

**Appendix C:
Geotechnical Supporting Information**

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C.1 - Geotechnical Investigation

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**GEOTECHNICAL INVESTIGATION
QUAKER HILL PROPERTY
HEALDSBURG, CALIFORNIA**

PROJECT NO. 20181540.001A

September 15, 2017

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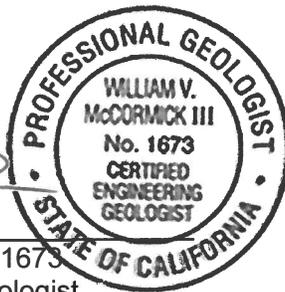
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**GEOTECHNICAL INVESTIGATION
QUAKER HILL PROPERTY
HEALDSBURG, CALIFORNIA**

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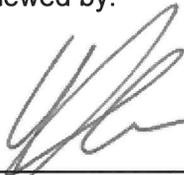


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**GEOTECHNICAL INVESTIGATION
QUAKER HILL PROPERTY
HEALDSBURG, CALIFORNIA**

1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed mixed use development portion of the Quaker Hill property located at 16977 Healdsburg Avenue in Healdsburg, California. Kleinfelder previously performed a Geologic and Geotechnical Investigation at this site and summarized our findings in our previous report dated April 16, 2003. This report has been prepared based on our previous work at the site and the results of subsurface explorations for this study. The following report describes the investigative procedures utilized during this study and presents the results of the investigation and engineering analyses for the site and the proposed development. **This study supersedes our previous study of the overall Quaker Hill property (dated April 16, 2003). Recommendations presented herein are specifically for the mixed-use concept provided by Comstock, which is referenced later in this report.**

1.1 SITE LOCATION AND DESCRIPTION

The site is located at 16977 Healdsburg Avenue, at the northern limits of the City of Healdsburg, as shown on the Site Location, Figure 1. The approximately 32-acre, irregularly-shaped site is bounded on the west by Highway 101, on the north by privately-owned undeveloped property, on the east by North Coast Railroad Authority (NCRA) easement which parallels Healdsburg Avenue, and on the south by the Simi Winery property.

It is our understanding that the site has been historically used as a lumber mill, storage or fabrication facility dating back at least as far as the 1950s. The mill equipment and all previous developments have been removed from the site. Our previous report originally identified areas of fill stock piles, and other undocumented fills throughout much of the site. As part of our work in 2004, Kleinfelder provided observation and testing during removal of old undocumented fills, and regrading of the site with compacted engineered fill. The site grading was completed in compliance with our 2003 recommendations.

Topographically, the site is characterized by a large relatively flat area surrounded on the west and north by elevated benches. Past grading has generated cut and fill slopes in excess of 40 feet in height with gradients of up to 1.25H:1V (horizontal to vertical). While the majority of the site is essentially flat, maximum topographic relief is about 82 vertical feet, ranging from 262 feet above mean sea level (msl) in the southwest corner to 180 feet above msl within the drainage channel at the southern property boundary. Cut slopes at the northern property boundary extend to elevations of 250 msl as well. The northern and western edges of the site have been graded in the past to create level log decks/benches and equipment storage areas associated with the former mill.

In general, the site drains predominantly by sheet flow and artificially created drainage channels towards the south. A south-flowing drainage channel approximately four feet in depth bisects the western portion of the site, extending from the northwest to the southeast corner where it outfalls into an ephemeral drainage offsite. Another channel parallels the NCRA easement and the eastern property line and outfalls into the storm drain system at the site entrance.

1.2 PROJECT DESCRIPTION

It is our understanding that a final development plan has not been prepared for this site and that proposed development could change. According to a conceptual development plan by Dicecco Architecture (dated April 20, 2017), it is our understanding that the project may include a private school site, a hotel site, an independent living site, a Zen center site with cottages, and an additional future development parcel. Our geotechnical report assumes that no individual residential ownership development will be included as part of the development on this site. Currently the school site is planned to include 1- and 2-story structures. The proposed hotel and independent living sites will accommodate structures up to 3 stories. Cottages on the Zen site are presumed to be single-story structures. The proposed (conceptual) development is shown on Figure 2, Conceptual Development Plan.

It is our understanding that mass earthwork grading is not proposed for this site since it has already been graded in accordance with our previous recommendations. We have assumed that future grading will be minimal, and consist mainly of relatively minor cuts and fills necessary to create the road improvements, parking areas and building pads as well as site drainage.

1.3 PURPOSE AND SCOPE OF SERVICES

The purpose of this study was to identify and evaluate the geologic and geotechnical conditions at the site, assess the overall feasibility of the proposed commercial project, and to provide geotechnical design-level recommendations for project development. The scope of this investigation included the following:

- Literature research and review of our previous work on this site
- Geologic reconnaissance and mapping of the site and vicinity
- Drilling, logging, and sampling of fourteen (14) new exploratory borings
- Laboratory testing of selected soil samples for engineering properties
- Geologic and geotechnical engineering analysis
- Preparation of this summary report.

The conclusions and recommendations presented in this report are based on data acquired and analyzed during this study and the review and evaluation of pertinent subsurface explorations and data from other studies in the vicinity, including our 2003 study of the Quaker Hill property, and our grading observation and testing services performed in 2004. This report is intended to provide information for project planning and design including mitigation of adverse geologic and geotechnical conditions, site grading, surface and subsurface drainage control, retaining walls, building foundations and pavement design. This report is considered valid for the conceptual commercial development described above, subject to a thorough peer review by the City's geotechnical consultant and a thorough plan review by Kleinfelder to check that our conclusions and recommendations presented herein are valid for the proposed project. Future reviews may develop design questions that require further investigation and/or analysis prior to finalization of plans, issuance of building permits, and commencement of construction.

The recommendations and conclusions in this report assume that the project will be designed in accordance with the 2016 CBC, which has been adopted since January 1, 2017. This code is based on the 2015 International Building Code and on ASCE 7-10. This report provides seismic design parameters based on the 2016 CBC.

This study did not include an evaluation of potentially hazardous or toxic materials that may or may not be present on or beneath this site, including Naturally Occurring Asbestos (NOA). This study is not intended to provide a comprehensive assessment of soil-related corrosivity. It is our understanding that assessment is the responsibility of the project corrosion engineer.



1.4 AUTHORIZATION

This study/report was authorized by the Professional Services Agreement contract by and between Comstock, Crosser & Associates Development Company, Inc. and Kleinfelder, Inc., dated July 5, 2017; amended August 23, 2017 and executed by Mr. David Lauletta, CEO and Mr. William McCormick, Area Manager of Kleinfelder, Inc.

2.0 EXECUTIVE SUMMARY

Based on the data acquired and analyzed during this investigation, it is our opinion that from a geologic and geotechnical viewpoint, the site is considered suitable for future commercial development provided the recommendations contained in this report are implemented into the planning, design and construction of the project.

The majority of the site is mantled by variable thickness engineered fill that was observed and tested by Kleinfelder during site preparation grading in 2004. Locally, the fill is underlain by native alluvial deposits. The alluvial deposits consist of stiff to very stiff clay. The alluvial deposits, as well as the sloped portions of the site, are underlain by weathered bedrock of the Great Valley Sequence and serpentinite. Great Valley Sequence bedrock consists predominantly of interbedded shale and minor sandstone. Serpentinite, considered to be part of the Franciscan Complex, consists of highly sheared to rocky exposures with little to no asbestos-form minerals observed. These two bedrock units are in fault contact with each other on the site.

Two previously unidentified faults were found crossing the northern and southern portion of the site. Based on exposures in our trenches on this site from our previous 2003 study, the Foss Creek Detention Basin site (Kleinfelder, 1996) and radiocarbon age-dating results, both faults are considered to be active (having experienced ground rupture within the last 11,000 years). The potential for future ground rupture to occur at this site is considered high along the identified fault traces, moderate within a 50-foot zone on either side of the faults, and low elsewhere on the site. Buildings intended for human occupancy should not be constructed within the Building Setback Zones presented on Figure 3.

The site will be subject to strong seismic ground shaking resulting from future moderate to major earthquakes on active faults in the region. As such, structures should be designed to accommodate predicted shaking in accordance with the appropriate seismic design codes and regulations. The potential for secondary seismic hazards to occur such as liquefaction, lateral spreading, lurch cracking, and settlement is considered to be low at this site.

Three, relatively small landslides exist near the southwest corner of the site. These slides are shallow (<10 feet thick), limited in extent and do not pose a significant hazard to development. These landslides can be readily removed during future grading and/or reconstructed if development encroaches to within 30 feet of their boundaries.

The principal geotechnical factors that may affect proposed development at this site are the presence of variable thickness engineered fill, expansive soils and bedrock, and locally shallow (perched) groundwater. Geotechnical engineering recommendations and design criteria for both ground improvement and alternative foundations types have been developed to address and mitigate these and other potentially adverse conditions. These recommendations are presented in the body of this report.

3.0 RESEARCH AND REVIEW

3.1 REGIONAL GEOLOGY

The site is located in the low hills that define the west edge of Alexander Valley in the Coast Range Geomorphic Province of Northern California. This province is generally characterized by northwest-trending mountain ranges and intervening valleys, which are a reflection of the dominant northwest structural trend of the bedrock in the region. The basement rock in the northern portion of this province is presumed to consist of Franciscan Complex, a diverse group of igneous, sedimentary, and metamorphic rocks of Upper Jurassic to Cretaceous age (140 to 65 million years old). The Franciscan Complex is part of a northwest trending belt of material, immediately adjacent to the eastern edge of the San Andreas fault system, which is located approximately 20 miles southwest of the site. In the site vicinity, the Franciscan Complex rocks have been unconformably overlain by Tertiary age continental and marine sedimentary and volcanic rocks. These Tertiary age rocks have been locally overlain by younger Quaternary alluvial, colluvial, and landslide deposits.

3.2 SITE GEOLOGY

The site and vicinity have been mapped by Huffman and Armstrong (1980, California Division of Mines and Geology, Special Report 120) and Delattre and McLaughlin (2011). Huffman and Armstrong indicate that the majority of the site is underlain by siltstone and sandstone of the Jurassic-Cretaceous age Great Valley Sequence. The upper elevations in the northeast corner of the site and the ridgeline adjacent to the northern site boundary are mapped as underlain by a fault-bound band of Jurassic-Cretaceous age Franciscan Complex (Formation) serpentinite. The central portion of the site is shown to be underlain by alluvial deposits consisting of clay, silt, sand and gravel. Huffman and Armstrong (1980) have designated the upper elevations at the north end and southwest corner of the site as located within Slope Stability Zone "C;" consisting of areas of relatively unstable rock and soil units on slopes greater than 15%, typically containing abundant landslides. The remainder of the site has been zoned "Bf," a locally level area within hilly terrain which may be underlain or bounded by unstable or potentially unstable rock materials. No landslides have been mapped by Huffman and Armstrong (1980) on the site.

Delatrrre and McLaughlin (2011) show the majority of the site to be underlain by undivided Holocene to latest Pleistocene alluvium, consisting of sand, gravel, silt and clay. The majority of the elevated portion of the site is shown to be underlain by Late Cretaceous to Early Jurassic Franciscan mélangé, which is a tectonic mixture of pervasively sheared argillite and graywacke that forms a matrix of more coherent rock masses of varied lithology. They also show a thin band of silica carbonate (altered serpentinite) and Great Valley Sequence mudstone, shale and sandstone near the southwestern end of the site. That portion of their map that includes the site is presented on Figure 4, Area Geology.

3.3 FAULTING AND SEISMICITY

The site, as well as the entire Northern California Coastal Region, is located within a seismically active portion of the state dominated by the presence of the San Andreas fault system, which forms the boundary between two tectonic plates of the earth's crust. At this boundary, the Pacific Plate (west of the fault) is moving north relative to the North American Plate (east of the fault). In the Northern San Francisco Bay/Coastal Region, this movement is distributed across a complex system of strike-slip, right-lateral, parallel and sub-parallel faults which include the San Andreas, Healdsburg/Rodgers Creek and Maacama, among others.

The site is not (currently) located within an Earthquake Fault Zone (EFZ) as defined by the California Geological Survey (CGS, formerly California Division of Mines and Geology, 1983) in accordance with the Alquist–Priolo Earthquake Fault Zone Act of 1972. In the 1976 published version of the Jimtown quadrangle map, the site was located within an Earthquake Fault Zone (CGS, 1976). The 1976 version shows two discontinuous traces of the Healdsburg fault (northern extension of the Healdsburg/Rodgers Creek fault) approximately 300 feet southeast (of the southeast corner) and approximately 100 feet northeast (of the northeast corner) of the site. A third trace was mapped approximately 1,000 feet east of the site. No faults were mapped on the site proper.

At the time of the 1976 version of the EFZ map, criteria for zonation included active and potentially active fault traces. Since that time, criteria for zonation by the CGS has been modified to include only those traces that are “well-defined” and “sufficiently active.” Based on an apparent lack of such criteria in this area, active fault zonation for the Healdsburg fault was removed by the CGS

(1983) in the Healdsburg area. Currently the site is not located within Earthquake Fault Zone as mapped by the CGS.

The nearest known active fault (from the literature) is the Maacama fault, located approximately 4.2 miles northeast of the site at its closest point, which is capable of producing a maximum earthquake Richter Magnitude event of 6.9. Moderate to major earthquakes generated on the Maacama fault can be expected to cause strong ground shaking at the site. Strong ground shaking can be expected from moderate to major earthquakes generated on other faults in the region such as the Healdsburg/Rodgers Creek fault, (located 5.5 miles northeast of the site as per CGS, 1983), and the San Andreas fault (located 20 miles southwest of the site). A number of large earthquakes have occurred within this region in the historic past. Some of the significant nearby events include two 1969 Santa Rosa earthquakes (M5.6, 5.7), the 2000 Napa earthquake (M5.0), 2014 West Napa earthquake (M6.0) and the 1906 San Francisco earthquake (M8+). Future seismic events in this region can be expected to produce strong seismic ground shaking at this site. The intensity of future shaking will depend on the distance from the site to the earthquake focus, magnitude of the earthquake, and the response of the underlying soil and bedrock.

Two previously unknown active faults were identified on this site during our April, 2003 study (discussed below). It should be noted that a light earthquake with a magnitude of 4.0 occurred on July 29, 2003. The epicenter of this earthquake was located approximately 3 miles northwest of the site, along a straight-line projection of one of the fault traces discovered during our 2003 study.

3.4 FLOODING

According to the Federal Emergency Management Agency (FEMA), Flood Insurance Rate Map panel 06097C0363E dated December 2, 2008, the site is not located within within a flood zone.

3.5 PREVIOUS SITE-SPECIFIC STUDIES

In addition to the above noted published references, our literature review included the following geologic and geotechnical reports previously produced for the site and site vicinity by private consultants:

- Geologic Evaluation-Phase 1: The Ridge Property, Healdsburg, CA, Harding Lawson Associates, November 1, 1985
- Preliminary Environmental Assessment: BMC West Corporation, Healdsburg Facility, Healdsburg, CA, Dames and Moore, February 23, 1990
- Preliminary Environmental Site Assessment, Phase 2 Study: BMC West Corporation, Healdsburg Facility, Healdsburg, CA, Hallenbeck and Associates, June 6, 1990
- Hydrogeologic Investigation: RJW Lumber Company, 16977 Healdsburg Avenue, Healdsburg, CA, Hallenbeck and Associates, February 13, 1991
- Soil Remediation and Groundwater Assessment: RJW Lumber Company, 16977 Redwood Highway North, Healdsburg, CA, LOK Environmental, Inc., June 1993
- Geologic Investigation and Fault Hazards Evaluation Report, Foss Creek Detention Basin, Kleinfelder, Inc., January 19, 1996(a)
- Geotechnical Investigation Report: Foss Creek Detention Basin, Healdsburg, CA, Kleinfelder, Inc., April 8, 1996(b)
- Geologic and Geotechnical Investigation, Healdsburg 32-Acre Parcel, Healdsburg, California, Kleinfelder, Inc., April 17, 2003

The majority of the reports were reviewed to obtain subsurface geologic information from the subsurface explorations conducted during those studies. Relevant data compiled during review of these reports has been considered and incorporated into both our current geologic hazards assessment and our geotechnical investigation for the project, as appropriate.

Of particular importance is subsurface data obtained from trenching on the Foss Creek Detention Basin (FCDB) site located immediately east of this site, and our previous report on this 32-acre (Quaker Hill) Parcel, by Kleinfelder in 1996(a) and 2003, respectively. Numerous photo lineaments were identified on the FCDB site and to the east that were suspected to be related to faulting. As part of the study, Kleinfelder excavated and logged two trenches across two of the photo lineaments, which were mapped near the southeastern boundary of the 32-acre Quaker Hill site. Faulting was exposed in both of those trenches, offsetting near-surface, recent alluvial soils and bounding active and buried stream channels. Kleinfelder (1996a) concluded that these faults, with shears extending to within 6 inches of the ground surface, showed strong evidence of Holocene activity and exhibit the potential for future surface fault rupture.

During our 2003 study on this site (which included five trenches and logging of two pipeline trenches) and our previous study at the FCDB site immediately to the east, our trenching program exposed two previously unidentified strike-slip faults (predominant movement side to side with minor vertical displacement), associated with the Healdsburg fault, which cross this site. Exposures within trenches excavated for these studies and radiocarbon age dating indicate that these two faults are active (defined by CGS as faults with displacement within the last 11,000 years). Based on the accumulated data, it is our opinion that the potential for ground surface rupture at the site is high along the identified fault traces and moderate within a 50-foot zone on either side of the fault traces. The location of these fault splays on this site are shown on Figures 2 (Conceptual Development Plan) and 3 (Site Plan and Geologic Map).

Taking into account the segmented and distributive nature of these faults and other segments of the Healdsburg fault previously mapped by the CGS (1976), in our 2003 study we estimated that future horizontal displacements on these faults could range from a few inches to possibly 2 feet during a moderate to major event centered in this area. The potential for ground rupture elsewhere on the site is considered to be low.

3.6 AERIAL PHOTOGRAPH INTERPRETATION

The following aerial photographs, from our files, were used in this study:

<u>Date</u>	<u>Scale</u>	<u>Type</u>
October 24, 1952	1:9,600	Black and White
July 11, 1980	1:24,000	Black and White
April 21, 2002	1:12,000	Black and White
Google Earth Images – various dates		

These photos were scrutinized for the presence of terrain features indicative of landslides, fills and active fault zones, and particularly lineaments. A lineament is seen on a stereo aerial photo pair as a feature with tonal differences on either side. These differences may be indicative of changes in soils types, vegetation, groundwater levels or sedimentary bedding characteristics. Lineaments are often indicative of the presence of geologic structures such as folds and faults. Data from the aerial photograph interpretation, geologic reconnaissance and subsurface exploration on this site has been summarized on the Site Plan and Geologic Map, Figure 4.

As discussed earlier, Kleinfelder (1996a) identified several photo lineaments during the investigation on the FCDB site, east of this site. Some of those lineaments correspond to previously mapped traces of the Healdsburg fault (CGS, 1976) as well as to the two faults exposed by trenching in that study. None of the previous lineaments identified in the Kleinfelder 1996(a) study were traced northwest as far or across the 32-acre site.

Further review of aerial photographs during this current study identified faint lineaments crossing the site at two locations. One of the lineaments is a projection of the lineament/fault identified in trench T-1 on the FCDB site. This lineament crosses the southern end of the site along a northwest trend to a drainage swale at the western boundary of the site. The lineament could not be traced farther to the northwest beneath the roadway fill prism for Highway 101. The other lineament crosses the northeastern portion of the site and can be traced from a swale (north of the site adjacent to Highway 101) southeastward across Healdsburg Avenue. Southeast of Healdsburg Avenue the lineament becomes more pronounced where it runs along the western edge of a suspected fault-bounded pressure ridge of serpentinite, east of the FCDB site. Another strong lineament can be seen in the photos along the eastern side of the aforementioned pressure ridge, extending northwestward to within approximately 100 feet (and farther northwestward) of the northeast corner of the site. This particular lineament coincides with what was commonly referred to as the main trace of the Healdsburg fault in this area. That trace is depicted on Figure 3.

A few small landslides and areas of incised erosion are visible in the earlier photos on and adjacent to the site. The locations of localized slope instabilities identified on the photos as well as those observed during our geologic reconnaissance are presented on Figure 3.

4.0 FIELD EXPLORATION

4.1 GEOLOGIC RECONNAISSANCE

Geologic reconnaissance and mapping of the site and vicinity were performed by our Certified Engineering Geologist during a site visit in August, 2017. The purpose of the reconnaissance and mapping was to identify any changes in the surficial geologic and geomorphic conditions including existing and potentially adverse geologic conditions that could affect site development, since remedial grading was performed in 2004. The results of the reconnaissance and mapping are summarized on the Site Plan and Geologic Map, Figure 3.

Relatively few natural bedrock outcrops were exposed on the site, with the exception of cut slopes on the western boundaries. The majority of the site is covered with engineered fill that was placed during remedial grading operations in 2004 on the site. Natural slopes elsewhere on the site are covered with residual soil and colluvial deposits. Within the bedrock exposures on the west side of the site, two predominant and varied lithologies were identified. These include interbedded shale and sandstone of the Great Valley Sequence, and serpentinite associated with the Franciscan Complex. These rock formations are considered to be Upper Jurassic to Cretaceous in age (65 to 140 million years old).

The Great Valley Sequence bedrock typically consisted of highly fractured to locally sheared, moderately weathered shale with interbeds of sandstone. Locally, moderately weathered sandstone exposures were prevalent and massive. The Great Valley bedrock was generally oriented to the northwest with variable, steeply inclined dips. In contrast, highly weathered, highly sheared exposures of serpentinite were exposed in the northeastern corner and in an isolated area on the west property line. Little to no asbestos-form minerals were visually noticed within the serpentinite exposures, however it is likely that some asbestos would exist in this type of rock.

Three small, shallow (2-10 feet deep) slump and/or translational landslides were observed locally in the southwest corner of the site. These features are relatively small and do not cross the small drainage ditch at the base of the slope

The contacts between the Great Valley and serpentinite bedrock were obscured in the field by soil and colluvial cover, but they occurred rather abruptly in spatial distance and coincided relatively closely with a change in geomorphic character of the slopes, namely adjacent to swales. The relatively abrupt change in bedrock and geomorphology at these two areas on the west and north ends of the site suggested that the contact between the two different formations could be faulted. As throughout much of Northern California, serpentinite is commonly found along fault zones.

Areas of groundwater (surface) seepage were not observed on the site, however, water and phreatophyte (“water-loving”) vegetation was observed within the on-site drainage channels and in the drainage swale area at the base of the highway fill prism, at the northwestern corner of the site.

4.2 EXPLORATORY BORINGS

Our geotechnical field exploration program for this study, which consisted of fourteen borings across the site and within conceptual building locations, was performed on August 7 and 21, 2017. The purpose of the borings was to provide subsurface geologic and geotechnical data across the site and at conceptual building footprints. The boring locations are shown on Figure 3. The borings were approximately located by Kleinfelder personnel by pacing from existing features shown on the site plan prepared by Carlile•Macy. As such, the locations of the explorations should not be considered more accurate than the locating methods used.

The borings were drilled with a Deep Rock DR8K and a Mobile Drill B-53 truck-mounted rig utilizing 4- and 6-inch-diameter solid-flight augers, respectively. Borings were drilled and sampled to depths of up to 15 feet below the existing ground surface. Upon completion, the borings were backfilled with soil cuttings.

Our Certified Engineering Geologist observed the drilling, logged the conditions encountered, and obtained samples for visual classification and pertinent laboratory testing. Samples of the soil and bedrock were obtained using 2.43-inch (inside diameter) California and 1.4-inch (inside diameter) Standard Penetrometer Test samplers driven with a 140-pound automatic hammer dropping 30 inches. The logs of exploratory borings are presented on Figures A-4 through A-17 in Appendix

A. Visual classifications were made in accordance with the Graphics Key, Soil Description Key and Rock Description Key presented on Figures A-1 through A-3, respectively.

5.0 LABORATORY TESTING

Selected soil samples recovered from the borings were tested in our laboratory to evaluate pertinent engineering and physical properties. The laboratory-testing program evaluated the natural moisture content, density, plasticity (Atterberg Limits), Expansion Index, unconsolidated undrained shear strength, and particle size analysis of selected soil samples. Classifications made in the field were modified, as appropriate, based on the laboratory test results. Classifications presented on the boring logs reflect the laboratory test results. The results of our geotechnical laboratory tests are discussed below in Section 7.0, Laboratory Test Results and are summarized on the Boring Logs. All test results are included in Appendix B.

One bulk sample of near-surface engineered fill soil from Boring KB-5 and KB-6 was submitted to Cerco Analytical of Concord, California for preliminary corrosivity analysis/screening. The results of their tests are briefly discussed in Section 7.3, and the full report of the results is presented in Appendix C.

6.0 SUBSURFACE CONDITIONS

In general, surface and near-surface conditions on-site consist of engineered (compacted) fill soils, underlain by alluvial clay and sandy clay basin deposits. Throughout the site, the fill and alluvial deposits overlie various bedrock lithologies consisting of shale, sandstone, minor claystone and serpentinite. Slopes at the west and north edge of the site expose shale, sandstone, and serpentinite bedrock overlain by residual and colluvial soil layers. Previous cut slopes on the north end of the site have been re-graded as engineered fill at the base of the slope.

Engineered fill across the site varies but generally can be described as consisting of gravelly clay with sand (CL) to clayey gravel with sand (GC). The fill is typically dry to moist, and hard to dense. Fill thickness varies across the site, and was encountered in all of the borings drilled for this study except Boring KB-3. Relative fill thickness at each boring is indicated within parenthesis on Figure 3.

Alluvial deposits were encountered within the central portion of the site beneath the fill in Borings KB-4, KB-5, KB-6, KB-8, KB-9, KB-10, KB-11, KB-12 and KB-14. The alluvium typically consisted of a fat (plastic) clay/basin deposit, which ranged in consistency from stiff to very stiff.

Within the majority of the site, the engineered fill and alluvial soils overlie sandstone and shale of the Great Valley Sequence. The sandstone encountered in the borings near the contact of the overlying alluvium or fill is moderately weathered, extremely weak, very closely fractured and fine to medium grained. The shale encountered is moderately weathered to decomposed, extremely weak, intensely fractured and very thinly bedded. We expect the sandstone and shale relative hardness / strength increases with depth below the depths of our borings. Franciscan Complex serpentinite exists in the northeastern and southwestern ends of the site. This ultramafic rock is highly weathered to decomposed, extremely weak, highly plastic, pervasively sheared and has little to no visible asbestos-form minerals. These two bedrock units are in fault contact with each other, at two locations on the site.

Previously, in our 2003 study, groundwater (perched) was encountered in some borings at depths as shallow as 1.5 feet below the existing ground surface (at that time). Only one boring, KB-7, encountered groundwater seepage during our current study, which was at a depth of about 8 feet. Our experience in the project vicinity indicates that local groundwater levels can fluctuate depending on factors such as seasonal rainfall, groundwater withdrawal, and construction activities on this or adjacent properties. In general, winter to early summer construction can experience extra earthwork costs related to the presence of groundwater or seepage, depending on the magnitude of prior seasonal rainfall.

