Appendix F-1
Geotechnical Investigation to Support the Programmatic EIR, Petra Geosciences, February 15, 2017
REVISED GEOTECHNICAL INVESTIGATION TO SUPPORT THE PROGRAMMATIC ENVIRONMENTAL IMPACT REPORT FOR THE PARADISE VALLEY PROJECT RIVERSIDE COUNTY, CALIFORNIA

GLORIOUS LAND COMPANY, LLC

February 15, 2017
J.N. 11-346
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GLORIOUS LAND COMPANY, LLC
39820 Portola Avenue, Suite 2
Palm Desert, California  92260

Attention: Mr. Frans Bigelow

Subject: Revised Geotechnical Investigation to Support the Programmatic Environmental Impact Report for the Paradise Valley Project, Riverside County, California

Dear Mr. Bigelow

Petra Geosciences, Inc. (Petra) is presenting herein the results of our revised preliminary geotechnical investigation for the proposed Paradise Valley project, located along Interstate 10, approximately 15 miles east of the city of Coachella, Riverside County, California.

PROJECT UNDERSTANDING

The Paradise Valley Specific Plan Project would develop a master-planned “new town” in Shavers Valley, an unincorporated area of Riverside County approximately 15 miles east of the city of Coachella and about 10 miles west of Chiriaco Summit. Regional access to the project site is via I-10, which traverses the northern portion of the site. The project site consists of approximately 5,000 contiguous acres located both north and south of I-10, of which approximately 1,800 acres would be developed with a mix of residential (8,500 units), commercial, light industrial, schools, parks and other public facilities.

Options for providing electricity to the site were provided by Envicom Corporation and are presented below in italics.

The project is located within Imperial Irrigation District’s (IID’s) service area. One option for providing electrical power for the project includes the transmission of electrical power generated from IID’s existing power generation facility located at 52nd Avenue in Coachella, California, to the project site via a new transmission line constructed within Caltrans’ existing frontage road right-of-way located adjacent to the Interstate 10 Freeway. A new substation will be constructed on site to accept power from the transmission line, and a distribution system will be constructed from the substation in order to deliver electricity throughout the project site. The electrical distribution system will be installed underground along with gas, telephone and cable television facilities. All electrical transmission and distribution facilities will be designed and constructed in accordance with IID’s adopted guidelines, policies and procedures.
A second option for providing electrical power for the project involves the construction of an on-site gas fired electrical generation facility. On site power generation will be achieved by a central plant, augmented by renewable energy generation, with electricity to be distributed throughout the project by an independent micro grid. As with electricity to be delivered via the transmission line discussed above, on-site power generation facilities would include an underground electrical distribution system installed along with gas, telephone and cable television facilities, all designed and constructed in accordance with IID’s adopted guidelines, policies and procedures.

A third option for providing electrical power for the project involves an interconnection to Southern California Edison’s (SCE’s) existing transmission lines which traverse the project site. This option will involve the construction of a substation stepdown transformer and associated substation required for acceptance of high voltage electricity, voltage reduction and distribution throughout the project. An interconnection to SCE’s facilities would involve a cooperative arrangement between IID, SCE and the California Independent System Operator (CAISO).

SITE LOCATION AND DESCRIPTION

The site comprises approximately 5,000 acres and is bisected by Interstate 10 (I-10) in unincorporated Riverside County, California, approximately 15 miles east of the city of Coachella. Access to the site is from the Frontage Road Exit off I-10 at the Sempra Energy pumping station. The majority of the site is undeveloped native desert alluvial fan topography. The northernmost portion of the project, north of I-10, lies within the lower portions of the Cottonwood Mountains. The Mecca Hills and the Orocopia Mountains lie to the southwest and south, respectively. Existing improvements within the project site include the Sempra Energy natural gas pumping plant, a well which services the pumping plant, an associated buried gas pipeline, and high-voltage transmission lines. The improvements all lie within utility easements.

The site includes portions of Sections 2 and 3 and all of Sections 1, 9, 10, 11, 13, 14 and 15 of Township 6 South and Range 10 East, SBBM. The topography of most of the site is gently sloping to the southeast. Based on information shown on the published U.S, Geological Survey (USGS) topographic map for the area, elevations range from approximately 2,200 feet above mean sea level in the Cottonwood Mountains in the northwest part of the site, to approximately 1,100 feet in Shavers Valley at the southeast corner. A prominent drainage, Pinkham Wash, traverses through the western portion of the site. The location of the project is shown on Figure 1 (Location Map).

OBJECTIVE AND SCOPE OF WORK

The objective of our work is to support the Programmatic Environmental Impact Report for the project by obtaining additional information pertaining to geotechnical/geologic conditions at the site and to provide a summary of the geotechnical constraints that may have an impact on proposed design. We also provide
general mitigation measures, in order to facilitate the design process. In addition, we provide recommendations for the utilization of on-site resources, as well as address options for rock disposal/utilization in an effort to optimize development costs. Petra utilized the findings of the previous geotechnical/geologic studies, where deemed appropriate, to minimize the duplication of effort.

Because development plans are still in the formative stage, the approach to this phase of geotechnical investigation is to first refine the geologic map prepared in the earlier studies. Field exploration consisted of geologic mapping and subsurface exploration for the purpose of geologic definition and obtaining representative samples of each of the geologic units. Laboratory testing focused on the engineering properties inherent in each of these geologic units. Subsequent analysis identified the potential impact of these properties on various aspects of development in order to assist in the planning process. By quantifying the geotechnical impact, such as potential fault traces, dynamic settlement-prone areas, presence of collapsible soils, removal depths, slope stability issues, oversize rock generation and disposal, etc., the development plan can be superimposed on the geologic map to evaluate the cost impacts and thereby improve the cost-effectiveness of the concept.

