GEOTECHNICAL ENGINEERING REPORT
for
North Base Lodge
Homewood Mountain Resort
Homewood/Placer County, California

Prepared for:
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Project No. 41278-03
January 21, 2010

HOLDREGE & KULL
CONSULTING ENGINEERS • GEOLOGISTS

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JMA Ventures, LLC
11025 Pioneer Trail, Suite 100B
Truckee, California 96161

Attention: Mr. Joseph Walsh

Reference: ***North Base Lodge***
Homewood Mountain Resort
Homewood/Placer County, California

Subject: ***Geotechnical Engineering Report***

This report presents the results of our geotechnical engineering investigation for the proposed Homewood Mountain Resort North Base Lodge to be constructed at Homewood in Placer County, California. The proposed project will involve redevelopment and expansion of the existing Homewood base area (North Base Lodge) including a mixed use of residential and commercial development, a hotel lodge and skier services building, residential condominiums, a parking structure, and workforce housing. Appurtenant construction will include an attached gondola, surface parking, an ice rink, landscaping, and drainage improvements.

Based on our findings, our professional opinion is that the site is suitable for the proposed development using conventional earthwork grading and foundation construction techniques. Subsurface conditions vary substantially across the site. Due to the presence of very dense soil and rock underlying the proposed lodge structure, rock excavation methods may be required for foundation construction. The proposed parking structure and Buildings C, D, and E will be founded on lacustrine sediments (lake deposits). Specific recommendations regarding the geotechnical aspects of project design and construction are presented in the following report.

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Please contact us if you have any questions regarding this report or if we can be of additional service.

Sincerely,
Holdrege & Kull

Prepared By:	Reviewed By:
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1. INTRODUCTION

This report presents the results of our design level geotechnical engineering investigation for the proposed North Base Area to be constructed at Homewood Mountain Resort in Homewood/Placer County, California. We performed our investigation in general accordance with our December 8, 2008 proposal for the project. For your review, Appendix A contains a document prepared by ASFE entitled *Important Information About Your Geotechnical Engineering Report.* This document summarizes the general limitations, responsibilities, and use of geotechnical engineering reports.

1.1 Purpose

The purpose of our investigation was to explore and evaluate the subsurface conditions at the project site, and to provide our geotechnical engineering recommendations for project design and construction.

Our findings are based on our subsurface exploration, laboratory test results, and our experience in the project area. We recommend retaining our firm to provide construction monitoring services during earthwork and foundation excavation to observe subsurface conditions encountered with respect to our recommendations.

1.2 Scope of Services

To prepare this report we performed the following scope of services:

- Review of our previous report prepared for the site, titled, “Preliminary Geotechnical Engineering Report, for Homewood Mountain resort, North Base Area, Homewood/Placer County, California”, dated September 9, 2009.

- Completion of a subsurface investigation by drilling, logging and collecting relatively undisturbed soil samples from four exploratory borings drilled with a truck-mounted drill rig. The subsurface investigation also included excavating, logging, and collecting bulk soil samples for laboratory analyses from eleven test pits and advancing nine cone penetrometers (CPTs) across the eastern portion of the site.

- Laboratory tests on selected soil samples obtained during our subsurface investigation to help evaluate material properties.

- Engineering analyses to develop design level geotechnical engineering recommendations for project design and construction; and,

- Preparation of this report.

1.3 Site Description

The project site consists of approximately 18 acres of developed land, as shown on Figure 1, Site Vicinity Map. The site is located on the west shore of Lake Tahoe on Highway 89 about 5 miles south of Tahoe City. In general, the site consists of paved and unpaved parking lots surrounding a lodge used for ski resort purposes. Preliminary land development plans indicate that the proposed project will involve removal of an existing two-story wood frame lodge and replacement with several mixed used residential and commercial structures.

The project site is bounded by residential properties to the north, existing Homewood ski runs to the west, a mixture of commercial and residential properties to the south, and Highway 89 South (West Lake Boulevard) to the east. Lake Tahoe is approximately 500 feet east of the site. A site vicinity map is shown on Figure 1.

The subject site is located in a portion of Section 1, Township 14 North, and Range 16 East (1992 edition of the Homewood California 7.5-minute quadrangle map published by the United States Geological Survey (USGS). The North Base Lodge area elevations range from approximately 6,330 feet above mean sea level (MSL) near the northwest corner of the project area to approximately 6,237 feet MSL along the east edge of the proposed development area. Surface water drainage consists of overland flow in a general west to east direction. The western portion of the site slopes moderately to gently down from west to east. The eastern portion of the site is relatively level. Vegetation consists of a few large trees, grass and brushes on the ski run, and willow bushes at the south property boundary.

1.4 Proposed Improvements

Information about the proposed project was obtained from our site visits, conversations with the project structural engineer, Levon Nishkian of Nishkian Menninger, staff members of JMA Ventures, LLC, and preliminary project plans prepared by HKS Hill Glazier Studio, Design Workshop, and Nichols Consulting Engineers, Ltd. Overall, the Homewood Mountain Resort (HMR) project will involve redevelopment and expansion of the existing Homewood base area (North Base Area), the Tahoe Ski Bowl base area (South Base Area), and the mid-mountain facilities (Mid-Mountain Area). This report focuses on the North Base Area.

The North Base Area project consists of removal of the existing two-story wood frame lodge and replacement with several mixed use residential and commercial structures. The project will include construction of residential condominiums, fractional ownership units, for-sale condominium/hotel units, and up to 25,000 square feet of commercial floor space and workforce housing units. The North Base redevelopment will include a main lodge at the base of the ski runs, a parking structure south of Fawn Street, and three separate buildings (Buildings C, D, and E) in the eastern portion of the site. A new base mountain facility will replace the existing day skier services structure.

We understand that the new lodge will be a 6- to 7-story, 350,000 square foot structure. The lower one- to-two stories will likely be cast-in-place concrete with steel frame above. The proposed 16 residential condominiums and approximately 40 fractional ownership units will be separate one-, two-, and three-story structures.

Approximately 810 parking spaces are planned, including approximately 300 day-use spaces in a three-to four-level parking structure with commercial floor space and workforce housing around the perimeter of the structure. Approximately 60 surface parking spaces will be constructed at the retail and skier drop off area, and approximately 450 underground parking spaces directly below the building footprint of the new hotel and ski services facility.

Estimated vertical structural loads for the new hotel/lodge structure are expected to be on the order of 1,200 to 1,500 kips at isolated columns and 7 kips per linear foot along continuous wall foundations for long-term loading. We understand that the earthwork cuts and fills for building construction may be on the order of approximately 20 feet. Post-tensioned concrete slabs are being considered for the floors. We anticipate the new hotel/lodge structure will be supported on a spread foundation system.

We anticipate that the parking structure will be supported on a rigid mat foundation system. Estimated vertical structural loads for the proposed workforce housing/parking structure are expected to be on the order of 600 to 700 kips at isolated columns for long-term loading conditions.

The separate residential buildings in the east portion of the project area (Buildings C, D, and E) will be constructed near existing site grade. The lower levels will include parking. We expect the structures will be wood and/or steel frame supported on a rigid mat foundation.

Appurtenant construction will include a gondola, asphalt concrete paved driveways and roadways, a large stone pavement plaza and ice rink area, site retaining structures, storm drainage facilities, landscaping, and underground utilities. The proposed development is shown on Figure 2.

2. GEOLOGY SETTING

2.1 Site Geology

We reviewed available geologic and soil literature to help evaluate geologic, seismic, and subsurface conditions at the project site. The *Geologic Map of the Lake Tahoe Basin, California and Nevada,* by G.J. Saucedo, California Geological Survey, 2005 shows the east portion of the project site is underlain by Holocene age lake deposits. The western portion of the project area is underlain by volcanic andesite rock of Miocene age (approximately 23.7 to 5.3 million years before the present). Much of the western portion of the site is underlain by shore zone deposits from previous higher stands of Lake Tahoe. The shore zone deposits consist of sand to coarse gravel and cobbles. The lake deposits consist of interbedded silt, sand and gravel.

