APPENDIX H

Geology and Soils

APPENDIX H-1

Holdrege and Kull Geotech 2003

GEOTECHNICAL ENGINEERING REPORT
for
ALPINE SIERRA SUBDIVISION
Alpine Meadows/Placer County, California

Prepared for:
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Project No. 40376-01
December 11, 2003

HOLDREGE & KULL
CONSULTING ENGINEERS • GEOLOGISTS

Project No. 40376-01
December 11, 2003

Alpine Sierra Partners, LLC
1015 NW 11th Avenue Suite 243
Portland, Oregon 97209

Attention: Chris Nelson

Reference: ***Alpine Sierra***
Alpine Meadows/Placer County, California

Subject: ***Geotechnical Engineering Report***

This report presents the results of our geotechnical engineering investigation for the proposed Alpine Sierra subdivision to be located in Alpine Meadows/Placer County, California. The proposed project involves subdividing the previously undeveloped 45 acre site into a single-family residential subdivision. Appurtenant construction will include underground utilities, earth retaining structures and asphalt concrete paved roads.

Based on our work completed to date, no severe soil or groundwater constraints were observed which would preclude the proposed development. Shallow rock may require special rock excavation techniques for site grading and underground utility construction. The proposed project is located in a relatively steep mountain area. Portions of the project's second phase may be subject to small stream flooding and debris flows that are typical of many local subdivisions in steep mountain areas. Our opinion is that the project can be completed as proposed, provided that the recommendations presented in this report are implemented. Specific recommendations regarding the potential geologic hazards and geotechnical aspects of project design and construction are presented in the following report.

The findings presented in this report are based on our site observations, subsurface exploration, laboratory test results and our experience in the project area. Holdrege & Kull should be retained to provide construction monitoring services during earthwork and foundation construction to observe subsurface conditions encountered with respect to our engineering recommendations provided in this report.

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

Sincerely,
HOLDREGE & KULL

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1 INTRODUCTION

Holdrege & Kull (H&K) is pleased to present our geotechnical engineering report for the proposed Alpine Sierra subdivision located in Alpine Meadows/Placer County, California. We performed our investigation in general accordance with our October 23, 2003 proposal for the project. A copy of the proposal is included as Appendix A of this report. For your review, Appendix B 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 Site Description

The project site consists of approximately 45 acres of undeveloped property in Alpine Meadows/Placer County, California. A Site Vicinity Map is presented as Figure 1. The existing Bear Creek subdivision is located adjacent to the site on the north and the Scott Peak and Alpine Chalet condominiums are on the south. The project site is bounded by undeveloped property to the east and Alpine Meadows Road to the west. Site grades vary significantly across the site, typically sloping moderately to steeply from south to north. A low rock knoll is located in the east central part of the site. Ground surface elevations range from approximately 7076 feet in the southeast corner of the site to 6600 feet in the north central portion of the site. Bear Creek crosses the far western part of the site and a tributary creek crosses the eastern part of the site.

Vegetation consists of dense conifer trees, mountain bushes and shrubs. Portions of the site consist of rock outcrops with sparse vegetation.

1.2 Proposed Improvements

We understand that the proposed development will consist of subdividing the existing property into approximately 31 single family residential lots. Current plans indicate that the development will occur in three phases. Appurtenant construction will include associated paved roads and underground utility construction. Earthwork cuts and fills of 20 feet or more will be required to grade all three phases of construction. Approximately 2760 lineal feet of retaining wall and three "Conspan" bridges are currently planned to construct the roads.

We anticipate that future residential structures will consist of two- to three-story, wood, concrete and steel-framed structures founded on shallow foundations, with raised wood and/or concrete slab-on-grade floors. Concrete retaining walls may also be incorporated in residential structures. Structural loading information was not available at this time.

Information pertaining to the proposed development and the project site was obtained from our site visits and the Alpine Sierra Subdivision Pre-Tentative Map, dated June 2003, prepared by Terrence E. Lowell & Associates, Inc. We also reviewed an Interim Avalanche Hazard Study for Alpine Sierra, dated May 2003, and an Addendum dated June 2003, prepared by Norman A. Wilson.

1.3 Purpose

The purpose of our geotechnical engineering report was to provide geotechnical engineering recommendations for project design and construction based on our surface observations and subsurface exploration at the project site. We recommend retaining our firm to provide construction monitoring services during earthwork and foundation excavation to observe subsurface conditions encountered with respect to our recommendations.

1.4 Scope of Services

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

- We performed a site reconnaissance, literature review and subsurface exploration involving backhoe excavated test pits.

- We logged the subsurface conditions encountered and collected bulk soil samples for classification and laboratory testing.

- We performed laboratory tests on selected soil samples obtained during our subsurface exploration to help evaluate material properties.

- Based on our subsurface exploration and the results of our laboratory testing, we performed engineering analyses to develop geotechnical engineering recommendations for project design and construction.

2 SITE GEOLOGY

The proposed Alpine Sierra subdivision is located in the northern Sierra Nevada geologic province about two miles east of the Sierra Nevada crest at an elevation of approximately 6,800 feet. The Sierra Nevada Mountains were formed by large intrusions of granitic rock during late Mesozoic time. Subsequent faulting and volcanic activity during Tertiary time raised the mountain range to its present position.

The northern Sierra Nevada geology is dominated by volcanic rocks of Tertiary to Quaternary age. Scott Peak, located south of the site, is made up of Pliocene (late Tertiary) volcanic rock consisting of andesite flows and lahar (mud flow) deposits. Volcanic rock that is slightly older than the Scott Peak volcanic rocks underlies much of the project site, but still consists of andesite and lahar deposits. Rock outcrops are present primarily in the southern part of the site at many areas within the site boundary. Volcanic rock can be observed in isolated outcrops in the northern part of the site and in the John Scott Trail road cut. It appears that volcanic rock extends under most of the site at relatively shallow depths. Granitic bedrock is present within the stream channel of Bear Creek at the proposed Road A crossing.

The Ward Peak area and the crest of the Sierra were occupied during much of the Quaternary Period by alpine glaciers that flowed down Bear Creek to the Truckee River. The northern portion of the site is underlain by Tioga age glacial till deposits (Birkeland, 1963). Till deposits consist of silty sandy gravel and gravelly sand with numerous cobbles and boulders. This is evident on site with the presence of many large granitic boulders. Surficial geology is shown on Figure 2, Test Pit Location Plan and Surficial Geologic Map.

2.1 Slope Stability

The proposed subdivision is located within and near steep slopes. Slope instability includes avalanches, landslides, debris flows, and rock fall. Site soil types are relatively competent and are not prone to landsliding. However, the area south of the site is very steep and some debris flows were observed in the site area. Debris piles and hummocky topography, characteristic of debris deposition, were observed in the north-central part of the site. Debris flows usually originate within saturated soil on very steep slopes and are transported in a fluid state into

the stream channels. Debris flows are usually confined to the stream channel until the stream gradient changes and becomes blocked or the channel is no longer confined. The volcanic rock in this area is not deeply weathered or altered and soils are well drained, not usually saturated. The slopes above the site have not developed a thick soil overlying the rock that may contribute to debris flows. Consequently, Phase II of the proposed subdivision does not have greater potential for debris flows than many other locations in the Bear Creek drainage.

The northeast portion of the site is underlain by alluvial deposits, which consist of sand and gravel with some cobbles and boulders transported by water. This area is located northeast of the stream channel that flows down from the Alpine Meadows ski run known as the Munchkin Chutes. The stream channel abruptly decreases gradient and forms a rough alluvial apron or fan below approximate elevation 6700 feet. The ground surface is characterized by numerous discontinuous stream channels and hummocky terrain. It appears that over geologic time the stream channel has shifted across the alluvial apron. The present stream channel is incised at the western edge of the apron, between the alluvium and andesite rock. Based on the observed geomorphology, this area may be subject to small stream flooding and debris flows. The small stream channel located in the northeast corner of the site also appears to have some associated alluvial deposits. Debris flows at the site are a rare event that would be associated with large storm events that may occur on the order of one hundred years.

Options to reduce the potential hazard due to debris flows and/or rockfall include the construction of barriers such as levees, stream channel modification and/or reinforcement of individual homes. Reinforcement of individual homes would be similar to design for avalanche protection. For protection from debris flows, we recommend that lot grading and house placement provide positive surface drainage away from structures. Reinforced concrete foundations for structures should extend a minimum of 4 feet above the existing ground surface.

2.2 Rock Fall

The potential rock fall hazard at the site is relatively low. The parent rock above the site consists of Lahar (or volcanic mudflow) deposits that tend to weather into relatively small, non-spherical boulders that are not prone to rolling down slope.

Colluvium and some large boulders are present at the base of the slope above the subject site. The slope gradient decreases below 7000 feet and the majority of this material appears to stop before reaching the Alpine Sierra site. The dense vegetation at and near the site significantly reduces the potential for both debris flows and rock fall. In the event of an intense forest fire, the potential for slope instability may increase.

3 SUBSURFACE EXPLORATION

We performed our subsurface exploration to help characterize subsurface conditions at the site. Our exploration included literature review and field exploration as described below.

3.1 Field Exploration

Subsurface conditions at the site were explored on October 28, 2003, by excavating 8 test pits to depths ranging from 4.0 to 7.5 feet below the existing ground surface (bgs). The test pits were excavated with a Takeuchi TB135 mini-excavator equipped with an 18-inch bucket. As excavation of a test pit became more difficult, the excavating contractor switched to a 12-inch bucket. Test pit locations were selected based on locations of proposed improvements and site access.

An engineer from our firm logged the soil conditions exposed in the test pits, visually classified soil and collected bulk soil samples for laboratory testing. Soil samples were packaged and sealed in the field to reduce moisture loss and were returned to our laboratory for further testing. Upon completion, the test pits were backfilled with the excavated soil. The approximate locations of our test pits are shown in the Test Pit Location Plan and Surficial Geologic Map, Figure 2.

3.2 Subsurface Soil Conditions

Approximately 4 to 18 inches of dark brown, silty sand top soil was encountered across a majority of the site. Near surface soil encountered underlying the top soil consisted of medium dense to very dense Silty Sand (SM) with Gravel and Silty Gravel with Sand (GM), containing varying amounts of cobbles and boulders up to approximately 4 feet in diameter. All test pit excavations encountered refusal

at depths of 4 to 7½ feet below the ground surface (bgs). Refusal was encountered on what appeared to be large boulders and/or volcanic rock. The rock quality varied from moderately weathered to fresh, closely to moderately fractured and moderately strong to strong. More detailed descriptions of subsurface conditions are presented in our test pit logs in Appendix C.

3.4 Groundwater Conditions

We did not encounter groundwater during our subsurface exploration. However, based on our experience in the project area, seasonal saturation of near-surface soil should be anticipated at the site. Groundwater may become perched above near-surface rock encountered at varying depths over the site. Fluctuations in soil moisture content and groundwater levels should be anticipated depending on precipitation, irrigation, runoff conditions, snow melt and other factors.

4 LABORATORY TESTING

We performed laboratory tests on bulk soil samples collected from our exploratory trenches to help evaluate their engineering properties. Laboratory tests performed included Moisture Content (ASTM Test Method D 2216) and Sieve Analysis (ASTM D422). Sieve analysis data were used to evaluate the Unified Soil Classification System (USCS) classifications as described in Section 2.3, Subsurface Soil Conditions. Specific soil classification and laboratory test data is included on the trench logs and in Appendix D.

5 CONCLUSIONS

The following conclusions are based on our field observations, laboratory test results, and our experience in the area.

1. Soil conditions encountered in our subsurface exploration generally consisted of medium dense to dense Silty Sand with Gravel (SM) and Silty Gravel with Sand (GM). The observed soil typically contained varying amounts cobbles and boulders up to approximately 4 feet in diameter. These soil types should provide adequate support for planned structures on conventional spread foundations.

2. Underlying near-surface soil, the excavator used for our field exploration encountered essential refusal at depths of 4 to 7½ feet bgs on what appeared to be volcanic rock and/or boulders in all test pit excavations. We also observed rock outcrop and boulders in numerous areas across the site. Excavation for roads, footings and underground utilities that extend into rock may be difficult and require special excavation techniques. Material excavated from these areas may require significant processing or crushing to remove oversize material prior to reuse as structural fill or utility trench backfill. Recommendations for the reuse of site soil for fill and backfill are discussed below.

3. Groundwater was not encountered during our subsurface exploration in our test pit excavations. However, we expect that near-surface soil at the site may become seasonally saturated, particularly during spring snow melt. Positive surface water drainage, subsurface drains behind below grade walls and beneath concrete slabs-on-grade and asphalt pavement will be important for performance of structures and other site features.

4. Due to the relatively steep terrain at the project site, steep constructed slopes and high retaining structures are planned for the project. Stability of cut and fill slopes involves true slope stability related to landslides and erosion potential. Site soil types are relatively competent and may be constructed at steep angles from a slope stability standpoint. Fill slopes may be stabilized using internally stabilized systems such as mechanically stabilized embankments. We anticipate that steep cut slopes will be stable in near-surface rock. Proposed slopes may also be retained with rockery walls and/or conventional cantilever retaining walls. Final design should be completed at specific locations as project planning is completed. A combination of vegetation and Rock Slope Protection (RSP) should be placed on cut and fill slopes for erosion protection.

5. Geotechnical engineering design criteria provided in this report are for native soil encountered in our test pit excavations at the site. Our experience is that site specific design criteria may be necessary to design larger retaining walls, bridge foundations and other structures for unusual load combinations (i.e. heavy snow loads, avalanche impact loads, traffic loads, etc.). Due to the variation in soil and rock condition observed, site specific design criteria may

be provided on a case by case basis. As project design continues we should evaluate future building, retaining wall and bridge sites to provide specific design criteria for individual loading conditions.

6. The proposed subdivision is located within and near steep slopes. Portions of the site are subject to potential hazards typical of steep mountain terrain, such as avalanches, rock fall and debris flows. A separate avalanche report with specific recommendations was prepared by others for this project. Rock fall is relatively uncommon and not a significant hazard. Structures in portions of Phase II can be designed using conventional construction to resist impacts from small stream flooding and debris flows based on their actual site location. Homes to be constructed within avalanche zones or areas of Phase II with potential for small stream flooding and debris flows should engage professional engineering assistance in design and construction of the foundation.

6 RECOMMENDATIONS

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

6.1 Grading

6.1.1 Clearing and Grubbing

Proposed fill and building areas should be cleared and grubbed of vegetation and other deleterious materials as described below.

1. Prior to site grading all existing fill, vegetation, organic topsoil and any debris should be stripped and disposed of outside the construction limits. Based on our subsurface exploration, we expect that approximately 12 inches may be used as an estimate for average depth of stripping within wooded areas. Stripping will be considerably less in rock areas. The actual depth of stripping will vary across the site. Organic surface soil should be stockpiled for future use in landscape area and is not suitable for use as structural fill or backfill.

2. Uncompacted soil or other unsuitable material in excavations resulting from tree removal or test pits, etc., should be over-excavated and dish-shaped to permit access for compaction equipment. Any 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.

3. Rocks greater than 8 inches (oversized rock) may be left in place in areas where fill will be deeper than 2 feet from finish grade, or removed and stockpiled for later use as RSP and landscaping. In areas where foundation structures or underground utility construction is deeper than 2 feet, oversized rock in fill should be deeper than the bottom of the proposed structures. Oversized rock should be placed so that nesting does not occur. Our personnel should be onsite to evaluate locations and methods for oversized rock placement as well as observation of rock fill placement.

6.1.2 Preparation for Fill Placement

Where fill placement is planned, near-surface soil exposed by clearing and grubbing should be prepared as described below.

1. The near-surface soil should be scarified to approximately 12 inches below existing ground surface or to competent material and then uniformly moisture conditioned to within approximately 2 percent of the ASTM D1557 optimum moisture content.

2. Areas to receive fill should be compacted with appropriate compaction equipment and proof rolled with a loaded, tandem-axle truck under observation of a representative of H&K. Areas that exhibit pumping or rutting should be over excavated and replaced with compacted fill placed according to the recommendations discussed below.

6.1.3 Fill Placement

Fill should be placed according to the following guidelines:

1. Material used for fill construction should consist of uncontaminated, predominantly granular, non-expansive native soil or approved import soil. Imported fill material should be predominantly granular, non-expansive, have a 8-inch maximum particle size and free of deleterious or organic material. Import material that is proposed for use onsite should be submitted to H&K for laboratory analysis at least 72 hours prior to import.

2. If site grading is performed during periods of wet weather or in the spring or early summer, 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 processing and drying to facilitate placement and compaction, particularly during or following the wet season or spring snow melt.

3. 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.

4. 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.

6.1.4 Cut/Fill Slope Grading

Stability of cut and fill slopes involves two separate aspects. The first concerns true slope stability related to mass wasting and landslides. True slope stability is dependent upon shear strength, unit weight, moisture content of soil and slope angle. The second aspect of slope stability involves erosion potential and is

dependent on numerous factors including grain size distribution, cohesion, moisture content, slope inclination, slope length and velocity of water or wind on the slope face. Steep constructed slopes reduce the amount of disturbed area and result in shorter slope lengths. Slope performance will be primarily affected by surface runoff and erosion. Permanent slope grading (cut/fill slopes) should be constructed according to the following recommendations.

1. Cut and fill slopes up to 20 feet in vertical extent consisting of soil may be constructed at inclinations of 1:1 (H:V) provided the slopes are surfaced with rock slope protection (RSP). Steep constructed slopes reduce the amount of disturbed area and result in shorter slope lengths. Slope performance at the site will primarily be affected by surface runoff, erosion and composition (soil or rock).

2. Cut slopes consisting of rock may excavated to grades up to 0.5:1, H:V. However, actual cut slope gradients and overall slope geometry should be evaluated on a case by case basis in the field by our firm.

3. Slopes steeper than 2:1, H:V, but no steeper than 1:1, H:V, should be protected with RSP in combination with vegetation. We recommend a minimum size of 12-inch diameter boulders for RSP on slopes up to 8 feet in vertical extent, 12- to 18-inch boulders for slopes up to 12 feet in vertical extent, and 18- to 24-inch boulders for slopes up to 20 feet in vertical extent. Slopes steeper than 1:1, H:V should be designed as stacked rockery walls. We can provide these recommendations on a case by case basis as necessary.

4. RSP should be placed in accordance with Caltrans Standard Specifications Method A Placement. Placing of rocks by dumping should not be permitted. Rocks should decrease in size from the bottom to the top of the slope. Rocks should have a specific gravity of at least 2.5 and a percentage of wear (per ASTM C535) of not more than 45.

5. 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.

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

6.1.5 Erosion Control

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

Erosion and sediment control measures can be categorized as temporary or permanent. Temporary measures are to be installed to provide short-term protection until the permanent measures are installed and effective. Temporary erosion and sediment control includes ground surface treatments and installation of sediment barriers and detention facilities. Sediment barriers and detention facilities are placed down gradient from all disturbed areas during construction and remain after construction until permanent measures become effective. 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.

These structures are designed to slow down water flow and intercept suspended sediment to prevent sediment discharge from the construction area while allowing runoff to continue down gradient. The requirement to install a sediment barrier is dependent on the following factors: 1) slope angle, 2) slope length, and 3) soil type (texture and coarse fragment content). 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 gently sloping swales. The slope length above each barrier should not exceed 100 feet. 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.

