CORNERSTONE
EARTH GROUP

Type of Services	**Preliminary Geotechnical and Geologic Hazard Investigation**
Project Name	**The Stanford Wedge**
Location	**Alpine Road** **Portola Valley, California**
Client	**Stanford Real Estate**
Client Address	**3160 Porter Drive, Suite 200** **Palo Alto, California**
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DRAFT

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SECTION 1: INTRODUCTION

This preliminary geotechnical and geologic hazard investigation and report was prepared for the sole use of Stanford Real Estate for The Stanford Wedge project in Portola Valley, California. The location of the site is shown on the Vicinity Map, Figure 1. For our use, we were provided with the following documents:

- A concept package displaying three site plan options titled, "The Stanford Wedge, Portola Valley, CA," prepared by Hospitality Group, dated August 10, 2016.

- An untitled, undated exhibits package displaying government jurisdictions, the total property area owned by Stanford including topographic elevations of the property (datum unknown), and the approximate site area within the property owned by Stanford that is to be developed.

1.1 PROJECT DESCRIPTION

The project site is located on the west side of Alpine Road and to the south of Westridge Drive in Portola Valley, California. We understand that Stanford owns an approximately 75-acre plot of land along Alpine Road and plans to develop the flatter, approximately 5-acre portion, closest to Alpine Road. This approximately 5-acre portion is called "The Stanford Wedge" and is generally undeveloped and occupied by horse corrals and horse grazing areas.

Site development plans are in the early planning stages. However, based on the site plan options provided (Options A to C) and discussions with you, we understand the project will be a residential development consisting of approximately 30 or so single-family detached homes on small clustered lots. Quad units will average approximately 2,400 to 2,500 square feet (excluding garage) and will be one and two stories. Affordable housing units will average about 700 to 750 square feet and will be one and/or two stories. We anticipate the residential development will be wood-framed construction. Appurtenant parking and drive aisles, utilities, landscaping and other improvements necessary for site development are also anticipated.

Structural loads are not known at this time for the proposed structures; however, structural loads are expected to be typical of similar type structures. Grading information is not available at this time but is anticipated to generally include cuts and fills less than 5 feet to grade the site for the building pads. Some larger cuts on the order of 5 to 10 feet may be required to grade building pads along the northwest perimeter of the site. As a result, retaining structures may be required in various locations across the site.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated June 15, 2017, and consisted of field and laboratory programs to evaluate physical and engineering properties of the subsurface soils, engineering analysis to prepare preliminary recommendations for site work and grading, building foundations, flatwork, retaining walls, and pavements, preliminary fault and slope stability screening, and preparation of this report. Brief descriptions of our exploration and laboratory programs are presented below.

1.3 EXPLORATION PROGRAM

Our exploration consisted of literature review, site reconnaissance, geologic mapping, and physical exploration. The following sections describe each task.

1.3.1 Literature Review

Published geologic maps and aerial photographs were researched and reviewed for this investigation and are listed in the “References” section of this report.

1.3.2 Site Reconnaissance and Geologic Mapping

Our geologist performed a site reconnaissance on July 2, 2017 to map the aerial extent of geologic deposits and obtain other details regarding the site geologic conditions and potential geologic and geotechnical hazards at and immediately adjacent to the site. The results of the reconnaissance and mapping are discussed in the following sections.

1.3.3 Field Exploration

Field exploration consisted of five borings drilled on July 20 and 21, 2017, with truck-mounted, hollow-stem auger drilling equipment and three Cone Penetration Tests (CPTs) advanced on July 7, 2017. The borings were drilled to depths of 8 to 30½ feet; the CPTs were advanced to depths of about 3 to 13½ feet. All borings encountered difficult drilling. Practical refusal was encountered in Boring EB-4 at a depth of 8 feet. Boring EB-4A was performed approximately 5 feet away and encountered practical refusal at 9½ feet. Practical refusal was encountered in all CPTs. Shear wave velocity measurements were collected from CPT-1 and CPT-2 to the depth of refusal. Borings EB-1, EB-2, and EB-3 were advanced adjacent to CPT-1, CPT-2, and CPT-3, respectively, for direct evaluation of physical samples to correlated soil behavior.

The borings and CPTs 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 and CPTs are shown on the Site Plan and Geologic Map, Figure 2. Details regarding our field program are included in Appendix A. Figure 2 also shows our previous field investigation from the C-1 Trails project; the details of the previous investigation are included in Appendix C.

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, washed sieve analyses, 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, the project environmental consultant should review our geotechnical recommendations for compatibility with the environmental concerns.

SECTION 2: REGIONAL SETTING

2.1 REGIONAL GEOLOGICAL SETTING

The site is located within the marginal foothills of the Peninsula segment of the Santa Cruz Mountains (in the southwest). This portion of the foothills has been dissected by creeks including the Los Trancos Creek on the southeast and east of the site. A number of published regional-scale maps cover the general region around the site including: Dibblee (1966), Pampeyan (1970 and 1993), Brabb et al. (1998), and Dibblee and Minch (2007). The most detailed mapping covering the area of the site is that of the Town of Portola Valley geologic peer review consultant Cotton Shires and Associates, Inc. (CSA). Their map is published at a scale of 1” = 500 feet. CSA indicates that the nearby foothills to the southwest are composed of primarily Upper Pliocene and older sedimentary rocks (Ladera and Whisky Hill formations), which in the valley bottom alongside Los Trancos Creek are mantled by Quaternary alluvium (gravel, sand, silt, and clay). The majority of the published maps indicate the contact between the Whisky Hill Formation and the Ladera Sandstone has been understood and depicted as a fault contact as discussed in the “Fault Rupture” section below. These maps show this faulted contact trending through the center (Pampeyan, 1970, 1993) or through the western edge of the site (CSA, 2010). CSA’s 2010 published map for the Town of Portola Valley was used as the base for our Vicinity Geologic Map, Figure 4.

2.2 REGIONAL SEISMICITY

While seismologists cannot predict earthquake events, geologists from the U.S. Geological Survey have recently updated earlier estimates from their 2015 Uniform California Earthquake Rupture Forecast (Version 3) 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%), Rodgers Creek (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 or Rodgers Creek Faults. During such an earthquake, very strong to severe ground shaking would occur.

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)
Monte Vista-Shannon	0.7	1.0
San Andreas (1906)	1.9	3.0
San Gregorio	12.7	20.5

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 RECONNAISSANCE AND SURFACE DESCRIPTION

The site was visited by our Certified Engineering Geologist on July 2, 2017. The triangular shaped site is currently accessed by a dirt road that extends into the site from Alpine Road at the northeast property corner. At the time of the reconnaissance the property was being used as an equestrian center with stables and fenced dirt exercise areas. The central portion of the site includes horse pens and numerous wood-framed storage sheds as well as miscellaneous stockpiled construction materials and some man-made debris.

The majority of the site is nearly flat to very gently sloping down toward Alpine Road to the east and northeast. A subtle, southeasterly trending slope break occurs through the central portion of the site. The northeastern facing slope along the southwest perimeter of the site is moderately inclined (approximately 6:1 horizontal: vertical) then becomes steep within about 70 feet southwest of the southwest property line and beyond (to approximately 2:1). Based on the topography provided on the site plans for the project, the site slopes gradually up from approximately Elevation 315 to 320 feet at the northeast corner to approximately Elevation 355 to 360 feet near the southwest side and southeast corner of the site.

An unnamed creek channel varying from 6 feet to 9 feet deep trends northeasterly along the north property line (refer to Figure 5). The channel of Los Trancos Creek (roughly 20 feet deep) exists approximately 150 to 250 feet southeast of the site on the far side of Alpine Road from the site.

3.2 SITE GEOLOGY AND SUBSURFACE CONDITIONS

As already mentioned, mapping by Cotton Shires and Associates (2010) shows the area of the site and adjacent sloping areas and hillsides as dominated by both the Ladera Sandstone and the Whisky Hill Formation, with the majority of the site shown as overlain by a relatively thin mantle of Quaternary alluvium overlying the older formations. The Ladera Sandstone is shown as concealed beneath the alluvium in the northeastern portion or half of the property. The Ladera Sandstone consists of marine silty sandstone to sandy siltstone. Pampeyan 1994 describes the Ladera as “mainly consists of fine-grained sandy siltstones to silty sandstones.” In their geologic guide for the Stanford Linear Accelerator site, Ehman and Witebsky describe the Ladera as “massive, lightcolored, fine- to medium-grained sandstone that was well sorted and sparsely fossiliferous…The unweathered Ladera Sandstone is composed of dark greenish gray to light olive gray, mottled sandy siltstone and silty sandstone that is soft and friable…The typical Ladera Sandstone observed in core is composed of intensely burrowed and homogenized sandy siltstone and silty sandstone. Well-defined bedding is rare because of the extensive bioturbation. If preserved, parallel laminations are the most common sedimentary structure, and rare fine ripple laminations” (Ehman and Witebsky, 2006).

The Whiskey Hill Formation is shown as underlying the alluvium within the southwestern portion of the property. Paymeyan (1994) indicates that the Whiskey Hill Formation consists of “Pale yellowish-orange to pale yellowish-brown, poorly cemented to very well cemented, poorly sorted, coarse-grained, thick bedded, feldspathic sandstone and interbedded silty claystone. The formation contains chaotic zones which consist of large blocks of sandstone in a sheared claystone matrix and also contains exotic blocks of metabasalt, quartzite, and granitic rocks. The Whiskey Hill Formation was probably deposited in a deep-marine slope and basin.”

Whiskey Hill Formation and Ladera formations are exposed locally along Westridge Drive and also locally within the channel along Los Trancos Creek. Along Westridge Drive the Whiskey Hill is moderately-severely weathered and massive but contains fractures and joints. The Ladera along the road cuts is severely weathered and friable. A review of an exposure of Whiskey Hill Formation within the creek indicates thick to medium bedding with a structural trend of N75°E/35°NW. The CSA (2010) map shows bedding (overturned) at a creek exposure of the Ladera Sandstone as dipping steeply (75°) toward the southwest. They (CSA) also show that bedding within the Whiskey Hill Formation on the hillside located just west of the property as variable in direction and dip angle; however, most of the outcrop locations have become overgrown and/or obscured by development since their original collection and these locations are located on private property. A bedding measurement taken within the Whiskey Hill

Formation at a location just west of the site shows a 60-degree dip toward the southeast. We have taken these bedding structural measurements into account (interpolated) and the fact that the two formations are in depositional contact with one another in reconstructing a geologic cross section along A-A'. This cross section depicts the apparent dip of bedding and the geologic contact along that trend.

Our borings extended to depths ranging from 8 to 30½ feet and encountered thin surficial fill/tilled and disturbed material forming a veneer over the Quaternary Alluvium geologic unit. The Quaternary alluvium consisted of alternating layers of generally medium dense to very dense silty sand with gravel, clayey sand with variable amounts of gravel, well-graded sand with silt and gravel, well-graded gravel with sand, and hard lean clays with variable amounts of sand and silt. The base of the alluvial stratigraphic section contains a concentration of boulders and cobbles of cemented conglomerate, which generally impeded auger advancement and at Borings EB-4 and EB-4A led to practical sampling and drilling refusal. Whiskey Hill Formation materials (sandstone and claystone) were encountered in Borings EB-2 and EB-3 at depths of 16 and 7 feet, respectively. Ladera sandstone was encountered at a depth of 18 feet in Boring EB-1.

3.2.1 Plasticity/Expansion Potential

We performed six Plasticity Index (PI) tests on representative samples from the upper 10½ feet of the existing grades. Test results were used to evaluate expansion potential of surficial soils, plasticity of the underlying claystone, and the plasticity of the fines in potentially liquefiable layers. The results of the surficial soil tests indicated PIs ranging from 4 to 22, indicating low to moderate expansion potential to wetting and drying cycles. The results of the claystone PI test indicated a PI of 37, indicating a high expansion potential. Results of the PI tests in potentially liquefiable layers indicated the soils to be too plastic to liquefy, in our opinion, with PIs greater than 7.

3.2.2 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents of the soils within the upper 10 feet range from 5 or more percent below optimum to about 2 to 3 percent over the estimated laboratory optimum moisture.

3.3 GROUND WATER

Free ground water was not encountered in any of our borings during drilling. The California Geologic Survey (Palo Alto 7.5 Minute Quadrangle, 2006) maps ground water in the site vicinity as being greater than 30 feet. However, based on the site geology, we anticipate temporary perched water conditions, especially in winter months, could occur at shallower depths within the alluvial layers.

Fluctuations in ground water levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors.

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT RUPTURE

The Monte Vista – Shannon Fault Zone is the closest active fault and it is located approximately 0.7 miles to the southeast. As discussed above, two other significant faults are located within 25 kilometers of the site. The active San Andreas Fault is located about 2 miles southwest of the site. More locally, there are several thrust and reverse faults in the vicinity of the Stanford campus which are coeval with folding in the area. These Quaternary reverse or thrust faults are subparallel with the San Andreas Fault zone and include; the Pulgas Fault, the San Juan Hill Fault, the Willow Road Bridge Fault, the Deer Creek Fault, the Arastadero Thrust Fault, the Hermit Fault, the Stanford Fault, and the Frenchman's Road Fault (Dames & Moore, 1995). Although all of these structures are contemporary with relatively recent activity on the San Andreas Fault Zone and in general with movements along the modern transform plate boundary, to date no compelling evidence has been encountered at the ground surface indicating Holocene fault movement on these structures (Kovach and Page, 1995). The Stock Farm Monocline is a late Pleistocene or Holocene structure originating from subsurface slip on a blind thrust fault. The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone, known formerly as a Special Studies Zone (CDMG, 1974). More locally, the Town of Portola Valley's Ground Movement Potential Map and Geologic Map (available through the Town's Planning Department webpage) indicates a fault trends through the southwestern portion of the site along a northerly trending natural break-in-slope (moderate to gently inclined) (CSA, 2010). The contact between the Whiskey Hill (on the west) against the Ladera (on the east) has traditionally been depicted as a fault on various geologic maps covering the area; Pampeyan (1970 and 1993), Brabb and Pampeyan (1983), Brabb et al. (1998); Bryant (2000), USGS (2006), and Dibblee and Minch (2007).

When asked recently by email communication about the status of this mapped fault, John Wallace (Senior Geologist) of Cotton Shires and Associates indicated:

> "We have been trying to find this fault for 30 years with no success in this area. We believe it was mapped based upon the steep east-facing hillside west of your site, and the change between Whiskey Hill and the Ladera Sandstone to the north. However, the excavation for the Stanford Linear Accelerator did not find this to be a faulted contact, but depositional. Lots of borings and trenches have been done in the area and found no signs of faulting.[1]"

Accordingly, Mr. Wallace provided us a revised copy of the Town's Ground Movement Potential Map attached as Figure 5, which depicts the contact as a depositional contact and the fault has been removed from the map. It should be noted the revised version of this map is not yet posted on the Town's webpage. During our field investigation we did encounter evidence of a contact between the Tw and Tl units (concealed beneath the overlying Quaternary Alluvium) extending with a southeast trend through the central portion of the site as suggested by the mapping of Pampeyan (1970, 1993) and Brabb and Others (1998). This contact is generally

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[1] John Wallace, email communication with Craig Harwood, CEG, July 27, 2017

coincident with a subtle, southeast trending break-in-slope. This slope break however is likely due to differential erosion across the geologic contact as the Whiskey Hill formation is harder and therefore more resistant to erosion than the adjacent Ladera Formation. Our review of historic aerial photography did not reveal any tonal variations suggesting faulting on the aerial photographs of the site, nor did we encounter any evidence in the field suggesting the presence of faulting in the immediate area.

In our judgment, the proposed development is not potentially impacted by primary fault surface rupture.

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 $F_{PGA} \times$ PGA, as allowed in the 2016 edition of the California Building Code. For our analyses we used a PGA_M of 1.090g.

4.3 LIQUEFACTION POTENTIAL

The eastern portion of the site is within a State-designated Liquefaction Hazard Zone (CGS, Palo Alto Quadrangle, 2006), as depicted on Figure 6. Additionally, as shown on the “Ground Movement Potential Map,” Figure 5, most of the site is depicted as being in area “Sun” described as “…liquefaction possible at valley floor sites during strong earthquakes.” Our field and laboratory programs addressed this issue by testing and sampling potentially liquefiable layers, performing visual classification on sampled materials, evaluating CPT data, and performing various tests to further classify soil properties.

4.3.1 Background

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.

4.3.2 Analysis

As discussed in the “Subsurface” section above, sand layers were encountered in the Quanternary Alluvium above bedrock. Ground water was not encountered in any of our borings and is mapped at depths greater than 30 feet beneath the surface. However, as mentioned, we anticipate temporary perch ground water conditions could occur above the bedrock within the Quanternary Alluvium at times of the year. As such, to be conservative, we evaluated the potential for liquefaction utilizing a ground water depth of 2 feet below the ground surface. Following the liquefaction analysis framework in the 2008 monograph, *Soil Liquefaction During Earthquakes* (Idriss and Boulanger, 2008), incorporating updates in *CPT and SPT Based Liquefaction Triggering Procedures* (Boulanger and Idriss, 2014), and in accordance with CDMG Special Publication 117A guidelines (CDMG, 2008) for quantitative analysis, these layers were analyzed for liquefaction triggering and potential post-liquefaction settlement. These methods compare the ratio of the estimated cyclic shaking (Cyclic Stress Ratio - CSR) to the soil's estimated resistance to cyclic shaking (Cyclic Resistance Ratio - CRR), providing a factor of safety against liquefaction triggering. Factors of safety less than or equal to 1.3 are considered to be potentially liquefiable and capable of post-liquefaction re-consolidation (i.e. settlement).

