CORNERSTONE EARTH GROUP

Date:	July 23, 2021
Project No.:	1142-1-2
Prepared For:	Mr. Ken White **PENINSULA HUMANE SOCIETY & SPCA** 1450 Rollins Road Burlingame, California 94010
Re:	Geotechnical Review of Use Permit Plans Peninsula Humane Society Animal Sanctuary Plan Review 12429 Pescadero Creek Road Loma Mar, California

Dear Mr. White:

As requested, we reviewed the geotechnical aspects of the architectural, civil and landscape Use Permit plans for the above-referenced project. We previously performed a geotechnical investigation for the project and presented our findings in our report titled, "Geotechnical Investigation Peninsula Humane Society Animal Sanctuary, 12429 Pescadero Creek Road, Loma Mar, California," dated July 23, 2021.

The documents reviewed include the following:

- Project plan set including architectural, titled, "Animal Sanctuary Peninsula Humane Society & SPCA, Loma Mar, CA, Sheets A0.00, A0.01, A1.02, A1.03, A1.2, A1.3, A1.4, AA2.0, AA3.0, AB.1, AC.1, AD.1AE2.0, AE3.0, AF.1, and AG.1," prepared by KSH Architects dated July 12, 2021, Use Permit Submittal.
- Project plan set including civil, titled, "Peninsula Humane Society Animal Sanctuary, 12429 Pescadero Creek Road, Loma Mar, CA, Sheets C-1.0, C-1.1, C-3.0, C-3.1, C-3.2, C-3.3, C-3.4, C-3.5, C-3.6, C-3.7, C-3.8, C-3.9, C-3.10, C-3.11, C-3.12, C-3.13, C-3.14, C-4.0, C-4.1, C-4.2, C-4.3, C-4.4, C-4.5, C-4.6, C-4.7, C-4.8, C-4.9, C-4.10, C-4.11, C-4.12, C-4.13, C-4.14, C-4.15, C-4.16, C-4.17, C-4.18, C-4.19, C-4.20, SS-1, SS-2, SS-3, and SS-4," prepared by Lea & Braze Engineering Inc. dated July 12, 2021.
- Project plan set including civil, titled, "Peninsula Humane Society Animal Sanctuary, Loma Mar, CA, Sheets L1.0, L1.1, L1.2, L1.3, L2.0, L2.1, L2.2, L3.0, L4.0, L6.0, L6.1, L6.2, and L7.0," prepared by The Guzzardo Partnership Inc., dated July 12, 2021, Use Permit Submittal.

Based on our review, the architectural, civil and landscape plans are in general conformance with the recommendations in our geotechnical report.

As recommended in our report, we should be retained to provide geotechnical observation and testing services during construction to complete our role as the Geotechnical Engineer-of-Record for the project.

Closure

This review of plans has been prepared for the sole use of Peninsula Humane Society & SPCA in accordance with generally accepted geotechnical engineering principles and practices in the San Francisco Bay Area at this time. No warranties are either expressed or implied.

Should you have any questions, or if we may be of further service, please contact us at your convenience.

Sincerely,

Cornerstone Earth Group, Inc.

Stephen C. Ohlsen, P.E.
Project Engineer

Danh T. Tran, P.E.
Senior Principal Engineer

SCO:DTT

Copies: Addressee (1 by email)

CORNERSTONE
EARTH GROUP

TYPE OF SERVICES	Geotechnical Investigation
PROJECT NAME	Peninsula Humane Society Animal Sanctuary
LOCATION	12429 Pescadero Road Loma Mar, California
CLIENT	Peninsula Humane Society & SPCA
PROJECT NUMBER	1142-1-1
DATE	July 23, 2021

CORNERSTONE EARTH GROUP

Type of Services	**Geotechnical Investigation**
Project Name	**Peninsula Humane Society Animal Sanctuary**
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Client Address	**1450 Rollins Road Burlingame, California**
Project Number	**1142-1-1**
Date	**July 23, 2021**

Prepared by

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CORNERSTONE EARTH GROUP

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Type of Services	**Geotechnical Investigation**
Project Name	**Peninsula Humane Society Animal Sanctuary**
Location	**12429 Pescadero Road**
	Loma Mar, California

SECTION 1: INTRODUCTION

This geotechnical report was prepared for the sole use of Peninsula Humane Society & SPCA for the Peninsula Humane Society Animal Sanctuary project in Loma Mar, California. The approximate location of the site is shown on the Vicinity Map, Figure 1. For our use, we were provided with the following documents:

- A set of architectural plans, titled, “Peninsula Humane Society Animal Sanctuary,” prepared by KSH Architects, County of San Mateo Use Permit Submittal, dated July 12, 2021.
- A set of civil plans titled, “Peninsula Humane Society Animal Sanctuary, 12429 Pescadero Creek Road, Loma Mar, California,” prepared by Lea & Braze Engineering, Inc., dated July 12, 2021.
- A set of landscape plans titled, “Peninsula Humane Society Animal Sanctuary,” prepared by The Guzzardo Partnership Inc., County of San Mateo Use Permit Submittal, dated July 12, 2021.

1.1 PROJECT DESCRIPTION

The irregularly shaped 213-acre project site is located off of Pescadero Road in Loma Mar, California, about 3500 feet west of the intersection of Pescadero Road and Alpine Road. The site is bounded by Pescadero Road to the east and essentially undeveloped properties surrounding the project site. The site is mostly undeveloped, with a fire road crossing the site transverse to the hillside and an existing barn and caretaker residence to the north of the fire road. Based on the provided architectural plans, we understand that an animal sanctuary campus is planned consisting of a two-level administrator/visitor structure (“Building 2”), cat enclosures (“Buildings B and C”), the restored existing barn (“Building 1”), a new 2,000-square-foot farm animal barn with covered corral (“Building 4”), a 3,000-square-foot covered dog arena, access roads, new caretaker residence with garage (“Building 3”), several maintenance buildings (“Buildings A”), a fire prevention water storage tank and associated pump station, a service yard for generators and a new domestic and landscape irrigation tank and associated pump station, a solar array, and dog enclosures (“Buildings D, E, and F”). Additionally, an on-site septic system with leach field is proposed southwest of the dog enclosures and new animal barn. This development will be clustered along the ridge top and most of the remainder of the site will remain undeveloped with a new gravel road connecting the improvements.

It is expected that the structures will likely be single-story wood-frame structures. Appurtenant parking, utilities, access roads and paths, landscaping and other improvements necessary for site development are also planned.

Structural loads are not currently known for the proposed structures; however, structural loads are expected to be light and typical of similar type structures. Based on our preliminary discussions with the project structural engineer we understand that the cat and dog enclosures will be supported by slabs-on-grade, that the maintenance buildings and animal barns will likely be supported on shallow spread footing foundations, and that the administrator/veterinary building, and caretaker residence will likely be supported on drilled pier foundations. The tank foundation type is unknown at this time. Based on the results of our site investigation and lab testing, we are providing our geotechnical recommendations for these structures in this report.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated June 12, 2019 and consisted of field and laboratory programs to evaluate physical and engineering properties of the subsurface soils, 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 are presented below.

1.3 EXPLORATION PROGRAM

Field exploration consisted of seven borings drilled on January 20 and 21, 2020 with track-mounted, limited-access hollow-stem auger drilling equipment and two borings drilled on January 21, 2020 with hand-auger equipment. The borings were drilled to depths ranging from 13½ to 21½ feet, while the hand augers were advanced to depths of 4 to 4½ feet. The borings and hand augers were backfilled with cement grout in accordance with local requirements; exploration permits were obtained as required by local jurisdictions.

The approximate locations of our exploratory borings are shown on the Geologic Site Plan, Figure 2, respectively. Details regarding our field program are included in Appendix A.

1.4 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 Plasticity Index tests. Details regarding our laboratory program are included in Appendix B.

1.5 ENVIRONMENTAL SERVICES

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

SECTION 2: REGIONAL SETTING

2.1 REGIONAL GEOLOGIC SETTING

The site is located within the north-central Santa Cruz Mountains, a northwest-southeast mountain range within the Coast Range Geomorphic Province. The Santa Cruz Mountains are within the San Francisco Bay Block, which is bounded to the east by the Hayward and Calaveras Faults and to the west by the San Andreas Fault. The San Andreas Fault is a NW-trending, right-lateral, strike-slip fault that is comprised of many strands that form a zone, which is up to 1 km wide within the area. The fault system distributes shearing across a complex system of primarily northwest trending, right-lateral, strike-slip faults that includes the Hayward and Calaveras Faults.

The geology of the La Honda 7.5-minute Quadrangle is characterized by two basement assemblages that are separated by the San Andreas Fault, which extends through the northeastern corner of the quadrangle. Northeast of the San Andreas Fault is a composite Mesozoic basement assemblage consisting of the Franciscan Complex, Coast Range Ophiolite, and the Great Valley Sequence. Southwest of the San Andreas Fault is the Salinian Terrane of the Santa Cruz block, a basement assemblage of granitic and metamorphic crystalline rocks. Rocks within the north-central Santa Cruz Mountains have undergone a complex structural history and have been strongly deformed by faulting and folding. The basement is overlain by Miocene marine strata and Pliocene and Pleistocene sediment. Miocene and later strata have been deformed by reverse faulting along the Sargent, Berrocal and Shannon Fault zones (Hitchcock et a.,1994).

2.2 REGIONAL SEISMICITY

While seismologists cannot predict earthquake events, geologists from the U.S. Geological Survey have recently updated (in 2015) earlier estimates from their 2014 Uniform California Earthquake Rupture Forecast (Version 3; UCERF3) publication. The estimated probability of one or more magnitude 6.7 earthquakes (the size of the destructive 1994 Northridge earthquake) expected to occur somewhere in the San Francisco Bay Area has been revised (increased) to 72 percent for the period 2014 to 2043 (Aagaard et al., 2016). The faults in the region with the highest estimated probability of generating damaging earthquakes between 2014 and 2043 are the Hayward (33%), Calaveras (26%), and San Andreas Faults (22%). In this 30-year period, the probability of an earthquake of magnitude 6.7 or larger occurring is 22 percent along the San Andreas Fault and 33 percent for the Hayward Fault.

The faults considered capable of generating significant earthquakes are generally associated with the well-defined areas of crustal movement, which trend northwesterly. The table below presents the State-considered active faults within 25 kilometers of the site.

Table 1: Approximate Fault Distances

Fault Name	Distance	
	(miles)	(kilometers)
San Andreas (1906)	5.5	8.8
Sargent-Berrocal	6.2	10
Monte Vista-Shannon	6.7	10.8
San Gregorio	7.5	12
Zayante	8.4	13.5

In addition, the Hayward Fault, Calaveras Fault zone, and the San Gregorio Fault Zone (major branching faults of the San Andreas system) are located 24 miles (38.3 km) northeast, 27.7 miles (44.5 km) northeast, and 7.5 miles west of the site. Additionally, two undifferentiated Quaternary faults exist in the general area including: the Butano Fault located about 2 miles (3.2 km) south of the site and the Pilarcitos Fault is located about 4.76 (7.6 km) miles northeast of the site. More locally, Jennings and Bryant (2010) show the (pre-Quaternary) La Honda Fault as projected toward the site with a southeasterly trend. It would intersect the far eastern edge of the site near Pescadero Creek Road (Jennings and Bryant, 2010). Pre-Quaternary Faults are not considered potential seismic sources and do not represent a geologic constraint for fault surface rupture.

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

SECTION 3: SITE CONDITIONS

3.1 SITE HISTORY AND AIR PHOTO REVIEW

A review of historic topographic maps extending back to 1940 and aerial photos extending back to 1931 shows that the site has been used as livestock rangeland for decades. As of the date of the 1931 aerial photos, the site appears to be totally undeveloped with no dirt roads and no structures present.

A review of the historical topographic maps (U.S.G.S.) indicates that a dirt access road ("Burns Chalk Fire Road") has existed along the spine of the ridge since at least as early as 1940. A barn structure was constructed at its current location and a stock pond established just downslope of the access road in the central portion of the site sometime between 1968 and 1980. Between 1982 and 1991 a residence was constructed just on the west side of the barn. Sometime between 1991 and 2005 numerous fenced livestock pens were constructed adjacent to the barn. Sometime between 2005 and 2009 additional soil was placed across from the barn in order to extend a parking area alongside the dirt road for parking of storage vehicles and farm equipment. Additional dirt roads were established along the top of the ridge further to the west in this period.

3.2 SURFACE DESCRIPTION AND TOPOGRAPHY

The site is located on a northwesterly trending ridge southern flank in an area of complex and highly varied topography. The southerly flank of the ridge varies from gently inclined to moderately inclined and steep. The areas where the proposed improvements are to be located can generally be characterized according to the following:

3.2.1 Area of Existing Barn/Adjacent Parking Lot Area

The area of the existing barn and existing caretaker's residence is relatively flat with steep downslopes located within 40 feet north of the existing structures. Although this area is largely flat, there are local variations resulting in approximately 2 feet of topographic relief across the pad area. We understand that the existing caretaker's residence will be demolished and a new fire prevention water tank and pump station will be constructed in its place. The proposed domestic water tank, pump station, and maintenance building located just east of the existing barn is on flat ground, however, there is an existing (undocumented) wedge of fill along the northern edge of this proposed improvement area the slopes become steep immediately adjacent to the area.

The proposed maintenance building and adjacent service yard for generators is located adjacent to the northern edge of the relatively flat area, which is at the crest of a steep slope where localized fill has been placed in order to create a flat pad.

3.2.2 Proposed Caretaker's Residence, Dog Enclosures, and New Barn Area

The proposed caretaker's residence ("Building 3"), dog enclosures ("Buildings D, E, and F"), and new farm animal barn ("Building 4") is on a moderately inclined slope on the downhill side of the existing fire access road.

There is approximately 6 to 8 feet of topographic relief across the pad area. Claystone bedrock is exposed at shallow depths within erosion gullies located just downslope of the building pad area.

3.2.3 Proposed Veterinarian/Administration Building

The area of the proposed Vet/Admin building (Building 2) is in a transitional area where the ground changes from nearly level to gently inclined toward the south. The northern and eastern portion of the building footprint is in an area where undocumented fills exist. These fill berms occur on both the west and the east side of the building footprint and, based on a review of the surrounding natural topography, may be up to 10 feet thick. There is approximately 8 to 12 feet of topographic relief across the pad area. Based on the provided topographic and architectural plans, we understand that the downslope side of the vet/admin building will have a basement level, which will be cut into the existing slope.

The group of proposed “cat enclosures” are located on a gently to moderately inclined slope just to the west of the Administration building. Relief across these pads is on the order to 4 to 6 feet. Bedrock is not exposed in this area of the site.

3.2.4 South Dog Loop Area

The “South Dog Loop” is a proposed group of kennels will include a 3,000 s.f. enclosed “dog arena”, and a series of large and small dog “cottages” around the brow or crest of the flanking slopes around the perimeter of the knoll. The proposed road at the “east dog loop” is located on the top a of a knoll where the slopes are gently inclined to moderately inclined. There is approximately 4 to 6 feet of topographic relief across the dog cottages pads and there is approximately 2 to 3 feet of relief across the the dog arena area. Sandstone bedrock is exposed locally at the ground surface on the top of the knoll.

3.3 SITE GEOLOGY AND SUBSURFACE CONDITIONS

Several regional geologic maps have been prepared of the area surrounding the campus, including those by; Rogers (1971), Brabb (1970 and 1980). We have adopted the nomenclature of Brabb (1980) in assigning geologic unit names for our characterization of the site. Brabb shows the bedrock in the area of the site as the Tahana member of the (Tertiary) Purisima Formation. A vicinity geologic map is presented as figure 6. The geologic units are characterized by Brabb as follows: “Greenish-gray to white or buff, medium to very fine grained sandstone and siltstone, with some silty mudstone. Locally the sandstone is tuffaceous and it weathers white. Pebble conglomerate occurs near the base.” In terms of rock characterization, the bedrock is generally weak, friable, moderately severely weathered.

Our site reconnaissance resulted in the following observations: Bedrock is exposed at road cuts, at erosion scars on site slopes, and at a large cut located just northeast of the proposed caretaker’s residence. A large exposure of bedrock located just on the north side of the caretaker’s residence exposes interbedded silty sandstone and thin bedded siltstone. Claystone is exposed within erosion gullies located on the south of these proposed structure. Our borings encountered primarily claystone with some layers of sandstone. The bedrock is thin to medium bedded (laminated locally) folded locally and displays a variety of structural trends varying from northwesterly, moderately dipping to southwesterly, steeply dipping.

The sloping portions of the site have experienced severe erosion where runoff is not controlled or, alternatively where the surface runoff is focused by roadways or culverts, or swales or gullies. This severe erosion appears to be exacerbated by an abrupt permeability contract between the sandy (erodible) surficial soils and the underlying consolidated sedimentary bedrock units that are more resistant to erosion. The erosion gullies trend downslope toward the southwest and vary from 3 feet deep to as much as 10 feet deep onsite.

Existing stockpiled fill: Two large accumulations of fill exist just south of the access road in the area of the barn, existing caretaker's residence, proposed new fire prevention water tank and pump station, domestic water tank and pump station, and maintenance building. This material forms a sliver of material that extends outward toward the south from the existing dirt road. This material is non-engineered and apparently was placed in order to create additional parking area for farm machinery and vehicles. This fill cannot be relied upon for support of improvements (see Recommendations).

Our site exploration consisted of drilling, logging and sampling within seven conventional geotechnical borings and two hand auger borings at various locations at the site. The exploration was accomplished with a track-mounted drill rig using hollow stem augers and standard geotechnical sampling equipment. The results of the borings are presented below according to location:

3.3.1 Area of Existing Barn/Adjacent Parking Lot Area

Boring EB-6 was located near the northwest corner of the current fenced in "corral" area, the future location of a domestic water tank and associated pump station, and maintenance building. Here the subsurface profile consisted of a 3½ foot-thick layer of surficial (undocumented fill) sandy lean clay. The fill was underlain by black fat clay (residual soil) to a depth of 7½ feet. Below the depth of 7½ feet is the sandy claystone bedrock. The fill and residual soil layers were found to be in a stiff to very stiff condition, however the undocumented fill is judged to be moderately compressible. The claystone was found to be in a generally weak condition in terms of bedrock characterization and produced standard penetration test blow counts that ranged from10 blows-per-foot (bpf) to 21 bpf. We understand that the existing caretaker's residence will be demolished and a fire prevention water tank and associated pump station will be constructed partially within the old residence footprint. We anticipate that up to several feet of undocumented fill may be encountered due to the previous development.

3.3.2 Proposed Caretaker's and New Barn

Boring EB-2 was located in the general area of the Caretaker's cottage and new barn. As noted already, a large exposure of bedrock located just on the north side of the caretaker's residence and guest cottages exposes interbedded silty sandstone and thin bedded siltstone. Claystone is exposed within erosion gullies located on the south of these proposed structures. The change in lithology between the cut exposure and the exploratory boring and erosion gullies further downslope is likely due to the result of folding that trends through the immediate area. At the Boring EB-2 location, the subsurface profile consisted of a 2½-foot-thick layer of surficial (colluvium) fat clay with sand. The residual soil was underlain by claystone bedrock. The residual soil layer was found to be in a medium stiff condition in terms of soil characterization. The claystone was found to be in a generally weak condition in terms of bedrock characterization and produced standard penetration test blow counts that ranged from 22 blows-per-foot (bpf) to 34 bpf. A geologic cross section A-A' developed for this area is shown on Figure G.

3.3.3 Proposed Veterinarian/Administration Building

Boring EB-7 was located in the general area of the veterinarian/administration building. Here the subsurface profile consisted of a 1½-foot-thick layer of surficial (colluvium) clayey sand. The residual soil was underlain by sandstone bedrock. The residual soil layer was found to be in a medium dense condition. The sandstone and claystone was found to be in a generally weak condition in terms of bedrock characterization and produced standard penetration test blow counts that ranged from 21 blows-per-foot (bpf) to 36 bpf. As discussed earlier, there are fill berms on both the west and the east side of the building footprint that may be up to 10 feet thick based on a review of the surrounding natural topography.

3.3.4 Proposed Cat and Dog Enclosure Area

Boring EB-3 and EB-7 was located in the general area of the cat enclosure area. Here the subsurface profile consisted of a 2- to 4-foot-thick layer of surficial (colluvium) fat clay with sand. The residual soil was underlain by sandstone bedrock. The residual soil layer was found to be in a stiff condition. The claystone was found to be in a generally weak condition in terms of bedrock characterization and produced standard penetration test blow counts that ranged from 18 blows-per-foot (bpf) to 37 bpf.

3.3.5 Plasticity/Expansion Potential

We performed two Plasticity Index (PI) tests on representative samples. Test results were used to evaluate expansion potential of surficial soils and underlying bedrock. The result of the surficial PI test indicated a PI of 34, indicating very high expansion potential to wetting and drying cycles. The result of the PI test on the underlying claystone indicated a PI of 60, which indicates very high expansive potential to wetting and drying cycles.

3.3.6 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents within the upper 10 feet range from 2 percent under to 15 percent over the estimated laboratory optimum moisture.

3.4 GROUNDWATER

The site encompasses high elevation ground along the top and southerly crest of a ridgetop in the rugged La Honda region of the Santa Cruz Mountains. The site is underlain at shallow depths by sedimentary bedrock and our research suggests this formation does not serve as a laterally continuous shallow aquifer. The only water noted at the site exists within two large stock ponds that exist in the lower portion of the site slopes located well below (downslope) of the proposed improvements. These stock ponds are fed by surface runoff. We did not encounter evidence of groundwater in any of our explorations. It should be noted that, in general, fluctuations in groundwater levels could occur due to many factors including perched water, and regional groundwater variations, and rainfall or irrigation. We note that perched groundwater conditions are often present in the bedrock on hillside sites.

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT SURFACE RUPTURE

As stated earlier, published maps do not show any faults trending through the subject site (Rogers, 1971; Brabb, 1970 and 1980; Brabb and Olsen 1983; Jennings and Bryant, 2010; CDMG, 2003; USGS Fault and Fault database, 2006). The site is not located within a State Earthquake Fault Zone (CDMG 2003). We did not encounter evidence during our research or site reconnaissance of faults trending through the site. The potential for fault surface rupture occurring at the site should be considered low.

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) was estimated for analysis using a value equal to F_{PGA}*PGA, as allowed in the 2019 edition of the California Building Code per Exception 2 of Section 11.4.8 of ASCE 7-16. For our analyses, we used a PGA of 1.114g.

4.3 LIQUEFACTION POTENTIAL

Published geotechnical hazard maps do not show the site in an area identified as having a liquefaction potential. This is due primarily to the fact that very shallow bedrock exists at the site and it is located at a high elevation in rugged terrain. The site is not located within a County-designated Liquefaction Hazard Zone (San Mateo County, 2008), and is within a zone mapped as having a low liquefaction potential by the Association of Bay Area Governments (ABAG). We screened the site for liquefaction during our site exploration by retrieving samples from the site, performing visual classification on sampled materials, and performing various tests to further classify the soil properties.

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.

As discussed in the “Subsurface” section above, we primarily encountered surficial soils consisting of lean clays or sandstone, siltstone and claystone bedrock. These materials are generally not susceptible to liquefaction. Based on the above, our screening of the site for liquefaction indicates a low potential for liquefaction.

