APPENDIX G-2

THIRD PARTY GEOTECHNICAL REVIEW

Gilpin Geosciences, Inc
Earthquake & Engineering Geology

March 29, 2013
Project 91456.01

Mr. Geoffrey A. Reilly
WRA
2169-G East Francisco Boulevard
San Rafael, California

Subject: Third Party Geotechnical/Geological Review
Berg Subdivision EIR Project
Upper Road
Ross, California

Dear Mr. Reilly:

This letter presents Gilpin Geosciences' (GGI) third party geotechnical and geological review comments for the proposed Berg Subdivision project on Upper Road in Ross, California. This review was performed in conjunction with services and technical review comments provided by Mr. Craig Shields of Rockridge Geotechnical, Inc. (RGI). The comments presented in this letter are provided to WRA for use during their preparation for the Environmental Impact Report (EIR).

SCOPE OF SERVICES

GGI's and RGI's scope of services included:

- reviewing available geotechnical and geologic information submitted by the project applicant;
- compiling and reviewing published and unpublished geologic and seismicity data for the site vicinity;

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- preliminary review of selected historical aerial photography of the site to identify features that may be associated with areas of slope instability, areas of fill, or other geologic conditions of concern;

- performing a site reconnaissance to review geologic mapping by applicant's consultants and observe site surface conditions for evidence of geotechnical and geologic hazards and unstable site conditions; and

- preparing written documents describing the results of the study, including a discussion of potential geotechnical and geological concerns, such as site seismicity, strong shaking from nearby earthquakes, lateral spreading, expansive

- During the preparation of this letter, we reviewed the following documents:

    - *Geotechnical Feasibility Evaluation Upper Road Land Division – Vesting Tentative Map Assessor's Parcel 073-011-26 Ross, California,* prepared by Herzog Geotechnical Consulting Engineers, dated 20 July 2012;

    - *Upper Road Land Division Vesting Tentative Map Project Report,* prepared for the Town of Ross by CSW/Stuber-Stroeh Engineering Group, dated May 2012;

    - *Administrative Draft Subsequent Environmental Impact Report Upper Road Land Division*, prepared by Donaldson Associates., dated 28 March 2006.

    - *Geologic Review and Update Proposed Monte Bello Subdivision Upper Road, Ross, California,* prepared by Phoenix Consultants, dated 16 November 2001;

    - *Geotechnical Report: Geological Hazards Investigation, Lot 3 Monte Bello Subdivision, Ross, California,* Herzog and Associates, dated 12 July 1993;

    - *Access Road Exploration Monte Bello Subdivision Ross, California*, prepared by Herzog and Associates, Inc., dated 9 August 1990;

- *Geotechnical Report Geological Hazards Investigation Lot 3 Monte Bello Subdivision Ross, California*, prepared by Herzog and Associates, Inc., dated 12 July 1990;
- *Geotechnical Investigation Proposed Property Subdivision Upper Road Ross, California*, prepared by Herzog Associates, Inc., dated 12 October 1989;
- *Geotechnical Investigation Driveway Fill 15 Upper Road Ross, California,* prepared by Herzog and Associates, Inc., dated 6 September 1983;
- *Mudd Properties Upper Road Ross, California*, prepared by Herzog and Associates, Inc., dated 10 August 1983;
- *Geotechnical Review Subdivision Feasibility Upper Road Ross, California,* prepared by Herzog And Associates, Inc., dated 4 May 1983;
- *Geotechnical Feasibility Investigation Mudd Property Ross, California,* prepared by Herzog and Associates, Inc., dated 8 October 1982; and
- *Conclusions Geotechnical Suitability 13 acre parcel*, prepared by Herzog and Associates, Inc., dated 5 October 1982.

During the history of site investigation the project geotechnical consultant has changed from Herzog Associates, Inc. to Phoenix Consultants in 2001. Presently Herzog Geotechnical Consultants are the project geotechnical engineer. In their 2012 letter, Herzog Geotechnical judged that the proposed project depicted on the 7 May 2012 Vesting Tentative Map submittal is feasible from a geotechnical standpoint. Pending a design-level geotechnical report for the present project, we are assuming for the purpose of this review that Herzog Geotechnical is accepting the general geotechnical, grading, and drainage recommendations contained in reports for previous project design layouts.

