RGH
CONSULTANTS

Experience is the difference

GEOTECHNICAL STUDY REPORT

BRIAN ARDEN WINERY
APN 011-050-030
SILVERADO TRAIL
CALISTOGA, CALIFORNIA

Project Number:

6569.01.04.2

Prepared For:

Burt and Brian Harlan
P.O. Box 96
Calistoga, California 94515

Prepared By:

RGH Consultants, Inc.

Napa Office
P.O. Box 10830
Napa, California 94581
(707) 252-8105

Juan B. Hidalgo
Principal Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
JUAN B. HIDALGO
GE 398
Exp. 3/31/13
GEOTECHNICAL
STATE OF CALIFORNIA

Travis A. Whitted
Senior Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
TRAVIS A. WHITTED
No. 2782
Exp. 9/30/2012
GEOTECHNICAL
STATE OF CALIFORNIA

June 8, 2011

TABLE OF CONTENTS

TABLE OF CONTENTS (cont'd)

INTRODUCTION

This report presents the results of our geotechnical study for the Brian Arden Winery to be constructed at APN 011-050-030 off Silverado Trail in Calistoga, California. The ± 2-acre, triangular-shaped property extends primarily over relatively level alluvial fan terrain near the base of the eastern flank of an isolated wooded knoll (Mount Washington) and sits about 1500 feet southeast of the intersection of Rosedale Road and Silverado Trail. The site location is shown on Plate 1, Appendix A.

We understand the project includes the construction of a winery building, a guest house and a crush/fermentation tank pad. We understand the winery building will be a two-story, wood-frame structure with slab-on-grade floors and will likely be set into the knoll. The guest house will be a single-story, wood-frame structure with either slab-on-grade or structurally supported wood floors. The crush/fermentation tank pad will sit on adjoining concrete aprons and will be covered by a steel frame canopy. Retaining walls may be needed to provide level breaks across the building site. Vehicular access and parking will be provided by asphalt paved driveways and stalls.

Foundation loads are expected to be typical of the light to moderately heavy type of construction proposed. We anticipate wall and isolated column loads will range from 1 to 2 kips per lineal foot and 40 to 60 kips, respectively. We understand site grading will require from a few to several feet of cut and fill to construct level building pads and paved areas with positive drainage.

SCOPE

The purpose of our study, as outlined in our Professional Service Agreement dated April 18, 2011, was to generate geotechnical information for the design and construction of the project. Our scope of services included reviewing selected published geologic data pertinent to the site; evaluating subsurface conditions with borings and laboratory tests; analyzing the field and laboratory data; and presenting this report with the following geotechnical information:

1. A brief description of soil and groundwater conditions observed during our study;
2. A discussion of seismic hazards that may affect the proposed project;
3. Seismic design criteria per guidelines in the 2010 CBC; and
4. Conclusions and recommendations regarding:
    a. Primary geotechnical engineering concerns and mitigating measures, as applicable;
    b. Site preparation and grading including remedial grading of weak, porous, compressible and/or expansive, creep-prone surface soils;
    c. Foundation type(s), design criteria and estimated settlement behavior;
    d. Lateral loads for retaining wall design;
    e. Support of concrete slabs-on-grade;

f. Preliminary pavement thickness based on our experience with similar soils and projects;

g. Utility trench backfill;

h. Geotechnical engineering drainage improvements; and

i. Supplemental geotechnical engineering services.

STUDY

Site Exploration

We reviewed our previous geotechnical studies in the vicinity and selected geologic references pertinent to the site. The geologic literature reviewed is listed in Appendix B.

On May 9, 2011, we performed a geotechnical reconnaissance of the site and explored the subsurface conditions by drilling four test borings to depths ranging from about 10½ to 26½ feet. The borings were drilled with a truck-mounted drill rig equipped with 6-inch diameter, solid stem and 8-inch diameter hollow stem augers at the approximate locations shown on the Exploration Plan, Plate 2. The test boring locations were determined approximately by pacing their distance from features shown on the Exploration Plan and should be considered accurate only to the degree implied by the method used. Our geotechnical engineer located and logged the borings and obtained samples of the materials encountered for visual examination, classification and laboratory testing.

Relatively undisturbed samples were obtained from the borings at selected intervals by driving a 2.43-inch inside diameter, split spoon sampler, containing 6-inch long brass liners, using a 140-pound hammer dropping approximately 30 inches. The sampler was driven 12 to 18 inches. The blows required to drive each 6-inch increment were recorded and the blows required to drive the last 12 inches, or portion thereof, were converted to equivalent Standard Penetration Test (SPT) blow counts for correlation with empirical data. Disturbed samples were also obtained at selected depths by driving a 1.375-inch inside diameter (2-inch outside diameter) SPT sampler, without liners or rings, using a 140-pound hammer dropping approximately 30 inches. The sampler was driven 12 to 18 inches, the blows to drive each 6-inch increment were recorded, and the blows required to drive the final 12 inches, or portion thereof, are provided on the boring logs. Disturbed "grab" samples were also obtained at selected depths from the borings and placed in plastic bags.

The logs of the borings showing the materials encountered, groundwater conditions, converted blow counts and sample depths are presented on Plates 3 through 6. The soils are described in accordance with the Unified Soil Classification System, outlined on Plate 7.

The boring logs show our interpretation of subsurface soil and groundwater conditions on the date and at the locations indicated. Subsurface conditions may vary at other locations and times. Our interpretation is based on visual inspection of soil samples, laboratory test results, and interpretation of drilling and sampling resistance. The location of the soil boundaries should be considered approximate. The transition between soil types may be gradual.

Laboratory Testing

The samples obtained from the borings were transported to our office and re-examined by the project engineer to verify soil classifications, evaluate characteristics, and assign tests pertinent to our analysis. Selected samples were laboratory tested to determine their classification (Atterberg Limits, percent of silt and clay). The test results are presented on the boring logs and on Plate 8.

SITE CONDITIONS

General

Napa County is located within the California Coast Range geomorphic province. This province is a geologically complex and seismically active region characterized by sub-parallel northwest-trending faults, mountain ranges and valleys. The oldest bedrock units are the Jurassic-Cretaceous Franciscan Complex and Great Valley sequence sediments originally deposited in a marine environment. Subsequently, younger rocks such as the Tertiary-age Sonoma Volcanics group, the Plio-Pleistocene-age Clear Lake Volcanics and sedimentary rocks such as the Guinda, Domengine, Petaluma, Wilson Grove, Cache, Huichica and Glen Ellen formations were deposited throughout the province. Extensive folding and thrust faulting during late Cretaceous through early Tertiary geologic time created complex geologic conditions that underlie the highly varied topography of today. In valleys, the bedrock is covered by thick alluvial soils. The site is located within the northwestern portion of the Napa Valley. The Napa Valley is a long, narrow northwest-trending alluvial plain flanked by northwest-trending mountain ridges.

Geology

Published geologic maps (Graymer et al., 2007) indicate the property is underlain by Holocene and Late Pleistocene alluvial fan deposits. The alluvium is shown to comprise poorly sorted, moderately to poorly bedded sand, gravel, silt and clay deposited in gently sloping fans.

Surface

The property extends primarily over relatively level to gently sloping alluvial terrain near the base of an isolated wooded knoll. The vegetation consists primarily of native grass. Some of the proposed building sites will be tucked into the moderately sloping flanks of the knoll.

In general, the ground surface is soft and spongy. This is a condition generally associated with weak, porous surface soils. On sloping terrain (5:1 or steeper), such as on the knoll flanks, the surface materials (topsoil, colluvium) undergo a gradual downhill movement known as creep. Soil creep is inherent to hillsides in the area and its force is directly proportional to slope inclination, the soil's plasticity, water content and expansion potential.

Natural drainage consists of sheet flow over the ground surface and slopes. The runoff tends to concentrate in man made surface drainage elements such as roadside ditches and natural drainage elements such as nearby swales, ravines and creeks.

Subsurface

Our borings and laboratory tests indicate that the portion of the site we studied is blanketed by
3½ to 6 feet of weak, porous, compressible, clayey surface soils. Porous soils appear hard and
strong when dry but become weak and compressible as their moisture content increases
towards saturation. These soils exhibit low to moderate plasticity (LL = 38; PI = 13) and low
expansion potential. These surface materials generally are underlain by relatively strong and
incompressible, discontinuous and alternating layers of clayey sands and gravels to the
maximum depths explored. In Boring B-3 we encountered loose clayey gravel to a depth of 9
feet. As previously discussed, on hillsides 5:1 or steeper, such as the knoll flanks, the surface
materials typically creep.

A detailed description of subsurface conditions found in our borings is given on Plates 3 through 6, Appendix A. Based on Table 1613.5.2 of the 2010 California Building Code (CBC), we have determined a Site Class of D should be used for the site.

Corrosion Potential

Mapping by the Natural Resources Conservation Service (2011) indicates that the corrosion potential of the near surface soil is high for uncoated steel and moderate for concrete. Performing corrosivity tests to verify these values was not part of our requested and/or proposed scope of work. Should the need arise, we would be pleased to provide a proposal to evaluate these characteristics.

Groundwater

Free groundwater was first detected in Borings 1, 2 and 3 at depths ranging from 5½ to 7½ feet below the ground surface at the time of drilling. Free groundwater was not observed in Boring B-4. Fluctuation in the groundwater level typically occurs because of a variation in rainfall intensity, duration and other factors such as flooding and periodic irrigation.

Flooding

Our review of the Federal Emergency Management Agency (FEMA) Flood Zone Map for Napa County, California, City of Calistoga (Map No. 06055C0229E dated September 26,2008, indicates that the proposed building site is located within Zone "X," an area determined to be outside the 0.2 percent annual chance floodplain. Evaluation of flooding potential is typically the responsibility of the project civil engineer.

DISCUSSION AND CONCLUSIONS

Seismic Hazards

General

We did not observe subsurface conditions within the portion of the property we studied that would suggest the presence of materials that may be susceptible to seismically induced lurching. Therefore, we judge the potential for the occurrence of these phenomena at the site to be low.

Seismicity

Data presented by the Working Group on California Earthquake Probabilities (2007) estimates the chance of one or more large earthquakes (Magnitude 6.7 or greater) in the San Francisco Bay region within the next 30 years to be approximately 63 percent. Therefore, future seismic shaking should be anticipated at the site. It will be necessary to design and construct the proposed winery facility in strict adherence with current standards for earthquake-resistant construction.