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. As the soils encountered at the site were predominantly medium stiff to very stiff clays, and medium dense clayey sands, or claystone and sandstone bedrock, in our opinion, the potential for significant differential seismic settlement affecting the proposed improvements is low.

4.6 LANDSLIDING

4.6.1 General

The California Geological Survey (CGS) has not yet produced a Seismic Hazard Zone report or accompanying map for the La Honda 7.5-minute quadrangle during their ongoing program to map Seismic Hazard Zones on a 7.5-minute quadrangle scale (1:24,000) in the Bay Area. The County of San Mateo has not established regulatory zones for landsliding, however, the planning department maintains a map of “Existing Landslides” in the county (based on the USGS publication), open File Report 975-C. The published landslide-themed map of Brabb and Pampeyan (1972) which covers the County of San Mateo shows the site in an area of suspected large-scale landsliding (Figure 5 is a partial reproduction of the map of Brabb and Pampeyan. Specifically, the ridge top and the crests of adjacent slope son the south side are shown in a headscarp area of a large-scale landslide complex, which is shown s encompassing the rolling topography on the slopes below the slope crests. The proposed improvements are outside the mapped landslide mass. The county planning department shows the site in a zone designated as “areas of mostly landslides”. The CGS interactive map showing reported recent landslides (CGS, 2018) does not show any reported landslides in the immediate area. These mapped landslides and classifications are the result of interpretive mapping and are not based on site-specific studies. These maps serve as a planning resource. Maps and publications published after the damaging El Niño rainfall events in 1982, and 1995 (Ellen & Weiczorek, 1982; Ellen et al., 1997) depicting landslides that resulted from those large-scale damaging events do not show any landslides that occurred from those events at the site.

Our site-specific geologic evaluation has resulted in an interpretation that differs from the published mapping in terms of the nature and extent of landsliding at the site.

4.6.2 Site-Specific

Our review of aerial photos, our site reconnaissance and subsurface exploration has led to our conclusion that, although the lower portions of slopes on the south flank of the ridge display rolling topography, these slopes are not part of a large-scale landslide as suggested on the map of Brabb and Pampeyan (1972). Landsliding identified in this evaluation is based on geomorphic features discernible at the ground surface and in stereo aerial photographs. We have mapped several landslides on the subject property and have depicted these features on the site plan (Figure 2) and have designated some of these individual slides on the map with numbers as a convenience in description in this text. Some of these identified features are located well beyond the proposed improvements and are not considered a constraint to the siting of structures or grading. The establishment of a septic system leachfield at the site is located closer to these identified landslides (see Figure 7) and the layout and design of these leachfields should take into account the constraints (see Recommendations section). Of the landslides that have been mapped during our study, the following landslides are located more proximal to the proposed features and are discussed below:

Qls1: This slide is located just downslope of the existing and proposed access road in the north-central portion of the property (see Figure 2). This feature is a slump-type failure and, based on the relative topography surrounding this feature, is inferred to be relatively shallow (approximately 15 feet thick or less) and consists of colluvial soils overlying thin bedded mudstone and sandstone. A culvert trends beneath the road which delivers surface runoff from the road into the headscarp of this feature. This may have served as the triggering mechanism for this shallow landslide. Drainage improvements should be modified in this area in order to help mitigate this condition. Recommendations are offered for reducing this constraint (see Section 6.12 titled “Site Drainage”).

Qls2: This suspected landslide is a relatively small, shallow landslide (a slump) located adjacent to the downslope side of the vet/admin building and several cat enclosure structures (see Figure 2). Although poorly defined in terms of slope morphology. The scarp area is located less than 10 feet from the nearest proposed enclosure and admin building. Our exploratory boring (EB-7) drilled near the scarp of this mapped slide indicates bedrock is shallow in this area. This feature may have been triggered by a lack of surface runoff coming off the top of the ridge. This runoff pattern my no longer exist due to the establishment of the graded dirt access road and fill berms that have been placed in the last 30 or so years.

Qls4: This is a suspected landslide scarp, however, it lacks topographic patterns that would suggest a debris field is present below the scarp (see Figure 2). This feature is located adjacent to the main site access road. A landslide below this scarp would most probably move downslope and away from the road, however, the scarp would not be expected to “back step” over time into the roadway area provided that surface runoff is controlled and directed away from this feature.

Qls3 and Qls5 are all located well outside any proposed developed areas and therefore do not pose a constraint to any proposed features for the current version of the development concept (see Figures 2). Aside from seismic shaking, proximity to some small to moderate sized landslides, and the more general hazard of erosion, there are no other geologic constraints that potentially impact the proposed project as currently conceived.

Control of construction phase runoff and long-term runoff is essential for the stability of slopes at the site. All runoff should be collected and directed to suitable discharge points which specifically avoid the mapped landslides and these discharge points should be located well downslope of the proposed development features, including roads. We do not recommend allowing or directing development runoff toward the very steep slopes on the north side of the north property line (see Site Drainage Recommendations).

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.

- Presence of highly expansive soil and bedrock
- Presence of undocumented fills
- Potential for cut/fill transitions
- Redevelopment considerations
- Slope stability and building/leach field setbacks
- Presence of cohesionless soils
- Potential for difficult excavation
- Soil Corrosion Potential

5.1.1 Presence of Highly Expansive Soil and Bedrock

Our borings disclosed the presence of both sandstone and claystone bedrock of the Tahana formation at the site. Our Plasticity Index testing of the claystone and residual clay soils indicate that these materials are highly to very highly expansive. 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 or the structures should be supported on a drilled pier foundation system. Because of these expansive soils and the close proximity of the bedrock, we recommend the care takers residence, fire prevention water tank and pump station, domestic water tank and pump station, maintenance building, and vet/admin building should be supported on drilled pier foundations. While the PI testing indicates highly expansive soils and bedrock, we are not aware of any published geologic or geotechnical information which suggests these materials are subject to extreme uplift pressures and movement as claystone bedrock of the Whiskey Hill formation is known for, which is located in the vicinity of Menlo Park. This report does not provide recommendations to address extreme uplift and movement of claystone because it has not been documented for this unit in the published literature or in our experience with this geologic unit. However, we would recommend that the grading plan be developed to limit cuts to about 3 feet to mitigate potential heave of the very highly expansive claystone. In areas of the structures where there will be greater than 3 feet of cut into the claystone, we recommend the minimum drilled pier embedment be increased to 15 feet. 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 these expansive soil and bedrock concerns are presented in the following "Earthwork" and "Foundation" sections.

5.1.2 Presence of Undocumented Fills

Our borings encountered undocumented fill ranging up to 3½ feet in depth, and two fill berms were observed the west and the east side of the approximate vet/admin building footprint that may be up to 10 feet thick based on our review of the surrounding natural topography. To reduce the potential for differential settlement, we recommend that the undocumented fill be over-excavated and recompacted following the recommendations presented in the "Earthwork" section below. In addition, where fill placement results in a cut/fill transition within a building pad that will be supported on shallow foundations, we recommend that the entire building pad be overexcavated to provide uniform support. Additional recommendations are provided in the "Earthwork" section of this report.

5.1.3 Potential for Cut/Fill Transitions

Based on the proposed level building pads for many of the structures, and the existing topography of the site, new structures could potentially span cut/fill transitions, if not mitigated. The performance of a structure supported on a shallow foundation overlying a cut/fill transition could result in increased differential settlement. Therefore, we recommend that cut/fill transitions be over-excavated and that shallow foundations bear uniformly on similar, undisturbed native soil or bedrock, or a relatively uniform section of engineered fill over undisturbed native soil and/or bedrock. Recommendations addressing this are presented in the "Earthwork" section.

5.1.4 Redevelopment Considerations

As discussed, the site is currently occupied by existing buildings, site fixtures, and landscaping. We understand that some of the existing improvements, such as the existing caretaker's residence, will be demolished for the construction of the new site improvements. We understand the new fire prevention water tank and pump station will be constructed partially within the footprint of the existing residence. Potential issues that are often associated with redeveloping sites include demolition of existing improvements, abandonment of existing utilities, and undocumented fills. Please refer to the "Earthwork" section below for further recommendations.

5.1.5 Slope Stability and Building Setbacks

Several potential landslides and areas of slope instability were identified during our investigation. However, it appears that the proposed project layout has been made to avoid these areas. Our recommendations for building and leach field setbacks are presented in the "Earthwork" section of this report.

5.1.6 Presence of Cohesionless Soils

As mentioned, some areas of the site are underlain by cohesionless, sandy soils with low fines content. The sandy soils may not stand vertical when excavated and excavation sidewalls for foundations, utility trenches, temporary slopes, basement excavation, etc., may cave in or accumulate significant amount of slough. Grading and excavation contractors should be made aware of this condition and plan on forming footings, preparing slab-on-grade subgrade just prior to concrete placement, and other similar construction issues as relates to temporary shoring, utility excavations, etc. Our recommendations for excavation of cohesionless soils are presented in the "Earthwork" section of this report.

5.1.7 Potential for Difficult Excavation

Our borings encountered moderately hard, moderately to deeply weathered Tahana Claystone and Sandstone. Based on the project plans, excavations into claystone and sandstone is anticipated and should be anticipated. In our opinion, moderately to deeply weathered areas of bedrock would be excavatable with heavy-duty excavating equipment (such as large backhoes or excavators). However, slightly weathered to fresh bedrock areas, if encountered, will likely require excavation with a hoe-ram. Additionally, drilled pier contractors should anticipate difficult drilling conditions and should be experienced in drilling in bedrock conditions and the use of appropriate equipment (such as coring barrels) to advance the piers to design depths. Additional recommendations are provided in the "Earthwork" and "Foundation" sections of this report.

5.1.8 Soil Corrosion Potential

Soil corrosion screening was not performed during our investigation; however, based on our experience with similar soil, the subsurface soil is likely to be considered corrosive to buried metal and potentially concrete as well. We recommend soil corrosion screening be performed during design.

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

All existing improvements not to be reused for the current development, including all foundations, flatwork, pavements, utilities, and other improvements should be demolished and removed from the site. Recommendations in this section apply to the removal of these improvements, which may be present on the site, prior to the start of mass grading or the construction of new improvements for the project. It is noted that “unknown” buried structures such as septic systems, leach fields, seepage piles, debris pits, and/or wells, etc. may be encountered during grading. If these are encountered during grading, we should provide recommendations to address them on a case-by-case basis.

Cornerstone should be notified prior to the start of demolition, and should be present on at least a part-time basis during all backfill and mass grading as a result of demolition.

6.1.1 Abandonment of Existing Utilities

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 risk for owners 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.

6.2 SITE CLEARING AND PREPARATION

6.2.1 Site Stripping

The site should be stripped of all surface vegetation, and surface and subsurface improvements to be removed within the proposed development area. Demolition of existing improvements is discussed in the prior paragraphs. A detailed discussion of removal of existing fills is provided later in this report. 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 4 to 6 inches below existing grade in vegetated areas.

6.2.2 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.

6.3 REMOVAL OF EXISTING FILLS

As discussed, our borings encountered undocumented fill to depths of 3½ feet and two fill berms observed directly west of and within the east side of the vet/admin building footprint that may be up to 10 feet thick, much of this fill will likely be removed during grading. In addition, we anticipate up to several feet of undocumented fill may be encountered below and in the vicinity of the existing caretaker’s residence due to previous site grading activities. All fills should be completely removed from within building areas and tank areas, and to a lateral distance of at least 5 feet beyond the building footprint or to a lateral distance equal to fill depth below the perimeter footing, whichever is greater. We also recommend that all undocumented fill be removed from pavement and flatwork areas. Provided the fills meet the “Material for Fill” requirements below, the fills may be reused when backfilling the excavations. Based on review of the fill berms, the material may be reused if all debris, wood, trash, and other unsuitable material is screened out of the remaining material and removed from the site. If materials are encountered that do not meet the requirements, such as debris, wood, trash, those materials should screened out of the remaining material and be removed from the site. Backfill of excavations should be placed in lifts and compacted in accordance with the "Compaction" section below.

6.4 BUILDING AND LEACH FIELD SETBACKS

In general, we recommend that the proposed buildings, equipment pads, and water tanks be setback at least 25 feet from the mapped landslides and 15 feet from the top of slopes. Where structures are within 15 feet of a slope, we recommend they be supported on drill piers designed in accordance with the recommendations in this report. This would apply to the caretaker residence, fire prevention and domestic water tank pads and associated pump stations, maintenance building, and administration/veterinary clinic building. We note that one of the cat enclosures is positioned about 10 feet away from the top of Landslide #2. We note that EB-7 was drilled between the Cat Enclosure and the top of Landslide #2. Since the boring disclosed that the sandstone bedrock is at a shallow depth in this area, the location of this Cat Enclosure is acceptable from a geologic viewpoint. The leach field should be set back at least 50 feet from the top of the mapped landslides. General recommendations for release of water onto the slopes is presented in the "Site Drainage" portion of this report.

6.5 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 C 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 no greater than 1:1 (horizontal:vertical) within the upper 5 feet below building subgrade, unless the OSHA soil classification indicates that slope should be flatter.

6.6 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.

6.7 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 natural 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 potential methods to address potential 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.

6.7.1 Scarification and Drying

The subgrade may be scarified to a depth of 12 to 18 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.

6.7.2 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.

6.7.3 Chemical Treatment

Where the unstable area exceeds about 5,000 to 10,000 square feet and/or site winterization is desired, chemical treatment with quicklime (CaO), kiln-dust, or cement may be more cost-effective than removal and replacement. Recommended chemical treatment depths will typically range from 12 to 18 inches depending on the magnitude of the instability.

6.8 MATERIAL FOR FILL

6.8.1 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 below the non-expansive fill section. 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 oversize 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.

6.8.2 Potential Import Sources

Imported and non-expansive material should be inorganic with a Plasticity Index (PI) of 15 or less, and not contain recycled asphalt concrete where it will be used within the habitable building areas. 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.

6.8.3 Non-Expansive Fill Using Lime Treatment

As discussed above, non-expansive fill should have a Plasticity Index (PI) of 15 or less. Due to the high clay content and PI of the on-site soil and bedrock materials, it is not likely that sufficient quantities of non-expansive fill would be generated from cut materials. As an alternative to importing non-expansive fill, chemical treatment can be considered to create non-expansive fill. If this option is considered, additional laboratory tests should be performed prior to initial site grading to further evaluate the optimum percentage of quicklime required.

6.9 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 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 2: 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
	Low Expansion Soils	90	>1
General Fill (below a depth of 5 feet)	On-Site Expansive Soils	95	>3
	Low Expansion Soils	95	>1
Trench Backfill	On-Site Expansive Soils	87 – 92	>3
Trench Backfill	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	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	Low Expansion Soils	95	>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)

6.9.1 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.

6.10 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 (⅜-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.

Where utility lines will cross perpendicular to strip footings, the footing should be deepened to encase the utility line, providing sleeves or flexible cushions to protect the pipes from anticipated foundation settlement, or the utility lines should be backfilled to the bottom of footing with sand-cement slurry or lean concrete. Where utility lines will parallel footings and will extend below the "foundation plane of influence," an imaginary 1:1 plane projected down from the bottom edge of the footing, either the footing will need to be deepened so that the pipe is above the foundation plane of influence or the utility trench will need to be backfilled with sand-cement slurry or lean concrete within the influence zone. Sand-cement slurry used within foundation influence zones should have a minimum compressive strength of 75 psi.

On expansive soils sites it is desirable to reduce the potential for water migration into building and pavement 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 and pavement areas.

6.11 PERMANENT CUT AND FILL SLOPES

All permanent cut and fill 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 (H:V). Fill slopes should be overbuilt and trimmed back, exposing engineered fill when complete. We would also recommend that in the building areas cuts be limited to 3 feet to reduce the potential for heave in the claystone bedrock. Refer to the "Erosion Control" section of this report for a discussion regarding protection of slope surfaces.

6.11.1 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 8 feet wide. Depending on the thickness of any colluvial/residual soil layer 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 moderately weathered bedrock, and be at least 12 feet wide. A typical key and construction is depicted in Figure 8.

6.11.2 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.

The drainage system should be constructed in small trenches or v-ditches and consist of a minimum 4-inch-diameter perforated (perforations placed downward) pipe, bedded and shaded in Caltrans Class 2 Permeable Material (latest version) or ¾-inch crushed rock; if crushed rock is used, the rock should be encapsulated in filter fabric (Mirafi 140N or equivalent). The bedding should be at least 2 inches, and the trench should be at least 8 inches in width and depth. Alternatively, geocomposite strip drains may be used. All drainage lines should slope towards suitable outlet structures at an inclination of at least 0.5 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.

6.11.3 Plan Review and Construction Monitoring

We should be retained to review the conceptual 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 keyway and fill slope construction. Field modifications to the planned keyway and benching may be required based on encountered field conditions. In addition, it has been our experience that cut slopes in the Tahana Formation 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.

6.12 CUT/FILL TRANSITION OVER-EXCAVATION

Structures underlain by cut/fill transitions should be over-excavated to provide a relatively uniform fill thickness beneath the structure footprint. The depth of over-excavation below pad grade should be equal to at least 3 feet below the bottom of foundations to provide a uniform engineered fill pad. The final depth of the over-excavation will depend on the type of material exposed, and will be determined in the field during construction. In general, over-excavation should extend to at least 5 feet beyond the building footprint. Adjustments to the depth and lateral limits of the over-excavation may need to be made at the time of construction depending on the actual conditions encountered during grading.

6.13 SITE DRAINAGE

6.13.1 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. We recommend that the development runoff be directed through solid drain pipes to suitable discharge facilities located well downslope of the developed areas. Alternatively, runoff may be directed in solid pipes to the existing stock ponds located in the western and in the eastern portions of the site. Discharge areas for runoff should be setback a minimum distance of 100 feet from identified landslides scarps. Runoff should not be allowed to flow over the steep to very steep slopes that are adjacent to the north property line. Ponding should also not be allowed on or adjacent to building foundations, slabs-on-grade, or pavements. Hardscape surfaces should slope at least 2 percent towards suitable discharge facilities; landscape areas should slope at least 3 percent towards suitable discharge facilities. Roof runoff should be directed away from building areas in closed conduits, to approved infiltration facilities, or on to hardscaped surfaces that drain to suitable facilities. Retention, detention or infiltration facilities should be spaced at least 10 feet from buildings, and preferably at least 5 feet from slabs-on-grade or pavements. These facilities are not recommended where stormwater infiltration may affect slopes at lower elevations on or adjacent to the site. However, if slopes are not present at lower elevations that could potentially be affected, and if retention, detention or infiltration facilities are located within these zones, we recommend that these treatment facilities meet the requirements in the Storm Water Treatment Design Considerations section of this report.

Lined v-ditches should be included at the top of slopes and intermediate benches, and at the toe of slopes or behind retaining walls adjacent to planned or existing 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 of is present or planned, additional surface and subsurface drainage, or construction of drained buttress fills may be needed to protect site improvements. We should be consulted if this issue will affect the project.

We recommend that the septic leach fields are designed to disperse effluent over as large an area as practicable, or alternatively, that the effluent be directed deeper into the subsurface profile within sandstone that underlies the surficial soils and claystone layers. The infiltration or percolation rate should be evaluated by the leach field designer.

6.13.2 Subsurface Drainage

As discussed in the "Permanent Cut and Fill Slopes" section, subsurface drainage improvements might be installed as part of earthwork for fill construction if perched groundwater is observed. These improvements should include positive surface gradients for keyways and benches and the installation of a subdrain system consisting of perforated pipe and permeable gravel or drain rock. If drain rock is used, the rock and pipe should be entirely wrapped with a permeable geotextile fabric. Subdrains should also be installed at the toe of any proposed cut slopes depending on the actual conditions observed during construction. As previously discussed, a conceptual subdrain plan should be prepared once preliminary grading plans are finalized. The actual location of subdrains should be determined in the field at the time of construction.

6.14 LOW-IMPACT DEVELOPMENT (LID) IMPROVEMENTS

The Municipal Regional Permit (MRP) requires regulated projects to treat 100 percent of the amount of runoff identified in Provision C.3.d from a regulated project's drainage area with low impact development (LID) treatment measures onsite or at a joint stormwater treatment facility. LID treatment measures are defined as rainwater harvesting and use, infiltration, evapotranspiration, or biotreatment. A biotreatment system may only be used if it is infeasible to implement harvesting and use, infiltration, or evapotranspiration at a project site.

Technical infeasibility of infiltration may result from site conditions that restrict the operability of infiltration measures and devices. Various factors affecting the feasibility of infiltration treatment may create an environmental risk, structural stability risk, or physically restrict infiltration. The presence of any of these limiting factors may render infiltration technically infeasible for a proposed project. To aid in determining if infiltration may be feasible at the site, we provide the following site information regarding factors that may aid in determining the feasibility of infiltration facilities at the site.

- The near-surface soils at the site are clayey, and categorized as Hydrologic Soil Group D, and is expected to have infiltration rates of less than 0.2 inches per hour. In our opinion, these clayey soils will significantly limit the infiltration of stormwater.

- No groundwater production wells are within 100 feet of potential locations for infiltration facilities.

- The site is not known, to our knowledge, to have pollutants with the potential for mobilization as a result of stormwater infiltration.

- The site has a known geotechnical hazard consisting of steep slopes and areas with landslide potential; therefore, stormwater infiltration facilities may not be feasible.

- In our opinion, infiltration locations within 10 feet of the buildings and top of slopes or on the slopes would create a geotechnical hazard.

6.14.1 Storm Water Treatment Design Considerations

If storm water treatment improvements, such as shallow bio-retention swales, basins or pervious pavements, are required as part of the site improvements to satisfy Storm Water Quality (C.3) requirements, we recommend the following items be considered for design and construction.

6.14.1.1 General Bioswale Design Guidelines

- If possible, avoid placing bioswales or basins within 10 feet of the building perimeter or within 5 feet of exterior flatwork or pavements. If bioswales must be constructed within these setbacks, the side(s) and bottom of the trench excavation should be lined with 10-mil visqueen to reduce water infiltration into the surrounding expansive clay.

- Bioswales constructed within 3 feet of proposed buildings may be within the foundation zone of influence for perimeter wall loads. Therefore, where bioswales will parallel foundations and will extend below the "foundation plane of influence," an imaginary 1:1 plane projected down from the bottom edge of the foundation, the foundation will need to be deepened so that the bottom edge of the bioswale filter material is above the foundation plane of influence.

- The bottom of bioswale or detention areas should include a perforated drain placed at a low point, such as a shallow trench or sloped bottom, to reduce water infiltration into the surrounding soils near structural improvements, and to address the low infiltration capacity of the on-site clay soils.

6.14.1.2 Bioswale Infiltration Material

- Gradation specifications for bioswale filter material, if required, should be specified on the grading and improvement plans.

- Compaction requirements for bioswale filter material in non-landscaped areas or in pervious pavement areas, if any, should be indicated on the plans and specifications to satisfy the anticipated use of the infiltration area.

- If required, infiltration (percolation) testing should be performed on representative samples of potential bioswale materials prior to construction to check for general conformance with the specified infiltration rates.

- It should be noted that multiple laboratory tests may be required to evaluate the properties of the bioswale materials, including percolation, landscape suitability and possibly environmental analytical testing depending on the source of the material. We recommend that the landscape architect provide input on the required landscape suitability tests if bioswales are to be planted.

- If bioswales are to be vegetated, the landscape architect should select planting materials that do not reduce or inhibit the water infiltration rate, such as covering the bioswale with grass sod containing a clayey soil base.