The soil and groundwater conditions described above, as encountered in the borings for this current investigation, have been simplified for ease of report presentation. A more detailed description of the conditions encountered is presented on the boring logs in Appendix A. Our interpretation of the subsurface geologic conditions across the site is presented on the Geologic Cross Section A-A', located on Figure 3.

7.0 LABORATORY TEST RESULTS

7.1 SOIL INDEX PROPERTIES AND STRENGTHS

Intermittent soil and rock samples were collected during our investigation and tested in our laboratory to evaluate physical properties pertinent to the proposed development. Atterberg Limits testing provides an indication of soil and rock plasticity and expansion potential. The results of the Atterberg Limits tests indicate Plasticity Indices ranging from 20 to 33, and Liquid Limits ranging from 37 to 65. Based on the Plasticity Index results, the tested soil and rock samples indicate a medium to high expansion potential. Based on the Liquid Limit results, the tested soil and rock samples indicate medium to very high expansion potential. An Expansion Index test result on one sample from Boring KB-3 was measured to be 63, which is considered to indicate medium expansion potential. Based on the Atterberg Limit and Expansion Index testing, the near-surface soil and rock have the potential to undergo strength and volume changes with variations in moisture content.

Soil and rock samples were selected for undrained shear strength testing to represent various soil and rock types encountered near the proposed improvements. These results were used to evaluate available allowable bearing capacities for shallow foundations and skin friction values for drilled pier design. The strength test results are summarized on the boring logs and are graphically shown in Appendix B.

The results of dry density and moisture content testing along with the results of the strength tests are shown on the boring logs adjacent to the samples tested.

7.2 PRELIMINARY CORROSIVITY

7.2.1 Current Study

One bulk composite sample of near-surface engineered fill soil from Borings KB-5 and KB-6 was submitted to Cerco Analytical of Concord, California for preliminary corrosivity analysis/screening. The composite sample was subjected to chemical analysis for the purpose of preliminary corrosion assessment. The samples were tested for the following by CERCO Analytical:

- Redox (ASTM D1498)
- pH (ASTM D4972)
- Resistivity, as-received and 100% saturation (ASTM G57)
- Soluble Sulfide Content (ASTM D4658M)
- Soluble Chloride and Sulfate Content (ASTM D4327)

The test results are tabulated below in Table 7.1, and are presented in Appendix C.

Table 7.1 Summary of Corrosion Test Results

Boring	Sample Depth (feet)	Redox (mV)	pH	Resistivity, As-received (Ohm-Cm)	Resistivity, 100% saturation (Ohm-Cm)	Water Soluble Chlorides (ppm)	Water Soluble Sulfates (ppm)
KB-5 / KB-6	0 - 6	+310	8.29	7,400	1,400	27	32

According to ACI 318 Section 4.3, Table 4.3.1, a sulfate concentration below 1,000 ppm is negligible (no restrictions on concrete type). A water-soluble chloride content of less than 500 ppm is generally considered non-corrosive to reinforced concrete.

Based on the ACI 318 guidelines for sulfate and chloride concentrations, the sample tested is considered non-corrosive to concrete, there is no specification for maximum water cement ratio, and concrete should have a minimum compressive strength of 2,500 psi.

The measured (as-received) minimum resistivity value indicates that the tested sample is “moderately corrosive” to buried ferrous metals. This correlation is based on Corrosion Basics, 2nd edition (Roberge, 2006).

The measured (100% saturated) minimum resistivity value indicates that the tested sample is “highly corrosive” to buried ferrous metals. This correlation is based on Corrosion Basics, 2nd edition (Roberge, 2006).

Kleinfelder had laboratory testing performed to provide data regarding corrosivity of on-site soils. The above corrosivity results are preliminary and are an indicator of potential soil corrosivity for the samples tested. Other soils found on the site may be more, less, or of a similar corrosive

nature. Our scope of services does not include corrosion engineering, and therefore, a detailed analysis of the corrosion test results is not included in this report. A qualified corrosion engineer should be retained to review the test results and design protective systems that may be required.

7.2.1 2003 Study

Two samples of near-surface alluvial soil from Borings K-3 and K-5 (previous study) were tested in 2003 by ETS of Petaluma, California for preliminary corrosivity analysis/screening (File # 17383). The samples were tested for the following by ETS:

- pH (ASTM G51)
- Resistivity (ASTM G57)
- Soluble Chloride Content (ASTM D512)
- Soluble Sulfate Content (ASTM D516)

The test results are summarized below in Table 7.2.

Table 7.2 Summary of Corrosion Test Results

Boring	Sample Depth (feet)	pH	Resistivity (Ohm-Cm)	Water Soluble Chlorides (ppm)	Water Soluble Sulfates (ppm)
K-3	3.5	5.32	2,560	65	255
K-5	4.0	6.25	2,780	82	66

According to ACI 318 Section 4.3, Table 4.3.1, a sulfate concentration below 1,000 ppm is negligible (no restrictions on concrete type). A water-soluble chloride content of less than 500 ppm is generally considered non-corrosive to reinforced concrete.

Based on current ACI 318 guidelines for sulfate and chloride concentrations, the samples tested are considered non-corrosive to concrete, there is no specification for maximum water cement ratio, and concrete should have a minimum compressive strength of 2,500 psi.

The measured minimum resistivity values indicate that the tested samples are “highly corrosive” to buried ferrous metals. This correlation is based on Corrosion Basics, 2nd edition (Roberge, 2006).

8.0 CONCLUSIONS

8.1 GENERAL

Based on the data acquired and analyzed during this investigation, it is our opinion that from a geologic and geotechnical viewpoint, the site is considered suitable for future commercial development provided the recommendations provided in this report are implemented into the project planning, design and construction. The most significant geologic and geotechnical engineering factors that must be considered in development design and construction are the presence of active faults and possible future ground rupture, moderate to highly expansive surface and near-surface soils and bedrock, variable thickness engineered fill soils, seasonal high groundwater, the control of site drainage, and the potential for strong seismic shaking generated from earthquakes on active faults in the region.

8.2 GEOLOGIC HAZARDS

8.2.1 Expansive Soils and Bedrock

The upper native in-place alluvial soils, colluvial soils and engineered fill soils at the site are, in general, moderately to highly expansive. Colluvial soils on slopes are also prone to creep (slow downslope movements of soil and weathered bedrock by gravity). In addition, the highly weathered serpentinite and possibly some areas of highly weathered shale are considered to be highly expansive. Expansive soils and bedrock exposed to seasonal variations in moisture content may undergo volume changes generating heave and resultant distress to lightly loaded footings or slabs. Therefore, we believe that expansive soils and highly weathered bedrock are not suitable for support of shallow foundations in their present condition. The potential for foundation heave from expansive soil/bedrock can be reduced by capping these materials with a 36-inch-thick layer of low expansion potential (select) fill material in conjunction with relatively standard concrete slabs-on-grade with spread footing foundations. As an alternative, the soils could be lime-treated to a depth of 36 inches. Another option would be to support proposed structures at this site on post-tensioned mat or pier and grade beam foundation systems underlain by on-site expansive soil or bedrock that is properly moisture conditioned. As such, recommendations for both earthwork and foundation support alternatives are provided in the Site Preparation and Grading Section (Section 9.2) and the Foundations Section (Section 9.5) of this report.

8.2.2 Serpentinite

Serpentinite, considered to be part of the Franciscan Complex, exists within limited slope exposures on the west and north ends of the site, and at shallow depths (i.e. Boring KB-13) beneath engineered fill in the northeast corner of the site. Kleinfelder did not perform an assessment of Naturally Occuring Asbestos Mineral-(NOAM) rocks or deposits for this site. Since there are no NOAM-bearing rocks or deposits identified in the footprint of the Conceptual Development area, it is not expected that the development at this site will involve significant grading/disturbance of serpentinite. Some exposure may occur in foundation excavations or pipeline trench work in the northeast corner of the site. If that occurs, further assessment of potential NOAM-bearing rocks and deposits should be performed to address potential construction and long-term exposure hazards. Standard grading procedures for the handling of serpentinite and/or NOAM materials will most likely be needed during the construction in the northeast portion of the site (i.e. near the gym building).

8.2.3 Landslides

Three, relatively small slump/translational landslides exist in the southwest corner of the site. These landslides are shallow (<10 feet thick), limited in extent and do not pose a significant hazard to development. These landslides can be readily mitigated during any future grading and/or reconstructed if development encroaches to within 30 feet of their boundaries. If highly weathered serpentinite bedrock is exposed or is in the near surface of slopes, it could be susceptible to slope instability (depending on slope gradient). Existing slopes that are cut into native materials at gradients steeper than 3H:1V (in serpentinite) and 2H:1V (other materials) should be considered to have a moderate potential for future slumping if not mitigated.

The remaining majority of the proposed development is located on relatively gentle topography with low slope inclination, therefore the potential for landsliding is low to non-existent. No evidence of massive landsliding was observed on this site.

8.2.4 Groundwater and Surface Drainage

Groundwater (perched) was encountered at relatively shallow depths in some of the exploratory excavations on this site during our 2003 study. In addition, standing water and wet areas were observed in and around the existing drainage channels on and immediately off-site. The existence and control of shallow groundwater should be taken into account in the planning and construction

of the development on this site. In addition, the control of off-site surface and subsurface drainage entering the site is critical to development on this site. This is especially the case along the western boundary of the site where fills have been placed in previous drainage swales

8.3 SEISMIC HAZARDS

8.3.1 Ground Surface Rupture

Ground (surface) rupture typically occurs in the immediate vicinity of active faults and is related to a moderate to high magnitude seismic event. The site currently lies outside the state-mandated Earthquake Fault Zone (CGS, 1983), as previously discussed. During our previous studies on and adjacent to this site, we have identified two previously unidentified strike-slip faults (predominant movement side to side with minor vertical displacement), associated with the Healdsburg fault, which cross this site. Exposures within trenches excavated for these studies and radiocarbon age dating indicate that these two faults are active. Based on the accumulated data, it is our opinion that the potential for ground surface rupture at the site is high along the identified fault traces and moderate within a 50-foot zone on either side of the fault traces. Taking into account the segmented and distributive nature of these faults and other segments of the Healdsburg fault previously mapped by the CGS (1976), we estimate that future horizontal displacements on these faults could range from a few inches to possibly 2 feet during a moderate to major event centered in this area. The potential for ground rupture elsewhere on the site is considered to be low.

NOTE: It should be noted that the current conceptual development plan shown on Figure 2 shows portions of buildings crossing into the recommended 50-foot-wide building setback zone associated with the identified faults. The conceptual plans should be modified to stay outside of these setback zones for buildings considered for human habitation.

8.3.2 Seismic Shaking

This site will likely experience strong seismic ground shaking as a result of future moderate to major earthquakes on nearby active faults, namely the Healdsburg/Rodgers Creek, Maacama and the San Andreas faults, as well as the faults identified on this site.

In recent years, many modern structures located near the seismic source have been severely damaged or collapsed. The severe damage and/or collapse are attributed to near fault motions that are characterized by energetic unidirectional velocity pulses. What makes these motions particularly damaging is the impulse (area under the acceleration time history multiplied by the mass). A structural system that yields during a long duration pulse (impulse loading) may experience very large permanent deformations and/or collapse. The extent of these actions depends on the strength and natural period of the structure and the structure articulation, as well as the amplitude, duration, and shape of the pulse. The near fault pulse type motions can be particularly damaging because they can accumulate inelastic deformations in one direction and their considerations in the near fault conditions should be properly evaluated.

Due to potential near-fault motion resulting from seismic activity on the faults on this site and the Maacama and other traces of the Healdsburg/Rodgers Creek faults, near source effects should be considered in the structural design of the proposed facility. Structures with strength discontinuities, soft stories, plan irregularities, discontinuous shear walls and ductile moment frames are particularly vulnerable to these type of motions and should either be avoided or properly evaluated.

8.3.3 Seismically Induced Ground Failure

Modes of seismically induced ground failure include liquefaction, lateral spreading, lurching, dynamic compaction and settlement, and seismically induced landslides. The potential at the site for seismically induced ground failures is discussed for each potential failure mode in the following sections.

8.3.3.1 *Liquefaction, Lateral Spreading and Lurching*

Soil liquefaction is a condition where saturated, granular soils undergo a substantial loss of strength and deformation due to pore pressure increase resulting from cyclic stress application induced by earthquakes. In the process, the soil acquires a mobility sufficient to permit both horizontal and vertical movements if the soil mass is not confined. Soils most susceptible to liquefaction are saturated, loose, clean, uniformly graded, Holocene age and fine grained sand deposits. Lateral spreading is a potential hazard commonly associated with liquefaction where extensional ground cracking and settlement occur as a response to lateral migration of subsurface liquefiable material. This phenomenon typically occurs adjacent to slopes or channels. Lurching

describes the detachment, lateral movement and failure of steep slopes or cut banks related to channels, and occurs as the result of strong seismically induced shaking.

We interpret the site to be covered by soils cohesive in nature underlain by relatively shallow bedrock. The susceptibility to liquefaction at the site is thus considered to be low. The potential for other secondary seismic effects related to liquefaction such as lateral spreading or lurching is also considered low.

8.3.3.2 *Dynamic Compaction*

Dynamic compaction is a phenomenon that can occur when granular soils undergo densification during seismic shaking. Because the site is interpreted to be underlain by soils that are predominantly cohesive in nature as well as bedrock, the potential for dynamic compaction to occur at the site is considered to be low.

8.3.3.3 *Landslides and Seismically Induced Slope Failures*

A majority of the site is located on flat ground or relatively gentle topography. Therefore the potential for seismically induced slope failures is considered to be low within or immediately adjacent to the proposed development at this site. There is a moderate potential for seismically induced slope failure in the existing, small landslides in the southwestern corner of the site, if not mitigated.

8.4 SEISMIC DESIGN CONSIDERATIONS

8.4.1 Site Class

In developing seismic design criteria, the characteristics of the soils underlying the site are an important input to evaluate the site response. Based on the results of our field investigation and review of available geologic information, the site can be classified as Site Class C according to Section 1613.3.2 of the 2016 CBC. Values for Site Class C are presented in the following section.

8.4.2 2016 CBC Seismic Design Parameters

For a 2016 California Building Code (CBC) based design, the estimated Maximum Considered Earthquake (MCE) mapped spectral accelerations for 0.2 second and 1 second periods (S_s and S_1), associated soil amplification factors (F_a and F_v), and mapped peak ground acceleration (PGA) are presented in Table 8.1. Corresponding site modified (S_{MS} and S_{M1}) and

design (S_{DS} and S_{D1}) spectral accelerations, PGA modification coefficient (F_{PGA}), PGA_M , risk coefficients (C_{RS} and C_{R1}), and long-period transition period (T_L) are also presented in Table 8.1. Presented values were estimated using Section 1613.3 of the 2016 California Building Code (CBC), chapters 11 and 22 of ASCE 7-10, and the United States Geological Survey (USGS) U.S. seismic design maps¹.

TABLE 8.1 Ground Motion Parameters Based on 2016 CBC

Parameter	Site Class C	Reference
	Value	
S_s	1.587	2016 CBC Section 1613.3.1
S_1	0.625g	2016 CBC Section 1613.3.1
Site Class	C	2016 CBC Section 1613.3.2
Seismic Design Category	D	2016 CBC Tables 1613.3.5 (1) and (2)
F_a	1.000	2016 CBC Table 1613.3.3(1)
F_v	1.300	2016 CBC Table 1613.3.3(2)
S_{MS}	1.587	2016 CBC Section 1613.3.3
S_{M1}	0.813	2016 CBC Section 1613.3.3
S_{DS}	1.058g	2016 CBC Section 1613.4.4
S_{D1}	0.542g	2016 CBC Section 1613.4.4
PGA	0.618g	ASCE 7-10 Figure 22-7
F_{PGA}	1.000	ASCE 7-10 Table 11.8-1
PGA_M	0.618g	ASCE 7-10 Section 11.8.3
C_{RS}	0.987	ASCE 7-10 Figure 22-17
C_{R1}	0.972	ASCE 7-10 Figure 22-18
T_L	8 seconds	ASCE 7-10 Figure 22-12

¹ <https://earthquake.usgs.gov/designmaps/us/application.php>

8.5 SETTLEMENT DUE TO NEW FILL

Based on our understanding of conceptual development and grading, new fills, if any, would generally be less than 3 feet within planned building areas. Assuming the new fills are properly compacted and prepared in accordance with the recommendations of this report, the potential for adverse fill settlement is low. Settlement potential within the in-place native alluvial deposits below the fill on the site is low to moderate and of limited magnitude due to the relatively low to moderate compressibility. If new fills greater than 3 feet in height are planned, we can provide an evaluation of fill-induced settlement estimates at the locations of the planned fills.

9.0 RECOMMENDATIONS

9.1 BUILDING SETBACKS

Based on the data collected and analyzed during our previous study, the faults identified on this site should be considered active and capable of future ground rupture and horizontal, and possibly some vertical displacement. Therefore, buildings intended for human habitation (2,000 hours or more per year) should not be constructed across the mapped traces or within a Building Setback Zone established on either side of the faults. Other improvements such as streets, parking lots or landscape areas can be constructed within the setback zone. Development planning at this site should recognize that utilities may cross the faults on the site. Therefore, we recommend that specialized construction techniques or designs be incorporated into the utility details, such as flexible connections or shutoff valves located on either side of the fault traces.

9.2 SITE PREPARATION AND GRADING

We anticipate that grading within the central flat portion of the site will consist mainly of cuts and fills less than 3 feet deep (or less) to raise or lower existing grades, and to create relatively level building pads and pavement areas having positive surface drainage. If larger grading is proposed for this site, we should be contacted to provide specific grading recommendations.

The site should be stripped of vegetation and organic debris before any grading commences. We anticipate that the stripping operation will require the removal of 2 to 3 inches of surface soil. Deeper stripping or grubbing will be required where concentrations or pockets of organic-laden soil and/or undocumented fills are encountered. The stripped, organic-rich material may be stockpiled and used for future landscaping purposes; however, this material should not be re-used as engineered fill.

Underground obstructions encountered during grading should be removed or abandoned in place. The resultant voids should be filled with lean concrete, or backfilled with soil that is compacted as subsequently recommended in this report. The removal/abandonment of underground obstructions encountered should be performed in accordance with the standards of the local governing agency.

Within the proposed building areas, and to a minimum of 5 feet laterally beyond each building footprint and 3 feet beyond exterior adjacent concrete flatwork (defined herein as building envelope), any weak soils, organic-rich soils and any dry expansive soils should be removed for their full depth (or reconditioned in place, if applicable). The Geotechnical Engineer/Engineering Geologist should observe excavation bottoms following removals. Where unsuitable (soft, organic, etc.) materials are observed within the excavation bottom, removal of such material will be required (over-excavation). The Geotechnical Engineer/Engineering Geologist must approve the excavated or over-excavated materials prior to their reuse as fill. Material approved for reuse must be thoroughly moisture conditioned (discussed below) prior to replacement and compaction.

Over-excavation depths can be difficult to predict. During the dry season, we have observed that shrinkage cracks in expansive soils can be as deep as five feet, or more. However, our experience with similar site soil conditions would indicate that over-excavation depths on the order of about 3 feet are typical and actual depths depend on such factors as the time of year grading is performed and seasonal rainfall totals. Because the exact depth of required moisture conditioning will not be known prior to site grading, we suggest that the contract documents contain provisions for over-excavation on a unit price basis.

Areas approved to receive fill should be scarified to a depth of at least 8 inches, moisture conditioned to above the optimum moisture content (at least 4 percent above optimum for expansive on-site clay soils having a plasticity index of 15 percent or greater) and compacted to at least 90 percent relative compaction¹ (or between 88 and 92 percent for expansive soils). Within the building envelopes, above optimum moisture content of expansive soil areas must be evaluated by the Geotechnical Engineer/Engineering Geologist through laboratory testing, visual inspection or both, to a depth of at least 3 feet below subgrade prior to fill placement. Approved fill material can then be placed in layers no more than approximately 8 inches in loose thickness and similarly moisture conditioned and compacted to achieve planned grades. For buildings supported on standard concrete slabs-on-grade and shallow spread footings, only engineered fill of low expansion potential should be used as fill in the upper 36 inches of planned finished grade within the building envelopes, as previously recommended.

¹ Relative compaction refers to the in-place dry density of fill expressed as a percentage of maximum dry density of the same material determined in accordance with the ASTM D1557 laboratory test procedure. Optimum moisture content refers to the water content (percentage by dry weight) corresponding to the maximum dry density.

9.2.1 Engineered Fill

Non-expansive engineered fill and imported engineered fill should be of low expansion potential and free of organic matter and should conform, in general, to the requirements presented below in Table 9.1.

Table 9.1 Non-Expansive Engineered Fill Recommendations

Fill Requirement		Test Procedures	
		ASTM ¹	Caltrans ²
Gradation			
Sieve Size	Percent Passing		
3 inch	100	D 422	202
¾ inch	70-100	D 422	202
No. 200	15-70	D 422	202
Plasticity			
Liquid Limit	Plasticity Index		
<30	<15	D 4318	204
Organic Content			
Less than 3%		D 2974	---
Expansion Potential			
20 or less		D 4829	---
¹ American Society for Testing and Materials Standard (latest edition) ² State of California, Department of Transportation, Standard Test Methods (latest edition)			

In general, the existing fill and near-surface soils encountered do not meet the criteria outlined above and are not considered suitable for reuse as non-expansive engineered fill unless chemically modified by means of lime treatment or similar method.

Any imported fill materials to be used for non-expansive engineered fill should be sampled, tested, and approved by the project geotechnical engineer prior to being used at the site. Lime treatment recommendations are provided below.

9.2.2 Lime Treatment

As an alternative to importing non-expansive fill, treatment of the onsite soil with high-calcium pulverized quicklime may be considered to reduce the expansive potential of the onsite soils. The amount of quicklime to be added should be evaluated by Kleinfelder prior to construction with additional laboratory testing. The application rate should be sufficient enough to result in an expansion index of 20 or less, based on ASTM D4829. For preliminary estimation purposes, it has been our experience that the high-calcium quicklime application rate can likely be in the range of 4 to 6 pounds of quicklime per cubic foot of soil treated.

Lime-treated soils should be uniformly moisture conditioned to at least 2 percent above the optimum moisture content and be mechanically compacted to the requirements for engineered fill compaction presented in Section 9.2.3. Depending on the contractor's equipment, the maximum lift thickness prior to compaction should generally be no more than 12 inches unless the contractor can demonstrate that a thicker lift can be properly compacted.

9.2.3 Engineered Fill Compaction Criteria

Fill should be spread in thin lifts, moisture conditioned to near optimum moisture content, and compacted to the relative compaction specifications presented below. Fill (or cut) subgrade soils should be finished to present smooth, unyielding surfaces. Subgrade soils should be maintained at their moist or above optimum moisture contents and be free of shrinkage cracks, until covered by permanent construction. A summary of our compaction requirements is presented in the following Table 9.2.

Table 9.2 Summary of Compaction Requirements

<u>Area</u>	<u>Compaction Requirements</u>
General and Select Engineered Fill and Trenches	In lifts, a maximum of 8 inches loose thickness, compact to a minimum of 90 percent relative compaction at, or within, 2 percent of the optimum moisture content for select (low expansion potential) fill. On-site expansive soil, if used outside of, or below, select fill zones, should be moisture conditioned to, and maintained at, 4 percent or more above optimum moisture content and compacted to between at least 88 and 92 percent relative compaction.
Parking, Streets and Driveways	Compact the top 6 inches of subgrade soil to at least 95 percent relative compaction at or within 2 percent of optimum moisture content for select fill of low expansion potential. On-site expansive soil should be moisture conditioned to at least 4 percent above optimum moisture content, and compacted between 90 and 92 percent relative compaction. Soils should be maintained at their moist or above optimum moisture contents until covered by permanent construction.

Where building pads are to accommodate conventional foundations (spread footings and/or conventional slabs-on-grade) supported by select fill, the differential fill thickness beneath the structure should not be more than 2 vertical feet across a horizontal distance of 40 feet. If the fill thickness is greater than that, additional sub-excavation and filling will be required in order to meet this criterion. Likewise, conventional foundations should not be supported by cut/ fill transition lots. Cut/fill transition lots should be over-excavated in the cut area and replaced with compacted engineered fill (with a minimum of 3 feet of select fill in the upper portion of the pad) in order to meet the differential fill thickness criteria presented above.

When grading is performed in the winter, spring or early summer, there is a risk that the site may be saturated and too soft to support construction equipment. Normally suitable fill material may be too wet to properly compact and excavation subgrades can become unstable. Such soil conditions could be mitigated by over-excavation and backfilling with imported dry fill by lime-treating on-site soils, and/or by other means (such as by incorporating geo-synthetics).

In general, site preparation and grading operations should be observed by a representative of Kleinfelder. This will allow us to check whether unforeseen or detrimental materials are exposed by the construction equipment and to modify our recommendations, if necessary.

9.3 SLOPE CONSTRUCTION

Currently we are unaware of any new planned slope construction. If new slopes are proposed we should be contacted for specific recommendations. In general, slope construction should be performed as follows.

Fills constructed on existing slopes that are steeper than 6H: 1V should be keyed into the existing slope at the base of the fill and benched along the bottom of the fill, as shown on Figure 5, Typical Fill Slope and Subdrain Details. A keyway should be excavated at least 2 feet into competent material (as determined by the Geotechnical Engineer/Engineering Geologist) along the entire downslope edge of planned fill and a keyway subdrain constructed at the rear of the keyway. However, the actual keyway depth and extent should be determined by the Geotechnical Engineer/Engineering Geologist during construction. Benches should be cut into competent soil or bedrock along the back of the fill area, as fill placement progresses upslope.

Subdrainage should be incorporated at the rear (upslope side) of keyways and may be needed at intermittent bench locations as determined by the Geotechnical Engineer/Engineering Geologist during grading. Subdrainage should be constructed using the materials and methods discussed above and shown on Figure 5.

Finished fill slopes should be trimmed to expose a firm surface free of loose material and should be no steeper than 3H:1V if more than 5 feet in height using on-site materials. Fill slopes less than 5 feet in height can be inclined at 2H:1V using on-site materials. Steeper fill slopes are possible if import select materials are used or if engineering reinforcement (such as geogrid) is utilized. Kleinfelder should be contacted to provide more specific design recommendations if fill slopes steeper than 3H:1V are desired. In general, cut slopes should be no steeper than 3H:1V. Cut slopes as steep as 2H:1V are considered feasible where competent shale or sandstone is exposed, subject to field evaluation by the Engineering Geologist. Slopes higher than 30 feet in vertical height should be benched near mid-slope in accordance with building code requirements. The bench should be minimum of 6-feet-wide and should contain a lined V-ditch near the back of the bench. Slope drainage should be directed to an appropriate drainage collection facility.

