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

Prepared for:
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Project No. 41278-01
September 9, 2009

FIGURES

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 Area***
Homewood Mountain Resort
Homewood/Placer County, California

Subject: ***Preliminary Geotechnical Engineering Report***

This report presents the results of our preliminary geotechnical engineering investigation for the proposed Homewood Mountain Resort North Base Area 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). The Homewood re-development project will also include residential development at the South Base Area, single family homes at the North Base Area, and a Mid-Mountain Lodge. At the North Base Area, the existing two-story wood frame lodge structure will be removed and replaced with several mixed use residential and commercial structures including a multi-story hotel/lodge. Appurtenant construction will include a gondola, a parking structure, an ice rink, landscaping, and drainage improvements.

This report is based on information collected during the drilling and logging of nine exploratory borings, previous geologic and geotechnical investigations performed in the site area, review of geologic maps, and our previous experience in the site area. The scope of this investigation was limited due to the amount of snow at the site and on-going ski area operations. A more detailed subsurface investigation must be performed prior to construction to further evaluate the subsurface conditions across the site and to confirm the preliminary subsurface conditions used to prepare this report.

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We appreciate the opportunity of providing our services for this project. Please contact the undersigned if you have any questions regarding this report.

Sincerely,
Holdrege & Kull

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TABLE OF CONTENTS

1. INTRODUCTION 1
   1.1 Purpose 1
   1.2 Scope of Services 1
   1.3 Site Description 2
   1.4 Proposed Improvements 2
2. Literature Review and site reconnaisance 3
   2.1 Site Geology 3
   2.2 Regional Faulting 4
   2.3 Secondary Seismic Hazards 5
3. Subsurface conditions 6
   3.1 Field Exploration 6
   3.2 Subsurface Soil Conditions 7
      3.2.1 Borings B-1 through B-3 and B-6 through B-9 – Proposed North Base Lodge 7
      3.2.2 Borings B-4 and B-5 – Proposed Parking Structure 7
   3.3 Groundwater Conditions 8
4. LABORATORY TESTING 8
   Table 4.1 – Summary of Laboratory Test Results 9
5. CONCLUSIONS 9
6. RECOMMENDATIONS 10
   6.1 Grading 11
      6.1.1 Clearing and Grubbing 11
      6.1.2 Preparation for Fill Placement 12
      6.1.3 Fill Placement 12
      6.1.4 Cut/Fill Slope Grading 13
      6.1.5 Temporary Unconfined Excavations 14
      6.1.6 Best Management Practices and Erosion Control 14
      6.1.7 Underground Utility Trenches 16
      6.1.8 Construction Dewatering 16
      6.1.9 Surface Water Drainage 17
      6.1.10 Plan Review and Construction Monitoring 18
   6.2 Preliminary Structural Improvement Design Criteria 18
      6.2.1 Continuous Flight Auger Drilled Pier Foundations 19
      6.2.2 Mat Foundations 20
      6.2.3 Seismic Design Criteria 21
      6.2.4 Slab-on-Grade Construction 21
      6.2.5 Retaining Wall Design Criteria 22
      Table 6.2.5.1 – Equivalent Fluid Unit Weights* 23
      6.2.6 Pavement Design 24
7. LIMITATIONS 25

FIGURES

Figure 1 – Site Vicinity Map
Figure 2 – Boring Location Plan

APPENDICES

Appendix A Important Information About Your Geotechnical Engineering Report
Appendix B Boring Logs
Appendix C Laboratory Test Results

1. INTRODUCTION

This report presents the results of our preliminary 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 this preliminary geotechnical report is to provide general geotechnical and geologic information to be considered during the planning and preliminary design of the project. Our evaluation addresses the general soil and groundwater conditions at the project site, with emphasis on how the conditions are expected to affect the proposed construction. This report also considers potential geologic hazards including faulting and seismicity, slope instability, liquefaction, and other secondary seismic hazards. The preliminary recommendations contained in this report should not be used for final design, extrapolated to other areas, or used for other developments. Our current scope of services includes a subsurface investigation; however, a more detailed geotechnical investigation must be performed prior to construction. We will prepare a design level report following the completion of a more detailed geotechnical investigation that will include collecting additional subsurface information.

1.2 Scope of Services

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

- A site reconnaissance and a review of available literature and subsurface information contained in our files pertinent to the proposed construction and project site;

- Review of the report titled, “Geologic Hazards and Geotechnical Evaluation, Homewood Mountain Resort, Homewood, California”, prepared by Kleinfelder, dated November 1, 2007.

- Completion of a preliminary subsurface investigation by drilling, logging and collecting relatively undisturbed soil samples from nine exploratory borings drilled with a truck-mounted drill rig.

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

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

- Preparation of this preliminary engineering 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. A plan view of the proposed redevelopment is shown on Figure 2, Boring Location Plan.

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.

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 site slopes steeply to gently down from west to east.

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 preliminary report focuses on the North Base Area.

As currently proposed, 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. A new base mountain facility will replace the existing day skier services structure. 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 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.

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

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 for building construction may be on the order of approximately 35 feet. Anticipated fills may be on the order of about 25 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 and/or a rigid mat foundation.

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 parking structure may be supported on a rigid mat foundation system.

Appurtenant construction will likely 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.

2. LITERATURE REVIEW AND SITE RECONNAISANCE

We reviewed available geologic and soil literature in order to evaluate geologic, seismic, and anticipated subsurface conditions at the project site. The following section of this report incorporates geologic features observed during our site reconnaissance.

2.1 Site Geology

We reviewed the *Geologic Map of the Chico Quadrangle, California*, by G.J. Saucedo and D.L. Wagner, California Division of Mines and Geology, 1992 and the *Geologic Map of the Lake Tahoe Basin, California and Nevada,* by G.J. Saucedo, California Geological Survey, 2005. The geologic maps indicate that the east portion of the

project area is underlain by lake deposits of Quaternary age (less than approximately 1.6 million years before the present). 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).

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.

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 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 through the northeast corner of the site. It is our professional opinion that building set back distances from the unnamed inferred fault are not warranted and no further study is necessary.

2.3 Secondary Seismic Hazards

Secondary seismic hazards include liquefaction, lateral spreading, and seismically induced slope instability and rock fall. Liquefaction is a phenomenon where loose, saturated, granular soil deposits lose a significant portion of their shear strength due to excess pore water pressure buildup. Cyclic loading, such as an earthquake, typically causes the increase in pore water pressure and subsequent liquefaction. Based on the results of our subsurface investigation, loose to medium dense sand underlies the eastern portion of the site. This soil may be prone to liquefaction and should be confirmed during a design level geotechnical investigation.

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.

Several options are available to reduce site liquefaction potential and/or adverse effects to structures located above potentially liquefiable soils. A list of generalized options considered appropriate to this project is provided below.

1. Densify potentially liquefiable soils with an in-situ ground improvement technique such as vibro-compaction, vibro-replacement, permeation or compaction grouting, or other similar methods.

2. Support the proposed structures on a deep foundation system.

3. Strengthen foundations (e.g., post-tensioned slab, reinforced mat or grid foundation, or other similar system) to resist excessive differential settlement associated with seismically-induced liquefaction.

4. Support the proposed structures on an engineered fill pad in order to reduce differential settlement resulting from seismically-induced liquefaction and post-seismic pore pressure dissipation.

In general, options listed above are in descending order of cost effectiveness. In our opinion, Option 1 would essentially eliminate potential liquefaction (and hence associated risks), Option 2 would not reduce the potential for liquefaction but would limit total and differential settlement of load-bearing structural elements, and Options 3 and 4 would limit differential settlement but not reduce total settlement of the proposed structures. We anticipate that a combination of Options may be used for the proposed structures at the site. Again, the potential for liquefaction should be further investigated during a design level geotechnical investigation.

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.

3. SUBSURFACE CONDITIONS

We performed our subsurface exploration to characterize typical subsurface conditions at the site, as described below.

3.1 Field Exploration

The subsurface conditions at the site were investigated on January 13 through 15, 2009 by drilling nine exploratory borings (B-1 through B-9) to depths ranging from 27 to 60 feet below the ground surface (bgs). The borings were drilled 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. Borings B-1, B-2, B-3, B-6, B-7, B-8, and B-9 were drilled within the asphalt paved parking lot in the area of the proposed North Base Lodge. Borings B-4 and B-5 were drilled within the unpaved overflow parking lot located south Fawn Street in the area of the proposed parking structure.

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. The approximate locations of our borings are shown on Figure 2, Boring Location Plan.

3.2 Subsurface Soil Conditions

3.2.1 Borings B-1 through B-3 and B-6 through B-9 – Proposed North Base Lodge

The asphalt concrete pavement section varied in thickness from approximately 2 to 6 inches with 1 to 3 inches of aggregate base, across the paved parking lot portion of the site.

Fill was encountered to depths of approximately 1.5 to 3 feet bgs in borings drilled in the paved parking lot near the existing lodge. The fill consisted of loose to medium dense silty sand (SM) with varying amounts of gravel. Underlying the fill, the borings generally encountered loose to dense silty sand with gravel (SM), poorly graded sand with gravel (SP), and poorly graded gravel with sand (GP) to depths ranging from approximately 14 to 21 feet bgs. Poorly graded sand (SP) was then encountered to depths ranging from approximately 20 to 60 feet bgs (maximum boring depth). The poorly graded sand (SP) was saturated, loose, and heaved inside the augers creating sampling difficulty. In general, a 1- to 2-foot thick cobble layer was encountered at the base of the poorly graded sand (SP).

Refusal on strong to very strong volcanic rock (RX) was encountered in Borings B-1, B-2, B-6 and possibly in B-9 at depths of approximately 22, 27, 39.5 and 54 feet bgs. Drilling methods were switched from hollow stem auger to continuous core in Boring B-1 to verify refusal on rock and to evaluate the rock quality designation (RQD). Continuous coring was completed in Boring B-1 from approximately 22 to 30 feet bgs. The rock consisted of a gray to dark gray andesite that was slightly weathered, widely fractured, moderately strong to strong with an RQD range from poor (43%) to good (76%).

3.2.2 Borings B-4 and B-5 – Proposed Parking Structure

The unpaved overflow parking lot was generally covered with approximately 1 to 2 inches of ice/snow that was underlain by approximately 2 to 3 inches of gravel. Underlying the ice/snow and gravel, Borings B-4 and B-5 generally encountered silty sand (SM), sandy clay to sandy silt (CL/ML), and poorly graded gravel with sand (GP) to depths ranging from approximately 19.5 to 21.5 feet bgs. Poorly sorted sand (SP) was then encountered to depths ranging from approximately 27 to 34 feet bgs. The poorly graded sand (SP) was saturated, loose, and heaved inside the augers creating sampling difficulty. In general, a 1- to 2-foot thick cobble layer was encountered at the base of the poorly graded sand (SP).

Refusal on strong to very strong volcanic rock (RX) was encountered in Borings B-4 and B-5 at depths of approximately 34 to 27 feet bgs, respectively. The rock consisted of a gray to dark gray andesite that was slightly weathered, widely fractured, and was strong to very strong (consistent to that encountered in Borings B-1 and B-2).

More detailed descriptions of the subsurface conditions observed are presented on the attached boring logs in Appendix B.

3.3 Groundwater Conditions

In general, groundwater was consistently encountered at the top of the poorly graded sand (SP) at depths ranging from approximately 13 to 16 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 relatively undisturbed soil samples collected from our exploratory borings to help evaluate their engineering properties. The following laboratory tests were performed:

- Atterberg Limits/Plasticity (ASTM Test Method D4318)
- Sieve Analysis (ASTM D422)
- Expansion Index (ASTM D4829)

Sieve analysis and Atterberg Limits data tests resulted in USCS classifications of Poorly Graded Sand (SP), Sandy Silt (ML), Silt with Sand (ML), Poorly Graded Sand with Silt and Gravel (SP), and Poorly Graded Sand with Gravel (SP). Expansion index testing of two soil samples collected from Boring B-4 at depth intervals ranging from 8.5 to 9 feet bgs indicated that the soil has a very low potential for expansion. More specific soil classification and laboratory test results are included in Appendix C. USCS classification and Atterberg indices are summarized below.

Table 4.1 – Summary of Laboratory Test Results				
Boring Number	Depth (feet)	USCS Classification	Liquid Limit	Plastic Limit
B-2	21	Poorly Graded Sand (SP)	--	--
B-4	6.5, 8.5, and 9	Sandy Silt (ML)	--	--
B-5	7.5 and 8	Silt with Sand (ML)	46	32
B-6	3	Silty Sand (SM)	40	40
B-7	16	Poorly Graded Sand with Silt and Gravel (SP)	--	--
B-8	11	Poorly Graded Sand with Gravel (SP)	--	--
B-8	16	Poorly Graded Sand with Silt and Gravel (SP)	--	--

5. CONCLUSIONS

The following conclusions are based on our limited subsurface investigation, laboratory testing, literature review, engineering analysis, and experience in the project area. A more detailed subsurface exploration must be performed prior to construction to evaluate the potential for liquefiable soil in the east portion of the site, further evaluate depth to bedrock throughout the project area, and confirm site subsurface conditions used to provide conclusions and recommendations in this report.

1. Subsurface conditions varied at the site from silty Sand with gravel to fine sandy Silt overlying poorly graded Sand with some areas of relatively near-surface rock. Based on limited subsurface information to date in the proposed lodge area, we expect that the lodge will be supported partly on volcanic rock and partly on lacustrine soils (lake deposits). Portions of the lodge may need to be supported on a deep foundation system. However, a design level investigation should further evaluate the foundation options.

2. Soil underlying the proposed parking structure consisted of medium stiff fine sandy Silt overlying saturated poorly graded Sand. Near-surface soil has a low expansion potential. There is potential for excessive settlement within the near-surface silt, and the potential for loss of support in the event of an earthquake that results in the liquefaction of the underlying sand. Prior to construction, a design level geotechnical investigation should be performed.

3 Soil underlying the proposed structures in the eastern portion of the site consists of a thin layer of existing fill and silty sand soil overlying saturated sand. Again, there is a potential for excessive settlement and the potential for loss of support in the event of liquefaction of the underlying sand. Further evaluation should be performed during a design level investigation. Possible foundation systems for these structures include post tensioned slabs and foundation system.

4. Some areas of near surface rock may be encountered during excavation for the lodge foundations. Additional subsurface information is needed at the actual building location. With the exception of the organic surface soil and silt soil encountered in the area of the proposed parking structure, site soil is generally suitable for reuse as structural fill. Processing to remove oversized material may be necessary for reuse of near-surface soil in the western part of the site.