- If required by governing agencies, field infiltration testing should be specified on the grading and improvement plans. The appropriate infiltration test method, duration and frequency of testing should be specified in accordance with local requirements.

- Due to the relatively loose consistency and/or high organic content of many bioswale filter materials, long-term settlement of the bioswale medium should be anticipated. To reduce initial volume loss, bioswale filter material should be wetted in 12 inch lifts during placement to pre-consolidate the material. Mechanical compaction should not be allowed, unless specified on the grading and improvement plans, since this could significantly decrease the infiltration rate of the bioswale materials.

- It should be noted that the volume of bioswale filter material may decrease over time depending on the organic content of the material. Additional filter material may need to be added to bioswales after the initial exposure to winter rains and periodically over the life of the bioswale areas, as needed.

6.14.1.3 Bioswale Construction Adjacent to Pavements

If bio-infiltration swales or basins are considered adjacent to proposed parking lots or exterior flatwork, we recommend that mitigative measures be considered in the design and construction of these facilities to reduce potential impacts to flatwork or pavements. Exterior flatwork, concrete curbs, and pavements located directly adjacent to bio-swales may be susceptible to settlement or lateral movement, depending on the configuration of the bioswale and the setback between the improvements and edge of the swale. To reduce the potential for distress to these improvements due to vertical or lateral movement, the following options should be considered by the project civil engineer:

- Improvements should be setback from the vertical edge of a bioswale such that there is at least 1 foot of horizontal distance between the edge of improvements and the top edge of the bioswale excavation for every 1 foot of vertical bioswale depth, or

- Concrete curbs for pavements, or lateral restraint for exterior flatwork, located directly adjacent to a vertical bioswale cut should be designed to resist lateral earth pressures in accordance with the recommendations in the "Retaining Walls" section of this report, or concrete curbs or edge restraint should be adequately keyed into the native soil or engineered to reduce the potential for rotation or lateral movement of the curbs.

6.15 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 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.

6.16 LANDSCAPE CONSIDERATIONS

Since the near-surface soils are moderately to highly expansive, we recommend greatly reducing the amount of surface water infiltrating these soils near foundations and exterior slabs-on-grade. This can typically be achieved by:

- Using drip irrigation
- Avoiding open planting within 3 feet of the building perimeter or near the top of existing slopes
- Regulating the amount of water distributed to planter areas by using irrigation timers
- Selecting landscaping that requires little or no watering, especially near foundations.

We recommend that the landscape architect consider these items when developing landscaping plans.

SECTION 7: FOUNDATIONS

7.1 SUMMARY OF RECOMMENDATIONS

In our opinion, the proposed structures may be supported on shallow foundations and/or drilled piers provided the recommendations in the "Earthwork" section and the sections below are followed.

7.2 SEISMIC DESIGN CRITERIA

Our explorations generally encountered colluvium and residual soil overlying Tahana Formation claystone and sandstone to depths of 21½ feet, the maximum depth explored. Based on our borings and review of local geology, the site is underlain by shallow alluvial soils underlain by shallow rock with typical SPT "N" values above 50 blows per foot. Therefore, we have classified the site as Soil Classification C. The mapped spectral acceleration parameters S_s and S_1 were calculated using the web-based program ATC Hazards by Locations, located at https://hazards.atcouncil.org/, based on the site coordinates presented below and the site classification. Recommended values for design are presented in Table 3. The table below lists the various factors used to determine the seismic coefficients and other parameters.

Table 3: 2019 CBC Site Categorization and Site Coefficients

Classification/Coefficient	Design Value
Site Class	D
Site Latitude	37.302572°
Site Longitude	-122.279724°
0.2-second Period Mapped Spectral Acceleration[1], S_S	2.11g
1-second Period Mapped Spectral Acceleration[1], S_1	0.815g
Short-Period Site Coefficient – Fa	1.2
Long-Period Site Coefficient – Fv	1.4
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S_{MS}	2.532g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	1.141g
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.688g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	0.76g
MCE_G Peak Ground Acceleration – PGA	0.929g
Site Amplification Factor at PGA – F_{PGA}	1.2
Site Modified Peak Ground Acceleration – PGA_M	1.114g

7.3 SHALLOW FOUNDATIONS

7.3.1 Spread Footings – Animal Barn and Enclosed Dog Arena

The proposed animal barn and enclosed dog arena may be supported on shallow spread footings. Spread footings should bear on natural, undisturbed soil or engineered fill, be at least 12 inches wide, and extend at least 30 inches below the lowest adjacent grade. Lowest adjacent grade is defined as the deeper of the following: 1) bottom of the adjacent interior slab-on-grade, or 2) finished exterior grade, excluding landscaping topsoil. The deeper footing embedment is due to the presence of highly expansive soils, and is intended to embed the footing below the zone of significant seasonal moisture fluctuation, reducing the potential for differential movement.

Footings constructed to the above dimensions and in accordance with the "Earthwork" recommendations of this report are capable of supporting maximum allowable bearing pressures of 2,500 psf for dead loads, 3,750 psf for combined dead plus live loads, and 5,000 psf for all loads including wind and seismic. These pressures are based on factors of safety of 3.0, 2.0, and 1.5 applied to the ultimate bearing pressure for dead, dead plus live, and all loads, respectively. These pressures are net values; the weight of the footing may be neglected for the portion of the footing extending below grade (typically, the full footing depth). Top and bottom mats of reinforcing steel should be included in continuous footings to help span irregularities and differential settlement.

7.3.2 Footing Settlement

Structural loads were not provided to us at the time this report was prepared; therefore, we assumed isolated column loading of 30 to 50 kips. Based on the assumed loading and the allowable bearing pressures presented above, we estimate that the total static footing settlement will be on the order of ½-inch, with about ¼-inch of post-construction differential settlement between adjacent foundation elements. As our footing loads were assumed, we recommend we be retained to review the final footing layout and loading, and verify the settlement estimates above.

7.3.3 Lateral Loading

Lateral loads may be resisted by friction between the bottom of footing and the supporting subgrade, and also by passive pressures generated against footing sidewalls. An ultimate frictional resistance of 0.45 applied to the footing dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 450 pcf may be used in design. The structural engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate values above. Where footings are adjacent to landscape areas without hardscape, the upper 12 inches of soil should be neglected when determining passive pressure capacity.

7.3.4 Spread Footing Construction Considerations

Where utility lines will cross perpendicular to strip footings, the footing should be deepened to encase the utility line, providing sleeves or flexible cushions to protect the pipes from anticipated foundation settlement, or the utility lines should be backfilled to the bottom of footing with sand-cement slurry or lean concrete. Where utility lines will parallel footings and will extend below the "foundation plane of influence," an imaginary 1:1 plane projected down from the bottom edge of the footing, either the footing will need to be deepened so that the pipe is above the foundation plane of influence or the utility trench will need to be backfilled with sand-cement slurry or lean concrete within the influence zone. Sand-cement slurry used within foundation influence zones should have a minimum compressive strength of 75 psi.

Footing excavations should be filled as soon as possible or be kept moist until concrete placement by regular sprinkling to prevent desiccation. A Cornerstone representative should observe all footing excavations prior to placing reinforcing steel and concrete. If there is a significant schedule delay between our initial observation and concrete placement, we may need to re-observe the excavations.

7.4 DRILLED PIER FOUNDATIONS – CARETAKER RESIDENCE, MAINTENANCE BUILDING, VETERINARY/ADMINISTRATION BUILDING, AND FIRE PREVENTION AND DOMESTIC WATER TANK PADS AND PUMP STATIONS

As discussed, the proposed caretaker residence, maintenance building, and fire prevention and domestic water tank pads and associated pump stations sit near/at the top of a slope while the veterinary/admin building is in close proximity to the landslide labeled Qls #2 on our Site Plan. We recommend that these structures be supported on drilled, cast-in-place, straight-shaft friction piers with a structural slab spanning between. The piers should have a minimum diameter of 18 inches and extend to a depth of at least 10 feet into bedrock beneath the fill, residual soils, and colluvium. In areas of the building where there will be cuts into the claystone greater than 3 feet, we recommend the minimum pier embedment be increased to 15 feet into bedrock. Adjacent piers 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 may be used provided the subgrade soils are prepared in accordance with the "Earthwork" section.

7.4.1 Vertical Capacity and Estimated Settlement

The vertical capacity of the piers may be designed based on an allowable skin friction of 750 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 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 450 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 should not exceed ½-inch to mobilize static capacities and post-construction differential settlement between each pier should not exceed ¼-inch due to static loads.

7.4.2 Lateral Capacity

Lateral loads exerted on the structure 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. The lateral pressure may be increased up to a maximum uniform pressure of 4,000 psf at depth. The upper 5 feet of soil should be neglected when determining lateral capacity due to the sloping ground conditions. The structural engineer should apply an appropriate factor of safety to the ultimate passive pressures.

7.4.3 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 groundwater 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.

Based on our explorations, medium dense to dense clayey sands were encountered at the site. We performed our borings with hollow-stem auger drilling equipment and as such were not able to evaluate the potential for caving soils, which can create difficult drilling conditions. Additionally, the soils are generally fill material and may contain adverse materials. The contractor should plan on encountering potentially caving soils and other materials that may require casing or other stability measures to prevent caving and sloughing into the pier foundations.

Contractors should note that embedment is into bedrock materials, and difficult drilling conditions may occur. Equipment capable of excavating the rock materials will be required. Equipment that includes rock bits, core barrels, downhole percussion hammers, and techniques such as pilot holes may also be required and should be anticipated.

SECTION 8: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

8.1 SLABS-ON-GRADE

The structural engineer should determine the appropriate slab reinforcement for the loading requirements and considering the expansion potential of the underlying soils. For unreinforced concrete slabs, ACI 302.1R recommends limiting control joint spacing to 24 to 36 times the slab thickness in each direction, or a maximum of 18 feet.

8.1.1 Animal Barn

As the Plasticity Index (PI) of the surficial soils ranges up to 34, the proposed interior slabs-on-grade should be at supported on at least 18 inches of non-expansive fill (NEF) to reduce the potential for slab damage due to soil heave. 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.

8.1.2 Cat and Dog Enclosures

As the Plasticity Index (PI) of the surficial soils ranges up to 34, the proposed slabs-on-grade should be supported on at least 12 inches of non-expansive fill (NEF) to reduce the potential for slab damage due to soil heave. Per discussions with the design team, we understand that the cat and dog enclosures are not sensitive structures and some movement of the slabs-on-grade might occur and is considered acceptable. 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.

8.1.3 Maintenance Buildings

As the Plasticity Index (PI) of the surficial soils ranges up to 34, the proposed slabs-on-grade should be supported on at least 18 inches of non-expansive fill (NEF) to reduce the potential for slab damage due to soil heave. 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.

8.1.4 Fire Water Storage Tank

As discussed above, we recommend that the fire water storage tank be constructed on a built up level pad and slab-on-grade supported on drilled piers due to the close proximity to steep slopes to the north. As the Plasticity Index (PI) of the surficial soils ranges up to 34, the proposed slab-on-grade should be supported on at least 18 inches of non-expansive fill (NEF) to reduce the potential for slab damage due to soil heave. The NEF layer should be constructed over subgrade prepared in accordance with the recommendations in the “Earthwork” section of this report. 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.

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 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 crushed rock should be placed below the vapor retarder and consolidated in place with vibratory equipment. The mineral aggregate shall be of such size that the percentage composition by dry weight as determined by laboratory sieves will conform to the following gradation:

Sieve Size	Percentage Passing Sieve
1"	100
¾"	90 – 100
No. 4	0 - 10

  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 EXTERIOR FLATWORK

Exterior flatwork, such as pedestrian walkways, patios, driveways, and sidewalks, may experience seasonal movement due to the native expansive soils; therefore, some cracking or vertical movement of conventional slabs should be anticipated where imported fill is not planned in flatwork areas. There are several alternatives for mitigating the impacts of expansive soils beneath concrete flatwork. We are providing recommendations to reduce distress to concrete flatwork that includes moisture conditioning the subgrade soils, using non-expansive fill, and providing adequate construction and control joints to control cracks that do occur. It should be noted that minor slab movement or localized cracking and/or distress could still occur.

- The minimum recommendation for concrete flatwork constructed on moderately to highly expansive soils is to properly prepare the clayey soils prior to placing concrete. This is typically achieved by scarifying, moisture conditioning, and re-compacting the subgrade soil. Subgrade soil should be moisture conditioned to at least 3 percent over the laboratory optimum and compacted using moderate compaction effort to a relative compaction of 87 to 92 percent (ASTM Test Method D1557). Since the near surface soils may have been previously compacted and tested, the subgrade soils could possibly be moisture conditioned by gradually wetting the soil, depending on the time of year slab construction occurs. This should not include flooding or excessively watering the soil, which would likely result in a soft, unstable subgrade condition, and possible delays in the construction while waiting for the soil to dry out. In general, the subgrade should be relatively firm and non-yielding prior to construction.

- Concrete flatwork, excluding pavements that would be subject to wheel loads, should be at least 4 inches thick and underlain by at least 12 inches of non-expansive fill. Non-expansive fill may include aggregate base, crushed rock, or imported soil with a PI of 15 or less. Non-expansive fill should be compacted to at least 90 percent relative compaction. 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.

- We recommend a maximum control joint spacing of about 2 feet in each direction for each inch of concrete thickness and a construction joint spacing of 10 to 12 feet. Construction joints that abut the foundations or garage slabs should include a felt strip, or approved equivalent, that extends the full depth of the exterior slab. This will help to reduce the potential for permanent vertical offset between the slabs due to friction between the concrete edges. We recommend that exterior slabs be isolated from adjacent foundations.

At the owner’s option, if desired to reduce the potential for vertical offset or widening of concrete cracks, consideration should be given to using reinforcing steel, such as No. 3 rebar spaced at 18 inches on center each direction.

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 the results of the laboratory testing performed on a surficial sample collected from the proposed pavement area and engineering judgment considering the variable surface conditions.

Table 4: 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	9.5	12.0
5.0	3.0	10.0	13.0
5.5	3.0	12.0	15.0
6.0	3.5	12.5	16.0
6.5	4.0	14.0	18.0

*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 use 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. Another alternative is to lime treat the subgrade. We also recommend limiting cuts to 3 feet to reduce the potential for heave of the claystone bedrock.

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

Allowable ADTT	Minimum PCC Thickness (inches)
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 4 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 surficial 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 native 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 with level backfill be designed for the following pressures:

Table 6: Recommended Lateral Earth Pressures

Sloping Backfill Inclination (horizontal:vertical)	Lateral Earth Pressure*	
	Unrestrained – Cantilever Wall	Restrained – Braced Wall
Level	45 pcf	45 pcf + 8H**
2:1	65 pcf	65 pcf + 8H**

* 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

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

10.2.1 Basement Walls

The 2019 California Building Code (CBC) states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. We checked seismic earth pressures for the proposed restrained and unrestrained (cantilever) retaining walls in accordance with CBC 1803.5.12 and ASCE 7-16 Section 11.8.3 using the Design level earthquake. We developed seismic earth pressures for the proposed basement using interim recommendations generally based on refinement of the Mononobe-Okabe method (Lew et al., SEAOC 2010).

Because the veterinary/admin building basement walls will be at or greater than 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. Basement walls are not free to deflect, and should therefore be designed for static conditions as a restrained wall, which is also a CBC requirement. Based on current recommendations for seismic earth pressures, it appears that active earth pressures plus a seismic increment exceed the restrained (i.e. at-rest), static wall earth pressures. Therefore, we recommend checking the walls for the seismic condition in accordance with the interim recommendations of the above referenced paper and the 2013 CBC.

The CBC prescribes basic load combinations for structures, components and foundations with the intention that their design strength equals or exceeds the effects of the factored loads. With respect to the load from lateral earth pressure and ground water pressure, the CBC prescribes the basic combinations shown in CBC equations 16-2 and 16-7 below.

$1.2(D + F) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$ [Eq. 16-2]

In Eq. 16-2: H - should represent the total static lateral earth pressure, which for the basement wall will be restrained (use 45 pcf + 8H psf)

$0.9(D + F) + 1.0E + 1.6H$ [Eq. 16-7]

In Eq. 16-7: H - should represent the static “active” earth pressure component under seismic loading conditions (use 45 pcf)

E - should represent the seismic increment component in Eq. 16-7, a triangular load with a resultant force of $8H^2$, which should be applied one third of the height up from the base of the wall (and which can also be expressed as an equivalent fluid pressure equal to 24 pcf).

The interim recommendations in the SEAOC paper more appropriately split out "active" earth pressure (and not the restrained "at-rest" pressure) from our report and provide the total seismic increment so that different load factors can be applied in accordance with different risk levels.

10.2.2 Site Walls

The 2019 CBC states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. At this time, we are not aware of any site retaining walls for the project. However, minor landscaping walls (i.e. walls 6 feet or less in height) may be proposed. In our opinion, design of these walls for seismic lateral earth pressures in addition to static earth pressures is not warranted.

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). 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 95 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.

10.5 FOUNDATIONS

Retaining walls may be supported on a continuous spread footing or 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 Peninsula Humane Society & SPCA specifically to support the design of the Peninsula Humane Society Animal Sanctuary project in Loma Mar, 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 groundwater conditions encountered during our subsurface exploration. If variations or unsuitable conditions are encountered during construction, Cornerstone must be contacted to provide supplemental recommendations, as needed.

Peninsula Humane Society & SPCA may have provided Cornerstone with plans, reports and other documents prepared by others. Peninsula Humane Society & SPCA 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

Aagaard, B.T., Blair, J.L., Boatwright, J., Garcia, S.H., Harris, R.A., Michael, A.J., Schwartz, D.P., and DiLeo, J.S., 2016, Earthquake outlook for the San Francisco Bay region 2014–2043 (ver. 1.1, August 2016): U.S. Geological Survey Fact Sheet 2016–3020, 6 p., http://dx.doi.org/10.3133/fs20163020.

ATC Hazards by Location, *Hazards by Location*, 2020, *https://hazards.atcouncil.org/*

Brabb, E.E., 1970, Preliminary geologic map of the central Santa Cruz Mountains, California: U.S. Geological Survey, Open-File Report OF-70-36, scale 1:62,500.

Brabb, E.E., 1980, Preliminary geologic map of the La Honda and San Gregorio quadrangles, San Mateo County, California: U.S. Geological Survey, Open-File Report OF-80-245, scale 1:24,000.

Brabb, E.E., and Pampeyan, E.H., 1983, Geologic map of San Mateo County, California: U.S. Geological Survey, Miscellaneous Investigations Series Map I-1257-A, scale 1:62,500.

Brabb, E.E., 1993, Preliminary geologic map of the onshore part of the Palo Alto 1:100,000 quadrangle, California: U.S. Geological Survey, Open-File Report OF-93-271, scale 1:62,500.

Brabb, E.E., and Olson, J.A., 1986, Map showing faults and earthquake epicenters in San Mateo County, California: U.S. Geological Survey, Miscellaneous Investigations Series Map I-1257-F, scale 1:62,500.

Brabb, E. E., Graymer, R.W. Jones, D.L., 1998, Geology of the Palo Alto 30 X 60 Minute Quadrangle, California: a digital database, U.S. Geological Survey Open File Report 98-348.

California Building Code, 2019, Structural Engineering Design Provisions, Vol. 2.

California Geological Survey, (Updated January 26, 2016), Landslide Inventory and Deep-

Seated Landslide Susceptibility Interactive Mapping (Beta version) http://maps.conservation.ca.gov/cgs/lsi/, accessed on April 11, 2019

DMG Staff, 2000, Digital images of Official Maps of Alquist-Priolo Earthquake Fault Zones of California, central coast region: California Division of Mines and Geology, CD 2000-004, scale 1:24,000.

Ellen, Stephen D, and Weiczorek, 1988, Landslides, Floods and Marine Effects of the Storm of January 3-5, 1982, in the San Francisco Bay Region, California, U.S. Geological Survey Paper 1434.

Ellen, S.D., Mark, R.K., Wieczorek, G.F., Wentworth, C.M., Ramsey, D.W., May, T.E., Graham, S.E., Beukelman,G.S., Barron, A.D., and Graymer, R.W., 1997, Map showing principal debris-flow source areas in the San Francisco Bay region, California: U.S. Geological Survey, Open-File Report OF-97-745-E, scale 1:125,000.

Ellis, W.L., Harp, E.L., Arnal, C.H., and Godt, J.W., 1999, Map showing locations of damaging landslides in Santa Clara County, California, resulting from 1997-98 El Nino rainstorms: U.S. Geological Survey, Miscellaneous Field Studies Map MF-2325-J, scale 1:125,000.

Haugerud, R.A., and Ellen, S.D., 1990, Coseismic ground deformation along the northeast margin of the Santa Cruz Mountains: U.S. Geological Survey Open-File Report 90-274, p. 32-37.

Hitchcock, C. S., Kelson, K. I., and Thompson, S. C., 1994, Geomorphic Investigations of Deformation Along the Northeastern Margin of the Santa Cruz Mountains, U. S. Geological Survey Open-File Report OFR 94-187.

Lawson, A.C., chairman, 1908, The California earthquake of April 18, 1906; Report of the State Earthquake Investigation Commission: Carnegie Institute Washington Publication 87, v. I, 451 p. (Reprinted, 1969).

Mark, R.K., 1992, Map of debris-flow probability, San Mateo County, California: U.S. Geological Survey, Miscellaneous Investigations Series Map I-1257-M, scale 1:62,500.
Portland Cement Association, 1984, Thickness Design for Concrete Highway and Street Pavements: report.

Pradell, D., 1988, Procedure to Evaluate Earthquake-Induced Settlements in Dry Sandy Soils, Journal of Geotechnical and Environmental Engineering, April 1998, p. 364 – 368 and Errata October 1998 p. 1048.

Rogers, T.H., 1971, Environmental geologic analysis, Santa Cruz mountain study area, Santa Clara County, California. California Division of Mines and Geology open- file report 72-21.

State of California Department of Transportation, 2018, Highway Design Manual, Sixth Edition, December 14, 2018.

Tokimatsu, K., and Seed, H. Bolton, 1987, Evaluation of Settlements in Sands due to Earthquake Shaking, ASCE Journal of Geotechnical Engineering, Vol. 113, August 1987, pp. 861-878.

San Mateo County, 2008, Hazards Mitigation Maps: on-line site at http://www.co.sanmateo.ca.us/smc/department/home/0,,5557771_5558929_436489912,00.html

U.S. Geological Survey, California Geological Survey, 2006, Quaternary fault and fold database for the United States, accessed March 30, 2019, http//earthquake.usgs.gov/hazards/qfaults/.

Wieczorek, G.F., 1982, Map showing recently active and dormant landslides near La Honda, central Santa Cruz Mountains, California: U.S. Geological Survey, Miscellaneous Field Studies Map MF-1422, scale 1:48,000.

Working Group on California Earthquake Probabilities, 2015, The Third Uniform California Earthquake Rupture Forecast, Version 3 (UCERF), U.S. Geological Survey Open File Report 2013-1165 (CGS Special Report 228). KMZ files available at: www.scec.org/ucerf/images/ucerf3_timedep_30yr_probs.kmz

Aerial Photos Reviewed

Vertical photos:
1953, 1956, 1960, 1968, 1980, 1982, 1991, 2005, 2009, 2010, 2012, 2014, 2016

Stereo Aerial Photos:
March 30, 1931, black and white, flight C-1471, frames 118, 119, scale: 1:18,000.

