GEOTECHNICAL INVESTIGATION
SILVER ROSE REPORT
400 SILVERADO HILL
Calistoga, California

Silver Rose Venture, LLC
c/o Mr. Kelly Foster
Bald Mountain Development
Aspen, Colorado

11 November 2011
Project No. 730453902

Treadwell&Rollo
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11 November 2011
Project 730453902

Silver Rose Venture LLC
c/o Mr. Kelly Foster
Bald Mountain Development
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Aspen, CO 81612

Subject: Geotechnical Investigation
Silver Rose Resort
400 Silverado Trail
Calistoga, California

Dear Mr. Foster:

We are pleased to submit our geotechnical investigation report for proposed Silver Rose Resort at 400 Silverado Trail in Calistoga, California. We understand the existing Winery/Conference/Events Center will be renovated and the other existing buildings at the site will be razed. Conceptual plans for the resort include a substantial development in the northwestern portion of the site that will include hotel facilities, commercial areas, a spa, swimming pools, and a below-grade parking lot and wine cave. On the eastern portion of the site, we understand about 32 cottages/residences will be constructed surrounded by vineyard and landscaped areas.

Based on the results of our investigation, we judge that the project is feasible as planned from a geotechnical standpoint. The subsurface materials encountered at the site generally consist of a relatively thin layer of alluvium over bedrock. The alluvium generally consists of interbedded layers of sand, gravel, and clay. Where encountered, the sand and gravel was generally loose to dense and the clay was medium stiff to hard, with a low to moderate expansion potential. Details regarding these subsurface conditions and their effect on foundation design are contained in this report; therefore, anyone relying on this report should read it in its entirety.

The conclusions and recommendations contained in this report are based on a limited subsurface exploration program. Consequently, variations between expected and actual soil conditions may be found during construction. We should be retained to observe the construction conditions during site grading and fill placement, foundation installation, and testing of fill and backfill, during which time we may make changes in our recommendations, if deemed necessary.

It has been a pleasure assisting Silver Rose Venture LLC and we look forward to our continued involvement with this exciting project. If you have any questions, please call.

Sincerely yours,
Treadwell & Rollo, A Langan Company

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1.0 INTRODUCTION AND BACKGROUND

This report presents the results of our geotechnical investigation for the proposed Silver Rose Resort at 400 Silverado Trail in Calistoga, California. The resort site is southeast of the intersection of Silverado Trail and Rosedale Road in Calistoga, as shown on the Site Location Map (Figure 1). It is triangular shaped with dimensions of approximately 1,600 by 1,800 by 1,150 feet and is bound by Silverado Trail on the southwest, Rosedale Road on the north, and a dirt road and adjacent properties on the southeast, as shown on the Site Plan (Figure 2). The site is comprised of 23 acres and is currently occupied by the Silver Rose Inn, Cellars, and Spas. This facility includes two inns (one of which is now closed), a winery and tasting room, parking lots, a vineyard, two pools, an irrigation pond, and a leach field. With the exception of the leach field, the existing improvements are generally located on the western portion of the site.

In 2007 we performed a geotechnical investigation at the resort site for a different developer. Data from this previous investigation aids our general understanding of the subsurface conditions at the site. The locations where our previous borings were advanced are shown on the Site Plan, Figure 2.

2.0 PROJECT DESCRIPTION

We understand the existing Winery/Tasting Room will be renovated while the other existing buildings at the site will be razed. In addition, conceptual plans for the resort include a substantial development in the northwestern portion of the site that will include hotel facilities, commercial areas, a spa, swimming pools, and a below-grade parking level and wine cave. This complex, hereafter referred to as the Hotel/Resort complex, will be one to two stories tall over one basement parking level. We understand the lower level of this portion of the development will match the lower grade of the existing winery facility (about Elevation 360 feet[1]) where it abuts the existing winery and slopes down to the south, with a lowest finished floor elevation of about 355 feet. The Hotel/Resort complex is also planned to extend into portions of the existing irrigation pond. In these areas the existing irrigation pond will be filled. The

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1 All elevations reference Mean Sea Level (MSL, NGVD 1929), and are based on a topographic map titled "Map of Topography of the Lands of Silver Rose Inn" by Albion Surveys, Inc., revision date 12/15/10.

remaining portion of the pond will be reshaped and deepened to provide water for irrigation and frost control. The general layout of the proposed improvements is shown on Figure 2.

Beyond the area of the new Hotel/Resort complex, a new parking area is also planned in the central portion of the site along Silverado Trail. On the eastern portion of the site, we understand about 32 cottages and/or single family residences (SFRs) will be constructed across the eastern portion of the site. The cottages and SFRs will be up to two stories tall, constructed at grade, and generally consist of wood-framed construction. The area between the cottages/SFRs will be filled with vineyard and landscaping. The existing leach field will be removed and generally converted into vineyard (where it is not occupied by cottages).

The development will also include private driveways, utilities, concrete flatwork (i.e. patios), and landscaping.

3.0 SCOPE OF SERVICES

Our scope of our services was outlined in our revised proposal dated 12 April 2011. The purpose of our investigation was to evaluate the subsurface conditions at the site and develop conclusions and recommendations regarding the geotechnical and foundation aspects of the proposed resort project.

To augment the existing subsurface information at the site we drilled 13 new borings, each drilled to bedrock. On the basis of the results of our field investigation, laboratory tests and our engineering studies we developed conclusions and recommendations for design and construction of the project, including:

- soil, bedrock, and groundwater conditions at the site
- appropriate foundation type(s) for the proposed structures and other improvements
- design criteria for the recommended foundation type(s), including values for vertical and lateral resistance
- estimated foundation settlement, including total and differential settlements
- appropriate retaining wall types and lateral earth pressures for the below-grade structures
- site seismicity and seismic hazards, including liquefaction potential, lateral spreading, and cyclic densification

- seismic design criteria in accordance with 2010 California Building Code (CBC)
- site grading, including criteria for fill quality and compaction
- flexible pavement design
- temporary shoring and excavation
- underpinning of existing structures, as necessary
- construction considerations.

4.0 FIELD EXPLORATION AND LABORATORY TESTING

To augment the existing subsurface information and further evaluate the subsurface conditions in different areas of the site, we drilled 13 new borings across the site at locations where improvements are planned. The approximate locations of the borings are presented on Figure 2, the Site Plan. Prior to drilling, we obtained a soil boring permit and monitoring well permits from the Napa County Department of Environmental Health and notified Underground Service Alert (USA). In addition we hired a private utility locator to check for underground utilities in the vicinity of our borings.

Four of the borings, labeled TR-1 through TR-3 and TR-8 were advanced on 13 June through 15 June 2011 using a track-mounted hollow-stem auger drill rig by PC Exploration of Lincoln, California. PC Exploration also advanced three borings, labeled TR-9, TR-10, and TR-13 on 14 and 15 June 2011 using a portable drill rig equipped with solid-stem augers. The remaining borings, labeled TR-4 through TR-7, TR-11 and TR-12, were advanced on 15 June 2011 by Exploration GeoServices of San Jose, California using hollow-stem-auger drilling equipment.

The borings were drilled to depths of about 14 to 30 feet beneath the existing ground surface at each location. The borings were drilled under the direction of our engineer, who logged the soil and bedrock encountered and obtained representative samples for visual classification and laboratory testing.

Logs of the borings are presented on Figures A-1 through A-13 in Appendix A. The soil and rock encountered in the borings was classified in accordance with the Classification Chart and Physical Properties Criteria for rock Descriptions, presented on Figures A-14 and A-15, respectively.

Soil samples were obtained using two types of driven samplers:

- Sprague & Henwood (S&H) split-barrel sampler with a 3.0-inch outside diameter and 2.5-inch inside diameter, lined with steel or brass tubes with an inside diameter of 2.43 inches
- Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and 1.43-inch inside diameter.

The sampler types were chosen on the basis of soil type being sampled and desired sample quality for laboratory testing. The SPT and S&H samplers were driven with a 140-pound, above-ground, safety hammer falling 30 inches. The samplers were driven up to 18 inches and the hammer blows required to advance the samplers every six inches of penetration were recorded and are presented on the boring logs. A "blow count" is defined as the number of hammer blows per six inches of penetration or 50 blows for six inches or less of penetration. The driving of samplers was discontinued if the observed (recorded) blow count was 50 for six inches or less of penetration. The blow counts required to drive the samplers were converted to approximate SPT N-values and are shown on the boring log. The factors used in this conversion depend on the type of hammer used and are presented on the boring logs. The blow counts used for this conversion were: 1) the last two blow counts if the sampler was driven more than 12 inches, 2) the last one blow count if the sampler was driven more than six inches but less than 12 inches, and 3) the only blow count if the sampler was driven six inches or less.

Upon completion, two of the boreholes, TR-8 and TR-12, were converted into piezometers. Details regarding the piezometer installation are shown on the boring logs. The remaining boreholes were backfilled with a cement grout in accordance with the requirements of the Napa County. The soil cuttings were spread near each of the borings performed in 'green' areas or in landscaped areas near the borings performed in the parking lot.

Soil samples recovered from our field exploration program were re-examined in the office by a geotechnical engineer and/or a geologist and samples were selected for laboratory testing. The laboratory testing program was designed to correlate and evaluate engineering properties of the soil at the site. Samples were tested to measure moisture content, dry density, Atterberg Limits, grain size distribution, and corrosivity. Results of the laboratory tests are included on the boring logs and in Appendix B. The results of the corrosivity testing are presented in Appendix C.

5.0 SITE AND SUBSURFACE CONDITIONS

5.1 Site Conditions

The triangular site is generally occupied by the infrastructure and improvements associated with Silver Rose Inn, Cellars, and Spa. This includes two inns (one of which is currently closed), a winery/tasting room building, two pools, two parking lots, and a vineyard. The existing structures are generally located at the western portion of the site. The existing parking lot for the winery/tasting room is located in the central portion of the site and is accessed off of Silverado Trail. This parking lot has been graded relatively flat, with elevations ranging from 358 to 356 feet. We understand about three to six feet of fill was likely placed to create this flat area and surrounding vineyard slopes.

An irrigation pond is located in the northwestern portion of the site. The irrigation pond is on the order of 15 feet deep, with a maximum crest elevation at about Elevation 360 feet and a base elevation at about 345 feet. We understand the pond surface was lined with expansive clay to limit infiltration losses.

A leach field is situated along the southeastern property boundary within the vineyard. Separate from the existing resort is an existing residence on the northern-central portion of the site. In addition, on an undated map titled "Parcel Map of the Lands of Dumont Enterprises" prepared by Michael W. Brooks & Associates an easement is noted titled "private utility easement" along the southeastern site boundary. This easement is shown to be 15 feet wide and is setback 20 feet from the property line. It is unknown if this utility easement is current or if any utilities have been installed in the easement.

With the exception of several key surface features described above, the site generally slopes gently from northwest to southeast. However, in the northeastern portion of the site a rock knoll rises up above the surrounding grade, ranging in elevation from about 370 to 385 feet. This knoll is currently occupied by one of the inn structures, and a parking lot. The existing winery and tasting room have been excavated into the knoll. Beyond the extents of the knoll the site ranges in elevation from about 363 (at the northeastern corner of the site) to 348 feet at the southern corner.

5.2 Subsurface Conditions

Borings and CPTs performed at the site encountered two distinct subsurface profiles. Specifically, the knoll is underlain by 2 to 4½ feet of thin surficial soil, which is underlain by agglomeritic and pumicitic ash flow tuff. The soil encountered above the tuff was generally very stiff to hard sandy and gravelly silt and clay and very dense sandy gravel. The tuff encountered in Borings B-1 through B-3 (all on or near the knoll) was primarily moderately hard, moderately strong, and moderately weathered with occasional fractures and weaker zones. The tuff encountered in the remaining borings on or near the knoll was less competent and consisted of soft to moderately hard, was friable to weak, was moderately to deeply weathered, and occasionally plastic.

In the lower lying areas of the site we encountered 7 to 17½ feet of alluvium overlying the tuff bedrock. The alluvium consists of alternating layers of medium dense to very dense clayey sand and stiff to hard clays and silts with varying amounts of gravel. Where clayey soils were exposed at the ground surface, they generally had a low to moderate expansion potential, with the exception of TR-12 (discussed below). In the lower lying areas and where overlain by significant alluvium, the tuff bedrock at the site is generally weak, has low hardness, and is moderately to deeply weathered.

In isolated areas fill was encountered above the alluvium. The fill material encountered in boring TR-7 appeared to be similar in nature and consistency to the surrounding alluvium, and was likely 'borrowed' from another location on site. However, in borings TR-11 and TR-12 we encountered 13 to 14½ feet of weak soil that is likely fill. The material encountered, which was consistent between these two borings, consisted of 7 feet of stiff clay over 6 to 7½ feet of very loose to loose clean saturated sand. This sand appears to be much cleaner and looser than any of the other native sandy strata encountered on site. In addition, the clay encountered in TR-12 had a high expansion potential (much higher than the native materials encountered in the other borings). These two borings were advanced adjacent to the "private utility easement" and we postulate that these weak materials are fill placed following the installation of a utility (i.e. utility trench backfill). The lateral limits of this fill are not fully known at this time.

During our previous investigation we advanced several borings in the leach field. These borings encountered soft to stiff clay to the depths explored, ranging from 5 to 6½ feet deep. This is likely due to a combination of uncompacted soil backfill placed over the leach field pipes and saturation of the clayey soil by the wastewater. The borings were only advanced in the strata above the leach field pipes.

5.3 Groundwater

Groundwater was measured in most of the borings across the site. In addition, piezometers were previously installed in Borings B-2 and B-3 (extending into bedrock in the knoll) during our 2007 study. During drilling of B-1 and B-2 in January of 2007, groundwater was observed at depths of 16 and 17½ feet (Elevations 354 and 355 feet), respectively. In the remaining borings drilled in 2007 groundwater, where measured, was between 17½ and 25 feet deep, corresponding to Elevation 337 to about 324 feet. Groundwater was obscured by the drilling method in Borings B-3 and B-4 and no groundwater was encountered in borings B-7 through B-9. Water was observed in the leach field at 0, 2½ and 4½ feet deep at Borings LF-2, LF-3, and LF-5, respectively. The water level in the leach field would not be representative of a natural groundwater level; however, the contractor should expect to find perched water condition and saturated soils during construction in these areas.

During our current investigation, we encountered groundwater in most of our borings at depths between 6 and 13 feet, corresponding to Elevations 355 feet on the western side of the site and about 340 feet on the southeastern corner of the site. Free groundwater was not observed in borings TR-3, TR-8, or TR-9. Two of the borings drilled during this investigation, TR-8 and TR-12, were converted into standpipe piezometers.

Onsite maintenance personnel began taking groundwater level readings on 28 March 2011 in the original two piezometers at the site (in borings B-2 and B-3) which are one the knoll. In addition, following the installation of the two new piezometers in borings TR-8 and TR-12, on site personnel began reading these piezometers as well. The recorded groundwater level information was relayed to us on 4 November 2011 and has been included in Appendix D.

Groundwater readings taken in the piezometers indicate the high groundwater levels range from about 368 feet in the knoll area to about Elevation 343 feet in the eastern portion of the site. More notably, between 28 March and 24 October 2011, the groundwater levels beneath the knoll dropped about 13 to 15 feet below their original readings. In the southeastern portion of the site groundwater levels have fallen about 6 to 7 feet between 27 June and 24 October of 2011; see Appendix D.

Generally speaking, much of the groundwater observed on site appeared to be perched in sandy strata above the bedrock strata. In borings B-1, B-2, and B-3, the groundwater is within the bedrock. The anticipated finished floor elevation of the Hotel/Resort complex is 360 feet, about eight feet lower than the groundwater level measured in March of 2011. Significant dewatering and subsurface drainage will be required to properly lower the groundwater and mitigate the effects on the proposed structure.

6.0 REGIONAL SEISMICITY AND FAULTING

The coastal areas of Northern California are seismically active, and the site can be expected to experience both periodic minor earthquakes and likely a major earthquake (moment magnitude 7 or greater) on one of the nearby active faults during the life of the project.

The seismicity in the site vicinity is related to activity on the San Andreas Fault Zone. The faults in this system are characterized by right-lateral, predominantly strike-slip movement (movement is mainly horizontal). The major active faults in the area are the Maacama-Garberville, Hayward-Rogers Creek, and San Andreas Faults. These and other faults of the region are shown on Figure 3. For each of the active faults, the distance from the site and estimated mean characteristic Moment magnitude[2] [2007 Working Group on California Earthquake Probabilities (WGCEP) (2007) and Cao et al. (2003)] are summarized in Table 1.

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2 Moment magnitude is an energy-based scale and provides a physically meaningful measure of the size of a faulting event. Moment magnitude is directly related to average slip and fault rupture area.

TABLE 1
Regional Faults and Seismicity

Fault Segment	Approx. Distance from fault (km)	Direction from Site	Mean Characteristic Moment Magnitude
Maacama-Garberville	11.0	West	7.40
Rodgers Creek	16	West	7.07
Total Hayward-Rodgers Creek	16	West	7.33
Hunting Creek-Berryessa	23	East	7.10
West Napa	24	Southeast	6.70
Collayomi	26	Northwest	6.70
Green Valley Connected	39	East	6.80
Bartlett Springs	41	North	7.30
Great Valley 4a, Trout Creek	44	East	6.60
Great Valley 4b, Gordon Valley	45	East	6.80
Great Valley 3, Mysterious Ridge	47	East	7.10
N. San Andreas - North Coast	49	West	7.51

Figure 4 also shows the earthquake epicenters for events with magnitude greater than 5.0 from January 1800 through December 2000.

The Maacama-Garberville fault is a right lateral strike-slip fault located in the Coast Ranges of northwestern California, generally extending from just west of Calistoga to the City of Ukiah. This fault is generally considered the northern-most extension of the Hayward Fault system. Fault movement in the past 700 years has been mostly creep, however, large earthquakes have occurred on the fault over the last 3,500 years. Creep along the Maacama fault has been determined to be between 6 and 8 mm per year, based upon trench exposures and radio carbon dating. These rates are consistent with the steady movement along the rest of the Hayward Fault system. No major earthquakes have been recorded on this fault in recorded history.

The Rodgers Creek fault is a right lateral strike slip fault located south of the Maacama Fault and north of the northern segment of the Hayward fault. The fault extends from San Pablo Bay to an area east of

Healdsburg, passing in close proximity to the east side of Santa Rosa. A dilational step-over between the Hayward and Rodgers Creek faults lies beneath San Pablo Bay, and it is believed that these two faults are connected by a series of en echelon fault strands located beneath the bay. Historic creep rates on the Rodgers Creek fault are about 6 mm per year. The Rodgers Creek fault is considered to be active, and capable of producing earthquakes with significant fault rupture, or moving resulting from sympathetic movement from a major earthquake on the Hayward fault. Several earthquake events in historical time probably occurred on the Rodgers Creek fault. These include the 31 March 1898 Mare Island Earthquake which had an estimated magnitude of 6.2 and the two Santa Rosa earthquakes on 2 October 1969 with magnitudes of 5.6 and 5.7.

In 1868 an earthquake with an estimated maximum intensity of X on the MM scale occurred on the southern segment (between San Leandro and Fremont) of the Hayward Fault. The estimated Mw for the earthquake is 7.0. In 1861, an earthquake of unknown magnitude (probably a Mw of about 6.5) was reported on the Calaveras Fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill earthquake (Mw = 6.2).