Berms, waterbars and ditches can be temporary or permanent and 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 stable, often well vegetated adjacent 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 flowline 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. Existing vegetation should be protected and undisturbed where possible. Revegetation should consist of native brush and grass species. All areas disturbed by construction should be re-vegetated. Previously stripped top soil should be replaced on disturbed areas where possible. Slope faces should be temporarily protected against erosion resulting from direct rain impact and melting snow 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.

6.1.6 Underground Utility Trenches

Underground utility trenches should be excavated and backfilled as described below.

1. We anticipate that the contractor will be able to excavate underground utility trenches using conventional earthmoving equipment for the majority of the planned Roads B and C, and Road A within Phase II. However, moderately strong bedrock and/or large boulders appear to underlie most of the site. The Takeuchi TB135 mini-excavator used in our field exploration encountered refusal at depths of 4 to 7½ feet below existing grade in all of our test pits. Based on conditions encountered in our trenches, we anticipate that a track mounted excavator equipped with a ripper or hydraulic hammer or other methods may be required for confined excavations.

2. The California Occupational Safety and Health Administration (OSHA) requires all utility trenches deeper than 5 feet bgs to be shored with bracing equipment or sloped back prior to entry.

3. We did not groundwater encounter in our test pit excavations. However depending on the time of year that excavation takes place, shallow subsurface seepage may be encountered in trench excavations, particularly if utility trenches are excavated below the rock surface. 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. In addition, to help prevent the migration of groundwater along utility trenches, cut-off walls should be considered for all trenches that exceed a 15 percent grade for a distance of 100 feet or more. Cut-off walls should consist of stacked bags of lean concrete with a subsurface drain extended to daylight on the upstream side.

4. Soil used as trench backfill should be granular and 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.

5. 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. The moisture content, density and relative compaction of fill should be evaluated by our firm at regular intervals during fill placement.

6. Trench backfill placed within 8 inches of subgrade in road and parking lot areas should be compacted to a minimum relative compaction of 95 percent of the maximum dry density per ASTM D1557.

6.1.7 Construction Dewatering

We did not encounter groundwater during our subsurface exploration. However, if grading is performed during or immediately following the wet season, seepage may be encountered in deeper excavations. If groundwater or saturated soil conditions are 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.

1. 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.

2. 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.

6.1.8 Surface Water Drainage

Proper surface water drainage is also important to the successful development of the project. We recommend the following measures to mitigate surface water drainage problems:

1. Slope final grade in structural areas so that surface water drains away from buildings at a minimum 2 percent slope for a minimum horizontal distance of 5 feet.

2. Compact and slope soil adjacent to building foundations so that water is not allowed to pond or infiltrate.

3. Direct roof downspouts to a closed collector pipe which discharges flow to positive drainage.

4. Paved areas should be sloped and drainage gradients maintained to carry all surface water to properly designed infiltration, detention basins or other storm water drainage structures.

5. Interior grades in crawl spaces beneath proposed structures should be higher than the exterior ground surface. The crawl space should be sloped to collect and divert water to drains that exit under or through the foundation (positive crawl space drainage). We recommend placing 3 to 4 inches of clean gravel over the crawl space grade.

6. Adequate ventilation should be provided in crawl space and/or basement areas to promote drying.

7. If open graded gravel or other highly permeable material is used for underground utilities, the trench should slope away from structures or the potential flow path should be plugged at the exterior of foundations.

6.1.9 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 Structural Improvement Design Criteria

Design criteria presented below assume that near-surface soil conditions at locations of proposed structures consist of dense, granular, native soil. Actual subsurface conditions at proposed structure locations may differ. We expect that these other conditions may be rock of varying quality. If subsurface conditions encountered during construction are different than dense native soil, we should be notified so that we can review and modify our recommendations as needed. Modification of our recommendations will require a site visit and possibly additional subsurface exploration and/or laboratory testing. Our firm can provide these services on a case by case basis upon request.

In addition, the following loads and design criteria consider typical geotechnical engineering loads only. Based on the geology and location of the site, additional loads may be required for design of structures. These loads may include, but are not limited to, avalanche impacts, snow creep, debris flows and small stream flooding. Homes to be constructed within avalanche zones or areas of Phase II with potential for small stream flooding and debris flows should engage professional engineering assistance in design and construction of the foundations.

6.2.1 Foundations

Our opinion is that shallow spread foundations are suitable for support of typical residential structures and medium sized earth retaining structures and bridges proposed for this project. Construction of special or larger structures may require deep foundations, rock bolting or other specialty techniques. The following paragraphs discuss foundation design parameters and construction recommendations.

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

2. Foundations constructed in non-expansive, previously undisturbed native soil or compacted fill may be designed using an allowable bearing capacity of 3000 pounds per square foot (psf) for dead plus live loads. An additional increase of 500 psf per foot of additional embedment (beyond the minimum 24 inches) may also be used, up to a limiting value of 4500 psf. Allowable bearing pressures may be increased by 33 percent for additional transient loading such as wind or seismic loads.

3. Resistance to lateral loads (including transient loads) may be provided by frictional resistance between the bottom of concrete foundations and the underlying soils, and by passive soil pressure against the sides of foundations. Lateral resistance derived from passive earth pressure can be modeled as a triangular pressure distribution ranging from 0 psf at the ground surface to a maximum of 350d psf, where d equals the depth of the foundation in feet. 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. A coefficient of friction of 0.40 may be used between poured-in-place concrete foundations and underlying native soil.

4. 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 footings designed and constructed in accordance with our recommendations will be on the order of ½-inch. Differential settlement between similarly loaded, adjacent footings is expected to be less than ¼-inch, provided footings are founded on similar materials (e.g., all on engineered fill, native soil, or rock). Differential settlement between adjacent footings founded on dissimilar materials (e.g., one footing on soil and an adjacent footing on rock) may approach the maximum anticipated total settlement. Settlement of foundations is expected to occur rapidly and should be essentially complete shortly after initial application of loads.

5. Potentially expansive or loose material remaining in footing excavations should be removed to expose firm, non-expansive material. Footing excavations should be moistened prior to placing concrete to reduce risk of problems caused by wicking moisture from curing concrete. H&K should

observe footing excavations prior to reinforcing steel and concrete placement.

6.2.2 Seismic Design Criteria

The site is located in Seismic Zone 3 of the 1997 Uniform Building Code (UBC) and the 2001 California Building Code (CBC) Seismic Zone Map. Therefore, structural improvements should be designed using a seismic zone factor of Z=0.30 (Table 16-I). Based on our observations of subsurface soil/rock conditions and our literature review, we recommend using the S_C Soil Profile Type (Table 16-J, 1997 UBC/2001 CBC) to evaluate seismic loads. Site specific evaluation may be appropriate for houses constructed on rock.

6.2.3 Slab-on-Grade Floor Systems

Concrete slab-on-grade floors may be used in conjunction with perimeter concrete footings. We make the following recommendations for the design and construction of slabs-on-grade:

1. Slabs-on-grade should be a minimum of 4 inches thick and uniform thickness. 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.

2. Slab subgrade soil should consist of non-expansive, suitable soil. Prior to constructing concrete slabs, the upper 8 inches of slab subgrade should be scarified, uniformly moisture conditioned to within 2% of optimum moisture content and compacted to at least 90% of the maximum dry density per ASTM D1557.

3. Slabs should be underlain by at least 4 inches of Class 2 aggregate base to provide uniform support. The aggregate base should be compacted to a minimum of 95% of the maximum dry density per ASTM D1557. The slab subgrade should be protected against drying until concrete placement. If groundwater is encountered in slab areas, subsurface drains should be constructed.

4. In slab-on-grade areas where moisture sensitive floor coverings are proposed, an vapor retarder membrane (e.g. 10 mil thick polyethylene) should be placed over the base course to reduce the migration of moisture vapor through the concrete slab. A minimum of 2 inches of clean, moist sand should be spread over the vapor retarder. The sand will help protect the membrane and aid in curing the concrete. Prior to concrete placement, the sand should be moistened (not wet) and tamped to reduce moisture withdrawal from concrete during curing. Exposed concrete slabs should be moisture cured for at least seven days after placement.

5. Exterior slabs-on-grade such as sidewalks may be placed directly on compacted fill or scarified and recompacted, non-expansive native soil without the use of an aggregate base section. Potentially expansive, 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.

6. 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.

6.2.4 Retaining Wall Discussion and Design Criteria

To help reduce the amount of disturbed area and constructed slopes, relatively high retaining walls and steep constructed slopes are anticipated for the proposed project. Conventional cast-in-place reinforced concrete walls may be constructed with textured and colored concrete for improved appearance. However, numerous other options for retaining structures are possible. Retaining structures for fill slopes will likely consist of mechanically stabilized embankments (MSE) such as Hilficker™ walls or geogrid reinforced embankments with rock slope protection (RSP). MSE generally consist of reinforcing layers of metal wire or high density polyethylene placed in between compacted layers of the embankment with a slope facing on the outside of the embankment. Crib walls and rockery walls are also possible options for the fill slopes at the project site.

For soil or loose rock cut slopes at the project site greater than 15 or 20 feet in height, tie-back anchored retaining structures are possible. A tie-back wall generally consists of reinforcing tendons drilled and grouted into the cut slope and tensioned to support the retained mass. Within stable rock areas the lateral earth pressure should be relatively low, reducing the need for massive, heavily reinforced concrete structures. These slopes are commonly faced with shotcrete, thin cast-in-place concrete walls or pre-cast concrete panels. Installation of tie-back anchors is typically performed by a specialty contractor experienced in these construction techniques.

We expect that many of the proposed cut slopes for the site will be excavated in relatively strong rock. Consequently, it may be feasible to excavate some permanent slopes at very steep angles. These rock slopes would likely be prone to raveling, erosion and rock fall. A short retaining wall at the bottom of steep slopes may be necessary to catch rock fall. Another option would be a combination of short walls at the bottom of steep slopes covered with RSP.

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 the unit weights presented in the following table.

Table 6.2.4.1 - 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)	35	50
Passive Pressure (pcf)	350	350
At-Rest Pressure (pcf)	50	65
Coefficient of Friction	0.45	0.45

* 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 5.2.4.1 assume that retaining walls are constructed in native soil or compacted fill and will not exceed approximately 10 feet in height. For retaining walls greater than 10 feet and/or with surcharge loads (e.g., footings, vehicles), we anticipate that site specific analysis for individual structures will be necessary. We can provide retaining wall and rockery wall design criteria for specific loading and backfill configurations, if requested. Additional recommendations for design and construction of retaining walls are listed below:

1. Heavy compaction equipment should not be used directly adjacent to retaining walls unless the wall is designed or braced to resist the additional lateral forces.

2. If surface loads are closer to the top of the retaining wall than one-half of its height, H&K should review the loads and loading configuration. We should also review details and plans for any proposed wall over 10 feet in height.

3. Retaining wall design criteria presented in Table 5.2.4.1 assume that retaining walls are well drained reduce hydrostatic pressures. A drainage blanket should be installed to reduce additional lateral forces and reduce saturation of backfill soil. Drainage blankets may consist of graded rock drains or geosynthetic blankets.

4. Rock drains should consist of a minimum 12 inches of open-graded crushed rock, and placed directly behind the wall, wrapped in non-woven geotextile filter fabric such as Amoco 4545™ or equivalent. Drains should have a minimum 4-inch diameter, perforated, schedule 40 PVC pipe placed at the base of the wall, inside the drain rock, with perforations placed down. The PVC pipe should be sloped so that water is directed away from the wall by gravity. A geosynthetic drainage blanket such as Enkadrain™ or equivalent may be substituted for the rock drain, provided that water is channeled away from the wall. If a geosynthetic blanket is used, backfill must be compacted carefully so that equipment or soil does not tear or crush the drainage blanket.

5. Additional lateral loading on retaining structures due to seismic accelerations may be considered at the designer's option. For an earthquake producing a

design horizontal acceleration of 0.3g, we recommend that the resulting additional lateral force applied to unrestrained (cantilevered) retaining structures onsite be estimated as $P_{ae} = 14H^2$ pounds, where H is the height of the wall in feet. The additional seismic force is assumed to be applied at a height of 0.6H above the base of the wall.

6. Retaining wall backfill should conform to recommendations in section 6.1.3, Fill Placement.

6.2.5 Pavement Design

Site soil should provide suitable support for the driveway and roadway asphalt concrete (AC) pavement. The following table provides our pavement design recommendations for the traffic indices noted.

6.2.5.1 Recommended Pavement Sections	
Traffic Index: 5 Traffic Description: residential traffic	Pavement Section (inches)
Caltrans Section 26, Standard Specifications, Asphalt Concrete	3.0
Caltrans Section 26, Class 2 Aggregate Base compacted to at least 95%	8.0
Traffic Index: 6 Traffic Description: moderate truck traffic	Pavement Section (inches)
Caltrans Section 26, Standard Specifications, Asphalt Concrete	3.0
Caltrans Section 26, Class 2 Aggregate Base compacted to at least 95%	9.0

The upper 6 inches of native soil should be compacted to at least of 95 percent of the maximum dry density per ASTM D1557. Aggregate baserock should also be compacted to a minimum of 95 percent. Subgrade and AB dry density should be

evaluated by H&K. In addition to field density tests, subgrade should be proof rolled under the observation of H&K prior to baserock placement.

7 LIMITATIONS

Our professional services were performed consistent with the generally accepted geotechnical engineering principles and practices employed in the site area at the time the report was prepared. 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 this 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 have assumed that subsurface soil conditions encountered at the location of our test pits are generally representative of subsurface conditions across the project site. Actual subsurface conditions at locations between and beyond our 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.

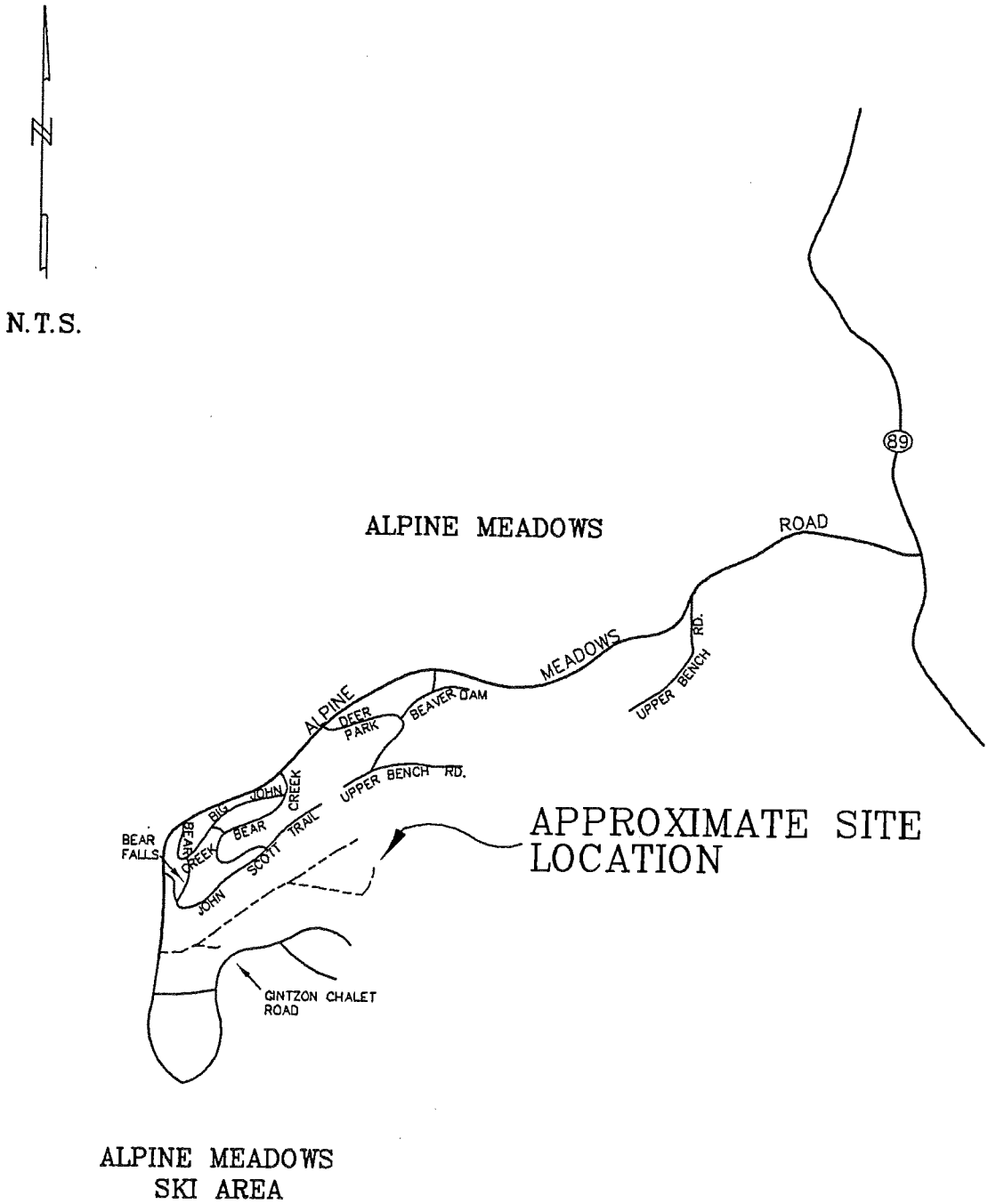
The elevation or depth to groundwater and soil moisture conditions underlying the project site may differ with time and location. The Test Pit Location Plan and Surficial Geologic Map shows approximate test pit locations as determined by pacing distances from identifiable site features. Therefore, test 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 be careful and 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 due to natural processes or works of man, at the site or adjacent properties. Changes in applicable or appropriate standards can occur, whether they result from legislation or broadening of knowledge. Therefore, the recommendations presented in this report should not be relied upon after a period of two years from the issue date without our review.

FIGURES

Figure 1	Site Vicinity Map
Figure 2	Test Pit Location Plan and Surficial Geologic Map



HOLDREGE & KULL
CONSULTING ENGINEERS • GEOLOGISTS
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(530) 587-5156

SITE VICINITY MAP
ALPINE SIERRA
ALPINE MEADOWS, CA

PROJECT NO.: 40376-01

DATE: 11/20/03

FIGURE NO.: 1

APPENDIX A PROPOSAL

HOLDREGE & KULL
CONSULTING ENGINEERS • GEOLOGISTS

October 23, 2003
Project No. 40376-01

Alpine Sierra Partners, LLC
1015 NW 11th Avenue, Suite 243
Portland, OR 97209

Attention: Chris Nelson

Reference: ***Proposed Alpine Sierra Subdivision***
Placer County, California

Subject: ***Proposal for Geotechnical Engineering Services***

We are pleased to provide this proposal for geotechnical engineering services for the proposed Alpine Sierra Subdivision in Alpine Meadows/Placer County, California. To prepare this proposal, we discussed the project with yourself and reviewed a Pre-Tentative Map prepared by Terrance E. Lowell & Associates, dated June 2003.

PROJECT DESCRIPTION

Currently, we understand that the proposed project will involve subdividing approximately 45 acres of previously undeveloped land into about 31 residential lots. We anticipate that future construction will consist of wood-framed structures with concrete slab-on-grade and/or raised-wood floors. Appurtenant construction will include asphalt concrete paved roads, earth retaining structures up to 35 feet high and underground utilities.

