September 21, 2009

Cornell Farms, LLC
% Davis Family Vineyards
Attention: Guy Davis
52 Front Street
Healdsburg, CA 95448

Response To BZA Comments
Cornell Winery
245 Wappo Road
Santa Rosa, California

PROJECT NUMBER: 2096.05.05.1

INTRODUCTION

This letter responds to questions from the Commissioners following the County of Sonoma Permit and Resource Management Department (PRMD) Board of Zoning Adjustments (BZA) hearing held on November 13, 2008 for the proposed Cornell Winery located at 245 Wappo Road in Sonoma County, California. The Commissioners questions along with questions and issues from PRMD staff are summarized in an e-mail correspondence dated December 15, 2008 (attached). The geologic and geotechnical related comments and issues address the proposed winery, the proposed leach field for the winery, and a landslide that occurred behind an adjacent residence. These areas are shown relative to one another on the Site Location Map (Plate 1) and the General Site Plan (Plate 2).

PURPOSE AND SCOPE

The purpose of this letter is to address comments, questions and issues by the BZA Commissioners and PRMD staff in order to obtain a land use permit for the proposed winery project. Our scope of services included reviewing our previous work and selected published geologic data pertinent to the site; evaluating subsurface conditions with test pits, profile holes, a mud rotary core boring and laboratory tests; analyzing the field and laboratory data; and summarizing the results of our work in this letter. Previous studies performed by RGH Consultants, Inc. for the proposed winery project include a Preliminary Geologic Study report, dated May 31, 2006 and our report update of April 22, 2008. This Preliminary Geologic Study and update included geologic mapping, subsurface exploration with test pits, core borings, and bucket auger borings and laboratory testing. Bucket auger borings were downhole logged by our geologist.
The BZA comments and PRMD questions and issues of the December 15, 2008 e-mail correspondence pertaining to RGH are as follows:

1. Comment 3 - Slope stability analysis of the winery site.

2. Comments 11 and 16 – Location of a landslide that occurred on the parcel in 2006 relative to the proposed winery site, and the cause of the 2006 landslide.

3. Comment 20 - Impacts of the proposed leachfield on potential landsliding and whether the septic system can add enough water to cause lubrication that could lead to sliding.


The areas of comment are shown and appropriately labeled on Plates 1 and 2.

COMMENT 3 – SLOPE STABILITY ANALYSIS FOR THE WINERY SITE

The proposed winery site is located on the southeastern quadrant of the parcel and on the northwesterly (downhill) side of Wappo Road (refer to Plate 2). A dormant landslide is located on the northeastern portion of the site that extends into the winery configuration. The dormant landslide exhibits rounded and subdued geomorphic features and locally dense re-vegetation.

SITE CONDITIONS

The proposed winery site is located on a 40-acre parcel that is accessed by Wappo Road from the north side of St. Helena Road. Wappo Road is partially paved and partially dirt and gravel. The southwestern quadrant of the parcel contains a residence, a leachfield, a well and a reportedly small "hobby" vineyard. The proposed winery site is situated on the parcel’s southeastern quadrant on the upper flanks of a knoll on the northwesterly (downhill) side of Wappo Road. The local terrain is characterized by westerly to northwesterly-sloping ridges and intervening ravines that drain into a southwesterly-flowing, Class IV watercourse. Slopes across the winery site generally range between 3½:1 (horizontal to vertical) and 7½:1. During the summer of 2005, the winery site was grubbed of natural scrub and is presently covered with erosion control protection including straw, bales, wattles, and dense perennial grasses.

SUPPLEMENTAL EXPLORATION

On January 19, 2009, we drilled and logged an additional core boring at the winery landslide location in order to analyze slope stability under long-term static conditions and seismic conditions. The core boring was drilled to a depth of 42 feet with a track-mounted mud rotary drill rig. Continuous core samples of the soils and bedrock were recovered at selected intervals by using pitcher barrels and a 101 mm geobarrel system. The core samples were visually classified and logged by our geologist, and stored in core boxes for transport to our laboratory.

