TO:          David Hardy  
Supervising Planner  
Sonoma County  
Permit and Resource Management Department  
2550 Ventura Avenue  
Santa Rosa, California 95403  

SUBJECT:  Supplemental Geologic and Geotechnical Peer Review  
RE:  Proposed Cornell Winery  
245 Wappo Road  

At your request, Cotton, Shires and Associates, Inc. (CSA) has completed a supplemental geologic and geotechnical peer review of supporting technical evaluations for the subject project construction using:


In addition, we have reviewed pertinent regional geologic maps, previously submitted project documents, multiple stereo pairs of historical aerial photographs of the site, conducted a reconnaissance of the current project site, and inspected samples for exploratory drilling. We have also been in communication with representatives of RGH during the progress of the referenced investigation.

DISCUSSION

The applicant is proposing to build a new winery facility in a rural portion of eastern Sonoma County. Access to the proposed winery buildings would be provided by improvement of an existing driveway extending from Wappo Road. We understand that the proposed project includes two one-story winery buildings with a total floor area of approximately 6,000 square feet, an adjacent patio area, a wine cave, a water tank site at the top of the local ridge, and a parking area located immediately west of the winery buildings. Various driveway improvements are also proposed. The layout of project improvements is illustrated on Plate 3 of the referenced report. We have not evaluated detailed design criteria associated with the wine cave (documents prepared by Condor); however, we understand that the proposed cave extends into intact bedrock materials that are not anticipated to be impacted by future landsliding.

In our previous project geologic and geotechnical peer review (letter dated February 16, 2010), we evaluated a different proposed construction site located north of the currently proposed improvements. We noted that this former site appeared to be constrained by potential reactivation of old/dormant landslides. We recommended that
consideration be given to selecting a construction site with favorable slope stability conditions and that areas of existing landslides be avoided.

The primary intent of our geologic and geotechnical peer review is to evaluate whether the referenced report by RGH, and revised project layout, adequately address apparent site geologic hazards and provide geotechnical design criteria appropriate to mitigate identified slope and earth materials conditions. Our understanding of site landslide distribution has relied significantly on our own site geologic mapping and aerial photographic analyses. All collected site subsurface data has also been considered. We have utilized Special Publication 117a and associated implementation documents as general guidelines for the level of investigation and analysis that should be performed to meet standards of geologic and geotechnical practice with respect to potential landslide hazards. Special Publication 117a contains provisions that allow areas of favorable ground stability with low potential for seismic hazards to be screened out of comprehensive quantitative evaluation requirements. Given concurrence from the lead agency technical reviewer, the Project Geotechnical Consultant may utilize a screening investigation to address sites with low hazard potential.

CONCLUSIONS AND RECOMMENDED ACTION

The applicant has elected to move proposed winery buildings to positions along the axis of a bedrock supported ridge. No indications of past landsliding are evident beneath proposed building footprints or in locations that appear to threaten proposed buildings during their intended design life. RGH concludes that the new winery site is stable under static and anticipated seismic conditions. The intact and competent nature of bedrock materials beneath the winery and tank sites has been investigated by seven core borings (39 to 119 feet in depth) and eleven test pits. In our opinion, site surface mapping and site subsurface exploration has been completed in a manner consistent with prevailing standards of geotechnical practice. We also concur that the winery site is favorably located on an intact, bedrock supported ridge displaying signs of long term stability. In our opinion, comprehensive quantitative slope stability analyses are not necessary to demonstrate the geotechnical suitability of the currently proposed building sites.

We do not have remaining geologic or geotechnical objections to the currently indicated site development layout or general presented project geotechnical design criteria. However, we do recommend that the applicant and RGH give additional consideration to the pier-supported foundation alternative for retaining walls depicted along the southeastern side of the winery patio (both spread footing and pier foundation alternatives for retaining walls have been presented). Given the proximity of steep adjoining slopes, the pier foundation option appears to be the most prudent approach. The final depths of wall piers should also reflect securing adequate downslope lateral coverage of piers prior to application of passive resistance pressures. RGH should verify that they have considered the above comments regarding retaining wall design during their formal geotechnical plan review of future construction documents.
We conclude that the proposed site improvements utilizing RGH recommended geotechnical engineering design measures appear sufficient to result in an "acceptable level" of risk as defined by Publication 117a. We also conclude that the project would not expose people or structures to fault rupture hazards. Utilization of seismic design parameters presented by RGH should be sufficient to address anticipated ground shaking conditions. We accept and concur with RGH that the potential for liquefaction or seismically-induced ground failure to result in substantial adverse impacts to winery buildings is low. As currently depicted, we also concur that the potential for landslides to result in substantial adverse impacts to the winery buildings, or other indicated site improvements, is low. Project grading could result in soil erosion but utilization of relatively standard erosion control methods (hydroseeding, siltation control, overland drainage control, and other best management practices) should be sufficient to prevent substantial soil erosion or topsoil loss. Because proposed buildings are now sited on stable geologic bedrock materials, adverse impacts from landsliding, liquefaction, lateral spreading, or soil collapse are not anticipated. Given implementation of appropriate erosion control methods, the potential for depicted site improvements to result in adverse off-site geotechnical impacts is low.

LIMITATIONS

This supplemental geologic and geotechnical peer review has been performed to provide technical advice to assist the County in its discretionary permit decisions. Our services have been limited to review of aerial photographs and the documents previously identified, examination of select earth material samples and a visual review of the buildings sites and adjoining slopes. Our opinions and conclusions are made in accordance with generally accepted principles and practices of the geotechnical profession. This warranty is in lieu of all other warranties, either expressed or implied.

Respectfully submitted,

COTTON, SHIRES AND ASSOCIATES, INC.

Ted Sayre
Principal Engineering Geologist
CEG 1795

David T. Schrier
Principal Geotechnical Engineer
GE 2334

TS:DTS:kd

COTTON, SHIRES AND ASSOCIATES, INC.
GEOTECHNICAL STUDY REPORT

CORNELL WINERY
245 WAPPO ROAD
SANTA ROSA, CALIFORNIA
APN 028-260-041

Project Number:
2096.05.05.1

Prepared For:
Cornell Farms, LLC
% Guy Davis
2555 Laguna Road
Santa Rosa, California 95401

Prepared By:
RGH Consultants, Inc.
Santa Rosa Office
1305 North Dutton Avenue
Santa Rosa, California 95401
(707) 544-1072

[Signatures and seals]

June 23, 2010
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INFORMATION ABOUT YOUR GEOTEchnICAL REPORT
INTRODUCTION

This report presents the results of our geotechnical study for the proposed Cornell winery to be constructed on the lands of Cornell Farms, LLC (Cornell Farms) in Sonoma County, California. The Cornell Farms land consists of 174 acres situated within rural open space and is approximately 1 mile east of the Sonoma-Napa County line. The proposed winery site, as referenced herein, is located at 245 Wappo Road (APN 028-260-041), the southwestern 40-acre parcel of the Cornell Farms property. The Cornell Farms property is accessed by Wappo Road, a partial paved/dirt and gravel road that extends generally northward off St. Helena Road. The 40-acre subject parcel and approximate winery building site relative to the Cornell Farms land is shown on the Site Vicinity Map, Plate 1, Appendix A.