5 Groundwater was encountered in our borings at depths of 14 ½ feet below the ground surface. We anticipate near-surface soil layers will become seasonally saturated. It is likely that groundwater seepage will be encountered within the lodge excavation. Based on previous studies, it appears that during the spring, groundwater may be encountered within a few feet of the existing ground surface. Near surface groundwater could limit the treatment of storm water runoff by infiltration. Fine sandy Silt soil encountered in Borings B-4 and B-5 likely has a very low infiltration rate and is not suitable for infiltration of storm water. Silty Sand soil encountered in the eastern portion of the likely has a low to moderate infiltration rate. However, seasonally high groundwater may be present in the eastern portion of the site and limit storm water treatment by infiltration.

6 Soil in the northern portion of the site should provide adequate pavement support. Fine sandy Silt encountered in Borings B-4 and B-5 may require a thicker pavement section than other areas of the site. Existing asphalt concrete pavement and concrete foundations may be crushed of pulverized and recycled for reuse as sub-base, aggregate base, and/or structural fill to be used on the site.

6. RECOMMENDATIONS

The following recommendations are based on our understanding of the project as currently proposed, our field observations, preliminary engineering analysis, and our experience in the project area. A more detailed subsurface investigation must be performed prior to development of final plans to further evaluate the potential for liquefiable soil in the east portion of the project area, depth to bedrock across the project area, and to confirm the assumed subsurface conditions used to prepare this

report. The following recommendations may change if additional information becomes available.

6.1 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.

6.1.1 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. Based on our experience in the area, we expect that 6 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 exploration to date, the depth of existing fill ranges from 3 to 4 feet across the site. The depth of fill should be confirmed during a design level investigation. 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.

6.1.2 Preparation for Fill Placement

Where fill placement is planned, the near-surface soil should be scarified to a depth of about 12 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 overexcavated and replaced with compacted fill placed according to the recommendations below.

6.1.3 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 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 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. 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 overexcavated 3 feet and rebuilt with compacted fill.

6.1.4 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.

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.

6.1.5 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 adjacent to the proposed structure.

Temporary Slope Inclination (Horizontal to Vertical)	Maximum Height (Feet)
0.5:1	8
0.75:1	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.

6.1.6 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.

6.1.7 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 bedrock. 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 with bracing equipment 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 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.

6.1.8 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.

6.1.9 Surface Water Drainage

Based on the results of our preliminary subsurface exploration 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 under-floor crawl 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. Near-surface and perched groundwater may enter under-floor crawl spaces, result in moisture intrusion through concrete floor slabs, degradation of asphalt concrete pavements, increased frost heave and other adverse conditions.

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 within 10 feet of 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 exterior foundations and should be placed along continuous interior wall foundations and in all crawl space areas. 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.

We recommend that the interior subgrade in the crawl space areas be sloped to collect and divert water to drains that exit under or through the foundation (positive crawl space

drainage). All vegetation and highly organic soil should be removed from the crawl space area. Adequate ventilation should be provided in all crawl space areas to promote drying. The project architect and owner should consider the need for an automated mechanical ventilation system.

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. 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.

Portions of the site appear to have poor infiltration capabilities and may generate large amounts of runoff. We recommend the project civil engineer in conjunction with the project geotechnical engineer develop appropriate measures to capture, detain, and manage surface water runoff.

6.1.10 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.

6.2 Preliminary Structural Improvement Design Criteria

The following sections provide preliminary design criteria for foundations, seismic design, slabs-on-grade, retaining walls, and pavement sections. Site specific subsurface exploration must be performed prior to preparation of design level drawings and specifications.

At this time our subsurface information is limited, we have not performed borings within the main lodge footprint. Based on the limited subsurface information obtained to date, it appears that the main lodge structure may be supported on a combination of spread foundations founded on rock and a deep foundation system. Several deep foundation

systems including driven piles and drilled piers are possible to adequately support the planned structure. Aggregate columns may not provide the required lateral support if the structure is supported on potentially liquefiable sand deposits. Cast-in-place concrete drilled shafts would provide high vertical and lateral support. However, drilled shafts are difficult to construct in the saturated flowable sand conditions encountered at the site. Our opinion is that continuous flight auger (CFA) drilled piles would provide potential cost saving over other types of deep foundations without sacrificing quality or performance. In the CFA method, shafts are drilled using a continuous flight auger with a hollow stem through which concrete is placed under pressure during extraction of the auger. CFA shafts use standard concrete with aggregate as large as ¾-inch and full-length rebar cages. The CFA shafts can be designed in the same way as a conventional drilled cast-in-place concrete shaft, but with a substantial installation cost savings.

It is our preliminary opinion that the lighter structures at the site and including the proposed parking structure may be supported on mat foundations. Additional subsurface exploration should be performed to evaluate the potential for liquefaction of clean sand deposits underlying the site. Based on the information available at this time, the sand is liquefiable resulting possible excessive settlement of overlying structures. The following paragraphs discuss foundation design parameters and construction recommendations.

6.2.1 Continuous Flight Auger Drilled Pier Foundations

Due to the potential for excessive settlement, relatively large anticipated foundation loads, and potential liquefaction of underlying sand deposits; we recommend the proposed lodge structure be supported on continuous flight auger (CFA) drilled, straight-shafted concrete piers or shafts. Continuous flight auger (CFA) drilled, straight-shafted concrete piers or shafts extending to bedrock or below the potential liquefaction zone and designed and constructed in accordance with our recommendations should provide adequate support for the planned lodge structure. However, recommendations for allowable end bearing pressure, unit skin friction, estimated settlements, lateral resistance values and specific design and construction recommendations are not possible at this time until end bearing and potential liquefaction conditions under the proposed building are evaluated.

Benefits of the CFA piles/shafts include reduced installation costs, reduced schedule duration, increased safety with no open holes, negligible vibration, low noise, ease of design (similar to conventional drilled shafts), no need for casing or slurry, and high confidence in the constructed shaft.

6.2.2 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 lower-story buildings 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 requirements for foundations should be determined 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 loading. 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.

6.2.3 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.

S_s = 109.8%g	Figure 1613.5(3), 2007 CBC
S_1 = 37.2%g	Figure 1613.5(4), 2007 CBC
F_a = 1.0	Table1613.5.3(1), 2007 CBC
F_v = 1.428	Table 1613.5.3(2), 2007 CBC

Based on our subsurface information to date, we recommend using Site Class D (Table 1613.5.2, 2007 CBC) to evaluate seismic loads.

6.2.4 Slab-on-Grade Construction

Concrete slabs-on-grade may be used in conjunction with perimeter concrete footings. 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.

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.

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 (i.e., exterior parking areas). The American Concrete Institute (ACI), recommends placing concrete directly on the vapor retarder; therefore, we do not recommend placing sand between the vapor retarder and the slab. All slab concrete should have a water-cement ratio of 0.45 or less.

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.

Exterior slabs-on-grade such as sidewalks may be placed directly on compacted fill or scarified and recompacted native soil without the use of an aggregate base section. Deleterious material should be removed from floor slab subgrades prior to concrete placement. For exterior slabs, the native soil should be ripped, moisture conditioned and recompacted to an 8-inch depth.

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.

6.2.5 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 6.2.5.1 – 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 6.2.5.1 assume that the retained soil will not exceed approximately 18 feet in height and that no surcharge loads (e.g., footings, vehicles) are anticipated within a horizontal distance of approximately 12 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 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 0.29g 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 6.2.5.1 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. A geosynthetic drainage blanket such as Enkadrain™ or equivalent should also be placed against the back of the wall. 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.

6.2.6 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 access drive/parking area pavement section of 3 inches of AC on 8 inches of aggregate base (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 baserock. Aggregate baserock 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 baserock 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 baserock, promoting asphalt failure. Cut-off curbs should be a minimum of 4-inches wide, and extend through the aggregate baserock a minimum of 4 inches into subgrade soil.

7. LIMITATIONS

The recommendations in this report are preliminary in nature. Additional subsurface conditions may vary from those described above. A final geotechnical investigation must be performed prior to construction. This report is only valid if Holdrege & Kull performs subsurface exploration prior to or at the time of construction.

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. This warranty is in lieu of all other warranties, either expressed or implied.

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 to review our conclusions and recommendations. Additional field work and laboratory tests may be required to revise our recommendations. Costs to review project changes, perform additional field work and laboratory testing 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.

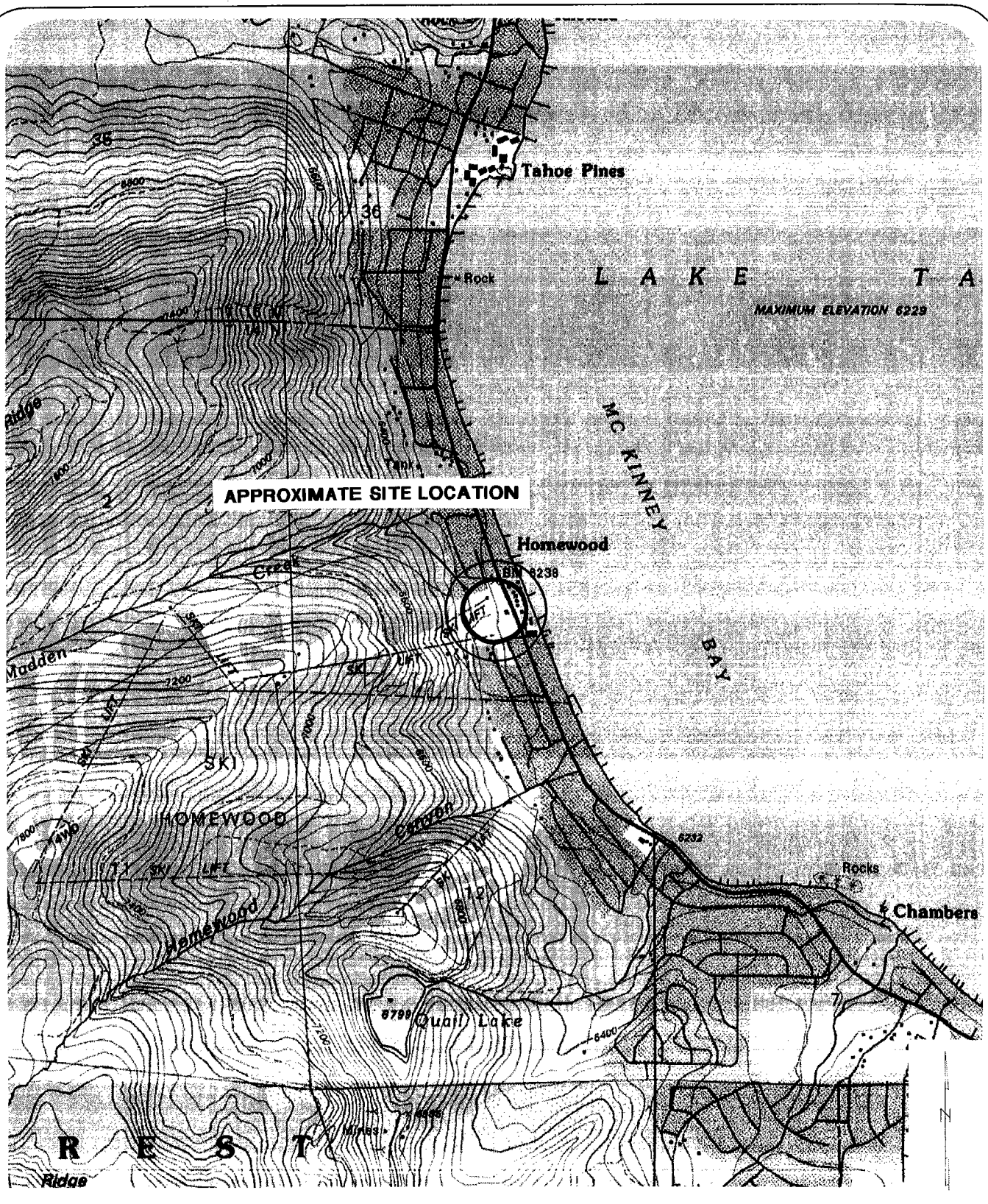
Analyses, conclusions and recommendations presented in this report are based on site conditions as they existed at the time we performed our subsurface exploration. We assumed that subsurface soil conditions encountered at the location of our exploratory test pits are generally representative of subsurface conditions across the project site. Actual subsurface conditions at locations between and beyond our exploratory test pits 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 locations as determined by pacing distances from identifiable site features. Therefore, test pit 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.

Figure 1 **Site Vicinity Map**
Figure 2 **Boring Location Plan**



N.T.S.