Based on our preliminary review of the previous geotechnical reports and available background literature, and on the experience of our firm with similar projects in the area of the subject site, the primary geotechnical issues addressed in this report include the following:

- Faulting, particularly the presence of previously noted lineaments, and local seismicity.
- The presence of native alluvial soils that may be prone to excessive consolidation, hydro-collapse, or dynamic settlement.
- The abundance of cobble- and boulder-sized rock in the natural alluvial soils that will generate significant quantities of oversize material requiring both special handling and rock disposal during grading.
- The alluvium that may be suitable for processing as aggregate base, pipe bedding, riprap, and other rock products.
- Approximate overexcavation and re-compaction depths that will be required to mitigate excessive settlement.
- Shrinkage and subsidence estimates.
- Presence of locally adverse geologic structure (slope stability).
To address the above concerns and provide sufficient geotechnical/geologic information for conceptual design, as previously noted, our scope of services consisted of the following:

- Performed supplemental geologic mapping within the site.
- Reviewed available published geotechnical literature and maps, as well as the previous site-specific geologic/geotechnical reports prepared by previous consultants to determine existing soil and geologic conditions within and adjacent to the subject property.
- Coordinated with the local underground utilities locating service (Underground Service Alert) to obtain an underground utility clearance prior to commencement of the subsurface investigation.
- Excavated 12 exploratory test pits with a backhoe to further evaluate near-surface geologic conditions. This activity was also combined with hand-auger drilling and in-situ moisture and density determination within selected test pits.
- Collected representative bulk and undisturbed soil samples for laboratory analysis.
- Performed appropriate laboratory analyses on soil samples, including determination of some or all of the following: in-situ density and moisture content, maximum density and optimum moisture content, consolidation and hydro-collapse characteristics, shear strength, grain size distribution, sand equivalent, R-value, abrasion resistance, soundness, and general corrosivity indicators (soluble sulfate and chloride content, soil pH and minimum resistivity).
- Performed analyses of potential dynamic settlement of dry sands.
- Performed slope stability analyses for typical cut and fill slope configurations anticipated to be part of the project design.
- Prepared a geotechnical map showing the distribution of geologic units, faults.
- Consulted with you at the completion of our field and laboratory work to discuss our findings and conclusions.
- Performed appropriate geologic and engineering analyses on all data collected.
- Prepared a preliminary geotechnical investigation report presenting our findings, conclusions and recommendations.

**FIELD INVESTIGATIONS AND LABORATORY TESTING**

The field exploration of the site was conducted in September 2013 and consisted of geologic mapping, and the excavating logging and sampling of 12 exploratory test pits to depths ranging from approximately 6 to 14 feet. These test pits were excavated utilizing a rubber-tired, 4-wheel drive backhoe. A geologist from this firm logged the test pits. Soil and bedrock materials encountered within the test pits were visually
classified and logged in general accordance with the Unified Soil Classification System (ASTM D2488) and the Engineering Geology Field Manual by the U.S. Department of the Interior, Bureau of Reclamation, respectively. Approximate locations of the exploratory test pits are shown on the Geotechnical Map, Figure 2, and descriptive logs for the test pits are presented in Appendix A.

**FINDINGS**

**Regional Geology**

The site is located at the southern edge of the Eastern Transverse Ranges geomorphic province, which is a subprovince of the east-west trending Transverse Ranges geomorphic province of California. The Coachella Valley just to the west of the site is part of the Colorado Desert geomorphic province. The San Andreas Fault, located along the northeastern edge of the Coachella Valley, forms the approximate boundary between the provinces. The Eastern Transverse Ranges are comprised of en echelon east-west trending mountain ranges that include the San Bernardino, Little San Bernardino, Pinto, Hexie, Cottonwood, Eagle Mountains, Orocopia, Chuckwalla and Little Chuckwalla Mountains (Powell, 1981).

Typical lithographic units within the Eastern Transverse Ranges consist of pre-Tertiary crystalline rocks of plutonic and metamorphic origin. Tertiary sedimentary and volcanic rocks are also present, but less common. Between the mountain ranges are Quaternary sediments that consist of coarse-grained alluvial fan and wash deposits with finer grained sediments in the central basin areas. The internal structural trend of the province is controlled by several east-west fault zones, including the Pinto Mountain, Blue Cut, Chiriaco, Salton Creek and Aztec Mine Wash faults.

The site lies within alluvial Shavers Valley and the southern edge of the Cottonwood Mountains, where granitic and metamorphic rocks are exposed (see Regional Geologic Map, Figure 3). The Orocopia Mountains are located southeast of the site and the Mecca Hills are to the southwest.

**Local Geology**

Most of the site is underlain by Quaternary alluvium (Qal) and Quaternary older alluvium (Qoa₁ and Qoa₂). A limited exposition of the Quaternary Ocotillo Formation is located at the southwest corner of the site and pre-Tertiary granitic and gneissic rocks are located in the Cottonwood Mountains at the northern edge of the site (see Geotechnical Map, Figure 2). The geologic units are discussed further below.

**Quaternary Younger Alluvium:** Quaternary younger alluvium (Qal) is common through the central part of the site, dominated by braided stream, and alluvial fan deposits along the Pinkham Wash, a large drainage
that originates from the area between the Little San Bernardino Mountains and the Cottonwood Mountains northwest of the site. Younger alluvium is also present in the active drainage channels emanating from the Cottonwood Mountains at the north of the site. The younger alluvium primarily consists of sandy gravel and gravelly sand, with areas of clean sand. The gravel is characteristically coarse, with abundant cobbles. Boulders are also present and generally increase in size and number towards the mountains.

**Quaternary Older Alluvium**: Quaternary older alluvium is located in the north-central and southwest portions of the site (Figure 2). The older alluvium also consists of coarse-grained sandy gravel, but differentiated by its reddish brown color, partial cementation and the presence of desert varnish on the surface. Two units of older alluvium (Qoa₁ and Qoa₂) are designated on the Site Geotechnical Map (Figure 2) based on the development of desert varnish. The desert varnish is significantly more developed on Qoa₂ compared to Qoa₁, which indicates a long stable ground surface. Both the Qoa₁ and Qoa₂ are estimated to consist of Pleistocene-age (>11,000 years old) sediments.

**Quaternary Ocotillo Formation**: Quaternary Ocotillo Formation (Qo) is limited to the southwest corner of the site (Section 15), which slightly encroaches into the Mecca Hills. The Ocotillo Formation consists of Pleistocene-age gravelly sand/sandy gravel. The Ocotillo Formation is distinguished from adjacent older alluvium by an erosional elevated surface versus the stable desert varnish surface of the older alluvium.

