GEOTECHNICAL ENGINEERING REPORT
for
AMAZING FACTS PROPERTY
Sierra College Boulevard and Nightwatch Drive
APNs 046-050-006 and -008
Placer County, California

Prepared for:
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Project No. 3307A-02
April 15, 2009

HOLDREGE & KULL
CONSULTING ENGINEERS • GEOLOGISTS

Project No. 3307A-02
April 15, 2009

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Vancouver, Washington 98685

Reference: ***Amazing Facts Property***
Sierra College Boulevard and Nightwatch Drive
APNs 046-050-006 and 046-050-008
Placer County, California

Subject: ***Geotechnical Engineering Report***

Dear Mr. Shearer:

This report presents the results of our geotechnical engineering investigation for the Amazing Facts property on Sierra College Boulevard in Placer County, California. Based on our conversation with Russ King of King Engineering, we understand that a church facility and associated driveway, parking areas and underground utilities are proposed for the northern portion of the approximate 75-acre property.

The findings presented in this report are based on our subsurface investigation, laboratory test results, and our experience with subsurface conditions in the area. Our opinion is that the project can be completed as proposed, provided the recommendations presented in this report are implemented. From a geotechnical engineering standpoint, our primary concern includes resistant rock at shallow depths. Recommendations for rock excavation are presented in the report.

Please contact us if you have any questions regarding our observations or the recommendations presented in this report.

Sincerely,

HOLDREGE & KULL

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Appendix C Exploratory Trench Logs
Appendix D Laboratory Test Data

1 INTRODUCTION

At the request of Rod Shearer of Shearer & Associates, Inc., Holdrege & Kull (H&K) performed a geotechnical investigation at the Amazing Facts property designated as Placer County Assessor's Parcel Numbers (APNs) 046-050-006 and 046-050-008, near the intersection of Sierra College Boulevard and Nightwatch Drive, in Placer County, California. The geotechnical investigation was performed in general accordance with our February 17, 2009 proposal for the project, a copy of which 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,* which summarizes the general limitations, responsibilities, and use of geotechnical reports.

1.1 SITE DESCRIPTION

The approximate 75-acre property is located southeast of the intersection of Sierra College Boulevard and Nightwatch Drive in Placer County, California. The property is bordered by rural residential property to the south and west, by a residential subdivision to the north and by Placer County Water Agency property to the east.

At the time of our field investigation, the project site was undeveloped except for partial clearing, dirt access roads, and a previously graded pond area in the southern portion of the site. Site topography varied from relatively flat in the northernmost portion near the area of proposed improvements, to steeply sloping in the eastern and southern portions of the site.

1.2 PROPOSED IMPROVEMENTS

Based on our review of an April 2007 site plan provided by King Engineering, we understand that the proposed project will likely include the construction of a multi structure church facility with paved parking areas and driveways, landscaping and underground utilities. The improvements will generally be located in the northern portion of the site adjacent to Sierra College Boulevard on south facing slopes.

Based on discussions with representatives of Shearer & Associates, we understand that the project is still in the conceptual or preliminary design stage and is subject to change. We anticipate construction may include wood-framed, steel and concrete multi-story structures utilizing slabs-on grade and retaining walls.

We anticipate that grading for the project will include excavation of surface soil in the proposed parking areas to be used for fill material within proposed building

pads, as well as grading for driveways and excavation for underground utilities. Estimated fill depths of up to 20 feet and 10-foot cuts are proposed.

1.3 PURPOSE

We performed a surface reconnaissance and subsurface geotechnical investigation at the site in the area proposed for improvements, collected soil samples for laboratory testing, and performed engineering calculations including a slope stability analysis. The purpose of our investigation was to provide grading and drainage recommendations, foundation and retaining wall design criteria, slab-on-grade recommendations, and pavement design for the proposed improvements.

1.4 SCOPE-OF-SERVICES

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

- We performed a site investigation, including a literature review and a limited subsurface investigation.
- We collected bulk soil samples from selected exploratory trenches.
- We performed laboratory tests on select soil samples obtained during our subsurface investigation to determine their engineering material properties.
- Based on observations made during our subsurface investigation and the results of laboratory testing, we performed engineering calculations to provide geotechnical engineering recommendations for earthwork and structural improvements.

Our scope of services did not include a groundwater flow analysis nor an evaluation of the site for the presence of hazardous materials, historic mining features, asbestiform minerals, mold, or corrosive subsurface conditions. We also did not review the conditions of the existing earth dam near the southern portion of the property.

2 SITE INVESTIGATION

We performed a site investigation to characterize the existing surface conditions and shallow subsurface soil/rock conditions. Our site investigation included a literature review and field investigation as described below.

2.1 LITERATURE REVIEW

We performed a limited review of geologic literature pertaining to the project site. The following sections summarize our findings.

2.1.1 Soil Survey

As part of our study, we reviewed the *Soil Survey of Placer County, California, Western Part,* prepared by the USDA Soil Conservation Service, 1980. The soil survey indicates that the project site is located in an area containing the Exchequer very stony loam soil series.

The soil survey describes the Exchequer soil as shallow, somewhat excessively drained, very stony soil underlain by hard andesitic breccia. Typically, the surface soil consists of brown, very stony loam and cobbly loam, which extends to an approximate depth of 11 inches below the ground surface (bgs). The brown loam is typically underlain by hard andesitic breccia. The shallow soil depth and the presence of resistant shallow rock are noted as potential limitations to development on this soil type.

2.1.2 Geologic Setting

We reviewed the Geologic Map of the Sacramento Quadrangle, California Division of Mines and Geology, 1987, for information about site geology. The map indicates that early Pliocene - late Miocene age andesitic conglomerate and mudflow breccia (lahar) of the Mehrten Formation underlie the site.

The andesitic lava flows that underlie the site contain subrounded to subangular boulders of andesite and other rock types that were entrained by the lava as it flowed downslope and solidified. The Miocene and Pliocene epochs are considered to have occurred between 22 to 5 million years and 5 to 2 million years before present, respectively.

We reviewed California Geological Survey Open File Report 96-08, Probabilistic Seismic Hazard Assessment for the State of California, and the 2002 update entitled California Fault Parameters. The documents indicate the property is located within the Foothills Fault System. The Foothills Fault System is designated as a Type C fault zone, with low seismicity and a low rate of recurrence. The 1997 edition of California Geological Survey Special Publication 43, Fault Rupture Hazard Zones in California, describes active faults and fault zones (activity within 11,000 years), as part of the Alquist-Priolo Earthquake Fault Zoning Act. The map

and document indicate the site is not located within an Alquist-Priolo active fault zone.

2.1.3 Previous Site Investigations

H&K previously performed a preliminary geotechnical investigation and a Phase I Environmental Site Assessment for the Amazing Facts project.

- Our June 2007 preliminary geotechnical investigation included a shallow subsurface investigation to determine the depth to rock across the area of proposed improvements in the northern portion of the site. The preliminary investigation included the excavation of 16 trenches which generally revealed a shallow layer of surface soil underlain by resistant rock. Our report included a discussion of the shallow depth to resistant rock and provided preliminary recommendations regarding drainage and grading. We did not perform laboratory testing as part of our preliminary investigation.
- Our *Phase I Environmental Site Assessment,* dated June 13, 2007, discussed past uses of the property and evaluated potential environmental concerns.

2.2 FIELD INVESTIGATION

We performed our field investigation on February 27, 2009. During our field investigation, we observed the local topography and surface conditions and performed a limited subsurface investigation. The following sections summarize surface and subsurface conditions observed during our field investigation.

Our subsurface investigation included the excavation of 4 shallow exploratory trenches within the proposed improvement area in the northern portion of the property. We excavated to depths ranging between 8 inches and 18 inches below the ground surface (bgs) using a Kubota KX-121 excavator equipped with an 18-inch bucket. Collection of undisturbed samples was limited due to the shallow depth of resistant rock onsite. We collected grab and bulk samples from the exploratory trenches for laboratory testing. An engineer from our firm logged the soil conditions revealed in the exploratory trenches. Figure 2 shows the approximate exploratory trench locations.

2.2.1 Surface Conditions

At the time of our investigation, the site was undeveloped. The site generally slopes to the southeast with a large triangular plateau in the northernmost portion of the property where improvements are proposed. Vegetation on the plateau

consists of grasses and forbs with few to no trees. Vegetation on the slopes consists of grasses, forbs, pine and oak trees as well as manzanita and poison oak.

2.2.2 Subsurface Soil Conditions

The soil conditions described are generalized, based on our observations of the subsurface conditions revealed in our exploratory trenches. More detailed information can be found in the trench logs in Appendix C.

Trenches T-1 through T-4 all revealed similar subsurface conditions. The trenches were excavated through surface soil consisting of reddish brown, moist, loose silty gravel with sand. The silty gravel with sand was underlain by resistant rock at depths ranging from 8 inches bgs to 18 inches bgs. Our trenches terminated at refusal within the resistant rock.

2.2.3 Ground Water Conditions

During our site investigation, we did not encounter groundwater seepage in our exploratory trenches, nor did we observe onsite springs or seeps emanating from the ground surface. We did observe several pools of standing water onsite.

Our exploratory trench observations of subsurface conditions were made in February 2009 following a period of wet weather. Although we did not observe groundwater in our exploratory trenches, our experience has shown that isolated areas of seepage and saturated soil will likely be encountered in excavations which reveal the soil/weathered rock transition and rock fractures, particularly during or shortly after the rainy season.

3 LABORATORY TESTING

We performed laboratory tests on selected soil samples collected from our subsurface exploratory trenches to determine their engineering material properties. These engineering material properties were used to develop geotechnical engineering recommendations for earthwork and structural improvements. We performed the following laboratory tests:

- Direct Shear Strength (ASTM D3080),
- Resistance Value (ASTM D2844)
- Particle Size Determination (ASTM D422), and
- Expansion Index (ASTM 4829).

Collection of undisturbed soil samples for laboratory testing was limited by shallow depth to resistant rock and significant gravel content in the subsurface soil revealed in the exploratory trenches. Appendix D presents graphical direct shear, expansion index and particle size test results.

We performed a particle size determination on a sample of reddish brown silty gravel with sand collected from the surface to approximately 10 inches bgs in trench T-2. The test revealed the sample consisted of approximately 41 percent gravel, 30 percent sand and 29 percent silt and clay. Based on the particle size determination, we classified the soil as silty gravel with sand (GW).

We also performed expansion index testing on a portion of sample PB 2-1 described as a reddish brown silty gravel with sand. A portion of the sample was remolded in a 1.0-inch-high ring and submerged in water under an applied loading of 144 pounds per square foot (psf). We observed the loaded sample for a minimum of 24 hours. During that time we measured the swell (or settlement) with a dial micrometer. Expansion index test results of 10 indicate the sample exhibited very low expansion potential, as classified by UBC guidelines.

An R-value test was performed on a composite bulk sample obtained from the ground surface to 1-foot bgs in trenches T-1 through T-4. The sample was composed of reddish brown, silty gravel with sand. The test indicated that the predominantly granular soil had an R-value of 11, by exudation pressure.

We performed a direct shear test on a remolded portion of composite sample PB 4-1, collected at depths ranging from the surface to 1.5 feet bgs in exploratory trench T-4. The sample was composed of silty gravel with sand. The soil was consolidated under an applied normal load in a loose state and sheared. The direct shear test indicated the soil exhibited a friction angle of 30 and cohesion of 620 psf.

4 SLOPE STABILITY ANALYSIS

The stability of a slope is evaluated by calculating its "factor of safety". The factor of safety (FS) is a ratio obtained by dividing the resisting forces (i.e., the strength of the material comprising the slope) by the driving forces (resulting from the slope gradient, groundwater, and earthquake loading). If the factor of safety is greater than 1, the slope is theoretically stable. A factor of safety equal to or less than 1 means the slope is theoretically unstable. Required factors of safety are selected in an effort to address uncertainties in the conditions as well as the anticipated consequences of slope instability. Typically, when evaluating slope stability,

minimum factors of safety of 1.5 or higher are considered to account for variability in groundwater, subsurface soil and rock conditions, and laboratory test results.

The slope configuration used in our slope stability analyses was based on review of topographic site plans provided by King Engineering. The slope model used was based on trenches we excavated in our previous investigation on or near the sloping portion of the site, and on material properties determined from our laboratory testing. Our analysis was performed using Stabl6™ software utilizing Bishop's simplified methods of slices.

Our slope stability analyses were based on a variety of assumptions and variables, including:

1. Strength data variables - The strength data used in our calculations were varied based on several assumptions. The strength of the material making up the slope was estimated by considering the direct shear test results and our previous experience with slopes in the area.

2. Geometry - Our stability analyses considered slope geometry and slope gradient configurations based on the topographic site plan.

3. Piezometric Surfaces – Although we did not encounter subsurface water or seepage during our site investigation, we included a piezometric surface in the stability analysis to account for possible seasonal fluctuation in groundwater or saturated soil conditions. Typically, the assumed water table acts to reduce the stability of the slope by increasing the driving forces tending to cause a failure, while simultaneously reducing the shear strength available to resist a failure. We used soil strength parameters obtained from our laboratory tests to evaluate the stability of trial cut and fill slope configurations in the area of the proposed building pad and driveway.

For the purposes of our engineering analysis, we modeled the soil onsite as having an internal friction angle of 30 degrees, 200 pounds per square foot of cohesion, and a total unit weight of 100 pounds per cubic foot. We also conservatively modeled the subsurface rock onsite as having an internal friction angle of 40 degrees, 1000 pounds per square foot of cohesion, and a total unit weight of 145 pounds per cubic foot. Our slope stability analysis was based on a computer assisted method of slices. In addition, we considered a minimum acceptable factor of safety of 1.5 under static conditions.

The slope stability analysis considered large scale or global failure of the slope. Because the site is generally underlain by resistant rock at relatively shallow depths, it is our opinion the likelihood of deep-seated failure is very low. However, sloughing, raveling, and erosion of relatively loose material on the slope faces were not considered in the analysis and should be expected to occur, particularly on slopes steeper than approximately 2:1, H:V. Exposed slopes will require periodic maintenance to remove surficial erosion and debris.

5 CONCLUSIONS

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

1. Our opinion is that the site is suitable for the proposed improvements, provided that the geotechnical engineering recommendations and design criteria presented in this report are incorporated into the project plans.
2. Our primary concern is the presence of resistant rock at shallow depths which may affect excavatability.
3. Based on our site observations, the geology of the region and our experience in the area, our opinion is that the risk of seismically induced hazards such as liquefaction and surface rupture is remote at the project site.
4. Because the site is generally underlain by resistant rock at relatively shallow depths, it is our opinion the likelihood of deep-seated failure is very low. However, sloughing, raveling, and erosion of relatively loose material on the slope faces should be expected to occur, particularly on slopes steeper than approximately 2:1, H:V.
5. Based on the site geology and our observation of the surface conditions, we anticipate that grading and excavation onsite will reveal variably weathered rock. Spoil resulting from excavation onsite may consist of predominantly angular, gravel to cobble-sized rock fragments. This material may be suitable for use as fill, depending on the nominal size of the rock fragments, but will likely require specific recommendations for fill placement and observation to confirm compaction. Recommendations addressing rock fill placement are included in this report.
6. We did not observe existing fill in our exploratory trenches. However, if fill is encountered during grading, fill should not be relied upon to support proposed improvements. Recommendations for overexcavation and replacement of existing fill, if encountered, are provided in the following sections.

7. During our site investigation, we did not observe groundwater or seepage within our exploratory trenches. We anticipate that moist to saturated soil conditions and groundwater may be encountered during grading, particularly in excavations that reveal the soil/rock transition. Recommendations addressing moisture conditioning, drainage, and fill placement are presented in the following sections of this report.

8. Prior to grading and construction, we should be retained to review the proposed grading plan and structural improvements to confirm our recommendations.

6 RECOMMENDATIONS

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

6.1 GRADING

The following sections present our grading recommendations. The grading recommendations address clearing and grubbing, soil preparation, cut slope grading, fill placement, fill slope grading, erosion control, subsurface drainage, surface water drainage, construction dewatering, underground utility trenches, soil corrosion potential, plan review, and construction monitoring.

6.1.1 Clearing and Grubbing

The areas to be graded should be cleared and grubbed to remove vegetation and other deleterious materials as described below.

1. Strip and remove debris from clearing operations and the top 1 to 2 inches of soil containing shallow vegetation, roots and other deleterious materials. The organic topsoil can be stockpiled onsite and used in landscape areas but is not suitable for use as fill. The project geotechnical engineer should approve any proposed use of the spoil generated from stripping prior to placement.

2. Although not observed during our investigation, if loose, untested fill is encountered during site development, overexcavate to competent native soil or weathered rock a minimum of 5 feet beyond the areas of proposed improvements.