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

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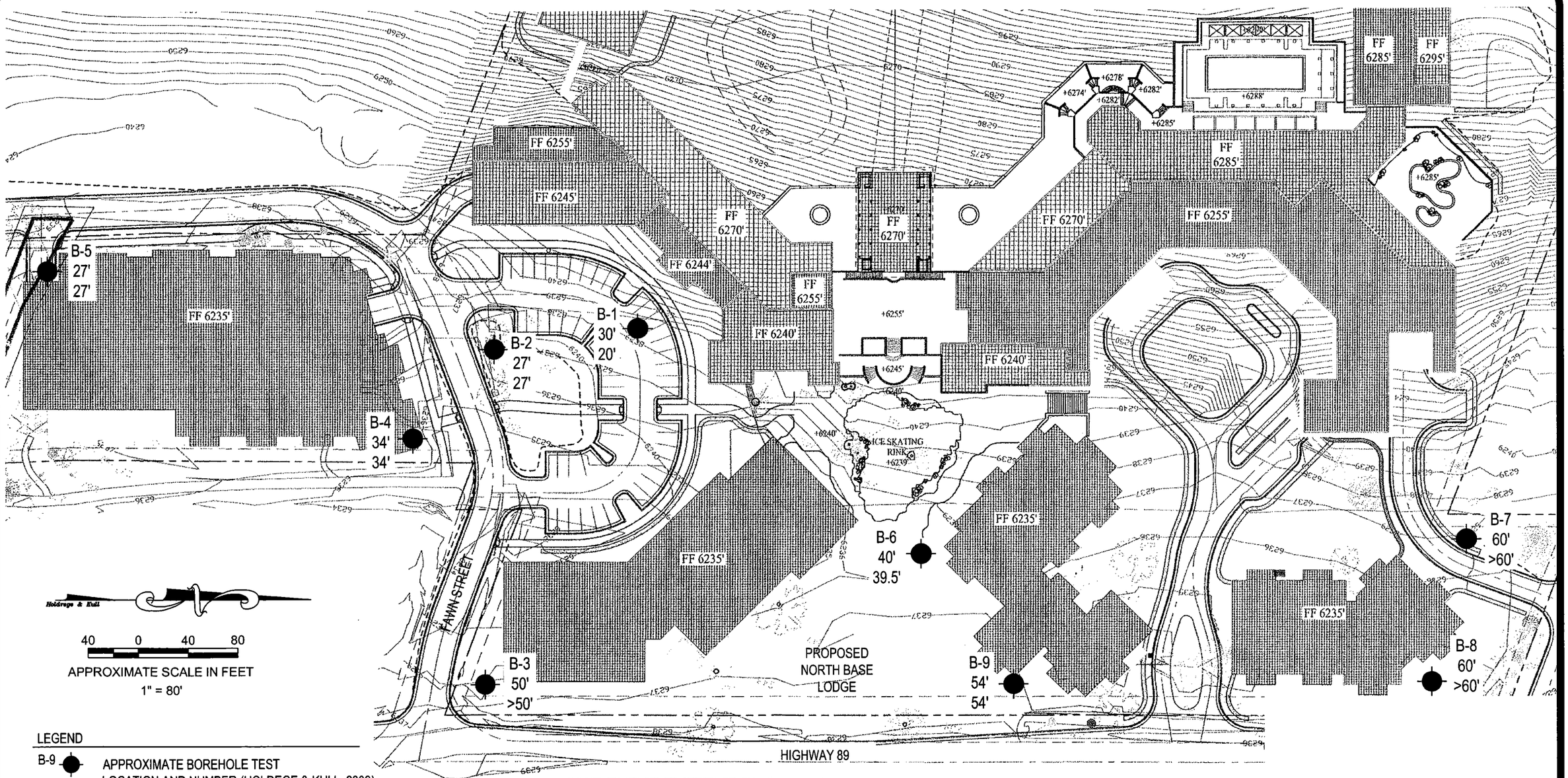
SITE VICINITY MAP
HOMEWOOD MOUNTAIN RESORT
NORTH BASE AREA
HOMEWOOD, CALIFORNIA

PROJECT NO.: 41278-01

DATE: APRIL 2009

FIGURE NO.: 1



LEGEND

B-9 — APPROXIMATE BOREHOLE TEST LOCATION AND NUMBER (HOLDEGE & KULL, 2009)

B-9 50' >50' — BOREHOLE NUMBER, MAXIMUM DEPTH EXPLORED, AND DEPTH TO ROCK.

SITE LAYOUT PLAN WAS PROVIDED BY HILL GLAZIER STUDIO DATED MARCH 31, 2008.

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BOREHOLE TEST LOCATION PLAN
HOMEWOOD MOUNTAIN RESORT
PROPOSED NORTH BASE LODGE
HOMEWOOD/PLACER COUNTY, CALIFORNIA

DRAWN BY: MED	CHECKED BY: JKH
PROJECT NO.: 41278-01	
DATE: APRIL, 2009	
FIGURE NO.: 2	

APPENDIX A ***Important Information About Your Geotechnical Engineering Report***
(Included with permission of ASFE, copyright 2004)

Important Information About Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual

subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.

ASFE
The Best People on Earth

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APPENDIX B Boring Logs

BORING B-1

PROJECT NO.	PROJECT NAME	ELEVATION (APPROX)	DATE	PAGE	BORING NO.
41278-01	HOMEWOOD MOUNTAIN RESORT	6238 FT.	01/13/09	1 OF 2	B-1

DRILLING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CME 75	HOLLOW STEM AUGER MODIFIED CALIFORNIA (2.5" DIA.)	YES	YES

SAMPLE NO.	BLOW COUNTS (N) *	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
				1	--	PAVEMENT SECTION: 1 TO 2 INCHES OF ASPHALT CONCRETE; 2 TO 3 INCHES OF AGGREGATE BASE.
				2	FILL (SM)	FILL: YELLOW BROWN SILTY SAND WITH GRAVEL (SM); MOIST, MEDIUM DENSE, ESTIMATED 30% PASSING NO. 200 SIEVE, FINE GRAVEL.
				3	SM	YELLOW BROWN SILTY SAND WITH GRAVEL (SM); MOIST, LOOSE, ESTIMATED 30% PASSING NO. 200 SIEVE, FINE ROUNDED GRAVEL.
1-1-2	6	--	--	4		
1-1-1		--	--			
				5		
				6		GRADES TO MEDIUM DENSE, ESTIMATED 15 TO 20% PASSING NO. 200 SIEVE, ROUNDED GRAVEL.
				7		
				8		
1-2-2	21	--	--	9		
1-2-1		--	--			
				10		GRADES TO: INCREASING GRAVEL IN CUTTINGS.
				11	SP/GP	GRAY GRAVELLY SAND TO SANDY GRAVEL (SP/GP); MOIST, DENSE, MEDIUM TO COARSE SAND, ROUNDED TO SUBROUNDED GRAVEL, SCATTERED COBBLES.
				12		
				13		
1-3-2	40	--	--	14		GRADES TO: DENSE.
1-3-1		--	--			
				15		
				16	▽ SP	GRAY POORLY GRADED SAND (SP); WET, LOOSE, FINE TO MEDIUM SAND, CONSISTENCY OF MATERIAL IS REPRESENTATIVE OF THE DRILLING OBSERVATIONS AND FLOWING AND HEAVING SAND AND NOT THE BLOW COUNTS.
				17		
				18		
NO	64/4"	--	--	19		GRADES TO: VERY DENSE, NO SAMPLE RECOVERY, 50 BLOWS PER 4 INCHES, HARDER DRILLING AT 20 FEET BGS.
RECOVERY		--	--			
				20		
					▽ ▽ RX	

* BLOW COUNTS WERE CORRECTED FOR SAMPLER TYPE.

HOLDREGE & KULL

BORING B-1

PROJECT NO.	PROJECT NAME	ELEVATION (APPROX)	DATE	PAGE	BORING NO.
41278-01	HOMEWOOD MOUNTAIN RESORT	6238 FT.	01/13/09	2 OF 2	B-1

DRILLING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CME 75	HOLLOW STEM AUGER MODIFIED CALIFORNIA (2.5" DIA.)	YES	YES

SAMPLE NO.	BLOW COUNTS (N) *	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
				21	RX	GRAY ANDESITE (RX); SLIGHTLY WEATHERED, SLIGHTLY TO MODERATELY FRACTURED, STRONG TO VERY STRONG.
1-5-1	50/5"			22		BEGAN CORING AT 22.5 FEET. RQD: 22.5 TO 25 FEET = 43% (POOR)
CORE				23		
				24		
				25		RQD: 25 TO 30 FEET = 76% (GOOD)
				26		
				27		
				28		
				29		
				30		BOREHOLE TERMINATED AT 30 FEET. CAVING WAS NOTED IN FLOWING SANDS.
				31		
				32		
				33		
				34		
				35		
				36		
				37		
				38		
				39		
				40		

* BLOW COUNTS WERE CORRECTED FOR SAMPLER TYPE.

HOLDREGE & KULL

BORING B-2

PROJECT NO.	PROJECT NAME	ELEVATION (APPROX)	DATE	PAGE	BORING NO.
41278-01	HOMEWOOD MOUNTAIN RESORT	6236 FT	01/13/09	1 OF 2	B-2

DRILLING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CME 75	HOLLOW STEM AUGER MODIFIED CALIFORNIA (2.5" DIA.)	YES	YES

SAMPLE NO.	BLOW COUNTS (N) *	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
				1	--	PAVEMENT SECTION: 5 TO 6 INCHES OF ASPHALT CONCRETE; 2 TO 3 INCHES OF AGGREGATE BASE.
				2	FILL (SM)	FILL: DARK YELLOW BROWN SILTY SAND WITH GRAVEL (SM); MOIST, LOOSE TO MEDIUM DENSE, ESTIMATED 20-30% PASSING NO. 200 SIEVE, FINE ROUNDED GRAVEL.
				3	SM	DARK YELLOW BROWN SILTY SAND WITH GRAVEL (SM); MOIST, LOOSE TO MEDIUM DENSE, ESTIMATED 20-30% PASSING NO. 200 SIEVE, ROUNDED GRAVEL.
				4		ENCOUNTERED AT COBBLE LAYER BETWEEN 3 TO 4 FEET BGS.
				5		
2-1-2	8	--	--	6		
2-1-1		--	--			GRADES WITH DECREASING FINES CONTENT.
				7		
				8		
				9		
				10		
2-2-2	42	--	--	11		
2-2-1		--	--		SP/GP	GRAY GRAVELLY SAND TO SANDY GRAVEL (SP/GP); MOIST, DENSE, MEDIUM TO COARSE SAND, ROUNDED TO SUBROUNDED GRAVEL, SCATTERED COBBLES.
				12		
				13		
				14		
				15		
2-3-2	42	--	--	16	▽	
2-3-1		--	--		SP	GRAY POORY GRADED SAND (SP); WET, LOOSE, FINE TO MEDIUM SAND, CONSISTENCY OF MATERIAL IS REPRESENTATIVE OF THE DRILLING OBSERVATIONS AND FLOWING AND HEAVING SAND AND NOT THE BLOW COUNTS.
				17		
				18		
				19		
				20		

* BLOW COUNTS WERE CORRECTED FOR SAMPLER TYPE.

HOLDREGE & KULL

BORING B-2

PROJECT NO.	PROJECT NAME	ELEVATION (APPROX)	DATE	PAGE	BORING NO.
41278-01	HOMEWOOD MOUNTAIN RESORT	6236 FT	01/13/09	2 OF 2	B-2

DRILLING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CME 75	HOLLOW STEM AUGER MODIFIED CALIFORNIA (2.5" DIA.)	YES	YES

SAMPLE NO.	BLOW COUNTS (N) *	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
2-4-2	48	--	--	21	SP	GRAY POORY GRADED SAND (SP); WET, LOOSE, FINE TO MEDIUM SAND, CONSISTENCY OF MATERIAL IS REPRESENTATIVE OF THE DRILLING OBSERVATIONS AND FLOWING AND HEAVING SAND AND NOT THE BLOW COUNTS.
2-4-1		--	--			
				22		GRADES TO: COBBLY AT 23 TO 24 FT BGS.
				23		
				24		
				25		GRADES TO: VERY DENSE
2-5-2	56	--	--	26		
2-5-1		--	--			
				27	RX	GRAY ANDESITE (RX); SLIGHTLY WEATHERED, SLIGHTLY TO MODERATELY FRACTURED, STRONG TO VERY STRONG, POOR RQD BETWEEN 22 TO 25 FEET.
				28		BOREHOLE MET REFUSAL AT 27 FEET ON ROCK. CAVING WAS NOTED IN FLOWING SANDS.
				29		
				30		
				31		
				32		
				33		
				34		
				35		
				36		
				37		
				38		
				39		
				40		

* BLOW COUNTS WERE CORRECTED FOR SAMPLER TYPE.

HOLDREGE & KULL

BORING B-3

PROJECT NO.	PROJECT NAME	ELEVATION (APPROX)	DATE	PAGE	BORING NO.
41278-01	HOMEWOOD MOUNTAIN RESORT	6236 FT	01/14/09	1 OF 3	B-3

DRILLING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CME 75	HOLLOW STEM AUGER MODIFIED CALIFORNIA (2.5" DIA.)	YES	YES

SAMPLE NO.	BLOW COUNTS (N) *	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
					--	PAVEMENT STRUCTURAL SECTION: 4 TO 5 INCHES OF ASPHALT CONCRETE; 3 INCHES OF AGGREGATE BASE.
				1	FILL (SM)	FILL: DARK BROWN SILTY SAND WITH GRAVEL (SM); MOIST, LOOSE, ESTIMATED 15-20% PAASING NO. 200 SIEVE, FINE ROUNDED GRAVEL.
				2		
				3	SM/SC	DARK YELLOW BROWN SILTY SAND TO CLAYEY SAND (SM/SC); MOIST, LOOSE TO MEDIUM DENSE, FINE SAND ESTIMATED 30-35% PASSING NO. 200 SIEVE, TRACE FINE ROUNDED GRAVEL, WATER OBSERVED IN CUTTINGS BUT NO FREE WATER.
				4		
				5		
NO RECOVERY	14	--	--	6		GRADES TO: MEDIUM DENSE
		--	--			
				7		
				8	GW	GRAY SANDY GRAVEL WITH CLAY (GW); WET, MEDIUM DENSE, FINE TO MEDIUM SAND, FINE SUBROUNDED TO ROUNDED GRAVEL WITH CALCAREOUS STAINING, ROUNDED GRAVEL.
				9		
				10		
3-1-2	18	--	--	11		
3-1-1		--	--			
				12	SP/GP	GRAY POORLY GRADED SAND WITH GRAVEL TO POORLY GRADED GRAVEL WITH SAND (SP/GP); MOIST, DENSE, MEDIUM TO COARSE SAND, ROUNDED TO SUBROUNDED GRAVEL, SCATTERED COBBLES.
				13		
				14		
3-2-2	36	--	--	15	SP/GP	DARK BROWN POORLY GRADED SAND WITH GRAVEL TO POORLY GRADED GRAVEL WITH SAND (SP/GP); WET, DENSE, COARSE SAND, COARSE SUBROUNDED TO ROUNDED GRAVEL, FLOWING AND HEAVING SAND.
3-2-1		--	--			
				16		
				17		CONTAINS LENSES OF COARSE SAND
				18		
				19		
3-3-2	+64	--	--	20		DRILLING SLOWED DOWN DUE TO PRESENCE OF COBBLES.
3-3-1		--	--			

* BLOW COUNTS WERE CORRECTED FOR SAMPLER TYPE.

HOLDREGE & KULL

BORING B-3

PROJECT NO.	PROJECT NAME	ELEVATION (APPROX)	DATE	PAGE	BORING NO.
41278-01	HOMEWOOD MOUNTAIN RESORT	6236 FT	01/14/09	2 OF 3	B-3

DRILLING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CME 75	HOLLOW STEM AUGER MODIFIED CALIFORNIA (2.5" DIA.)	YES	YES

SAMPLE NO.	BLOW COUNTS (N) *	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
						DRILLING SLOWED DOWN DUE TO PRESENCE OF COBBLES.
				21	SP	GRAY POORY GRADED SAND (SP); WET, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM SAND, CONSISTENCY OF MATERIAL IS REPRESENTATIVE OF THE DRILLING OBSERVATIONS AND FLOWING AND HEAVING SAND AND NOT THE BLOW COUNTS.
				22		
				23		
				24		
				25		
				26		
				27		
				28		
				29		
				30		
				31		
				32		
				33		
				34		
				35		
				36		
				37		
				38		
				39		
				40		

* BLOW COUNTS WERE CORRECTED FOR SAMPLER TYPE.

