GEOTECHNICAL ENGINEERING REPORT
for
10166 Alta Sierra Drive
APN 25-430-08
Nevada County, California

Prepared for:
CJS Development II, LLC
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Project No. 4268-01
June 26, 2014
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Dan Biswas
CJS Development II, LLC
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Scottsdale, Arizona 85250

Reference: 10166 Alta Sierra Drive
APN 25-430-08
Nevada County, California

Subject: Geotechnical Engineering Report

Dear Mr. Biswas,

This report presents the results of our geotechnical engineering investigation for the approximate 1-acre property located at 10166 Alta Sierra Drive in Nevada County, California. As proposed, the project is to include development of a single story, 9,100-square-foot retail building, associated parking lot, sidewalks and underground utilities.

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. Our primary concerns, from a geotechnical engineering standpoint, include encountering shallow resistant rock in areas of proposed deep cuts near the north side of the property.

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

Sincerely,

HOLDREGE & KULL

Prepared by:
Bryan Botsford
Staff Geologist

Reviewed by:
Chuck Kull, G.E. 2359
Principle Engineer

copies: 5 to CJS Development II / Attn: Dan Biswas
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  (included with permission of ASFE, Copyright 2004)
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1 INTRODUCTION

At the request of CJS Development II, LLC, Holdrege & Kull (H&K) performed a geotechnical investigation at 10166 Alta Sierra Drive in Nevada County, California. The geotechnical investigation was performed in general accordance with our June 12, 2014 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 1-acre commercial site is located between Alta Sierra Drive and Little Valley Road, approximately 500 feet east of the Highway 49 intersection with Alta Sierra Drive, in Nevada County, California. The property is bordered by Alta Sierra Drive to the west, Little Valley Road to the east and by commercial development to the north and south.

At the time of our field investigation, the project site was undeveloped. Site topography varied from gently sloping in the north and central portion of the property to slightly steeper slopes in the southwest portion. In general, the property sloped towards the southwest.

1.2 PROPOSED IMPROVEMENTS

Based on our review of a preliminary undated site plan for the project provided by CJS Development II, LLC, we understand that the proposed improvements will likely include a 9,100-square-foot retail building, associated parking areas, sidewalks and underground utilities. We also estimate that the maximum anticipated wall and column loads will be approximately 4 kips per lineal foot and 40 kips, respectively. We anticipate that grading for the project will include cut and fill for the building and parking lot areas and excavation for underground utilities. We anticipate that retaining structures will also be utilized.

1.3 PURPOSE

We performed a surface reconnaissance and subsurface geotechnical investigation at the site, collected soil samples for laboratory testing, and performed engineering calculations 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 relatively undisturbed soil samples and 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.

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 USDA Soil Conservation Service Web Soil Survey, accessed online in June 2014, and the Soil Survey of the Nevada County Area, California, USDA Soil Conservation Service (1993). The soil survey indicated that the site is located in an area containing the Secca-Rock Outcrop Complex.
The soil survey describes the Secca series soil as having a surface layer of 15 inches of brown and reddish brown gravelly silt loam. The surface layer is underlain by about 30 inches of yellowish red cobbly silty clay loam, strong brown cobbly clay and light yellowish brown gravelly light clay. Partly weathered basic rock is typically encountered at a depth of 45 inches. The Secca series soil is described as being moderately well-drained, and having medium to rapid runoff with slow permeability. About 10 to 40 percent of the Secca-Rock Outcrop Complex is rock outcrop.

2.1.2 Geologic Setting

According to the Geologic Map of the Chico Quadrangle (California Division of Mines and Geology, 1992), the area containing the project site is generally underlain by Paleozoic-aged metavolcanic rocks. The Paleozoic era spans the period of time between 251 and 542 million years before present.

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.

2.2 FIELD INVESTIGATION

We performed our field investigation on June 20, 2014. 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 seven exploratory trenches across the project site. We excavated to depths ranging between 2.5 and 7.5 feet below the ground surface (bgs) using a Caterpillar 430D backhoe equipped with a 24-inch bucket. We obtained samples using a hand-actuated slide sampler and shovel. A staff geologist from our firm logged the soil conditions revealed in the exploratory trenches and collected relatively undisturbed and bulk soil samples for laboratory testing. Figure 2 shows the approximate exploratory trench locations.
2.2.1 Surface Conditions

At the time of our investigation, the site appeared to be unimproved. Site topography generally sloped to the southwest and southeast, and ranged from 10 percent to an approximate 25 percent gradient in the southeast portion of the property. Referencing the United States Geological Survey (USGS) quadrangle map of Grass Valley, the site elevation is approximately 1980 feet above mean sea level (MSL).