3. Remove rocks greater than 8 inches in greatest dimension (oversized rock) from native soil by scarifying to a depth of 12 inches below finish grade or to resistant rock in areas to support pavement, slabs-on-grade or other flatwork. Oversized rock may be used in landscape areas, rock landscape walls, or removed from the site. Oversized rock can be stockpiled onsite and used to construct fills, but must be placed at or near the bottom of deep fills and must be placed in windrows to avoid nesting. No oversized rock should be placed in the upper 3 feet of any structural fill. The project geotechnical engineer should approve the use of oversized rock prior to constructing fill.
4. Fine grained, potentially expansive soil, as determined by H&K, that is encountered during grading should be mixed with granular soil, or overexcavated and stockpiled for removal from the project site or for later use in landscape areas. A typical mixing ratio for granular to expansive soil is 4 to 1. The actual mixing ratio should be determined by H&K.
5. Vegetation, deleterious materials, structural debris, and oversized rocks not used in landscape areas, drainage channels, or other non-structural uses should be removed from the site.

6.1.2 Cut Slope Grading

Based on our understanding of the project at this time, we anticipate that permanent cut slopes up to 5 feet in height may be created during grading of the proposed improvements. In general, permanent cut slopes in soil should not be steeper than 2:1, horizontal to vertical (H:V). However, cut slopes may be graded to 1:1, H:V, where excavations reveal resistant rock. The upper portion of cut slopes revealing soil, typically the upper one to two feet, should be graded to an approximate 2:1, H:V, slope to reduce sloughing and erosion of looser surface soil.

Temporary cut slopes may be constructed to facilitate retaining wall construction. We anticipate that resistant rock subsurface conditions will be favorable for construction of temporary cut slopes no steeper than ½:1, H:V, for a maximum height of approximately 10 feet. To reduce the likelihood of sloughing or failure, temporary cut slopes should not remain over the winter.

A representative of H&K must observe temporary cut slopes steeper than 2:1, H:V, during grading to confirm the soil and rock conditions encountered. We recommend that personnel not be allowed between the cut slope and the proposed retaining structure, form work, grading equipment, or parked vehicles during construction, unless the stability of the slope has been reviewed by H&K or the slope has been confirmed to meet OSHA excavation standards.

6.1.3 Soil Preparation for Fill Placement

Where fill placement is proposed, the surface soil exposed by site clearing and grubbing should be prepared as described below.

1. The surface soil should be scarified to a minimum depth of 8 inches below the existing ground surface, or to resistant rock, whichever is shallower. Following scarification, the soil should be uniformly moisture conditioned to within approximately 3 percentage points of the ASTM D1557 optimum moisture content.
2. The scarified and moisture conditioned soil should then be compacted to achieve a minimum relative compaction of 90 percent based on ASTM D1557 maximum dry density. The moisture content, density, and relative percent compaction should be verified by a representative of H&K. The earthwork contractor should assist our representative by excavating test pads with onsite earth moving equipment.
3. Where fill placement is proposed on native slopes steeper than approximately 5:1, H:V, a base key and routine benches must be provided. Unless otherwise recommended by the project geotechnical engineer, the base key should be excavated at the toe of the fill to resistant rock or a minimum of 2 feet into competent stratum, as determined by a representative of H&K during construction observation. The bottom of the base key should be approximately level or sloped slightly into the hillside.
4. The fill must be benched into existing side slopes as fill placement progresses. Benching must extend through loose surface soil into firm material, and at intervals such that no loose surface soil is beneath the fill. As a minimum, a horizontal bench should be excavated every 5 vertical feet or as determined by a representative of H&K.

6.1.4 Soil Fill Placement

Soil fill placement proposed for the project should incorporate the following recommendations:

1. Soil used for fill should consist of uncontaminated, predominantly granular, non-expansive native soil or approved import soil. Rock used in fill should be broken into pieces no larger than 8 inches in diameter. Rocks larger than 8 inches are considered oversized material and should be stockpiled for offhaul or later use in landscape areas and drainage channels.

2. Import soil should be predominantly granular, non-expansive and free of deleterious material. Import material that is proposed for use onsite should be submitted to H&K for approval and possible laboratory testing at least 72 hours prior to transport to the site.
3. Cohesive, predominantly fine grained, or potentially expansive soil encountered during grading should be stockpiled for removal, mixed as directed by H&K, or used in landscape areas.
4. As an option, cohesive fine grained, or potentially expansive soil can often be placed in the deeper portions of proposed fill (e.g., depths greater than 3 feet below subgrade in building footprints). However, this option would have to be evaluated on a case-by-case basis with consideration of the fill depth and proposed loading.
5. Soil used to construct fill should be uniformly moisture conditioned to within approximately 3 percentage points of the ASTM D1557 optimum moisture content. Wet soil may need to be air dried or mixed with drier material to facilitate placement and compaction, particularly during or following the wet season.
6. Fill should be constructed by placing uniformly moisture conditioned soil in maximum 8-inch-thick loose, horizontal lifts (layers) prior to compacting.
7. All fill should be compacted to a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density. The upper 12 inches of fill in paved areas, beneath proposed slabs-on-grade, and within the proposed building footprint should be compacted to a minimum of 95 percent relative compaction.

The moisture content, density and relative percent compaction of fill should be confirmed by a representative of H&K during construction.

6.1.5 Differential Fill Depth

The recommendations presented in this section are intended to reduce the magnitude of differential settlement-induced structural distress associated with variable fill depth beneath structures.

In general, site grading should be performed so that cut-fill transition lines do not occur directly beneath any structures. In addition, differential fill depths beneath structures should not exceed 5 feet. However, because of the relatively shallow depth to resistant rock observed at the site, we anticipate that it will not be feasible to overexcavate the cut portions of cut/fill building pads in an effort to reduce differential settlement. The use of modified foundation systems or deepened

footings may be appropriate to reduce settlement at locations where significant differential fill depths will occur. We can provide site specific settlement analysis, and recommendations to mitigate the potential for differential settlement, once grading plans and design of the proposed structures has been completed.

6.1.6 Rock Fill Placement

Based on our observation of the rocky nature of the subsurface conditions revealed in our exploratory trenches, we anticipate that fill material generated from the project site may contain significant rock fragments, and that compaction testing with conventional methods may be difficult or inappropriate. Typically, fill that consists primarily of soil can be tested for relative compaction by using a nuclear density gauge. Our opinion is that rock fill cannot be reliably tested using this method.

We recommend that quality assurance during rock fill placement be based on a procedural approach, or method specification, rather than a specified relative compaction. The procedural requirements will depend on the equipment used, as well as the nature of the fill material, and will need to be determined by the geotechnical engineering firm onsite. Typically, procedural recommendations are based on the measured relative compaction of a test fill constructed onsite.

Based on our experience in the area, we anticipate that the procedural specification will require a minimum of six passes (back and forth equaling one pass) with a Cat 563 or similar, self-propelled, vibratory compactor to compact a maximum 8-inch thick, loose lift. Processing or screening of the fill material will be needed to remove rocks larger than approximately 8 inches in maximum dimension. Continuous or nearly continuous observation by a representative of H&K would be required during fill placement to confirm that procedural specifications have been met.

6.1.7 Fill Slope Grading

Based on our understanding of the project, we anticipate that fill slopes up to 30 feet in height may be created as part of the proposed improvements. In general, permanent fill slopes created onsite should be no steeper than 2:1, H:V. H&K should review fill slope configurations greater than approximately 15 feet in height, if proposed, prior to fill placement. Compaction and fill slope grading must be confirmed by H&K in the field.

Steeper fill slopes may be feasible with the use of geotextile reinforcement and/or rock facing. We can provide reinforced or buttressed fill slope design for the project, if requested.

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.

6.1.8 Erosion Controls

Graded portions of the site should be seeded as soon as possible to allow vegetation to become established prior to and during the rainy season. In addition, grading that results in greater than one acre of soil disturbance or in sensitive areas may require the preparation of a site-specific storm water pollution prevention plan. As a minimum, the following controls should be installed prior to and during grading to reduce erosion.

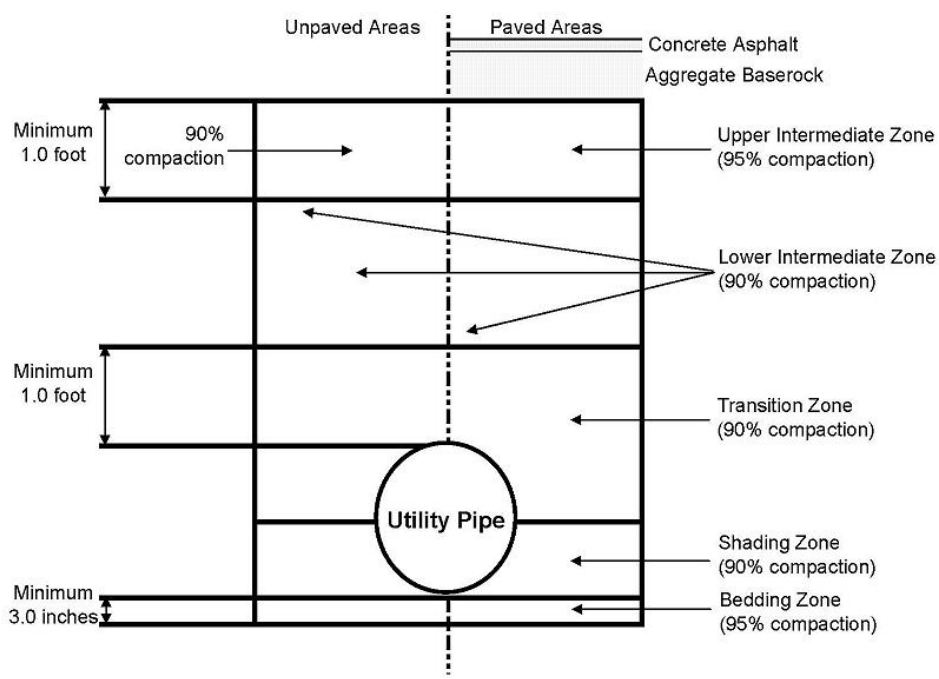
1. Prior to commencement of site work, fiber rolls should be installed down slope of the proposed area of disturbance to reduce migration of sediment from the site. Fiber rolls on slopes are intended to reduce sediment discharge from disturbed areas, reduce the velocity of water flow, and aid in the overall revegetation of slopes. The fiber rolls should remain in place until construction activity is complete and vegetation becomes established.
2. Soil exposed in permanent slope faces should be hydroseeded or hand seeded/strawed with an appropriate seed mixture compatible with the soil and climate conditions of the site as recommended by the local Resource Conservation District.
3. Following seeding, jute netting or erosion control blankets should be placed and secured over the slopes steeper than 2:1, H.V.
4. Surface water drainage ditches should be established as necessary to intercept and redirect concentrated surface water away from cut and fill slope faces. The intercepted water should be discharged into natural drainage courses or into other collection and disposal structures.

6.1.9 Underground Utility Trenches

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

1. Based on subsurface conditions observed in our exploratory trenches, we anticipate that resistant rock at shallow depths will limit utility trench excavations. Pre-ripping of the trench alignment, using trenchers or rock-wheels may be required, particularly if utility trench excavations are deeper than three feet.
2. The California Occupational Safety and Health Administration (OSHA) requires all utility trenches deeper than 4 feet bgs be shored with bracing equipment prior to being entered by any individuals, whether or not they are associated with the project.
3. We anticipate that shallow subsurface seepage may be encountered, particularly if utility trenches are excavated during the winter, spring, or early summer. The earthwork contractor may need to employ dewatering methods as discussed in the Construction Dewatering section on page 17 to excavate, place and compact the trench backfill materials.
4. Trench backfill used within the bedding zone, shading zone, and transition zones, as shown on the following figure, should consist of ¾-inch minus crushed rock.

TRENCH BACKFILL



5. Soil used as trench backfill within the lower and upper intermediate zones, as shown on the above figure, should consist of non-expansive soil with a PI of less than or equal to 15 (based on ASTM D4318) and should not contain rocks greater than 3 inches in greatest dimension.
6. Soil used to construct trench backfill should be uniformly moisture conditioned to within 3 percentage points of the ASTM D1557 optimum moisture content.
7. Trench backfill should be constructed by placing uniformly moisture conditioned soil in maximum 12-inch-thick loose lifts (layers) prior to compacting.
8. Pipe bedding zone: Trench backfill placed in the pipe bedding zone (beneath the utilities) should be compacted to a minimum relative compaction of 95 percent of the ASTM D1557 maximum dry density.
9. Pipe shading zone: Trench backfill placed within the pipe shading zone (above the bedding zone and to a height of one pipe radius above the pipe spring line) should be compacted to a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density.
10. Pipe transition zone: Trench backfill placed within the pipe transition zone (above the pipe shading zone to one foot over the pipe top surface) should be compacted to a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density.
11. Lower intermediate zone: Trench backfill placed within the lower intermediate zone (above the pipe transition zone to 1 foot below the finished subgrade surface) should be compacted to a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density.
12. Upper intermediate zone (unpaved areas): Trench backfill placed within the upper intermediate zone (above the lower intermediate zone to the finished subgrade surface) in unpaved (non-road and non-parking lot) areas should be compacted to a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density.
13. Upper intermediate zone (paved areas): Trench backfill placed within the upper intermediate zone (above the lower intermediate zone to the finished subgrade surface) in paved (road and parking lot) areas should be compacted to a minimum relative compaction of 95 percent of the ASTM D1557 maximum dry density.
14. The loose lift thickness, moisture, density and relative compaction of the trench backfill soil should be verified by a representative of H&K.

15. Construction quality assurance tests should be performed at a frequency determined by the project geotechnical engineer.
16. The earthwork contractor should assist our representative by excavating test pads with onsite earth moving equipment.

6.1.10 Construction Dewatering

The earthwork contractor should be prepared to dewater excavations if seepage is encountered during grading. Seepage will likely be encountered if grading is performed during or immediately after the rainy season. In addition, perched groundwater may be encountered on resistant rock layers even during the summer months.

If subsurface seepage or groundwater conditions are encountered which prevent or restrict fill placement or construction of the proposed improvements, subdrains may be necessary. If groundwater or saturated soil conditions are encountered during grading, we should be retained to observe the conditions and provide site specific subsurface drainage recommendations. The following typical measures can be employed to mitigate the presence of seepage in excavations.

1. We anticipate that dewatering of utility trenches can be performed by constructing sumps to depths below the trench bottom and removing the water with sump pumps.
2. Additional sump excavations and pumps should be added as necessary to keep the excavation bottom free of standing water and relatively dry when placing and compacting the trench backfill material.

6.1.11 Soil Corrosion Potential

Index testing of the soil in an effort to evaluate corrosion potential was not performed as a part of our soil evaluation. Based on review of soil survey information the native soil conditions onsite possess a low and moderate corrosion potential for uncoated steel and concrete, respectively.

To reduce the likelihood of corrosion problems, materials used for underground utilities, permanent subsurface drainage improvements, and foundation systems should be selected based on local experience and practice. If alternative or new construction methods or materials are being proposed, it may be appropriate to have the selected materials evaluated by a corrosion engineer for compatibility with the onsite soil and groundwater conditions.

6.1.12 Surface Water Drainage

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

1. Slope final grades in structural areas so that surface water drains away from building pad finish subgrade at a minimum 2 percent slope for a minimum distance of 10 feet. For structures utilizing slab-on-grade interior floor systems we recommend increasing the slope to 4 percent.
2. To reduce surface water infiltration, compact and slope all soil placed adjacent to building foundations such that water is not allowed to pond. Backfill should be free of deleterious materials.
3. Direct downspouts to positive drainage or a closed collector pipe that discharges flow to positive drainage.
4. Construct V-ditches at the top of cut and fill slopes where necessary to reduce concentrated surface water flow over slope faces. Typically, V-ditches should be 3 feet wide and at least 6 inches deep. Surface water collected in V-ditches should be directed away and downslope from proposed building pads and driveways into a drainage channel.

6.1.13 Grading Plan Review and Construction Monitoring

Construction quality assurance includes review of plans and specifications and performing construction monitoring as described below.

1. H&K should be retained to review the final grading plans prior to construction to confirm our understanding of the project at the time of our investigation, to determine whether our recommendations have been implemented, and to provide additional and/or modified recommendations, if necessary.
2. H&K should be retained to perform construction quality assurance (CQA) monitoring of all earthwork grading performed by the contractor to determine whether our recommendations have been implemented, and if necessary, provide additional and/or modified recommendations.

6.2 STRUCTURAL IMPROVEMENT DESIGN CRITERIA

The following sections present our structural improvement design criteria and recommendations. The recommendations address foundations, seismic parameters, concrete slabs-on-grade, retaining walls and pavement design.