Faulting

We did not observe landforms within the area that would indicate the presence of active faults and the site is not within a current Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007). Therefore, we believe the risk of fault rupture at the site is low. However, the site is within an area affected by strong seismic activity. Several northwest-trending Earthquake Fault Zones exist in close proximity to and within several miles of the site (Bortugno, 1982). The shortest distances from the site to the mapped surface expression of these faults are presented in the table below.

ACTIVE FAULT PROXIMITY

Fault	Direction	Distance-Miles
San Andreas	SW	32
Healdsburg-Rodgers Creek	SW	20½
Concord-Green Valley	SE	26
Cordelia	SE	32
West Napa	SE	16
Maacama	W	8
Konocti	N	23
Hunting Creek	NE	16

Liquefaction

Liquefaction is a rapid loss of shear strength experienced in saturated, predominantly granular soils below the groundwater level during strong earthquake ground shaking due to an increase in pore water pressure. The occurrence of this phenomenon is dependent on many complex factors including the intensity and duration of ground shaking, particle size distribution and density of the soil.

Granular soils were encountered at the site below the groundwater table. Therefore, we performed an analysis of the blow count data from our borings using the methods of Seed and Idriss (1982), Seed and others (1985), Youd and Idriss (2001), Idriss and Boulanger (2004) and Idriss and Boulanger (2008). These procedures normalize the blow counts to account for overburden pressure, rod length, hammer energy, and fines (percent of silt and clay) content. Once the blow counts are normalized and adjusted to a clean sand blow count, the cyclic resistance ratio (CRR) for each blow count is then determined using the same procedures referenced above. The CRR is compared to the cyclic stress ratio (CSR) induced by the earthquake. Calculating the CSR requires a peak ground acceleration and design earthquake magnitude.

Peak ground acceleration (PGA) was determined using the methods in the 2010 California Building Code (CBC) and Chapter 11 of the American Society of Civil Engineers (ASCE) Standard 7-05, titled "Minimum Design Loads for Buildings and Other Structures" (2006). Section 11.8.3 of ASCE Standard 7-05 states that the PGA for liquefaction evaluation can be defined as the design spectral response acceleration at short periods with 5 percent damping (S_{DS}) divided by 2.5. The S_{DS} value is determined using the United States Geological Survey's Earthquake Ground Motion Parameter Java Application (2008). Based on the site's latitude and longitude of 38.5828°N and -122.5660°W, respectively, and a site soil Class of D, the S_{DS} value is 0.85g. Therefore, the PGA used for our evaluation is 0.34g.

The West Napa fault is most likely controlling the ground motions at the site. According to Petersen (1996), the West Napa fault is capable of a M_M 6.5 earthquake. Using the scaling factors and confining stress corrections presented in Idriss and Boulanger (2004), the critical CSR at the site ranges from 0.22 to 0.31. Some of the CRR values at the site do not exceed these values. Therefore, we judge that the potential for liquefaction at the site is moderate to high, particularly in the area of Boring B-3 where we encountered a layer of loose gravel from 3½ to 9 feet.

There are three potential consequences of liquefaction: bearing capacity failure, lateral spreading toward a free face (e.g. riverbank) and settlement. Bearing capacity failure is sudden and extreme settlement of foundations that typically occurs when the liquefied layer is relatively close (typically within two times the footing width, depending on the loads) to the bottom of the foundation. Because the liquefiable layer is 3½ feet below the ground surface, we judge that there is a moderate to high potential for bearing capacity failure in the area of Boring B-3. If grading is performed to strengthen the upper 5 to 6 feet of loose gravel as subsequently discussed, the potential for bearing capacity failure will be low.

Lateral spreading can occur where continuous layers of liquefiable soil extend to a free face, such as a creek bank. There are no significant free faces in the vicinity of the site. Therefore, we judge the potential for liquefaction-induced lateral spreading at the site is low.

The third potential consequence of liquefaction is settlement due to densification of the liquefied soils. Potential settlements based on the blow count data and cyclic stress ratio were calculated using the methods of Ishihara and Yoshimine (1992). For the layers encountered in Borings 1, 2 and 4 we calculated total settlement ranging from ¼ to ½ inch. For the layers encountered in Boring B-3, we calculated total settlement ranging from 1¼ to 2½ inches. Differential settlement could range from 1¼ to 2½ inches. If grading is performed to strengthen the upper 5 to 6 feet of loose gravel, as subsequently discussed, the expected settlement in the area of Boring B-3 would be reduced to about 1 inch.

Densification

Densification is the settlement of loose, granular soils above the groundwater level due to earthquake shaking. Typically, granular soils that would be susceptible to liquefaction, if saturated, are susceptible to densification. As discussed in the "Liquefaction" section, the soils at the site have a moderate to high potential for liquefaction. However, because the loose soils are generally saturated, we judge that there is a low potential for densification to impact structures at the site.

Geotechnical Issues

General

Based on our study, we judge the proposed Winery facility can be built as planned, provided the recommendations presented in this report are incorporated into its design and construction. The primary geotechnical concerns during design and construction of the project are:

1. The presence of 3½ to 5½ feet of weak, porous, compressible, clayey surface soils and local areas with loose gravel to a depth of about 9 feet;

2. The detrimental effects of uncontrolled surface runoff and groundwater seepage on the long-term satisfactory performance of wineries, especially those constructed on, or near, hillsides/alluvial fans, given the erosion potential and porous nature of the surface soils; and

3. The strong ground shaking predicted to impact the site during the life of the project.

Weak, Porous Surface Soils and Loose Gravels

Weak, porous surface soils, such as those found at the site, appear hard and strong when dry but will lose strength rapidly and settle under the load of fills, foundations, slabs, and pavements as their moisture content increases and approaches saturation. The moisture content of these soils can increase as the result of rainfall, periodic irrigation or when the natural upward migration of water vapor through the soils is impeded by, and condenses under fills, foundations, slabs, and pavements. Loose gravels such as those encountered in Boring B-3 can settle due to liquefaction. The detrimental effects of such movements can be remediated by strengthening the soils during grading. This can be achieved by excavating the weak, compressible soils and loose gravels, and replacing them as properly compacted (engineered) fill. Alternatively, satisfactory foundation support could be obtained from the relatively strong and incompressible, gravelly soils found below the weak surface soils.

Downslope Creep

Weak, creep-prone surface soils, such as those found on the sloping portions of the site, tend to naturally consolidate and settle on sloping terrain that is 5:1 (horizontal to vertical) or steeper. Fills and foundations deriving support from these materials will be susceptible and contribute to the downslope creep and settlement unless properly embedded in bedrock or buttressed (keyed, benched, drained and compacted). The settlement causes cracks in the slabs and structural distress in the form of cracked plaster and sticky doors and windows. Therefore, it will be necessary to obtain fill and/or foundation support below the creeping soils and, outside buttressed areas, design the foundations to resist stresses imposed by the creeping soils.

Fill Support - Hillside fills need to be constructed on level keyways and benches excavated entirely on rock. However, regardless of the care used during grading, buttressed fills of uneven thickness such as those typically built on hillsides, will settle differentially. Satisfactory performance of structural elements constructed on hillside fills will require the use of specialized grading techniques discussed in the following sections of this report. These include excavating all creeping soils and replacing these materials as a buttressed fill of even thickness or constructing the improvements entirely on cut. For the purpose of this discussion, fills with a differential thickness of less than 5 feet can be assumed to have equal thickness. In order to provide the equal thicknesses, it may be necessary to overexcavate at least a few feet in cut areas. Where the total fill thickness is less than 3 feet, the fill can be placed at 95 percent relative compaction in lieu of overexcavation in cut areas.

Foundation, Slab and Pavement Support - Satisfactory foundation support for the proposed structures can be obtained from spread footings that bottom at minimum depth on engineered fill, on firm bedrock exposed by planned excavations or in bedrock reached by footings excavated through the creeping soils. Where the creeping soils are not buttressed or removed by grading, the footings must be designed to resist creep forces. Spread footings can also be used for foundation support where the building pad transitions from bedrock to fill and the fill is less than 3 feet thick, provided the fills are compacted to at least 95 percent relative compaction. Interior slab-on-grade floors, exterior slabs and pavements can also be satisfactorily supported on the engineered fill.

Floor Systems - Slab-on-grade floors can be used in the interior area provided that:

1. The planned grading either removes the weak, compressible and/or creep-prone surface soils or increases their supporting capacity by mechanical compaction;

2. The upper 5 to 6 feet of loose gravel in the area of Boring B-3 is strengthened during grading;

3. The subgrade materials are pre-swelled by soaking prior to installation of the slabs;

4. The slabs are reinforced to reduce cracks;

5. The slabs are grooved to induce cracking in a non-obtrusive manner; and

6. The slab area is underlain by engineered fill, firm rock or bedrock and fill (3 feet thick or less) placed at 95 percent relative compaction or buttressed fills of even thickness, entirely.

Excavation Difficulty

Site excavation, especially in sloping areas, may encounter hard, resistant bedrock a few feet below the surface. Site excavations, including utility trenches will require heavy ripping and jack hammering. The contractors and subcontractors bidding this job should read this report and become familiar with site conditions as they pertain to their operation and the appropriate equipment needed to perform their tasks. If more detailed information regarding excavatability of the bedrock is required, a seismic refraction study should be performed or additional test pits should be excavated using the type and size of equipment planned for construction.

On-Site Soil Quality

All fill materials used in the building area must be select, as subsequently described in "Recommendations." We anticipate that, with the exception of organic matter and of rocks or lumps larger than 6 inches in diameter, the excavated material will be suitable for re-use as general and select fill.

Select Fill

The select fill can consist of approved on-site soils or import materials with a low expansion potential. The geotechnical engineer must approve the use of on-site soils as select fill during grading.

Settlement

If remedial grading is performed and the spread footings are installed in accordance with the recommendations presented in this report, we estimate that post-construction differential settlements across the building will be about one inch. Seismically induced liquefaction settlement could also be about 1 inch.

Surface Drainage

Because of topography and location, the site will be impacted by surface runoff from the upgradient slopes. In addition, the site soils are susceptible to erosion and sloughing. Surface runoff typically sheet flows over the ground surface and slopes but can be concentrated by the planned site grading, landscaping, and drainage. The ensuing erosion can create sloughing and promote slope instability or the surface runoff can pond against structures and seep into the slab rock. Therefore, strict control of surface runoff is necessary to provide long-term satisfactory performance of projects constructed on, or near, hillsides. It will be necessary to divert surface runoff around slopes and improvements, provide positive drainage away from structures, and install energy dissipaters at discharge points of concentrated runoff. This can be achieved by constructing the building pad several inches above the surrounding area and conveying the runoff into man made drainage elements or natural swales that lead downgradient of the site.