Vegetation on the site was typical of the Sierra Nevada Foothills, with areas of dense oak and pine trees, manzanita and poison oak. Site drainage generally trends to the southwest and southeast.

2.2.2 Subsurface Soil Conditions

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

Trench T-1 was excavated from the ground surface to a depth of approximately 1.5 feet below the ground surface (bgs) through reddish brown, loose, moist, silty sand with gravel. The silty sand with gravel was underlain by highly weathered metavolcanic rock that became more resistant with depth. The weathered rock excavated as reddish brown, silty sand with 8- to 10-inch diameter angular cobbles. Trench T-1 was terminated at 5 feet bgs at near refusal. Trench T-2 was excavated in a depression and was terminated at approximately 2.5 feet. The purpose of the trench was to determine if it was mining related or an old debris depression.

Trench T-3 was excavated into existing fill to determine the material type. The trench revealed approximately 3 feet of fill. The excavation of trenches T-4 and T-6 revealed reddish brown, loose, moist silty sand with gravel to depths of approximately 1.5 to 2 feet. The silty sand was underlain by very dense, moist silty sand with gravel. Trenches T-4 and T-6 were terminated at depths of 6 and 7.5 bgs, respectively. Trenches T-5 and T-7, located in the area of the proposed building, were excavated through approximately 1 to 2 feet of silty sand with gravel. Beneath the silty sand with gravel highly to slightly weathered resistant metavolcanic rock with cobbles and boulders up 35 inches in diameter was encountered. Trenches T-5 and T-7 were terminated at 3.5 feet bgs at near refusal.
2.2.3 Groundwater 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 drainage channels and swales on the west side of the property that indicate seasonal flow of surface water.

Our observations of groundwater conditions were made in June 2014 following a period of dry weather. Although we did not observe groundwater in our exploratory trenches, our experience has shown that seepage may be encountered in excavations which reveal the soil/weathered rock transition, particularly during or 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 design recommendations for earthwork and structural improvements.

We performed the following laboratory tests:

- Moisture Content, (ASTM D2216),
- Density (unit weight), (ASTM D2937),
- Atterberg Limits (ASTM D4318),
- Particle Size (ASTM D422),
- Unconfined Compression Test (ASTM D2166), and
- Resistance Value (ASTM D2844).

\[1\] Due to rock content the unconfined compression test failed to provide meaningful results

In general, relatively undisturbed soil samples were collected for laboratory testing within the upper 2 feet of the trenches. Significant rock content prevented the collection of undisturbed soil samples below 2 feet.

Table 3.1 below summarizes moisture/density test results. Appendix D presents graphical Atterberg limits, particle size, and R-value test results.

<table>
<thead>
<tr>
<th>Trench Number</th>
<th>Sample Number</th>
<th>Depth</th>
<th>Dry Density (pcf)</th>
<th>Moisture Content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T-4</td>
<td>T4-01</td>
<td>4 to 10 inches</td>
<td>89.5</td>
<td>14.4</td>
</tr>
</tbody>
</table>
We performed a particle size determination on a sample of red sandy silt collected between approximately 2.5 feet to 4 feet in trench T-4. The test revealed the sample consisted of approximately 47 percent sand, and 53 percent silt and clay. Based on the particle size determination, we classified the soil as a sandy silt.

We performed an Atterberg limits determination on sample 4-2 collected from a depth of 1.5 feet to 2 feet in Trench 4. The Atterberg limits determination revealed that the portion of the sample passing the No. 40 sieve had a liquid limit of 54 and a plastic limit of 30, resulting in a plasticity index of 24. Based on the Atterberg limits determination we classified the soil as a silt with medium plasticity (MH).

An R-value test was performed on a bulk sample obtained from 5 to 6 feet bgs in trench T-4. The sample was described in the field as consisting of yellowish brown silty sand with gravel. The test indicated that the predominantly granular soil had an R-value of 47, by exudation pressure. Based on our experience in the area, the subsurface conditions revealed during our investigation, and the expansion pressure, we elected to use a design R-value of 40.

4 CONCLUSIONS

The following conclusions are based on our field observations, laboratory test results, 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 slope instability, liquefaction, and surface rupture are remote at the project site.

4. Based on the site geology and our observation of the surface conditions, we anticipate that grading and excavation onsite will reveal variably weathered, fractured, metamorphic rock. Areas of resistant rock may be encountered which may require splitting, hammering, or blasting to increase the rate of excavation. In addition, spoil resulting from excavation onsite will likely 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. Preliminary recommendations addressing rock fill placement are included in this report.

5. We encountered existing fill and disturbed soil that varied in depth up to approximately 3 feet bgs in exploratory trench T-3. Existing fill was also observed adjacent to trench T-7. Existing fill, if encountered, should not be relied upon to support proposed improvements. We provide recommendations for existing fill in Section 5.1.2, below.