The terrain on the subject parcel extends over variably sloping shrubland and woodland characterized by westerly facing spur ridges and intervening ravines off a southerly trending ridge (proposed winery location) and knoll top that borders the eastern parcel boundary. In general, the ground slopes across the site range locally between about 1½:1 (Horizontal to Vertical) and 13:1:1V. Ground slopes generally range between 3½:1:1V and 5:1V across the proposed winery building site. The U.S. Geological Survey (USGS) 1½ Minute series topographic map of the Calistoga Quadrangle (1997) indicates the topography ranges from over 1680 feet above mean sea level along the eastern parcel boundary to approximately 1360 feet within a deep ravine on the west. It should be noted that the contours shown on the USGS map do not agree with the contours shown on the site specific topographic map, presented herein as the Geologic Map (Plate 2) and the Exploration Plan (Plate 3). Wappo Road traverses northeasterly through the southeastern quadrant of the subject parcel. Site improvements include a residence, a leachfield, a small vineyard and a well on the southern portion of the property. The proposed winery site is located on the southeastern quadrant of the 40-acre parcel, and immediately southeasterly and adjacent to Wappo Road. A water tank pad will be located northeast of the planned winery as shown on Plate 3.

We have been performing ongoing geologic and geotechnical evaluations for the Cornell Farms property. Our previous studies are presented in the list of references presented in Appendix B. At the time of our previous studies, the concept was to construct the winery about 200 feet north of the current location and across two spur ridges that are separated by the head of a narrow ravine. That proposed site was abandoned and relocated to the subject ridge area.

We understand it is planned to construct two one-story structures with a combined total of about 6000 square feet of slab-on-grade floor area and metal roofing. Roof loads will be transmitted to the ground by perimeter wall and isolated column footings.

The upper winery building will be recessed into the hillside creating an approximately 20-foot high cut. This cut will be retained by a soil nail cave portal that will not be part of the winery building. Retaining walls will be needed along a portion of Wappo Road and along the southeastern side of the winery. A water tank pad will be constructed northeast of the winery site and may require retaining walls to create a level pad.

Auto access will be provided by Wappo Road. Parking will be provided west of the winery and Wappo Road on the top of a relatively level spur ridge.

Actual foundation loads are not known at this time. We anticipate the loads will be typical for the light to moderately heavy type of construction planned and that wall loads will range from about 1 to 2 kips per lineal foot.
Grading plans are not available, but we anticipate that the planned grading will be the minimum amount needed to construct a level building pad and provide the building site and parking areas with positive drainage, and could include cuts and fills on the order of 1 to 20 feet.

**SCOPE**

The purpose of our study was to generate geotechnical information for the design and construction of the project. Our scope of services included reviewing our previous studies at the property, selected published geologic data pertinent to the site; evaluating subsurface conditions with borings, test pits and laboratory tests; analyzing the field and laboratory data; and presenting this report with the following geotechnical information:

1. A brief description of soil, bedrock and groundwater conditions observed during our study;
2. A discussion of seismic hazards that may affect the proposed winery improvements; and
3. 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 and the construction of hillside fills;
   c. Foundation type(s), design criteria, and estimated settlement behavior;
   d. Lateral loads for cantilevered 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.
Site Reconnaissance

Our geologist conducted a surficial reconnaissance of the property to observe features detected in aerial photographs, exposed topographic features, surface soils, rock outcroppings and cut banks. A site geologic map is presented on Plate 2.

Subsurface Exploration

On April 1 through April 16, 2010, we explored subsurface conditions across the winery site by drilling, logging and sampling seven core borings to depths ranging from 39½ to 119 feet. The core borings were drilled with a track-mounted mud rotary drill rig capable of coring with an HQ barrel or 101mm sampler. In the upper 5 to 10 feet of each hole and where we were getting limited and disturbed core recovery, relatively undisturbed samples were obtained at selected intervals by driving a 1.375-inch inside diameter (2-inch outside diameter) Standard Penetration Test (SPT) sampler, without liners or rings, and a 2.43-inch inside diameter Sprague and Henwood split barrel sampler, containing 6-inch long brass liners, using a 140-pound hammer falling approximately 30 inches. The samplers were 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 test data. Sampler penetration resistance (blow counts) provides a relative measure of soil/bedrock consistency and strength. The samples were visually classified and logged by our field geologist, and stored in core-boxes for transport to the office. The core boring locations are presented on the exploration plan, Plate 3. The core boring logs are presented on Plates 4 through 10.

On April 21 and 22, 2010, we supplemented the core borings by excavating eleven test pits to depths ranging from about 5 to 11½ feet with a track-mounted excavator. Two test pits were excavated on January 15, 2008, near the planned water tank site. Test pit sidewalls were cleaned of smear and logged in detail. The test pit locations are presented on Plate 3. The test pit logs are presented on Plates 11 through 15.

The soils are described in accordance with the Unified Soil Classification System, outlined on Plate 16, Appendix A. Bedrock materials are described in accordance with Engineering Geology Rock Terms shown on Plate 17, Appendix A.

Laboratory Testing

The samples obtained from the borings and test pits were transported to our office and re-examined to verify soil classifications, evaluate characteristics, and assign tests pertinent to our analysis. Selected samples were laboratory tested to determine their water content, dry density, classification (Atterberg Limits, percent of silt and clay), triaxial compressive strength and expansion potential (Expansion Index - EI). Results of the classification and triaxial compression strength tests are presented on Plates 18 through 23.
SITE CONDITIONS

General

Sonoma 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 unit in the area is the lower Jurassic to upper Cretaceous Franciscan Complex, 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.