2.2 Regional Faulting

The project is located in a potentially active seismic area. To evaluate the location of mapped faults relative to the project site, we reviewed the following maps:

- *Fault Activity Map of California and Adjacent Areas;* by Charles W. Jennings, California Department of Conservation, Division of Mines and Geology, 1994.

- *Geologic Map of the Chico Quadrangle, California,* by G.J. Saucedo and D.L. Wagner, California Division of Mines and Geology, 1992.

- *Geologic Map of the Lake Tahoe Basin, California and Nevada,* by G.J. Saucedo, California Geological Survey, 2005.

- *New Constraints on Deformation, Slip Rate, and Timing of the Most Recent Earthquake on the West Tahoe – Dollar Point Fault, Lake Tahoe Basin, California,* by Daniel S. Brothers, et. al., Bulletin of the Seismological Society of America, April 2009.

The potential risk of fault rupture is based on the concept of recency and recurrence. The more recently a particular fault has ruptured, the more likely it will rupture again. The California State Mining and Geology Board define an “active fault” as one that has had surface displacement within the past 11,000 years (Holocene). Potentially active faults are defined as those that have ruptured between 11,000 and 1.6 million years before the present (Quaternary). Faults are generally considered inactive if there is no evidence of displacement during the Quaternary.

The referenced geologic maps show several active and potentially active faults located near the project site, including the Dog Valley Fault (active, approximately 20 miles north-northwest), a group of unnamed faults southeast of Truckee (potentially active, approximately 15 miles north), the West Tahoe – Dollar Point fault (active, approximately 3 miles east), and the North Tahoe Fault (active, approximately 6 miles northeast). The Genoa Fault trends in a north-south direction approximately 18 miles east of the site and is capable of very large earthquakes. Earthquakes associated with these faults may cause strong ground shaking and secondary hazards such as landslides and/or rock fall at the project site.

An unnamed, discontinuous, fault is shown on the Geologic Map of the Lake Tahoe Basin (Saucedo, 2005) that trends in a northwest direction near the base of the slope through the project area. This fault is relatively short, about one mile long, and is shown as approximately located (dashed) and uncertain as to existence (queried) on the Saucedo (2005) map. In addition, this fault is not shown on the Chico Quadrangle Map (Saucedo and Wagner, 1992).

The potential hazard associated with earthquake faults involves surface rupture and strong ground motion. The unnamed fault discussed above is discontinuous and questionable as to presence and location. Therefore, the hazard from surface rupture on this unnamed fault is considered low. The hazard associated with strong ground motion is dependent on the location and magnitude of the source earthquake, which is related to the size of the fault (length and height). The mapped unnamed fault is one mile long and is not capable of producing large earthquakes. Earthquakes on regional faults in the area, such as the West Tahoe fault or Genoa fault, would likely result in higher ground motion at the site than earthquakes on the unnamed fault inferred to trend approximately 200 feet west of the North Base Lodge. It is our professional opinion that building set back distances from the unnamed inferred fault are not warranted and no further study is necessary.

We reviewed the 1997 version of Special Publication 42, Fault Rupture Hazard Zones in California, which describes active faults and fault zones (activity within 11,00 years), as part of the Alquist-Priolo Earthquake Fault Zoning Act. The document and 1999 on-line update indicate that the site is not located within an Alquist-Priolo active fault zone.

3. SUBSURFACE EXPLORATION

On October 2, 2009, eleven test pits (TP-1 through TP-11) were excavated across the western portion of the site to depths ranging from approximately 7 to 18 feet below the ground surface (bgs). The test pits were excavated with a John Deere 200C track-mounted excavator equipped with a 24-inch bucket. An engineer from our firm logged the soil conditions exposed in the test pits, visually classified soil, and collected bulk soil samples for laboratory testing. Soil samples were packaged and sealed in the field to reduce moisture loss and were returned to our laboratory for testing. Upon completion, the test pits were backfilled with the excavated soil.

As previously stated, Holdrege & Kull prepared a preliminary report for this project. The scope of work for that report included drilling nine borings to depths of approximately 27 to 60 feet below the ground surface. The borings were drilled on January 13 through 15, 2009, with a truck-mounted CME 75 drill rig equipped with 6-inch outside and 3.25-inch inside diameter hollow stem augers and 4.5-inch diameter continuous core pipe with a diamond core bit.

In addition to the nine previous borings, the subsurface conditions beneath the eastern portion of the site were investigated on October 6 and 7, 2009, by drilling four more exploratory borings (B-10 through B-13) to depths ranging from approximately 23.1 to 50 feet bgs. The borings were drilled with a truck-mounted Mobile B-61 drill rig equipped with 6-inch outside diameter hollow stem augers and 4.5-inch diameter mud rotary continuous pipe with a rock bit. A geologist from our firm logged the soil conditions exposed in drill cuttings and samples, visually classified soil, and collected relatively undisturbed soil samples for laboratory testing. Soil samples were retrieved by driving a split-spoon Modified California sampler lined with 2-inch diameter stainless steel liners with a 140 pound automatic hammer falling 30 inches. Blow counts were recorded for the bottom 12 inches of an 18-inch drive of the sampler. The samples were sealed with plastic end caps in the field to reduce moisture loss and labeled for laboratory testing. Upon completion, the borings were backfilled with soil cuttings and capped with neat cement.

On October 16, 2009, nine cone penetrometers (CPT-1 through CPT-9) were advanced across the eastern portion of the site to depths ranging from approximately 14.5 to 18 feet bgs. The cone penetrometers (CPTs) were advanced with a truck-mounted direct push rig equipped with 1.5-inch rods. Each test was performed by driving a cylindrical penetrometer with a conical tip (cone) into the ground at a constant rate. Each CPT run met refusal at the depth indicated. Forces between the friction sleeve and cone were measured with a Hogentogler data acquisition system and software and recorded on logs included in Appendix B. Upon completion, the CPTs were backfilled with neat cement grout to the ground surface. The approximate locations of our test pits, borings, and CPTs are shown on Figure 2.

Shear wave velocity measurements were performed using the SeisOpt ReMi Refraction Microtremor method within the upper 110 to 120 feet in the eastern portion of the site on November 22, 2009. The measurements were performed along two northwest-southeast trending seismic lines (Line 1 and 2) located in the northern and southern portion of the main parking lot, as shown on Figure 2.

3.1 Subsurface Soil Conditions

The following are summaries of the subsurface conditions encountered in our test pits and borings drilled during our previous preliminary and design level investigation. More detailed descriptions of the subsurface conditions observed are presented in our Test Pit, Boring, and CPT Logs in Appendix B.

3.1.1 North Base Main Lodge – West Area

The western portion of the North Base Main Lodge is underlain by a relatively thin layer (1.5 to 3 feet thick) of silty Sand (SM) soil overlying very dense sand and gravel, and volcanic lahar rock. This portion of the site is generally underlain by shore line deposits consisting of sand and gravel with cobbles and boulders.

Test Pits TP-1 and TP-2 encountered refusal on volcanic lahar rock at depths of 16 and 11 feet bgs, respectively. In addition, we encountered refusal on volcanic lahar rock in Boring B-13 at a depth of approximately 23.1 feet bgs. The volcanic lahar rock was highly weathered, moderately fractured, and weak to moderately strong. The remaining test pits (except Test Pit TP-3) encountered refusal on strongly cemented cobbles and boulders at depths ranging from approximately 7 to 18 feet bgs. We encountered refusal on very dense, moderately cemented granular soil at a depth of 8 feet bgs in Test Pit TP-3. Loose to medium dense fill was encountered in Test Pits TP-7 and TP-10 to depths of 3 and 10 feet, respectively.