Groundwater

We anticipate that rainwater will percolate through the porous topsoil and migrate downslope at the interface of the topsoil and bedrock and through fractures in the bedrock and seep into the slab rock. Groundwater will also seep into excavations, such as structures partially recessed into hillsides, that expose the water migration zone or into hillside fills. Therefore, it will be necessary to intercept, collect and divert groundwater outside of the proposed improvements. This can be accomplished by installing retaining wall backdrains and slab underdrains, as recommended herein.

RECOMMENDATIONS

Seismic Design

Seismic design parameters presented below are based on Section 1613 titled "Earthquake Loads" of the 2010 California Building Code (CBC). Based on CBC Table 1613.5.2, we have determined a Site Class D should be used for the subject site. Using a site latitude and longitude of 38.5828°N and -122.5660°W, respectively, and the United States Geological Survey's Earthquake Ground Motion Parameter Java Application (USGS, 2008) we recommend that the following seismic design criteria be used for structures at the site.

2010 CBC Seismic Criteria	
Spectral Response Parameter	**Acceleration (g)**
S_S (0.2 second period)	1.27
S_1 (1 second period)	0.54
S_{MS} (0.2 second period)	1.27
S_{M1} (1 second period)	0.80
S_{DS} (0.2 second period)	0.85
S_{D1} (1 second period)	0.54

Grading

Site Preparation

Areas to be developed should be cleared of vegetation and debris. Trees and shrubs that will not be part of the proposed development should be removed and their primary root systems grubbed. Cleared and grubbed material should be removed from the site and disposed of in accordance with County Health Department guidelines. We did not observe septic tanks, leach lines or underground fuel tanks during our study. Any such appurtenances found during grading should be capped and sealed and/or excavated and removed from the site, respectively, in accordance with established guidelines and requirements of the County Health Department. Voids created during clearing should be backfilled with engineered fill as recommended herein.

Stripping

Areas to be graded should be stripped of the upper few inches of soil containing organic matter. Soil containing more than two percent by weight of organic matter should be considered organic. Actual stripping depth should be determined by a representative of the geotechnical engineer in the field at the time of stripping. The strippings should be removed from the site, or if suitable, stockpiled for re-use as topsoil in landscaping.

Excavations

Following initial site preparation, excavation should be performed as planned or recommended herein. Excavations extending below the proposed finished grade should be backfilled with suitable materials compacted to the requirements given below.

Within fill and building areas, the weak, porous, compressible and or creep-prone surface soils should be excavated to within 6 inches of their entire depth (up to about 5½ feet in our borings). In addition, the loose gravel in the area of Boring B-3 should be excavated to a depth of about 6 feet. The excavation of weak, compressible surface soils should also extend at least 12 inches below exterior slab and pavement subgrade (where planned excavations do not completely remove the weak soils). On sloping terrain 5:1 or steeper, fills should be constructed by excavating level keyways that expose undisturbed bedrock. The keyways should be at least 10 feet wide, extend at least one 3 feet below the existing ground surface on the downhill side and should be sloped to drain to the rear. Keyway excavations should extend laterally to at least a 1:1 imaginary line extending down from the toe of the fill. Keyway subdrains are discussed hereinafter in "Subsurface Drainage."

The excavation of loose gravel and weak, porous, compressible, creep-prone surface materials should extend at least 5 feet beyond the outside edge of the exterior footings of the proposed buildings and tank farm, and 3 feet beyond the edge of exterior slabs and pavements and three feet beyond the toe of new fills that are not supported by keyways. The excavated materials should be stockpiled for later use as compacted fill, or removed from the site, as applicable. Excavation of hard resistant bedrock at the site may require heavy ripping and/or jack hammering. The grading contractor should review this report, become familiar with site conditions as they pertain to his operation and draw his own conclusions regarding excavation difficulty and suitable grading equipment. The bids for accomplishing the remedial grading recommended herein should include the cost for overexcavation and recompaction either on a lump sum or unit price basis.

At all times, temporary construction excavations should conform to the regulations of the State of California, Department of Industrial Relations, Division of Industrial Safety or other stricter governing regulations. The stability of temporary cut slopes, such as those constructed during the installation of underground utilities, should be the responsibility of the contractor. Depending on the time of year when grading is performed, and the surface conditions exposed, temporary cut slopes may need to be excavated to 1½:1, or flatter. The tops of the temporary cut slopes should be rounded back to 2:1 in weak soil zones.

Subsurface Drainage

A subdrain should be installed at the rear of the keyways and/or where evidence of seepage is observed. The subdrain should consist of a 4-inch diameter (minimum) perforated plastic pipe with SDR 35 or better embedded in Class 2 permeable material. The permeable material should be at least 12 inches thick and extend at least 48 inches above the bottom of the keyway (see Plate 9) and/or 12 inches above and below the seepage zone.

The depth and extent of subdrains should be determined and approved by the geotechnical engineer in the field during construction. In addition, subdrains should be installed at a minimum slope of 1 percent and should have cleanouts located at their ends and at turning points. "Sweep" type elbows and wyes should be used at all turning points and cleanouts, respectively. Subdrain outlets and riser cleanouts should be fabricated of the same material as the subdrain pipe as specified herein. Outlet and riser pipe fittings should not be perforated. A licensed land surveyor or civil engineer should provide "record drawings" depicting the locations of subdrains and cleanouts.

Fill Quality

All fill materials should be free of perishable matter and rocks or lumps over 6 inches in diameter, meet the criteria set forth herein for select fill and must be approved by the geotechnical engineer prior to use. We judge the on-site soils are generally suitable for use as general and select fill. The suitability of the on-site soils for use as select fill should be verified during grading.

Select Fill

Select fill should be free of organic matter, have a low expansion potential, and conform in general to the following requirements:

SIEVE SIZE	PERCENT PASSING (by dry weight)
6 inch	100
4 inch	90 – 100
No. 200	10 – 60

Liquid Limit – 40 Percent Maximum
Plasticity Index – 15 Percent Maximum

In general, imported fill, if needed, should be select. Material not conforming to these requirements may be suitable for use as import fill; however, it shall be the contractor's responsibility to demonstrate that the proposed material will perform in an equivalent manner. The geotechnical engineer should approve imported materials prior to use as compacted fill. The grading contractor is responsible for submitting, at least 72 hours (3 days) in advance of its intended use, samples of the proposed import materials for laboratory testing and approval by the soils engineer.

Fill Placement

The surface exposed by stripping and removal of loose gravel or weak, compressible, creep-prone surface soils should be scarified to a depth of at least 6 inches, uniformly moisture-conditioned to near optimum and compacted to at least 90 percent of the maximum dry density of the materials as determined by ASTM Test Method D-1557. Approved fill material should then be spread in thin lifts, uniformly moisture-conditioned to near optimum and properly compacted. All structural fills, including those placed to establish site surface drainage, should be compacted to at least 90 percent relative compaction. Fills placed on terrain sloping at 5:1 or steeper should be continually keyed and benched into firm, undisturbed bedrock.

Permanent Cut and Fill Slopes

In general, cut and fill slopes should be designed and constructed at slope gradients of 2:1 (horizontal to vertical) or flatter, unless otherwise approved by the geotechnical engineer in specified areas. Fill slopes should be constructed by overfilling and cutting the slope to final grade. "Track walking" of a slope to achieve slope compaction is not an acceptable procedure for slope construction. Permanent cut slopes should be observed in the field by the geotechnical engineer to verify that the exposed soil/bedrock conditions are as anticipated. The geotechnical engineer is not responsible for measuring the angles of these slopes. Denuded slopes should be planted with fast-growing, deep-rooted groundcover to reduce sloughing or erosion.

Wet Weather Grading

Generally, grading is performed more economically during the summer months when on-site soils are usually dry of optimum moisture content. Delays should be anticipated in site grading performed during the rainy season or early spring due to excessive moisture in on-site soils. Special and relatively expensive construction procedures, including dewatering of excavations and importing granular soils, should be anticipated if grading must be completed during the winter and early spring or if localized areas of soft saturated soils are found during grading in the summer and fall.

Open excavations also tend to be more unstable during wet weather as groundwater seeps towards the exposed cut slope. Severe sloughing and occasional slope failures should be anticipated. The occurrence of these events will require extensive clean up and the installation of slope protection measures, thus delaying projects. The general contractor is responsible for the performance, maintenance and repair of temporary cut slopes.

Foundation Support

Provided the loose gravel and weak surface soils are removed by planned grading or strengthened by remedial grading as recommended herein, the proposed structures can be supported on continuous and isolated spread footings that bottom on select engineered fill or undisturbed bedrock.

Spread Footings

Spread footings should be at least 12 inches wide and should bottom on select engineered fill or on undisturbed bedrock, as applicable, at least 12 inches below pad subgrade. Additional embedment or width may be needed to satisfy code and/or structural requirements.

The bottoms of all footing excavations should be thoroughly cleaned out or wetted, and compacted using hand-operated tamping equipment prior to placing steel and concrete. This will remove the soils disturbed during footing excavations, or restore their adequate bearing capacity, and reduce post-construction settlements. Footing excavations should not be allowed to dry before placing concrete. If shrinkage cracks appear in soils exposed in the footing excavations, the soil should be thoroughly moistened to close all cracks prior to concrete placement. The moisture condition of the foundation excavations should be checked by the geotechnical engineer no more than 24 hours prior to placing concrete.

Bearing Pressures - Footings installed in accordance with these recommendations may be designed using allowable bearing pressures of 1800, 2700 and 3600 pounds per square foot (psf), for dead loads, dead plus code live loads, and total loads (including wind and seismic), respectively.

Lateral Pressures - The portion of spread footing foundations extending into select engineered fill may impose a passive equivalent fluid pressure and a friction factor of 350 pcf and 0.35, respectively, to resist sliding. Passive pressure should be neglected within the upper 6 inches, unless the soils are confined by concrete slabs or pavements.

Retaining Walls

Retaining walls constructed at the site must be designed to resist lateral earth pressures plus additional lateral pressures that may be caused by surcharge loads applied at the ground surface behind the walls. Retaining walls free to rotate (yielding greater than 0.1 percent of the wall height at the top of the backfill) should be designed for active lateral earth pressures. If walls are restrained by rigid elements to prevent rotation, they should be designed for "at rest" lateral earth pressures. In the absence of backdrains, the retaining walls should be designed to resist full hydrostatic pressures.