Geology and Soils

Published geologic maps prepared by Huffman and Armstrong (1980), shown on Plate 24 in Appendix A of this report, indicate an unnamed thrust fault that extends northwesterly through the northern portion of the parcel. The thrust fault is shown to juxtapose sheared shale and sandstone (mélange unit) of the Franciscan Complex northeast of the fault over the younger tuffaceous unit of the Sonoma Volcanics Group on the southwest (Huffman and Armstrong, 1980; Fox, 1973). The mélange unit is shown to consist of sheared shale and sandstone that contains generally resistant masses of chert, high-grade metamorphic rocks, variable shattered sandstone and greenstone, metagreenstone and generally less resistant serpentinite. The tuffaceous rocks are reported to comprise pumiceous ash-flow tuff that is locally or partly welded, and contains bedded agglomeratic tuff, andesitic or basaltic lava flows, tuff breccia, bedded tuff and pumiceous tuff.

Based on Huffman and Armstrong (1980) and Fox (1973), the winery site is shown to be underlain by the Sonoma Volcanics; however, our findings indicate the site to be underlain by the Franciscan Complex. The geologic map prepared by Graymer et al., 2007, (Plate 25), generally depicts the same findings as the Huffman and Armstrong’s mapping. During our geologic reconnaissance of the winery site, we observed outcrops of the Franciscan Complex on the knoll/spur ridge on the eastern portion of the parcel and along a portion of the road cut for Wapoo Road. We also observed Franciscan Complex bedrock along the road past the electronic gate on the eastern side of the subject parcel. On the western portions of the parcel, we observed local outcrops of tuff breccia of the Sonoma Volcanics. Our test pits and borings confirm the winery site is underlain by materials of the sandstone and mélange units of the Franciscan Complex which overlie and are in fault contact with the Sonoma Volcanics.

Landslides

Published maps (Huffman and Armstrong, 1980) show the tongue of a large landslide encompassing a portion of the site. However, Dwyer, 1976, does not show the presence of landslides at the proposed winery site. We documented active, dormant and ancient landslides in the vicinity during the course of our study. Brief descriptions of active, dormant and ancient landslides are summarized herein:
Active - Active landslides, including recently active, exhibit areas of unstable ground with fresh geomorphic features. Common fresh features can include hummocky topography, abrupt grade breaks, ground cracks, exposed soils and disrupted vegetation. Active landslides typically range in age from recent to ±50 years.

Dormant - Dormant landslides appear to be quasi-stable with a mature and subdued surface expression. Fresh features generally become vague or indistinct, and vegetation generally re-establishes itself but is typically different in type and/or density than original. The age of dormant landslides is estimated to range from ±50 years to several hundred years.

Ancient - Ancient landslides differ from dormant landslides in that the landslide features are highly eroded and subdued, and vegetation is more heavily re-established and with a similar type as the surrounding terrain.

The “Landslide and Relative Slope Stability” maps (Plate 26, Appendix A) by Huffman and Armstrong (1980) indicate a portion of the site is within a relative slope stability category “C” - areas of relatively unstable rock and soil units, and slopes of greater than 15 percent are said to contain abundant landslides. There are three landslides mapped by Huffman and Armstrong within the area:

1. A very large landslide is shown to the northeast of the winery site. A narrow tongue of this landslide is shown to encompass a portion of the ridge and knoll at the site.

2. A second landslide is shown approximately 500 feet away on the far western parcel boundary that encompasses the residence and also includes the southern and southeastern flanks of a knoll across the ravine. We confirmed the presence of at least a portion of this landslide during our previous subsurface exploration on March 11 through 13, 2008; and

3. A third, queried (?) landslide is shown 900 feet away two ravines to the east.

The "Reconnaissance Photo-Interpretation Map of Landslides" by Dwyer, et al (1976) does not indicate the presence of landslides at the winery site (Plate 27, Appendix A). Further, Dwyer et al (1976) shows a much smaller landslide to the northeast. That landslide is mapped to be at least 840 feet northeast of the proposed winery site. The Dwyer landslide is mapped on the southern flanks of a very narrow ridgeline. The second and third landslides mapped by Huffman and Armstrong (1980) were not presented by Dwyer’s (1976) map.

Surface

The roughly square-shaped, 40-acre parcel is located within rural open space at 245 Wappo Road. The terrain extends over shrubland and woodland characterized by westerly-facing spur ridges and intervening ravines off the southerly trending ridge (winery site) and knoll top that borders the eastern parcel boundary. Plate 1 shows that the topography ranges from over 1680 feet on a prominent knoll top on the eastern part of the parcel to approximately 1360 feet on a deeply incised ravine on the west. As previously discussed, the contours shown on the USGS map do not agree with the contours shown on the Site Geologic Map (Plate 2) and Exploration Plan (Plate 3). Wappo Road initially extends northerly off St. Helena Road and traverses northeasterly through the
southeastern portion of the parcel. Wappo Road is paved for the first approximately 300 feet then transitions to a dirt and gravel road. The parcel contains a residence, a leachfield, a well and a small vineyard on the southwestern quadrant of the property.

The proposed winery site is located in approximately the southeasterly quadrant of the property and immediately southeasterly and adjacent to Wappo Road. The terrain is characterized by a northwesterly-trending knoll and spur ridge. Groundslopes across the winery site generally range between 3½:1V and 5:1V.

In general, the ground surface is soft and spongy in the winter months and dry and hard in the summer months. These soil conditions are generally associated with weak, porous surface soils. On sloping terrain 5:1V or steeper, the weak, surface materials are prone to undergo a gradual downhill movement known as creep. Areas of active soil creep are shown on the Site Geologic Map (Plate 2). Soil creep is inherent to hillsides in the area and its force is directly proportional to slope inclination, the soils plasticity, water content and expansion potential.

Natural drainage consists of overland flow from the knoll and spur ridge that concentrates on natural drainage elements such as the swales, ravines and to the creek southeast of the winery site. The natural drainages to the west of the site trend westerly into a deep ravine that trends southwesterly through the northwestern portion of the subject parcel. The ravine trends off the parcel and into a second south-westerly-flowing intermittent blue-line stream that empties into Mark West Creek off the property. Mark West Creek is a perennial blue-line stream that flows westerly adjacent to St. Helena Road.

Subsurface

Our subsurface exploration indicates that, in general, the ridge area of the site (borings 2, 4, 5 and 7, and test pits 4 through 10) are underlain by about 1 to 4 feet of topsoil, residual soil and colluvium (surface soils). Beyond the winery building site, on the eastern flanking slopes of the ridge, these soils thicken to up to 15 feet (SCB-3). The winery building site is underlain by about 2 to 4 feet of weak, porous and compressible clayey and gravelly surface soil. These surface soils generally appear hard and strong when dry but become weak and compressible as their moisture content increases towards saturation. These soils can be considered to be actively creeping, such as found on the steeper ravine slopes northeast of the proposed winery. In general, surface materials are considered prone to creep on hillsides through most of the site sloping at 5:1V or steeper.