We have performed a review of the geologic and geotechnical aspects of this proposed project. Our recommendations for addressing the geologic hazards and geotechnical issues at the site are presented in this letter.

SITE CONDITIONS AND PROJECT DESCRIPTION

The proposed project will consist of subdividing an approximately 36-acre property into 3 residential lots. From the new project entrance at Upper Road, a 20-foot wide access way would extend about 992 feet connecting Upper Road to 12-foot wide driveways for Parcels 1, 2 and 3. The common road is shown climbing the ridge on Parcel 3 in a series of switchbacks that are graded in cuts. However, the profile requires stacked (up to 3 shown) 6-foot high concrete retaining walls above the road and in some sections up to 2 6-fot high retaining walls on the downslope side of the alignment. The curving entranceway would have a maximum slope of 18 percent compared to the 27 percent average slope of the existing topography at this location.

The project objectives of balancing cut and fill on-site and reducing road grades is proposed to be accomplished by taking the cut material from the road system and incorporating it into a single surplus fill pad on Parcel 1 supported by up to six stacked 6-foot high concrete retaining walls. The fill is graded with irregular contours which is intended to preserve the adjacent Redwood grove and swales. The result is that no material would be off-hauled by truck. Total cut and fill has been reduced 62.5 percent from 61,500 cubic yards (CY) in the prior design to 23,100 CY in the proposed project.

The eastern property boundary lies just upslope of the main trunk of Ross Creek which flows in a prominent northeast-trending canyon. Tributary drainages cross easterly across the site in two ravines Swan Swale, and Frog Swale, and empty into Ross Creek.

REGIONAL GEOLOGY & SEISMICITY

The site is located in the Coast Ranges geomorphic province that is characterized by northwest-southeast trending valleys and ridges. These are controlled by folds and faults that resulted from the collision of the Farallon and North American plates and subsequent shearing along the San Andreas fault. Bedrock in the region is primarily comprised of Upper Jurassic to Lower Cretaceous (~160-100 million years ago) Franciscan Complex rocks consisting of greenstone, sandstone, shale, chert, and localized limestone overlain by Quaternary alluvium, and colluvium (Wagner, 1991). The site vicinity has been mapped as Franciscan Complex interbedded shale and sandstone with areas of mélange (Rice and others, 1976; Blake and others, 2000). Landslide deposits are mapped blanketing most of the bedrock beneath the site.

Regional Seismicity

The major active faults in the area are the San Andreas, Rodgers Creek, Hayward Faults, and Concord/Green Valley, and West Napa. For each of the active faults, the distance from the site and estimated maximum Moment magnitude[1] (USGS, 2008) and Cao et al. (2003) are summarized in Table 1.

TABLE 1
Regional Faults and Seismicity

Fault Segment	Approx. Distance from fault (km)	Direction from Site	Mean Characteristic Moment Magnitude
San Andreas North Coast	11	West	7.5
Total Hayward	18	East	6.9
Total Hayward – Rodgers Creek	18	East	7.3
North Hayward	18	East	6.5
Rodgers Creek	19	Northeast	7.0
West Napa	37	Northeast	6.5
Concord/Green Valley	44	East	6.7
Mount Diablo	48	East	6.7

The 1906 San Francisco earthquake had an estimated Moment Magnitude (M_w) of 7.8 and created a surface rupture along the San Andreas fault approximately 290 miles long, with a maximum horizontal surface displacement of about 21 feet. The epicenter of the 1906 event is estimated to be offshore of the San Francisco coastline near the Golden Gate, southwest of the site. Strong shaking also occurred at many sites in the East Bay and extensive damage was documented.

[1] 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.

The recent Loma Prieta Earthquake (M_w 6.9) was centered on or near the San Andreas fault more than 70 miles from the site. It produced moderate ground shaking and minor damage in the San Rafael area.

