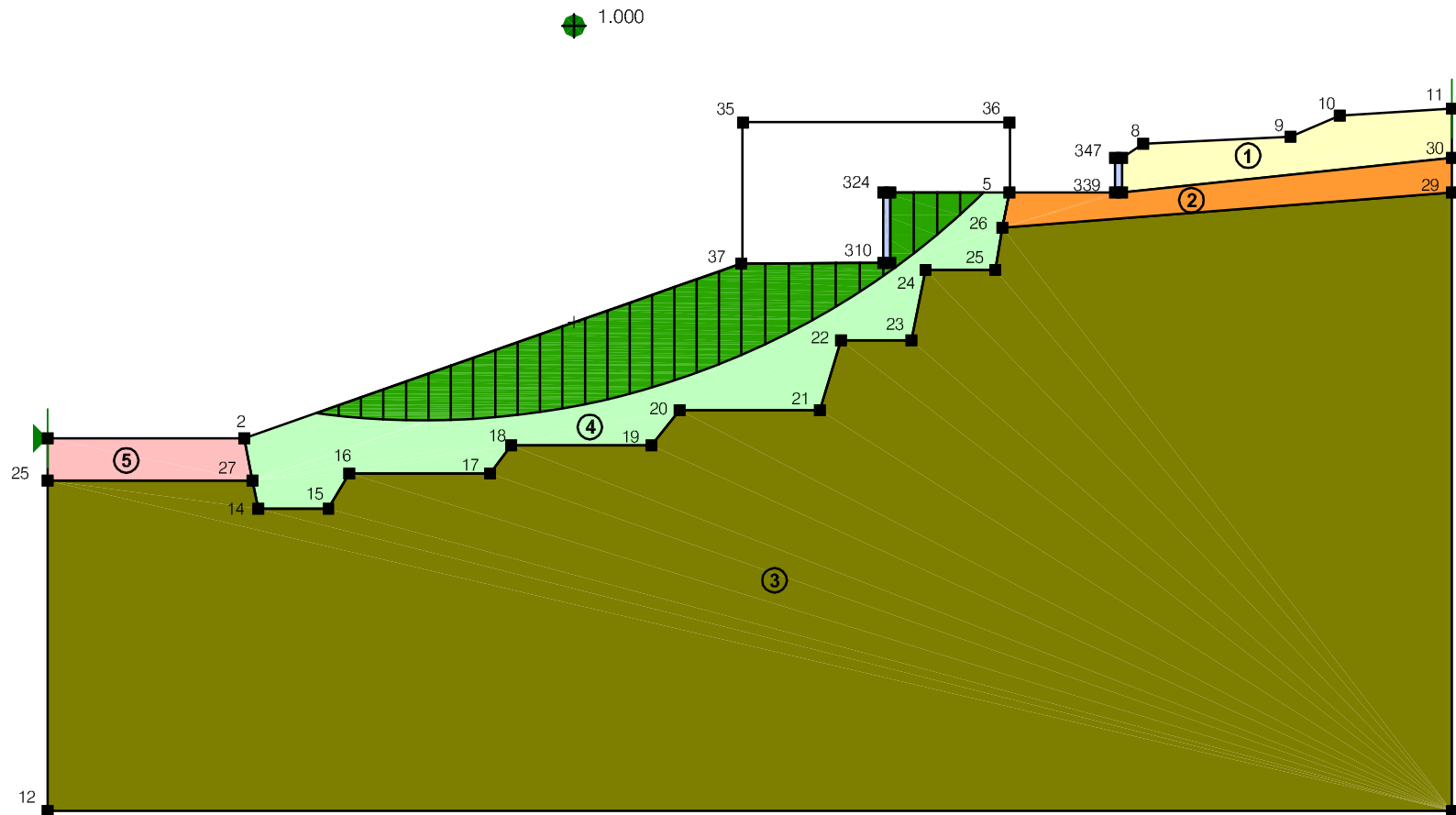
Treadwell & Rollo

**PSEUDO-STATIC
 SLOPE STABILITY ANALYSIS**

Date 08/21/09

Project No. 4872.02

Figure C-3



Material # 1

Description: Existing Fill
 Model: MohrCoulomb
 Wt: 110
 Cohesion: 500
 Phi: 26

Material # 2

Description: Colluvium
 Model: MohrCoulomb
 Wt: 120
 Cohesion: 700
 Phi: 22

Material # 3

Description: Franciscan Melange
 Model: MohrCoulomb
 Wt: 135
 Cohesion: 800
 Phi: 22

Material # 4

Description: Proposed Buttress Fill
 Model: MohrCoulomb
 Wt: 124
 Cohesion: 60
 Phi: 32.3

Material # 5

Description: Landslide Deposits
 Model: MohrCoulomb
 Wt: 110
 Cohesion: 700
 Phi: 11

HIGHLAND ESTATES
 San Mateo County, California

Treadwell & Rollo

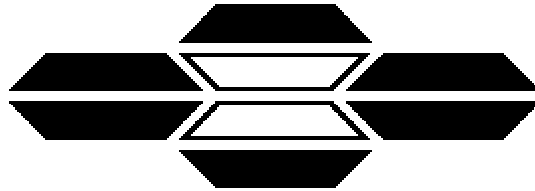
YIELD SLOPE STABILITY ANALYSIS

Date 08/21/09

Project No. 4872.02

Figure C-4

APPENDIX D: SITE ASBESTOS EVALUATION



ASBESTOS TEM LABORATORIES, INC.

**CARB Method 435
Polarized Light Microscopy
Analytical Report**

Laboratory Job # 1206-00077

630 Bancroft Way
Berkeley, CA 94710
(510) 704-8930
FAX (510) 704-8429



ASBESTOS TEM LABORATORIES, INC

CA DPH ELAP
Lab No. 1866



NVLAP Lab Code: 101891-0
Berkeley, CA

Oct/05/2015

Matt Schaffer
Cornerstone Earth Group, Inc.
1259 Oakmead Parkway
Sunnyvale, CA 94085

RE: LABORATORY JOB # 1206-00077
Polarized light microscopy analytical results for 1 bulk sample(s).
Job Site: 230-1-5
Job No.: Highland Estates Lots 5-11

Enclosed please find the bulk material analytical results for one or more samples submitted for asbestos analysis. The analyses were performed in accordance with the California Air Resources Board (ARB) Method 435 for the determination of asbestos in serpentine aggregate samples.