The surface soils are underlain by Franciscan Complex bedrock that extends from beneath the surface materials to depths of 65 to 111 feet. Locally the Franciscan Complex bedrock consists of a sandstone block overlaying mélangé. The sandstone contains minor shale and siltstone that is generally firm to hard, friable to strong and moderately to slightly weathered. The sandstone is matrix and rock supported, and exhibits extremely close to closely spaced fractures and quartz veining. In core borings 1, 2, 3 and 5, it ranges to depths of 20½ to 31½ feet, while in core borings 4 and 7, the sandstone ranged in depth from 12½ to 13½ feet. The Franciscan mélangé unit underlies the sandstone, and generally consists of intermixed graywacke sandstone and sheared shale. Mélange is formed by tectonic processes, and by definition generally consists of fragments and blocks (some exotic) of various rock types embedded in a fragmented and generally sheared matrix. The mélange is generally firm to moderately hard, plastic to weak and fresh to slightly weathered. The sheared material is locally...
foliated and matrix supported sandstone and shale, and exhibits extremely closely spaced fractures and quartz veining. The sandstone and mélange units are overlain and in fault contact with Tertiary age Sonoma Volcanics. The volcanics were encountered in core borings 2, 3, 4 and 5 at depths ranging from 65 to 111 feet and consist of crushed to moderately fractured, firm to weak, slightly weathered lithic tuff.

A core boring by Condor, 2009, upslope of WTP-1 and 2 at the tank site encountered clayey soils to a depth of about 2½ feet. Highly sheared and weathered shale underlies the surface soils.

Our aerial photo review and subsurface exploration (borings and test pits) of the ridge top did not identify evidence of the northeastern landslide shown on Huffman and Armstrong, 1980. However, as shown on the site geologic map, Plate 3, a small, active landslide was observed on the base of the steep slope above a creek southeast of the winery. The debris is estimated to be about 3 to 5 feet thick. Test pits STP-2 and 3 are located within a larger dormant landslide on the steep slopes easterly of the winery. Test pits WTP-1 and 2, located near the water tank site (northeast of the knoll), encountered landslide debris underlain by Franciscan Complex sandstone. The head scarp was exposed in WTP-2.

The active and dormant landslide debris generally consists of sheared, shattered shale that is firm to moderately hard, plastic to weak and moderately weathered in a matrix of sand with clay and gravel.

**Corrosion Potential**

Mapping by the Natural Resources Conservation Service (2010) indicates that the corrosion potential of the near surface soil is moderate 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 not observed in our borings and test pits. On hillsides, rainwater typically percolates through the porous surface materials and migrates downslope in the form of seepage at the interface of the surface materials and bedrock, and through fractures in the bedrock. Fluctuations in the seepage rates typically occur due to variations in rainfall intensity, duration and other factors such as periodic irrigation. A well is located 120 feet southwest of the winery site at an elevation of 1888 feet. Reported water levels of 96 feet below the surface at an elevation of 1792 feet were recorded by the well drillers during well development (Todd, 2006).
DISCUSSION AND CONCLUSIONS

Seismic Hazards

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 project in strict adherence with current standards for earthquake-resistant construction.

Faulting

There are faults mapped in close proximity to the site. Huffman and Armstrong (1980) shows an unnamed thrust fault that extends northwesterly through the northern portion of the subject parcel, and a parallel fault mapped approximately 1½ miles to the southwest. An easterly-westerly fault that branches off the thrust fault is mapped approximately 1000 feet to the southeast of the subject parcel. The branch fault shows a dip angle of 60 degrees northerly. Two short, northeasterly-trending faults are mapped approximately 800 feet southeast and about 5000 feet northwest of the parcel. All of these faults are said to show evidence of faulting during the Pleistocene age (700,000 to 2 million years) (Bortugno, 1982).

Recent geologic mapping of the adjacent Mark West Springs 7½ minute quadrangle by Mr. Robert McLaughlin of the USGS is published as Scientific Investigations Map (SIM) 2858 (2004). SIM 2858 indicates the formerly unnamed thrust fault is the Petrified Forest Thrust (PFT) zone. Our personal communication with Mr. McLaughlin (2006) indicates the PFT fault dip ranges from approximately 50 degrees northeasterly to near vertical at the surface, and that the age of faulting could be younger than about 2.8 million years, but no evidence of Holocene rupture was observed.

We did not observe landforms at the winery site that would indicate the presence of active faults and the site is not within a current Alquist-Priolo (A-P) Earthquake Fault Zone (Bryant and Hart, 2007). Active faults are defined by the CGS as one which has had surface displacement within Holocene time (last 11,000 years). Since the site is not within a current (A-P) Earthquake Fault Zone, we believe the risk of surface 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

<table>
<thead>
<tr>
<th>Fault</th>
<th>Direction</th>
<th>Distance-Miles</th>
</tr>
</thead>
<tbody>
<tr>
<td>San Andreas</td>
<td>SW</td>
<td>28 1/2</td>
</tr>
<tr>
<td>Healdsburg-Rodgers Creek</td>
<td>SW</td>
<td>7</td>
</tr>
<tr>
<td>Redwood Hill</td>
<td>W-NW</td>
<td>7 1/2</td>
</tr>
<tr>
<td>Yountville Earthquake of 2000</td>
<td>SE</td>
<td>13</td>
</tr>
<tr>
<td>West Napa</td>
<td>SE</td>
<td>14</td>
</tr>
<tr>
<td>Maacama</td>
<td>NW</td>
<td>8</td>
</tr>
</tbody>
</table>

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. Most soils at the site were neither granular nor saturated or located below the groundwater surface. The soils at the site were generally stiff to very stiff sandy clays with some gravel. Therefore, we judge the potential for liquefaction at the site is low.