6. During our site investigation, we did not observe groundwater or seepage within our exploratory trenches. However, we did observe evidence that surface water is seasonally transported through drainage channels and swales on the property. 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.

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

5 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.

5.1 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.

5.1.1 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. Overexcavate any relatively loose debris and soil that is encountered in our exploratory trenches or any other onsite excavations to underlying, competent material. Possible excavations include exploratory trenches excavated by others, mantles or soil test pits, holes resulting from tree stump or boulder removal, and mining relics.

3. Loose, untested fill encountered during site development should be overexcavated to competent native soil or weathered rock a minimum of 5 feet beyond the areas of proposed improvements.

4. 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 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. Unless used as rip-rap, oversized rock placed in fill should not be located within 5 feet horizontally of the finished fill slope face. The project geotechnical engineer should approve the use of oversized rock prior to constructing fill.

5. 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.

6. 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.

5.1.2 Existing Fill

One of our concerns regarding the project site is the presence of existing untested fill within the proposed improvement areas. Loose fill beneath footings may contribute to future differential settlement-induced distress. Our opinion is that the
existing fill should not be relied upon to support the proposed improvements without mitigation, as described in the following paragraphs.

Relatively loose fill, within and a minimum of 5 feet beyond the proposed structure footprints, shall be overexcavated and stockpiled onsite. The depth of the overexcavation should extend through all loose soil to competent native soil or rock. The fill shall be replaced and compacted using the recommendations presented in the Fill Placement section of this report.

5.1.3 Cut Slope Grading

Based on our understanding of the project at this time, we anticipate that permanent cut slopes up to 17 feet in height will be created during grading of the proposed improvements. The cut slopes may be free standing or retained. In general, permanent cut slopes should not be steeper than 1.5:1, horizontal to vertical (H:V). Steeper cut slopes may be feasible, depending on the soil/rock conditions encountered and should be reviewed on a case-by-case basis. The upper two feet of all cut slopes 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 subsurface conditions will be favorable for construction of temporary cut slopes no steeper than ½:1, H:V, for a maximum height of approximately 20 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 1:1, 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.

5.1.4 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 12 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 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 sloped slightly into the hillside at an approximate gradient of 5 percent or greater.

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.

5.1.5 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. If encountered, 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. If approved by the project geotechnical engineer, oversized rock may be placed at or near the bottom of deep fills. Oversized rock must be placed in windrows to avoid nesting and to facilitate the placement of compacted fill. 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.
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.

   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.

4. 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.

5. Fill should be constructed by placing uniformly moisture conditioned soil in maximum 8-inch-thick loose, horizontal lifts (layers) prior to compacting.

6. 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.

5.1.6 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.

1. Site grading should be performed so that cut-fill transition lines do not occur directly beneath any structures. The cut portion of the cut-fill building pads, if proposed, should be scarified to a minimum depth of 8 inches, and recompacted to 95 percent relative compaction.
2. Differential fill depths beneath structures should not exceed 5 feet. For example, if the maximum fill depth is 8 feet across a building pad, the minimum fill depth beneath that pad should not be less than 3 feet. If a cut-fill building pad is used in this example, the cut portion would need to be overexcavated 3 feet and rebuilt with compacted fill.

5.1.7 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.

5.1.8 Fill Slope Grading

Based on our understanding of the project, we anticipate that fill may be placed up to 13 feet in height 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 10 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.

Where placement of oversized rock in deep fill is proposed, the oversized rock should be placed a minimum of 5 feet horizontally from the finished fill slope face.

**5.1.9 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. All 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. Under no circumstances should concentrated surface water be directed over slope faces. The intercepted water should be discharged into natural drainage courses or into other collection and disposal structures.
5.1.10 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, blasting, or splitting may be required, particularly if utility trench excavations are deeper than five feet below existing grade.

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 15 to excavate, place and compact the trench backfill materials.

4. Trench backfill used within the bedding and shading zones should consist of ¾-inch minus crushed rock, granular material with a sand equivalent greater than 30, or similar material approved by the project engineer.

5. Soil used as trench backfill should consist of non-expansive soil with a plasticity index (PI) less than or equal to 15 and should not contain rocks greater than 3 inches in greatest dimension unless otherwise approved by the geotechnical engineer.

6. Where utility trenches will intersect perimeter footings or pass within the proposed building footprint, we recommend that a low permeability backfill plug be placed to reduce water migration and infiltration. In general, a low permeability, predominantly fine-grained soil backfill, sand-cement slurry, or other approved material should be placed within five feet of the building exterior.

7. Trench backfill should be constructed by placing uniformly moisture conditioned soil in maximum 12-inch-thick loose lifts prior to compacting.