Upon completion of grading, denuded slopes should be planted with fast-growing, deep-rooted groundcover and erosion-control matting to reduce the risk of erosion, as determined by the project Civil Engineer. Finished soil subgrade areas should generally be sloped, and drainage gradients maintained to carry surface water away from construction areas.

9.4 TEMPORARY EXCAVATION AND BACKFILL

Shallow excavations for utility trenches can readily be made with low to moderate difficulty using either a backhoe or trencher. We expect the walls of trenches less than 5 feet deep to remain in a near-vertical configuration for short periods during utility construction provided equipment or excavated spoil surcharges are not located near the top of the excavation and groundwater is not encountered. Where trenches extend deeper than 5-feet, the excavation can become unstable. Trenches, regardless of depth, should be evaluated for stability prior to personnel entering them. Shoring or sloping of the deeper trench walls will be necessary to protect personnel and to provide stability. At a minimum, trenches should conform to the current CAL-OSHA requirements for worker safety.

We recommend trench backfill be placed and compacted to the requirements of engineered fill, as previously presented in Section 9.2, Table 9.2. Care should be taken to adequately compact utility trench backfill in structure, flatwork, and pavement areas. Poor compaction will likely cause subsequent settlement of the trench, resulting in possible distress cracking to the overlying improvements.

9.5 FOUNDATIONS

Three foundation systems are recommended for the proposed buildings:

- Standard spread footing foundations and slabs-on-grade are suitable for 1- to 2-story structures if the upper 36 inches of the subgrade is replaced with non-expansive select (imported) engineered fill.
- 2- to 3-story structures may require support from deeper foundations such as reinforced concrete drilled piers if shallow foundations may be subject to excessive settlement under heavy loads.
- Post-tensioned slabs on native soil or bedrock can be used if construction is on relatively flat building pads.

- Drilled pier-and-grade-beam foundations with structural slabs or timber floors are suitable for support of all structures. This option would not require a level building pad.

9.5.1 Drilled Pier Design Criteria

Piers should be designed for the following parameters:

- **Vertical Loads.** *Shale or Sandstone Bedrock.* Skin Friction (downward or uplift) = 1,250 psf, Soil or Serpentine Skin Friction (downward or uplift) = 275 psf. Neglect end bearing due to settlement concerns.

Drilled piers can be designed using an allowable skin friction of 1,250 pounds psf for the length of penetration into shale or sandstone bedrock. This applies to dead plus long-term live loads and includes a factor of safety of at least 2.0. It is intended for use in a working stress analysis (allowable stress design). The strength of overlying engineered fill or engineered fill over properly prepared native soil can be included in pier design calculations. However, because of potential differences in the amount of strain necessary to develop full shear strength, we recommend that the allowable skin friction be limited to 275 pounds per square foot in fill and native clay soils. The above friction values can be increased by one-third for the inclusion of transitory loads such as wind or seismic forces. We recommend that end bearing be neglected because of the difficulty of cleaning out small diameter pier holes.

- **Tensile Capacity.** *Tensile capacity may be obtained by multiplying the above compressive capacity by a factor of 0.8, and adding the weight of the foundation.*
- **Lateral Loads.** Passive pressure (soil or serpentine) = 250 pcf (equivalent fluid pressure) applied over a two pier diameter width. Passive pressure (shale or sandstone bedrock) = 400 pcf (equivalent fluid pressure) applied over a two pier diameter width.

It is customary to design piers with length to diameter ratios less than 10 as rigid elements using an allowable passive pressure for lateral resistance. Lateral loads can be resisted by passive pressure against both the pier caps and shafts. Drilled piers may be designed for a passive equivalent fluid pressure of 250 pcf acting over a width of two pier diameters, up to a maximum of 2,500 pounds per square foot for the portions that are embedded in soil or serpentine. This is a design value for use in working stress analysis and contains a factor of safety of approximately 1.5. This value can be increased to an equivalent fluid pressure of 400 pcf acting over a width of two pier diameters, up to a maximum of 4,000

pounds per square foot, for the portions of the piers that are embedded in shale or sandstone bedrock.

The above resistance values can be increased by one-third for the inclusion of transitory loads such as wind or seismic forces. Passive pressure should not be included in the top three feet below surface grade, due to the expansive soil conditions. Additional drilled pier lateral resistance evaluations can be performed if required, when the foundation systems and structural loading conditions are designed in more detail.

- **Combined Loading.** Allowable vertical and lateral loads may be combined without reduction.

- **Estimated Settlement:** Less than one-half inch (total and differential)

This is our best estimate of actual settlement that may occur under wall loads of up to 4 kips per foot and column loads up to 50 kips. Differential settlements are estimated to be less than ½ inch over a span of 25 feet between adjacent, similarly loaded, foundations.

- **Adjacent Slopes.**

Based on our review of the proposed improvements, we do not anticipate drilled pier foundations will be adjacent to existing or planned slopes. If this assumption is incorrect, we should be notified to provide recommendations regarding a reduction in allowable lateral resistance, or the necessity to include creep forces in the drilled pier design.

- **Minimum Diameter.** 18 inches

- **Minimum Spacing.** 3 pier diameters

- **Minimum Depth.** 8 feet

- **Grade Beams.** Design for expansion pressure

An air space (2-inch minimum) should be provided beneath grade beams connecting the drilled piers to reduce the potential for soil or rock contact and subsequent uplift expansion pressures. Alternatively, the grade beams can be designed to withstand an expansive soil uplift pressure. However, soil and rock expansion pressures can vary greatly throughout the site. If grade beams are designed to resist expansive soil uplift pressures, the locations of those foundation elements should be provided to Kleinfelder so that we can provide estimates of design expansion forces based on correlations to index testing or further laboratory testing.

Pier depths and exposed soils in pier holes should be checked by Kleinfelder before reinforcing steel is installed and concrete is placed. The contractor should anticipate that drilled piers might encounter boulders or zones of hard bedrock. If piers encounter relatively hard rock it may be possible to shorten the pier length, but this should be evaluated on an individual basis.

9.5.2 Shallow (Spread) Foundations

Standard spread footing foundations and slabs-on-grade are suitable for building support if the upper 36 inches of the subgrade is replaced with low-expansion select (imported) engineered fill that is compacted to a minimum of 90% relative compaction in accordance with previous sections of this report, and expected settlements are tolerable. The select fill should be placed to such a depth as to provide a minimum of 36 inches below the building slab and 12 inches below the base of footings. The fill pad should extend a minimum of five feet beyond the perimeter of the building in all directions. It is possible that lime-treated on-site soil could be used instead of imported fill, but the effectiveness of lime treatment is highly dependent on soil mineralogy. The effectiveness of lime treatment should be verified by laboratory testing. If lime treatment is a desired option, we could provide a proposal for a feasibility study.

Foundation Loads: Wall loads < 5 kips per foot; column loads < 50 kips.

These loads pertain to dead plus frequently applied live loads (not including wind or seismic). If heavier loads are anticipated, we should be contacted to review our recommendations.

Allowable Maximum Bearing Pressure: 2,500 psf

This is a net pressure. Therefore, the weight of the foundation and backfill over the foundation can be neglected. This value contains a factor of safety of at least 3 and is intended for use in a working stress analysis with “un-factored” loads. A one-third increase may be applied to this value when considering the effects of transient loads such as wind or seismic.

Allowable coefficient of friction on the bottom of foundation: 0.35

This value contains a factor of safety of at least 1.5 and assumes good contact between a concrete foundation and the underlying soil. A one-third increase may be applied to this value when considering the effects of transient loads such as wind or seismic. **If moisture barriers or other substances are placed beneath footings, the coefficient of friction can be significantly lower.**

Allowable lateral (passive) pressure: 250 pounds per cubic foot (pcf) (triangular distribution).

Passive pressure should be neglected in the top one foot of soil unless confined by slabs or pavements. This value includes a factor of safety of at least 1.5, which generally corresponds to a predicted lateral deflection of less than one-half inch. The above resistance value can be increased by one-third for the inclusion of transitory loads such as wind or seismic forces.

Combined Loading. Allowable vertical and lateral loads may be combined without reduction.

Passive pressure and bottom friction can also be combined without reduction.

Estimated Total Settlement: Less than one inch

This is our best estimate of actual settlement that may occur under wall loads of up to 5 kips per foot and column loads up to 50 kips. Differential settlements are estimated to be less than ½ inch over a span of 25 feet between adjacent, similarly loaded, foundations.

Minimum Footing Width: 18 inches

Minimum Footing Depth: 24 inches

Footing concrete should be poured neat against compacted fill. A representative from Kleinfelder should observe all footing excavations prior to placement of concrete to check that the conditions exposed are as anticipated, or to modify our recommendations, if necessary.

9.5.3 Post-Tensioned Slabs or Mat Foundations

Structures can also be supported on post-tensioned slabs or rigid mat foundations that are designed by a structural engineer experienced in design for expansive soil conditions. If post-tensioned slabs or mats are designed in accordance with this report section, the 36-inch select fill layer is not required beneath the structures. We recommend that preliminary design be based on the parameters listed below. These parameters are based on our experience, and the procedures presented in the Uniform Building Code (2012) and the Post Tensioning Institute’s publication “Design and Construction of Post-Tensioned Slabs on Ground” (2004).

<u>Parameter</u>	<u>Estimated Value</u>
Edge Moisture Variation Distance (e_m) - Center Lift	3.5 feet
Edge Moisture Variation Distance (e_m) - Edge Lift	5.0 feet
Differential Soil Movement (y_m) - Center Lift (Shrink)	1.5 inches
Differential Soil Movement (y_m) - Edge Lift (Swell)	1.7 inches

The preceding parameters are intended for preliminary design and cost estimating purposes only. Because of the soil type and variety of anticipated conditions, if post tensioned slabs are desired, we recommend that they be individually analyzed.

Post-tensioned slabs can be designed to impose dead (plus code live) and total design (including wind or seismic forces) load bearing pressures of 2,000 and 3,000 psf, respectively. A modulus of subgrade reaction value of $k = 100$ pounds per cubic inch (pci) can be used for slabs on fill or expansive soil.

Estimated mat or PT slab settlements are as follows (these values do not include edge curling due to expansive soil conditions, as discussed previously in this section):

1) Mat foundations and PT slabs with net load < 500psf

- Maximum estimated average settlement - $\frac{1}{2}$ " (unless greater deflections are calculated based on structural properties and a modulus of subgrade reaction of 100 pci)
- Maximum estimated differential settlement over a span of 25 feet - $\frac{1}{2}$ inch (unless greater deflections are calculated based on structural properties and a modulus of subgrade reaction of 100 pci)

An allowable passive pressure of 250 pcf (equivalent fluid pressure) may be used to resist lateral forces. This value includes a factor of safety of at least 1.5. If the mat is formed against unimproved soil, then passive pressure should be neglected in the upper three feet. If the mat is backfilled with select material (non-expansive) extending a minimum of five feet beyond the edge of the mat, then passive pressures can extend to within one foot of the exterior grade (if landscaped), or to the ground surface (if covered by pavement or a concrete slab). The coefficient of sliding friction on the bottom of the slab should be 0.30 in areas where there is no underlying plastic membrane or similar vapor protection. This is an allowable value and includes a factor of safety of at least 1.5. Passive pressure and skin friction on the bottom of the slab can be combined without reduction.

We recommend a minimum slab thickness of 10 inches for post-tensioned slabs. Underslab moisture/vapor control is discussed in a later section of this report. However, we point out that, because the post-tensioned mat slabs are much thicker than conventional slabs, the moisture dissipation will take longer than that of thin slabs. With a typical construction schedule, the

moisture content of the slab prior to the placement of the flooring may exceed that specified. Therefore, measures should generally be taken to evaluate whether the moisture, prior to placement of the flooring, is below that specified by the flooring manufacturer. Actual test results should generally be obtained, checked for acceptability by the contractor, and kept in the project construction files. Some designers prefer to use a “turned-down” edge of slab to reduce moisture infiltration; although this is not necessary in our design, it can provide an additional level of protection.

9.5.4 Under-Slab Moisture Vapor

Subsurface moisture and moisture vapor naturally migrate upward through soil and, where the soil is covered by a building or pavement, this subsurface moisture will collect. To reduce the impact of this subsurface moisture and the potential impact of moisture that could be introduced in the future (such as landscape irrigation, precipitation or leaking pipes) the current industry standard is to provide a vapor retardant membrane. This membrane typically consists of visqueen or polyvinyl plastic sheeting at least-10 mil in thickness. It should be noted that although vapor barrier systems are currently the industry standard, this system may not be completely effective in preventing floor slab moisture problems. These systems typically will not necessarily assure that floor slab moisture transmission rates will meet floor-covering manufacturer standards and that indoor humidity levels be appropriate to inhibit mold growth. The design and construction of such systems are totally dependent on the proposed use and design of the proposed building and elements of building design and function should be considered in the slab-on-grade floor design. Building design and construction may have a greater role in perceived moisture problems since sealed buildings/rooms or inadequate ventilation may produce excessive moisture in a building and affect indoor air quality.

Various factors such as surface grades, adjacent planters, the quality of slab concrete and the permeability of the on-site soils affect slab moisture can control future performance. In many cases, floor moisture problems are the result of either improper curing of floor slabs or improper application of flooring adhesives. We recommend contacting a flooring consultant experienced in the area of concrete, slab-on-grade floors for specific recommendations regarding your proposed flooring applications.

Special precautions must be taken during the placement and curing of concrete slabs. Excessive slump (high water-cement ratio) of the concrete and/or improper curing procedures used during either hot or cold weather conditions could lead to excessive shrinkage, cracking or curling of the slabs. High water-cement ratio and/or improper curing also greatly increase the water vapor permeability of concrete. We recommend that concrete placement and curing operations be performed in accordance with the American Concrete Institute (ACI) Manual.

It should be emphasized that Kleinfelder personnel are not moisture proofing experts for floors. We make no guarantee, nor provide any assurance, that use of the capillary break/vapor retardant system will reduce concrete slab-on-grade floor moisture penetration to any specific rate or level, particularly those required by floor covering manufacturers. The builder and designers should consider all available measures for slab moisture protection and should consult with the project architect for appropriate moisture barrier design.

Because the post-tensioned mat slabs are much thicker than conventional slabs, the moisture dissipation will take longer than that of thin slabs. With a typical construction schedule, the moisture content of the slab prior to the placement of the flooring may exceed that specified. Therefore, measures should generally be taken to evaluate whether the moisture, prior to placement of the flooring, is below that specified by the flooring manufacturer. Actual test results should generally be obtained, checked for acceptability by the contractor and kept in the project construction files.

9.6 RETAINING WALLS

Currently, we are unaware of any proposed retaining walls for this site. If walls are required, Kleinfelder should be contacted for specific design recommendations. General retaining wall parameters are presented below.

As indicated in the grading section of this report, cut or fill slopes higher than 5 feet should not generally be inclined steeper than 3H:1V above the top of a wall. Retaining walls that are free to rotate slightly and support a level back-slope should be, in general, designed to resist an active equivalent fluid weight of 40 pounds per cubic foot (pcf) acting in a triangular pressure distribution. For slopes inclined up to 3H:1V, walls should be designed for an equivalent fluid weight of 60 pcf. If walls are constrained at the top and cannot tilt, the pressures are higher and walls should be

designed for “at-rest” equivalent fluid weights of 60 pcf and 80 pcf, respectively (triangular distribution). For intermediate slope inclinations, the equivalent fluid weights for wall design can be interpolated between these values. At building corners, the walls should be designed for “at-rest” pressures for a distance equal to the wall height away from the corner. We can assist in evaluating the effects of possible wall surcharges, such as where traffic loading or steeper back slopes are anticipated. The pressures presented above represent our best estimate of actual pressures that may develop against a retaining wall and **do not include a factor of safety. The above pressures also assume that hydrostatic pressures will not develop behind walls.**

To help reduce lateral pressures against retaining walls as a result of expansive soil swelling, we recommend that retaining wall backfill consist of granular, angular material that is low in expansion potential for a horizontal distance equal to one-half the wall height. Unless designed to resist the additional hydrostatic pressure, walls should be drained in accordance with the Retaining Wall Backfill and Drainage Detail shown on Figure 6. Retaining wall backfill should be observed, tested and approved by the Geotechnical Engineer/Engineering Geologist.

Where migration of moisture through retaining walls would be detrimental to the intended interior use, retaining walls should be treated in some manner so as to be water-resistant or waterproof. It should be emphasized that Kleinfelder representatives are not moisture-proofing experts. The project Architect or Civil Engineer should be consulted for specific moisture-proofing recommendations. The use of weep-holes to drain the walls (Plate 6) assumes that water collecting at the wall front will not be detrimental to the function of this area. Retaining walls will yield slightly during backfilling and, therefore, should be backfilled prior to building on or adjacent to the walls. Retaining walls can be supported on spread footings in accordance with recommendations previously presented in Section 9.5 of this report. However, if they are not underlain by select fill per Section 9.2, these spread footings should be founded at least 36 inches below the lowest adjacent grade.

9.7 CONCRETE SLABS-ON-GRADE

Other than the aforementioned interior slabs, concrete slabs-on-grade for this project will most likely consist of exterior flatwork. On-site soils will provide sufficient support for the concrete slabs-on-grade where prepared as previously recommended in this report. However, to minimize seasonal fluctuations in moisture content and, thus, reduce the potential for swelling, planned slabs beyond the building envelopes could be underlain by a layer at least 18 inches in thickness of select fill of low expansion potential; relatively granular material such as soil meeting the specifications for imported fill presented in Section 9.2 of this report, or; State of California Department of Transportation (Caltrans) Class 2 Aggregate Base. Such fill should extend laterally at least 3 feet beyond the edges of the concrete slab or flatwork. Even with an 18-inch-thick layer of such material, some differential movement as a result of seasonal soil heave/shrink cycles should be expected (on the order of approximately one inch between adjacent slabs and/or scores) in construction areas underlain by expansive material. Alternatively, perimeter moisture cutoffs (e.g., 4-inch-wide trenches filled with lean concrete) extending at least 40 inches below lowest adjacent grade to act as a barrier to moisture gain-loss beneath the exterior flatwork may be used to minimize heave. If installed in conjunction with properly prepared slab subgrade soils, landscape irrigation around the flatwork perimeter can be effective in minimizing shrink-swell movements; however, this involves a greater risk and requires continual monitoring to regulate soil moisture.

We recommend that exterior concrete slabs be a minimum of four inches thick and be reinforced according to the recommendations set forth by the structural designer. During construction, care should be taken such that reinforcement is placed at the slab mid-height, particularly when using welded-wire fabric.

9.8 ASPHALT CONCRETE PAVEMENT SECTIONS

Pavement for this project will consist primarily of new asphaltic concrete (AC) paved driveways and parking. Our pavement thickness recommendations are based on the assumption that the pavement subgrade soils may consist of moderate to highly plastic soil. Based on our experience with soils of this nature, a Resistance (R-) Value of 5 was chosen as being representative. This value, in addition to Traffic Indices (T.I.s) ranging from 4 to 8, and the State of California Department of Transportation (Caltrans) Flexible Pavement Design Method including Class 2 Aggregate Base (AB) was used to provide the following recommended pavement sections:

ASPHALT CONCRETE PAVEMENT DESIGN R-VALUE = 5		
	Pavement Section (inches)	
T.I.	AC	AB
4	2.5	9.0
5	3.0	10.0
6	3.0	14.0
7	4.5	15.0
8	5.0	18.0
AC = Type B Asphalt Concrete AB = Class 2 Aggregate Base (Minimum R-Value = 78)		

The above thicknesses for the AC and AB and corresponding T.I.s should be checked by the project Civil Engineer for their applicability prior to final design and use. If desired, we can evaluate final design pavement sections once rough grading is near completion by performing R-value tests at more specific subgrade locations. This may affect construction sequencing for one to two weeks until laboratory testing is completed. Earthwork quantities may also be affected depending on the actual R-value results and the possible revised pavement subgrade elevations.

Prior to subgrade preparation, utility trench backfills should be properly placed and compacted. The upper six inches of subgrade should be re-rolled to provide a smooth, unyielding surface, compacted to between 90 and 92 percent relative compaction for native expansive soils (at a moisture content at least 4 percent above optimum to cause expansion to occur prior to capping with permanent construction) and to at least 95 percent for select import material of low expansion potential. The subgrade soils should be maintained in a moist condition and free of shrinkage cracks until covered with the complete pavement section. Because of the high potential for soil expansion in the subgrade soil during seasonal moisture changes, a barrier at pavement edges (to reduce potential detrimental swelling in pavement subgrade soils) should be considered. The actual need for moisture barriers can be further evaluated during future planning and or when rough subgrade is achieved.

Class 2 Aggregate Base should conform to the requirements of the City of Healdsburg (City) and/or Section 26, Caltrans Standard Specifications (latest edition), as applicable. Aggregate base should be placed in thin lifts in a manner to prevent segregation, uniformly moisture conditioned, and compacted to at least 95 percent relative compaction to provide a smooth, unyielding surface. The asphalt concrete surfacing should conform to the quality requirements of the City and/or Caltrans Standard Specifications (latest edition), as applicable.

Considering the size of the project and the predominantly highly expansive nature of the soils encountered in the surface and near-surface soil zone, it may be prudent to consider lime-treatment of pavement subgrade soils and/or building pads. Lime treatment improves the strength of the subgrade soils and will likely reduce the thickness of the aggregate base layer of the pavement section design. Lime-treatment also tends to reduce the potential for swelling due to moisture variation in the expansive soil subgrade, and may eliminate the need for moisture barriers. A lime-treatment testing program can be provided by Kleinfelder at an additional fee if requested. Typical lime treatment consists of high-calcium quick lime in the range of 4 to 6 pounds per cubic foot of soil treated.

9.9 SURFACE AND SUBSURFACE DRAINAGE CONTROL

We recommend improving surface drainage throughout the site. Drainage improvements should be installed upslope of improvement areas to collect and divert surface water off slopes to planned drainage facilities. Improvements should consist of a series of concrete-lined V-ditches, drop-

inlets or other drainage control features. Site drainage systems should be designed by the project Civil Engineer.

We anticipate that improvements (such as storm drain trenches) will mitigate a majority of the subsurface water that seasonally and locally exists on the site. Accordingly, the need to provide intermittent inlets from trench backfill into storm drain facilities should be recognized. However, we anticipate that an interceptor subdrain could be needed locally on the site. The need for interceptor subdrains should be evaluated in the field during grading when actual conditions are exposed. We recommend that a budget be established to allow for intermittent placement of subdrains during grading.

It is important that the area adjacent to structures be sloped so as to provide positive surface drainage away from structures. Grades should be sloped a minimum of 5% extending at least 10 feet out from the buildings. The roofs should be provided with gutters and downspouts that discharge into a closed pipe system with outflow into an appropriate drainage facility.

With a drilled pier-and-grade-beam foundation (where used), there is a potential for surface and subsurface water to seep under grade beams and collect in under-floor areas. To help reduce such seepage, foundation drains should be provided adjacent to the upslope perimeter grade beams and possibly at intermediate grade beam locations. Foundation drains should consist of trenches at least 18 inches deep (and extending at least 8 inches below the bottom of the grade beam) that are sloped to drain to outlets by gravity. The side of the trench adjacent to the grade beam should be lined with an impermeable membrane (10-mil visqueen or equivalent). Three-inch-diameter perforated, rigid plastic pipe (SDR 35), sloped to drain, should be placed in the bottom of the trenches on a bed of drain rock (Caltrans Class 2 Permeable). The trenches then should be backfilled to within 8 inches of the ground surface and at least 4 inches above the bottom of the grade beam with drain rock. The upper 8 inches should then be backfilled with compacted clayey soils to inhibit surface water infiltration. Where pier-and-grade-beam foundations are used on relatively flat pads, the soils beneath the structure should be shaped to drain to surface drains that collect under house water and discharge it to an appropriate drainage facility by a closed pipe system.

Where irrigated landscape areas abut structures, excess water can be introduced into soil layers along building edges, tending to soften soils and increase the risk of potential migration of moisture into under-floor areas. Planned landscaping may need surface and subsurface drainage improvements. We can evaluate the need for such drainage facilities during final design, if requested.

It should be recognized that concrete curbs, sidewalks and mowing strips, and header boards and raised berms can impede the flow of surface water away from buildings, promote soil saturation and contribute to seepage of water into under-slab rock. Where such landscaping elements are planned, surface and subsurface drainage features may need to be incorporated into the plans. We can develop specific recommendations, if desired.

9.10 UTILITIES

It is our understanding that utilities/utility services for this site will be provided by the City, most likely accessing the site from Healdsburg Avenue and through the proposed roadways for this project. If utilities do cross the active faults on this site, they should be outfitted with shut off valves on either side of the fault zone, flexible connections and/or other engineering solutions to reduce the potential for damage or rupture of utility lines in this part of the site. Where utility trenches extend from the exterior to the interior limits of a building, native clayey soils or lean concrete should be used as backfill material for a distance of 2 feet laterally on each side of the exterior building line to reduce the probability of the trench from acting as a conduit for groundwater flow.

9.10.1 Trench Backfill

Backfill should consist of either approved on-site excavation or approved granular import material. Subsequent backfill should be free from organics and other deleterious substances, and be of such size (gradation) to allow relatively uniform compaction to the specified relative compaction. Rocks larger than 6 inches in greatest dimension will not be permitted as subsequent backfill.

Groundwater entering trenches at the time of excavation should be removed by positive means to a controlled outlet as approved by the Geotechnical Engineer/Engineering Geologist.

Kleinfelder should observe and periodically test the backfill during the underground construction to assess that the work is constructed in compliance with the recommendations presented herein.

9.10.2 Trench Cutoffs

If utilities are sloped 14 percent and steeper, we recommend a cutoff structure be constructed every 150 lineal feet in order to intercept seepage and discharge it to an appropriate location. The trench cutoff should consist of a minimum two-foot thick barrier consisting of low permeability material extending 6 inches below the bottom of the trench, 6 inches beyond the sides of the trench, and one foot above the pipe bedding into low permeability trench backfill. Typically, such a barrier is constructed of concrete or low permeability controlled density fill, although other materials can be considered. A drain should be constructed at the base of the trench on the uphill side of the cutoff to prevent the buildup of water in the pipe bedding. The drain should consist of a 2-foot long section of 3-inch perforated PVC pipe surrounded with 3 cubic feet of Caltrans Class 2 permeable material or $\frac{3}{4}$ inch drain rock wrapped in filter fabric. The drain should be connected to a 3-inch solid PVC pipe that flows under gravity to a storm drain or daylights to the slope face with energy dissipaters as needed.

In utilities sloped 8 to 14 percent, we do not feel it is necessary to construct additional barriers in the pipe bedding material. However, we do feel it is necessary to provide conduits for the release of water from the pipe bedding where the pipe bedding is interrupted (such as where a manhole had been set with a concrete plug). In such cases we recommend a drain be provided uphill of where the bedding material is interrupted. In the case of storm drain manholes, weep holes may be constructed directly into the manhole. Weep holes should be a minimum of 1½ inches in diameter and should be covered with filter fabric on the outside of the manhole.