Retaining walls should be designed to resist the following earth equivalent fluid pressures (triangular distribution):

EARTH EQUIVALENT FLUID PRESSURES

Active Pressures (level backfill)	40 pcf
Active Pressures (3:1 or steeper backfill)	60 pcf
At Rest Pressures	70 pcf

Where required by the building code, retaining walls with horizontal backfill should be designed to resist a seismic pressure of 8H (in psf) applied with an equivalent point load at a distance equal to 0.6H from the base of the wall (where H is the height of the wall in feet). Where a sloping backfill condition is planned and seismic pressures are required, we should be consulted to provide the appropriate design parameters. These pressures do not consider additional loads resulting from adjacent foundations or other loads. If these additional surcharge loadings are anticipated, we can assist in evaluating their effects. Where retaining wall backfill is subject to vehicular traffic, the walls should be designed to resist an additional surcharge pressure equivalent to two feet of additional backfill.

Retaining walls will yield slightly during backfilling. Therefore, walls should be backfilled prior to building on, or adjacent to, the walls. Backfill against retaining walls should be compacted to at least 90 and not more than 95 percent relative compaction. Over-compaction or the use of large compaction equipment should be avoided because increased compactive effort can result in lateral pressures higher than those recommended above.

<u>Foundation Support</u>

Retaining walls should be supported on spread footings designed in accordance with the recommendations presented in this report. Retaining wall foundations should be designed by the project civil or structural engineer to resist the lateral forces set forth in this section.

<u>Wall Drainage and Backfill</u>

Retaining walls should be backdrained as shown on Plate 10, Appendix A. The backdrains should consist of 4-inch diameter, rigid perforated pipe embedded in Class 2 permeable material. The pipe should be PVC Schedule 40 or ABS with SDR 35 or better, and the pipe should be sloped to drain to outlets by gravity. The top of the pipe should be at least 8 inches below lowest adjacent grade. The Class 2 permeable material should extend to within 1½ feet of the surface. The upper 1½ feet should be backfilled with compacted soil to exclude surface water. Retaining walls designed to resist full hydrostatic pressures need not be backdrained. Expansive soils should not be used for wall backfill. Where expansive soils are present in the excavation made to install the retaining wall, the excavation should be sloped back 1:1 from the back of the footing or grade beam. The ground surface behind retaining walls should be sloped to drain. Where migration of moisture through retaining walls would be detrimental, retaining walls should be waterproofed.

Slab-On-Grade

Provided grading is performed in accordance with the recommendations presented herein, interior and exterior slabs should be underlain by undisturbed bedrock and/or select engineered fill.

Slab-on-grade subgrade should be rolled to produce a dense, uniform surface. The future expansion potential of the subgrade soils should be reduced by thoroughly presoaking the slab subgrade prior to concrete placement. The moisture condition of the subgrade soils should be checked by the geotechnical engineer no more than 24 hours prior to placing the capillary moisture break. The slabs should be underlain with a capillary moisture break consisting of at least 4 inches of clean, free-draining crushed rock or gravel (excluding pea gravel) at least ¼-inch and no larger than ¾-inch in size. Interior slabs subject to vehicular traffic may be underlain by Class 2 aggregate base. The use of Class 2 aggregate base should be reviewed on a case by case basis. Class 2 aggregate base can be used for slab rock under exterior slabs. Interior area slabs should be provided with an underdrain system. The installation of this subdrain system is discussed in the "Geotechnical Drainage" section.

Slabs should be designed by the project civil or structural engineer to support the anticipated loads, reduce cracking and provide protection against the infiltration of moisture vapor. Slabs subjected to heavy concentrated wheel loads, such as forklift or trailer-trucks, should be designed to carry the anticipated wheel loads.

A vapor barrier should be placed under all slabs-on-grade that are likely to receive an impermeable floor finish or be used for any purpose where the passage of water vapor through the floor is undesirable. RGH does not practice in the field of moisture vapor transmission evaluation or mitigation. Therefore, we recommend that a qualified person be consulted to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. This person should provide recommendations for mitigation of the potential adverse impact of moisture vapor transmission on various components of the structure as deemed appropriate.

Utility Trenches

The shoring and safety of trench excavations is solely the responsibility of the contractor. Attention is drawn to the State of California Safety Orders dealing with "Excavations and Trenches."

Unless otherwise specified by the County of Napa on-site, inorganic soil may be used as (general) utility trench backfill. Where utility trenches support pavements, slabs and foundations, trench backfill should consist of aggregate baserock. The baserock should comply with the minimum requirements in Caltrans Standard Specifications, Section 26 for Class 2 Aggregate Base. Trench backfill should be moisture-conditioned as necessary, and placed in horizontal layers not exceeding 8 inches in thickness, before compaction. Each layer should be compacted to at least 90 percent relative compaction as determined by ASTM Test Method D-1557. The top 6 inches of trench backfill below vehicle pavement subgrades should be moisture-conditioned as necessary and compacted to at least 95 percent relative compaction. Jetting or ponding of trench backfill to aid in achieving the recommended degree of compaction should not be attempted.

Pavements

Based on our study, we believe the near-surface soils will have a low to moderate supporting capacity, after proper compaction, when used as a pavement subgrade. An R-value of 20 was assumed for use in pavement design calculations.

Based on the assumed selected R-value, we have computed pavement sections for Traffic Indices (TI) ranging from 5.0 to 7.0 in the table below. The project engineer, in consultation with City/County officials, should choose the pertinent (TI) for this project.

TI	ASPHALT CONCRETE (feet)	CLASS 2 AGGREGATE BASE (feet)	AGGREGATE SUBBASE (feet)
PAVEMENT SECTIONS			
7.0	0.35	0.50	---
6.0	0.30	0.50	---
5.0	0.20	0.50	---

Pavement thicknesses were computed using Method 301 F of the Caltrans Highway Design Manual and are based on a pavement life of 20 years. These recommendations are intended to provide support for the auto and light truck traffic represented by the indicated Traffic Indices. They are not intended to provide pavement sections for heavy concentrated construction storage or wheel loads such as forklifts, parked truck-trailers and concrete trucks or for post-construction concentrated wheel loads such as self-loading dumpster trucks.

In areas where heavy construction storage and wheel loads are anticipated, the pavements should be designed to support these loads. Support could be provided by increasing pavement sections or by providing reinforced concrete slabs. Alternatively, paving can be deferred until heavy construction storage and wheel loads are no longer present. Loading areas for self-loading dumpster trucks should be provided with reinforced concrete slabs at least 6 inches thick, and reinforced with No. 4 bars at 12-inch centers each way. Alternatively, the asphalt concrete section should be increased to at least 12 inches in these areas.

Prior to placement of aggregate base, the upper 6 inches of the pavement subgrade soils should be scarified, uniformly moisture-conditioned to near optimum, and compacted to at least 95 percent relative compaction to form a firm, non-yielding surface.

Aggregate base materials should be spread in thin layers, uniformly moisture-conditioned, and compacted to at least 95 percent relative compaction to form a firm, non-yielding surface. The materials and methods used should conform to the requirements of the County of Napa and the current edition of the Caltrans Standard Specifications, except that compaction requirements should be based on ASTM Test Method D-1557. Aggregate used for the base course should comply with the minimum requirements specified in Caltrans Standard Specifications, Section 26 for Class 2 Aggregate Base.

Wet Weather Paving

In general, the pavements should be constructed during the dry season to avoid the saturation of the subgrade and base materials, which often occurs during the wet winter months. If pavements are constructed during the winter, a cost increase relative to drier weather construction should be anticipated. Unstable areas may have to be overexcavated to remove soft soils. The excavations will probably require backfilling with imported crushed (ballast) rock. The geotechnical engineer should be consulted for recommendations at the time of construction.

Geotechnical Drainage

This section presents recommendations for surface and subsurface drainage. For the discussion of subsurface drainage related to grading, especially on hillsides, refer to the "Subsurface Drainage" section.

Surface

Surface water should be diverted away from slopes, foundations and edges of pavements. Surface drainage gradients should slope away from building foundations in accordance with the requirements of the CBC or local governing agency. Where a gradient flatter than 2 percent for paved areas and 4 percent for unpaved areas is required to satisfy design constraints, area drains should be installed with a spacing no greater than about 20 feet. Roofs should be provided with gutters and the downspouts should be connected to closed (glued Schedule 40 PVC or ABS with SDR of 35 or better) conduits discharging well away from foundations, onto paved areas or into the site's surface drainage system. Roof downspouts and surface drains must be maintained entirely separate from the slab underdrains recommended hereinafter.

Water seepage or the spread of extensive root systems into the soil subgrade of footings, slabs or pavements could cause differential movements and consequent distress in these structural elements. Landscaping should be planned with consideration for these potential problems.

Slab Underdrains

Where interior slab subgrades are less than 6 inches above adjacent exterior grade and where migration of moisture through the slab would be detrimental, slab underdrains should be installed to dispose of surface and/or groundwater that may seep and collect in the slab rock. Slab underdrains should consist of 6-inch wide trenches that extend at least 6 inches below the bottom of the slab rock and slope to drain by gravity. The slab underdrain trenches should be spaced no further than 15 feet from exterior walls. Additional drain trenches should be installed, as necessary, to drain wider and isolated under slab areas. Four-inch diameter perforated pipe (SDR 35 or better) sloped to drain to outlets by gravity should be placed in the bottom of the trenches. Slab underdrain trenches should be backfilled to subgrade level with clean, free draining slab rock. An illustration of this system is shown on Plate 11. If slab underdrains are not used, it should be anticipated that water will enter the slab rock, permeate through the concrete slab and ruin floor coverings.

Crawl Space Drains

Crawl spaces are inherently damp and humid. In addition, groundwater seepage is unpredictable and difficult to control and, regardless of the care used in installing perimeter foundation drains, can find its way into crawl spaces. The ground surface within the crawl space should be sloped to drain away from foundations and toward a 12 inch square drain trench that is excavated through the longitudinal axis of the crawl space. A 4-inch diameter perforated drain pipe (SDR 35 or better) should be embedded in Class 2 permeable materials near the bottom of the trench. The drain rock should extend to the surface of the crawl space (see Plate 11). Piped outlets should be provided to allow drainage of the collected water through foundations and discharge into the storm drain system. Additional protection against water seepage into crawl spaces can be obtained by compacting fill placed adjacent to perimeter walls to at least 90 percent relative compaction.