The CSR for each layer quantifies the stresses anticipated to be generated due to a design-level seismic event, is based on the peak horizontal acceleration generated at the ground surface discussed in the "Estimated Ground Shaking" section above, and is corrected for overburden and stress reduction factors as discussed in the procedure developed by Seed and Idriss (1971) and updated in the 2008 Idriss and Boulanger monograph.

For CPT analysis, the soil's CRR is estimated from the in-situ measurements from CPTs and laboratory testing on samples retrieved from our borings. The tip pressures are corrected for effective overburden stresses, taking into consideration both the ground water level at the time of exploration and the design ground water level, and stress reduction versus depth factors. The CPT method utilizes the soil behavior type index (I_C) to estimate the plasticity of the layers. Selected soil samples collected from our borings advanced adjacent to the CPT were tested for fines contents and plasticity, as well as visually observed for comparison of CPT soil behavior types. For SPT analysis, the soil's CRR is estimated from the in-situ density and strength obtained from field SPT blow counts ("N" value). The "N" values are corrected for effective overburden stresses, taking into consideration both the ground water level at the time of exploration and the design ground water level, and stress reduction versus depth factors. The "N" values are also corrected for fines content, hammer efficiency, boring diameter, rod length, and sampler type (with or without liners).

Soils with significant quantities of plastic fines are typically considered too plastic or too dense/stiff to liquefy. These soil layers have been screened out during our analyses based on laboratory analysis.

Based on the soils encountered, the soils below a depth of 2 feet are generally hard cohesive clays, dense to very dense sands and gravels, or medium dense sands with plastic fines with PIs of 8 or greater, which in our opinion are too plastic to liquefy. Based on the above, our analysis indicates a low potential for liquefaction to occur at the site.

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.

As discussed, an unnamed creek channel trends northeasterly along the north property line. Additionally, Los Trancos Creek is located on the far (easterly) side of Alpine Road from the site. However, as the potential for liquefaction at the site is low, in our opinion the potential for lateral spreading to affect the site is low.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose to medium dense unsaturated sandy soils can settle during strong seismic shaking. Based on the work by Robertson and Shao (2010), subsurface conditions encountered including soil density, fines contents, and plasticity of fines, and our engineering judgement, indicate the potential for differential seismic settlement affecting the proposed improvements is low, in our opinion.

4.6 SLOPE STABILITY

The various published landslide-themed maps at the site (Brabb and Pampeyan, 1972; Jayko et al., 1989; California Geological Survey, 2006 and 2015) do not show landslides mapped at or near the site. Additionally, there are no landslides depicted as occurring at the site on the majority of regional scale geologic maps (CSA, 2010; San Mateo County, 2006). An exception to this interpretation however is the mapping of Pampeyan (1970 and 1993) that shows a moderate sized (“old”) landslide on the moderately inclined slope that exists within the southwestern margin of the site and extends beyond the southwest property line. However, the more detailed and more recent mapping by the Town’s geologic peer reviewer Cotton Shires and Associates did not adopt this mapping and therefore does not show any landslides in this same area (CSA, 2010). The CSA map however does show four landslides ranging from small to moderate in size within the easterly trending drainage canyon (south of Westridge Drive) located about 1,200 feet west of the site. These slope failures are topographically isolated from the site, have moved either in a northerly direction or in a southerly direction within the drainage and do not potentially impact the subject site. An additional slope failure is shown as crossing Westridge Drive northwest of the site. This slide has also moved in a direction that does not potentially impact the site.

As shown on the “CGS Seismic Hazard Map” Figure 6, an earthquake-induced landslide zone is mapped by the California Geologic Survey adjacent to the southwest end of the site within the slope along the southwest perimeter of the site (CGS, 2006). Also, the ground movement potential map (by CSA, 2017) indicates slopes in this area of the site are designated with a “sbr” mapping symbol. The “sbr” designation is characterized as “Level ground to moderately steep slopes underlain by bedrock within approximately three feet of the ground surface or less; relatively thin soil mantle may be subject to shallow landsliding, settlement, and soil creep.” Our review of maps, reconnaissance and observations of outcrops in the area suggest this slope is underlain by the Whiskey Hill Formation which is generally semi-cemented and in a very dense condition.

A drainage swale located just to the west of the site that feeds the unnamed, seasonal creek along the north property line is characterized on the Town’s GMP map as being within a “Ps” zone. A narrow band of Ps zone is also shown as being along the southwest property line. The Ps zone is described as; “Unstable, unconsolidated material, commonly more than 10 feet in thickness, on moderate to steep slopes subject to shallow landsliding, slumping, settlement and soil creep.” This characterization is not based on site-specific investigations but on interpretative work using remote sensing and generalized bedrock characteristics. This Ps zone appears to be concentrated within the trough of the drainage channel. This drainage is directed toward the adjacent properties located just beyond the north property line and does not potentially impact the development. An additional area of “Ps” zoning is shown on the southerly facing slope that descends from Westridge Road, toward the northwest corner of the site. The toe of this slope is crossed by the seasonal creek which is about 8 to 12 feet deep in this area. We noted no evidence of previous slope failures extending onto this portion of the site in aerial photos or geomorphic evidence noted in our site reconnaissance. In all likelihood, the creek would provide a catchment area for a shallow slope failure emanating from this slope. However, this portion of the site should be further explored in the design phase with test pit excavations to determine possible evidence of past slope failures toeing out in this portion of the property.

Our site reconnaissance did not reveal any unusual topography in this area and the aerial photos reviewed did not show any features that would suggest that this sloping area had been subjected to landsliding or slope debris events. Roadcut exposures of bedrock along Alpine Road suggests the bedrock underlying this slope is likely at shallow depths. Shallow slumping of the colluvial mantle is possible.

4.6.1 Preliminary Slope Stability Screening

As discussed, project development plans are preliminary and grading plans are not yet available. However, we understand some larger cuts on the order of 5 to 10 feet may be required to grade building pads along the northwest perimeter of the site. As a result, retaining structures may be required in various locations across the site. As part of this preliminary investigation, we performed a preliminary slope stability screening. Our preliminary analysis was performed using data obtained from our site observations, mapped geology, topographic information provided, our onsite boring explorations, and published Whisky Hill Formation strength parameters from CGS. We also utilized available acceleration data from the 1989 Loma Prieta earthquake to back calculate minimum bedrock strength parameters for our analysis.

Based on our preliminary screening, our analysis using the computer program SLIDE indicates the slope could potentially experience instability during a seismic event. As we have adopted published bedrock structural measurements (strike/dip) from areas located off site and projected them onto our cross section, we did not have the opportunity to confirm the bedrock structural trends on site in this initial study. Our current cross section indicated bedding potentially dipping steeply in the direction of the site but not necessarily daylighting on the natural slope. We recommend additional field exploration including test pits and exploratory borings/rock cores be performed during a design-level investigation to further evaluate the bedrock bedding conditions and to collect samples for bedrock strength testing of the northeast facing slope beyond the southwest property line. This will enable us to evaluate whether favorable or adverse bedding conditions exist and to better estimate the strength of the rock and overlaying colluvium to further evaluate the stability of the slope and determine whether any mitigation is required to protect the development from a slope failure during a significant seismic event.

The slope screening performed generally addresses deep seated land sliding. Shallow slumping of the colluvial mantle is possible and should be further addressed in the design-level report. Additional freeboard on retaining walls or infrastructure for debris retention may be required.

4.7 FLOODING

Based on our internet search of the Federal Emergency Management Agency (FEMA) flood map public database, the site is located within Zone X, described as "Areas determined to be outside the 2% annual chance floodplain." We recommend the project civil engineer be retained to confirm this information.

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. The preliminary recommendations that follow are intended for conceptual planning and preliminary design. A design-level geotechnical investigation should be performed once final site development plans are prepared showing the final building layouts, grading information, and other proposed improvements. The design-level investigation findings will be used to confirm the preliminary recommendations and provide more detailed recommendations, as needed, for the final design and construction. Descriptions of each geotechnical concern with brief outlines of our preliminary recommendations follow the listed concerns.

- Presence of expansive bedrock and soils
- Potential for slope instability
- Cut/fill transitions
- Potential for cobbles and/or boulders beneath foundations
- Undocumented fills, tilled/disturbed soils, and previous site development
- Potential excavation difficulties in sands
- Potential for hard bedrock

5.1.1 Presence of Expansive Bedrock and Soils

The Whiskey Hill Formation is known for highly expansive claystone beds interbedded with sandstone beds. The Whiskey Hill Claystone has severe design complications. Care should be taken to isolate water from this stratum as well as reducing or eliminating cuts into this material. The claystone is very sensitive to load relief (i.e. cuts) and, for example, may expand on the order of 10 inches for a 10-foot-cut above a 30-foot-thick claystone bed. In addition, expansion may not be apparent until 1 to 2 years after cuts are completed (Meehan, et al., 1975). The addition of water will increase the amount of heave. Differential movement may occur between the claystone and sandstone beds if heave within the claystone beds occurs.

As discussed, we encountered Whiskey Hill Claystone in two borings at depths of 7 and 16¼ feet beneath the surface. At this time, grading plans are not available. On a preliminary basis, if grades are not cut significantly (more than approximately 1 to 2 feet) we anticipate the underlying Whiskey Hill Claystone would not be significantly affected and buildings could be founded on shallow foundations. However, if grades are more significantly cut, deep foundations may be required along with structural slabs and void forms beneath the slabs. During a design-level investigation once grading plans have been determined, the potential for Whiskey Hill Claystone heave should be further evaluated and recommendations provided. Test pits with an excavator are recommended.

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

5.1.2 Potential for Slope Instability

As discussed, we performed a preliminary slope stability screening of the slope along the southwest perimeter of the site. Based on site observations and review of published documents, the slope directly above the site does not appear to have been affected by past landslides. However, the portion of the slope near the southwest end of the site is mapped by CGS as an earthquake-induced landslide zone and the CSA mapping shows active landslides on the more northern facing slope to the northwest.

Based on mapped and observable site geology, topographic information provided, and published bedrock strength parameters, our preliminary stability screening indicates the potential for deep seated slope instability during a seismic event. We recommend additional exploration, including test pits with an excavator on the slope be performed to further evaluate the geology and bedding conditions within the slope. Laboratory testing should also be performed on the materials collected from the explorations to evaluate the slope specific material strength parameters. Further stability analysis should then be performed to evaluate potential instability and any mitigation, if required.

Shallow slumping of the colluvial mantle is possible and should be further addressed in the design-level report. Additional freeboard on retaining walls or infrastructure for debris may be required.

5.1.3 Cut/Fill and Material Transitions

As discussed, site grading plans are not available at this time; however, we anticipate several cut and fill areas throughout the site during grading for future building pads, and therefore several cut/fill transitions. Building foundations and improvements that span over cut/fill transitions will be susceptible to differential settlement. Additionally, differing subsurface conditions at the site could increase the potential differential movement underneath the buildings at transitions. To limit potential differential settlements, there should be a relatively uniform fill thickness beneath building footprints. Grading recommendations addressing this concern are presented in the following section of this report.

5.1.4 Potential for Cobbles Beneath Foundations

Concentrations of cobbles and boulders were generally encountered in the base of the alluvial layer. These cobbles and boulders can adversely affect foundations (sources of hard points) for foundations. As such, we recommend these cobbles and boulders be removed when located close to the bottom of foundations. Preliminary grading recommendations addressing this concern are presented in the following section of this report.

The grading contractor should anticipate difficult grading conditions where cuts will extend near the base of the alluvial stratigraphic section, where the concentration of boulders and cobbles of cemented conglomerate will be encountered.

5.1.5 Undocumented Fills, Tilled/Disturbed Soils, and Previous Site Development

As discussed, the site is currently being used as an equestrian center with stables and fenced dirt exercise areas and stables. Wood framed storage sheds and stockpiled construction materials and some man-made debris are also present at the site. Additionally, thin surficial fills were observed in locations of the site and the surficial soils appear tilled and disturbed in various locations. We recommend undocumented fills, existing improvements, and tilled/disturbed soils be completely removed and replaced as engineered fill within building locations. Recommendations addressing this concern are presented in the “Earthwork” section below.

5.1.6 Potential Excavation Difficulties in Sands

As discussed in the subsurface section, the site is underlain by alluvium comprised of hard clays and medium dense to very dense sands with varying amounts of fines. Additionally, large gravels and cobbles were encountered in these materials. Depending on final site grading, excavations may extend into these sandier materials. Excavations into the medium dense sands with gravels and cobbles may result in caving of sidewalls. In addition, excavation subgrades and bottom of footings within the sandier soils can be easily disturbed by foot traffic and/or construction equipment. To reduce the potential for caving, we recommend that the construction schedule be coordinated to minimize the amount of time excavations are left open. In addition, the contractor should be prepared to form or shore excavations or slope excavation walls, if necessary. If caving does occur, it may result in additional work to remove caved soils and additional concrete and forming of footings may be necessary. Manual segregation of large gravel and cobbles from excavated materials may be necessary prior to reusing for fill. If the bottom of a footing excavation is disturbed, it may be necessary to compact the excavation subgrade using hand tools or vibratory compaction equipment prior to placing reinforcing steel. We should observe all footing excavations at the time of construction and confirm these recommendations.

5.1.7 Potential for Hard Bedrock

As mentioned above, Whiskey Hill Sandstone and Claystone and Ladera Sandstone were encountered in our borings beginning at depths ranging from 7 to 18 feet; however, the bedrock may be encountered at shallower depths across the site during site grading, depending on the final grading plan. Based on the bedrock at the site, we anticipate the bedrock would generally be excavatable with typical excavating equipment, however, some areas of harder and more resistant rock may pose difficulties during the excavations for grading. Contractors should anticipate using appropriate means and methods to excavate through areas of harder bedrock. Difficult excavation conditions resulting in slower production should be anticipated. Based on our borings and experience, we anticipate that the harder sandstone can be removed with a hoe-ram and an excavator.

5.2 DESIGN-LEVEL GEOTECHNICAL INVESTIGATION

The preliminary recommendations contained in this study were based on the current preliminary site development plans. As site conditions may vary significantly between the small-diameter borings performed during this investigation and recommendations could vary based on final proposed plans and building layout, we also recommend that we be retained to 1) provide a design-level geotechnical investigation and report, once more detailed site development plans are available; 2) 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; and 3) be present to provide geotechnical observation and testing during earthwork and foundation construction.

SECTION 6: PRELIMINARY EARTHWORK MEASURES

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.

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. Occasionally, other types of buried structures (wells, cisterns, debris pits, etc.) can be found on sites with prior development. If encountered, Cornerstone should be contacted to address these types of structures on a case-by-case basis.

6.1.1 Demolition of Existing Slabs, Foundations and Pavements

All slabs, foundations, and pavements should be completely removed from within planned building areas.

Special care should be taken during the demolition and removal of existing floor slabs, foundations, utilities and pavements to minimize disturbance of the subgrade. Excessive disturbance of the subgrade, which includes either native or previously placed engineered fill, resulting from demolition activities can have serious detrimental effects on planned foundation and paving elements.

Existing foundations are typically mat-slabs, shallow footings, or piers/piles. If slab or shallow footings are encountered, they should be completely removed. If drilled piers are encountered, they should be cut off at an elevation at least 60-inches below proposed footings or the final subgrade elevation, whichever is deeper. The remainder of the drilled pier could remain in place. Foundation elements to remain in place should be surveyed and superimposed on the proposed development plans to determine the potential for conflicts or detrimental impacts to the planned construction. Following review, additional mitigation or planned foundation elements may need to be modified.

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

6.2 CLEARING AND PREPARATION

6.2.1 Site Stripping

The site should be stripped of all surface vegetation, and surface and subsurface improvements within the proposed development area. Demolition of existing improvements is discussed in detail below. Surface vegetation and topsoil should be stripped to a sufficient depth to remove all material greater than 3 percent organic content by weight.

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

Fills were not encountered in our borings. However, the upper soils in various locations of the site appeared tilled and disturbed. Also, manure and landscaping debris appeared scattered in areas. Tilled and disturbed soils, manure and landscaping debris, and any fills encountered following site stripping should be completely removed from within building 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. Provided the fills and tilled and disturbed soils meet the “Material for Fill” requirements below, the fills may be reused when backfilling the excavations. If materials are encountered that do not meet the requirements, such as debris, wood, trash, those materials should be screened out of the remaining material and be removed from the site. Manure and landscaping debris should not be reused as backfill material. Backfill of excavations should be placed in lifts and compacted in accordance with the “Compaction” section below.

Fills and tilled and disturbed soils extending into planned pavement and flatwork areas may be left in place provided they are determined to be a low risk for future differential settlement and that the upper 12 to 18 inches of fill, tilled, and disturbed soils below pavement subgrade is re-worked and compacted as discussed in the “Compaction” section below.