The location of the core boring is shown on the Map of Dormant Landslide and Winery, Plate 3. Plate 3 also includes the exploration work we performed for the Preliminary Geologic Report and other mapped geologic features. Where features are located outside of the dormant landslide they have been shaded back. The log of the core boring showing the materials encountered and core sample recovery is shown on Plates 4a and 4b. The soils are described in accordance with the
Unified Soil Classification System, outlined on Plate 5. Bedrock is described in accordance with Engineering Geology Rock Terms, shown on Plate 6.

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

LABORATORY TESTING

The samples obtained from the core boring were transported to our office and re-examined to verify classifications, evaluate characteristics, and assign tests pertinent to our analysis. Selected samples were laboratory tested to determine their strength characteristics, liquid and plastic limits and grain size distribution needed for our analysis.

SUBSURFACE

This section describes the subsurface conditions encountered within the core boring (CB-7) drilled within the dormant landslide. The subsurface conditions and geology through the winery and the dormant landslide are depicted in our geologic profile presented on Plate 7.

The core boring indicates that colluvium generally comprised of clayey soils extends to a depth of approximately 8 feet, and exhibits moderate plasticity and low strength. Dormant landslide debris was encountered from below the colluvium to a depth of approximately 21½ feet. The dormant landslide debris generally consists of sheared, shattered shale that is firm, plastic and moderately weathered. Fractures are generally very closely to extremely closely spaced. The dormant landslide debris contains talc patches, root mats and pinched clay seams.

Franciscan Complex bedrock extends from beneath the dormant landslide debris (21½ feet) to the maximum depth explored (42 feet). The bedrock consists of the sheared shale melange unit, and locally contains intermixed graywacke sandstone. Melange 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 sheared shale is generally firm to moderately hard, friable to weak and moderately to slightly weathered. The sheared material is locally foliated and exhibits extremely closely spaced fractures with hard graywacke sandstone and quartz veining. Few distinct and continuous fracture planes were observed within the Franciscan rocks and bedding is generally absent.

SLOPE STABILITY

This section presents a discussion of slope stability related to the proposed winery site. We also performed slope stability analysis for the proposed leachfield site. That analysis is discussed in the response to the related question. All of our slope stability analyses were performed using the computer program Slope/W (GEO-SLOPE International, Ltd., 2007). Stability was evaluated under long-term static conditions and seismic conditions. Laboratory test data and slope stability printouts will be presented in a Geotechnical Study Report for the winery project.

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 proposed buttressed fill and the
underlying 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 using both methods and, where multiple data points were available, we averaged the results. Comparing our strength parameters for the bedrock materials at the site with those determined by Condor Earth Technology Inc (Condor) for the winery cave portals, our parameters are appropriately conservative for this analysis. The parameters used in our analysis and those determined for the Franciscan Melange by Condor are presented in the table below.

<table>
<thead>
<tr>
<th>Unit</th>
<th>$\phi'$ (degrees)</th>
<th>$c'$ (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Landslide Debris</td>
<td>19</td>
<td>75</td>
</tr>
<tr>
<td>Buttress Fill Material</td>
<td>26</td>
<td>150</td>
</tr>
<tr>
<td>Franciscan Melange (RGH)</td>
<td>24</td>
<td>249</td>
</tr>
<tr>
<td>Franciscan Melange (Condor, 2009)</td>
<td>26</td>
<td>600</td>
</tr>
</tbody>
</table>

Using these parameters, our stability analysis yielded a Factor of Safety greater than or equal to 1.51 for the proposed removal of dormant landslide debris and construction of a drained, buttress fill.

**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 buttress fill 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, 1997), which is a guideline for evaluation and mitigation of landslide hazards. 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 0.2159.