Maintenance

Periodic land maintenance, especially on hillsides, will be required. Surface and subsurface drainage facilities should be checked frequently, and cleaned and maintained as necessary or at least annually. A dense growth of deep-rooted ground cover must be maintained on all slopes to reduce sloughing and erosion. Sloughing and erosion that occurs must be repaired promptly before it can enlarge.

Supplemental Services

RGH Consultants, Inc. (RGH) recommends that we be retained to review the project plans and specifications to determine if they are consistent with our recommendations. In addition, we should be retained to observe construction, particularly site excavations, compaction of fills and backfills, foundation and subdrain installations, and perform field and laboratory testing. As part of these services, we recommend that prior to construction a meeting be held at the site that includes, but is not limited to, the owner or owner's representative, the general contractor, the grading contractor, the foundation contractor, the underground contractor, any specialty contractors, the project civil engineer, other members of the project design team and RGH. This meeting should serve as a time to discuss and answer questions regarding the recommendations presented herein and to establish the coordination procedure between the contractors and RGH.

If, during construction, we observe subsurface conditions different from those encountered during the explorations, we should be allowed to amend our recommendations accordingly. If different conditions are observed by others, or appear to be present beneath excavations, RGH should be advised at once so that these conditions may be evaluated and our recommendations reviewed and updated, if warranted. The validity of recommendations made in this report is contingent upon our being notified and retained to review the changed conditions.

If more than 18 months have elapsed between the submission of this report and the start of work at the site, or if conditions have changed because of natural causes or construction operations at, or adjacent to, the site, the recommendations made in this report may no longer be valid or appropriate. In such case, we recommend that we be retained to review this report and verify the applicability of the conclusions and recommendations or modify the same considering the time lapsed or changed conditions. The validity of recommendations made in this report is contingent upon such review.

These supplemental services are performed on an as-requested basis and are in addition to this geotechnical study. We cannot accept responsibility for items that we are not notified to observe or for changed conditions we are not allowed to review.

LIMITATIONS

This report has been prepared by RGH for the exclusive use of Burt and Brian Harlan and their consultants as an aid in the design and construction of the proposed winery project described in this report.

The validity of the recommendations contained in this report depends upon an adequate testing and monitoring program during the construction phase. Unless the construction monitoring and testing program is provided by our firm, we will not be held responsible for compliance with design recommendations presented in this report and other addendum submitted as part of this report.

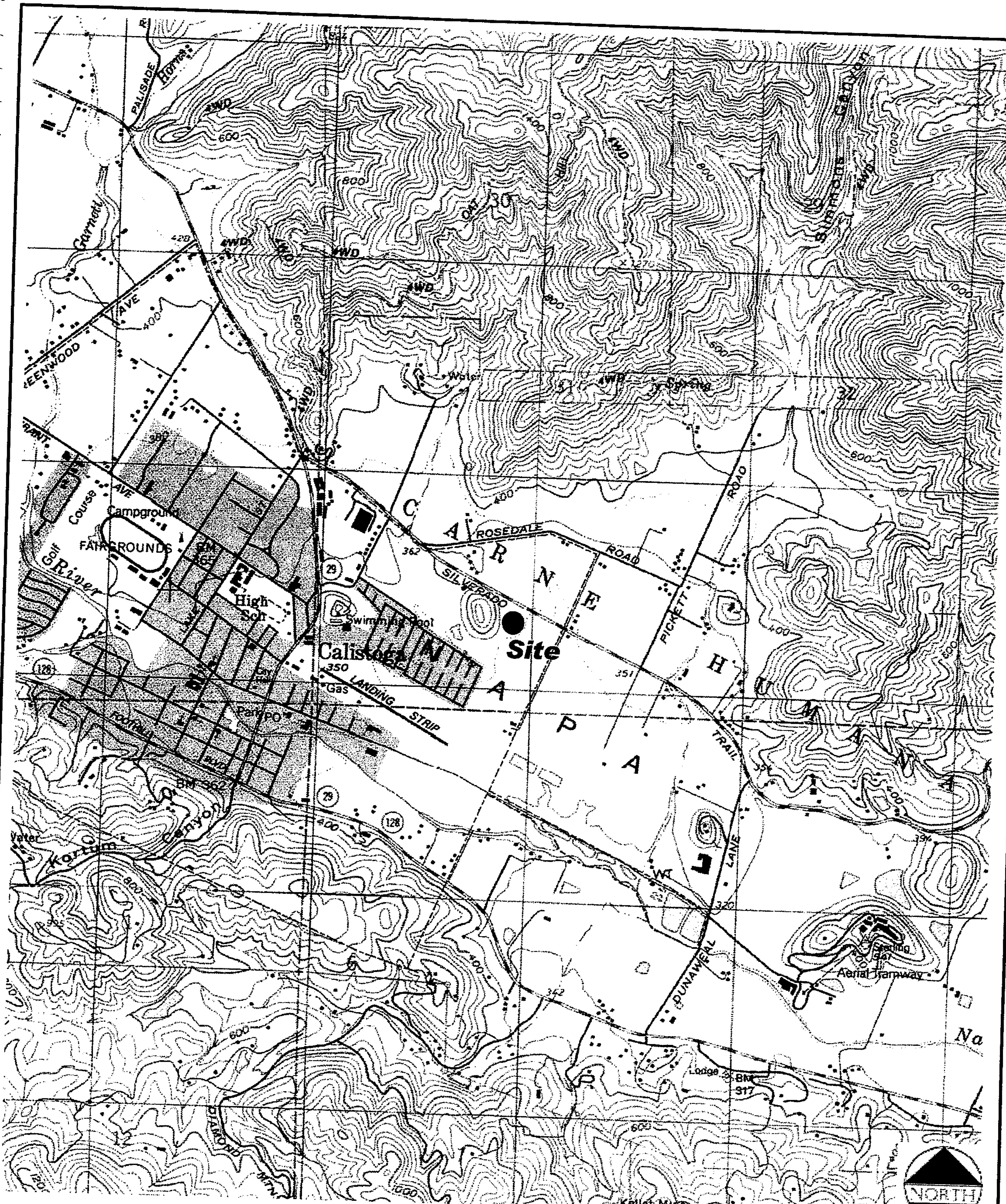
Our services consist of professional opinions and conclusions developed in accordance with generally accepted geotechnical engineering principles and practices. We provide no warranty, either expressed or implied. Our conclusions and recommendations are based on the information provided to us regarding the proposed construction, the results of our field exploration, laboratory testing program, and professional judgment. Verification of our conclusions and recommendations is subject to our review of the project plans and specifications, and our observation of construction.

The borings represent subsurface conditions at the locations and on the date indicated. It is not warranted that they are representative of such conditions elsewhere or at other times. Site conditions and cultural features described in the text of this report are those existing at the time of our field exploration on May 9, 2011 and may not necessarily be the same or comparable at other times.

The scope of our services did not include an environmental assessment or a study of the presence or absence of toxic mold and/or hazardous, toxic or corrosive materials in the soil, surface water, groundwater or air (on, below or around this site), nor did it include an evaluation or study for the presence or absence of wetlands. These studies should be conducted under separate cover, scope and fee and should be provided by a qualified expert in those fields.

APPENDIX A - PLATES

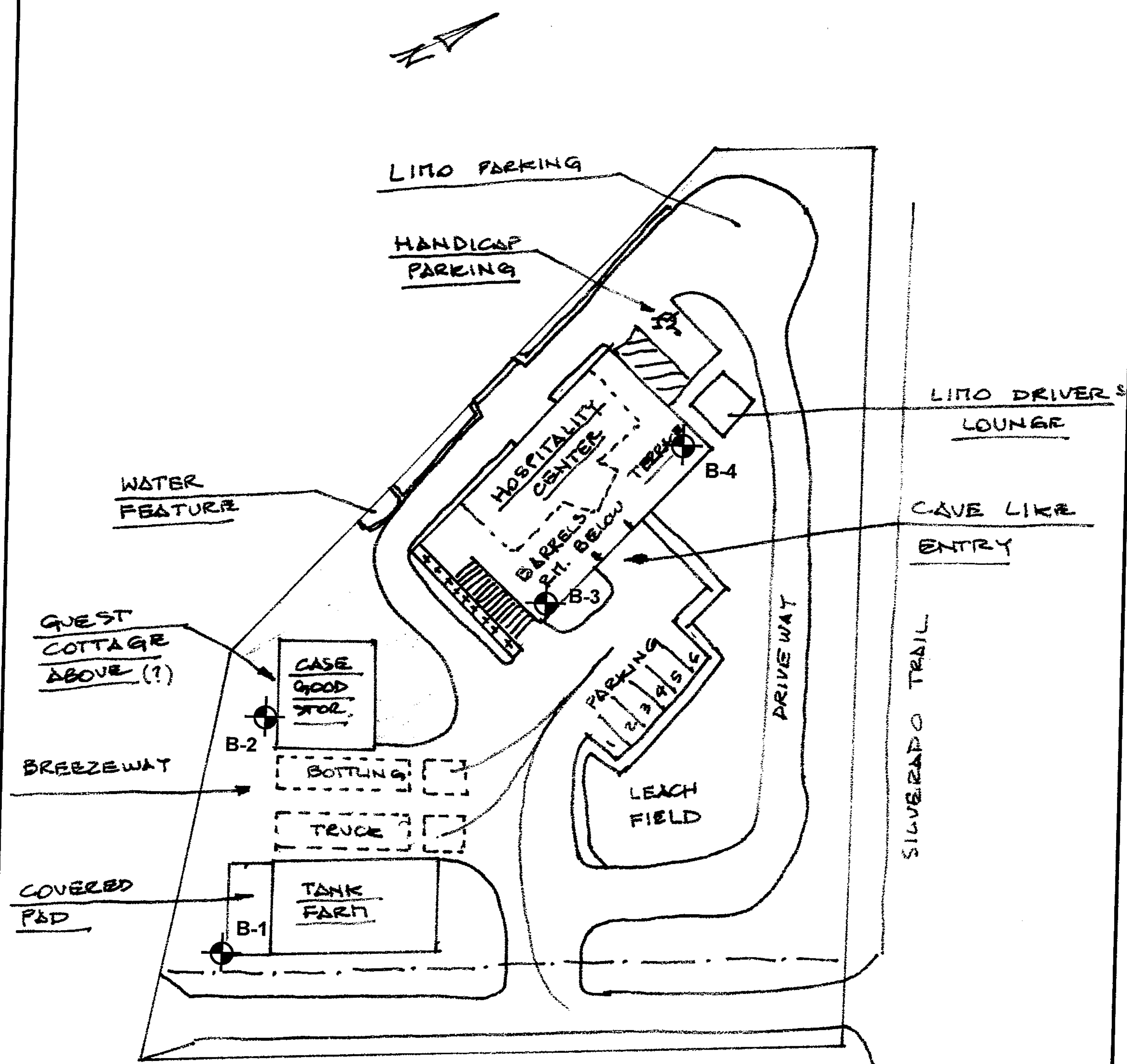
LIST OF PLATES



Reference: Maptech Topoquad, Calistoga, California Quadrangle

Scale: 1" = 2000'