Prior to analysis, samples are logged-in and all data pertinent to the sample recorded. The samples are checked for damage or disruption of any chain-of-custody seals. A unique laboratory ID number is assigned to each sample. A hard copy log-in sheet containing all pertinent information concerning the sample is generated. This and all other relevant paper work are kept with the sample throughout the analytical procedures to assure proper analysis.

Sample preparation follows a standard CARB 435 prep method. The entire sample is dried at 135-150 C and then crushed to ~3/8" gravel size using a Bico Chipmunk crusher. If the submitted sample is >1 pint, the sample was split using a 1/2" riffle splitter following ASTM Method C-702-98 to obtain a 1 pint aliquot. The entire 1 pint aliquot, or entire original sample, is then pulverized in a Bico Braun disc pulverizer calibrated to produce a nominal 200 mesh final product. If necessary, additional homogenization steps are undertaken using a 3/8" riffle splitter. Small aliquots are collected from throughout the pulverized material to create three separate microscope slide mounts containing the appropriate refractive index oil. The prepared slides are placed under a polarizing light microscope where standard mineralogical techniques are used to analyze the various materials present, including asbestos. If asbestos is identified and of less than 10% concentration by visual area estimate then an additional five sample mounts are prepared. Quantification of asbestos concentration is obtained using the standard CAL ARB Method 435 point count protocol. For samples observed to contain visible asbestos of less than 10% concentration, a point counting technique is used with 50 points counted on each of eight sample mounts for a total of 400 points. The data is then compiled into standard report format and subjected to a thorough quality assurance check before the information is released to the client.

While the CARB 435 method has much to commend it, there are a number of situations where it fails to provide sufficient accuracy to make a definitive determination of the presence/absence of asbestos and/or an accurate count of the asbestos concentration present in a given sample. These problems include, but are not limited to, 1) statistical uncertainty with samples containing <1% asbestos when too few particles are counted, 2) definitive identification and discrimination between various fibrous amphibole minerals such as tremolite/actinolite/hornblende and the "Libby amphiboles" such as tremolite/winchite/richterite/arfvedsonite, and C) small asbestiform fibers which are near or below the resolution limit of the PLM microscope such as those found in various California coast range serpentine bodies. In these cases, further analysis by transmission electron microscopy is recommended to obtain a more accurate result.

Sincerely Yours,

Lab Manager
ASBESTOS TEM LABORATORIES, INC.

--- These results relate only to the samples tested and must not be reproduced, except in full, without the approval of the laboratory. ---

630 BANCROFT WAY • BERKELEY, CA 94710 • PH. (510) 704-8930 • FAX (510) 704-8429

With Branch Offices Located At: 1350 FREEPORT BLVD. UNIT 104, SPARKS, NV 89431

POLARIZED LIGHT MICROSCOPY CARB 435 ANALYTICAL REPORT

Contact: Matt Schaffer	Samples Submitted: 1	Report No. 336724
Address: Cornerstone Earth Group, Inc. 1259 Oakmead Parkway Sunnyvale, CA 94085	Samples Analyzed: 1	Date Submitted: Sep-30-15
	Job Site / No. Highland Estates Lots 5-11 230-1-5	Date Reported: Oct-05-15

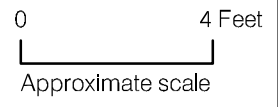
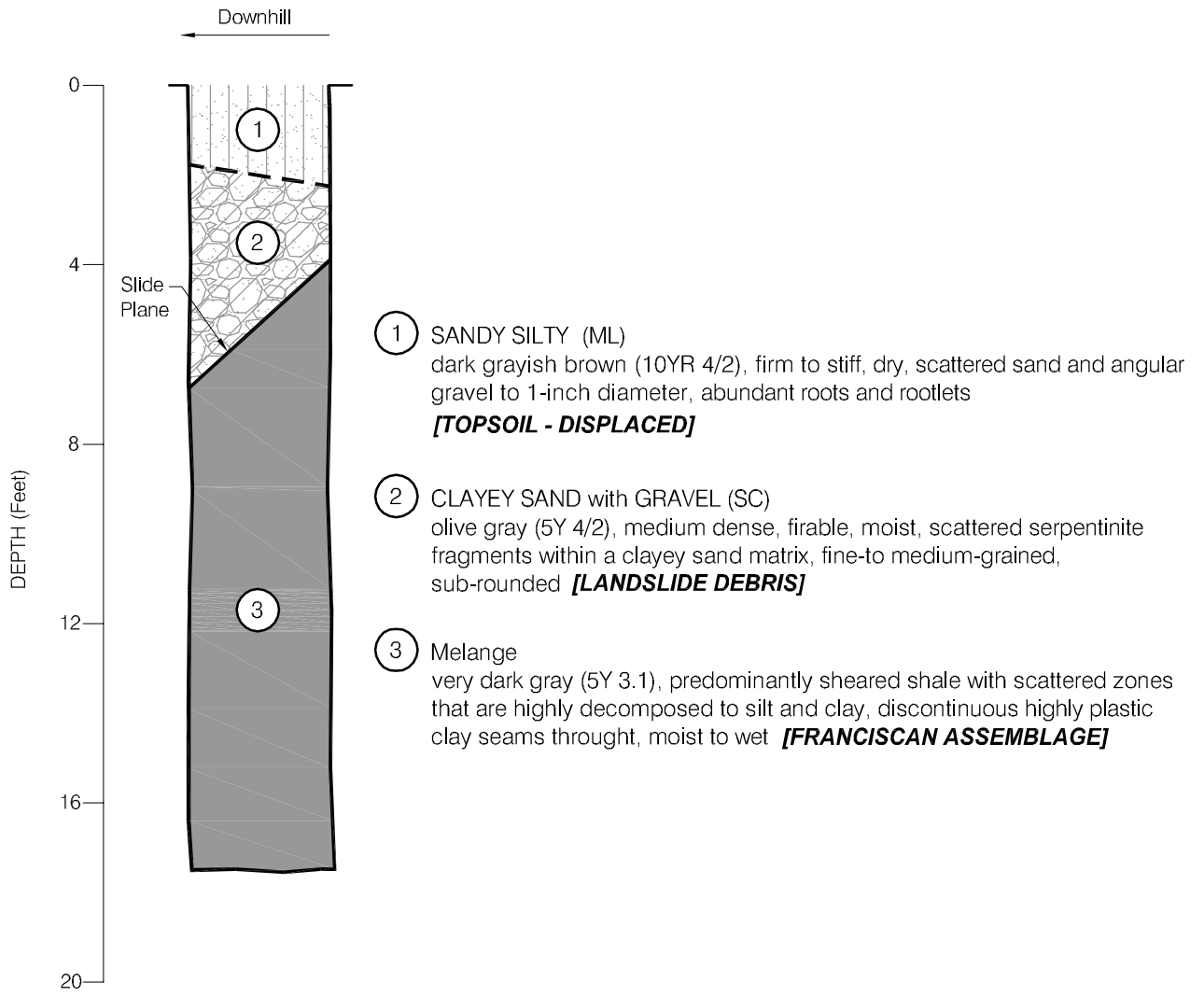
SAMPLE ID	POINTS COUNTED	ASBESTOS %	TYPE	LOCATION / DESCRIPTION
EB-1 (8.5-15)		<0.25%	None Detected	Soil/Bedrock
Lab ID # 1206-00077-001	400 - Total Points			No Asbestos Detected - ARB Exception I
Lab ID #	- Total Points			
Lab ID #	- Total Points			
Lab ID #	- Total Points			
Lab ID #	- Total Points			
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Lab ID #	- Total Points			
Lab ID #	- Total Points			