3.1.2 North Base Main Lodge – East Area

Borings B-1 through B-3 and B-6 through B-9 were drilled in the eastern portion of the main lodge area during our preliminary investigation. Boring B-10 was also drilled in the eastern portion of the main lodge area as part of our recent investigation. In general, the existing asphalt pavement section through the eastern portion of the site consists of approximately 2 to 6 inches of asphalt concrete (AC) overlying 1 to 3 inches of aggregate base (AB).

The eastern portion of the North Base Main Lodge is underlain by a relatively thin layer of fill (1.5 to 3 feet thick) that overlies medium dense to dense, poorly graded, saturated sand with varying amounts of silt, gravel and cobbles (SM, SP, and GP). Refusal on strong to very strong volcanic lahar rock was encountered in Borings B-1, B-2, B-6, and B-10 at depths of approximately 22, 27, 39.5, and 46.2 feet bgs, respectively. The volcanic lahar rock was similar to that encountered in our test pits excavated in the western portions of the site.

3.1.3 Proposed Parking Structure

Borings B-4 and B-5 (preliminary investigation) and B-11 and B-12 (recent investigation) were drilled in the unpaved overflow parking area at the site. In general, the subsurface soil encountered in our borings consisted of soft to hard fine-grained soil types to depth of approximately 12 feet bgs. Underlying the fine-grained soil, the soil generally consisted of medium dense to very dense granular soil types with varying amounts of silt (SP, SW-SM, and GW-GM) to the maximum depth explored of 50 feet bgs. Boring B-11 encountered refusal on slightly weathered, widely fractured, strong to very strong volcanic lahar rock at a depth of approximately 25.5 feet bgs.

Based on the results of our shear wave velocity measurements performed in the upper 110 to 120 feet in the eastern portions of the site, it appears that the site is underlain by three principal soil layers, each with characteristic shear wave velocities. The depth and relative location of the soil layers evident from the shear wave velocity measurements appear to correspond to the soil layers encountered in our borings. The shear wave profiles depicting the soil profiles described above are presented in Appendix D.

3.2 Groundwater Conditions

We observed groundwater in our borings drilled during the subsurface exploration at depths ranging from approximately 10 to 18 feet bgs. Fluctuations in soil moisture content and groundwater levels should be anticipated depending on precipitation, irrigation, runoff conditions and other factors. Based on our experience in the project area, seasonal saturation of near-surface soil should be anticipated, especially during and immediately after seasonal snowmelt.

4. LABORATORY TESTING

We performed laboratory tests on bulk soil samples collected from our exploratory test pits and relatively undisturbed samples collected from our borings to help evaluate their engineering properties. The following laboratory tests were performed:

- Atterberg Limits/Plasticity (ASTM Test Method D4318)
- Sieve Analysis (ASTM D422)
- Moisture-Density (ASTM D2216 and D2937)
- Unconfined Compression (ASTM D2166)

- Consolidation (ASTM D2435)
- Corrosion Potential (pH, resistivity, sulfates, and chlorides)

Sieve analysis, Atterberg Limits, and Moisture-Density data resulted in USCS classifications that varied from coarse to fine-grained soil, as summarized in Table 3.1, below.

Unconfined compression strength test results of a soil sample collected from Boring B-12 at a depth of 9 feet indicated that the soil has an unconfined compressive strength of 1,412 pounds per square foot (psf). Consolidation tests were performed on samples collected in Borings B-10 and B-11 at depths of 7.5 and 3 feet bgs, respectively. The results of consolidation test indicated that soil at a depth of approximately 7.5 feet bgs has a slight compressibility in the area of Boring B-10 (south central portion of main parking lot). Soil at a depth of about 3 feet bgs in the northwest corner of the proposed parking structure (current unpaved overflow parking lot) has a moderate compressibility.

The results of corrosivity testing of soil samples collected from Borings B-10, B-11, and B-13 indicated negligible potential for sulfate attack on concrete. Therefore, use of Type II cement is acceptable. The resistivity results indicated a very low potential (6,000 ohm-cm and higher) of corrosion of metal exposed to native soils. More specific soil classification and laboratory test data is included in Appendix C. In addition, laboratory test results of soil samples collected from our previous borings (Borings 1 through 9) are also included in Appendix C. USCS classification and Atterberg indices are summarized in Table 4.1 below.

Table 4.1 – Summary of Laboratory Test Results				
Test Pit/Boring Number	Depth (feet)	USCS Classification	Liquid Limit	Plastic Limit
TP-4	2.5	Well-Graded Gravel with Sand (GW)	--	--
TP-4	5	Poorly Graded Gravel with Sand (GP)	--	--
TP-5	5	Well-Graded Sand with Silt and Gravel (SW-SM)	--	--
B-10	3	Sandy Lean Clay (CL)	45	27
B-10	8	Silty Sand with Gravel (SM)	--	--
B-10	19	Well-Graded Sand with Silt and Gravel (SW-SM)	--	--
B-10	24.3	Poorly Graded Sand with Silt (SP-SM)	--	--
B-10	29	Well-Graded Gravel with Silt and Sand (GW-GM)	--	--
B-10	33	Poorly Graded Sand with Gravel (SP)	--	--
B-11	7.5	Clayey Sand (SC)	35	23
B-11	13	Well-Graded Sand with Silt and Gravel (SW-SM)	--	--
B-12	3.5	Sandy Silt with Gravel (ML)	--	--
B-12	4	Silty Clay (CL-ML)	--	--
B-12	8.5	Silty Sand with Gravel (SM)	--	--
B-12	9	Sandy Silt with Gravel (ML)	--	--
B-12	14	Silty Sand with Gravel (SM)	--	--
B-12	18.5	Silty Sand with Gravel (SM)	--	--
B-12	29	Silty Gravel with Sand (GM)	--	--
B-12	38.5	Silty Sand with Gravel (SM)	--	--

5. SECONDARY SEISMIC HAZARDS

Secondary seismic hazards include lateral spreading, seismically induced slope instability, rock fall, and liquefaction. Lateral spreading is the lateral movement toward a free face of soil resulting from liquefaction of subadjacent materials. Based on the subsurface information available to date, and the results of our analysis, site soil located at depths of approximately 15 to 50 feet below the existing site grade could undergo significant strength loss due to liquefaction. Surface manifestations of liquefaction (and resulting strength loss) could involve subsidence and/or lateral spreading of the ground surface and partial bearing failure (resulting in excessive settlement) of structures supported on shallow foundations.

Slope instability includes landslides, debris flows, and rock fall. No recent landslides, debris flows or rock fall hazards were observed in the site area. Due to the granular and rocky nature of the proposed site and general surrounding area, the potential for slope instability is considered low.

Liquefaction is a phenomenon whereby loose, saturated, granular soil deposits lose shear strength due to excess pore water pressure buildup resulting from cyclic loading, such as that caused by an earthquake. Among other effects, liquefaction can result in lateral spreading and/or densification of such deposits (and hence settlement of overlying structures) after an earthquake as excess pore water pressures are dissipated. The primary factors affecting liquefaction potential of a soil deposit are level and duration of seismic ground motions, soil type and consistency, and depth to groundwater. Based on the results of our subsurface investigation, medium to very dense sand and gravel underlies the eastern portion of the site. Groundwater was encountered at approximately 15 feet below the ground surface during our subsurface investigation in October 2009. Based on information provided by Kleinfelder (2007), we expect groundwater to seasonally rise.

Data obtained from our exploratory borings, CPT probes, and shear wave velocity measurements were utilized to evaluate the liquefaction potential of saturated sand and gravel in the eastern and southern portions of the site. We used methods of analyses presented in "*Soil Liquefaction during Earthquakes*", by I.M. Idriss and R.W. Bouanger, Earthquake Engineering Research Institute, 2008. The CPTs met refusal on dense gravel and cobble layers at depths ranging from approximately 14 to 18 feet bgs. The blow counts obtained from the saturated sand indicate that the sand is medium dense to very dense. In addition, several medium dense to dense gravel layers were encountered in the saturated sand. Soil densities measured in the CPTs indicated that the upper 16 to 18 feet of soil is medium dense/medium stiff to dense/stiff. This soil profile will have a low potential for liquefaction.