RGH CONSULTANTS	SITE LOCATION MAP Brian Arden Winery APN 011-050-030 Calistoga, California	PLATE 1
Job No: 6569.01.04.2 · Date: JUN 2011		



Explanation

B-4 ⊕ Approximate Boring Location

Reference: Triad Design Studios

Scale: 1" = +/-60'

RGH CONSULTANTS

Job No: 6569.01.04.2 | Date: JUN 2011

EXPLORATION PLAN
Brian Arden Winery
APN 011-050-030
Calistoga, California

PLATE
2

DATE DRILLED: 5/9/11	
DRILLING CONTRACTOR: Paul Pearson Drilling	
DRILLING METHOD: 8-inch Hollow Stem Augers	
HAMMER WEIGHT: 140 lbs	DROP: 30 inches
LOGGED BY: JBH	ELEVATION: 346 feet **

NOTES:

*Equivalent Standard Penetration Test (SPT) blow count.
** Countour lines on topo may by Albion Surveys, Inc., dated 4/20/11

FIELD					LABORATORY							
DEPTH (FEET)	SAMPLE	BLOWS/FOOT *	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	% < 200 SIEVE	PLASTICITY INDEX (PI)	LIQUID LIMIT (LL)	EXPANSION INDEX (EI)	OTHER TESTS	DEPTH (FEET)
0				BROWN sANYD CLAY (CL), medium stiff, wet, with abundant small gravel, porous to 5.5 feet, compressible								0
		6					51.2	13	38			
		4										
5		7										5
				BROWN CLAYEY SAND (GC), medium dense, wet			53.1					
				BROWN CLAYEY SAND (SC), medium dense, wet								
		11					39.7					
10												10
				BROWN CLAYEY GRAVEL (GC), medium dense to dense, wet								
15		32										15
		20					18.5					
20		24					15.5					20
				Becoming dense below 22 feet								
25		30					12.6					25
				Bottom of Boring B-1 at 26.5 feet Groundwater Encountered at 7.5 feet								
30												30

RGH CONSULTANTS

Job No: 6569.01.04.2 Date: 6/6/2011

LOG OF BORING B-1

Brian Arden Winery
APN 011-050-030
Calistoga, California

PLATE 3

DATE DRILLED: 5/9/11		**NOTES:**	
DRILLING CONTRACTOR: Paul Pearson Drilling		*Equivalent Standard Penetration Test (SPT) blow count. ** Countour lines on topo may by Albion Surveys, Inc., dated 4/20/11	
DRILLING METHOD: 6-inch Solid Stem Augers			
HAMMER WEIGHT: 140 lbs	**DROP:** 30 inches		
LOGGED BY: JBH	**ELEVATION:** 352 feet **		

FIELD					LABORATORY							
DEPTH (FEET)	SAMPLE	BLOWS/FOOT *	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	% < 200 SIEVE	PLASTICITY INDEX (PI)	LIQUID LIMIT (LL)	EXPANSION INDEX (EI)	OTHER TESTS	DEPTH (FEET)
0				BROWN SANDY CLAY (CL), soft to medium stiff, with abundant gravel, very porous, compressible, with abundant small gravel (Topsoil)								0
		4										
		4										
5		6										5
				MOTTLED BROWN CLAYEY GRAVEL (GC), medium dense, wet								
		11					16.8					
10												10
		23					21.8					
15												15
		38		GREEN BROWN CLAYEY SAND (SC), dense, wet								
				Bottom of Boring B-2 at 16.5 feet Groundwater Encountered at 6.5 feet								
20												20
25												25
30												30

RGH CONSULTANTS

Job No: 6569.01.04.2 Date: 6/6/2011

LOG OF BORING B-2

Brian Arden Winery
APN 011-050-030
Calistoga, California

PLATE 4

DATE DRILLED: 5/9/11		NOTES:	
DRILLING CONTRACTOR: Paul Pearson Drilling		*Equivalent Standard Penetration Test (SPT) blow count. ** Countour lines on topo may by Albion Surveys, Inc., dated 4/20/11	
DRILLING METHOD: 6-inch Solid Stem Augers			
HAMMER WEIGHT: 140 lbs	DROP: 30 inches		
LOGGED BY: JBH	ELEVATION: 352 feet **		

FIELD					LABORATORY							
DEPTH (FEET)	SAMPLE	BLOWS/FOOT *	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	% < 200 SIEVE	PLASTICITY INDEX (PI)	LIQUID LIMIT (LL)	EXPANSION INDEX (EI)	OTHER TESTS	DEPTH (FEET)
0				BROWN SANDY CLAY (CL), medium stiff, wet, with abundant gravel, porous, compressible to 3.5 feet (Topsoil)								0
		6										
		7		MOTTLED BROWN CLAYEY GRAVEL (GC), loose, wet								
5												5
		5					37.3					
10				BROWN CLAYEY SAND (SC), medium dense, wet								10
		19		Becoming dense below 12 feet								
		35		MOTTLED BROWN CLAYEY SAND (SC), dense, wet								
15				Bottom of Boring B-3 at 14 feet Groundwater Encountered at 5.5 feet								15
20												20
25												25
30												30

RGH CONSULTANTS

Job No: 6569.01.04.2 | Date: 6/6/2011

LOG OF BORING B-3

Brian Arden Winery
APN 011-050-030
Calistoga, California

PLATE
5

DATE DRILLED: 5/9/11	NOTES:
DRILLING CONTRACTOR: Paul Pearson Drilling	*Equivalent Standard Penetration Test (SPT) blow count.
DRILLING METHOD: 6-inch Solid Stem Augers	** Countour lines on topo may by Albion Surveys, Inc., dated 4/20/11
HAMMER WEIGHT: 140 lbs / DROP: 30 inches	
LOGGED BY: JBH / ELEVATION: 357 feet **	

FIELD					LABORATORY								
DEPTH (FEET)	SAMPLE	BLOWS/FOOT *	GRAPHIC LOG	MATERIAL DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	% < 200 SIEVE	PLASTICITY INDEX (PI)	LIQUID LIMIT (LL)	EXPANSION INDEX (EI)	OTHER TESTS	DEPTH (FEET)
0		7		DARK BROWN SANDY CLAY (CL), medium stiff, wet, porous compressible to 3.5 feet, with abundant gravel (Topsoil)								0
		8		MOTTLED BROWN CLAYEY SAND (SC), medium dense, wet								
5		16										5
		18		Becomming dense below 9 feet								
10		32		Bottom of Boring B-4 at 10.5 feet No Groundwater Encountered								10
15												15
20												20
25												25
30												30

RGH CONSULTANTS

Job No: 6569.01.04.2 | Date: 6/6/2011

LOG OF BORING B-4

Brian Arden Winery
APN 011-050-030
Calistoga, California

PLATE 6

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVEL (LITTLE OR FINES)		GW	WELL-GRADED GRAVEL, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVEL, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
		GRAVEL WITH FINES (OVER 12% OF FINES)		GM	WELL-GRADED GRAVEL, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
				GC	CLAYEY GRAVEL, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SAND, GRAVELLY SAND, LITTLE OR NO FINES
				SP	POORLY-GRADED SAND, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (OVER 12% OF FINES)		SM	SILTY SANDS, POORLY GRADED SAND-SILT MIXTURES
				SC	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50			ML	INORGANICS SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50			MH	ORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS AND OTHER SOILS WITH HIGH ORGANIC-CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

KEY TO TEST DATA

- ■ - "Undisturbed" Sample
- ⊠ - Bulk or Disturbed Sample
- ◪ - Standard Penetration Test
- ⧄ - Sample Attempt With No Recovery
- ☐ - Sample Recovered But Not Retained
- ▽ - Groundwater First Encountered
- ▼ - Groundwater Level at End of Exploration
- ⚲ - Seepage Observed

	Shear Strength, psf	Confining Pressure, psf	
Tx	320	(2600)	- Unconsolidated Undrained Traixial
TxCU	320	(2600)	- Consolidated Undrained Traixial
DS	2750	(2600)	- Consolidated Drained Direct Shear
UC	2000		- Unconfined Compression
FVS	470		- Field Vane Shear
LVS	700		- Laboratory Vane Shear
SS			- Shrink Swell
EXP			- Expansion
P			- Permeability

Note: All strength tests on 2.8-in. or 2.4-in. diameter sample, unless otherwise indicated.