QC Reviewer *R. Mc. B...*

Analyst *Jo Ann H...*

APPENDIX E: SELECTED PREVIOUS INVESTIGATION BORING AND TEST PIT LOGS

R:\Trgraphics\4800's\4872.02\4872.02 Log of Test Pits 1 to 3.dwg 8/27/09

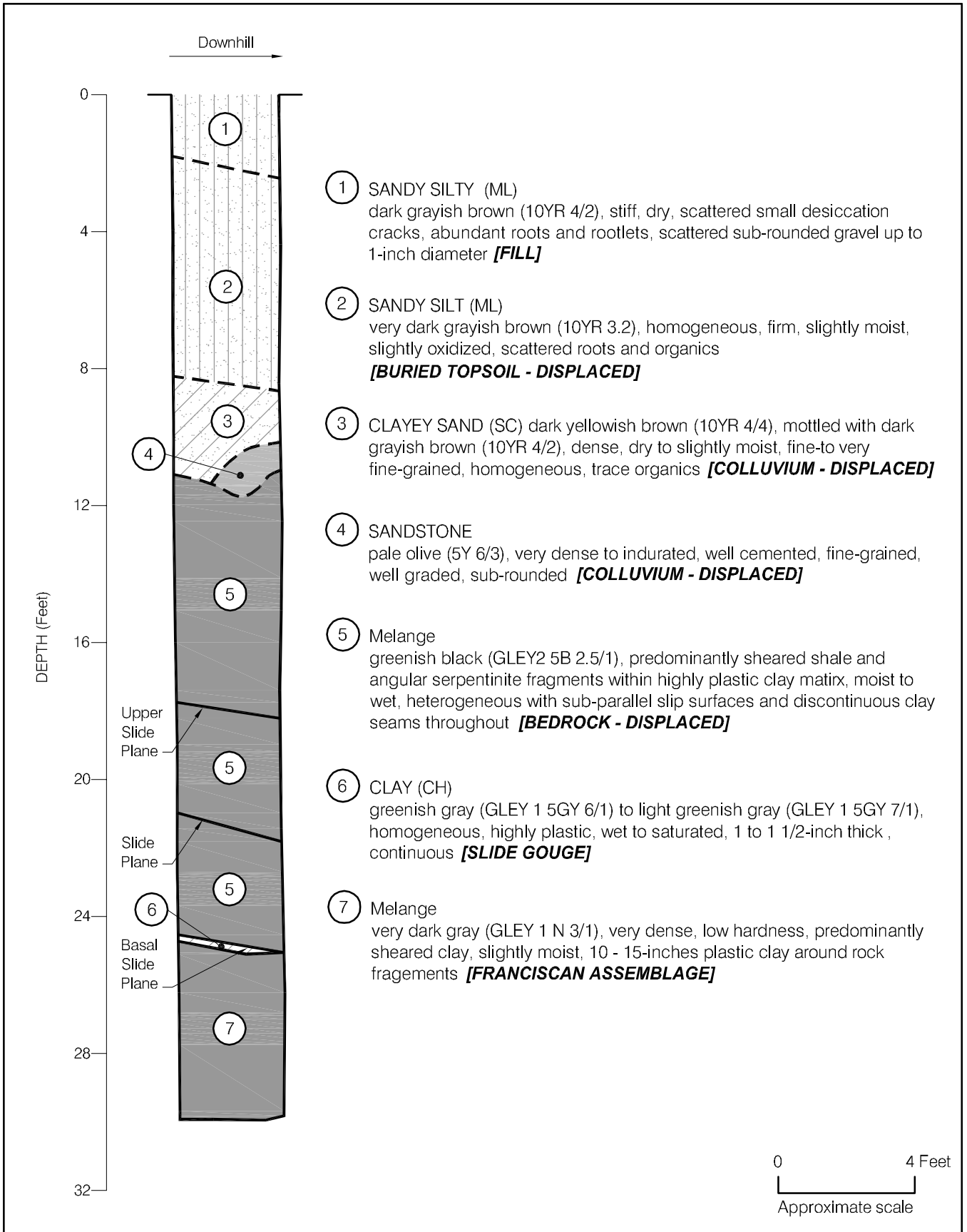


HIGHLAND ESTATES
San Mateo County, California

**LOG OF TEST PIT
TP-1**



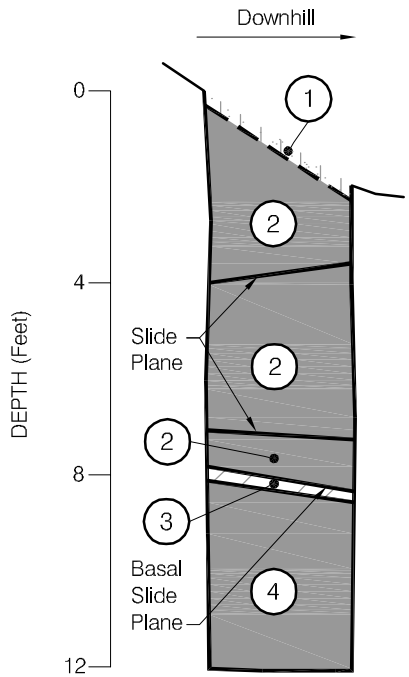
Date 08/25/09	Project No. 4872.02	Figure A-1
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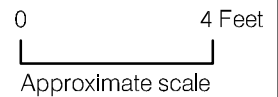
R:\Trgraphics\4800's\4872.02\4872.02 Log of Test Pits 1 to 3.dwg 8/27/09