9.10.3 Bedding Material

Bedding material should be placed below the pipe invert as well as around the pipe and above the pipe to provide support for the pipe and protect the pipe during backfill compaction. The bedding should be placed according to the pipe manufacturer's recommendations and as required by the local governing agency. As a minimum, we recommend there be at least 6 inches of bedding below the pipe and a minimum of 12 inches above the pipe. The bedding material should be rodded around the pipe to densify the material by reducing the voids around the pipe. Bedding material should be free of organic material, lumps or balls of clay, or other deleterious material. The bedding material should have a sand equivalent of 30, a durability index of 25 and conform to the following gradation:

Sieve Size	Percent Passing (by Weight)
#4 Sieve	90 - 100
#200 Sieve	Less than 15

9.10.4 Trench Stability

Trenches up to 5 feet deep without groundwater are expected to be stable for temporary work. Deeper trenches should be shored. The underground contractor is responsible for utility excavation safety and shoring in accordance with the current OSHA Health and Safety Standards for Excavations (29 CFR, Part 1926).

9.10.5 Pipe Thrust Resistance

At locations where the pipeline changes direction, or at valves or other attachments, fluid forces will develop within the pipe. In such cases, it may be necessary or beneficial to resist these forces externally through soil friction and thrust blocks. Soil friction can be calculated using a coefficient of friction of 0.3 between steel or concrete pipes and the surrounding soil. If plastic pipe, plastic or epoxy coated pipe or pipe wrapped in a protective coating are used the coefficient of friction should be reduced to 0.2. The pressure between the soil and the pipe can be computed by assuming a unit weight of 100 pcf above the groundwater and 40 pcf below the groundwater.

Thrust blocks are blocks of concrete attached or adjacent to the pipe to provide additional resistance to thrust forces. The resistance capacity of thrust blocks can be calculated using a combination of lateral bearing pressure and friction between the soil and the concrete. Thrust blocks should extend a minimum of one foot below the invert of the pipe and should have a minimum dimension of 2 feet. The top of the thrust block should be a minimum of 2 feet below the ground surface. Based on the above criteria, we recommend thrust blocks in the various reaches be designed using a friction coefficient of 0.25 and a lateral bearing pressure equal to the passive pressures provided previously in the foundations section of this report.

10.0 ADDITIONAL SERVICES AND LIMITATIONS

Additional information on subsurface conditions at the site will become available during subsequent project construction. As such, Kleinfelder's review of project plans and specifications, along with field observation and testing during project construction, are an integral part of the conclusions and recommendations made in this report. If Kleinfelder is not retained to provide these services, we will cease to be the Geotechnical-Engineer-of-Record.

We have provided Comstock, Crosser & Associates Development Company, Inc. one electronic copy of this report. If hard copies are required, we can provide them at an additional fee (in accordance with our current fee schedule) after receipt of a written request from our Client. **Under no circumstances will we provide a copy of the report to other design consultants or Contractors without written permission from our Client.**

The recommendations contained in this report are subject to the limitations presented herein. In addition, a brochure prepared by GBC (Geotechnical Business Council) has been included Appendix D. We recommend that all individuals reading this report also read this brochure.

This work was performed in a manner consistent with that level of care and skill ordinarily exercised by other members of Kleinfelder's profession practicing in the same locality, under similar conditions and at the date the services are provided. Our conclusions, opinions and recommendations are based on a limited number of observations and data. It is possible that conditions could vary between or beyond the data evaluated. Kleinfelder makes no other representation, guarantee or warranty, express or implied, regarding the services, communication (oral or written), report, opinion, or instrument of service provided.

This report may be used only by the Client and the registered design professional in responsible charge and only for the purposes stated for this specific engagement within a reasonable time from its issuance, but in no event later than two (2) years from the date of the report.

The scope of services was limited to conceptual commercial improvements only. Recommendations are subject to change during future planning stages and prior to finalization of development plans. It should be recognized that definition and evaluation of subsurface conditions are difficult. Judgments leading to conclusions and recommendations are generally made with incomplete knowledge of the subsurface conditions present due to the limitations of data from field studies. The conclusions and recommendations contained in this report are based on fourteen (14) soil test borings (maximum depth of 15 feet), limited laboratory testing, engineering analysis, review of our previous work on this site and our experience in the project area.

Kleinfelder offers various levels of investigative and engineering services to suit the varying needs of different clients. Although risk can never be eliminated, more detailed and extensive studies yield more information, which may help understand and manage the level of risk. Since detailed study and analysis involves greater expense, our clients participate in determining levels of service, which provide information for their purposes at acceptable levels of risk. The client and key members of the design team should discuss the issues covered in this report with Kleinfelder, so that the issues are understood and applied in a manner consistent with the owner's budget, tolerance of risk, and expectations for future performance and maintenance.

Recommendations contained in this report are based on our field observations and subsurface explorations, limited laboratory tests, and our present knowledge of the proposed construction. It is possible that soil, rock or groundwater conditions could vary between or beyond the points explored. If soil, rock or groundwater conditions are encountered during construction that differ from those described herein, the client is responsible for ensuring that Kleinfelder is notified immediately so that we may reevaluate the recommendations of this report. If the scope of the proposed construction, including the estimated building loads, and the design depths or locations of the foundations, changes from that described in this report, the conclusions and recommendations contained in this report are not considered valid unless the changes are reviewed, and the conclusions of this report are modified or approved in writing, by Kleinfelder.

As the geotechnical engineering firm that performed the geotechnical evaluation for this project, Kleinfelder should be retained to confirm that the recommendations of this report are properly incorporated in the design of this project, and properly implemented during construction. This may avoid misinterpretation of the information by other parties and will allow us to review and modify

our recommendations if variations in the soil conditions are encountered. As a minimum Kleinfelder should be retained to provide the following continuing services for the project:

- Review the project plans and specifications, including any revisions or modifications;
- Observe and evaluate the site earthwork operations to confirm subgrade soils are suitable for construction of foundations, slabs-on-grade, pavements and placement of engineered fill;
- Confirm engineered fill for the structures and other improvements is placed and compacted per the project specifications; and
- Observe foundation bearing soils to confirm conditions are as anticipated.
- Construction Special Inspection

Kleinfelder cannot be responsible for interpretation by others of this report or the conditions encountered in the field.

Kleinfelder must be retained so that all geotechnical aspects of construction will be monitored on a part-time to full-time basis (as required) by a representative from Kleinfelder, including site preparation, preparation of foundations, installation of piles, and placement of engineered fill and trench backfill. These services provide Kleinfelder the opportunity to observe the actual soil, rock and groundwater conditions encountered during construction and to evaluate the applicability of the recommendations presented in this report to the site conditions. If Kleinfelder is not retained to provide these services, we will cease to be the engineer of record for this project and will assume no responsibility for any potential claim during or after construction on this project. If changed site conditions affect the recommendations presented herein, Kleinfelder must also be retained to perform a supplemental evaluation and to issue a revision to our original report.

This report, and any future addenda or reports regarding this site, may be made available to bidders to supply them with only the data contained in the report regarding subsurface conditions and laboratory test results at the point and time noted. Bidders may not rely on interpretations, opinion, recommendations, or conclusions contained in the report. Because of the limited nature of any subsurface study, the contractor may encounter conditions during construction which differ from those presented in this report. In such event, the contractor should promptly notify the owner so that Kleinfelder's geotechnical engineer can be contacted to confirm those conditions. We recommend the contractor describe the nature and extent of the differing conditions in writing and

that the construction contract include provisions for dealing with differing conditions. Contingency funds should be reserved for potential problems during earthwork and foundation construction. Furthermore, the contractor should be prepared to handle contamination conditions encountered at this site, which may affect the excavation, removal, or disposal of soil; dewatering of excavations; and health and safety of workers.

The scope of services for this subsurface exploration and geotechnical report did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil, surface water, or groundwater at this site. This study did not include an assessment of NOAM-bearing deposits or rock.

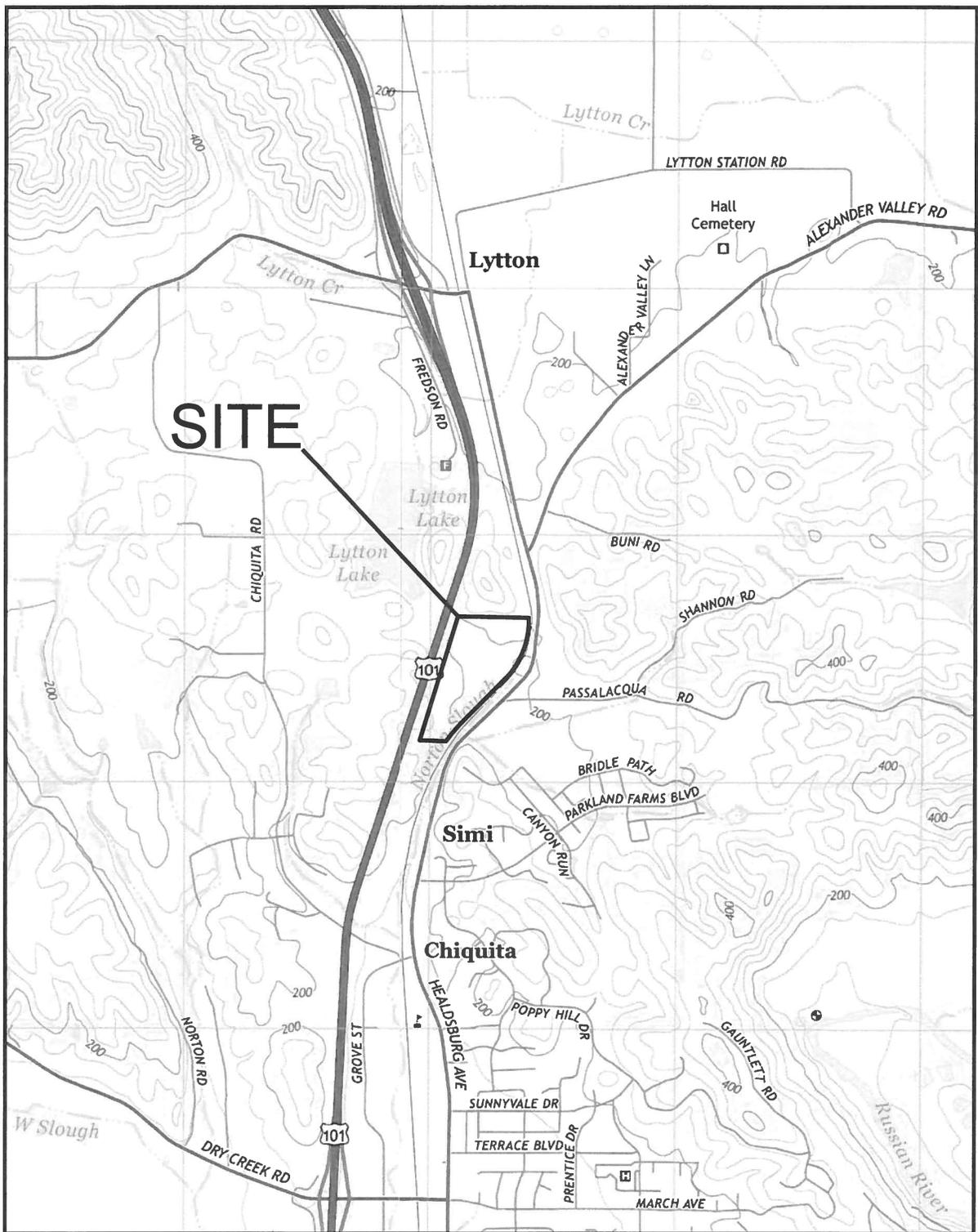
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Reference: USGS, 2015

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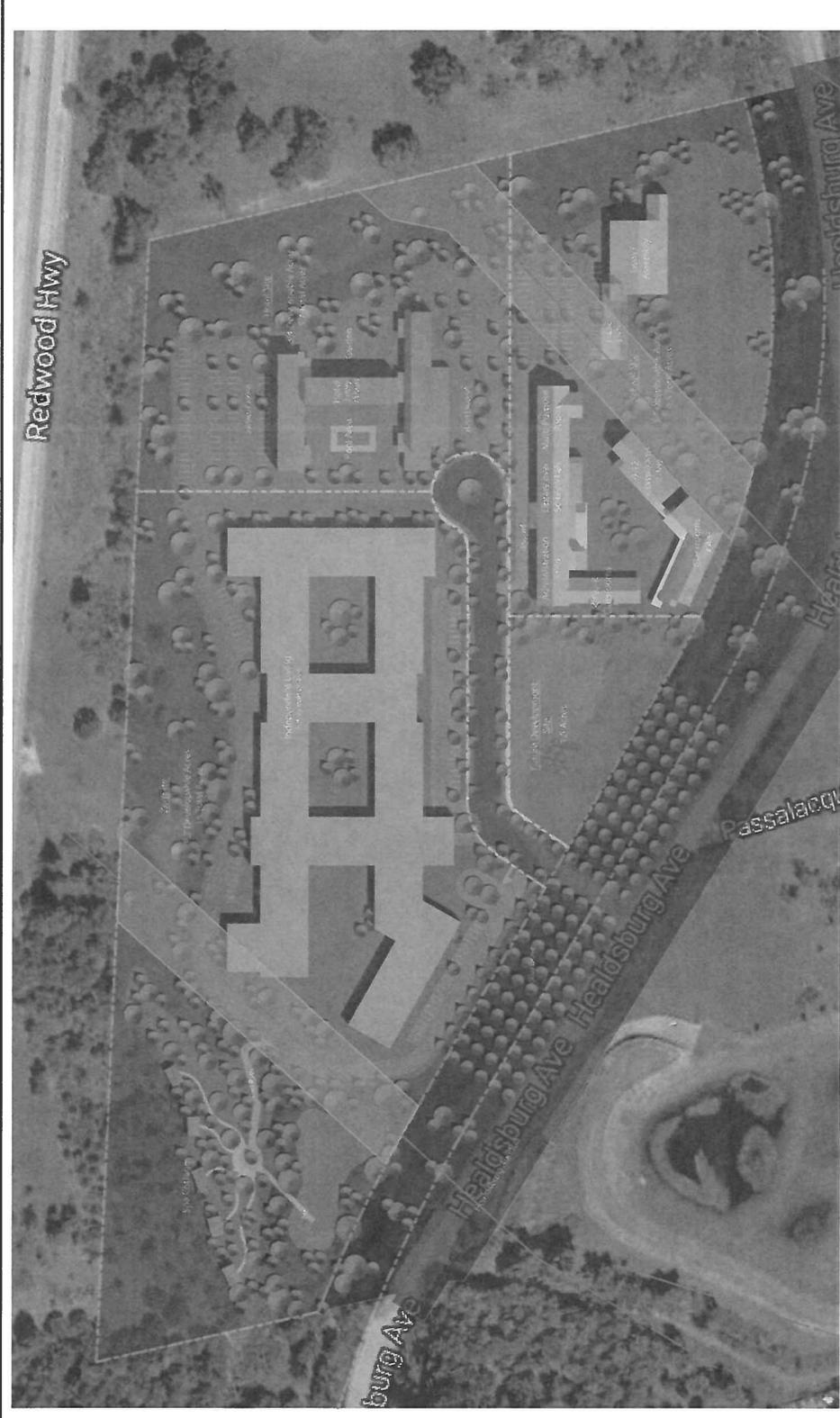


PROJECT NO.	20181540
DRAWN	SEP 2017
DRAWN BY	JCR
CHECKED BY	WVM
FILE NAME	Site Location.ai

SITE LOCATION

QUAKER HILL 32 ACRE PARCEL
HEALDSBURG, CALIFORNIA

FIGURE
1



COMSTOCK HOMES
Healdsburg Quaker Site
 Master Plan
 4/20/17

D I C E C C O
ARCHITECTURE
C O R P O R A T E D
 887 PATRIOT DRIVE, SUITE C, MOORPARK, CALIFORNIA 93011
 805.552.2088
 DICCCOARCH.COM

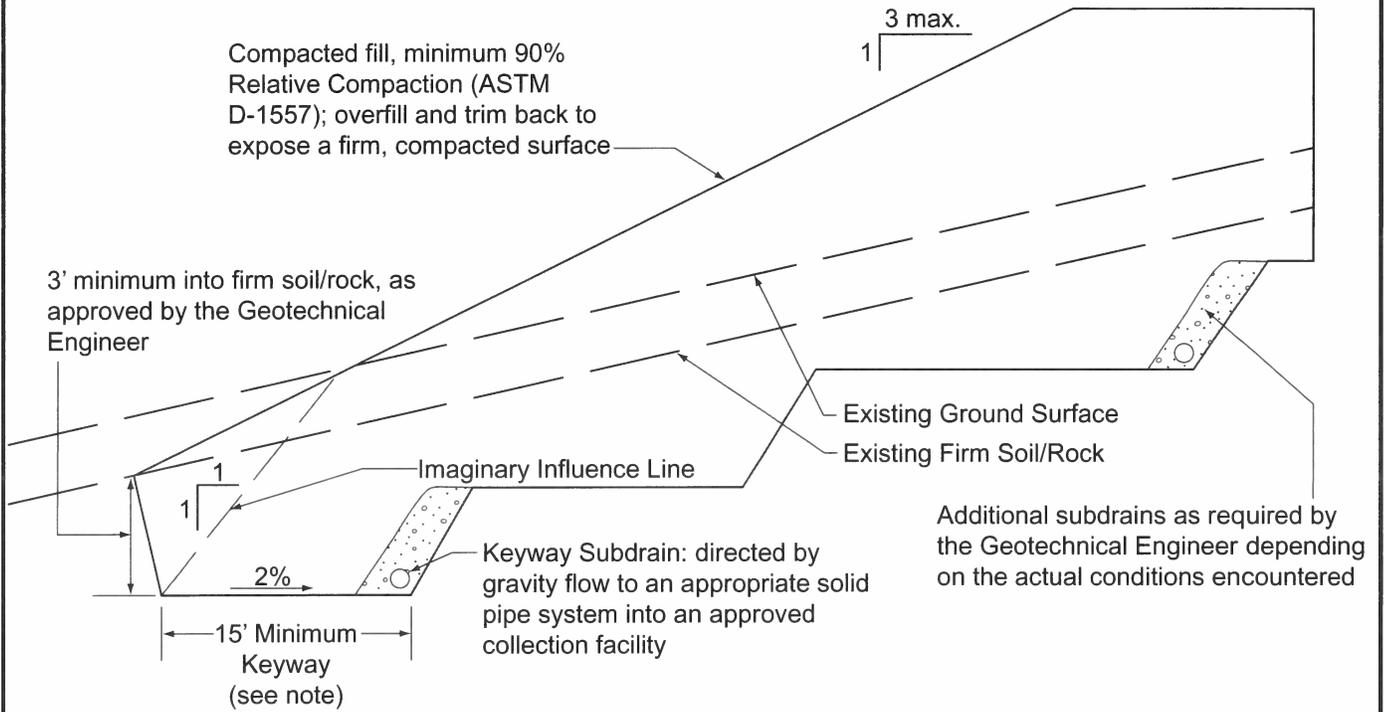
PROJECT NO. 20181540		DRAWN BY JOR		CONCEPTUAL DEVELOPMENT PLAN QUAKER HILL 32 ACRE PARCEL HEALDSBURG, CALIFORNIA
DRAWN BY JOR		CHECKED BY WJM		
FILE NAME		Conceptual Development.Lai		FIGURE 2


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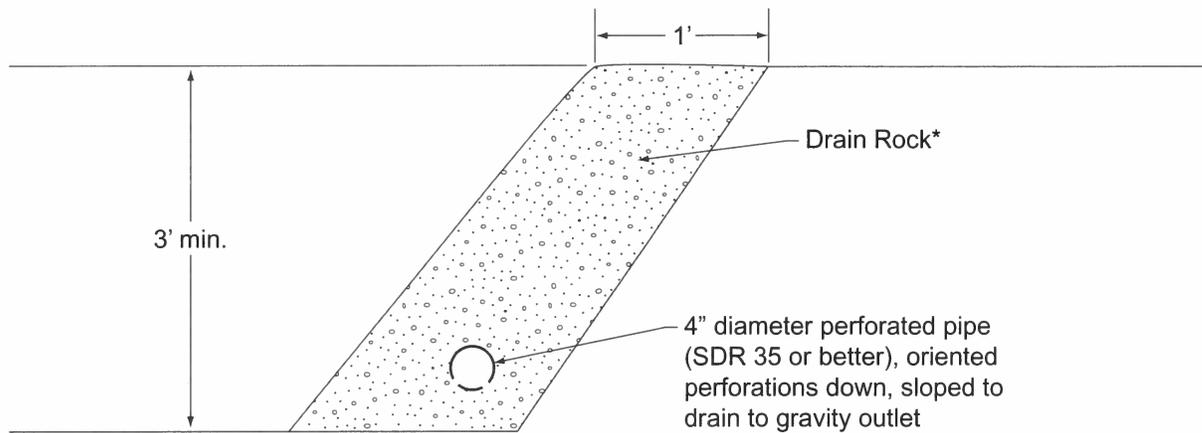
Surveyed Fault Locations (red) and Recommended Building Setback Zones (orange) from Kleinfelder, 2003

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Note: Keyway excavation and subdrain installation should be observed by the Geotechnical Engineer; where existing slope exceeds 6H:1V (Horizontal:Vertical), excavate series of benches into firm soil/rock, as indicated below.



Typical Fill Section - Keyway Construction
(not to scale)



Subdrain Detail
(not to scale)

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* Drain rock should be clean, free draining and meet the requirements for Class 2 permeable material Section 68 of State of California "Caltrans" Standard Specifications, or 0.75" diameter crushed rock encapsulated in filter fabric, such as Mirafi 140n



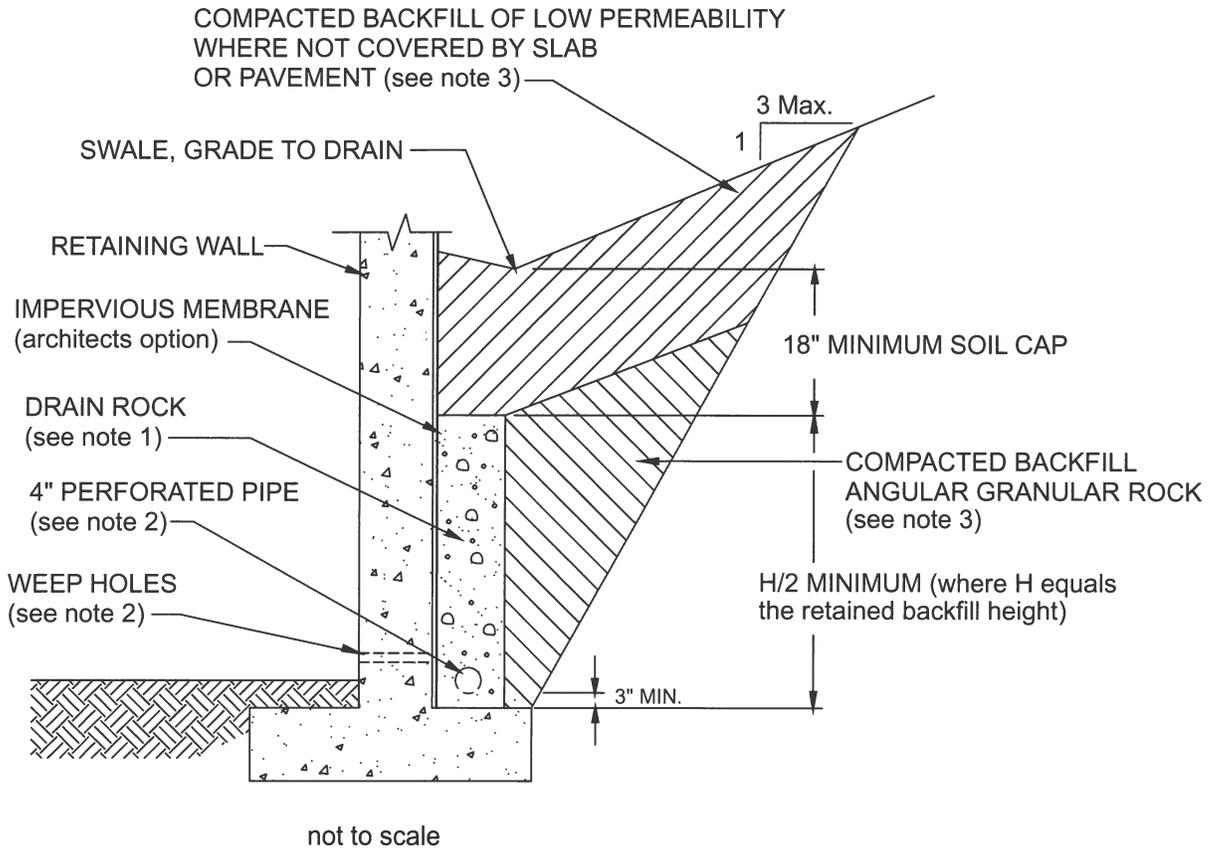
PROJECT NO.	20181540
DRAWN	SEP 2017
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CHECKED BY	WVM
FILE NAME	Site Location.ai

TYPICAL FILL SLOPE AND SUBDRAIN DETAILS

QUAKER HILL 32 ACRE PARCEL
HEALDSBURG, CALIFORNIA

FIGURE

5



NOTES:

1. Drain rock should be clean, free draining 3/4 inch crushed rock or gravel wrapped in filter fabric (Mirafi 140N or equivalent) or Class 2 Permeable Material, Section 68, State of California "Caltrans" Standard Specification, latest edition. Alternatively, a pre-fabricated drainage structure (Miradrain or equivalent) installed to the manufacturers recommendations, may be used in lieu of drainrock and fabric.
2. Pipe should conform to the requirements of Section 68 of the Caltrans Standard Specifications (minimum SDR 35), perforations placed down, sloped at 1% for gravity flow to outlet or sump with automatic pump. In addition, drainage outlet can be provided through 3" diameter weep holes spaced approximately 10' to 15' apart.
3. During compaction, the contractor should use appropriate methods (such as temporary bracing and/or light compaction equipment) to avoid overstressing the walls.