April 24, 1948, black and white, flight CDF5, frames 1-58, scale: 1:20,000.

May 1, 1965, black and white, flight CAS-65-130, frame 3-56, scale: 1:12,000.

APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using track-mounted, hollow-stem, limited-access auger drilling equipment. Seven 6½-inch-diameter exploratory borings were drilled on January 20 and 21, 2020 to depths of 15 to 21½ feet. Two 3-inch diameter exploratory hand auger borings were drilled on January 21, 2020, to a depth of 4 to 4½ feet. The approximate locations of exploratory borings are shown on the Site Plan, Figure 2. 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.

Boring locations were approximated using existing site boundaries, and other site features as references. Boring elevations were not determined. The locations of the borings should be considered accurate only to the degree implied by the method used.

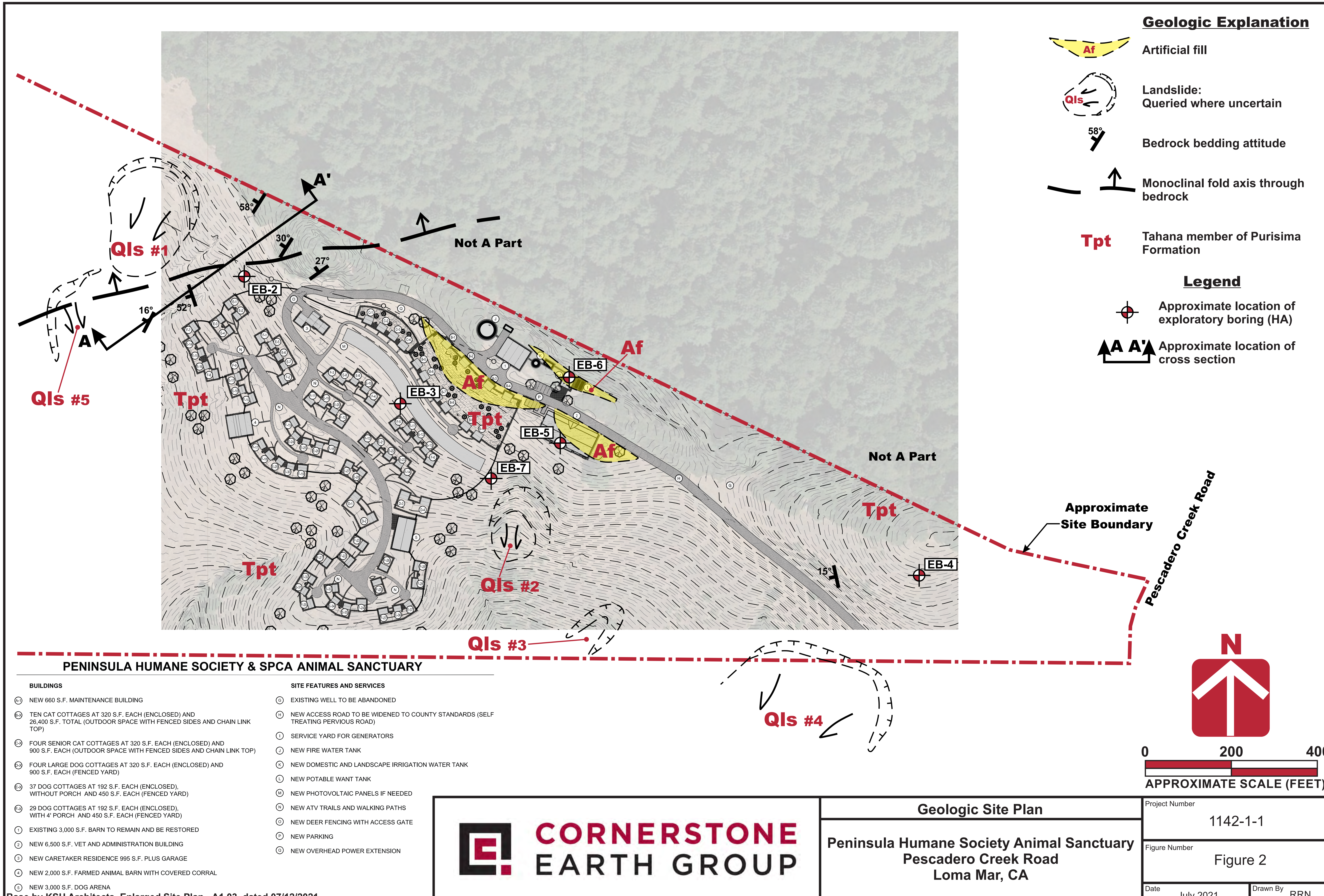
Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. The standard penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The 2-inch O.D. 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 logs.

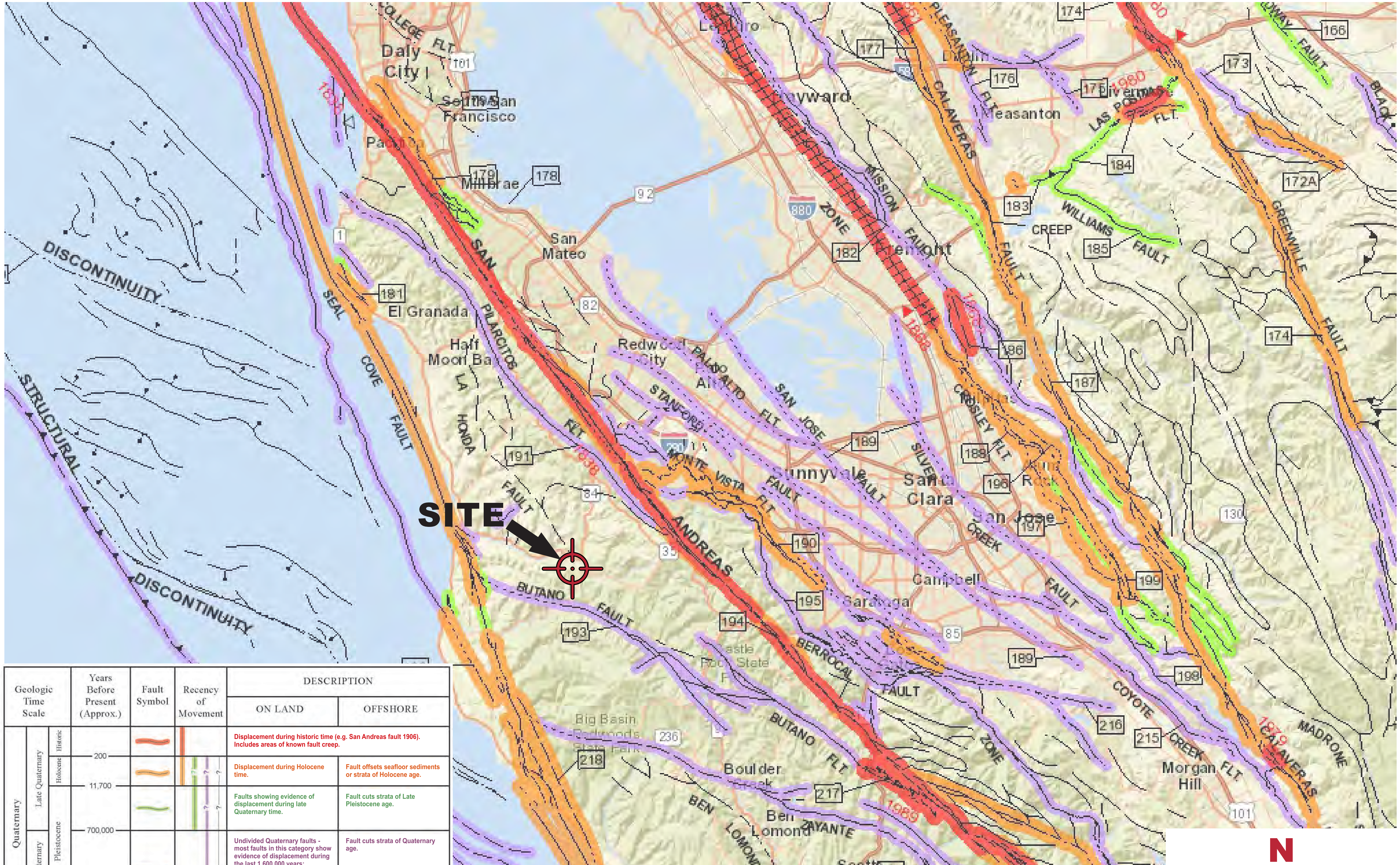
Field tests included an evaluation of the unconfined compressive strength of the soil samples using a pocket penetrometer device. The results of these tests are presented on the individual boring logs at the appropriate sample depths.

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

CORNERSTONE EARTH GROUP

Vicinity Map	Project Number 1142-1-1
Peninsula Humane Society Animal Sanctuary Pescadero Creek Road Loma Mar, CA	Figure Number Figure 1
	Date July 2021 / Drawn By RRN


Geologic Explanation
Af
Artificial fill
Qls
Landslide:
Queried where uncertain
58°
Bedrock bedding attitude
Monoclinal fold axis through bedrock
Tpt
Tahana member of Purisima Formation
Legend
Approximate location of exploratory boring (HA)
A A'
Approximate location of cross section
A'
A
58°
30°
27°
16°
52°
15°
Qls #1
Qls #2
Qls #3
Qls #4
Qls #5
Tpt
Af
EB-2
EB-3
EB-4
EB-5
EB-6
EB-7
Not A Part
Not A Part
Approximate Site Boundary
Pescadero Creek Road
N
0
200
400
APPROXIMATE SCALE (FEET)
PENINSULA HUMANE SOCIETY & SPCA ANIMAL SANCTUARY
BUILDINGS
A-1 NEW 660 S.F. MAINTENANCE BUILDING
B-X TEN CAT COTTAGES AT 320 S.F. EACH (ENCLOSED) AND 26,400 S.F. TOTAL (OUTDOOR SPACE WITH FENCED SIDES AND CHAIN LINK TOP)
C-X FOUR SENIOR CAT COTTAGES AT 320 S.F. EACH (ENCLOSED) AND 900 S.F. EACH (OUTDOOR SPACE WITH FENCED SIDES AND CHAIN LINK TOP)
D-X FOUR LARGE DOG COTTAGES AT 320 S.F. EACH (ENCLOSED) AND 900 S.F. EACH (FENCED YARD)
E-X 37 DOG COTTAGES AT 192 S.F. EACH (ENCLOSED), WITHOUT PORCH AND 450 S.F. EACH (FENCED YARD)
F-X 29 DOG COTTAGES AT 192 S.F. EACH (ENCLOSED), WITH 4' PORCH AND 450 S.F. EACH (FENCED YARD)
1 EXISTING 3,000 S.F. BARN TO REMAIN AND BE RESTORED
2 NEW 6,500 S.F. VET AND ADMINISTRATION BUILDING
3 NEW CARETAKER RESIDENCE 995 S.F. PLUS GARAGE
4 NEW 2,000 S.F. FARMED ANIMAL BARN WITH COVERED CORRAL
5 NEW 3,000 S.F. DOG ARENA
SITE FEATURES AND SERVICES
G EXISTING WELL TO BE ABANDONED
H NEW ACCESS ROAD TO BE WIDENED TO COUNTY STANDARDS (SELF TREATING PERVIOUS ROAD)
I SERVICE YARD FOR GENERATORS
J NEW FIRE WATER TANK
K NEW DOMESTIC AND LANDSCAPE IRRIGATION WATER TANK
L NEW POTABLE WANT TANK
M NEW PHOTOVOLTAIC PANELS IF NEEDED
N NEW ATV TRAILS AND WALKING PATHS
O NEW DEER FENCING WITH ACCESS GATE
P NEW PARKING
Q NEW OVERHEAD POWER EXTENSION
Base by KSH Architects, Enlarged Site Plan - A1.03, dated 07/12/2021
CORNERSTONE EARTH GROUP
Geologic Site Plan
Peninsula Humane Society Animal Sanctuary
Pescadero Creek Road
Loma Mar, CA
Project Number
1142-1-1
Figure Number
Figure 2
Date
July 2021
Drawn By
RRN



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION ON LAND	DESCRIPTION OFFSHORE
Quaternary	Late Quaternary	Historic				Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
			200				
		Holocene				Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
			11,700				
		Pleistocene				Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
			700,000				
	Early Quaternary					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 Pilo-Pleistocene age.	Fault cuts strata of Quaternary age.
			1,600,000				
Pre-Quaternary						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.
			4.5 billion (Age of Earth)				

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

APPROXIMATE SCALE (MILES)

Regional Fault Map

Peninsula Humane Society Animal Sanctuary
Pescadero Creek Road
Loma Mar, CA

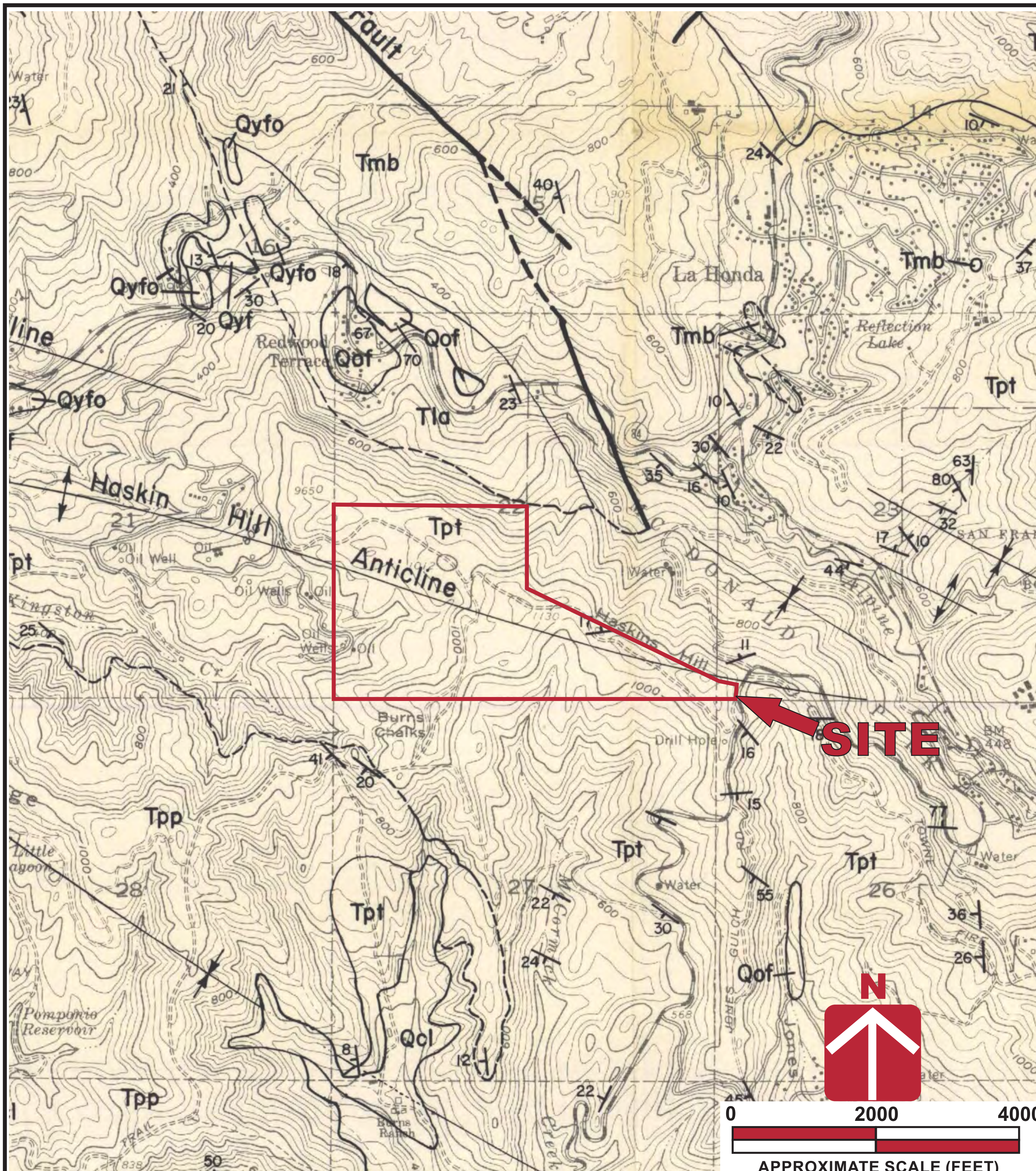
CORNERSTONE EARTH GROUP

Project Number 1142-1-1

Figure Number Figure 3

Date July 2021

Drawn By RRN



Geologic Units

Qof Older Aluvial Fan and Stream deposits (Pleistocene)

Tpt Tahana Member of Purisima Formation (upper Mioceene and Pliocene)

Tla Lambert Shale (Oligocene and lower Miocene)

Explanation

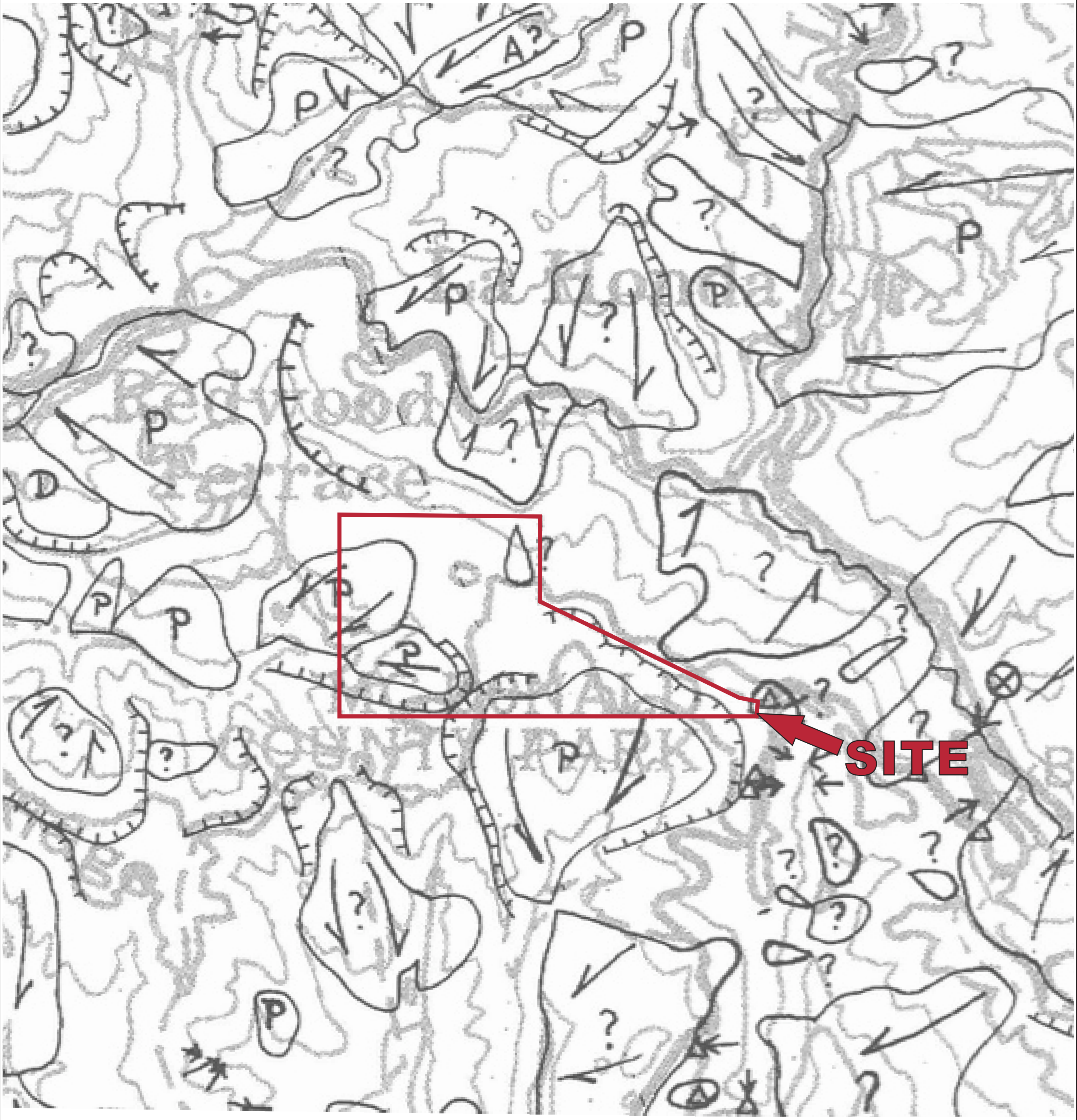
Contact- dashed where approximate, dotted where concealed

Fault- dashed where approximate, dotted where concealed

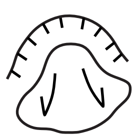
Anticlinal fold axis

Base by Earl E. Brabb, Preliminary Geologic Map of the La Honda and San Gregorio Quadrangles, dated 1980, Palo Alto 30'x60' Quad.

CORNERSTONE EARTH GROUP	**Vicinity Geologic Map**	Project Number 1142-1-1
	Peninsula Humane Society Animal Sanctuary Pescadero Creek Road Loma Mar, CA	Figure Number Figure 4
		Date July 2021 / Drawn By FLL/RRN



Explanation



Landslide: P = Probable, queried where interpretation is uncertain

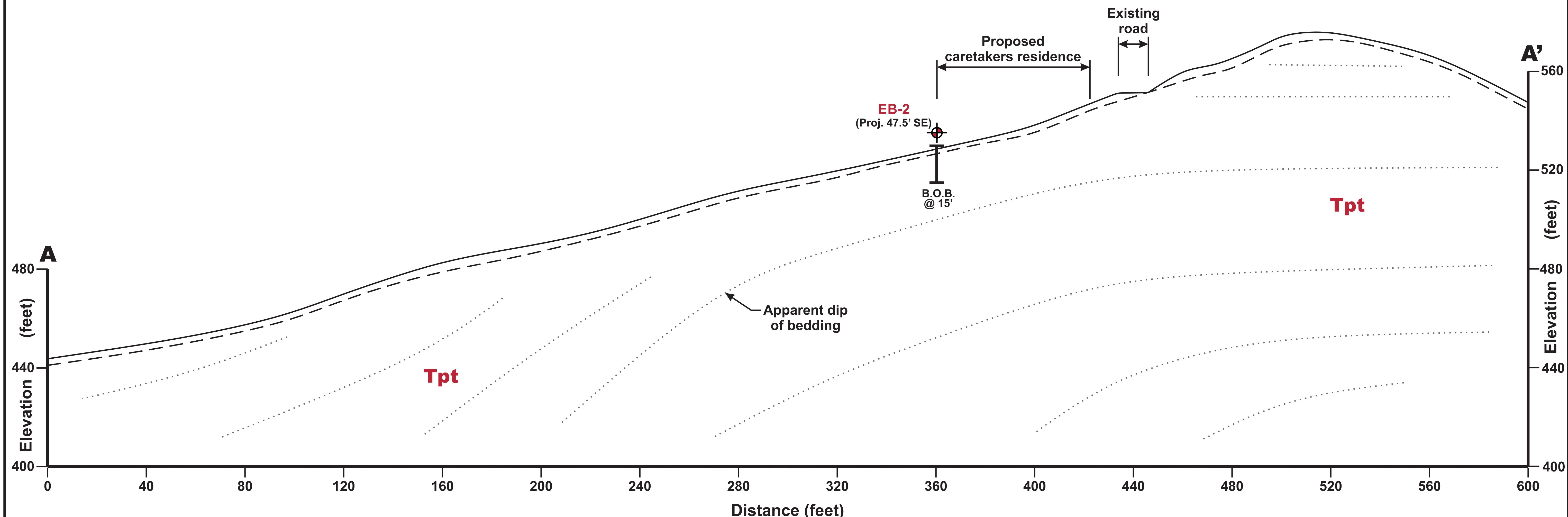
N

0 2000 4000

APPROXIMATE SCALE (FEET)

Base by Earl E. Brabb and Pampeyan, 1972.