For an alluvial site, we obtained a peak ground acceleration (PGA) of 0.316g from the California Geologic Survey Probabilistic Seismic Hazards Mapping Ground Motion website. Using this PGA and the blow counts obtained from our subsurface investigation, we calculated a minimum factor of safety against liquefaction of 1.15. If liquefaction were to occur, the potential hazard at the site would be ground settlement in the area of the proposed parking structure and Buildings C, D, and E. Based on recommended factors of safety obtained from *"Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Liquefaction Hazards in California",* Southern California Earthquake Center, 1999, this is an acceptable factor of safety where differential settlement is the hazard of concern. To mitigate the potential for differential settlement at the site, the proposed parking structure and Building C, D, and E, will be supported on mat foundations. Due to the thickness and relative competency of near-surface soil at the site, no surface manifestation of underlying potentially liquefiable soils would be expected at the site.

In addition, we measured shear wave velocities of subsurface soil along two lines located in the eastern portion of the site using the ReMi refraction micrometer method. The shear wave velocity measurements obtained during this investigation indicate that the saturated sand has a shear wave velocity on the order of 942 to 945 feet per second (ft/sec). Based on shear wave velocity and liquefaction correlation with the saturated sand, as presented in Idriss and Boulanger, 2008, Page 116, the soil has a low potential for liquefaction.

6. CONCLUSIONS

The following conclusions are based on our subsurface investigation, laboratory test results, and engineering analysis. These conclusions may change if additional information becomes available.

1. Subsurface conditions underlying the site vary from dense cemented sand and gravel in the sloping western portion of the site, to stiff silt and lean clay, overlying medium dense sand and gravel in the south and eastern portions of the site. Soil conditions encountered in the area of the proposed lodge during our field investigation generally consisted of medium dense to very dense granular soil of low plasticity that should provide suitable foundation support for the proposed structure on conventional spread foundations. The Lodge site has a negligible potential for liquefaction.

2. Subsurface conditions in the area of the proposed parking structure and residential buildings (Buildings C, D, and E), generally consisted of medium stiff to very stiff fine-grained soil with low expansion potential. The fine-grained soil types are underlain by medium dense to very dense coarse-grained sand and gravel that has a low potential for liquefaction. Based on consolidation tests of fine-grained soil samples collected at depths of 3 and 7.5 feet bgs, the soil is slightly to moderately compressible. The fine-grained soil in this area should provide adequate support for a rigid mat foundation.

3. In general, medium dense to very dense soil types were encountered in our test pits excavated in the western, sloping portions of the proposed lodge site. The John Deere 200C large excavator used during our field investigation encountered refusal on volcanic rock at depths of about 7 to 11 feet bgs. Based on our understanding of the project as currently proposed, cuts on the order of approximately 20 feet are planned within the Lodge foundation. Cuts extending beyond about 11 feet in this area may be difficult due to near surface rock and

cemented gravel. A significant amount of boulders and over-sized material should be anticipated in excavations in the western portion of the site. With the exception of the organic surface soil, site soil is generally suitable for reuse as structural fill; however, processing to remove oversized material will likely be necessary. The near-surface fine-grained soil encountered in the southern portion of the site will not suitable for reuse as engineered fill that will support structures.

4. Groundwater was not encountered in our test pits excavated during this investigation. However, groundwater was encountered in our borings at depths ranging from approximately 10 to 15 feet bgs. We anticipate that near-surface soil layers will become seasonally saturated and groundwater levels will rise. It is possible that groundwater seepage will be encountered during excavations for the lodge structure. Near-surface groundwater should be anticipated during the spring.

5. Existing fill soil was encountered in the area of the northeast corner of the proposed lodge structure. Due to the potential for excessive settlement, existing fill should not be used for foundation support. Foundation recommendations provided in this report are based on the assumption that all existing fill that would otherwise remain under future foundations will be removed prior to foundation construction.

6. The North Base Lodge will be located at the base of a moderately steep slope. Due to the relative competency of the underlying rock at the site, the potential for seismically induced slope instability is low. Similar to most locations in mountainous terrain, seismically induced rock fall is a potential hazard at the site. However, no rock outcrops are located on the slope above the site. The potential hazard from seismically induced rock fall at the site is low.

7. RECOMMENDATIONS

The following geotechnical engineering recommendations are based on our understanding of the project as currently proposed, our field observations, the results of our laboratory tests, engineering analysis, and our experience in the project area.

7.1 Grading

The following sections present our recommendations for site clearing and grubbing, preparation for and placement of fill material, temporary excavation and cut/fill slope grading, erosion control measures, utility trench construction, construction dewatering, surface water drainage, plan review, and construction monitoring.

7.1.1 Clearing and Grubbing

Areas proposed for fill placement, road and access drive construction, and building areas should be cleared and grubbed of vegetation and other deleterious materials. Existing vegetation, organic topsoil, and any debris should be stripped and hauled offsite or stockpiled outside the construction limits. We expect that 4 inches may be used as a reasonable estimate for average depth of stripping. Organic surface soil may be stockpiled for future use in landscape areas, but is not suitable for use as structural fill. The actual depth of stripping will vary across the site and may be greater in wooded areas.

Although not encountered during our field investigation, it is possible abandoned utility lines, septic or storage tanks, wells, and/or foundations may exist on site. If encountered within the area of construction, these items should be removed and disposed of off-site; existing wells should be abandoned in accordance with applicable regulatory requirements. Existing utility pipelines which extend beyond the limits of the proposed construction and will be abandoned in-place, should be plugged with cement grout to prevent migration of soil and/or water. All excavations resulting from removal activities should be cleaned of loose or disturbed material (including all previously placed backfill) and dish-shaped to permit access for compaction equipment.

All existing fill should be removed in areas that will support foundations and/or pavements. Based on our subsurface explorations completed to date, the depth of existing fill ranges from 3 to 4 feet across the eastern portion of the site. We encountered fill in Test Pit TP-10 (northwest portion of site) that generally consisted of silty sand with gravel and boulders, extending to a depth of approximately 10 feet. The existing fill should either be replaced with compacted structural fill or improvements may be founded directly on properly prepared underlying native soil. The existing fill material will likely be suitable for re-use as engineered fill material provided any debris exceeding 8 inches maximum dimension and all organic or deleterious material are removed and disposed off-site. Preparation of the subgrade exposed by overexcavation and requirements for engineered fill should be in accordance with recommendations provided below.

All rocks greater than 8 inches in greatest dimension (oversized rock) should be removed from the top 12 inches of soil, if encountered. Oversized rock may be used in landscape areas, rock faced slopes, or removed from the site. Oversized rock should not be placed in fill without prior approval by the project geotechnical engineer.

7.1.2 Preparation for Fill Placement

Prior to fill placement, all areas of existing fill material, man-made debris, or backfill soil should be removed to expose firm native soil as discussed in the previous section.

Where fill placement is planned, the near-surface soil should be scarified to a depth of about 8 inches below existing ground surface or to competent material and then uniformly moisture conditioned to within 2 percent of the ASTM D1557 optimum moisture content. Areas to receive fill should be compacted with appropriate compaction equipment to at least 90 percent of the maximum dry density per ASTM D1557, and proof rolled with a loaded, tandem-axle truck under the observation of a representative of Holdrege & Kull. Any areas that exhibit pumping or rutting should be over-excavated and replaced with compacted fill placed according to the recommendations below.

7.1.3 Fill Placement

Material used for fill construction should consist of uncontaminated, predominantly granular, non-expansive native soil or approved import soil. Engineered fill should consist of granular material, nearly free of organic debris, with liquid limit of less than 40, a plasticity index less than 15, 100 percent passing the 8-inch sieve, and less than 30 percent passing the No. 200 sieve. With the exception of the silty soil in the south parking structure area encountered in Borings B-4 and B-5, we anticipate that on-site soil may be used in a fill provided all oversized material is removed prior to placement and compaction. Rock used in fill should be broken into fragments no larger than 8 inches in diameter. Rocks larger than 8 inches are considered oversized material and should be stockpiled for offhaul, later use in rock faced slopes, or placement in landscape areas. The silt soil encountered in the area of the parking garage will likely be difficult to uniformly moisture condition to near optimum moisture content and compact. We recommend that this soil not be reused for structural fill.