HIGHLAND ESTATES San Mateo County, California	LOG OF TEST PIT TP-2		
	Date 08/25/09	Project No. 4872.02	Figure A-2

R:\Trgraphics\4800's\4872.02\4872.02 Log of Test Pits 1 to 3.dwg 8/27/09



- ① SANDY SILT (ML)
very dark grayish brown (10YR 3.2), homogeneous, firm, slightly moist, slightly oxidized, scattered roots and organics **[TOPSOIL - DISPLACED]**
- ② Melange
greenish black (GLE Y2 5B 2.5/1), predominantly sheared shale and angular serpentinite fragments within highly plastic clay matrix, moist to wet, heterogeneous with sub-parallel slip surfaces and discontinuous clay seams throughout **[BEDROCK - DISPLACED]**
- ③ CLAY (CH)
greenish gray (GLE Y 1 5GY 6/1) to light greenish gray (GLE Y 1 5GY 7/1), homogeneous, highly plastic, wet to saturated, 1 to 1 1/2-inch thick, continuous **[SLIDE GOUGE]**
- ④ Melange
very dark gray (GLE Y 1 N 3/1), very dense, low hardness, predominantly sheared clay, slightly moist, 10 - 15-inches plastic clay around rock fragments **[FRANCISCAN ASSEMBLAGE]**



HIGHLAND ESTATES
San Mateo County, California

LOG OF TEST PIT TP-3



Date 08/06/09	Project No. 4872.02	Figure A-3
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EXPLORATORY BORING: EB-1

Sheet 1 of 1

DRILL RIG: MINUTE MAN

BORING TYPE: 4 INCH FLIGHT AUGER

LOGGED BY: BM

START DATE: 3-9-05 FINISH DATE: 3-9-05

PROJECT NO: 1291-2B

PROJECT: TICONDEROGA DRIVE

LOCATION: SAN MATEO, CA

COMPLETION DEPTH: 20.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (kst)
525.0	0		SURFACE ELEVATION: 525 FT. (+/-)							
			LEAN CLAY WITH SAND (CL) [COLLUVIUM] very stiff, moist, brown with reddish brown mottles, fine sand, some fine and coarse gravel, low plasticity	CL	17	◆	14	101		○
519.8	5		LEAN CLAY (CL) [COLLUVIUM] medium stiff to stiff, moist, gray, some fine and coarse gravel, moderate plasticity	CL	14	◆	21	100		○
517.0	10		SANDSTONE [FRANCISCAN FORMATION (fsr)] moderately to severely weathered, very soft, olive to brown	fsr	18	◆	11	127		
	15			fsr	27	◆	6			
	20		completely weathered, soft with hard seams, gray with bluish gray mottles		32	◆	25	93		
505.0	20		Bottom of Boring at 20 feet							
	25									
	30									

Undrained Shear Strength (kst)
 ○ Pocket Penetrometer
 △ Torvane
 ● Unconfined Compression
 ▲ U-U Triaxial Compression

LA CORP GDT 2/2/06 MW* ELL

GROUND WATER OBSERVATIONS:
NO FREE GROUND WATER ENCOUNTERED

EXPLORATORY BORING: EB-2

Sheet 1 of 1

DRILL RIG: MINUTE MAN

BORING TYPE: 4 INCH FLIGHT AUGER

LOGGED BY: BM

START DATE: 3-8-05

FINISH DATE: 3-8-05

PROJECT NO: 1291-2B

PROJECT: TICONDEROGA DRIVE

LOCATION: SAN MATEO, CA

COMPLETION DEPTH: 20.0 FT.

This log is a part of a report by TRC Lowney, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksi)
524.0	0		SURFACE ELEVATION: 524 FT. (+/-)							
			LEAN CLAY WITH SAND (CL) [LANDSLIDE DEPOSIT] stiff, moist to wet, brown, fine sand, some fine and coarse gravel, low plasticity	CL	8	X	21	90		○
	5				9	X	22			○
517.0			SANDSTONE [FRANCISCAN FORMATION (fsr)] moderately to severely weathered, soft, dark brown, friable, some clay seams	fsr	50/6"	X	5	113		
	10				24	X	7			
	15				19	X				
	20		Plasticity Index = 13, Liquid Limit = 29		47	X	10			
504.0	20		Bottom of Boring at 20 feet							

GROUND WATER OBSERVATIONS:
NO FREE GROUND WATER ENCOUNTERED

LA CORP GDT 2/5/05 MW* FLL

EXPLORATORY BORING: EB-3

Sheet 1 of 1

DRILL RIG: MINUTE MAN
 BORING TYPE: 4 INCH FLIGHT AUGER
 LOGGED BY: BM
 START DATE: 3-8-05 FINISH DATE: 3-8-05

PROJECT NO: 1291-2B
 PROJECT: TICONDEROGA DRIVE
 LOCATION: SAN MATEO, CA
 COMPLETION DEPTH: 20.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE	Undrained Shear Strength (ksf)
500.0	0		SURFACE ELEVATION: 500 FT. (+/-)							
			LEAN CLAY WITH SAND (CL) [LANDSLIDE DEPOSIT] medium stiff, moist to wet, brown, fine sand, some fine and coarse gravel, trace organics, low plasticity	CL	13	X	20	100		○
496.5	5		SANDY LEAN CLAY (CL) [COLLUVIUM] very stiff, moist, gray, fine to coarse sand, some fine and coarse gravel, low plasticity	CL	24	X	13	113		○
491.5	10		SANDSTONE [FRANCISCAN FORMATION (fsr)] moderate to severely weathered, soft, dark brown, friable, some fine sand	fsr	50/3"	X	6	124		
	15		increasing clay		49	X				
			abruptly severely weathered silty yellowish olive graywacke		59	X	8			
					32	X				
480.0	20		Bottom of Boring at 20 feet		55	X	3			
	25									
	30									