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BORING B-3

PROJECT NO.	PROJECT NAME	ELEVATION (APPROX)	DATE	PAGE	BORING NO.
41278-01	HOMEWOOD MOUNTAIN RESORT	6236 FT	01/14/09	3 OF 3	B-3

DRILLING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CME 75	HOLLOW STEM AUGER MODIFIED CALIFORNIA (2.5" DIA.)	YES	YES

SAMPLE NO.	BLOW COUNTS (N) *	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
				41	SP	GRAY POORY GRADED SAND (SP); WET, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM SAND, FEW SUBROUNDED TO ROUNDED COBBLES BETWEEN 40 TO 45 FEET BGS, CONSISTENCY OF MATERIAL IS REPRESENTATIVE OF THE DRILLING OBSERVATIONS AND FLOWING AND HEAVING SAND AND NOT THE BLOW COUNTS.
				42		
				43		
				44		
				45		
				46		
				47		
				48		
				49		
				50		BOREHOLE TERMINATED AT 50 FEET. CAVING WAS NOTED IN FLOWING SANDS.
				51		
				52		
				53		
				54		
				55		
				56		
				57		
				58		
				59		
				60		

* BLOW COUNTS WERE CORRECTED FOR SAMPLER TYPE.

HOLDREGE & KULL

BORING B-4

PROJECT NO.	PROJECT NAME	ELEVATION (APPROX)	DATE	PAGE	BORING NO.
41278-01	HOMEWOOD MOUNTAIN RESORT	6236 FT	01/14/09	1 OF 2	B-4

DRILLING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CME 75	HOLLOW STEM AUGER MODIFIED CALIFORNIA (2.5" DIA.)	YES	YES

SAMPLE NO.	BLOW COUNTS (N) *	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
					--	GRAVEL PARKING LOT SURFACE: 2 TO 4 INCHES OF COARSE GRAVEL.
				1	SM	DARK BROWN SILTY SAND WITH GRAVEL (SM); MOIST, MEDIUM DENSE, FINE SAND, ESTIMATED 30-40% PASSING NO. 200 SIEVE, COARSE ROUNDED GRAVEL, TRACE COBBLES.
				2		
				3		
NO RECOVERY	26	--	--	4	SM/ML	DARK BROWN SILTY SAND TO SANDY SILT (SM/ML); MOIST, MEDIUM DENSE, FINE SAND, ESTIMATED 45-55% PASSING NO. 200 SIEVE, TRACE GRAVEL.
		--	--			
				5		
4-1-2	7	--	--	6	ML	LIGHT GRAY SANDY SILT (ML); MOIST, MEDIUM STIFF, FINE SAND, LOW PLASTIC FINES, TRACE ROOTLETS, TRACE FINE ROUNDED GRAVEL, MOTTLED WITH DARK YELLOW BROWN STREAKS, SAND AND GRAVEL LENSES.
4-1-1		--	--			
				7		
				8		
4-2-2	27	--	--	9		
4-2-1		--	--			GRADES TO: VERY STIFF.
				10		
				11		
				12	SP/GP	GRAY POORLY GRADED SAND WITH GRAVEL TO POORLY GRADED GRAVEL WITH SAND (SP/GP); WET, DENSE, COARSE SAND, FINE ROUNDED GRAVEL, FREE WATER IN SAMPLE BUT NOT IN BORELHOLE.
				13		
4-3-2	39	--	--	14		
4-3-1		--	--			
				15		
				16		GRADES TO DENSE, MEDIUM TO COARSE SAND, COARSE ROUNDED GRAVEL, MOTTLED WITH RED BROWN TO YELLOW BROWN.
				17		
				18		
4-4-2	58	--	--	19	SP	GRAY POORY GRADED SAND (SP); WET, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM SAND, CONSISTENCY OF MATERIAL IS REPRESENTATIVE OF THE DRILLING OBSERVATIONS AND FLOWING AND HEAVING SAND AND NOT THE BLOW COUNTS.
4-4-1		--	--	20		

* BLOW COUNTS WERE CORRECTED FOR SAMPLER TYPE.

HOLDREGE & KULL

BORING B-4

PROJECT NO.	PROJECT NAME	ELEVATION (APPROX)	DATE	PAGE	BORING NO.
41278-01	HOMEWOOD MOUNTAIN RESORT	6236 FT	01/14/09	2 OF 2	B-4

DRILLING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CME 75	HOLLOW STEM AUGER MODIFIED CALIFORNIA (2.5" DIA.)	YES	YES

SAMPLE NO.	BLOW COUNTS (N) *	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
				21	SP	GRAY POORY GRADED SAND (SP); WET, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM SAND, CONSISTENCY OF MATERIAL IS REPRESENTATIVE OF THE DRILLING OBSERVATIONS AND FLOWING AND HEAVING SAND AND NOT THE BLOW COUNTS.
				22		
				23		
4-5-2	34	--	--	24		5 FT OF SAND HEAVED INTO AUGER
4-5-1		--	--			
				25		
				26		
				27		
				28		
				29		
				30		5 FT OF SAND HEAVED INTO AUGER.
NO	+64	--	--	31		
RECOVERY		--	--			
				32		
				33		
				34		BOREHOLE MET REFUSAL AT 34 FEET ON ROCK; SIMILAR SUBSURFCE CONDITIONS AS OBSERVED IN B-2; VERY HARD DRILLING AT 34 FEET; ANDESITIC COBBLE IN TIP OF SAMPLER. CAVING WAS NOTED IN FLOWING SANDS.
				35		
				36		
				37		
				38		
				39		
				40		

* BLOW COUNTS WERE CORRECTED FOR SAMPLER TYPE.

HOLDREGE & KULL

BORING B-5

PROJECT NO.	PROJECT NAME	ELEVATION (APPROX)	DATE	PAGE	BORING NO.
41278-01	HOMEWOOD MOUNTAIN RESORT	6239 FT	01/14/09	1 OF 2	B-5

DRILLING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CME 75	HOLLOW STEM AUGER MODIFIED CALIFORNIA (2.5" DIA.)	YES	YES

SAMPLE NO.	BLOW COUNTS (N) *	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
				1	--	SNOW AND GRAVEL AT SURFACE IN PARKING LOT: 2 TO 4 INCHES OF COARSE GRAVEL.
				2	CL	DARK BROWN SILTY CLAY TO SANDY CLAY (CL); MOIST, MEDIUM STIFF, FINE SAND, LOW PLASTIC FINES, FINE ROUNDED GRAVEL, ORGANIC MATTER/ROOTS.
				3 4 5	CL	LIGHT GRAY SANDY CLAY TO SILTY CLAY (CL); MOIST, MEDIUM STIFF, FINE SAND, LOW PLASTIC FINES, TRACE ROOTLETS, TRACE FINE ROUNDED GRAVEL, MOTTLED WITH DARK YELLOW BROWN STREAKS, SAND AND GRAVEL LENSES, FINE CHARCOAL FRAGMENTS.
NO RECOVERY	34	-- --	-- --	6	ML	DARK YELLOW BROWN SILT WITH SAND (ML); MOIST, MEDIUM STIFF, FINE SAND, LOW PLASTIC FINES, TRACE ROOTLETS, TRACE FINE ROUNDED GRAVEL, MOTTLED WITH LIGTH GRAY, SAND AND GRAVEL LENSES, NO SAMPLE RECOVER AT 5-6.5 FT DUE TO COBBLE.
5-1-2 5-1-1	6	-- --	-- --	7 8		
5-2-2 5-2-1	10	-- --	-- --	9 10 11	SP/SM	GRAY POORLY GRADED SAND WITH SILT AND GRAVEL (SP/SM); MOIST TO WET, LOOSE TO MEDIUM DENSE, MEDIUM SAND, FINE ROUNDED GRAVEL, FREE WATER IN SAMPLE BUT NOT IN BORELHOLE, MOTTLED WITH DARK YELLOW BROWN.
5-3-2 5-3-1	33	-- --	-- --	12 13 14 15 16	SP/GP	GRAY POORLY GRADED SAND WITH GRAVEL TO POORLY GRADED GRAVEL WITH SAND (SP/GP); WET, DENSE, MEDIUM TO COARSE SAND, COARSE ROUNDED TO SUBROUNDED GRAVEL, MOTTLED WITH YELLOW BROWN.
				17 18 19 20		HARDER DRILLING AT 20 FT BGS.

* BLOW COUNTS WERE CORRECTED FOR SAMPLER TYPE.

HOLDREGE & KULL

BORING B-5

PROJECT NO.	PROJECT NAME	ELEVATION (APPROX)	DATE	PAGE	BORING NO.
41278-01	HOMEWOOD MOUNTAIN RESORT	6236 FT	01/14/09	2 OF 2	B-5

DRILLING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CME 75	HOLLOW STEM AUGER MODIFIED CALIFORNIA (2.5" DIA.)	YES	YES

SAMPLE NO.	BLOW COUNTS (N) *	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
5-4-2	50	--	--	21		SOFT DRILLING AT 21 FT BGS.
5-4-1		--	--			
				22	SP	GRAY POORY GRADED SAND (SP); WET, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM SAND, CONSISTENCY OF MATERIAL IS REPRESENTATIVE OF THE DRILLING OBSERVATIONS AND FLOWING AND HEAVING SAND AND NOT THE BLOW COUNTS.
				23		COBBLES BETWEEN 23 TO 24 FT BGS.
				24		
				25		SAND HEAVED INTO AUGER.
5-5-2	35	--	--	26	GP	GRAY POORLY GRADED GRAVEL WITH SAND (GP); WET, MEDIUM DENSE TO DENSE, FINE TO MEDIUM SAND, FINE TO COARSE SUBGROUNDED TO ROUNDED GRAVEL, DRILLING HARD BETWEEN 26 TO 27 FT AND VERY HARD AT 27 FEET BGS.
5-5-1		--	--			
				27		BOREHOLE MET REFUSAL AT 27 FEET ON ROCK.
				28		CAVING WAS NOTED IN FLOWING SANDS.
				29		
				30		
				31		
				32		
				33		
				34		
				35		
				36		
				37		
				38		
				39		
				40		

* BLOW COUNTS WERE CORRECTED FOR SAMPLER TYPE.

HOLDREGE & KULL

BORING B-6

PROJECT NO.	PROJECT NAME	ELEVATION (APPROX)	DATE	PAGE	BORING NO.
41278-01	HOMEWOOD MOUNTAIN RESORT	6236 FT	01/15/09	1 OF 2	B-6

DRILLING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CME 75	HOLLOW STEM AUGER MODIFIED CALIFORNIA (2.5" DIA.)	YES	YES

SAMPLE NO.	BLOW COUNTS (N) *	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
				1	--	PAVEMENT STRUCTURAL SECTION: 4 INCHES OF ASPHALT CONCRETE; 2 TO 3 INCHES OF AGGREGATE BASE.
				2	FILL (SM)	FILL; DARK BROWN SILTY SAND WITH GRAVEL (SM); MOIST, MEDIUM DENSE, FINE TO MEDIUM SAND, ESTIMATED 15-25% PASSING NO. 200 SIEVE, COARSE ROUNDED GRAVEL.
6-1-2	6	--	--			
6-1-1		--	--	3	SM	DARK YELLOW BROWN SILTY SAND TO SAND WITH SILT (SM/SP-SM); MOIST, LOOSE, FINE TO MEDIUM SAND, ESTIMATED 10-20% PASSING NO. 200 SIEVE, TRACE ROOTLETS.
				4	SM	DARK YELLOW BROWN SILTY SAND (SM); MOIST, MEDIUM DENSE, FINE TO MEDIUM SAND, LOW PLASTIC FINES, TRACE FINE ROUNDED GRAVEL, MOTTLED WITH LIGHT GRAY.
				5		
6-2-2	14	--	--			
6-2-1		--	--	6		
				7		
				8	SP/GP	GRAY POORLY GRADED SAND WITH GRAVEL TO POORLY GRADED GRAVEL WITH SAND (SP/GP); MOIST, LOOSE TO MEDIUM DENSE, MEDIUM SAND, FINE ROUNDED GRAVEL, CONTACT BASED ON SUBSURFACE DRILLING CONDITIONS.
				9		
				10		
6-3-2	20	--	--			
6-3-1		--	--	11		
				12		
				13	▽	GRADES TO WET, DENSE, MEDIUM TO COARSE SAND, COARSE ROUNDED TO SUBROUNDED GRAVEL.
				14		
				15		
6-4-2	41	--	--			
6-4-1		--	--	16		
				17	SP	GRAY POORY GRADED SAND (SP); WET, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM SAND, CONSISTENCY OF MATERIAL IS REPRESENTATIVE OF THE DRILLING OBSERVATIONS AND FLOWING AND HEAVING SAND AND NOT THE BLOW COUNTS.
				18		
				19		
				20		HEAVING SAND UP THROUGH AUGERS AT 20 FT.

* BLOW COUNTS WERE CORRECTED FOR SAMPLER TYPE.

HOLDREGE & KULL

BORING B-6

PROJECT NO.	PROJECT NAME	ELEVATION (APPROX)	DATE	PAGE	BORING NO.
41278-01	HOMEWOOD MOUNTAIN RESORT	6236 FT	01/15/09	2 OF 2	B-6

DRILLING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CME 75	HOLLOW STEM AUGER MODIFIED CALIFORNIA (2.5" DIA.)	YES	YES

SAMPLE NO.	BLOW COUNTS (N) *	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
				21	SP	GRAY POORY GRADED SAND (SP); WET, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM SAND, CONSISTENCY OF MATERIAL IS REPRESENTATIVE OF THE DRILLING OBSERVATIONS AND FLOWING AND HEAVING SAND AND NOT THE BLOW COUNTS.
				22		
				23		AUGERS FELL RAPIDLY AT 0 PSI DRILLING DOWN PRESSURE.
				24		
				25		
				26		
				27		
				28		
				29		
				30		
				31		
				32		
				33		
				34		
				35	GP	GRAY POORLY GRADED GRAVEL WITH SAND (GP); WET, MEDIUM DENSE TO DENSE, FINE TO MEDIUM SAND, FINE TO COARSE SUBGROUNDED TO ROUNDED GRAVEL, SOME COBBLES BETWEEN 35 TO 37 FT, AUGERS FELL FROM 37 TO 40 FT.
				36		
				37		
				38		
				39	RX	GRAY ANDESITE (RX); SLIGHTLY TO MODERATELY WEATHERED, MODERATELY FRACTURED, STRONG TO VERY STRONG.
				40		BOREHOLE MET REFUSAL AT 40 FEET ON ROCK. CAVING WAS NOTED IN FLOWING SANDS.