6.4 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. Due to the variability of the surficial soils, on a preliminary basis, site soils should be classified as OSHA Soil Type C materials.

Excavations performed during site demolition and fill removal should be sloped at 3:1 (horizontal:vertical) within the upper 5 feet below building subgrade. Excavations extending more than 5 feet below building subgrade and excavations in pavement and flatwork areas should be sloped at a 1.5:1 inclination unless the OSHA soil classification indicates otherwise.

6.5 CUT/FILL TRANSITION OVER-EXCAVATION

As mentioned above, several cut and fill areas, as well as cut/fill transitions throughout the site will most likely exist for the proposed development. Foundations constructed over cut/fill transitions may experience differential movements under static and seismic loading conditions. Additionally, differing subsurface conditions at the site could increase the potential differential movement underneath the buildings at transitions. To limit potential differential settlements, there should be a relatively uniform fill thickness beneath building footprints. Therefore, in areas where foundations and improvements that straddle cut/fill transitions, additional over-excavation and backfill should occur to provide a uniform fill thickness. On a preliminary basis, cut/fill pad grading should extend at least 5 feet beyond any building footprint.

We recommend that the buildings requiring over-excavation for cut/fill transitions be confirmed during a design-level investigation once final grading plans are completed. We also recommend that these buildings be verified by the grading contractor prior to the start of grading.

6.6 COBBLE OVER-EXCAVATION

As discussed, the base of the alluvial stratigraphic section contains a concentration of boulders and cobbles. These cobbles can create hard points under footings, which can adversely affect the footings and structures. Depending on the final grading plans, the soils with concentrations of cobbles and boulders may be close to the bottom of building foundations. Where cobbles and boulders are within 3 feet of the bottom of building foundations, these soils and cobbles/boulders should be over-excavated to a minimum of 3 feet below the footings. Excavations should then be backfilled with engineered fill not containing cobbles/boulders up to the bottom of foundations. Potential areas of concern should be further evaluated during design-level investigation once final grading plans have been developed.

6.7 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.

Depending on the final grading plans, sandier soils may be encountered at the subgrade elevation. If these sandier soils are encountered at subgrade, we recommend subgrade compaction and proof rolling be performed within 24 hours of capillary break layer or slab-on-grade construction.

6.8 SUBGRADE STABILIZATION MEASURES

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

As discussed in the “Subsurface” section in this report, the in-situ moisture contents range from 5 or more percent below optimum to about 2 to 3 percent over the estimated laboratory optimum in the upper 10 feet of the soil profile. The contractor should anticipate moisture-conditioning the soils prior to reusing them as fill.

As discussed, grading plans have not yet been determined. Depending on the final grades, claystone may be exposed and has a high potential for deep drying and shrinkage. Significant time may be required to moisture condition exposed claystone.

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

6.8.1 Scarification and Drying

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

6.8.2 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 geosynthethic (stabilization fabric or geogrid) is recommended, and what materials are recommended for backfill.

6.8.3 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 costeffective than removal and replacement. Recommended chemical treatment depths will typically range from 12 to 18 inches depending on the magnitude of the instability.

6.9 MATERIAL FOR FILL

6.9.1 Re-Use of On-site Soils

On-site soils with an organic content less than 3 percent by weight may be reused as general fill. General fill should not have lumps, clods or cobble pieces larger than 6 inches in diameter; 85 percent of the fill should be smaller than 2½ inches in diameter. Minor amounts of oversized material (smaller than 12 inches in diameter) may be allowed provided the oversized pieces are not allowed to nest together and the compaction method will allow for loosely placed lifts not exceeding 12 inches.

6.9.2 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 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.

6.10 COMPACTION REQUIREMENTS

All fills, and subgrade areas where fill, slabs-on-grade, and pavements are planned, should be placed in loose lifts 8 inches thick or less and compacted in accordance with ASTM D1557 (latest version) requirements as shown in the table below. In general, clayey soils should be compacted with sheepsfoot equipment and sandy/gravelly soils with vibratory equipment; open-graded materials such as crushed rock should be placed in lifts no thicker than 18 inches and consolidated in place with vibratory equipment. Each lift of fill and all subgrade should be firm and unyielding under construction equipment loading in addition to meeting the compaction requirements to be approved. The contractor (with input from a Cornerstone representative) should evaluate the in-situ moisture conditions, as the use of vibratory equipment on soils with high moistures can cause unstable conditions. General recommendations for soil stabilization are provided in the "Subgrade Stabilization Measures" section of this report. Where the soil's PI is 20 or greater, the expansive soil criteria should be used.

Table 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)

Earthwork contractors should be made aware of the moisture sensitivity of clayey soils and potential compaction difficulties. Transitions areas between sandstone and claystone beds are particularly sensitive, irrespective of the time of year grading occurs. If construction is undertaken during wet weather conditions, the surficial soils may become further saturated, soft and unworkable.

6.10.1 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.

6.11 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, including interior plumbing, 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.

6.12 PERMANENT CUT AND FILL SLOPES

For planning purposes, all permanent cut and fill slopes in soil should have a maximum inclination of 2½:1 (horizontal:vertical) for slopes up to 10 feet high; slopes greater than 10 feet should be inclined at no greater than 3:1 for the overall slope inclination. Fill slopes should be overbuilt and trimmed back, exposing engineered fill when complete. Refer to the “Erosion Control” section of this report for a discussion regarding protection of slope surfaces.

6.12.1 Keyways and Benches

Fill placed on existing ground inclined at 6:1 or greater, or new cut slope buttress fills, 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 and be spaced vertically at no greater than 5 feet between benches.

During a design-level investigation once more finalized grading plans have been established we can provide more specific input regarding the location of any required keyways and benching, the dimensions of keyways and benches, 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.

We recommend that the project civil engineer or land surveyor be retained to survey in place any 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.

6.13 SITE DRAINAGE

Surface runoff should not be allowed to flow over the top of or pond at the top or toe of engineered slopes or retaining walls. Ponding should also not be allowed on or adjacent to 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.

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 existing near-surface soils at the site are generally clayey with higher fines contents, anticipated to be categorized as Hydrologic Soil Group D, and are expected to have infiltration rates of less than 0.2 inches per hour. In our opinion, these clayey soils would significantly limit the infiltration of stormwater. However, the surficial clayey soils are generally underlain by sands and gravels with less fines that would likely be more conducive to infiltration. Once grading plans have been developed, we recommend areas of proposed infiltration be further analyzed and addition exploration and testing in these areas be performed to better determine potential infiltration in those locations.

- Locally, seasonal high free ground water is mapped at a depth of greater than 30 feet beneath the surface, and therefore is expected to be at least 10 feet below the base of the infiltration measure.

- In our opinion, infiltration locations within 10 feet of the buildings 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 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. Prior to the field infiltration testing, the bioswales should be pre-soaked to pre-consolidate the material.

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 jute netting, straw matting, or erosion control blankets used in combination with hydroseeding.

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

6.16 LANDSCAPE CONSIDERATIONS

Since moderately to highly expansive materials are present and may be close to the surface following grading, we recommend 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 lawns or 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, on a preliminary basis the proposed structures may be supported on shallow foundations provided permanent cuts (overburden relief) of the Whiskey Hill bedrock areas are relatively minor (i.e. less than about 2 feet), and the recommendations in the “Earthwork” section and the sections below are followed. Once final grading plans are available, a design-level investigation should be performed to evaluate the potential influences of the underlying Whisky Hill Claystone and whether an alternate foundation system would be required.

Mitigation in that case would typically consist of the use of deep drilled piers to resist differential movement of the expansive claystone.

7.2 SEISMIC DESIGN CRITERIA

The 2016 California Building Code (CBC) provides criteria for the seismic design of buildings in Chapter 16. The "Seismic Coefficients" used to design buildings are established based on a series of tables and figures addressing different site factors, including the soil profile in the upper 100 feet below grade and mapped spectral acceleration parameters based on distance to the controlling seismic source/fault system. Based on our borings and review of local geology, the site is underlain at shallow depths by Whisky Hill Formation or Ladera Sandstone with typical SPT "N" values greater than 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 USGS computer program *U.S. Seismic Design Maps*, located at http://earthquake.usgs.gov/designmaps/us/application.php, based on the site coordinates presented below and the site classification. The table below lists the various factors used to determine the seismic coefficients and other parameters.

Table 3: CBC Site Categorization and Site Coefficients

Classification/Coefficient	Design Value
Site Class	C
Site Latitude	37.38990°
Site Longitude	-122.19205°
0.2-second Period Mapped Spectral Acceleration[1], S_S	2.787g
1-second Period Mapped Spectral Acceleration[1], S_1	1.062g
Short-Period Site Coefficient – Fa	1.0
Long-Period Site Coefficient – Fv	1.3
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S_{MS}	2.787g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	1.380g
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.858g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	0.920g
Mapped MCE Geometric Mean Peak Ground Acceleration – PGA_M	1.090g
Site Coefficient Based on PGA and Site Class - F_{PGA}	1.0

[1] For Site Class B, 5 percent damped.

7.3 SHALLOW FOUNDATIONS

7.3.1 Spread Footings

On a preliminary basis, the proposed structures may be supported on shallow foundations. Spread footings should be at least 15 inches wide, and extend at least 18 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 moderately expansive soils, and is intended to embed the footing below the zone of significant seasonal moisture fluctuation, reducing the potential for differential movement.

Spread footings should bear entirely on natural, undisturbed soil or engineered fill. As previously mentioned, if fill pads are anticipated at the site, footings should bear on a uniform amount of fill to limit differential settlement. Additionally, on cut pads, if soils with cobbles are anticipated within three feet of the bottom of footings, soils should be over-excavated and backfilled as engineered fill to provide a uniform three-foot engineered fill layer beneath footings without cobbles.

Footings constructed to the above dimensions, in accordance with the “Earthwork” recommendations of this report, and underlain by engineered fill due to the conditions discussed above are capable of supporting maximum allowable bearing pressures of 2,000 psf for dead loads, 3,000 psf for combined dead plus live loads, and 4,000 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.

Where structures will not be underlain by engineered fill and footings will bear entirely on natural, undisturbed soil, these footings are capable of supporting maximum allowable bearing pressures of 3,000 psf for dead loads, 4,500 psf for combined dead plus live loads, and 6,000 psf for all loads including wind and seismic provided they are constructed to the above dimensions and in accordance with the “Earthwork” recommendations of this report.

7.3.2 Footing Settlement

Structural loads were not provided to us at the time this report was prepared; therefore, we assumed the typical loading in the following table.

Table 4: Assumed Preliminary Structural Loading

Foundation Area	Range of Assumed Loads
Interior Isolated Column Footing	40 to 90 kips
Perimeter Strip Footing	1 to 2 kips per lineal foot

Based on the above loading and the allowable bearing pressures presented above, on a preliminary basis we estimate that the total static footing settlement will be on the order of ½-inch, with about ½-inch of post-construction differential settlement between adjacent foundation elements. As our footing loads were assumed and grading plans have not yet been established, we recommend the above settlement estimates be verified during a design-level investigation.

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.

Depending on the final site grading, sandy soils may be encountered within footing excavations. Excavation walls may not stand vertical and may need to be sloped to a minimum 1:1 inclination where footings are located within sands or Stay-Form or similar be placed within the footing excavations as they are excavated during construction of the foundation elements. The granular material encountered in the footing bottoms may also be disturbed to a depth of 6 to 8 inches following excavation and may need to be compacted prior to steel placement. Care should be taken to not disturb the compacted granular material during steel placement. We should re-observe the footing excavations in granular materials after reinforcing steel has been placed and just prior to concrete placement. Footing excavations should also be kept moist by regular sprinkling with water to prevent desiccation and potential raveling of the granular materials. As an alternative, a rat slab can be placed over the granular material after we have observed the footing excavation to protect the granular material prior to steel placement.

SECTION 8: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

8.1 INTERIOR SLABS-ON-GRADE WITH SPREAD FOOTINGS

As the Plasticity Index (PI) of the surficial soils ranges up to 22, proposed slabs-on-grade should be supported on at least 9 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 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 in accordance with the “Compaction” section of this report. The structural engineer should determine the appropriate slab reinforcement for the loading requirements and considering the expansion potential of the underlying soils. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness.

As previously discussed, highly expansive Whisky Hill Claystone was encountered in some of our borings. Depending on the final grading plans, this highly expansive claystone may influence proposed structures. Once grading plans have been established, the depth to the highly expansive claystone should be further evaluated during a design-level investigations and additional slab recommendations provided, if required.

8.2 INTERIOR SLABS MOISTURE PROTECTION CONSIDERATIONS

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

- Place a minimum 10-mil-thick vapor retarder conforming to ASTM E 1745, Class C requirements or better directly below the concrete slab; the vapor retarder should extend to the slab edges and be sealed at all seams and penetrations in accordance with manufacturer's recommendations and ASTM E 1643 requirements. A 4-inch-thick capillary break, consisting of 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 layer 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 F1869-98 and F710-98 requirements and evaluated against the floor covering manufacturer's requirements prior to installation.

8.3 EXTERIOR FLATWORK

8.3.1 Pedestrian Concrete Flatwork

Exterior concrete flatwork subject to pedestrian and/or occasional light pick up loading should be at least 4 inches thick and supported on at least 6 inches of non-expansive fill overlying subgrade prepared in accordance with the "Earthwork" recommendations of this report. The top 4 inches of non-expansive fill should also meet Class 2 aggregate base specifications. Flatwork that will be subject to heavier or frequent vehicular loading should be designed in accordance with the recommendations in the "Vehicular Pavements" section below. To help reduce the potential for uncontrolled shrinkage cracking, adequate expansion and control joints should be included. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness. Flatwork should be isolated from adjacent foundations or retaining walls except where limited sections of structural slabs are included to help span irregularities in retaining wall backfill at the transitions between at-grade and on-structure flatwork.

8.3.2 Pedestrian Pavers

Concrete unit pavers subject to pedestrian and/or occasional light pick up loading should be at least 60 mm thick and supported on at least 6 inches of non-expansive fill overlying subgrade prepared in accordance with the "Earthwork" recommendations of this report. The top 4 inches of non-expansive fill should also meet Class 2 aggregate base specifications. Pavers that will be subject to heavier or frequent vehicular loading should be designed in accordance with the recommendations in the "Vehicular Pavements" section below. A maximum 1-inch-thick layer of sand may be used as a leveling/setting bed over the aggregate base.

8.3.3 Asphalt Concrete Path

Asphalt Concrete (AC) paths should consist of at least 6 inches of Class 2 aggregate base and 2 inches of asphalt concrete overlying subgrade prepared in accordance with the "Earthwork" recommendations of this report.

SECTION 9: VEHICULAR PAVEMENTS

9.1 ASPHALT CONCRETE

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

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

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base* (inches)	Total Pavement Section Thickness (inches)
4.0	2.5	7.5	10.0
4.5	2.5	9.5	12.0
5.0	3.0	10.0	13.0
5.5	3.0	12.0	15.0
6.0	3.5	13.0	16.5
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 be 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.

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

Table 6: PCC Pavement Recommendations

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 6 inches of Class 2 aggregate base compacted as recommended in the “Earthwork” section, and laterally restraining the PCC with curbs or concrete shoulders. Adequate expansion and control joints should be included. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness. Due to the expansive soils that may be present at the surface following grading, we recommend that the construction and expansion joints be dowelled.

9.3 TRASH ENCLOSURES

Trash enclosures and the associated stress pads should be supported on at least 8 inches of Portland cement concrete (PCC) over at least 8 inches of Class 2 aggregate base, where the aggregate base should be compacted to 95 percent relative compaction. The top 6 inches of the underlying subgrade should be moisture conditioned and compacted according to the “Compaction” section of this report. The compressive strength and construction details should be consistent with the above recommendations for PCC pavements.

9.4 VEHICULAR CONCRETE UNIT PAVERS

Where vehicular concrete unit pavers are desired in standard traffic areas, we recommend 80 mm thick unit pavers and that the pavers be underlain by a 6-inch-thick concrete sub-slab designed as discussed above, including the aggregate base section. Pavers should be placed on a bituminous or mortar setting bed over the concrete sub-slab. Where the pavers will be used as an emergency vehicle access (EVA) only in landscaping areas, the pavers should be placed over at least 12 inches of Class 2 aggregate base and prepared subgrade as recommended in the “Earthwork” section. A maximum 1 inch thick sand setting bed may be used to level the pavers on the aggregate base.

SECTION 10: RETAINING WALLS

10.1 STATIC LATERAL EARTH PRESSURES

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

Table 7: Recommended Lateral Earth Pressures

Sloping Backfill Inclination (horizontal:vertical)	**Lateral Earth Pressure***	
	Unrestrained – Cantilever Wall	Restrained – Braced Wall
Level	45 pcf	45 pcf + 8H
3:1	55 pcf	55 pcf + 8H
2½:1	60 pcf	60 pcf + 8H
2:1	65 pcf	65 pcf + 8H
Additional Surcharge Loads	⅓ of vertical loads at top of wall	½ of vertical loads at top of wall

* Lateral earth pressures are based on an equivalent fluid pressure
** H is the distance in feet between the bottom of footing and top of retained soil

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

10.2 SEISMIC LATERAL EARTH PRESSURES

The 2016 CBC states that lateral pressures from earthquakes should be considered in the design of retaining walls greater than about 6 feet. At this time, grading plans have not been provided and there has not been enough detail provided to evaluate if taller walls than 6 feet will be required. This should be further evaluated during the design-level investigation.