Imported fill material should be predominantly granular, non-expansive, and free of deleterious or organic material. Import material that is proposed for use onsite should be submitted to Holdrege & Kull for approval and laboratory analysis at least 72 hours prior to import.

If site grading is performed during periods of wet weather, near-surface site soil may be significantly above optimum moisture content. These conditions could hamper equipment maneuverability and efforts to compact fill materials to the recommended compaction criteria. Fill material may require drying to facilitate placement and compaction, particularly during or following the wet season or spring snowmelt. Suitable compaction results may be difficult to obtain without processing the soil (e.g., discing during favorable weather, covering stockpiles during periods of precipitation, etc.).

Fill should be uniformly moisture conditioned to within 2 percent of optimum moisture content and placed in maximum 8-inch thick, loose lifts (layers) prior to compacting. Fill should be compacted to at least of 90 percent of the maximum dry density per ASTM D1557. The upper 8 inches of fill in paved areas should be compacted to at least 95 percent of the maximum dry density per ASTM D1557. In addition, for fills where the total depth will exceed 10 feet, we recommend that the bottom ½ of the total fill depth be compacted to at least 95 percent relative compaction. Moisture content, dry density, and relative compaction of fill should be evaluated by our firm at regular intervals during fill placement. The earthwork contractor should assist our representative by preparing test pads with the onsite earth moving equipment.

Fill material with more than 30 percent rock larger than ¾-inch is not testable using conventional compaction testing equipment. We recommend that a procedural approach, or method specification, be used for quality assurance during rock fill placement rather than a specified relative compaction. The procedural requirements will depend on the equipment used, as well as the nature of the fill material, and will need to be determined by the geotechnical engineer on site. Based on our experience in the area, we anticipate that the procedural specification will require a minimum of six passes with a Cat 563 or similar, self-propelled vibratory compactor to compact a maximum 8-inch thick loose lift. Processing or screening of the fill may be required to remove rocks larger than 8-inches in maximum dimension. Continuous observation by a representative of Holdrege & Kull will be required during fill placement to confirm that procedural specifications have been met.

Differential fill depths beneath the structures should not exceed 5 feet. For example, if the maximum fill depth is 8 feet across a building pad, the minimum fill depth beneath that pad should not be less than 3 feet. If a cut-fill building pad were used in this example, the cut portion would need to be over-excavated 3 feet and rebuilt with compacted fill.

7.1.4 Cut/Fill Slope Grading

Site soil is generally anticipated to be granular material. Permanent cut and fill slopes at the subject site should be stable at inclinations of 2H:1V or flatter provided they are protected against water erosion. Steeper slopes may be possible at the site provided slopes are protected from excessive erosion. Recommendations for cut/fill slopes steeper than 2H:1V may be provided by request. Mid slope benches should be provided at a maximum vertical spacing of approximately 30 feet.

Fill should be placed in horizontal lifts to the lines and grades shown on the project plans. Slopes should be constructed by overbuilding the slope face and then cutting it back to the design slope gradient. Fill slopes should not be constructed or extended horizontally by placing soil on an existing slope face and/or compacted by track walking.

Equipment width keyways and benches should be provided where fill is placed on side-slopes with gradients steeper than 5H:1V. Benching must extend through loose surface soil into suitable material, and be performed at intervals such that no loose soil is left beneath the fill. Holdrege & Kull should observe keyways and benches prior to fill placement.

The upper two to five feet of cut slopes should be rounded into the existing terrain above the slope to remove loose material and produce a contoured transition from cut face to natural ground. Scaling to remove unstable cobbles and boulders may be necessary. Fill slopes should be compacted as recommended for the placement of engineered fill. The upper 4 to 8 inches may be scarified to help promote revegetation.

7.1.5 Temporary Unconfined Excavations

Based on our understanding of the proposed project, temporary unconfined excavations will be necessary for this project. The following criteria may be used for construction of temporary cut slopes excavated for the proposed structures.

Temporary Slope Inclination (Horizontal to Vertical)	**Maximum Height (Feet)**
0:1	5
0.5:1	12
0.75:1	20
Specific Design	>20

These temporary requirements may require modifications in the field after construction or where loose soil, groundwater seepage, or existing fill is encountered. The slope should be scaled of loose cobbles and boulders and covered with strong wire or fabric, firmly secured to prevent roll down of cobbles or other deleterious materials. The contractor is responsible for the safety of workers and should strictly observe federal and local OSHA requirements for excavation shoring and safety. Due to the granular nature of the surface soil, some raveling of temporary cut slopes should be anticipated. During wet weather, surface water runoff should be prevented from entering excavations. To reduce the likelihood of sloughing or failure, temporary cut slopes must not remain over the winter.

7.1.6 Best Management Practices and Erosion Control

Based on our site observations and experience in the area, the predominantly granular onsite soil will be moderately to highly susceptible to erosion, particularly on steep, unprotected slopes. Best management practices (BMPs) should be incorporated into the design and construction of this project. A reference regarding appropriate BMPs is the "Erosion and Sediment Control Guidelines for Developing Areas of the Sierra Foothills and Mountains", prepared by the High Sierra Resource Conversation and Development Council, 1991. The California Regional Water Quality Control Board, Lahontan Region, Best Management Practices Plan is anther source of BMPs.

Erosion and sediment control measures can be categorized as temporary or permanent. Temporary measures should be installed to provide short-term protection until the permanent measures are installed and effective. Temporary erosion control structures are designed to slow runoff velocity and intercept suspended sediment to prevent sediment discharge from the construction area while allowing runoff to continue down gradient. Typical temporary measures include properly installed silt fences, straw bales, sediment logs, water bars, detention basins, covering of exposed soil, channel linings, and inlet protection. Following completion of construction and planting/seeding, temporary erosion control measures may be left in place, possibly for a complete growing season. Temporary erosion control measures require regular inspection and maintenance.

The selection and sizing of a sediment barrier is dependent on slope angle, slope length, and soil type. Sediment barriers should be installed down gradient and at the edges of all disturbed areas and around topsoil and spoil piles where necessary. Sediment barriers should be placed as needed on slope contours, within small drainages, and in gently sloping swales. The unprotected slope length above each barrier should not exceed 100 feet.

Berms, waterbars and ditches should be used to divert or channel storm water runoff away from sensitive, disturbed or construction areas. Waterbars are intended to slow water traveling down a disturbed slope and divert water off disturbed soil into adjacent stable often well-vegetated areas. Where possible, interceptor ditches and waterbars should take advantage of existing terrain and vegetation to divert runoff before it reaches slopes and disturbed areas. Waterbars should be constructed above and within disturbed areas. The spacing for temporary waterbars should be as needed to divert water off the disturbed areas. Waterbars should be located adjacent to non-erodible (vegetated or rocky) receiving areas. If stable receiving areas are not present, flow energy dissipaters or "J-hook" shaped silt fences should be positioned at the waterbar outlet. In highly erodible soils, waterbar ditches should be protected by temporary lining or by decreasing waterbar spacing and length of flow line slopes.

Permanent erosion and sediment control measures may include rock slope protection (RSP), rock lined ditches and inlet/outlet protection, rock energy dissipaters, infiltration/detention basins, and vegetation. All areas disturbed by construction should be revegetated, and existing vegetation should be protected and undisturbed where possible. Revegetation should consist of native brush and grass species. Slope faces should be temporarily protected against erosion resulting from direct rain impact and melting snow using the methods described above until permanent vegetation can be established. Surface water drainage should not be directed to flow over slope faces. Interceptor (brow) ditches should be considered at the tops of slopes in order to collect and divert runoff which otherwise would flow over the slope face. The intercepted water should be discharged into natural drainage courses or into other collection and disposal structures.