Since 1800, four major earthquakes have been recorded on the San Andreas Fault. In 1836 an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale (Figure 4) occurred east of Monterey Bay on the San Andreas Fault (Toppozada and Borchardt 1998). The estimated Moment magnitude, Mw, for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to a Mw of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas Fault from Shelter Cove to San Juan Bautista approximately 470 kilometers in length. It had a maximum intensity of XI (MM), a Mw of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The most recent earthquake to affect the Bay Area was the Loma Prieta Earthquake of 17 October 1989, in the Santa Cruz Mountains with a Mw of 6.9, approximately 180 km from the site.

The 2008 WGCEP at the U.S. Geologic Survey (USGS) predicted a 63 percent chance of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Area in 30 years. More specific estimates of the probabilities for different faults in the Bay Area are presented in Table 2.

TABLE 2
WGCEP (2008) Estimates of 30-Year Probability of a Magnitude 6.7 or Greater Earthquake

Fault	Probability (percent)
Hayward-Rodgers Creek	31
N. San Andreas	21
Concord-Green Valley	3

7.0 DISCUSSION AND CONCLUSIONS

On the basis of our knowledge of the subsurface conditions at the project site, we conclude the project may be constructed as planned from a geotechnical engineering standpoint, provided the recommendations provided herein are incorporated into the foundation design, project plans and construction.

7.1 Seismic Hazards

Historically, ground surface ruptures closely follow traces of geologically young faults. The site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act and no known active or potentially active faults exist on the site. In a seismically active area, the remote possibility exists for future faulting in areas where no faults previously existed; however, we conclude the risk of surface faulting and consequent secondary ground failure at this site is low.

Considering the proximity of the site to the major active faults in the San Francisco Bay Area, we conclude there is a high potential for the site to experience moderate to very strong ground shaking during a major earthquake. The intensity of earthquake ground motion at the site will depend on the characteristics of the generating fault, the distance to the earthquake fault, and the magnitude and duration of the earthquake, and subsurface conditions.

Strong ground shaking during an earthquake can result in ground failure such as that associated with soil liquefaction[3], lateral spreading[4], and cyclic densification[5]. We used the results of our investigation to evaluate the potential of these phenomena occurring at the project site.

7.1.1 Liquefaction

When a saturated, cohesionless soil liquefies during a major earthquake, it experiences a temporary loss of shear strength due to a transient rise in excess pore water pressure generated by strong ground motions. Flow failure, lateral spreading, differential settlement, loss of bearing strength, ground fissures, and sand boils are evidence of excess pore pressure generation and liquefaction. We used the results from our borings to evaluate the potential for liquefaction and subsequent settlement using the methodology outlined in the Proceedings of the NCEER Workshop on the Evaluation of Liquefaction of Soils (Youd et al. 2001).

In our liquefaction analyses, a peak ground acceleration (PGA) of 0.34 times gravity was used. This PGA was calculated using the procedures specified in Section 1613 of the 2010 CBC for the Design Earthquake. An earthquake magnitude of 7.1 was also assumed in our analyses. Lastly, we assumed the groundwater surface was slightly higher than the highest water level observed in either the borings or piezometers.

The vast majority of the soil encountered beneath the groundwater table consists of either stiff to hard clay and medium dense clayey and gravelly sand, which we judge is sufficiently dense or cohesive to resist liquefaction. However, in three borings we encountered layers of soil that were not sufficiently strong or cohesive, and are therefore susceptible to liquefaction and significant strength loss.

In boring TR-2 we encountered a 3½-foot-thick layer of potentially liquefiable, loose, saturated clayey sand at a depth of 8 feet beneath the existing ground surface, corresponding to elevation 353 feet. The

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[3] Liquefaction is a transformation of soil from a solid to a liquefied state during which saturated soil temporarily loses strength resulting from the buildup of excess pore water pressure, especially during earthquake-induced cyclic loading. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits.

[4] Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported downslope or in the direction of a free face by earthquake and gravitational forces.

[5] Cyclic Densification (also referred to as Differential Compaction) is a phenomenon in which non-saturated, cohesionless soil is densified by earthquake vibrations, causing ground surface settlement.

strain potential of an identified potentially liquefiable layers can be estimated using the method developed by Tokimatsu and Seed (1984), which relates $(N1)_{60,CS}$ values to strain potential. Based on the results of this analysis, we estimate the ground surface at the location of boring TR-2 may settle up to ¾ inches during and immediately following a major earthquake from liquefaction induced settlement. TR-2 is located at the western edge of the Hotel/Resort complex. Foundation considerations for this portion of the complex will take this layer into account (i.e. this soil will not be relied upon to support the building or floor slab).

In borings TR-11 and TR-12, we encountered 6 to 7½ feet of very loose to loose clean sand fill beneath the groundwater table. These sands are also susceptible to liquefaction and strength loss during a major earthquake. Using the Tokimatsu and Seed method, we estimate the ground surface may settle on the order of 2½ to 3½ inches during and immediately following an earthquake in the vicinity of this fill material.

The settlements due to liquefaction will likely be random and erratic and will cause an equal amount of differential settlement over short distances.

7.1.2 Lateral Spreading

Lateral spreading occurs as surficial soil displaces along a uniform shear zone that has formed within an underlying continuous liquefied layer. The surficial blocks are transported downslope or in the direction of a free face, such as a channel, by earthquake and gravitational forces. On the basis of the results of our investigation, we only encountered one potentially continuous liquefiable layer, in TR-11 and TR-12. However, we believe this layer consists of trench backfill in and near the public easement, and is therefore limited laterally by the confines of the trench. With the exception of this backfill zone we conclude the potential for lateral spreading beneath the site is low.

7.1.3 Cyclic Densification

Cyclic Densification can occur in non-saturated sand (sand above the groundwater table) caused by earthquake vibrations, resulting in settlement of the ground surface. Many of the borings encountered layers of medium dense sand above the groundwater table in the alluvium. Using the Tokimatsu and Seed (1984) method for evaluating settlement of dry sands, we estimate that portions of the site may settle up to ¼ inch. These settlements will likely be random and erratic.

7.2 Expansive Soils

Atterberg limits tests were performed several soil samples where the near-surface soil was clayey. The results of our testing indicate that for the majority of the site, the near-surface clayey soils have a low to moderate expansion potential, with plasticity indices ranging from 13 to 20. In boring TR-12 however, we encountered near-surface clay in the fill with a high expansion potential, with a plastic index equal to 35. As discussed in below in Section 7.3, the fill encountered in this boring will be removed and replaced with engineered fill if it extends beneath any proposed buildings. Therefore, the recommendations contained herein assume that the foundation soil will have a low to moderate expansion potential. During construction we should be on-site during the excavation of footings to check that highly expansive clays are not encountered at the foundation depths.

Expansive soils are those that shrink or swell significantly with changes in moisture content. The clay content and porosity of the soil also influence the change in volume. The shrinking and swelling caused by expansive clay-rich soil often results in damage to overlying structures. Therefore, foundations and slabs should be designed and constructed to resist the effects of the expansive soil. These effects can be mitigated by moisture conditioning the expansive soil and/or providing select, non-expansive fill below interior and exterior slabs and roadways and supporting foundations below the zone of severe moisture change. Detailed recommendations for mitigating the effects of the moderately expansive near-surface soils are described in Sections 8.2.4 and 8.2.5.

7.3 Foundations and Settlement

Based on our current understanding of the project, the Winery and Hotel/Resort complex will be situated in the vicinity of the rock knoll. At the design lower level (about Elevation 355 to 360 feet), the majority of this building will be founded directly in bedrock. In the western and southwestern portion of the complex, the building will be constructed above new engineered fill placed to backfill the existing pond. If shallow foundations are founded on this fill, excessive settlements could occur. Therefore, to limit total and differential settlements of this structure, the entire footprint should be supported on bedrock. Where bedrock is exposed at the design foundation level or the depth to bedrock is less than about five feet beneath the finished floor elevations (typically an economical depth to excavate), the building can be supported on spread footings bearing on bedrock.

Within the Hotel/Resort complex where the depth to bedrock is more than about five feet below the floor elevation, drilled piers or auger cast piles gaining support in bedrock may be a more suitable and economical foundation alternative than footings. At the western and southern portions of the complex) the bedrock will likely be more than five feet below the basement level. The elevation where we encountered bedrock in each of our borings is presented on Figure 2.

The proposed cabins and single family residences (SFRs), can generally be supported in the alluvium or engineered fill using conventional shallow foundations and/or PT slabs. A conventional shallow foundation system generally consists of a continuous perimeter footing and isolated interior footings.

The fill material encountered in Borings TR-11 and TR-12 is not suitable for the support of the proposed buildings, excessive and erratic settlements would occur during and following an earthquake. The fill is on the order of 13 to 14½ feet deep where encountered and the lateral extent of this fill is not currently known. As discussed above, it is postulated that the fill is localized to the limits of the trench backfill for utilities that have been installed in easement. Where this weak soil could be within the vicinity of new improvements (such as SFRs or cabins), its extent should be identified (see section 8.12). If the fill extends beneath the proposed buildings, it should be removed and replaced with engineered fill. Where the poor soil is removed and replaced with engineered fill, the liquefaction potential will be mitigated and the new fill will be suitable for bearing. The foundation of the proposed buildings can then be supported on the engineered fill. Additional details regarding these recommendations are presented in Section 8.1.2.

In addition, the weak material encountered within the leach field should be removed within the vicinity of newly proposed buildings or improvements and replaced with engineered fill. The proposed building cabins/SFRs may be supported on the engineered fill.

7.3.1 Foundation Settlement

The amount of anticipated foundation settlement will depend on the foundation type and construction practices. Shallow foundations bearing on competent rock should experience less than ¾ inch of total settlement. Differential settlement between columns should be less than ½ inch. Properly constructed drilled piers or auger cast piles gaining support in competent rock should have a total settlement less than 1 inch, with less than ½ inch of differential settlements between columns. Most of these settlements are expected to occur during construction.

For properly constructed shallow foundations bearing on alluvium or engineered fill less than 5 feet thick, we estimate total static settlement will be on the order of 1 inch, with differential settlements less than ½ inch between columns. We anticipate about half of this settlement will occur during construction. In addition, we anticipate an additional ¼ inch of erratic settlement may occur across the eastern portion of the site (underlain by alluvium) from cyclic densification, as discussed in Section 7.1.3.

If the new foundations are supported on fills greater than 5 feet thick (i.e. where significant over excavation has been required to remove unsuitable soils) the total and differential settlements will be higher. If a cabin or SFR is supported on an average of 10 feet of new fill, we anticipate the building will have a total settlement of about 1½ to 2 inches. As discussed in Section 8.1.2 we recommend a maximum differential fill thickness of 6 feet under any building. For this case, the anticipated differential settlement could be about 1 inch across the building. If this differential settlement is unacceptable, the differential fill thickness should be reduced.

7.4 Floor Slabs

Floor slabs for the cabins and SFRs may be supported on grade. In addition, where bedrock is shallow and the Hotel/Resort complex is supported on shallow footings, the floor slabs may be supported on grade.

Where the Hotel/Resort complex is supported on drilled piers or piles, the new fill and/or thick alluvium is expected to settle somewhat over time. Therefore, floor slabs within the pier/pile supported area of the Hotel/Resort complex should be designed to span between pier caps and/or grade beams, and the fill should not be relied upon for support.

7.5 Excavation and Shoring

For the construction of the Hotel/Resort Complex, we anticipate an excavation on the order of 10 to 25 feet will be required prior to construction of the planned building. The majority of the excavation will be to remove a significant portion of the knoll. At the edges of the proposed Hotel/Resort complex, we anticipate the cuts will be less than 15 feet. The majority of this excavation will be in Tuff bedrock. We understand a study was performed by Norcal Geophysical to evaluate the rippability of the Tuff bedrock on the knoll and the results indicate that the material is rippable.

Where there is sufficient space, the required excavation for the building may be sloped. If there is insufficient space to slope the excavation, shoring will be needed. There are several key considerations in selecting a suitable shoring system. Those we consider to be primary concerns are:

- protection of surrounding improvements
- proper construction of the shoring system to minimize the potential for ground movement
- cost.

We judge the most cost effective and appropriate temporary shoring system for this project is a soil/rock nail wall. Soil/Rock-nail shoring systems consist of reinforcing bars, which are grouted in predrilled holes through the face of the excavation, and shotcrete facing.

In addition, a typical soldier-pile-and-lagging system would also be appropriate for the project. For this type of system, soldier piles are placed in predrilled holes which will be backfilled with concrete. Wood lagging will be placed between the soldier piles as the excavation proceeds. Drilling of the boreholes for the soldier piles may require casing to prevent caving of the soil above the bedrock). If the shoring retains more than about 15 feet of soil and bedrock, the shoring system may require additional lateral restraint. Either grouted tiebacks or internal bracing is acceptable to provide this additional restraint.

Minor deflections of the ground surface and adjacent structures should be expected with shoring systems. The amount of movement and distress to adjacent improvements will depend on the workmanship of the contractor. During excavation, the shoring system is expected to yield and deform, which could cause surrounding improvements to settle and move. The magnitude of shoring movements and resulting settlements are difficult to estimate because they depend on many factors, including the method and the specialty shoring contractor's skill in the installation. We estimate a properly installed system will limit settlements to adjacent improvements to less than one inch. The settlement should decrease linearly with distance from the excavation and should be relatively insignificant at a distance twice the excavation depth.

The selection, design, construction, and performance of the shoring system should be the responsibility of the contractor. A structural/civil engineer knowledgeable in this type of construction should be retained to design the shoring. We should review the final shoring plans to check that they are consistent with the recommendations presented in this report.

During excavation, the groundwater level in the knoll area will likely need to be lowered. During construction the groundwater should be lowered to a depth of at least three feet below the bottom of the planned excavation and maintained at that level throughout construction. Variables which significantly influence the performance of the dewatering system and the quantity of water produced include the number of wells, the depth and positioning of the wells, the interval over which each well is screened, and the rate at which each well is pumped. Different combinations of these variables can be used to dewater the site. The selection and design of the dewatering system should be the responsibility of the contractor. However, we should check the design of the proposed dewatering system prior to installation.

7.6 Underpinning

Where the proposed new Hotel/Resort complex abuts the existing Winery building, the new foundation elements may extend below the existing building foundations. If this condition exists, the existing building should be underpinned to temporarily support the structure during construction of the new complex.

Underpinning can consist of hand-excavated piers that extend at least three feet below the planned bottom of excavation. Hand-dug underpinning piers are usually 30 by 48 inches in plan and are shored using pressure-treated lagging. The open piers are reinforced with steel and are filled with concrete, the top of the pier is jacked and dry-packed to fit tightly with the base of the underpinned foundation. The piers should act in end bearing in the bearing strata below the depth of the proposed excavation and should be designed to resist at-rest lateral pressures imposed by the soil beneath the building (see Section 8.3).

7.7 Surface Water and Groundwater Seepage

Surface runoff in the vicinity of the proposed project should be collected and directed to discharge at a location(s) where it should not impact the project development. We understand the several "drain tile" subdrains have been installed across the site to help capture excess surface and subsurface water. However, we also understand that minor flooding was observed in the southern corner of the site during the winter of 2010-2011.

In the piezometers installed in Borings B-2 and B-3, groundwater was observed at a depth of 7.3 and 6.9 feet, corresponding to about Elevations 368 and 365 feet. The Hotel/Resort complex will be cut into bedrock, with a lower level finished floor elevation at about 360 feet; below the observed groundwater elevations. However, a significant portion of the knoll will be removed and covered with the proposed Hotel/Resort complex as part of this project. In addition, the ground surface elevations surrounding the knoll are generally at or lower than Elevation 360 feet. With the removal of much of the knoll and ground around the knoll being at a similar elevation to the building finished floor, it is our opinion that the groundwater level in the knoll area will be lowered through the course of construction and will remain lower than the building finished floor. However, localized seepage through the bedrock will still likely occur and should be controlled through the use of a subsurface drainage system consisting of wall backdrains and an underslab subdrain.

All below-grade walls should be properly backdrained to prevent the buildup of hydrostatic pressures. Because the groundwater within the vicinity of the knoll travels in fractures within the rock, the wall drains may not intercept all of the groundwater. To prevent a buildup of hydrostatic pressures on the floor slabs from groundwater in the bedrock (or in bedrock fractures), an underslab drainage system should also be installed. This system should consist of a continuous below-slab gravel blanket with a network of perforated pipes embedded in the gravel. The perforated pipes should collect the water captured by the gravel blanket and carry it to a solid pipe that should conduct the water to a suitable outlet. Special care should be taken during design to eliminate the possibility of conducting water from the wall backdrain system into the underslab drainage system and the outlet pipes for these two systems should be kept separate.

To prevent water and moisture migration into the below-grade portions of the building, the below-grade walls and floors should be waterproofed and waterstops should be provided across all below grade construction joints. This is particularly important in areas where there will be finished space, such as residential, retail, or commercial space.

7.8 New Irrigation Pond

The new irrigation pond will be divided into two levels: the northwestern portion will have a high water elevation of 359 feet and the southeast portion will step down to Elevation 356 feet. The finished floor in the Hotel/Resort complex is at Elevation 360 feet and will be underlain by a underslab subdrain system. To reduce the potential for pond water to enter the subdrain system and to limit overall infiltration and associated water loss, we conclude that the irrigation ponds should be lined with a permanent impermeable membrane.

7.9 Soil Corrosivity

Two soil corrosivity tests were performed on samples of near-surface soil at the site. Testing was performed on samples from TR-3 (on the west side of the site) and from TR-11 (on the east side of the site). The results of these tests are presented in Appendix C. Design of the proposed development should account for the soil corrosivity.

8.0 RECOMMENDATIONS

Our recommendations regarding site preparation, foundations, site drainage, seismic design, and other geotechnical aspects of this project are presented in the following sections.

8.1 Site Preparation and Fill Placement

Areas of the site that will receive improvements (including fill, building pads, pavements, and exterior concrete slabs) should be cleared and grubbed of all vegetation, and the site should be stripped of organic topsoil containing over three percent organic matter. In the vineyard areas, this may include the removal of up to two feet of organic-rich soil. Stripped materials should be removed from the site or stockpiled for later use in landscaped areas, if approved by the architect.

Demolition of the existing site improvements will include the removal of existing utility lines and removal of foundations beneath the buildings that are being removed. All utilities and existing foundation elements should be removed in the vicinity of the proposed improvements. Voids resulting from demolition activities should be properly backfilled with engineered fill or lean concrete.

Where utilities that are removed extend off site, they should be capped or plugged with cement grout at the property line. It may be feasible to abandoned utilities in-place by filling them with grout, provided they will not impact future utilities or building foundations. The utility lines, if encountered, should be addressed on a case-by-case basis.

As part of the future re-shaping of the irrigation pond, significant fill will be placed in the eastern portion of the existing pond. Prior to placing fill, all of the weak slough, sediment, organic material, and clayey pond liner material should be removed to expose firm native soils. In addition, as the fill is being placed the side-slope of the existing pond should be benched into native soils or bedrock.

8.1.1 Cut Slopes

Current design plans include a significant excavation in the knoll area to allow construction of the new Hotel/Resort complex. The safety of workers and equipment in or near excavations is the responsibility of the contractor. The contractor should be familiar with the most recent OSHA Trench and Excavation Safety standards. We should review plans for temporary sloping prior to construction. During construction, we should observe cut slopes to verify the inclinations are appropriate for the conditions encountered.

Temporary unretained cut slopes more than 5 feet high in alluvium or other on-site soils should be graded no steeper than 1.5:1 (horizontal:vertical).