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 <p>KLEINFELDER Bright People. Right Solutions. www.kleinfelder.com</p>	PROJECT NO. 20181540	<p>RETAINING WALL BACKFILL AND DRAINAGE DETAIL</p>	<p>FIGURE</p> <p>6</p>
	DRAWN SEP 2017		
	DRAWN BY JCR	<p>QUAKER HILL 32 ACRE PARCEL HEALDSBURG, CALIFORNIA</p>	
	CHECKED BY WVM		
	FILE NAME Site Location.ai		



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SAMPLER AND DRILLING METHOD GRAPHICS

	BULK / GRAB / BAG SAMPLE
	MODIFIED CALIFORNIA SAMPLER (2 or 2-1/2 in. (50.8 or 63.5 mm.) outer diameter)
	CALIFORNIA SAMPLER (3 in. (76.2 mm.) outer diameter)
	STANDARD PENETRATION SPLIT SPOON SAMPLER (2 in. (50.8 mm.) outer diameter and 1-3/8 in. (34.9 mm.) inner diameter)
	SHELBY TUBE SAMPLER
	HOLLOW STEM AUGER
	SOLID STEM AUGER
	WASH BORING

GROUND WATER GRAPHICS

	WATER LEVEL (level where first observed)
	WATER LEVEL (level after exploration completion)
	WATER LEVEL (additional levels after exploration)
	OBSERVED SEEPAGE

NOTES

- The report and graphics key are an integral part of these logs. All data and interpretations in this log are subject to the explanations and limitations stated in the report.
- Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual or differ from those shown.
- No warranty is provided as to the continuity of soil or rock conditions between individual sample locations.
- Logs represent general soil or rock conditions observed at the point of exploration on the date indicated.
- In general, Unified Soil Classification System designations presented on the logs were based on visual classification in the field and were modified where appropriate based on gradation and index property testing.
- Fine grained soils that plot within the hatched area on the Plasticity Chart, and coarse grained soils with between 5% and 12% passing the No. 200 sieve require dual USCS symbols, i.e., GW-GM, GP-GM, GW-GC, GP-GC, GC-GM, SW-SM, SP-SM, SW-SC, SP-SC, SC-SM.
- If sampler is not able to be driven at least 6 inches then 50/X indicates number of blows required to drive the identified sampler X inches with a 140 pound hammer falling 30 inches.

ABBREVIATIONS

WOH - Weight of Hammer
WOR - Weight of Rod

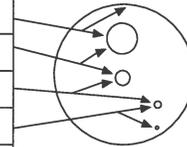
UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

GRAVELS (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVEL WITH <5% FINES	Cu ≥ 4 and 1 ≤ Cc ≤ 3		GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		Cu < 4 and/or 1 > Cc > 3		GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
	GRAVELS WITH 5% TO 12% FINES	Cu ≥ 4 and 1 ≤ Cc ≤ 3		GW-GM	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GW-GC	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
		Cu < 4 and/or 1 > Cc > 3		GP-GM	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GP-GC	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
	GRAVELS WITH > 12% FINES			GM	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
				GC-GM	CLAYEY GRAVELS, GRAVEL-SAND-CLAY-SILT MIXTURES
	SANDS (More than half of coarse fraction is smaller than the #4 sieve)	CLEAN SANDS WITH <5% FINES	Cu ≥ 6 and 1 ≤ Cc ≤ 3		SW
		Cu < 6 and/or 1 > Cc > 3		SP	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
SANDS WITH 5% TO 12% FINES		Cu ≥ 6 and 1 ≤ Cc ≤ 3		SW-SM	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SW-SC	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
		Cu < 6 and/or 1 > Cc > 3		SP-SM	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SP-SC	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
SANDS WITH > 12% FINES				SM	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-GRAVEL-CLAY MIXTURES
				SC-SM	CLAYEY SANDS, SAND-SILT-CLAY MIXTURES
FINE GRAINED SOILS (More than half of material is smaller than the #200 sieve)		SILTS AND CLAYS (Liquid Limit less than 50)			ML
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				CL-ML	INORGANIC CLAYS-SILTS OF LOW PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
	SILTS AND CLAYS (Liquid Limit greater than 50)			OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY
				MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT
				CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
		OH	ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY		

 Bright People. Right Solutions.	PROJECT NO.: 20181540	GRAPHICS KEY Quaker Hill 32 Acre Parcel Healdsburg, California	FIGURE
	DRAWN BY: JDS CHECKED BY: JCR DATE: 9/15/2017 REVISED: -		A-1

GRAIN SIZE

DESCRIPTION	SIEVE SIZE	GRAIN SIZE	APPROXIMATE SIZE
Boulders	>12 in. (304.8 mm.)	>12 in. (304.8 mm.)	Larger than basketball-sized
Cobbles	3 - 12 in. (76.2 - 304.8 mm.)	3 - 12 in. (76.2 - 304.8 mm.)	Fist-sized to basketball-sized
Gravel	coarse	3/4 - 3 in. (19 - 76.2 mm.)	Thumb-sized to fist-sized
	fine	#4 - 3/4 in. (#4 - 19 mm.)	Pea-sized to thumb-sized
Sand	coarse	#10 - #4	Rock salt-sized to pea-sized
	medium	#40 - #10	Sugar-sized to rock salt-sized
	fine	#200 - #40	Flour-sized to sugar-sized
Fines	Passing #200	<0.0029 in. (<0.07 mm.)	Flour-sized and smaller



SECONDARY CONSTITUENT

Term of Use	AMOUNT	
	Secondary Constituent is Fine Grained	Secondary Constituent is Coarse Grained
Trace	<5%	<15%
With	≥ 5 to <15%	≥ 15 to <30%
Modifier	≥ 15%	≥ 30%

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

CEMENTATION

DESCRIPTION	FIELD TEST
Weakly	Crumbles or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

CONSISTENCY - FINE-GRAINED SOIL

CONSISTENCY	SPT - N ₆₀ (# blows / ft)	Pocket Pen (tsf)	UNCONFINED COMPRESSIVE STRENGTH (Q _u)(psf)	VISUAL / MANUAL CRITERIA
Very Soft	<2	PP < 0.25	<500	Thumb will penetrate more than 1 inch (25 mm). Extrudes between fingers when squeezed.
Soft	2 - 4	0.25 ≤ PP < 0.5	500 - 1000	Thumb will penetrate soil about 1 inch (25 mm). Remolded by light finger pressure.
Medium Stiff	4 - 8	0.5 ≤ PP < 1	1000 - 2000	Thumb will penetrate soil about 1/4 inch (6 mm). Remolded by strong finger pressure.
Stiff	8 - 15	1 ≤ PP < 2	2000 - 4000	Can be imprinted with considerable pressure from thumb.
Very Stiff	15 - 30	2 ≤ PP < 4	4000 - 8000	Thumb will not indent soil but readily indented with thumbnail.
Hard	>30	4 ≤ PP	>8000	Thumbnail will not indent soil.

FROM TERZAGHI AND PECK, 1948; LAMBE AND WHITMAN, 1969; FHWA, 2002; AND ASTM D2488

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPT-N ₆₀ (# blows/ft)	MODIFIED CA SAMPLER (# blows/ft)	CALIFORNIA SAMPLER (# blows/ft)	RELATIVE DENSITY (%)
Very Loose	<4	<4	<5	0 - 15
Loose	4 - 10	5 - 12	5 - 15	15 - 35
Medium Dense	10 - 30	12 - 35	15 - 40	35 - 65
Dense	30 - 50	35 - 60	40 - 70	65 - 85
Very Dense	>50	>60	>70	85 - 100

FROM TERZAGHI AND PECK, 1948

STRUCTURE

DESCRIPTION	CRITERIA
Stratified	Alternating layers of varying material or color with layers at least 1/4-in. thick, note thickness.
Laminated	Alternating layers of varying material or color with the layer less than 1/4-in. thick, note thickness.
Fissured	Breaks along definite planes of fracture with little resistance to fracturing.
Slickensided	Fracture planes appear polished or glossy, sometimes striated.
Blocky	Cohesive soil that can be broken down into small angular lumps which resist further breakdown.
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay; note thickness.

REACTION WITH HYDROCHLORIC ACID

DESCRIPTION	FIELD TEST
None	No visible reaction
Weak	Some reaction, with bubbles forming slowly
Strong	Violent reaction, with bubbles forming immediately

PLASTICITY

DESCRIPTION	LL	FIELD TEST
Non-plastic	NP	A 1/8-in. (3 mm.) thread cannot be rolled at any water content.
Low (L)	< 30	The thread can barely be rolled and the lump or thread cannot be formed when drier than the plastic limit.
Medium (M)	30 - 50	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump or thread crumbles when drier than the plastic limit.
High (H)	> 50	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump or thread can be formed without crumbling when drier than the plastic limit.

ANGULARITY

DESCRIPTION	CRITERIA
Angular	Particles have sharp edges and relatively plane sides with unpolished surfaces.
Subangular	Particles are similar to angular description but have rounded edges.
Subrounded	Particles have nearly plane sides but have well-rounded corners and edges.
Rounded	Particles have smoothly curved sides and no edges.



PROJECT NO.: 20181540
 DRAWN BY: JDS
 CHECKED BY: JCR
 DATE: 9/15/2017
 REVISED: -

SOIL DESCRIPTION KEY

Quaker Hill 32 Acre Parcel
 Healdsburg, California

FIGURE

A-2

INFILLING TYPE

NAME	ABBR	NAME	ABBR
Albite	Al	Muscovite	Mus
Apatite	Ap	None	No
Biotite	Bi	Pyrite	Py
Clay	Cl	Quartz	Qz
Calcite	Ca	Sand	Sd
Chlorite	Ch	Sericite	Ser
Epidote	Ep	Silt	Si
Iron Oxide	Fe	Talc	Ta
Manganese	Mn	Unknown	Uk

DENSITY/SPACING OF DISCONTINUITIES

DESCRIPTION	SPACING CRITERIA
Unfractured	>6 ft. (>1.83 meters)
Slightly Fractured	2 - 6 ft. (0.061 - 1.83 meters)
Moderately Fractured	8 in - 2 ft. (203.20 - 609.60 mm)
Highly Fractured	2 - 8 in (50.80 - 203.30 mm)
Intensely Fractured	<2 in (<50.80 mm)

ADDITIONAL TEXTURAL ADJECTIVES

DESCRIPTION	RECOGNITION
Pit (Pitted)	Pinhole to 0.03 ft. (3/8 in.) (>1 to 10 mm.) openings
Vug (Vuggy)	Small openings (usually lined with crystals) ranging in diameter from 0.03 ft. (3/8 in.) to 0.33 ft. (4 in.) (10 to 100 mm.)
Cavity	An opening larger than 0.33 ft. (4 in.) (100 mm.), size descriptions are required, and adjectives such as small, large, etc., may be used
Honeycombed	If numerous enough that only thin walls separate individual pits or vugs, this term further describes the preceding nomenclature to indicate cell-like form.
Vesicle (Vesicular)	Small openings in volcanic rocks of variable shape and size formed by entrapped gas bubbles during solidification.

ADDITIONAL TEXTURAL ADJECTIVES

DESCRIPTION	CRITERIA
Unweathered	No evidence of chemical / mechanical alternation; rings with hammer blow.
Slightly Weathered	Slight discoloration on surface; slight alteration along discontinuities; <10% rock volume altered.
Moderately Weathered	Discoloring evident; surface pitted and alteration penetration well below surface; Weathering "halos" evident; 10-50% rock altered.
Highly Weathered	Entire mass discolored; Alteration pervading most rock, some slight weathering pockets; some minerals may be leached out.
Decomposed	Rock reduced to soil with relic rock texture/structure; Generally molded and crumbled by hand.

RELATIVE HARDNESS / STRENGTH DESCRIPTIONS

GRADE	UCS (Mpa)	FIELD TEST	
R0	Extremely Weak	0.25 - 1.0	Indented by thumbnail
R1	Very Weak	1.0 - 5.0	Crumbles under firm blows of geological hammer, can be peeled by a pocket knife.
R2	Weak	5.0 - 25	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer.
R3	Medium Strong	25 - 50	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single firm blow of a geological hammer.
R4	Strong	50 - 100	Specimen requires more than one blow of geological hammer to fracture it.
R5	Very Strong	100 - 250	Specimen requires many blows of geological hammer to fracture it.
R6	Extremely Strong	> 250	Specimen can only be chipped with a geological hammer.

ROCK QUALITY DESIGNATION (RQD)

DESCRIPTION	RQD (%)
Very Poor	0 - 25
Poor	25 - 50
Fair	50 - 75
Good	75 - 90
Excellent	90 - 100

APERTURE

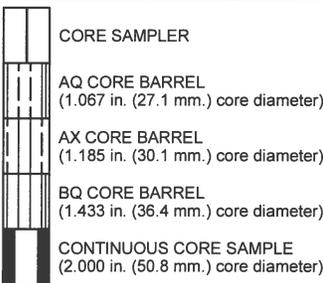
DESCRIPTION	CRITERIA [in (mm)]
Tight	<0.04 (<1)
Open	0.04 - 0.20 (1 - 5)
Wide	>0.20 (>5)

BEDDING CHARACTERISTICS

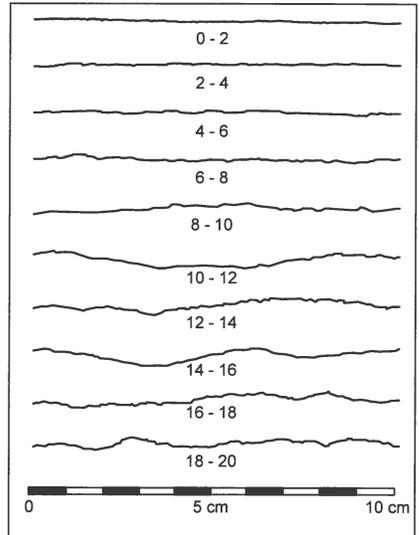
DESCRIPTION	Thickness [in (mm)]
Very Thick Bedded	>36 (>915)
Thick Bedded	12 - 36 (305 - 915)
Moderately Bedded	4 - 12 (102 - 305)
Thin Bedded	1 - 4 (25 - 102)
Very Thin Bedded	0.4 - 1 (10 - 25)
Laminated	0.1 - 0.4 (2.5 - 10)
Thinly Laminated	<0.1 (<2.5)

Bedding Planes: Planes dividing the individual layers, beds, or stratigraphy of rocks.
 Joint: Fracture in rock, generally more or less vertical or traverse to bedding.
 Seam: Applies to bedding plane with unspecified degree of weather.

CORE SAMPLER TYPE GRAPHICS

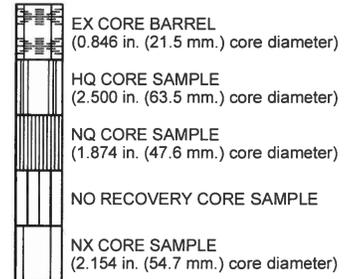


JOINT ROUGHNESS COEFFICIENT (JRC)



From Barton and Choubey, 1977

RQD: Rock-quality designation (RQD) Rough measure of the degree of jointing or fracture in a rock mass, measured as a percentage of the drill core in lengths of 10 cm. or more.



PROJECT NO.: 20181540
 DRAWN BY: JDS
 CHECKED BY: JCR
 DATE: 9/15/2017
 REVISED: -

ROCK DESCRIPTION KEY

Quaker Hill 32 Acre Parcel
 Healdsburg, California

FIGURE

A-3

PLOTTED - 09/15/2017 07:43 AM BY: JSala

PROJECT NUMBER 20181540 OFFICE FILTER - SANTA ROSA
 GINT TEMPLATE E-KLF STANDARD_GINT LIBRARY 2017 GLB [KLF BORING/TEST PIT SOIL LOG]

Date Begin - End: 8/07/2017 **Drilling Company:** Clear Heart Drilling **BORING LOG KB-1**
Logged By: J. Richmond **Drill Crew:** Ricky
Hor.-Vert. Datum: Not Available **Drilling Equipment:** DR8K **Hammer Type - Drop:** 140 lb. Auto - 30 in.
Plunge: -90 degrees **Drilling Method:** Solid Stem Auger
Weather: Overcast **Exploration Diameter:** 4 in. O.D.

Depth (feet)	Graphical Log	FIELD EXPLORATION				LABORATORY RESULTS							
		Surface Condition: Grass	Sample Type	Blow Counts (BC) = Blow/ft. Blows/ft. in. Pocket Pen (PP) = tsf Torque (TV) = tsf RQD = %	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/Remarks
Lithologic Description													
		Sandy SILT with Gravel (ML): low plasticity, gray brown, dry, medium stiff, fine to coarse grained sand, subrounded gravel to 0.5" (Fill)											
		Gravelly CLAY with Sand (CL): medium plasticity, brown, gray brown, dry to moist, very stiff to hard, fine to coarse grained sand, subangular gravel to 0.75", occasional organics (Fill)	BC=17 24 19	56%		11.2	121.6			40	21	Rig chatter at 1'	
5		brown, dark gray, subangular gravel to 0.5"	BC=7 11 15	56%									
		SANDSTONE: yellow brown, highly weathered, extremely weak (R0), fine grained	BC=7 12 13	44%									
10			BC=31 50	67%									
			BC=21 50/5"	36%									
15		The boring was terminated at approximately 15 ft. below ground surface. The boring was backfilled with cuttings on August 07, 2017.		GROUNDWATER LEVEL INFORMATION: Groundwater was not observed during drilling or after completion. GENERAL NOTES:									

	PROJECT NO.: 20181540	BORING LOG KB-1	FIGURE
	DRAWN BY: SDC CHECKED BY: JCR DATE: 9/15/2017 REVISED: -		
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Date Begin - End: 8/07/2017	Drilling Company: Clear Heart Drilling	BORING LOG KB-2
Logged By: J. Richmond	Drill Crew: Ricky	
Hor.-Vert. Datum: Not Available	Drilling Equipment: DR8K	Hammer Type - Drop: 140 lb. Auto - 30 in.
Plunge: -90 degrees	Drilling Method: Solid Stem Auger	
Weather: Overcast	Exploration Diameter: 4 in. O.D.	

Depth (feet)	Graphical Log	FIELD EXPLORATION				LABORATORY RESULTS						
		Surface Condition: Grass	Sample Type	Blow Counts (BC) = Incor. Blows/ft Pocket Pen (PP) = tsf Torque (TV) = tsf RDB = %	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)
Lithologic Description												
5		Clayey Sandy GRAVEL (GC): light brown, gray brown, dry to moist, dense, fine to coarse grained sand, subangular gravel to 0.75" (Fill)	BC=18 33 50	78%		8.9	121.5					
		SANDSTONE: yellow brown, highly weathered, extremely weak (R0), fine grained	BC=18 42 50/5"	100%								

The boring was terminated at approximately 5 ft. below ground surface. The boring was backfilled with cuttings on August 07, 2017.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.
GENERAL NOTES:

GINT FILE: Klf_gint_masier_2017 PROJECT NUMBER: 20181540 OFFICE FILTER SANTA ROSA GINT TEMPLATE E.KLF STANDARD GINT LIBRARY 2017 GLB [KLF BORING/TEST PIT SOIL LOG]

 <p>KLEINFELDER Bright People. Right Solutions.</p>	PROJECT NO.: 20181540	BORING LOG KB-2	FIGURE
	DRAWN BY: SDC CHECKED BY: JCR DATE: 9/15/2017 REVISED: -	Quaker Hill 32 Acre Parcel Healdsburg, California	A-5
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PROJECT NUMBER: 20181540 OFFICE FILTER: SANTA ROSA
 GINT FILE: Klf_gint_master_2017 GINT TEMPLATE: E:\KLF_STANDARD_GINT_LIBRARY_2017.GLB [KLF_BORING/TEST PIT SOIL LOG]

Date Begin - End: 8/07/2017 **Drilling Company:** Clear Heart Drilling **BORING LOG KB-3**
Logged By: J. Richmond **Drill Crew:** Ricky
Hor.-Vert. Datum: Not Available **Drilling Equipment:** DR8K **Hammer Type - Drop:** 140 lb. Auto - 30 in.
Plunge: -90 degrees **Drilling Method:** Solid Stem Auger
Weather: Overcast **Exploration Diameter:** 4 in. O.D.

Depth (feet)	Graphical Log	FIELD EXPLORATION				LABORATORY RESULTS							
		Surface Condition: Grass/Soil	Sample Type	Blow Counts (BC) Uncorr. Blows/ft. Pocket Pen (PP) = tsf Torque (TV) = tsf RQD = %	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
Lithologic Description													
0 - 5		SHALE: gray and red brown, moderately weathered, extremely weak (R0), upper 3-6" reworked, local FeO mineralization on fracture faces	BC=13 18 28	78%		12.3	120.2						TXUU: c = 7.79 ksf
5 - 10		dark gray, slightly weathered, extremely weak (R0), friable, locally sheared to clay	BC=12 20 13	100%									
10 - 13.5		highly sheared	BC=13 16 27	100%									
13.5 - 15			BC=8 8 8	100%									
		The boring was terminated at approximately 13.5 ft. below ground surface. The boring was backfilled with cuttings on August 07, 2017.				GROUNDWATER LEVEL INFORMATION: Groundwater was not observed during drilling or after completion. GENERAL NOTES:							

	PROJECT NO.: 20181540	BORING LOG KB-3 Quaker Hill 32 Acre Parcel Healdsburg, California	FIGURE
	DRAWN BY: SDC CHECKED BY: JCR DATE: 9/15/2017 REVISED: -		A-6

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PROJECT NUMBER 20181540 OFFICE FILTER SANTA ROSA
 GINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY 2017 GLB _KLF_BORING/TEST PIT SOIL LOG

Date Begin - End: 8/07/2017 **Drilling Company:** Clear Heart Drilling **BORING LOG KB-4**
Logged By: J. Richmond **Drill Crew:** Ricky
Hor.-Vert. Datum: Not Available **Drilling Equipment:** DR8K **Hammer Type - Drop:** 140 lb. Auto - 30 in.
Plunge: -90 degrees **Drilling Method:** Solid Stem Auger
Weather: Overcast **Exploration Diameter:** 4 in. O.D.

Depth (feet)	Graphical Log	FIELD EXPLORATION				LABORATORY RESULTS							
		Surface Condition: Grass	Sample Type	Blow Counts (BC) = Blow/ft. Blow/5 ft. Pocket Pen(PP) = tsf Torque(TV) = tsf RQD = %	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/Remarks
Lithologic Description													
0 - 5		Sandy CLAY with Gravel (CL-CH): medium plasticity, brown, yellow brown, dry to moist, hard, fine to coarse grained sand, subangular gravel to 1" (Fill)	BC=16 15 19	33%		13.4	103.1				48	28	
5 - 10		medium plasticity, moist, hard, dark gray brown, fine to coarse grained sand, subrounded gravel to 2"	BC=15 20 22	56%									
10 - 15		Sandy Fat CLAY with Gravel (CH): high plasticity, gray blue, gray brown, yellow brown, moist, very stiff, fine to coarse grained sand, subrounded gravel to 1.5", trace organics (Fill)	BC=10 12 18	50%		9.0	124.9						
15 - 16		CLAY with Sand (CL): medium plasticity, mottled red brown and yellow brown, moist, hard, fine grained sand (Alluvium/Basin Deposit)	BC=10 10 17	56%									
16 - 17			PP=4.5+										
17 - 18			BC=9 12 14	56%									
18 - 19			PP=3.25										
19 - 20		Clayey SHALE (SHALE): gray brown, light brown, highly weathered, extremely weak (R0), gray blue fracture infill	BC=6 12 16	100%									
20 - 21			BC=6 14 24	17%									

The boring was terminated at approximately 15.5 ft. below ground surface. The boring was backfilled with cuttings on August 07, 2017.

GROUNDWATER LEVEL INFORMATION:
 Groundwater was not observed during drilling or after completion.
GENERAL NOTES:

	PROJECT NO.: 20181540	BORING LOG KB-4 Quaker Hill 32 Acre Parcel Healdsburg, California	FIGURE
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PROJECT NUMBER 20181540 OFFICE FILTER SANTA ROSA
 GINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY 2017 GLB [KLF_BORING/TEST PIT SOIL LOG]

Date Begin - End: 8/07/2017 **Drilling Company:** Clear Heart Drilling **BORING LOG KB-5**
Logged By: J. Richmond **Drill Crew:** Ricky
Hor.-Vert. Datum: Not Available **Drilling Equipment:** DR8K **Hammer Type - Drop:** 140 lb. Auto - 30 in.
Plunge: -90 degrees **Drilling Method:** Solid Stem Auger
Weather: Overcast **Exploration Diameter:** 4 in. O.D.

Depth (feet)	Graphical Log	FIELD EXPLORATION				LABORATORY RESULTS							
		Surface Condition: Grass	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf Torvane(TV)= tsf RQD=%	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
Lithologic Description													
		Sandy Gravelly CLAY (CL-CH): medium to high plasticity, brown, gray, gray brown, dry to moist, hard, fine to coarse grained sand, subangular gravel to 0.5" (Fill)	BC=17 23 30	33%		9.6	120.5						
			BC=20 50/5"	36%								Likely pushing a cobble	
5		Clayey SAND with Gravel (SC): low plasticity, gray blue, gray and brown, moist, dense, fine to coarse grained sand, subangular gravel to 0.5", serpentinite derived (Fill)	BC=10 16 29	28%									
		increased plasticity											
10		Fat CLAY (CH): high plasticity, dark gray to black, moist, very stiff (Alluvium/Basin Deposit)	BC=7 10 10 PP=2.25	44%									
			BC=6 10 12 PP=2.5	56%									
15		Fat CLAY with Sand (CH): high plasticity, olive gray brown, moist, very stiff, fine to coarse grained sand											

GROUNDWATER LEVEL INFORMATION:
 Groundwater was not observed during drilling or after completion.
GENERAL NOTES:

The boring was terminated at approximately 15.5 ft. below ground surface. The boring was backfilled with cuttings on August 07, 2017.

	PROJECT NO.: 20181540	BORING LOG KB-5 Quaker Hill 32 Acre Parcel Healdsburg, California	FIGURE
	DRAWN BY: SDC CHECKED BY: JCR DATE: 9/15/2017 REVISED: -		A-8
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PROJECT NUMBER: 20181540 OFFICE FILTER: SANTA ROSA
 GINT TEMPLATE: E-KLF STANDARD_GINT_LIBRARY_2017.GLB GINT FILE: Klf_gint_master_2017
 GINT_TEMPLATE E-KLF_BORING/TEST PIT SOIL LOG

Date Begin - End: 8/07/2017	Drilling Company: Clear Heart Drilling	BORING LOG KB-6
Logged By: J. Richmond	Drill Crew: Ricky	
Hor.-Vert. Datum: Not Available	Drilling Equipment: DR8K	Hammer Type - Drop: 140 lb. Auto - 30 in.
Plunge: -90 degrees	Drilling Method: Solid Stem Auger	
Weather: Overcast	Exploration Diameter: 4 in. O.D.	

Depth (feet)	Graphical Log	FIELD EXPLORATION				LABORATORY RESULTS						
		Surface Condition: Grass	Sample Type	Blow Counts/BC= Uncorr. Blows/5 ft. Pocket Pen(PP)= tsf Torque(TV)= tsf ROD= %	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)
Lithologic Description												
5	[Diagonal Hatching]	Sandy Gravelly CLAY (CL-CH): medium to high plasticity, brown, yellow brown, gray brown, dry to moist, hard, fine to coarse grained sand, subangular gravel to 2" (Fill)	BC=17 28 23	50%		9.9	116.4					
		medium to high plasticity, moist, hard	BC=12 18 18	28%								
		high plasticity, black, gray blue, yellow brown, moist, stiff, fine to coarse grained sand, subangular gravel to 1.5"	BC=9 17 16	44%								
		high plasticity, gray, gray blue, reddish brown, moist, stiff, fine to coarse grained sand, subrounded gravel to 2", increased plasticity	BC=6 8 14 PP=2.0	50%		14.9	117.4					
10	[Diagonal Hatching]	Fat CLAY with Sand (CH): high plasticity, dark gray, gray blue, moist, stiff, fine to medium grained sand, faint organic decay odor (Alluvium/Basin Deposit)	BC=6 8 10 PP=2.0	61%		30.6	90.0					TXUU: c = 1.08 ksf
		Fat CLAY (CH): high plasticity, olive to light olive brown, moist, very stiff (Alluvium/Basin Deposit)	BC=6 9 12 PP=3.0	100%								
15	[Horizontal Hatching]	Clayey SHALE: olive brown, highly weathered, extremely weak (R0)	BC=12 18 26	78%								

The boring was terminated at approximately 16.5 ft. below ground surface. The boring was backfilled with cuttings on August 07, 2017.