As discussed previously, there is a dormant landslide at the winery site and the portion of the landslide beneath the winery will be removed and replaced as a buttressed fill. For the conditions at the site, the first most likely failure during an earthquake would be a failure of a portion of the debris within the dormant landslide. The next likely scenario would be the reactivation of the existing dormant landslide feature. Therefore, our seismic analysis was focused on evaluating the reactivation of the existing landslide and its impact on the buttressed fill constructed for the winery.

Seismic stability can be evaluated using the strength parameters determined above or shear strength ($s_u$) for the materials. Shear strength is determined from unconsolidated-undrained triaxial compression tests (TXUU). The shear strength parameters we used for our analysis were based on laboratory tests on the subsurface materials or, in the case of the fill, estimated based on our experience with similar materials. The parameters we used for our evaluation are presented in the table below.
Evaluating the seismic stability using shear strength parameters, our stability analysis yielded a Factor of Safety greater than or equal to 1.10.

CONCLUSIONS AND RECOMMENDATIONS

Based on our slope stability analysis under long-term static conditions and seismic conditions, the landslide can be remediated by excavating the debris to its full depth and rebuilding the slope as a drained, buttressed fill. For this type of repair, a level keyway will be excavated into bedrock at the toe of the repair, and subdrains constructed to dewater the keyway. Subsequent level benches will be excavated into firm bedrock, and the excavated landslide debris replaced as compacted fill to the top of the landslide. Subsequent subdrains will be installed at the rear of benches every 25 feet vertically and as recommended by RGH during construction. Detailed earthwork recommendations for keyway and benches, subdrains, fill placement and other geotechnical considerations will be provided in a Geotechnical Study Report for the winery project, along with laboratory test data, analyzed cross sections and slope stability printouts. Such reports are considered appropriate within the local industry and are required by PRMD for design and construction of structures in the County.

COMMENTS 11 AND 16 – 2006 LANDSLIDE

The landslide that reportedly occurred in April 2006 at the 245 Wappo Road address is located at least 650 feet downhill and southwest of the proposed winery site. The terrain at the winery site is characterized by westerly to northwesterly-sloping ridges and three primary intervening ravines that drain surface runoff into a southwesterly-flowing, Class IV watercourse downhill of the site. It has been suggested by others that local grubbing at the winery site in 2005 was a major contributor to the 2006 landslide. In our opinion, grubbing at the winery site is unrelated to the cause of the subject landslide as the ravines at the winery site do not drain onto the landslide. All three ravines at the winery site flow into the watercourse in locations upstream from the landslide area.

The nearest ravine to the subject landslide is on the southwest side of the winery site. We performed a reconnaissance along this ravine path to determine whether it flows onto the subject landslide area or to find evidence that surface runoff overtopped the ravine to flow onto the subject landslide. We found that the ravine completely bypasses the landslide area. Plate 8 is a Google Earth map that demonstrates that the path of the ravine courses downhill into the Class IV watercourse, bypassing the subject landslide area.

Further, we did not observe any evidence that surface runoff from the winery site is channeled onto the subject landslide or overtops the ravine banks to flow onto the landslide area. We observed no evidence of erosion, siltation, or downed vegetation indicating the ravine banks were overtopped. The subject landslide area, which is located on the opposite side of the southwesterly ravine, generally receives surface runoff from the immediate (uphill) area and from direct rainfall, but not from the proposed winery site.