Temporary slopes in competent bedrock may be made vertical; however, the height of any vertical segment should not exceed five feet unless shoring is used. Temporary cut slopes in rock higher than five feet may be graded as steep as 1:1, depending on the rock fracturing, hardness, and weathering. If poor rock quality or adverse bedding is present, these slopes should be flattened.

Based on our understanding of the site development, we do not anticipate the site will have significant permanent cut slopes. If present, permanent cut slopes in alluvium and rock should be no steeper than 2:1 and 1.5:1, respectively. All permanent cut slopes should be observed by our engineering geologist at the time of grading to assess the applicability of our recommendations and make supplemental recommendations, if necessary.

It is the responsibility of the contractor to maintain safe and stable slopes during construction. Heavy construction equipment, building materials, excavated soil, and vehicle traffic should not be allowed within seven feet of the top of excavations. During wet weather, runoff should be prevented from running across slopes and from entering excavations.

8.1.2 Site Fill

We anticipate fills less than five feet thick will be required across the site to construct building pads and landscape areas at their anticipated elevations. Thicker fills will be required where the southeastern portion of the irrigation pond is being filled.

After stripping the existing soil subgrade, areas to receive fill should be scarified to a depth of eight inches, moisture-conditioned and recompacted to at least 90 percent relative compaction.[6] If soft or loose soil is encountered after stripping, the unsuitable material should be excavated and replaced with suitable fill material. Slopes steeper than 5:1 that will receive fill should be benched as the fill operation proceeds upslope. The benches should have a minimum width of 6 feet, maximum height of 5 feet, and an overall inclination no steeper than 1:1.

All materials to be used as general engineered fill, including onsite soil, should be free of organic material, contain no rocks or lumps larger than 4 inches in greatest dimension, be non-corrosive, have low to moderate expansive potential (Plasticity Index less than 20), and be approved by the geotechnical engineer. Select fill should also meet these requirements as well as have a low expansion potential. Low expansion potential is defined by a liquid limit of less than 25 and a plasticity index lower than 12, as well as contain at least 10 percent fines. In isolated areas in the fill, we encountered plastic material with a Plasticity Index of 35. This material should not be used as fill unless it is placed in landscape areas at least 10 feet from any proposed improvements.

The onsite soil likely meets the requirements for general fill. The rock material generated by onsite cuts in bedrock will likely meet the requirements for general fill; however, screening and crushing of the material will be required. During construction we will perform additional laboratory testing as needed to check that the proposed fill material meets the project requirements.

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[6] Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557-00 laboratory compaction procedure.

Fill should be placed in horizontal lifts not exceeding eight inches in uncompacted thickness, moisture-conditioned to above the optimum moisture content and compacted to at least 90 percent relative compaction. The soil subgrade beneath slab-on-grade floors and/or PT-slabs should be compacted to at least 90 percent relative compaction proof rolled to provide a firm, unyielding surface prior to placement of the vapor barrier or underslab drainage system.

The upper six inches of the pavement subgrade should be scarified, moisture-conditioned to above optimum moisture content and compacted to at least 95 percent relative compaction. The subgrade should be proofrolled to provide a firm, non-yielding surface. Aggregate base (AB) placed beneath roadways and/or exterior slabs-on-grade (such as patios and sidewalks) should be moisture-conditioned to near the optimum moisture content and compacted to at least 95 percent relative compaction.

Following fill placement, the soil subgrade should be kept moist until it is covered by fill, a building, or the pavement section.

If loose, saturated sand (similar to the material encountered in TR-11 and TR-12) is present beneath any proposed buildings, it should be removed and replaced with engineered fill. The fill should generally be placed in accordance with the recommendations presented above for general fill, except that fill thicker than five feet thick should be compacted to 95 percent relative compaction. In addition, the upper three feet of the new fill should consist of non-expansive 'select' fill.

Significant variations in fill thickness can result in significant differential settlement of the structure. Therefore, the differential fill thickness beneath any structure should not be greater than six feet. If only a portion of a footprint of a cabin or SFR building is underlain by more than six feet of unsuitable fill, the remaining portion of the building pad, after the unsuitable soils are removed, should also be partially overexcavated to reduce the differential thickness of engineered fill.

8.1.3 Utility Trench Backfill

Utility trenches should be excavated a minimum of four inches below the bottom of pipes or conduits and have clearances of at least six inches on both sides. Where necessary, trench excavations should be shored and braced to prevent cave-ins and/or in accordance with safety regulations. To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of sand or fine gravel. After pipes and conduits are tested, inspected (if required), and approved, they should be covered to a depth of six inches with moisture conditioned sand, which should then be mechanically tamped.

Backfill for utility trenches and other excavations is also considered fill, and should be compacted according to the recommendations previously presented. If imported clean sand or gravel (defined as soil with less than 10 percent fines) is used as backfill, it should be compacted to at least 95 percent relative compaction. The upper three feet of backfill placed in utility trenches within the building pads should consist of select fill. Jetting of trench backfill should not be permitted. Special care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to the pavement section.

In areas where granular deposits are encountered, some sloughing of soil into trench excavations may occur. If sloughing or caving should occur, trenches will require temporary shoring or sloping of the sidewalls. All trenches should conform to the current OSHA requirements for work safety. Temporary dewatering may be required during construction if excavations extend below the groundwater table.

Where utility trenches extend from the exterior to the interior limits of a building, native clayey soil or lean concrete should be used as backfill material for a distance of two feet laterally on each side of the exterior building line to reduce the potential for the trench to act as a conduit for external water to enter the building footprint. Utility trenches in landscape areas should also be capped with a minimum of 12 inches of compacted on-site clayey soils.

8.1.4 Grading Construction Considerations

During construction, we should check that the onsite and/or any proposed import material is suitable for use as fill. Samples of all imported fill should be submitted to the geotechnical engineer for testing at least 72 hours before delivery to the site. A flowable cement grout should be used to backfill areas not accessible to compaction equipment.

If grading is performed during or following the wet winter months, moisture conditioning may consist of lowering the moisture content to a level that will promote proper compaction. One method to lower the moisture content consists of mixing and turning (aerating) the soil to naturally dry the soil and lower the moisture content to an acceptable level. Aeration typically requires at least a week of warm, dry weather to effectively dry the material. Material to be dried by aeration should be scarified to a depth of at least twelve inches; the scarified material should be turned at least twice a day promote uniform drying. Once the moisture content of the aerated soil has been reduced to acceptable levels, the soil should be compacted in accordance with our previous recommendations. Aeration typically is the least costly method used to stabilize the subgrade soil; however, it generally requires the most time to complete.

In some cases, aeration may not be effective in lowering the moisture content of the soil and/or achieving the desired degree of compaction within an acceptable period of time. Various methods are available to effectively treat wet soil. These include mixing the soil with lime, cement, or kiln dust. Detailed recommendations can be provided during construction as necessary.

If unstable, soft soil is encountered in pavement or building pad areas during site grading, it should be removed and replaced with engineered fill as described in Section 8.1.2. If the areas are isolated, it may be more cost-effective to fill the void created by overexcavation with lean concrete. Recommendations for stabilization of deflecting soil will be provided as necessary on a case-by-case basis.

8.2 Foundations

The proposed Hotel/Resort complex buildings should be supported on foundations gaining support in bedrock. Footings should be used where bedrock is shallow, and drilled piers or auger cast piles are recommended where the depth to bedrock is greater than about five feet beneath the finished floor elevations.

The proposed cabins and SFRs may be supported on conventional shallow footings or P-T slabs bearing in alluvium and/or engineered fill. Recommendations for each foundation type are presented in the remaining subsections.

8.2.1 Shallow Footings in Bedrock

The proposed Hotel/Resort complex may be supported on continuous and/or individual footings bearing in bedrock. Continuous footings should be at least 18 inches wide and isolated footings should have a minimum width of at least 24 inches. The footings should be embedded at least 18 inches below lowest adjacent grade and at least 12 inches into bedrock. Footings adjacent to utility trenches or other footings should bear below an imaginary 30 degree line projected upward form the bottom edge of the utility trench or adjacent footing.

For footings bearing in rock, we recommend using an allowable bearing pressure of 6,000 pounds per square foot (psf) for dead plus live loads (DL+LL), with a one-third increase for total loads, including wind or seismic.

Because the quality and consistency of the Tuff bedrock varies, highly weathered or soft rock may be encountered at the footing subgrade elevation. This weak rock may locally need to be overexcavated and replaced with lean concrete.

8.2.2 Drilled Piers

Drilled cast-in-place concrete piers extending into bedrock should be used to support the Hotel/Resort complex where the depth to bedrock makes it impractical to use shallow footings. Piers should derive vertical capacities through skin friction in bedrock. Skin friction in the fill (if present) and alluvium, as well as the contribution of end bearing, should be ignored for drilled piers. As a minimum, piers should extend at least five feet into bedrock. The elevation where we encountered bedrock is presented on the Site Plan, Figure 2, at each boring location.

To compute the axial capacity of drilled piers, we recommend using an allowable skin friction of 1,250 pounds per square foot (psf) for dead and live loads. For temporary, compressive, total loads, including wind and/or seismic load, the skin friction value can be increased by one third. For temporary uplift loads, we recommend an allowable skin friction of 1,250 psf in the bedrock and 140 psf in overlying soils.

The above capacities are based on earth material capacity only, individual piers should be structurally designed and reinforced to support horizontal and vertical loads. Drilled piers should be at least 18 inches in diameter and should have a minimum bedrock embedment of five feet. Piers installed in a group should be spaced at least three diameters on center.

Drilled piers should be installed by a qualified contractor with demonstrated experience in this type of foundation and subsurface conditions. Groundwater and potentially caving sand will likely be encountered during drilling. Therefore, casing and/or drilling fluid will likely be required to prevent caving. Concrete placement in drilled piers should start upon completion of the drilling and clean out. As a minimum, the piers should be poured the same day they are drilled. The bottoms of the pier excavations should be free of all loose cuttings and soil fall-in prior to the installation of the reinforcing steel and the placement of the concrete. Any accumulated water in the pier excavations should be removed prior to the placement of the reinforcing steel and concrete or the concrete may be placed underwater by the tremie method. Concrete should be placed from the bottom up in a single operation. The tremie pipe should be maintained at least five feet below the upper surface of the concrete during casting of the piers. As the concrete is placed, casing used to stabilize the hole can be withdrawn. The bottom of the casing should be maintained at least three feet below the surface of the concrete.

8.2.3 Auger Cast Piles

Auger cast piles may be used as an alternative to drilled piers to support the Hotel/Resort complex where the depth to bedrock makes it impractical to use shallow footings. An auger cast pile is a pile that is drilled with a hollow-stem, continuous-flight auger. When the auger reaches the required depth concrete or gout is injected as the auger is slowly withdrawn. While the grout is still fluid, a steel reinforcing cage or steel beam is inserted into the shaft. Auger cast piles can range in diameter; an 18-inch-diameter pile is typical.

Auger cast piles should derive their axial capacity through friction and end-bearing in the bedrock below the fill and alluvium. They are typically designed and installed by design-build contractors. As such, the final design capacity of the piles should be provided by the contractor. Based on our experience with similar sites, we estimate with 15 feet embedment in bedrock these piles could have an allowable capacity on the order of 300 kips. The actual design capacities, developed by the design-build contractor, should be verified by a load test program.

If auger cast piles are used, we recommend an indicator pile program be performed to provide data for production pile installation. The foundation contractor should evaluate the potential for variations throughout the site and, given their ability to accommodate these variations, determine the appropriate number of indicator piles to install to evaluate these variations; we recommend a minimum of 6 indicator piles be installed. Indicator piles may be installed at column locations and can be used for support of the building. They should be installed with the same equipment that will be used to install the production piles.

The load test program should consist of testing at least two piles in compression and one pile in tension. In compression, the piles should be loaded to at least 200 percent of the design compression load (dead plus live conditions) plus the contribution in friction from the fill. In tension, the pile should be loaded to at least 150 percent of the design seismic uplift load. The test piles should be installed and tested from building pad subgrade elevation; if they will be installed and tested from a higher elevation, additional measures will need to be taken to eliminate the extra friction and the influence of the additional overburden.

Once the indicator piles have been installed we will select the appropriate piles to test and provide the recommended test load for each pile. The piles should be tested in accordance with ASTM Standards D1143-07 and D3689-07 for static load tests in compression and tension, respectively. During the test we should observe the behavior of the pile under the test loads and confirm that the piles are performing as planned.

8.2.4 Footings in Alluvium and/or Engineered Fill

The cabins and single family residences may be supported on a combination of shallow isolated interior footings and continuous perimeter footings. Continuous footings should be at least 18 inches wide, and isolated footings should have a minimum width of at least 24 inches. To limit the potential detrimental effects of the moderately expansive near-surface soils at the site, the continuous perimeter footings should bear at least 24 inches below the lowest adjacent grade. Interior footings should have an embedment of at least 18 inches below the lowest adjacent grade. Also, footings adjacent to utility trenches or other footings should bear below an imaginary 30-degree line projected upward from the bottom edge of the utility trench or adjacent footing.

For footings bearing in native alluvium or newly placed engineered fill, we recommend using an allowable bearing pressure of 1,600 pounds per square foot (psf) for dead plus live loads (DL+LL), with a one-third increase for total loads, including wind or seismic.

In Section 8.1.2 we recommend a maximum differential fill thickness of 6 feet under any building.
For this case, the anticipated differential settlement could be about one inch across the building. If this differential settlement is unacceptable, the differential fill thickness should be reduced.

Footing excavations should be free of standing water, debris, and disturbed materials prior to placing concrete. The bottoms and sides of excavations should be wetted following excavation and maintained in a moist condition until concrete is placed. We should check foundation excavations prior to placement of reinforcing steel.

8.2.5 P-T Slabs in Alluvium and/or Engineered Fill

As an alternate to conventional footings, the cabins and SFRs may be supported on P-T slabs bearing on native alluvium or engineered fill prepared in accordance with our previous recommendations. For design of P-T slabs, we recommend using the parameters presented in Table 3.

The soil differential movement may be controlled by the amount of differential settlement expected rather than the potential for seasonal differential movement. Therefore, we recommend the slabs be checked for the edge-lift condition using special "no-swell" design equations specified by the Third Edition (2008) of the "Design of Post-Tensioned Slabs-on-Ground" publication of the Post-Tensioning Institute. For this procedure, we recommend the total settlement value used in the equations be equal to 1 inch and the soil differential movement value should be 1/2 inch in 20 feet.

TABLE 3
P-T Slab Design Parameters (Third Edition)

Parameter	Value
Thornwaite Moisture Index	20
Edge moisture variation distance edge lift center lift	 4.0 feet 7.8 feet
Depth to constant soil suction	5 feet
Constant soil suction	3.8 pF
Soil differential movement edge lift center lift	 1.8 inches 1.2 inches

The P-T slabs should be at least 8 inches thick, with a thickened edge that should be embedded at least 12 inches below the lowest adjacent outside grade or six inches below the water vapor retarder (described below in Section 8.5), whichever is lower. The maximum bearing pressure beneath the P-T slabs should not exceed 1,600 psf for dead plus live loads, with a one third increase for total load conditions. These parameters are based on the subgrade soil having a Plasticity Index (PI) of less than 20. If index testing during construction indicates a PI greater than 20 at subgrade then the upper three feet of soil should be removed and replaced with engineered fill with a PI less than 20.

We should check the P-T slab subgrade prior to placing reinforcing steel or a moisture barrier. Because of the limited edge embedment, we should check the condition of the subgrade around the building prior to placing fill adjacent to the P-T slabs to confirm there are no voids beneath the edges of the P-T slabs. The excavations should be free of standing water, debris, and disturbed materials prior to placing concrete. The bottom of the excavation and P-T slab subgrade should be kept moistened before concrete or vapor barrier is placed.

8.2.6 Lateral Resistance

For shallow foundations, lateral loads can be resisted by a combination of passive resistance on the vertical faces of the foundation elements and friction along the bases of the footings. Passive resistance may be estimated using an equivalent fluid weight (triangular) distribution of 250 pounds per cubic foot (pcf) in compacted fill or native alluvium. In bedrock passive resistance may be estimated using a uniform pressure equal to 4,000 pounds per square foot (psf). For passive resistance the upper one foot of soil should be ignored unless it is confined by slabs or pavement. If the ground surface slopes away from foundations, passive resistance will be developed below an imaginary horizontal line that intersects the ground surface a distance of 10 feet from the face of the foundation system. If this distance cannot be obtained with shallow footings, deepened footings, drilled piers or auger cast piles can be used.

Frictional resistance may be estimated using a base friction coefficient of 0.25 for native alluvium and engineered fill. In bedrock, the allowable friction coefficient is 0.32. These values should be used where the foundation is in direct contact with the subgrade soil. Where a P-T slab is underlain by a vapor retarder, a base friction coefficient of 0.2 should be used.

The passive resistance and frictional resistance values include a factor of safety of about 1.5 and may be used in combination without reduction

Where the proposed buildings will be supported on piers or auger cast piles, lateral loads imposed on the building foundations can be resisted by a the lateral capacity of the pile and passive resistance acting against the vertical faces of the foundations and grade beams.

Lateral resistance of piers and auger cast piles will depend on their diameter, head condition (restrained or unrestrained), allowable deflection of the pier/pile top, and the bending moment resistance of the piers/piles, as well as the strength of the surrounding soil. For design of drilled piers and auger cast piles, we have assumed that piers/piles will have a diameter of either 18 or 24 inches diameter, be installed in a level ground condition, and be designed for a deflection of ½ inch at the top of the pier/pile. The results of our analyses are presented in Table 4.

TABLE 4
Results of Lateral Load Analyses for ½-Inch Deflection at Pier/Pile Top

Pier/Pile Diameter (inches)	Pier/Pile Top Condition	Computed Lateral Load at 1/2-inch Deflection (kips)	Computed Maximum Bending Moment (kip-feet)	Depth to Maximum Bending Moment (feet)
18	Restrained	22	113	0
18	Unrestrained	9	40	12
24	Restrained	49	330	0
24	Unrestrained	27	118	14

1. Analyses assume a minimum pier/pile length of 15 feet beneath pier cap.

If the ground surface will slope away from piers or piles (i.e. near the new irrigation pond), the passive resistance in the upper portion of the piers/piles will be reduced, and these piers/piles will have a lower lateral capacity. Once the final pier/pile layout and ground surface configuration has been established, we can perform additional analyses for these piers/piles upon request.

The lateral resistances tabulated above in Table 4 are for single piers/piles. To account for group effects, the lateral load capacity of a single pier/pile should be multiplied by the appropriate reduction factors shown on Table 5. However, the maximum bending moment for a single pier/pile with an unfactored load should be used to check the design of individual piers/piles in a group. The reduction factors are based on a minimum center-to-center spacing of three pier diameters. Reduction for other pile group spacing can be provided once the number and arrangement of piles are known.

TABLE 5
Lateral Group Reduction Factors

Number of Piers/Piles within Pier Cap	Lateral Group Reduction Factor
2	0.9
3 to 5	0.8
≥6	0.7

8.3 Shoring and Underpinning

If there is insufficient space to slope cut the proposed excavation for the Hotel/Resort complex, either a soil/rock nail wall or a soldier-pile-and-lagging system can be used to retain the excavation.

8.3.1 Soil/Rock Nails

Several computer programs, such as SNAILZ (California Department of Transportation, 1999) and GoldNail (Golder Associates, 1996), are available for designing a soil/rock-nail wall. SNAILZ uses a force equilibrium method of analysis; the failure planes are assumed bi-linear if they pass through the toe of the wall and tri-linear if they pass below the toe of the wall. GoldNail uses a slope-stability model that satisfies overall limiting equilibrium of free bodies defined by circular slip surfaces. For input parameters, we recommend the values presented in Table 6.