GROUND WATER OBSERVATIONS:
 NO FREE GROUND WATER ENCOUNTERED

LA CORP.GDT. 2/2/08 MV-FLL

EXPLORATORY BORING LOG					LABORATORY TESTS									
Sample Number	Depth, feet	Boring Log	Unified Soil Classification System Symbols	Description	Standard Penetration Test, blows/foot	Moisture Content, %	Dry Density, p. c. f.	Unconfined Compressive Strength, k. s. f.	Direct Shear Test		Liquid Limit, %	Plasticity Index, %	Triaxial Compression	
									"C", k. s. f.	"φ", degrees				
Boring No. B-6					Date of Drilling: 7/20/92									
6-1	2	X		Fill, clayey Silt, dark brown-gray	* 43	16.5	119.4	1.2	15					
	4			(Q _{af})										
6-2	6	X		mottled with med. gray clay & SS inclusions	* 51	11.1	123.2						x	
	8			Shale/Siltstone clay gouge, purple tinted gray-brown cuttings, damp, hard										
	10			Claystone, sandy, purple blue, becoming blue-gray (F _m)										
	12			becoming blocky, slightly weathered										
	14													
	16													
Bottom at 17 feet														
Boring No. B-7					Date of Drilling: 7/20/92									
7-1	2	X		Colluvium, sandy clayey Silt, tan-brown, sl. damp, loose	* 50/5"	11.5	106.3							
	4			Siltstone, moderately weathered, pale yellowish brown, sheared, damp, soft (stiff clayey Silt)										
	6			(F _m)										
	8			Graywacke Sandstone, sl. weathered, sl. damp, hard (S _s)										
	10													
	12													
	14													
Bottom at 15 feet														

Plate 9 - Logs of Test Borings: B-6 & B-7

SOIL FOUNDATION SYSTEMS, INC.







EXPLORATORY BORING LOG					LABORATORY TESTS									
Sample Number	Depth, feet	Boring Log	Unified Soil Classification System Symbols	Description	Standard Penetration Test, blows/foot	Moisture Content, %	Dry Density, p. c. f.	Unconfined Compressive Strength, k. s. f.	Direct Shear Test		Liquid Limit, %	Plasticity Index, %		
									"C", k. s. f.	"φ", degrees				
Boring No. B-14					Date of Drilling: 7/30/92									
14-1	2			(W/L: 17:30, 7/30/92) Silty Clay, black sheared Serpentine, sev. weathered, blue-gray, moist (poor sample)	**18	13.4	-				34	12		
	4			(Sp)										
	6													
	8					Clay gouge zone								
14-2	10					sheared Serpentine, sl. damp	**27	11.8						
	12					(Sp)								
	14					massive, blocky								
	16					Note: ** denotes penetration resistance of Standard penentrometer driven with a 70-pound hammer dropping a distance of 30 inches.								
	18					Note: this hole drilled with portable rig								
						Bottom at 19 feet								
Boring No. B-15					Date of Drilling: 7/24/92									
	2					clayey Silt, dark brown to black, damp, sl. organic								
	4					Serpentine, very severely to severely weathered, silty Clay, pale green- gray, with angular fragments (Sp)								
	6					very hard at 7'								
						Bottom at 8 feet								
						Note: this hole drilled with portable rig								

Plate 14 - Logs of Test Borings: B-14 & B-15

SOIL FOUNDATION SYSTEMS, INC.

EXPLORATORY BORING LOG					LABORATORY TESTS							
Sample Number	Depth, feet	Boring Log	Unified Soil Classification System Symbols	Description	Standard Penetration Test, blows/foot	Moisture Content, %	Dry Density, p. c. f.	Unconfined Compressive Strength, k. s. f.	Direct Shear Test		Liquid Limit, %	Plasticity Index, %
									"C", k. s. f.	"φ", degrees		
Boring No. B-16					Date of Drilling: 7/30/92							
	2			silty Clay, dark brown								
	4			▼ sl. organic = (W/L: 11:00; 7/30/92)								
16-1	6			sheared Serpentine, very severely weathered, silty Clay with serpentine fragments, pale green-gray	*** 10	28.6	95.9		0.4	25		
	8											
	10			(Sp)								
16-2	12				*** 22	11.8	107.3		0.5	38		
	14											
	16			becoming hard								
16-3	18			(Sample not recovered)***	18							
				Bottom at 20 feet								
				Note: *** denotes penetration resistance of 2-½ I.D. sampler driven with a 70-pound hammer dropping a distance of 30 inches.								
				Note: This hole drilled with a portable rig								

July 20, 1993

EXPLORATORY BORING LOG					LABORATORY TESTS								
Sample Number	Depth, feet	Boring Log	Unified Soil Classification System Symbols	Description	Standard Penetration Test, blows/foot	Moisture Content, %	Dry Density, p. c. f.	Unconfined Compressive Strength, k. s. f.	Direct Shear Test		Liquid Limit, %	Plasticity Index, %	Triaxial Compression
									"C", k. s. f.	"φ", degrees			
Boring No. B-17					Date of Drilling: 7/20/92								
	4			Fill, silty Clay, dark brown to green-black (W/L: 16:00; 7/20/92)									
17-1	8			Serpentine, very severely weathered (W/L: 10:00; 7/20/92)	* 83	11.6	126.1						x
17-2	12			sheared Serpentine, mod. weathered, blue-gray, moist	* 25	11.8	132.5						x
17-3	16			(Sp)	* 48	9.5	129.9			20	4		x
17-4	28			Clay gouge zone	* 40	11.4	120.5						
17-5	36			sheared Serpentine, sl. weathered, dark gray, sl. damp (stiff silty Clay with serpentine fragments)	* 100	10.4	122.3						
Bottom at 42 feet													
<p>Note: * denotes penetration resistance of 2 1/2-inch I.D. sampler driven with a 140-pound hammer dropping a distance of 30 inches.</p>													

Job No. 805-10

TEST PIT LOGS

<u>Test Pit Number</u>	<u>Depth (ft.)</u>	<u>Description</u>
TP-1	0-6	Fill: heterogeneous mixture of sandy clay and gravelly clay, brown and light brown, damp, medium stiff, (W < PL), some gravels to 6" across; a 5/8" diameter cable at 4'; base marked by 2" to 3" brown organic material.
	6-7 1/4	Soil: sandy clay, brown, damp (W < PL), medium to low plasticity, soft in upper few inches, then medium stiff, minor angular fragments of sandstone.
	7 1/4-11	Subsoil: gravelly clay, brown, damp (W < PL), medium to low plasticity, gravels > 4" across comprise approximately 50 percent of this material, and percentage increasing with depth to possible bedrock at the bottom of the test pit. Total depth 11 feet; no free groundwater.
TP-2	0-1 1/4	Soil: sandy clay, brown, damp (W < PL), medium to low plasticity, soft in upper few inches, then medium stiff, minor angular fragments of sandstone.
	1 1/4-3	Subsoil: sandy clay with gravel, light brown, damp (W < PL), medium plasticity; increasing gravels with depth, fragments of sandstone commonly 1' to 3' across. Total depth 3 feet; no free groundwater.
	0-1 1/4	Soil: sandy clay, brown, damp (W < PL), medium to low plasticity, soft in upper few inches, then medium stiff, minor angular fragments of sandstone.
TP-3	0-1 1/4	Soil: sandy clay, brown, damp (W < PL), medium to low plasticity, soft in upper few inches, then medium stiff, minor angular fragments of sandstone.
	1 1/4-6 1/4	Subsoil: sandy clay with gravel, grading to gravelly clay or bedrock at depth, light brown, damp (W < PL), fragments of sandstone generally < 3" across. Total depth 6 1/4'; no free groundwater.