SCOPE OF SERVICES

Based on our understanding of the project, we propose to provide design level geotechnical engineering recommendations for project design and construction. Prior to our field exploration, we will review existing geologic and soil references to develop anticipated site conditions. Our engineering geologist will prepare a surface geology map to help evaluate the geotechnical engineering conditions at the site.

(530) 587-5156 • FAX (530) 587-5196 • E-mail: handk@HandK.net • 15826 Donner Pass Rd., Suite 101 • Truckee, CA 96161 • A California Corporation

Field Investigation

Our field investigation will involve the excavation of 8 to 10 exploratory trenches up to about 12 feet deep. Trench locations will be selected based on anticipated project and layout and site access. Exploratory trenches will be excavated using a rubber tired or track mounted backhoe. During our field investigation we will obtain bulk samples of the soil/rock encountered for laboratory testing. Trenches will be logged in the field by an engineer or geologist from our firm.

Laboratory Testing

Laboratory tests will include moisture/density determination, direct shear test, Atterberg Limits determinations, sieve analyses, and R-value testing. Direct shear test results will be used to calculate foundation and retaining wall design criteria for the soil encountered at the site. R-value test results will be used in asphalt pavement design. Sieve analyses and Atterberg Limits testing will provide information for soil classification and engineering properties of select soil samples.

Engineering

Using the results of our laboratory tests, we will develop geotechnical engineering recommendations to provide foundation and retaining wall design criteria and recommended pavement sections.

Geotechnical Report

Following completion of the above tasks, we will compile a report which will include:

- Logs of exploratory trenches;
- Site plan showing approximate locations of exploratory trenches and a reconnaissance level geologic map;

- Description of the soil and rock conditions encountered at the site with emphasis on how the conditions should affect project design and construction;

- Recommendations for earthwork construction, including site preparation recommendations, discussion of reuse of on site soil as structural or non-structural fill, and a discussion of remedial earthwork recommendations, if warranted;

- Recommendations for temporary excavations and trench backfill including permanent and temporary near-surface dewatering recommendations;

- Recommendations for permanent cut and fill slopes;

- Design criteria for foundations, including depth of embedment, allowable bearing capacity for the recommended foundation system(s) and UBC Soil Profile Type to be used for seismic design;

- Lateral earth pressures and drainage recommendations for retaining wall design and a discussion of possible types of retaining structures;

- Recommended asphalt concrete pavement sections; and

- Slab-on-grade concrete subgrade preparation recommendations.

FEES

Our lump sum fee to provide the scope of services described above would be $7500 based on the enclosed 2003 fee schedule. This fee includes the cost of providing a backhoe for our field investigation. If this proposal meets your approval, please sign the enclosed terms and conditions and return one copy as our authorization to proceed.

TIMING

We can begin our field investigation within approximately one week of notice to proceed depending on weather and site moisture conditions. If requested, we will be able to provide verbal design criteria within one week following completion of our field investigation. The final report will be submitted within approximately three to four weeks following completion of our field investigation.

We appreciate the opportunity to submit this proposal and look forward to working with you on this project. If you have any questions or need additional information, please do not hesitate to contact us.

Sincerely,
HOLDREGE & KULL

Gabe Hudson/for
Greg Porter, P.E.
Project Engineer

Gabe Hudson
John K. Hudson, P.E., C.E.G.
Senior Engineer

encl: Terms and Conditions
2003 Fee Schedule

APPENDIX B *IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL ENGINEERING REPORT*

(included with permission of ASFE, Copyright 1998)

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.

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 such risks, 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.*

Rely on Your Geotechnical Engineer for Additional Assistance

Membership in ASFE 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 your ASFE-member geotechnical engineer for more information.

ASFE PROFESSIONAL FIRMS PRACTICING IN THE GEOSCIENCES

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APPENDIX C EXPLORATORY TRENCH LOGS

TEST PIT 1

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE
40376-01	ALPINE SIERRA	6838 FEET	10/28/03	1

EXCAVATING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
TAKEUCHI TB 135 MINI-EX. (12" & 18" BUCKET)	GRAB/BULK	N/A	N/A

SAMPLE NO.	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
			1	SM	BROWN, SLIGHTLY DAMP, LOOSE TO MEDIUM DENSE SILTY SAND (FINE TO COARSE), WITH FINE TO COARSE GRAVEL; CONTAINS ABUNDANT FINE TO COARSE ROOTS & ORGANICS, COBBLES TO 4" (ESTIMATED 5% +3")
			2	SM	BROWN, MOIST, MEDIUM DENSE TO DENSE SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL, CONTAINS COBBLES AND BOULDERS TO 6" (ESTIMATED 5 % +3")
1-1	- -	- -	3		
			4	SM	OANGE-BROWN, MOIST, DENSE TO VERY DENSE SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL; CONTAINS COBBLES AND BOULDERS TO 12" (ESTIMATED 5 % +3")
			5		
			6		BUCKET SCRAPING @ 3.0'; 12" BOULDER OBSERVED
1-2	- -	- -	7		
			8		EXCAVATION REFUSAL @ 7.0'; BOULDERS IN BOTTOM OF EXCAVATION
			9		
			10		
			11		
			12		
			13		
			14		
			15		
			16		
			17		
			18		
			19		
			20		

TEST PIT 2

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE
40376-01	ALPINE SIERRA	6797 FEET	10/28/03	1

EXCAVATING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
TAKEUCHI TB 135 MINI-EX. (12" & 18" BUCKET)	GRAB/BULK	N/A	N/A

SAMPLE NO.	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
			1	SM	BROWN, SLIGHTLY MOIST, LOOSE SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL; CONTAINS ABUNDANT FINE TO COARSE ROOTS AND COBBLES TO 6" (ESTIMATED 5 % + 3")
			2	SM	LIGHT BROWN, MOIST, MEDIUM DENSE TO DENSE SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL; CONTAINS COBBLES AND BOULDERS TO 12"
2-1	--	--	3		OBSERVED 24" BOULDER @ 1.5'
			4		
			5		
2-2	--	--	6		DIFFICULT EXCAVATION
			7		
			8		EXCAVATION REFUSAL @ 7.5'
			9		
			10		
			11		
			12		
			13		
			14		
			15		
			16		
			17		
			18		
			19		
			20		

HOLDREGE & KULL

TEST PIT 3

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE
40376-01	ALPINE SIERRA	6775 FEET	10/28/03	1

EXCAVATING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
TAKEUCHI TB 135 MINI-EX. (12" & 18" BUCKET)	GRAB/BULK	N/A	N/A

SAMPLE NO.	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
			1	SM	BROWN, SLIGHTLY DAMP, LOOSE SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL; CONTAINS ABUNDANT FINE TO COARSE ROOTS AND ORGANICS
			2	GP-GM	BROWN, MOIST POORLY GRADED FINE TO COARSE GRAVEL WITH FINE TO COARSE SAND (WEATHERED ROCK FRAGMENTS)
3-1	--	--	3	RX	ANDESITE ROCK; GRAY, CLOSELY FRACTURED, HIGHLY TO MODERATELY WEATHERED, MODERATELY STRONG
			4		EXCAVATION REFUSAL @ 4.0'
			5		
			6		
			7		
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TEST PIT 4

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE
40376-01	ALPINE SIERRA	6773 FEET	10/28/03	1

EXCAVATING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
TAKEUCHI TB 135 MINI-EX. (12" & 18" BUCKET)	GRAB/BULK	N/A	N/A

SAMPLE NO.	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
			1	SM	BROWN, SLIGHTLY DAMP, LOOSE SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL; CONTAINS ROOTS, ORGANICS AND COBBLES AND BOULDERS TO 24"
4-1	--	--	2	SM	BROWN, SLIGHTLY MOIST, LOOSE TO MEDIUM DENSE SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL; CONTAINS COBBLES AND BOULDERS TO 24" (ESTIMATED 20 % +3")
			3		BUCKET SCRAPING @ 2.5'; SWITCH TO 12" BUCKET
			4		48" BOULDER EXCAVATION @ 3.5'
					EXCAVATION REFUSAL @ 4.0'
			5		
			6		
			7		
			8		
			9		
			10		
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			14		
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			16		
			17		
			18		
			19		
			20		

TEST PIT 5

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE
40376-01	ALPINE SIERRA	6616 FEET	10/28/03	1

EXCAVATING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
TAKEUCHI TB 135 MINI-EX. (12" & 18" BUCKET)	GRAB/BULK	N/A	N/A

SAMPLE NO.	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
			1	SM	BROWN, DRY, LOOSE SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL; CONTAINS ABUNDANT FINE TO COARSE ROOTS AND ORGANICS, COBBLES AND BOULDERS TO 18" (ESITMATED 15 % + 3")
			2	SM	BROWN, MOIST, MEDIUM DENSE SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL, CONTAINS COBBLES AND BOULDERS TO 18"
5-1	--	--	3		DIFFICULT EXCAVATION; SWITCHED BUCKETS
			4	SM	ORANGE BROWN MOIST, DENSE TO VERY DENSE, SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL, COBBLES AND BOULDERS TO 18"
			5		
5-2	--	--	6	RX	ANDESITE; GREY AND WHITE, HIGHLY TO MODERATELY WEATHERED, CLOSELY TO MODERATLY FRACTURED, WEAK TO MODERATELY STRONG
			7		
			8		EXCAVATION REFUSAL @ 7.5'
			9		
			10		SWITCHED TO 12" BUCKET @ 6.0'
			11		
			12		
			13		
			14		
			15		
			16		
			17		
			18		
			19		
			20		

TEST PIT 6

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE
40376-01	ALPINE SIERRA	6784 FEET	10/28/03	1

EXCAVATING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
TAKEUCHI TB 135 MINI-EX. (12" & 18" BUCKET)	GRAB/BULK	N/A	N/A

SAMPLE NO.	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
			1	SM	BROWN, DRY, LOOSE, SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL, CONTAINS FINE TO MEDIUM ROOTS
6-1	--	--	2	SM	BROWN, MOIST, MEDIUM DENSE SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL COBBLES TO 4" (5-10 % + 3")
			3		
			4	RX	ANDESITE; GRAY BROWN, COMPLETLY TO HIGHLY WEATHERED CLOSELY FRACTURED, WEAK TO MODERATLY STRONG
			5		SWITCHED TO 12" BUCKET @ 4.0', DIFFICULT EXCAVATION
			6		EXCAVATION REFUSAL @ 6.0'
			7		
			8		
			9		
			10		
			11		
			12		
			13		
			14		
			15		
			16		
			17		
			18		
			19		
			20		

TEST PIT 7

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE
40376-01	ALPINE SIERRA	6783 FEET	10/28/03	1

EXCAVATING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
TAKEUCHI TB 135 MINI-EX. (12" & 18" BUCKET)	GRAB/BULK	N/A	N/A

SAMPLE NO.	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
			1	SM	BROWN, DRY, LOOSE SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL; COBBLES AND BOULDERS TO 18"; FINE TO MEDIUM ROOT AND ABUNDANT ORGANICS
			2	SM	BROWN MOIST MEDIUM DENSE SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL; COBBLE AND BOULDERS TO 2'
			3	SM	GRADES TO ORANGE BROWN @ 3.0'
			4		BUCKET SCRAPING, DIFFICULT EXCAVATION @ 4.0'; SWITCHED TO 12" BUCKET @ 4.0'
			5		
			6		EXCAVATION REFUSAL @ 6.0'
			7		
			8		
			9		
			10		
			11		
			12		
			13		
			14		
			15		
			16		
			17		
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TEST PIT 8

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE
40376-01	ALPINE SIERRA	6666 FEET	10/28/03	1

EXCAVATING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
TAKEUCHI TB 135 MINI-EX. (12" & 18" BUCKET)	GRAB/BULK	N/A	N/A

SAMPLE NO.	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
			1	SM	BROWN, DRY, LOOSE SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL; CONTAINS COBBLES AND BOULDERS TO 12" AND FINE TO MEDIUM ROOT AND ABUNDANT ORGANICS
			2	SM	LIGHT BROWN, MOIST, MEDIUM DENSE SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL; CONTAINS COBBLES AND BOULDERS TO ESTIMATED 12" (ESTIMATED 10 % +3")
8-1	--	--	3		
			4		BUCKET SCRAPING @ 4.0'; SWITCHED TO 12" BUCKET
			5		
			6		EXCAVATION REFUSAL @ 6.0'
			7		
			8		
			9		
			10		
			11		
			12		
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			15		
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			19		
			20		

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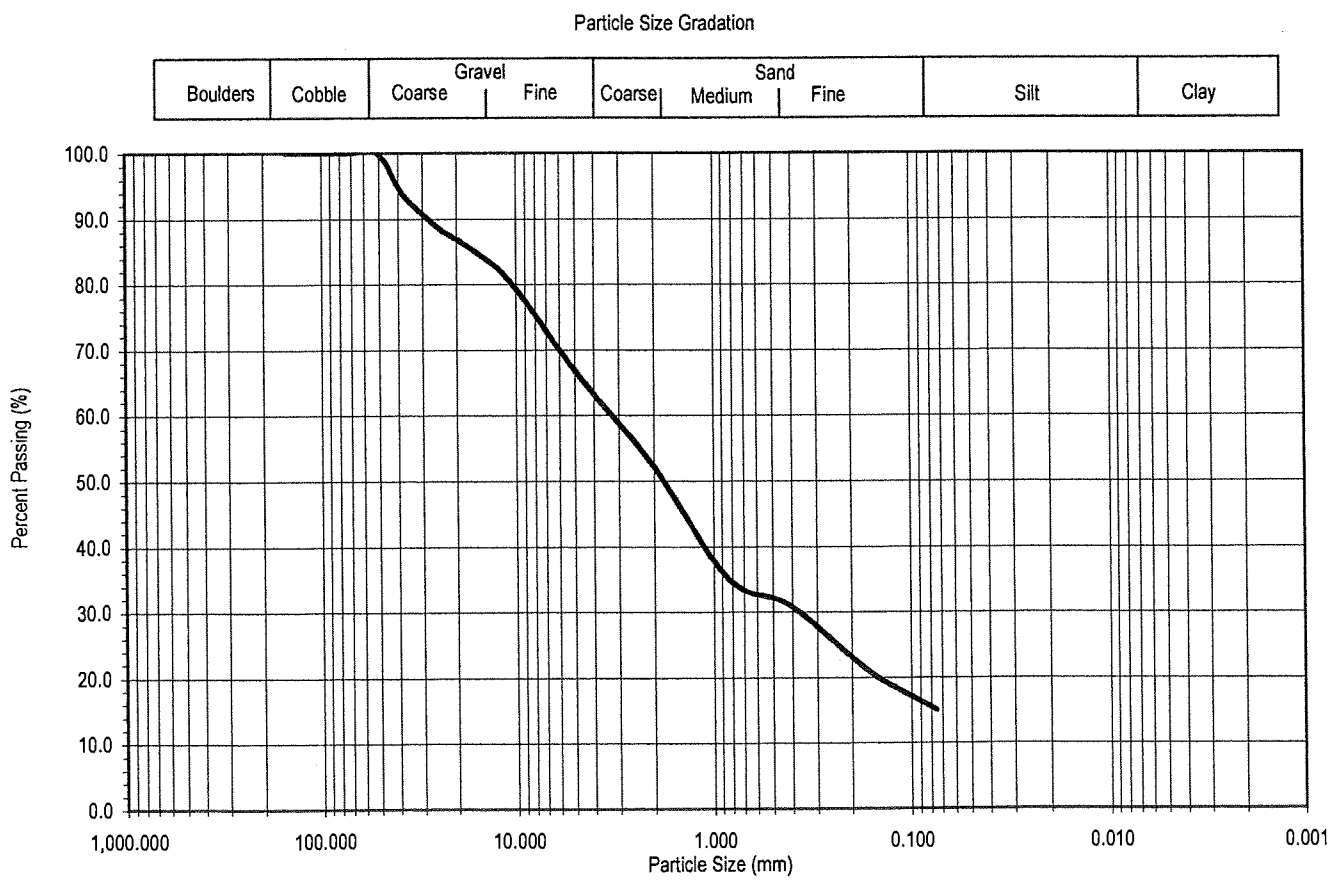
APPENDIX D LABORATORY TEST RESULTS

Particle Size Distribution

ASTM D422

Project No.:	**40376-01**	Project Name:	**Alpine Sierra**		Date: **10/31/2003**
Sample No.:	**1-1**	Boring/Trench:	**T-1**	Depth, (ft.): **3**	Tested By: **JCS**
Description:	**Yellow Brown Silty Sand with Gravel (SM)**				Checked By: **GNP**
Sample Location:	**See Test Pit Location Plan, Figure 2**				Lab. No.: **3-754**

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	4,550.3	**100.0**
3 Inch	3.0000	76.2	0.00	0.0	4,550.3	**100.0**
2 Inch	2.0000	50.8	0.00	0.0	4,550.3	**100.0**
1.5 Inch	1.5000	38.1	279.60	279.6	4,270.7	**93.9**
1.0 Inch	1.0000	25.4	226.54	506.1	4,044.2	**88.9**
3/4 Inch	0.7500	19.1	111.70	617.8	3,932.5	**86.4**
1/2 Inch	0.5000	12.7	166.94	784.8	3,765.6	**82.8**
3/8 Inch	0.3750	9.5	190.72	975.5	3,574.8	**78.6**
#4	0.1870	4.7500	567.50	1,543.0	3,007.3	**66.1**
#10	0.0787	2.0000	633.99	2,177.0	2,373.3	**52.2**
#20	0.0335	0.8500	771.87	2,948.9	1,601.5	**35.2**
#40	0.0167	0.4250	175.95	3,124.8	1,425.5	**31.3**
#60	0.0098	0.2500	263.06	3,387.9	1,162.5	**25.5**
#100	0.0059	0.1500	243.33	3,631.2	919.1	**20.2**
#200	0.0030	0.0750	233.64	3,864.8	685.5	**15.1**