TABLE 6
Recommended Input Parameters for Design of
A Soil-Nail Wall

Soil Type	Total Density (pcf)	Ultimate Soil-Nail Friction (psf)	Shear Strength c[1] (psf)	• [2] (deg)
Soil (alluvium and fill)	125	1,000	100	30
Rock	135	2,000	500	30

Notes:
1. Cohesion intercept, without a safety factor.
2. Angle of internal friction, without a safety factor.

The anticipated depth to bedrock across the site may be estimated using the elevations of the bedrock surface presented on Figure 2 and the results of the seismic velocity study performed by Norcal Geophysical, Inc.

The soil/rock-nail wall should be backdrained using prefabricated drainage panels (at least two feet wide) between the nails. Where construction equipment will be working or driving upslope of the walls, the design should include a vertical surcharge pressure of 250 psf acting a horizontal distance between 5 and 25 feet from the wall.

In accordance with the FHWA manual on soil/rock nail walls (2003), we recommend designing the soil/rock nail walls using the minimum safety factor listed in Table 7, below:

Table 7
Recommended Safety Factors for Design of Soil/Rock-Nail Walls

Failure Mode	Resisting Component	Minimum Factor of Safety for Temporary Structures
External Global Stability	Final Condition	1.35
	Interim Condition	1.25
Internal Stability	Grout-Soil Bond Strength	2.0
	Bar Tensile Strength	1.8
Shotcrete Facing	Punching Shear	1.35

Notes:
1. Interim condition corresponds to the case where temporary excavation lifts are unsupported for up to 24 hours before nails are installed.

If the soil/rock-nail walls are designed using a common safety factor against the external global stability, grout-soil bond strength, bar tensile strength, and facing punching shear failure modes, we recommend designing the soil-nail walls with a minimum safety factor of 1.5 for the final condition and 1.3 for the interim condition. These values are for a temporary soil/rock nail wall.

9.3.1.1 Soil/Rock Nail Testing

Test nails should be installed using the same equipment, method, and hole diameter as planned for the production nails. Verification tests should be performed prior to production nail installation to verify the pullout resistance (bond strength) value used in design. Two verification tests should be performed for each soil/rock type assumed in design. Proof tests are performed during construction to verify that the contractor's procedure remains the same or that the nails are not installed in a soil type not tested during the verification stage testing. At least five percent of the production nails should be proof tested.

Tests should be performed on production or sacrificial nails to a test load corresponding to the ultimate pullout resistance value used in the design. Test nails should have at least one foot of unbonded length and 10 feet of bond length. The nail bar grade and size should be designed such that the bar stress does not exceed 80 percent of its ultimate strength during testing.

In the verification and proof tests, the load is applied to the nails in four increments. The maximum test load should be held for a minimum of 10 minutes; the movements of the nails should be recorded at 0, 1, 2, 3, 4, 5, 6, and 10 minutes. If the difference in movement between the 1- and 10-minute reading is less than 0.04 inch, the test is discontinued. If the difference is more than 0.04 inch, the holding period is extended to 60 minutes, and the movements should be recorded at 15, 20, 25, 30, 45, and 60 minutes.

We should evaluate the test results and determine whether the test nail performance is acceptable. Generally, a test with a ten-minute hold is acceptable if the nail carries the maximum test load with less than 0.04 inch movement between one and 10 minutes. A test with a 60-minute hold is acceptable if the nail carries the maximum test load with less than 0.08 inch movement between six and 60 minutes.

8.3.2 Soldier-Pile-and-Lagging Shoring

We anticipate the sides of the excavation will be less than 15 feet tall and that a cantilever system may be used. Cantilever soldier-pile-and-lagging walls should be design to resist an active earth pressure corresponding to an equivalent fluid weigh of 38 pounds per cubic foot (pcf) in the alluvium and 32 pcf in the bedrock. This lateral force may be resisted by passive earth pressures against the embedded vertical faces of the piers. We recommend passive resistance be calculated using an equivalent fluid weight of 120 pcf in the native alluvium and a uniform pressure of 4,000 psf in the bedrock. These values assume that the toe of the soldier piles could extend below the groundwater table. The calculated passive pressure may be applied over three pier diameters.

If traffic occurs within 10 feet of the shoring depth, a uniform surcharge load of 100 psf should be added to the design. An increase in lateral design pressure for the shoring may be required where heavy construction equipment or stockpiled materials are within a distance equal to the shoring depth. Construction equipment should not be allowed within five feet from the edge of the excavation unless the shoring is specifically designed for the appropriate surcharge. The increase in pressure should be computed after the surcharge loads are known.

If the depth of the excavation is too great to be restrained using a cantilevered shoring system, the shoring can be tied back or internally braced. If a tied-back or braced system is used, we should be contacted to provide additional recommendations for these types of shoring.

The selection, design, construction, and performance of the shoring system should be the responsibility of the contractor. The shoring system should be designed by a licensed structural engineer experienced in the design of retaining systems, and installed by an experienced shoring specialty contractor. The shoring engineer should be responsible for the design of temporary shoring in accordance with applicable regulatory requirements. Control of ground movement will depend as much on the timeliness of installation of lateral restraint as on the design. We should review the shoring plans and a representative from our office should observe the installation of the shoring.

8.3.3 Underpinning Design

If the construction of the new Hotel/Resort complex requires an excavation below the depth of the existing foundations of the winery, the winery foundations should be underpinned. The depth of the foundation system of the existing winery is not currently known. Intermittent hand-excavated piers may be used to underpin the existing foundations of this building. If intermittent piers are used, the excavation face between the underpinning piers should be retained using lagging provided the existing footing can span between piles/piers. Underpinning piers should extend at least two feet below the bottom of the planned excavation and should be designed to retain an at-rest pressure equal to 48 pcf. This value assumes the site has been dewatered to at least the depth of the bottom of the underpinning pit. Lateral pressures may be resisted by passive resistance against the embedded portion of the pier using passive pressures equal to 4,000 psf in the bedrock.

For vertical support, the hand-dug piers should act in end bearing. Piers extending into bedrock may be designed using an allowable bearing pressure of 6,000 psf for dead plus live loads. The recommended allowable bearing capacity can be used in the underpinning design provided the underpinning pits are dry, all loose soil is removed, and the pits can be visually inspected. We should observe the subgrade of the underpinning pier excavations to check that it is properly cleaned and can support the design pressure.

To reduce movement and provide adequate foundation support during installation of the underpinning piers, adjacent piers should not be excavated concurrently. We recommend underpinning piers be preloaded prior to dry packing to reduce settlement as the foundation load is transferred to the piers.

8.4 Permanent below-grade Walls

Below-grade walls should be designed to resist lateral pressures imposed by the adjacent soil and rock and any surcharge effects caused by loads adjacent to the wall (i.e. traffic, other retaining walls, and building loads). Because basement walls are not free to rotate they should be designed to accommodate the at-rest soil pressures presented in Table 8. These pressures assume that backfill has been placed against the newly constructed walls. Because the site is in a seismically active area, the design should also be checked for seismic condition. Under seismic loading conditions, there will be an additional seismic increment that should be added to active earth pressures (Lew et al. 2010). We used the procedures outlined in Lew et al. (2010) to compute the seismic active pressure. In this procedure, the seismic increment is a function the anticipated peak ground acceleration at the project site. Table 8 presents the at-rest and seismic pressures (active plus seismic pressure increment) for two code levels of shaking, the DE and the MCE event. All of these values assume the soil upslope of the permanent walls is relatively flat. All parameters are presented as equivalent fluid weights (triangular distribution).

TABLE 8
Retaining Wall Design Earth Pressures
(Drained Conditions)

	Static Conditions	**Seismic Condition* DE (PGA = 0.33g)**	**Seismic Condition* MCE (PGA = 0.51g)**
	Restrained Walls At-Rest	**Active Plus Seismic Pressure Increment**	**Active Plus Seismic Pressure Increment**
Level Backfill	58 pcf	38 + 0 pcf§	38 +25 pcf

* The more critical condition of either at-rest pressure for static conditions or seismic condition should be used. The appropriate design earthquake level for wall design should be evaluated by the structural engineer/wall designer.

§ No seismic increment is recommended when PGA is less than 0.4g.

If surcharge loads occur above an imaginary 45-degree line (from the horizontal) projected up from the bottom of a retaining wall, a surcharge pressure should be included in the wall design. If this condition exists, we should be consulted to estimate the added pressure on a case-by-case basis. Where vehicular traffic will pass within 10 feet of permanent retaining walls, temporary traffic loads should be considered in the design of the walls. Traffic loads may be modeled by a uniform pressure of 100 pounds per square foot applied in the upper 10 feet of the walls.

The lateral earth pressures presented in Table 8 are applicable to walls that are backdrained to prevent the buildup of hydrostatic pressure. One acceptable method for backdraining the wall is to place a prefabricated drainage panel (Miradrain 6000 or equivalent) against the backside of the wall. The drainage panel should extend down to a perforated PVC collector pipe at the base of the wall. The pipe should be surrounded on all sides by at least four inches of crushed rock (1-1/2- to 3/4-inch gradation) wrapped in filter fabric (Mirafi 140N or equivalent). AdvanEDGE pipe (or equivalent) may be used in lieu of the PVC pipe surrounded by crushed rock. The pipe should be sloped to drain into a closed collection system, such as the storm drain system. We should review the manufacturer's specifications for proposed prefabricated drainage panel material and drain pipe to verify they are appropriate for the intended use.

To protect against moisture migration, basement walls should be waterproofed and water stops placed at all construction joints. The waterproofing should be placed directly against the backside of the walls.

Where wall backfill is required, it should meet the requirements presented in Section 8.1.2 for on-site or imported fill and should be compacted to at least 90 percent relative compaction using light (hand-operated) compaction equipment. If heavy equipment is used, the wall should be appropriately designed to withstand loads exerted by the equipment and/or temporarily braced.

8.5 Floor Slabs

Where footings are used for building support, floor slabs may be supported on grade. Concrete slab-on-grade floors should have a minimum thickness of five inches and should be well reinforced (with at least No. 4 reinforcing steel bars, 16 inches on center). The soil subgrade beneath slab-on-grade floors should be compacted to at least 90 percent relative compaction. If the soil subgrade is disturbed during utility or foundation installation it should be scarified, moisture-conditioned, and rerolled to provide a firm, unyielding surface prior to placement of the vapor barrier or under slab drainage system.

Where the Hotel/Resort complex is supported on drilled piers or piles, the new fill placed to fill a portion of the pond and to create a level site is expected to settle over time. Therefore, floor slabs within this area of the Hotel/Resort complex should be designed to span between pier caps and/or grade beams, and the fill should not be relied upon for support.

To prevent a buildup of hydrostatic pressures on the Hotel/Resort complex, the floor slabs should be underlain entirely by a subdrain system over the bedrock or engineered fill. Because there is the potential for free water to be present in the subdrain system immediately beneath the floor slab, we recommend the floor slab in the Hotel/Resort complex be waterproofed.

If slab-on-grade floors or PT-slabs are used for proposed cabins and/or SFRs, moisture transmission through the slabs should be reduced by installing a capillary moisture break and a water vapor retarder beneath the slabs. A capillary moisture break should consist of at least four inches of clean, free-draining gravel or crushed rock. The vapor retarder should meet the requirements for Class C vapor retarders provided in ASTM E1745-97. The vapor retarder, which should be placed over the capillary break material, should be placed in accordance with the requirements of ASTM E1643-98. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder. The vapor retarder should be covered with two inches of sand to aid in concrete curing and to protect the vapor retarder during construction. The particle size of the gravel/crushed rock and sand should meet the gradation requirements presented in Table 9.

The sand overlying the membrane should be dry at the time concrete is placed. Excess water trapped in the sand could eventually be transmitted as vapor through the slab. If rain is forecast prior to pouring the slab, the sand should be covered with plastic sheeting to avoid wetting. If the sand becomes wet, concrete should not be placed until the sand has been dried or replaced.

TABLE 9
Gradation Requirements for Capillary Moisture Break

Sieve Size	Percentage Passing Sieve
Gravel or Crushed Rock	
1 inch	90–100
3/4 inch	30–100
1/2 inch	5–25
3/8 inch	0–6
Sand	
No. 4	100
No. 200	0–5

Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab. Therefore, concrete for the slabs should have a low w/c ratio - less than 0.50. If approved by the project structural engineer, the sand can be eliminated and the concrete can be placed directly over the vapor retarder, provided the w/c ratio of the concrete does not exceed 0.45 and water is not added in the field. If necessary, workability should be increased by adding plasticizers. In addition, the slabs should be properly cured.

Before the floor coverings are placed, the contractor should check that the concrete surface and the moisture emission levels meet the manufacturer’s requirements.

8.6 Surface Drainage Control

Surface drainage should be provided to collect surface runoff, prevent surface erosion, contain slough, and prevent saturation of the engineered fill. All surfaces should be sloped to drain and all water should be directed to a suitable outlet or retention basin.

Positive surface drainage should be provided around the buildings to direct surface water away from the foundation elements (a minimum of two percent for a lateral distance of at least five feet). In addition, roof downspouts should be discharged into controlled drainage facilities to keep the water away from the foundations. The use of water-intensive landscaping around the perimeter of the building should be avoided to reduce the amount of water introduced to the subgrade soils. Water-loving trees (i.e. eucalyptus trees) should not be planted within a horizontal distance equal to the mature height of the tree to prevent drying of the soil, which may result in ground-surface settlement.

We did not perform any percolation tests on site, as the type of potential infiltration system nor its location are known at this time.

The successful installation and operation of a surface water infiltration system will likely be difficult at this project site. The near-surface soils encountered varied considerably across the site. In addition, the site has a high groundwater table during the winter months and bedrock is relatively shallow. Lastly, where the near-surface soils are clayey, the infiltration rate will be very slow. If surface water infiltration systems are installed, no permeable pavements, detention basins, or bioswales should be within five feet of shallow foundations or floor slabs. If these types of features are required to be within five feet of the buildings, the bottoms and sides of these features should be lined with an impermeable membrane so that infiltration does not occur within five feet of the buildings.

8.7 Subsurface Drainage

The below-grade walls of the Hotel/Resort complex should be backdrained and an underslab subdrain systems should be installed beneath the floor slabs of the complex.

The wall backdrains should be installed as described in Section 8.4. The underslab drainage system installed beneath the Hotel/Resort complex should consist of at least 12 inches of gravel consisting of Class 2 permeable material or open graded crushed rock (1-1/2- to 3/4-inch gradation) placed on a slightly sloping subgrade (1 percent). The gravel blanket should extend across the entire building footprint. Perforated collector pipes should be installed to collect the water captured by the gravel blanket and to transmit the water to solid pipes that carry the water to a suitable outlet. The perforated pipes should have a minimum diameter of four inches, be installed in trenches with a minimum slope of one percent (a minimum of six inches deep) and have a maximum horizontal spacing of 45 feet. The perforated pipes should be connected to a solid 6-inch-diameter (minimum) pipe that conducts the water to a suitable outlet. Subdrain pipes should consist of either ABS (SDR-35) or PVC (Schedule 40 minimum) meeting Caltrans and/or ASTM requirements.

We understand a pump will be provided with the underslab drainage system. The pump will reduce the possibility that the underslab drainage system will receive water from the storm drain system during a storm event. It is difficult to anticipate the amount of water that will be generated by the underslab drainage system. As a minimum, we judge that the pump should be sized to accommodate a steady state flow of the capacity of the pipes. In addition, the pump should be connected to the emergency backup power system for the building and should be outfitted with an alarm to notify maintenance personnel if the pump fails to turn on during a storm event. We should be consulted during subdrain layout and design by the civil engineer.

Clean-outs should be provided for the underslab drain. Cleanouts should be provided for each length of pipe that has a bend sharper than 45 degrees and at approximately 200-foot intervals for straight pipe.

Because the purpose of the drainage blanket is to collect water, it cannot be relied on as a capillary break to protect the floor slab from vapor transmission; therefore the floor slab should be waterproofed.

8.8 Flexible Pavement

The State of California flexible pavement design method was used to develop the recommended asphalt concrete pavement sections. We expect the final soil subgrade in asphalt-paved areas will generally consist of native alluvial soils; sand and/or moderately expansive clay. Laboratory testing in 2007 on a sample of sandy clay with gravel resulted in an R-value of 41. However, based on the materials encountered in our recent borings, this R-value does not appear to be appropriate for all of the soils encountered at the site. Based on our experience with similar soils, we selected an R-value of 12 for design. If fill is placed in the area that will lie beneath paved areas, the fill material should have an R-value of at least 12. Additional tests may be performed during construction to confirm the use of a higher R-value, if deemed appropriate. Depending on the results of the tests, the pavement design can be revised.

We understand the proposed pavement at the site will consist of drive isles and parking spaces. Recommended pavement sections for several traffic indices are presented in Table 10. The appropriate traffic indices (TIs) for the new pavement should be selected by the project civil engineer.

TABLE 10
Asphaltic Concrete Pavement Section Design

TI	Asphaltic Concrete (inches)[1]	Class 2 Aggregate Base[2] (inches)[3]
4.5	3	7
5.5	3.5	10
6.5	4.0	12

Notes:
1. Asphaltic Concrete should have a minimum thickness of 2.5 inches
2. Class 2 Aggregate Base material should have a minimum R-Value of 78
3. Class 2 Aggregate Base should have a minimum thickness of 6 inches

Pavement components should conform to the current Caltrans Standard Specifications. The upper six inches of the soil subgrade in pavement areas should be moisture-conditioned to above optimum and compacted to at least 95 percent relative compaction and rolled to provide a smooth non-yielding surface. Aggregate base should be compacted to at least 95 percent relative compaction.

8.9 2010 California Building Code Mapped Values

For seismic design in accordance with the provisions of 2010 California Building Code (CBC) we recommend the following:

- Maximum Considered Earthquake (MCE) S_s and S_1 values of 1.27g and 0.54g, respectively.
- Site Class C
- Site Coefficients F_a and F_v of 1.0 and 1.3
- Maximum Considered Earthquake (MCE) spectral response acceleration parameters at short periods, S_{MS}, and at one-second period, S_{M1}, of 1.27g and 0.70g, respectively.
- Design Earthquake (DE) spectral response acceleration parameters at short period, S_{DS}, and at one-second period, S_{D1}, of 0.85g and 0.47g, respectively.

8.10 Concrete Flatwork

In areas to receive concrete patios, sidewalks or other flatwork, the subgrade should be scarified to a depth of at least 12 inches, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent relative compaction. Exterior concrete flatwork should be underlain by at least four inches of Class 2 aggregate base. The aggregate base should extend at least six inches beyond the slab edges.

8.11 Swimming Pools

Four in-ground pools are planned for the project; a fitness pool near the northwestern portion of the site and three pools in the central portion of the site, just east of the knoll. We anticipate that the three pools in the central portion of the site will be excavated into stiff clay and/or medium dense sand within the alluvium. The fitness pool will likely extend into bedrock. Excavations that will be deeper than five feet and will be entered by workers should be shored or sloped in accordance with the OSHA standards (29 CFR Part 1926). Because the soil through which the pools will extend is moderately expansive, we recommend that the sides and bottom of the pool excavation be kept wet following excavation and their moist condition maintained until concrete is placed. We should check the condition of the pool excavations just prior to concrete placement to confirm that the excavations are sufficiently moist.