8. Trench backfill should be compacted to a minimum relative compaction of 90 percent of the ASTM D1557 maximum dry density. In areas of proposed pavement or concrete flatwork, the upper 12 inches of backfill should be
compacted to a minimum relative compaction of 95 percent of the ASTM D1557 maximum dry density. Jetting is not an acceptable method of compacting trench backfill or bedding sand.

9. The loose lift thickness, moisture, density and relative compaction of the trench backfill soil should be observed by a representative of H&K during placement.

10. Construction quality assurance tests should be performed at a frequency determined by the project geotechnical engineer. Where trench backfill is placed at depths greater than approximately 4 feet, or where potentially unstable sidewall conditions exist, shoring may need to be provided by the contractor to facilitate compaction testing. If shoring is not provided or unsafe conditions are encountered, full time observation will likely be required to confirm compactive effort.

5.1.11 Construction Dewatering

Seepage may be encountered during grading, particularly in deeper excavations made during site preparation. The earthwork contractor should be prepared to dewater excavations if seepage is encountered during grading. Seepage may be encountered if grading is performed during or immediately after the rainy season. In addition, perched groundwater may be encountered on low permeability soil or weathered 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.

3. If groundwater enters the trench faster than it can be removed by the dewatering system, the underlying compacted soil may become unstable while compacting successive soil lifts. If this occurs, the unstable soil may need to be removed and replaced with free draining open graded drain rock.
rock is used, it should meet or exceed the following gradation specifications:
100 percent passing the $\frac{3}{4}$-inch sieve, 95 to 100 percent passing the $\frac{1}{2}$-inch sieve, 70 to 100 percent passing the $\frac{3}{8}$-inch sieve, 0 to 55 percent passing the No. 4 sieve, 0 to 10 percent passing the No. 8 sieve, and 0 to 3 percent passing the No. 200 sieve. Other approved backfill materials can again be used after placing the drain rock to an elevation that is higher than the groundwater.

5.1.12 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 and our experience, the native soil conditions onsite possess a mild to moderate corrosion potential for uncoated steel and concrete.

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.

5.1.13 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.

**5.1.14 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.

**5.2 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.

**5.2.1 Seismic Design Criteria**

Our classification of on-site soil conditions is based on field observations and laboratory tests. When grading is complete, the near surface soil within the building envelope will consist of predominantly granular soil composed of silty sand with angular rock fragments. Based on the presence of predominantly granular soil and resistant, ultramafic rock at relatively shallow depths, we classified the on-site soil as SM for design purposes.

Table 5.2.1.1 below summarizes seismic design criteria based on Section 1613 of the 2013 California Building Code, CCR Title 24, Part 2. We used Section 1613 of the 2013 California Building Code (CBC) and the United States Geological Survey (USGS), *Java Ground Motion Parameter Calculator, Earthquake Ground Motion Tools, Version 5.1.0*, to develop the following seismic design parameters:
### 5.2.1.1 - 2013 Seismic Design Parameters

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Latitude</td>
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<td>1</td>
</tr>
<tr>
<td>Longitude</td>
<td>-121.0690</td>
<td>1</td>
</tr>
<tr>
<td>Site Coefficient, $F_A$</td>
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<td>6</td>
</tr>
<tr>
<td>Site Coefficient, $F_V$</td>
<td>1.6</td>
<td>7</td>
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<td>Short (0.2 sec) Spectral Response, $S_S$</td>
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<tr>
<td>Spectral Response, $S_1$</td>
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<td>4, 5</td>
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<tr>
<td>$S_S$ modified for Site Class Effects, $S_{MS}$</td>
<td>0.65g</td>
<td>8, 5</td>
</tr>
<tr>
<td>$S_1$ modified for Site Class Effects, $S_{M1}$</td>
<td>0.38g</td>
<td>9, 5</td>
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<tr>
<td>Design Spectral Response Acceleration, Short Periods, $S_{DS}$</td>
<td>0.44g</td>
<td>10, 5</td>
</tr>
<tr>
<td>Design Spectral Response Acceleration, Long Periods, $S_{DL}$</td>
<td>0.25g</td>
<td>11, 5</td>
</tr>
</tbody>
</table>

References:
1. USGS 7.5 min
2. 2013 CBC, Table 1613.5.2
3. CBC Figure 1613.3.1(1)
4. CBC Figure 1613.3.1(1)
5. USGS Uniform Hazard Response Spectra, v 5.1.0
6. 2013 CBC, Table 1613.3.3(1)
7. 2013 CBC, Table 1613.3.3(2)
8. 2013 CBC, Equation 16-37
9. 2013 CBC, Equation 16-38
10. 2013 CBC, Equation 16-39
11. 2013 CBC, Equation 16-40

### 5.2.2 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. 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. In general, structures constructed adjacent to descending slopes should employ a minimum setback of either 1/3 the height of the slope, or 40 feet, whichever is less. The setback for ascending slopes is either 1/2 the slope height or 15 feet, whichever is less. Where footings are proposed within these code-based setbacks, the project geotechnical engineer should review the proposed slope configuration and provide revised setback recommendations, if appropriate. Footings can be constructed adjacent to or as an integral part of retaining walls provided the walls are designed for the surcharge loads.