**Pre-Tertiary Granitic and Gneissic Rocks**: The pre-Tertiary granitic and gneissic rocks (gr/gn) are located at the north edge of the site in the Cottonwood Mountains. The gneiss consists of a coarse-crystalline layered metamorphic rock and the granitic rock consists mostly of a coarse-crystalline granodiorite. The granitic and gneissic rocks are moderately to slightly weathered and are characteristically hard.

**Groundwater**

The site lies in the Orocopia Groundwater Basin. Based on well records, the groundwater generally exceeds a depth of 200 feet in the site area. Two well records are available from the California Department of Water Resources (DWR), one of which is the onsite well in the central part of the site; the other is located approximate two miles east of the site (see Figure 1). Records from the onsite well (06S10E11N001S) indicate a depth to groundwater of about 330 feet (~ water surface elevation, 945 feet above mean sea level) between 1952 and 1985. DWR records for the offsite well (06S11E16E001S) indicate a depth of approximately 245 feet (~water surface elevation, 1,074 feet above mean sea level) between 1933 and 1994.

As part of the Paradise Valley Project a groundwater study was conducted by Dudek (2016). As part of the study groundwater monitoring wells were installed in the central portion of the project. Depth to
groundwater measured in December 2014 January 2014 showed depth groundwater between 202 and 364 feet below ground surface corresponding to a groundwater elevation of 887 and 948 feet above mean sea level. That report is in Draft form as of the writing of this report.

**GEOLOGIC HAZARDS**

The following section discusses various potential geologic hazards with respect to the proposed project site. The issues addressed include risks associated with active faults, strong seismic ground shaking, seismic-related ground failure such as liquefaction, landslides, subsidence, and flooding.

**Fault Rupture**

The site is not located within a currently delineated State of California Alquist-Priolo Earthquake Fault Zone (Hart and Bryant, 1999), or a Riverside County identified fault zone. In addition, no known active faults have been identified on the site. Previous regional geologic maps (Jennings 1967, Gutierrez and other, 2010, and Jennings and Bryant, 2011), however, show a fault within older alluvium just west of the site, which projects through the southwest corner of the site (Section 15). In addition, the ESS (2005) report discusses some possible fault-related aerial photo lineaments they observed, although the report does not define the location of these. These lineaments likely correspond to the faults shown on their Geologic Map (ESS Figure 4), which is based on the previous regional maps. Two fault identified as the North and South Chiriaco Faults have been mapped as trending into the site from the east and are mapped as concealed (i.e. they do not reach the surface). They are not considered active they do not reach the surface and we did not observe evidence of activity during our investigation. The Regional Geologic Map (Figure 3) and the Regional Fault Map (Figure 4) shows these faults. One of these is a northwest trending buried fault (Dillon Fault) drawn between bedrock faults in the Little San Bernardino and Orocopia Mountains. The other fault, mentioned above, is at the southwest part of the site. An approximately east-west trending fault in the pre-Tertiary bedrock of the Cottonwood Mountains is shown. On the State of California fault activity map (Jennings and Bryant, 2010-included as Figure 4), only the fault at the southwest corner is shown as Quaternary (potentially active). This south west corner fault does encroach in to the planned development area. The other two faults that traverse the site are shown as inactive.

To better assess photo-lineaments on the site, Petra obtained and reviewed several sets of photos (see references). The northwest trending fault traversing through the middle of the site (see Figures 3 and 4) corresponds to the trend of the Pinkham Wash drainage, however no lineaments beyond that of the drainage pattern are observed. The “potentially active” fault mapped at the southwest corner of the site also roughly follows drainage patterns, from an unnamed wash originating from the east end of the Little San Bernardino
Mountains. A possible lineament is also formed by the nearby contact between the older alluvium (Qoa2) and the Ocotillo Formation (Qo). Both of these possible lineaments, however, project to areas of the older alluvium, which is an old (Pleistocene) surface. In addition, no offset drainages or other evidence of faulting was observed on the photos. Based on our observations, it is our opinion that a potentially active fault is not located in this area. Regional mapping by Dibblee (2008) did not map these faults either.

A geophysical study by GeoVision (2005) detected faults within the basement (crystalline bedrock) buried by hundreds of feet of alluvium. These faults are considered to be part of the Chiriaco Fault Zone. The Chiriaco fault is one of the prominent east-west trending faults that separate structural blocks of the Eastern Transverse Ranges, but has not been active in the Quaternary (Powell, 1981, Spinler and others, 2010) and is therefore considered inactive. Accordingly, the Chiriaco Faults is shown as inactive on the State’s Fault Activity Map (see Figure 4).

Based on the information summarized above, it is our opinion that on-site lineaments are primarily related to drainage courses. Evidence for the presence of a potentially active fault is not present and a subsurface fault investigation, as suggested by ESS, does not appear to be warranted. It is our opinion, therefore, that the potential for active fault rupture on the site is very low.

According to State of California fault definitions (CGS, 2003), an “active” fault has had displacement within the Holocene epoch or about the last 11,000 years. A “potentially active” fault is a fault that does not have evidence of movement within the last 11,000 years, but has moved within the Quaternary period, the last 2.6 million years. “Potentially active” faults are not placed within Alquist-Priolo Earthquake Fault Zones, but are considered when placing such critical structures as dams and nuclear power plants, etc.

**Seismic Shaking**

The site is located within an active tectonic area with several significant faults capable of producing strong earthquakes. The Regional Fault Map, Figure 4, shows the significant faults in the site area. The closest known active fault is the northwest trending San Andreas Fault, located approximately 5 miles southwest of the site. Other important regional faults include the San Jacinto fault zone located approximately 26 miles southwest of the site, and the Pinto Mountain Fault located approximately 28 miles to the north. The Mojave Desert area contains several northwest trending right lateral strike-slip faults that could also affect the site area, such as the Johnson Valley, Eureka Peak and Burnt Mountain faults, which ruptured during the 1992 Landers Earthquake.
The nearby San Andreas Fault, however, represents the most significant risk to strong seismic shaking at the site. Based on studies by the in the Third Uniform California Earthquake Forecast (UCERF3) Prepared by the Working Group on California Probabilities (Field et al, 2014), the probability of an earthquake of \( \geq 6.7 \) magnitude within the next 30 years (starting in 2014) is 24 percent for the Coachella Section of the Southern San Andreas fault. The last earthquake on this segment occurred in about the year 1680.