Job No. 805-10

TEST PIT LOGS

<u>Test Pit Number</u>	<u>Depth (ft.)</u>	<u>Description</u>
TP-4	0-3 1/4	Soil: sandy clay, brown, slightly damp (W < PL), medium to low plasticity, soft, with gravelly clay 3" thick at the base; contact with underlying subsoil approximately 25' downhill, no shearing observed.
	3 1/4-6	Subsoil: sandy clay with gravel, grading to gravelly clay or bedrock at depth, light brown, damp (W < PL), fragments of sandstone generally < 3" across. Total depth 6 feet; no free groundwater.
	0-2	Soil: sandy clay, brown, damp (W < PL), medium to low plasticity, soft in upper few inches, then medium stiff, minor angular fragments of sandstone.
TP-5	2-4 1/4	Subsoil: sandy clay to gravelly clay, light brown, slightly damp (W < PL), gravels generally < 3" across.
	4 1/4-5	Bedrock: sandstone, fine- to medium-grained, light gray to light brown, micaceous, massive, very well indurated; generally breaks into pieces 6" to 3'. Total depth 5 feet; no free groundwater.
TP-6	0-4 1/4	Soil: silty gravel, dark brown, moist (W > PL), fragments of sandstone generally 6" across; very hard digging. Total depth 4 1/4 feet; no free groundwater.

TEST PIT LOGS

<u>Test Pit Number</u>	<u>Depth (ft.)</u>	<u>Description</u>
TP-7	0-6½	Soil: sandy clay, brown, damp (W<PL), medium to low plasticity, soft in upper few inches, then medium stiff, minor angular fragments of sandstone.
	6½-10½	Talus: sandy gravel with minor clay, light brown, fragments of sandstone 6" to 1' across in sandy matrix, generally loose.
	10½-12	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 1" across. Total depth 12 feet; no free groundwater.
TP-8	0-2	Soil: sandy clay, brown, damp (W<PL), medium to low plasticity, soft in upper few inches, then medium stiff, minor angular fragments of sandstone.
	2-5	Colluvium?: clayey sand, brownish orange, damp (W<PL), friable.
	5-7½	Landslide shear zone?: clay to sandy clay, dark gray, moist (W>PL), stiff, high plasticity.
	7½-10½	Colluvium?: clayey sand as above between 2 and 5 feet. Total depth 10½'; no free groundwater.
TP-9	0-2	Soil: sandy clay, brown, damp (W<PL), medium to low plasticity, soft in upper few inches, then medium stiff, minor angular fragments of sandstone.
	2-5	Colluvium?: clayey sand, brownish orange, damp (W<PL), friable.
	5-8	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 1" across. Total depth 8'; no free groundwater.

TEST PIT LOGS

<u>Test Pit Number</u>	<u>Depth (ft.)</u>	<u>Description</u>
TP-10	0-1½	Soil: sandy clay, brown, damp (W<PL), medium to low plasticity, soft in upper few inches, then medium stiff, minor angular fragments of sandstone.
	1½-4	Bedrock: sandstone, fine- to medium-grained, light gray to light brown, micaceous, massive, very well indurated; generally breaks into pieces 6" to 3'. Total depth 4'; no free groundwater.
TP-11	0-1	Soil: sandy clay, brown, damp (W<PL), medium to low plasticity, soft in upper few inches, then medium stiff, minor angular fragments of sandstone.
	1-3¾	Bedrock: sandstone, fine- to medium-grained, light gray to light brown, micaceous, massive, very well indurated; generally breaks into pieces 6" to 3'. Total depth 3¾'; no free groundwater.
TP-12	0-2	Soil: sandy clay, brown, damp (W<PL), medium to low plasticity, soft in upper few inches, then medium stiff, minor angular fragments of sandstone.
	2-4	Bedrock: sandstone, fine- to medium-grained, light gray to light brown, micaceous, massive, very well indurated; generally breaks into pieces 6" to 3'. Total depth 4'; no free groundwater.

Job No. 805-10

TEST PIT LOGS

<u>Test Pit Number</u>	<u>Depth (ft.)</u>	<u>Description</u>
TP-13	0-2	Soil: sandy clay, brown, damp (M<PL), medium to low plasticity, soft in upper few inches, then medium stiff, minor angular fragments of sandstone.
	2-3½	Bedrock: sandstone, fine- to medium-grained, light gray to light brown, micaceous, massive, very well indurated; generally breaks into pieces 6" to 2". Total depth 3½'; no free groundwater.
TP-14	0-4½	Soil: sandy clay, dark brown, damp (M<PL), firm to 2½', low plasticity; medium stiff below 2½'.
	4½-6	Subsoil: silty clay with minor sand, gray, damp to moist (M₂PL), medium stiff to stiff, high plasticity.
	6-7	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 2" across. Total depth 7'; no free groundwater.
TP-15	0-2½	Fill: sandy clay, mottled dark brown and reddish-brown, slightly damp (M<PL), medium stiff to stiff, medium plasticity, layered structure (horizontal).
	2½-4	Soil: sandy clay, dark brown, damp (M<PL), firm to 3½', low plasticity; medium stiff below 3½'.
	4-5½	Subsoil: silty clay with minor sand, gray, damp to moist (M₂PL), medium stiff to stiff, high plasticity.
	5½-7	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 2" across. Total depth 7'; no free groundwater.

Job No. 805-10

TEST PIT LOGS

<u>Test Pit Number</u>	<u>Depth (ft.)</u>	<u>Description</u>
TP-16	0-6	Fill: sandy clay as in TP-15; mottled light and dark brown, damp (M<PL), and soft in top 2'; dark gray to dark brown, firm to very stiff, slightly damp (M<PL) below 2'.
	6-9	Soil: sandy clay, dark brown, damp (M<PL), firm to 7', low plasticity; medium stiff below 7'.
	9-10½	Subsoil: silty clay with minor sand, gray, damp to moist (M₂PL), medium stiff to stiff, high plasticity.
	10½-11½	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 1" across. Total depth 11½'; no free groundwater.
TP-17	0-2	Soil: silty clay with minor sand, gray, damp to moist (M<PL), medium stiff to stiff, high plasticity, soft in top foot.
	2-5½	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 1" across. Damp from 2 to 3½' (M<PL), slightly damp (M₂PL) below 3½'; large block of very fractured but hard greenstone at 5'. Total depth 5½'; no free groundwater.