7.1.7 Underground Utility Trenches

We anticipate that the contractor will be able to excavate underground utility trenches using conventional earthmoving equipment across the majority of the site. However, trenches in the west sloping portion of the site may encounter moderately strong cemented soil, bedrock and/or boulders. We anticipate that a track mounted excavator equipped with a ripper or hydraulic hammer, or spot blasting may be required in the western portion of the site. An excavator with a "thumb" attachment may increase ease of boulder removal at the site.

Due to the granular nature of the near-surface soil, we expect that some rock fall, caving and sloughing of utility trench sidewalls may occur. The California Occupational Safety and Health Administration (OSHA) requires all utility trenches deeper than 5 feet bgs be shored or sloped back prior to entry.

Shallow subsurface seepage may be encountered in trench excavations, particularly if utility trenches are excavated during the spring or early summer. The earthwork contractor may need to employ dewatering methods as discussed in the *Construction Dewatering* section below to excavate, place and compact trench backfill materials.

If underground utilities are to be installed on the western sloping portion of the site, we recommend utility trench cut off walls and/or relief drains be considered for any proposed steep utility lines greater than 100 feet in length or any lines entering a building. We can provide details for cut off drain construction as necessary.

Soil used as trench backfill should be non-expansive and should not contain rocks greater than 3 inches in maximum dimension. Trench backfill should consist of uniformly moisture conditioned soil and be placed in maximum 8-inch thick loose lifts prior to compacting. Unless otherwise specified by the applicable local utility district, pipe bedding and trench backfill should be compacted to at least 90 percent of the maximum dry density per ASTM D1557. Trench backfill placed within 8 inches of subgrade in building, road and parking lot areas should be compacted to a minimum relative compaction of 95 percent of the maximum dry density per ASTM D1557. The moisture content, density and relative compaction of fill should be tested by Holdrege & Kull at regular intervals during fill placement.

7.1.8 Construction Dewatering

If grading is performed during or immediately following the wet season or spring snowmelt, seepage may be encountered during grading. We should observe those conditions and provide site specific subsurface drainage recommendations. The following recommendations are preliminary and are not based on a groundwater flow analysis.

We anticipate that dewatering of excavations can be performed by gravity or by constructing sumps to depths below the excavation and removing water with pumps. To maintain stability of the excavation when placing and compacting the trench backfill, groundwater levels should be drawn down a minimum of 2 feet below the lowest point of the excavation.

If seepage is encountered during trench excavation, it may be necessary to remove underlying saturated soil and replace it with free draining, open-graded crushed rock. Soil backfill may be placed after backfilling with drain rock to an elevation higher than encountered groundwater.

7.1.9 Surface Water Drainage

Based on the results of our subsurface explorations completed at the site, and our past experience with geotechnical investigations in the project vicinity, there is a relatively high potential for seasonal saturation of near-surface soil and groundwater seepage into the foundation areas. Near-surface groundwater may enter through retaining walls into subsurface spaces, migrate through concrete floor slabs, degrade asphalt concrete pavements, increase frost heave and contribute to other adverse conditions. In addition, we anticipate that groundwater may be perched on top of near-surface rock or less permeable silt layers.

Final elevations at the site should be planned so that drainage is directed away from all foundations and pavements. Ponding of surface water should not be allowed near pavements or structures. Infiltration of roof or pavement runoff should not be allowed adjacent to structures. Paved areas should be sloped and drainage gradients maintained to carry all surface water to a properly designed infiltration or detention basin.

Drains should be constructed on the upslope side of retaining walls and should be placed along continuous interior wall foundations. Drains should extend to a properly designed infiltration gallery. Recommended subsurface drain locations can be provided at the time of construction and when foundation elevations are known.

All foundation and slab-on-grade concrete should have a water to cement ratio of 0.45 or less. Underslab or blanket drains should be considered in floor pavement areas to reduce moisture transmission through the floor and help maintain subgrade support.

If open-graded gravel or other permeable material is used for underground utilities, the trench should slope away from the structure or the potential flow path should be plugged with a less permeable material at the exterior of the foundation. All utility pipes should have sealed joints.

Roof drip-lines should be protected from erosion with a gravel layer and riprap. Roof downspouts should be directed to a closed collector pipe that discharges flow to positive drainage away from structures. Backfill soil placed adjacent to building foundations should be placed and compacted such that water is not allowed to pond or infiltrate. Backfill should be free of deleterious material and placed and compacted in accordance with the above earthwork recommendations.

The south and east portions of the site appear to be underlain by fine-grained soil that may have low infiltration capabilities. We recommend the project civil engineer in conjunction with the project geotechnical engineer develop appropriate measures to capture, detain, and manage surface water runoff.

7.1.10 Plan Review and Construction Monitoring

Construction monitoring includes review of plans and specifications and observation of onsite activities during construction as described below. We should review final grading and foundation plans prior to construction to evaluate whether our recommendations have been implemented and to provide additional and/or modified recommendations, if necessary. We also recommend that our firm be retained to provide construction monitoring and testing services during site grading, foundation, retaining wall, underground utility and road construction to observe subsurface conditions with respect to our engineering recommendations.

7.2 Structural Improvement Design Criteria

The following sections provide design criteria for foundations, seismic design, slabs-on-grade, retaining walls, and pavement sections.

Our opinion is that conventional spread foundations are suitable for support of the proposed Lodge structure in the western portion of the site. Due to the potential for excessive settlement, we recommend that the proposed Parking structure and Buildings C, D, and E be supported on a rigid mat foundation. The following paragraphs discuss foundation design parameters and construction recommendations.

7.2.1 Spread Foundations

Exterior foundations should be embedded a minimum of 24 inches below the lowest adjacent exterior finish grade for frost protection and confinement. The bottom of interior footings should be at least 12 inches below lowest adjacent finish grade for confinement. Reinforcing steel requirements for foundations should be determined by the project structural engineer.

Prior to foundation construction all existing fill should be removed to firm, previously undisturbed soil. Foundations founded in competent, undisturbed native soil or compacted fill may be designed using an allowable bearing capacity of 4,500 pounds per square foot (psf) for dead plus live loads. The allowable bearing pressure is a net value: therefore, the weight of the foundation that extends below grade may be neglected when computing dead loads. Allowable bearing pressures may be increased by 33 percent for transient loading such as wind or seismic loads.

Resistance to lateral loads (including transient loads) may be provided by frictional resistance between the bottom of concrete foundations and the underlying soil, and by passive soil pressure against the sides of foundations. Due to potential variability of soil consistency at finish grade, potential surface soil desiccation and disturbance, we recommend the upper 6 inches of soil be neglected when estimating lateral resistance. Lateral resistance derived from passive earth pressure can be modeled as a triangular pressure distribution ranging from 0 psf at six-inches below the ground surface to a maximum of 400d psf, where d equals the depth of the foundation in feet. A coefficient of friction of 0.40 may be used between poured-in-place concrete foundations and the underlying native soil.

Total settlement of individual foundations will vary depending on the plan dimensions of the foundation and actual structural loads. Based on anticipated foundation dimensions and loads, we estimate that total post-construction settlement of footings designed and constructed in accordance with our recommendations will be on the order of 3/4-inch.

Differential settlement between similarly loaded, adjacent footings is expected to be less than 3/8-inch, provided footings are founded on similar materials (e.g., all on engineered fill, native soil, or rock). Differential settlement between adjacent footings founded on dissimilar materials (e.g., one footing on soil and an adjacent footing on rock) may approach the maximum anticipated total settlement. Settlement of foundations is expected to occur rapidly and should be essentially complete shortly after initial application of loads.

Loose material remaining in footing excavations should be removed to expose firm, unyielding material or compacted to at least 90 percent relative compaction. Footing excavations should be moistened prior to placing concrete to reduce risk of problems caused by wicking of moisture from curing concrete. Holdrege & Kull should observe footing excavations prior to reinforcing steel and concrete placement.