* BLOW COUNTS WERE CORRECTED FOR SAMPLER TYPE.

HOLDREGE & KULL

BORING B-7

PROJECT NO.	PROJECT NAME	ELEVATION (APPROX)	DATE	PAGE	BORING NO.
41278-01	HOMEWOOD MOUNTAIN RESORT	6237 FT	01/15/09	1 OF 3	B-7

DRILLING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CME 75	HOLLOW STEM AUGER MODIFIED CALIFORNIA (2.5" DIA.)	YES	YES

SAMPLE NO.	BLOW COUNTS (N) *	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
				1	--	PAVEMENT STRUCTURAL SECTION: 2 TO 3 INCHES OF ASPHALT CONCRETE; 2 TO 3 INCHES OF AGGREGATE BASE.
7-1-1	14	-- / --	-- / --	2	FILL (SM)	FILL: DARK BROWN SILTY SAND WITH GRAVEL (SM); MOIST, MEDIUM DENSE, FINE TO COARSE SAND, ESTIMATED 15-20% FINES, FINE ROUNDED GRAVEL.
				3	SM/SC	DARK YELLOW BROWN SILTY SAND TO CLAYEY SAND (SM/SC); DRY, MEDIUM DENSE, FINE TO MEDIUM SAND ESTIMATED 30-35% PASSING NO. 200 SIEVE, TRACE FINE ROUNDED GRAVEL.
				4	SM	DARK YELLOW BROWN SILTY SAND (SM); MOIST, MEDIUM DENSE, FINE TO MEDIUM SAND, ESTIMATED 20-30% LOW PLASTIC FINES, TRACE FINE ROUNDED GRAVEL, MOTTLED WITH LIGHT GRAY, CEMENTED.
				5		
7-2-2 / 7-2-1	56	-- / --	-- / --	6		GRADES TO: VERY DENSE
				7		
				8		HARD DRILLING, GRANULAR SOIL.
				9		
				10		
7-3-2 / 7-3-1	25	-- / --	-- / --	11		
				12	SP-SM	GRAY POORLY GRADED SAND WITH SILT AND GRAVEL (SP-SM); MOIST, MEDIUM DENSE, MEDIUM TO COARSE SAND, FINE ROUNDED TO SUBROUNDED GRAVEL.
				13		
				14	▽	EASIER DRILLING AT 14 FT BGS.
				15	SP	GRAY POORY GRADED SAND (SP); WET, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM SAND, CONSISTENCY OF MATERIAL IS REPRESENTATIVE OF THE DRILLING OBSERVATIONS AND FLOWING AND HEAVING SAND AND NOT THE BLOW COUNTS.
7-4-2 / 7-4-1	42	-- / --	-- / --	16		AUGER FELL RAPIDLY AT 0 PSI DRILLING PRESSURE.
				17		
				18		
				19		TRACE COBBLE LAYERS.
				20		

* BLOW COUNTS WERE CORRECTED FOR SAMPLER TYPE.

HOLDREGE & KULL

BORING B-7

PROJECT NO.	PROJECT NAME	ELEVATION (APPROX)	DATE	PAGE	BORING NO.
41278-01	HOMEWOOD MOUNTAIN RESORT	6237 FT	01/15/09	2 OF 3	B-7

DRILLING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CME 75	HOLLOW STEM AUGER MODIFIED CALIFORNIA (2.5" DIA.)	YES	YES

SAMPLE NO.	BLOW COUNTS (N) *	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
					SP	GRAY POORY GRADED SAND (SP); WET, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM SAND, CONSISTENCY OF MATERIAL IS REPRESENTATIVE OF THE DRILLING OBSERVATIONS AND FLOWING AND HEAVING SAND AND NOT THE BLOW COUNTS.
				21		
				22		
				23		
				24		
				25		
				26		COBBLES BETWEEN 26 TO 27 FT.
				27		
				28		
				29		
				30		
				31		COBBLES BETWEEN 31 TO 33 FT.
				32		
				33		HARD DRILLING BETWEEN 33 TO 33.5 FT.
				34		SOFT DRILLING / FLOWING SAND.
				35		
				36		
				37		
				38		
				39		
				40		

* BLOW COUNTS WERE CORRECTED FOR SAMPLER TYPE.

HOLDREGE & KULL

BORING B-7

PROJECT NO.	PROJECT NAME	ELEVATION (APPROX)	DATE	PAGE	BORING NO.
41278-01	HOMEWOOD MOUNTAIN RESORT	6237 FT	01/15/09	3 OF 3	B-7

DRILLING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CME 75	HOLLOW STEM AUGER MODIFIED CALIFORNIA (2.5" DIA.)	YES	YES

SAMPLE NO.	BLOW COUNTS (N) *	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
					SP	GRAY POORY GRADED SAND (SP); WET, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM SAND, FEW SUBROUNDED TO ROUNDED COBBLES.
				41		SOFT DRILLING / FLOWING SAND.
				42		
				43		
				44		
				45		
				46		
				47		
				48		
				49		
				50		
				51		
				52		HARDER DRILLING BETWEEN 52 TO 52.5 FT.
				53		FLOWING SAND HEAVED INTO AUGER BEYOND 52.5 FT.
				54		
				55		
				56		
				57		HARDER DRILLING BETWEEN 57 TO 60 FT.
				58		
				59		
				60		BOREHOLE TERMINATED AT 60 FEET.
						CAVING WAS NOTED IN FLOWING SANDS.

* BLOW COUNTS WERE CORRECTED FOR SAMPLER TYPE.

HOLDREGE & KULL

BORING B-8

PROJECT NO.	PROJECT NAME	ELEVATION (APPROX)	DATE	PAGE	BORING NO.
41278-01	HOMEWOOD MOUNTAIN RESORT	6235 FT	01/15/09	1 OF 3	B-8

DRILLING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CME 75	HOLLOW STEM AUGER MODIFIED CALIFORNIA (2.5" DIA.)	YES	YES

SAMPLE NO.	BLOW COUNTS (N) *	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
					--	PAVEMENT STRUCTURAL SECTION: 5 TO 6 INCHES OF ASPHALT CONCRETE; 2 TO 3 INCHES OF AGGREGATE BASE.
				1	FILL (SM)	FILL; DARK BROWN SILTY SAND WITH GRAVEL (SM); MOIST, MEDIUM DENSE, FINE TO COARSE SAND, ESTIMATED 15-20% PASSING NO. 200 SIEVE, FINE ROUNDED GRAVEL.
				2		
8-1-2	9	--	--		SM	DARK YELLOW BROWN SILTY SAND (SM); MOIST, MEDIUM DENSE, FINE TO MEDIUM SAND, ESTIMATED 30-35% PASSING NO. 200 SIEVE, TRACE FINE ROUNDED GRAVEL, TRACE ROOTLETS.
8-1-1		--	--	3		
				4	SP-SM	GRAY TO OLIVE BROWN POORLY GRADED SAND WITH SILT (SP-SM); MOIST, MEDIUM DENSE, MEDIUM SAND, LOW PLASTIC FINES, TRACE FINE ROUNDED GRAVEL.
				5		
8-2-2	49	--	--			GRADES TO: DENSE
8-2-1		--	--	6		
				7		
				8	SP	GRAY POORLY GRADED SAND WITH GRAVEL (SP); MOIST, MEDIUM DENSE, MEDIUM TO COARSE SAND, FINE ROUNDED TO ANGULAR GRAVEL, MOTTLED WITH RED BROWN.
				9		
				10		
8-3-2	29	--	--			
8-3-1		--	--	11		
				12		
				13		
				14		SOFT DRILLING AT 14.5 FT, FLOWING SAND HEAVING INTO AUGER.
				15	SP	DARK BROWN POORLY GRADED SAND (SP); WET, LOOSE TO MEDIUM DENSE, COARSE SAND, COARSE ROUNDED GRAVEL, FLOWING AND HEAVING SAND, CONSISTENCY OF MATERIAL IS REPRESENTATIVE OF THE DRILLING OBSERVATIONS AND FLOWING AND HEAVING SAND AND NOT THE BLOW COUNTS.
8-4-2	41	--	--			
8-4-1		--	--	16		
				17		
				18		
				19		
				20		DRILLING SLOWED DOWN DUE TO PRESENCE OF COBBLES.

* BLOW COUNTS WERE CORRECTED FOR SAMPLER TYPE.

HOLDREGE & KULL

BORING B-8

PROJECT NO.	PROJECT NAME	ELEVATION (APPROX)	DATE	PAGE	BORING NO.
41278-01	HOMEWOOD MOUNTAIN RESORT	6235 FT	01/15/09	2 OF 3	B-8

DRILLING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CME 75	HOLLOW STEM AUGER MODIFIED CALIFORNIA (2.5" DIA.)	YES	YES

SAMPLE NO.	BLOW COUNTS (N) *	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
					SP	GRAY POORY GRADED SAND (SP); WET, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM SAND, CONSISTENCY OF MATERIAL IS REPRESENTATIVE OF THE DRILLING OBSERVATIONS AND FLOWING AND HEAVING SAND AND NOT THE BLOW COUNTS.
				21		
				22		COBBLES BETWEEN 21 AND 22 FT.
						VERY SOFT DRILLING
				23		
				24		
				25		
				26		
				27		
				28		
				29		
				30		
				31		
				32		
				33		
				34		
				35		
				36		DRILLED 5 FT PER 1 MINUTE
				37		
				38		
				39		
				40		

* BLOW COUNTS WERE CORRECTED FOR SAMPLER TYPE.

HOLDREGE & KULL

BORING B-8

PROJECT NO.	PROJECT NAME	ELEVATION (APPROX)	DATE	PAGE	BORING NO.
41278-01	HOMEWOOD MOUNTAIN RESORT	6235 FT	01/15/09	3 OF 3	B-8

DRILLING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CME 75	HOLLOW STEM AUGER MODIFIED CALIFORNIA (2.5" DIA.)	YES	YES

SAMPLE NO.	BLOW COUNTS (N) *	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
					SP	GRAY POORY GRADED SAND (SP); WET, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM SAND, CONSISTENCY OF MATERIAL IS REPRESENTATIVE OF THE DRILLING OBSERVATIONS AND FLOWING AND HEAVING SAND AND NOT THE BLOW COUNTS.
				41		
				42		DRILLED 5 FT PER MINUTE
				43		
				44		
				45		
				46		
				47		
				48		
				49		COBBLES BETWEEN 49 AND 49.5 FT.
				50		
				51		HARDER DRILLING AT 51 FT, STALLED DRILL RIG.
				52		EASY DRILLING AT 52.5 FT,
				53		
				54		
				55		
				56		SLOWER DRILLING AT 56 FT,
				57		DRILLED 5 FT PER MINUTE LOWER 15 FEET OF AUGER FILLED WITH SAND/HEAVING
				58		
				59		
				60		BOREHOLE TERMINATED AT 60 FEET. CAVING WAS NOTED IN FLOWING SANDS.

* BLOW COUNTS WERE CORRECTED FOR SAMPLER TYPE.

HOLDREGE & KULL

BORING B-9

PROJECT NO.	PROJECT NAME	ELEVATION (APPROX)	DATE	PAGE	BORING NO.
41278-01	HOMEWOOD MOUNTAIN RESORT	6237 FT	01/15/09	1 OF 3	B-9

DRILLING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CME 75	HOLLOW STEM AUGER MODIFIED CALIFORNIA (2.5" DIA.)	YES	YES

SAMPLE NO.	BLOW COUNTS (N) *	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
				1	--	PAVEMENT STRUCTURAL SECTION: 4 TO 5 INCHES OF ASPHALT CONCRETE; 2 TO 3 INCHES OF AGGREGATE BASE.
9-1-2	28	--	--	2	FILL (SM)	FILL; DARK BROWN SILTY SAND WITH GRAVEL (SM); MOIST, MEDIUM DENSE, FINE TO COARSE SAND, ESTIMATED 15-20% PASSING NO. 200 SIEVE, COARSE ROUNDED GRAVEL.
9-1-1		--	--			
				3	SP	OLIVE BROWN POORLY GRADED SAND WITH GRAVEL (SP); MOIST, MEDIUM DENSE, FINE TO MEDIUM SAND ESTIMATED 30-35% PASSING NO. 200 SIEVE, COARSE SUBROUNDED GRAVEL.
9-2-2	21	--	--	4		
9-2-1		--	--			
				5		
				6		
				7		
				8		
9-3-2	38	--	--	9	SP	YELLOW BROWN TO GRAY BROWN POORLY GRADED SAND WITH GRAVEL (SP); MOIST, DENSE, FINE TO MEDIUM SAND, COARSE ROUNDED GRAVEL.
9-3-1		--	--			
				10		
				11		
				12		
				13		
9-4-2	27	--	--	14	▽	SOFTER DRILLING AT 14 FT.
9-4-1		--	--		SP/GP	GRAY POORLY GRADED SAND WITH GRAVEL TO POORLY GRADED GRAVEL WITH SAND (SP/GP); WET, LOOSE TO MEDIUM DENSE, MEDIUM TO COARSE SAND, COARSE ROUNDED GRAVEL.
				15		
				16		
				17		SOFTER DRILLING, FLOWING AND HEAVING SAND.
9-4-2	+64	--	--	18	SP	GRAY POORY GRADED SAND (SP); WET, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM SAND, CONSISTENCY OF MATERIAL IS REPRESENTATIVE OF THE DRILLING OBSERVATIONS AND FLOWING AND HEAVING SAND AND NOT THE BLOW COUNTS.
9-4-1		--	--			
				19		VARIOUS GRAVEL AND COBBLE LENSES, COBBLE IN DRILL BIT.
				20		

* BLOW COUNTS WERE CORRECTED FOR SAMPLER TYPE.

HOLDREGE & KULL

BORING B-9

PROJECT NO.	PROJECT NAME	ELEVATION (APPROX)	DATE	PAGE	BORING NO.
41278-01	HOMEWOOD MOUNTAIN RESORT	6237 FT	01/15/09	2 OF 3	B-9

DRILLING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CME 75	HOLLOW STEM AUGER MODIFIED CALIFORNIA (2.5" DIA.)	YES	YES

SAMPLE NO.	BLOW COUNTS (N) *	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
					SP	GRAY POORY GRADED SAND (SP); WET, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM SAND, CONSISTENCY OF MATERIAL IS REPRESENTATIVE OF THE DRILLING OBSERVATIONS AND FLOWING AND HEAVING SAND AND NOT THE BLOW COUNTS.
				21		
				22		DRILLED 5 FT PER 1 MINUTE
				23		
				24		
				25		
				26		
				27		DRILLED 5 FT PER 1 MINUTE
				28		
				29		
				30		
				31		
				32		DRILLED 5 FT PER 1 MINUTE
				33		
				34		
				35		
				36		
				37		DRILLED 5 FT PER 1 MINUTE
				38		
				39		
				40		

* BLOW COUNTS WERE CORRECTED FOR SAMPLER TYPE.