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 with a PI less than 20 should be compacted to at least 95 percent relative compaction using light compaction equipment. If the soil’s PI is 20 or greater, expansive soil criteria should be used as discussed in the “Compaction” section of this report. Where no surface improvements are planned, backfill should be compacted to at least 90 percent for soils with a PI less than 20. Expansive soil criteria should be followed for soils with a PI of 20 or greater. If heavy compaction equipment is used, the walls should be temporarily braced.

10.5 FOUNDATIONS

On a preliminary basis, retaining walls may be supported on a continuous spread footing designed in accordance with the recommendations presented in the “Foundations” section of this report. Based on final grading plans, alternative foundations may be required depending and should be further evaluated during a design-level investigation.

SECTION 11: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of Stanford Real Estate specifically to support the design of The Stanford Wedge project in Portola Valley, California. The opinions, conclusions, and recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in Northern California at the time this report was prepared. No warranty, expressed or implied, is made or should be inferred.

Recommendations in this report are based upon the soil and ground water conditions encountered during our subsurface exploration. If variations or unsuitable conditions are encountered during construction, Cornerstone must be contacted to provide supplemental recommendations, as needed.

Stanford Real Estate may have provided Cornerstone with plans, reports and other documents prepared by others. Stanford Real Estate 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.

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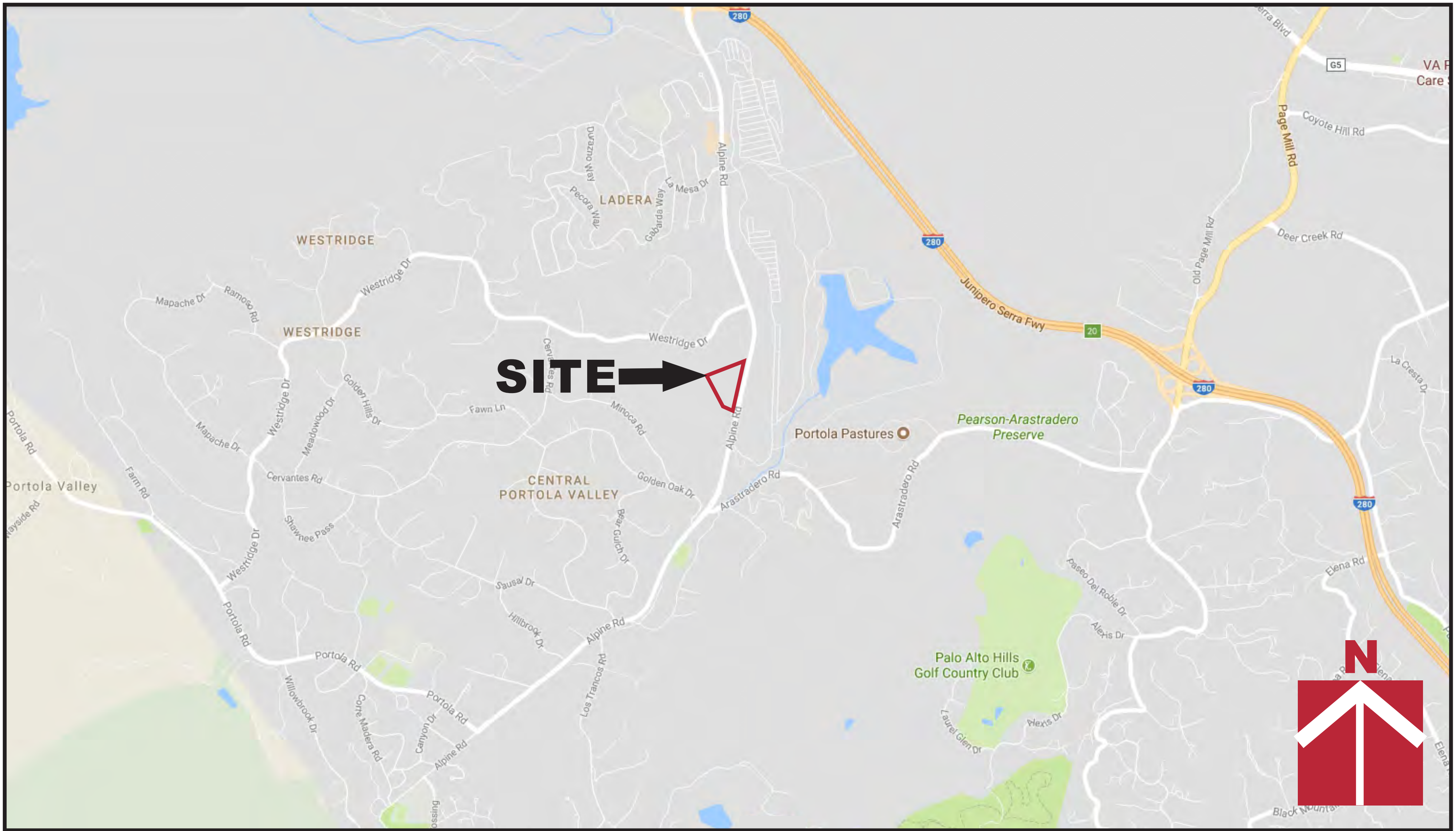
Aerial Photographs

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

Date	Flight	Frames	Scale
August 1, 1939	CIV 286	675, 110	1:20,000 black & white
October 5, 1943	DDB-1B	48, 49	1:20,000 black & white
July 19, 1963	DDB-1DD	7, 8	1:20,000 black & white
June 7, 1973	3567-5	016, 017	1:12,000 black & white



CORNERSTONE
EARTH GROUP

Vicinity Map

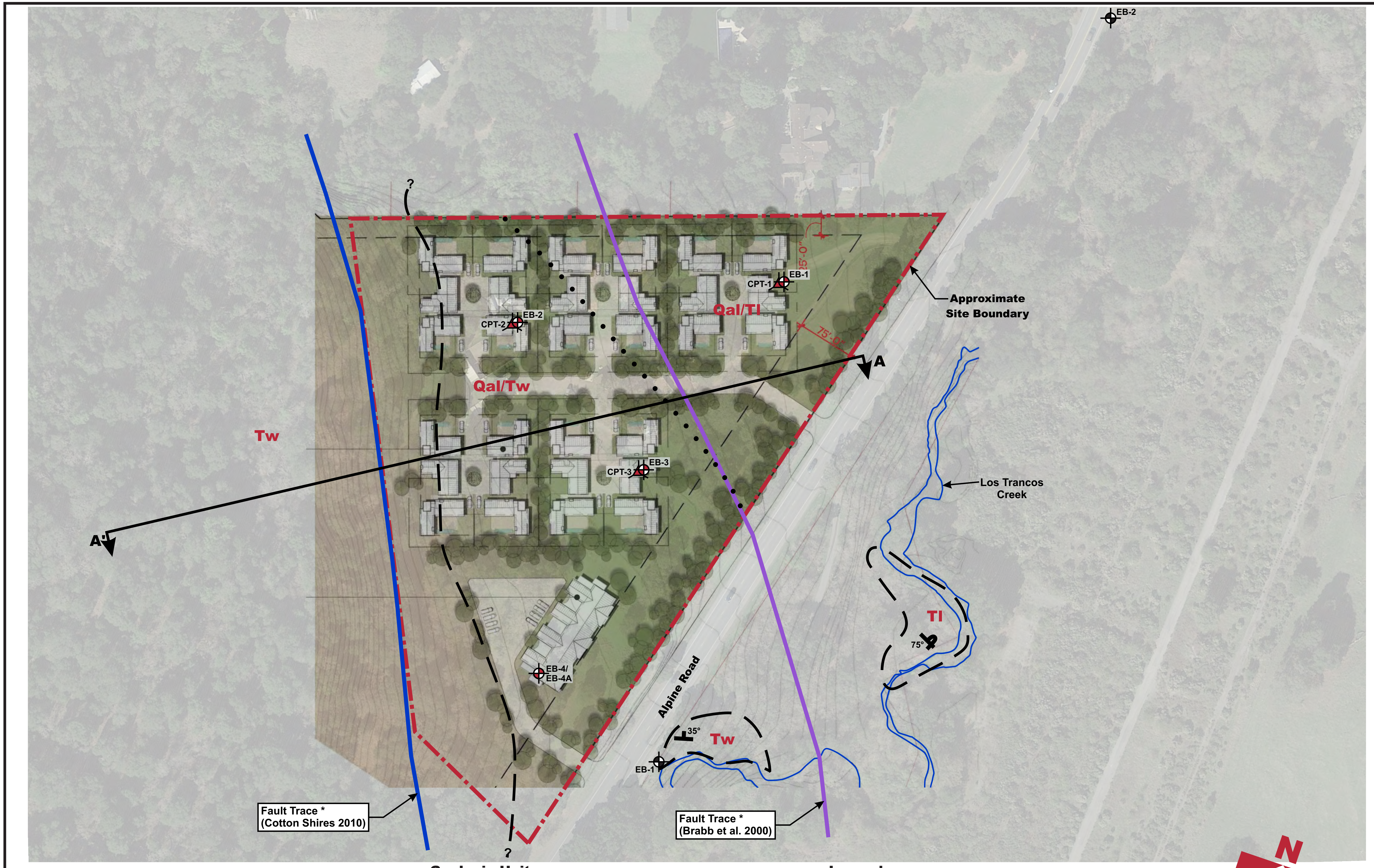
The Stanford Wedge
Portola Valley, CA

Project Number
105-117-1

Figure Number
Figure 1

Date
August 2017

Drawn By
RRN



* Mapped fault traces dropped from current town geologic map due to lack of evidence. Brabb et. al mapping of fault is considered to be a depositional contact.

Geologic Units

Qal	Quaternary alluvium
Tw	Whiskey Hill Formation
Tl	Ladera Sandstone

Legend

A A'	Approximate location of cross section (See Figures 7A and 7B)
35°	Bedding altitude (field measured by Craig Harwood, C.E.G.)
75°	Overturned bedding (Cotton, Shires and Associates, Inc., 2010)
···—?—	Geologic contact; dashed where approximate, dotted where concealed
⊕	Approximate location of exploratory boring (EB)
▲	Approximate location of cone penetration test (CPT)
⊕	Approximate location of exploratory boring (EB) (Cornerstone, 2007)

N

0 125 250
APPROXIMATE SCALE (FEET)

Base by Google Earth, dated 11/2/2017
Overlay by HKS Hospitality Group, Site Plan: Option A, dated 8/10/2016

Project Number
105-117-1

Figure Number
Figure 2

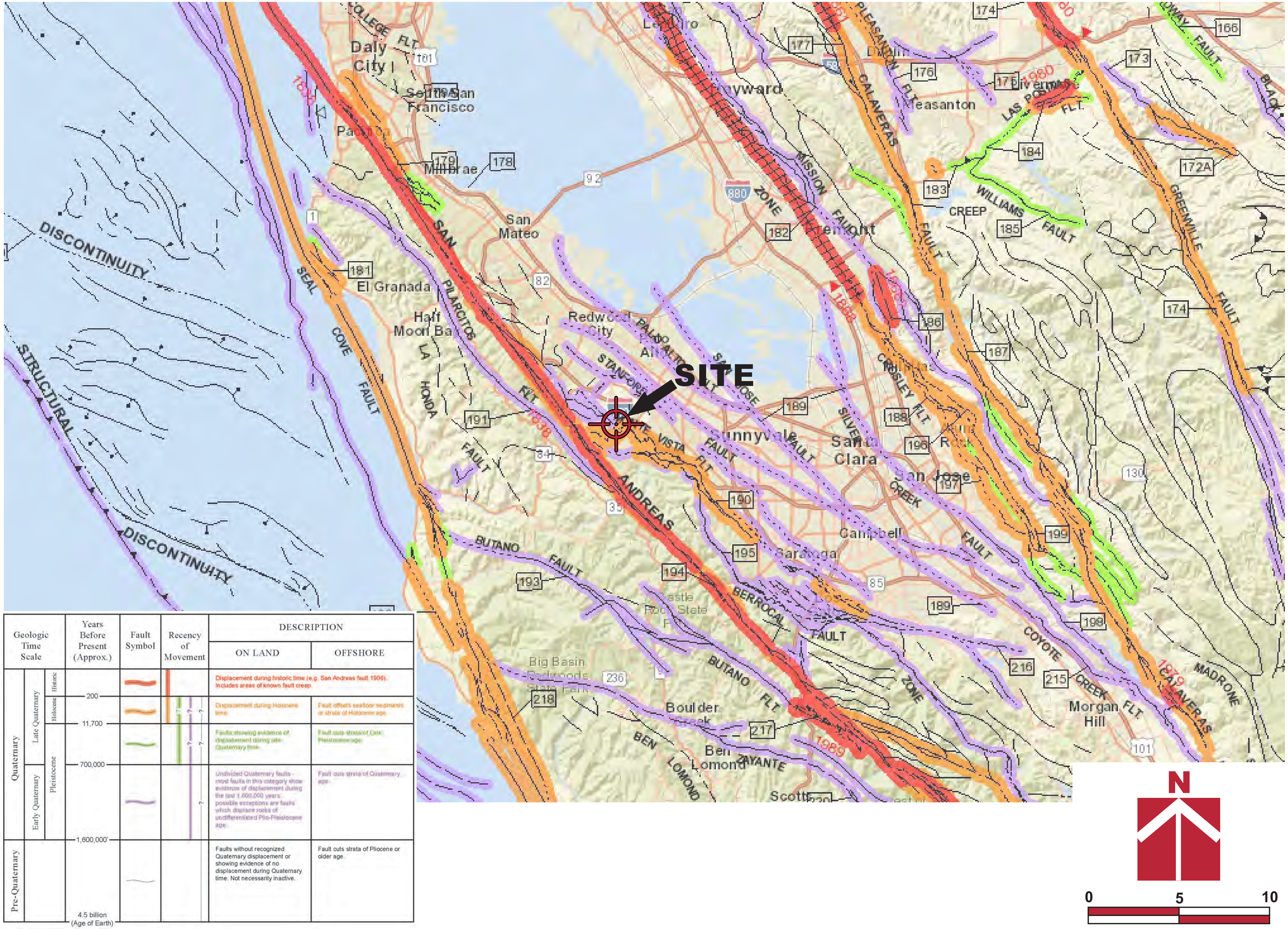
Date
August 2017

Drawn By
RRN

Site Plan and Geologic Map

The Stanford Wedge
Portola Valley, CA

CORNERSTONE
EARTH GROUP



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.	
		Holocene	200	🟧	? ? ?	Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	11,700	🟩	? ?	Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		700,000	🟪	?	Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary			1,600,000 4.5 billion (Age of Earth)	~		Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

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

Project Number 105-117-1

Figure Number Figure 3

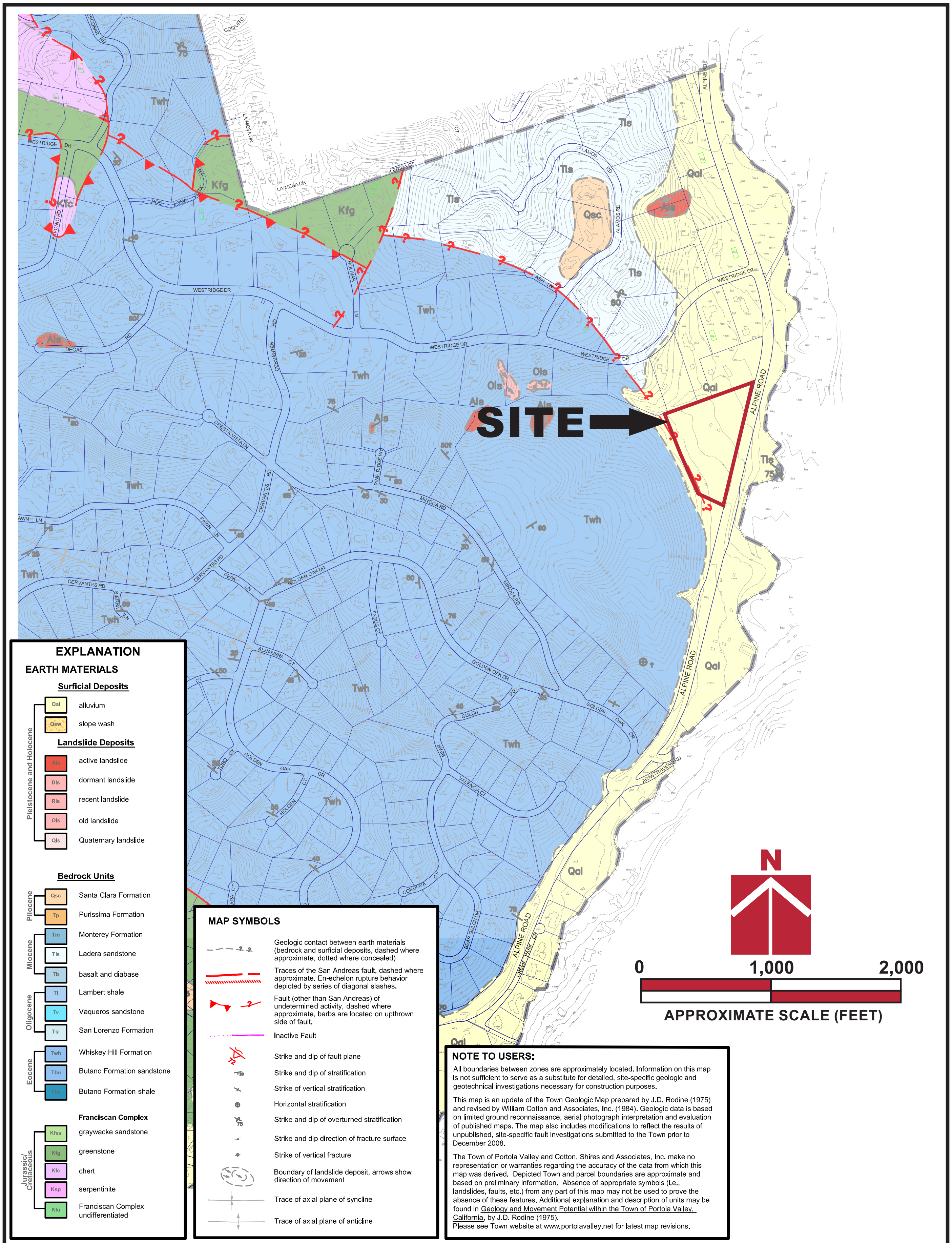
Drawn By RRN

Date August 2017

Regional Fault Map

The Stanford Wedge
Portola Valley, CA

CORNERSTONE EARTH GROUP



EXPLANATION

EARTH MATERIALS

Surficial Deposits

Pleistocene and Holocene	Qal	alluvium
	Qsw	slope wash

Landslide Deposits

Pleistocene and Holocene	Als	active landslide
	Dls	dormant landslide
	Rls	recent landslide
	Ols	old landslide
	Qls	Quaternary landslide