6. 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.

7. 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 soil be sized for an allowable bearing capacity of 3,000 psf for dead plus live loads. This value can be increased by 300 psf for each additional foot of embedment up to a limiting value of 3,600 psf. Allowable bearing may be increased by 33 percent for additional transient loading, such as wind or seismic loads.

8. 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 0.4 may be used. These values can be used together, provided the higher of the two values is reduced by 50 percent.

9. 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.

5.2.3 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 residential 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 24-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 vapor transmissivity prior to the installation of moisture sensitive flooring. 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.

5.2.4 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.

5.2.5 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 5.2.5.1 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>
<thead>
<tr>
<th>Loading Condition</th>
<th>Retained Cut or Compacted Fill (approximately horizontal backfill)</th>
<th>Retained Cut or Compacted Fill (retained slope up to 2:1, H:V)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Active Pressure (pcf)</td>
<td>35</td>
<td>45</td>
</tr>
<tr>
<td>Passive Pressure (pcf)</td>
<td>300</td>
<td>300</td>
</tr>
<tr>
<td>At-Rest Pressure (pcf)</td>
<td>55</td>
<td>65</td>
</tr>
<tr>
<td>Coefficient of Friction</td>
<td>0.40</td>
<td>0.40</td>
</tr>
</tbody>
</table>

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.

7. Mechanically Stabilized Earth (MSE) or stacked rock walls have performed well in Nevada County. The design may want to consider the use of these types of walls for the project. Typically, MSE walls have a geogrid length equal to approximately 0.7 times the height of the wall.

5.2.6 Pavement Design

The R-value sample was collected on June 20, 2014. We used a design R-value of 40 and preliminary traffic indices (TIs) of 4 and 5. 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 5.2.6.1 below.

<table>
<thead>
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<th>Table 5.2.6.1 - Recommended Pavement Sections</th>
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<td>Design R-Value: 40</td>
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<tr>
<td>Traffic Description: light auto traffic</td>
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<tr>
<td>Caltrans Section 26, Standard Specifications,</td>
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<td>Asphalt Concrete</td>
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<tr>
<td>Caltrans Section 26, Class 2 Baserock</td>
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<tr>
<td>95% compaction</td>
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<tr>
<td>Subgrade Soil</td>
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<tr>
<td>95% compaction</td>
</tr>
<tr>
<td>Traffic Index: 5</td>
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<tr>
<td>Design R-Value: 40</td>
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<tr>
<td>Traffic Description: light truck traffic</td>
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<td>Caltrans Section 26, Standard Specifications,</td>
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<td>Asphalt Concrete</td>
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<tr>
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</tr>
<tr>
<td>Subgrade Soil</td>
</tr>
<tr>
<td>95% compaction</td>
</tr>
</tbody>
</table>
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.

6 **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. No warranty is 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
APPENDIX A     PROPOSAL
Proposal No. PN14097
June 12, 2014

Simon Commercial Real Estate  CJS Development II, LLC
5111 N. Scottsdale Road, Suite 200
Scottsdale, Arizona 85250

Attention: Dan Biswas, V.P. of Development

Reference: 10166 Alta Sierra Drive
APN 25-430-08
Nevada County, California

Subject: Proposal for Geotechnical Engineering Services

Dear Mr. Biswas:

At your request, we prepared this proposal to provide geotechnical engineering services for 10166 Alta Sierra Drive in Nevada County, California. We understand that an approximate 9,100-square-foot retail building with associated parking, underground utilities, and signage is proposed for the site. To prepare this proposal, we discussed the project with you and reviewed an undated Preliminary Site Plan. We understand that excavations up to 16 feet deep are proposed. The purpose of our services will be to provide a design-level geotechnical investigation for the project. A proposal for soil evaluation for wastewater disposal is also being provided under separate cover.

SCOPE OF SERVICES

We propose the following scope of services based on our understanding of the project.

\* Must satisfy all requirements for county and all municipal approvals

Literature Review

H＆K will perform a map and literature review of published documents pertinent to the project site including geologic maps and soil survey maps.