6.2.1 Seismic Design Criteria

Our classification of on-site soil conditions is based on field observations and laboratory tests. The on-site soil primarily consists of granular soil composed of silty gravel with sand. Based on the presence of predominantly granular soil and resistant rock at relatively shallow depths, we classified the on-site soil as GW for design purposes.

Table 6.2.1.1 below summarizes seismic design criteria based on Section 1613 of the 2007 California Building Code, CCR Title 24, Part 2. The building code updates are effective as of January 2008.

We used Section 1613 of the 2007 California Building Code (CBC) and the United States Geological Survey (USGS), *Java Ground Motion Parameter Calculator, Earthquake Ground Motion Tools, Version 5.0.8*, to develop the following seismic design parameters:

6.2.1.1 - 2007 Seismic Design Parameters

Description	Value	Reference	Description	Value	Reference
Latitude Longitude	38.774 -121.209	1	Site Class	B	2
Site Coefficient, F_A	1.00	6	Site Coefficient, F_V	1.00	7
Short (0.2 sec) Spectral Response, S_S	0.398*g*	3, 5	Long (1.0 sec) Spectral Response, S_1	0.197*g*	4, 5
S_S modified for Site Class Effects, S_{MS}	0.398*g*	8, 5	S_1 modified for Site Class Effects, S_{M1}	0.197*g*	9, 5

References:

1. *USGS 7.5 min*
2. *2007 CBC, Table 1613.5.2*
3. *CBC Figure 1613.5(3)*
4. *CBC Figure 1613.5(4)*
5. *USGS Uniform Hazard Response Spectra, v 5.0.8*
6. *2007 CBC, Table 1613.5.3(1)*
7. *2007 CBC, Table 1613.5.3(2)*
8. *2007 CBC, Equation 16-37*
9. *2007 CBC, Equation 16-38*

6.2.2 Foundations

Provided that the grading for the project is performed in accordance with the recommendations presented in this report, our opinion is that the site will be suitable for the use of conventional perimeter foundations, isolated interior footings, and interior slabs-on-grade. Following are our recommendations for foundations constructed on compacted and tested fill or competent native soil:

1. Footings for single story structures should be a minimum of 12 inches wide and trenched through any loose surface material, potentially expansive soil, or untested fill, and a minimum of 12 inches into competent native soil, weathered rock or compacted fill. Footings for two-story structures should be a minimum of 15 inches wide and trenched a minimum of 18 inches into competent native soil, weathered rock or compacted fill. If clay is encountered at the base of footing excavations, the footing should be deepened through the clay lens into underlying granular material or weathered rock, as determined in the field by H&K.
2. The base of the footing excavation should be approximately level. On sloping sites, it will be necessary to step the base of the footing excavation as necessary to maintain a slope of less than 10 percent at the base of the footing.
3. Footing trenches should be cleaned of all loose soil and construction debris prior to placing concrete. A representative from H&K should observe the footing excavations prior to concrete placement.
4. As a minimum, the footings should be designed with two No. 4 rebar reinforcement, one near the top of the footing and one near the bottom. A minimum of 3 inches of concrete coverage should surround the bars.
5. Footing excavations should be saturated prior to placing concrete to reduce the risk of problems caused by wicking of moisture from curing concrete. However, concrete should not be placed through standing water in the footing excavations.
6. In an effort to reduce the likelihood of settlement-induced distress to the proposed structures, we recommend that strip and isolated footings with a minimum embedment depth of 12 inches in competent native soil or compacted and tested fill be sized for an allowable bearing capacity of 3000 psf for dead plus live loads. This value can be increased by 500 psf for each additional foot of embedment up to a limiting value of 3500 psf. Allowable bearing may be increased by 33 percent for additional transient loading, such

as wind or seismic loads. Footings placed on resistant rock may use a higher allowable bearing capacity of 4,000 psf.

7. A triangularly-distributed lateral resistance (passive soil resistance) of 300d psf, where d is footing depth, may be used for footings. This value may be increased by 33 percent for wind and seismic. As an alternate to the passive soil resistance described above, a coefficient of friction for resistance to sliding of 0.35 may be used.

8. 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 one-half inch. Differential settlement between similarly loaded, adjacent footings is expected to be less than one-quarter inch, provided footings are founded on similar materials (e.g., all on structural 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.

9. We anticipate that construction may include concrete “tilt up” structures. Concrete tilt-up construction transfers loads to the footings relatively quickly. Therefore, elastic settlement of the footings may become a factor when aligning the panels. We estimate that, using the recommended allowable bearing pressures presented in this report, total elastic footing settlement would be on the order of ¾ inch with differential settlement of ½ inch. Elastic settlement occurs relatively quickly (several days). Therefore, it is expected that the top of the panels may yield slightly during placement. Alignment of the top of the panels will be governed by their height. Therefore, taller panels will typically reveal more horizontal displacement at the top than shorter walls. We recommend that the crane setting the panels hold the load as long as possible to allow mobilization of the footings. Some adjustment of the panels may be required following placement.

6.2.3 Slab-on-Grade Floor Systems

Our opinion is that interior concrete slab-on-grade floors may be used in conjunction with perimeter concrete foundations for the proposed improvements. The project structural engineer should design slabs-on-grade with regard to the

anticipated loading. This section presents typical slab sections and reinforcement schedules used for construction in the region and presents construction recommendations. We can provide project specific slab-on-grade design for the proposed improvements once anticipated loading and serviceability criteria have been established.

1. The slab-on-grade should be a minimum of 4 inches thick. If floor loads higher than 250 psf or intermittent live loads are anticipated, a structural engineer should determine the slab thickness and steel reinforcing schedule.
2. The subgrade soil around the slabs-on-grade should be sloped away from the proposed slab subgrade a minimum of 4 percent for a distance of 10 feet as discussed in the Surface Water Drainage section of this report. A representative from H&K should observe pad and subgrade elevations prior to forming the slab footings.
3. As a minimum, No. 3 rebar on 18-inch centers or flat sheets of 6x6, W4.0xW4.0 welded wire mesh (WWM) should be used as slab reinforcement. We do not recommend using rolls of WWM because vertically centered placement of rolled mesh within the slab is difficult to achieve. All rebar and sheets of WWM should be placed in the center of the slab and supported on concrete "dobies". We do not recommend "hooking and pulling" of steel during concrete placement.
4. Prior to placing the vapor retarder and concrete, slab subgrade soil must be moisture conditioned to between 75 and 90 percent saturation to a depth of 24 inches. Moisture conditioning should be performed for a minimum of 24 hours prior to concrete placement. Clayey soil may take up to 72 hours to reach this required degree of saturation. If the soil is not moisture conditioned prior to placing concrete, moisture will be wicked out of the concrete, possibly contributing to shrinkage cracks. Additionally, our opinion is that moisture conditioning the soil prior to placing concrete will reduce the likelihood of soil swell or heave following construction at locations where fine grained, potentially expansive soil is encountered. To facilitate slab-on-grade construction, we recommend that the slab subgrade soil be moisture conditioned following rock placement. Following moisture conditioning, the vapor retarder should be placed.
5. Slabs should be underlain by 4 inches of washed rock. The rock should be uniformly graded so that 100% passes the 1-inch sieve, with 0% to 5% passing the No. 4 sieve. Following rock placement, the subgrade soil should be moisture conditioned for 24 hours. The rock should then be overlain by a

vapor retarder at least 15 mils thick. All penetrations through the vapor retarder should be taped or sealed to reduce vapor. Laps in the vapor retarder should be taped. If requested, H&K can provide observation of the vapor retarder prior to placing concrete. The vapor retarder may be omitted in areas that do not have moisture sensitive floor coverings (i.e., exterior parking areas).

6. Regardless of the type of vapor retarder used, moisture can wick up through a concrete slab. Excessive moisture transmission through a slab can cause adhesion loss, warping and peeling of resilient floor coverings, deterioration of adhesive, seam separation, formation of air pockets, mineral deposition beneath flooring, odor and fungi growth. Slabs can be tested for water transmissivity in areas that are moisture sensitive. Commercial sealants, entrained air, fly ash and a reduced water to cement ratio can be incorporated into the concrete to reduce slab permeability. A waterproofing consultant should be contacted if moisture sensitive flooring is proposed.
7. Expansion joints should be provided between the slab and perimeter footings. Control joints should bisect the length and width of the slab at intervals specified by the American Concrete Institute (ACI) or Portland Concrete Association (PCA).
8. Exterior slabs-on-grade, such as sidewalks, may be placed directly on compacted fill without the use of a baserock section. For exterior slabs, the native soil should be ripped, moisture conditioned and recompacted to an 8-inch depth per the grading recommendations presented in this report.
9. All deleterious material must be removed prior to placing concrete.
10. We recommend that concrete have a water/cement ratio no greater than 0.45. Pozzolans or other additives may be added to increase workability.
11. Concrete slabs should be moisture cured for at least seven days after placement. Excessive curling of the slab may occur if moisture conditioning is not performed. This is especially critical for slabs that are cast during the warm summer months.
12. Concrete slabs impart a relatively small load on the subgrade (approximately 50 psf). Therefore, some vertical movement should be anticipated from possible expansion or differential loading.

6.2.4 Rock Anchors

Rock anchors or doweling may be used to provide lateral and uplift resistance where shallow, competent rock limits footing excavation. Rock anchors should

only be installed in competent rock, to be determined in the field by a representative of H&K. The design of rock anchors should include the following criteria.

1. Pull-out resistance for rock anchors will generally be limited by the shear resistance between the grout and the native rock. For design purposes, a pull-out resistance of 50 pounds per square inch of grout/competent rock contact may be used. Because of the strain in the anchor steel during pull-out, we recommend that the upper 6 inches of grout/competent rock contact be neglected when sizing for uplift.
2. We recommend that the drilled hole have a minimum ½-inch annular clearance between the steel and surrounding rock. Thus, grouting a No. 4 rebar would require a 1½-inch diameter hole.
3. Lateral shear resistance for rock anchors should be designed using V_s=0.45 F_y, where F_y equals the tensile strength of the steel. To develop this shear resistance, a minimum steel embedment of 24 inches into undisturbed, competent rock should be used.
4. Prior to anchor placement, loose debris, dust, and standing water in the hole must be removed by blowing with oil-free compressed air, cleaning the hole with a nylon brush, and then blowing out the remaining dust. Dust and debris left in the hole will significantly reduce anchor capacity.
5. We recommend using a cement grout that has a water/cement ratio of less than 0.6 to construct rock anchors. If high strength epoxy or other adhesives are proposed, H&K should review the proposed rock anchor detail prior to construction.
6. If rock anchors are used on more than 10 percent of the foundation system of any given structure, a representative of H&K should perform pull tests on select anchors.

6.2.5 Retaining Wall Design Criteria

The following active and passive pressures are for retaining walls in cut native soil or backfilled with granular onsite soil. If import soil is used, a representative from our firm should be retained to observe and test the soil to determine its strength properties. The pressures exerted against retaining walls may be assumed to be equal to a fluid of equivalent unit weight.

Table 6.2.5.1 presents equivalent fluid unit weights for cut native soil and onsite fill compacted per the grading recommendations presented in this report. For approximately horizontal backfill we assume that the retained fill surface will be no steeper than 10% for a minimum distance of the wall height from the back of the retaining wall. If surcharge loads (such as adjacent building foundations) or live loads will be applied within a distance of the wall height from the back of the wall, we should be retained to review the loading conditions and revise our recommendations, if necessary.

Table 6.2.5.1 - Equivalent Fluid Unit Weights *(1)*

Loading Condition	Retained Cut or Compacted Fill (approximately horizontal backfill)	Retained Cut or Compacted Fill (retained slope up to 2:1, H:V)
Active Pressure (pcf)	30	50
Passive Pressure (pcf)	300	300
At-Rest Pressure (pcf)	50	65
Coefficient of Friction	0.35	0.35

Note: (1) The equivalent fluid unit weights presented are ultimate values and do not include a factor of safety. The passive pressures provided assume footings are founded in competent native soil or engineered fill.

Please note that the use of the tabulated active pressure unit weight requires that the wall design accommodate sufficient deflection for mobilization of the retained soil to occur. Typically, a wall yield of less than 1 percent of the wall height is sufficient to mobilize active conditions in granular soil. However, if the walls are rigid or restrained to prevent rotation, at-rest conditions should be used for design.

Recommendations for design and construction of retaining walls are listed below:

1. Compaction equipment should not be used directly adjacent to retaining walls unless the wall is designed or braced to resist the additional lateral pressures.
2. If any surface loads are closer to the top of the retaining wall than its height, H&K should review the loads and loading configuration. We should be retained to review wall details and plans for any wall over 10 feet in height.
3. All retaining walls must be well drained to reduce hydrostatic pressures. Walls should be provided with a drainage blanket to reduce additional lateral forces and minimize saturation of the backfill soil. Drainage blankets may consist of graded rock drains or geosynthetic blankets.

4. Rock drains should consist of a minimum 12-inch wide, Caltrans Class II, permeable drainage blanket, placed directly behind the wall; or crushed washed rock enveloped in a non-woven geotextile filter fabric such as Amoco 4546™ or equivalent. Drains should have a minimum 4-inch diameter, perforated, schedule 40, PVC pipe placed at the base of the wall, inside the drainrock, with the 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 the collected 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. Adequate drainage and waterproofing for retaining walls associated with finished interior spaces are essential to reduce the likelihood of seepage and vapor transmission into the living space. We recommend that an appropriate waterproofing sealant be applied to the exterior surface of such retaining walls. A waterproofing consultant may be contacted to further review seepage and vapor transmission.
6. 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 0.2g, we recommend that the resulting additional lateral force applied to unrestrained (cantilevered) retaining structures with drained level backfill onsite be estimated as $P_{ae}=9H^2$ pounds, where H is the height of the wall in feet. The additional seismic force may be assumed to be applied at a height of 0.6H above the base of the wall. This seismic loading is for a drained, level backfill condition only; H&K should be consulted for values of seismic loading due to non-level or non-drained backfill conditions. The use of reduced factors of safety is often appropriate when reviewing overturning and sliding resistance during seismic events.

6.2.6 Pavement Design

Based on our review of the project grading plans, we understand that the parking area in the northernmost portion of the property will be located in an area of cut within the resistant rock. We also anticipate that the fill used for the other parking areas will consist of both onsite soil as well as gravel-size rock fragments derived from rock excavation. Our R-value testing was performed on the shallow surface soil observed onsite, and resulted in an R-value of 11 by exudation pressure. Considering the depth of rock excavation expected onsite, and the likelihood of

significant rock content in the resulting fill material, we have elected to use an R-value of 30 for preliminary design of asphalt pavement sections.

The following recommended asphalt concrete flexible pavement sections are based on preliminary traffic indices (TIs) of 5.5 and 6.5 as requested by King Engineering. The TIs are being considered on a preliminary basis to facilitate planning of the proposed onsite and offsite roadways. Other TIs may need to be considered in design if heavy vehicle loads, truck traffic, or improvements to the adjacent streets are proposed. Pavement design is presented in Table 6.2.6.1 below.

Table 6.2.6.1 - Recommended Pavement Sections

Traffic Index: 5.5 Design R-Value: 30	Alternate A Pavement Section (inches)	Alternate B Pavement Section (inches)
Caltrans Section 26, Standard Specifications, Asphalt Concrete	3.0	3.5
Caltrans Section 26, Class 2 Baserock 95% compaction	7.0	6.0
Subgrade Soil 95% compaction	8.0	8.0
Traffic Index: 6.5 Design R-Value: 30	**Alternate A Pavement Section (inches)**	**Alternate B Pavement Section (inches)**
Caltrans Section 26, Standard Specifications, Asphalt Concrete	3.5	4.0
Caltrans Section 26, Class 2 Baserock 95% compaction	9.0	8.0
Subgrade Soil 95% compaction	8.0	8.0

We make the following recommendations regarding paving at the site.

1. Fill must be compacted to a minimum of 90 percent of the maximum dry density per ASTM D 1557, Modified Proctor. The upper 8 inches of subgrade in areas to be paved must be compacted to a minimum of 95 percent per ASTM D 1557. Baserock should be compacted to a minimum of 95 percent

per ASTM D 1557. Moisture content, density and relative percent compaction should be verified by H&K. In addition to density testing, the subgrade must be proofrolled under the observation of a representative of H&K, prior to baserock placement.

2. Subgrade should be sloped to drain away from the proposed road alignment.
3. Import soil, if used, should be predominantly granular, non-expansive and free of deleterious material. Proposed import should be submitted to H&K for testing prior to transport to the site.
4. Steel reinforced concrete slabs should be considered for use in loading bays, service docks, garbage facilities, and other areas where frequent, heavy vehicle loads are anticipated. The project structural engineer should determine slab thickness and steel reinforcement.
5. Depending on the subsurface conditions encountered and the sources of fill, the actual subgrade material may vary significantly from that tested during this investigation. Representative subgrade samples should be obtained and additional R-value tests performed, if appropriate, to confirm the recommendations in this report. If the results of confirmation testing vary significantly from those used in design, the recommended pavement sections may need to be revised.