### SEISMIC STRENGTH PARAMETERS

<table>
<thead>
<tr>
<th>Unit</th>
<th>$\varphi'$ (degrees)</th>
<th>$S_u$ (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Landslide Debris</td>
<td>0</td>
<td>1680</td>
</tr>
<tr>
<td>Buttress Fill Material</td>
<td>5</td>
<td>2000</td>
</tr>
</tbody>
</table>
At the landslide site, a residence is situated on relatively level terrain approximately 75 feet from the landslide headscarp. A small vineyard and leachfield are situated further east on relatively gentle terrain. The terrain steepens downhill of the residence at a gradient of approximately 2:1 to 2½:1. The gradient is steeper than 2:1 near the top bank of the Class IV watercourse. At this location, the watercourse is very narrow with steep sides and a free face. Volcanic bedrock outcrops are exposed outside of the subject landslide on the northeastern side. Volcanic bedrock was also observed at about 6 feet below the former ground surface at the bottom of the landslide, or about 9 feet above the bottom of the watercourse. The landslide exhibited translational movement (sliding along a roughly planar surface), and measures approximately 215 feet long, 40 to 60 feet wide and up to about 7 feet deep. These measurements calculate to an approximate volume of 2800 to 3400 cubic yards, and not 10,000 cubic yards as was suggested by others.

Heavy and prolonged rainfall occurred in northern California during the 2005-2006 rainfall season. Our historical data search for rainfall data in the Santa Rosa area (http://www.pressdemocrat.com/article/99999999/MISC/553810672&template=art plain) indicates the rainfall season lasted about 8 months, stretching from October 2005 through May 2006. Average annual rainfall in the Santa Rosa area is reportedly about 30 inches. Recorded rainfall in Santa Rosa was approximately 46 inches, or about 150 percent of normal, during this period. Prolonged rainfall was especially prevalent during December 2005 when nearly 18 inches of rain fell. Nearly 10 inches of rain fell from December 25 to 31, with 4 inches falling on New Years Eve. Hundreds of landslides occurred in the region in response to this rainfall. Heavy rains fell well into March and April with a combined total of about 15 inches. In March, rain fell during 25 of 31 days of the month and 16 of the first 17 days of April were rainy.

In general, landslides are often triggered when water rapidly accumulates in the surface soil. This can occur during a single intense storm or during a series of storms accompanied with prolonged, heavy rainfall. As heavy and/or prolonged rainfall occurs, permeability decreases with depth. A perched water table develops, in effect trapping the rainwater in the surface soil. The surface soil becomes filled with rainwater and pore-water pressures increase to a point that reduces the shear strength of the material to a critical level, resulting in failure. In geologic terms, the driving force (saturated mass on a steep slope) exceeds the resisting force (soil strength) resulting in slope failure. At the subject landslide, the volcanic bedrock acted as the impermeable surface described above. In addition to the heavy rains, factors contributing to the landslide activation were the steep slope and a free face at the bank of the watercourse at the bottom.

In summary, the landslide occurred from several factors including the slope steepness (2:1 to 2½:1), an impermeable surface (bedrock) that keeps rainwater in the soil mass, an unsupported bank (free face) at the top of the watercourse and intense and prolonged rainfall that was greater than is typically experienced in this area. However, the water source at the landslide was from the immediate drainage area upslope and from direct rainfall, and not from channeled or concentrated flow from the ravines at the winery site. These ravines bypass the subject landslide area and drain directly from the winery site into the watercourse downhill, and we did not observe evidence that surface water overtopped the ravine banks. Therefore, we judge that neither surface runoff from the winery site nor grubbing contributed to the subject landslide.

**COMMENT 20 – IMPACTS OF LEACHFIELD ON SLOPE STABILITY**

The leachfield site was moved from its original location to a new site at 560 Wappo Road. This site is on the northwestern portion of the Cornell Farms land, as shown on Plates 1 and 2, and is accessed by a dirt road that traverses along the northern side of a westerly trending ridge. We understand that the leachfield will be a subsurface drip system.
SITE CONDITIONS

The leachfield site extends over terrain sloping at a gradient of between 5:1 and 9:1, and is vegetated with fir and oak trees and natural grasses. The leachfield site is depicted on Plate 9. Graymer et al. (2007) maps this site as being underlain by rocks of the graywacke and melange unit of the Franciscan Complex. Neither Huffman and Armstrong (1980) nor Dwyer et al. (1976) show this ridgetop site as being underlain by a landslide, and we did not observe evidence of landslides at this site during our study. Geologic profiles depicting the subsurface geology are presented on Plates 10 and 11. As shown on Plates 9 and 10, a landslide feature was observed on the slope below the roadway that is located downslope of the proposed leachfield. This relatively shallow feature appears to be related to grading for an old roadway. The roadway crosses above the headscarp and switches back across the toe and may have concentrated drainage and contributed to the failure.