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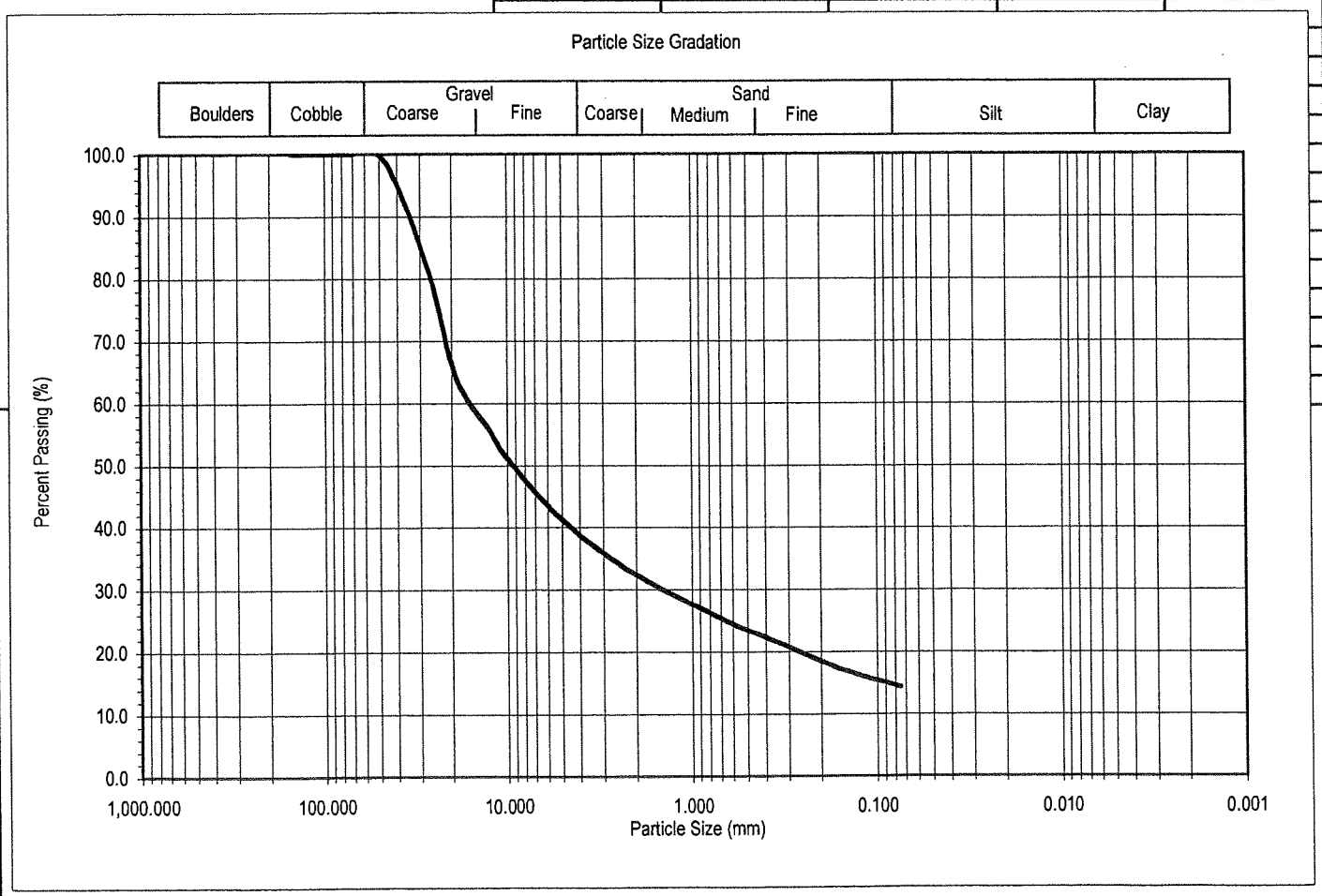
Particle Size Distribution

ASTM D422

Project No.:	**40376-01**	Project Name:	**Alpine Sierra**		Date: **10/31/2003**
Sample No.:	**3-1**	Boring/Trench:	**T-3**	Depth, (ft.): **2**	Tested By: **JCS**
Description:	**Dark Brown Silty Gravel with Sand (SM)**				Checked By: **GNP**
Sample Location:	**See Test Pit Location Plan, Figure 2**				Lab. No.: **3-754**

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	3,842.1	**100.0**
3 Inch	3.0000	76.2	0.00	0.0	3,842.1	**100.0**
2 Inch	2.0000	50.8	0.00	0.0	3,842.1	**100.0**
1.5 Inch	1.5000	38.1	242.10	242.1	3,600.0	**93.7**
1.0 Inch	1.0000	25.4	571.60	813.7	3,028.4	**78.8**
3/4 Inch	0.7500	19.1	566.58	1,380.3	2,461.8	**64.1**
1/2 Inch	0.5000	12.7	312.90	1,693.2	2,148.9	**55.9**
3/8 Inch	0.3750	9.5	215.75	1,908.9	1,933.1	**50.3**
#4	0.1870	4.7500	369.17	2,278.1	1,564.0	**40.7**
#10	0.0787	2.0000	319.04	2,597.1	1,244.9	**32.4**
#30	0.0236	0.6000	304.02	2,901.2	940.9	**24.5**
#40	0.0167	0.4250	69.09	2,970.3	871.8	**22.7**
#60	0.0098	0.2500	113.70	3,083.9	758.1	**19.7**
#100	0.0059	0.1500	104.85	3,188.8	653.3	**17.0**
#200	0.0030	0.0750	101.76	3,290.6	551.5	**14.4**



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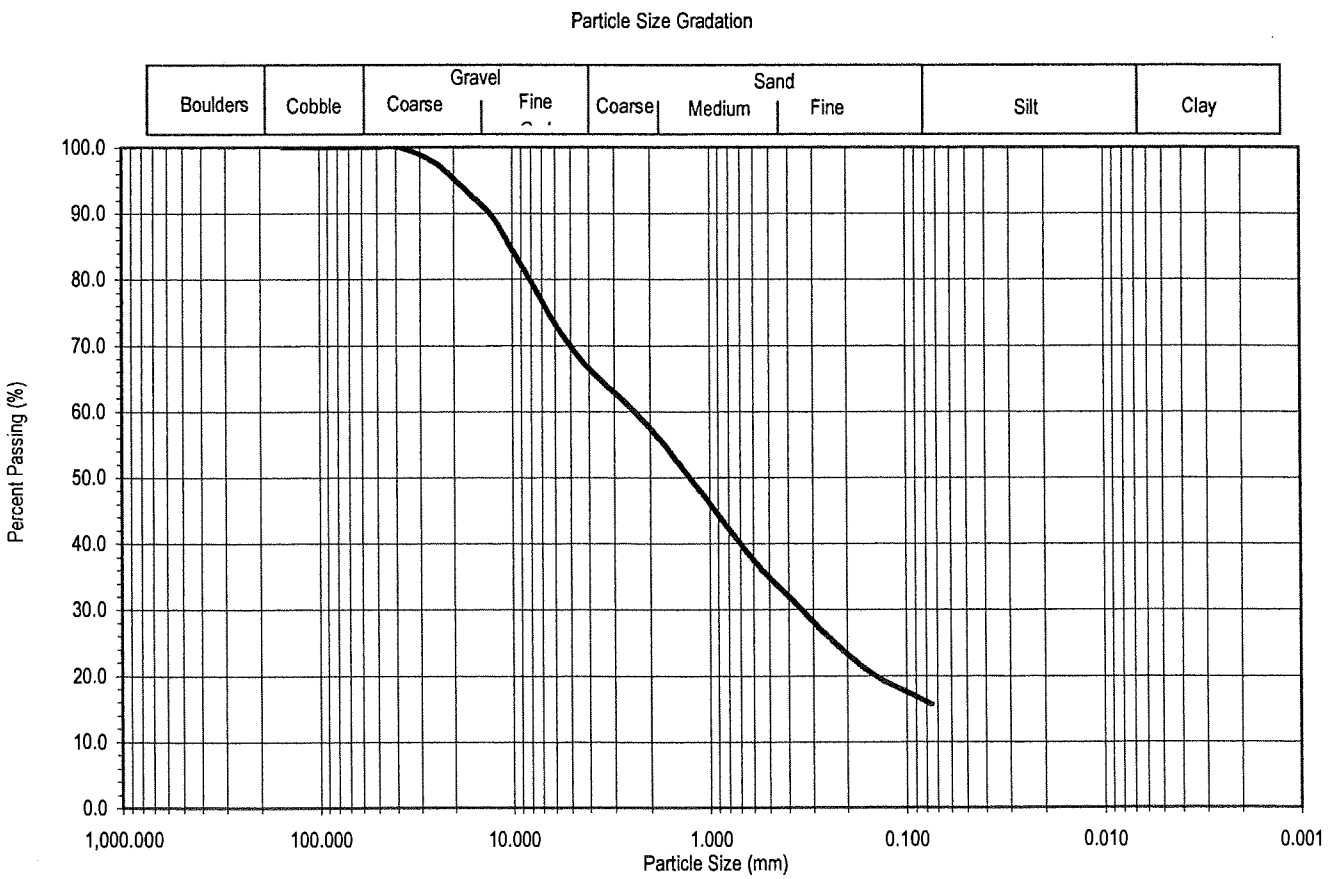
Particle Size Distribution

ASTM D422

Project No.:	**40376-01**	Project Name:	**Alpine Sierra**		Date: **10/31/2003**
Sample No.:	**5-2**	Boring/Trench:	**T-5**	Depth, (ft.): **6**	Tested By: **JCS**
Description:	**Brown Silty Sand with Gravel (SM)**				Checked By: **GNP**
Sample Location:	**See Test Pit Location Plan, Figure 2**				Lab. No.: **3-754**

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	3,581.2	**100.0**
3 Inch	3.0000	76.2	0.00	0.0	3,581.2	**100.0**
2 Inch	2.0000	50.8	0.00	0.0	3,581.2	**100.0**
1.5 Inch	1.5000	38.1	0.00	0.0	3,581.2	**100.0**
1.0 Inch	1.0000	25.4	76.22	76.2	3,505.0	**97.9**
3/4 Inch	0.7500	19.1	117.97	194.2	3,387.0	**94.6**
1/2 Inch	0.5000	12.7	178.70	372.9	3,208.3	**89.6**
3/8 Inch	0.3750	9.5	217.47	590.4	2,990.8	**83.5**
#4	0.1870	4.7500	515.24	1,105.6	2,475.6	**69.1**
#10	0.0787	2.0000	420.18	1,525.8	2,055.4	**57.4**
#30	0.0236	0.6000	713.78	2,239.6	1,341.6	**37.5**
#40	0.0167	0.4250	166.87	2,406.4	1,174.7	**32.8**
#60	0.0098	0.2500	249.66	2,656.1	925.1	**25.8**
#100	0.0059	0.1500	196.16	2,852.3	728.9	**20.4**
#200	0.0030	0.0750	169.69	3,021.9	559.2	**15.6**



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APPENDIX H-2

Holdrege and Kull Geotech 2013

GEOTECHNICAL ENGINEERING REPORT UPDATE for Alpine Sierra Development Alpine Meadows/Placer County, California

Prepared for:
Alpine Sierra Partners, LLC
1015 NW 11th Avenue, Suite 243
Portland, Oregon

Prepared by:
Holdrege & Kull
10775 Pioneer Trail, Suite 213
Truckee, California 96161

Project No. 40376-02
September 20, 2013

HOLDREGE & KULL
CONSULTING ENGINEERS • GEOLOGISTS

Project No. 40376-02
September 20, 2013

Alpine Sierra Partners, LLC
1015 NW 11th Avenue, Suite 243
Portland, Oregon 97209

Attention: Mr. Chris Nelson

Reference: ***Alpine Sierra Subdivision***
Alpine Meadows/Placer County, California

Subject: ***Geotechnical Engineering Report Update Letter***

This letter presents our geotechnical engineering report update for the proposed Alpine Sierra Subdivision to be constructed in Alpine Meadows/Placer County, California. Our current scope of services included a site visit, review of plans titled "Alpine Sierra Subdivision, PD-Use Permit/Site Plan" dated August, 2013, review of our previously prepared geotechnical report for the Alpine Sierra Subdivision, dated December 11, 2003, and preparation of this letter. This letter is considered a part of the geotechnical engineering report for the Alpine Sierra Subdivision and should be bound to it.

Our professional opinion is that with the exception of the updated recommendations provided in the following letter, the recommendations provided in our geotechnical engineering report for the Alpine Sierra Subdivision, dated December 11, 2003, are applicable to this project site. This report update letter should be used in conjunction with our previously prepared geotechnical engineering report for project design, submittal, and construction. A subsurface exploration was not part of our scope of services for this report update letter. Holdrege & Kull should be retained to provide earthwork observation and materials testing services during grading, and to develop site specific design criteria as project plans are developed.

This letter contains updated geotechnical recommendations that will supersede portions or all of the following section(s) of the Alpine Sierra Subdivision report:

- 6.1.8 Surface Water Drainage;
- 6.2.2 Seismic Design Criteria;
- 6.2.3 Slab-on-Grade Floor Systems; and,
- 6.2.4 Retaining Wall Design Criteria.

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Site Description

The project site consists of an approximately 45-acre undeveloped parcel located in Alpine Meadows, California. The project site is bounded by the existing Bear Creek subdivision to the north, an undeveloped property to the east, the Scott Peak and Alpine Chalet Condominiums to the south, and Alpine Meadows Road to the west. Vegetation at the site consists of dense conifer trees and brush. Rock outcrops are located near the east central portion of the site, and are generally devoid of vegetation.

Site elevations range from approximately 6,600 feet above mean sea level (MSL) near the north central portion of the site to approximately 7,076 feet MSL near the southeast corner of the site. Site grades vary significantly across the property but typically slope moderately to steeply down from south to north. Surface water drainage consists of overland flow in a general south to north direction.

Proposed Improvements

Information about the proposed project was obtained from our site visit, conversations with Mr. Chris Nelson of Alpine Sierra Partners, LLC, review of site plans dated August 2013, and our previous experience in the site area. Project plans were preliminary at the time we prepared this report; however, we understand that the project will involve subdividing the property into 47 single-family lots. Appurtenant construction will include paved roadways and underground utility construction. In addition, approximately 2,700 lineal feet of retaining walls and three "Conspan" bridges are currently planned to construct the roadways.

We anticipate the future proposed residences will consist of two- to three-story wood and steel framed structures with concrete slab-on-grade and/or raised-wood floors supported on conventional shallow spread foundations. We understand that some structures may have daylight basements/garages that will be partially supported on cast-in-placed reinforced concrete retaining walls. Assumed maximum wall and column loads will be about 4 kips per lineal foot and 80 kips, respectively. We anticipate that earthwork cuts and fills up to about 20 feet and construction of retaining walls up to about 20 feet in height will be required for construction of roadways.

Anticipated Subsurface Conditions

Based on the results of our previous subsurface investigation, we anticipate that near-surface soil will consist of about 4 to 18 inches of silty sand (SM) containing organic material (topsoil) over a majority of the site. The silty sand topsoil will likely be underlain by medium dense to very dense silty sand with gravel (SM) and silty gravel with sand (GM) containing varying amounts of cobbles and boulders up to approximately 4 feet in

diameter. Each of our previous test pits encountered refusal at depths ranging from 4 to 7.5 feet below the ground surface on either boulders or volcanic rock. The quality of volcanic rock varied from moderately weathered to fresh, closely to moderately fractured, and moderately strong to strong. Depth to rock will likely vary across the site and should be anticipated at or near the ground surface within outcrop areas exposed in the eastern and central portions of the site.

Groundwater was not encountered in test pits excavated during our previous investigation at the site. However, 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 or heavy rain events.

Regional Faulting and Seismicity

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;* by Charles W. Jennings and William A. Bryant, California Geological Survey, 2010 (CGS, 2010).
- *Geologic Map of the North Lake Tahoe-Donner Pass Region, Northern Sierra Nevada, California;* by Arthur G. Sylvester et.al., California Geological Survey, 2012 (Sylvester et.al., 2012).
- *Geologic Map of the Lake Tahoe Basin, California and Nevada,* by George J. Saucedo, California Geological Survey, 2005 (Saucedo, 2005).
- *Geologic Map of the Chico Quadrangle, California,* by G.J. Saucedo and D.L. Wagner, California Division of Mines and Geology, 1992 (DMG, 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 12 miles

northwest), a group of unnamed faults southeast of Truckee (active and potentially active, approximately 8 and 10 miles northeast, respectively), the Polaris Fault (active, approximately 12 miles northeast), the North Tahoe Fault (active, approximately 8 miles southeast), and the West Tahoe – Dollar Point Fault (active, approximately 8 miles southeast). Earthquakes associated with these faults may cause strong ground shaking at the project site.

A trace of the Tahoe Sierra Frontal Fault Zone (TSFFZ) is shown as trending near or through the southwest corner of the project area on the geologic maps referenced above. This fault zone trends in a general northwest direction sub-parallel to the crest of the Sierra from south or Lake Tahoe to north of Donner Summit. The fault is considered to be Quaternary aged (showing displacement within the past 1.6 million years) and represents the structural boundary between the Sierra Nevada and Basin and Range geomorphic provinces. The TSFFZ may be a splay or step off of the West Tahoe Fault, which has relatively recently been shown to be an active fault (Brothers, et. al, 2009) located about 8.7 miles southeast of the site.

The maps prepared by Saucedo and Wagner (1992) and Saucedo (2005) show the trace of the TSFFZ trending west of the building sites and dotted (concealed) beneath glacial till deposits. The map prepared by Sylvester et. al. (2012) shows the trace as solid (accurately located) and trending through the western portion of the site. The geologic map prepared by Sylvester et.al. (2012) also shows an unnamed fault trace crossing near or through the northeast corner of the site. This fault trace is shown as dashed (approximately located) and discontinuous (approximately 6 miles in length).

The CGS (2010) fault activity map of California shows the TSFFZ trending in a general northwest direction east of the project area. Two fault traces designated as Quaternary age or of indeterminate activity are shown as trending through the southwest corner of the project area and to the west. CGS (2010) identifies the TSFFZ as potentially active. Fault splays west of the TSFFZ are shown as Quaternary age or indeterminate activity.

We reviewed the "Digital Images of Official Maps of Alquist-Priolo Earthquake Fault Zones of California, Northern California Region", which describes active faults and fault zones (activity within 11,000 years), as part of the Alquist-Priolo Earthquake Fault Zoning Act. The document and the on-line update indicate the site is not located within or near an Alquist-Priolo active fault zone.

We reviewed aerial photographs of the site including an Aerial Topographic Survey of Squaw Valley and Alpine Meadows by Andregg Geomatics, dated May 30, 2012, scale 1:6,000. We have also made a site reconnaissance to look for evidence of fault traces. The fault trace can be seen on aerial photographs and on the ground both north and south of the site area generally as a contact between different rock types and as

topographic features, such as gullies. There are no prominent fault scarps, ponds or depressions that are indicative of active faulting. We did not observe any evidence of the fault within the area covered by glacial till.

The potential hazard associated with earthquake faults involves surface rupture and strong ground motion. Recently a report published in the Geological Society of America Bulletin on May 18, 2012, concluded that the Tahoe Sierra frontal fault zone is potentially active and may be active. Given this information, there is a potential for surface rupture at or near the site. However, the recurrence interval of earthquakes on the Tahoe Sierra Frontal Fault Zone is likely on the order of several hundred to several thousand years. Therefore, the potential for surface rupture at the site in any one year is low. Establishing building setback distances along the fault trace as it trends through the southwest corner of the project area is not warranted. If structures are planned in this area of the site, further investigation should be completed to evaluate the location and activity of the fault trace.

Concerning strong ground motion, earthquakes centered on regional faults in the area, such as the West Tahoe fault, would likely result in higher ground motion at the site than earthquakes centered on smaller faults that are mapped closer to the site.

Conclusions

The following conclusions are based on our field observations, our experience in the project area, and the results of our previously prepared geotechnical engineering report.

1. We anticipate that the majority of the site is underlain by granular native soil and rock that will be suitable for support of roadways, utilities, and structures. Cobbles, boulders, and near-surface volcanic rock should be anticipated in site excavations. Excavations for roadways, foundations, and underground utilities that extend into rock may be difficult and require special excavation techniques, such as spot blasting. No highly plastic, potentially expansive, or compressible soil is known to be present at the site. The potential for liquefaction and lateral spreading at the site is considered low. We anticipate that site soil will be generally suitable for reuse as structural fill; however, processing to remove oversize material will likely be necessary.