7.2.2 Mat Foundations

We recommend a mat foundation constructed of reinforced concrete or post tensioned concrete and founded on undisturbed soil or engineered fill be used to support the proposed parking structure and Buildings C, D, and E in the eastern portion of the site. For frost protection and confinement the mat should be surrounded by a continuous perimeter foundation embedded a minimum of 24 inches below the lowest adjacent grade. An allowable bearing pressure of 3,500 pounds per square foot (psf) may be used for a mat foundation. The allowable bearing pressure is a net value; therefore, the weight of the foundation which extends below grade may be neglected when computing dead loads. Allowable bearing pressures may be increased by 1/3 for transient loading such as wind or seismic loads. Reinforcing steel and post tensioning requirements for foundations should be designed by the project structural engineer.

For design of mat foundations, a modulus of subgrade reaction of 225 pounds per square inch per inch of deflection (based on a loaded area of 30 inches square in plan dimensions and 0.1 inches of vertical deflection) may be used.

Resistance to lateral loads (including transient loads) may be provided by frictional resistance between the bottom of concrete foundations and the underlying soil, and by passive soil pressure against the sides of foundations. Due to potential variability of soil consistency at finish grade, potential surface soil desiccation and disturbance, we recommend the upper 6 inches of soil be neglected when estimating lateral resistance. Lateral resistance derived from passive earth pressure can be modeled as a triangular pressure distribution ranging from 0 psf at six-inches below the ground surface to a maximum of 350d psf, where d equals the depth of the foundation in feet. A coefficient of friction of 0.33 may be used between poured-in-place concrete foundations and the underlying native soil.

Total settlement of individual foundations will vary depending on the plan dimensions of the foundation and actual structural loads. Based on anticipated foundation dimensions and loads, we estimate that total post-construction settlement of a mat foundation designed and constructed in accordance with our recommendations will be on the order of ½-inch. Settlement of foundations is expected to occur relatively rapidly and should be essentially complete shortly after initial application of loads.

Loose material remaining in footing excavations should be removed to expose firm, unyielding material or compacted to at least 90 percent relative compaction. Within large mat foundations subgrade soil may be above optimum moisture content and may become unstable under equipment traffic. Equipment traffic should be limited within foundation areas and/or an aggregate sub-base may be necessary for construction. Foundation excavations should be moistened prior to placing concrete to reduce risk of problems caused by wicking of moisture from curing concrete. Holdrege & Kull should observe foundation excavations prior to reinforcing steel and concrete placement.

7.2.3 Seismic Design Criteria

In accordance with the 2007 CBC, the mapped maximum considered earthquake spectral response acceleration at short periods (S_s) and at the 1-second period (S_1) shown in the table below should be used for the project site. The values were obtained for the site using the USGS Earthquake Hazards Program Ground Motion Calculator. The values were generated based on the site's approximate latitude and longitude (39.0856^0 N and 120.1605^0 W, respectively) obtained from Google Earth.

Due to the varied subsurface conditions at the site, we recommend using Site Class B for the North Base Lodge Building (Table 1613.5.2, 2007 CBC) to evaluate seismic loads. We recommend using Site Class D for the Parking Structure and Buildings C, D, and E.

For Site Class B, the following seismic parameters should be used for earthquake design loads:

S_s = 115.5%g	Figure 1613.5(3), 2007 CBC
S_1 = 41.5%g	Figure 1613.5(4), 2007 CBC
F_a = 1.0	Table1613.5.3(1), 2007 CBC
F_v = 1.0	Table 1613.5.3(2), 2007 CBC

For Site Class D, the following seismic parameters should be used for earthquake design loads:

S_s = 115.5%g	Figure 1613.5(3), 2007 CBC
S_1 = 41.5%g	Figure 1613.5(4), 2007 CBC
F_a = 1.038	Table1613.5.3(1), 2007 CBC
F_v = 1.585	Table 1613.5.3(2), 2007 CBC

7.2.4 Slab-on-Grade Construction

Prior to constructing concrete slabs, the upper 8 inches of slab subgrade should be scarified, uniformly moisture conditioned to within 2 percent of optimum moisture content and compacted to at least 90 percent of the maximum dry density per ASTM D1557. Scarification and recompaction may not be required if floor slabs are placed directly on undisturbed compacted structural fill. The subgrade should be protected against drying until the concrete slab is placed.

Slabs-on-grade should be a minimum of 4 inches thick. If floor loads higher than 250 psf, intermittent live loads, or vehicle loads are anticipated, the project structural engineer should provide slab thickness and steel reinforcing requirements. For design of slabs and estimating slab deflections, a modulus of subgrade reaction of 250 pounds per square inch of deflection (based on a loaded area of 30 inches square in plan dimensions and 0.1 inch of vertical deflection) may be used. Settlement of lightly loaded floor slabs (i.e. less than 200 psf) is estimated to be less than ¼ inch.

Due to the potential for seasonal saturation of near-surface soil and to reduce the potential for moisture intrusion, the project architect and/or owner should consider constructing a drain beneath concrete slabs on grade that will enclose livable space. Subdrains should consist of a minimum of 4-inches of clean crushed gravel placed over native subgrade sloped a minimum of 2 percent towards a 4-inch diameter perforated drain pipe. The drain pipe should be placed perforations face down and sloped to drain water from beneath the slab to a properly constructed infiltration gallery or detention basin. A minimum of one pipe should be installed in each area of the slab surrounded by continuous perimeter foundation elements.

Slabs should be underlain by at least 4 inches of Class 2 aggregate base placed over the prepared subgrade or subdrain to provide uniform support. The aggregate base should be compacted to a minimum of 95 percent of the maximum dry density per ASTM D1557. If groundwater is encountered in slab areas, subsurface drains should be constructed.

We recommend all slab-on-grade areas, regardless of floor coverings, be underlain by a vapor retarder (e.g. 15 mil thick plastic water vapor retarder per ASTM E 1745-97 2004). The retarder should be placed over the base course to reduce the migration of moisture vapor through the concrete slab. All penetrations through the vapor retarder should be taped or sealed to reduce vapor. Laps in the vapor retarder should be taped. The vapor retarder may be omitted in areas that do not have moisture sensitive floor coverings, such as parking areas. The American Concrete Institute (ACI) approves of placing concrete directly on the vapor retarder. We do not recommend placing sand between the vapor retarder and the slab. However, all slab concrete should have a water-cement ratio of 0.45 or less and concrete mixes should be designed for placement directly on a vapor barrier.

Regardless of the type of vapor retarder used, moisture can wick up through a concrete slab. Excessive moisture transmission through a slab can cause adhesion loss, warping, and peeling of resilient floor coverings, deterioration of adhesive, seam separation, formation of air pockets, mineral deposition beneath flooring, odor, and fungi growth. Slabs can be tested for water transmissivity in areas that are moisture sensitive. Commercial sealants, moisture retarding admixtures, fly ash, and a reduced water-to-cement ratio can be incorporated into the concrete to reduce slab permeability. To further reduce the chance of moisture transmission, a waterproofing consultant should be contacted.

Concrete slabs impart a relatively small load on the subgrade (approximately 50 psf). Therefore, some vertical movement should be anticipated from possible expansion, freeze-thaw cycles, or differential loading.

7.2.5 Retaining Wall Design Criteria

Retaining walls should be designed to resist lateral earth pressures exerted by retained, compacted backfill plus additional lateral forces (i.e. surcharge loads) that will be applied to walls. The following active and passive pressures are for well drained walls retaining native soil. If import soil is used for fill or backfill, we should review our recommendations. Pressures exerted against retaining walls may be calculated by modeling soil as an equivalent fluid with unit weights presented in the following table.