The Rodgers Creek and Hayward faults form the main subsidiary faults making up the San Andreas Fault System in the East Bay and Northern San Francisco Bay Region. These faults lie approximately 18 km from the site and are capable of generating magnitude 7.0 to 7.3 earthquakes.

Two moderate earthquakes (Richter Magnitude 5.6 and 5.7) occurred on the Rodgers Creek fault near Santa Rosa in 1969. These earthquakes resulted in widespread minor damage and localized structural damage in Sonoma County but no significant damage in the San Rafael area.

The U.S. Geological Survey's (2008) 2007 Working Group on California Earthquake Probabilities has compiled the earthquake fault research for the San Francisco Bay area in order to estimate the probability of fault segment rupture. They have determined that the overall probability of moment magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Region during the next 30 years is 63 percent. The highest probabilities are assigned to the Hayward/Rodgers Creek and the Northern segment of the San Andreas faults. These probabilities are 31 and 21 percent, respectively (USGS, 2008).

SITE GEOLOGY

The site has been investigated by Herzog Associates (1982; 1989; 1990). Mapping was included in their various reports, Site Plan and Plate 1 Geology Map Toigo Property Subdivision Marin County, California (Herzog, 1989), Plate 1 Site Plan Lot 3 – Monte Bello Subdivision Ross, California (Herzog Associates, Inc., 1990); however, the site geology is most recently compiled in their 1989 report; the 1990 report supplements the subsurface exploration of one area of the site.

Gilpin Geosciences conducted a site field reconnaissance and mapping on 16 January 2013.

The bedrock within the site vicinity has been mapped as Franciscan Complex sandstone with shale and mélange. The mélange consists of blocks of sandstone, shale, and greenstone within a matrix of sheared shale (Rice and others, 1976; Blake et al., 2000). The entire site has been mapped as overlain by a massive complex landslide deposit (Rice and others, 1976; Wentworth and Frizzell, 1975).

Subsurface Conditions

Subsurface conditions at the site were previously explored by Herzog and Associates, Inc. by excavating a total of 60 test pits excavated at the approximate locations shown on the Site Plans and Geologic Maps in their reports as follows:

- Test Pits (TP- 1 to TP-21), October 8, 1982;
- Test pits (TP-1A to TP-17A), July 25, 1989;
- Supplemental Lot 3 Test Pits (S-1 to S-9), July 2, 1990; and,
- Test Pits (TP-B1 to TP- B13), August, 1990.

Subsurface exploration indicates that bedrock conditions (lithology and depth to rock) vary markedly throughout the site. The areas of proposed development are underlain primarily by sandstone and shale. The sandstone and shale are typically moderately strong, closely to intensely fractured, deeply to moderately weathered and non-expansive.

Mélange matrix encountered was weak, pervasively sheared, deeply weathered and weathers to expansive clay locally. Greenstone occurs locally, predominately in the form of large blocks up to 50 feet across within the mélange. Occasional hard and strong greenstone and graywacke blocks and inclusions were encountered along spur ridges and within drainages. In particular, there is a band of greenstone blocks (up to 40 feet high) that lines the head of the "Frog" Swale at an elevation of 350-400 feet, that we interpret as either a dike or more likely blocks within the mélange that have been exposed by weathering. Sandstone and shale bedrock was mapped and encountered upslope and downslope of this band of greenstone outcrops. In addition, smaller (car-size) greenstone blocks lie downslope from this outcrop within the Frog Swale drainage but appear to have been detached from this outcrop and have slid downslope. Due to subsequent erosion, these blocks now lie on shallow dipping slopes and do not appear to be a rockfall or landslide hazard.

The depth to competent bedrock varies throughout the site from 1 to over 13 feet below the existing ground surface (Herzog, 1982, 1989). Benched areas on the site slopes commonly correspond to changes in bedrock composition.

Deep colluvial soils of stiff sandy silts, sandy clays, and clayey sands are common throughout the site. Generally, these soils appeared well consolidated, only slightly compressible, and non-expansive. There was no testing to back up the soil plasticity index; only visual identification, the sandstone and shale frequently weather to expansive soil (Herzog, 1982).