2. A trace of the Tahoe Sierra Frontal Fault Zone is mapped as crossing near or through the southwest corner of the project area. This fault zone is considered to be Quaternary aged (showing displacement within the past 1.6 million years) and represents the structural boundary between the Sierra Nevada and Basin and Range geomorphic provinces. The TSFFZ is currently considered potential active and research into the activity of the fault is ongoing. The recurrence

interval of earthquakes on the Tahoe Sierra Frontal Fault Zone is likely on the order of several hundred to thousands of years. Therefore, if the fault is active, the potential for surface rupture in any one year at the site is relatively low. The project area is not located within an Alquist-Priolo Earthquake Fault Zone. Therefore, establishing setbacks along the fault trace as it trends through the southwest corner of the project area is not warranted. If structures are planned in this area of the site, further investigation should be completed to evaluate the location and activity of the fault trace.

3. Although groundwater was not encountered in our test pits to the maximum depth explored, near-surface soil layers will likely become seasonally saturated. Positive surface water drainage will be important across the site to help reduce the potential for groundwater to collect in residence crawl spaces, cause moisture migration through concrete slabs-on-grade, cause degradation of asphalt concrete pavements, and contribute to frost heave and other adverse conditions. We have provided updated recommendations to reduce the potential for these adverse effects in the recommendations section below.

Recommendations

The following recommendations are based on our understanding of the project as currently proposed, our site visit, and our experience in the project area. The recommendations provided below supersede those provided in the Alpine Sierra Subdivision Geotechnical Engineering Report dated December 11, 2003.

Surface Water Drainage

The following paragraphs should replace all text in Section 6.1.8 of the Alpine Sierra Subdivision Geotechnical Engineering Report.

Based on our observations and 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. In addition, near-surface rock was encountered in our test pits at depth of about 4 to 7.5 feet below existing site grade. Depending on final site grades, rainfall, irrigation practices, and other factors beyond the scope of this study, perched groundwater will likely seasonally develop above onsite rock. 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.

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.

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. If physical obstructions or lot lines prohibit drainage away from buildings, a 5 percent slope should be constructed towards a drainage swale or other conveyance system that diverts water away from the foundation. Infiltration of roof or pavement runoff should not be allowed within 10 feet of structures. Paved areas should be sloped away from structures a minimum of 2 percent and drainage gradients should be maintained to carry all surface water to a properly designed infiltration or detention basin.

Drains should be constructed on the upslope side of continuous interior wall foundations and in the 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.

We recommend that the elevation of the interior subgrade in the crawl space be higher than the exterior ground surface. If the design of the residence is such that the crawl space must be lower than the surrounding grade, drains should be installed in the crawl space area. The subgrade should 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.

Seismic Design Criteria

The following paragraphs should replace all text in Section 6.2.2 of the Alpine Sierra Subdivision Geotechnical Engineering Report.

In accordance with the 2010 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 (39.1728^0 N and -120.2310^0 W, respectively) obtained from Google Earth.

S_s = 1.127g	Figure 1613.5(3), 2010 CBC
S_1 = 0.397g	Figure 1613.5(4), 2010 CBC
F_a = 1.0	Table1613.5.3(1), 2010 CBC
F_v = 1.403	Table 1613.5.3(2), 2010 CBC

Based on our literature review, our site reconnaissance, and our experience in the area, we recommend using Site Class C (Table 1613.5.2, 2010 CBC) to evaluate seismic loads.

Slab-on-Grade Construction

The following paragraphs should replace paragraph 4 of Section 6.2.3 of the Alpine Sierra Subdivision Geotechnical Engineering Report.

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 receive moisture-sensitive floor coverings, or in areas where groundwater is encountered during grading. Subdrains should consist of a minimum of 4-inches of clean crushed gravel placed over native subgrade leveled or sloped at 2 percent towards a 4-inch diameter perforated drain pipe. The drain pipe should be placed with perforations face down in a minimum 12 inch wide gravel filled trench. The depth of the trench may vary depending on cover requirements for the drain pipe and the slope required 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.

In slab-on-grade areas where moisture sensitive floor coverings are proposed, a vapor barrier (e.g. 15 mil Stego® Wrap) should be placed over the base course or gravel subdrain to reduce the migration of moisture vapor through the concrete slab. The Stego® Wrap should be installed in accordance with the manufacturer's instructions.

Concrete should be placed directly on the vapor barrier. All slab concrete should have a water-cement ratio of 0.45 or less.

Regardless of the type of vapor barrier 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.

Retaining Wall Design Criteria

The following paragraphs should replace paragraph 5 (Page 22) of Section 6.2.4 of the Alpine Sierra Subdivision Geotechnical Engineering Report.

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 0.30g with the Mononobe-Okabe/Seed Whitman procedure to evaluate seismic loading on retaining walls.

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

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.

Retaining wall backfill 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 35 percent passing the No. 200 sieve. Backfill should be uniformly moisture conditioned to within 2 percent of the ASTM D1557 optimum moisture content and compacted with appropriate compaction equipment to at least 90

percent of the maximum dry density per ASTM D1557. If the retaining wall backfill will support foundations or rigid pavements, the backfill should be compacted to at least 95 percent of the maximum dry density per ASTM D1557. We should review and provide specific backfill criteria for all retaining walls over 10 feet in height. Utilities that run through retaining wall backfill should not pass through the wall or other rigid structures without allowance for vertical movement of several inches.

Closing

The limitations outlined in the Alpine Sierra Subdivision Geotechnical Engineering Report are considered applicable to this report update. 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. We have prepared this letter for your exclusive use in accordance with the generally accepted geotechnical engineering practice as it exists in the site area at the time of our services. No warranty, express or implied, is intended.

We appreciate the opportunity to provide assistance on this project. If you have any questions regarding this letter, please contact the undersigned.

Sincerely,

Holdrege & Kull

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Copies: 4 to Doug Clyde, Alpine Sierra Partners, LLC

GEOTECHNICAL ENGINEERING REPORT
for
ALPINE SIERRA SUBDIVISION
Alpine Meadows/Placer County, California

[redacted]

Prepared by:
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Project No. 40376-01
December 11, 2003

Project No. 40376-01
December 11, 2003

Alpine Sierra Partners, LLC
1015 NW 11th Avenue Suite 243
Portland, Oregon 97209

Attention: Chris Nelson

Reference: ***Alpine Sierra***
Alpine Meadows/Placer County, California

Subject: ***Geotechnical Engineering Report***

This report presents the results of our geotechnical engineering investigation for the proposed Alpine Sierra subdivision to be located in Alpine Meadows/Placer County, California. The proposed project involves subdividing the previously undeveloped 45 acre site into a single-family residential subdivision. Appurtenant construction will include underground utilities, earth retaining structures and asphalt concrete paved roads.

Based on our work completed to date, no severe soil or groundwater constraints were observed which would preclude the proposed development. Shallow rock may require special rock excavation techniques for site grading and underground utility construction. The proposed project is located in a relatively steep mountain area. Portions of the project's second phase may be subject to small stream flooding and debris flows that are typical of many local subdivisions in steep mountain areas. Our opinion is that the project can be completed as proposed, provided that the recommendations presented in this report are implemented. Specific recommendations regarding the potential geologic hazards and geotechnical aspects of project design and construction are presented in the following report.

The findings presented in this report are based on our site observations, subsurface exploration, laboratory test results and our experience in the project area. Holdrege & Kull should be retained to provide construction monitoring services during earthwork and foundation construction to observe subsurface conditions encountered with respect to our engineering recommendations provided in this report.

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

Sincerely,
HOLDREGE & KULL

Gregory N. Porter, P.E.
Project Engineer

John K. Hudson, P.E., C.E.G.
Senior Engineer

copies: Ed Staniforth, TLA

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APPENDICES

Appendix A Proposal
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Appendix C Exploratory Test Pit Logs
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1 INTRODUCTION

Holdrege & Kull (H&K) is pleased to present our geotechnical engineering report for the proposed Alpine Sierra subdivision located in Alpine Meadows/Placer County, California. We performed our investigation in general accordance with our October 23, 2003 proposal for the project. A copy of the proposal is included as Appendix A of this report. For your review, Appendix B 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 Site Description

The project site consists of approximately 45 acres of undeveloped property in Alpine Meadows/Placer County, California. A Site Vicinity Map is presented as Figure 1. The existing Bear Creek subdivision is located adjacent to the site on the north and the Scott Peak and Alpine Chalet condominiums are on the south. The project site is bounded by undeveloped property to the east and Alpine Meadows Road to the west. Site grades vary significantly across the site, typically sloping moderately to steeply from south to north. A low rock knoll is located in the east central part of the site. Ground surface elevations range from approximately 7076 feet in the southeast corner of the site to 6600 feet in the north central portion of the site. Bear Creek crosses the far western part of the site and a tributary creek crosses the eastern part of the site.

Vegetation consists of dense conifer trees, mountain bushes and shrubs. Portions of the site consist of rock outcrops with sparse vegetation.

1.2 Proposed Improvements

We understand that the proposed development will consist of subdividing the existing property into approximately 31 single family residential lots. Current plans indicate that the development will occur in three phases. Appurtenant construction will include associated paved roads and underground utility construction. Earthwork cuts and fills of 20 feet or more will be required to grade all three phases of construction. Approximately 2760 lineal feet of retaining wall and three "Conspan" bridges are currently planned to construct the roads.

We anticipate that future residential structures will consist of two- to three-story, wood, concrete and steel-framed structures founded on shallow foundations, with raised wood and/or concrete slab-on-grade floors. Concrete retaining walls may also be incorporated in residential structures. Structural loading information was not available at this time.

Information pertaining to the proposed development and the project site was obtained from our site visits and the Alpine Sierra Subdivision Pre-Tentative Map, dated June 2003, prepared by Terrence E. Lowell & Associates, Inc. We also reviewed an Interim Avalanche Hazard Study for Alpine Sierra, dated May 2003, and an Addendum dated June 2003, prepared by Norman A. Wilson.

1.3 Purpose

The purpose of our geotechnical engineering report was to provide geotechnical engineering recommendations for project design and construction based on our surface observations and subsurface exploration at the project site. We recommend retaining our firm to provide construction monitoring services during earthwork and foundation excavation to observe subsurface conditions encountered with respect to our recommendations.

1.4 Scope of Services

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

- We performed a site reconnaissance, literature review and subsurface exploration involving backhoe excavated test pits.
- We logged the subsurface conditions encountered and collected bulk soil samples for classification and laboratory testing.
- We performed laboratory tests on selected soil samples obtained during our subsurface exploration to help evaluate material properties.
- Based on our subsurface exploration and the results of our laboratory testing, we performed engineering analyses to develop geotechnical engineering recommendations for project design and construction.

2 SITE GEOLOGY

The proposed Alpine Sierra subdivision is located in the northern Sierra Nevada geologic province about two miles east of the Sierra Nevada crest at an elevation of approximately 6,800 feet. The Sierra Nevada Mountains were formed by large intrusions of granitic rock during late Mesozoic time. Subsequent faulting and volcanic activity during Tertiary time raised the mountain range to its present position.

The northern Sierra Nevada geology is dominated by volcanic rocks of Tertiary to Quaternary age. Scott Peak, located south of the site, is made up of Pliocene (late Tertiary) volcanic rock consisting of andesite flows and lahar (mud flow) deposits. Volcanic rock that is slightly older than the Scott Peak volcanic rocks underlies much of the project site, but still consists of andesite and lahar deposits. Rock outcrops are present primarily in the southern part of the site at many areas within the site boundary. Volcanic rock can be observed in isolated outcrops in the northern part of the site and in the John Scott Trail road cut. It appears that volcanic rock extends under most of the site at relatively shallow depths. Granitic bedrock is present within the stream channel of Bear Creek at the proposed Road A crossing.

The Ward Peak area and the crest of the Sierra were occupied during much of the Quaternary Period by alpine glaciers that flowed down Bear Creek to the Truckee River. The northern portion of the site is underlain by Tioga age glacial till deposits (Birkeland, 1963). Till deposits consist of silty sandy gravel and gravelly sand with numerous cobbles and boulders. This is evident on site with the presence of many large granitic boulders. Surficial geology is shown on Figure 2, Test Pit Location Plan and Surficial Geologic Map.

2.1 Slope Stability

The proposed subdivision is located within and near steep slopes. Slope instability includes avalanches, landslides, debris flows, and rock fall. Site soil types are relatively competent and are not prone to landsliding. However, the area south of the site is very steep and some debris flows were observed in the site area. Debris piles and hummocky topography, characteristic of debris deposition, were observed in the north-central part of the site. Debris flows usually originate within saturated soil on very steep slopes and are transported in a fluid state into

the stream channels. Debris flows are usually confined to the stream channel until the stream gradient changes and becomes blocked or the channel is no longer confined. The volcanic rock in this area is not deeply weathered or altered and soils are well drained, not usually saturated. The slopes above the site have not developed a thick soil overlying the rock that may contribute to debris flows. Consequently, Phase II of the proposed subdivision does not have greater potential for debris flows than many other locations in the Bear Creek drainage.

The northeast portion of the site is underlain by alluvial deposits, which consist of sand and gravel with some cobbles and boulders transported by water. This area is located northeast of the stream channel that flows down from the Alpine Meadows ski run known as the Munchkin Chutes. The stream channel abruptly decreases gradient and forms a rough alluvial apron or fan below approximate elevation 6700 feet. The ground surface is characterized by numerous discontinuous stream channels and hummocky terrain. It appears that over geologic time the stream channel has shifted across the alluvial apron. The present stream channel is incised at the western edge of the apron, between the alluvium and andesite rock. Based on the observed geomorphology, this area may be subject to small stream flooding and debris flows. The small stream channel located in the northeast corner of the site also appears to have some associated alluvial deposits. Debris flows at the site are a rare event that would be associated with large storm events that may occur on the order of one hundred years.

Options to reduce the potential hazard due to debris flows and/or rockfall include the construction of barriers such as levees, stream channel modification and/or reinforcement of individual homes. Reinforcement of individual homes would be similar to design for avalanche protection. For protection from debris flows, we recommend that lot grading and house placement provide positive surface drainage away from structures. Reinforced concrete foundations for structures should extend a minimum of 4 feet above the existing ground surface.

2.2 Rock Fall

The potential rock fall hazard at the site is relatively low. The parent rock above the site consists of Lahar (or volcanic mudflow) deposits that tend to weather into relatively small, non-spherical boulders that are not prone to rolling down slope.

Colluvium and some large boulders are present at the base of the slope above the subject site. The slope gradient decreases below 7000 feet and the majority of this material appears to stop before reaching the Alpine Sierra site. The dense vegetation at and near the site significantly reduces the potential for both debris flows and rock fall. In the event of an intense forest fire, the potential for slope instability may increase.

3 SUBSURFACE EXPLORATION

We performed our subsurface exploration to help characterize subsurface conditions at the site. Our exploration included literature review and field exploration as described below.

3.1 Field Exploration

Subsurface conditions at the site were explored on October 28, 2003, by excavating 8 test pits to depths ranging from 4.0 to 7.5 feet below the existing ground surface (bgs). The test pits were excavated with a Takeuchi TB135 mini-excavator equipped with an 18-inch bucket. As excavation of a test pit became more difficult, the excavating contractor switched to a 12-inch bucket. Test pit locations were selected based on locations of proposed improvements and site access.

An engineer from our firm logged the soil conditions exposed in the test pits, visually classified soil and collected bulk soil samples for laboratory testing. Soil samples were packaged and sealed in the field to reduce moisture loss and were returned to our laboratory for further testing. Upon completion, the test pits were backfilled with the excavated soil. The approximate locations of our test pits are shown in the Test Pit Location Plan and Surficial Geologic Map, Figure 2.

3.2 Subsurface Soil Conditions

Approximately 4 to 18 inches of dark brown, silty sand top soil was encountered across a majority of the site. Near surface soil encountered underlying the top soil consisted of medium dense to very dense Silty Sand (SM) with Gravel and Silty Gravel with Sand (GM), containing varying amounts of cobbles and boulders up to approximately 4 feet in diameter. All test pit excavations encountered refusal

at depths of 4 to 7½ feet below the ground surface (bgs). Refusal was encountered on what appeared to be large boulders and/or volcanic rock. The rock quality varied from moderately weathered to fresh, closely to moderately fractured and moderately strong to strong. More detailed descriptions of subsurface conditions are presented in our test pit logs in Appendix C.

3.4 Groundwater Conditions

We did not encounter groundwater during our subsurface exploration. However, based on our experience in the project area, seasonal saturation of near-surface soil should be anticipated at the site. Groundwater may become perched above near-surface rock encountered at varying depths over the site. Fluctuations in soil moisture content and groundwater levels should be anticipated depending on precipitation, irrigation, runoff conditions, snow melt and other factors.

4 LABORATORY TESTING

We performed laboratory tests on bulk soil samples collected from our exploratory trenches to help evaluate their engineering properties. Laboratory tests performed included Moisture Content (ASTM Test Method D 2216) and Sieve Analysis (ASTM D422). Sieve analysis data were used to evaluate the Unified Soil Classification System (USCS) classifications as described in Section 2.3, Subsurface Soil Conditions. Specific soil classification and laboratory test data is included on the trench logs and in Appendix D.

5 CONCLUSIONS

The following conclusions are based on our field observations, laboratory test results, and our experience in the area.

1. Soil conditions encountered in our subsurface exploration generally consisted of medium dense to dense Silty Sand with Gravel (SM) and Silty Gravel with Sand (GM). The observed soil typically contained varying amounts cobbles and boulders up to approximately 4 feet in diameter. These soil types should provide adequate support for planned structures on conventional spread foundations.

2. Underlying near-surface soil, the excavator used for our field exploration encountered essential refusal at depths of 4 to 7½ feet bgs on what appeared to be volcanic rock and/or boulders in all test pit excavations. We also observed rock outcrop and boulders in numerous areas across the site. Excavation for roads, footings and underground utilities that extend into rock may be difficult and require special excavation techniques. Material excavated from these areas may require significant processing or crushing to remove oversize material prior to reuse as structural fill or utility trench backfill. Recommendations for the reuse of site soil for fill and backfill are discussed below.

3. Groundwater was not encountered during our subsurface exploration in our test pit excavations. However, we expect that near-surface soil at the site may become seasonally saturated, particularly during spring snow melt. Positive surface water drainage, subsurface drains behind below grade walls and beneath concrete slabs-on-grade and asphalt pavement will be important for performance of structures and other site features.

4. Due to the relatively steep terrain at the project site, steep constructed slopes and high retaining structures are planned for the project. Stability of cut and fill slopes involves true slope stability related to landslides and erosion potential. Site soil types are relatively competent and may be constructed at steep angles from a slope stability standpoint. Fill slopes may be stabilized using internally stabilized systems such as mechanically stabilized embankments. We anticipate that steep cut slopes will be stable in near-surface rock. Proposed slopes may also be retained with rockery walls and/or conventional cantilever retaining walls. Final design should be completed at specific locations as project planning is completed. A combination of vegetation and Rock Slope Protection (RSP) should be placed on cut and fill slopes for erosion protection.

5. Geotechnical engineering design criteria provided in this report are for native soil encountered in our test pit excavations at the site. Our experience is that site specific design criteria may be necessary to design larger retaining walls, bridge foundations and other structures for unusual load combinations (i.e. heavy snow loads, avalanche impact loads, traffic loads, etc.). Due to the variation in soil and rock condition observed, site specific design criteria may

be provided on a case by case basis. As project design continues we should evaluate future building, retaining wall and bridge sites to provide specific design criteria for individual loading conditions.