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 low potential for liquefaction. Therefore, 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 project 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 steep slopes downhill of the proposed buildings and tank sites;
2. The presence of 1 1/2 to 6 feet, but typically in the range of 2 to 4 feet, of weak, compressible, occasionally expansive, creep-prone clayey surface soils overlying expansive bedrock materials;
3. The detrimental effects of uncontrolled surface runoff and groundwater seepage on the long-term satisfactory performance of wineries especially those
constructed on hillsides, given the erosion potential and porous nature of the surface soils; and

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

Slope Stability Analysis

This section presents a discussion of slope stability related to the proposed winery site. Our slope stability analyses were performed using the computer program Slope/W (GEO-SLOPE International, Ltd., 2007) and the geologic cross sections shown on Plates 29 and 30 (B-B' and C-C'). These sections each represent a critical slope condition for the main winery building. They also extend down the steepest slope adjacent to the winery. The steepness is further complicated by a relatively small landslide at the toe of the slope and a slight dip slope contact between the sandstone unit and the mélange unit of the Franciscan Complex. The cross sections were drawn relatively perpendicular to the slope from the front and rear corner of the winery, which represents the direction of potential failure of the slope. Slope stability was evaluated under long-term static conditions and seismic conditions. For our analyses we considered the slope with the landslide debris completely removed, which is the most conservative assessment.

Groundwater elevation plays an important role in slope stability analysis. As discussed previously, water level measurements in the well near the winery site indicate groundwater is at about Elevation 1792 feet. Accounting for water rising to the top of the Sonoma Volcanics encountered in our borings and shown on the well log, we assumed for our analysis groundwater could be as high as 1815 feet, which is approximately 75 feet below the existing ground surface.

Static Stability - Static stability for long-term conditions was evaluated for a Factor of Safety against failure of 1.5. This Factor of Safety is generally accepted as the standard of practice for these conditions. Our analysis required strength parameters for both the near surface bedrock materials and the underlying mélange bedrock materials. For the evaluation of long-term static loading conditions, we used the effective strength parameters including internal friction angle ($\phi'$) and cohesion ($c'$). These parameters can be determined or estimated by testing in-situ samples or by using correlations relating effective internal friction angle with plasticity index (Ladd et al., 1977). For our analysis we determined these parameters by testing in-situ samples of each unit and then compared the effective internal friction angles to those obtained from correlations. In all cases the correlation values exceeded our in-situ test values (see Table below). Furthermore, our strength parameters for the mélange bedrock materials at the site compared reasonably well with those determined by Condor Earth Technology, Inc. (Condor) for the previously planned winery cave portals. We also compared our in-situ values with those determined for our stability analysis of the previously planned winery site further up the road. Those values ranged from less than to more than the values obtained from site specific testing of the current winery site. When averaged together, the resulting $\phi'$ and $c'$ are less than the in-situ values from our core borings. A detailed review of the core logs for this study and the test pit and core logs from our previous work at the property (RGH, 2008; and RGH, 2009) indicate that the mélange in and around the previous winery site contains significant amounts of talc, while the mélange at the new site has minimal talc. The presence of talc would lead to lower effective internal friction angles, which would in turn lead to the lower average value used for the
previous winery site. The parameters used in our analysis and those determined for the Franciscan mélange by Condor are presented in the table below.

<table>
<thead>
<tr>
<th>STATIC STRENGTH PARAMETERS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit</td>
</tr>
<tr>
<td>Sandstone Bedrock (RGH, in-situ)</td>
</tr>
<tr>
<td>Sandstone Bedrock (correlations)</td>
</tr>
<tr>
<td>Franciscan Mélange (RGH)</td>
</tr>
<tr>
<td>Franciscan Mélange (correlations)</td>
</tr>
<tr>
<td>Franciscan Mélange (Condor, 2009)</td>
</tr>
</tbody>
</table>

Using the approximate groundwater elevation from the well and the in-situ parameters for the sandstone and the mélange, our stability analysis yielded Factors of Safety greater than or equal to 1.5 for both sections B-B' and C-C'. Plots of these analyses are presented on Plates 31 and 32. Based on the results of our analysis, the winery site described herein is considered stable under static conditions.

**Seismic Stability -** Seismic stability was evaluated for a Factor of Safety against failure of 1.0. This Factor of Safety is generally accepted as the standard of practice for these conditions. Seismic stability of the site was analyzed using the procedures prepared by the Southern California Earthquake Center (SCEC) in 2002 (SCEC, 2002). These procedures were prepared in association with Special Publication 117 (California Geological Survey, 2008), which is a guideline for evaluation and mitigation of seismic induced hazards including landslides. The procedure factors in the magnitude of the controlling earthquake, the anticipated peak ground acceleration at the site, and the F_{eq} factor developed by SCEC in order to determine the horizontal seismic coefficient. For the winery site, this coefficient was determined to be 0.215g.

Seismic stability was evaluated using the strength parameters determined above. Our analysis yielded Factors of Safety greater than or equal to 1.0 for both sections B-B' and C-C'. Plots of these analyses are presented on Plates 33 and 34. Based on our analysis, the winery site described herein is considered stable under seismic conditions.

**Weak Surface Soils**

Weak 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. The detrimental effects of such movements can be remediated by strengthening the soils during grading. This can be achieved by excavating the weak soils and replacing them as properly compacted (engineered) fill.
Expansive Soil and Bedrock

In addition, the surface soils are occasionally expansive and the bedrock is also expansive. Expansive soils and bedrock exposed at the surface will shrink and swell as they lose and gain moisture throughout the yearly weather cycle. Near the surface, the resulting movements can heave and crack lightly loaded shallow foundations (spread footings) and slabs and pavements. The zone of significant moisture variation (active layer) is dependent on the expansion potential of the soil and the extent of the dry season. In the Sonoma County area, the active layer is generally considered to range in thickness from about 2 to 3 feet. The detrimental effects of the above-described movements can be remediated by pre-swelling the expansive soils and covering them with a moisture fixing and confining blanket of properly compacted select fill, as subsequently defined. In building and tank areas, the blanket thickness required depends on the expansion potential of the soil and bedrock and the anticipated performance of the foundations and slabs. In order to effectively reduce foundation and slab heave given the expansion potential of the site’s soils and bedrock, a blanket thickness of 30 inches will be needed. In exterior slab and paved areas, the select fill blanket need only be 12 inches thick.

Downslope Creep

Weak, creep-prone surface soils, such as those found at the site, tend to naturally consolidate and settle on sloping terrain. 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 - All 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 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.

Foundation and Slab Support - Provided grading is performed as discussed above for expansive soil and bedrock, and creeping soils, satisfactory foundation support for structures and the water tanks can be obtained from spread footings and mat slabs, respectively, that bottom on the select engineered fill. Satisfactory support for the interior slabs can also be obtained from the select engineered fill. Satisfactory support for site retaining walls can be obtained by spread footings that bear on select engineered fill or firm, undisturbed bedrock at the depths recommended herein. Alternatively, site retaining walls can be supported on drilled piers.
Excavation Difficulty

Site excavation will 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.