7 LIMITATIONS

The following limitations apply to the findings, conclusions and recommendations presented in this report:

1. Our professional services were performed consistent with the generally accepted geotechnical engineering principles and practices employed in northern California. This warranty is in lieu of all other warranties, either expressed or implied.
2. These services were performed consistent with our agreement with our client. We are not responsible for the impacts of any 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 unless noted otherwise. Any reliance on this report by a third party is at the party's sole risk.
3. If changes are made to the nature or design of the project as described in this report, then the conclusions and recommendations presented in this report

should be considered invalid. Only our firm can determine the validity of the conclusions and recommendations presented in this report. Therefore, we should be retained to review all project changes and prepare written responses with regards to their impacts on our conclusions and recommendations. However, we may require additional fieldwork and laboratory testing to develop any modifications to our recommendations. Costs to review project changes and perform additional fieldwork and laboratory testing necessary to modify our recommendations are beyond the scope of services presented in this report. Any additional work will be performed only after receipt of an approved scope of services, budget, and written authorization to proceed.

4. The analyses, conclusions and recommendations presented in this report are based on site conditions as they existed at the time we performed our surface and subsurface field investigations. We have assumed that the subsurface soil and groundwater conditions encountered at the location of our exploratory trenches are generally representative of the subsurface conditions throughout the entire project site. However, the actual subsurface conditions at locations between and beyond our exploratory trenches may differ. Therefore, if the subsurface conditions encountered during construction are different than those described in this report, then we should be notified immediately so that we can review these differences and, if necessary, modify our recommendations.

5. The elevation or depth to groundwater underlying the project site may differ with time and location.

6. The project site map shows approximate exploratory trench locations as determined by pacing distances from identifiable site features. Therefore, the trench locations should not be relied upon as being exact nor located with surveying methods.

7. Our geotechnical investigation scope of services did not include evaluating the project site for the presence of historic mining operations or hazardous materials. Although we did not observe evidence of historic mining activity or hazardous materials within the proposed building area at the time of our field investigation, all project personnel should be careful and take the necessary precautions should hazardous materials be encountered during construction. Possible historic mining excavation not detected during our investigation may impact the proposed improvements.

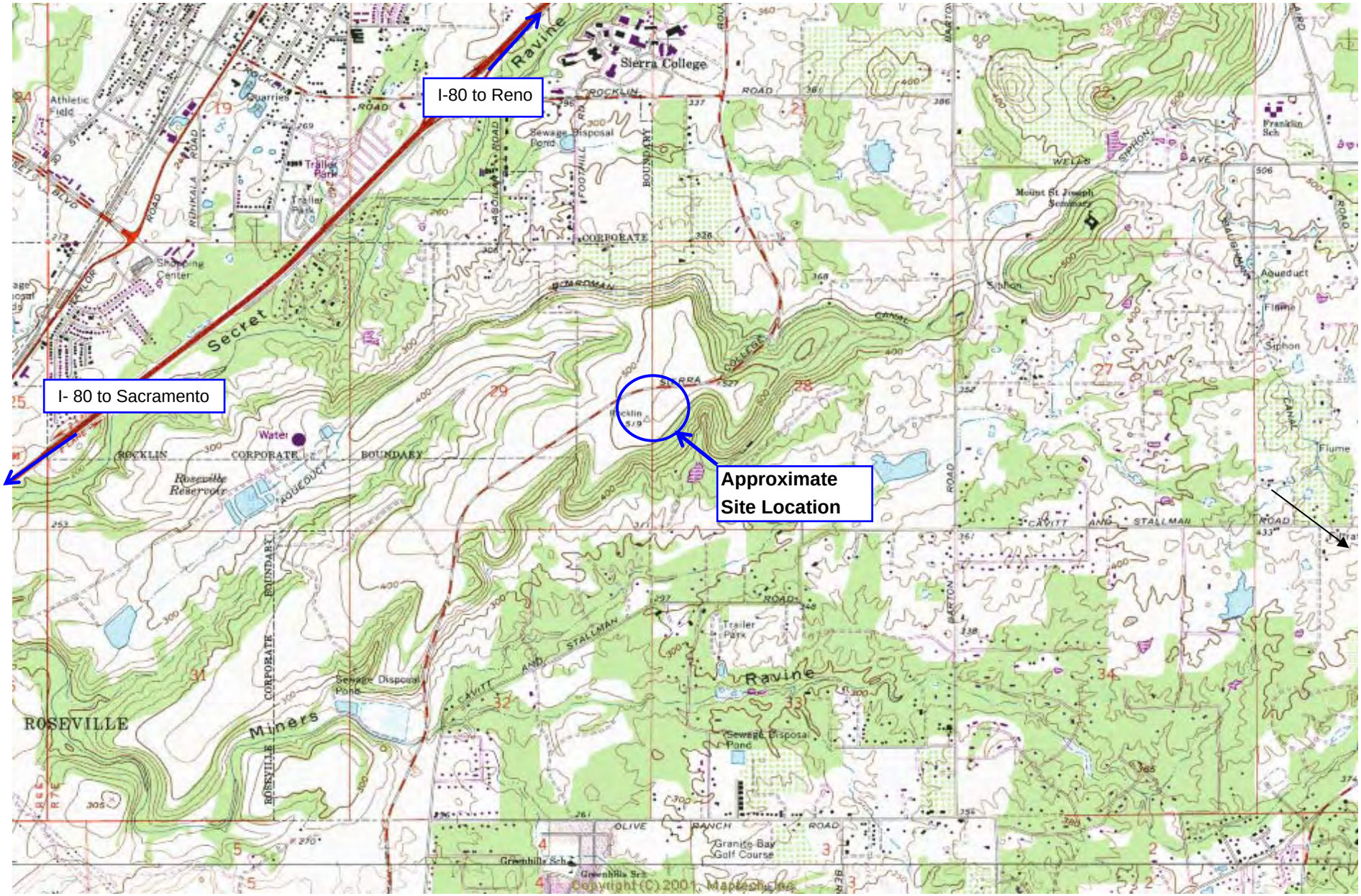
8. The findings of this report are valid as of the present date. However, changes in the conditions of the property can occur with the passage of time. The changes may be due to natural processes or to the works of man, on the

project site or adjacent properties. In addition, changes in applicable or appropriate standards can occur, whether they result from legislation or the 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 **Exploratory Trench Location Map**



SOURCE: MAPTECH, Terrain Navigator Pro, ver. 7.0 - USGS 7.5 minute topographic map, Rocklin Quadrangle, 1981.

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TOPOGRAPHIC VICINITY MAP
APN's 046-050-006 and -008
Placer County, California

PROJECT NO. 3307A-02

FIGURE 1

April 2009

APPENDIX A PROPOSAL

HOLDREGE & KULL
CONSULTING ENGINEERS • GEOLOGISTS

Proposal No. PN08237
February 17, 2009

Mr. Rod Shearer
Shearer & Associates
1404 Northeast 134th Street, Suite 200
Vancouver, WA 98685

Reference: ***Proposed Amazing Facts Site***
Sierra College Boulevard and Nightwatch Drive
APNs 046-050-006 and 046-050-008
Placer County, California

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

Dear Mr. Shearer:

At your request, Holdrege & Kull (H&K) prepared this proposal to provide geotechnical engineering services for the Amazing Facts property at APNs 046-050-006 and 046-050-008 in Placer County, California. We understand that the project will consist of the construction of a church facility and associated parking areas, driveways and underground utilities. The purpose of our services will be to provide a design-level geotechnical report for the project to satisfy the requirements of Placer County. We previously performed a limited subsurface investigation and prepared a preliminary geotechnical report for the property dated May 25, 2007.

SCOPE OF SERVICES

Based on our understanding of the project, we propose to perform the following scope of services.

Site Investigation

H&K's site investigation will include only the area currently proposed for improvement. We will perform a map and literature search, and review published documents pertinent to the area proposed for improvement. We will perform a site reconnaissance to identify surface conditions that may impact the proposed development plans.

Our subsurface investigation will include the excavation of 3 to 4 shallow exploratory trenches in the vicinity of the proposed improvements to depths up to 10 feet or to resistant rock. We will obtain undisturbed and bulk samples of soil for laboratory testing. An engineer or geologist from our firm will log the trenches in the field.

Laboratory Testing

H&K will perform laboratory tests on selected soil samples to determine their engineering material properties. We anticipate our laboratory testing will include moisture/density determination, Atterberg limits, expansion index (if appropriate), direct shear testing, and R-value determination. Direct shear test results are used to calculate foundation and retaining wall design criteria. Moisture/density tests are used to determine the dry density and moisture content of relatively undisturbed soil samples. R-value test results will be used to provide pavement design recommendations for the proposed parking area and driveways.

If clayey, potentially expansive soil is observed, we will perform expansion index and/or Atterberg limits testing to evaluate the expansion potential of the soil.

Engineering

Using the laboratory test results, we will perform the necessary calculations to provide foundation and retaining wall design criteria, recommended pavement thicknesses based on assumed traffic indices, and general grading and drainage recommendations.

Geotechnical Report

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

- Logs of exploratory trenches;
- Site plan showing approximate location of exploratory trenches;
- Description of site geology, and the soil and rock conditions encountered;
- Conclusions regarding the feasibility of the proposed improvements, including retaining structures, from a geotechnical standpoint; and
- Site class and seismic design criteria based on the 2007 CBC.

As required by the County of Placer's memorandum dated December 24, 2008, H&K's geotechnical report will address:

- Pavement section design for parking areas and driveways;
- Foundation and retaining wall design criteria;
- General recommendations for grading and erosion control;
- Observations regarding groundwater and expansive soil; and
- Conclusions regarding slope stability.

FEES

We performed a preliminary investigation at the site in 2007. This design-level investigation requires that we perform additional subsurface investigations and soil sampling, as the samples we collected in 2007 were discarded after one year. We are also required to address County requirements regarding slope stability, which was not included in our previous proposal.

Our services will be performed on a time and materials basis per the attached 2009 fee schedule. Our fee will not exceed $4,000 and may be less depending upon the time required to obtain the soil samples and perform stability analysis. Progress billing will be monthly based on the fee schedule.

TIMING

We will perform our field investigation within one week of receiving authorization to proceed, weather permitting. We can usually provide verbal, preliminary design recommendations within two weeks of our field investigation. We anticipate the geotechnical report would be issued within three weeks of the field investigation.

AUTHORIZATION TO PROCEED

If you are in agreement with the scope of services and fee presented in this proposal, please sign the attached agreement for geotechnical engineering services and return one copy along with a retainer in the amount of $2,000 as our authorization to proceed.

We appreciate the opportunity to provide you with this proposal. If you have any questions regarding this proposal, please feel free to contact our office.

Sincerely,

HOLDREGE & KULL

Jason W. Muir, P.E., G.E.
Principal Engineer

encl: Agreement for Geotechnical Engineering Services

copy: Rod Shearer via email, RShearer@shearerandassociates.com

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HOLDREGE & KULL
CONSULTING ENGINEERS • GEOLOGISTS

AGREEMENT FOR GEOTECHNICAL ENGINEERING SERVICES

THIS AGREEMENT, effective as of this 17 day of February 2009, is by and between ("Client") and Holdrege & Kull Consulting Engineers and Geologists ("Engineer").

THE PROJECT is generally described as: Amazing Facts and is located at Sierra College Boulevard and Nightwatch Drive, Placer County, California.

THIS AGREEMENT consists of the following documents which are incorporated herein by reference:

- GENERAL CONDITIONS FOR GEOTECHNICAL ENGINEERING SERVICES; and
- Engineer's PROPOSAL PN08237 dated February 17, 2009 and FEE SCHEDULE; and
- Any documents specifically listed below or incorporated by reference in the listed documents.

N/A
N/A
N/A
N/A

Engineer agrees to perform the Services set forth in this Agreement and in accordance with its terms, including all attachments incorporated herein by reference. This Agreement may not be modified or altered, except in writing as specifically described in this Agreement.

	CLIENT:	ENGINEER:
Signature:		
Print Name:		JASON W. MUIR, CE 60167
Title:		Principal
Company:		Holdrege & Kull
Street Address:		792 Searls Avenue
City, State, Zip Code:		Nevada City, CA 95959
Email:		jmuir@handk.net or rmack@handk.net
Phone:		530 478 1305
Fax:		530 478 1019
Date:		February 17, 2009

GENERAL CONDITIONS FOR GEOTECHNICAL ENGINEERING SERVICES

1. DEFINITIONS

1.1. Contract Documents. Plans, specifications, and agreements between Client and Contractors, including addenda, amendments, supplementary instructions, and change orders.

1.2. Contractor. The contractor or contractors, including its/their subcontractors of every tier, retained to construct the Project for which Engineer is providing Services under this Agreement.

1.3. Day(s). Calendar day(s) unless otherwise stated.

1.4. Hazardous Materials. The term Hazardous Materials means any toxic substances, chemicals, radioactivity, pollutants or other materials, in whatever form or state, known or suspected to impair the environment in any way whatsoever. Hazardous Materials include, but are not limited to, those substances defined, designated or listed in any federal, state or local law, regulation or ordinance concerning hazardous wastes, toxic substances or pollution.

1.5. Services. The Services provided by Engineer as set forth in this Agreement, the PROPOSAL and any written amendment to this Agreement.

1.6. Work. The labor, materials, equipment and services required to complete the work described in the Contract Documents.

2. SCOPE OF SERVICES

Engineer will perform the Services described in the attached PROPOSAL.

2.1. Changes in Scope. If Engineer provides Client with a written confirmation of a change in the scope of services outlined in the PROPOSAL, it will become an amendment to this Agreement unless Client objects in writing within 5 business days after receipt. All Services performed by Engineer on the Project are subject to the terms and limitations of this Agreement. If Services are performed, but the parties do not reach agreement concerning modifications to the scope of services outlined in the PROPOSAL or compensation, then the terms and limitations of this Agreement apply to such Services, except for the payment terms. The parties agree to resolve disputes concerning modifications to scope or compensation pursuant to Section 19, "Disputes."

2.2. Licenses. Engineer will procure and maintain business and professional licenses and registrations necessary to provide its Services.

2.3. Excluded Services. Engineer's Services under this Agreement include only those Services specified in the PROPOSAL.

2.3.1. General. Client expressly waives any claim against Engineer resulting from its failure to perform recommended additional Services that Client has not authorized Engineer to perform, and any claim that Engineer failed to perform services that Client instructs Engineer not to perform.

2.3.2. Biological Pollutants. Engineer's scope of services outlined in the PROPOSAL specifically excludes the investigation, detection, prevention or assessment of the presence of Biological Pollutants. The term "Biological Pollutants" includes, but is not limited to, molds, fungi, spores, bacteria, viruses, and/or any of their byproducts. Engineer's scope of services outlined in the PROPOSAL will not include any interpretations, recommendations or findings pertaining to Biological Pollutants. Client agrees that Engineer has no liability for any claims alleging a failure to investigate, detect, prevent, assess, or make recommendations for preventing, controlling, or abating Biological Pollutants. Furthermore, Client agrees to defend, indemnify, and hold harmless Engineer from all claims by any third party concerning Biological Pollutants, except for damages caused by Engineer's sole negligence.

3. PAYMENTS TO ENGINEER

3.1. Basic Services. Engineer will perform Services set forth in the attached PROPOSAL and FEE SCHEDULE (if applicable) for the amount(s) set forth therein.

3.2. Additional Services. Any Services performed under this Agreement, except those Services expressly identified in the attached PROPOSAL, will be provided on a time and materials basis unless otherwise specifically agreed to in writing by both parties.

3.3. Estimate of Fees. Engineer will, to the best of its ability, perform the Services and accomplish the objectives defined in this Agreement within any written cost estimate provided by Engineer. Client recognizes that changes in scope and schedule, and unforeseen circumstances can all influence the successful completion of Services within the estimated cost. The use of an estimate of fees or of a "not to exceed" limitation is not a guarantee that the Services will be completed for that amount; rather, it indicates that Engineer shall not incur fees and expenses in excess of the estimate or limitation amount without obtaining Client's agreement to do so.

3.4. Rates. Client will pay Engineer at the rates set forth in the PROPOSAL and FEE SCHEDULE, as applicable.

3.4.1. Changes to Rates. Client and Engineer agree that the FEE SCHEDULE is subject to periodic review and amendment, as appropriate to reflect Engineer's then-current fee structure. Engineer will give Client at least 30 days advance notice of any

changes. Unless Client objects in writing to the proposed amended fee structure within 30 days of notification, the amended fee structure will be incorporated into this Agreement and will then supersede any prior fee structure. If Client timely objects to the amended fee structure, and Engineer and Client cannot agree upon a new fee structure within 30 days after notice, Engineer may terminate this Agreement and be compensated as set forth under Section 18, "Termination."