CORNERSTONE EARTH GROUP	Published Landslide Map	Project Number 1142-1-1
	Peninsula Humane Society Animal Sanctuary Pescadero Creek Road Loma Mar, CA	Figure Number Figure 5
		Date July 2021 / Drawn By RRN



Section A-A'

(View Looking Northwest)
1"=40' H:V

Explanation

Geologic Units

Tpt Tahana member of Purisima Formation

Symbols

Approximate location of exploratory boring (EB)

Notes:
1) Surficial fills associated with existing pavements, landscaping or utilities are not shown.
2) The subsurface profile is conceptual and is based on limited subsurface data obtained from widely spaced borings. Actual subsurface conditions may vary significantly between borings.
3) See Figure 2 for location of cross section.

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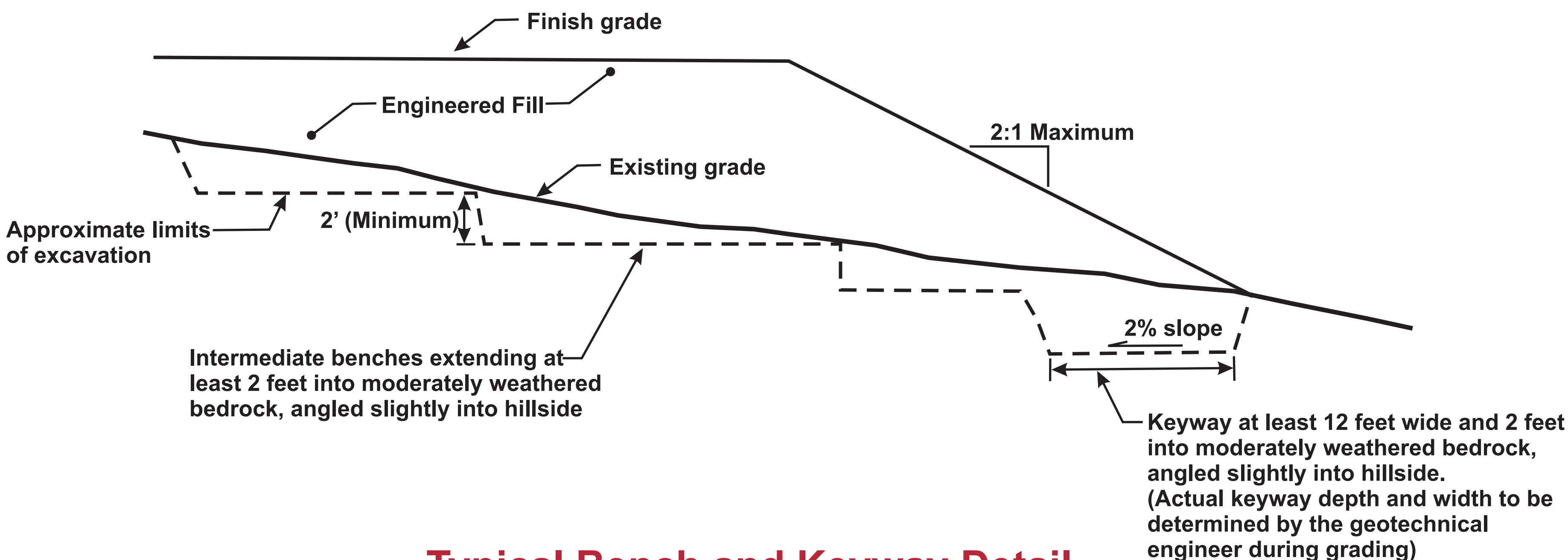
EARTH GROUP

Geologic Cross Section A-A'	Project Number 1142-1-1	
Peninsula Humane Society Animal Sanctuary Pescadero Creek Road Loma Mar, CA	Figure Number Figure 6	
	Date July 2021	Drawn By RRN

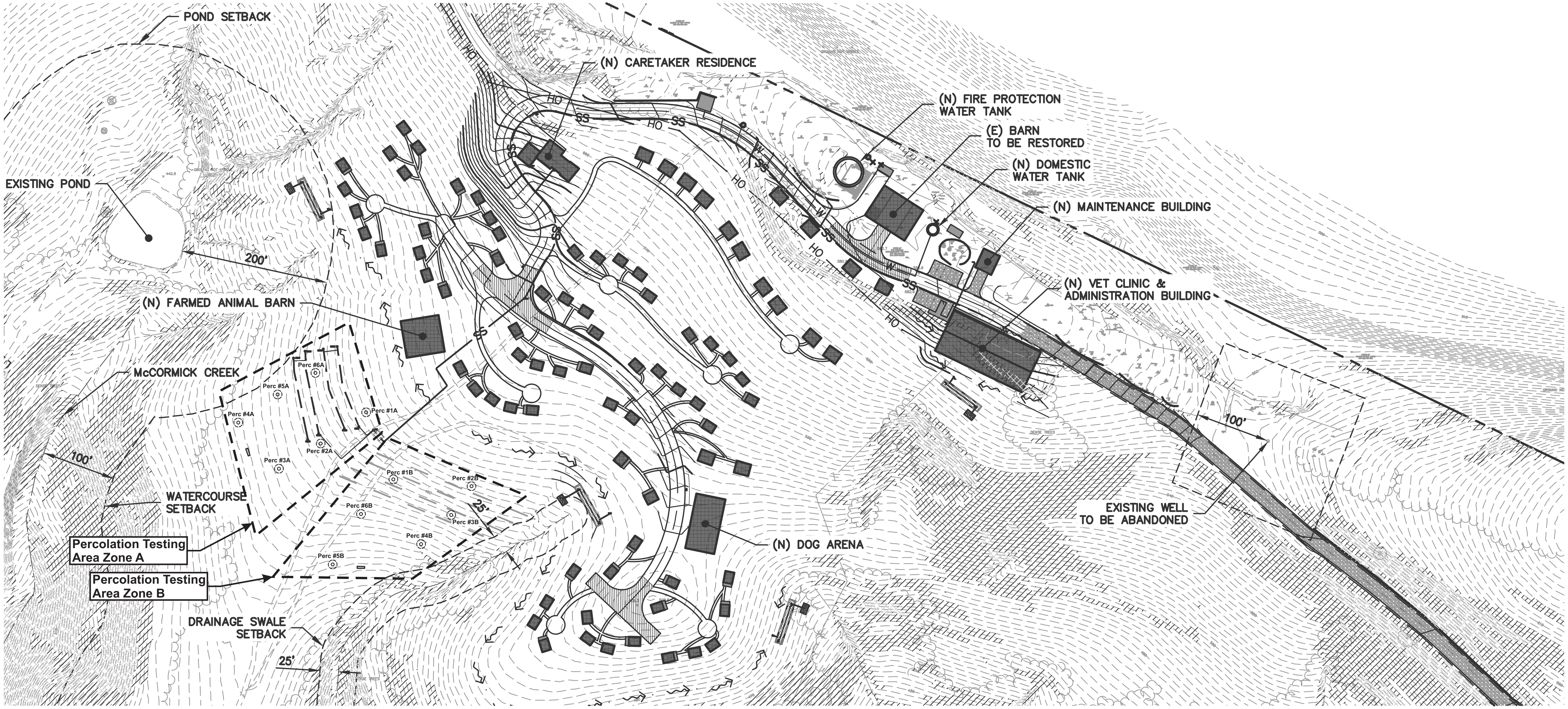


Typical Bench and Keyway Detail

Not to scale

Note: Fill slopes should be over-built at least 18 to 24 inches and trimmed to expose compacted fill.

CORNERSTONE EARTH GROUP	**Typical Bench and Keyway Detail**	Project Number 1142-1-1
	Peninsula Humane Society Animal Sanctuary **Pescadero Creek Road** **Loma Mar, CA**	Figure Number Figure 7
		Date July 2021 / Drawn By RRN



Proposed Septic System Leachfield Area

Legend

- BOUNDARY
- PRIMARY LEACH LINE
- EXPANSION LEACH LINE
- EXISTING LEACH LINE
- EXISTING LEACH LINE TO BE REMOVED
- PERCOLATION TESTING LOCATION

N

0 150 300

APPROXIMATE SCALE (FEET)

Base by Lea & Braze Engineering, Inc., Septic Construction Plan - SS-1, dated 06/18/2021

Proposed Septic System Leachfield Areas

Peninsula Humane Society Animal Sanctuary
Pescadero Creek Road
Loma Mar, CA

Project Number 1142-1-1

Figure Number Figure A-1

Date July 2021

Drawn By RRN

CORNERSTONE EARTH GROUP

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 41 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 17 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.

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-10)

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

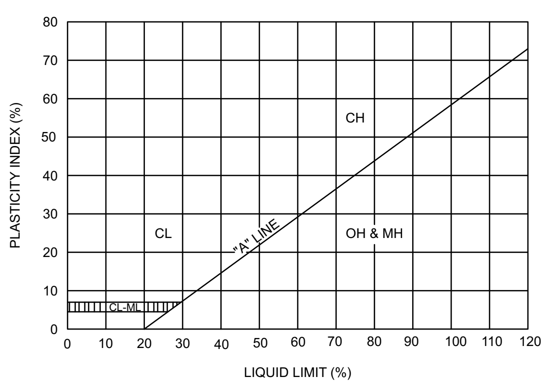
SAMPLER TYPES

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.

CORNERSTONE EARTH GROUP

LEGEND TO SOIL DESCRIPTIONS

Figure Number A-1

HARDNESS

Soft – Reserved for plastic material alone.
Low hardness – Can be gouged deeply or carved easily with a knife blade.
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.
Hard – Can be scratched with difficulty: scratch produces little powder and is often faintly visible.
Very hard – Cannot be scratched with knife blade: leaves a metallic streak.

STRENGTH

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

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.
Moderate – Slight change or partial decomposition of minerals: little disintegration: cementation little to unaffected. Moderate to occasionally intense discoloration. Moderately coated fractures.
Little – No megascopic decomposition of minerals: little or no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains or fracture surfaces.
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

BORING NUMBER EB-1

CORNERSTONE EARTH GROUP

PROJECT NAME Haskins Ridge
PROJECT NUMBER 1142-1-1
PROJECT LOCATION La Honda, CA

DATE STARTED 1/20/20 **DATE COMPLETED** 1/20/20
GROUND ELEVATION **BORING DEPTH** 16.5 ft.
DRILLING CONTRACTOR Cuesta Geo
LATITUDE **LONGITUDE**
DRILLING METHOD MPP LAD Track Rig, 6½ inch Hollow-Stem Auger
GROUNDWATER LEVELS:
LOGGED BY CSH
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0, 2.0, 3.0, 4.0)

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
	0		**Fat Clay with Sand (CH) [Residual Soil]** medium stiff, moist, dark brown to brown, fine sand, high plasticity Liquid Limit = 57, Plastic Limit = 23	7	MC-1B	95	28	34		○
			becomes very stiff	13	MC-2B	95	28			○
	5		**Claystone [Tpt]** low hardness, weak, deep weathering, olive gray with brown mottles, moderate to high plasticity Liquid Limit = 95, Plastic Limit = 35	10	MC-3A	71	43	60		○ >4.5
				6	SPT-4		43			
	10									
				16	MC-5B	77	46			
	15			19	SPT					
			Bottom of Boring at 16.5 feet.							
	20									
	25									

BORING NUMBER EB-2

CORNERSTONE EARTH GROUP

PROJECT NAME Haskins Ridge
PROJECT NUMBER 1142-1-1
PROJECT LOCATION La Honda, CA

DATE STARTED 1/20/20 **DATE COMPLETED** 1/20/20
GROUND ELEVATION **BORING DEPTH** 15 ft.
DRILLING CONTRACTOR Cuesta Geo
LATITUDE **LONGITUDE**
DRILLING METHOD MPP LAD Track Rig, 6½ inch Hollow-Stem Auger
GROUNDWATER LEVELS:
LOGGED BY CSH
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0

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
	0		**Fat Clay with Sand (CH) [Colluvium]** medium stiff, moist, dark brown to brown, fine sand, high plasticity							
				19	MC-1A	93	30			○
			Sandy Claystone [Tpt] low hardness, weak, deep weathering, olive gray with reddish brown mottles, fine sand, high plasticity							
				36	MC-2A	95	26			
	5									
				22	SPT-3		21			
			Claystone [Tpt] low hardness, weak, deep weathering, olive gray with brown mottles, high plasticity							
				20	SPT-4		34			
	10									
				34	MC-5B	80	44			
	15		Bottom of Boring at 15.0 feet.							
	20									
	25									

BORING NUMBER EB-3

CORNERSTONE EARTH GROUP

PROJECT NAME Haskins Ridge
PROJECT NUMBER 1142-1-1
PROJECT LOCATION La Honda, CA

DATE STARTED 1/20/20 **DATE COMPLETED** 1/20/20
DRILLING CONTRACTOR Cuesta Geo
DRILLING METHOD MPP LAD Track Rig, 6½ inch Hollow-Stem Auger
LOGGED BY CSH
NOTES

GROUND ELEVATION **BORING DEPTH** 15 ft.
LATITUDE **LONGITUDE**
GROUNDWATER LEVELS:
AT TIME OF DRILLING Not Encountered
AT END OF DRILLING Not Encountered

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UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (ft)	DEPTH (ft)	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
	0	**Fat Clay with Sand (CH) [Colluvium]** stiff, moist, dark brown to brown, fine sand, high plasticity	7	MC-1B	95	30			○ 1.0
			13	MC-2B	85	32			○ 1.8
	4	**Sandy Claystone [Tpt]** low hardness, weak, deep weathering, gray with brown mottles, high plasticity	24	SPT-3		37			
	8	**Silty Sandstone [Tpt]** low hardness, weak, deep weathering, gray with brown mottles, fine sand	37	SPT-4		19			
			40	SPT-5		18			
	15	Bottom of Boring at 15.0 feet.							

BORING NUMBER EB-4

CORNERSTONE EARTH GROUP

PROJECT NAME Haskins Ridge
PROJECT NUMBER 1142-1-1
PROJECT LOCATION La Honda, CA

DATE STARTED 1/21/20 **DATE COMPLETED** 1/21/20
GROUND ELEVATION **BORING DEPTH** 15 ft.
DRILLING CONTRACTOR Cuesta Geo
LATITUDE **LONGITUDE**
DRILLING METHOD MPP LAD Track Rig, 6½ inch Hollow-Stem Auger
GROUNDWATER LEVELS:
LOGGED BY CSH
∇ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL; scale 1.0, 2.0, 3.0, 4.0

ELEVATION (ft)	DEPTH (ft)	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
	0	**Fat Clay with Sand (CH) [Residual soil]** medium stiff, moist, dark brown to brown, fine sand, high plasticity						
		Sandy Claystone [Tpt] low hardness, weak, deep weathering, gray with brown mottles, high plasticity	25	MC-1B	97	27		
			48	MC-2B	98	28		
	5	**Silty Sandstone [Tpt]** low hardness, weak, deep weathering, gray with brown mottles, fine sand	31	SPT-3		21		
		Sandy Claystone [Tpt] low hardness, weak, deep weathering, gray with brown mottles, high plasticity	73	SPT-4		22		
	10							
		Silty Sandstone [Tpt] low hardness, weak, deep weathering, gray with brown mottles, fine sand	60	SPT-5		19		
	15	Bottom of Boring at 15.0 feet.						
	20							
	25							

BORING NUMBER EB-5

CORNERSTONE EARTH GROUP

PROJECT NAME Haskins Ridge
PROJECT NUMBER 1142-1-1
PROJECT LOCATION La Honda, CA
DATE STARTED 1/21/20 **DATE COMPLETED** 1/21/20
GROUND ELEVATION **BORING DEPTH** 20 ft.
DRILLING CONTRACTOR Cuesta Geo
LATITUDE **LONGITUDE**
DRILLING METHOD MPP LAD Track Rig, 6½ inch Hollow-Stem Auger
GROUNDWATER LEVELS:
LOGGED BY CSH
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION
	0		**Fat Clay with Sand (CH) [Colluvium]** very stiff, moist, dark brown to brown, fine sand, high plasticity
			Sandy Claystone [Tpt] low hardness, weak, deep weathering, gray with reddish brown mottles, fine sand, high plasticity
	20		Bottom of Boring at 20.0 feet.

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
34	MC-1A	119	28			○ (approx. 2.5)
41	MC-2B	99	25			
18	SPT-3		29			
29	MC-4A		26			
17	SPT-5		34			
24	SPT-6		32			

BORING NUMBER EB-6

CORNERSTONE EARTH GROUP

PROJECT NAME Haskins Ridge
PROJECT NUMBER 1142-1-1
PROJECT LOCATION La Honda, CA

DATE STARTED 1/21/20 **DATE COMPLETED** 1/21/20
GROUND ELEVATION **BORING DEPTH** 21.5 ft.
DRILLING CONTRACTOR Cuesta Geo
LATITUDE **LONGITUDE**
DRILLING METHOD MPP LAD Track Rig, 6½ inch Hollow-Stem Auger
GROUNDWATER LEVELS:
LOGGED BY CSH
AT TIME OF DRILLING Not Encountered
NOTES
AT END OF DRILLING Not Encountered

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UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (ft)	DEPTH (ft)	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
	0	**Sandy Lean Clay (CL) [Fill]** very stiff, moist, dark brown with brown mottles, fine to coarse sand, moderate plasticity	14	MC-1B	98	23			○
	3.5	**Fat Clay (CH) [Colluvium]** stiff, moist, brown with gray mottles, fine sand, high plasticity	12	MC-2B	85	38			○
	5		7	SPT-3		34			○
	7.5	**Sandy Claystone [Tpt]** low hardness, weak, deep weathering, gray with brown mottles, fine sand, high plasticity	13	MC-4B	90	32			○
	14		10	SPT-5		32			
	19		16	SPT-6		31			
	20.5		20	SPT-7		31			
	21.5	Bottom of Boring at 21.5 feet.							

BORING NUMBER EB-7

CORNERSTONE EARTH GROUP

PROJECT NAME Haskins Ridge
PROJECT NUMBER 1142-1-1
PROJECT LOCATION La Honda, CA

DATE STARTED 1/20/20 **DATE COMPLETED** 1/20/20
GROUND ELEVATION **BORING DEPTH** 13.5 ft.
DRILLING CONTRACTOR Cuesta Geo
LATITUDE **LONGITUDE**
DRILLING METHOD MPP LAD Track Rig, 6½ inch Hollow-Stem Auger
GROUNDWATER LEVELS:
LOGGED BY CSH
AT TIME OF DRILLING Not Encountered
NOTES
AT END OF DRILLING Not Encountered

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UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0, 2.0, 3.0, 4.0)

ELEVATION (ft)	DEPTH (ft)	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
	0	**Clayey Sand (SC) [Colluvium]** medium dense, moist, dark brown to brown, fine sand							
		Silty Sandstone [Tpt] low hardness, weak, deep weathering, gray with brown mottles, fine sand	30	MC-1A	100	25			
			31	SPT-2		25			
	5		29	SPT-3		28			
		some interbedded claystone layers							
		Sandy Claystone [Tpt] low hardness, weak, deep weathering, gray with brown mottles, fine sand, high plasticity	31	SPT-4		24			
	10	some interbedded sandstone layers							
			21	SPT-5		28			
		Bottom of Boring at 13.5 feet.							
	15								
	20								
	25								

BORING NUMBER HA-1

CORNERSTONE EARTH GROUP

PROJECT NAME Haskins Ridge
PROJECT NUMBER 1142-1-1
PROJECT LOCATION La Honda, CA

DATE STARTED 1/21/20 **DATE COMPLETED** 1/21/20
GROUND ELEVATION **BORING DEPTH** 4.25 ft.
DRILLING CONTRACTOR N/A
LATITUDE **LONGITUDE**
DRILLING METHOD
GROUNDWATER LEVELS:
LOGGED BY
AT TIME OF DRILLING Not Encountered
NOTES
AT END OF DRILLING Not Encountered

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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		**Fat Clay with Sand (CH) [Residual Soil]** very stiff, moist, dark brown to brown, fine sand, high plasticity		GB-1		27			
			Sandy Claystone [Tpt] low hardness, weak, deep weathering, gray with brown mottles, fine sand, high plasticity		GB					
			Bottom of Boring at 4.3 feet.							
	5									
	10									
	15									
	20									
	25									

BORING NUMBER HA-2

CORNERSTONE EARTH GROUP

PROJECT NAME Haskins Ridge
PROJECT NUMBER 1142-1-1
PROJECT LOCATION La Honda, CA

DATE STARTED 1/21/20 **DATE COMPLETED** 1/21/20
GROUND ELEVATION **BORING DEPTH** 4.25 ft.
DRILLING CONTRACTOR N/A
LATITUDE **LONGITUDE**
DRILLING METHOD
GROUNDWATER LEVELS:
LOGGED BY
AT TIME OF DRILLING Not Encountered
NOTES
AT END OF DRILLING Not Encountered

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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
	0		**Fat Clay with Sand (CH) [Residual Soil]** very stiff, moist, dark brown to brown, fine sand, high plasticity		GB-1		23			
			Sandy Claystone [Tpt] low hardness, weak, deep weathering, gray with brown mottles, fine sand, high plasticity		GB-2		28			
			Bottom of Boring at 4.3 feet.							
	5									
	10									
	15									
	20									
	25									

UNDRAINED SHEAR STRENGTH, ksf legend: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL. Scale: 1.0 2.0 3.0 4.0

CORNERSTONE
EARTH GROUP

Type of Services	Geotechnical Investigation
Project Name	Peninsula Humane Society Animal Sanctuary
Location	12429 Pescadero Road Loma Mar, California
Client	Peninsula Humane Society & SPCA
Client Address	1450 Rollins Road Burlingame, California
Project Number	1142-1-1
Date	July 23, 2021

Prepared by

Stephen C. Ohlsen, P.E.
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Geotechnical Project Manager

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Quality Assurance Reviewer

CORNERSTONE EARTH GROUP

TABLE OF CONTENTS

CORNERSTONE
EARTH GROUP

Type of Services	**Geotechnical Investigation**
Project Name	**Peninsula Humane Society Animal Sanctuary**
Location	**12429 Pescadero Road Loma Mar, California**

SECTION 1: INTRODUCTION

This geotechnical report was prepared for the sole use of Peninsula Humane Society & SPCA for the Peninsula Humane Society Animal Sanctuary project in Loma Mar, California. The approximate location of the site is shown on the Vicinity Map, Figure 1. For our use, we were provided with the following documents:

- A set of architectural plans, titled, "Peninsula Humane Society Animal Sanctuary," prepared by KSH Architects, County of San Mateo Use Permit Submittal, dated July 12, 2021.
- A set of civil plans titled, "Peninsula Humane Society Animal Sanctuary, 12429 Pescadero Creek Road, Loma Mar, California," prepared by Lea & Braze Engineering, Inc., dated July 12, 2021.
- A set of landscape plans titled, "Peninsula Humane Society Animal Sanctuary," prepared by The Guzzardo Partnership Inc., County of San Mateo Use Permit Submittal, dated July 12, 2021.