6. The proposed subdivision is located within and near steep slopes. Portions of the site are subject to potential hazards typical of steep mountain terrain, such as avalanches, rock fall and debris flows. A separate avalanche report with specific recommendations was prepared by others for this project. Rock fall is relatively uncommon and not a significant hazard. Structures in portions of Phase II can be designed using conventional construction to resist impacts from small stream flooding and debris flows based on their actual site location. Homes to be constructed within avalanche zones or areas of Phase II with potential for small stream flooding and debris flows should engage professional engineering assistance in design and construction of the foundation.

6 RECOMMENDATIONS

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

6.1 Grading

6.1.1 Clearing and Grubbing

Proposed fill and building areas should be cleared and grubbed of vegetation and other deleterious materials as described below.

1. Prior to site grading all existing fill, vegetation, organic topsoil and any debris should be stripped and disposed of outside the construction limits. Based on our subsurface exploration, we expect that approximately 12 inches may be used as an estimate for average depth of stripping within wooded areas. Stripping will be considerably less in rock areas. The actual depth of stripping will vary across the site. Organic surface soil should be stockpiled for future use in landscape area and is not suitable for use as structural fill or backfill.

2. Uncompacted soil or other unsuitable material in excavations resulting from tree removal or test pits, etc., should be over-excavated and dish-shaped to permit access for compaction equipment. Any 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.

3. Rocks greater than 8 inches (oversized rock) may be left in place in areas where fill will be deeper than 2 feet from finish grade, or removed and stockpiled for later use as RSP and landscaping. In areas where foundation structures or underground utility construction is deeper than 2 feet, oversized rock in fill should be deeper than the bottom of the proposed structures. Oversized rock should be placed so that nesting does not occur. Our personnel should be onsite to evaluate locations and methods for oversized rock placement as well as observation of rock fill placement.

6.1.2 Preparation for Fill Placement

Where fill placement is planned, near-surface soil exposed by clearing and grubbing should be prepared as described below.

1. The near-surface soil should be scarified to approximately 12 inches below existing ground surface or to competent material and then uniformly moisture conditioned to within approximately 2 percent of the ASTM D1557 optimum moisture content.

2. Areas to receive fill should be compacted with appropriate compaction equipment and proof rolled with a loaded, tandem-axle truck under observation of a representative of H&K. Areas that exhibit pumping or rutting should be over excavated and replaced with compacted fill placed according to the recommendations discussed below.

6.1.3 Fill Placement

Fill should be placed according to the following guidelines:

1. Material used for fill construction should consist of uncontaminated, predominantly granular, non-expansive native soil or approved import soil. Imported fill material should be predominantly granular, non-expansive, have a 8-inch maximum particle size and free of deleterious or organic material. Import material that is proposed for use onsite should be submitted to H&K for laboratory analysis at least 72 hours prior to import.

2. If site grading is performed during periods of wet weather or in the spring or early summer, 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 processing and drying to facilitate placement and compaction, particularly during or following the wet season or spring snow melt.

3. 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.

4. 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.

6.1.4 Cut/Fill Slope Grading

Stability of cut and fill slopes involves two separate aspects. The first concerns true slope stability related to mass wasting and landslides. True slope stability is dependent upon shear strength, unit weight, moisture content of soil and slope angle. The second aspect of slope stability involves erosion potential and is

dependent on numerous factors including grain size distribution, cohesion, moisture content, slope inclination, slope length and velocity of water or wind on the slope face. Steep constructed slopes reduce the amount of disturbed area and result in shorter slope lengths. Slope performance will be primarily affected by surface runoff and erosion. Permanent slope grading (cut/fill slopes) should be constructed according to the following recommendations.

1. Cut and fill slopes up to 20 feet in vertical extent consisting of soil may be constructed at inclinations of 1:1 (H:V) provided the slopes are surfaced with rock slope protection (RSP). Steep constructed slopes reduce the amount of disturbed area and result in shorter slope lengths. Slope performance at the site will primarily be affected by surface runoff, erosion and composition (soil or rock).

2. Cut slopes consisting of rock may excavated to grades up to 0.5:1, H:V. However, actual cut slope gradients and overall slope geometry should be evaluated on a case by case basis in the field by our firm.

3. Slopes steeper than 2:1, H:V, but no steeper than 1:1, H:V, should be protected with RSP in combination with vegetation. We recommend a minimum size of 12-inch diameter boulders for RSP on slopes up to 8 feet in vertical extent, 12- to 18-inch boulders for slopes up to 12 feet in vertical extent, and 18- to 24-inch boulders for slopes up to 20 feet in vertical extent. Slopes steeper than 1:1, H:V should be designed as stacked rockery walls. We can provide these recommendations on a case by case basis as necessary.

4. RSP should be placed in accordance with Caltrans Standard Specifications Method A Placement. Placing of rocks by dumping should not be permitted. Rocks should decrease in size from the bottom to the top of the slope. Rocks should have a specific gravity of at least 2.5 and a percentage of wear (per ASTM C535) of not more than 45.

5. 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.

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

6.1.5 Erosion Control

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

Erosion and sediment control measures can be categorized as temporary or permanent. Temporary measures are to be installed to provide short-term protection until the permanent measures are installed and effective. Temporary erosion and sediment control includes ground surface treatments and installation of sediment barriers and detention facilities. Sediment barriers and detention facilities are placed down gradient from all disturbed areas during construction and remain after construction until permanent measures become effective. 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.

These structures are designed to slow down water flow and intercept suspended sediment to prevent sediment discharge from the construction area while allowing runoff to continue down gradient. The requirement to install a sediment barrier is dependent on the following factors: 1) slope angle, 2) slope length, and 3) soil type (texture and coarse fragment content). 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 gently sloping swales. The slope length above each barrier should not exceed 100 feet. 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.

Berms, waterbars and ditches can be temporary or permanent and 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 stable, often well vegetated adjacent 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 flowline 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. Existing vegetation should be protected and undisturbed where possible. Revegetation should consist of native brush and grass species. All areas disturbed by construction should be re-vegetated. Previously stripped top soil should be replaced on disturbed areas where possible. Slope faces should be temporarily protected against erosion resulting from direct rain impact and melting snow 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.

6.1.6 Underground Utility Trenches

Underground utility trenches should be excavated and backfilled as described below.

1. We anticipate that the contractor will be able to excavate underground utility trenches using conventional earthmoving equipment for the majority of the planned Roads B and C, and Road A within Phase II. However, moderately strong bedrock and/or large boulders appear to underlie most of the site. The Takeuchi TB135 mini-excavator used in our field exploration encountered refusal at depths of 4 to 7½ feet below existing grade in all of our test pits. Based on conditions encountered in our trenches, we anticipate that a track mounted excavator equipped with a ripper or hydraulic hammer or other methods may be required for confined excavations.

2. The California Occupational Safety and Health Administration (OSHA) requires all utility trenches deeper than 5 feet bgs to be shored with bracing equipment or sloped back prior to entry.

3. We did not groundwater encounter in our test pit excavations. However depending on the time of year that excavation takes place, shallow subsurface seepage may be encountered in trench excavations, particularly if utility trenches are excavated below the rock surface. 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. In addition, to help prevent the migration of groundwater along utility trenches, cut-off walls should be considered for all trenches that exceed a 15 percent grade for a distance of 100 feet or more. Cut-off walls should consist of stacked bags of lean concrete with a subsurface drain extended to daylight on the upstream side.

4. Soil used as trench backfill should be granular and 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.

5. 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. The moisture content, density and relative compaction of fill should be evaluated by our firm at regular intervals during fill placement.

6. Trench backfill placed within 8 inches of subgrade in road and parking lot areas should be compacted to a minimum relative compaction of 95 percent of the maximum dry density per ASTM D1557.

6.1.7 Construction Dewatering

We did not encounter groundwater during our subsurface exploration. However, if grading is performed during or immediately following the wet season, seepage may be encountered in deeper excavations. If groundwater or saturated soil conditions are 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.

1. 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.

2. 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.

6.1.8 Surface Water Drainage

Proper surface water drainage is also important to the successful development of the project. We recommend the following measures to mitigate surface water drainage problems:

1. Slope final grade in structural areas so that surface water drains away from buildings at a minimum 2 percent slope for a minimum horizontal distance of 5 feet.

2. Compact and slope soil adjacent to building foundations so that water is not allowed to pond or infiltrate.

3. Direct roof downspouts to a closed collector pipe which discharges flow to positive drainage.

4. Paved areas should be sloped and drainage gradients maintained to carry all surface water to properly designed infiltration, detention basins or other storm water drainage structures.

5. Interior grades in crawl spaces beneath proposed structures should be higher than the exterior ground surface. The crawl space should be sloped to collect and divert water to drains that exit under or through the foundation (positive crawl space drainage). We recommend placing 3 to 4 inches of clean gravel over the crawl space grade.

6. Adequate ventilation should be provided in crawl space and/or basement areas to promote drying.

7. If open graded gravel or other highly permeable material is used for underground utilities, the trench should slope away from structures or the potential flow path should be plugged at the exterior of foundations.

6.1.9 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 Structural Improvement Design Criteria

Design criteria presented below assume that near-surface soil conditions at locations of proposed structures consist of dense, granular, native soil. Actual subsurface conditions at proposed structure locations may differ. We expect that these other conditions may be rock of varying quality. If subsurface conditions encountered during construction are different than dense native soil, we should be notified so that we can review and modify our recommendations as needed. Modification of our recommendations will require a site visit and possibly additional subsurface exploration and/or laboratory testing. Our firm can provide these services on a case by case basis upon request.

In addition, the following loads and design criteria consider typical geotechnical engineering loads only. Based on the geology and location of the site, additional loads may be required for design of structures. These loads may include, but are not limited to, avalanche impacts, snow creep, debris flows and small stream flooding. Homes to be constructed within avalanche zones or areas of Phase II with potential for small stream flooding and debris flows should engage professional engineering assistance in design and construction of the foundations.

6.2.1 Foundations

Our opinion is that shallow spread foundations are suitable for support of typical residential structures and medium sized earth retaining structures and bridges proposed for this project. Construction of special or larger structures may require deep foundations, rock bolting or other specialty techniques. The following paragraphs discuss foundation design parameters and construction recommendations.

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

2. Foundations constructed in non-expansive, previously undisturbed native soil or compacted fill may be designed using an allowable bearing capacity of 3000 pounds per square foot (psf) for dead plus live loads. An additional increase of 500 psf per foot of additional embedment (beyond the minimum 24 inches) may also be used, up to a limiting value of 4500 psf. Allowable bearing pressures may be increased by 33 percent for additional transient loading such as wind or seismic loads.

3. Resistance to lateral loads (including transient loads) may be provided by frictional resistance between the bottom of concrete foundations and the underlying soils, and by passive soil pressure against the sides of foundations. Lateral resistance derived from passive earth pressure can be modeled as a triangular pressure distribution ranging from 0 psf at the ground surface to a maximum of 350d psf, where d equals the depth of the foundation in feet. 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. A coefficient of friction of 0.40 may be used between poured-in-place concrete foundations and underlying native soil.

4. 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 footings designed and constructed in accordance with our recommendations will be on the order of ½-inch. Differential settlement between similarly loaded, adjacent footings is expected to be less than ¼-inch, provided footings are founded on similar materials (e.g., all on engineered fill, native soil, or rock). Differential settlement between adjacent footings founded on dissimilar materials (e.g., one footing on soil and an adjacent footing on rock) may approach the maximum anticipated total settlement. Settlement of foundations is expected to occur rapidly and should be essentially complete shortly after initial application of loads.

5. Potentially expansive or loose material remaining in footing excavations should be removed to expose firm, non-expansive material. Footing excavations should be moistened prior to placing concrete to reduce risk of problems caused by wicking moisture from curing concrete. H&K should

observe footing excavations prior to reinforcing steel and concrete placement.

6.2.2 Seismic Design Criteria

The site is located in Seismic Zone 3 of the 1997 Uniform Building Code (UBC) and the 2001 California Building Code (CBC) Seismic Zone Map. Therefore, structural improvements should be designed using a seismic zone factor of Z=0.30 (Table 16-I). Based on our observations of subsurface soil/rock conditions and our literature review, we recommend using the S_C Soil Profile Type (Table 16-J, 1997 UBC/2001 CBC) to evaluate seismic loads. Site specific evaluation may be appropriate for houses constructed on rock.

6.2.3 Slab-on-Grade Floor Systems

Concrete slab-on-grade floors may be used in conjunction with perimeter concrete footings. We make the following recommendations for the design and construction of slabs-on-grade:

1. Slabs-on-grade should be a minimum of 4 inches thick and uniform thickness. 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.

2. Slab subgrade soil should consist of non-expansive, suitable soil. Prior to constructing concrete slabs, the upper 8 inches of slab subgrade should be scarified, uniformly moisture conditioned to within 2% of optimum moisture content and compacted to at least 90% of the maximum dry density per ASTM D1557.

3. Slabs should be underlain by at least 4 inches of Class 2 aggregate base to provide uniform support. The aggregate base should be compacted to a minimum of 95% of the maximum dry density per ASTM D1557. The slab subgrade should be protected against drying until concrete placement. If groundwater is encountered in slab areas, subsurface drains should be constructed.

4. In slab-on-grade areas where moisture sensitive floor coverings are proposed, an vapor retarder membrane (e.g. 10 mil thick polyethylene) should be placed over the base course to reduce the migration of moisture vapor through the concrete slab. A minimum of 2 inches of clean, moist sand should be spread over the vapor retarder. The sand will help protect the membrane and aid in curing the concrete. Prior to concrete placement, the sand should be moistened (not wet) and tamped to reduce moisture withdrawal from concrete during curing. Exposed concrete slabs should be moisture cured for at least seven days after placement.

5. Exterior slabs-on-grade such as sidewalks may be placed directly on compacted fill or scarified and recompacted, non-expansive native soil without the use of an aggregate base section. Potentially expansive, 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.

6. 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.

6.2.4 Retaining Wall Discussion and Design Criteria

To help reduce the amount of disturbed area and constructed slopes, relatively high retaining walls and steep constructed slopes are anticipated for the proposed project. Conventional cast-in-place reinforced concrete walls may be constructed with textured and colored concrete for improved appearance. However, numerous other options for retaining structures are possible. Retaining structures for fill slopes will likely consist of mechanically stabilized embankments (MSE) such as Hilficker™ walls or geogrid reinforced embankments with rock slope protection (RSP). MSE generally consist of reinforcing layers of metal wire or high density polyethylene placed in between compacted layers of the embankment with a slope facing on the outside of the embankment. Crib walls and rockery walls are also possible options for the fill slopes at the project site.

For soil or loose rock cut slopes at the project site greater than 15 or 20 feet in height, tie-back anchored retaining structures are possible. A tie-back wall generally consists of reinforcing tendons drilled and grouted into the cut slope and tensioned to support the retained mass. Within stable rock areas the lateral earth pressure should be relatively low, reducing the need for massive, heavily reinforced concrete structures. These slopes are commonly faced with shotcrete, thin cast-in-place concrete walls or pre-cast concrete panels. Installation of tie-back anchors is typically performed by a specialty contractor experienced in these construction techniques.

We expect that many of the proposed cut slopes for the site will be excavated in relatively strong rock. Consequently, it may be feasible to excavate some permanent slopes at very steep angles. These rock slopes would likely be prone to raveling, erosion and rock fall. A short retaining wall at the bottom of steep slopes may be necessary to catch rock fall. Another option would be a combination of short walls at the bottom of steep slopes covered with RSP.

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 the unit weights presented in the following table.

Table 6.2.4.1 - 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)	35	50
Passive Pressure (pcf)	350	350
At-Rest Pressure (pcf)	50	65
Coefficient of Friction	0.45	0.45

* 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 5.2.4.1 assume that retaining walls are constructed in native soil or compacted fill and will not exceed approximately 10 feet in height. For retaining walls greater than 10 feet and/or with surcharge loads (e.g., footings, vehicles), we anticipate that site specific analysis for individual structures will be necessary. We can provide retaining wall and rockery wall design criteria for specific loading and backfill configurations, if requested. Additional recommendations for design and construction of retaining walls are listed below:

1. Heavy compaction equipment should not be used directly adjacent to retaining walls unless the wall is designed or braced to resist the additional lateral forces.

2. If surface loads are closer to the top of the retaining wall than one-half of its height, H&K should review the loads and loading configuration. We should also review details and plans for any proposed wall over 10 feet in height.

3. Retaining wall design criteria presented in Table 5.2.4.1 assume that retaining walls are well drained reduce hydrostatic pressures. A drainage blanket should be installed to reduce additional lateral forces and reduce saturation of backfill soil. Drainage blankets may consist of graded rock drains or geosynthetic blankets.

4. Rock drains should consist of a minimum 12 inches of open-graded crushed rock, and placed directly behind the wall, wrapped in non-woven geotextile filter fabric such as Amoco 4545™ or equivalent. Drains should have a minimum 4-inch diameter, perforated, schedule 40 PVC pipe placed at the base of the wall, inside the drain rock, with perforations placed down. The PVC pipe should be sloped so that water is directed away from the wall by gravity. A geosynthetic drainage blanket such as Enkadrain™ or equivalent may be substituted for the rock drain, provided that water is channeled away from the wall. If a geosynthetic blanket is used, backfill must be compacted carefully so that equipment or soil does not tear or crush the drainage blanket.

5. Additional lateral loading on retaining structures due to seismic accelerations may be considered at the designer's option. For an earthquake producing a

design horizontal acceleration of 0.3g, we recommend that the resulting additional lateral force applied to unrestrained (cantilevered) retaining structures onsite be estimated as $P_{ae} = 14H^2$ pounds, where H is the height of the wall in feet. The additional seismic force is assumed to be applied at a height of 0.6H above the base of the wall.

6. Retaining wall backfill should conform to recommendations in section 6.1.3, Fill Placement.

6.2.5 Pavement Design

Site soil should provide suitable support for the driveway and roadway asphalt concrete (AC) pavement. The following table provides our pavement design recommendations for the traffic indices noted.

6.2.5.1 Recommended Pavement Sections	
Traffic Index: 5 Traffic Description: residential traffic	Pavement Section (inches)
Caltrans Section 26, Standard Specifications, Asphalt Concrete	3.0
Caltrans Section 26, Class 2 Aggregate Base compacted to at least 95%	8.0
Traffic Index: 6 Traffic Description: moderate truck traffic	Pavement Section (inches)
Caltrans Section 26, Standard Specifications, Asphalt Concrete	3.0
Caltrans Section 26, Class 2 Aggregate Base compacted to at least 95%	9.0

The upper 6 inches of native soil should be compacted to at least of 95 percent of the maximum dry density per ASTM D1557. Aggregate baserock should also be compacted to a minimum of 95 percent. Subgrade and AB dry density should be

evaluated by H&K. In addition to field density tests, subgrade should be proof rolled under the observation of H&K prior to baserock placement.

7 LIMITATIONS

Our professional services were performed consistent with the generally accepted geotechnical engineering principles and practices employed in the site area at the time the report was prepared. 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.