The faults discussed above, as well as other regional faults, contribute to the potential ground shaking at the subject site. Based on probabilistic analysis from the California Geological Survey, peak ground acceleration at the site is estimated to be approximately 0.55g. This probability analysis takes into account the earthquake histories; slip rates, and potential earthquake magnitudes of significant regional faults. The proposed project will be constructed in conformance with the California Building Standards Code (CBC) in order to minimize seismic impacts.

### Table 1

**Significant Historical Earthquakes**

<table>
<thead>
<tr>
<th>Historical Earthquake</th>
<th>Approximate Epicentral Distance from center of Project Site (mi)</th>
<th>Moment Magnitude (Mw)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hector Mine (Oct. 16, 1999)</td>
<td>67</td>
<td>7.1</td>
</tr>
<tr>
<td>Landers (June 28, 1992)</td>
<td>47</td>
<td>7.6</td>
</tr>
<tr>
<td>Big Bear (June 28, 1992)</td>
<td>64</td>
<td>6.7</td>
</tr>
<tr>
<td>Superstition Hills (Nov. 24, 1987)</td>
<td>44</td>
<td>6.4</td>
</tr>
<tr>
<td>Borrego Mountain (April 8, 1968)</td>
<td>34</td>
<td>7.7</td>
</tr>
<tr>
<td>Imperial Valley (May 18, 1940)</td>
<td>68</td>
<td>7.8</td>
</tr>
</tbody>
</table>

**Secondary Effects of Seismic Activity**

Secondary effects of seismic activity normally considered as possible hazards to a site include liquefaction, several types of ground failure, and earthquake-induced flooding. The potential for liquefaction at the site is likely to be negligible due to the absence of shallow groundwater. However, the site is located within a Riverside County zone with low to moderate potential for liquefaction. Future geotechnical studies are therefore required to assess the potential for liquefaction at the site.

Similar to liquefaction, settlement of dry sand can be induced by strong earthquake shaking. Deposits of young, deep, loose, low density (unconsolidated alluvium) may be prone to dry sand settlement. Young, shallow, dense sandy soils are more resistant to dry sand settlements. Older consolidated alluvium and
bedrock units are not likely to be prone settlement from earthquake shaking. Where younger deposits are present, but underlain at shallow depths by older alluvium, the amount of dry sand settlement may be relatively small. Future geotechnical studies may consider dry sand settlements were younger low density sandy alluvial soils are encountered and structures are planned.

Various general types of ground failures, which might occur as a consequence of severe ground shaking at the site, include landsliding, ground subsidence, and ground lurching. The probability of occurrence of each type of ground failure depends on the severity of the earthquake, distance from faults, topography, subsoils and groundwater conditions, in addition to other factors. Based on the site conditions and gently sloping topography, the above secondary effects of seismic activity are considered unlikely at the site.

Seismically induced flooding that might be considered a potential hazard to a site normally includes flooding due to tsunami or seiche (i.e., a wave-like oscillation of the surface of water in an enclosed basin that may be initiated by a strong earthquake) or failure of a major reservoir or retention structure upstream of the site. No major reservoir is located near, or upstream of the site so the potential for seiche or inundation is considered negligible. Because of the inland location of the site, flooding due to a tsunami is also considered non-existent at the site.

**Landslides and Slope Instability**

No landslides are known to exist on the site. Within the steeper areas in the Cottonwood Mountain of the northern edge of the site, there is a moderate to high potential for landsliding. However, the shear strength of the granitic and metamorphic rock in this area is likely to be strong and not conducive to rotational type landslides. More likely, the common type of slope failure in the northern mountainous part of the site are topples and rock fall, particularly when induced by strong earthquakes. For the majority of the site, where there are relatively gentle slope gradients, the potential for landslides is negligible.

**Debris Flow**

The potential for debris flow occurrence is high within the active drainage of the site. Debris flows will occur during the sporadic, but strong torrential rains that occur in the site area.

**Areal Subsidence**

Based on the coarse alluvium at the site, it is unlikely that the area is susceptible to subsidence due to withdrawal of groundwater. The site is, however, located in an area designated by the Riverside County (Riverside County Information Technology Website - Parcel Reports) as active or susceptible to subsidence.
from seismic shaking due to earthquakes. Future geotechnical studies will be required to evaluate potential subsidence, which may more likely be related to seismic settlement, or consolidation/hydroconsolidation of the surficial soils.

**Expansive Soils**

Expansive soils generally result from specific clay minerals that expand in volume when saturated and shrink in volume when dry. Based on the sandy alluvium at the site, the potential for expansive soil at the site is considered to be very low.

**Flooding and Erosion**

The high potential for local flooding and erosion exists in the drainage courses within the site. Most of the area mapped as younger alluvium (Qal) is included in a Riverside County Flood Zone. The areas mapped as older alluvium on the site, have a low flooding potential, based on the stable ground surface that has developed desert varnish.

**CONCLUSIONS AND RECOMMENDATIONS**

**General Feasibility**

Based on our preliminary geotechnical investigation, the Paradise Valley project is underlain primarily by alluvial soils that may present minor geologic/geotechnical issues relative to the feasibility of the proposed project. These include coarse-grained soils, and local flooding. These issues, among others, are discussed further below.

**Fault Rupture**

Several faults have been previously mapped on or near the site and possible fault related photo-lineaments were observed by ESS (2005). A more detailed aerial photograph analysis was conducted for this investigation. It is our opinion that the photo-lineaments observed are primarily related to site drainages and do not indicate the presence of active faults on the site. A subsurface fault investigation is not warranted, based on our findings. The site is not located within a State of California Alquist-Priolo Earthquake Fault Zone (Hart and Bryant, 1999), or a Riverside County fault zone. The potential for fault rupture at the site is considered to be low.

**Seismic Shaking**

Strong ground shaking is expected at the site as a result of an earthquake generated by the San Andreas fault. The proposed development should be designed and constructed according to the latest building codes.
Coarse Alluvium

Both the younger and the older alluvium at the site is characteristically coarse. Cobble sized gravel (6 to 12 inches in diameter) is common. Boulders from 1 to over 5 feet in diameter are also present, and generally increase in size and number toward the north. This material will require special handling during the proposed grading of the site, such as using crushing operations to reduce grain size in order to place compacted fill, placing oversize material in rock blankets/windrows in areas of deeper fill (likely greater than 10 feet below finish grades), or placement in non-structural areas (subject Riverside County approval).