HOLDREGE & KULL

BORING B-9

PROJECT NO.	PROJECT NAME	ELEVATION (APPROX)	DATE	PAGE	BORING NO.
41278-01	HOMEWOOD MOUNTAIN RESORT	6237 FT	01/15/09	3 OF 3	B-9

DRILLING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
CME 75	HOLLOW STEM AUGER MODIFIED CALIFORNIA (2.5" DIA.)	YES	YES

SAMPLE NO.	BLOW COUNTS (N) *	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
					SP	GRAY POORY GRADED SAND (SP); WET, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM SAND, CONSISTENCY OF MATERIAL IS REPRESENTATIVE OF THE DRILLING OBSERVATIONS AND FLOWING AND HEAVING SAND AND NOT THE BLOW COUNTS.
				41		
				42		DRILLED 5 FT PER 1 MINUTE
				43		
				44		
				45		
				46		
				47		DRILLED 5 FT PER 1 MINUTE
				48		
				49		
				50		
				51		
				52		
				53		COBBLY AT 53 FT.
				54		BOREHOLE MET REFUSAL AT 54 FEET ON ROCK. CAVING WAS NOTED IN FLOWING SANDS.
				55		
				56		
				57		
				58		
				59		
				60		

* BLOW COUNTS WERE CORRECTED FOR SAMPLER TYPE.

HOLDREGE & KULL

APPENDIX C Laboratory Test Results

Particle Size Distribution

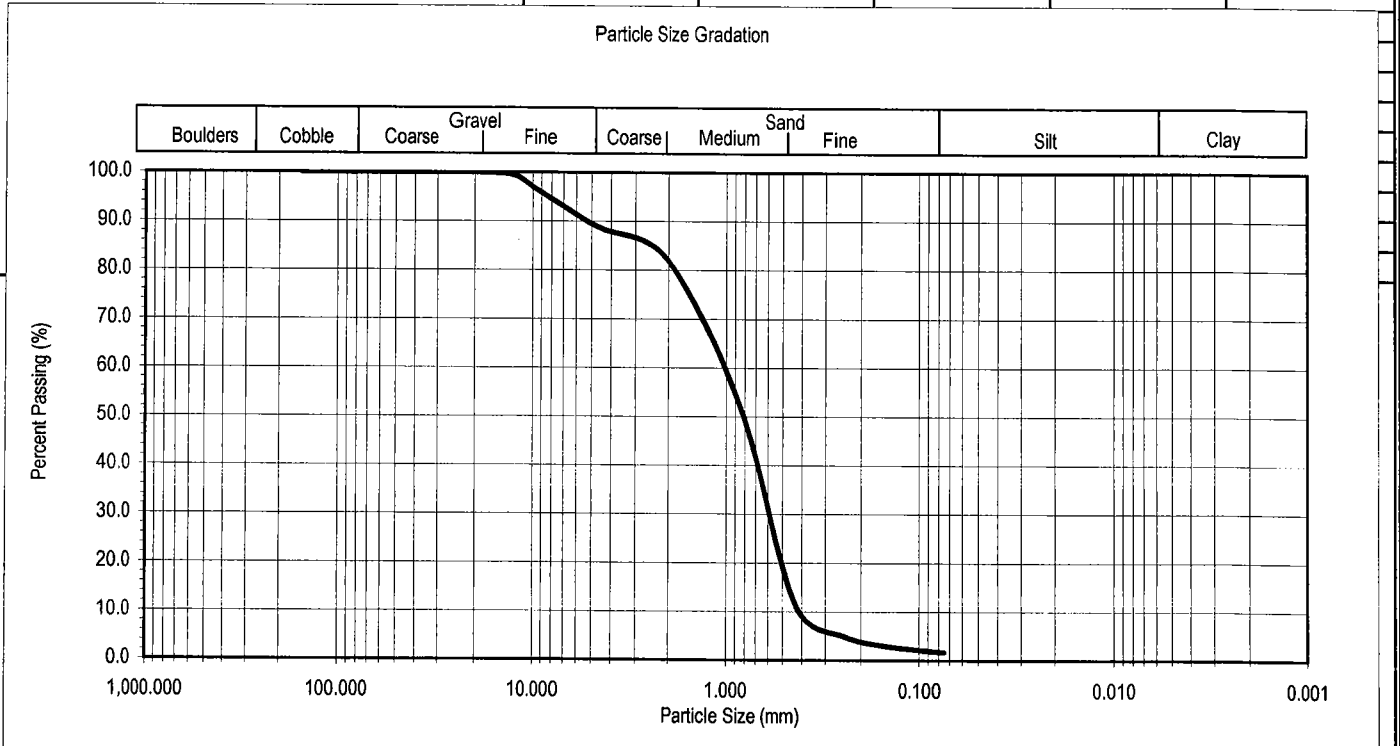
ASTM D422

Project No.:	41278-01	Project Name:	Homewood-North Base Lodge		Date: 2/4/2009
Sample No.:	2-4-2	Boring/Trench:	B-2	Depth, (ft.): 21	Tested By: BLk/MLF
Description:	Dark Gray (GLEY2 4/1) Poorly Graded Sand (SP)				Checked By: MLHF
Sample Location:					Lab. No.: 9-018

Sieve Size (U.S. Standard)	Particle Diameter Inches (in.)	Particle Diameter Millimeter (mm)	Dry Weight on Sieve Retained On Sieve (gm)	Dry Weight on Sieve Accumulated On Sieve (gm)	Dry Weight on Sieve Passing Sieve (gm)	Percent Passing (%)
6 Inch	6.0000	152.4	0.00	0.0	515.6	**100.0**
3 Inch	3.0000	76.2	0.00	0.0	515.6	**100.0**
2 Inch	2.0000	50.8	0.00	0.0	515.6	**100.0**
1.5 Inch	1.5000	38.1	0.00	0.0	515.6	**100.0**
1.0 Inch	1.0000	25.4	0.00	0.0	515.6	**100.0**
3/4 Inch	0.7500	19.1	0.00	0.0	515.6	**100.0**
1/2 Inch	0.5000	12.7	2.79	2.8	512.9	**99.5**
3/8 Inch	0.3750	9.5	15.26	18.1	497.6	**96.5**
#4	0.1870	4.7500	38.52	56.6	459.1	**89.0**
#10	0.0787	2.0000	37.01	93.6	422.1	**81.9**
#20	0.0335	0.8500	152.87	246.5	269.2	**52.2**
#40	0.0167	0.4250	213.36	459.8	55.8	**10.8**
#60	0.0098	0.2500	29.83	489.6	26.0	**5.0**
#100	0.0059	0.1500	10.65	500.3	15.4	**3.0**
#200	0.0030	0.0750	6.47	506.8	8.9	**1.7**

Cc = 0.84

Cu = 2.50



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Expansion Index/Swell

ASTM D4829/UBC 18.2

Project No.:	41278-01	Project Name:	Homewood-North Base Lodge		Date: 2/4/2009
Sample No.:	4-2-1&4-2-2	Boring/Trench No.:	B-4	Depth (ft.) 8.5-9	Tested By: BLK/MLF
Soil Description:	Grayish Brown (10YR 5/2) Sandy Silt (ML)				Checked By: MLHF
Estimated % of sample retained on #4:					Lab. No.: 9-018

Specimen Type:	Undisturbed:	Disturbed:	Remolded to:	ASTM Guidelines
Tube Dia. (Inch) =		Ring Dia. (Inch) = 4	Ring Height (Inch) = 1.00	

FIELD DATA Tube Sample Moisture & Density			LAB DATA	Test wt. 144 Initial	Final	Test wt. Initial	Final	Test wt. Initial	Final
Tare Tube Number			Tare Number	E					
Tare Weight	(gr)	50.77	Tare Ring Weight (gr)	200.53	200.53				
Wet Soil + Tare	(gr)	162.44	Tare Pan Weight (gr)	0.00	156.55				
Dry Soil + Tare	(gr)	149.75	Wet Soil + Tare (gr)	543.00	725.00				
Weight of Water	(gr)	12.69	Dry Soil + Tare (gr)	503.95	660.50	0.00		0.00	
Dry Soil Weight	(gr)	98.98	Weight of Water (gr)	39.05	64.50	0.00	0.00	0.00	0.00
Moisture Content	(%)	12.82	Dry Soil Weight (gr)	303.42	303.42	0.00	0.00	0.00	0.00
Soil Height	(In.)		Moisture Content (%)	12.87	21.26	0.00	0.00	0.00	0.00
Wet Unit Weight	(pcf)	#REF!	Wet Unit Weight (pcf)	103.83	111.27				
Dry Unit Weight	(pcf)	#REF!	Dry Unit Weight (pcf)	91.99	91.76				
			Sample Height (Inches)	1.00	1.003				
Specific Gravity		2.7	Percent Saturation	41.79	68.65				

Expansion Index Number

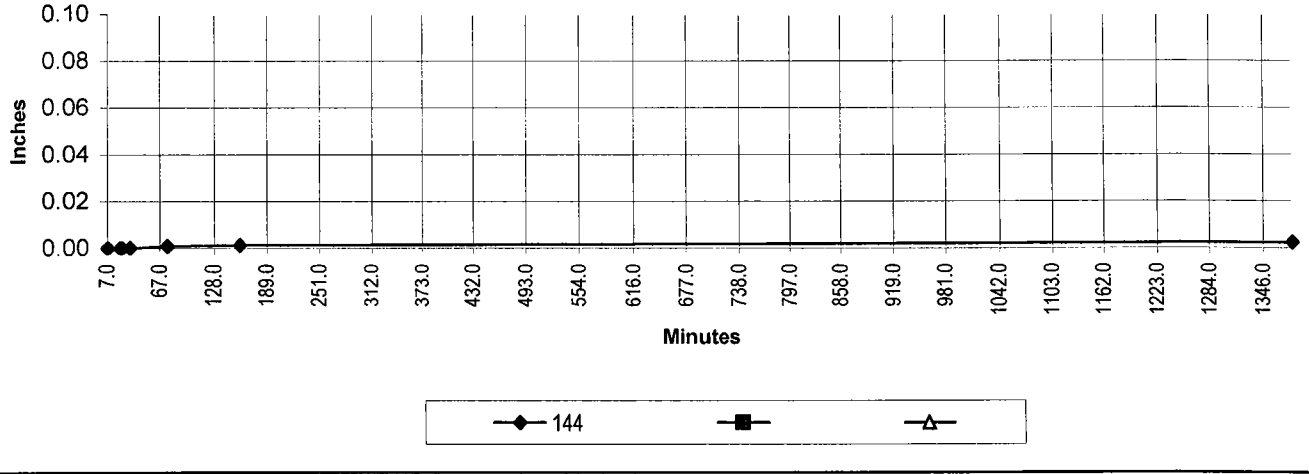
Surcharge (psf)	Uncorrected	Corrected to 50% Saturation
Test wt. 144	2	-1
Test wt.		
Test wt.		

Elapsed Time (m:s)	Change in Height (Inches)	Elapsed Time (m:s)	Change in Height (Inches)	Elapsed Time (m:s)	Change in Height (Inches)
7.0	0.0000				
21.0	0.0000				
23.0	0.0000				
32.0	0.0001				
75.0	0.0009				
157.0	0.0013				
1380.0	0.0025				

Expansion Index Values and Descriptions

Expansion Index	Potential Expansion
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
Above 130	Very High

Expansion Versus Time



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Atterberg Indices

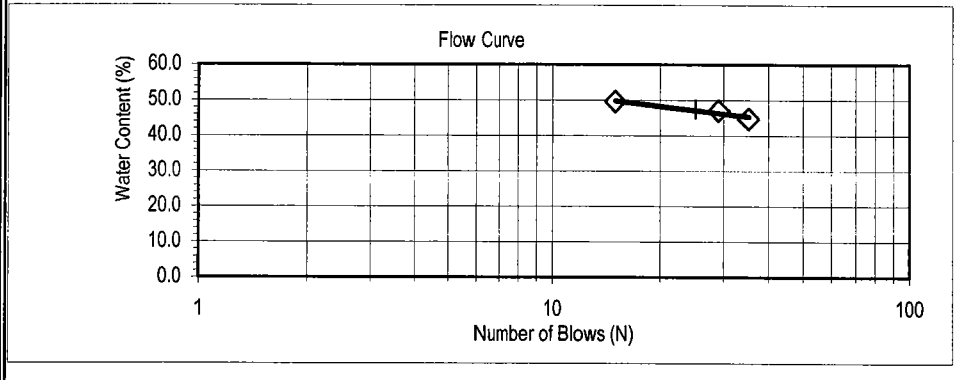
ASTM D4318

Project No.:	41278-01	Project Name:	Homewood-North Base Lodge	Date:	2/4/2009
Sample No.:	5-1-1&5-1-2	Boring/Trench: B-5	Depth, (ft.): 7.5-8	Tested By:	BLk/MLF
Description:	Light Olive Brown (2.5UR 5/3) Silt with Sand (ML)			Checked By:	MLHF
Sample Location:				Lab. No.:	9-018