Considering the locally high groundwater in the vicinity of the proposed pools, we recommend the vertical sides of the pools be designed as retaining walls restrained from rotation using an at-rest pressure of 90 pcf, equivalent fluid weight. This value assumes that the groundwater could be as high as the future ground surface. The pool bottom should be underlain by an eight-inch gravel blanket and be equipped with hydrostatic pressure relief valves to prevent excess hydrostatic pressure from damaging the pool in the event it is drained.

8.12 Additional Geotechnical Exploration

As discussed in sections 5.2 and 7.1, borings TR-11 and TR-12 encountered loose, clean, saturated sand between 7 and 14.5 feet beneath the existing ground surface that is potentially liquefiable. As discussed in Section 7.2 this material is not suitable for foundation support. We have postulated that this clean sand is fill that was placed as utility trench backfill following the installation of a utility along the private utility easement.

To evaluate the extent of these potentially liquefiable soils within the nearby proposed building footprints, we recommend at least two trenches be excavated. The trenches should start at the alignment of the public easement and extend northwest towards the future building pads. The trenches should be at least 8 feet deep to expose the top of the potentially liquefiable soils observed in our borings. If encountered beneath the proposed buildings, the full extent of the potentially liquefiable soils should be evaluated during trenching. We should be on-site to observe the conditions exposed in the trenches and to evaluate the need for additional exploration.

Once the lateral limits of this potentially liquefiable clean sand have been identified additional supplemental recommendations can be provided, as necessary. Recommendations for limiting the detrimental effects of this material are presented in Section 8.1.2, if these soils are present beneath the new building pads.

8.13 Construction Considerations

Much of the near surface soils consist of stiff clay. If construction activities are performed during the winter/rainy season, the near-surface soils will be saturated and easily remolded. The contractor should be prepared to handle this material. In addition, the exposed material in the existing irrigation pond is likely soft and saturated. All of the soft and saturated material must be removed prior to placing any new engineered fill in the limits of the irrigation pond. To facilitate this removal of soft material, the pond should be emptied several weeks prior to the beginning of excavation activities.

Foundation excavations should be free of standing water, debris, and disturbed materials prior to placing concrete. As discussed in Section 8.2.2, groundwater and potentially caving soils may be encountered during drilled pier installation. The contractor should anticipate this condition and be prepared to prevent caving of the drilled pier holes.

During the excavation for the new Hotel/Resort complex the groundwater at the project site should be lowered to a depth of at least three feet below the bottom of the planned maximum excavation depth. Elevator and sump pits should be locally dewatered.

9.0 ADDITIONAL GEOTECHNICAL SERVICES

In addition to the services described in Section 8.12, during final design we should be retained to consult with the design team as geotechnical and foundation questions arise. Prior to construction, Treadwell & Rollo, A Langan Company, should review the project plans and specifications to verify that they conform to the intent of our recommendations. During construction, our field engineer should provide on-site observation. These observations will allow us to compare actual with anticipated soil conditions and to verify that the contractor's work conforms with the geotechnical aspects of the plans and specifications.

Specifically, all site preparation and fill placement should be observed by Treadwell & Rollo, A Langan Company. It is important that, during the stripping, scarification, and fill process, a representative of Treadwell & Rollo be present to observe whether any undesirable material is encountered in the construction areas and provide supplemental recommendations. In addition, we should be on site during the excavation activities and installation of new foundations and drilled piers, temporary shoring and underpinning, subdrain systems, permanent retaining walls, and backfill of utilities.

10.0 LIMITATIONS

The conclusions and recommendations presented in this report apply to the site and construction conditions as we have described them and are the result of engineering studies and our interpretations of the existing geotechnical conditions at the time of our field activities. Actual subsurface conditions may vary. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that described in this report, Treadwell & Rollo, A Langan Company should be notified so that supplemental recommendations can be developed, as necessary.

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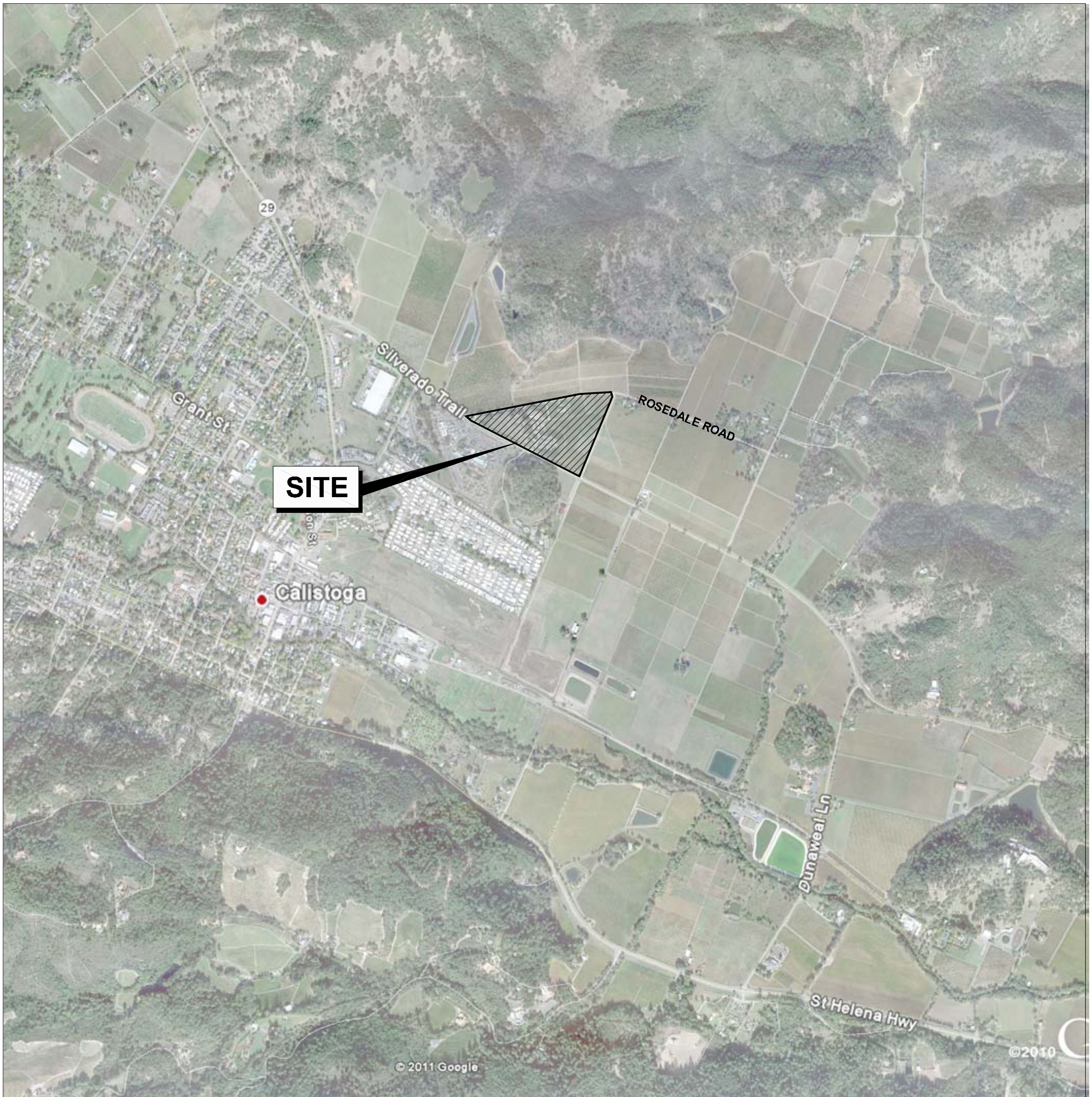
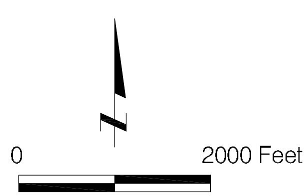
FIGURES





Reference: Goolge Earth Pro, 2011.

SILVER ROSE RESORT **400 SILVERADO TRAIL** Calistoga, California	**SITE LOCATION MAP**
Treadwell&Rollo A LANGAN COMPANY	Date 07/12/11 | Project No. 730453902 | Figure 1

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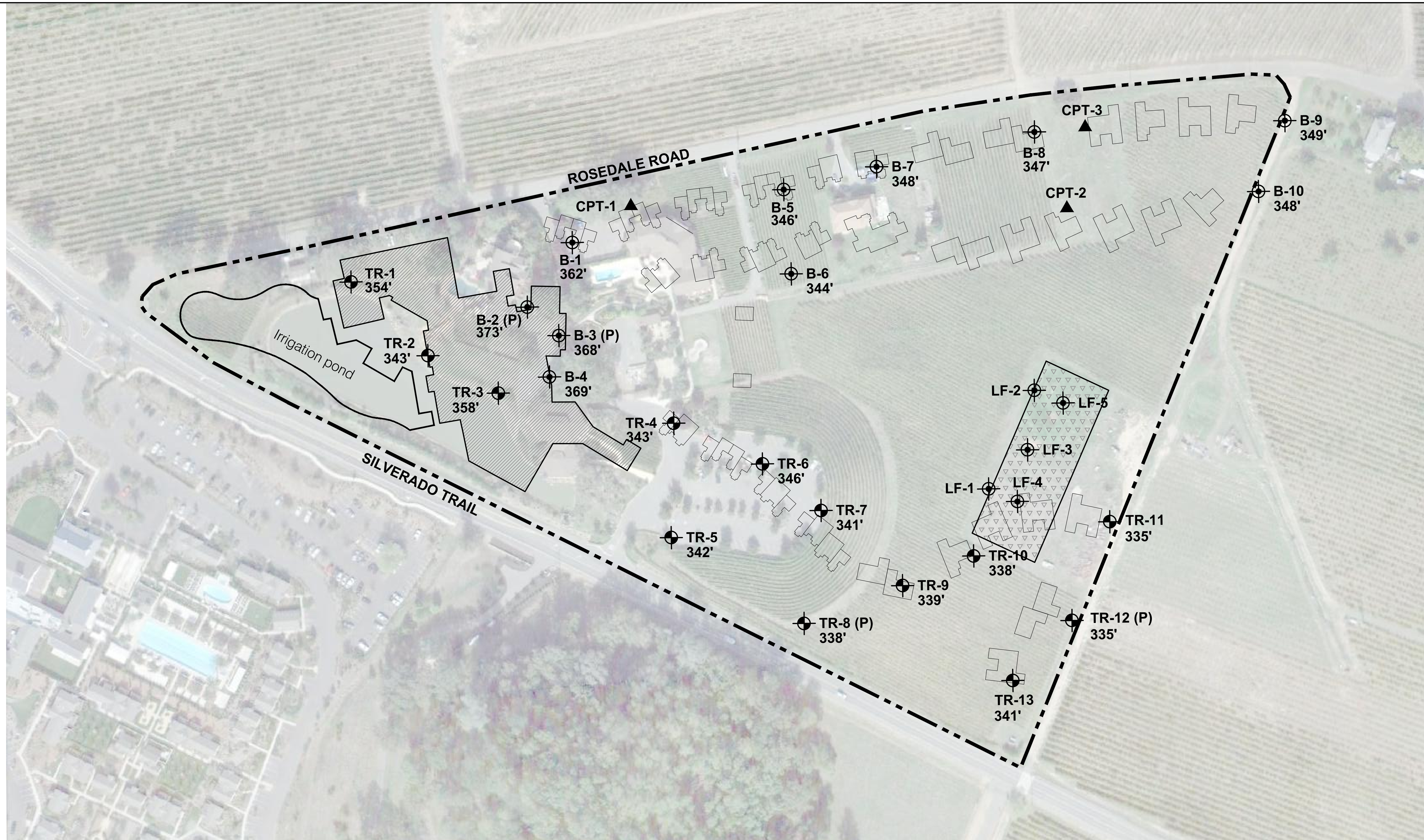

Reference: Google Earth Pro, 2011.

EXPLANATION

TR-1 354' — Approximate location of boring by Treadwell & Rollo, A Langan Company, June 2011 and approximate Elevation of bedrock, MSL

CPT-1 — Approximate location of cone penetration test by Treadwell & Rollo, Inc., January 2007

B-1 — Approximate location of boring by Treadwell & Rollo, Inc., December 2006 and January 2007

LF-1 — Approximate location of boring by Treadwell & Rollo, Inc. in leach field, January 2007

Approximate site boundary

Approximate location of Hotel/Resort Complex

Approximate location of existing leach field

Approximate location of cabins/single family homes

0 160 Feet

Approximate scale

SILVER ROSE RESORT
400 SILVERADO TRAIL
Calistoga, California

SITE PLAN

Date 11/04/11	Project No. 730453902	Figure 2

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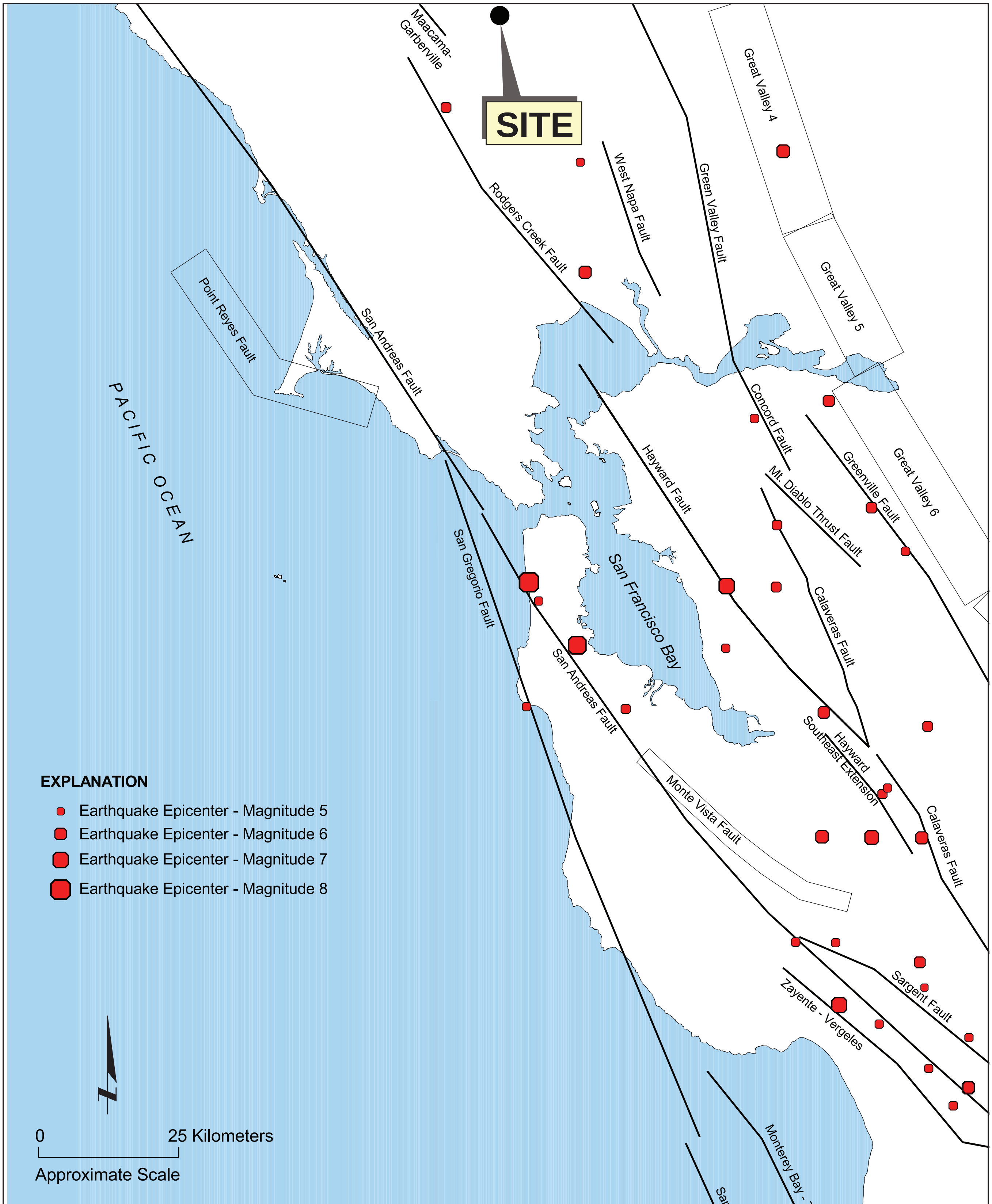

NOTES:
Digitized data for fault coordinates and earthquake catalog was developed by the California Department of Conservation Division of Mines and Geology. The historic earthquake catalog includes events from January 1800 to December 2000.

SILVER ROSE RESORT
400 SILVERADO TRAIL
Calistoga, California

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MAP OF MAJOR FAULTS AND EARTHQUAKE EPICENTERS IN THE SAN FRANCISCO BAY AREA