GROUNDWATER LEVEL INFORMATION:
 Groundwater was not observed during drilling or after completion.
GENERAL NOTES:

 KLEINFELDER <i>Bright People. Right Solutions.</i>	PROJECT NO.: 20181540	BORING LOG KB-6	FIGURE
	DRAWN BY: SDC CHECKED BY: JCR DATE: 9/15/2017 REVISED: -	Quaker Hill 32 Acre Parcel Healdsburg, California	A-9
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GINT FILE: KLF_gint_master_2017 PROJECT NUMBER: 20181540 OFFICE FILTER: SANTA ROSA GINT TEMPLATE: E:\KLF_STANDARD_GINT_LIBRARY_2017.GLB [KLF_BORING/TEST PIT SOIL LOG]

Date Begin - End: 8/07/2017	Drilling Company: Clear Heart Drilling	BORING LOG KB-7
Logged By: J. Richmond	Drill Crew: Ricky	
Hor.-Vert. Datum: Not Available	Drilling Equipment: DR8K	Hammer Type - Drop: 140 lb. Auto - 30 in.
Plunge: -90 degrees	Drilling Method: Solid Stem Auger	
Weather: Overcast	Exploration Diameter: 4 in. O.D.	

Depth (feet)	Graphical Log	FIELD EXPLORATION					LABORATORY RESULTS						
		Surface Condition: Grass	Sample Type	Blow Counts (BC) Uncorr. Blows/ft. Pocket Pen(PP)= tsf Torvane(TV)= tsf RQD=%	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/Remarks
Lithologic Description													
5		Sandy CLAY with Gravel (CL-CH): brown, gray brown, reddish brown, dry to moist, hard, fine to coarse grained sand, subangular gravel to 1", occasional organics (Fill) moist 3" gravel in shoe minor organic decay odor ----- SERPENTINITE: yellow brown, gray blue, whitish gray, highly weathered, extremely weak (R0), intensely fractured (sheared) local moderately weathered zones	BC=16 24 30 BC=12 19 19 BC=6 9 10 BC=19 25 38	61% 22% 78% 100%			12.1 70.2	121.2 54.4		37 20	TXUU: c = 2.46 ksf TXUU: c = 1.37 ksf		
10		The boring was terminated because of refusal (↑) at approximately 10 ft. below ground surface. The boring was backfilled with cuttings on August 07, 2017.					GROUNDWATER LEVEL INFORMATION: ∇ Groundwater was observed at approximately 8 ft. below ground surface during drilling. GENERAL NOTES:						

	PROJECT NO.: 20181540	BORING LOG KB-7	FIGURE
	DRAWN BY: SDC CHECKED BY: JCR DATE: 9/15/2017 REVISED: -	Quaker Hill 32 Acre Parcel Healdsburg, California	A-10
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OFFICE FILTER: SANTA ROSA
PROJECT NUMBER: 20181540
GINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2017.GLB
KLF BORING/TEST PIT SOIL LOG

Date Begin - End: 8/07/2017
Logged By: J. Richmond
Hor.-Vert. Datum: Not Available
Plunge: -90 degrees
Weather: Not Available

Drilling Company: Pearson
Drill Crew:
Drilling Equipment: Mobile B-53
Drilling Method: Solid Stem Auger
Exploration Diameter: 6 in. O.D.

BORING LOG KB-8

Hammer Type - Drop: 140 lb. Auto - 30 in.

Depth (feet)	Graphical Log	FIELD EXPLORATION					LABORATORY RESULTS						
		Surface Condition: Grass	Sample Type	Blow Counts (BC) = Corrected Blow/ft In. Pocket Pen(PP) = tsf Torvane(TV) = tsf RQD = %	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/Remarks
Lithologic Description													
5		Clayey Sandy GRAVEL (GC): low plasticity, variegated, dry to moist, dense, fine to coarse grained sand, subangular to subrounded gravel to 1" (Fill)	BC=21 26 28	78%		9.0	124.6	36					
		subangular to subrounded gravel to 1.5"	BC=15 17 25	50%									
		moist, medium dense, subangular to subrounded gravel to 2"	BC=6 8 11	56%									
10		Sandy Gravelly CLAY (CL): medium plasticity, dark gray blue, black, moist, very stiff, fine to coarse grained sand, subangular gravel to 2" (Alluvium/Basin Deposit)	BC=8 16 14	44%									
15		Fat CLAY (CH): high plasticity, gray to dark gray, moist, very stiff, trace fine pores (Alluvium/Basin Deposit)	BC=4 7 9 PP=2.0	44%									
<p>The boring was terminated at approximately 14.5 ft. below ground surface. The boring was backfilled with cuttings on August 07, 2017.</p> <p>GROUNDWATER LEVEL INFORMATION: Groundwater was not observed during drilling or after completion.</p> <p>GENERAL NOTES:</p>													

 BRIGHT PEOPLE. RIGHT SOLUTIONS.	PROJECT NO.: 20181540	BORING LOG KB-8 Quaker Hill 32 Acre Parcel Healdsburg, California	FIGURE
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PROJECT NUMBER: 20181540 OFFICE FILTER: SANTA ROSA
 GINT FILE: Klf.gint_master_2017 GINT TEMPLATE: E:\KLF_STANDARD_GINT_LIBRARY_2017\GLB_KLF_BORING\TEST_PIT_SOIL_LOG

Date Begin - End: 8/21/2017	Drilling Company: Pearson	BORING LOG KB-9
Logged By: J. Richmond	Drill Crew:	
Hor.-Vert. Datum: Not Available	Drilling Equipment: Mobile B-53	Hammer Type - Drop: 140 lb. Auto - 30 in.
Plunge: -90 degrees	Drilling Method: Solid Stem Auger	
Weather: Not Available	Exploration Diameter: 6 in. O.D.	

Depth (feet)	Graphical Log	FIELD EXPLORATION				LABORATORY RESULTS							Additional Tests/Remarks	
		Surface Condition: Grass	Sample Type	Blow Counts/BC Uncorr. Blows/6 In. Pocket Pen(PP)= tsf Torque(TV)= tsf RQP=%	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)		
		Lithologic Description												
5	[Hatched Pattern]	Sandy Gravelly CLAY (CL): medium plasticity, brown, red brown, gray, dry to moist, very stiff, fine to coarse grained sand, subangular to subrounded gravel to 1.5" (Fill) stiff, marginally increased clay content brown, gray blue, fine to coarse grained sand, subangular gravel to 2"	BC=9 16 28 BC=13 19 28 BC=5 6 8 BC=4 9 10 BC=5 6 9 PP=1.5 BC=7 14 23	39% 50% 100% 50% 78% 50%		10.7	119.0		42					
10	[Diagonal Pattern]	CLAY (CH): high plasticity, dark gray, moist, stiff (Alluvium/Basin Deposit)										TXUU: c = 1.3 ksf		
15	[Dotted Pattern]	SANDSTONE: yellow brown, highly weathered to decomposed, extremely weak (R0), weathered locally to Clayey SAND with relic structure										Increased drilling resistance at 12.5'		
The boring was terminated at approximately 16.5 ft. below ground surface. The boring was backfilled with cuttings on August 21, 2017.						GROUNDWATER LEVEL INFORMATION: Groundwater was not observed during drilling or after completion.								
						GENERAL NOTES:								

 KLEINFELDER <i>Bright People. Right Solutions.</i>	PROJECT NO.: 20181540	BORING LOG KB-9	FIGURE
	DRAWN BY: SDC CHECKED BY: JCR DATE: 9/15/2017 REVISED: -	Quaker Hill 32 Acre Parcel Healdsburg, California	A-12
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OFFICE FILTER: SANTA ROSA

PROJECT NUMBER: 20181540

GINT FILE: KLF_gint_master_2017
GINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2017.GLB [KLF_BORING/TEST PIT SOIL LOG]

Date Begin - End: 8/21/2017	Drilling Company: Pearson	BORING LOG KB-10	
Logged By: J. Richmond	Drill Crew:		
Hor.-Vert. Datum: Not Available	Drilling Equipment: Mobile B-53		Hammer Type - Drop: 140 lb. Auto - 30 in.
Plunge: -90 degrees	Drilling Method: Solid Stem Auger		
Weather: Not Available	Exploration Diameter: 6 in. O.D.		

Depth (feet)	Graphical Log	FIELD EXPLORATION					LABORATORY RESULTS						
		Surface Condition: Grass	Sample Type	Blow Counts(BC)= Uncorr. Blows/ft Pocket Pen(PP)= tsf Torrane(TV)= tsf RQD=%	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
Lithologic Description													
5	Clayey Gravelly SAND/Sandy GRAVEL (SC): low plasticity, dry, dense, fine to coarse grained sand, subangular to subrounded gravel to 2", rootlets to 6" (Fill)	BC=7 26 40	44%										
	moist, medium dense, increased clay content	BC=9 9 9	100%	SC	10.4			27	39	20			
		BC=8 11 13	39%		10.0	122.9						TXUU: c = 1.67 ksf Rig chatter at 6' (large clast)	
10	Fat CLAY (CH): high plasticity, mottled olive and red brown, moist, very stiff, trace fine pores (Alluvium/Basin Deposit)	BC=7 8 8 PP=2.25	67%										
	SHALE: mottled olive, olive brown, gray blue, highly weathered to decomposed, extremely weak (R0), intensely fractured, calcium carbonate	BC=8 17 25	44%										
15	The boring was terminated at approximately 14.5 ft. below ground surface. The boring was backfilled with cuttings on August 21, 2017.					GROUNDWATER LEVEL INFORMATION: Groundwater was not observed during drilling or after completion. GENERAL NOTES:							

	PROJECT NO.: 20181540	BORING LOG KB-10 Quaker Hill 32 Acre Parcel Healdsburg, California	FIGURE
	DRAWN BY: SDC CHECKED BY: JCR DATE: 9/15/2017 REVISED: -		A-13
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PROJECT NUMBER: 20181540
 OFFICE FILTER: SANTA ROSA
 GINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2017_GLB_KLF_BORING/TEST PIT SOIL LOG

Date Begin - End: 8/21/2017	Drilling Company: Pearson	BORING LOG KB-11
Logged By: J. Richmond	Drill Crew:	
Hor.-Vert. Datum: Not Available	Drilling Equipment: Mobile B-53	Hammer Type - Drop: 140 lb. Auto - 30 in.
Plunge: -90 degrees	Drilling Method: Solid Stem Auger	
Weather: Not Available	Exploration Diameter: 6 in. O.D.	

Depth (feet)	Graphical Log	FIELD EXPLORATION					LABORATORY RESULTS						
		Surface Condition: Grass	Sample Type	Blow Counts (BC) = Blow/ft. Blow/sq. ft. Pocket Pen(PP) = tsf Tonvane(TV) = tsf RQD = %	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/Remarks
Lithologic Description													
5		Sandy Clayey GRAVEL (GC): low to medium plasticity, gray, brown, gray blue, dry to moist, dense, fine to coarse grained sand, subangular to subrounded gravel to 2", rootlets to 6" (Fill)	BC=19 26 24	56%	SC	11.0	119.5	37	58	33			
		low to medium plasticity, gray blue, dark gray, moist, medium dense, fine to coarse grained sand, subrounded gravel to 1.5"	BC=18 18 20	44%									
			BC=5 7 9	100%									
		Fat CLAY (CH): high plasticity, dark brown, moist, stiff to very stiff (Alluvium/Basin Deposit)										Rig chatter Reduced drilling resistance from 8-8.5'	
10		Fat CLAY with Sand (CH): high plasticity, mottled olive, red brown, gray blue, moist, very stiff, fine to coarse grained sand (Alluvium/Basin Deposit)	BC=4 6 9 PP=1.0-2.75	67%		24.4	101.2					TXUU: c = 0.71 ksf	
		SHALE: olive, gray blue, highly weahtered, extremely weak (R0), intensely fractured											
			BC=8 14 17	44%									

The boring was terminated at approximately 15.5 ft. below ground surface. The boring was backfilled with cuttings on August 21, 2017.

GROUNDWATER LEVEL INFORMATION:
 Groundwater was not observed during drilling or after completion.

GENERAL NOTES:

 BRIGHT PEOPLE. RIGHT SOLUTIONS.	PROJECT NO.: 20181540	BORING LOG KB-11	FIGURE
	DRAWN BY: SDC	Quaker Hill 32 Acre Parcel Healdsburg, California	A-14
CHECKED BY: JCR	DATE: 9/15/2017		
REvised: -			PAGE: 1 of 1

PLOTTED: 09/15/2017 07:45 AM BY: JSala

OFFICE FILTER SANTA ROSA

PROJECT NUMBER: 20181540
 GINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2017.GLB
 GINT FILE: KLF_gint_master_2017

Date Begin - End: 8/21/2017	Drilling Company: Pearson	BORING LOG KB-12
Logged By: J. Richmond	Drill Crew:	
Hor.-Vert. Datum: Not Available	Drilling Equipment: Mobile B-53	Hammer Type - Drop: 140 lb. Auto - 30 in.
Plunge: -90 degrees	Drilling Method: Solid Stem Auger	
Weather: Not Available	Exploration Diameter: 6 in. O.D.	

Depth (feet)	Graphical Log	FIELD EXPLORATION					LABORATORY RESULTS						
		Surface Condition: Grass	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 In. Pocket Pen(PP)= tsf Tonane(TV)= tsf RQD=%	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
Lithologic Description													
0	Surface Condition: Grass												
0-5	Sandy GRAVEL with Clay (GP-GC): low plasticity, dry to moist, dense, fine to coarse grained sand, subangular gravel to 2.5+", rootlets to 6" (Fill)												
5	reworked at contact												
5-7	Fat CLAY (CH): high plasticity, dark gray brown, moist, stiff, trace fine to coarse grained sand, subrounded gravel to 1" (Reworked Alluvium/Basin Deposit)	BC=21 29 50	61%										
7-8	Fat CLAY (CH): high plasticity, dark gray brown, moist, stiff, trace fine to coarse grained sand, subrounded gravel to 1" (Reworked Alluvium/Basin Deposit)	BC=12 18 22	33%			10.8	126.7						
8-9	Fat CLAY (CH): high plasticity, dark gray brown, moist, stiff, trace fine to coarse grained sand, subrounded gravel to 1" (Reworked Alluvium/Basin Deposit)	BC=4 7 7 PP=1.75	56%			23.5	101.2						
9-10	SERPENTINITE: red brown, gray blue, slightly weathered, extremely weak (R0), pervasively sheared												
10-11	SERPENTINITE: red brown, gray blue, slightly weathered, extremely weak (R0), pervasively sheared	BC=21 30	67%										
11-13.5	SERPENTINITE: red brown, gray blue, slightly weathered, extremely weak (R0), pervasively sheared	BC=8 20 24	100%										
13.5	The boring was terminated at approximately 13.5 ft. below ground surface. The boring was backfilled with cuttings on August 21, 2017.					GROUNDWATER LEVEL INFORMATION: Groundwater was not observed during drilling or after completion. GENERAL NOTES:							

 BRIGHT PEOPLE. RIGHT SOLUTIONS.	PROJECT NO.: 20181540	BORING LOG KB-12	FIGURE
	DRAWN BY: SDC	Quaker Hill 32 Acre Parcel Healdsburg, California	A-15
CHECKED BY: JCR	DATE: 9/15/2017		
REVISD: -			PAGE: 1 of 1

PLOTTED: 09/15/2017 07:45 AM BY: JSala

Date Begin - End: 8/21/2017 **Drilling Company:** Pearson **BORING LOG KB-13**
Logged By: J. Richmond **Drill Crew:** _____
Hor.-Vert. Datum: Not Available **Drilling Equipment:** Mobile B-53 **Hammer Type - Drop:** 140 lb. Auto - 30 in.
Plunge: -90 degrees **Drilling Method:** Solid Stem Auger
Weather: Not Available **Exploration Diameter:** 6 in. O.D.

Depth (feet)	Graphical Log	FIELD EXPLORATION					LABORATORY RESULTS						
		Surface Condition: Grass	Lithologic Description	Sample Type	Blow Counts (BC) = Incorf. Blows/6 in. Pocket Pen (PP) = tsf Tonane(TV) = tsf RQD = %	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)
5		Clayey Sandy GRAVEL (GC): low plasticity, variegated, dry to moist, dense, fine to coarse grained sand, subangular gravel to 1.5", rootlets to 6" (Fill)	BC=8 19 38	50%		16.1	114.2						
		SERPENTINITE: gray blue, gray, slightly weathered, extremely weak to weak (R0-R1), pervasively sheared	BC=19 29 50/4"	50%							65	22	
			BC=10 14 16	67%									

The boring was terminated at approximately 5.5 ft. below ground surface. The boring was backfilled with cuttings on August 21, 2017.

GROUNDWATER LEVEL INFORMATION:
 Groundwater was not observed during drilling or after completion.
GENERAL NOTES:

PROJECT NUMBER: 20181540 OFFICE FILTER: SANTA ROSA
 GINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2017.GLB [KLF_BORING/TEST PIT SOIL LOG]

	PROJECT NO.: 20181540	BORING LOG KB-13 Quaker Hill 32 Acre Parcel Healdsburg, California	FIGURE
	DRAWN BY: SDC CHECKED BY: JCR DATE: 9/15/2017 REVISED: -		A-16
			PAGE: 1 of 1

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PROJECT NUMBER: 20181540
 OFFICE FILTER: SANTA ROSA
 GINT FILE: KLF_gint_master_2017
 GINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2017.GLB [KLF_BORING/TEST PIT SOIL LOG]

Date Begin - End: 8/21/2017	Drilling Company: Pearson	BORING LOG KB-14	
Logged By: J. Richmond	Drill Crew:		
Hor.-Vert. Datum: Not Available	Drilling Equipment: Mobile B-53		Hammer Type - Drop: 140 lb. Auto - 30 in.
Plunge: -90 degrees	Drilling Method: Solid Stem Auger		
Weather: Not Available	Exploration Diameter: 6 in. O.D.		

Depth (feet)	Graphical Log	FIELD EXPLORATION					LABORATORY RESULTS							
		Surface Condition: Grass	Lithologic Description	Sample Type	Blow Counts(BC)= D100: Blows/6 in. Pocket Pen(PP)= tsf Tonimeter(TV)= tsf RQD=%	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/Remarks
5		Sandy GRAVEL with Silt (GP-GM): gray, dry, dense, fine to coarse grained sand, subangular gravel to 0.75" (AB)												
		Sandy Clayey GRAVEL (GC): low to medium plasticity, brown, red brown, gray, moist, dense, fine to coarse grained sand, subangular gravel to 2" (Fill)	BC=8 15 17	78%		SC	9.9	123.7		24	47	26		
		low plasticity, medium dense, Serpentinite derived	BC=5 6 12	100%										
		Fat CLAY with Sand (CH): high plasticity, mottled light brown and gray blue, moist, stiff, fine to coarse grained sand (Alluvium/Basin Deposit)	BC=3 4 5 PP=2.5	100%										
10		dark gray to gray blue, very stiff	BC=4 6 10 PP=2.0	50%			33.6	87.7						
		SERPENTINITE: olive, decomposed, extremely weak (R0), pervasively sheared												
15			BC=9 19 19	44%			12.7	123.0						

The boring was terminated at approximately 15.5 ft. below ground surface. The boring was backfilled with cuttings on August 21, 2017.

GROUNDWATER LEVEL INFORMATION:
 Groundwater was not observed during drilling or after completion.
GENERAL NOTES:

	PROJECT NO.: 20181540	BORING LOG KB-14	FIGURE
	DRAWN BY: SDC		
	CHECKED BY: JCR	Quaker Hill 32 Acre Parcel Healdsburg, California	A-17
	DATE: 9/15/2017		
	REVISED: -		PAGE: 1 of 1



KLEINFELDER

Bright People. Right Solutions.

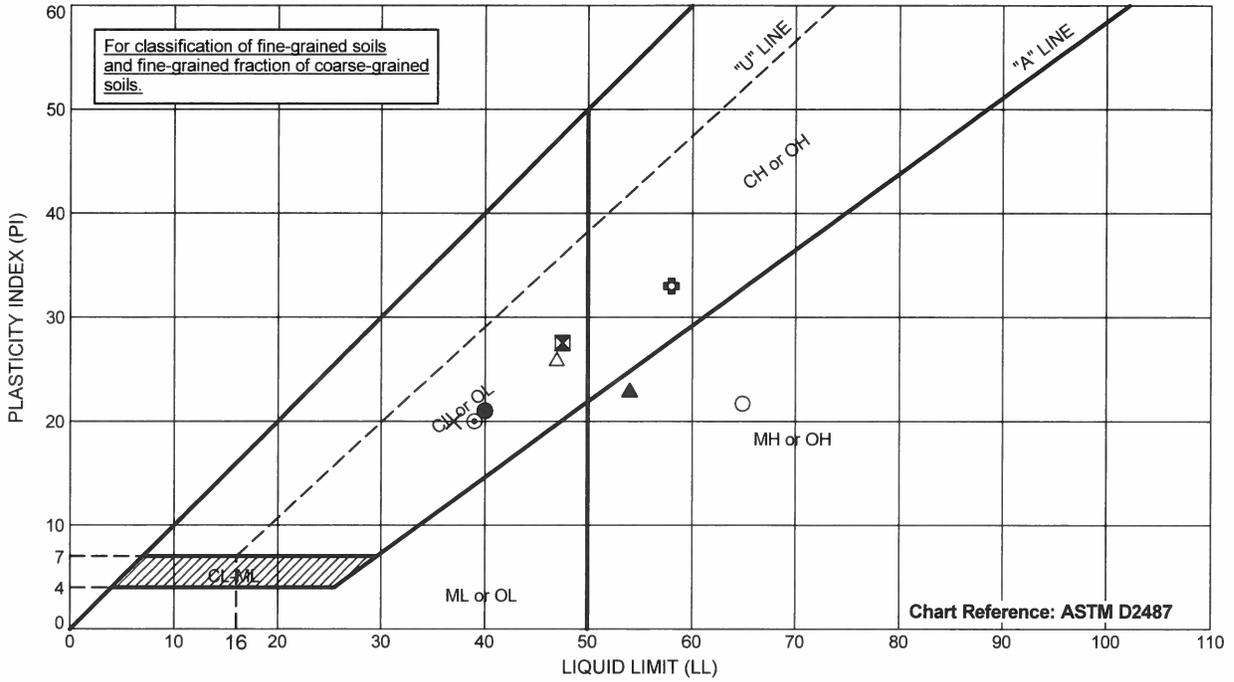
Appendix B

Exploration ID	Depth (ft.)	Sample Description	Water Content (%)	Dry Unit Wt. (pcf)	Sieve Analysis (%)			Atterberg Limits			Additional Tests
					Passing 3/4"	Passing #4	Passing #200	Liquid Limit	Plastic Limit	Plasticity Index	
KB-1	2.5	OLIVE BROWN SANDY LEAN CLAY (CL)	11.2	121.6				40	19	21	
KB-2	1.5		8.9	121.5							TXUU: c = 7.79 ksf
KB-3	1.5		12.3	120.2							
KB-4	1.0	OLIVE BROWN SANDY LEAN CLAY (CL)	13.4	103.1				48	20	28	
KB-4	5.0		9.0	124.9							
KB-5	2.0		9.6	120.5							
KB-5/6	0.0 - 6.0	OLIVE BROWN SANDY ELASTIC SILT (MH)						54	31	23	
KB-6	1.0		9.9	116.4							
KB-6	7.0		14.9	117.4							
KB-6	10.0		30.6	90.0				37	17	20	TXUU: c = 1.08 ksf TXUU: c = 2.46 ksf
KB-7	2.0	DARK YELLOWISH BROWN CLAYEY SAND WITH GRAVEL (SC)	12.1	121.2							TXUU: c = 1.37 ksf
KB-7	6.0		70.2	54.4							
KB-8	2.0	OLIVE BROWN CLAYEY SAND WITH GRAVEL (SC)	9.0	124.6							
KB-9	1.0	OLIVE BROWN CLAYEY SAND (SC)	10.7	119.0							
KB-9	11.0		23.2	98.8							TXUU: c = 1.3 ksf
KB-10	2.0	DARK OLIVE BROWN CLAYEY SAND WITH GRAVEL (SC)	10.4					39	19	20	
KB-10	5.0		10.0	122.9							TXUU: c = 1.67 ksf
KB-11	2.0	DARK OLIVE BROWN CLAYEY SAND (SC)	11.0	119.5				37	58	33	
KB-11	10.0		24.4	101.2							TXUU: c = 0.71 ksf
KB-12	4.0		10.8	126.7							
KB-12	6.0		23.5	101.2							
KB-13	1.0		16.1	114.2							
KB-13	3.0	DARK GREENISH GRAY SERPENTINITE									
KB-14	3.0	DARK OLIVE BROWN CLAYEY GRAVEL WITH SAND (GC)	9.9	123.7				65	43	22	
KB-14	10.0		33.6	87.7				24	47	21	26
KB-14	15.0		12.7	123.0							



Refer to the Geotechnical Evaluation Report or the supplemental plates for the method used for the testing performed above.
 NP = NonPlastic

PROJECT NO.: 20181540	DRAWN BY: JDS	CHECKED BY: JCR	DATE: 9/15/2017	REVISED: -
LABORATORY TEST RESULT SUMMARY		Quaker Hill 32 Acre Parcel Healdsburg, California		
FIGURE		B-1		



Exploration ID	Depth (ft.)	Sample Description	Passing #200	LL	PL	PI
● KB-1	2.5	OLIVE BROWN SANDY LEAN CLAY (CL)	NM	40	19	21
⊠ KB-4	1	OLIVE BROWN SANDY LEAN CLAY (CL)	NM	48	20	28
▲ KB-5/6	0 - 6	OLIVE BROWN SANDY ELASTIC SILT (MH)	NM	54	31	23
⊗ KB-7	2	DARK YELLOWISH BROWN CLAYEY SAND WITH GRAVEL (SC)	NM	37	17	20
⊙ KB-10	2	DARK OLIVE BROWN CLAYEY SAND WITH GRAVEL (SC)	27	39	19	20
⊕ KB-11	2	DARK OLIVE BROWN CLAYEY SAND (SC)	37	58	25	33
○ KB-13	3	DARK GREENISH GRAY SERPENTINITE	NM	65	43	22
△ KB-14	3	DARK OLIVE BROWN CLAYEY GRAVEL WITH SAND (GC)	24	47	21	26