3.4.2. Prevailing Wages. Unless Client specifically informs Engineer in writing that prevailing wage regulations cover the Project and the PROPOSAL identifies it as covered by such regulations, Client will reimburse, defend, indemnify and hold harmless Engineer from and against any liability resulting from a subsequent determination that prevailing wage regulations cover the Project, including all costs, fines and attorneys' fees.

3.5. Payment Timing; Late Charge. All invoices are due upon receipt. All amounts unpaid 30 days after the invoice date will include a late payment charge from the date of the invoice, at the rate of 1-1/2% per month or the highest rate permitted by law. Client will reimburse Engineer for any costs, including legal fees, associated with the collection of unpaid amounts.

4. STANDARD OF PERFORMANCE; DISCLAIMER OF WARRANTIES

4.1. Level of Service. Engineer offers different levels of geotechnical engineering Services to suit the desires and needs of different clients. Although the possibility of error can never be eliminated, more detailed and extensive Services yield more information and reduce the probability of error, but at increased cost. Client must determine the level of Services adequate for its purposes. Client has reviewed the PROPOSAL and has determined that it does not need or want a greater level of Services than that being provided.

4.2. Standard of Care. Subject to the limitations inherent in the agreed scope of services outlined in the PROPOSAL as to the degree of care, the amount of time and expenses to be incurred, and subject to any other limitations contained in this Agreement, Engineer may perform its Services consistent with that level of care and skill ordinarily exercised by other professional engineers practicing in the same locale and under similar circumstances at the time the Services are performed.

4.3. No Warranty. No warranty, express or implied, is included or intended by this Agreement.

5. ESTIMATE OF CONSTRUCTION COSTS

Client acknowledges that construction and Project development are subject to many influences that are not subject to precise forecasting and are outside of Engineer's control. Client further acknowledges that actual costs incurred may vary substantially from the estimates prepared by Engineer and that Engineer does not warrant or guaranty the accuracy of construction or development cost estimates.

6. CONSTRUCTION PHASE SERVICES

If Engineer's scope of services outlined in the PROPOSAL includes observation and/or testing during the course of construction, Engineer may:

6.1. Construction Observation.

6.1.1. Site Meetings & Visits. Engineer will participate in job site meetings as requested by Client, and, unless otherwise requested by Client, visit the site at times specified in the PROPOSAL or, if not specified in the PROPOSAL, at intervals as Engineer deems appropriate to the various stages of construction to observe the geotechnical conditions encountered by Contractor and the progress and quality of the geotechnical aspects of the Work. Engineer will rely on Client or Client's representative for timely notification of changes to the construction schedule, so that Engineer can schedule site visits for testing and observation accordingly. Based on information obtained during such visits and on such observations, Engineer may inform Client of the progress of the geotechnical aspects of the Work. Client understands that Engineer may not be on site continuously; and, unless expressly agreed otherwise, Engineer will not observe all of the Work.

6.1.2. Contractor's Performance. Engineer does not, and cannot, warrant or guarantee that all of the geotechnical Work performed by Contractor meets the requirements of Engineer's geotechnical recommendations or the plans and specifications for such geotechnical Work; nor can Engineer be responsible for Contractor's failure to perform the Work in accordance with the plans, specifications or the recommendations of Engineer.

6.1.3. Contractor's Responsibilities. Engineer will not supervise, direct or have control over the Work nor will Engineer have authority over or responsibility for the means, methods, techniques, sequences or procedures of construction selected by Contractor for the geotechnical aspects of the Project; for safety precautions and programs incident to the Work; nor for any failure of Contractor to comply with Laws and Regulations applicable to Contractor furnishing and performing its Work.

6.1.4. Final Report. At the conclusion of Construction Phase Services, Engineer will provide Client with a written report summarizing the tests and observations, if any, made by Engineer.

6.2. Review of Contractor's Submittals. If included in the scope of services outlined in the PROPOSAL, Engineer will review and take appropriate action on the Contractor's submittals, such as shop drawings, product data, samples, and other required submittals. Engineer will review such submittals solely for general conformance with Engineer's design, and will not

include review for the following, all of which will remain the responsibility of the Contractor: accuracy or completeness of details, quantities or dimensions; construction means, methods, sequences or procedures; coordination among trades; or construction safety.

6.3. Tests. Tests performed by Engineer on finished Work or Work in progress are taken intermittently and indicate the general acceptability of the Work on a statistical basis. Engineer's tests and observations of the Work are not a guarantee of the quality of Work and do not relieve other parties from their responsibility to perform their Work in accordance with applicable plans, specifications and requirements.

7. CLIENT'S RESPONSIBILITIES

In addition to payment for the Services performed under this Agreement, Client agrees to:

7.1. Cooperation. Assist and cooperate with Engineer in any manner necessary and within its ability to facilitate Engineer's performance under this Agreement.

7.2. Representative. Designate a representative with authority to receive all notices and information pertaining to this Agreement, communicate Client's policies and decisions, and assist as necessary in matters pertaining to the Project and this Agreement. Client's representative will be subject to change by written notice.

7.3. Rights of Entry. Provide access to and/or obtain permission for Engineer to enter upon all property, whether or not owned by Client, as required to perform and complete the Services. Engineer will operate with reasonable care to reduce damage to the Project Site(s). However, Client recognizes that Engineer's operations and the use of investigative equipment may unavoidably alter conditions or affect the environment at the existing Project Site(s). The cost of repairing such damage will be borne by Client and is not included in the fee unless otherwise stated.

7.4. Relevant Information. Supply Engineer with all information and documents in Client's possession or knowledge which are relevant to Engineer's Services. Client warrants the accuracy of any information supplied by it to Engineer, and acknowledges that Engineer is entitled to rely upon such information without verifying its accuracy. Prior to the commencement of any Services in connection with a specific property, Client will notify Engineer of any known potential or possible health or safety hazard existing on or near the Project Site, with particular reference to Hazardous Materials or conditions.

7.5. Subsurface Structures. Correctly designate on plans to be furnished to Engineer, the location of all subsurface structures, such as pipes, tanks, cables and utilities within the property lines of the Project Site(s), and be responsible for any damage inadvertently caused by Engineer to any such structure or utility not so designated. Engineer is not liable to Client for any losses, damages or claims arising from damage to subterranean structures or utilities that were not correctly shown on plans furnished by Client to Engineer.

8. CHANGED CONDITIONS

If Engineer discovers conditions or circumstances that it had not contemplated at the commencement of this Agreement ("Changed Conditions"), Engineer will notify Client of the Changed Conditions. Client and Engineer agree to that they will then renegotiate in good faith the terms and conditions of this Agreement. If Engineer and Client cannot agree upon amended terms and conditions within 30 days after notice, Engineer may terminate this Agreement and be compensated as set forth in Section 18, "Termination."

9. HAZARDOUS MATERIALS

Client understands that Engineer's Services under this Agreement are limited to geotechnical engineering and that Engineer has no responsibility to locate, identify, evaluate, treat or otherwise consider or deal with Hazardous Materials. Client is solely responsible for notifying all appropriate federal, state, municipal or other governmental agencies, including the potentially affected public, of the existence of any Hazardous Materials located on or in the Project site, or located during the performance of this Agreement. The existence or discovery of Hazardous Materials constitutes a Changed Condition under this Agreement.

10. CERTIFICATIONS

Client agrees not to require that Engineer execute any certification with regard to Services performed or Work tested and/or observed under this Agreement unless: 1) Engineer believes that it has performed sufficient Services to provide a sufficient basis to issue the certification; 2) Engineer believes that the Services performed or Work tested and/or observed meet the criteria of the certification; and 3) Engineer has reviewed and approved in writing the exact form of such certification prior to execution of this Agreement. Any certification by Engineer is limited to an expression of professional opinion based upon the Services performed by Engineer, and does not constitute a warranty or guaranty, either expressed or implied.

11. ALLOCATION OF RISK

11.1. Limitation of Remedies. The total cumulative liability of Engineer, its subEngineers and subcontractors, and all of their respective shareholders, directors, officers, employees and agents (collectively "Engineer Entities"), to Client arising from Services under this Agreement, including attorney's fees due under this Agreement, will not exceed the gross compensation received by Engineer under this Agreement or $50,000, whichever is

greater; provided, however, that such liability is further limited as described below. This limitation applies to all lawsuits, claims or actions that allege errors or omissions in Engineer's Services, whether alleged to arise in tort, contract, warranty, or other legal theory. Upon Client's written request, Engineer and Client may agree to increase the limitation to a greater amount in exchange for a negotiated increase in Engineer's fee, provided that they amend this Agreement in writing as provided in Section 20.

11.2. Indemnification.

11.2.1. Indemnification of Client. Subject to the provisions and limitations of this Agreement and all otherwise applicable statutes of limitations and repose, Engineer agrees to indemnify and hold harmless Client, its shareholders, officers, directors and employees from and against any and all claims, suits, liabilities, damages, expenses (including reasonable attorney's fees and costs of defense), or other losses (collectively "Losses") to the extent caused by Engineer's negligent performance of its Services under this Agreement.

11.2.2. Indemnification of Engineer. Client will indemnify and hold harmless Engineer Entities from and against any and all Losses to the extent caused by the negligence of Client, its employees, agents and contractors. In addition, except to the extent caused by Engineer's sole negligence, Client expressly agrees to defend, indemnify and hold harmless Engineer Entities from and against any and all Losses arising from or related to the existence, disposal, release, discharge, treatment or transportation of Hazardous Materials, or the exposure of any person to Hazardous Materials, or the degradation of the environment due to the presence, discharge, disposal, release of or exposure to Hazardous Material.

11.3. Consequential Damages. Neither Client nor Engineer will be liable to the other for any special, consequential, incidental or penal losses or damages including but not limited to losses, damages or claims related to the unavailability of property or facilities, shutdowns or service interruptions, loss of use, profits, revenue, or inventory, or for use charges, cost of capital, or claims of the other party and/or its customers.

11.4. ***No Personal Liability.*** Client expressly waives that right to sue or otherwise make any claim against any of the Engineer's officers or employees, past or present, as individuals, for any cause.

11.5. Continuing Agreement. The indemnity obligations and the limitations of liability established under this Agreement will survive the expiration or termination of this Agreement. If Engineer provides Services to Client that the parties do not confirm through execution of an amendment to this Agreement, the obligations of the parties to indemnify each other and the limitations on liability established under this Agreement apply to such Services as if the parties had executed an amendment.

12. INSURANCE

12.1. Engineer's Insurance. Engineer will obtain, if reasonably available, the following coverage:

12.1.1. Statutory Workers' Compensation/ Employer's Liability Insurance;

12.1.2. Commercial General Liability Insurance with a limit of not less than $1,000,000 per occurrence and $1,000,000 aggregate limit;

12.1.3. Automobile Liability Insurance, including liability for all owned, hired and non-owned vehicles with a combined single limit per occurrence of $1,000,000; and

12.1.4. Professional Liability Insurance in amounts of $1,000,000 per claim and annual aggregate.

12.2. Contractor's Insurance. Client shall require that all Contractors and subcontractors for the Project name Engineer as an additional insured under their General Liability and Automobile Liability insurance policies. If Client is not the Project owner, Client will require the Project owner to require the owner's Contractor to purchase and maintain General Liability, Builder's Risk, Automobile Liability, Workers' Compensation, and Employer's Liability insurance with limits no less than as set forth above, and to name Engineer and its subcontractors and subconsultants as additional insureds on the owners' General Liability insurance.

12.3. Certificates of Insurance. Upon request, Engineer and Client will each provide the other with certificate(s) of insurance evidencing the existence of the policies required herein. Except for Professional Liability and Workers' Compensation Insurance, all policies required herein shall contain a waiver of subrogation.

13. OWNERSHIP AND USE OF DOCUMENTS

13.1. Client Documents. All documents provided by Client will remain the property of Client. Engineer will return all such documents to Client upon request, but may retain file copies of such documents.

13.2. Engineer's Documents. Unless otherwise agreed in writing, all documents and information prepared by Engineer or obtained by Engineer from any third party in connection with the performance of Services, including, but not limited to, Engineer's reports, boring logs, maps, field data, field notes, drawings and specifications, laboratory test data and other similar documents (collectively "Documents") are the property of Engineer. Engineer has the right, in its sole discretion, to dispose of or retain the Documents.

13.3. Use of Documents. All Documents prepared by Engineer are solely for use by Client.

13.3.1. Use by Client. Client has the right to reuse the Documents for purposes reasonably connected with the Project for which the Services are

provided, including without limitation design and licensing requirements of the Project.

13.3.2. Use by Engineer. Engineer retains the right of ownership with respect to any patentable concepts or copyrightable materials arising from its Services and the right to use the Documents for any purpose.

13.4. Electronic Media. Engineer may agree at Client's request to provide Documents and information in an electronic format. Client recognizes that Documents or other information recorded on or transmitted as electronic media are subject to undetectable alteration due to (among other causes) transmission, conversion, media degradation, software error, or human alteration. Accordingly, all Documents and information provided by Engineer in electronic media are for informational purposes only and not as final documentation. Unless otherwise defined in the PROPOSAL, Engineer's electronic Documents and media will conform to Engineer's standards. Engineer will provide any requested electronic Documents for a 30-day acceptance period, and Engineer will correct any defects reported by Client to Engineer during this period. Engineer makes no warranties, either express or implied, regarding the fitness or suitability of any electronic Documents or media.

13.5. Unauthorized Reuse. No party other than Client may rely, and Client will not represent to any other party that it may rely on Documents without Engineer's express prior written consent and receipt of additional compensation. Client will not permit disclosure, mention, or communication of, or reference to the Documents in any offering circular, securities offering, loan application, real estate sales documentation, or similar promotional material without Engineer's express prior written consent. Client waives any and all claims against Engineer resulting in any way from the unauthorized reuse or alteration of Documents by itself or anyone obtaining them through Client. Client will defend, indemnify and hold harmless Engineer from and against any claim, action or proceeding brought by any party (including reasonable attorneys fees, expert fees and other costs of defense) arising out of the reuse, alteration, or reliance on the Documents or information or opinions contained in Documents without having obtained Engineer's prior written consent.

14. SAMPLES AND CUTTINGS

14.1. Sample Retention. If Engineer provides laboratory testing or analytic Services, Engineer will preserve such soil, rock, water, or other samples as it deems necessary for the Project, but no longer than 45 days after issuance of any Documents that include the data obtained from these samples. Client will promptly pay and be responsible for the removal and lawful disposal of all contaminated samples, cuttings, Hazardous Materials, and other hazardous substances.

14.2. Monitoring Wells. Client will take custody of all monitoring wells and probes installed during any investigation by Engineer, and will take any and all necessary steps for the proper maintenance, repair or closure of such wells or probes at Client's expense.

15. RELATIONSHIP OF THE PARTIES

Engineer will perform Services under this Agreement as an independent contractor.

16. CONSENT TO ASSIGNMENT

Client and Engineer, respectively, each binds itself and its successors and assigns to the other and its successors and assigns with respect to all covenants of this Agreement. Neither Client nor Engineer shall assign, sublet or transfer any rights under or interest in this Agreement without the prior written consent of the other party, including but not limited to: (a) any interest in the proceeds of this Agreement, or any proceeds of claims arising from or under this Agreement; (b) any claims, causes of action or rights against the other party arising from or under this Agreement; (c) the control of claims or causes of action against the other party arising from or under this Agreement; and (d) any proceeds from claims or causes of action as security, collateral or the source of payment for any notes or liabilities to any third party. This section shall not, however, apply to any subrogation rights (if any) of any insurer of either party. This section shall survive the completion or termination of this Agreement for any reason and shall remain enforceable between parties.

Engineer may subcontract for the services of others without obtaining Client's consent if Engineer deems it necessary or desirable for others to perform certain Services.

17. SUSPENSION AND DELAYS

17.1. Procedures. Client may, at any time by 10 days written notice suspend performance of all or any part of the Services by Engineer. Engineer may terminate this Agreement if Client suspends Engineer's Services for more than 60 days and Client will pay Engineer as set forth under Section 18, "Termination." If Client suspends Engineer's Services, or if Client or others delay Engineer's Services, Client and Engineer agree to equitably adjust: (1) the time for completion of the Services; and (2) Engineer's compensation in accordance with Engineer's then current Fee Schedule for the additional labor, equipment, and other charges associated with maintaining its workforce for Client's benefit during the delay or suspension, or charges incurred by Engineer for demobilization and subsequent remobilization.

17.2. Liability. Engineer is not liable to Client for any failure to perform or delay in performance due to circumstances beyond Engineer's control, including but not limited to pollution, contamination, or release of hazardous substances, strikes, lockouts, riots, wars, fires, flood, explosion, "acts of God," adverse weather

conditions, acts of government, labor disputes, delays in transportation or inability to obtain material and equipment in the open market.