Date 07/01/11 | Project No. 730453902 | Figure 3

I	**Not felt by people, except under especially favorable circumstances. However, dizziness or nausea may be experienced.** Sometimes birds and animals are uneasy or disturbed. Trees, structures, liquids, bodies of water may sway gently, and doors may swing very slowly.
II	**Felt indoors by a few people, especially on upper floors of multi-story buildings, and by sensitive or nervous persons.** As in Grade I, birds and animals are disturbed, and trees, structures, liquids and bodies of water may sway. Hanging objects swing, especially if they are delicately suspended.
III	**Felt indoors by several people, usually as a rapid vibration that may not be recognized as an earthquake at first. Vibration is similar to that of a light, or lightly loaded trucks, or heavy trucks some distance away. Duration may be estimated in some cases.** Movements may be appreciable on upper levels of tall structures. Standing motor cars may rock slightly.
IV	**Felt indoors by many, outdoors by a few. Awakens a few individuals, particularly light sleepers, but frightens no one except those apprehensive from previous experience. Vibration like that due to passing of heavy, or heavily loaded trucks. Sensation like a heavy body striking building, or the falling of heavy objects inside.** Dishes, windows and doors rattle; glassware and crockery clink and clash. Walls and house frames creak, especially if intensity is in the upper range of this grade. Hanging objects often swing. Liquids in open vessels are disturbed slightly. Stationary automobiles rock noticeably.
V	**Felt indoors by practically everyone, outdoors by most people. Direction can often be estimated by those outdoors. Awakens many, or most sleepers. Frightens a few people, with slight excitement; some persons run outdoors.** Buildings tremble throughout. Dishes and glassware break to some extent. Windows crack in some cases, but not generally. Vases and small or unstable objects overturn in many instances, and a few fall. Hanging objects and doors swing generally or considerably. Pictures knock against walls, or swing out of place. Doors and shutters open or close abruptly. Pendulum clocks stop, or run fast or slow. Small objects move, and furnishings may shift to a slight extent. Small amounts of liquids spill from well-filled open containers. Trees and bushes shake slightly.
VI	**Felt by everyone, indoors and outdoors. Awakens all sleepers. Frightens many people; general excitement, and some persons run outdoors.** Persons move unsteadily. Trees and bushes shake slightly to moderately. Liquids are set in strong motion. Small bells in churches and schools ring. Poorly built buildings may be damaged. Plaster falls in small amounts. Other plaster cracks somewhat. Many dishes and glasses, and a few windows break. Knickknacks, books and pictures fall. Furniture overturns in many instances. Heavy furnishings move.
VII	**Frightens everyone. General alarm, and everyone runs outdoors.** People find it difficult to stand. Persons driving cars notice shaking. Trees and bushes shake moderately to strongly. Waves form on ponds, lakes and streams. Water is muddied. Gravel or sand stream banks cave in. Large church bells ring. Suspended objects quiver. Damage is negligible in buildings of good design and construction; slight to moderate in well-built ordinary buildings; considerable in poorly built or badly designed buildings, adobe houses, old walls (especially where laid up without mortar), spires, etc. Plaster and some stucco fall. Many windows and some furniture break. Loosened brickwork and tiles shake down. Weak chimneys break at the roofline. Cornices fall from towers and high buildings. Bricks and stones are dislodged. Heavy furniture overturns. Concrete irrigation ditches are considerably damaged.
VIII	**General fright, and alarm approaches panic.** Persons driving cars are disturbed. Trees shake strongly, and branches and trunks break off (especially palm trees). Sand and mud erupts in small amounts. Flow of springs and wells is temporarily and sometimes permanently changed. Dry wells renew flow. Temperatures of spring and well waters varies. Damage slight in brick structures built especially to withstand earthquakes; considerable in ordinary substantial buildings, with some partial collapse; heavy in some wooden houses, with some tumbling down. Panel walls break away in frame structures. Decayed pilings break off. Walls fall. Solid stone walls crack and break seriously. Wet grounds and steep slopes crack to some extent. Chimneys, columns, monuments and factory stacks and towers twist and fall. Very heavy furniture moves conspicuously or overturns.
IX	**Panic is general.** Ground cracks conspicuously. Damage is considerable in masonry structures built especially to withstand earthquakes; great in other masonry buildings - some collapse in large part. Some wood frame houses built especially to withstand earthquakes are thrown out of plumb, others are shifted wholly off foundations. Reservoirs are seriously damaged and underground pipes sometimes break.
X	**Panic is general.** Ground, especially when loose and wet, cracks up to widths of several inches; fissures up to a yard in width run parallel to canal and stream banks. Landsliding is considerable from river banks and steep coasts. Sand and mud shifts horizontally on beaches and flat land. Water level changes in wells. Water is thrown on banks of canals, lakes, rivers, etc. Dams, dikes, embankments are seriously damaged. Well-built wooden structures and bridges are severely damaged, and some collapse. Dangerous cracks develop in excellent brick walls. Most masonry and frame structures, and their foundations are destroyed. Railroad rails bend slightly. Pipe lines buried in earth tear apart or are crushed endwise. Open cracks and broad wavy folds open in cement pavements and asphalt road surfaces.
XI	**Panic is general.** Disturbances in ground are many and widespread, varying with the ground material. Broad fissures, earth slumps, and land slips develop in soft, wet ground. Water charged with sand and mud is ejected in large amounts. Sea waves of significant magnitude may develop. Damage is severe to wood frame structures, especially near shock centers, great to dams, dikes and embankments, even at long distances. Few if any masonry structures remain standing. Supporting piers or pillars of large, well-built bridges are wrecked. Wooden bridges that "give" are less affected. Railroad rails bend greatly and some thrust endwise. Pipe lines buried in earth are put completely out of service.
XII	**Panic is general.** Damage is total, and practically all works of construction are damaged greatly or destroyed. Disturbances in the ground are great and varied, and numerous shearing cracks develop. Landslides, rock falls, and slumps in river banks are numerous and extensive. Large rock masses are wrenched loose and torn off. Fault slips develop in firm rock, and horizontal and vertical offset displacements are notable. Water channels, both surface and underground, are disturbed and modified greatly. Lakes are dammed, new waterfalls are produced, rivers are deflected, etc. Surface waves are seen on ground surfaces. Lines of sight and level are distorted. Objects are thrown upward into the air.

SILVER ROSE RESORT
400 SILVERADO TRAIL
Calistoga, California

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MODIFIED MERCALLI INTENSITY SCALE

Date 07/01/11	Project No. 730453902	Figure 4

APPENDIX A
Logs of Borings

PROJECT:	SILVER ROSE RESORT 400 SILVERADO TRAIL Calistoga, California	Log of Boring TR-1 PAGE 1 OF 1

Boring location: See Site Plan, Figure 2	Logged by: S. Magallon
Date started: 6/13/11 — Date finished: 6/13/11	
Drilling method: Hollow Stem Auger	
Hammer weight/drop: 140 lbs./30 inches — Hammer type: Automatic	
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)	

DEPTH (feet)	Sampler Type	Blows/6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION (Ground Surface Elevation: ~361 foot[2])	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–2.5				SP	SAND (SP) gray-brown, loose, moist						
2.5–4.5	S&H	5 4 4	6	CL	CLAY (CL) brown, medium stiff, moist, trace coarse-grained sand						
4.5–7.5	S&H	4 4 6	7	SC	CLAYEY SAND (SC) yellow-brown, loose, moist, trace fine gravel ▽ (06/13/11, 11:10 a.m.) LL = 36, PI = 16				39.5	23.0	98
7.5–17.25	S&H	8 20 27	33		TUFF yellow-brown and gray mottled, moderately to closely fractured, low hardness, friable to weak, moist ▽ (06/13/11, 11:05 a.m.)						
	S&H	8 19 40	41								
	SPT	32 50/4"	60/4"		moderately hard, weak to moderately strong yellow-brown, moist, white parting in upper 1" (quartz)						
	SPT	50/3"	60/3"								
	SPT	50/3"	60/3"								

Boring terminated at a depth of 17.25 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 10 feet below ground surface during drilling.
LL = liquid limit, PI = plastic index

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.
[2] Elevation based on topographic map titled "Map of Topography of the Lands of Silver Rose Inn," by Albion Surveys, Inc., revison dated 12/15/10.

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Project No.: 730453902 — Figure: A-1

TEST GEOTECH LOG 730453902.GPJ TR.GDT 9/6/11

PROJECT:	SILVER ROSE RESORT 400 SILVERADO TRAIL Calistoga, California	Log of Boring TR-2 PAGE 1 OF 1

Boring location: See Site Plan, Figure 2

Logged by: S. Magallon

Date started: 6/13/11 — Date finished: 6/13/11

Drilling method: Hollow Stem Auger

Hammer weight/drop: 140 lbs./30 inches — Hammer type: Automatic

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Ground Surface Elevation: ~361 foot[2]

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–3.5	S&H (at ~2–3.5)	3, 3, 4	5	CL	CLAY (CL) brown, medium stiff, moist, trace fine gravel and fine-grained sand. LL = 41, PI = 17					23.5	96
3.5–6.5	S&H (at ~5–6.5)	3, 3, 3	4	SP	SAND (SP) brown to yellow-brown, loose, moist				2.9	8.0	
6.5–8				CL	SANDY CLAY (CL) yellow-brown mottled yellow, soft, moist, fine- to coarse-grained sand						
8–11.5	S&H (at ~8–9.5)	2, 2, 4	4	SC	CLAYEY SAND (SC) yellow-brown and brown, loose, wet, fine- to coarse-grained sand and trace gravel. ▽ (06/13/11, 2:45 p.m.) Sieve Analysis, see Figure B-1					32.1	89
	S&H (at ~10–11.5)	4, 4, 6	7						20.2	28.2	90
11.5–14.5	SPT (at ~12–13.5)	4, 7, 11	22	SC	CLAYEY SAND (SC) yellow-brown, medium dense, wet						
14.5–17.5	S&H (at ~15–16.5)	6, 15, 12	19	SW	SAND (SW) yellow-brown, gray, white, and red (varicolored), medium dense, wet, subrounded to subangular sand and gravel				3.9	15.0	
17.5–27.25	SPT (at ~20–21.5)	8, 35, 34	83		TUFF yellow-brown and gray mottled, moderately hard, friable to weak, moderately to deeply weathered						
	SPT (at ~25–26)	19, 50/ 6"	60/ 6"								
	SPT (at ~26.5–27)	50/ 6"	60/ 6"								

Boring terminated at a depth of 27.25 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 10 feet below ground surface during drilling.
LL = liquid limit, PI = plastic index

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.
[2] Elevation based on topographic map titled "Map of Topography of the Lands of Silver Rose Inn," by Albion Surveys, Inc., revison dated 12/15/10.

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Project No.: 730453902 — Figure: A-2

PROJECT: **SILVER ROSE RESORT**
400 SILVERADO TRAIL
Calistoga, California

Log of Boring TR-3

Boring location:	See Site Plan, Figure 2
Date started:	6/14/11
Date finished:	6/14/11
Drilling method:	Hollow Stem Auger
Hammer weight/drop:	140 lbs./30 inches
Hammer type:	Automatic
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by:	S. Magallon

Ground Surface Elevation: ~366 foot[2]

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Blows/6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
1				CL	SANDY CLAY (CL) brown, stiff, moist						
3	S&H	6 6 7	9								
6	S&H	5 6 7	9							16.6	109
8					TUFF yellow to yellow-brown, low hardness, weak, moderately weathered						
9	S&H	5 15 45	42								
10	SPT	33 50/ 1"	60/ 1"								
14	SPT	50/ 4"	60/ 4"		little weathered						

Boring terminated at a depth of 13.83 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.
[2] Elevation based on topographic map titled "Map of Topography of the Lands of Silver Rose Inn," by Albion Surveys, Inc., revison dated 12/15/10.

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Project No.: 730453902
Figure: A-3

PROJECT:	SILVER ROSE RESORT 400 SILVERADO TRAIL Calistoga, California	# Log of Boring TR-4

PAGE 1 OF 1

Boring location:	See Site Plan, Figure 2	Logged by:	S. Walker
Date started:	6/15/11	Date finished:	6/15/11
Drilling method:	Hollow Stem Auger		
Hammer weight/drop:	140 lbs./30 inches	Hammer type:	Hydraulic Trip
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT)		

MATERIAL DESCRIPTION — Ground Surface Elevation: ~359 foot[2]

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–1					2-inches Asphalt Concrete (AC)						
					8-inches Aggregate Base (AB)						
1–6	S&H	4, 3, 3	4	SC	CLAYEY SAND with GRAVEL (SC) brown, loose, moist to wet, angular gravel from sand size up to 3/8-inch in diameter LL = 34, PI = 13				24.0	18.8	91
	S&H	4, 4, 7	7								
6–10	S&H	5, 6, 8	8	CL	SANDY CLAY with GRAVEL (CL) yellow to yellow-brown with red-brown mottling, medium stiff, angular to subrounded gravel in plastic matrix, fine- to medium-grained sand				58.8	22.3	
10–12.5	S&H	4, 7, 13	12	SC	CLAYEY SAND with GRAVEL (SC) ▽ yellow-brown, medium dense, wet (06/15/11, 9:00 a.m.) Sieve Analysis, see Figure B-1				25.6	18.8	
12.5–15.5	S&H	6, 10, 11	13	GP-GM	SANDY GRAVEL with SILT (GP-GM) variegated red, brown, and white, medium dense, wet, yellow to yellow-brown angular to subangular fine to coarse sand, open graded Sieve Analysis, see Figure B-1				5.1	16.0	
					plastic clayey sand						
16–19.5					TUFF red-brown, soft to low hardness, friable, deeply weathered, wet						
	S&H	13, 16, 22	23		yellow to yellow-brown, very stiff, wet						
19.5–25					TUFF weathered brown to light gray, closely fractured to intensely fractured, soft to low hardness, weak, moderately weathered						
23	S&H	50/ 6"	30/ 6"								
24	SPT	35, 50/ 6"	50/ 6"								

Boring terminated at a depth of 25 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 11 feet below ground surface during drilling.
LL = liquid limit, PI = plastic index

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.
[2] Elevation based on topographic map titled "Map of Topography of the Lands of Silver Rose Inn," by Albion Surveys, Inc., revison dated 12/15/10.

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Project No.: 730453902 | Figure: A-4

PROJECT:	SILVER ROSE RESORT 400 SILVERADO TRAIL Calistoga, California	**Log of Boring TR-5**
		PAGE 1 OF 1

Boring location:	See Site Plan, Figure 2
Date started:	6/15/11
Date finished:	6/15/11
Drilling method:	Hollow Stem Auger
Hammer weight/drop:	140 lbs./30 inches
Hammer type:	Hydraulic Trip
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by:	S. Walker

MATERIAL DESCRIPTION — Ground Surface Elevation: ~356 foot[2]

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0				CL	3-inches Asphalt Concrete (AC)						
1					CLAY (CL) brown, medium stiff, moist						
2–3	S&H	4 4 7	7	SP-SC	GRAVELLY SAND with CLAY (SP-SC) mottled yellow-brown and dark brown with red and red-brown, loose, moist, fine- to medium gravel					19.5	100
5–6	S&H	6 5 4	5							20.1	93
6–8				SC	CLAYEY SAND (SC) brown, loose, moist, fine- to medium-grained sand, trace fine- to medium-grained subrounded gravel decreasing clay content						
8–9	S&H	5 5 6	7	GP-GC	SANDY GRAVEL with CLAY (GP-GC) yellow-brown, loose, wet, fine- to coarse subangular to angular sand and gravel ∇ (06/15/11, 2:30 p.m.) Sieve Analysis, see Figure B-1				10.6	18.9	
10–11	S&H	15 16 30	28								
12				SC	CLAYEY SAND with GRAVEL (SC) yellow-brown, medium dense, wet, with gravel up to 2 inches in diameter grades light brown						
14	S&H	17 50/ 6"	30/ 6"		TUFF yellow to yellow-brown, low hardness, weak, moderately to deeply weathered						
19	S&H	50/ 5"	30/ 5"		light brown with gray spots, low hardness to moderately hard, weak, deeply weathered, plastic						
24	SPT	13 28 46	74		with dark gray, white and light pink clasts in light brown matrix, plastic						
25–30											

Boring terminated at a depth of 25 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 9.5 feet below ground surface during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.
[2] Elevation based on topographic map titled "Map of Topography of the Lands of Silver Rose Inn," by Albion Surveys, Inc., revison dated 12/15/10.

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Project No.: 730453902 | Figure: A-5

PROJECT:	SILVER ROSE RESORT 400 SILVERADO TRAIL Calistoga, California	Log of Boring TR-6 PAGE 1 OF 1

Boring location:	See Site Plan, Figure 2
Date started: 6/15/11	Date finished: 6/15/11
Drilling method:	Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches	Hammer type: Hydraulic Trip
Sampler:	Sprague & Henwood (S&H)
Logged by:	S. Walker

DEPTH (feet)	Sampler Type	Blows/6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: ~359 foot[2]	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					3-inches Asphaltic Concrete (AC)						
0–1					7-inches Aggregate Base (AB)						
1–4	S&H	7 10 12	13	SC	CLAYEY SAND (SC) mottled yellow-brown, medium dense, dry, with trace sandstone gravel up to 2 inches in diameter						
4–6	S&H	5 6 7	8	CL	SANDY CLAY with GRAVEL (CL) dark brown, medium stiff, moist, fine- to medium-grained sand, fine gravel LL = 35, PI = 12					21.7	94
8–9	S&H	4 5 6	7	CL	brown to yellow-brown grades wet					18.5	99
10–11	S&H	5 6 6	7	CL	yellow-brown to yellow increased gravel content white, yellow, and red gravel					20.7	103
12.5–15	S&H	8 8 13	13		∇ (06/15/11, 11:45 a.m.) TUFF yellow-brown with gray and red-brown mottling, soft to low hardness, friable, deeply weathered, plastic LL = 32, PI = 11						
19–20	S&H	18 30 43	44		gray-brown, moderately weathered, weak, low hardness						
24–25	S&H	15 28 35	38		with black spots, near vertical bed/ contact, platic						

Boring terminated at a depth of 25 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 13 feet below ground surface during drilling.
LL = liquid limit, PI = plastic index

[1] S&H blow counts for the last two increments were converted to SPT N-Values using a factor of 0.6, to account for sampler type and hammer energy.
[2] Elevation based on topographic map titled "Map of Topography of the Lands of Silver Rose Inn," by Albion Surveys, Inc., revison dated 12/15/10.

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Project No.:	Figure:
730453902	A-6

PROJECT:	SILVER ROSE RESORT 400 SILVERADO TRAIL Calistoga, California	**Log of Boring TR-7** PAGE 1 OF 1

Boring location: See Site Plan, Figure 2	Logged by: S. Walker
Date started: 6/15/11	Date finished: 6/15/11
Drilling method: Hollow Stem Auger	
Hammer weight/drop: 140 lbs./30 inches	Hammer type: Hydraulic Trip
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)	

MATERIAL DESCRIPTION — Ground Surface Elevation: ~359 foot[2]

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Blows/6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					3-inches Asphaltic Concrete (AC)						
0.25–0.75					6-inches Aggregate Base (AB)						
1–4 (FILL)											
2–3.5	S&H	10, 9, 9	11	GC	CLAYEY GRAVEL with SAND (GC) yellow to yellow-brown, medium dense, wet, fine- to coarse-grained sand					16.7	96
5–6.5	S&H	10, 11, 11	13	CL	SILTY CLAY (CL) brown, dark brown to olive-brown, stiff, moist, trace fine gravel LL = 37, PI = 14						
8–9.5	S&H	5, 6, 9	9	SC	CLAYEY SAND with GRAVEL (SC) yellow-brown, loose to medium dense, wet, fine- to coarse rounded gravel Sieve Analysis, see Figure B-2 LL = 33, PI = 9				37.3	16.9	
10–11.5	S&H	8, 8, 11	11	SC	white, red, and brown, subangular to subrounded fine- to coarse gravel						
13–14.5	S&H	12, 20, 24	26	ML	▽ SANDY SILT with GRAVEL (ML) light brown with orange mottling, very stiff, moist (06/15/11, 1:00 p.m.)						
15				ML	near vertical gray sand bed encountered in sample						
18–19.5	S&H	7, 7, 11	11		TUFF yellow to yellow-brown, soft to low hardness, friable to weak, deeply weathered						
20–21.5	SPT	10, 28, 40	68		yellow to yellow-brown with dark brown spots						
23.5–25	S&H	30, 25, 24	29		yellow-brown to olive-brown mottled with red, white, and black, wet						

Boring terminated at a depth of 25 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 13 feet below ground surface during drilling.
LL = liquid limit, PI = plastic index

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.
[2] Elevation based on topographic map titled "Map of Topography of the Lands of Silver Rose Inn," by Albion Surveys, Inc., revison dated 12/15/10.