Job No. 805-10

TEST PIT LOGS

<u>Test Pit Number</u>	<u>Depth (ft.)</u>	<u>Description</u>
TP-18	0-14	Soil: sandy clay, brown, damp (W<PL), medium to low plasticity, soft in upper few inches, then medium stiff, minor angular fragments of sandstone.
	14-2	Subsoil: sandy clay with gravel, grading to gravelly clay or bedrock at depth, light brown, damp (W<PL), fragments of sandstone generally <3" across.
	2-5	Bedrock: sandstone, fine- to medium-grained, light gray to light brown, micaceous, massive, very well indurated; generally breaks into pieces 6" to 14".
		Total depth 5'; no free groundwater.
TP-19	0-1	Soil: sandy clay, dark brown, damp (W<PL), medium stiff, low plasticity.
	1-2	Subsoil: silty clay with minor sand, gray, damp to moist (W2PL), medium stiff to stiff, high plasticity.
	2-5	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 2" across.
		Total depth 5'; no free groundwater.
TP-20	0-14	Soil: sandy clay, dark brown, damp (W<PL), medium stiff, low plasticity.
	14-5	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 1" across.
		Total depth 5'; no free groundwater.

Job No. 805-10

TEST PIT LOGS

<u>Test Pit Number</u>	<u>Depth (ft.)</u>	<u>Description</u>
P-21	0-1	Soil: sandy clay, dark brown, damp (W<PL), medium stiff, low plasticity.
	1-2	Subsoil: silty clay with minor sand, gray, damp to moist (W2PL), medium stiff to stiff, high plasticity.
	2-64	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 3" across, damp (W<PL).
		Total depth 64'; no free groundwater.
P-22	0-24	Soil: sandy clay, dark brown, damp (W<PL), firm to 24', low plasticity; medium stiff below 24'.
	24-3	Subsoil: silty clay with minor sand, gray, damp to moist (W2PL), medium stiff to stiff, high plasticity.
	3-84	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 3" across.
		Total depth 84'; no free groundwater.
P-23	0-4	Soil: sandy clay, dark brown, damp (W<PL), firm to 24', low plasticity; medium stiff below 24'.
	4-7	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 1" across, a 6" steel pipe 54' deep headed toward a man-hole.
		Total depth 7'; no free groundwater.

-B.16-

Job No. 805-10

TEST PIT LOGS

<u>Test Pit Number</u>	<u>Depth (ft.)</u>	<u>Description</u>
TP-24	0-1	Soil: sandy clay, dark brown, damp (W<PL), medium stiff, low plasticity.
	1-6	Colluvium: sandy clay, dark brown, slightly damp (W<PL), medium stiff to stiff, medium to high plasticity.
	6-9	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 1" across. Total depth 9'; no free groundwater.
TP-25	0-4	Soil: sandy clay, dark brown, damp (W<PL), medium stiff, low plasticity.
	4-8½	Bedrock: sandstone, fine- to medium-grained, light gray to light brown, micaceous, massive, very well indurated; generally breaks into pieces 6" to 2'. Total depth 8½'; no free groundwater.
TP-26	0-4	Soil: sandy clay, dark brown, damp (W<PL), medium stiff, low plasticity.
	4-11	Colluvium: sandy clay, dark brown, slightly damp (W<PL), medium stiff to stiff, medium to high plasticity, with increasing gravel to bottom. Total depth 11'; no free groundwater.

Job No. 805-10

TEST PIT LOGS

<u>Test Pit Number</u>	<u>Depth (ft.)</u>	<u>Description</u>
TP-27	0-3	Soil: sandy clay, dark brown, damp (W<PL), firm to 2', low plasticity; medium stiff below 2'.
	3-5	Subsoil: silty clay with minor sand, gray, damp to moist (W>PL), medium stiff to stiff, high plasticity.
	5-10¼	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 1" across, moist to very moist (W>PL). Total depth 10¼'; no free groundwater.
TP-28	0-3	Soil: sandy clay, dark brown, damp (W<PL), medium stiff, low plasticity.
	3-6	Colluvium: sandy clay, dark brown, slightly damp (W<PL), medium stiff to stiff, medium to high plasticity, with dispersed gravel and layers of gravel.
	6-11	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 1" across, slightly damp (W<PL); contact with overlying colluvium is oriented downhill about 23 degrees and is distinct, no shearing observed. Total depth 11'; no free groundwater.

Job No. 805-10

TEST PIT LOGS

<u>Test Pit Number</u>	<u>Depth (ft.)</u>	<u>Description</u>
TP-29	0-2	Soil: sandy clay, dark brown, damp (W<PL), medium stiff to 1 1/4, low plasticity.
	2-5	Colluvium?: sandy clay, dark brown, slightly damp (W<PL), firm to stiff, medium to high plasticity.
	5-8 1/4	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 3" across, slightly damp (W<PL). Total depth 8 1/4'; no free groundwater.
TP-30	0-2	Soil: sandy clay, dark brown, damp (W<PL), firm to 1, low plasticity; medium stiff below 1'.
	2-3 1/4	Subsoil: sandy clay with gravel, grading to gravelly clay or bedrock at depth, light brown, damp (W<PL), fragments of sandstone generally <3" across.
	3 1/4-5 1/4	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 2" across, slightly damp (W<PL). Total depth 5 1/4'; no free groundwater.
TP-31	0-1	Soil: sandy clay, dark brown, damp (W<PL), medium stiff, low plasticity.
	1-5	Bedrock: sandstone, fine- to medium-grained, light gray to light brown, micaceous, massive, very well indurated; generally breaks into pieces 6" to 2'. Total depth 5'; no free groundwater.

Job No. 805-10

TEST PIT LOGS

<u>Test Pit Number</u>	<u>Depth (ft.)</u>	<u>Description</u>
TP-32	0-1 1/4	Soil: sandy clay, dark brown, damp (W<PL), medium stiff, low plasticity.
	1 1/4-6	Bedrock: contact between sandstone and Franciscan sheared rock, sandstone to west. Total depth 6'; no free groundwater.
TP-33	0-1 1/4	Soil: sandy clay, dark brown, damp (W<PL), firm to 1', low plasticity; medium stiff below 1'.
	1 1/4-9	Colluvium: sandy clay, light brown, moist (W>PL), medium to high plasticity; gray, with common organic material below 6', low plasticity.
	9-12 1/4	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 3" across. Total depth 12 1/4'; no free groundwater.
TP-34	0-1 1/4	Soil: sandy clay, dark brown, damp (W<PL), medium stiff, low plasticity.
	1 1/4-4	Bedrock: sandstone, fine- to medium-grained, light gray to light brown, micaceous, massive, very well indurated; generally breaks into pieces 6" to 1 1/4'. Total depth 4'; no free groundwater.