Site Investigation

H＆K will perform an investigation of the project site to characterize shallow soil, rock and groundwater conditions. The information gathered during the investigation will be used to prepare geotechnical engineering design recommendations for earthwork and structural improvements.
Surface Reconnaissance

H&K will perform a site reconnaissance to identify surface conditions that may impact the proposed development plans. An engineer/geologist from our firm will observe and describe surface exposures of the following existing site conditions:

1. Site and surrounding land uses.
2. Surface soil conditions.
3. Site topography and drainage.
4. Vegetation.
5. Geologic units exposed at the surface.

Subsurface Investigation

We will perform a subsurface investigation to obtain an understanding of the soil, rock and groundwater conditions underlying the project site to the maximum depth excavated. Prior to our field investigation, we will obtain underground service alert clearance for the site. Our field investigation will include the excavation of 4 to 5 exploratory trenches in the vicinity of the proposed improvements to depths up to 8 feet below the ground surface. One excavation in the area of proposed deep cuts will be advanced to 15 feet or refusal. If refusal is met at shallower depths, we may propose a cursory seismic refraction survey at the site to determine the shear wave velocity of the subsurface materials for rippability. A seismic refraction survey is not proposed in our current scope of services. Our field investigation may also include 2 to 3 shallow excavations in the proposed parking areas to depths up to 3 feet deep to facilitate the collection of composite soil samples. Excavated soil will be placed back into the exploratory trenches, but will not be compacted. Recompaction of the trenches should be accomplished during grading for the project.

During our field investigation, an engineer/geologist from our firm will log the soil conditions, and collect relatively undisturbed and bulk soil samples. Relatively undisturbed soil samples will be collected with a 2.0-inch-diameter (inside diameter) hand-activated sampler equipped with brass liner tubes. Additional soil samples may be taken and/or the sample intervals may be changed depending upon the soil conditions encountered. The soil samples will be labeled, sealed, and transported to our laboratory where selected samples will be tested to determine their engineering material properties.
Laboratory Testing

H&K will perform laboratory tests on selected soil samples to determine their engineering material properties. Laboratory tests will be performed using American Society for Testing and Materials (ASTM) and Caltrans methods, as guidelines.

Depending on the subsurface conditions encountered, we anticipate that laboratory testing will include:

- D2216, Moisture Content
- D2487, Unified Soil Classification System
- D2488, Soil Description Visual Manual Method
- D2844, Resistance Value (R-Value)
- D2937, Density
- D3080, Direct Shear Strength
- D4318, Atterberg Plasticity Indices (if appropriate)
- D4829, Expansion Index (if appropriate)

Data Analysis and Engineering

Data will be analyzed and engineering calculations will be performed to determine the following:

1. Soil bearing capacity for shallow foundations.
2. Lateral earth pressures for foundation and retaining wall design.
4. Soil shear strength.
5. Soil expansion and swell potential (if appropriate).
6. Design sections for asphalt pavement.

H&K will develop geotechnical engineering recommendations for earthwork and structural improvements and provide applicable recommendations. The geotechnical engineering recommendations will include the following.

Earthwork Improvement Recommendations

1. Site clearing and subgrade preparation.
2. Fill moisture conditioning, placement, and compaction requirements.
3. Utility trench backfill placement and compaction requirements.
4. Retaining wall backfill specifications.
5. Retaining wall drainage.
6. Surface water drainage.
7. Expansive soil mitigation (if appropriate).
8. Temporary construction dewatering methods.
9. Subdrain recommendations (if appropriate).

**Structural Improvement Recommendations**

1. Foundation types and embedment depths.
2. Allowable soil bearing capacity.
3. Foundation soil friction coefficients.
4. Concrete slab-on-grade floors.
5. Cantilever retaining wall lateral earth pressure coefficients, including effects of surcharge and seismic loading.
6. Seismic design parameters.
7. Asphalt pavement section design.

**Report Preparation**

We will prepare a geotechnical engineering report that will present our findings, conclusions, and recommendations. The report will include descriptions of site conditions, our field investigation, laboratory testing, and geotechnical engineering recommendations for the proposed earthwork and structural improvements. The report will also include a site plan showing the approximate locations of the exploratory trenches, proposed buildings, and property boundaries. The report appendices will present the exploratory trench logs and laboratory test data.

Please note that the proposed scope of services for a geotechnical investigation does not include an assessment of onsite environmental conditions or potential impacts from past use of the site.

**ASSUMPTIONS AND CLIENT RESPONSIBILITIES**

The proposed scope of services is based on the following assumptions:

- H&K will provide excavation equipment for the exploratory trenches.
- The client will provide H&K with the authorization to access the site. Although reasonable care will be used during excavation, the client understands that unmarked underground utilities may be damaged. H&K will not be responsible for repair of utilities that were not marked or were improperly marked prior to the investigation.
• Five copies of the geotechnical report will be delivered to the client and/or the client's engineers and architects. In addition, we will provide a pdf version of the report for electronic distribution.