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Project No.: 730453902 | Figure: A-7

TEST GEOTECH LOG 730453902.GPJ TR.GDT 9/2/11

PROJECT: **SILVER ROSE RESORT**
400 SILVERADO TRAIL
Calistoga, California

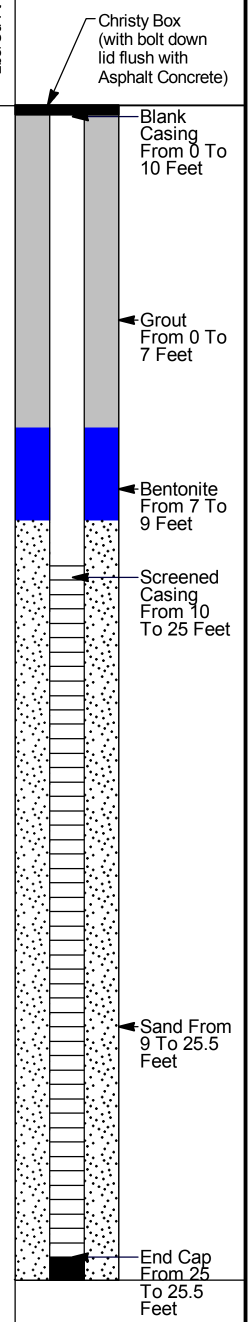
Log of Boring TR-8

Boring location:	See Site Plan, Figure 2
Date started:	6/15/11
Date finished:	6/15/11
Drilling method:	Hollow Stem Auger
Hammer weight/drop:	140 lbs./30 inches
Hammer type:	Automatic
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by:	S. Magallon

Ground Surface Elevation: ~353 foot[2]

DEPTH (feet)	Sampler Type	Blows/6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–4.5				CL	CLAY (CL) brown, stiff, moist, trace fine-grained sand, trace silt						
2–3.5	S&H	7, 7, 6	9		LL = 36, PI = 16						
4.5–13.5				CL	SANDY CLAY (CL) yellow-brown, stiff, moist, trace fine gravel, fine- to medium-grained sand						
5–6.5	S&H	4, 7, 7	10							23.4	99
8–9.5	S&H	6, 9, 11	14								
10–11.5	S&H	5, 7, 14	15								
13.5–15				CL	SANDY CLAY with GRAVEL (CL) yellow-brown, very stiff, moist, fine- to coarse sand, fine gravel						
15–25.5					TUFF yellow-brown, unknown fracturing, soft to low hardness, friable, deeply weathered						
15–16.5	S&H	11, 22, 24	32								
20–21.5	S&H	9, 25, 50/3"	53/9"		yellow and tan, low hardness, weak, wet						
25	SPT	50/2"	60/2"		closely fractured, low hardness, weak, deeply weathered						

Piezometer Completion Detail



Boring terminated at a depth of 25.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.
LL = liquid limit, PI = plastic index

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively to account for sampler type and hammer energy.
[2] Elevation based on topographic map titled "Map of Topography of the Lands of Silver Rose Inn," by Albion Surveys, Inc., revison dated 12/15/10.

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Project No.: 730453902
Figure: A-8

PROJECT:	SILVER ROSE RESORT 400 SILVERADO TRAIL Calistoga, California	Log of Boring TR-9 PAGE 1 OF 1

Boring location: See Site Plan, Figure 2

Logged by: S. Magallon

Date started: 6/14/11 | Date finished: 6/14/11

Drilling method: Solid Flight Auger, Portable Minute Man

Hammer weight/drop: 140 lbs./30 inches | Hammer type: Rope & Cathead

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

DEPTH (feet)	Sampler Type	Blows/6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: ~353 foot[2]	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–2				CL	SANDY CLAY (CL) yellow-brown, stiff, moist, fine- to coarse-grained sand, trace rootlets						
3–4	S&H	6 10 9	11		LL = 35, PI = 15					18.8	101
6–7	S&H	19 39 33	43		hard no rootlets, trace subangular gravel					20.8	103
9–10	S&H	30 50/ 6"	30/ 6"		porous structure						
15	S&H	50/ 6"	30/ 6"		TUFF yellow-brown, unknown fracturing, low hardness, weak to friable, moderate to deeply weathered, moist to wet						
16	SPT	23 50/ 6"	50/ 6"								

Boring terminated at a depth of 16.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.
LL = liquid limit, PI = plastic index

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.
[2] Elevation based on topographic map titled "Map of Topography of the Lands of Silver Rose Inn," by Albion Surveys, Inc., revison dated 12/15/10.

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Project No.: 730453902 | Figure: A-9

TEST GEOTECH LOG 730453902.GPJ TR.GDT 9/2/11

PROJECT:	SILVER ROSE RESORT 400 SILVERADO TRAIL Calistoga, California	Log of Boring TR-10 PAGE 1 OF 1

Boring location: See Site Plan, Figure 2

Date started: 6/15/11 | Date finished: 6/15/11

Drilling method: Solid Flight Auger, Portable Minute Man

Hammer weight/drop: 140 lbs./30 inches | Hammer type: Rope & Cathead

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Logged by: S. Magallon

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Ground Surface Elevation: ~351 foot[2]	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–5	S&H (3–4.5)		8, 7, 6	8	SC	CLAYEY SAND with GRAVEL (SC) brown, olive-brown to dark brown fine- to coarse-grained sand, loose, moist, fine red and gray gravel						
5–13.5	S&H (6–7.5)		11, 11, 17	17	CL	SANDY CLAY (CL) dark brown to yellow-brown mottling yellow, very stiff, moist, trace fine gravel, fine- to medium-grained sand ▽ (06/15/11, 10:15 a.m.)					17.6	106
	S&H (9–10.5)		12, 50/ 6"	30		increase in clay content, yellow-brown, decrease in fine gravel content						
13.5–15.83	S&H (15–15.83)		38, 50/ 4"	30/ 4"		TUFF yellow-brown and gray, unknown fracturing, low hardness, friable, moderately weathered, wet ▽ (06/15/11, 10:00 a.m.)						

Boring terminated at a depth of 15.83 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 15 feet below ground surface during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.
[2] Elevation based on topographic map titled "Map of Topography of the Lands of Silver Rose Inn," by Albion Surveys, Inc., revison dated 12/15/10.

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Project No.: 730453902 | Figure: A-10

PROJECT: **SILVER ROSE RESORT**
400 SILVERADO TRAIL
Calistoga, California

Log of Boring TR-11

Boring location:	See Site Plan, Figure 2
Date started: 6/15/11	Date finished: 6/15/11
Drilling method:	Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches	Hammer type: Hydraulic Trip
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by:	S. Walker

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Blows/6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION — Ground Surface Elevation: ~350 foot[2]	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–1				CL	SANDY CLAY (CL) brown with red and brown gravel , stiff, moist, fine- to coarse-grained sand (FILL)						
3	S&H	4 5 4	5								
5–6	S&H	4 5 3	5		medium stiff					22.0	90
6.5					▼ (06/15/11, 5:40 p.m.)						
7				SP	SAND (SP) ▽ gray-brown, very loose, wet, fine- to coarse sand (06/15/11, 4:20 p.m.)						
9	S&H	3 2 2	2								
11	S&H	4 6 4	6	SP-SC	SAND with CLAY and GRAVEL (SP-SC) gray-brown, loose, wet				6.5	20.2	
12	SPT	5 5 6	11	SP	SAND (SP) gray, medium dense, wet, with rounded to subrounded gravel, fine- to coarse-grained sand						
14	SPT	1 2 8	10						3.3	16.8	
14.5					1-inch rounded gravel layer, loose						
15–16	SPT	5 11 11	22		TUFF brown, low hardness, friable, deeply weathered						
19–20	SPT	14 21 15	36		variegated color, gray, white, dark red, weak, moderately weathered						
23–24	SPT	9 12 13	25		light gray and light brown						
29–30	SPT	12 13 26	39		gray, yellow, black, and olive, weak, little to moderately weathered						

Boring terminated at a depth of 30 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 8 feet below ground surface during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.
[2] Elevation based on topographic map titled "Map of Topography of the Lands of Silver Rose Inn," by Albion Surveys, Inc., revison dated 12/15/10.

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Project No.: 730453902
Figure: A-11

PROJECT:	SILVER ROSE RESORT 400 SILVERADO TRAIL Calistoga, California	Log of Boring TR-12 PAGE 1 OF 1

Boring location: See Site Plan, Figure 2
Logged by: S. Magallon/S. Walker
Date started: 6/15/11 Date finished: 6/15/11
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches Hammer type: Hydraulic Trip
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

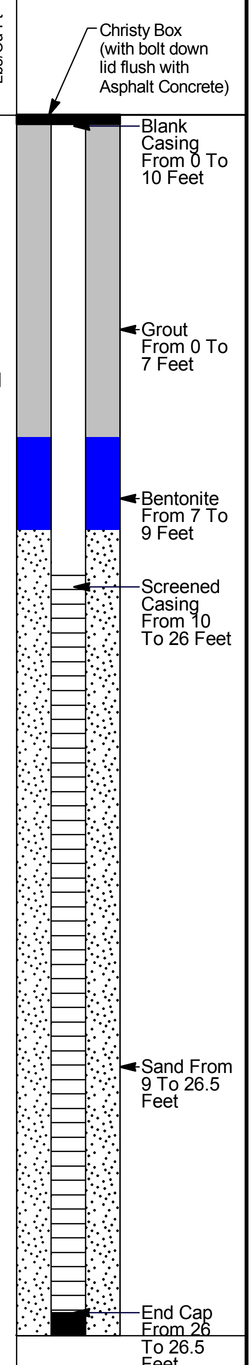
Ground Surface Elevation: ~349 foot[2]

DEPTH (feet)	Sampler Type	Blows/6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–4				CH	CLAY (CH) brown to dark brown, stiff, wet		
2–3.5	S&H	5, 7, 10	10		LL = 61, PI = 35		
4.5–7.5	S&H	5, 5, 4	5	CL	SANDY CLAY (CL) gray-brown, medium stiff, wet, with trace gravel ▽ (06/15/11, 6:20 p.m.)	21.3	101
7.5–13	S&H	2, 3, 3	4	SP-SC	SAND (SP) brown, loose, wet, fine- to coarse-grained sand		
10–11.5	S&H	2, 3, 3	4			12.1	
11.5–13	SPT	3, 2, 2	4				
13–26.5	SPT	7, 10, 11	21		TUFF variegated color, brown, tan, red and gray, low hardness, weak, deeply weathered matrix		
15–16.5	SPT	8, 8, 15	23				
18.5–20	SPT	8, 12, 16	28				
23.5–25	SPT	8, 12, 25	37				
25–26.5	SPT	15, 28, 29	57		little weathered		

FILL (0 to 13 feet)

LABORATORY TEST DATA: Type of Strength Test; Confining Pressure Lbs/Sq Ft; Shear Strength Lbs/Sq Ft; Fines %; Natural Moisture Content, %; Dry Density Lbs/Cu Ft

Piezometer Completion Detail



Boring terminated at a depth of 26.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 7 feet below ground surface during drilling.
LL = liquid limit, PI = plastic index

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.
[2] Elevation based on topographic map titled "Map of Topography of the Lands of Silver Rose Inn," by Albion Surveys, Inc., revison dated 12/15/10.

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Project No.: 730453902 Figure: A-12

PROJECT: **SILVER ROSE RESORT**
400 SILVERADO TRAIL
Calistoga, California

Log of Boring TR-13

Boring location:	See Site Plan, Figure 2
Date started:	6/15/11
Date finished:	6/15/11
Drilling method:	Solid Flight Auger, Portable Minute Man
Hammer weight/drop:	140 lbs./30 inches
Hammer type:	Rope & Cathead
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by:	S. Magallon

Ground Surface Elevation: ~349 foot[2]

DEPTH (feet)	Sampler Type	Blows/6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–5				CL	CLAY (CL) yellow-brown mottling yellow, very stiff, moist, trace fine-grained sand						
3–4.5	S&H	12, 18, 18	22		LL = 45, PI = 20						
5–8				SC	CLAYEY SAND (SC) gray-brown, medium dense, moist, with occasional coarse gravel						
6–7.5	S&H	14, 19, 20	23						22.0	14.0	
8–19					∇ TUFF variegated brown, yellow-brown, red, low hardness, weak to friable, moderately to deeply weathered, wet						
9–10	S&H	35, 50/6"	30/6"								
15–16.5	SPT	19, 18, 26	44								
18	SPT	50/6"	50/6"		weak, moderate to little weathered						

Boring terminated at a depth of 26.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 8.5 feet below ground surface during drilling.
LL = liquid limit, PI = plastic index

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively to account for sampler type and hammer energy.
[2] Elevation based on topographic map titled "Map of Topography of the Lands of Silver Rose Inn," by Albion Surveys, Inc., revison dated 12/15/10.

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Project No.: 730453902
Figure: A-13

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions		Symbols	Typical Names
Coarse-Grained Soils (more than half of soil > no. 200 sieve size)	Gravels (More than half of coarse fraction > no. 4 sieve size)	GW	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	Sands (More than half of coarse fraction < no. 4 sieve size)	SW	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
Fine -Grained Soils (more than half of soil < no. 200 sieve size)	Silts and Clays LL = < 50	ML	Inorganic silts and clayey silts of low plasticity, sandy silts, gravelly silts
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	Silts and Clays LL = > 50	MH	Inorganic silts of high plasticity
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic silts and clays of high plasticity
Highly Organic Soils		PT	Peat and other highly organic soils

GRAIN SIZE CHART

Classification	Range of Grain Sizes	
	U.S. Standard Sieve Size	Grain Size in Millimeters
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel	3" to No. 4	76.2 to 4.76
coarse	3" to 3/4"	76.2 to 19.1
fine	3/4" to No. 4	19.1 to 4.76
Sand	No. 4 to No. 200	4.76 to 0.075
coarse	No. 4 to No. 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	0.420 to 0.075
Silt and Clay	Below No. 200	Below 0.075

▽ Unstabilized groundwater level

▼ Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

Sample taken with Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter. Darkened area indicates soil recovered

Classification sample taken with Standard Penetration Test sampler

Undisturbed sample taken with thin-walled tube

Disturbed sample, hand auger

Sampling attempted with no recovery

Core sample

Analytical laboratory sample

Sample taken with Direct Push sampler

SAMPLER TYPE

C — Core barrel

CA — California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter

D&M — Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube

O — Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube

PT — Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube

S&H — Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter

SPT — Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.5-inch inside diameter

ST — Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

I FRACTURING

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

II HARDNESS

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

III STRENGTH

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

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

D. **Deeply Weathered** - moderate to complete mineral decomposition; extensive disintegration; deep and thorough discoloration; many fractures, all extensively coated or filled with oxides, carbonates and/or clay or silt.

M. **Moderately Weathered** - slight change or partial decomposition of minerals; little disintegration; cementation little to unaffected. Moderate to occasionally intense discoloration. Moderately coated fractures.

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

F. **Fresh** - unaffected by weathering agents. No disintegration of discoloration. Fractures usually less numerous than joints.

ADDITIONAL COMMENTS:

V CONSOLIDATION OF SEDIMENTARY ROCKS: usually determined from unweathered samples. Largely dependent on cementation.

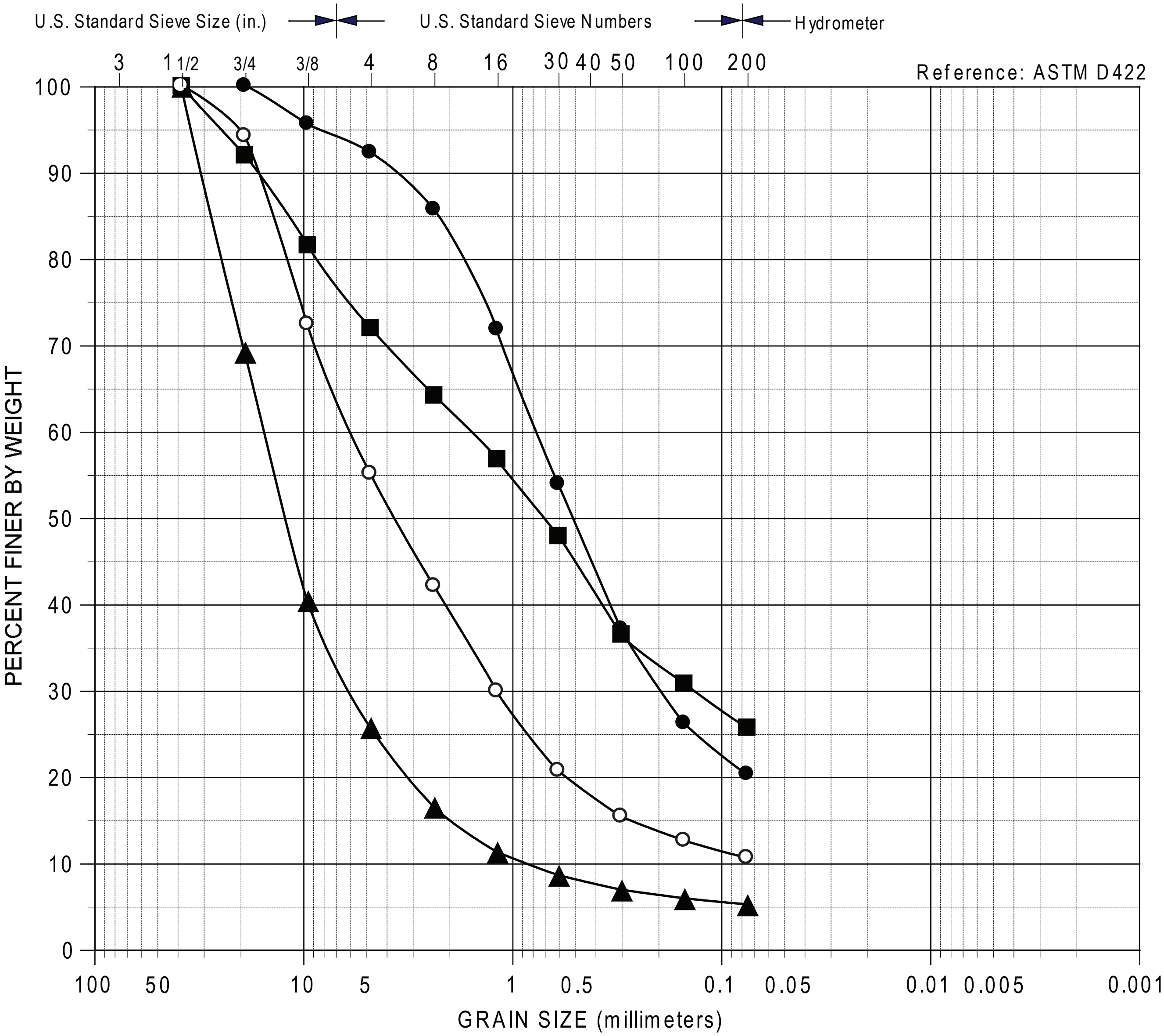
U = unconsolidated
P = poorly consolidated
M = moderately consolidated
W = well consolidated

VI BEDDING OF SEDIMENTARY ROCKS

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

VII DIP ANGLE REFERS TO THE ANGLE OF BEDDING (B) OR FRACTURING (F) OF THE ROCK

APPENDIX B
Results of Laboratory Testing



Sample	% Gravel		%Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay

Symbol	Sample Source	Classification
●	TR-2 at 11 feet	CLAYEY SAND (SC), yellow-brown and brown
■	TR-4 at 10.5-11 feet	CLAYEY SAND with GRAVEL (SC), yellow to yellow-brown with red and brown mottling
▲	TR-4 at 13.5 feet	SANDY GRAVEL with SILT (GP-GM), varigated red, brown and white
○	TR-5 at 8.5-9 feet	SANDY GRAVEL with CLAY (GP-GC), yellow-brown