EXPLORATION

On March 24, 2009, we performed a reconnaissance and logged three test pits excavated at the site with a Takeuchi TB145 track-mounted mini-excavator equipped with a 24-inch bucket. These test pit locations were surveyed and are shown on Plate 9. On August 19, 2009, we supplemented the three test pits by excavating four test pits at the approximate locations shown on Plate 9. Our geologist located and logged the test pits and obtained samples of the materials encountered for visual examination and classification. The logs of the test pits are shown graphically on Plates 12 and 13. The soils are described in accordance with the Unified Soil Classification System shown on Plate 5. Bedrock is described with Engineering Geology Rock Terms shown on Plate 6.

SUBSURFACE

Based on our observations of the test pits, the surface soil consists of porous clayey or silty sands, sandy clay and clayey gravel that extend to depths of between ½ and 2½ feet below the ground surface. Completely weathered bedrock was found in all of the test pits except Pit J, where it was presumably removed during the grading of the road. The completely weathered bedrock extends from below the surface soil to a depth of between 2 and 7 feet below the ground surface, and has predominantly weathered to a clayey sand or sandy clay and contains varying quantities and sizes of remnant harder sandstone clasts. Sandstone bedrock of the Franciscan Complex was encountered below the completely weathered bedrock to the maximum depth explored (8½+ feet). The sandstone is generally fine to medium grained, firm, friable to weak, slightly to moderately weathered, and exhibits very closely spaced fractures.

Free groundwater was not observed in our test pits at the time of excavation. 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.

SLOPE STABILITY

This section presents a discussion of slope stability related to the proposed leachfield site. We also performed slope stability analysis for the proposed winery site. That analysis is discussed in the response to the related question. Slope stability printouts will be presented in a Geotechnical Study Report for the winery project.
Static Stability - Static stability for long-term conditions was evaluated for a Factor of Safety against failure of 1.5, which as discussed earlier is generally accepted as the standard of practice for these conditions. In order to evaluate the impact of the leachfield on the slope, we first evaluated the static stability of the existing conditions as shown on Geologic Profiles LFA-LFA' and LFB-LFB presented on Plates 10 and 11, respectively. We then analyzed the static condition with water from the leachfield saturating the subsurface materials. We believe the leachate water will generally flow downward into the formation. However, for our analysis we conservatively assumed that the water flows both downward and horizontally and modeled this by extending the water surface down at a 2:1 (horizontal to vertical) inclination from the edge of the leachfield.

Our analysis required strength parameters for both the proposed near surface completely weathered bedrock and the underlying less weathered bedrock materials. For the evaluation of long-term static loading conditions, we used the effective strength parameters including internal friction angle and cohesion. We estimated the strength parameters for the less weathered bedrock using the computer program RocLab (2007). These values are presented in the table below. For the completely weathered bedrock, the effective internal friction angle was developed from correlations related to plasticity index presented in Ladd et al (1977). This correlation yielded an effective internal friction angle that ranges from about 28 to 34 degrees. We typically choose a value in the middle of this range, but because a landslide feature is present within this material we used 28 degrees for our analysis. In order to determine the effective cohesion of the completely weathered bedrock, we performed a global stability analysis for geologic Profile LFA-LFA' which is the profile that crosses through the landslide feature we observed. In our analysis, we back-calculated the effective cohesion using the effective internal friction angle of 28 degrees and the assumption that the slope currently has a Factor of Safety of about 1.0. If the Factor of Safety was less than 1.0, the slope should actively be failing, which is not the case. With these assumptions, we calculated an effective cohesion of 30 pounds per square foot (psf). We then used these parameters to evaluate the global stability of the slope represented by Geologic Profile LFB-LFB'. The Factor of Safety was calculated to be 1.0, which is consistent with the results of the analysis for LFA-LFA' and indicates the values used are appropriate. In summary, the strength parameters used for our evaluation are presented in the table below.