Bedrock Units

Pliocene	Qsc	Santa Clara Formation
	Tp	Purissima Formation
Miocene	Tm	Monterey Formation
	Tls	Ladera sandstone
	Tb	basalt and diabase
Oligocene	Tl	Lambert shale
	Tv	Vaqueros sandstone
	Tsl	San Lorenzo Formation
Eocene	Twh	Whiskey Hill Formation
	Tbu	Butano Formation sandstone
	Tbs	Butano Formation shale

Franciscan Complex

Jurassic/ Cretaceous	Kfss	graywacke sandstone
	Kfg	greenstone
	Kfc	chert
	Ksp	serpentinite
	Kfu	Franciscan Complex undifferentiated

MAP SYMBOLS

- Geologic contact between earth materials (bedrock and surficial deposits, dashed where approximate, dotted where concealed)
- Traces of the San Andreas fault, dashed where approximate. En-echelon rupture behavior depicted by series of diagonal slashes.
- Fault (other than San Andreas) of undetermined activity, dashed where approximate, barbs are located on upthrown side of fault.
- Inactive Fault
- Strike and dip of fault plane
- Strike and dip of stratification
- Strike of vertical stratification
- Horizontal stratification
- Strike and dip of overturned stratification
- Strike and dip direction of fracture surface
- Strike of vertical fracture
- Boundary of landslide deposit, arrows show direction of movement
- Trace of axial plane of syncline
- Trace of axial plane of anticline

NOTE TO USERS:

All boundaries between zones are approximately located. Information on this map is not sufficient to serve as a substitute for detailed, site-specific geologic and geotechnical investigations necessary for construction purposes.

This map is an update of the Town Geologic Map prepared by J.D. Rodine (1975) and revised by William Cotton and Associates, Inc. (1984). Geologic data is based on limited ground reconnaissance, aerial photograph interpretation and evaluation of published maps. The map also includes modifications to reflect the results of unpublished, site-specific fault investigations submitted to the Town prior to December 2008.

The Town of Portola Valley and Cotton, Shires and Associates, Inc. make no representation or warranties regarding the accuracy of the data from which this map was derived. Depicted Town and parcel boundaries are approximate and based on preliminary information. Absence of appropriate symbols (i.e., landslides, faults, etc.) from any part of this map may not be used to prove the absence of these features. Additional explanation and description of units may be found in Geology and Movement Potential within the Town of Portola Valley, California, by J.D. Rodine (1975).
Please see Town website at www.portolavalley.net for latest map revisions.

Base provided by Cotton, Shires and Associates, Inc., Geologic Map - Sheet 1 of 1, dated September 2010

CORNERSTONE
EARTH GROUP

Vicinity Geologic Map

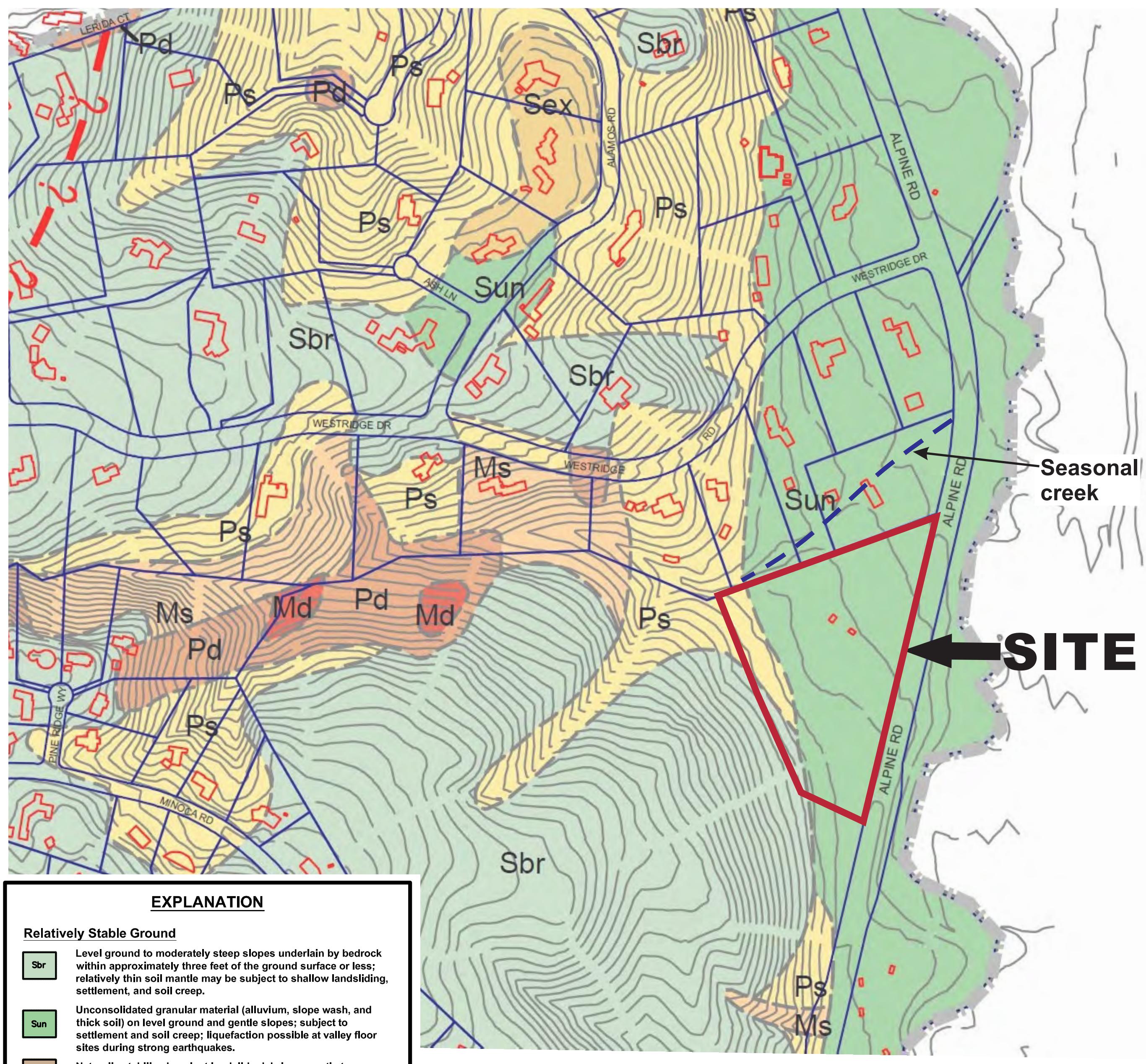
The Stanford Wedge
Portola Valley, CA

Project Number
105-117-1

Figure Number
Figure 4

Date
August 2017

Drawn By
RRN



EXPLANATION

Relatively Stable Ground

Sbr	Level ground to moderately steep slopes underlain by bedrock within approximately three feet of the ground surface or less; relatively thin soil mantle may be subject to shallow landsliding, settlement, and soil creep.
Sun	Unconsolidated granular material (alluvium, slope wash, and thick soil) on level ground and gentle slopes; subject to settlement and soil creep; liquefaction possible at valley floor sites during strong earthquakes.
Sls	Naturally stabilized ancient landslide debris on gentle to moderate slopes; subject to settlement and soil creep.
Sex	Generally highly expansive, clay-rich soils and bedrock. Subject to seasonal shrink-swell, rapid soil creep, and settlement. May include areas of non-expansive material. Expansive soils may also occur within other map units.

Areas with Significant Potential for Downslope Movement

Pmw	Steep to very steep slopes generally underlain by weathered and fractured bedrock subject to mass-wasting by rockfall, slumping, and raveling.
Ps	Unstable, unconsolidated material, commonly less than 10 feet in thickness, on gentle to moderately steep slopes subject to shallow landsliding, slumping, settlement, and soil creep.
Pd	Unstable, unconsolidated material, commonly more than 10 feet in thickness, on moderate to steep slopes; subject to deep landsliding.
Pdf	Debris flows, (shallow, rapidly moving landslides) including recognized source areas, flow paths and depositional runout areas.

Areas with Potential for Primary Ground Rupture from Active Faults

Pf	Zone of potential primary surface fault rupture.

Unstable Ground Characterized by Seasonally Active Downslope Movement

Ms	Moving shallow landslides, commonly less than 10 feet in thickness.
Md	Moving deep landslides, commonly more than 10 feet in thickness.

Base provided by Cotton, Shires and Associates, Inc., dated 2017.

NOTE TO USERS:

This is an interpretive map derived both from the Geologic Map of the Town of Portola Valley and from additional field observations and geologic experience in the Portola Valley region. All boundaries between zones are located approximately. Information on this map is NOT sufficient to serve as a substitute for detailed, site-specific geologic and geotechnical investigations necessary for construction. It illustrates the relative stability or movement potential, in the Portola Valley area, of ground in its natural undisturbed state. Works of man may seriously alter the natural stability of the ground. Potential impacts of graded cut and fill slopes are not addressed movement potential interpretations.

This map is an update of the initial Movement Potential of Undisturbed Ground Map prepared by J.D. Rodine (1975) and revised by William Cotton and Associates, Inc. (1984). For additional information about this map and the Geologic Map of the Town of Portola Valley, see: Geology and Movement Potential within the Town of Portola Valley, California, February 1975, by J.D. Rodine.

Please see Town website at www.portolavalley.net for latest map revisions.

MAP SYMBOLS

Traces of the San Andreas fault, dashed where approximate. En-echelon rupture behavior depicted by series of diagonal slashes.

Fault (other than San Andreas) of undetermined activity, dashed where approximate, barbs are located on upthrown side of fault

Inactive Fault

N

0 500 1000

APPROXIMATE SCALE (FEET)

CORNERSTONE EARTH GROUP

Ground Movement Potential Map

The Stanford Wedge
Portola Valley, CA

Project Number
105-117-1

Figure Number
Figure 5

Date
August 2017

Drawn By
RRN

Explanation

Liquefaction

🟩 Areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslides

🟦 Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

N

0 1,000 2,000

APPROXIMATE SCALE (FEET)

Base provided by CGS, Earthquake Zones of Required Investigation, Palo Alto Quadrangle, dated 2006

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CGS Seismic Hazard Map

The Stanford Wedge
Portola Valley, CA

Project Number
105-117-1

Figure Number
Figure 6

Date
August 2017

Drawn By
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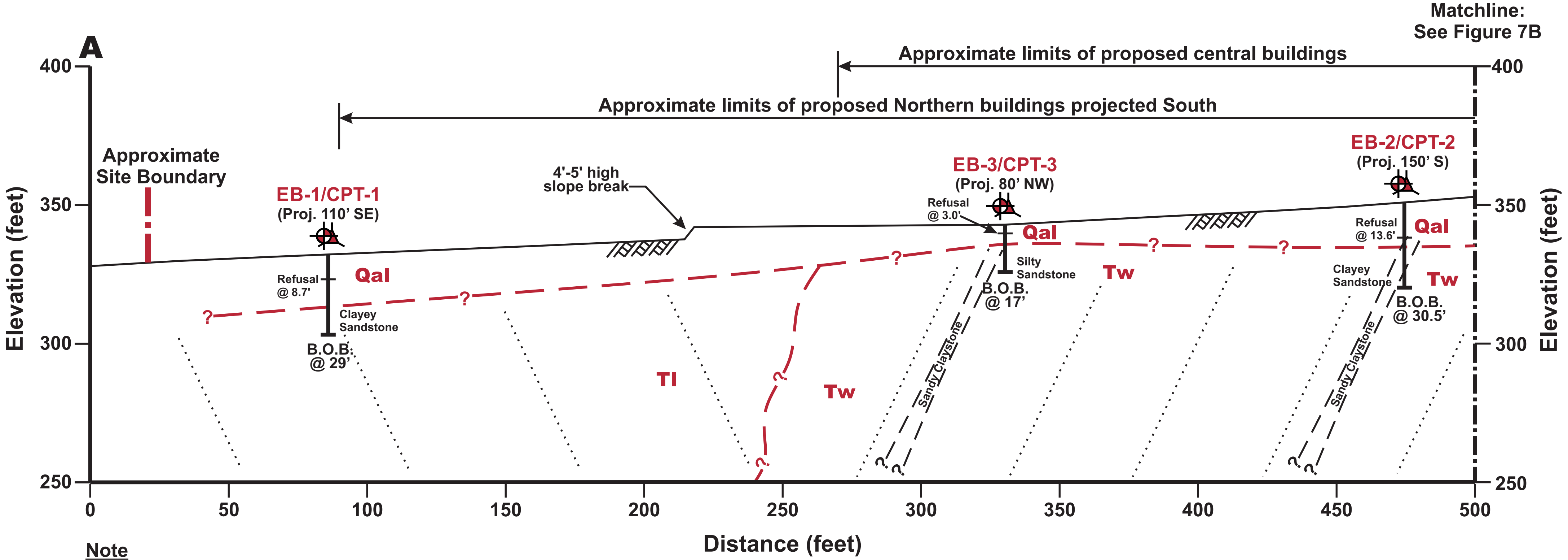


Note
Bedrock dip angle by
Craig Harwood, C.E.G.

Section A-A'

(View Looking Southeast)
1"=50' H:V

Explanation

Geologic Units

Qal	Alluvium
Tl	Ladera sandstone
Tw	Whiskey Hill Formation

Symbols

⌖	Approximate location of exploratory boring (EB)
▲	Approximate location of cone penetration test (CPT)

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

Project Number
105-117-1

Figure Number
Figure 7A

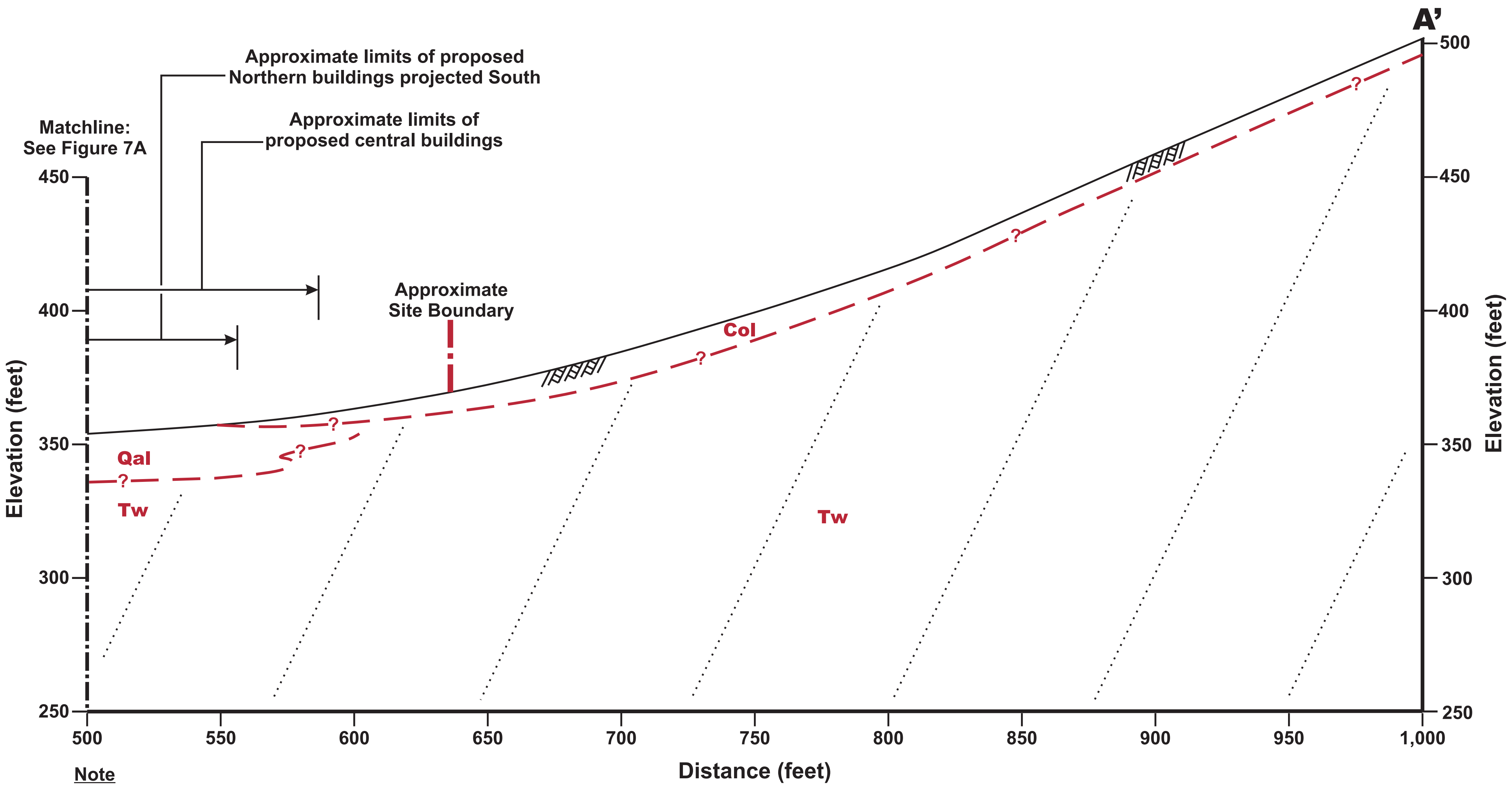
Drawn By
RRN

Date
August 2017

Geologic Cross Section A-A'

The Stanford Wedge
Portola Valley, CA

CORNERSTONE
EARTH GROUP



Note
Bedrock dip angle by
Craig Harwood, C.E.G.