SILVER ROSE RESORT
400 SIVERADO TRAIL
Calistoga, California

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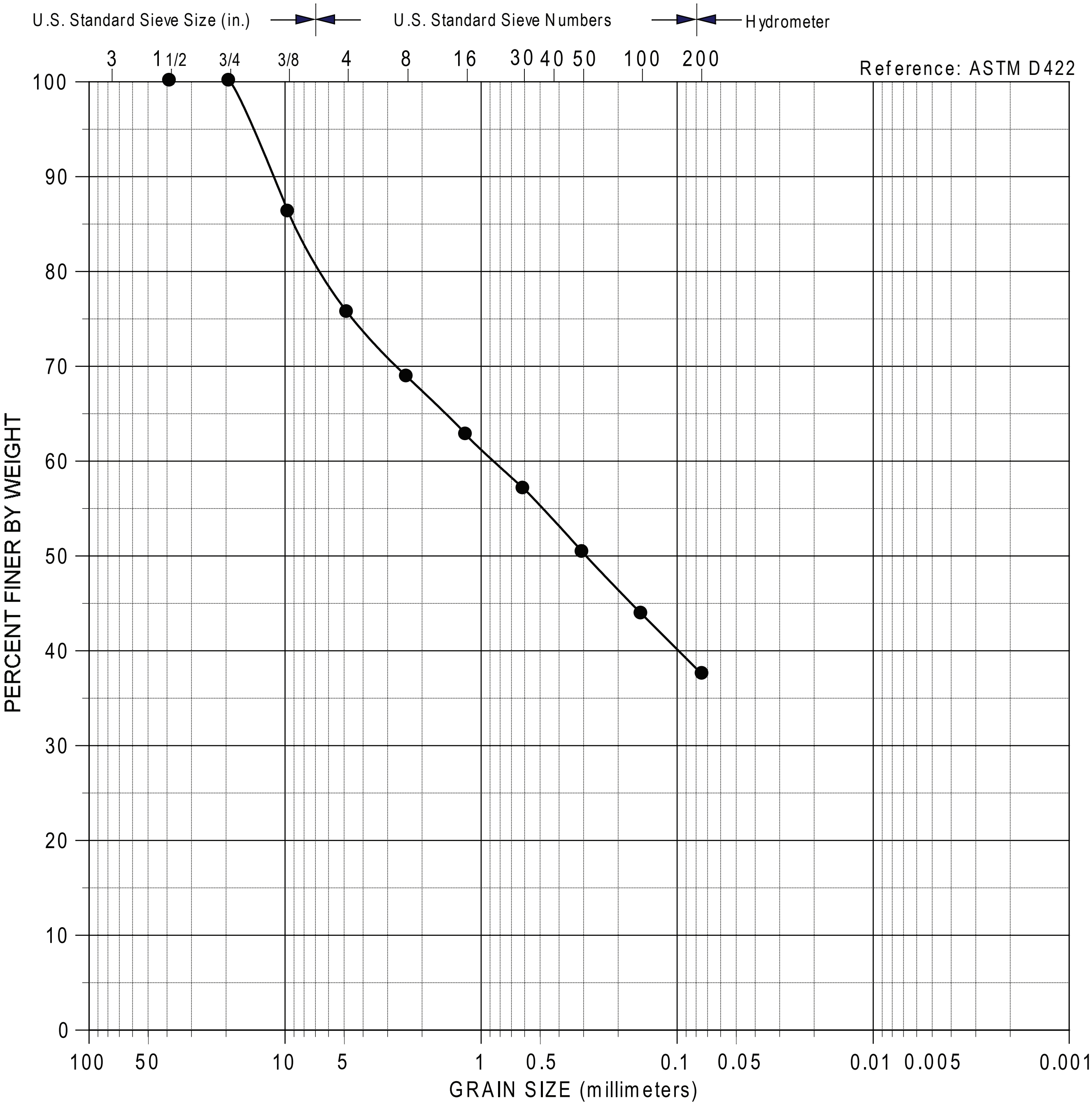
PARTICLE SIZE ANALYSIS

Date 07/12/11	Project No. 730453902	Figure B-1



% Gravel		%Sand			% Fines	
Coarse	Fine	Coarse	Medium	Fine	Silt	Clay

Symbol	Sample Source	Classification
●	TR-7 at 9-11.5 feet	CLAYEY SAND with GRAVEL (SC), yellow-brown

SILVER ROSE RESORT
400 SIVERADO TRAIL
Calistoga, California

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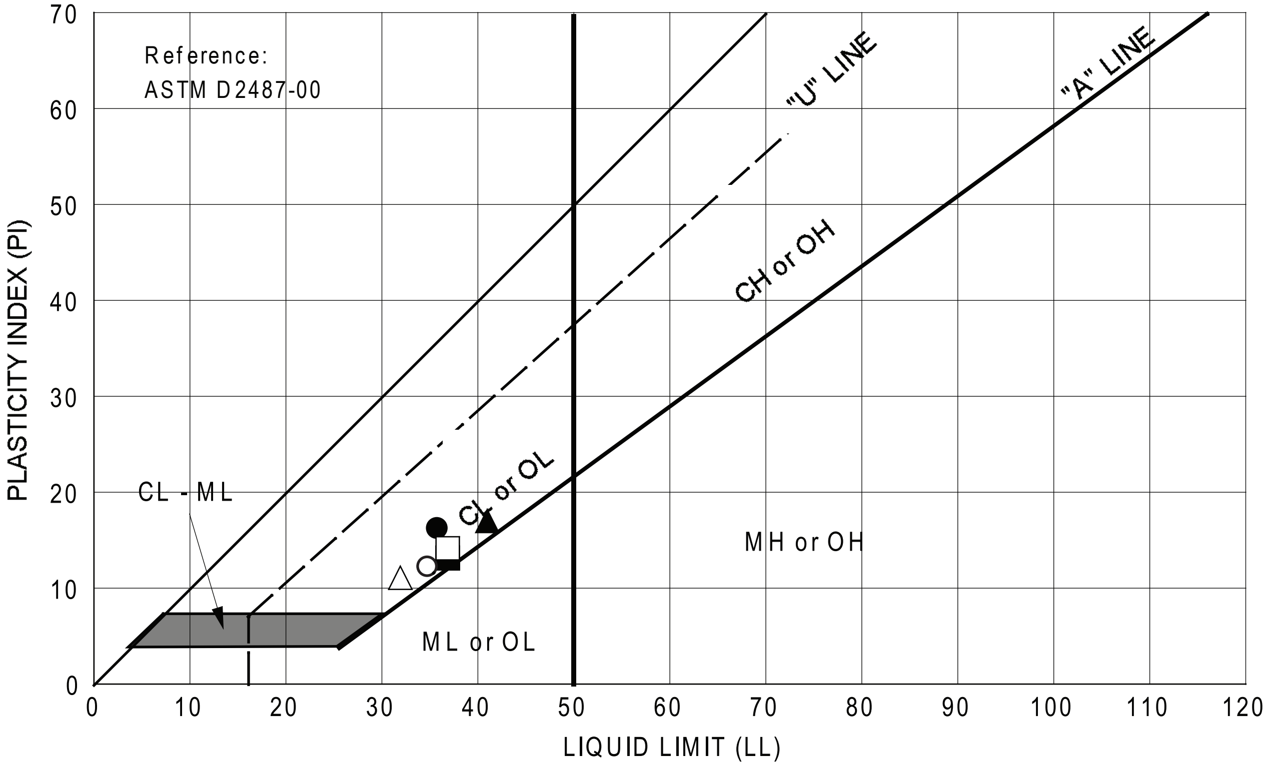
PARTICLE SIZE ANALYSIS

Date 07/11/11 | Project No. 730453902 | Figure B-2



Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	TR-1 at 6 feet	CLAYEY SAND (SC), yellow-brown	23.0	36	16	39.5
▲	TR-2 at 3 feet	CLAY (CL), brown	23.5	41	17	--
■	TR-4 at 3 feet	CLAYEY SAND with GRAVEL (SC), brown	18.8	34	13	24.0
○	TR-6 at 6 feet	SANDY CLAY with GRAVEL (CL), dark brown	21.7	35	12	--
△	TR-6 at 14.5 feet	TUFF, yellow-brown with gray and red-brown mottling	--	32	11	--
□	TR-7 at 6 feet	SILTY CLAY (CL), brown, dark brown to olive-brown	--	37	14	--

SILVER ROSE RESORT
400 SILVERADO TRAIL
Calistoga, California

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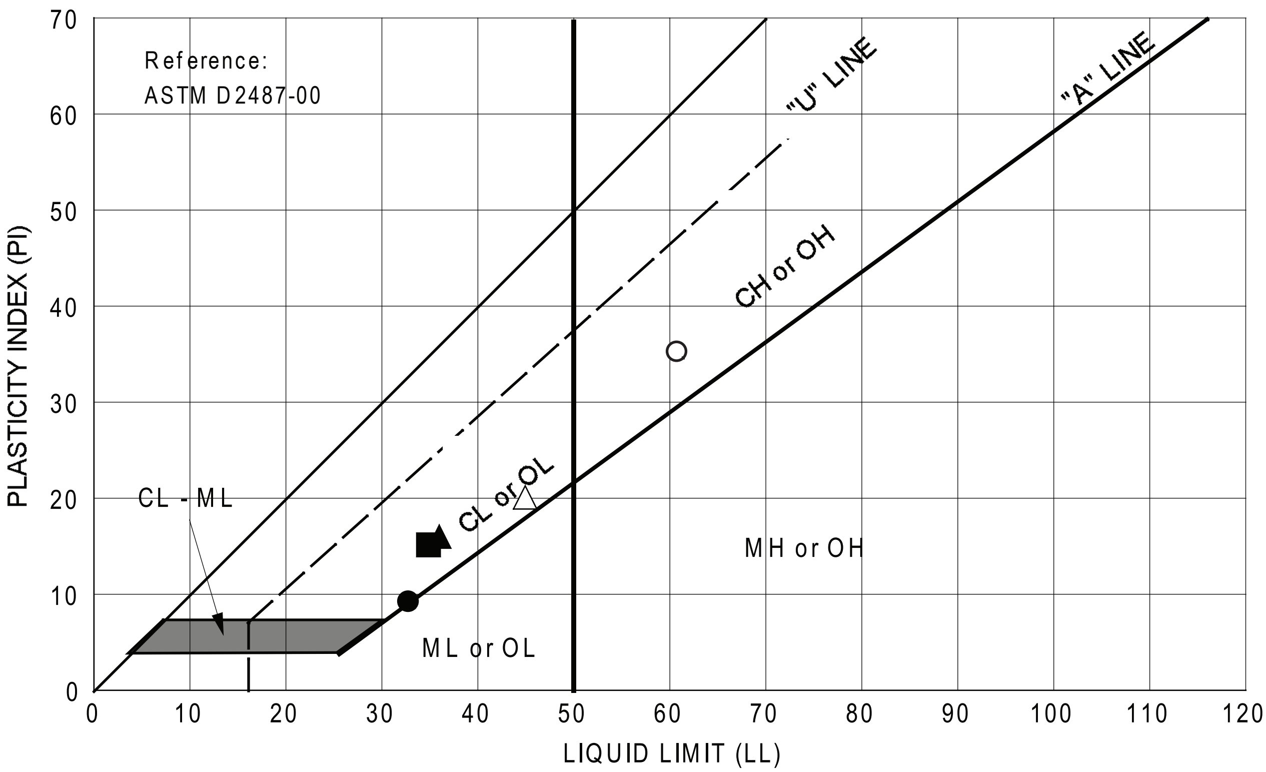
PLASTICITY CHART

Date 07/12/11 | Project No. 730453902 | Figure B-3



Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	TR-7 at 9-11.5 feet	CLAYEY SAND with GRAVEL (SC), yellow-brown	16.9	33	9	37.3
▲	TR-8 at 3 feet	CLAY (CL), brown	--	36	16	--
■	TR-9 at 4 feet	SANDY CLAY (CL), yellow-brown	18.8	35	15	--
○	TR-12 at 3 feet	CLAY (CH), brown to dark brown	--	61	35	--
△	TR-13 at 4 feet	CLAY (CL), yellow-brown mottling yellow	--	45	20	--

SILVER ROSE RESORT
400 SILVERADO TRAIL
Calistoga, California

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PLASTICITY CHART

Date 07/12/11 | Project No. 730453902 | Figure B-4

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APPENDIX C
Results of Corrosion Testing

E T S

Environmental Technical Services

975 Transport Way, Suite 2
Petaluma, CA 94954
(707) 778-9605/FAX 778-9612
e-mail: entech@pacbell.net

-Soil, Water & Air Testing & Monitoring
-Analytical Labs
-Technical Support

Serving people and the environment so that both benefit.

COMPANY:	Treadwell & Rollo, 501 14th Street, 3rd Floor, Oakland, CA 94612			ANALYST(S)	SUPERVISOR
ATTN:	Scott Walker		DATE of	D. Salinas	D. Jacobson
JOB SITE:	Silver Rose Resort, Calistoga, California	DATE RECEIVED	COMPLETION	S. Santos	LAB DIRECTOR
PROJ NUM #:	730453902	7/1/2011	7/8/2011		G.S. Conrad PhD

LAB SAMPLE NUMBER	SAMPLE ID	DESCRIPTION of SOIL and/or SEDIMENT	SOIL pH -log[H+]	NOMINAL RESISTIVITY ohm-cm	ELECTRICAL CONDUCTIVITY µmhos/cm	SULFATE SO4 ppm	CHLORIDE Cl ppm
04481-1	SRR1/C	TR-3-1 @ 3.0'	**4.57**	**1,790**	**[559]**	**24**	**48**
04481-2	SRR2/C	TR-11-1 @ 3.0'	**4.92**	**1,860**	**[538]**	**20**	**39**
Method	Detection	Limits --->	---	1	0.1	1	1

LAB SAMPLE NUMBER	SAMPLE ID	DESCRIPTION of SOIL and/or SEDIMENT	SALINITY ECe mmhos/cm	SOLUBLE SULFIDES (S=) ppm	SOLUBLE CYANIDES (CN=) ppm	REDOX mV	PERCENT MOISTURE %
Method	Detection	Limits --->	---	0.1	0.1	1	0.1

** COMMENTS **

Resistivities are in the 1,500 to 2,000 ohm-cm range which is mediocre; soil reactions (i.e., pHs) are strongly acidic which does not help; sulfates and chlorides are low. The standard CalTrans times to perforation are as follows: for SRR1 and 18 ga steel the time is just 7.4 yrs, and for 12 ga it goes up to ≈16 yrs; for SRR2 the respective times are just 8.7 yrs, and ≈19 yrs. For gray/ductile steel/iron the calculated average pitting rate for SRR1 is ≈0.33 mm/yr and for SRR2 is 0.32 mm/yr, so pitting to a 2 mm depth is at ≈6.1 yrs and ≈6.2 yrs, respectively; double the times for a 4 mm depth. Chloride levels are so low that this anion should not have any measurable corrosion impact on steel reinforcement; and sulfate is also low so it should not have any adverse impact on concrete, cements, mortars or grouts. These soils could benefit greatly from alkaline treatment (i.e., lime or cement) such that raising their pHs to the 7.5-8.5 range would increase the 18 ga times to perf to 31 yrs and 32 yrs, respectively; and the pitting rates would decline to 0.092 mm/yr and 0.091 mm/yr, respectively, putting the 2 mm depth times at >21 yrs and ≈22 yrs. Keep in mind that lime treatment does not last more than a few years in an unprotected setting, but has much greater longevity in protected environments (i.e., underneath slabs, buildings, etc.); cement treatment is often superior to lime treatment. Otherwise, to increase metals longevity in these soils and/or insure a specific longevity would require upgrading (e.g. increased gauge or more resistant steels, etc.); and/or cathodic protection along with coating or wrapping the steel could be utilized (the number and size of sacrificial anodes and impressed cathodic current could be significant); other alternatives include more engineering fill, or employing the use of plastic, fiberglass or concrete pipe, etc. Last, standard concrete mixes should be fine in these soils based on the results.

\\\\NOTES: Methods are from following sources: extractions by Cal Trans protocols as per Cal Test 417 (SO4), 422 (Cl), and 532/643 (pH & resistivity); &/or by ASTM Vol. 4.08 & ASTM Vol. 11.01 (=EPA Methods of Chemical Analysis, or Standard Methods); pH - ASTM G 51; Spec. Cond. - ASTM D 1125; resistivity - ASTM G 57; redox - Pt probe/ISE; sulfate - extraction Title 22, detection ASTM D 516 (=EPA 375.4); chloride - extraction Title 22, detection ASTM D 512 (=EPA 325.3); sulfides - extraction by Title 22, and detection EPA 376.2 (= SMEWW 4500-S D); cyanides - extraction by Title 22, and detection by ASTM D 4374 (=EPA 335.2).

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APPENDIX D
Ground Water Level Readings

Silver Rose Ground water readings taken by Silver Rose personnel.[1]								
Boring Name	**B-3**		**B-2**		**B-12**		**B-8**	
Silver Rose Designation	Well #1		Well #2		Well #3		Well #4	
Ground Surface Elevation (MSL)[2]	375 feet		372 feet		348 feet		353 feet	
Date	Measured Depth beneath Existing Ground Suface (feet)	Corresponding Groundwater Surface Elev. (feet, MSL)	Measured Depth beneath Existing Ground Suface (feet)	Corresponding Groundwater Surface Elev. (feet, MSL)	Measured Depth beneath Existing Ground Suface (feet)	Corresponding Groundwater Surface Elev. (feet, MSL)	Measured Depth beneath Existing Ground Suface (feet)	Corresponding Groundwater Surface Elev. (feet, MSL)
28-Mar-11	7.3	367.8	6.9	365.1				
4-Apr-11	10.7	364.3	8.7	363.3				
8-Apr-11	11.3	363.7	9.0	363.0				
11-Apr-11	12.2	362.8	10.3	361.8				
18-Apr-11	13.0	362.0	11.0	361.0				
25-Apr-11	13.5	361.5	11.8	360.3				
2-May-11	14.0	361.0	12.7	359.3				
9-May-11	14.3	360.8	13.3	358.8				
16-May-11	14.7	360.3	13.6	358.4				
23-May-11	14.3	360.7	14.0	358.0				
30-May-11	14.7	360.3	14.3	357.8				
6-Jun-11	14.3	360.7	14.7	357.3				
13-Jun-11	14.7	360.3	14.7	357.3				
20-Jun-11	15.3	359.7	15.3	356.8				
27-Jun-11	16.0	359.0	15.8	356.3	7.2	340.9	10.0	343.0
4-Jul-11	16.5	358.5	16.3	355.8	7.5	340.5	10.7	342.3
11-Jul-11	17.0	358.0	16.8	355.3	7.8	340.2	11.0	342.0
18-Jul-11	17.2	357.9	17.0	355.0	8.5	339.5	11.7	341.3
25-Jul-11	17.3	357.8	17.7	354.4	8.9	339.2	12.3	340.8
1-Aug-11	17.7	357.3	18.0	354.0	9.3	338.8	13.0	340.0
8-Aug-11	18.2	356.8	18.3	353.8	9.9	338.1	13.7	339.3
15-Aug-11	18.5	356.5	18.6	353.4	10.6	337.4	14.5	338.5
22-Aug-11	19.0	356.0	19.0	353.0	11.0	337.0	14.8	338.2
29-Aug-11	19.50	355.5	19.33	352.7	12.25	335.8	15.70	337.3
5-Sep-11	19.75	355.3	19.75	352.3	12.75	335.3	16.00	337.0
12-Sep-11	20.40	354.6	20.00	352.0	13.00	335.0	17.00	336.0
19-Sep-11	20.82	354.2	20.33	351.7	13.25	334.8	17.82	335.2
26-Sep-11	21.33	353.7	20.33	351.7	13.33	334.7	18.00	335.0
3-Oct-11	21.50	353.5	20.40	351.6	13.33	334.7	18.00	335.0
10-Oct-11	21.75	353.3	19.75	352.3	13.25	334.8	17.33	335.7
17-Oct-11	22.00	353.0	20.50	351.5	13.25	334.8	17.25	335.8
24-Oct-11	22.25	352.8	20.50	351.5	13.50	334.5	17.66	335.3

Notes:

[1] Groundwater Depths were obtained by Silver Rose personnel. Data received via electronic mail on 4 November 2011.

[1] Groundwater Elevations estimated from top of boring Elevations. These elevations were estimated using a the map titled "Map of Topography of the Lands of Silver Rose Inn" prepared by Albion Surveys, Inc. and dated 15 December 2010.

Appendix D

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