Exterior Slabs and Pavements

Exterior slabs and pavements will heave and crack as the expansive soils and bedrock shrink and swell through the yearly weather cycle. Slab and pavement cracking and distress are typically concentrated along edges where moisture content variation is more prevalent within subgrade soils. Slab and pavement performance and the incidence of repair can be reduced by covering the pre-swelled expansive soils with at least 12 inches of select fill (see “On-Site Soil Quality” section) prior to constructing the slab or pavement required to carry the anticipated traffic.

On-Site Soil Quality

All fill materials used in the upper 30 inches of the building and tank areas and the upper 12 inches of exterior slab and pavement subgrade 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 fill, but will not be suitable for use as 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

Provided remedial grading is performed as recommended herein, we estimate that total settlements of heavily loaded column footings will be about 1-inch and settlement of the strip footings will be ½-inch. We estimate that post-construction differential settlements between columns and lightly loaded perimeter footings will be about ½-inch. We also estimate that post construction differential settlement across the tank area will be less than 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 surface soils are susceptible to erosion and sloughing. Surface runoff typically sheet flows over the ground surface 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 cause deeper than normal soil heave and/or 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 surface soils and migrate downslope at the interface of the surface soil and bedrock and through fractures in the bedrock and seep into the slab rock. Groundwater will also seep into excavations exposing 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 subsurface drainage and slab underdrains as recommended herein.

Serpentinite Bedrock Grading

We did not encounter serpentinite bedrock in our explorations at the new winery site. However, serpentinite has been encountered at the Cornell property. Serpentinite can contain naturally occurring asbestos fibers. There are State enforced regulations giving specific measures that must be used during grading to mitigate hazards associated with airborne asbestos particles. If these materials are exposed during grading, we, in accordance with these regulations, will notify the general contractor and State of the potential hazard, and recommend you retain certified asbestos personnel to sample and test the presence or absence of asbestos and issue a work plan to mitigate airborne asbestos during grading, if necessary.

RECOMMENDATIONS

Seismic Design

Seismic design parameters presented below are based on Section 1613 titled “Earthquake Loads” of the 2007 California Building Code (CBC). Based on CBC Table 1613.5.2, we have determined a Site Class C should be used for the subject site. Using a site latitude and longitude of 38.5188°N and -122.5838°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.

<table>
<thead>
<tr>
<th>Spectral Response Parameter</th>
<th>Acceleration (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ss (0.2 second period)</td>
<td>1.450</td>
</tr>
<tr>
<td>S1 (1 second period)</td>
<td>0.595</td>
</tr>
<tr>
<td>SsS (0.2 second period)</td>
<td>1.450</td>
</tr>
<tr>
<td>Sss (1 second period)</td>
<td>0.774</td>
</tr>
<tr>
<td>Sds (0.2 second period)</td>
<td>0.967</td>
</tr>
<tr>
<td>Sd1 (1 second period)</td>
<td>0.516</td>
</tr>
</tbody>
</table>
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 stripplings 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, interior slab-on-grade, and tank areas, the weak, compressible, expansive surface soils should be excavated to within 6 inches of their entire depth. Additional excavation should be performed, as necessary, to allow space for the installation of a blanket of select fill, at least 30 inches thick, beneath the building and tank pad subgrades. This additional excavation may require the removal of expansive bedrock. Additional excavation may also be required to maintain a differential fill thickness of less than 5 feet. The excavation of weak, compressible, expansive soils and bedrock should also extend at least 12 inches below exterior slab and pavement subgrade to allow space for the installation of the select fill blanket discussed in the conclusions section of this report. Fills should be constructed by excavating level keyways that expose undisturbed bedrock. The keyways should be at least 10 feet wide, extend at least 2 feet below the bedrock 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 weak, compressible, expansive materials should extend at least 5 feet beyond the outside edge of the exterior footings of the proposed buildings and 3 feet beyond the edge of exterior slabs and pavements. 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.
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 35) 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, and must be approved by the geotechnical engineer prior to use. The upper 30 inches of fill beneath and within 5 feet of the building and tank areas and the upper 12 inches of fill beneath and within 3 feet of exterior slabs and pavement edges should be select fill. We judge the on-site soils are generally suitable for use as general fill but will not be suitable for use as 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:

<table>
<thead>
<tr>
<th>SIEVE SIZE</th>
<th>PERCENT PASSING (by dry weight)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6 inch</td>
<td>100</td>
</tr>
<tr>
<td>4 inch</td>
<td>90 – 100</td>
</tr>
<tr>
<td>No. 200</td>
<td>10 – 60</td>
</tr>
</tbody>
</table>

- Liquid Limit – 40 Percent Maximum
- Plasticity Index – 15 Percent Maximum
- R-value – 20 Minimum (pavement and tank areas only)
- Effective Internal Friction Angle – 30 degrees (retaining wall backfill/fill slopes steeper than 3:1)
- Maximum Total Unit Weight – 130 pcf

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. With the exception of being used as wall backfill, 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. For retaining wall backfill approval, the grading contractor should allow 10 days for laboratory testing and approval.

Fill Placement

The surface exposed by stripping and removal of weak, compressible, expansive surface soils should be scarified to a depth of at least 6 inches, uniformly moisture-conditioned to near optimum (about 4 percent above optimum where expansive soil or bedrock is present) and compacted to at least 90 percent of the maximum dry density of the materials as determined by ASTM Test Method D-1557. In expansive soil and bedrock areas, moisture conditioning should be sufficient to completely close all shrinkage cracks for their full depth within pavement, exterior slab, building, and tank areas. If grading is performed during the dry season, the shrinkage cracks may extend to a few feet below the surface. Therefore, it may be necessary to excavate a portion of the cracked soils to obtain the proper moisture condition and degree of compaction. Approved fill material should then be spread in thin lifts (8 inch loose thickness), 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. Expansive soils used as fill should be moisture-conditioned to about 4 percent above optimum. Only approved select materials should be used for fill within the upper 30 inches of interior slab and tank pad subgrade and within the upper 12 inches of exterior slab and pavement subgrade. Fills should be continually keyed and benched into firm, undisturbed bedrock. The benches should allow space for the placement of select fill of even thickness under settlement sensitive structural elements supported directly on the fill. An illustration of this grading technique is shown on Plate 35.
# SUMMARY OF COMPACTION RECOMMENDATIONS