Section A-A'

(View Looking Southeast)
1"=50' H:V

Explanation

Geologic Units

Col	Colluvium
Qal	Alluvium
Tw	Whiskey Hill Formation

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

Project Number
105-117-1

Figure Number
Figure 7B

Drawn By
RRN

Date
August 2017

Geologic Cross Section A-A'

The Stanford Wedge
Portola Valley, CA

CORNERSTONE
EARTH GROUP

APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, hollow-stem auger drilling equipment and 20-ton truck-mounted Cone Penetration Test equipment. Five 8-inch-diameter exploratory borings were drilled on July 20 and 21 to depths of 8 to 30½ feet. Three CPT soundings were also performed in accordance with ASTM D 5778-95 (revised, 2002) on July 7, 2017, to depths ranging from about 3 to 13½ feet, where refusal was encountered. The approximate locations of exploratory borings and CPTs are shown on the Site Plan and Geologic Map, Figure 2. The soils and bedrock 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 and CPT locations were approximated using existing site boundaries, a hand held GPS unit, and other site features as references. Boring elevations were based on interpolation of the topographic information provided. The locations and elevations of the borings and CPTs should be considered accurate only to the degree implied by the method used.

Representative soil and bedrock 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- and 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.

The CPT involved advancing an instrumented cone-tipped probe into the ground while simultaneously recording the resistance at the cone tip (q_c) and along the friction sleeve (f_s) at approximately 5-centimeter intervals. Based on the tip resistance and tip to sleeve ratio (R_f), the CPT classified the soil behavior type and estimated engineering properties of the soil, such as equivalent Standard Penetration Test (SPT) blow count, internal friction angle within sand layers, and undrained shear strength in silts and clays. A pressure transducer behind the tip of the CPT cone measured pore water pressure (u_2). Graphical logs of the CPT data is included as part of this appendix.

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 and CPT 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 and CPT 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.

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-98)

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

OTHER MATERIAL SYMBOLS

- ▨ Poorly-Graded Sand with Clay
- ▨ Clayey Sand
- ▥ Sandy Silt
- ▦ Artificial/Undocumented Fill
- ⁘ Poorly-Graded Gravelly Sand
- ⁘ Topsoil
- ● Well-Graded Gravel with Clay
- ● Well-Graded Gravel with Silt
- ⁘ Sand
- ▥ Silt
- ⁘ Well Graded Gravelly Sand
- ▥ Gravelly Silt
- ■ Asphalt
- ● Boulders and Cobble

SAMPLER TYPES

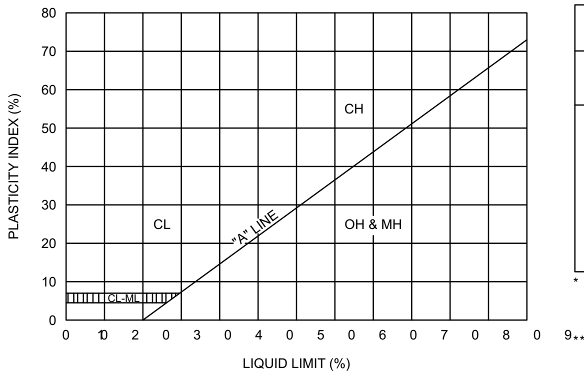
- ☒ SPT
- ⧗ Modified California (2.5" I.D.)
- ▮ Rock Core
- ■ Shelby Tube
- ○ No Recovery
- ✋ Grab Sample

ADDITIONAL TESTS

CA - CHEMICAL ANALYSIS (CORROSIVITY)
CD - CONSOLIDATED DRAINED TRIAXIAL
CN - CONSOLIDATION
CU - CONSOLIDATED UNDRAINED TRIAXIAL
DS - DIRECT SHEAR
PP - POCKET PENETROMETER (TSF)
(3.0) - (WITH SHEAR STRENGTH IN KSF)
RV - R-VALUE
SA - SIEVE ANALYSIS: % PASSING #200 SIEVE
▼ - WATER LEVEL

PI - PLASTICITY INDEX
SW SWELL TEST
TC - CYCLIC TRIAXIAL
TV - TORVANE SHEAR
UC - UNCONFINED COMPRESSION
(1.5) - (WITH SHEAR STRENGTH IN KSF)
UU - UNCONSOLIDATED UNDRAINED TRIAXIAL

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

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

STRENGTH

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

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

D. **Deep –** Moderate to complete mineral decomposition: extensive disintegration: deep and thorough discoloration: many fractures, all extensively coated or filled with oxides, carbonates and/or clay or silt.

M. **Moderate –** Slight change or partial decomposition of minerals: little disintegration: cementation little to unaffected. Moderate to occasionally intense discoloration. Moderately coated fractures.

L. **Little –** No megascopic decomposition of minerals: little or no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains or fracture surfaces.

F. **Fresh –** Unaffected by weathering agents. No disintegration or discoloration. Fractures usually less numerous than joints.

FRACTURING

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

BEDDING OF SEDIMENTARY ROCKS

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

CORNERSTONE
EARTH GROUP

Physical Properties of Rock Descriptions

Figure Number
A-2

CORNERSTONE
EARTH GROUP

BORING NUMBER EB-1

PROJECT NAME The Stanford Wedge
PROJECT NUMBER 105-117-1
PROJECT LOCATION Alpine Road, Portola Valley, CA
DATE STARTED 7/20/17 **DATE COMPLETED** 7/20/17
GROUND ELEVATION 334 FT +/- **BORING DEPTH** 29 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.3905° **LONGITUDE** -122.1915°
DRILLING METHOD Mobile B-61, 8 inch Hollow-Stem Auger
GROUND WATER 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

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 (1.0 2.0 3.0 4.0)
334.0	0	**Silty Sand with Gravel (SM) [Quaternary Alluvium-Qal]**							
333.0		**Sandy Lean Clay (CL) [Qal]** stiff to hard, moist, dark brown to brown, fine to medium sand, moderate plasticity Liquid Limit = 35, Plastic Limit = 16	22	MC-2	100	11	19	63	>4.5 ○
			23	MC-4	97	10			
329.5	5	**Lean Clay with Sand (CL) [Qal]** hard, moist, brown with light brown mottles, fine sand, moderate plasticity	17	MC-6	104	19			>4.5 ○
			46	MC-8	111	10		31	
326.0	10	**Clayey Sand (SC) [Qal]** very dense, moist, brown with light brown and olive mottles, fine to coarse sand, some fine to coarse subangular to subrounded gravel							
			50 6"	SPT-9		12			
322.3		**Well-Graded Sand with Silt and Gravel (SW-SM) [Qal]** very dense, moist, gray, fine subangular to subrounded gravel, some cobbles	50 4"	SPT					
320.0	15	**Well-Graded Gravel with Sand (GW) [Qal]** very dense, dry, gray, coarse gravel, fine to coarse sand, some cobbles to boulders							
316.0	20	**Clayey Sandstone [Ladera Sandstone-Tl]** moderately hard, weak, deep weathering, gray and brown mottled, fine sand	50 6"	SPT-10		14			
			50 4"	SPT-11		10			
311.0									

Continued Next Page

BORING NUMBER EB-1

PROJECT NAME The Stanford Wedge

PROJECT NUMBER 105-117-1

PROJECT LOCATION Alpine Road, Portola Valley, CA

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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
311.0			**Clayey Sandstone [Ladera Sandstone-Tl]** moderately hard, weak, deep weathering, gray and brown mottled, fine sand	50 6"	MC-12	113	13			
	25									
				50 6"	SPT-13		14			
305.0			Bottom of Boring at 29.0 feet.							
	30									
	35									
	40									
	45									

CORNERSTONE EARTH GROUP

BORING NUMBER EB-2

PAGE 1 OF 2

PROJECT NAME The Stanford Wedge
PROJECT NUMBER 105-117-1
PROJECT LOCATION Alpine Road, Portola Valley, CA
DATE STARTED 7/21/17 **DATE COMPLETED** 7/21/17
GROUND ELEVATION 351 FT +/- **BORING DEPTH** 30.5 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.3900° **LONGITUDE** -122.1926°
DRILLING METHOD Mobile B-61, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY CSH
▽ **AT TIME OF DRILLING** Not Encountered
NOTES *-designates 2" diameter Modified Calif. Sampler
▼ **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)	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
351.0	0	**Sandy Silty Clay (CL-ML) [Quaternary Alluvium-Qal]** hard, moist, reddish brown, fine sand, low plasticity Liquid Limit = 17, Plastic Limit = 13							
349.5		**Sandy Lean Clay (CL) [Qal]** hard, moist, reddish brown with gray mottles, fine sand, trace subrounded gravel, moderate plasticity Liquid Limit = 36, Plastic Limit = 14	18	1 MC 2	85 113	8 9	4	54	>4.5 ○
			12	SPT-3		15	22	55	>4.5 ○
	5		18	SPT-4		17			>4.5 ○
			28	MC-6	107	16			>4.5 ○
341.0	10	**Clayey Sand with Gravel (SC) [Qal]** medium dense, moist, brown with light brown and olive mottles, fine to coarse sand, fine to coarse subangular to subrounded gravel Liquid Limit = 31, Plastic Limit = 16	10	SPT-7		20	15	49	
338.5		**Well-Graded Sand with Silt and Gravel (SW-SM) [Qal]** dense, moist, brown with light brown mottles, fine to coarse sand, fine to coarse subangular gravel	72	MC-8	133	12		12	
	15								
334.8		**Sandy Claystone [Whiskey Hill Formation-Tw]** moderately hard, weak, deep weathering, gray and brown mottled	50 6"	SPT-10		23			
			34	SPT-11		23			
	20								
328.0									

Continued Next Page

BORING NUMBER EB-2

PAGE 2 OF 2

CORNERSTONE EARTH GROUP

PROJECT NAME The Stanford Wedge

PROJECT NUMBER 105-117-1

PROJECT LOCATION Alpine Road, Portola Valley, CA

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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
328.0 328.0		▨	**Clayey Sandstone [Whiskey Hill Formation-Tw]** moderately hard, weak, deep weathering, dark gray brown, fine sand	50 6"	☒ SPT					
	25									
				50 5"	☒ SPT-13*		12			
320.8	30			50 3"	☒ SPT-14*		12			
			Bottom of Boring at 30.5 feet.							
	35									
	40									
	45									

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 9/13/17 15:16 - P:\DRAFTING\GINT FILES\105-117-1 STANFORD WEDGE.GPJ

BORING NUMBER EB-3

PAGE 1 OF 1

CORNERSTONE
EARTH GROUP

PROJECT NAME The Stanford Wedge
PROJECT NUMBER 105-117-1
PROJECT LOCATION Alpine Road, Portola Valley, CA
DATE STARTED 7/20/17 **DATE COMPLETED** 7/21/17
GROUND ELEVATION 345 FT +/- **BORING DEPTH** 17 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.3897° **LONGITUDE** -122.1918°
DRILLING METHOD Mobile B-61, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY CSH
▽ **AT TIME OF DRILLING** Not Encountered
NOTES *-designates 2" diameter Modified Calif. Sampler
▼ **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
345.0	0		**Silty Sand with Gravel (SM) [Quaternary Alluvium-Qal]** very dense, dry, brown, fine to coarse sand, fine to coarse gravel, some cobbles	50/5"	MC-1	102	4			
343.0			**Clayey Sand with Gravel (SC) [Qal]** very dense, moist, brown with light brown and olive mottles, fine to coarse sand, fine to coarse subangular to subrounded gravel	50/4"	MC-3	99	11		20	
340.0	5		**Well-Graded Sand with Silt and Gravel (SW-SM) [Qal]** very dense, moist, gray and brown, fine to medium sand, fine to coarse subangular to subrounded gravel	50/6"	MC-5	103	8		11	
338.0			**Sandy Claystone [Whiskey Hill Formation-Tw]** low hardness, weak, deep weathering, reddish brown with gray mottles, high plasticity Liquid Limit = 69, Plastic Limit = 32	25	SPT-6		29	37		
				20	SPT-7		27			
	10									
334.0			**Silty Sandstone [Whiskey Hill Formation-Tw]** moderately hard, weak, deep weathering, brown with yellowish brown and reddish brown mottles brown, fine sand	50/3"	SPT-8*					
				50/5"	SPT-9*		11			
	15									
				63	SPT-10*		13			
328.0			Bottom of Boring at 17.0 feet.							
	20									

BORING NUMBER EB-4

PAGE 1 OF 1

CORNERSTONE
EARTH GROUP

PROJECT NAME The Stanford Wedge
PROJECT NUMBER 105-117-1
PROJECT LOCATION Alpine Road, Portola Valley, CA
DATE STARTED 7/21/17 **DATE COMPLETED** 7/21/17
GROUND ELEVATION 353 FT +/- **BORING DEPTH** 8 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.3890° **LONGITUDE** -122.1919°
DRILLING METHOD Mobile B-61, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY CSH
▽ **AT TIME OF DRILLING** Not Encountered
NOTES Abandoned hole, could not advance augers beyond 8'.
▼ **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
353.0	0	▨	**Clayey Sand (SC) [Quaternary Alluvium-Qal]** loose to medium dense, moist, yellow brown to brown, fine to medium sand, some fine subangular to subrounded gravel Liquid Limit = 21, Plastic Limit = 13	18	MC-2	103	8	8	48
				19	SPT-3		13		
348.5	5	▨	**Clayey Sand with Gravel (SC) [Qal]** dense, moist, brown with light brown and olive mottles, fine to coarse sand, fine to coarse subangular to subrounded gravel, some cobbles	44	MC-5	105	12		
346.0			**Well Graded Gravel with Sand (GW) [Qal]** very dense, dry, gray, fine to coarse gravel, fine to coarse sand, some cobbles to boulders						
345.0			Bottom of Boring at 8.0 feet.						
	10								
	15								
	20								

CORNERSTONE
EARTH GROUP

BORING NUMBER EB-4A

PROJECT NAME The Stanford Wedge
PROJECT NUMBER 105-117-1
PROJECT LOCATION Alpine Road, Portola Valley, CA
DATE STARTED 7/21/17 **DATE COMPLETED** 7/21/17
GROUND ELEVATION 353 FT +/- **BORING DEPTH** 9.5 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.3890° **LONGITUDE** -122.1919°
DRILLING METHOD Mobile B-61, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY CSH
▽ **AT TIME OF DRILLING** Not Encountered
NOTES Continuation of EB-4, could not advance augers beyond 9.5'.
▼ **AT END OF DRILLING** Not Encountered