Areas of extensive landslide debris were mapped in combination with colluvium as unit “Qsc”, predominately along slopes lining the eastern "Swan" Swale. These soils varied from dry and stiff clayey gravels, to wet and soft expansive sandy clays (Herzog, 1982).

Other landslide deposits appeared to be relatively old dormant features (Herzog, 1982). There is an area of active sliding downslope and to the south and west of the abandoned "cabin" about 65 feet downslope of the proposed road alignment. A fresh scarp and debris slide of approximately 4- 6 feet deep extends downslope with the toe encroaching Swan Swale.

There does not appear to be any significant active slope failures within the areas of proposed development or in the areas upslope from the property that could be attributed to the intense winter storms of 1981-1982 (Herzog, 1982).

Groundwater was not encountered in any of the test pits. However, shallow subsurface groundwater seepage was encountered during our site reconnaissance in the central-northern part of the site approximately 200 feet (horizontal) west of, and 100 feet upslope of Swan Swale in a small tributary.

3.2 Site History

Standard aerial photograph review techniques were employed to identify slope stability features at the site, such as tonal contrasts, vegetation patterns, and abrupt changes in topographic slope. The following discussion provides a limited chronology of site development based on the maps and photographs. Photos reviewed date from 1946 to 2005.

The earliest available photographs, dated 1946, showed two small scars where a small structure "cabin" and landscaped area were present on the ridge along the eastern property line where there currently lies an abandoned structure ("cabin") that has access from a sharp bend in Upper Road. Upper Road exists and a few large houses are visible to the east of the property. The site is heavily vegetated and there is no evidence of landsliding, scars or downed trees that could be attributed to slope instability. The site appears unchanged in photos from 1958 to 2005, other than development of houses to the east. Small landslides and minor erosion are not visible due to the extensive tree canopy.

GEOLOGIC AND SEISMIC HAZARDS

Potential geologic and seismic hazards at the project site include strong ground shaking, landslides, and rock falls. These hazards are discussed in the following sections.

Strong Ground Shaking

The 2006 Administrative Draft EIR indicates the San Andreas fault is the controlling fault in terms of future ground shaking estimates. Probabilistic seismic hazard analysis from the State of California Geological Survey estimates a peak horizontal ground acceleration at the site having a 10 percent probability of exceedance in 50 years to be 0.483g (CGS, 2005). We concur with this assessment of the potential for very strong shaking at the subject site.

Seismic Hazards

During a major earthquake on one of the active or potentially active nearby faults, strong to very strong ground shaking is expected to occur at the project site. Strong shaking can result in ground failures, such as those associated with soil liquefaction[2], lateral spreading[3], post-liquefaction reconsolidation[4], and cyclic soil densification[5], and seismic slope instability.

Based on a review of boring and test pit logs, and site conditions, we concur with 2006 EIR conclusion that the potential for liquefaction at the site is nil to very low because the materials necessary for the liquefaction condition do not exist at the site.

Based on our review and experience, soil liquefaction could result in limited localized ground failures, such as lateral spreading where proximity to steep-sided stream banks could result in localized failures. To mitigate the potential for adverse impacts associated with lateral spreading, Herzog has set back the building envelope areas from the edges of the steep-sided stream banks.

Cyclic Densification

The Herzog Associates reports discusses the potential for non-uniform compaction of soil strata and concludes either: 1) the subsurface soil and bedrock are stiff to hard, or 2) where softer colluvial or landslide deposits exist at proposed development areas, these materials will be over-excavated and recompacted during site grading. Therefore, Herzog Associates concludes the potential for differential ground movement is low. We concur with Herzog Associates' conclusions.

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[2] 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.

[3] 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.

[4] Post-liquefaction reconsolidation is a phenomenon in which a previously liquefied sand layer settles into a denser soil arrangement after dissipation of pore water pressures.

[5] Cyclic soil densification is a phenomenon in which non-saturated, cohesionless soil is densified by earthquake vibrations, resulting in ground surface settlement.

Seismic Design

Herzog Associates has not provided parameters for design in accordance with the California Building Code (CBC) seismic code. Seismic parameters should be provided for the CBC being used at the time of permit application.