<table>
<thead>
<tr>
<th>Area</th>
<th>Compaction Recommendation (ASTM D-1557)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Preparation for areas to receive fill</td>
<td>After preparation in accordance with this report, compact upper 6 inches to a minimum of 90 percent relative compaction. The moisture content should be at least 4 percent over optimum where expansive soils are present.</td>
</tr>
<tr>
<td>General fill (native or import)</td>
<td>Compact to a minimum of 90 percent relative compaction. Expansive soils used as fill should be moisture conditioned to at least 4 percent above optimum.</td>
</tr>
<tr>
<td>Structural fill beneath buildings and the tank pad, extending outward to 5' beyond building perimeter</td>
<td>Compact to a minimum of 90 percent relative compaction.</td>
</tr>
<tr>
<td>Structural fill beneath building pads that transition between bedrock and fills less than 3 feet thick</td>
<td>Compact to a minimum of 95 percent relative compaction</td>
</tr>
<tr>
<td>Trenches</td>
<td>Compact to a minimum of 90 percent relative compaction. Compact the top 6 inches below vehicle pavement subgrade to a minimum of 95 percent relative compaction.</td>
</tr>
<tr>
<td>Retaining wall backfill</td>
<td>Compact to a minimum of 90 percent relative compaction, but not more than 95 percent.</td>
</tr>
<tr>
<td>Pavements, extending outward to 3' beyond edge of pavement</td>
<td>Compact upper 6 inches of subgrade to a minimum of 95 percent relative compaction.</td>
</tr>
<tr>
<td>Concrete flatwork and exterior slabs, extending outward to 3' beyond edge of slab</td>
<td>Compact subgrade to a minimum of 90 percent relative compaction. Where subject to vehicle traffic, compact upper 6 inches of subgrade to at least 95 percent relative compaction.</td>
</tr>
<tr>
<td>Aggregate Base</td>
<td>Compact aggregate base to at least 95 percent relative compaction.</td>
</tr>
</tbody>
</table>

**Permanent Cut and Fill Slopes**

In general, cut and fill slopes should be designed and constructed at slope gradients of 3:1 (horizontal to vertical) or flatter, unless otherwise approved by the geotechnical engineer in specified areas. Where 2:1 fill slopes are required, they should be constructed with the outer 10
feet of the fill slope consisting of select fill. Where steeper slopes are required, retaining walls should be used. Fill slopes steeper than 2:1 will require the use of geogrid to increase stability. Providing recommendations for grid type and spacing was not part of our requested and/or proposed scope of work. Should the need to use geogrid arise, additional laboratory testing and stability analyses will be required. 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 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. The cut and fill slope inclinations recommended herein address only the stability of the slopes. It should not be inferred that they address the feasibility of landscaping and weed control. Where these are concerns, the slopes should be flattened accordingly.

**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.

**Serpentinite Bedrock Grading**

If serpentine bedrock is encountered during grading, a certified asbestos professional must be retained to collect and test samples for the presence of naturally occurring asbestos. If asbestos is detected, measures to mitigate the health hazards to workers associated with airborne asbestos fiber particles generated during grading should be implemented. These should include consulting with the regulatory agencies, preparing a work plan and verifying this plan is implemented.

**Foundation Support**

Provided remedial grading is performed as recommended herein, the proposed structures can be supported on continuous and isolated spread footings that bottom on firm, undisturbed non-expansive bedrock or select engineered fill. Site retaining walls outside of the proposed structures can be supported on spread footings or drilled piers. The tanks can be founded on a concrete mat slab. Specific recommendations for each alternative are given in the following sections of the report.
Spread Footings

Spread footings should be sized by the structural engineer and should be no less than 12 inches wide. Footings should bottom on firm, undisturbed, non-expansive bedrock or select engineered fill, as applicable, at least 18 inches below lowest adjacent grade. Where expansive bedrock is present at site retaining wall excavations, this depth should be increased to 36 inches. Footings for structures may need to be deepened to maintain a differential fill thickness of less than 5 feet beneath the footings for a given structure. Additional embedment or width may be needed to satisfy code and/or structural requirements. On ungraded sloping terrain, the footings should be stepped as necessary to produce level tops and bottoms. Footings should be deepened as necessary to provide at least 7 feet of horizontal confinement between the footing bottoms and the face of the nearest slope.

The bottoms of all footing excavations should be thoroughly cleaned out or wetted and compacted using hand-operated tampering 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 2000, 3000 and 4000 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 firm, undisturbed non-expansive bedrock or 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.

Drilled Piers

Drilled, cast-in-place, reinforced concrete piers can be used for foundation support for site retaining walls. Drilled piers should be at least 16 inches in diameter and should extend at least 6 feet below the bedrock surface. Larger piers and deeper embedment may be needed to resist the lateral forces imposed by earthquakes per the 2007 California Building Code. Piers should be spaced no closer than 3 pier diameters, center to center.

Skin Friction - The portion of the piers extending below the bedrock surface may be designed using an allowable skin friction of 600 psf for dead load plus long term live loads. This value can be increased by ½ for total loads, including downward vertical wind or seismic forces. A skin friction value of 400 psf should be used to resist uplift forces. End bearing should be neglected because of the difficulty of cleaning out small diameter pier holes, and the uncertainty of mobilizing end bearing and skin friction simultaneously.

Lateral Forces - Lateral loads on piers will be resisted by passive pressure on the bedrock. An equivalent fluid pressure of 350 pcf acting on two pier diameters should be used. Confinement
for passive pressure may be assumed from below the bedrock surface. Once wall locations have been determined, we should be consulted to provide the depth to confinement for each wall.

The piers should be interconnected with grade beams to support wall loads and to redistribute stresses imposed by wind or earthquakes and the expansive materials. The grade beams should be designed to span between the piers in accordance with structural requirements. The steel from the piers should extend sufficient distance into the grade beams to develop its full bond strength.

Uplift Forces - The piers and grade beams should be designed to resist uplift pressures imposed by expansive soils. The uplift pressure should be assumed to be 1,500 psf of grade beam surface contact.

Pier Drilling - We did not encounter groundwater and/or caving-prone soils within the planned pier depth at the potential wall locations during our study. If groundwater is encountered during drilling, it may be necessary to de-water the holes and/or place the concrete by the tremie method. If caving soils are encountered, it may be necessary to case the holes. Difficult drilling may be required to achieve the required penetration. The drilling subcontractor should review this report, become familiar with site conditions as they pertain to his operation and draw his own conclusions regarding drilling difficulty, suitable drill rigs and the need for casing and dewatering prior to bidding.