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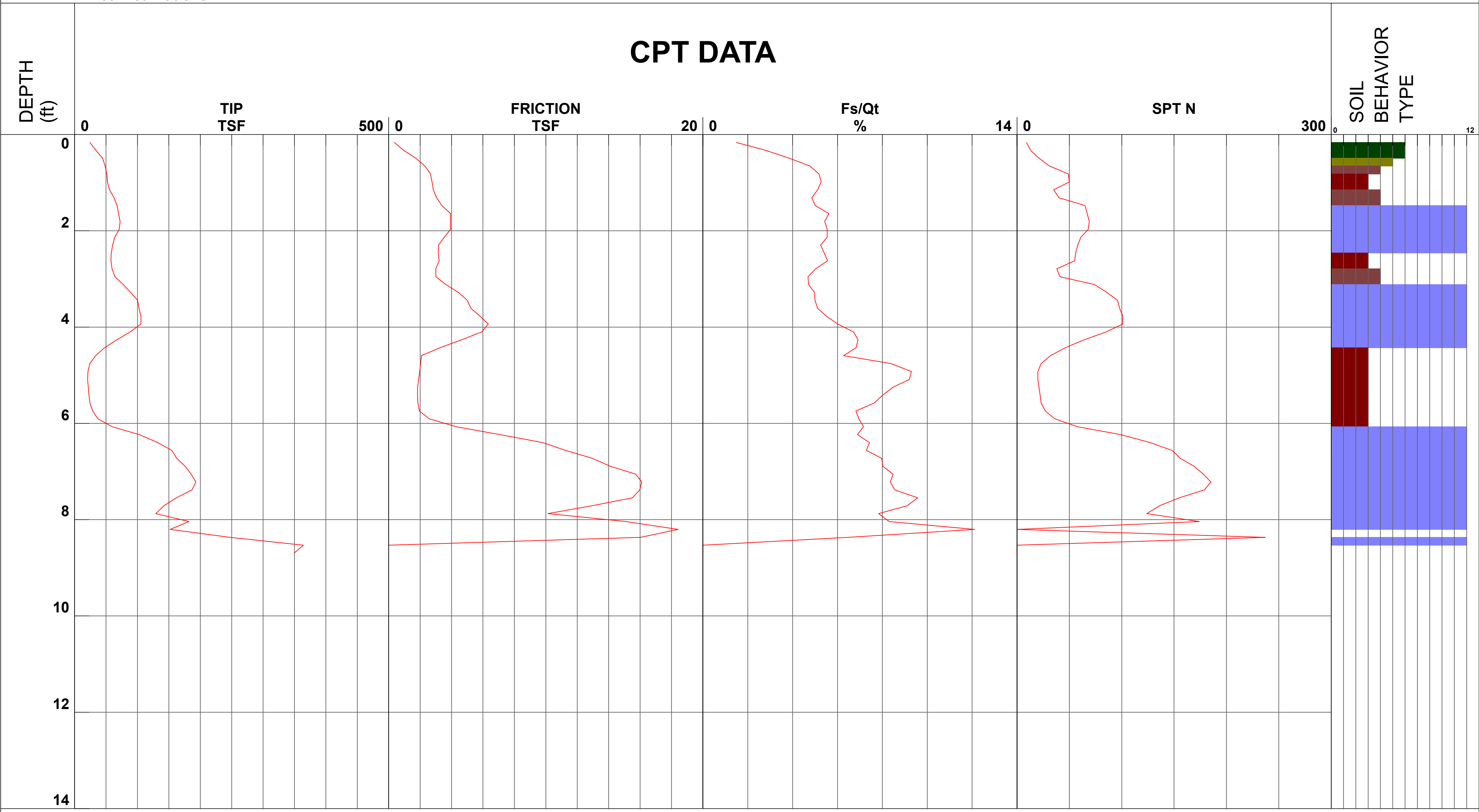
ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf ○ HAND PENETROMETER △ TORVANE ● UNCONFINED COMPRESSION ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL 1.0 2.0 3.0 4.0
353.0	0	▨	Drilled to 7' before sampling. Cuttings observed are same as EB-4.							
	5	▨								
346.0		●	**Well-Graded Gravel with Sand (GW)** **[Quaternary Alluvium-Qal]** very dense, dry, gray, fine to coarse gravel, fine to coarse sand, some cobbles to boulders	50 4"	○ NR-6*					
				50 1"	⊠ SPT					
				50 0"	NR					
343.5			Bottom of Boring at 9.5 feet.	50 1"	⊠ SPT					
	10									
	15									
	20									

Middle Earth
GEO TESTING INC.

Cornerstone Earth Group

Project The Stanford Wedge
Job Number 105-117-1
Hole Number CPT-01
EST GW Depth During Test >8.69 ft
Operator RB KK
Cone Number DDG1333
Date and Time 7/7/2017 1:20:22 PM
Filename SDF(080).cpt
GPS
Maximum Depth 8.69 ft

Net Area Ratio .8



- ■ 1 - sensitive fine grained
- ■ 2 - organic material
- ■ 3 - clay
- ■ 4 - silty clay to clay
- ■ 5 - clayey silt to silty clay
- ■ 6 - sandy silt to clayey silt
- ■ 7 - silty sand to sandy silt
- ■ 8 - sand to silty sand
- ■ 9 - sand
- ■ 10 - gravelly sand to sand
- ■ 11 - very stiff fine grained (*)
- ■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

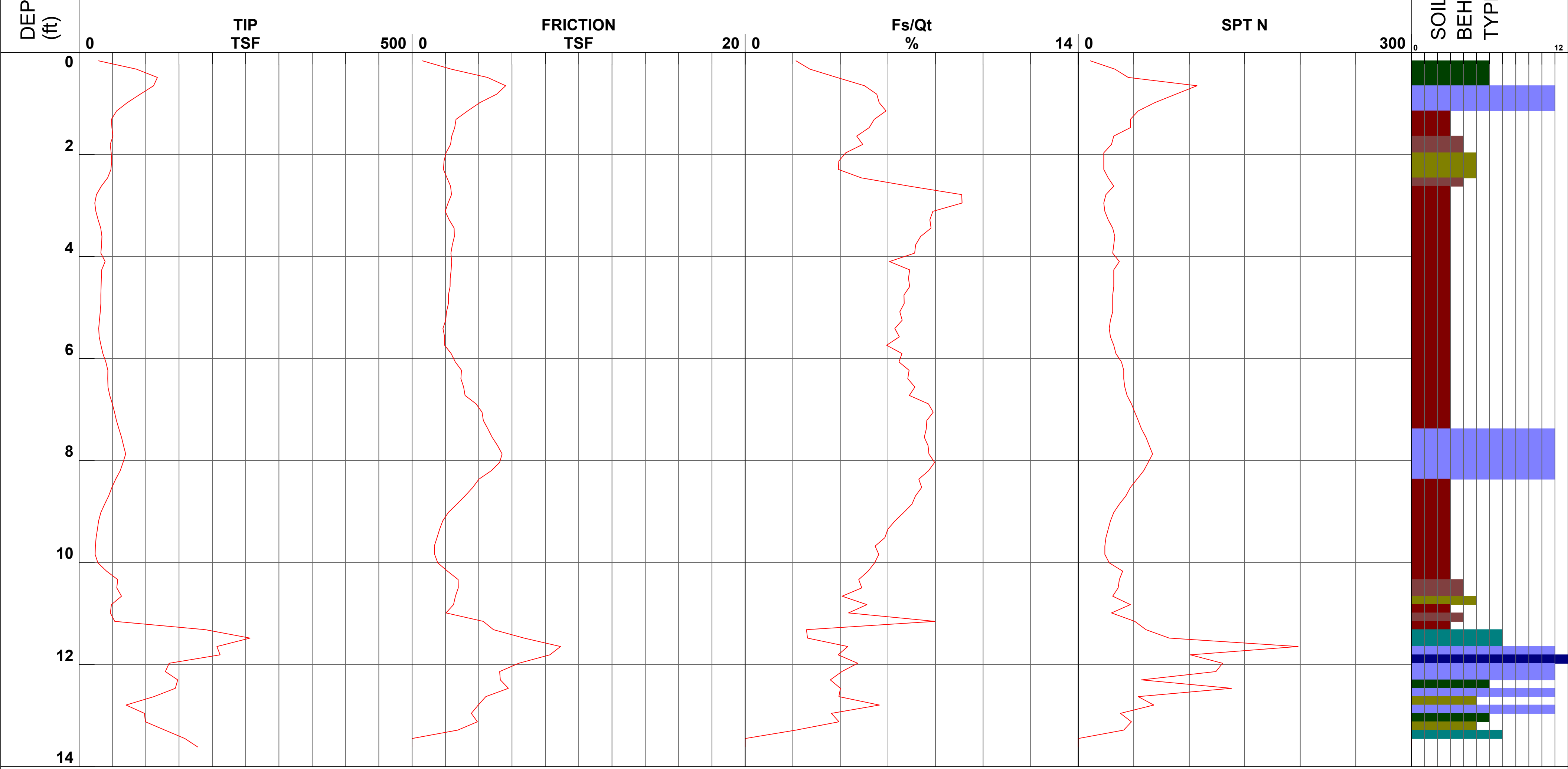
S*Soil behavior type and SPT based on data from UBC-1983

Cornerstone Earth Group

Project	The Stanford Wedge	**Operator**	RB KK	**Filename**	SDF(081).cpt
Job Number	105-117-1	**Cone Number**	DDG1333	**GPS**	
Hole Number	CPT-02	**Date and Time**	7/7/2017 1:54:37 PM	**Maximum Depth**	13.62 ft
EST GW Depth During Test		>13.62 ft			

Net Area Ratio .8

CPT DATA



🟫 1 - sensitive fine grained
🟫 2 - organic material
🟥 3 - clay
🟫 4 - silty clay to clay
🟫 5 - clayey silt to silty clay
🟩 6 - sandy silt to clayey silt
🟦 7 - silty sand to sandy silt
🟪 8 - sand to silty sand
🟧 9 - sand
🟥 10 - gravelly sand to sand
🟦 11 - very stiff fine grained (*)
🟦 12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

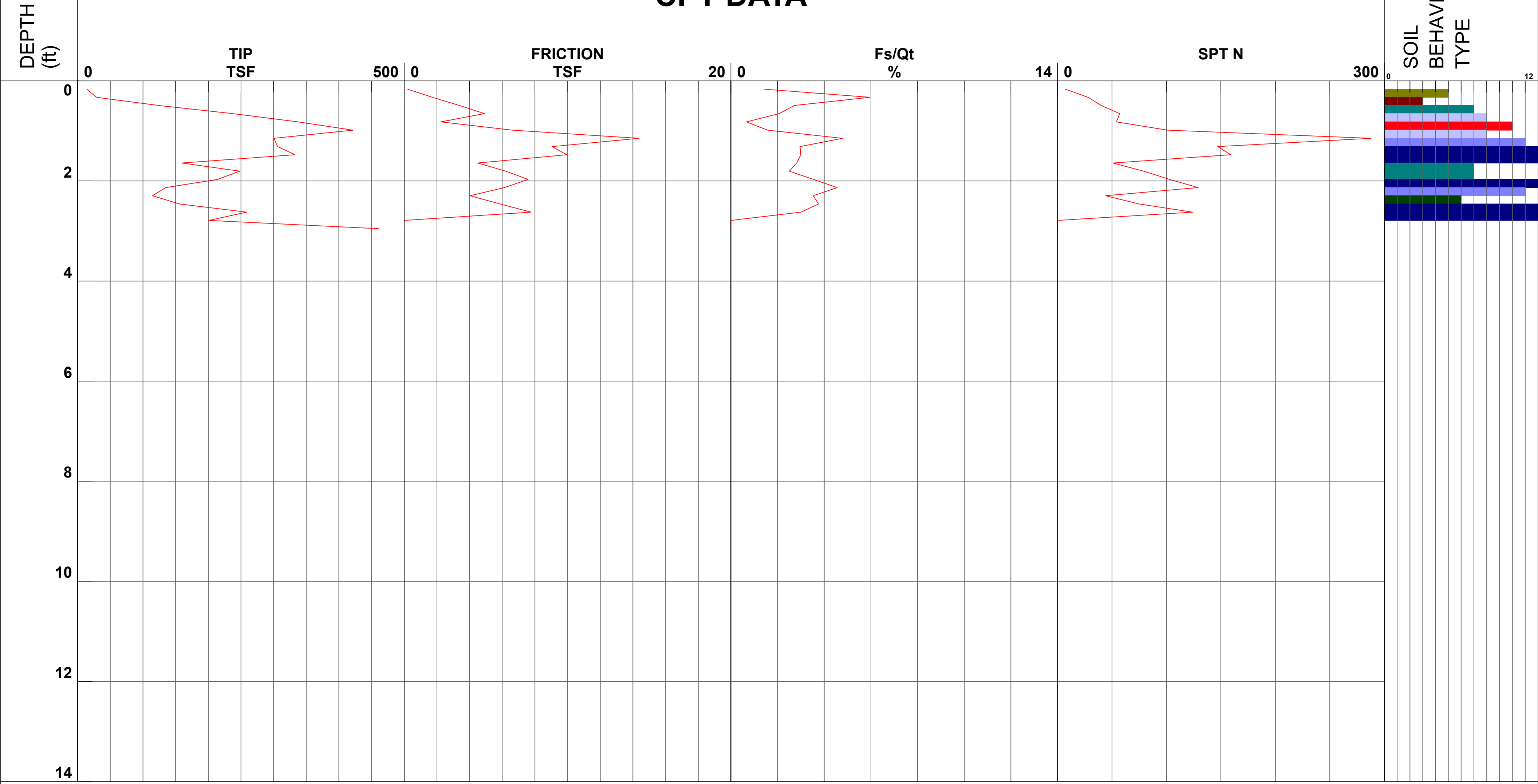
Middle Earth
Geo Testing Inc.

Cornerstone Earth Group

Project The Stanford Wedge
Job Number 105-117-1
Hole Number CPT-03
EST GW Depth During Test >2.95 ft
Operator RB KK
Cone Number DDG1333
Date and Time 7/7/2017 2:32:47 PM
Filename SDF(082).cpt
GPS
Maximum Depth 2.95 ft

Net Area Ratio .8

CPT DATA



■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay
■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt
■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

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.

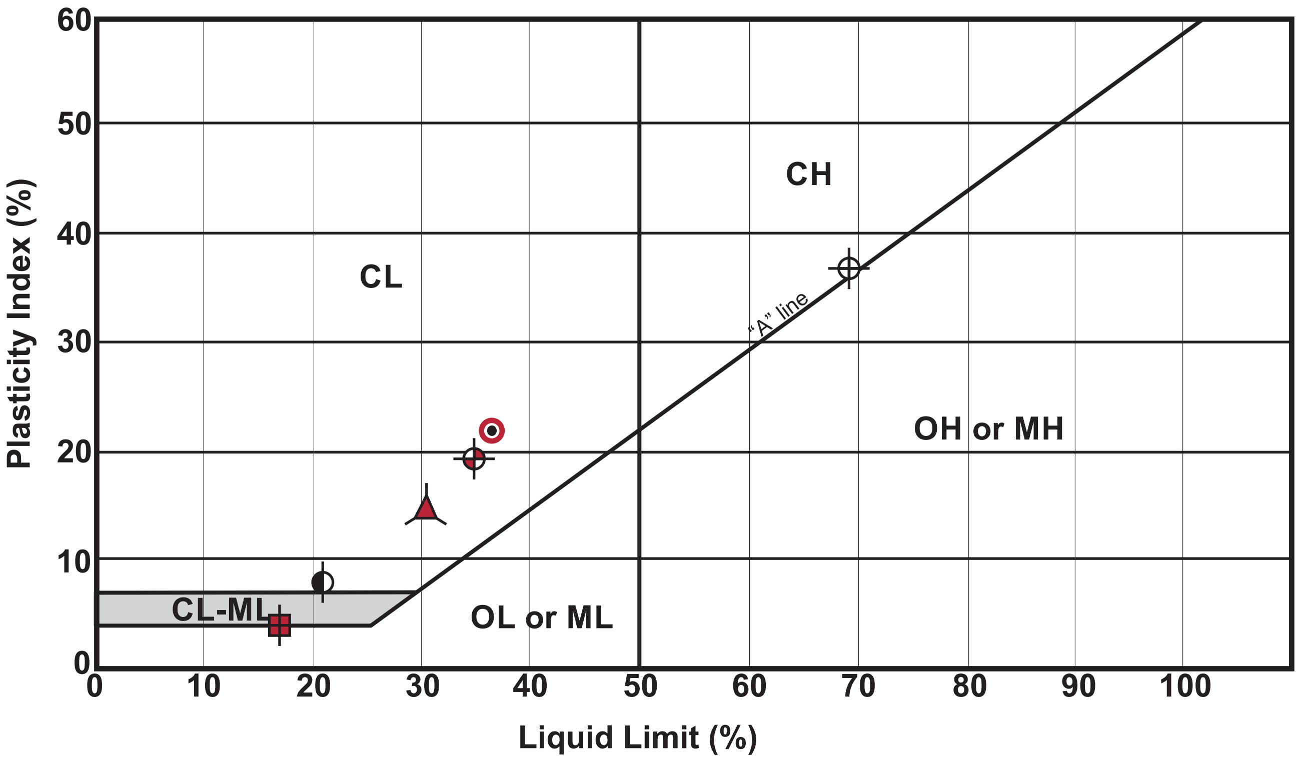
Moisture Content: The natural water content was determined (ASTM D2216) on 30 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities: In place dry density determinations (ASTM D2937) were performed on 14 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.

Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on nine samples of the subsurface soils to aid in the classification of these soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Plasticity Index: Six Plasticity Index determinations (ASTM D4318) were performed on samples of the subsurface soils to measure the range of water contents over which the 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 these tests are shown on the boring logs at the appropriate sample depths.