TEST PIT LOGS

<u>Test Pit Number</u>	<u>Depth (ft.)</u>	<u>Description</u>
P-35	0-24	Soil: gravelly silt with some clay, abundant organic material; dark brown, slightly damp (W<PL), low plasticity, soft; gravels to 1' across.
	24-84	Colluvium: gravelly clay, light brown, slightly damp (W<PL), medium to low plasticity, medium stiff.
	84-184	Talus: sandy gravel with minor clay, light brown, slightly damp, very loose; gravels all subangular sandstone commonly 6" to 8" across, but some 2' to 3'. Total depth 184'; no free groundwater.
P-36	0-3	Soil: gravelly silt with some clay, abundant organic material; dark brown, slightly damp (W<PL), low plasticity, soft; gravels to 1' across.
	3-64	Colluvium and Talus: sandy gravel with minor clay to gravelly clay.
	64-84	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 1" across, slightly damp (W<PL). Total depth 84'; no free groundwater.

TEST PIT LOGS

<u>Test Pit Number</u>	<u>Depth (ft.)</u>	<u>Description</u>
P-37	0-3	Soil: gravelly silt with some clay, abundant organic material; dark brown, slightly damp (W<PL), low plasticity, soft; gravels to 1' across.
	3-12	Talus: sandy gravel with minor clay, light brown, slightly damp, very loose; gravels all subangular sandstone commonly 6" to 8" across, but some 2' to 3'.
P-38	0-3	Soil: sandy clay, dark brown, damp (W<PL), firm to 2 1/2', low plasticity; medium stiff below 2 1/2'.
	3-4	Colluvium: gravelly clay, light brown, slightly damp (W<PL), medium to low plasticity, medium stiff.
	4-6	Bedrock: sandstone, fine- to medium-grained, light gray to light brown, micaceous, massive, very well indurated; generally breaks into pieces 6" to 14'.
		Total depth 6'; no free groundwater.
P-39	0-24	Soil: sandy clay with minor gravel.
	24-9	Bedrock: sheared sandstone, probably intermediate between sandstone as in TP-18 and Franciscan sheared rock as in TP-7.
		Total depth 9'; no free groundwater.

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TEST PIT LOGS

<u>Test Pit Number</u>	<u>Depth (ft.)</u>	<u>Description</u>
FP-40	0-1½	Soil: sandy clay, dark brown, damp (W<PL), firm to 1', low plasticity; medium stiff below 1'.
	14-5	Bedrock: sandstone, fine- to medium-grained, light gray to light brown, micaceous, massive, very well indurated; generally breaks into pieces 6" to 18". Total depth 3'; no free groundwater.
FP-41	0-3	Soil: sandy clay with gravel.
	3-8	Talus: sandy gravel with minor clay, light brown, slightly damp, very loose; gravels all subangular sandstone commonly 6" to 8" across, but some 2' to 3'. Total depth 8'; no free groundwater.
FP-42	0-3	Soil: sandy clay with gravel.
	3-6	Colluvium: gravelly clay, light brown, slightly damp (W<PL), medium to low plasticity, medium stiff.
	6-10	Bedrock: sandstone, fine- to medium-grained, light gray to light brown, micaceous, massive, very well indurated; generally breaks into pieces 6" to 2'. Total depth 10'; no free groundwater.

Job No. 805-10

TEST PIT LOGS

<u>Test Pit Number</u>	<u>Depth (ft.)</u>	<u>Description</u>
FP-43	0-3	Soil: sandy clay with minor gravel.
	3-5½	Colluvium: gravelly clay with some large boulders.
	5½-7½	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 1" across, damp (W<PL). Total depth 7½'; no free groundwater.
FP-44	0-2½	Soil: sandy clay, dark brown, damp (W<PL), firm to 2½', low plasticity; medium stiff below 2½'.
	2½-5	Colluvium: sandy clay with minor gravel, light brown, slightly damp (W<PL), medium to high plasticity.
	5-8½	Bedrock: sandstone, medium- to coarse-grained, light brown, slightly damp, poorly indurated, friable, broken into pieces ¾" to 2"; easily excavated. Total depth 8½'; no free groundwater.
FP-45	0-2	Soil: sandy clay, dark brown, damp (W<PL), firm to 1', low plasticity; medium stiff below 1'.
	2-11	Colluvium: sandy clay, dark brown, slightly damp (W<PL), firm to stiff, medium to high plasticity. Total depth 11'; no free groundwater.

Job No. 805-10

TEST PIT LOGS

<u>Test Pit Number</u>	<u>Depth (ft.)</u>	<u>Description</u>
P-46	0-3	Soil: sandy clay, dark brown, damp (N<PL), firm to 2½', low plasticity; medium stiff below 2½'.
	3-9	Colluvium: sandy clay with minor gravel.
	9-11	Bedrock (in place?): sandstone, fine- to medium grained, light gray to light brown, micaceous, massive, very well indurated; generally breaks into pieces 6" to 3'.
		Total depth 11'; no free groundwater.
P-47	0-4½	Soil: sandy clay, dark brown, damp (N<PL), firm to 2½', low plasticity; medium stiff below 2½'.
	4½-6	Alluvium: sandy clay with large gravel up to 3' across, light brown, moist (N₂PL), high plasticity.
	6-12	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 1' across.
		Total depth 12'; free groundwater at 6'.

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TEST PIT LOGS

<u>Test Pit Number</u>	<u>Depth (ft.)</u>	<u>Description</u>
TP-48	0-2½	Soil: sandy clay with gravel.
	2½-7	Talus: sandy clay with large gravel up to 3' across.
	7-9	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 1' across.
		Total depth 9'; no free groundwater.
TP-49	0-2	Soil: sandy clay, dark brown, damp (N<PL), firm to 2½', low plasticity; medium stiff below 2½'.
	2-6	Talus: sandy clay with large gravels.
	6-8	Bedrock: Franciscan sheared rock, dark gray, dominantly slickensided and sheared clay with subrounded inclusions up to 1' across.
		Total depth 8'; no free groundwater.