1.1 PROJECT DESCRIPTION

The irregularly shaped 213-acre project site is located off of Pescadero Road in Loma Mar, California, about 3500 feet west of the intersection of Pescadero Road and Alpine Road. The site is bounded by Pescadero Road to the east and essentially undeveloped properties surrounding the project site. The site is mostly undeveloped, with a fire road crossing the site transverse to the hillside and an existing barn and caretaker residence to the north of the fire road. Based on the provided architectural plans, we understand that an animal sanctuary campus is planned consisting of a two-level administrator/visitor structure ("Building 2"), cat enclosures ("Buildings B and C"), the restored existing barn ("Building 1"), a new 2,000-square-foot farm animal barn with covered corral ("Building 4"), a 3,000-square-foot covered dog arena, access roads, new caretaker residence with garage ("Building 3"), several maintenance buildings ("Buildings A"), a fire prevention water storage tank and associated pump station, a service yard for generators and a new domestic and landscape irrigation tank and associated pump station, a solar array, and dog enclosures ("Buildings D, E, and F"). Additionally, an on-site septic system with leach field is proposed southwest of the dog enclosures and new animal barn. This development will be clustered along the ridge top and most of the remainder of the site will remain undeveloped with a new gravel road connecting the improvements.

It is expected that the structures will likely be single-story wood-frame structures. Appurtenant parking, utilities, access roads and paths, landscaping and other improvements necessary for site development are also planned.

Structural loads are not currently known for the proposed structures; however, structural loads are expected to be light and typical of similar type structures. Based on our preliminary discussions with the project structural engineer we understand that the cat and dog enclosures will be supported by slabs-on-grade, that the maintenance buildings and animal barns will likely be supported on shallow spread footing foundations, and that the administrator/veterinary building, and caretaker residence will likely be supported on drilled pier foundations. The tank foundation type is unknown at this time. Based on the results of our site investigation and lab testing, we are providing our geotechnical recommendations for these structures in this report.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated June 12, 2019 and consisted of field and laboratory programs to evaluate physical and engineering properties of the subsurface soils, 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 are presented below.

1.3 EXPLORATION PROGRAM

Field exploration consisted of seven borings drilled on January 20 and 21, 2020 with track-mounted, limited-access hollow-stem auger drilling equipment and two borings drilled on January 21, 2020 with hand-auger equipment. The borings were drilled to depths ranging from 13½ to 21½ feet, while the hand augers were advanced to depths of 4 to 4½ feet. The borings and hand augers were backfilled with cement grout in accordance with local requirements; exploration permits were obtained as required by local jurisdictions.

The approximate locations of our exploratory borings are shown on the Geologic Site Plan, Figure 2, respectively. Details regarding our field program are included in Appendix A.

1.4 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 Plasticity Index tests. Details regarding our laboratory program are included in Appendix B.

1.5 ENVIRONMENTAL SERVICES

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

SECTION 2: REGIONAL SETTING

2.1 REGIONAL GEOLOGIC SETTING

The site is located within the north-central Santa Cruz Mountains, a northwest-southeast mountain range within the Coast Range Geomorphic Province. The Santa Cruz Mountains are within the San Francisco Bay Block, which is bounded to the east by the Hayward and Calaveras Faults and to the west by the San Andreas Fault. The San Andreas Fault is a NW-trending, right-lateral, strike-slip fault that is comprised of many strands that form a zone, which is up to 1 km wide within the area. The fault system distributes shearing across a complex system of primarily northwest trending, right-lateral, strike-slip faults that includes the Hayward and Calaveras Faults.

The geology of the La Honda 7.5-minute Quadrangle is characterized by two basement assemblages that are separated by the San Andreas Fault, which extends through the northeastern corner of the quadrangle. Northeast of the San Andreas Fault is a composite Mesozoic basement assemblage consisting of the Franciscan Complex, Coast Range Ophiolite, and the Great Valley Sequence. Southwest of the San Andreas Fault is the Salinian Terrane of the Santa Cruz block, a basement assemblage of granitic and metamorphic crystalline rocks. Rocks within the north-central Santa Cruz Mountains have undergone a complex structural history and have been strongly deformed by faulting and folding. The basement is overlain by Miocene marine strata and Pliocene and Pleistocene sediment. Miocene and later strata have been deformed by reverse faulting along the Sargent, Berrocal and Shannon Fault zones (Hitchcock et a.,1994).

2.2 REGIONAL SEISMICITY

While seismologists cannot predict earthquake events, geologists from the U.S. Geological Survey have recently updated (in 2015) earlier estimates from their 2014 Uniform California Earthquake Rupture Forecast (Version 3; UCERF3) publication. The estimated probability of one or more magnitude 6.7 earthquakes (the size of the destructive 1994 Northridge earthquake) expected to occur somewhere in the San Francisco Bay Area has been revised (increased) to 72 percent for the period 2014 to 2043 (Aagaard et al., 2016). The faults in the region with the highest estimated probability of generating damaging earthquakes between 2014 and 2043 are the Hayward (33%), Calaveras (26%), and San Andreas Faults (22%). In this 30-year period, the probability of an earthquake of magnitude 6.7 or larger occurring is 22 percent along the San Andreas Fault and 33 percent for the Hayward Fault.

The faults considered capable of generating significant earthquakes are generally associated with the well-defined areas of crustal movement, which trend northwesterly. The table below presents the State-considered active faults within 25 kilometers of the site.

Table 1: Approximate Fault Distances

Fault Name	Distance	
	(miles)	(kilometers)
San Andreas (1906)	5.5	8.8
Sargent-Berrocal	6.2	10
Monte Vista-Shannon	6.7	10.8
San Gregorio	7.5	12
Zayante	8.4	13.5

In addition, the Hayward Fault, Calaveras Fault zone, and the San Gregorio Fault Zone (major branching faults of the San Andreas system) are located 24 miles (38.3 km) northeast, 27.7 miles (44.5 km) northeast, and 7.5 miles west of the site. Additionally, two undifferentiated Quaternary faults exist in the general area including: the Butano Fault located about 2 miles (3.2 km) south of the site and the Pilarcitos Fault is located about 4.76 (7.6 km) miles northeast of the site. More locally, Jennings and Bryant (2010) show the (pre-Quaternary) La Honda Fault as projected toward the site with a southeasterly trend. It would intersect the far eastern edge of the site near Pescadero Creek Road (Jennings and Bryant, 2010). Pre-Quaternary Faults are not considered potential seismic sources and do not represent a geologic constraint for fault surface rupture.

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

SECTION 3: SITE CONDITIONS

3.1 SITE HISTORY AND AIR PHOTO REVIEW

A review of historic topographic maps extending back to 1940 and aerial photos extending back to 1931 shows that the site has been used as livestock rangeland for decades. As of the date of the 1931 aerial photos, the site appears to be totally undeveloped with no dirt roads and no structures present.

A review of the historical topographic maps (U.S.G.S.) indicates that a dirt access road (“Burns Chalk Fire Road”) has existed along the spine of the ridge since at least as early as 1940. A barn structure was constructed at its current location and a stock pond established just downslope of the access road in the central portion of the site sometime between 1968 and 1980. Between 1982 and 1991 a residence was constructed just on the west side of the barn. Sometime between 1991 and 2005 numerous fenced livestock pens were constructed adjacent to the barn. Sometime between 2005 and 2009 additional soil was placed across from the barn in order to extend a parking area alongside the dirt road for parking of storage vehicles and farm equipment. Additional dirt roads were established along the top of the ridge further to the west in this period.

3.2 SURFACE DESCRIPTION AND TOPOGRAPHY

The site is located on a northwesterly trending ridge southern flank in an area of complex and highly varied topography. The southerly flank of the ridge varies from gently inclined to moderately inclined and steep. The areas where the proposed improvements are to be located can generally be characterized according to the following:

3.2.1 Area of Existing Barn/Adjacent Parking Lot Area

The area of the existing barn and existing caretaker's residence is relatively flat with steep downslopes located within 40 feet north of the existing structures. Although this area is largely flat, there are local variations resulting in approximately 2 feet of topographic relief across the pad area. We understand that the existing caretaker's residence will be demolished and a new fire prevention water tank and pump station will be constructed in its place. The proposed domestic water tank, pump station, and maintenance building located just east of the existing barn is on flat ground, however, there is an existing (undocumented) wedge of fill along the northern edge of this proposed improvement area the slopes become steep immediately adjacent to the area.

The proposed maintenance building and adjacent service yard for generators is located adjacent to the northern edge of the relatively flat area, which is at the crest of a steep slope where localized fill has been placed in order to create a flat pad.

3.2.2 Proposed Caretaker's Residence, Dog Enclosures, and New Barn Area

The proposed caretaker's residence ("Building 3"), dog enclosures ("Buildings D, E, and F"), and new farm animal barn ("Building 4") is on a moderately inclined slope on the downhill side of the existing fire access road.

There is approximately 6 to 8 feet of topographic relief across the pad area. Claystone bedrock is exposed at shallow depths within erosion gullies located just downslope of the building pad area.

3.2.3 Proposed Veterinarian/Administration Building

The area of the proposed Vet/Admin building (Building 2) is in a transitional area where the ground changes from nearly level to gently inclined toward the south. The northern and eastern portion of the building footprint is in an area where undocumented fills exist. These fill berms occur on both the west and the east side of the building footprint and, based on a review of the surrounding natural topography, may be up to 10 feet thick. There is approximately 8 to 12 feet of topographic relief across the pad area. Based on the provided topographic and architectural plans, we understand that the downslope side of the vet/admin building will have a basement level, which will be cut into the existing slope.

The group of proposed “cat enclosures” are located on a gently to moderately inclined slope just to the west of the Administration building. Relief across these pads is on the order to 4 to 6 feet. Bedrock is not exposed in this area of the site.

3.2.4 South Dog Loop Area

The “South Dog Loop” is a proposed group of kennels will include a 3,000 s.f. enclosed “dog arena”, and a series of large and small dog “cottages” around the brow or crest of the flanking slopes around the perimeter of the knoll. The proposed road at the “east dog loop” is located on the top a of a knoll where the slopes are gently inclined to moderately inclined. There is approximately 4 to 6 feet of topographic relief across the dog cottages pads and there is approximately 2 to 3 feet of relief across the the dog arena area. Sandstone bedrock is exposed locally at the ground surface on the top of the knoll.

3.3 SITE GEOLOGY AND SUBSURFACE CONDITIONS

Several regional geologic maps have been prepared of the area surrounding the campus, including those by; Rogers (1971), Brabb (1970 and 1980). We have adopted the nomenclature of Brabb (1980) in assigning geologic unit names for our characterization of the site. Brabb shows the bedrock in the area of the site as the Tahana member of the (Tertiary) Purisima Formation. A vicinity geologic map is presented as figure 6. The geologic units are characterized by Brabb as follows: “Greenish-gray to white or buff, medium to very fine grained sandstone and siltstone, with some silty mudstone. Locally the sandstone is tuffaceous and it weathers white. Pebble conglomerate occurs near the base.” In terms of rock characterization, the bedrock is generally weak, friable, moderately severely weathered.

Our site reconnaissance resulted in the following observations: Bedrock is exposed at road cuts, at erosion scars on site slopes, and at a large cut located just northeast of the proposed caretaker’s residence. A large exposure of bedrock located just on the north side of the caretaker’s residence exposes interbedded silty sandstone and thin bedded siltstone. Claystone is exposed within erosion gullies located on the south of these proposed structure. Our borings encountered primarily claystone with some layers of sandstone. The bedrock is thin to medium bedded (laminated locally) folded locally and displays a variety of structural trends varying from northwesterly, moderately dipping to southwesterly, steeply dipping.

The sloping portions of the site have experienced severe erosion where runoff is not controlled or, alternatively where the surface runoff is focused by roadways or culverts, or swales or gullies. This severe erosion appears to be exacerbated by an abrupt permeability contract between the sandy (erodible) surficial soils and the underlying consolidated sedimentary bedrock units that are more resistant to erosion. The erosion gullies trend downslope toward the southwest and vary from 3 feet deep to as much as 10 feet deep onsite.

Existing stockpiled fill: Two large accumulations of fill exist just south of the access road in the area of the barn, existing caretaker's residence, proposed new fire prevention water tank and pump station, domestic water tank and pump station, and maintenance building. This material forms a sliver of material that extends outward toward the south from the existing dirt road. This material is non-engineered and apparently was placed in order to create additional parking area for farm machinery and vehicles. This fill cannot be relied upon for support of improvements (see Recommendations).

Our site exploration consisted of drilling, logging and sampling within seven conventional geotechnical borings and two hand auger borings at various locations at the site. The exploration was accomplished with a track-mounted drill rig using hollow stem augers and standard geotechnical sampling equipment. The results of the borings are presented below according to location:

3.3.1 Area of Existing Barn/Adjacent Parking Lot Area

Boring EB-6 was located near the northwest corner of the current fenced in "corral" area, the future location of a domestic water tank and associated pump station, and maintenance building. Here the subsurface profile consisted of a 3½ foot-thick layer of surficial (undocumented fill) sandy lean clay. The fill was underlain by black fat clay (residual soil) to a depth of 7½ feet. Below the depth of 7½ feet is the sandy claystone bedrock. The fill and residual soil layers were found to be in a stiff to very stiff condition, however the undocumented fill is judged to be moderately compressible. The claystone was found to be in a generally weak condition in terms of bedrock characterization and produced standard penetration test blow counts that ranged from10 blows-per-foot (bpf) to 21 bpf. We understand that the existing caretaker's residence will be demolished and a fire prevention water tank and associated pump station will be constructed partially within the old residence footprint. We anticipate that up to several feet of undocumented fill may be encountered due to the previous development.

3.3.2 Proposed Caretaker's and New Barn

Boring EB-2 was located in the general area of the Caretaker's cottage and new barn. As noted already, a large exposure of bedrock located just on the north side of the caretaker's residence and guest cottages exposes interbedded silty sandstone and thin bedded siltstone. Claystone is exposed within erosion gullies located on the south of these proposed structures. The change in lithology between the cut exposure and the exploratory boring and erosion gullies further downslope is likely due to the result of folding that trends through the immediate area. At the Boring EB-2 location, the subsurface profile consisted of a 2½-foot-thick layer of surficial (colluvium) fat clay with sand. The residual soil was underlain by claystone bedrock. The residual soil layer was found to be in a medium stiff condition in terms of soil characterization. The claystone was found to be in a generally weak condition in terms of bedrock characterization and produced standard penetration test blow counts that ranged from 22 blows-per-foot (bpf) to 34 bpf. A geologic cross section A-A' developed for this area is shown on Figure G.

3.3.3 Proposed Veterinarian/Administration Building

Boring EB-7 was located in the general area of the veterinarian/administration building. Here the subsurface profile consisted of a 1½-foot-thick layer of surficial (colluvium) clayey sand. The residual soil was underlain by sandstone bedrock. The residual soil layer was found to be in a medium dense condition. The sandstone and claystone was found to be in a generally weak condition in terms of bedrock characterization and produced standard penetration test blow counts that ranged from 21 blows-per-foot (bpf) to 36 bpf. As discussed earlier, there are fill berms on both the west and the east side of the building footprint that may be up to 10 feet thick based on a review of the surrounding natural topography.

3.3.4 Proposed Cat and Dog Enclosure Area

Boring EB-3 and EB-7 was located in the general area of the cat enclosure area. Here the subsurface profile consisted of a 2- to 4-foot-thick layer of surficial (colluvium) fat clay with sand. The residual soil was underlain by sandstone bedrock. The residual soil layer was found to be in a stiff condition. The claystone was found to be in a generally weak condition in terms of bedrock characterization and produced standard penetration test blow counts that ranged from 18 blows-per-foot (bpf) to 37 bpf.

3.3.5 Plasticity/Expansion Potential

We performed two Plasticity Index (PI) tests on representative samples. Test results were used to evaluate expansion potential of surficial soils and underlying bedrock. The result of the surficial PI test indicated a PI of 34, indicating very high expansion potential to wetting and drying cycles. The result of the PI test on the underlying claystone indicated a PI of 60, which indicates very high expansive potential to wetting and drying cycles.

3.3.6 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents within the upper 10 feet range from 2 percent under to 15 percent over the estimated laboratory optimum moisture.

3.4 GROUNDWATER

The site encompasses high elevation ground along the top and southerly crest of a ridgetop in the rugged La Honda region of the Santa Cruz Mountains. The site is underlain at shallow depths by sedimentary bedrock and our research suggests this formation does not serve as a laterally continuous shallow aquifer. The only water noted at the site exists within two large stock ponds that exist in the lower portion of the site slopes located well below (downslope) of the proposed improvements. These stock ponds are fed by surface runoff. We did not encounter evidence of groundwater in any of our explorations. It should be noted that, in general, fluctuations in groundwater levels could occur due to many factors including perched water, and regional groundwater variations, and rainfall or irrigation. We note that perched groundwater conditions are often present in the bedrock on hillside sites.

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT SURFACE RUPTURE

As stated earlier, published maps do not show any faults trending through the subject site (Rogers, 1971; Brabb, 1970 and 1980; Brabb and Olsen 1983; Jennings and Bryant, 2010; CDMG, 2003; USGS Fault and Fault database, 2006). The site is not located within a State Earthquake Fault Zone (CDMG 2003). We did not encounter evidence during our research or site reconnaissance of faults trending through the site. The potential for fault surface rupture occurring at the site should be considered low.

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) was estimated for analysis using a value equal to F_{PGA}*PGA, as allowed in the 2019 edition of the California Building Code per Exception 2 of Section 11.4.8 of ASCE 7-16. For our analyses, we used a PGA of 1.114g.

4.3 LIQUEFACTION POTENTIAL

Published geotechnical hazard maps do not show the site in an area identified as having a liquefaction potential. This is due primarily to the fact that very shallow bedrock exists at the site and it is located at a high elevation in rugged terrain. The site is not located within a County-designated Liquefaction Hazard Zone (San Mateo County, 2008), and is within a zone mapped as having a low liquefaction potential by the Association of Bay Area Governments (ABAG). We screened the site for liquefaction during our site exploration by retrieving samples from the site, performing visual classification on sampled materials, and performing various tests to further classify the soil properties.

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.

As discussed in the “Subsurface” section above, we primarily encountered surficial soils consisting of lean clays or sandstone, siltstone and claystone bedrock. These materials are generally not susceptible to liquefaction. Based on the above, our screening of the site for liquefaction indicates a low potential for liquefaction.

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. As the soils encountered at the site were predominantly medium stiff to very stiff clays, and medium dense clayey sands, or claystone and sandstone bedrock, in our opinion, the potential for significant differential seismic settlement affecting the proposed improvements is low.

4.6 LANDSLIDING

4.6.1 General

The California Geological Survey (CGS) has not yet produced a Seismic Hazard Zone report or accompanying map for the La Honda 7.5-minute quadrangle during their ongoing program to map Seismic Hazard Zones on a 7.5-minute quadrangle scale (1:24,000) in the Bay Area. The County of San Mateo has not established regulatory zones for landsliding, however, the planning department maintains a map of “Existing Landslides” in the county (based on the USGS publication), open File Report 975-C. The published landslide-themed map of Brabb and Pampeyan (1972) which covers the County of San Mateo shows the site in an area of suspected large-scale landsliding (Figure 5 is a partial reproduction of the map of Brabb and Pampeyan. Specifically, the ridge top and the crests of adjacent slope son the south side are shown in a headscarp area of a large-scale landslide complex, which is shown s encompassing the rolling topography on the slopes below the slope crests. The proposed improvements are outside the mapped landslide mass. The county planning department shows the site in a zone designated as “areas of mostly landslides”. The CGS interactive map showing reported recent landslides (CGS, 2018) does not show any reported landslides in the immediate area. These mapped landslides and classifications are the result of interpretive mapping and are not based on site-specific studies. These maps serve as a planning resource. Maps and publications published after the damaging El Niño rainfall events in 1982, and 1995 (Ellen & Weiczorek, 1982; Ellen et al., 1997) depicting landslides that resulted from those large-scale damaging events do not show any landslides that occurred from those events at the site.

Our site-specific geologic evaluation has resulted in an interpretation that differs from the published mapping in terms of the nature and extent of landsliding at the site.

4.6.2 Site-Specific

Our review of aerial photos, our site reconnaissance and subsurface exploration has led to our conclusion that, although the lower portions of slopes on the south flank of the ridge display rolling topography, these slopes are not part of a large-scale landslide as suggested on the map of Brabb and Pampeyan (1972). Landsliding identified in this evaluation is based on geomorphic features discernible at the ground surface and in stereo aerial photographs. We have mapped several landslides on the subject property and have depicted these features on the site plan (Figure 2) and have designated some of these individual slides on the map with numbers as a convenience in description in this text. Some of these identified features are located well beyond the proposed improvements and are not considered a constraint to the siting of structures or grading. The establishment of a septic system leachfield at the site is located closer to these identified landslides (see Figure 7) and the layout and design of these leachfields should take into account the constraints (see Recommendations section). Of the landslides that have been mapped during our study, the following landslides are located more proximal to the proposed features and are discussed below:

Qls1: This slide is located just downslope of the existing and proposed access road in the north-central portion of the property (see Figure 2). This feature is a slump-type failure and, based on the relative topography surrounding this feature, is inferred to be relatively shallow (approximately 15 feet thick or less) and consists of colluvial soils overlying thin bedded mudstone and sandstone. A culvert trends beneath the road which delivers surface runoff from the road into the headscarp of this feature. This may have served as the triggering mechanism for this shallow landslide. Drainage improvements should be modified in this area in order to help mitigate this condition. Recommendations are offered for reducing this constraint (see Section 6.12 titled “Site Drainage”).

Qls2: This suspected landslide is a relatively small, shallow landslide (a slump) located adjacent to the downslope side of the vet/admin building and several cat enclosure structures (see Figure 2). Although poorly defined in terms of slope morphology. The scarp area is located less than 10 feet from the nearest proposed enclosure and admin building. Our exploratory boring (EB-7) drilled near the scarp of this mapped slide indicates bedrock is shallow in this area. This feature may have been triggered by a lack of surface runoff coming off the top of the ridge. This runoff pattern my no longer exist due to the establishment of the graded dirt access road and fill berms that have been placed in the last 30 or so years.

Qls4: This is a suspected landslide scarp, however, it lacks topographic patterns that would suggest a debris field is present below the scarp (see Figure 2). This feature is located adjacent to the main site access road. A landslide below this scarp would most probably move downslope and away from the road, however, the scarp would not be expected to “back step” over time into the roadway area provided that surface runoff is controlled and directed away from this feature.

Qls3 and Qls5 are all located well outside any proposed developed areas and therefore do not pose a constraint to any proposed features for the current version of the development concept (see Figures 2). Aside from seismic shaking, proximity to some small to moderate sized landslides, and the more general hazard of erosion, there are no other geologic constraints that potentially impact the proposed project as currently conceived.

Control of construction phase runoff and long-term runoff is essential for the stability of slopes at the site. All runoff should be collected and directed to suitable discharge points which specifically avoid the mapped landslides and these discharge points should be located well downslope of the proposed development features, including roads. We do not recommend allowing or directing development runoff toward the very steep slopes on the north side of the north property line (see Site Drainage Recommendations).