Hard Bedrock

Hard granitic and gneissic rocks are located at the northern part of the site in the Cottonwood Mountains. While it is our understanding that the primary development will not encroach into the Cottonwood Mountains, a tank site may be proposed at the higher elevations. Grading in this area may require special excavation techniques, such as pneumatic hammers or blasting.

Subsidence/Settlement

Riverside County has designated the area as active or susceptible to subsidence. Future geotechnical studies will be required to evaluate potential subsidence, as well as the potential for seismic settlement, and consolidation/hydroconsolidation of the surficial soils.

Flooding, Erosion and Debris Flow

The high potential for local flooding, erosion and debris flow exists in the drainage courses within the site. Civil engineering design will be necessary to develop the areas located in potential flood hazards.

Grading Plan Review

Grading plans (one inch = 400 feet scale) prepared by KWC Engineers were reviewed to provide a general comment as to the feasibility from a geotechnical standpoint. The mass grading concepts discussed below in italics for the site were prepared by KWC Engineers in a Technical Memorandum dated May 1, 2015.

Mass Grading Concepts

The proposed mass grading concepts intend to mimic existing topography conditions by protecting existing wash banks and utilizing the natural course of runoff, while still protecting the proposed development from substantial flood events. Grading operations in the northern area will be minimal due to site constraints which include rocky sub-material and rock outcroppings. Existing rock outcroppings will serve as buffer zones between the areas that will be mass graded, and areas that are too steep and will remain undisturbed.
In an effort to limit unnecessary disturbed acreage within each phase of development, the earthwork in each grading area of the project will balance within itself. This will insure that the development is in accordance with the requirements set forth in the Multiple Species Habitat Conservation Plan.

In developing a mass grading concept for the Paradise Valley Project, drainage conveyance was given special attention in an effort to accept and deliver storm runoff to and from existing drainage paths and flowlines.

As Interstate-10 bisects the project from east to west, it creates several points of concentrated flow between the northern planning areas and the southern planning areas. There are approximately 24 existing culverts and bridges along Interstate 10 with direct conveyance to the project property. The intent of the grading concept is to modify the landform enough to take these concentrated flows and re-distribute them in a less concentrated form. In order to minimize the effects of the concentrated flows through the project site, and through the Southern California Edison (SCE) Easement, the project proposes to design and construct reinforced concrete boxes, and/or natural channels to deliver storm runoff through the site. This will reduce the effects of scouring throughout the site. The challenge in mitigating the effects of the concentrated flows and the flows generated by our proposed development is to disburse the runoff in a natural and less concentrated fashion. Onsite runoff will be picked up by a combination of grate inlets, catch basins, and swales as part of the onsite storm drain system.

Onsite grading and drainage facilities will include the development of detention/infiltration facilities in each of the planning areas for runoff conveyance and treatment. Furthermore, import and export of earthwork material will balance within each phase and/or proposed Village. Excess rocks and boulders generated during grading operations will be reincorporated for base material, aesthetics, or reburied on-site in appropriate areas. Proposed drainage areas and flow lines will be designed at a minimum grade when feasible to allow for maximum percolation and reduced erosion.

**Northern Grading**

In general, grading operations north of the I-10 freeway will vary in relation to operations south of the freeway. The exposed outcroppings along the mountain range typically consist of a granitic and gneissic rock. North of the I-10 Freeway, existing grades are generally between 6 percent and 9 percent. Additional design criteria for the northern area includes:

- Project related grading will occur approximately 20’ from the banks of the existing drainage channels.
- Grading located adjacent to preserved rock outcrops will provide embankments to capture rock fall.
- Proposed cuts for the area north of I-10 range from approximately 1’ to 10’.
- All disturbed areas will be phased in accordance with the Multiple Species Habitat Conservation Plan regulations.
- Proposed concepts for the northern area will generally follow the existing grades and create slopes between 0.5 percent for the commercial areas, and 9 percent for all other grading areas.
- The related cuts range from approximately 0’ to 10’ and fills range from 0’ to 30’ with the exception of the interchange underpass grading requirements.
- Reservoir Water Tanks sites will be located along the western and eastern slopes of the Cottonwood Mountains and located where grading/access is feasible and cuts are minimized.

- Grading design and operation will also include the development of detention / Water Quality Management Plan facilities for runoff conveyance and treatment.

- Import and export of earthwork material will balance within each grading area.

- Excess rocks or boulders generated during grading operations will be reincorporated for base material, aesthetics, or reburied on-site in appropriate areas.

- Proposed drainage areas and flow lines will be designed at a minimum grade when feasible to allow for maximum percolation and reduced erosion.

**Southern Grading**

Existing terrain south of Interstate 10, gradually increases from the southern planning areas from about 2 percent to about 6 percent north towards the freeway right-of-way. Furthermore, on-site materials are suitable for engineered fills for the construction of roads, fill slopes and building pads. Additional design criteria for the southern area includes:

- Along the western project footprint, also being the eastern banks of Pinkham Wash, the proposed development will be graded to a minimum of five (5) feet above the existing terrain.

- Along the eastern project footprint the proposed development will be graded to a minimum of five (5) feet above the existing terrain.

- The main site entrance will be graded in anticipation of the proposed ultimate underpass construction.

- All disturbed areas will be phased in accordance with the Multiple Species Habitat Conservation Plan regulations.

- Proposed concepts for the southern area will generally follow the existing grades and create slopes between 0.5 percent (for the commercial areas) and 6 percent.

- Project related cuts range from 0’ to 30’ and fills range from 0’-40’.

- Grading design and operations will include the development of three major off-site drainage interceptor inlets to divert flows to an on-site storm drain conveyance system delivering runoff to the existing flow lines along the southern/downstream ends of the project footprint.

- Grading design and operation will also include the development of detention / Water Quality Management Plan facilities for runoff conveyance and treatment.

- Import and export of earthwork material will balance within each grading area.

- Excess rocks or boulders generated during grading operations will be reincorporated for base material, aesthetics, or reburied on-site in appropriate areas.
• Proposed drainage areas and flow lines will be designed at a minimum grade when feasible to allow for maximum percolation and reduced erosion.