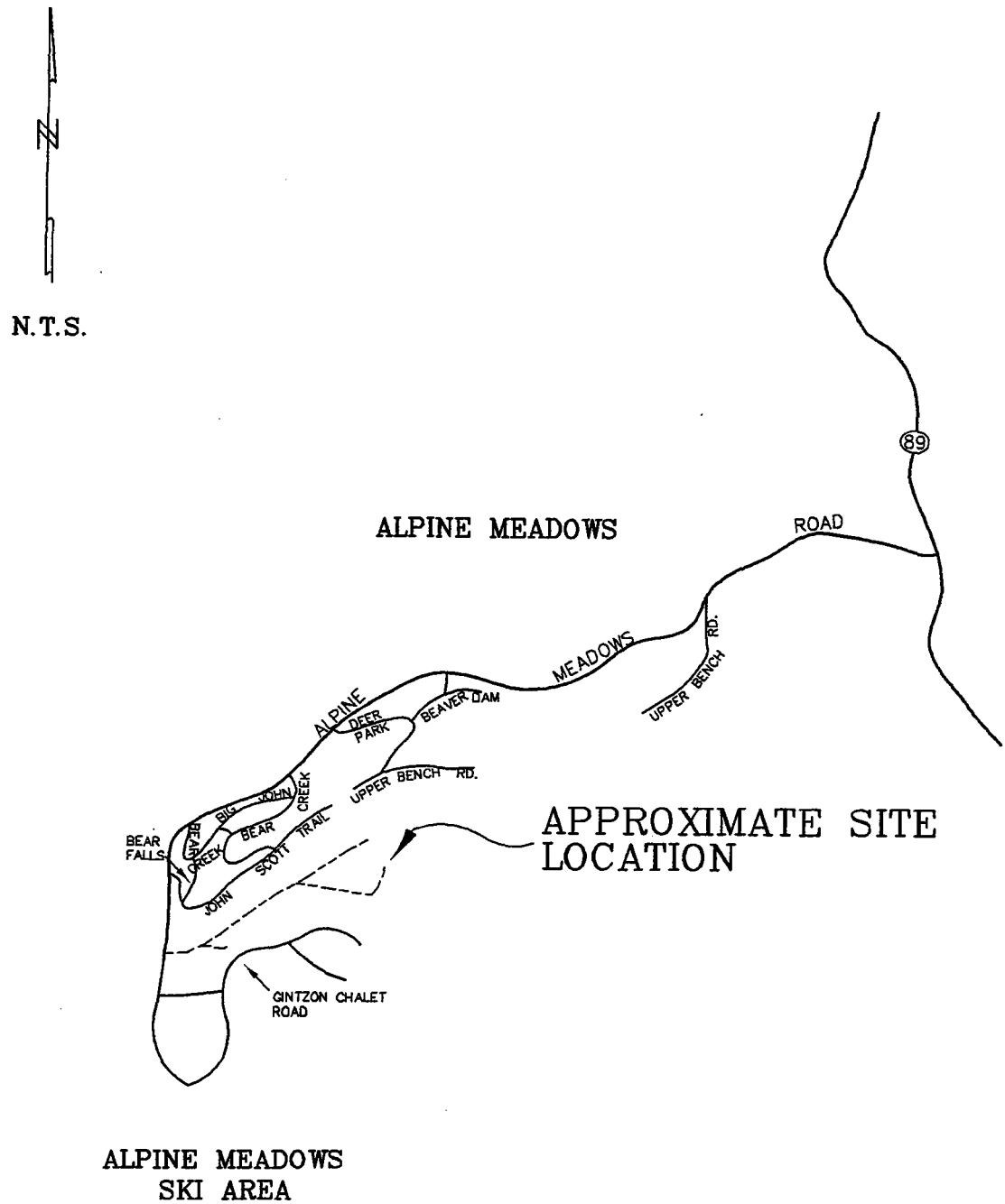
If changes are made to the nature or design of the project as described in this report, then our conclusions and recommendations presented in this report should be reviewed by Holdrege & Kull to review our conclusions and recommendations. Additional field work and laboratory tests may be required to 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 have assumed that subsurface soil conditions encountered at the location of our test pits are generally representative of subsurface conditions across the project site. Actual subsurface conditions at locations between and beyond our 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.

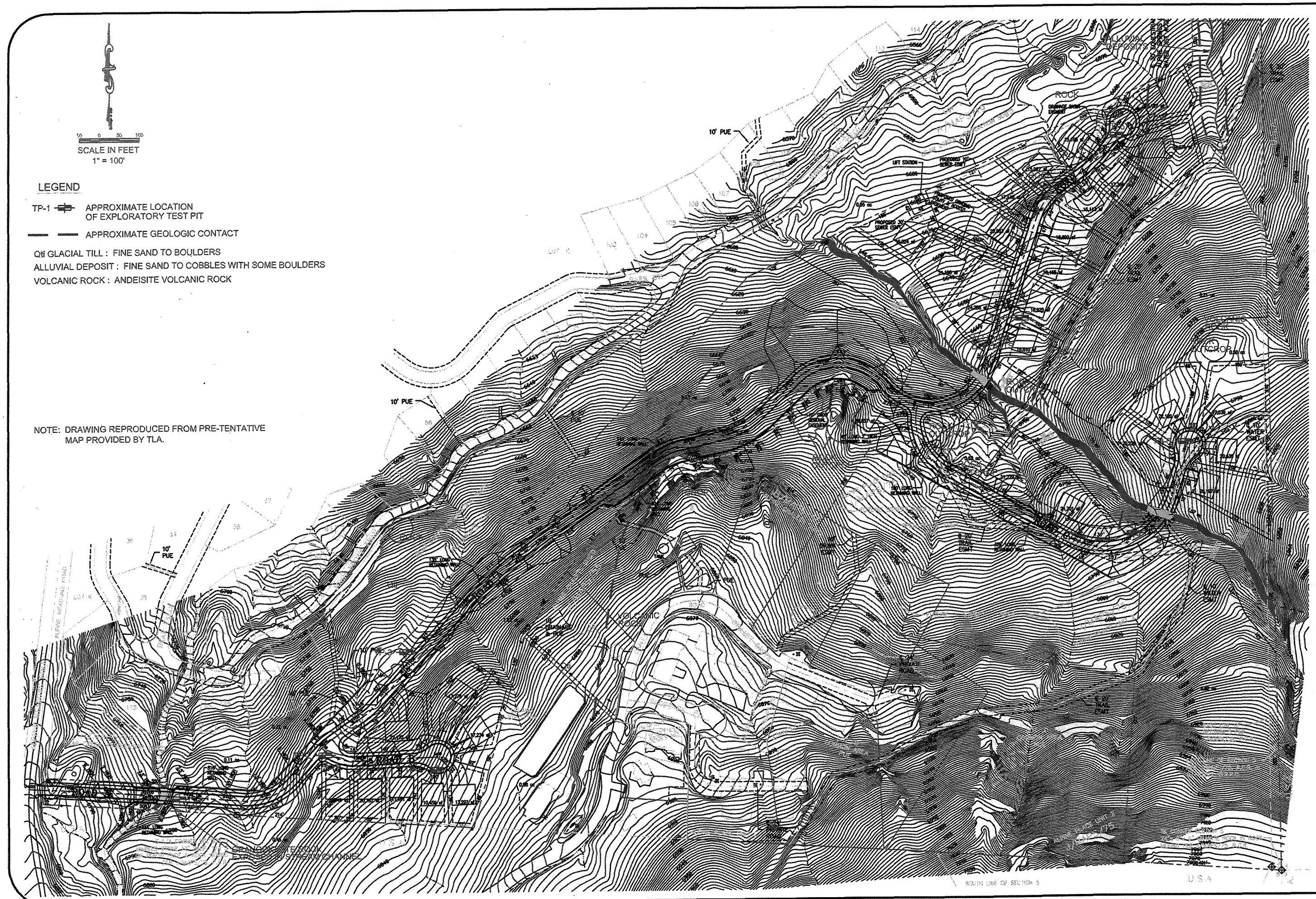
The elevation or depth to groundwater and soil moisture conditions underlying the project site may differ with time and location. The Test Pit Location Plan and Surficial Geologic Map shows approximate test pit locations as determined by pacing distances from identifiable site features. Therefore, test 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 be careful and 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 due to natural processes or works of man, at the site or adjacent properties. 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.



HOLDREGE & KULL *CONSULTING ENGINEERS • GEOLOGISTS* *15826 DONNER PASS ROAD* *TRUCKEE, CA 96160* *(530) 587-5156*	SITE VICINITY MAP ALPINE SIERRA ALPINE MEADOWS, CA	PROJECT NO.: 40376-01 DATE: 11/20/03 FIGURE NO.: 1


SCALE IN FEET
1" = 100'
LEGEND
TP-1 APPROXIMATE LOCATION OF EXPLORATORY TEST PIT
APPROXIMATE GEOLOGIC CONTACT
Qtl GLACIAL TILL : FINE SAND TO BOULDERS
ALLUVIAL DEPOSIT : FINE SAND TO COBBLES WITH SOME BOULDERS
VOLCANIC ROCK : ANDEISITE VOLCANIC ROCK
NOTE: DRAWING REPRODUCED FROM PRE-TENTATIVE MAP PROVIDED BY TLA.
10' PUE
LIFT STATION
VOLCANIC ROCK
ROCK
USA
DESIGNED BY: GNP
DRAWN BY: EBA
DATE: DECEMBER 2003
DRAWING NAME: FIG. 2
PROJECT No.: 40376-01
DATE
REVISIONS
NO.
TEST PIT LOCATION PLAN AND
SURFICIAL GEOLOGIC MAP FOR ALPINE SIERRA
ALPINE MEADOWS / PLACER COUNTY, CALIFORNIA
HOLDREGE & KULL
CONSULTING ENGINEERS • GEOLOGISTS
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HK1

HOLDREGE & KULL
CONSULTING ENGINEERS • GEOLOGISTS

October 23, 2003
Project No. 40376-01

Alpine Sierra Partners, LLC
1015 NW 11th Avenue, Suite 243
Portland, OR 97209

Attention: Chris Nelson

Reference: ***Proposed Alpine Sierra Subdivision***
Placer County, California

Subject: ***Proposal for Geotechnical Engineering Services***

We are pleased to provide this proposal for geotechnical engineering services for the proposed Alpine Sierra Subdivision in Alpine Meadows/Placer County, California. To prepare this proposal, we discussed the project with yourself and reviewed a Pre-Tentative Map prepared by Terrance E. Lowell & Associates, dated June 2003.

PROJECT DESCRIPTION

Currently, we understand that the proposed project will involve subdividing approximately 45 acres of previously undeveloped land into about 31 residential lots. We anticipate that future construction will consist of wood-framed structures with concrete slab-on-grade and/or raised-wood floors. Appurtenant construction will include asphalt concrete paved roads, earth retaining structures up to 35 feet high and underground utilities.

SCOPE OF SERVICES

Based on our understanding of the project, we propose to provide design level geotechnical engineering recommendations for project design and construction. Prior to our field exploration, we will review existing geologic and soil references to develop anticipated site conditions. Our engineering geologist will prepare a surface geology map to help evaluate the geotechnical engineering conditions at the site.

Field Investigation

Our field investigation will involve the excavation of 8 to 10 exploratory trenches up to about 12 feet deep. Trench locations will be selected based on anticipated project and layout and site access. Exploratory trenches will be excavated using a rubber tired or track mounted backhoe. During our field investigation we will obtain bulk samples of the soil/rock encountered for laboratory testing. Trenches will be logged in the field by an engineer or geologist from our firm.

Laboratory Testing

Laboratory tests will include moisture/density determination, direct shear test, Atterberg Limits determinations, sieve analyses, and R-value testing. Direct shear test results will be used to calculate foundation and retaining wall design criteria for the soil encountered at the site. R-value test results will be used in asphalt pavement design. Sieve analyses and Atterberg Limits testing will provide information for soil classification and engineering properties of select soil samples.

Engineering

Using the results of our laboratory tests, we will develop geotechnical engineering recommendations to provide foundation and retaining wall design criteria and recommended pavement sections.

Geotechnical Report

Following completion of the above tasks, we will compile a report which will include:

- Logs of exploratory trenches;
- Site plan showing approximate locations of exploratory trenches and a reconnaissance level geologic map;

- Description of the soil and rock conditions encountered at the site with emphasis on how the conditions should affect project design and construction;

- Recommendations for earthwork construction, including site preparation recommendations, discussion of reuse of on site soil as structural or non-structural fill, and a discussion of remedial earthwork recommendations, if warranted;

- Recommendations for temporary excavations and trench backfill including permanent and temporary near-surface dewatering recommendations;

- Recommendations for permanent cut and fill slopes;

- Design criteria for foundations, including depth of embedment, allowable bearing capacity for the recommended foundation system(s) and UBC Soil Profile Type to be used for seismic design;

- Lateral earth pressures and drainage recommendations for retaining wall design and a discussion of possible types of retaining structures;

- Recommended asphalt concrete pavement sections; and

- Slab-on-grade concrete subgrade preparation recommendations.

FEES

Our lump sum fee to provide the scope of services described above would be based on the enclosed 2003 fee schedule. This fee includes the cost of providing a backhoe for our field investigation. If this proposal meets your approval, please sign the enclosed terms and conditions and return one copy as our authorization to proceed.

TIMING

We can begin our field investigation within approximately one week of notice to proceed depending on weather and site moisture conditions. If requested, we will be able to provide verbal design criteria within one week following completion of our field investigation. The final report will be submitted within approximately three to four weeks following completion of our field investigation.

We appreciate the opportunity to submit this proposal and look forward to working with you on this project. If you have any questions or need additional information, please do not hesitate to contact us.

Sincerely,
HOLDREGE & KULL

Jake Hudson/for

Greg Porter, P.E.
Project Engineer

Jake Hudson

John K. Hudson, P.E., C.E.G.
Senior Engineer

encl: Terms and Conditions
2003 Fee Schedule

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.

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 such risks, 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.*

Rely on Your Geotechnical Engineer for Additional Assistance

Membership in ASFE 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 your ASFE-member geotechnical engineer for more information.

ASFE PROFESSIONAL FIRMS PRACTICING IN THE GEOSCIENCES

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TEST PIT 1

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE
40376-01	ALPINE SIERRA	6838 FEET	10/28/03	1

EXCAVATING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
TAKEUCHI TB 135 MINI-EX. (12" & 18" BUCKET)	GRAB/BULK	N/A	N/A

SAMPLE NO.	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
			1	SM	BROWN, SLIGHTLY DAMP, LOOSE TO MEDIUM DENSE SILTY SAND (FINE TO COARSE), WITH FINE TO COARSE GRAVEL; CONTAINS ABUNDANT FINE TO COARSE ROOTS & ORGANICS, COBBLES TO 4" (ESTIMATED 5% +3")
			2	SM	BROWN, MOIST, MEDIUM DENSE TO DENSE SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL, CONTAINS COBBLES AND BOULDERS TO 6" (ESTIMATED 5 % +3")
1-1	- -	- -	3		
			4	SM	OANGE-BROWN, MOIST, DENSE TO VERY DENSE SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL; CONTAINS COBBLES AND BOULDERS TO 12" (ESTIMATED 5 % +3")
			5		
			6		BUCKET SCRAPING @ 3.0'; 12" BOULDER OBSERVED
1-2	- -	- -	7		
			8		EXCAVATION REFUSAL @ 7.0'; BOULDERS IN BOTTOM OF EXCAVATION
			9		
			10		
			11		
			12		
			13		
			14		
			15		
			16		
			17		
			18		
			19		
			20		

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TEST PIT 2

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE
40376-01	ALPINE SIERRA	6797 FEET	10/28/03	1

EXCAVATING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
TAKEUCHI TB 135 MINI-EX. (12" & 18" BUCKET)	GRAB/BULK	N/A	N/A

SAMPLE NO.	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
			1	SM	BROWN, SLIGHTLY MOIST, LOOSE SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL; CONTAINS ABUNDANT FINE TO COARSE ROOTS AND COBBLES TO 6" (ESTIMATED 5 % + 3")
			2	SM	LIGHT BROWN, MOIST, MEDIUM DENSE TO DENSE SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL; CONTAINS COBBLES AND BOULDERS TO 12"
2-1	--	--	3		OBSERVED 24" BOULDER @ 1.5'
			4		
			5		
2-2	--	--	6		DIFFICULT EXCAVATION
			7		
			8		EXCAVATION REFUSAL @ 7.5'
			9		
			10		
			11		
			12		
			13		
			14		
			15		
			16		
			17		
			18		
			19		
			20		

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TEST PIT 3

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE
40376-01	ALPINE SIERRA	6775 FEET	10/28/03	1

EXCAVATING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
TAKEUCHI TB 135 MINI-EX. (12" & 18" BUCKET)	GRAB/BULK	N/A	N/A

SAMPLE NO.	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
			1	SM	BROWN, SLIGHTLY DAMP, LOOSE SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL; CONTAINS ABUNDANT FINE TO COARSE ROOTS AND ORGANICS
			2	GP-GM	BROWN, MOIST POORLY GRADED FINE TO COARSE GRAVEL WITH FINE TO COARSE SAND (WEATHERED ROCK FRAGMENTS)
3-1	--	--	3	RX	ANDESITE ROCK; GRAY, CLOSELY FRACTURED, HIGHLY TO MODERATELY WEATHERED, MODERATELY STRONG
			4		EXCAVATION REFUSAL @ 4.0'
			5		
			6		
			7		
			8		
			9		
			10		
			11		
			12		
			13		
			14		
			15		
			16		
			17		
			18		
			19		
			20		

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TEST PIT 4

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE
40376-01	ALPINE SIERRA	6773 FEET	10/28/03	1

EXCAVATING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
TAKEUCHI TB 135 MINI-EX. (12" & 18" BUCKET)	GRAB/BULK	N/A	N/A

SAMPLE NO.	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
			1	SM	BROWN, SLIGHTLY DAMP, LOOSE SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL; CONTAINS ROOTS, ORGANICS AND COBBLES AND BOULDERS TO 24"
4-1	--	--	2	SM	BROWN, SLIGHTLY MOIST, LOOSE TO MEDIUM DENSE SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL; CONTAINS COBBLES AND BOULDERS TO 24" (ESTIMATED 20 % +3")
			3		BUCKET SCRAPING @ 2.5'; SWITCH TO 12" BUCKET
			4		48" BOULDER EXCAVATION @ 3.5'
					EXCAVATION REFUSAL @ 4.0'
			5		
			6		
			7		
			8		
			9		
			10		
			11		
			12		
			13		
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TEST PIT 5

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE
40376-01	ALPINE SIERRA	6616 FEET	10/28/03	1

EXCAVATING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
TAKEUCHI TB 135 MINI-EX. (12" & 18" BUCKET)	GRAB/BULK	N/A	N/A

SAMPLE NO.	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
			1	SM	BROWN, DRY, LOOSE SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL; CONTAINS ABUNDANT FINE TO COARSE ROOTS AND ORGANICS, COBBLES AND BOULDERS TO 18" (ESITMATED 15 % + 3")
			2	SM	BROWN, MOIST, MEDIUM DENSE SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL, CONTAINS COBBLES AND BOULDERS TO 18"
5-1	--	--	3		DIFFICULT EXCAVATION; SWITCHED BUCKETS
			4	SM	ORANGE BROWN MOIST, DENSE TO VERY DENSE, SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL, COBBLES AND BOULDERS TO 18"
			5		
			6		
5-2	--	--		RX	ANDESITE; GREY AND WHITE, HIGHLY TO MODERATELY WEATHERED, CLOSELY TO MODERATLY FRACTURED, WEAK TO MODERATELY STRONG
			7		
			8		EXCAVATION REFUSAL @ 7.5'
			9		
			10		SWITCHED TO 12" BUCKET @ 6.0'
			11		
			12		
			13		
			14		
			15		
			16		
			17		
			18		
			19		
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TEST PIT 6