Testing performed in general accordance with ASTM D4318.
 NP = Nonplastic
 NM = Not Measured



PROJECT NO.: 20181540
 DRAWN BY: SDC
 CHECKED BY: JCR
 DATE: 9/15/2017
 REVISED: -

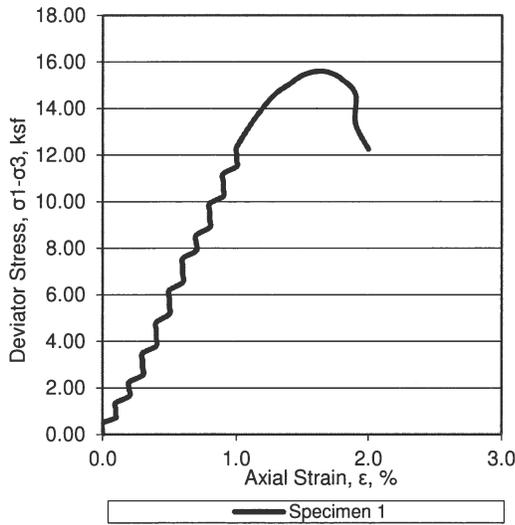
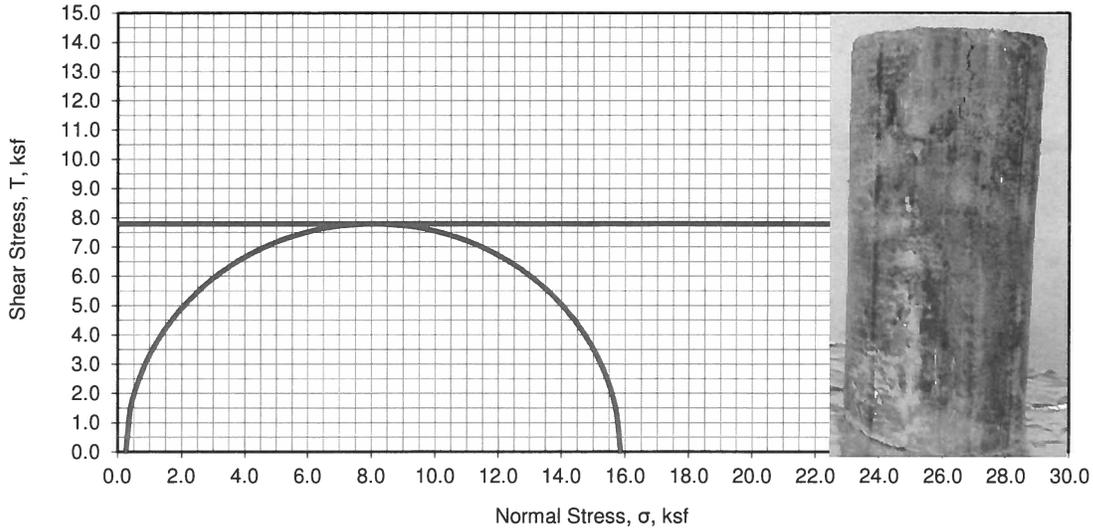
ATTERBERG LIMITS

Quaker Hill 32 Acre Parcel
 Healdsburg, California

FIGURE
B-2

Total	
c =	7.79 ksf

Specimen Shear Picture



Specimen No.		1	
Initial	Diameter, in	D _o	2.41
	Height, in	H _o	5.53
	Water Content, %	ω _o	12.3
	Dry Density, lbs/ft ³	γ _d _o	120.2
	Saturation, %	S _o	87
	Void Ratio	e _o	0.375
Minor Principal Stress, ksf		σ ₃	0.25
Maximum Deviator Stress, ksf		(σ ₁ -σ ₃) _{max}	15.59
Time to (σ ₁ -σ ₃) _{max} , min		t _f	1.63
Deviator Stress @ 15% Axial Strain, ksf		(σ ₁ -σ ₃) _{15%}	12.25
Ultimate Deviator Stress, ksf		(σ ₁ -σ ₃) _{ult}	na
Rate of strain, %/min		ε̇	1.00
Axial Strain at Failure, %		ε _f	1.63

Description of Specimen: Dark Grayish Brown Lean Clay with Sand (CL)			
Amount of Material Finer than the No. 200, %:		nm	
LL: nm	PL: nm	PI: nm	G _s : 2.65 Assumed
Specimen Type: Undisturbed		Test Method: ASTM D2850	

Membrane correction applied

Boring:	KB-3	Remarks: nm= not measured, na = not applicable
Sample:	1C	
Depth, ft:	1.5	
Test Date:	8/21/17	



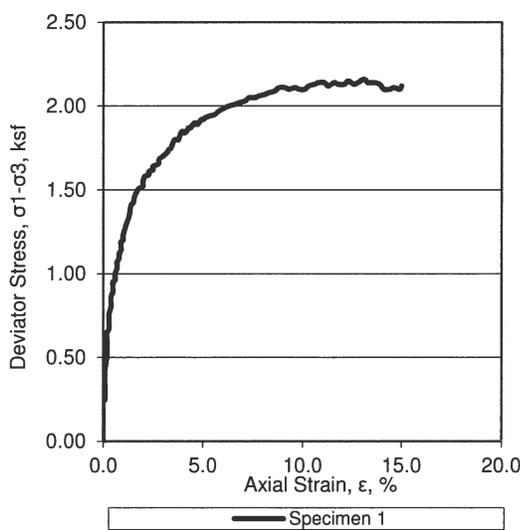
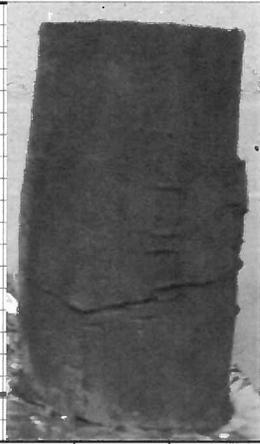
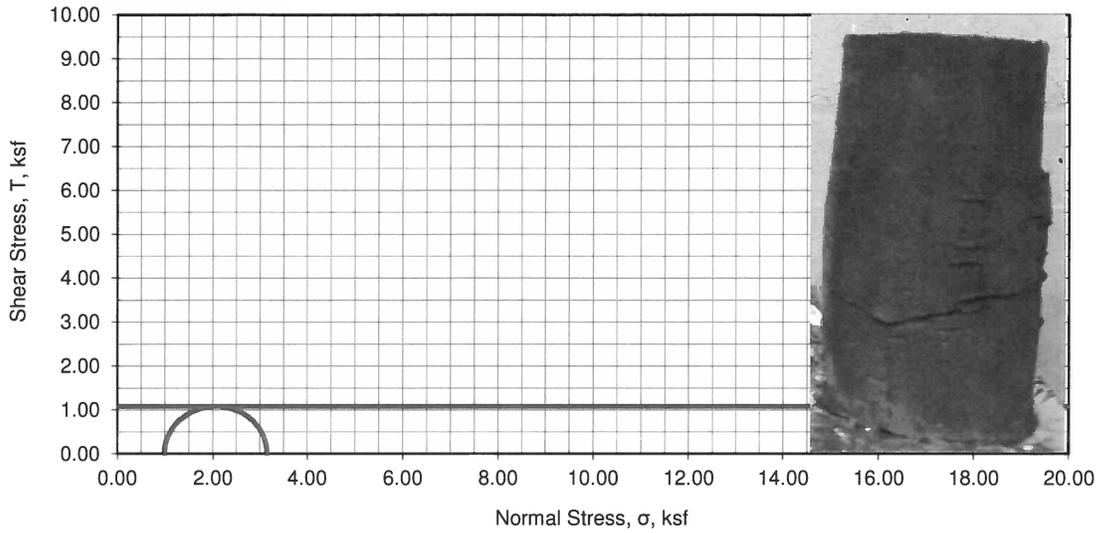
Project No.:	20181540
Date:	8/25/17
Entry By:	CP
Checked By:	CP
File Name:	HL10476

TRIAXIAL COMPRESSION
TEST (UU)
Quaker Hill 32 Acre Parcel
Healdsburg, California

Figure
 1 of 1
B-3

Total	
c =	1.08 ksf

Specimen Shear Picture



Specimen No.		1	
Initial	Diameter, in	D _o	2.40
	Height, in	H _o	5.61
	Water Content, %	w _o	30.6
	Dry Density, lbs/ft ³	γ _d _o	90.0
	Saturation, %	S _o	97
	Void Ratio	e _o	0.838
Minor Principal Stress, ksf		σ ₃	0.99
Maximum Deviator Stress, ksf		(σ ₁ -σ ₃) _{max}	2.16
Time to (σ ₁ -σ ₃) _{max} , min		t _f	13.08
Deviator Stress @ 15% Axial Strain, ksf		(σ ₁ -σ ₃) _{15%}	2.12
Ultimate Deviator Stress, ksf		(σ ₁ -σ ₃) _{ult}	na
Rate of strain, %/min		'ε	1.00
Axial Strain at Failure, %		ε _f	13.08

Description of Specimen:		Black Sandy Lean Clay (CL)	
Amount of Material Finer than the No. 200, %:		nm	
LL: nm	PL: nm	PI: nm	G _s : 2.65 Assumed
Specimen Type:		Undisturbed	
Test Method:		ASTM D2850	

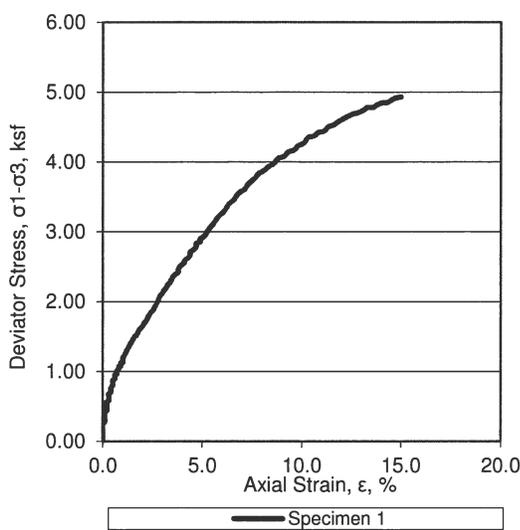
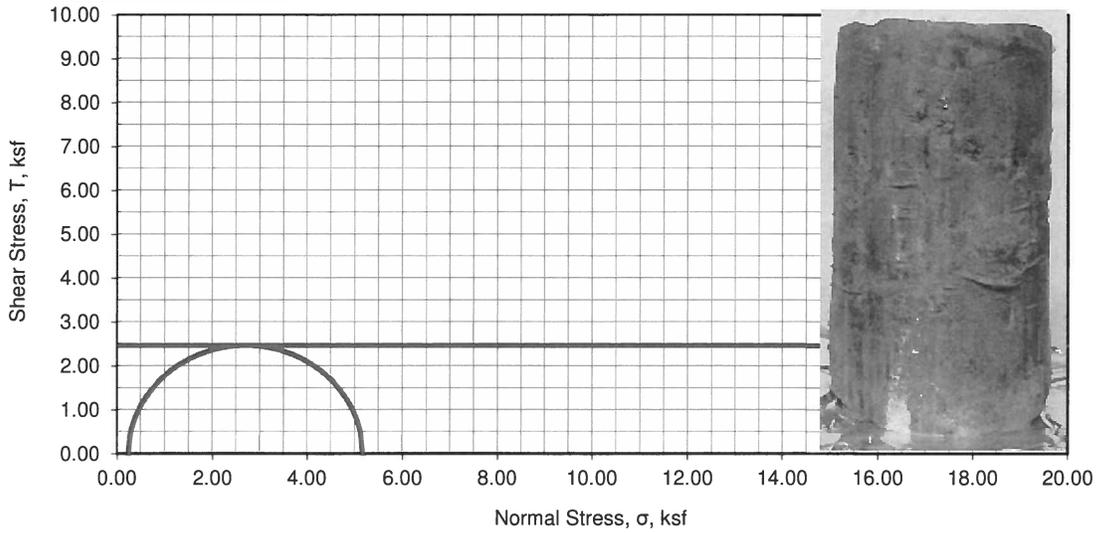
Membrane correction applied

Boring:	KB-6	Remarks: nm= not measured, na = not applicable * Sample saturated prior to testing.
Sample:	5C	
Depth, ft:	10.0	
Test Date:	8/22/17	

	Project No.:	20181540	TRIAXIAL COMPRESSION TEST (UU)	Figure 1 of 1
	Date:	8/25/17		
	Entry By:	CP	Quaker Hill 32 Acre Parcel Healdsburg, California	B-4
	Checked By:	CP		
File Name:	HL10476			

Total	
c =	2.46 ksf

Specimen Shear Picture



Specimen No.		1
Initial	Diameter, in	D _o 2.40
	Height, in	H _o 5.51
	Water Content, %	w _o 12.1
	Dry Density, lbs/ft ³	γ _d 121.2
	Saturation, %	S _o 88
	Void Ratio	e _o 0.364
Minor Principal Stress, ksf		σ ₃ 0.24
Maximum Deviator Stress, ksf		(σ ₁ -σ ₃) _{max} 4.93
Time to (σ ₁ -σ ₃) _{max} , min		t _f 15.02
Deviator Stress @ 15% Axial Strain, ksf		(σ ₁ -σ ₃) _{15%} 4.93
Ultimate Deviator Stress, ksf		(σ ₁ -σ ₃) _{ult} na
Rate of strain, %/min		ε̇ 1.00
Axial Strain at Failure, %		ε _f 15.02

Description of Specimen:		Dark Yellowish Brown Clayey Sand with Gravel (SC)	
Amount of Material Finer than the No. 200, %:		nm	
LL: 37	PL: 17	PI: 20	G _s : 2.65 Assumed
Specimen Type:		Undisturbed	Test Method: ASTM D2850

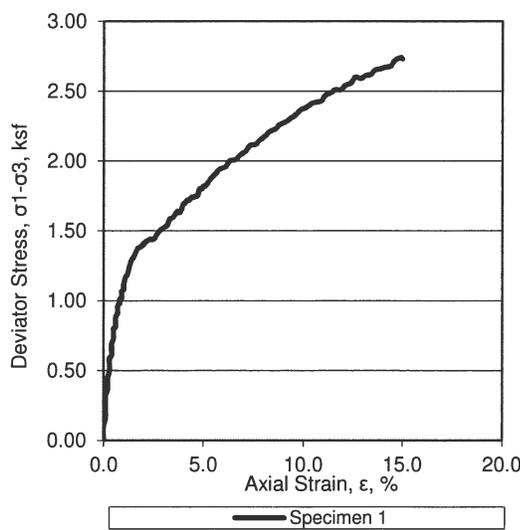
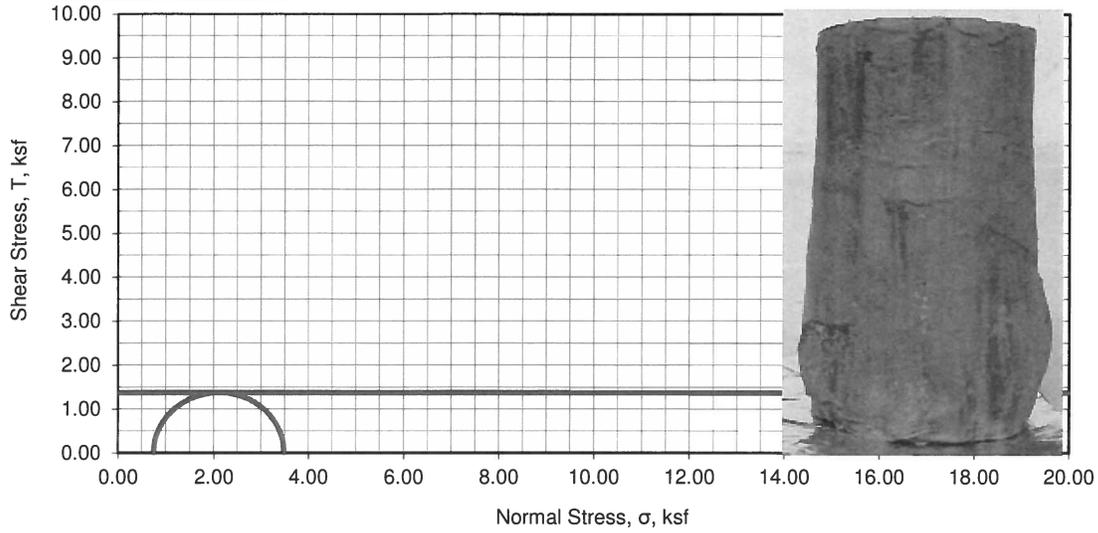
Membrane correction applied

Boring:	KB-7	Remarks: nm= not measured, na = not applicable * Sample saturated prior to testing. Gravel found in sample after test completed.
Sample:	1C	
Depth, ft:	2.0	
Test Date:	8/21/17	

<p>KLEINFELDER Bright People. Right Solutions. 2601 Barrington Ct, Hayward, CA 94545</p>	Project No.:	20181540	TRIAXIAL COMPRESSION	Figure 1 of 1
	Date:	8/25/17		
	Entry By:	CP	Quaker Hill 32 Acre Parcel Healdsburg, California	B-5
	Checked By:	CP		
File Name:	HL10476			

Total	
c =	1.37 ksf

Specimen Shear Picture



Specimen No.		1
Initial	Diameter, in	D _o 2.40
	Height, in	H _o 5.59
	Water Content, %	ω _o 70.2
	Dry Density, lbs/ft ³	γ _d _o 54.4
	Saturation, %	S _o 91
	Void Ratio	e _o 2.042
Minor Principal Stress, ksf		σ ₃ 0.75
Maximum Deviator Stress, ksf		(σ ₁ -σ ₃) _{max} 2.74
Time to (σ ₁ -σ ₃) _{max} , min		t _f 14.88
Deviator Stress @ 15% Axial Strain, ksf		(σ ₁ -σ ₃) _{15%} 2.73
Ultimate Deviator Stress, ksf		(σ ₁ -σ ₃) _{ult} na
Rate of strain, %/min		'ε 1.00
Axial Strain at Failure, %		ε _f 14.88

Description of Specimen: Olive Brown Sandy Lean Clay (CL)			
Amount of Material Finer than the No. 200, %: nm			
LL: nm	PL: nm	PI: nm	G _s : 2.65 Assumed
Specimen Type: Undisturbed		Test Method: ASTM D2850	

Membrane correction applied

Boring:	KB-7	Remarks: nm= not measured, na = not applicable
Sample:	3C	
Depth, ft:	6.0	
Test Date:	8/22/17	



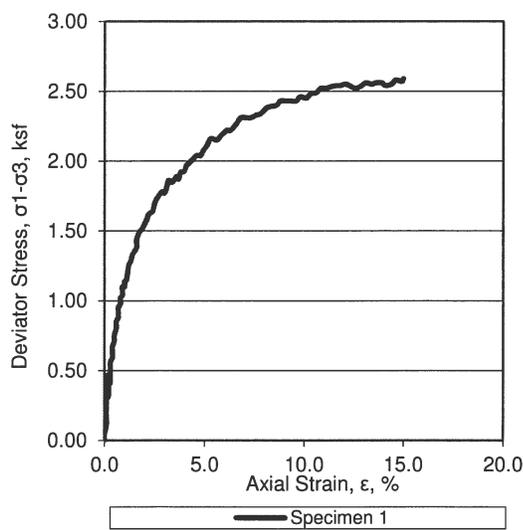
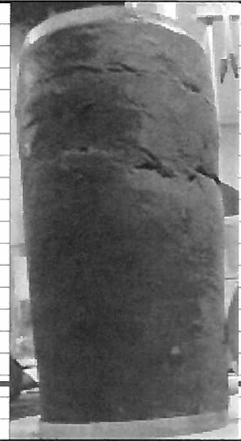
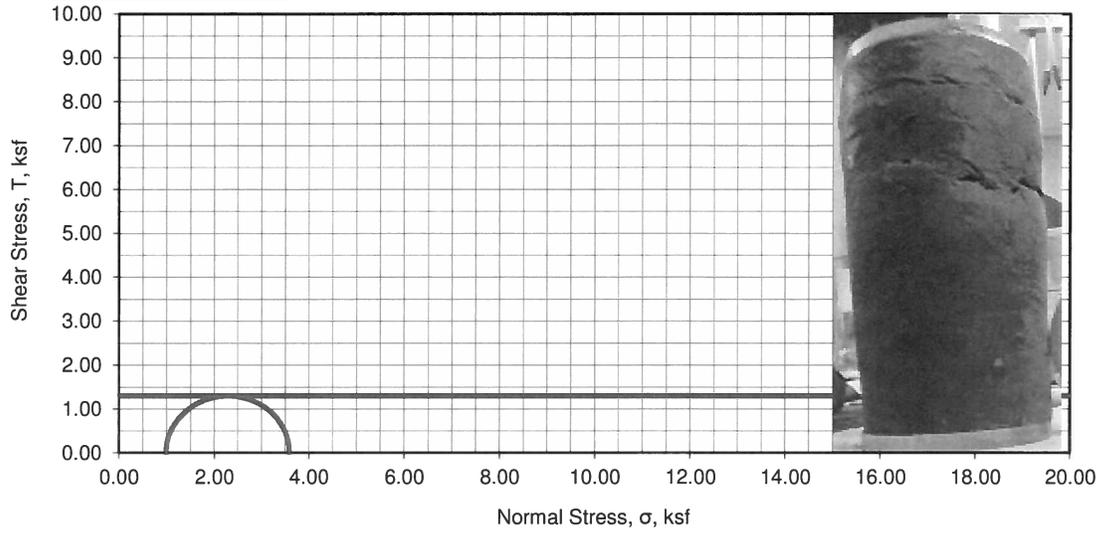
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TRIAxIAL COMPRESSION
TEST (UU)
Quaker Hill 32 Acre Parcel
Healdsburg, California

Figure
1 of 1
B-6

Total	
c =	1.30 ksf

Specimen Shear Picture



Specimen No.			1
Initial	Diameter, in	D _o	2.40
	Height, in	H _o	5.76
	Water Content, %	w _o	23.2
	Dry Density, lbs/ft ³	γ _{d_o}	98.8
	Saturation, %	S _o	91
	Void Ratio	e _o	0.673
Minor Principal Stress, ksf		σ ₃	0.99
Maximum Deviator Stress, ksf		(σ ₁ -σ ₃) _{max}	2.59
Time to (σ ₁ -σ ₃) _{max} , min		t _f	15.00
Deviator Stress @ 15% Axial Strain, ksf		(σ ₁ -σ ₃) _{15%}	2.59
Ultimate Deviator Stress, ksf		(σ ₁ -σ ₃) _{ult}	na
Rate of strain, %/min		ε̇	1.00
Axial Strain at Failure, %		ε _f	15.00

Description of Specimen:		Brown Lean Clay (CL)	
Amount of Material Finer than the No. 200, %:		nm	
LL: nm	PL: nm	PI: nm	G _s : 2.65 Assumed
Specimen Type:		Undisturbed	Test Method: ASTM D2850

Membrane correction applied

Boring:	KB-9	Remarks: nm= not measured, na = not applicable
Sample:	5C	
Depth, ft:	11.0	
Test Date:	9/5/17	



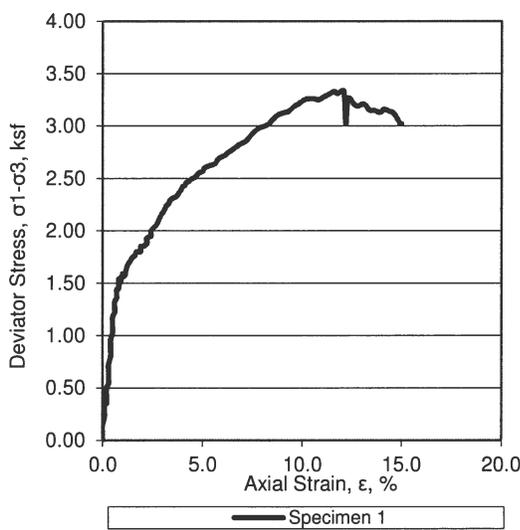
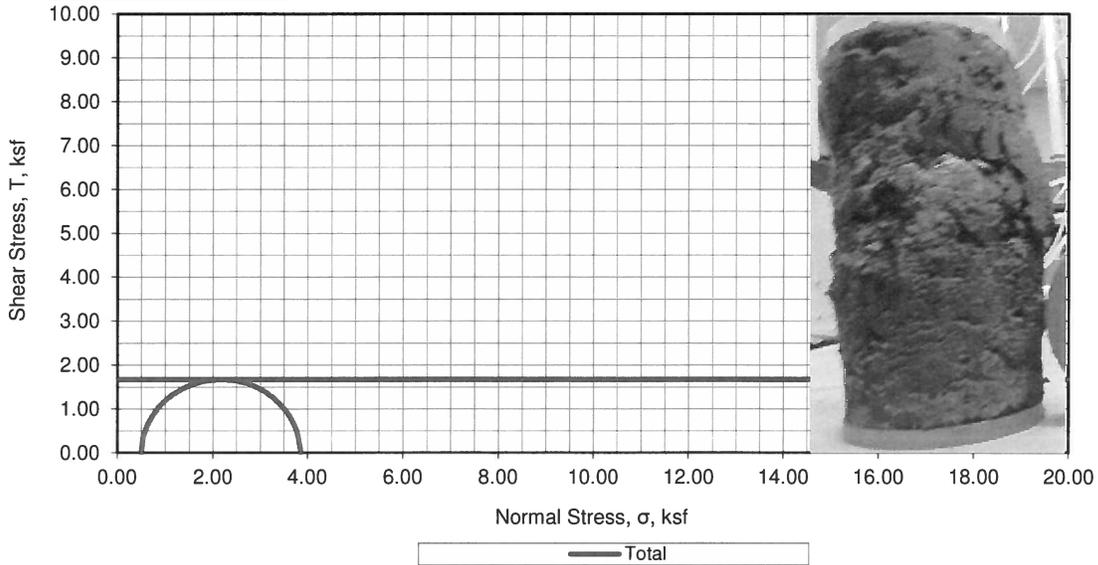
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Checked By:	CP
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**TRIAXIAL COMPRESSION
TEST (UU)**
**Quaker Hill 32 Acre Parcel
Healdsburg, California**

Figure
1 of 1
B-7

Total	
c =	1.67 ksf

Specimen Shear Picture



Specimen No.		1
Initial	Diameter, in	D ₀ 2.39
	Height, in	H ₀ 5.60
	Water Content, %	ω ₀ 10.0
	Dry Density, lbs/ft ³	γ _{d0} 122.9
	Saturation, %	S ₀ 77
	Void Ratio	e ₀ 0.346
Minor Principal Stress, ksf		σ ₃ 0.50
Maximum Deviator Stress, ksf		(σ ₁ -σ ₃) _{max} 3.33
Time to (σ ₁ -σ ₃) _{max} , min		t _f 12.10
Deviator Stress @ 15% Axial Strain, ksf		(σ ₁ -σ ₃) _{15%} 3.02
Ultimate Deviator Stress, ksf		(σ ₁ -σ ₃) _{ult} na
Rate of strain, %/min		'ε 1.00
Axial Strain at Failure, %		ε _f 12.10