Table 7.2.5.1 – Equivalent Fluid Unit Weights*		
Loading Condition	Retained Cut or Compacted Fill (Level Backfill)	Retained Cut or Compacted Fill (Backfill Slopes up to 2:1, H:V)
Active Pressure (pcf)	30	45
Passive Pressure (pcf)	350	350
At-Rest Pressure (pcf)	50	65
Coefficient of Friction	0.40	0.40

* Equivalent fluid unit weights presented are ultimate values and do not include a factor of safety. Passive pressures provided assume footings are founded in competent native soil or compacted and tested fill.

The values presented in Table 7.2.5.1 assume that the retained soil will not exceed approximately 22 feet in height and that no surcharge loads (e.g., footings, vehicles) are anticipated within a horizontal distance of approximately 14 feet from the face of the wall. If additional surcharge loads are anticipated, we should review the proposed loading configuration to provide loading-specific design criteria. In addition, we can provide retaining wall and rockery wall design criteria for specific loading and backfill configurations, if requested.

The use of the tabulated active pressure unit weight requires that the wall design accommodate sufficient deflection for mobilization of the retained soil to occur. Typically, a wall yield of less than 0.1 percent of the wall height is sufficient to mobilize active conditions in granular soil. If the walls are rigid or restrained to prevent rotation, at-rest conditions should be used for design.

Additional lateral loading on retaining structures due to seismic accelerations may be considered at the designer's option. For this site, we recommend using a design ground acceleration (K_h) of 0.31g (Class B soil type) with the Mononobe-Okabe/Seed Whitman procedure to evaluate seismic loading on retaining walls.

Compaction equipment should not be used directly adjacent to retaining walls unless the wall is designed or braced to resist the additional lateral forces. If surface loads are closer to the top of the retaining wall than one-half of its height, Holdrege & Kull should review the loads and loading configuration. We should also review details and plans for any proposed wall over 10 feet in height.

Retaining wall design criteria presented in Table 7.2.5.1 assume that retaining walls are well drained to reduce hydrostatic pressures. Drainage blankets consisting of graded rock drains and geosynthetic blankets should be installed to reduce hydrostatic pressures. Rock drains should consist of a minimum 18 inches of open-graded crushed rock, and placed directly behind the wall, wrapped in non-woven geotextile filter fabric such as Mirafi 140N or approved equivalent. Drains should have a minimum 4-inch diameter, perforated drain pipe placed at the base of the wall, inside the drain rock, with perforations placed down. The pipe should be sloped so that water is directed away from the wall by gravity. Gravel drains should extend to the top of the wall or the ground surface. A geosynthetic drainage blanket such as Enkadrain™ or equivalent may be placed against the back of the wall above the gravel drain. Backfill must be compacted carefully so that equipment or soil does not tear or crush the drainage blanket.

If constructed, we recommend that subsurface walls and slabs be treated to resist moisture migration. Moisture retarding material should consist of sheet membrane rubberized asphalt, polymer-modified asphalt, butyl rubber, or other approved material capable of bridging nonstructural cracks, applied in accordance with the manufacturers recommendations. Extra attention should be paid to concrete cold joints between walls and footings. A manufactured water-stop or key should be placed at all cold joints. The project architect or contractor may wish to consult with a waterproofing expert regarding additional options for reducing moisture migration into living areas.

7.2.6 Pavement Design

Based on our experience in the Tahoe-Truckee area, environmental factors, such as freeze-thaw cycles and thermal cracking will usually govern the life of asphalt concrete (AC) pavements. Thermal cracking of asphalt pavement allows more water to enter the pavement section, which promotes deterioration and increases maintenance costs. In addition, snow removal activities on site will result in heavy traffic loads. For these reasons, we recommend a minimum parking area pavement section of 3 inches of AC on 8 inches of aggregate base (AB). We recommend main access driveways and all delivery areas have a minimum pavement section of 4 inches of AC on 8 inches of AB.

We recommend that paving stones in non-traffic areas be supported by a minimum of 6-inches of Caltrans Class 2 aggregate base (AB). We do not recommend paving stones in vehicle traffic areas. An underlying concrete slab is not necessary for non-traffic areas. Prior to placing aggregate base, the subgrade should be prepared in accordance with the recommendations provided below.

Due to seasonal saturation of the underlying AB and freeze-thaw cycles, some vertical movement of paving stones over time should be anticipated. This movement can likely be reduced by constructing a drainage layer beneath paving stone pavements. The drainage layer should consist of 4 inches of compacted clean angular gravel. The gravel layer should be underlain by a minimum 4-inch diameter perforated pipe, sloped to drain water from beneath the pavement towards an infiltration gallery. A minimum 4-

ounce non-woven filter fabric such as Mirafi 140 or approved equivalent should be placed between the compacted gravel subdrain and aggregate base layer.

The upper 6 inches of native soil should be compacted to at least of 95 percent of the maximum dry density per ASTM D1557 prior to placing aggregate base. Aggregate base should also be compacted to a minimum of 95 percent. Subgrade and AB dry density should be evaluated by Holdrege & Kull. In addition to field density tests, subgrade should be proof rolled under the observation of Holdrege & Kull prior to base placement.

Pavement subgrade should be graded and prepared such that water drains from beneath pavement section and to a properly designed infiltration or detention basin. In addition, we recommend installing cut-off curbs where paved areas abut landscaped areas to reduce migration of irrigation water into subgrade soil or base, promoting asphalt failure. Cut-off curbs should be a minimum of 4-inches wide, and extend through the aggregate base a minimum of 4 inches into subgrade soil.

8. LIMITATIONS

Our professional services were performed consistent with the generally accepted geotechnical engineering principles and practices employed in the site area at the time the report was prepared. No warranty, either express or implied, is intended.

Our services were performed consistent with our agreement with our client. We are not responsible for the impacts of changes in environmental standards, practices or regulations subsequent to performance of our services. We do not warrant the accuracy of information supplied by others, or the use of segregated portions of this report. This report is solely for the use of our client. Reliance on this report by a third party is at the risk of that party.

If changes are made to the nature or design of the project as described in this report, then our conclusions and recommendations presented in the report should be reviewed by Holdrege & Kull. Additional field work and laboratory tests may be required to revise our recommendations. Costs to review project changes, perform additional field work and laboratory tests necessary to modify our recommendations are beyond the scope of services provided for this report. Additional work will be performed only after receipt of an approved scope of services, budget, and written authorization to proceed.

Analyses, conclusions and recommendations presented in this report are based on site conditions as they existed at the time we performed our subsurface exploration. We assumed that subsurface soil conditions encountered at the location of our exploratory test pits, borings, and CPTs are generally representative of subsurface conditions across the project site. Actual subsurface conditions at locations between and beyond our exploratory test pits, borings, and CPTs may differ. If subsurface conditions encountered during construction are different than those described in this report, we should be notified so that we can review and modify our recommendations as needed.

The elevation or depth to groundwater and soil moisture conditions underlying the project site may differ with time and location. The project site map shows approximate exploratory test pit, boring, and CPT locations as determined by pacing distances from identifiable site features. Therefore, test pit, boring, and CPT locations should not be relied upon as being exact.

Our scope of services did not include evaluating the project site for the presence of hazardous materials or petroleum products. Although we did not observe evidence of hazardous materials or petroleum products at the time of our field investigation, project personnel should take necessary precautions should hazardous materials be encountered during construction.
The findings of this report are valid as of the present date. Changes in the conditions of the property can occur with the passage of time. These changes may be due to natural processes or works of man, at the project site or adjacent properties. In addition, changes in applicable or appropriate standards can occur, whether they result from legislation or broadening of knowledge. Therefore, the recommendations presented in this report should not be relied upon after a period of two years from the issue date without our review.

FIGURES

SOURCE: USGS HOMEWOOD, CA, 7.5 MINUTE TOPOGRAPHIC MAP, 1992.

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SITE VICINITY MAP
HOMEWOOD MOUNTAIN RESORT
PROPOSED NORTH BASE LODGE
HOMEWOOD/PLACER COUNTY, CALIFORNIA

PROJECT NO.: 41278-03
DATE: JANUARY, 2010
FIGURE NO.: 1