Estimated % of Sample Retained on No. 40 Sieve: _____ Sample Air Dried: yes

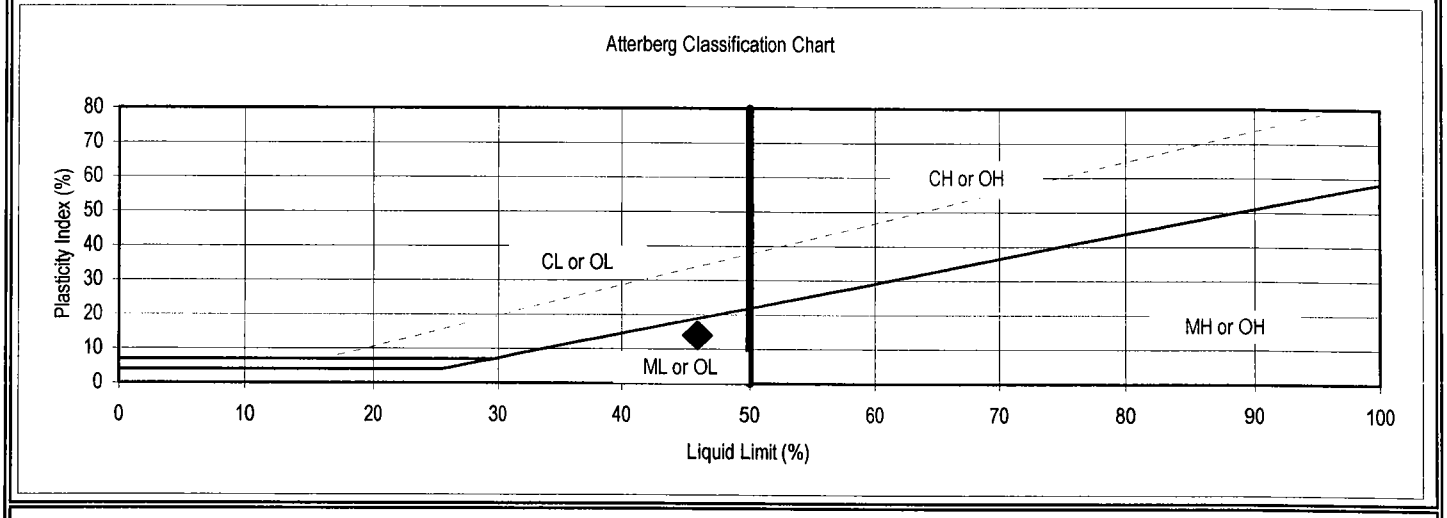
Test Method A or B: A

LIQUID LIMIT:						PLASTIC LIMIT:		
Sample No.:	1	2	3	4	5	1	2	3
Pan ID:	HK	LC	AT			LF	I	
Wt. Pan (gr)	14.95	15.02	15.29			10.83	11.29	
Wt. Wet Soil + Pan (gr)	24.84	20.84	25.38			14.84	16.47	
Wt. Dry Soil + Pan (gr)	21.56	18.98	22.26			13.87	15.23	
Wt. Water (gr)	3.28	1.86	3.12			0.97	1.24	
Wt. Dry Soil (gr)	6.61	3.96	6.97			3.04	3.94	
Water Content (%)	49.6	47.0	44.8			31.9	31.5	
Number of Blows, N	15	29	35					
				LIQUID LIMIT =	46		PLASTIC LIMIT =	32



Plasticity Index = 14

Group Symbol = ML



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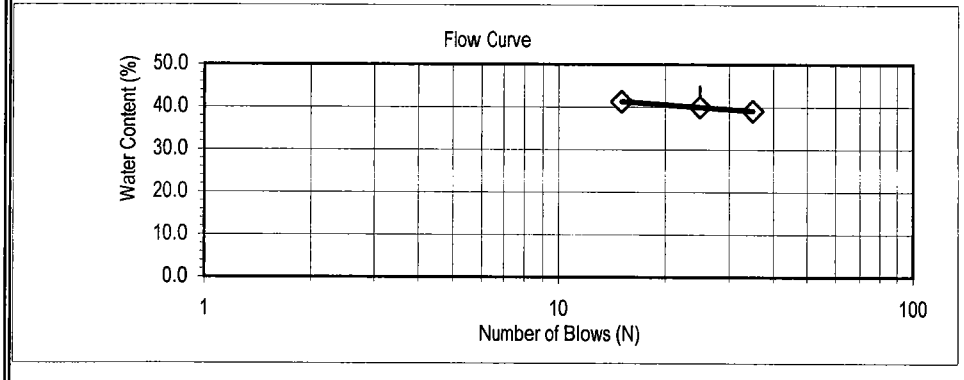
Atterberg Indices

ASTM D4318

Project No.: 41278-01 Project Name: Homewood-North Base Lodge Date: 2/4/2009
Sample No.: 6-1-2 Boring/Trench: B-6 Depth, (ft.): 3 Tested By: BLk/MLF
Description: Dark Reddish Brown (5YR 2.5/2) Silty Sand (SM) Checked By: MLHF
Sample Location: Lab. No.: 9-018

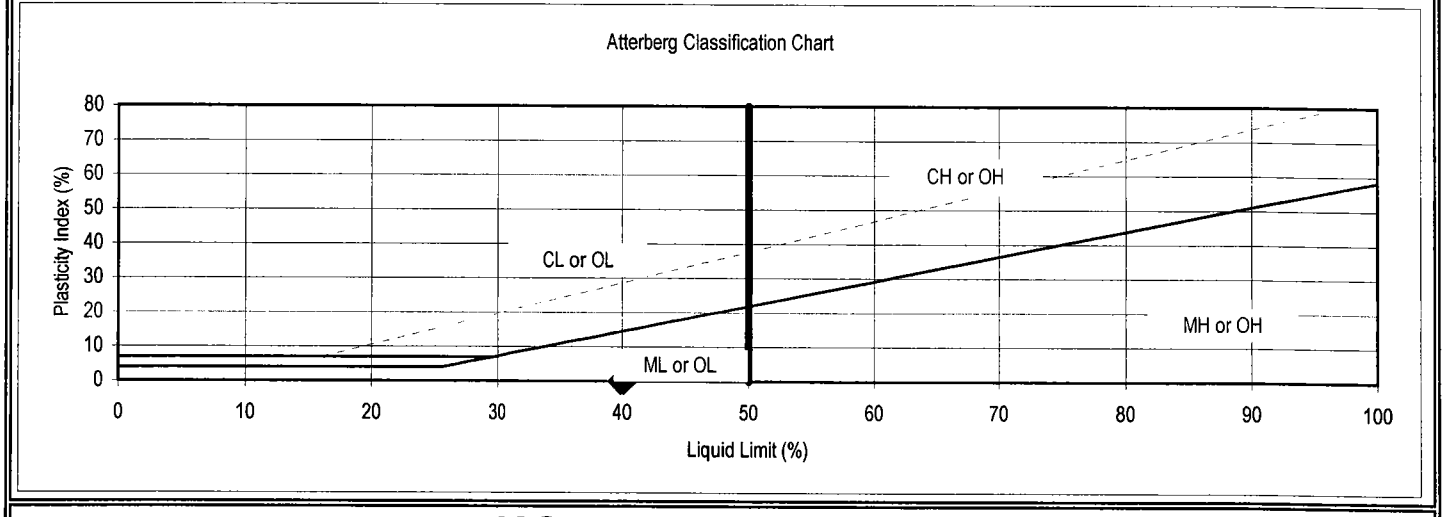
Estimated % of Sample Retained on No. 40 Sieve: _____ Sample Air Dried: yes
Test Method A or B: A

	LIQUID LIMIT:					PLASTIC LIMIT:		
Sample No.:	1	2	3	4	5	1	2	3
Pan ID:	AT	LC	HK			I	BB	
Wt. Pan (gr)	15.31	15.02	15.00			11.28	11.15	
Wt. Wet Soil + Pan (gr)	22.70	25.35	24.04			18.23	15.87	
Wt. Dry Soil + Pan (gr)	20.54	22.40	21.50			16.25	14.53	
Wt. Water (gr)	2.16	2.95	2.54			1.98	1.34	
Wt. Dry Soil (gr)	5.23	7.38	6.50			4.97	3.38	
Water Content (%)	41.3	40.0	39.1			39.8	39.6	
Number of Blows, N	15	25	35					
				LIQUID LIMIT =	40		PLASTIC LIMIT =	40



Plasticity Index = 0

Group Symbol = ML



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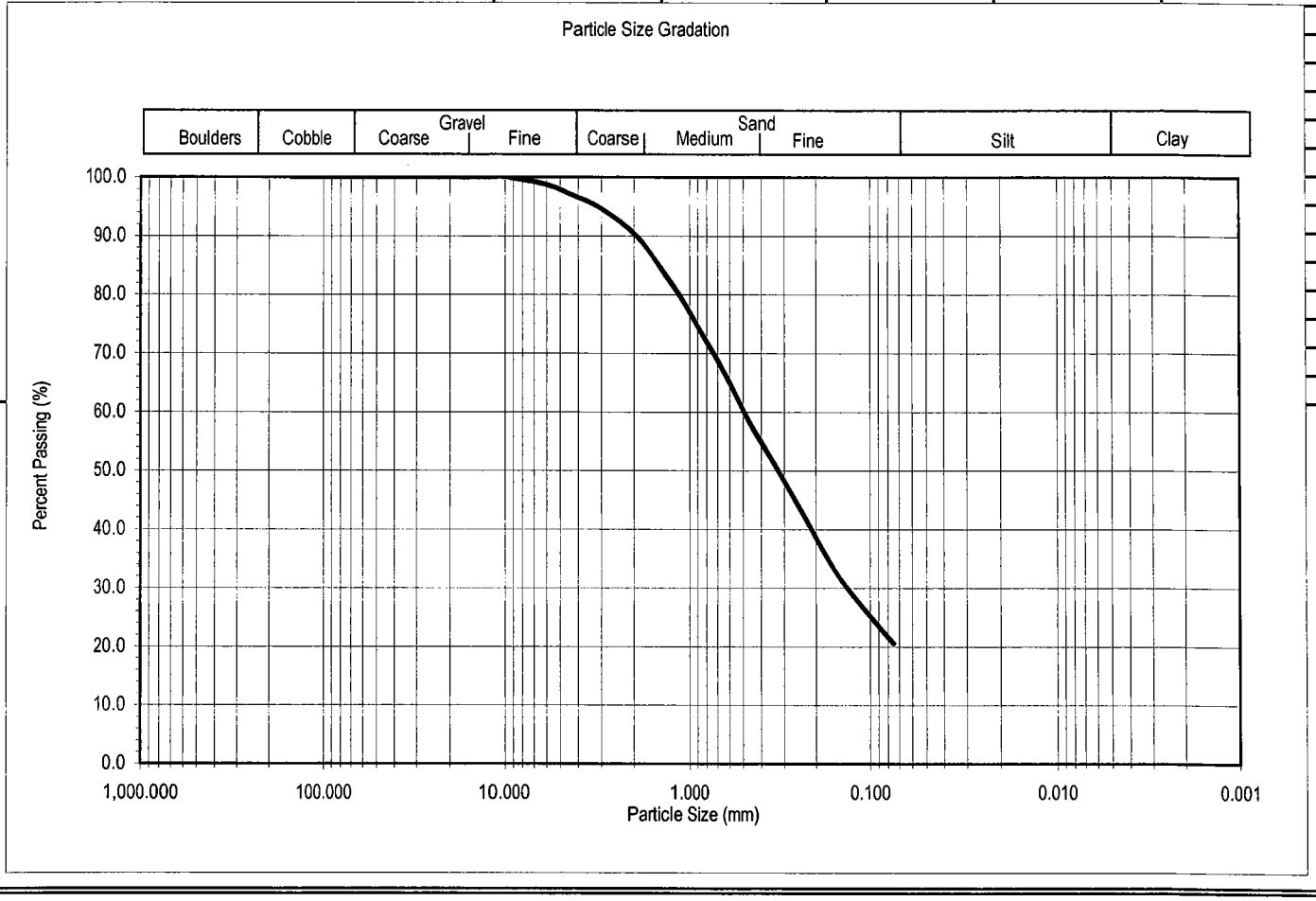
Particle Size Distribution

ASTM D422

Project No.:	41278-01	Project Name:	Homewood-North Base Lodge		Date: 2/4/2009
Sample No.:	6-1-2	Boring/Trench:	B-6	Depth, (ft.): 3	Tested By: BLk/MLF
Description:	Dark Reddish Brown (5YR 2.5/2) Silty Sand (SM)				Checked By: MLHF
Sample Location:					Lab. No.: 9-018

Sieve Size	Particle Diameter		Dry Weight on Sieve			Percent
	Inches	Millimeter	Retained On Sieve	Accumulated On Sieve	Passing Sieve	Passing
(U.S. Standard)	(in.)	(mm)	(gm)	(gm)	(gm)	(%)
6 Inch	6.0000	152.4	0.00	0.0	331.9	**100.0**
3 Inch	3.0000	76.2	0.00	0.0	331.9	**100.0**
2 Inch	2.0000	50.8	0.00	0.0	331.9	**100.0**
1.5 Inch	1.5000	38.1	0.00	0.0	331.9	**100.0**
1.0 Inch	1.0000	25.4	0.00	0.0	331.9	**100.0**
3/4 Inch	0.7500	19.1	0.00	0.0	331.9	**100.0**
1/2 Inch	0.5000	12.7	0.00	0.0	331.9	**100.0**
3/8 Inch	0.3750	9.5	0.00	0.0	331.9	**100.0**
#4	0.1870	4.7500	7.84	7.8	324.0	**97.6**
#10	0.0787	2.0000	23.89	31.7	300.1	**90.4**
#20	0.0335	0.8500	56.61	88.3	243.5	**73.4**
#40	0.0167	0.4250	56.53	144.9	187.0	**56.3**
#60	0.0098	0.2500	40.83	185.7	146.2	**44.0**
#100	0.0059	0.1500	39.43	225.1	106.7	**32.2**
#200	0.0030	0.0750	38.42	263.6	68.3	**20.6**