<table>
<thead>
<tr>
<th>STRENGTH PARAMETERS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit</td>
</tr>
<tr>
<td>Completely Weathered Bedrock</td>
</tr>
<tr>
<td>Sandstone Bedrock</td>
</tr>
</tbody>
</table>

Using these parameters, we evaluated the static stability of the steepest portion of the slope nearest to the downhill end of the leachfield, which turned out to be the cut for the roadway. We also performed an infinite slope stability analysis for the actual sloping portion where the leachfield is to be constructed. These evaluations were performed for both of the above referenced geologic profiles and with and without leachfield water. When we evaluated the addition of the leachfield water, we were looking to see if the Factor of Safety changed as it relates to the global stability analysis because we had assumed 1.0 in our evaluation. With regards to the other profiles, we were looking to see if the leachfield water lowered the Factor of Safety to less than 1.5. Using these strength parameters, our stability analysis yielded the Factors of Safety presented below.
SLOPE STABILITY FACTORS OF SAFETY

<table>
<thead>
<tr>
<th>Geologic Profile and Area Analyzed</th>
<th>Existing</th>
<th>Leachfield</th>
</tr>
</thead>
<tbody>
<tr>
<td>LFA-LFA' - Global</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>LFA-LFA' - Road Cut</td>
<td>1.88</td>
<td>1.88</td>
</tr>
<tr>
<td>LFA-LFA' - Infinite Slope</td>
<td>3.55</td>
<td>3.24</td>
</tr>
<tr>
<td>LFB-LFB’ - Global</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>LFB-LFB’ - Road Cut</td>
<td>2.08</td>
<td>2.08</td>
</tr>
<tr>
<td>LFB-LFB’ - Infinite Slope</td>
<td>4.36</td>
<td>3.76</td>
</tr>
</tbody>
</table>

With the exception of the previously noted global stability evaluations, the Factors of Safety before and after leachfield installation are greater than 1.5. The failure indicated by the global stability analysis is located in the steep slope below the roadway. These possible failure envelopes are not being influenced by the leachfield as indicated by the fact that the Factor of Safety does not change from the existing to the post leachfield installation condition.

Seismic Stability - Seismic stability of the leachfield was analyzed using the procedures and seismic coefficient (0.215) described herein for the winery site. Seismic stability that involves bedrock materials can be evaluated using the strength parameters determined above or shear strength for the materials. Because the bedrock at this location is significantly stronger and has more bedrock-like characteristics, we used the parameters presented above for our analysis. Using these strength parameters, our stability analysis yielded the Factors of Safety presented below.

SEISMIC SLOPE STABILITY FACTORS OF SAFETY

<table>
<thead>
<tr>
<th>Geologic Profile and Area Analyzed</th>
<th>Existing</th>
<th>Leachfield</th>
</tr>
</thead>
<tbody>
<tr>
<td>LFA-LFA’ - Global</td>
<td>0.68</td>
<td>0.68</td>
</tr>
<tr>
<td>LFA-LFA’ - Road Cut</td>
<td>1.14</td>
<td>1.14</td>
</tr>
<tr>
<td>LFA-LFA’ - Infinite Slope</td>
<td>1.50</td>
<td>1.38</td>
</tr>
<tr>
<td>LFB-LFB’ - Global</td>
<td>0.66</td>
<td>0.66</td>
</tr>
<tr>
<td>LFB-LFB’ - Road Cut</td>
<td>1.28</td>
<td>1.30</td>
</tr>
<tr>
<td>LFB-LFB’ - Infinite Slope</td>
<td>1.62</td>
<td>1.40</td>
</tr>
</tbody>
</table>

With the exception of the global stability evaluations for both profiles, the Factors of Safety before and after leachfield installation are greater than 1.0. As discussed above for the static analysis, the failures indicated by the global stability analyses are located in the steep slope below the roadway. These possible failure envelopes are not being influenced by the leachfield as indicated by the Factor of Safety not changing from the existing to the post leachfield installation condition.