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	11	35	16	19	63	Sandy Lean Clay (CL) [Qal]
⊞	EB-2	1.5	8	17	13	4	54	Sandy Silty Clay (CL-ML) [Qal]
◉	EB-2	3.0	15	36	14	22	55	Sandy Lean Clay (CL) [Qal]
▲	EB-2	10.5	20	31	16	15	49	Clayey Sand (SC) (CL fines) [Qal]
⊕	EB-3	7.0	29	69	32	37	---	Sandy Claystone [Tw]
◐	EB-4	2.0	8	21	13	8	48	Clayey Sand (SC) (CL fines) [Qal]

CORNERSTONE
EARTH GROUP

Plasticity Index Testing Summary

The Stanford Wedge
Portola Valley, CA

Project Number
105-117-1

Figure Number
Figure B1

Date
August 2017

Drawn By
FLL

APPENDIX C: PREVIOUS FIELD INVESTIGATION (C-1 TRAILS, CORNERSTONE EARTH GROUP 2007)

Los Trancos Creek
Approximate Alignment

Pedestrian Bridge Area

Creek Bank Stabilization Area

N

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EARTH GROUP

Vicinity Map

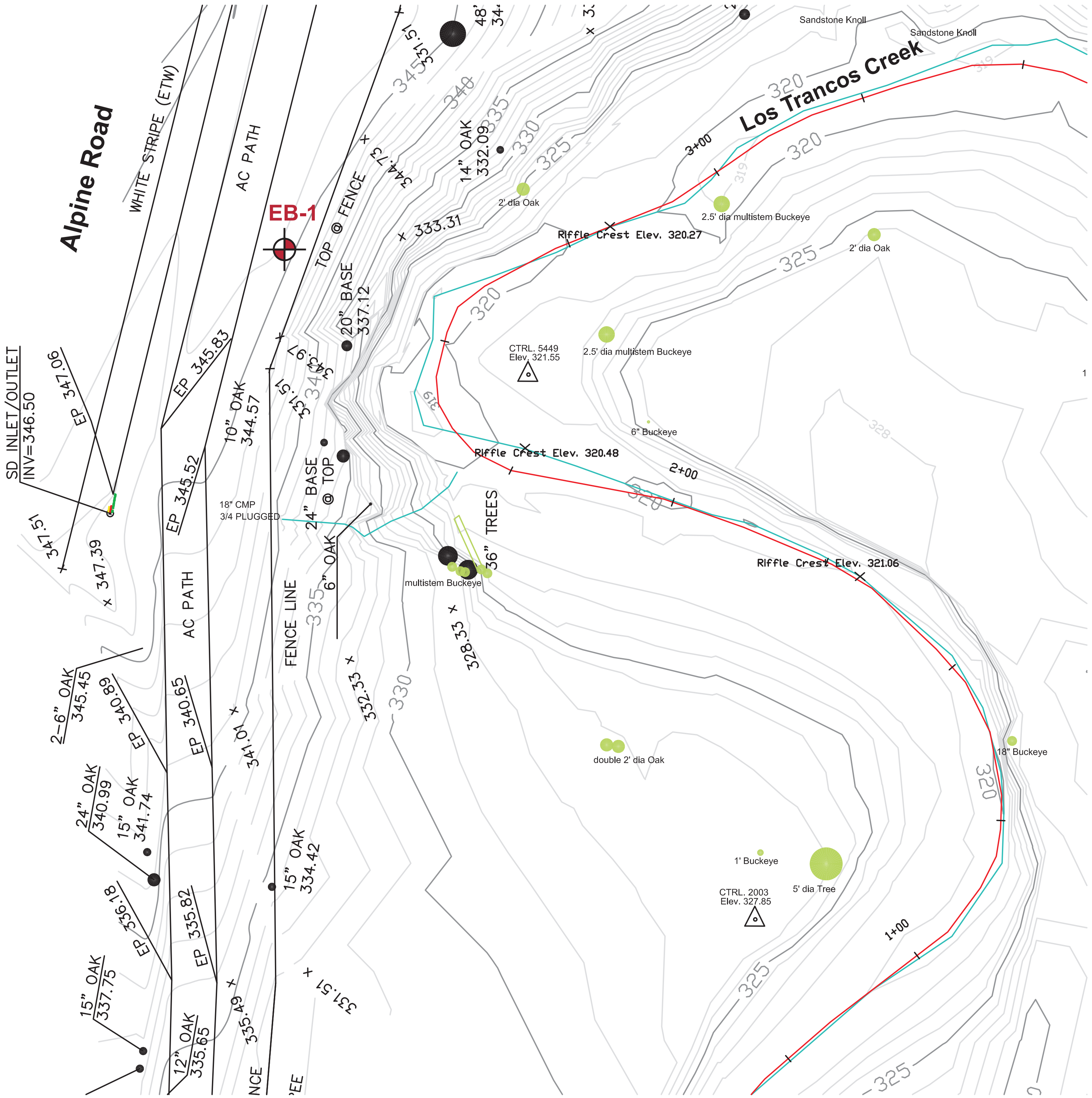
Stanford C-1 Trail, Alpine Road - Westridge Drive to Arastradero Road Portola Valley, CA

Project Number
105-10-1

Figure Number
Figure 1

Date
November 2007

Drawn By
MGV



⌖ **Approximate Location of Exploratory Boring**

Base by Balance Hydrologics, 2007.

Project Number
105-10-1

Figure Number
Figure 2A

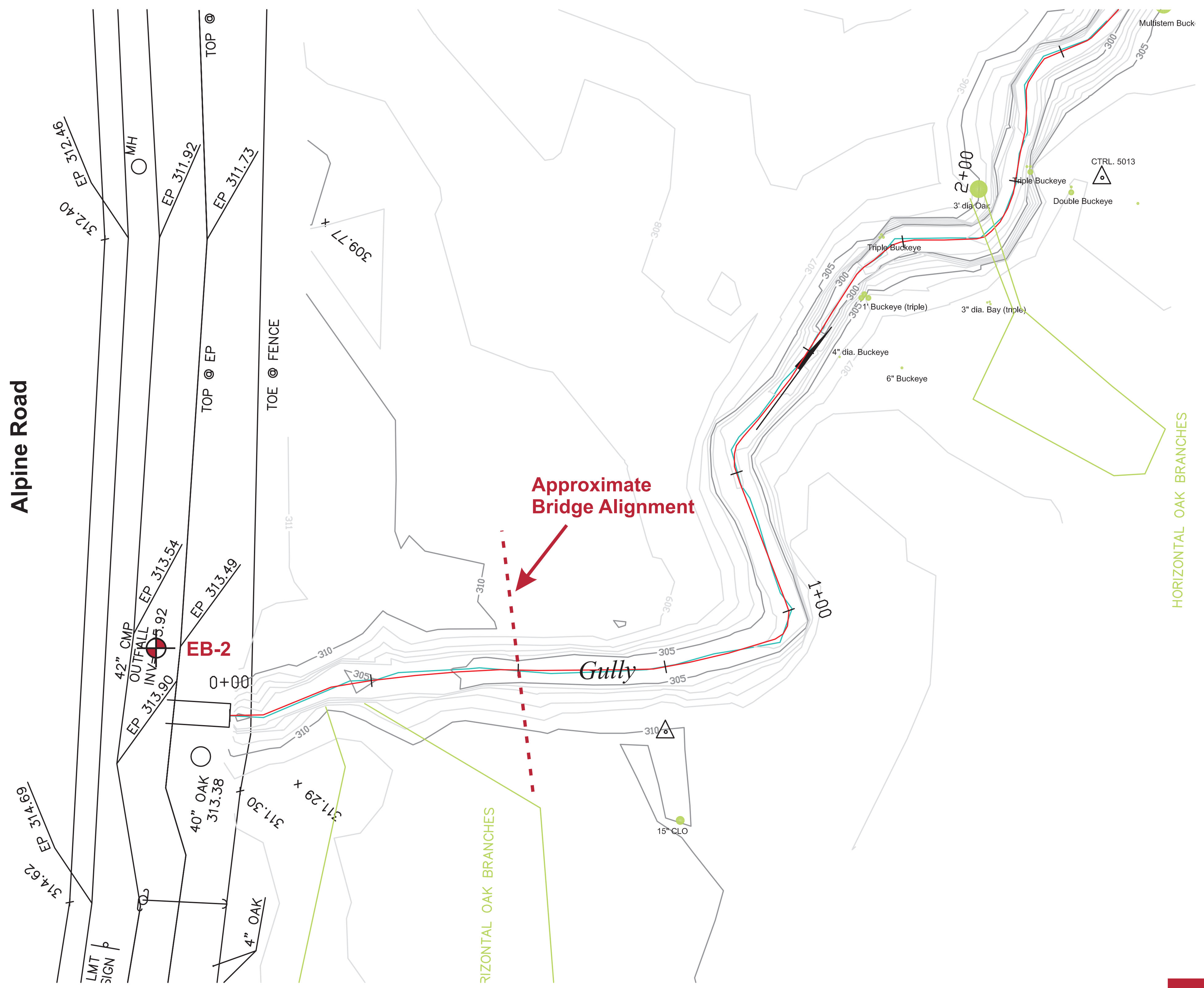
Date
November 2007

Drawn By
MGV

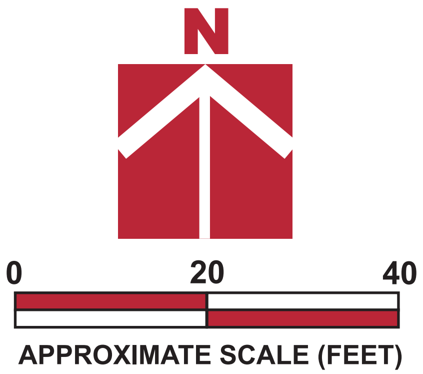
Site Plan - Creek Bank Stabilization Area

Stanford C-1 Trail, Alpine Road -
Westridge Drive to Arastradero Road
Portola Valley, CA

CORNERSTONE
EARTH GROUP



⊕ **Approximate Location of Exploratory Boring**



APPROXIMATE SCALE (FEET)

Base by Balance Hydrologics, 2007.

Project Number
105-10-1

Figure Number
Figure 2B

Drawn By
MGV

Date
November 2007

Site Plan - Pedestrian Bridge

Stanford C-1 Trail, Alpine Road -
Westridge Drive to Arastradero Road
Portola Valley, CA

CORNERSTONE
EARTH GROUP

APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, hollow-stem auger drilling equipment. Two 8-inch-diameter exploratory borings were drilled on October 4, 2007 to depths of 34½ and 49 feet. The approximate locations of exploratory borings are shown on the Site Plans, Figures 2A and 2B. The soils and bedrock 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 based on interpolation of plan contours. The locations and elevations 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.

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-98)

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

OTHER MATERIAL SYMBOLS

Poorly-Graded Sand with Clay	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

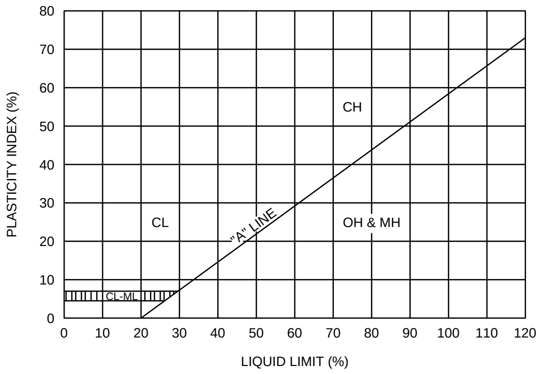
Rock Core

ADDITIONAL TESTS

CA - CHEMICAL ANALYSIS (CORROSIVITY)
CD - CONSOLIDATED DRAINED TRIAXIAL
CN - CONSOLIDATION
CU - CONSOLIDATED UNDRAINED TRIAXIAL
DS - DIRECT SHEAR
PP - POCKET PENETROMETER (TSF)
(3.0) - (WITH SHEAR STRENGTH IN KSF)
RV - R-VALUE
SA - SIEVE ANALYSIS: % PASSING #200 SIEVE
▼ - WATER LEVEL

PI - PLASTICITY INDEX
SW SWELL TEST
TC - CYCLIC TRIAXIAL
TV - TORVANE SHEAR
UC - UNCONFINED COMPRESSION
(1.5) - (WITH SHEAR STRENGTH IN KSF)
UU - UNCONSOLIDATED UNDRAINED TRIAXIAL

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 - 0.5
LOOSE	4 - 10	SOFT	2 - 4	0.5 - 1.0
MEDIUM DENSE	10 - 30	MEDIUM STIFF	4 - 8	1.0 - 2.0
DENSE	30 - 50	STIFF	8 - 15	2.0 - 4.0
VERY DENSE	OVER 50	VERY STIFF	15 - 30	4.0 - 8.0
		HARD	OVER 30	OVER 8.0

* 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

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

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

HARDNESS

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

STRENGTH

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

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

D. Deep – Moderate to complete mineral decomposition: extensive disintegration: deep and thorough discoloration: many fractures, all extensively coated or filled with oxides, carbonates and/or clay or silt.

M. Moderate – Slight change or partial decomposition of minerals: little disintegration: cementation little to unaffected. Moderate to occasionally intense discoloration. Moderately coated fractures.

L. Little – No megascopic decomposition of minerals: little or no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains or fracture surfaces.

F. Fresh – Unaffected by weathering agents. No disintegration or discoloration. Fractures usually less numerous than joints.

CORNERSTONE
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Physical Properties
for
Rock Description

Figure Number

Figure A-2

BORING NUMBER EB-1

CORNERSTONE EARTH GROUP

PROJECT NAME C-1 Trail
PROJECT NUMBER 105-10-1
PROJECT LOCATION Portola Valley
DATE STARTED 10/4/07 **DATE COMPLETED** 10/4/07
GROUND ELEVATION 346 FT +/- **BORING DEPTH** 49 ft.
DRILLING CONTRACTOR EGI
LATITUDE **LONGITUDE**
DRILLING METHOD Hollow Stem Auger
GROUND WATER LEVELS:
LOGGED BY HSR
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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

UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL (scale 1.0 2.0 3.0 4.0 5.0)

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
346.0	0	▨	**Sandy Clay (CL)** very stiff, moist, brown mottled gray, trace gravel and organics, moderate plasticity							
				33	MC-1A	95	18			○
				30	MC-2A	88	20			>>○
	5									
				52	MC-3A	100	16			>>○
337.0			**Claystone [Whiskey Hill Formation]** low hardness, deep weathering, brown mottled gray	42	MC-4A	101	19			>>○
	10									
				90 11"	MC-5A	111	16			
				63	SPT-6A					
	15									
				50 6"	SPT-7A		16			
	20									
322.0			**Claystone [Whiskey Hill Formation]** low hardness, deep weathering, gray	51	SPT-8A					
	25									
				89	SPT-9A		13			
	30									
				50 6"	SPT-10A		12			
311.0	35		*Continued Next Page*							

BORING NUMBER EB-1

PAGE 2 OF 2

CORNERSTONE EARTH GROUP

PROJECT NAME C-1 Trail

PROJECT NUMBER 105-10-1

PROJECT LOCATION Portola Valley

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

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

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 1.0 2.0 3.0 4.0 5.0
311.0	35	▨	**Claystone [Whiskey Hill Formation]** low hardness, deep weathering, gray							
		▨		50 6"	☒ SPT-11A		12			
	40	▨								
		▨		50 6"	☒ SPT-12A					
	45	▨								
		▨		50 6"	☒ SPT-13A					
297.0			Bottom of Boring at 49.0 feet.							
	50									
	55									
	60									
	65									
	70									
	75									

BORING NUMBER EB-2

CORNERSTONE EARTH GROUP

PROJECT NAME C-1 Trail
PROJECT NUMBER 105-10-1
PROJECT LOCATION Portola Valley
DATE STARTED 10/4/07 **DATE COMPLETED** 10/4/07
GROUND ELEVATION 313 FT +/- **BORING DEPTH** 34.5 ft.
DRILLING CONTRACTOR EGI
LATITUDE **LONGITUDE**
DRILLING METHOD Hollow Stem Auger
GROUND WATER LEVELS:
LOGGED BY HSR
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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

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

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 1.0 2.0 3.0 4.0 5.0
313.0	0	▨	**Sandy Clay (CL)** very stiff, moist, brown, trace cobble, gravel, organics, high plasticity							
		▨		55	MC-1A	119	9			>>○
		▨		49	MC-2A	111	12			>>○
	5	▨								
		▨		76	MC-3A	126	10			>>●
		▨		64	MC-4A	126	10			>>○
	10	▨	seam of sand							
		▨		54	MC-5A	121	12			>>●
	15	▨		48	MC-6A	121	11			>>○
		▨		27	MC-7A	115	18			●
	20	▨								
290.0		× ×	**Siltstone [Ladera Formation]** low hardness, deep weathering, brown with gray mottling							
	25	× ×		54	MC-8A	103	23			>>○
	30	× ×		62	MC-9A	106	18			>>○
278.6		× ×		50 6"	MC-10A					
	35		Bottom of Boring at 34.5 feet.							

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 18 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 14 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.

Unconfined Compression: The undrained shear strength (ASTM D 2166) was determined on three samples by unconfined compression strength testing. The results of these tests are shown on the log at the appropriate depth and included as part of this appendix.