18. TERMINATION

18.1. Termination for Convenience. Engineer and Client may terminate this Agreement for convenience upon 30 days written notice delivered or mailed to the other party.

18.2. Termination for Cause. In the event of material breach of this Agreement, the party not breaching the Agreement may terminate it upon 10 days written notice delivered or mailed to the other party. The termination notice shall state the basis for the termination. The Agreement may not be terminated for cause if the breaching party cures the breach within the 10-day period.

18.3. Payment on Termination. Following termination other than for Engineer's material breach of this Agreement, Client will pay Engineer for Services performed prior to the termination notice date, and for any necessary Services and expenses incurred in connection with the termination of the Project, including but not limited to, the costs of completing analysis, records and reports necessary to document job status at the time of termination and costs associated with termination of subcontractor contracts in accordance with Engineer's then current Fee Schedule.

19. DISPUTES

19.1. Mediation. All disputes between Engineer and Client are subject to mediation. Either party may demand mediation by serving a written notice stating the essential nature of the dispute, amount of time or money claimed, and requiring that the matter be mediated within 45 days of service of notice.

19.2. Precondition to Other Action. No action or suit may be commenced unless the mediation did not occur within 45 days after service of notice; or the mediation occurred but did not resolve the dispute; or a statute of limitation would elapse if suit was not filed prior to 45 days after service of notice.

19.3. Choice of Law; Venue. This Agreement will be construed in accordance with and governed by the laws of the state in which the Project is located. Unless the parties agree otherwise, any mediation or other legal proceeding will occur in the state in which the Project is located.

19.4. Statutes of Limitations. Any applicable statute of limitations will be deemed to commence running on the earlier of the date of substantial completion of Engineer's Services under this Agreement or the date on which claimant knew, or should have known, of facts giving rise to its claims.

20. MISCELLANEOUS

20.1. Integration and Severability. This Agreement reflects the entire agreement of the parties with respect to its terms and supersedes all prior agreements, whether written or oral. If any portion of this Agreement is void or voidable, such portion will be deemed stricken and the Agreement reformed to as closely approximate the stricken portions as the law allows.

20.2. Modification of this Agreement. This Agreement may not be modified or altered, except by a written agreement signed by authorized representatives of both parties and referring specifically to this Agreement.

20.3. Notices. Any and all notices, requests, instructions, or other communications given by either party to the other must be in writing and either hand delivered to the recipient, or delivered by first-class mail (postage prepaid), or express mail (billed to sender), by fax, or by email, at the addresses given in this Agreement.

20.4. Headings. The headings used in this Agreement are for convenience only and are not a part of this Agreement.

20.5. Waiver. The waiver of any term, conditions or breach of this Agreement will not operate as a subsequent waiver of the same term, condition, or breach.

End of General Conditions

APPENDIX B ***IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL ENGINEERING REPORT (Included with permission of ASFE, Copyright 2004)***

Important Information About Your

Geotechnical Engineering Report

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

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

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

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

Read the Full Report

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

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

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

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

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

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

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

Subsurface Conditions Can Change

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

Most Geotechnical Findings Are Professional Opinions

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

A Report's Recommendations Are *Not* Final

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

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

A Geotechnical Engineering Report Is Subject to Misinterpretation

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

Do Not Redraw the Engineer's Logs

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

Give Contractors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

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

Geoenvironmental Concerns Are Not Covered

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

Obtain Professional Assistance To Deal with Mold

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

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

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

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APPENDIX C EXPLORATORY TRENCH LOGS

TRENCH T-1

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	TRENCH NO.
3307A-02	AMAZING FACTS SITE	526 FEET MSL	02-27-09	1 OF 1	T-1

EXCAVATION METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
KUBOTA KX121-3 WITH 18-INCH BUCKET	NONE	NONE	NONE

SAMPLE NO.	POCKET PEN (tsf)	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
					GW	REDDISH BROWN (2.5YR 4/4), SILTY GRAVEL WITH SAND, MOIST, LOOSE, NON-PLASTIC, FINE ROOTS TO 2 INCHES BGS
				1		TRENCH TERMINATED AT 12 INCHES BGS, REFUSAL ON RESISTANT ROCK
				2		
				3		
				4		
				5		
				6		
				7		
				8		
				9		
				10		
				11		
				12		
				13		
				14		
				15		
				16		
				17		
				18		
				19		
				20		

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TRENCH T-2

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	TRENCH NO.
3307A-02	AMAZING FACTS SITE	520 FEET MSL	02-27-09	1 OF 1	T-2

EXCAVATION METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
KUBOTA KX121-3 WITH 18-INCH BUCKET	NONE	NONE	NONE

SAMPLE NO.	POCKET PEN (tsf)	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
					GW	REDDISH BROWN (2.5YR 4/4), SILTY GRAVEL WITH SAND, MOIST, LOOSE, NON-PLASTIC
				1		TRENCH TERMINATED AT 10 INCHES BGS, REFUSAL ON RESISTANT ROCK
				2		
				3		
				4		
				5		
				6		
				7		
				8		
				9		
				10		
				11		
				12		
				13		
				14		
				15		
				16		
				17		
				18		
				19		
				20		

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TRENCH T-3

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	TRENCH NO.
3307A-02	AMAZING FACTS SITE	517.5 FEET MSL	02-27-09	1 OF 1	T-3

EXCAVATION METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
KUBOTA KX121-3 WITH 18-INCH BUCKET	NONE	NONE	NONE

SAMPLE NO.	POCKET PEN (tsf)	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
					GW	REDDISH BROWN (2.5YR 4/4), SILTY GRAVEL WITH SAND, MOIST, LOOSE, NON-PLASTIC, FINE ROOTS TO 3 INCHES BGS
				1		TRENCH TERMINATED AT 8 INCHES BGS, REFUSAL ON RESISTANT ROCK
				2		
				3		
				4		
				5		
				6		
				7		
				8		
				9		
				10		
				11		
				12		
				13		
				14		
				15		
				16		
				17		
				18		
				19		
				20		

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TRENCH T-4

PROJECT NO.	PROJECT NAME	ELEVATION	DATE	PAGE	TRENCH NO.
3307A-02	AMAZING FACTS SITE	522.5 FEET MSL	02-27-09	1 OF 1	T-4

EXCAVATION METHOD	SAMPLING METHOD	GROUNDWATER ENCOUNTERED	CAVED
KUBOTA KX121-3 WITH 18-INCH BUCKET	NONE	NONE	NONE

SAMPLE NO.	POCKET PEN (tsf)	DRY DENSITY (PCF)	PERCENT MOISTURE	DEPTH (FT)	USCS	DESCRIPTIONS/REMARKS
				1	GW	REDDISH BROWN (2.5YR 4/4), SILTY GRAVEL WITH SAND, MOIST, LOOSE, NON-PLASTIC, FINE ROOTS TO 2 INCHES BGS
				2		TRENCH TERMINATED AT 18 INCHES BGS, REFUSAL ON RESISTANT ROCK
				3		
				4		
				5		
				6		
				7		
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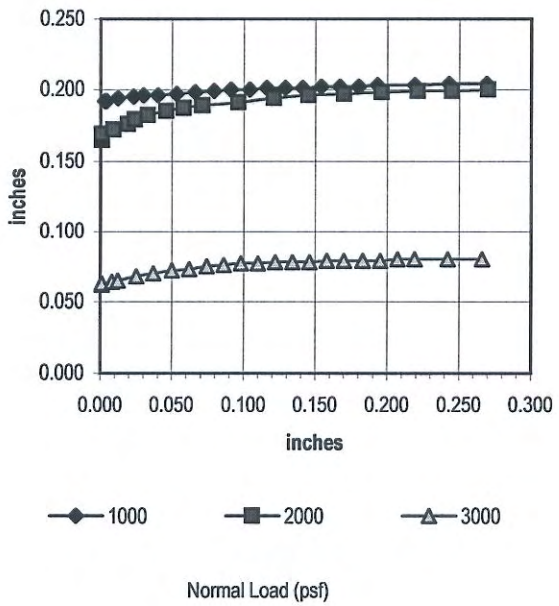
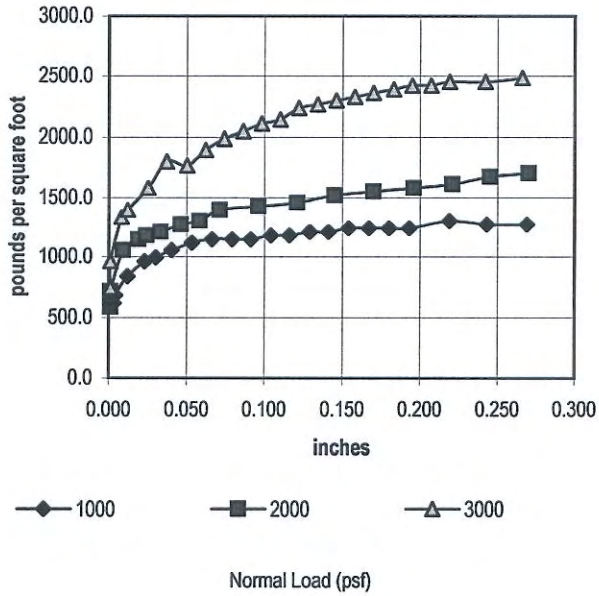
APPENDIX D LABORATORY TEST DATA

DIRECT SHEAR
TEST RESULTS

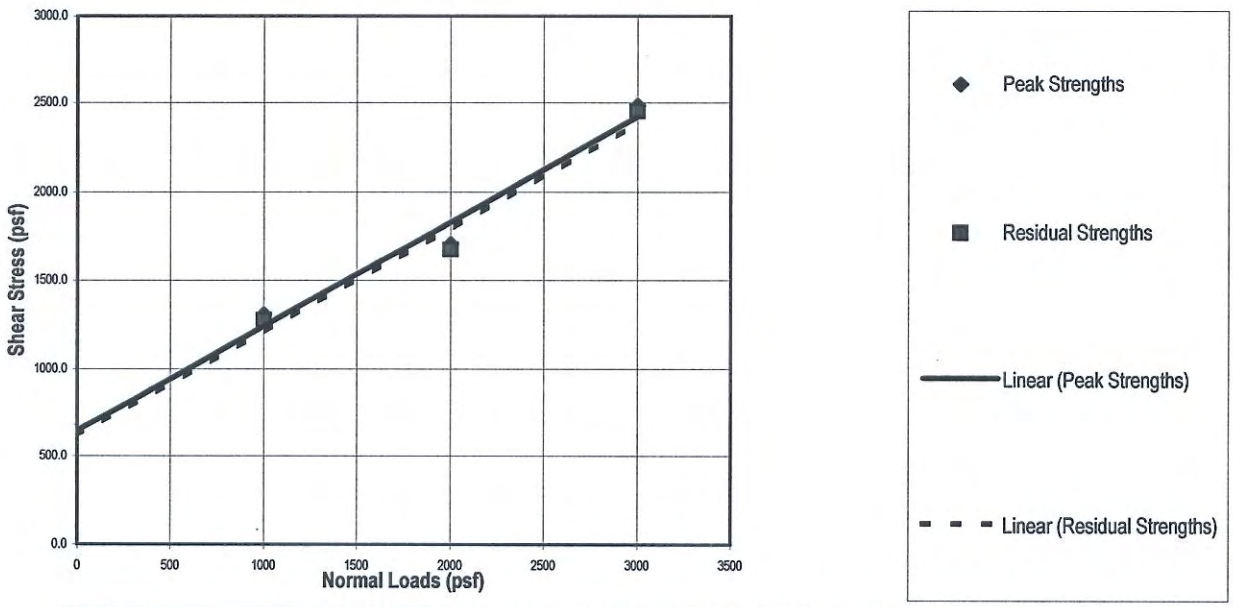
Shear Strain vs. Normal Strain



Shear Strain vs. Shear Stress



Mohr-Coulomb Failure Envelope



SHEAR STRENGTH TEST RESULTS		
PARAMETERS	PEAK STRENGTH:	RESIDUAL STRENGTH:
FRICTION ANGLE, (Degree)	30.5	30.5
COHESION, (psf)	652.0	621.0

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CONSULTING ENGINEERS • GEOLOGISTS
792 SEARLS AVENUE
NEVADA CITY, CA 95959
(530) 478-1305 FAX 478-1019

PROJECT NAME:	Amazing Facts		
PROJECT NO.:	3307A-02	DATE:	3/18/2009
BORING / TRENCH NO.:	4	LAB NO.:	9-028
SAMPLE NO.:	PB4-1	SAMPLE DEPTH (ft.):	0-1.5
DESCRIPTION:	Reddish Brown (2.5YR 4/4) Silty Gravel with Sand		

Expansion Index/Swell

ASTM D4829/UBC 18.2

Project No.:	3307A-02	Project Name:	Amazing Facts			Date:	3/27/2009
Sample No.:	PB2-1	Boring/Trench No.:	2	Depth (ft.)	0-10"	Tested By:	MLHF
Soil Description:	Reddish Brown (2.5YR 4/4) Silty Gravel with Sand					Checked By:	JHA
Estimated % of sample retained on #4:						Lab. No.:	15-09-39

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

FIELD DATA Tube Sample Moisture & Density			LAB DATA	Test wt.	144	Test wt.		Test wt.	
				Initial	Final	Initial	Final	Initial	Final
Tare Tube Number		TI	Tare Number	0.00					
Tare Weight	(gr)	51.05	Tare Ring Weight (gr)	200.93	0.00				
Wet Soil + Tare	(gr)	132.77	Tare Pan Weight (gr)	0.00	279.93				
Dry Soil + Tare	(gr)	125.02	Wet Soil + Tare (gr)	587.13	705.28				
Weight of Water	(gr)	7.75	Dry Soil + Tare (gr)	550.50	629.50	0.00		0.00	
Dry Soil Weight	(gr)	73.97	Weight of Water (gr)	36.63	75.78	0.00	0.00	0.00	0.00
Moisture Content	(%)	10.48	Dry Soil Weight (gr)	349.57	349.57	0.00	0.00	0.00	0.00
Soil Height	(In.)		Moisture Content (%)	10.48	21.68	0.00	0.00	0.00	0.00
Wet Unit Weight	(pcf)	#REF!	Wet Unit Weight (pcf)	117.09	127.58				
Dry Unit Weight	(pcf)	#REF!	Dry Unit Weight (pcf)	105.99	104.85				
			Sample Height (Inches)	1.00	1.011				
Specific Gravity		2.7	Percent Saturation	47.98	96.46				

Expansion Index Number

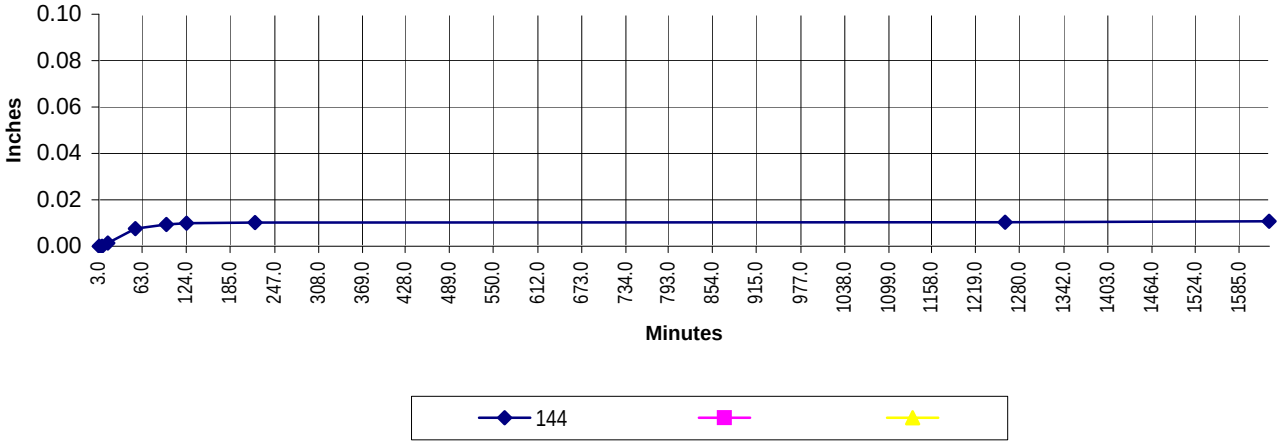
Surcharge (psf)	Uncorrected	Corrected to 50% Saturation
Test wt. 144	11	10
Test wt.		
Test wt.		