Landslides and Slope Stability

The document titled *Special Publication 117, Guidelines for Evaluating and Mitigating Seismic Hazards in California (SP117)*, prepared by the California Geological Survey, adopted 13 March 1997 and revised on 11 September 2008, presents screening and analysis methodology for evaluating the seismic hazard, including hazards associated with landslides, rockfalls, and proposed embankments. As discussed in SP117, an on-site engineering geologic mapping should be performed to document surface conditions which, in turn, provide a basis for projecting subsurface conditions that may be relevant to the stability of the site. The on-site engineering geologic mapping should identify, classify, and locate on a map the features and characteristics of existing landslides, and surficial and bedrock geologic materials.

We reviewed the subsurface exploration logs and methodology used by Herzog and Associates to evaluate the proposed construction and stability of the new cut and fill slopes.

We do not concur with the assessment summarized in the 2006 EIR on Table 9: "Interpreted Geologic Conditions". Where early test pits (1982) encountered unknown depths of colluvium of landslide material, later test pit exploration (1989; 1990) explored to depths where bedrock was encountered. Although the bedrock was described in places as weak or sheared, these represent typical conditions for Franciscan Complex rock that juxtapose both sandstone/shale and mélange units.

As part of the recommendations presented in their 1989 report:

> "New fills should be stable when keyed into suitable bedrock materials, drained, and compacted. The construction of stable fills will require the

> excavation of keyway into rock or approved stiff or dense soils, placement of subdrains (as necessary), and reconstruction of slope with compacted materials. The main consideration with respect to the construction of engineered fills is the depth of keying required to mitigate potential creep or fill failure. Although fills of only a few feet may be proposed, in some instances the presence of underlying weak material may require deep keyways."

Removal of surficial soils, colluvium, and landslide debris during grading and subsequent appropriate construction of keyways involves observation during grading by an engineering geologist to specify the depth and width of proposed keyways in competent bedrock material of suitable strength to support the proposed buttress fill slope.

It is noted that expansive soils mapped as colluvium and old slide debris (Qsc) are not suitable as access road fill material. Herzog Geotechnical, the project geotechnical engineer, should comment on whether construction materials of moderately to highly expansive clay and bedrock should be used as fill for embankment and foundation construction. We did not find any record of laboratory soil testing from the site in the reports we reviewed. An appropriate spectrum of soil tests, including tests to measure the shear strength and expansion potential, of the various earth materials, should be performed on the site areas proposed for improvements.

The proposed project includes stacked 6-foot high retaining walls on both the common driveway and to retain the surplus fill pad on Parcel 1. Since closely spaced retaining walls need to support the lateral loads of the total stacked height, the geotechnical designs may need to accommodate lateral loads similar to single 18- to 36- feet high retaining walls. Because of the weak colluvium and landslide deposits, steep slopes, and limited construction area with tight limits of grading imposed by the site constraints, we recommend that the "stacked retaining wall" design be analyzed for slope stability, preferably using laboratory test results from appropriate subsurface samples.

COMMENTS, CONCLUSIONS, AND RECOMMENDATIONS

Based on the results of the third party review of the geotechnical and geologic issues associated with the Upper Road Land Division project in the Town of Ross, GGI and RGI have the following comments and recommendations for issues that should be addressed by Herzog Geotechnical and/or the project applicant:

Slopes and Subsurface Drainages

The Herzog Associates report (1989) recommends maximum fill slope inclinations of 2:1 (horizontal to vertical) and maximum cut slope inclinations of 1-1/2:1. Herzog Associates recommends that their engineering geologist observe all cut slopes to check the exposed soil or bedrock and determine whether any localized adverse material or bedding exists. If adverse slope conditions are encountered, it may be necessary to decrease the inclination of the cut slope.