• Client meetings and revisions to reports are not included in the budget estimate, but can be provided on a time and materials basis at the client's request.

FEES

The geotechnical investigation and report can be provided for a lump sum fee of [ ] if performed in conjunction with the septic evaluation described in our separate, June 12, 2014 proposal to provide onsite wastewater disposal evaluation and design services. Our fee to perform the geotechnical engineering investigation will be [ ] if performed separately from the wastewater evaluation. A seismic refraction survey, if required, is not included in our estimate or scope of services. MUST SATISFY ALL MUNICIPAL AND COUNTY REQUIREMENTS Progress billing will be monthly on a percent complete basis. If this proposal is acceptable, please sign the enclosed terms and conditions, initial the scope of services requested, and return one copy to our office along with a $ retainer as our authorization to proceed.

TIMING

Timing is dependent on receipt of authorization to proceed and the availability of the Nevada County Environmental Health Department. County representatives would need to be present for the onsite wastewater soil evaluation.

We will schedule our field investigation within one week of receiving authorization to proceed. We understand that at this time you would like to submit plans by June 27, 2014. We anticipate that the investigation and reporting can be performed in two weeks which would meet your proposed time frame if authorization is promptly provided.

Holdrege & Kull can also provide testing and observation services during grading to confirm that the project plans and specifications are incorporated into the construction. Our cost to provide testing and observation services depends to a large extent on the quantity of earthwork to be performed during grading, as well as the contractor's schedule and efficiency. We can provide a detailed cost estimate for testing and observation services once the grading plan for the project has been prepared.
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

Chuck Kull, G.E. 2359
Principal Engineer

encl: Terms and 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 overly rely 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 Geotechnical 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.
### TRENCH T1

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**PROJECT NAME:** 10166 ALTA SIERRA DRIVE  
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<th>DEPTH (FT)</th>
<th>USCS</th>
<th>DESCRIPTIONS/REMARKS</th>
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<td>UPPER 2&quot; FOREST DUFF</td>
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<td>1</td>
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<td>1</td>
<td>SM</td>
<td>REDDISH BROWN (10YR 3/6), LOOSE, MOIST, SILTY SAND WITH GRAVEL</td>
</tr>
<tr>
<td>2</td>
<td></td>
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<td>2</td>
<td>SM</td>
<td>REDDISH BROWN (10YR 3/6), HIGHLY WEATHERED METAVOLCANIC ROCK, HIGHLY FRACTURED, LESS WEATHERED WITH DEPTH, EXCAVATED AS REDDISH BROWN SILTY SAND WITH ANGULAR COBBLES 8-30&quot;.</td>
</tr>
<tr>
<td>3</td>
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<td>3</td>
<td>SM</td>
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<td>4</td>
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<td>4</td>
<td></td>
<td>TRENCH TERMINATED AT 3.5 FEET BGS (NEAR REFUSAL)</td>
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</table>
**Sample No.:** 4-2  | **Boring/Trench:** T4  | **Depth, (ft.):** 1.5-2  | **Tested By:** MLH  
**Project No.:** 4268-01  | **Project Name:** 10166 Alta Sierra Drive  | **Date:** 6/23/2014  | **Checked By:** MLH  
**Sample Location:** Red (2.5YR 4/6) Sandy Elastic Silt  | **Lab. No.:** 15-14-151  

---

**Estimated % of Sample Retained on No. 40 Sieve:** 33%  | **Sample Air Dried:** yes  
**Test Method A or B:** A  

---

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Pan ID</th>
<th>Mt. Pan (gr)</th>
<th>Mt. Wet Soil + Pan (gr)</th>
<th>Mt. Dry Soil + Pan (gr)</th>
<th>Mt. Water (gr)</th>
<th>Water Content (%)</th>
<th>Number of Blows, N</th>
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<tr>
<td></td>
<td>LB</td>
<td>15.32</td>
<td>28.64</td>
<td>24.12</td>
<td>4.52</td>
<td>51.4</td>
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<td>LD</td>
<td>15.21</td>
<td>30.49</td>
<td>25.15</td>
<td>5.34</td>
<td>53.7</td>
<td>27</td>
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<td>AI</td>
<td>14.96</td>
<td>31.38</td>
<td>25.47</td>
<td>5.91</td>
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<td>LF</td>
<td>10.81</td>
<td>16.93</td>
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<td>1.44</td>
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<td>11.04</td>
<td>15.40</td>
<td>14.39</td>
<td>1.01</td>
<td>30.1</td>
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</table>

**LIQUID LIMIT:** 54  
**PLASTIC LIMIT:** 30  

---

**Plasticity Index:** 24  
**Group Symbol:** MH

---

**Atterberg Classification Chart**

---

**HOLDREGE & KULL**

(530) 478-1305 - Fax (530) 478-1019 - 792 Searls Ave.- Nevada City, CA 95959 - A California Corporation
## Particle Size Distribution