RGH CONSULTANTS

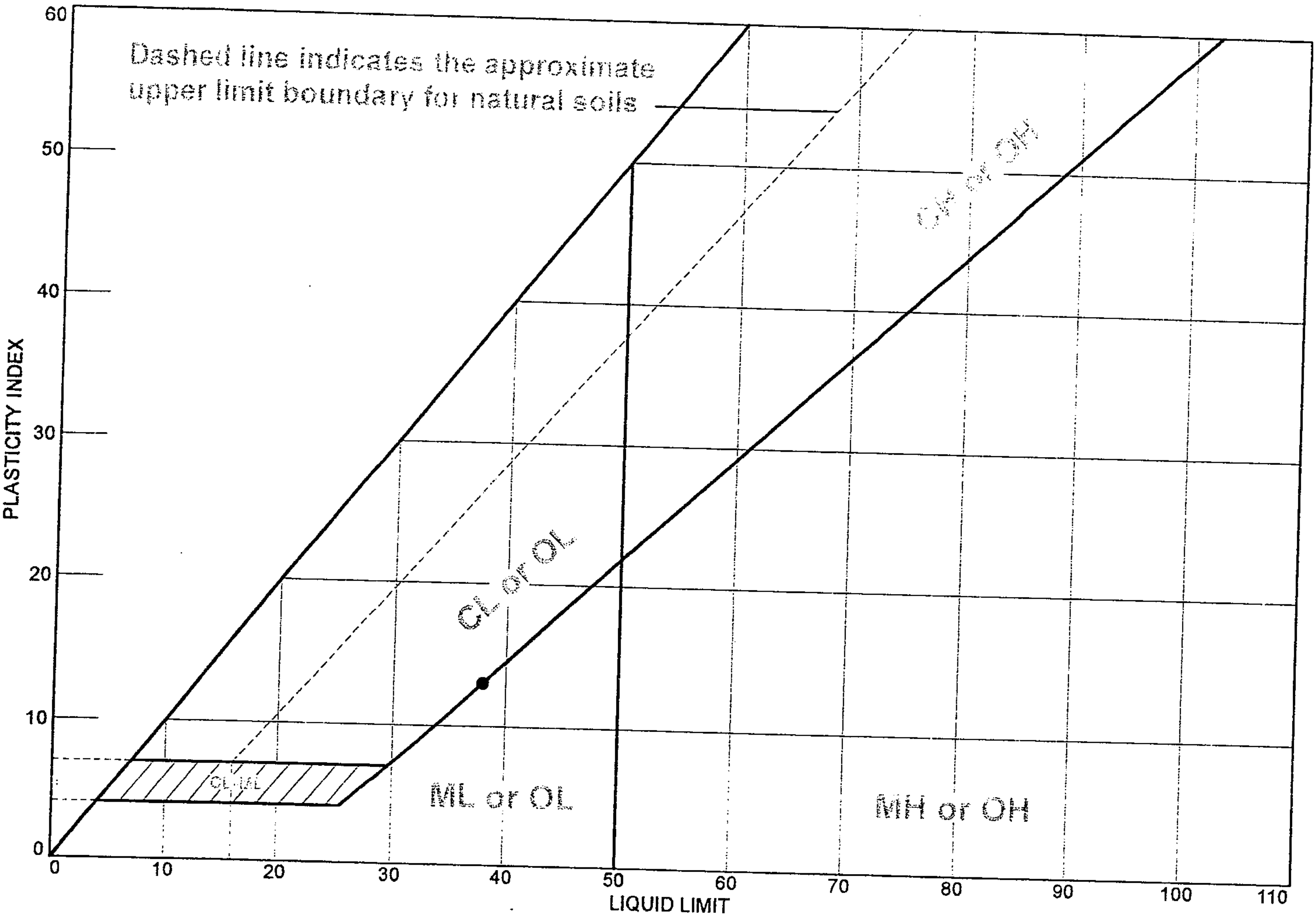
Job No: 6569.01.04.2 | Date: JUN 2011

SOIL CLASSIFICATION AND KEY TO TEST DATA

Brian Arden Winery
APN 011-050-030
Calistoga, California

PLATE 7

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Brn Sandy Clay (CL)	38	25	13		51.2	CL

Project No. 6569.01.04.2 **Client:** RGH Consultants

Project: Brian Arden Winery

● **Source of Sample:** B-1 **Depth:** 1.0'

Remarks:

RGH CONSULTANTS

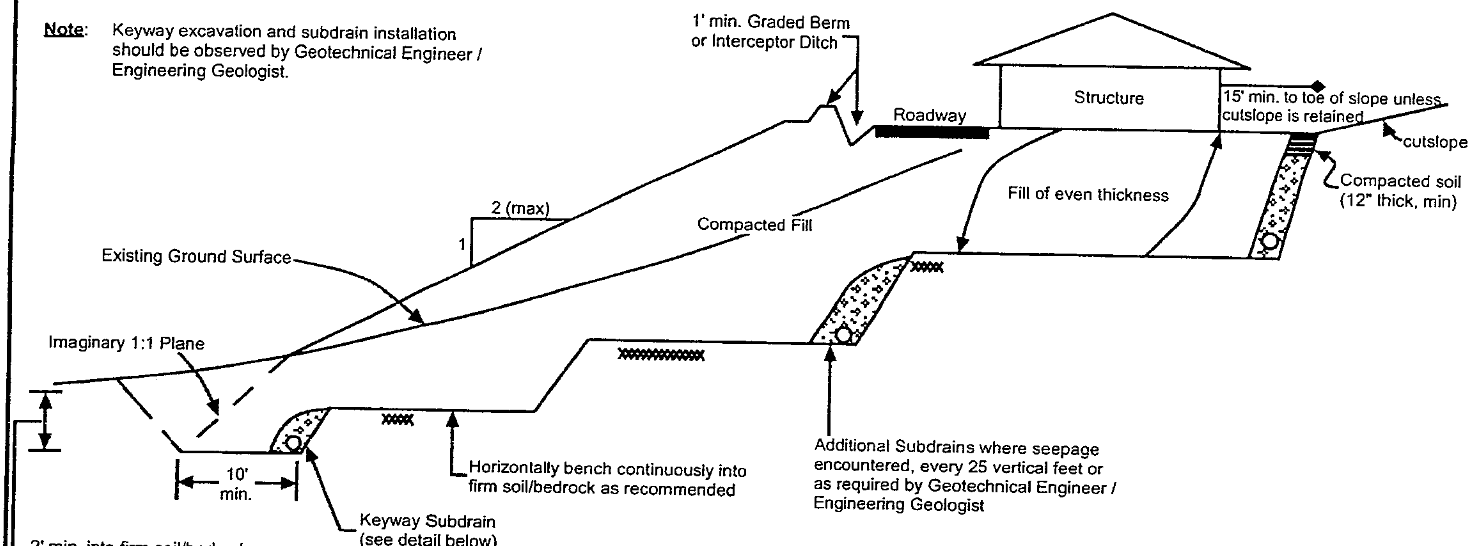
Job No: 6569.01.04.2 Date: JUN 2011

CLASSIFICATION TEST DATA

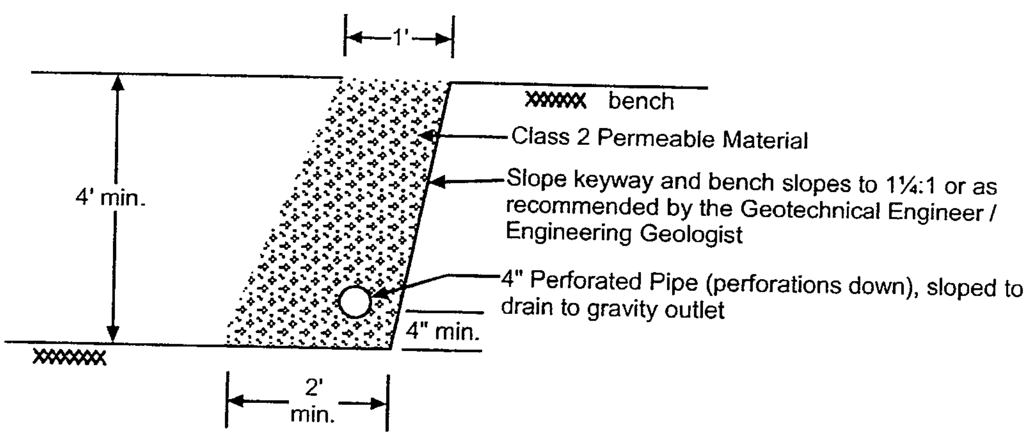
Brian Arden Winery
APN 011-050-030
Calistoga, California

PLATE

8

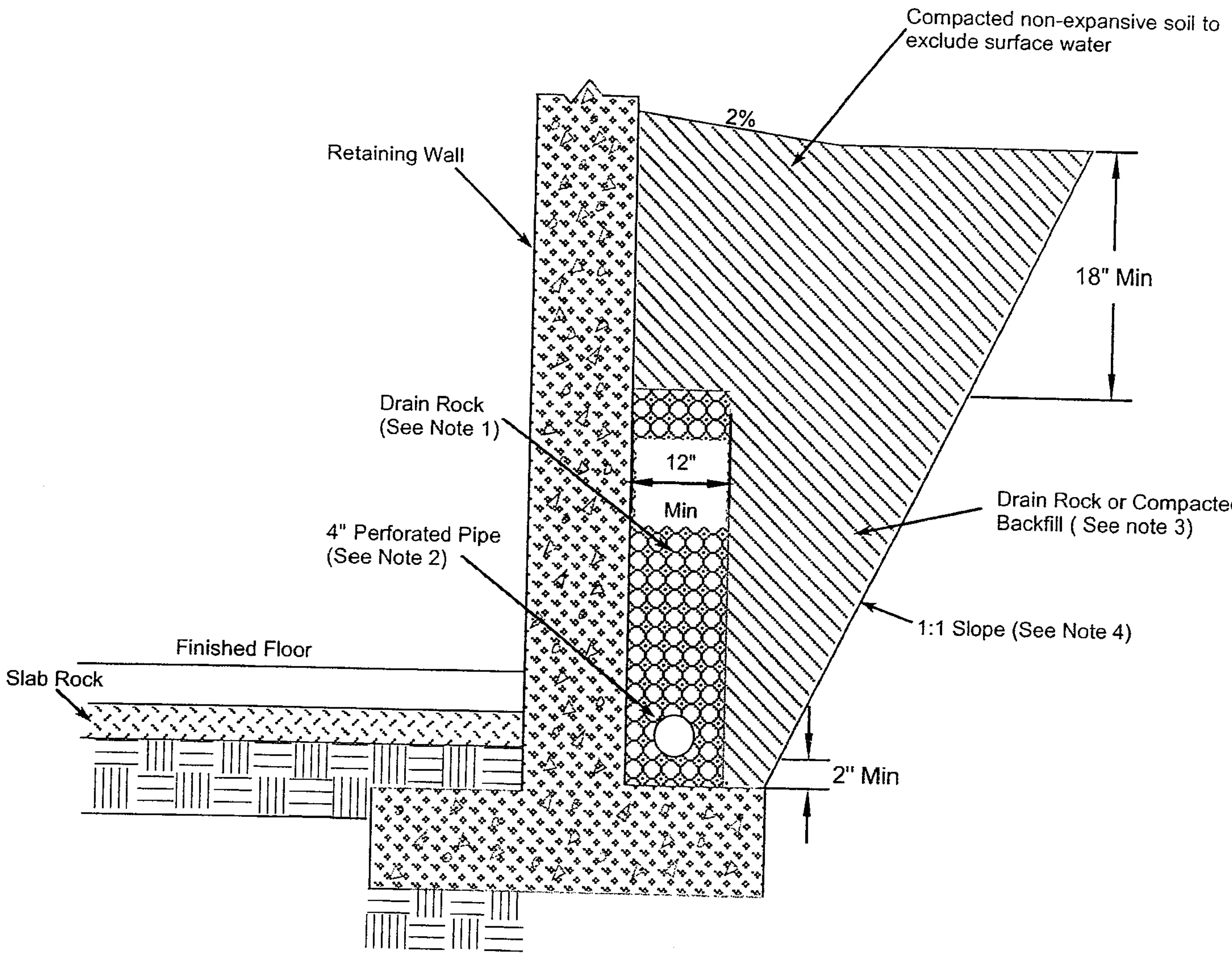


HILLSIDE GRADING ILLUSTRATION
(Not To Scale)



KEYWAY SUBDRAIN
(Not To Scale)

RGH CONSULTANTS	HILLSIDE GRADING ILLUSTRATION Brian Arden Winery APN 011-050-030 Calistoga, California	PLATE 9
Job No: 6569.01.04.2 — Date: JUN 2011		



Notes:

1. Drain rock should meet the requirements for Class 2 Permeable Material, Section 68, State of California "Caltrans" Standard Specification, latest edition. Drain rock should be placed to approximately three-quarters the height of the retaining wall.