Elapsed Time (m:s)	Change in Height (Inches)	Elapsed Time (m:s)	Change in Height (Inches)	Elapsed Time (m:s)	Change in Height (Inches)
3.0	0.0000				
7.0	0.0000				
15.0	0.0014				
53.0	0.0076				
96.0	0.0094				
124.0	0.0100				
219.0	0.0102				
1260.0	0.0104				
1626.0	0.0108				

Expansion Index Values and Descriptions

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

Expansion Versus Time



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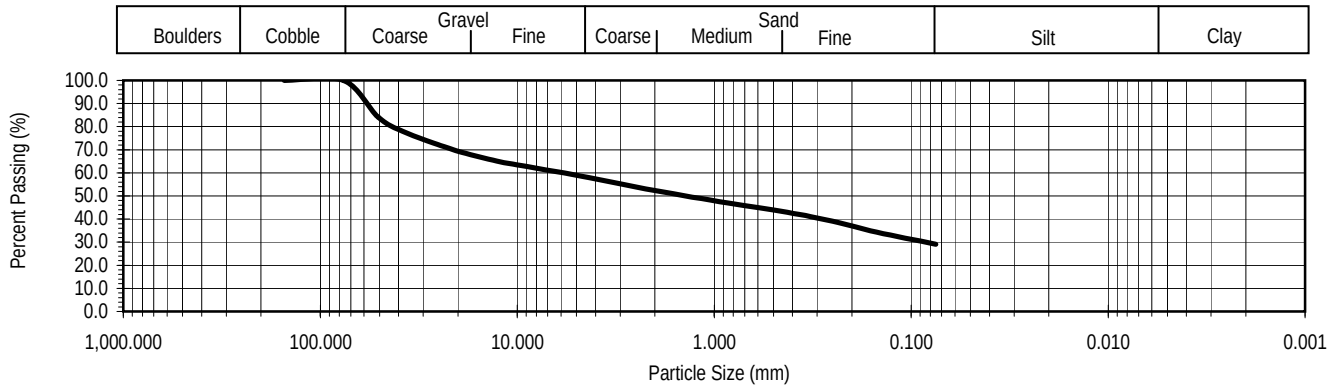
Particle Size Distribution

ASTM D422

Project No.:	**3307A-02**	Project Name:	**Amazing Facts**	Date:	**3/18/2009**
Sample No.:	**PB2-1**	Boring/Trench:	**2**	Depth, (ft.): **0-10"**	Tested By: **MLHF**
Description:	**Reddish Brown (2.5YR 4/4) Silty Gravel with Sand**			Checked By:	**JHA**
Sample Location:	**contains minor organics**			Lab. No.:	**9-028**

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	2,805.6	100.0
3 Inch	3.0000	76.2	0.00	0.0	2,805.6	100.0
2 Inch	2.0000	50.8	448.32	448.3	2,357.2	84.0
1.5 Inch	1.5000	38.1	169.01	617.3	2,188.2	78.0
1.0 Inch	1.0000	25.4	159.16	776.5	2,029.1	72.3
3/4 Inch	0.7500	19.1	95.88	872.4	1,933.2	68.9
1/2 Inch	0.5000	12.7	108.72	981.1	1,824.5	65.0
3/8 Inch	0.3750	9.5	53.76	1,034.9	1,770.7	63.1
#4	0.1870	4.7500	126.29	1,161.1	1,644.4	58.6
#10	0.0787	2.0000	177.32	1,338.5	1,467.1	52.3
#20	0.0335	0.8500	149.66	1,488.1	1,317.4	47.0
#40	0.0167	0.4250	114.24	1,602.4	1,203.2	42.9
#60	0.0098	0.2500	107.55	1,709.9	1,095.7	39.1
#100	0.0059	0.1500	131.02	1,840.9	964.6	34.4
#200	0.0030	0.0750	149.32	1,990.2	815.3	29.1
Hydrometer						

Particle Size Gradation



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PLANNING DEPT.

ENGINEERING GEOLOGIC EVALUATION
PROPOSED RESIDENTIAL SUBDIVISION
GRANITE BAY, CALIFORNIA

May 23, 2003

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An employee owned company

May 23, 2003
File: 29944

Mr. Larry Kelley
Sierra College Partners
3140 Peacekeeper Way
McClellan, CA 95652

Subject: Engineering Geologic Evaluation
Proposed Residential Subdivision
Granite Bay, California

Dear Mr. Kelley:

As you requested, we have performed a preliminary engineering geologic evaluation of the proposed subdivision along Sierra College Boulevard in Granite Bay, California. Our study has generally concluded that the site is developable for the intended use and will require attention to the following geologic, soils, and/or seismic issues during planning and design.

- Expansive Soils – Are typical as soils on Mehrten volcanics and may require removal and replacement with more granular soils or at-grade improvements to resist potential heave.
- Perched Groundwater – Is expected to be persistent particularly during winter/spring months and may require installation of subdrain systems and possible dewatering during construction.
- Excvatability – May be difficult in both Mehrten Formation in upper portions of the site and granitic rock in the lower portions of the site. Both of these units can be expected to require additional effort during earthwork and especially during trenching.
- Removals – Will be necessary where surficial soils (soil, colluvium and alluvium) are present within the areas of proposed development. These soils can be replaced as engineered fill once dry enough for compaction.

We sincerely appreciate the opportunity to be of service to you. Should you have any questions or require further information, please do not hesitate to contact either of the undersigned at your convenience.

Respectfully,

KLEINFELDER, INC.

Bruce R. Hilton R.G., C.E.G. 2212
Senior Engineering Geologist

Timothy A. Williams, P.E., G.E. 2404
Senior Geotechnical Engineer

BRH:TAW:sev

Distribution: (6) Addressee

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ES

- Site Location Map – USGS topo w/ site design
- Regional Faulting and Historic Seismicity
- Site Geologic Map – Above geologic contacts shown on site plan/photo

1 INTRODUCTION

As requested, Kleinfelder has performed a geologic and geotechnical review of site conditions relative to the proposed development. Our study has entailed a review of existing literature and maps in conjunction with review of aerial photographs and reconnaissance-scale geologic mapping. The purpose of our study was to support the environmental application process and provide geologic, geotechnical, and seismic input to the land-use planning and design process. The following report summarizes site conditions based on this study followed by a discussion of anticipated planning/design issues and recommendations for future studies and mitigation.

1.1. PLANNED DEVELOPMENT

Our evaluation has been performed on the basis of discussions with you, the project civil engineer, Spannagel & Associates, and the provided site plan. It is our understanding that the proposed project will consist of subdividing approximately 77 ± acres of hillside terrain into 16 residential, 2 to 5 acre custom lots with infrastructure including roads, utilities, and building pad access ramps. This proposed subdivision plan is provided as Plate 3, herein.

2 SITE CONDITIONS

The site is located along the south side of Sierra College Boulevard and to the north of Cavitt and Stallman Road in the northern portion of the community of Granite Bay, Placer County, California. Topographically, the upper northern portion of the site is relatively flat terrace with the central part of the site descending from the upper terrace at a moderate gradient and the lower southern portion of the site occupied by a gentle valley. The site and surrounding topography is shown on Plate 1, herein. Two ponds exist onsite including a smaller one in the north-east corner and another in the lower south-central part of the site. The upper terrace occurs at approximately elevation 520 feet MSL (above mean sea level). The site descends to the lower portion of the valley at a lowest elevation of about 300 feet, giving the site an approximate total relief of 220 feet vertically. Although localized areas of rock outcrops achieve steeper slopes, the average slope of the central portion of the site is approximately 3 ½ (horizontal to vertical ratio). Vegetation consists of annual grasses across the majority of the site with scattered oak trees and shrubs. Presently, the site is unoccupied but has several manmade features including pipelines, pump houses, and earthen-impounded ponds to retain water for local, downstream use.

2.1. GEOLOGIC EARTH UNITS

Geologic mapping has been performed at various scales at the site by Jennings (1977), Wagner (1981), and Loyd (1984). Jennings' statewide mapping is less accurate but identifies the site along the boundary between the Rocklin Penryn Pluton (to the east) and Plio-Pleistocene continental deposits (to the west) and outcrops of Tertiary volcanics and Eocene continental deposits as isolated occurrences along this main contact. At a more accurate scale, Wagner shows the site to expose Mehrten Formation volcanic conglomerate, sandstone, and breccia bedrock underlain by Mesozoic granitics of the Rocklin-Penryn Pluton. The most detailed and current geologic mapping onsite is provided by Loyd and shows the northern mesa or terrace portion of the site to be Mehrten Formation forming cliffs along it's boundary to the south where it is underlain by Mesozoic granitic bedrock. In all cases, these maps generally agree with one

We also reviewed aerial photographs and performed independent reconnaissance-scale geologic mapping at the site. Air photos show the vegetation contrast that occurs along the mapped contact between the upper Mehrten Formation volcanic rock and the underlying granitics. Vegetation along the contact consists of moderately dense shrubs and oak trees whereas the areas underlain by Mehrten Formation or granitic bedrock are primarily occupied by grasses. In addition to the primary contact between the Mehrten Formation and the underlying granitic bedrock, we also observed the contact between the upper breccia unit and the underlying conglomeratic unit within the Mehrten Formation. This contact may be significant to future assessment of excavatability as the conglomeratic unit typically has a weaker matrix and is therefore more excavatable than the breccia. Our interpretation of the generalized distribution of geologic units onsite is shown on Plate 3.

2.2. SOIL UNITS

Four general soil units are mapped by the USDA (1975) each corresponding well with the aforementioned geologic units. The northern, upper portion of the site is mapped as Exchequer and Inks soil series both described as stony soils developed on underlying hard, andesitic breccia. The Exchequer is described as hard volcanic rock while the Inks is more conglomeratic with cobbles. These two units probably correspond to the typical occurrence of hard Mehrten caprock in the extreme north underlain by the conglomeratic unit of the Mehrten forming the steeper upper portions of slopes. The central portion of the site is mapped as Andregg series described as well-drained soil developed over weathered granitic bedrock. The lower, southern part of the site is mapped as Xerofluvents soil series consisting of sandy and stoney soils associated with recent alluvium in or adjacent to drainage channels. Table 1 summarizes the engineering characteristics of these soil series along with their correlative geologic unit.

Table 1. USDA Soil Units

Soil Series	Corresponding Geologic Formation	Engineering Characteristics
Exchequer	Soil developed on Mehrten Formation-Breccia	ML, non plastic to PI = 10, erosion factor (T) = 1, pH of 5.6-6.5, and permeability (in/hr) of 0.6-2.0
Inks	Soil developed on Mehrten Formation-Conglomerate	GM-SC, PI = 10-20, erosion factor (T) = 1, pH of 5.6-6.5, and permeability (in/hr) of 0.6-2.0
Andregg	Soil developed on granitic bedrock	SM, non plastic, erosion factor (T) = 2, pH of 5.6-7.3, and permeability (in/hr) of 2.0-6.0
Xerofluvents	Recent alluvium	SP-SM, non plastic, erosion factor (T) = 5, pH of 6.1-7.8, and permeability (in/hr) of 2.0-20.0

2.3. FAULTING AND HISTORIC SEISMICITY

The subject site is situated in the eastern portion of the greater Sacramento metropolitan where historic seismicity is relatively moderate when compared to other regions of Cali This reduced seismic activity is largely due to the absence of nearby active or major source that generate large earthquakes. The nearest active (i.e. <10,000 years old; CGS, 2003) or faults in this part of the State include (from west to east) the San Andreas and other relate Area Faults (minimum distance 122 km to the west), Green Valley fault (89 km west) thrust Coast Range-Central Valley geomorphic block boundary (56 km west), Foothills System 16.5 km east), and the Sierra Nevada Frontal Fault System (106 km east). No A Priolo Earthquake Fault Zones (CGS, 2003) are mapped on, near, or projecting towa subject site. Although the Foothills Fault System (FFS) is relatively close to the site, h earthquake activity along this fault is relatively infrequent with small magnitude events an low slip rate value of 0.05 mm/year. These fault systems (Jennings, 1994) and h seismicity (Toppozada, 2000) are shown on Plate 2 relative to the site location.

The Willows blind fault is also mapped approximately 18 km west of the site. This blind fault occurs at a depth of more than 1,000 feet beneath the ground surface and is recognizable by deep well logs and groundwater elevation changes and does have historic micro-seismicity associated with it. Although some doming and other forms of ground surface warping may be active north of the Sutter Buttes (73 km north), it is not considered capable of generating large earthquakes and is not considered active with regard to ground surface rupture potential.

2.4. GROUNDWATER

Regional groundwater in this area is deep and contained within fractures in bedrock. The current DWR (Department of Water Resources) website does not show any wells in this region as few production wells occur in this bedrock terrain. Shallow groundwater is largely controlled by drainage swales and/or permeable earth units overlying less permeable units such as the soil-bedrock contacts at this site. Groundwater and seepage along these contacts can be expected to occur seasonally with volume controlled primarily by rainfall. The presence of leaky, upslope water impoundments may also influence downstream seepage conditions.

3 POTENTIAL GEOLOGIC, SOILS, AND SEISMIC HAZARDS

3.1. FAULTING AND SEISMICITY

No Alquist Priolo Earthquake Fault Zones are known to exist on or projecting toward the subject parcel nor are any major or active faults mapped on or near the site. The nearest fault with known Quarternary activity is the West Bear Mountains Fault Zone located about 16 km to the east of the site. Since no mapped, or otherwise known young faults occur on or adjacent to the site, ground rupture is not considered probable.

Onsite seismicity will be controlled by large earthquakes on distant, active faults and to a lesser extent by small events on nearby faults with lower activity. The onsite peak ground acceleration has been estimated using Probabilistic Seismic Hazard Analysis (PSHA) at 0.10 – 0.15 g (CGS, USGS, 1996) for bedrock conditions and 10% probability of exceedance in 50 years (i.e. Design Basis Earthquake; DBE).

3.2. EARTH UNIT CHARACTERISTICS

Each of the earth units described above are differentiated due to their discernable and unique engineering characteristics. Those characteristics are described in association with each onsite earth or soil unit below, from youngest to oldest.

3.2.1. Colluvium/Residual Soil Profile

Colluvium is a term for surficial soil that is generated by sheet flow erosion resulting in downslope deposition, or colluviation. Without subsurface exploration at the site, it is difficult to discern this form of soil from residual soil that develops through weathering in place (i.e. non-transported). Although these two soil units onsite may have unique engineering characteristics, they are not differentiated at this time and are combined as one soil unit. This soil unit can be expected to form a thin blanket across most of the site. It has been mapped as "Qcl" based on our field reconnaissance in areas where thicker soils may exist as shown on Plate 3. Although subsurface exploration was not performed at this time, our experience with soils in similar geologic settings suggests that this earth unit may consists of soft and potentially compressible

sand, silt, and clay with localized cobbles. Where it occurs over the Mehrten Formation, it is typically plastic and therefore also potentially expansive. It is very conducive to excavation and variably permeable.

3.2.2. Alluvium

Alluvium is a term for surficial soils formed by erosion and deposition along tributaries along which concentrated drainage has occurred, or alluviation. These soils therefore occur along drainage courses and are recognizable by the presence of flatter surface gradients than the surrounding slopes. This unit has been mapped as "Qal" on the basis of our field reconnaissance as shown on Plate 3. Because these materials are deposited in a higher energy environment than colluvial or residual soils, they tend to be coarser-grained. Although subsurface exploration was not performed at this time, our experience with soils in similar geologic settings suggests that this earth unit may consist of soft and potentially compressible sand, silt, and localized cobbles. It is typically low to non-plastic and therefore also not likely to be potentially expansive. It is very conducive to excavation, variably permeable, but may contain isolated cobble and boulders.

3.2.3. Mehrten Formation

The Mehrten Formation contains reworked and relocated volcaniclastic rock. Andesitic source rock well upslope into the Sierra Nevada Mountains east of the site represent the volcanic source terrain for this material. Much of the downslope occurrence of the Mehrten Formation represents mudflow and lahar (i.e. debris flows induced by rapid melting of snow covered slopes during volcanic events) that have transported this pyroclastic rock downslope. There are two discernable units within the Mehrten Formation onsite including an upper, harder breccia unit and the underlying conglomeratic unit. The upper breccia unit is mapped as "TMV" and the lower conglomeratic unit as "Tmc" on Plate 3. The breccia unit forms the upper terrace at the site as well as the upper, steeper slopes. Although subsurface exploration was not performed at this time, our experience with the Mehrten Formation in other regions suggests that this earth unit will consists of angular fragments (sand-gravel-cobble-boulder sized) of andesite (breccia unit) within a hard, lithified medium-grained matrix of tuff and pyroclastic debris or well-rounded andesitic cobbles (conglomeratic unit) within a hard matrix. It is typically difficult to excavate and generates variable quantities of oversized rock. It is relatively impermeable when compared to overlying soil mantles resulting in shallow seepage along the contact during winter and spring months.