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.

- Presence of highly expansive soil and bedrock
- Presence of undocumented fills
- Potential for cut/fill transitions
- Redevelopment considerations
- Slope stability and building/leach field setbacks
- Presence of cohesionless soils
- Potential for difficult excavation
- Soil Corrosion Potential

5.1.1 Presence of Highly Expansive Soil and Bedrock

Our borings disclosed the presence of both sandstone and claystone bedrock of the Tahana formation at the site. Our Plasticity Index testing of the claystone and residual clay soils indicate that these materials are highly to very highly expansive. 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 or the structures should be supported on a drilled pier foundation system. Because of these expansive soils and the close proximity of the bedrock, we recommend the care takers residence, fire prevention water tank and pump station, domestic water tank and pump station, maintenance building, and vet/admin building should be supported on drilled pier foundations. While the PI testing indicates highly expansive soils and bedrock, we are not aware of any published geologic or geotechnical information which suggests these materials are subject to extreme uplift pressures and movement as claystone bedrock of the Whiskey Hill formation is known for, which is located in the vicinity of Menlo Park. This report does not provide recommendations to address extreme uplift and movement of claystone because it has not been documented for this unit in the published literature or in our experience with this geologic unit. However, we would recommend that the grading plan be developed to limit cuts to about 3 feet to mitigate potential heave of the very highly expansive claystone. In areas of the structures where there will be greater than 3 feet of cut into the claystone, we recommend the minimum drilled pier embedment be increased to 15 feet. 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 these expansive soil and bedrock concerns are presented in the following “Earthwork” and “Foundation” sections.

5.1.2 Presence of Undocumented Fills

Our borings encountered undocumented fill ranging up to 3½ feet in depth, and two fill berms were observed the west and the east side of the approximate vet/admin building footprint that may be up to 10 feet thick based on our review of the surrounding natural topography. To reduce the potential for differential settlement, we recommend that the undocumented fill be over-excavated and recompacted following the recommendations presented in the “Earthwork” section below. In addition, where fill placement results in a cut/fill transition within a building pad that will be supported on shallow foundations, we recommend that the entire building pad be overexcavated to provide uniform support. Additional recommendations are provided in the “Earthwork” section of this report.

5.1.3 Potential for Cut/Fill Transitions

Based on the proposed level building pads for many of the structures, and the existing topography of the site, new structures could potentially span cut/fill transitions, if not mitigated. The performance of a structure supported on a shallow foundation overlying a cut/fill transition could result in increased differential settlement. Therefore, we recommend that cut/fill transitions be over-excavated and that shallow foundations bear uniformly on similar, undisturbed native soil or bedrock, or a relatively uniform section of engineered fill over undisturbed native soil and/or bedrock. Recommendations addressing this are presented in the “Earthwork” section.

5.1.4 Redevelopment Considerations

As discussed, the site is currently occupied by existing buildings, site fixtures, and landscaping. We understand that some of the existing improvements, such as the existing caretaker’s residence, will be demolished for the construction of the new site improvements. We understand the new fire prevention water tank and pump station will be constructed partially within the footprint of the existing residence. Potential issues that are often associated with redeveloping sites include demolition of existing improvements, abandonment of existing utilities, and undocumented fills. Please refer to the “Earthwork” section below for further recommendations.

5.1.5 Slope Stability and Building Setbacks

Several potential landslides and areas of slope instability were identified during our investigation. However, it appears that the proposed project layout has been made to avoid these areas. Our recommendations for building and leach field setbacks are presented in the “Earthwork” section of this report.

5.1.6 Presence of Cohesionless Soils

As mentioned, some areas of the site are underlain by cohesionless, sandy soils with low fines content. The sandy soils may not stand vertical when excavated and excavation sidewalls for foundations, utility trenches, temporary slopes, basement excavation, etc., may cave in or accumulate significant amount of slough. Grading and excavation contractors should be made aware of this condition and plan on forming footings, preparing slab-on-grade subgrade just prior to concrete placement, and other similar construction issues as relates to temporary shoring, utility excavations, etc. Our recommendations for excavation of cohesionless soils are presented in the “Earthwork” section of this report.

5.1.7 Potential for Difficult Excavation

Our borings encountered moderately hard, moderately to deeply weathered Tahana Claystone and Sandstone. Based on the project plans, excavations into claystone and sandstone is anticipated and should be anticipated. In our opinion, moderately to deeply weathered areas of bedrock would be excavatable with heavy-duty excavating equipment (such as large backhoes or excavators). However, slightly weathered to fresh bedrock areas, if encountered, will likely require excavation with a hoe-ram. Additionally, drilled pier contractors should anticipate difficult drilling conditions and should be experienced in drilling in bedrock conditions and the use of appropriate equipment (such as coring barrels) to advance the piers to design depths. Additional recommendations are provided in the “Earthwork” and “Foundation” sections of this report.

5.1.8 Soil Corrosion Potential

Soil corrosion screening was not performed during our investigation; however, based on our experience with similar soil, the subsurface soil is likely to be considered corrosive to buried metal and potentially concrete as well. We recommend soil corrosion screening be performed during design.

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

All existing improvements not to be reused for the current development, including all foundations, flatwork, pavements, utilities, and other improvements should be demolished and removed from the site. Recommendations in this section apply to the removal of these improvements, which may be present on the site, prior to the start of mass grading or the construction of new improvements for the project. It is noted that “unknown” buried structures such as septic systems, leach fields, seepage piles, debris pits, and/or wells, etc. may be encountered during grading. If these are encountered during grading, we should provide recommendations to address them on a case-by-case basis.

Cornerstone should be notified prior to the start of demolition, and should be present on at least a part-time basis during all backfill and mass grading as a result of demolition.

6.1.1 Abandonment of Existing Utilities

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 risk for owners 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.

6.2 SITE CLEARING AND PREPARATION

6.2.1 Site Stripping

The site should be stripped of all surface vegetation, and surface and subsurface improvements to be removed within the proposed development area. Demolition of existing improvements is discussed in the prior paragraphs. A detailed discussion of removal of existing fills is provided later in this report. 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 4 to 6 inches below existing grade in vegetated areas.

6.2.2 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.

6.3 REMOVAL OF EXISTING FILLS

As discussed, our borings encountered undocumented fill to depths of 3½ feet and two fill berms observed directly west of and within the east side of the vet/admin building footprint that may be up to 10 feet thick, much of this fill will likely be removed during grading. In addition, we anticipate up to several feet of undocumented fill may be encountered below and in the vicinity of the existing caretaker’s residence due to previous site grading activities. All fills should be completely removed from within building areas and tank areas, and to a lateral distance of at least 5 feet beyond the building footprint or to a lateral distance equal to fill depth below the perimeter footing, whichever is greater. We also recommend that all undocumented fill be removed from pavement and flatwork areas. Provided the fills meet the “Material for Fill” requirements below, the fills may be reused when backfilling the excavations. Based on review of the fill berms, the material may be reused if all debris, wood, trash, and other unsuitable material is screened out of the remaining material and removed from the site. If materials are encountered that do not meet the requirements, such as debris, wood, trash, those materials should screened out of the remaining material and be removed from the site. Backfill of excavations should be placed in lifts and compacted in accordance with the “Compaction” section below.

6.4 BUILDING AND LEACH FIELD SETBACKS

In general, we recommend that the proposed buildings, equipment pads, and water tanks be setback at least 25 feet from the mapped landslides and 15 feet from the top of slopes. Where structures are within 15 feet of a slope, we recommend they be supported on drill piers designed in accordance with the recommendations in this report. This would apply to the caretaker residence, fire prevention and domestic water tank pads and associated pump stations, maintenance building, and administration/veterinary clinic building. We note that one of the cat enclosures is positioned about 10 feet away from the top of Landslide #2. We note that EB-7 was drilled between the Cat Enclosure and the top of Landslide #2. Since the boring disclosed that the sandstone bedrock is at a shallow depth in this area, the location of this Cat Enclosure is acceptable from a geologic viewpoint. The leach field should be set back at least 50 feet from the top of the mapped landslides. General recommendations for release of water onto the slopes is presented in the “Site Drainage” portion of this report.

6.5 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 C 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 no greater than 1:1 (horizontal:vertical) within the upper 5 feet below building subgrade, unless the OSHA soil classification indicates that slope should be flatter.

6.6 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.

6.7 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 natural 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 potential methods to address potential 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.

6.7.1 Scarification and Drying

The subgrade may be scarified to a depth of 12 to 18 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.

6.7.2 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.

6.7.3 Chemical Treatment

Where the unstable area exceeds about 5,000 to 10,000 square feet and/or site winterization is desired, chemical treatment with quicklime (CaO), kiln-dust, or cement may be more cost-effective than removal and replacement. Recommended chemical treatment depths will typically range from 12 to 18 inches depending on the magnitude of the instability.

6.8 MATERIAL FOR FILL

6.8.1 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 below the non-expansive fill section. 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 oversize 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.

6.8.2 Potential Import Sources

Imported and non-expansive material should be inorganic with a Plasticity Index (PI) of 15 or less, and not contain recycled asphalt concrete where it will be used within the habitable building areas. 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.

6.8.3 Non-Expansive Fill Using Lime Treatment

As discussed above, non-expansive fill should have a Plasticity Index (PI) of 15 or less. Due to the high clay content and PI of the on-site soil and bedrock materials, it is not likely that sufficient quantities of non-expansive fill would be generated from cut materials. As an alternative to importing non-expansive fill, chemical treatment can be considered to create non-expansive fill. If this option is considered, additional laboratory tests should be performed prior to initial site grading to further evaluate the optimum percentage of quicklime required.

6.9 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 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 2: 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
	Low Expansion Soils	90	>1
General Fill (below a depth of 5 feet)	On-Site Expansive Soils	95	>3
	Low Expansion Soils	95	>1
Trench Backfill	On-Site Expansive Soils	87 – 92	>3
Trench Backfill	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	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	Low Expansion Soils	95	>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)

6.9.1 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.

6.10 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 (⅜-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.

Where utility lines will cross perpendicular to strip footings, the footing should be deepened to encase the utility line, providing sleeves or flexible cushions to protect the pipes from anticipated foundation settlement, or the utility lines should be backfilled to the bottom of footing with sand-cement slurry or lean concrete. Where utility lines will parallel footings and will extend below the "foundation plane of influence," an imaginary 1:1 plane projected down from the bottom edge of the footing, either the footing will need to be deepened so that the pipe is above the foundation plane of influence or the utility trench will need to be backfilled with sand-cement slurry or lean concrete within the influence zone. Sand-cement slurry used within foundation influence zones should have a minimum compressive strength of 75 psi.

On expansive soils sites it is desirable to reduce the potential for water migration into building and pavement 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 and pavement areas.

6.11 PERMANENT CUT AND FILL SLOPES

All permanent cut and fill 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 (H:V). Fill slopes should be overbuilt and trimmed back, exposing engineered fill when complete. We would also recommend that in the building areas cuts be limited to 3 feet to reduce the potential for heave in the claystone bedrock. Refer to the "Erosion Control" section of this report for a discussion regarding protection of slope surfaces.

6.11.1 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 8 feet wide. Depending on the thickness of any colluvial/residual soil layer 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 moderately weathered bedrock, and be at least 12 feet wide. A typical key and construction is depicted in Figure 8.

6.11.2 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.

The drainage system should be constructed in small trenches or v-ditches and consist of a minimum 4-inch-diameter perforated (perforations placed downward) pipe, bedded and shaded in Caltrans Class 2 Permeable Material (latest version) or ¾-inch crushed rock; if crushed rock is used, the rock should be encapsulated in filter fabric (Mirafi 140N or equivalent). The bedding should be at least 2 inches, and the trench should be at least 8 inches in width and depth. Alternatively, geocomposite strip drains may be used. All drainage lines should slope towards suitable outlet structures at an inclination of at least 0.5 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.

6.11.3 Plan Review and Construction Monitoring

We should be retained to review the conceptual 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 keyway and fill slope construction. Field modifications to the planned keyway and benching may be required based on encountered field conditions. In addition, it has been our experience that cut slopes in the Tahana Formation 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.

6.12 CUT/FILL TRANSITION OVER-EXCAVATION

Structures underlain by cut/fill transitions should be over-excavated to provide a relatively uniform fill thickness beneath the structure footprint. The depth of over-excavation below pad grade should be equal to at least 3 feet below the bottom of foundations to provide a uniform engineered fill pad. The final depth of the over-excavation will depend on the type of material exposed, and will be determined in the field during construction. In general, over-excavation should extend to at least 5 feet beyond the building footprint. Adjustments to the depth and lateral limits of the over-excavation may need to be made at the time of construction depending on the actual conditions encountered during grading.

6.13 SITE DRAINAGE

6.13.1 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. We recommend that the development runoff be directed through solid drain pipes to suitable discharge facilities located well downslope of the developed areas. Alternatively, runoff may be directed in solid pipes to the existing stock ponds located in the western and in the eastern portions of the site. Discharge areas for runoff should be setback a minimum distance of 100 feet from identified landslides scarps. Runoff should not be allowed to flow over the steep to very steep slopes that are adjacent to the north property line. Ponding should also not be allowed on or adjacent to building foundations, slabs-on-grade, or pavements. Hardscape surfaces should slope at least 2 percent towards suitable discharge facilities; landscape areas should slope at least 3 percent towards suitable discharge facilities. Roof runoff should be directed away from building areas in closed conduits, to approved infiltration facilities, or on to hardscaped surfaces that drain to suitable facilities. Retention, detention or infiltration facilities should be spaced at least 10 feet from buildings, and preferably at least 5 feet from slabs-on-grade or pavements. These facilities are not recommended where stormwater infiltration may affect slopes at lower elevations on or adjacent to the site. However, if slopes are not present at lower elevations that could potentially be affected, and if retention, detention or infiltration facilities are located within these zones, we recommend that these treatment facilities meet the requirements in the Storm Water Treatment Design Considerations section of this report.

Lined v-ditches should be included at the top of slopes and intermediate benches, and at the toe of slopes or behind retaining walls adjacent to planned or existing 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 of is present or planned, additional surface and subsurface drainage, or construction of drained buttress fills may be needed to protect site improvements. We should be consulted if this issue will affect the project.

We recommend that the septic leach fields are designed to disperse effluent over as large an area as practicable, or alternatively, that the effluent be directed deeper into the subsurface profile within sandstone that underlies the surficial soils and claystone layers. The infiltration or percolation rate should be evaluated by the leach field designer.

6.13.2 Subsurface Drainage

As discussed in the "Permanent Cut and Fill Slopes" section, subsurface drainage improvements might be installed as part of earthwork for fill construction if perched groundwater is observed. These improvements should include positive surface gradients for keyways and benches and the installation of a subdrain system consisting of perforated pipe and permeable gravel or drain rock. If drain rock is used, the rock and pipe should be entirely wrapped with a permeable geotextile fabric. Subdrains should also be installed at the toe of any proposed cut slopes depending on the actual conditions observed during construction. As previously discussed, a conceptual subdrain plan should be prepared once preliminary grading plans are finalized. The actual location of subdrains should be determined in the field at the time of construction.

6.14 LOW-IMPACT DEVELOPMENT (LID) IMPROVEMENTS

The Municipal Regional Permit (MRP) requires regulated projects to treat 100 percent of the amount of runoff identified in Provision C.3.d from a regulated project's drainage area with low impact development (LID) treatment measures onsite or at a joint stormwater treatment facility. LID treatment measures are defined as rainwater harvesting and use, infiltration, evapotranspiration, or biotreatment. A biotreatment system may only be used if it is infeasible to implement harvesting and use, infiltration, or evapotranspiration at a project site.

Technical infeasibility of infiltration may result from site conditions that restrict the operability of infiltration measures and devices. Various factors affecting the feasibility of infiltration treatment may create an environmental risk, structural stability risk, or physically restrict infiltration. The presence of any of these limiting factors may render infiltration technically infeasible for a proposed project. To aid in determining if infiltration may be feasible at the site, we provide the following site information regarding factors that may aid in determining the feasibility of infiltration facilities at the site.

- The near-surface soils at the site are clayey, and categorized as Hydrologic Soil Group D, and is expected to have infiltration rates of less than 0.2 inches per hour. In our opinion, these clayey soils will significantly limit the infiltration of stormwater.

- No groundwater production wells are within 100 feet of potential locations for infiltration facilities.

- The site is not known, to our knowledge, to have pollutants with the potential for mobilization as a result of stormwater infiltration.

- The site has a known geotechnical hazard consisting of steep slopes and areas with landslide potential; therefore, stormwater infiltration facilities may not be feasible.

- In our opinion, infiltration locations within 10 feet of the buildings and top of slopes or on the slopes would create a geotechnical hazard.

6.14.1 Storm Water Treatment Design Considerations

If storm water treatment improvements, such as shallow bio-retention swales, basins or pervious pavements, are required as part of the site improvements to satisfy Storm Water Quality (C.3) requirements, we recommend the following items be considered for design and construction.

6.14.1.1 General Bioswale Design Guidelines

- If possible, avoid placing bioswales or basins within 10 feet of the building perimeter or within 5 feet of exterior flatwork or pavements. If bioswales must be constructed within these setbacks, the side(s) and bottom of the trench excavation should be lined with 10-mil visqueen to reduce water infiltration into the surrounding expansive clay.

- Bioswales constructed within 3 feet of proposed buildings may be within the foundation zone of influence for perimeter wall loads. Therefore, where bioswales will parallel foundations and will extend below the "foundation plane of influence," an imaginary 1:1 plane projected down from the bottom edge of the foundation, the foundation will need to be deepened so that the bottom edge of the bioswale filter material is above the foundation plane of influence.

- The bottom of bioswale or detention areas should include a perforated drain placed at a low point, such as a shallow trench or sloped bottom, to reduce water infiltration into the surrounding soils near structural improvements, and to address the low infiltration capacity of the on-site clay soils.

6.14.1.2 Bioswale Infiltration Material

- Gradation specifications for bioswale filter material, if required, should be specified on the grading and improvement plans.

- Compaction requirements for bioswale filter material in non-landscaped areas or in pervious pavement areas, if any, should be indicated on the plans and specifications to satisfy the anticipated use of the infiltration area.

- If required, infiltration (percolation) testing should be performed on representative samples of potential bioswale materials prior to construction to check for general conformance with the specified infiltration rates.

- It should be noted that multiple laboratory tests may be required to evaluate the properties of the bioswale materials, including percolation, landscape suitability and possibly environmental analytical testing depending on the source of the material. We recommend that the landscape architect provide input on the required landscape suitability tests if bioswales are to be planted.

- If bioswales are to be vegetated, the landscape architect should select planting materials that do not reduce or inhibit the water infiltration rate, such as covering the bioswale with grass sod containing a clayey soil base.

- If required by governing agencies, field infiltration testing should be specified on the grading and improvement plans. The appropriate infiltration test method, duration and frequency of testing should be specified in accordance with local requirements.

- Due to the relatively loose consistency and/or high organic content of many bioswale filter materials, long-term settlement of the bioswale medium should be anticipated. To reduce initial volume loss, bioswale filter material should be wetted in 12 inch lifts during placement to pre-consolidate the material. Mechanical compaction should not be allowed, unless specified on the grading and improvement plans, since this could significantly decrease the infiltration rate of the bioswale materials.

- It should be noted that the volume of bioswale filter material may decrease over time depending on the organic content of the material. Additional filter material may need to be added to bioswales after the initial exposure to winter rains and periodically over the life of the bioswale areas, as needed.

6.14.1.3 Bioswale Construction Adjacent to Pavements

If bio-infiltration swales or basins are considered adjacent to proposed parking lots or exterior flatwork, we recommend that mitigative measures be considered in the design and construction of these facilities to reduce potential impacts to flatwork or pavements. Exterior flatwork, concrete curbs, and pavements located directly adjacent to bio-swales may be susceptible to settlement or lateral movement, depending on the configuration of the bioswale and the setback between the improvements and edge of the swale. To reduce the potential for distress to these improvements due to vertical or lateral movement, the following options should be considered by the project civil engineer:

- Improvements should be setback from the vertical edge of a bioswale such that there is at least 1 foot of horizontal distance between the edge of improvements and the top edge of the bioswale excavation for every 1 foot of vertical bioswale depth, or

- Concrete curbs for pavements, or lateral restraint for exterior flatwork, located directly adjacent to a vertical bioswale cut should be designed to resist lateral earth pressures in accordance with the recommendations in the “Retaining Walls” section of this report, or concrete curbs or edge restraint should be adequately keyed into the native soil or engineered to reduce the potential for rotation or lateral movement of the curbs.

6.15 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 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.

6.16 LANDSCAPE CONSIDERATIONS

Since the near-surface soils are moderately to highly expansive, we recommend greatly reducing the amount of surface water infiltrating these soils near foundations and exterior slabs-on-grade. This can typically be achieved by:

- Using drip irrigation
- Avoiding open planting within 3 feet of the building perimeter or near the top of existing slopes
- Regulating the amount of water distributed to planter areas by using irrigation timers
- Selecting landscaping that requires little or no watering, especially near foundations.

We recommend that the landscape architect consider these items when developing landscaping plans.

SECTION 7: FOUNDATIONS

7.1 SUMMARY OF RECOMMENDATIONS

In our opinion, the proposed structures may be supported on shallow foundations and/or drilled piers provided the recommendations in the "Earthwork" section and the sections below are followed.

7.2 SEISMIC DESIGN CRITERIA

Our explorations generally encountered colluvium and residual soil overlying Tahana Formation claystone and sandstone to depths of 21½ feet, the maximum depth explored. Based on our borings and review of local geology, the site is underlain by shallow alluvial soils underlain by shallow rock with typical SPT "N" values above 50 blows per foot. Therefore, we have classified the site as Soil Classification C. The mapped spectral acceleration parameters S_s and S_1 were calculated using the web-based program ATC Hazards by Locations, located at https://hazards.atcouncil.org/, based on the site coordinates presented below and the site classification. Recommended values for design are presented in Table 3. The table below lists the various factors used to determine the seismic coefficients and other parameters.

Table 3: 2019 CBC Site Categorization and Site Coefficients

Classification/Coefficient	Design Value
Site Class	D
Site Latitude	37.302572°
Site Longitude	-122.279724°
0.2-second Period Mapped Spectral Acceleration[1], S_S	2.11g
1-second Period Mapped Spectral Acceleration[1], S_1	0.815g
Short-Period Site Coefficient – Fa	1.2
Long-Period Site Coefficient – Fv	1.4
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S_{MS}	2.532g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	1.141g
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.688g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	0.76g
MCE_G Peak Ground Acceleration – PGA	0.929g
Site Amplification Factor at PGA – F_{PGA}	1.2
Site Modified Peak Ground Acceleration – PGA_M	1.114g

7.3 SHALLOW FOUNDATIONS

7.3.1 Spread Footings – Animal Barn and Enclosed Dog Arena

The proposed animal barn and enclosed dog arena may be supported on shallow spread footings. Spread footings should bear on natural, undisturbed soil or engineered fill, be at least 12 inches wide, and extend at least 30 inches below the lowest adjacent grade. Lowest adjacent grade is defined as the deeper of the following: 1) bottom of the adjacent interior slab-on-grade, or 2) finished exterior grade, excluding landscaping topsoil. The deeper footing embedment is due to the presence of highly expansive soils, and is intended to embed the footing below the zone of significant seasonal moisture fluctuation, reducing the potential for differential movement.