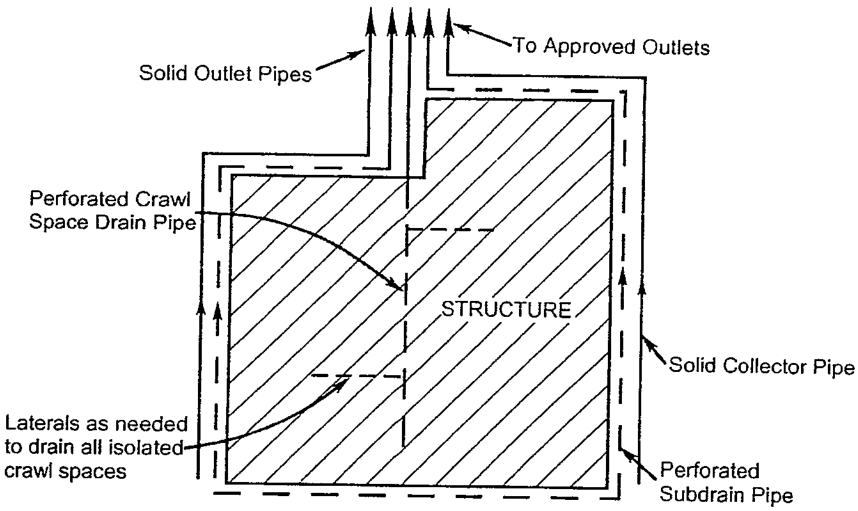
2. Pipe should conform to the requirements of Section 68 of State of California "Caltrans" Standards, perforations placed down, sloped at 1% for gravity flow to outlet or sump with automatic pump. The pipe invert should be located at least 8 inches below the lowest adjacent finished surface.

3. During construction the contractor should use appropriate methods such as temporary bracing and/or light compaction equipment to avoid overstressing the walls. Non-expansive soils to be used as backfill.

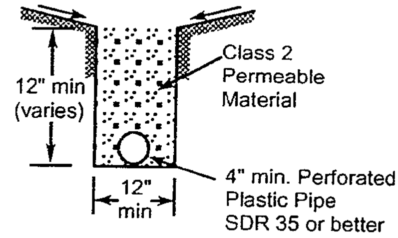
4. Slope excavation back at a 1:1 gradient from the back of footing where expansive materials are exposed.

Not to Scale

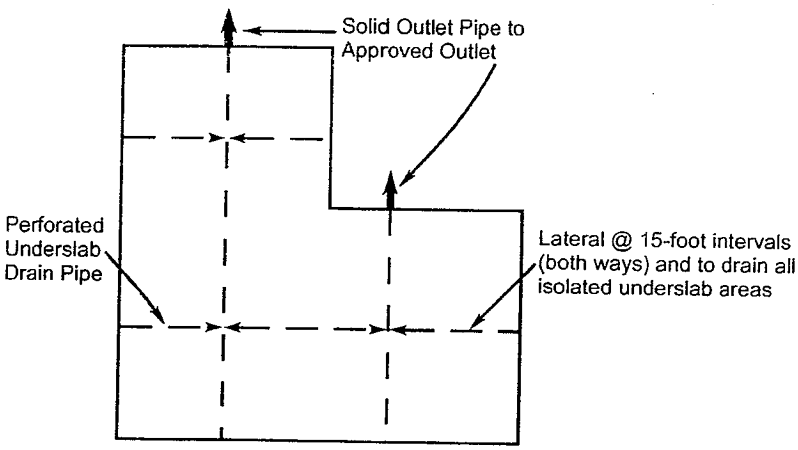
RGH CONSULTANTS	**RETAINING WALL BACKDRAIN ILLUSTRATION** Brian Arden Winery APN 011-050-030 Calistoga, California	PLATE **10**
Job No: 6569.01.04.2 Date: JUN 2011		



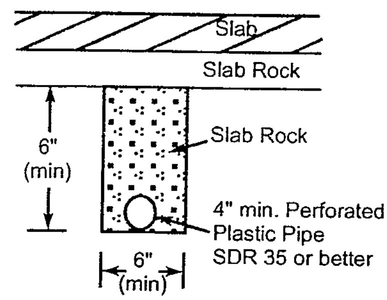
TYPICAL FOUNDATION DRAIN PLAN



CRAWL SPACE DRAIN



TYPICAL UNDERSLAB DRAIN PLAN



SLAB UNDERDRAIN

RGH
CONSULTANTS

Job No: 6569.01.04.2 | Date: JUN 2011

TYPICAL SUBDRAIN DETAILS
Brian Arden Winery
APN 011-050-030
Calistoga, California

PLATE
11

APPENDIX B - REFERENCES

American Society of Civil Engineers, 2006, Minimum Design Loads for Buildings and Other Structures, ASCE Standard ASCE/SEI 7-05.

Bortugno, E.J., 1982, Map Showing Recency of Faulting, Santa Rosa Quadrangle in Wagner and Bortugno, Geologic Map of the Santa Rosa Quadrangle: California Division of Mines and Geology, Regional Geologic Map Series, Map No. 2A, Santa Rosa Quadrangle, Scale 1:250,000.

Bryant, W.A., 1982, West Napa Fault Zone and Soda Creek (East Napa) Fault, Cuttings Wharf, Napa, Yountville, and part of Cordelia 7.5-Minute Quadrangles, Napa County, California: California Division of Mines and Geology Fault Evaluation Report FER-129, 10 p., maps, Scale 1:24,000.

Bryant, W.A., and Hart, E.W., Interim Revision 2007, Fault-Rupture Zones in California; California Geological Survey, Special Publication 42, p. 21 with Appendices A through F.

California Building Code, 2010, California Building Standard Commission.

Davis, J.F., 1983, Special Studies Zone Map, Cuttings Wharf Quadrangle: California Division of Mines and Geology, Scale 1:24,000.

Dwyer, M.J., Noguchi, N., and O'Rourke, J., 1976, Reconnaissance Photo-Interpretation Map of Landslides in 24 Selected 7.5-Minute Quadrangles in Lake, Napa, Solano, and Sonoma Counties, California: U.S. Geological Survey OFR 76-74, 25 Plates, Scale 1:24,000.

Federal Emergency Management Agency, National Flood Insurance Program, 2008, Flood Insurance Rate Map, Napa County, California and Incorporated Areas, Map Number 06055C0229E.

Graymer, R.W., Brabb, E.E., Jones, D.L., Barnes, J., Nicholson, R.S., and Stamski, R.E., 2007, Geologic Map and Map Database of Eastern Sonoma and Western Napa Counties, California, U.S. Department of the Interior, U.S. Geological Survey, Scientific Investigations Map 2956.

Idriss, I.M. and Boulanger, R.W., 2004, Semi-Empirical Procedures for Evaluating Liquefaction Potential During Earthquakes, Proceedings of the 11th ICSDEE and 3rd ICEGE, pp. 32-56.

Idriss, I.M. and Boulanger, R.W., 2008, Soil Liquefaction During Earthquakes.

Ishihara, K., and Yoshimine, M., 1992, Evaluation of settlements in sand deposits following liquefaction during earthquakes, *Soils and Foundations* 32(1), 173-88.

Knudsen, K.L., Sowers, J.M., Witter, R.C., Wentworth, C.M., and Helley, E.J., 2000, Preliminary Maps of Quaternary Deposits and Liquefaction Susceptibility, Nine-County San Francisco Bay Region, California, U.S. Geological Survey, Open File Report 00-444.

Natural Resources Conservation Service, United States Department of Agriculture, accessed 2011. Web Soil Survey, available online at http://websoilsurvey.nrcs.usda.gov/.

Petersen, et al., 1996, Probabilistic Seismic Hazard Assessment for the State of California, California Department of Conservation, Division of Mines and Geology, Open File Report 96-08.

Seed, H.B. and Idriss, I.M., 1982, Ground Motion and Soil Liquefaction During Earthquakes: Earthquake Engineering Research Institute, Berkeley, California, 134 p.

Seed, H.B., Tokimatsu, K., Harder, L.F., and Chung, R.M., 1985, Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations: Journal of Geotechnical Engineering Division, American Society of Civil Engineers, v. III, no. 12, December, p. 1425-1445.

United States Geological Survey, 2008, Earthquake Ground Motion Parameter Java Application, Version 5.0.9, updated October 6, 2008.

Working Group on California Earthquake Probabilities, 2007, Uniform California Earthquake Rupture Forecast (UCERF): Notes on Southern California Earthquake Center (SCEC) Web Site (http://www.scec.org/ucerf/).

Youd, T.L., and Idriss, I.M., and 19 others, 2001, Liquefaction Resistance of Soils: summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils: ASCE Geotechnical and Geoenvironmental Journal, v. 127, no. 10, p. 817-833.

APPENDIX C - DISTRIBUTION

Burt and Brian Harlan (1,0)
P.O. Box 96
Calistoga, CA 94515

Triade Design Studio (5,1)
Attention: Carlo Di Fede
3475 N. St. Helena Hwy.
St. Helena, CA 94574

JBH:TAW:jh:lw

s:\project files\6501-6750\6569.01.04.2 brian arden winery\gs report.doc