• Grading areas located adjacent to the proposed offsite storm inlet will be elevated a minimum of 10-feet above the existing terrain for flood protection purposes.

I-10 Interchange
As mentioned earlier, the existing I-10 Freeway and Frontage Road Interchange will be improved and phased along with the project. An additional underpass will be constructed along with later phases of the project and the grading associated with the preliminary underpass design was incorporated into the mass grading earthwork analysis.

It is our opinion, based on our review of the KWC Technical Memorandum dated May 1, 2015 and the exhibits provided by KWC Engineers, that the site grading is feasibly from a geotechnical Standpoint. Additional geotechnical investigation and/or review will be needed as larger scale grading plans are developed for the phases and prior to issuing grading plans.

Electrical Transmission Alignment
The options to get electricity may require a new electrical transmission line. The alignments under consideration are presented on Figure 5. The general geology is also shown on Figure 5. The geology along the alignments in the eastern ¾ is typically young alluvial soils similar to those encountered at the project site. In the western approximately ¼ the geologic units encountered are young sedimentary units consisting of siltstone/claystone, sandstone, and Colmerate of the Palm Spring Ocotillo, and Canebreak formations, respectively. All of the earth material expected be encountered along these alignment are expected to excavateable using conventional powerline construction equipment providing access is made available. The western ¼ of alignment 2 goes through relatively steep terrain and of access road for construction and later maintenance will be necessary.

The San Andreas fault is present several hundred feet east of the Avenue 52 (Coachella) Substation at the western end of the power line alignment options (see Figure 5). Existing power lines cross the San Andreas fault from the Coachella Substation to the 230 kv transmission line owned and operated by the IID. This situation, transmission lines crossing the surface trace of active faulting, is unavoidable and is present in several location in the Coachella Valley.
Engineering Characteristics of On-Site Soil Materials

Soil Material Classification

Grain size distribution tests were performed on selected samples to classify the on-site soils in accordance with the Unified Soil Classification System (ASTM C136). The predominate soil types comprising the older alluvium (Qoal₁ and Qoal₂) and younger alluvium (Qal) can generally be classified as poorly-graded gravel (GP), silty gravel (GM) and poorly-graded sand with gravel (SP). The amount of cobbles and boulders observed in the test pits was estimated at about 20 to 45 percent of the total volume of excavated materials.

A 2.5 foot thick layer of clayey sand (SC) was encountered at the surface in TP-8 and at a depth interval of 5.0 to 6.0 feet in TP-12.

Excavation Characteristics

Based upon the results of our subsurface exploration and experience associated with grading of projects underlain with similar soils, it is our opinion that the on-site earth materials can be excavated with conventional earth moving equipment. However, self-loading scrapers may experience difficulty in areas containing significant cobbles and boulders.

Soil Suitability for Use as Fill and Backfill

General – On-site earth materials are generally considered suitable for use as engineered fills in the construction of building pads, roadways and fill slopes.

Fill Slope Construction – Care should be taken in the selection of fill materials to be used in the construction of the outer 10 to 15 feet of fill slope faces to avoid clean sands with little or no fines content (silts and clays). Such materials are highly erodible and will require special treatment to improve the surficial stability of the slope face.

Building Pad Construction – Building pads should be capped with approximately 2 to 3 feet of soil containing a maximum rock size of 3 inches to facilitate excavation of footing and plumbing trenches.

Street Subgrade – As with building pad construction, the upper 1-foot of the subgrade soils in street areas should be void of cobbles exceeding a maximum size of 3 inches. This will mitigate the potential for protrusion of cobbles through the asphalt pavement due to repeated traffic loads.

Trench Backfill – The majority of the sands encountered on the site are considered well-suited for use as structural backfill behind retaining structures and for backfilling of utility trenches; however, the sands utilized for these purposes should first be screened to rid the soils of cobbles exceeding a maximum dimension of 3 inches.
Oversize Materials

Highly variable amounts of oversized materials (boulders greater than twelve inches in maximum diameter) will be generated during grading. Oversize materials also exist at the surface in many areas of the site. Oversized materials should be handled as recommended in the “Earthwork Recommendations” section of this report.

Earthwork Adjustments

Volumetric changes in earthwork quantities (shrinkage) will occur when excavated on-site soils are placed as compacted fill. Additional adjustment factors will be necessary due to subsidence when exposed bottom surfaces in over-excavated areas are scarified and recompacted in accordance with the recommendations presented in the “Earthwork” section of this report. The following estimated earthwork adjustment factors are recommended for use by project planners in an effort to balance earthwork quantities.

Table 2

<table>
<thead>
<tr>
<th>Geologic Unit</th>
<th>Map Symbol</th>
<th>Shrinkage Adjustment Factors</th>
<th>Subsidence Factors</th>
</tr>
</thead>
<tbody>
<tr>
<td>Topsoil</td>
<td>None</td>
<td>Shrinkage of 10 to 15 percent</td>
<td>Not applicable – soils requiring complete removal</td>
</tr>
<tr>
<td>Wash Deposits</td>
<td>None</td>
<td>Shrinkage of 10 to 15 percent</td>
<td>Not applicable – soils requiring complete removal</td>
</tr>
<tr>
<td>Younger Alluvium</td>
<td>Qal</td>
<td>Upper 2 feet: Shrinkage of 10 to 15 percent</td>
<td>0.10 to 0.15 feet</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Below 2 feet: Shrinkage of 5 to 10 percent</td>
<td></td>
</tr>
<tr>
<td>Older Alluvium</td>
<td>Qoa1</td>
<td>Upper 2 feet: Shrinkage of 10 to 15 percent</td>
<td>0.10 to 0.15 feet</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Below 2 feet: Shrinkage of 5 to 10 percent</td>
<td></td>
</tr>
<tr>
<td>Very Old Alluvium</td>
<td>Qoa2</td>
<td>Shrinkage of 5 to 15 percent</td>
<td>0.05 to 0.10 feet</td>
</tr>
</tbody>
</table>

The above estimates of shrinkage and subsidence are based on available subsurface data and should not be considered absolute values. Therefore, contingencies should be made during the initial phases of site development for balancing earthwork quantities based on actual shrinkage and subsidence that will occur during the initial phases of grading. Furthermore, the shrinkage factors will increase in areas where oversize materials are removed from the site and not buried on-site in designated rock disposal areas.