### ASTM D422

**Project No.:** 4268-01  
**Project Name:** 10166 Alta Sierra Drive  
**Date:** 6/23/2014  
**Sample No.:** comp  
**Boring/Trench:** T4  
**Depth, (ft.):** 2.5-4  
**Tested By:** MLH  
**Checked By:** MLH  
**Lab. No.:** 15-14-151

### Description:
- **Sample Location:** Red (2.5YR 4/6) Sandy Elastic Silt
- **Sample Location:** Lab. No.: 15-14-151

### Particle Size Gradation

<table>
<thead>
<tr>
<th>Particle Size Gradation</th>
<th>Boulders</th>
<th>Cobble</th>
<th>Coarse Gravel</th>
<th>Fine</th>
<th>Coarse Sand</th>
<th>Medium Sand</th>
<th>Fine Silt</th>
<th>Clay</th>
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</thead>
<tbody>
<tr>
<td>Percent Passing (%)</td>
<td>100.0</td>
<td>90.0</td>
<td>80.0</td>
<td>70.0</td>
<td>60.0</td>
<td>50.0</td>
<td>40.0</td>
<td>30.0</td>
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</table>

### Table: Particle Diameter Details

<table>
<thead>
<tr>
<th>Sieve Size (U.S. Standard)</th>
<th>Particle Diameter (U.S. Standard)</th>
<th>Particle Diameter (mm)</th>
<th>Retained On Sieve (gm)</th>
<th>Accumulated On Sieve (gm)</th>
<th>Passing Sieve (gm)</th>
<th>Percent Passing (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6 inch</td>
<td>0.0000</td>
<td>15.24</td>
<td>0.00</td>
<td>0.0</td>
<td>957.6</td>
<td>100.0</td>
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<tr>
<td>3 inch</td>
<td>0.0000</td>
<td>76.2</td>
<td>0.00</td>
<td>0.0</td>
<td>957.6</td>
<td>100.0</td>
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<tr>
<td>2 inch</td>
<td>0.0000</td>
<td>50.8</td>
<td>0.00</td>
<td>0.0</td>
<td>957.6</td>
<td>100.0</td>
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<tr>
<td>1.5 inch</td>
<td>0.0000</td>
<td>38.1</td>
<td>0.00</td>
<td>0.0</td>
<td>957.6</td>
<td>100.0</td>
</tr>
<tr>
<td>1.0 inch</td>
<td>0.0000</td>
<td>25.4</td>
<td>0.00</td>
<td>0.0</td>
<td>957.6</td>
<td>100.0</td>
</tr>
<tr>
<td>3/4 inch</td>
<td>0.0000</td>
<td>19.1</td>
<td>0.00</td>
<td>0.0</td>
<td>957.6</td>
<td>100.0</td>
</tr>
<tr>
<td>1/2 inch</td>
<td>0.0000</td>
<td>12.7</td>
<td>0.00</td>
<td>0.0</td>
<td>957.6</td>
<td>100.0</td>
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<tr>
<td>3/8 inch</td>
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<td>9.5</td>
<td>0.00</td>
<td>0.0</td>
<td>957.6</td>
<td>100.0</td>
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<tr>
<td>#4</td>
<td>0.0000</td>
<td>4.7500</td>
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<td>957.6</td>
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<td>201.5</td>
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<td>#40</td>
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<td>0.4250</td>
<td>70.44</td>
<td>271.3</td>
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<tr>
<td>#60</td>
<td>0.0000</td>
<td>0.2500</td>
<td>60.08</td>
<td>332.0</td>
<td>625.6</td>
<td>65.3</td>
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<td>#200</td>
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<td>62.16</td>
<td>447.8</td>
<td>509.8</td>
<td>53.2</td>
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</table>

HOLDREGE & KULL  
(530) 478-1305 - Fax (530) 478-1019 - 792 Searls Ave.- Nevada City, CA 95959 - A California Corporation
# RESISTANCE (R) VALUE TEST

**ASTM D 2844**

<table>
<thead>
<tr>
<th>Specimen No.</th>
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<th>2</th>
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<tbody>
<tr>
<td>Moisture Content (%)</td>
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<tr>
<td>Dry Density (PCF)</td>
<td>109.6</td>
<td>108.5</td>
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<td>Resistance Value (R)</td>
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<td>Exudation Pressure (PSI)</td>
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<tr>
<td>Expansion Pressure</td>
<td>48</td>
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As Received Moisture Content (%) | 10.7 |

**RESISTANCE VALUE AT 300 P.S.I.** | 47 |

Reviewed By: Brandon Rodebaugh  
Materials Engineer