Concrete - Concrete mix design and placement should be done in accordance with the current ADSC and/or ACI specifications. Concrete should not be allowed to mushroom at the top of the piers or below the bottom of grade beams.

Mat Slab

Mat slabs should bottom on select engineered fill and should be designed by the project structural engineer. We understand that mat slabs are either a uniform thickness or a thinner general slab with thickened areas or regions of heavier loading. The bottoms of thickened area excavations should be treated like footings and 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 beam excavations. These excavations should not be allowed to dry before placing concrete. The moisture condition of the beam excavations should be checked by the geotechnical engineer no more than 24 hours prior to placing concrete.

Provided grading is performed as recommended herein, a modulus of subgrade reaction of 100 pounds per cubic inch (pci) may be used for design. This value is based on an assumed tank pad subgrade R-value of 20. Mat slabs may be designed using allowable bearing pressures of 2000, 3000, and 4000 pounds per square foot for dead load, dead plus live load, and total loads (including wind and seismic forces), respectively. Resistance to lateral loads can be obtained from passive resistance against the edge of the mat and friction across the base of the mat. Passive resistance of 350 pcf and a friction factor of 0.35 may be used for design.
Retaining Walls

We understand that the soil nail wall will be designed by others and that we are not providing design criteria for that wall. We are, however, providing design criteria for non-soil nail walls at the site.

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.

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

<table>
<thead>
<tr>
<th>Earth Equivalent Fluid Pressures*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Active Pressures (level backfill)</td>
</tr>
<tr>
<td>Active Pressures (backfill slopes up to 2:1)</td>
</tr>
<tr>
<td>At Rest Pressures</td>
</tr>
</tbody>
</table>

* All pressures assume that import select fill is used as wall backfill in accordance with the "Wall Drainage and Backfill" section.

Where required by the building code, retaining walls with horizontal backfill should be designed to resist a seismic pressure of 17 pcf 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). For wall backfill sloping up to 3:1, a seismic earth equivalent pressure of 46 pcf should be used. Where wall backfill slopes are steeper than 3:1 and seismic wall pressures are required, we should be consulted to provide recommendations for pressure and backfill. The above 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.

Foundation Support

Retaining walls should be supported on spread footings or drilled piers, as applicable, 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.
Wall Drainage and Backfill

Retaining walls should be backdrained as shown on Plate 36, 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. 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. As discussed previously, the on-site soils available for use as wall backfill are expansive, and thus, import backfill will be required. 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

Because of expansive soils and bedrock, slab-on-grade floors should not be used in interior areas that are not underlain by at least 30 inches of select fill. Slabs-on-grade can be used in exterior flatwork areas provided the slabs are underlain by at least 12 inches of select fill (not counting the slab rock). Slab-on-grade subgrade should be rolled to produce a dense, uniform surface. 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 ⅝-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 Sonoma, 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**

Provided the site grading is performed to remediate expansive soil heave, as recommended herein, the uppermost 12-inches of pavement subgrade soils will be imported select fill with a minimum R-value of 20. Based on that R-value we recommend the pavement sections listed in the table below be used.

<p>| PAVEMENT SECTIONS WITH IMPORTED SELECT FILL SUBGRADE |
|---------------------------------------------|---------------------------------|-----------------|-----------------|</p>
<table>
<thead>
<tr>
<th>TI</th>
<th>ASPHALT CONCRETE (feet)</th>
<th>CLASS 2 AGGREGATE BASE (feet)</th>
<th>IMPORTED SELECT FILL* (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.0</td>
<td>0.30</td>
<td>1.15</td>
<td>1.0</td>
</tr>
<tr>
<td>6.0</td>
<td>0.25</td>
<td>1.05</td>
<td>1.0</td>
</tr>
<tr>
<td>5.0</td>
<td>0.20</td>
<td>0.90</td>
<td>1.0</td>
</tr>
</tbody>
</table>

* R-value ≥20

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. Alternatively, paving can be performed in two lifts with construction traffic using the first lift. Prior to placing the final lift, damaged areas should then be repaired. 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 8 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 Sonoma 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.

Parking Lot Drainage

Water tends to migrate under pavements and collect in the aggregate courses at low areas on parking lot subgrade soils, such as around storm drain inlets and the thread of paved swales leading to inlets. The ponded water will soften subgrade soils and, under repetitive heavy-wheel loads, will induce inordinately high stresses on the subgrade and pavement components that could result in untimely maintenance. Under-pavement drainage can be improved and maintenance reduced by replacing a 12-inch wide strip (extending at least 15 feet on either side of the inlet) of the select subbase layer or subgrade soils with a subdrain consisting of ¾-inch or 1⅛-inch free-draining Class 1 Permeable Material. The drain rock should be outletted into the storm drain inlet. Storm drain trenches can be made to serve as pavement subdrains. We should be consulted to verify the suitability of storm drain trenches as pavement subdrains in a case-specific basis.

Where pavements will abut landscaped areas, the pavement baserock layer and subgrade soils should be protected against saturation from irrigation and rainwater with a subdrain, similar to that previously discussed. The subdrain should extend to a depth of at least 6 inches below the bottom of the baserock layer. Alternatively, a grouted moisture cut-off that extends 12 inches below the bottom of the baserock layer should be provided below or immediately behind the curb and gutter.

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 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 erosion resistant natural drainages 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, both ways. Additional drain trenches should be installed, as necessary, to drain all 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 37. 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.

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 Cornell Farms, LLC 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 test pits and core 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 during April 2010 and may not necessarily be the same or comparable at other times.

It should be understood that slope failures including landslides, debris flows and erosion are ongoing natural processes which gradually wear away the landscape. Residual soils and weathered bedrock can be susceptible to downslope movement, even on apparently stable sites. Such inherent hillside-and-slope risks are generally more prevalent during periods of intense and prolonged rainfall, which occasionally occur, in northern California and/or during earthquakes. Therefore, it must be accepted that occasional, unpredictable slope failure and erosion and deposition of the residual soils and weathered bedrock materials are irreducible risks and hazards of building upon or near the base of any hillside or any steeper slope area throughout northern California. By accepting this report, the client and other recipients acknowledge their understanding and acceptance of these risks and hazards, and the terms and conditions herein.

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

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<td>Plate 37</td>
<td>Typical Subdrain Details Illustration</td>
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</table>
Reference: Maptech TopoQuad, Calistoga, California Quadrangle

Scale: 1" = 2000'

RGH CONSULTANTS

SITE LOCATION MAP
Cornell Winery Supplemental Study
Wappo Road
Santa Rosa, California

Job No: 2096.05.05.1 | Date: June 2010