Low-Density Surface Soils and Compressibility

Surficial soil deposits overlying the various geologic units are compressible in their existing state and will require removal to underlying competent materials from all areas planned to receive fill. Estimated depths
of removal of unsuitable materials are indicated in the following table. These materials, once properly moisture-conditioned, will be suitable for use as engineered fill.

### Table 3
**Removal Depths**

<table>
<thead>
<tr>
<th>Geologic Unit</th>
<th>Map Symbol</th>
<th>Estimated Depths of Unsuitable Surficial Deposits</th>
</tr>
</thead>
<tbody>
<tr>
<td>Younger Alluvium</td>
<td>Qal</td>
<td>Up to 10 feet</td>
</tr>
<tr>
<td>Older Alluvium</td>
<td>Qoa₁</td>
<td>Up to 6 feet</td>
</tr>
<tr>
<td>Very Old Alluvium</td>
<td>Qoa₂</td>
<td>Up to 6 feet</td>
</tr>
</tbody>
</table>

**Consolidation and Hydro-Collapse Potential**

Due to the generally gravelly nature of the onsite soils, and the presence of cobbles and boulders, on-site soils are not considered subject to excessive consolidation or hydro-collapse under existing or proposed overburden pressures.

**Seismically Induced Dynamic Settlement of Dry Sands**

Due to the absence of shallow groundwater and the physical characteristics of the onsite soils as described above, the potential for seismically-induced liquefaction is considered negligible. The potential for seismically-induced dynamic settlement of dry sands is considered very low for consolidated older alluvial areas of the site. Further site specific assessment of dry sand settlement potential should be conducted during engineering design development phases of the project where structures may be planned and young, loose, low density, deep deposits of unconsolidated alluvium are present (such as along active washes). Young, shallow, dense sandy soils are more resistant to dry sand settlements. The magnitude of any dry sand settlements are likely to be within the range that can be handled by structures designed according to current standards of practice where structures are designed to prevent collapse and maintain life safety during earthquakes, but may sustain major damage during the event, even to the point where they may be unusable afterword. Remedial grading can also be used to mitigate any potential for dry sand settlements of onsite soils.

**Expansion Potential**

Selected earth materials were tested for expansion potential in accordance with ASTM D4829. The test results indicate the majority of the onsite soils are granular in nature and non-expansive, exhibiting Expansion Indices of less than 20. However, an Expansion Index of 22 (Low expansion potential) was
determined for a sample of interbedded clayey sand encountered in TP-12 at a depth interval of 5.0 to 6.0 feet.

**Soil Corrosivity Potential**

As a screening level study, limited chemical testing was performed on selected representative samples of the on-site soils to identify potential corrosive characteristics in accordance with applicable California (Caltrans) test methods. The following table presents a summary of the preliminary testing for three geologic units. Soil corrosivity potential and mitigation measures are discussed in greater detail in a following section of this report.

**Table 4**

<table>
<thead>
<tr>
<th>Geologic Unit</th>
<th>Map Symbol</th>
<th>Soluble Sulfate (%)</th>
<th>pH</th>
<th>Chloride Content (ppm)</th>
<th>Resistivity (ohm-cm)</th>
<th>Corrosivity Potential</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Old Alluvium</td>
<td>Qoa₂</td>
<td>0.030</td>
<td>7.9</td>
<td>73</td>
<td>2,000</td>
<td>Concrete: Negligible Metals: Moderate</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.060</td>
<td>7.4</td>
<td>80</td>
<td>1,200</td>
<td></td>
</tr>
<tr>
<td>Younger Alluvium</td>
<td>Qal</td>
<td>0.42</td>
<td>7.5</td>
<td>185</td>
<td>620</td>
<td>Concrete: Severe Metals: Moderate</td>
</tr>
</tbody>
</table>

**Global Stability of Proposed Cut and Fill Slopes**

Cut and fill slopes proposed within the development should be planned at a maximum slope ratio of 2:1 (h:v). When specific grading plans have been prepared, limit equilibrium slope stability calculations should be performed for the most critical (highest) cut and fill slopes. Based on the direct shear tests presented in Appendix B, it is anticipated that 2:1 (h:v) cut and fill slopes may be safely constructed to heights of about 75 feet.

**Surficial Slope Stability**

Proposed cut and fill slopes are anticipated to be marginally surficially stable due to the granular nature of the onsite soils. The standard calculations for surficial stability are based on an infinite slope height and a vertical depth of saturation of 4 feet with seepage parallel to the slope face. The analysis utilizes soil shear strength parameters of angle of internal friction and cohesion at low stresses. However, because of the low values of cohesion inherent in the on-site materials, the calculations typically do not reflect the minimum factor of safety of 1.5.
It should be noted, however, that where the soils comprising the slope face are sufficiently dense and do not expose clean, running sands, the primary concern is that of erosion, rather than shallow instability. To that end, several methods are available to enhance erosion resistance of the slope face. While general measures are discussed in Slope Landscaping and Maintenance, measures specifically intended to reduce erosion potential of the slope face include:

**Enhanced Compaction of Fill Slope Face** – Grading specifications typically require that fill comprising the fill slope be compacted to 90 percent or more relative compaction, extending to the slope face. Specifying a higher compaction requirement, such as 95 percent relative compaction, reduces the permeability of the soils, thereby increasing the in-situ soil strength and reducing the potential for saturation. The compaction standard notwithstanding; however, overbuilding the slope face and trimming back to the compacted core is preferred over the method of constructing the slope face to finish grade with periodic grid-rolling and final track-walking of the slope surface. A uniformly compacted slope face is essential to both long-term surficial stability and erosion resistance.

**Select Landscaping** – All engineered slopes should be landscaped as soon as practical at the completion of grading. As noted, the landscaping should consist of a deep-rooted, drought-resistant, and maintenance-free plant species. Hydro-seeding with select plant species native to the area that exhibit the aforementioned properties can expedite the process of establishing vegetation prior to periods of heavy or extended rainfall. Depending on the time of year, initial irrigation may be required to establish the plant growth.

**Jute Mat** – If landscaping cannot be established within a reasonable period of time, jute matting or equivalent should be considered as a short-term measure to inhibit surface erosion.