Description of Specimen: Brown Sandy Lean Clay (CL)			
Amount of Material Finer than the No. 200, %: nm			
LL: nm	PL: nm	PI: nm	G _s : 2.65 Assumed
Specimen Type: Undisturbed		Test Method: ASTM D2850	

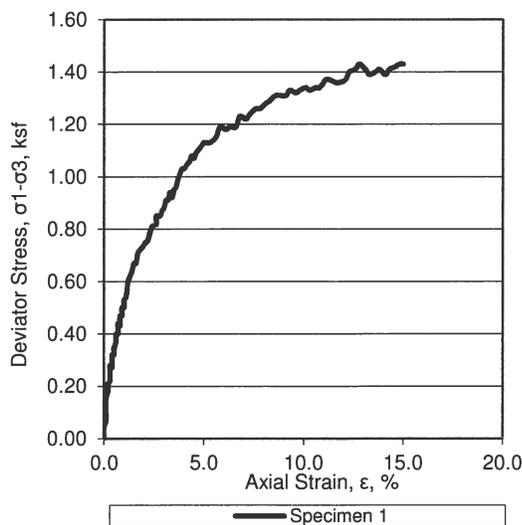
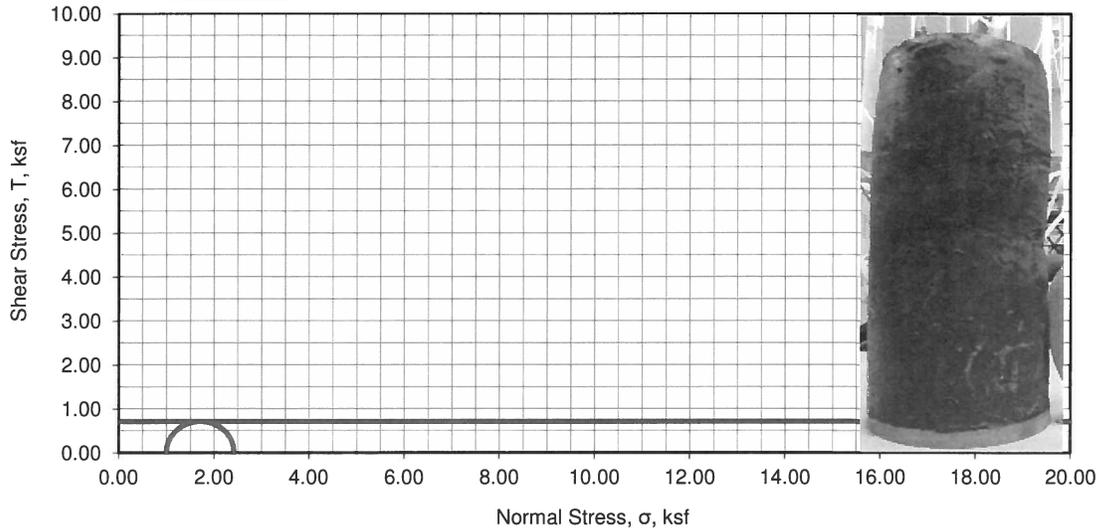
Membrane correction applied

Boring:	KB-10	Remarks: nm= not measured, na = not applicable
Sample:	3C	
Depth, ft:	5.0	
Test Date:	9/5/17	

	Project No.:	20181540	TRIAxIAL COMPRESSION TEST (UU) Quaker Hill 32 Acre Parcel Healdsburg, California	Figure 1 of 1 B-8
	Date:	9/6/17		
	Entry By:	CP		
	Checked By:	CP		
	File Name:	HL10523		

Total	
c =	0.71 ksf

Specimen Shear Picture



Specimen No.		1
Initial	Diameter, in	D _o 2.40
	Height, in	H _o 5.86
	Water Content, %	w _o 24.4
	Dry Density, lbs/ft ³	γ _d 101.2
	Saturation, %	S _o 102
	Void Ratio	e _o 0.634
Minor Principal Stress, ksf		σ ₃ 0.99
Maximum Deviator Stress, ksf		(σ ₁ -σ ₃) _{max} 1.43
Time to (σ ₁ -σ ₃) _{max} , min		t _f 14.85
Deviator Stress @ 15% Axial Strain, ksf		(σ ₁ -σ ₃) _{15%} 1.43
Ultimate Deviator Stress, ksf		(σ ₁ -σ ₃) _{ult} na
Rate of strain, %/min		'ε 1.00
Axial Strain at Failure, %		ε _f 14.85

Description of Specimen: Brown Lean Clay (CL)	
Amount of Material Finer than the No. 200, %:	nm
LL: nm	PL: nm
PI: nm	G _s : 2.65 Assumed
Specimen Type: Undisturbed	Test Method: ASTM D2850

Membrane correction applied

Boring:	KB-11	Remarks: nm= not measured, na = not applicable
Sample:	4B	
Depth, ft:	10.0	
Test Date:	9/5/17	

<p>KLEINFELDER Bright People. Right Solutions. 2601 Barrington Ct, Hayward, CA 94545</p>	Project No.:	20181540	TRIAXIAL COMPRESSION TEST (UU)	Figure 1 of 1
	Date:	9/6/17		
	Entry By:	CP	Quaker Hill 32 Acre Parcel Healdsburg, California	B-9
	Checked By:	CP		
	File Name:	HL10523		



Laboratory Test Report

Client: **Comstock, Crosser & Associates Development Company, Inc.** Report No.: **17-HAY-01424 Rev. 0** Issued: **8/29/2017**
Project: **20181540.001A** Field ID: **HL10476**
Quaker Hill Healdsburg 32 Acre Parcel Sampled by: Date: **8/7/2017**
03-000L - Laboratory Testing Submitted by: **M. Pucci** Date: **8/11/2017**

Expansion Index Test

Tested on **8/28/2017** by **C. Pimentel**
Material Description: **Dark Grayish Brown Lean Clay with Sand (CL)**
Sample Location: **KB-3 @ 1.5**

Test Method: **ASTM D4829**
Expansion Index : **63**
Dry Density, pcf : **106.6**
Water Content, as molded, %: **10.5**
Final Water Content, %: **25.3**
Initial Saturation, as molded %: **49.1**

Classification of Potentially Expansive Soil	
Expansion Index, EI	Potential Expansion
0 - 20	Very Low
21 - 50	Low
51 - 90	Medium
91 - 130	High
Above 130	Very High

Remarks:

Reviewed on 8/29/2017 by Cindy Pimentel,
Senior Technician

Limitations: Pursuant to applicable building codes, the results presented in this report are for the exclusive use of the client and the registered design professional in responsible charge. The results apply only to the samples tested. If changes to the specifications were made and not communicated to Kleinfelder, Kleinfelder assumes no responsibility for pass/fail statements (meets/did not meet), if provided. This report may not be reproduced, except in full, without written approval of Kleinfelder.



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Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the configuration-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be, and, in general, if you are the least bit uncertain about the continued reliability of this report, contact your geotechnical engineer before applying it.* A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual site-wide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may

perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old*.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, *proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration*. Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. *Geotechnical engineers are not building-envelope or mold specialists*.



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C.2 - Custom Soil Resource Report

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United States
Department of
Agriculture

NRCS

Natural
Resources
Conservation
Service

A product of the National
Cooperative Soil Survey,
a joint effort of the United
States Department of
Agriculture and other
Federal agencies, State
agencies including the
Agricultural Experiment
Stations, and local
participants

Custom Soil Resource Report for Sonoma County, California



Preface

Soil surveys contain information that affects land use planning in survey areas. They highlight soil limitations that affect various land uses and provide information about the properties of the soils in the survey areas. Soil surveys are designed for many different users, including farmers, ranchers, foresters, agronomists, urban planners, community officials, engineers, developers, builders, and home buyers. Also, conservationists, teachers, students, and specialists in recreation, waste disposal, and pollution control can use the surveys to help them understand, protect, or enhance the environment.

Various land use regulations of Federal, State, and local governments may impose special restrictions on land use or land treatment. Soil surveys identify soil properties that are used in making various land use or land treatment decisions. The information is intended to help the land users identify and reduce the effects of soil limitations on various land uses. The landowner or user is responsible for identifying and complying with existing laws and regulations.

Although soil survey information can be used for general farm, local, and wider area planning, onsite investigation is needed to supplement this information in some cases. Examples include soil quality assessments (<http://www.nrcs.usda.gov/wps/portal/nrcs/main/soils/health/>) and certain conservation and engineering applications. For more detailed information, contact your local USDA Service Center (<https://offices.sc.egov.usda.gov/locator/app?agency=nrcs>) or your NRCS State Soil Scientist (http://www.nrcs.usda.gov/wps/portal/nrcs/detail/soils/contactus/?cid=nrcs142p2_053951).

Great differences in soil properties can occur within short distances. Some soils are seasonally wet or subject to flooding. Some are too unstable to be used as a foundation for buildings or roads. Clayey or wet soils are poorly suited to use as septic tank absorption fields. A high water table makes a soil poorly suited to basements or underground installations.

The National Cooperative Soil Survey is a joint effort of the United States Department of Agriculture and other Federal agencies, State agencies including the Agricultural Experiment Stations, and local agencies. The Natural Resources Conservation Service (NRCS) has leadership for the Federal part of the National Cooperative Soil Survey.

Information about soils is updated periodically. Updated information is available through the NRCS Web Soil Survey, the site for official soil survey information.

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How Soil Surveys Are Made

Soil surveys are made to provide information about the soils and miscellaneous areas in a specific area. They include a description of the soils and miscellaneous areas and their location on the landscape and tables that show soil properties and limitations affecting various uses. Soil scientists observed the steepness, length, and shape of the slopes; the general pattern of drainage; the kinds of crops and native plants; and the kinds of bedrock. They observed and described many soil profiles. A soil profile is the sequence of natural layers, or horizons, in a soil. The profile extends from the surface down into the unconsolidated material in which the soil formed or from the surface down to bedrock. The unconsolidated material is devoid of roots and other living organisms and has not been changed by other biological activity.

Currently, soils are mapped according to the boundaries of major land resource areas (MLRAs). MLRAs are geographically associated land resource units that share common characteristics related to physiography, geology, climate, water resources, soils, biological resources, and land uses (USDA, 2006). Soil survey areas typically consist of parts of one or more MLRA.

The soils and miscellaneous areas in a survey area occur in an orderly pattern that is related to the geology, landforms, relief, climate, and natural vegetation of the area. Each kind of soil and miscellaneous area is associated with a particular kind of landform or with a segment of the landform. By observing the soils and miscellaneous areas in the survey area and relating their position to specific segments of the landform, a soil scientist develops a concept, or model, of how they were formed. Thus, during mapping, this model enables the soil scientist to predict with a considerable degree of accuracy the kind of soil or miscellaneous area at a specific location on the landscape.

Commonly, individual soils on the landscape merge into one another as their characteristics gradually change. To construct an accurate soil map, however, soil scientists must determine the boundaries between the soils. They can observe only a limited number of soil profiles. Nevertheless, these observations, supplemented by an understanding of the soil-vegetation-landscape relationship, are sufficient to verify predictions of the kinds of soil in an area and to determine the boundaries.

Soil scientists recorded the characteristics of the soil profiles that they studied. They noted soil color, texture, size and shape of soil aggregates, kind and amount of rock fragments, distribution of plant roots, reaction, and other features that enable them to identify soils. After describing the soils in the survey area and determining their properties, the soil scientists assigned the soils to taxonomic classes (units). Taxonomic classes are concepts. Each taxonomic class has a set of soil characteristics with precisely defined limits. The classes are used as a basis for comparison to classify soils systematically. Soil taxonomy, the system of taxonomic classification used in the United States, is based mainly on the kind and character of soil properties and the arrangement of horizons within the profile. After the soil

Custom Soil Resource Report

scientists classified and named the soils in the survey area, they compared the individual soils with similar soils in the same taxonomic class in other areas so that they could confirm data and assemble additional data based on experience and research.

The objective of soil mapping is not to delineate pure map unit components; the objective is to separate the landscape into landforms or landform segments that have similar use and management requirements. Each map unit is defined by a unique combination of soil components and/or miscellaneous areas in predictable proportions. Some components may be highly contrasting to the other components of the map unit. The presence of minor components in a map unit in no way diminishes the usefulness or accuracy of the data. The delineation of such landforms and landform segments on the map provides sufficient information for the development of resource plans. If intensive use of small areas is planned, onsite investigation is needed to define and locate the soils and miscellaneous areas.

Soil scientists make many field observations in the process of producing a soil map. The frequency of observation is dependent upon several factors, including scale of mapping, intensity of mapping, design of map units, complexity of the landscape, and experience of the soil scientist. Observations are made to test and refine the soil-landscape model and predictions and to verify the classification of the soils at specific locations. Once the soil-landscape model is refined, a significantly smaller number of measurements of individual soil properties are made and recorded. These measurements may include field measurements, such as those for color, depth to bedrock, and texture, and laboratory measurements, such as those for content of sand, silt, clay, salt, and other components. Properties of each soil typically vary from one point to another across the landscape.

Observations for map unit components are aggregated to develop ranges of characteristics for the components. The aggregated values are presented. Direct measurements do not exist for every property presented for every map unit component. Values for some properties are estimated from combinations of other properties.

While a soil survey is in progress, samples of some of the soils in the area generally are collected for laboratory analyses and for engineering tests. Soil scientists interpret the data from these analyses and tests as well as the field-observed characteristics and the soil properties to determine the expected behavior of the soils under different uses. Interpretations for all of the soils are field tested through observation of the soils in different uses and under different levels of management. Some interpretations are modified to fit local conditions, and some new interpretations are developed to meet local needs. Data are assembled from other sources, such as research information, production records, and field experience of specialists. For example, data on crop yields under defined levels of management are assembled from farm records and from field or plot experiments on the same kinds of soil.

Predictions about soil behavior are based not only on soil properties but also on such variables as climate and biological activity. Soil conditions are predictable over long periods of time, but they are not predictable from year to year. For example, soil scientists can predict with a fairly high degree of accuracy that a given soil will have a high water table within certain depths in most years, but they cannot predict that a high water table will always be at a specific level in the soil on a specific date.

After soil scientists located and identified the significant natural bodies of soil in the survey area, they drew the boundaries of these bodies on aerial photographs and

Custom Soil Resource Report

identified each as a specific map unit. Aerial photographs show trees, buildings, fields, roads, and rivers, all of which help in locating boundaries accurately.

Soil Map

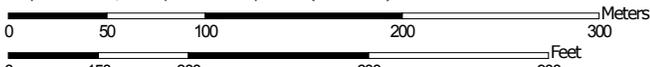
The soil map section includes the soil map for the defined area of interest, a list of soil map units on the map and extent of each map unit, and cartographic symbols displayed on the map. Also presented are various metadata about data used to produce the map, and a description of each soil map unit.

Custom Soil Resource Report Soil Map



Soil Map may not be valid at this scale.

Map Scale: 1:3,820 if printed on A portrait (8.5" x 11") sheet.



Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 10N WGS84

MAP LEGEND

Area of Interest (AOI)

 Area of Interest (AOI)

Soils

 Soil Map Unit Polygons

 Soil Map Unit Lines

 Soil Map Unit Points

Special Point Features

-  Blowout
-  Borrow Pit
-  Clay Spot
-  Closed Depression
-  Gravel Pit
-  Gravelly Spot
-  Landfill
-  Lava Flow
-  Marsh or swamp
-  Mine or Quarry
-  Miscellaneous Water
-  Perennial Water
-  Rock Outcrop
-  Saline Spot
-  Sandy Spot
-  Severely Eroded Spot
-  Sinkhole
-  Slide or Slip
-  Sodic Spot

-  Spoil Area
-  Stony Spot
-  Very Stony Spot
-  Wet Spot
-  Other
-  Special Line Features

Water Features

 Streams and Canals

Transportation

-  Rails
-  Interstate Highways
-  US Routes
-  Major Roads
-  Local Roads

Background

 Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:20,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
 Web Soil Survey URL:
 Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Sonoma County, California
 Survey Area Data: Version 11, Sep 21, 2017

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Dec 31, 2009—Nov 22, 2016

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
CgE	Clough gravelly loam, 15 to 30 percent slopes	7.2	16.2%
HbC	Haire gravelly loam, 0 to 9 percent slopes	4.9	11.0%
MoG	Montara cobbly clay loam, 30 to 75 percent slopes	10.7	24.3%
ZaA	Zamora silty clay loam, 0 to 2 percent slopes	21.5	48.5%
Totals for Area of Interest		44.3	100.0%

Map Unit Descriptions

The map units delineated on the detailed soil maps in a soil survey represent the soils or miscellaneous areas in the survey area. The map unit descriptions, along with the maps, can be used to determine the composition and properties of a unit.

A map unit delineation on a soil map represents an area dominated by one or more major kinds of soil or miscellaneous areas. A map unit is identified and named according to the taxonomic classification of the dominant soils. Within a taxonomic class there are precisely defined limits for the properties of the soils. On the landscape, however, the soils are natural phenomena, and they have the characteristic variability of all natural phenomena. Thus, the range of some observed properties may extend beyond the limits defined for a taxonomic class. Areas of soils of a single taxonomic class rarely, if ever, can be mapped without including areas of other taxonomic classes. Consequently, every map unit is made up of the soils or miscellaneous areas for which it is named and some minor components that belong to taxonomic classes other than those of the major soils.

Most minor soils have properties similar to those of the dominant soil or soils in the map unit, and thus they do not affect use and management. These are called noncontrasting, or similar, components. They may or may not be mentioned in a particular map unit description. Other minor components, however, have properties and behavioral characteristics divergent enough to affect use or to require different management. These are called contrasting, or dissimilar, components. They generally are in small areas and could not be mapped separately because of the scale used. Some small areas of strongly contrasting soils or miscellaneous areas are identified by a special symbol on the maps. If included in the database for a given area, the contrasting minor components are identified in the map unit descriptions along with some characteristics of each. A few areas of minor components may not have been observed, and consequently they are not mentioned in the descriptions, especially where the pattern was so complex that it was impractical to make enough observations to identify all the soils and miscellaneous areas on the landscape.

Custom Soil Resource Report

The presence of minor components in a map unit in no way diminishes the usefulness or accuracy of the data. The objective of mapping is not to delineate pure taxonomic classes but rather to separate the landscape into landforms or landform segments that have similar use and management requirements. The delineation of such segments on the map provides sufficient information for the development of resource plans. If intensive use of small areas is planned, however, onsite investigation is needed to define and locate the soils and miscellaneous areas.

An identifying symbol precedes the map unit name in the map unit descriptions. Each description includes general facts about the unit and gives important soil properties and qualities.

Soils that have profiles that are almost alike make up a *soil series*. Except for differences in texture of the surface layer, all the soils of a series have major horizons that are similar in composition, thickness, and arrangement.

Soils of one series can differ in texture of the surface layer, slope, stoniness, salinity, degree of erosion, and other characteristics that affect their use. On the basis of such differences, a soil series is divided into *soil phases*. Most of the areas shown on the detailed soil maps are phases of soil series. The name of a soil phase commonly indicates a feature that affects use or management. For example, Alpha silt loam, 0 to 2 percent slopes, is a phase of the Alpha series.

Some map units are made up of two or more major soils or miscellaneous areas. These map units are complexes, associations, or undifferentiated groups.

A *complex* consists of two or more soils or miscellaneous areas in such an intricate pattern or in such small areas that they cannot be shown separately on the maps. The pattern and proportion of the soils or miscellaneous areas are somewhat similar in all areas. Alpha-Beta complex, 0 to 6 percent slopes, is an example.

An *association* is made up of two or more geographically associated soils or miscellaneous areas that are shown as one unit on the maps. Because of present or anticipated uses of the map units in the survey area, it was not considered practical or necessary to map the soils or miscellaneous areas separately. The pattern and relative proportion of the soils or miscellaneous areas are somewhat similar. Alpha-Beta association, 0 to 2 percent slopes, is an example.

An *undifferentiated group* is made up of two or more soils or miscellaneous areas that could be mapped individually but are mapped as one unit because similar interpretations can be made for use and management. The pattern and proportion of the soils or miscellaneous areas in a mapped area are not uniform. An area can be made up of only one of the major soils or miscellaneous areas, or it can be made up of all of them. Alpha and Beta soils, 0 to 2 percent slopes, is an example.

Some surveys include *miscellaneous areas*. Such areas have little or no soil material and support little or no vegetation. Rock outcrop is an example.

Sonoma County, California

CgE—Clough gravelly loam, 15 to 30 percent slopes

Map Unit Setting

National map unit symbol: hfbq
Elevation: 200 to 1,000 feet
Mean annual precipitation: 35 inches
Mean annual air temperature: 61 degrees F
Frost-free period: 200 to 250 days
Farmland classification: Not prime farmland

Map Unit Composition

Clough and similar soils: 85 percent
Minor components: 15 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Clough

Setting

Landform: Terraces
Landform position (two-dimensional): Backslope
Landform position (three-dimensional): Tread
Down-slope shape: Concave
Across-slope shape: Convex
Parent material: Gravelly alluvium derived from sedimentary rock

Typical profile

H1 - 0 to 10 inches: gravelly loam
H2 - 10 to 23 inches: very gravelly clay
H3 - 23 to 38 inches: indurated
H4 - 38 to 60 inches: stratified very gravelly loam

Properties and qualities

Slope: 15 to 30 percent
Depth to restrictive feature: About 10 inches to abrupt textural change; 20 to 40 inches to duripan
Natural drainage class: Moderately well drained
Runoff class: Very high
Capacity of the most limiting layer to transmit water (Ksat): Very low (0.00 to 0.00 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Available water storage in profile: Very low (about 1.4 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 6e
Hydrologic Soil Group: D
Ecological site: SHALLOW LOAMY UPLANDS (R015XD129CA)
Hydric soil rating: No

Minor Components

Positas

Percent of map unit: 8 percent
Hydric soil rating: No

Haire

Percent of map unit: 7 percent
Hydric soil rating: No

HbC—Haire gravelly loam, 0 to 9 percent slopes

Map Unit Setting

National map unit symbol: hfdn
Elevation: 20 to 2,400 feet
Mean annual precipitation: 20 to 45 inches
Mean annual air temperature: 54 to 57 degrees F
Frost-free period: 200 to 300 days
Farmland classification: Farmland of statewide importance

Map Unit Composition

Haire and similar soils: 85 percent
Minor components: 15 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Haire

Setting

Landform: Terraces
Landform position (two-dimensional): Footslope
Landform position (three-dimensional): Base slope
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: Alluvium derived from sedimentary rock

Typical profile

H1 - 0 to 24 inches: gravelly loam
H2 - 24 to 36 inches: clay
H3 - 36 to 60 inches: very gravelly clay loam

Properties and qualities

Slope: 0 to 9 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Moderately well drained
Runoff class: High
Capacity of the most limiting layer to transmit water (Ksat): Moderately low to moderately high (0.06 to 0.20 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None

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Available water storage in profile: Moderate (about 6.3 inches)

Interpretive groups

*Land capability classification (irrigated): 2e
Land capability classification (nonirrigated): 3e
Hydrologic Soil Group: C
Ecological site: CLAYPAN (R014XC010CA)
Hydric soil rating: No*

Minor Components

Diablo

*Percent of map unit: 7 percent
Hydric soil rating: No*

Arbuckle

*Percent of map unit: 6 percent
Hydric soil rating: No*

Clear lake

*Percent of map unit: 2 percent
Landform: Depressions
Hydric soil rating: Yes*

MoG—Montara cobbly clay loam, 30 to 75 percent slopes

Map Unit Setting

*National map unit symbol: hfhd
Elevation: 100 to 3,000 feet
Mean annual precipitation: 12 to 50 inches
Mean annual air temperature: 57 to 61 degrees F
Frost-free period: 175 to 350 days
Farmland classification: Not prime farmland*

Map Unit Composition

*Montara and similar soils: 85 percent
Minor components: 15 percent
Estimates are based on observations, descriptions, and transects of the mapunit.*

Description of Montara

Setting

*Landform: Hills
Landform position (two-dimensional): Backslope
Landform position (three-dimensional): Side slope
Down-slope shape: Convex
Across-slope shape: Convex
Parent material: Residuum weathered from serpentinite*

Typical profile

*H1 - 0 to 10 inches: cobbly clay loam
H2 - 10 to 20 inches: unweathered bedrock*

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Properties and qualities

Slope: 30 to 50 percent
Depth to restrictive feature: 10 to 20 inches to lithic bedrock
Natural drainage class: Well drained
Runoff class: Very high
Capacity of the most limiting layer to transmit water (Ksat): Very low to moderately low (0.00 to 0.06 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Salinity, maximum in profile: Nonsaline to very slightly saline (0.0 to 2.0 mmhos/cm)
Available water storage in profile: Very low (about 1.3 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 7e
Hydrologic Soil Group: D
Ecological site: SERPENTINE LAND (R015XD133CA)
Hydric soil rating: No

Minor Components

Henneke

Percent of map unit: 5 percent
Hydric soil rating: No

Raynor

Percent of map unit: 5 percent
Hydric soil rating: No

Rock outcrop

Percent of map unit: 5 percent
Hydric soil rating: No

ZaA—Zamora silty clay loam, 0 to 2 percent slopes

Map Unit Setting

National map unit symbol: hfl3
Elevation: 30 to 1,300 feet
Mean annual precipitation: 22 inches
Mean annual air temperature: 61 degrees F
Frost-free period: 250 to 330 days
Farmland classification: Prime farmland if irrigated

Map Unit Composition

Zamora and similar soils: 85 percent
Minor components: 15 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Zamora

Setting

Landform: Alluvial fans
Landform position (two-dimensional): Backslope
Landform position (three-dimensional): Side slope
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: Alluvium derived from sedimentary rock

Typical profile

H1 - 0 to 5 inches: silty clay loam
H2 - 5 to 29 inches: clay loam
H3 - 29 to 41 inches: clay loam
H4 - 41 to 55 inches: sandy clay loam
H5 - 55 to 60 inches: gravelly clay

Properties and qualities

Slope: 0 to 2 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Well drained
Runoff class: Medium
Capacity of the most limiting layer to transmit water (Ksat): Moderately high (0.20 to 0.57 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Available water storage in profile: High (about 10.0 inches)

Interpretive groups

Land capability classification (irrigated): 1
Land capability classification (nonirrigated): 3c
Hydrologic Soil Group: C
Hydric soil rating: No

Minor Components

Cole

Percent of map unit: 4 percent
Hydric soil rating: No

Yolo

Percent of map unit: 4 percent
Hydric soil rating: No

Cortina

Percent of map unit: 3 percent
Hydric soil rating: No

Pajaro

Percent of map unit: 3 percent
Hydric soil rating: No

Unnamed

Percent of map unit: 1 percent
Landform: Depressions
Hydric soil rating: Yes

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