Footings constructed to the above dimensions and in accordance with the “Earthwork” recommendations of this report are capable of supporting maximum allowable bearing pressures of 2,500 psf for dead loads, 3,750 psf for combined dead plus live loads, and 5,000 psf for all loads including wind and seismic. These pressures are based on factors of safety of 3.0, 2.0, and 1.5 applied to the ultimate bearing pressure for dead, dead plus live, and all loads, respectively. These pressures are net values; the weight of the footing may be neglected for the portion of the footing extending below grade (typically, the full footing depth). Top and bottom mats of reinforcing steel should be included in continuous footings to help span irregularities and differential settlement.

7.3.2 Footing Settlement

Structural loads were not provided to us at the time this report was prepared; therefore, we assumed isolated column loading of 30 to 50 kips. Based on the assumed loading and the allowable bearing pressures presented above, we estimate that the total static footing settlement will be on the order of ½-inch, with about ¼-inch of post-construction differential settlement between adjacent foundation elements. As our footing loads were assumed, we recommend we be retained to review the final footing layout and loading, and verify the settlement estimates above.

7.3.3 Lateral Loading

Lateral loads may be resisted by friction between the bottom of footing and the supporting subgrade, and also by passive pressures generated against footing sidewalls. An ultimate frictional resistance of 0.45 applied to the footing dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 450 pcf may be used in design. The structural engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate values above. Where footings are adjacent to landscape areas without hardscape, the upper 12 inches of soil should be neglected when determining passive pressure capacity.

7.3.4 Spread Footing Construction Considerations

Where utility lines will cross perpendicular to strip footings, the footing should be deepened to encase the utility line, providing sleeves or flexible cushions to protect the pipes from anticipated foundation settlement, or the utility lines should be backfilled to the bottom of footing with sand-cement slurry or lean concrete. Where utility lines will parallel footings and will extend below the "foundation plane of influence," an imaginary 1:1 plane projected down from the bottom edge of the footing, either the footing will need to be deepened so that the pipe is above the foundation plane of influence or the utility trench will need to be backfilled with sand-cement slurry or lean concrete within the influence zone. Sand-cement slurry used within foundation influence zones should have a minimum compressive strength of 75 psi.

Footing excavations should be filled as soon as possible or be kept moist until concrete placement by regular sprinkling to prevent desiccation. A Cornerstone representative should observe all footing excavations prior to placing reinforcing steel and concrete. If there is a significant schedule delay between our initial observation and concrete placement, we may need to re-observe the excavations.

7.4 DRILLED PIER FOUNDATIONS – CARETAKER RESIDENCE, MAINTENANCE BUILDING, VETERINARY/ADMINISTRATION BUILDING, AND FIRE PREVENTION AND DOMESTIC WATER TANK PADS AND PUMP STATIONS

As discussed, the proposed caretaker residence, maintenance building, and fire prevention and domestic water tank pads and associated pump stations sit near/at the top of a slope while the veterinary/admin building is in close proximity to the landslide labeled Qls #2 on our Site Plan. We recommend that these structures be supported on drilled, cast-in-place, straight-shaft friction piers with a structural slab spanning between. The piers should have a minimum diameter of 18 inches and extend to a depth of at least 10 feet into bedrock beneath the fill, residual soils, and colluvium. In areas of the building where there will be cuts into the claystone greater than 3 feet, we recommend the minimum pier embedment be increased to 15 feet into bedrock. Adjacent piers 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 may be used provided the subgrade soils are prepared in accordance with the "Earthwork" section.

7.4.1 Vertical Capacity and Estimated Settlement

The vertical capacity of the piers may be designed based on an allowable skin friction of 750 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 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 450 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 should not exceed ½-inch to mobilize static capacities and post-construction differential settlement between each pier should not exceed ¼-inch due to static loads.

7.4.2 Lateral Capacity

Lateral loads exerted on the structure 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. The lateral pressure may be increased up to a maximum uniform pressure of 4,000 psf at depth. The upper 5 feet of soil should be neglected when determining lateral capacity due to the sloping ground conditions. The structural engineer should apply an appropriate factor of safety to the ultimate passive pressures.

7.4.3 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 groundwater 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.

Based on our explorations, medium dense to dense clayey sands were encountered at the site. We performed our borings with hollow-stem auger drilling equipment and as such were not able to evaluate the potential for caving soils, which can create difficult drilling conditions. Additionally, the soils are generally fill material and may contain adverse materials. The contractor should plan on encountering potentially caving soils and other materials that may require casing or other stability measures to prevent caving and sloughing into the pier foundations.

Contractors should note that embedment is into bedrock materials, and difficult drilling conditions may occur. Equipment capable of excavating the rock materials will be required. Equipment that includes rock bits, core barrels, downhole percussion hammers, and techniques such as pilot holes may also be required and should be anticipated.

SECTION 8: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

8.1 SLABS-ON-GRADE

The structural engineer should determine the appropriate slab reinforcement for the loading requirements and considering the expansion potential of the underlying soils. For unreinforced concrete slabs, ACI 302.1R recommends limiting control joint spacing to 24 to 36 times the slab thickness in each direction, or a maximum of 18 feet.

8.1.1 Animal Barn

As the Plasticity Index (PI) of the surficial soils ranges up to 34, the proposed interior slabs-on-grade should be at supported on at least 18 inches of non-expansive fill (NEF) to reduce the potential for slab damage due to soil heave. 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.

8.1.2 Cat and Dog Enclosures

As the Plasticity Index (PI) of the surficial soils ranges up to 34, the proposed slabs-on-grade should be supported on at least 12 inches of non-expansive fill (NEF) to reduce the potential for slab damage due to soil heave. Per discussions with the design team, we understand that the cat and dog enclosures are not sensitive structures and some movement of the slabs-on-grade might occur and is considered acceptable. 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.

8.1.3 Maintenance Buildings

As the Plasticity Index (PI) of the surficial soils ranges up to 34, the proposed slabs-on-grade should be supported on at least 18 inches of non-expansive fill (NEF) to reduce the potential for slab damage due to soil heave. 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.

8.1.4 Fire Water Storage Tank

As discussed above, we recommend that the fire water storage tank be constructed on a built up level pad and slab-on-grade supported on drilled piers due to the close proximity to steep slopes to the north. As the Plasticity Index (PI) of the surficial soils ranges up to 34, the proposed slab-on-grade should be supported on at least 18 inches of non-expansive fill (NEF) to reduce the potential for slab damage due to soil heave. The NEF layer should be constructed over subgrade prepared in accordance with the recommendations in the “Earthwork” section of this report. 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.

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 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 crushed rock should be placed below the vapor retarder and consolidated in place with vibratory equipment. The mineral aggregate shall be of such size that the percentage composition by dry weight as determined by laboratory sieves will conform to the following gradation:

Sieve Size	Percentage Passing Sieve
1"	100
¾"	90 – 100
No. 4	0 - 10

  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 EXTERIOR FLATWORK

Exterior flatwork, such as pedestrian walkways, patios, driveways, and sidewalks, may experience seasonal movement due to the native expansive soils; therefore, some cracking or vertical movement of conventional slabs should be anticipated where imported fill is not planned in flatwork areas. There are several alternatives for mitigating the impacts of expansive soils beneath concrete flatwork. We are providing recommendations to reduce distress to concrete flatwork that includes moisture conditioning the subgrade soils, using non-expansive fill, and providing adequate construction and control joints to control cracks that do occur. It should be noted that minor slab movement or localized cracking and/or distress could still occur.

- The minimum recommendation for concrete flatwork constructed on moderately to highly expansive soils is to properly prepare the clayey soils prior to placing concrete. This is typically achieved by scarifying, moisture conditioning, and re-compacting the subgrade soil. Subgrade soil should be moisture conditioned to at least 3 percent over the laboratory optimum and compacted using moderate compaction effort to a relative compaction of 87 to 92 percent (ASTM Test Method D1557). Since the near surface soils may have been previously compacted and tested, the subgrade soils could possibly be moisture conditioned by gradually wetting the soil, depending on the time of year slab construction occurs. This should not include flooding or excessively watering the soil, which would likely result in a soft, unstable subgrade condition, and possible delays in the construction while waiting for the soil to dry out. In general, the subgrade should be relatively firm and non-yielding prior to construction.

- Concrete flatwork, excluding pavements that would be subject to wheel loads, should be at least 4 inches thick and underlain by at least 12 inches of non-expansive fill. Non-expansive fill may include aggregate base, crushed rock, or imported soil with a PI of 15 or less. Non-expansive fill should be compacted to at least 90 percent relative compaction. 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.

- We recommend a maximum control joint spacing of about 2 feet in each direction for each inch of concrete thickness and a construction joint spacing of 10 to 12 feet. Construction joints that abut the foundations or garage slabs should include a felt strip, or approved equivalent, that extends the full depth of the exterior slab. This will help to reduce the potential for permanent vertical offset between the slabs due to friction between the concrete edges. We recommend that exterior slabs be isolated from adjacent foundations.

At the owner’s option, if desired to reduce the potential for vertical offset or widening of concrete cracks, consideration should be given to using reinforcing steel, such as No. 3 rebar spaced at 18 inches on center each direction.

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 the results of the laboratory testing performed on a surficial sample collected from the proposed pavement area and engineering judgment considering the variable surface conditions.

Table 4: 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	9.5	12.0
5.0	3.0	10.0	13.0
5.5	3.0	12.0	15.0
6.0	3.5	12.5	16.0
6.5	4.0	14.0	18.0

*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 use 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. Another alternative is to lime treat the subgrade. We also recommend limiting cuts to 3 feet to reduce the potential for heave of the claystone bedrock.

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

Allowable ADTT	Minimum PCC Thickness (inches)
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 4 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 surficial 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 native 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 with level backfill be designed for the following pressures:

Table 6: Recommended Lateral Earth Pressures

Sloping Backfill Inclination (horizontal:vertical)	Lateral Earth Pressure*	
	Unrestrained – Cantilever Wall	Restrained – Braced Wall
Level	45 pcf	45 pcf + 8H**
2:1	65 pcf	65 pcf + 8H**

* 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

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

10.2.1 Basement Walls

The 2019 California Building Code (CBC) states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. We checked seismic earth pressures for the proposed restrained and unrestrained (cantilever) retaining walls in accordance with CBC 1803.5.12 and ASCE 7-16 Section 11.8.3 using the Design level earthquake. We developed seismic earth pressures for the proposed basement using interim recommendations generally based on refinement of the Mononobe-Okabe method (Lew et al., SEAOC 2010).

Because the veterinary/admin building basement walls will be at or greater than 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. Basement walls are not free to deflect, and should therefore be designed for static conditions as a restrained wall, which is also a CBC requirement. Based on current recommendations for seismic earth pressures, it appears that active earth pressures plus a seismic increment exceed the restrained (i.e. at-rest), static wall earth pressures. Therefore, we recommend checking the walls for the seismic condition in accordance with the interim recommendations of the above referenced paper and the 2013 CBC.

The CBC prescribes basic load combinations for structures, components and foundations with the intention that their design strength equals or exceeds the effects of the factored loads. With respect to the load from lateral earth pressure and ground water pressure, the CBC prescribes the basic combinations shown in CBC equations 16-2 and 16-7 below.

$1.2(D + F) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$ [Eq. 16-2]

In Eq. 16-2: H - should represent the total static lateral earth pressure, which for the basement wall will be restrained (use 45 pcf + 8H psf)

$0.9(D + F) + 1.0E + 1.6H$ [Eq. 16-7]

In Eq. 16-7: H - should represent the static “active” earth pressure component under seismic loading conditions (use 45 pcf)

E - should represent the seismic increment component in Eq. 16-7, a triangular load with a resultant force of $8H^2$, which should be applied one third of the height up from the base of the wall (and which can also be expressed as an equivalent fluid pressure equal to 24 pcf).

The interim recommendations in the SEAOC paper more appropriately split out "active" earth pressure (and not the restrained "at-rest" pressure) from our report and provide the total seismic increment so that different load factors can be applied in accordance with different risk levels.

10.2.2 Site Walls

The 2019 CBC states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. At this time, we are not aware of any site retaining walls for the project. However, minor landscaping walls (i.e. walls 6 feet or less in height) may be proposed. In our opinion, design of these walls for seismic lateral earth pressures in addition to static earth pressures is not warranted.

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). 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 95 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.

10.5 FOUNDATIONS

Retaining walls may be supported on a continuous spread footing or 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 Peninsula Humane Society & SPCA specifically to support the design of the Peninsula Humane Society Animal Sanctuary project in Loma Mar, 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 groundwater conditions encountered during our subsurface exploration. If variations or unsuitable conditions are encountered during construction, Cornerstone must be contacted to provide supplemental recommendations, as needed.

Peninsula Humane Society & SPCA may have provided Cornerstone with plans, reports and other documents prepared by others. Peninsula Humane Society & SPCA 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

Aagaard, B.T., Blair, J.L., Boatwright, J., Garcia, S.H., Harris, R.A., Michael, A.J., Schwartz, D.P., and DiLeo, J.S., 2016, Earthquake outlook for the San Francisco Bay region 2014–2043 (ver. 1.1, August 2016): U.S. Geological Survey Fact Sheet 2016–3020, 6 p., http://dx.doi.org/10.3133/fs20163020.

ATC Hazards by Location, *Hazards by Location*, 2020, *https://hazards.atcouncil.org/*

Brabb, E.E., 1970, Preliminary geologic map of the central Santa Cruz Mountains, California: U.S. Geological Survey, Open-File Report OF-70-36, scale 1:62,500.

Brabb, E.E., 1980, Preliminary geologic map of the La Honda and San Gregorio quadrangles, San Mateo County, California: U.S. Geological Survey, Open-File Report OF-80-245, scale 1:24,000.

Brabb, E.E., and Pampeyan, E.H., 1983, Geologic map of San Mateo County, California: U.S. Geological Survey, Miscellaneous Investigations Series Map I-1257-A, scale 1:62,500.

Brabb, E.E., 1993, Preliminary geologic map of the onshore part of the Palo Alto 1:100,000 quadrangle, California: U.S. Geological Survey, Open-File Report OF-93-271, scale 1:62,500.

Brabb, E.E., and Olson, J.A., 1986, Map showing faults and earthquake epicenters in San Mateo County, California: U.S. Geological Survey, Miscellaneous Investigations Series Map I-1257-F, scale 1:62,500.

Brabb, E. E., Graymer, R.W. Jones, D.L., 1998, Geology of the Palo Alto 30 X 60 Minute Quadrangle, California: a digital database, U.S. Geological Survey Open File Report 98-348.

California Building Code, 2019, Structural Engineering Design Provisions, Vol. 2.

California Geological Survey, (Updated January 26, 2016), Landslide Inventory and Deep-

Seated Landslide Susceptibility Interactive Mapping (Beta version) http://maps.conservation.ca.gov/cgs/lsi/, accessed on April 11, 2019

DMG Staff, 2000, Digital images of Official Maps of Alquist-Priolo Earthquake Fault Zones of California, central coast region: California Division of Mines and Geology, CD 2000-004, scale 1:24,000.

Ellen, Stephen D, and Weiczorek, 1988, Landslides, Floods and Marine Effects of the Storm of January 3-5, 1982, in the San Francisco Bay Region, California, U.S. Geological Survey Paper 1434.

Ellen, S.D., Mark, R.K., Wieczorek, G.F., Wentworth, C.M., Ramsey, D.W., May, T.E., Graham, S.E., Beukelman,G.S., Barron, A.D., and Graymer, R.W., 1997, Map showing principal debris-flow source areas in the San Francisco Bay region, California: U.S. Geological Survey, Open-File Report OF-97-745-E, scale 1:125,000.

Ellis, W.L., Harp, E.L., Arnal, C.H., and Godt, J.W., 1999, Map showing locations of damaging landslides in Santa Clara County, California, resulting from 1997-98 El Nino rainstorms: U.S. Geological Survey, Miscellaneous Field Studies Map MF-2325-J, scale 1:125,000.

Haugerud, R.A., and Ellen, S.D., 1990, Coseismic ground deformation along the northeast margin of the Santa Cruz Mountains: U.S. Geological Survey Open-File Report 90-274, p. 32-37.

Hitchcock, C. S., Kelson, K. I., and Thompson, S. C., 1994, Geomorphic Investigations of Deformation Along the Northeastern Margin of the Santa Cruz Mountains, U. S. Geological Survey Open-File Report OFR 94-187.

Lawson, A.C., chairman, 1908, The California earthquake of April 18, 1906; Report of the State Earthquake Investigation Commission: Carnegie Institute Washington Publication 87, v. I, 451 p. (Reprinted, 1969).

Mark, R.K., 1992, Map of debris-flow probability, San Mateo County, California: U.S. Geological Survey, Miscellaneous Investigations Series Map I-1257-M, scale 1:62,500.
Portland Cement Association, 1984, Thickness Design for Concrete Highway and Street Pavements: report.

Pradell, D., 1988, Procedure to Evaluate Earthquake-Induced Settlements in Dry Sandy Soils, Journal of Geotechnical and Environmental Engineering, April 1998, p. 364 – 368 and Errata October 1998 p. 1048.

Rogers, T.H., 1971, Environmental geologic analysis, Santa Cruz mountain study area, Santa Clara County, California. California Division of Mines and Geology open- file report 72-21.

State of California Department of Transportation, 2018, Highway Design Manual, Sixth Edition, December 14, 2018.

Tokimatsu, K., and Seed, H. Bolton, 1987, Evaluation of Settlements in Sands due to Earthquake Shaking, ASCE Journal of Geotechnical Engineering, Vol. 113, August 1987, pp. 861-878.

San Mateo County, 2008, Hazards Mitigation Maps: on-line site at http://www.co.sanmateo.ca.us/smc/department/home/0,,5557771_5558929_436489912,00.html

U.S. Geological Survey, California Geological Survey, 2006, Quaternary fault and fold database for the United States, accessed March 30, 2019, http//earthquake.usgs.gov/hazards/qfaults/.

Wieczorek, G.F., 1982, Map showing recently active and dormant landslides near La Honda, central Santa Cruz Mountains, California: U.S. Geological Survey, Miscellaneous Field Studies Map MF-1422, scale 1:48,000.

Working Group on California Earthquake Probabilities, 2015, The Third Uniform California Earthquake Rupture Forecast, Version 3 (UCERF), U.S. Geological Survey Open File Report 2013-1165 (CGS Special Report 228). KMZ files available at: www.scec.org/ucerf/images/ucerf3_timedep_30yr_probs.kmz

Aerial Photos Reviewed

Vertical photos:
1953, 1956, 1960, 1968, 1980, 1982, 1991, 2005, 2009, 2010, 2012, 2014, 2016

Stereo Aerial Photos:
March 30, 1931, black and white, flight C-1471, frames 118, 119, scale: 1:18,000.

April 24, 1948, black and white, flight CDF5, frames 1-58, scale: 1:20,000.

May 1, 1965, black and white, flight CAS-65-130, frame 3-56, scale: 1:12,000.

APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using track-mounted, hollow-stem, limited-access auger drilling equipment. Seven 6½-inch-diameter exploratory borings were drilled on January 20 and 21, 2020 to depths of 15 to 21½ feet. Two 3-inch diameter exploratory hand auger borings were drilled on January 21, 2020, to a depth of 4 to 4½ feet. The approximate locations of exploratory borings are shown on the Site Plan, Figure 2. 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.

Boring locations were approximated using existing site boundaries, and other site features as references. Boring elevations were not determined. The locations of the borings should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings 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 logs.

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.

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

APPENDIX B: LABORATORY TEST PROGRAM

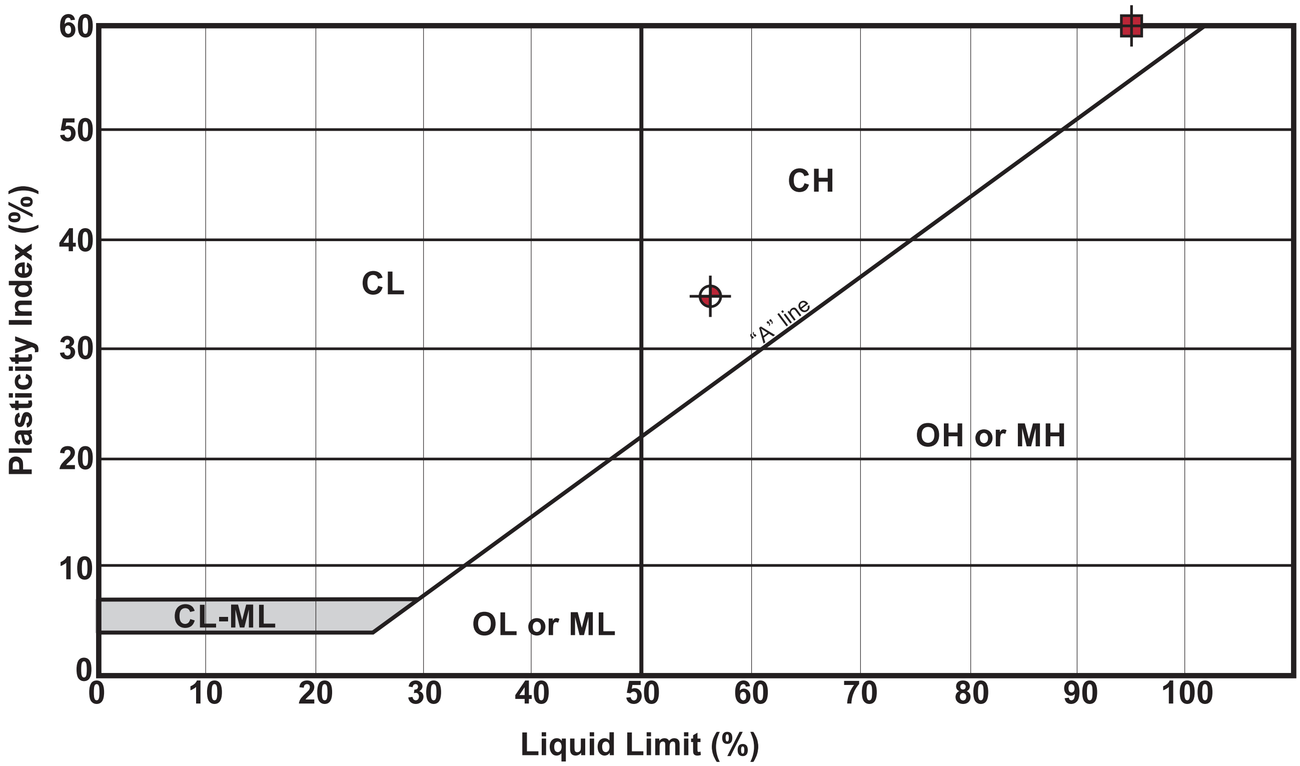
The laboratory testing program was performed to evaluate the physical and mechanical properties of the soils retrieved from the site to aid in verifying soil classification.

Moisture Content: The natural water content was determined (ASTM D2216) on 41 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 17 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	2.0	28	57	23	34	—	Fat Clay with Sand (CH) [Residual Soil]
	EB-1	5.5	43	95	35	60	—	Claystone [Tpt]

CORNERSTONE EARTH GROUP

Plasticity Index Testing Summary

Haskin's Ridge Animal Sanctuary
Pescadero Creek Road
La Honda, CA

Project Number 1142-1-1

Figure Number Figure B1

Date February 2020

Drawn By FLL