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3.2.4. Rocklin-Penryn Pluton Granitic Bedrock

The Rocklin-Penryn Pluton extends from Lake Folsom and Granite Bay northwest to just northeast of Lincoln. This pluton represents a Mesozoic intrusion related to plutonism of the Sierra Nevada Batholith and consists of diorite, granodiorite, and locally granite. In this location, it is generally medium-grained and visible outcrops consisted of granite and granodiorite. The hardness of this bedrock unit varies primarily with degree of weathering and fracturing. This unit is notorius throughout Rocklin as being deeply weathered in some locations with localized occurrences of essentially unweathered and thus very hard outcrops and zones beneath the surface. This earth unit is exposed and mapped as "gr" in the middle and lower portions of the site, as shown on Plate 3. Although subsurface exploration was not performed at this time, our experience with this geologic unit elsewhere suggests that it consists of variably weathered and fractured medium-grained rock. The depth and lateral extent of weathering is essentially unpredictable and yet very significant to it's engineering characteristics. In locations where it is deeply weathered, it occurs as a soil-like unit consisting of medium-grained permeable and generally non- to low plasticity soil. In locations where it is less weathered it forms a hard rock unit that is relatively impermeable and difficult to excavate.

4 PLANNING/DESIGN ISSUES AND RECOMMENDED SUBSEQUENT STUDIES

In general, there appear to be no geologic, soils, or seismic conditions that would preclude the proposed site development activities. Exploration will be necessary to further evaluate the extent and nature of onsite subsurface conditions. This effort is generally performed once design drawings become available such as the tentative map or grading plan preparation stage of the project. At that time, specific recommendations can be provided to mitigate any potentially adverse site geologic, soils, and/or seismic conditions. The following sections discuss the potential for potentially adverse conditions at the site based on our literature and map review and site reconnaissance. These sections are intended as planning and design input for the project.

4.1. NATURALLY OCCURRING ASBESTOS

Naturally Occurring Asbestos, or NOAMs have recently been identified as a source of potentially hazardous airborne asbestos fibers when disturbed by mining or earthwork activities, if present (CGS, 2002). NOAMs generally occur within mafic or ultramafic metamorphic rock units in which one of the asbestiform minerals defined by the CARB (2002) are present. These NOAM minerals include those minerals with unique flexibility and strength characteristics and include asbestiform varieties of the six minerals chrysotile, amosite, crocidolite, tremolite actinolite, and anthophyllite. The nearest mapped occurrence of these metamorphic rock units is the Foothills Metamorphic Belt along the western Sierra Nevada Foothill and located a minimum of 25 km south and east of the site. At this distance, NOAMs are not expected to influence site development. Imported rock sources should be carefully characterized relative to the potential content of NOAMs prior to transporting to the site.

4.2. CORROSION/SULFATE CONTENT

Chemical testing soil and/or rock is necessary to adequately evaluate the presence of high salinity (low pH) or high concentrations of sulfates that may influence the performance of buried metallic improvements and concrete, respectively. High salt content soils are generally found in evaporates or playas, where abnormally high evaporation rates concentrate salts, resulting in low pH (acidic) soil conditions. Sulfates are generally present where high concentrations of organic

have degraded and altered to sulfates and severely influence the curing and/or performance of concrete in contact with this chemistry. In either case, such soils are not known to be associated with onsite earth units or conditions; however, fine grained soils may be corrosive to buried metal objects. Therefore, subsequent studies should perform resistivity, pH, and sulfate chemical analyses to verify the absence of this adverse soil chemistry.

4.3. EXPANSION POTENTIAL

Fine-grained soils such as clays can undergo significant swelling when wetted and shrinkage during drying, resulting in damage to overlying improvements such as hardscape, footings, and slabs. Such fine-grained soils may be present onsite in surficial soils developed over either Mehrten Formation or granitic bedrock. This represents a majority of the site. Although these potentially expansive soils may exist across much of the site, they can be readily mitigated by recognition and analysis accompanied by proper design of footings, slabs, hardscape, and other proposed surface improvements. These soils also typically result in lower R-values that would increase the thickness of designed pavement sections. These conditions should also be anticipated in areas where these materials may be excavated and placed as compacted fill elsewhere.

4.4. COMPRESSIBLE/COLLAPSIBLE

Compressible soils are those that compress or consolidate when loaded and collapsible soils are those that hydroconsolidate rapidly when wetted. Either of these conditions can result in settlement of the ground surface adversely influencing associated surface improvements. These soils are generally characterized by low density and/or penetration resistance. Although subsurface exploration, sampling, and laboratory analysis will be necessary to adequately characterize these conditions, compressible/collapsible soils tend to be restricted to younger surficial soil deposits, such as residual, colluvial, and alluvial soils present onsite. Conversely, onsite granitic and volcanic bedrock units onsite are not likely to be susceptible to compressibility or collapse. Onsite compressible/collapsible soil units will typically be removed as part of the grading process and should be evaluated in the design level geotechnical report.

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4.5. EXCAVATION CHARACTERISTICS

Soil units onsite are anticipated to be readily excavatable with typical earthwork equipment in good repair. The granitic bedrock that forms the lower, southern portion of the site may encounter zones of less weathering and/or fracturing that can be expected to be difficult to excavate. Most of the Mehrten Formation volcanic rock unit is well known for it's difficult excavation characteristics. Typically, our experience has been that the upper, terrace-forming breccia caprock unit is harder and generates more oversized rock than the underlying conglomeratic unit, but the latter should still be considered difficult to excavate. Excavations in similar conditions have occurred extensively across Sierra College Blvd. north of the site and should provide valuable information regarding onsite excavation characterisitics. We recommend that future studies include seismic refraction survey of areas where outcrops suggest harder occurrences as well as other typical areas. These data can be compared to available seismic refraction data from the adjacent construction sites to correlate equipment performance. It should be noted that excavation characteristics are not only a function of hardness and fracture spacing of underlying rock but also the equipment proposed to perform those excavations. For example, large earthmoving equipment may be more capable of excavating rock units than utility, footing, and pool excavation equipment both due to the size of the equipment and the confinement of excavations. Future excavatability studies should address these proposed equipment options.

4.6. SHALLOW GROUNDWATER AND SEEPAGE

Shallow groundwater and seepage may result in saturation and ponding onsite if not anticipated and mitigated. This potential is anticipated to be very predominate at this site due to the presence of permeable soils overlying hard, less permeable rock units and also where excavated rock results in highly permeable fill over this same hard bedrock. Seepage along these contacts should be anticipated and mitigated once a grading plan is available and exploration can further evaluate the presence of these conditions. Mitigation of seepage may include placement of subdrains and cutoff drains to intercept and capture seepage and redirect it into unimproved areas

4.7. LIQUEFACTION AND LATERAL SPREADING

Liquefaction is a condition when low density soils behave much as a liquid when subjected to strong earthquake ground motions and thus lose their ability to support overlying loads. Lateral spreading is a similar condition but where lateral flow of these materials is allowed by the presence of exposed slopes or "free face" conditions such as along natural or manmade drainage or stream channels. The only onsite soils expected to be susceptible to this condition are those described under "Compressible/Collapsible Soils" above. These materials will be mitigated due to their potential for seismic consolidation and, once mitigated, are not expected to be liquefiable or susceptible to lateral spreading.

4.8. FLOODING

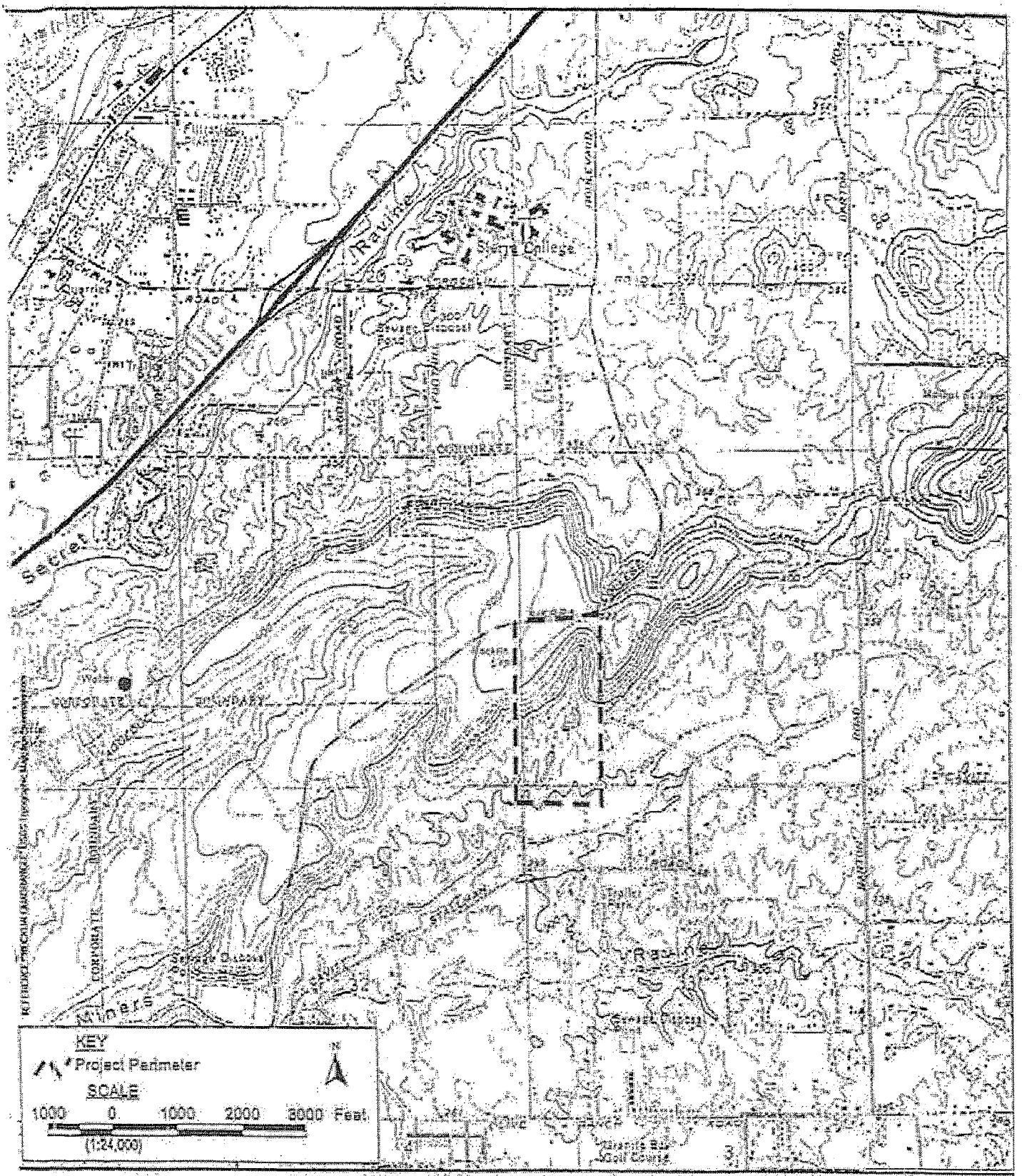
Federal Emergency Management Agency (FEMA) Flood Insurance Rate Maps (FIRMs) were reviewed to assess the presence of mapped 100-year or 500-year flood zones onsite. The FEMA website was reviewed and the nearest mapped flood zone is shown on the order of 1 km south of the site along Miners Ravine Creek.

4.9. SLOPE STABILITY AND ACCELERATED EROSION

Slope instability occurs when the surface gradient of underlying earth materials exceeds the strength of these materials to remain supported, especially when saturated. It is the shear strength character of these earth units that predominately controls the steepness of slopes that remain stable under saturated conditions. Slopes may occur as natural or manmade slopes. The stability of natural slopes only influences proposed development where adjacent (i.e. ascending above or descending below) proposed improvements within the area of influence of potential failure. The underlying bedrock units generally possess high shear strength and are capable of supporting steeper slopes than shallow surficial deposits represented by residual, colluvial, or alluvial soils. Although subsurface exploration, sampling, laboratory and engineering analyses will be necessary to adequately evaluate slope stability at the site, for planning purposes we suggest the following maximum slope gradients for onsite earth units:

Geologic Unit	Map Symbol (Plate 3)	Recommended Planned Slope Gradient (h : v)
Soil	NA	2 : 1
Mehrten Conglomerate	Tmc	1 : 1
Mehrten Breccia Caprock	Tmv	1 : 1
Moderate-Highly Weathered Granitic Bedrock	gr	1.5 : 1
Moderate-Highly Weathered Granitic Bedrock	gr	1 : 1
Alluvium	Qal	2 : 1
Engineered Fill Derived from Onsite Materials	N/A	2:1

Natural slopes approaching or exceeding these gradients may require stabilization proposed adjacent to proposed improvements. This would primarily apply to localized ou of steep to vertical exposures of rock in the north-central portion of the site. In all drainage conditions on and above the proposed and existing natural slopes may adv influence the stability of slopes and should be addressed by the project civil engineer.



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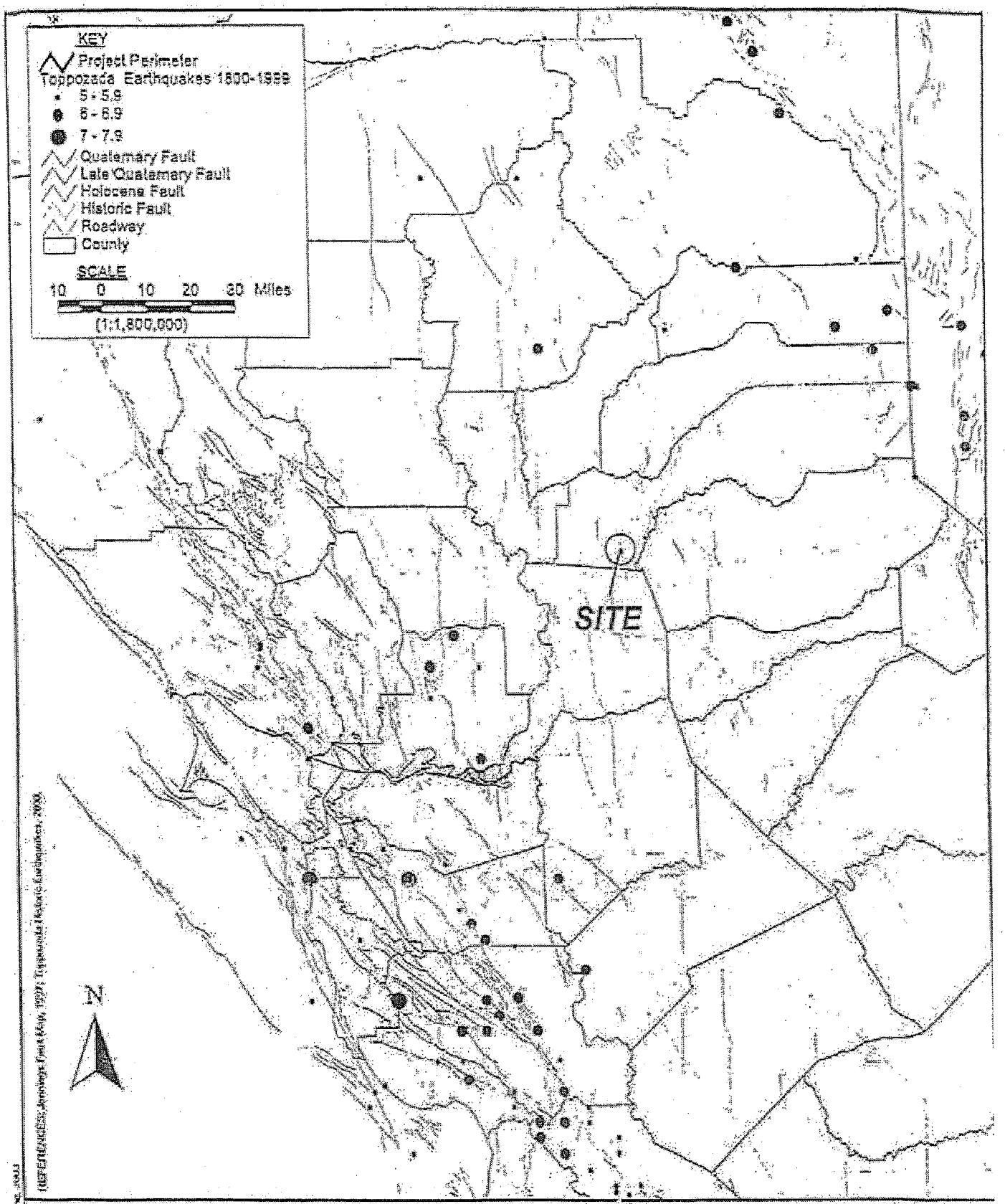
SITE LOCATION MAP

PROPOSED RESIDENTIAL SUBDIVISION
SIERRA COLLEGE BOULEVARD
GRANITE BAY, CALIFORNIA

Plate

1



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FAULTS AND HISTORIC SEISMICITY

PROPSED RESIDENTIAL SUBDIVISION
SIERRA COLLEGE BOULEVARD
GRANITE BAY, CALIFORNIA

Plate

2