CONCLUSIONS

The proposed leachfield site is situated on a relatively stable ridgetop and outside of landslides and steeply sloping terrain. Our stability analysis indicates that the leachfield site is stable under static
and seismic conditions both before and after the leachfield is constructed. Therefore, we judge that the proposed site is satisfactory for the planned leachfield.

COMMENT 21 – GRADING PLAN

We have reviewed the "Preliminary Grading Plan, UPE07-0008," prepared by Atterbury & Associates dated August 21 and September 14, 2009. As the title of the plans suggests, these plans are preliminary and subject to further review and modification based on the recommendations to be contained in a Geotechnical Study Report for the project upon its approval.

Sheet C1 contains a summary of earthwork estimates (appropriately surface only) for the building pad, caves, roadway, tank pad and total quantities. Total quantities are estimated to consist of 13,010 cubic yards of cut, 6670 cubic yards of fill and a net excess of 6340 cubic yards to be hauled off site.

Grading for the winery and roadways are shown to consist mostly of retained cuts and fills ranging from a few to 10 feet. Two small, minor fill slopes from 2 to 4 feet high are planned along the western side of the winery to fill low areas. The head portion of the ravine on the southwest is shown to be filled for roadway construction. The fill ranges in thickness up to about 10 feet and is inclined at a gradient of 3:1. A fire protection tank and process treatment area pad is shown on the east that will be recessed into the hillside up to 21 feet, and be retained with walls.

We judge the preliminary plans are satisfactory from a geotechnical standpoint at this time. The planned cut and fill slopes are appropriate given the site topography. Detailed earthwork recommendations for buttress keyways, subdrains, fill placement and compaction recommendations will be presented in a Geotechnical Study Report once the use permit is approved. Earthwork operations will be strictly monitored to verify our recommendations are implemented during construction.

SUMMARY

In summary, we believe this Response to BZA Comments has satisfactorily addressed the comments, questions and issues presented by BZA and PRMD staff regarding the Cornell Winery project. Upon issuance of a Use Permit we will prepare a Geotechnical Study Report that includes laboratory test data, analyzed cross sections and slope stability printouts, and recommendations for earthwork, foundations, retaining walls, slabs-on-grade, pavements and other geotechnical considerations for the design and construction of a properly built project.
We trust this provides the information you require at this time. If you have questions or wish to discuss this further, please call.

Very truly yours,
RGH Consultants, Inc.

Jared J. Pratt
Certified Engineering Geologist – 2453

Eric G. Chase
Geotechnical Engineer – 2628

cc: Atterbury & Associates
16109 Healdsburg Ave, Ste D
Healdsburg, CA 95448
(4 copies submitted)

Attachments:
Plate 1 Site Location Map
Plate 2 Overall Site Plan
Plate 3 Map of Dormant Landslide and Winery
Plates 4a and 4b Log of Core Boring CB7
Plate 5 Soil Classification Chart and Key to Test Data
Plate 6 Engineering Geology Rock Terms
Plate 7 Geologic Profile A-A’
Plate 8 Google Map of 2006 Landslide Relative to Winery Site
Plate 9 Map of Leachfield Site
Plate 10 Geologic Profile LFA-LFA’
Plate 11 Geologic Profile LFB-LFB’
Plate 12 Leachfield Site Pits E, F and G
Plate 13 Leachfield Site Pits H, I, J and K

References

List of Selected Previous Plates

Plate 16 Log of Core Boring 5
Plate 25 Log of test Pit TP-14
Plate 26 Log of test Pit TP-15
Plate 27 Log of test Pit TP-16
Plate 28 Log of test Pit TP-17

Email Comments