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE
40376-01	ALPINE SIERRA	6784 FEET	10/28/03	1

EXCAVATING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
TAKEUCHI TB 135 MINI-EX. (12" & 18" BUCKET)	GRAB/BULK	N/A	N/A

SAMPLE NO.	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
			1	SM	BROWN, DRY, LOOSE, SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL, CONTAINS FINE TO MEDIUM ROOTS
6-1	--	--	2	SM	BROWN, MOIST, MEDIUM DENSE SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL COBBLES TO 4" (5-10 % + 3")
			3		
			4	RX	ANDESITE; GRAY BROWN, COMPLETLY TO HIGHLY WEATHERED CLOSELY FRACTURED, WEAK TO MODERATLY STRONG
			5		SWITCHED TO 12" BUCKET @ 4.0', DIFFICULT EXCAVATION
			6		EXCAVATION REFUSAL @ 6.0'
			7		
			8		
			9		
			10		
			11		
			12		
			13		
			14		
			15		
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TEST PIT 7

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE
40376-01	ALPINE SIERRA	6783 FEET	10/28/03	1

EXCAVATING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
TAKEUCHI TB 135 MINI-EX. (12" & 18" BUCKET)	GRAB/BULK	N/A	N/A

SAMPLE NO.	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
			1	SM	BROWN, DRY, LOOSE SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL; COBBLES AND BOULDERS TO 18"; FINE TO MEDIUM ROOT AND ABUNDANT ORGANICS
			2	SM	BROWN MOIST MEDIUM DENSE SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL; COBBLE AND BOULDERS TO 2'
			3	SM	GRADES TO ORANGE BROWN @ 3.0'
			4		BUCKET SCRAPING, DIFFICULT EXCAVATION @ 4.0'; SWITCHED TO 12" BUCKET @ 4.0'
			5		
			6		EXCAVATION REFUSAL @ 6.0'
			7		
			8		
			9		
			10		
			11		
			12		
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TEST PIT 8

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE
40376-01	ALPINE SIERRA	6666 FEET	10/28/03	1

EXCAVATING METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
TAKEUCHI TB 135 MINI-EX. (12" & 18" BUCKET)	GRAB/BULK	N/A	N/A

SAMPLE NO.	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
			1	SM	BROWN, DRY, LOOSE SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL; CONTAINS COBBLES AND BOULDERS TO 12" AND FINE TO MEDIUM ROOT AND ABUNDANT ORGANICS
			2	SM	LIGHT BROWN, MOIST, MEDIUM DENSE SILTY FINE TO COARSE SAND WITH FINE TO COARSE GRAVEL; CONTAINS COBBLES AND BOULDERS TO ESTIMATED 12" (ESTIMATED 10 % +3")
8-1	--	--	3		
			4		BUCKET SCRAPING @ 4.0'; SWITCHED TO 12" BUCKET
			5		
			6		EXCAVATION REFUSAL @ 6.0'
			7		
			8		
			9		
			10		
			11		
			12		
			13		
			14		
			15		
			16		
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			19		
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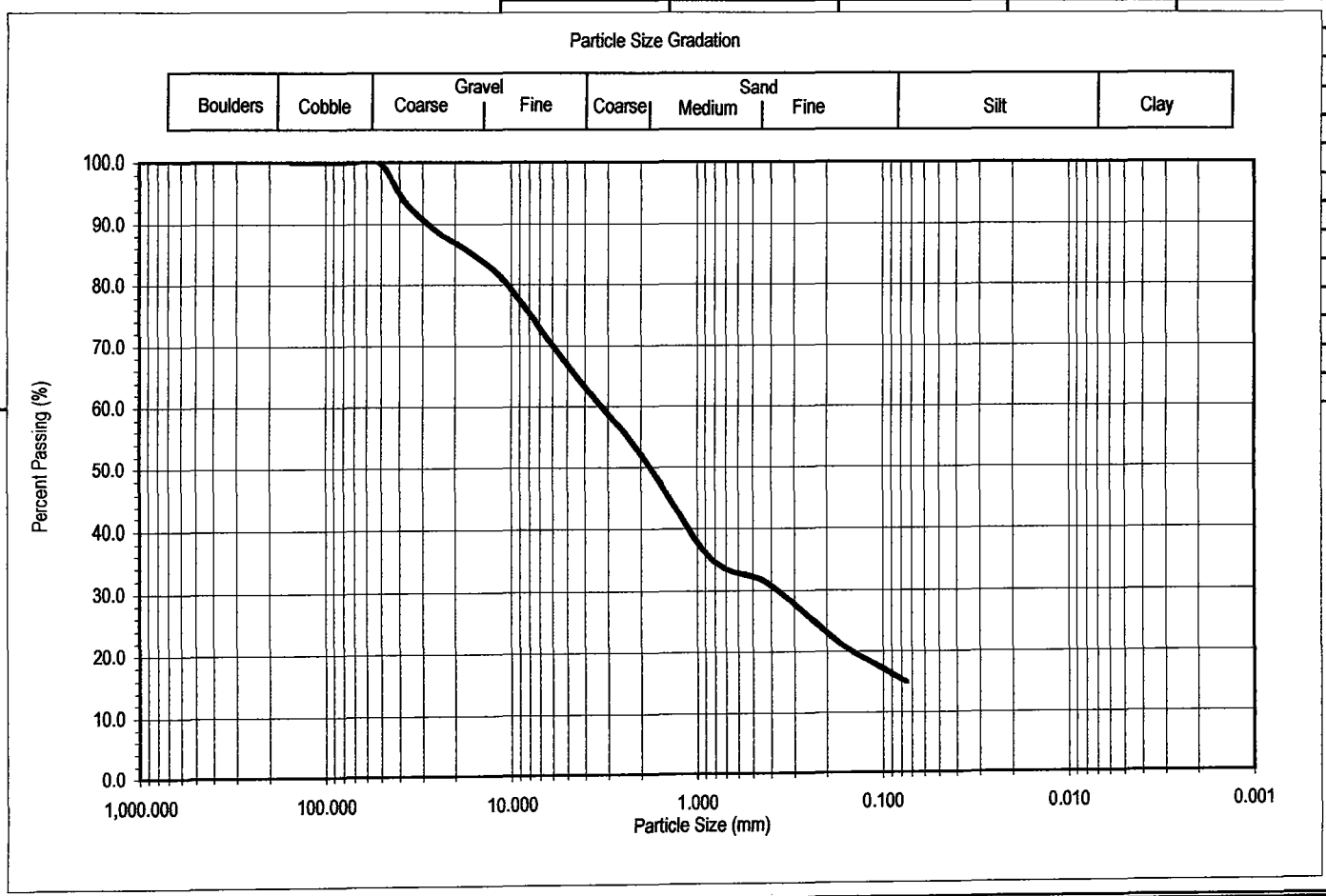
Particle Size Distribution

ASTM D422

Project No.:	40376-01	Project Name:	Alpine Sierra		Date: 10/31/2003
Sample No.:	1-1	Boring/Trench:	T-1	Depth, (ft.): 3	Tested By: JCS
Description:	Yellow Brown Silty Sand with Gravel (SM)				Checked By: GNP
Sample Location:	See Test Pit Location Plan, Figure 2				Lab. No.: 3-754

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	4,550.3	**100.0**
3 Inch	3.0000	76.2	0.00	0.0	4,550.3	**100.0**
2 Inch	2.0000	50.8	0.00	0.0	4,550.3	**100.0**
1.5 Inch	1.5000	38.1	279.60	279.6	4,270.7	**93.9**
1.0 Inch	1.0000	25.4	226.54	506.1	4,044.2	**88.9**
3/4 Inch	0.7500	19.1	111.70	617.8	3,932.5	**86.4**
1/2 Inch	0.5000	12.7	166.94	784.8	3,765.6	**82.8**
3/8 Inch	0.3750	9.5	190.72	975.5	3,574.8	**78.6**
#4	0.1870	4.7500	567.50	1,543.0	3,007.3	**66.1**
#10	0.0787	2.0000	633.99	2,177.0	2,373.3	**52.2**
#20	0.0335	0.8500	771.87	2,948.9	1,601.5	**35.2**
#40	0.0167	0.4250	175.95	3,124.8	1,425.5	**31.3**
#60	0.0098	0.2500	263.06	3,387.9	1,162.5	**25.5**
#100	0.0059	0.1500	243.33	3,631.2	919.1	**20.2**
#200	0.0030	0.0750	233.64	3,864.8	685.5	**15.1**



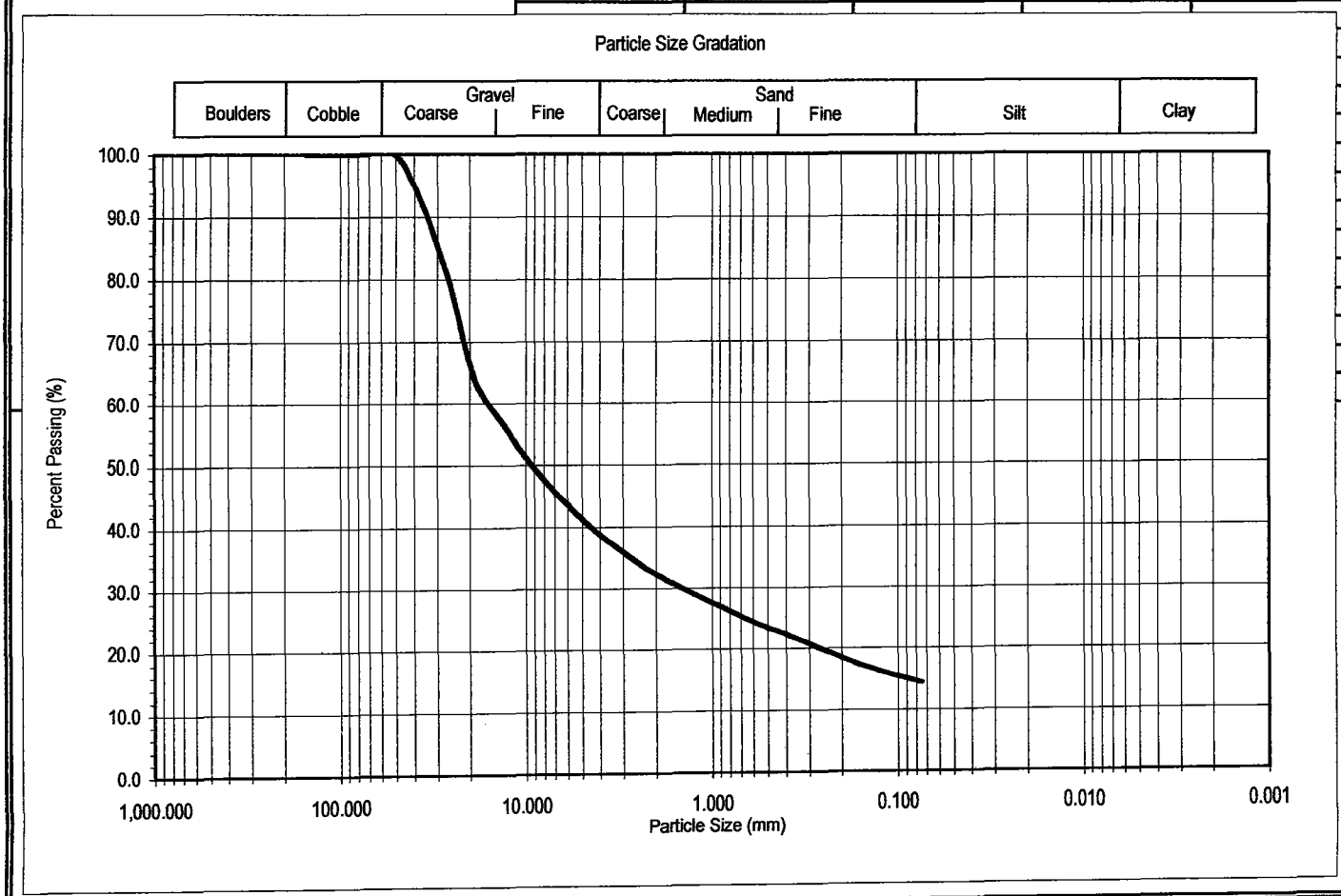
HOLDREGE & KULL

Particle Size Distribution

ASTM D422

Project No.:	40376-01	Project Name:	Alpine Sierra		Date: 10/31/2003
Sample No.:	3-1	Boring/Trench:	T-3	Depth, (ft.): 2	Tested By: JCS
Description:	Dark Brown Silty Gravel with Sand (SM)				Checked By: GNP
Sample Location:	See Test Pit Location Plan, Figure 2				Lab. No.: 3-754

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	3,842.1	**100.0**
3 Inch	3.0000	76.2	0.00	0.0	3,842.1	**100.0**
2 Inch	2.0000	50.8	0.00	0.0	3,842.1	**100.0**
1.5 Inch	1.5000	38.1	242.10	242.1	3,600.0	**93.7**
1.0 Inch	1.0000	25.4	571.60	813.7	3,028.4	**78.8**
3/4 Inch	0.7500	19.1	566.58	1,380.3	2,461.8	**64.1**
1/2 Inch	0.5000	12.7	312.90	1,693.2	2,148.9	**55.9**
3/8 Inch	0.3750	9.5	215.75	1,908.9	1,933.1	**50.3**
#4	0.1870	4.7500	369.17	2,278.1	1,564.0	**40.7**
#10	0.0787	2.0000	319.04	2,597.1	1,244.9	**32.4**
#30	0.0236	0.6000	304.02	2,901.2	940.9	**24.5**
#40	0.0167	0.4250	69.09	2,970.3	871.8	**22.7**
#60	0.0098	0.2500	113.70	3,083.9	758.1	**19.7**
#100	0.0059	0.1500	104.85	3,188.8	653.3	**17.0**
#200	0.0030	0.0750	101.76	3,290.6	551.5	**14.4**



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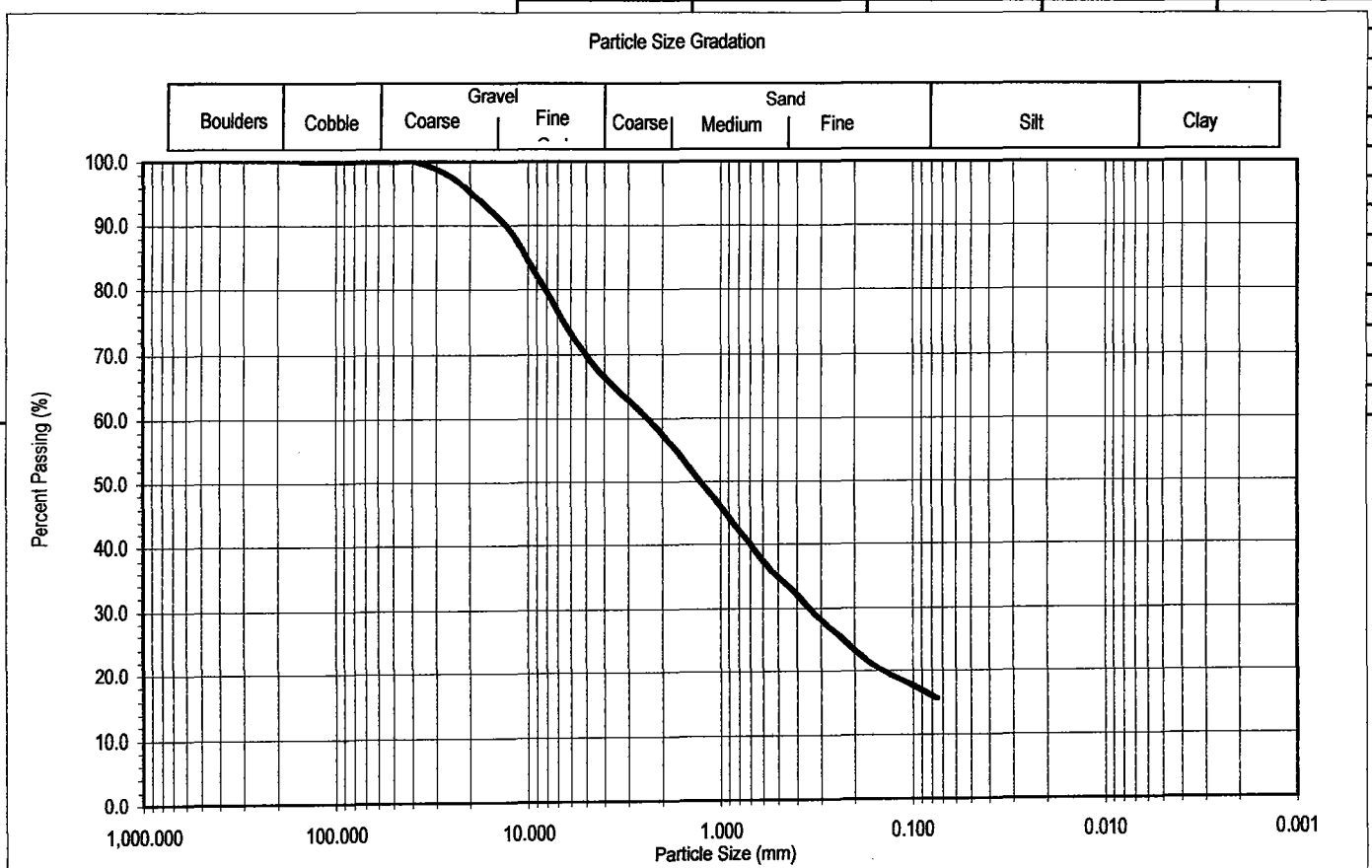
Particle Size Distribution

ASTM D422

Project No.:	40376-01	Project Name:	Alpine Sierra		Date: 10/31/2003
Sample No.:	5-2	Boring/Trench:	T-5	Depth, (ft.): 6	Tested By: JCS
Description:	Brown Silty Sand with Gravel (SM)				Checked By: GNP
Sample Location:	See Test Pit Location Plan, Figure 2				Lab. No.: 3-754

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	3,581.2	**100.0**
3 Inch	3.0000	76.2	0.00	0.0	3,581.2	**100.0**
2 Inch	2.0000	50.8	0.00	0.0	3,581.2	**100.0**
1.5 Inch	1.5000	38.1	0.00	0.0	3,581.2	**100.0**
1.0 Inch	1.0000	25.4	76.22	76.2	3,505.0	**97.9**
3/4 Inch	0.7500	19.1	117.97	194.2	3,387.0	**94.6**
1/2 Inch	0.5000	12.7	178.70	372.9	3,208.3	**89.6**
3/8 Inch	0.3750	9.5	217.47	590.4	2,990.8	**83.5**
#4	0.1870	4.7500	515.24	1,105.6	2,475.6	**69.1**
#10	0.0787	2.0000	420.18	1,525.8	2,055.4	**57.4**
#30	0.0236	0.6000	713.78	2,239.6	1,341.6	**37.5**
#40	0.0167	0.4250	166.87	2,406.4	1,174.7	**32.8**
#60	0.0098	0.2500	249.66	2,656.1	925.1	**25.8**
#100	0.0059	0.1500	196.16	2,852.3	728.9	**20.4**
#200	0.0030	0.0750	169.69	3,021.9	559.2	**15.6**



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