We generally concur with Herzog Associates recommendations for slopes and fill drainages, but also have the following comments: 1) highly expansive soil should not be used in unreinforced fill slopes inclined at 2:1, 2) cut slopes in highly expansive soil should not be inclined at 2:1, 3) because of the steep slopes, weak colluvium and landslide deposits, and the constrains on any construction operations (grading limits) the slope stability should be analyzed for the "stacked concrete retaining walls" proposed for the common driveway and surplus fill pad on Parcel 1, 4) "cleanouts" should be used at the ends of the subsurface drains to facilitate the long-term maintenance of the system, 5) Herzog Associates' engineering geologist should help evaluate and select appropriate drain outlet locations to reduce the potential for slope erosion and instability, and 6) a homeowner's association maintenance and monitoring program should be established to ensure that the subsurface drainage systems are operating properly; the homeowner's association should make repairs to the fill slopes and embankments, if necessary.

Drainage

The Herzog Associates report provides recommendations for surface drainage. In general, surface water should not be allowed to flow over the top of slopes or down engineered slope faces by appropriate grading or construction of ditches. Ditches should be provided behind tops of all retaining walls and all subdrain and retaining wall backdrain outlets should discharge into either erosion-resistant rip rap areas within the creeks or swales, or into lined ditches that tie into the storm drain system.

Also, Herzog Associates recommends that subsurface drains consist of perforated piping and permeable gravel or drain rock. If drain rock is used, the rock and pipe should be entirely enclosed with a permeable geotextile fabric. Subdrains should be installed where seepage is observed. Subdrains may also need to be installed at the toe of any proposed cut slopes depending on the actual condition observed during construction.

Herzog Associates does not discuss surface drainage near foundations. Positive drainage should be provided within five feet of buildings to direct surface water away from foundations and slabs towards suitable discharge facilities. Roof gutters should be used on all buildings. Roof downspouts should be connected to solid pipes that transmit storm water onto paved roadways, into drainage inlets, or into storm drains. Landscaping drainage inlets should be provided around the proposed foundations that adequately collect irrigation or rain water and direct the water onto pavement or into storm water systems.

Erosion Control

Herzog Associates recommends permanent erosion control measures be placed on all slopes. As a minimum, all slopes should be hydroseeded. Based on the results of the design-level investigation, more aggressive permanent erosion control measures will be evaluated to minimize surface runoff velocities and erosion potential.

We concur with Herzog Associates' erosion control recommendations.

Building Setbacks

CSW/Stuber Stroeh has presented building envelopes on the Vesting Tentative Map. We recommend that these minimum building setbacks be established adjacent to the top or toe of new slopes in accordance with the current CBC to reduce the potential for seismic slope deformation, lateral fill extension, and/or slope creep from impacting the structures.

Single-Family Homes

Herzog Geotechnical (1989) preliminarily recommends new residences constructed on level pads with bedrock exposed be supported on minimum footing dimensions in accordance with the 2001 CBC. Herzog Associates recommends new footings be preliminarily designed for maximum allowable bearing pressures on the order of 2,000, 3,000 and 4,000 pounds per square foot (psf) for dead loads, dead plus code live loads, and total loads (including wind and seismic), respectively.

Herzog Associates (1989) recommends that the portion of spread footing foundations extending into rock and at least 7 horizontal feet from the face of the nearest slope may impose a passive equivalent fluid pressure and a friction factor of 350 pcf and 0.40 respectively, to resist sliding.

The Herzog Associates report indicates that drilled piers should preliminarily be designed with at least four No. 5 bars and should be tied together with grade beams. The portion of the piers extending into undisturbed rock should be designed using an allowable skin friction of 800 pounds per square foot (psf). The portion of the piers in compacted fill or dense/stiff soil beneath colluvium may be designed using an allowable skin friction of 600 psf. End bearing should be neglected because of difficulty of cleaning out small diameter pier holes, and the uncertainty of mobilizing end bearing and skin friction simultaneously. Lateral loads on piers will be resisted by passive pressures in the fill and rock. Equivalent fluid pressures of 350 pcf for rock and 250 pcf for compacted fill or stiff soil, acting over two pier diameters, should be used. The stability of the system should be calculated using a minimum factor of safety of 1.5.

We concur with Herzog Associates' preliminary foundation recommendations for the new residences.

Concrete Slabs-on-Grade

Herzog Associates does not recommend either conventional slab-on-grade or raised wood floors for the project. However, they anticipate that the buildings will be supported on continuous and interconnected spread footings.