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Particle Size Distribution

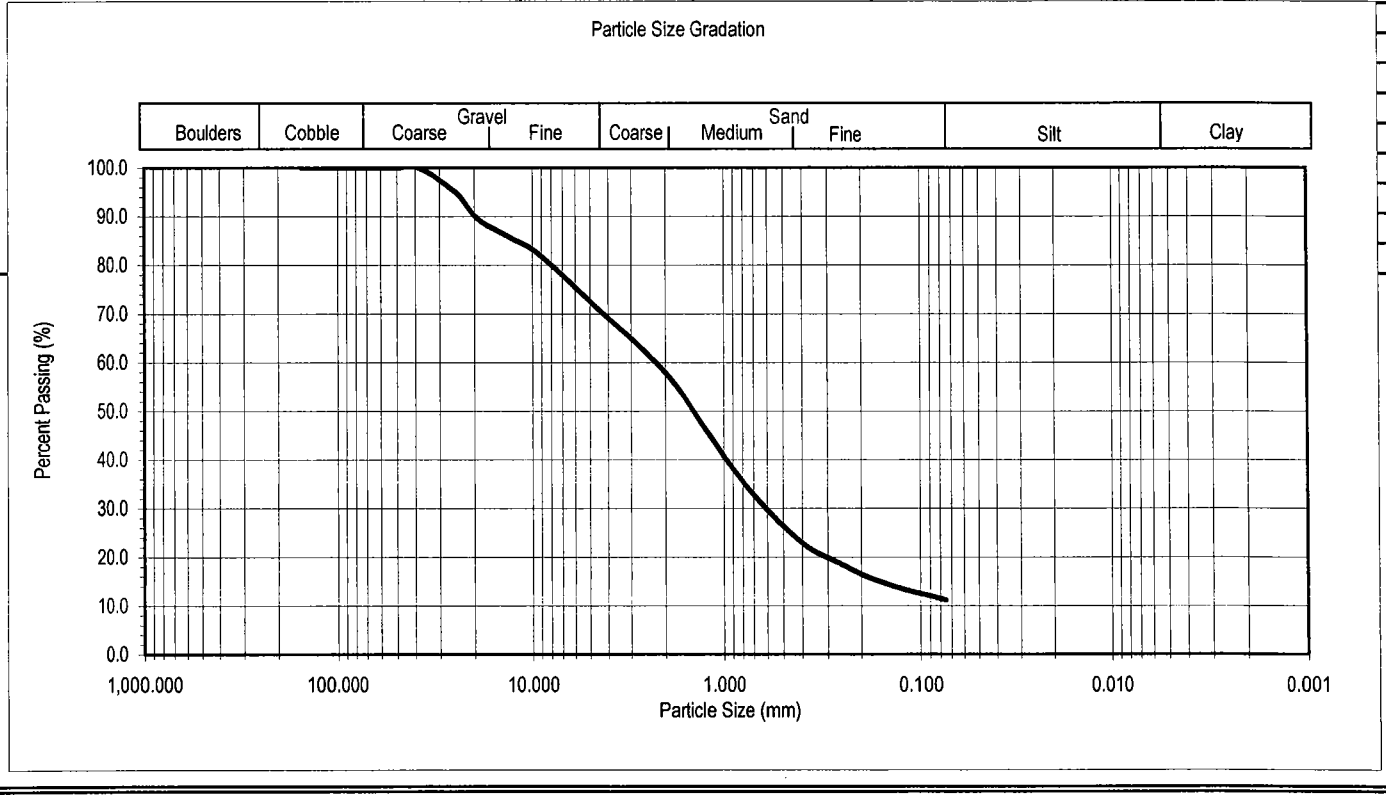
ASTM D422

Project No.:	41278-01	Project Name:	Homewood-North Base Lodge	Date:	2/4/2009
Sample No.:	7-4-2	Boring/Trench:	B-7	Depth, (ft.): 16	Tested By: BLk/MLF
Description:	Dark Gray (10YR 4/1) Poorly Graded Sand with Silt and Gravel (SP-SM)				Checked By: MLHF
Sample Location:					Lab. No.: 9-018

Sieve Size (U.S. Standard)	Particle Diameter Inches (in.)	Particle Diameter Millimeter (mm)	Dry Weight on Sieve Retained On Sieve (gm)	Dry Weight on Sieve Accumulated On Sieve (gm)	Dry Weight on Sieve Passing Sieve (gm)	Percent Passing (%)
6 Inch	6.0000	152.4	0.00	0.0	508.3	**100.0**
3 Inch	3.0000	76.2	0.00	0.0	508.3	**100.0**
2 Inch	2.0000	50.8	0.00	0.0	508.3	**100.0**
1.5 Inch	1.5000	38.1	0.00	0.0	508.3	**100.0**
1.0 Inch	1.0000	25.4	24.39	24.4	483.9	**95.2**
3/4 Inch	0.7500	19.1	29.33	53.7	454.6	**89.4**
1/2 Inch	0.5000	12.7	20.31	74.0	434.3	**85.4**
3/8 Inch	0.3750	9.5	14.24	88.3	420.0	**82.6**
#4	0.1870	4.7500	55.22	143.5	364.8	**71.8**
#10	0.0787	2.0000	70.40	213.9	294.4	**57.9**
#20	0.0335	0.8500	106.71	320.6	187.7	**36.9**
#40	0.0167	0.4250	66.72	387.3	121.0	**23.8**
#60	0.0098	0.2500	27.74	415.1	93.2	**18.3**
#100	0.0059	0.1500	19.42	434.5	73.8	**14.5**
#200	0.0030	0.0750	16.96	451.5	56.9	**11.2**

Cc = 3.96

Cu = 18.57



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Particle Size Distribution

ASTM D422

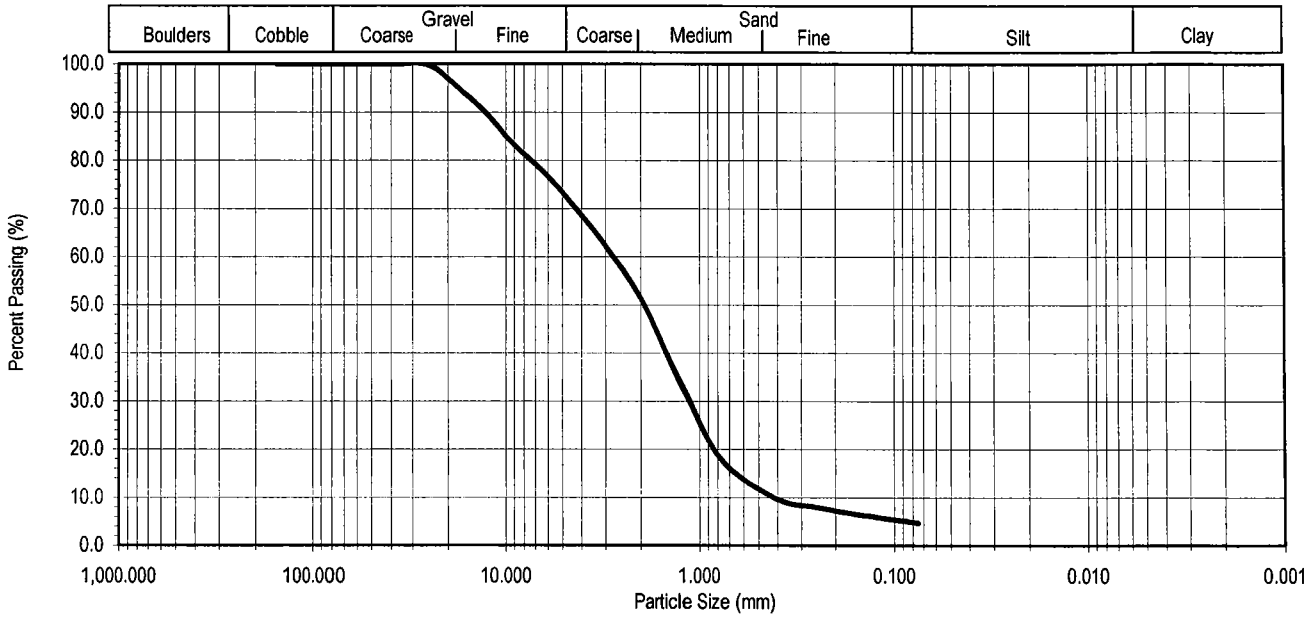
Project No.:	41278-01	Project Name:	Homewood-North Base Lodge		Date: 2/4/2009
Sample No.:	8-3-2	Boring/Trench:	B-8	Depth, (ft.): 11	Tested By: BLk/MLF
Description:	Dark Gray (5Y 4/1) Poorly Graded Sand with Gravel (SP)				Checked By: MLHF
Sample Location:					Lab. No.: 9-018

Sieve Size (U.S. Standard)	Particle Diameter Inches (in.)	Particle Diameter Millimeter (mm)	Dry Weight on Sieve Retained On Sieve (gm)	Dry Weight on Sieve Accumulated On Sieve (gm)	Dry Weight on Sieve Passing Sieve (gm)	Percent Passing (%)
6 Inch	6.0000	152.4	0.00	0.0	485.3	**100.0**
3 Inch	3.0000	76.2	0.00	0.0	485.3	**100.0**
2 Inch	2.0000	50.8	0.00	0.0	485.3	**100.0**
1.5 Inch	1.5000	38.1	0.00	0.0	485.3	**100.0**
1.0 Inch	1.0000	25.4	0.00	0.0	485.3	**100.0**
3/4 Inch	0.7500	19.1	18.10	18.1	467.2	**96.3**
1/2 Inch	0.5000	12.7	31.03	49.1	436.2	**89.9**
3/8 Inch	0.3750	9.5	27.52	76.7	408.7	**84.2**
#4	0.1870	4.7500	57.94	134.6	350.7	**72.3**
#10	0.0787	2.0000	101.60	236.2	249.1	**51.3**
#20	0.0335	0.8500	150.60	386.8	98.5	**20.3**
#40	0.0167	0.4250	49.26	436.1	49.3	**10.2**
#60	0.0098	0.2500	10.73	446.8	38.5	**7.9**
#100	0.0059	0.1500	7.37	454.1	31.2	**6.4**
#200	0.0030	0.0750	8.40	462.5	22.8	**4.7**

Cc = 1.51

Cu = 7.00

Particle Size Gradation



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Particle Size Distribution

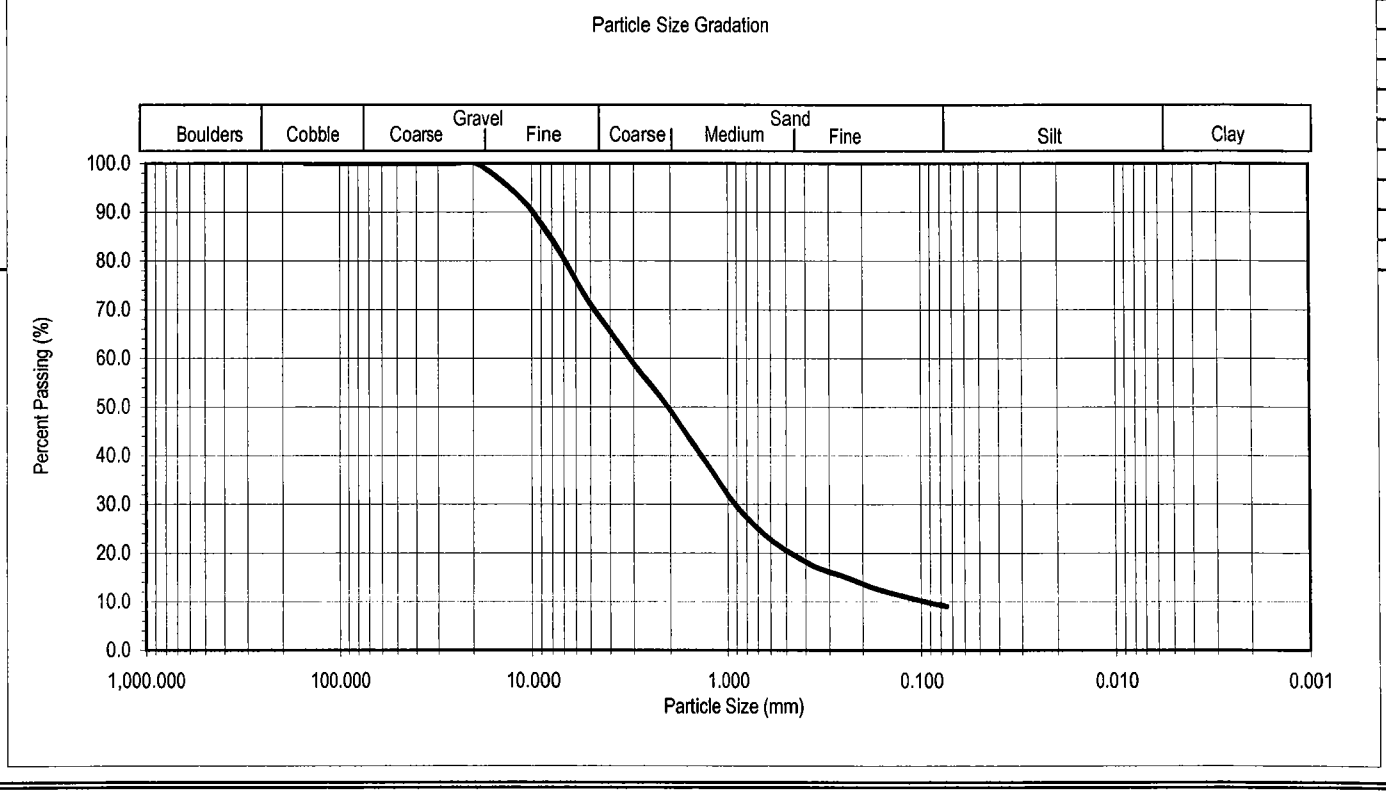
ASTM D422

Project No.:	**41278-01**	Project Name:	**Homewood-North Base Lodge**		Date: **2/4/2009**
Sample No.:	**8-4-2**	Boring/Trench:	**B-8**	Depth, (ft.): **16**	Tested By: **BLk/MLF**
Description:	**Dark Gray (5Y 4/1) Poorly Graded Sand with Silt and Gravel (SP)**				Checked By: **MLHF**
Sample Location:					Lab. No.: **9-018**

Sieve Size (U.S. Standard)	Particle Diameter Inches (in.)	Particle Diameter Millimeter (mm)	Dry Weight on Sieve Retained On Sieve (gm)	Dry Weight on Sieve Accumulated On Sieve (gm)	Dry Weight on Sieve Passing Sieve (gm)	Percent Passing (%)
6 Inch	6.0000	152.4	0.00	0.0	520.0	**100.0**
3 Inch	3.0000	76.2	0.00	0.0	520.0	**100.0**
2 Inch	2.0000	50.8	0.00	0.0	520.0	**100.0**
1.5 Inch	1.5000	38.1	0.00	0.0	520.0	**100.0**
1.0 Inch	1.0000	25.4	0.00	0.0	520.0	**100.0**
3/4 Inch	0.7500	19.1	0.00	0.0	520.0	**100.0**
1/2 Inch	0.5000	12.7	27.69	27.7	492.3	**94.7**
3/8 Inch	0.3750	9.5	28.27	56.0	464.0	**89.2**
#4	0.1870	4.7500	101.03	157.0	363.0	**69.8**
#10	0.0787	2.0000	106.42	263.4	256.6	**49.3**
#20	0.0335	0.8500	108.77	372.2	147.8	**28.4**
#40	0.0167	0.4250	50.17	422.4	97.6	**18.8**
#60	0.0098	0.2500	18.98	441.3	78.6	**15.1**
#100	0.0059	0.1500	15.97	457.3	62.7	**12.1**
#200	0.0030	0.0750	15.55	472.9	47.1	**9.1**

Cc = 3.27

Cu = 38.75



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APPENDIX A ***Important Information About Your Geotechnical Engineering Report***
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APPENDIX B Boring Logs

APPENDIX C Laboratory Test Results