We believe a concrete slab-on-grade floor is feasible for this project; however, if a slab-on-grade floor is used, we suggest a capillary break and water vapor retarder system be placed beneath slab-on-grade floors to reduce the potential for moisture migration through the slab and that expansive soils are replaced with 24 inches of nonexpansive select fill beneath the slab areas.

Pavements

Herzog Associates provided preliminary asphalt concrete pavement recommendations for driveways, 2.5 inches of asphalt and 6 inches of Class II AB, and for access roads, 3 inches of asphalt with 8 inches of Class II AB. They recommend that the actual R-value of subgrade soils be established after rough grading, and the pavement design modified as necessary. We concur with Herzog Associates preliminary pavement design recommendations.

Fill Placement and Compaction

Herzog Associates preliminarily recommends that general and select engineered fill be placed in eight inch loose lifts and compacted to at least 90 percent relative compaction in accordance with ASTM D1557. Parking and driveway subgrade should be compacted to at least 95 percent relative compaction.

The preliminary recommendations for moisture-conditioning and compaction of fill are not adequate to address the variable earth materials and fill thicknesses for the proposed development. Where fill is thicker than 5 feet, moisture-conditioning and compaction of the fill must minimize the potential for hydrocompression of the fill as the fill becomes wet from irrigation and rainfall. In addition, if highly expansive soil is used in fills, the moisture-conditioning and compaction of the fill must minimize potential for volume changes in the fill as the moisture content changes over time. The moisture-conditioning and compaction of thicker fills may vary depending on both the thickness of the fill and the plasticity of the fill material. The final geotechnical investigation should include sufficient laboratory testing to develop recommendations to address potential for soil volume changes resulting from both hydrocompression of all fill materials and shrink/swell of expansive soil.

In conclusion, GGI and RGI recommend that Herzog Geotechnical consider the comments and recommendations presented above and provide a response or acknowledgement that the comments presented above will be addressed during the final design of the project.

We appreciate the opportunity to assist you with the evaluation of geotechnical and geological issues for this project. If you have any questions or require additional information, please call.

Sincerely yours,
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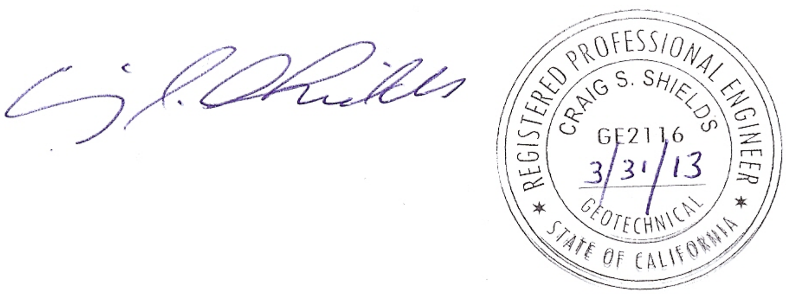

Craig S. Shields
Geotechnical Engineer

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AERIAL PHOTOGRAPHS

Photo	**No.**		**Scale**	**Date**
KAV9010	17	3	1:10000	03-06-05
KAV9010	18	1	1:10000	03-06-05
KAV9010	19	2	1:10000	03-06-05
AV 6540	122	65	1:12000	05-03-00
AV 6540	123	62	1:12000	05-03-00
KAVP6087	21	2	1:12000	10-23-98
AV 4890	16	50	1:12000	08-14-95
AV 4890	17	53	1:12000	08-14-95
AV 4252	27	77	1:12000	04-27-92
AV 4252	328	7	1:12000	04-27-92

AV 3766	9	28	1:12000	03-15-90
AV 3766	8	29	1:12000	03-15-90
AV 2860	10	18	1:12000	04-19-86
AV 2140	3	24	1:12000	05-03-82
AV 1840	3	28	1:12000	04-01-80
AV 1187	4	23	1:12000	04-17-75
AV 1187	3	25	1:12000	04-17-75
AV 957	4	24	1:12000	07-02-70
AV 957	3	25	1:12000	07-02-70
SF-AREA	1	6	1:36000	03-01-58
AV 9	2	1	1:23600	09-06-46