**Chemical Stabilizer** – Spray-on products designed to seal slope surfaces and reduce erosion potential are an option to jute mat.

**Geo-Textile Fabrics** – Geotextile fabrics are commercially available for use in the construction of fill slopes which are either over-steepened or subject to erosion. Such fabrics are utilized in the placement of engineered fill to construct a geotextile-wrapped slope face. Such measures are comparatively expensive and are usually reserved for specialty applications.

**Earthwork Recommendations**

**General Earthwork and Grading Specifications**

All earthwork and grading should be accomplished under the observation and testing of the project soils engineer and engineering geologist or his/her authorized representative. Earthwork and grading should be performed in accordance with the following recommendations prepared by Petra, the grading code of the County of Riverside, Chapter 33 Appendix J of the 2013 California Building Code (CBC), and Petra’s “Standard Grading Specifications” presented in Appendix C.
Site Clearing and Grubbing

All weeds, grasses, brush, shrubs, trees and similar vegetation should be stripped and removed from the site prior to any grading, as should all trash and debris. Large shrubs and trees, when removed, should be grubbed out so as to include their stumps and major root systems, and these organic materials removed from the site. Remaining roots exposed during grading will require hand labor for proper removal.

The project geotechnical consultant should be notified at the appropriate times to provide observation and testing services during clearing operations to verify compliance with the above recommendations. In addition, should any buried structures or unusual or adverse soil conditions be encountered during grading that are not described or anticipated herein, these conditions should be brought to the immediate attention of the project geotechnical consultant for corrective recommendations.

Pre-Watering

In-situ moisture contents of the on-site earth materials range from approximately 1 to 2 percent at the time of investigation. Due to the dry condition of the on-site soils, it is recommended that the site be pre-watered prior to grading to facilitate moisture-conditioning of the soils. Pre-watering should be performed over a period of 2 to 3 weeks utilizing a rainbird system.

Ground Preparation

Low-density surficial soil deposits overlying the various geologic units are compressible in their existing state and will require removal to underlying competent native materials from all areas planned to receive fill to mitigate possible excessive settlement of building foundations and street improvements. Competent native materials are defined as undisturbed soils possessing an in-place density equivalent to a relative compaction of 90 percent or more. Estimated depths of removal of unsuitable materials range from 1-foot to locally greater than 10 feet (see table of removals provided in a previous section). The estimated removal depths are based on exploratory test pit data, and our laboratory test results. However, it is emphasized that the required depths of removal can, and usually do, vary between points of exploration. For this reason, the actual removal depths will have to be determined during grading on the basis of in-grading observations and testing performed by representatives of the project geotechnical consultant.

All existing low-density surficial soils, once properly moisture-conditioned, will be suitable for use as engineered fill. Remedial grading and ground preparation should be performed prior to placing any planned fills. Prior to placing structural fill, exposed bottom surfaces in each removal area approved for fill should first be scarified to a depth of at least 6 inches, watered as necessary to achieve a moisture content that is
equal to or slightly above optimum moisture content, and then recompacted in-place to a minimum relative compaction of 90 percent.

Any low-density surficial soils exposed at finish grade in cut areas should also be removed and replaced as compacted fill.

**Observation of Ground Preparation**

All removal bottoms should be observed, tested and approved by the project geotechnical consultant and/or engineering geologist prior to fill placement.

**Canyon Subdrains**

Because the majority of the site is underlain by alluvial materials that are themselves fairly permeable, the potential build-up of hydrostatic pressures below compacted fills due to infiltration of surface waters is not likely. Therefore, canyon subdrains should not be required; however, the final determination should be based on additional onsite studies when definitive grading plans have been prepared. The project geotechnical consultant should designate the location of any required subdrain systems on the grading plans. Required subdrains should be constructed in accordance with Plate SG-1, Appendix D.

**Fill Placement**

All fills should be placed in 6- to 8-inch-thick maximum lifts, watered as necessary to achieve a moisture content that is equal to or slightly above optimum moisture content, and then compacted in-place to 90 percent or more relative compaction with reference to ASTM D1557. All fill subsequently placed shall be compacted to 90 percent or more relative compaction, except where the fill depths exceed 50 feet, in which case all fill placed deeper than 50 feet from finish grade should be compacted to 95 percent or more relative compaction. The laboratory maximum dry density and optimum moisture content for each change in soil type should be determined in accordance with Test Method ASTM D 1557.

**Settlement Monitoring**

Post-grading settlement of deep fills will occur due to their own weight. The fills within the site will be derived from soil that are primarily granular, have low clay contents, and have a very low expansion potential. Based on these conditions, it is expected that total primary consolidation of deep fill materials will be reached immediately at the completion of grading within areas underlain by 50 feet of compacted fill or less. In addition, considering the anticipated granular nature of the fill materials, long-term secondary
settlement of these materials is expected to be negligible. However, in areas underlain by approximately 50 feet (or more) of compacted fill, it is recommended that settlement monitoring be performed.

Surface monuments should be installed at finished grade in these deep fill areas immediately following completion of grading to verify post-grading settlement. Where relatively deep fills are proposed within the main canyon of a temporarily mass-graded area, monuments should be installed as soon as temporary design grades are reached. The survey monuments should be monitored on a weekly basis for the first three weeks, then once every two weeks for a total of one month. Subsequent readings should be taken once a month for three months, or whenever the settlement appears to stabilize. Additional settlement monuments may be required once finish pad grades are reached following future rough grading. Building construction should not proceed until it is determined by this firm that primary consolidation has occurred and that any further anticipated settlement will be within acceptable tolerable limits.

**Benching**

Fills placed on or against natural slope surfaces inclining at 5:1 (h:v) or steeper should be placed on a series of level benches excavated into competent bedrock or competent native soil materials. These benches should be provided at vertical intervals of approximately 3 to 5 feet. Typical benching details are shown on Plates SG-5 through SG-8, Appendix C.

**Mixing**

In order to prevent layering of different soil types and/or different moisture contents, thorough mixing of materials will be necessary prior to final compaction of each fill lift. Discing may be required for mixing of excessively dry materials.

**Disposal of Oversize Rock**

Oversize rock is defined as hard irreducible boulders exceeding 12 inches in maximum dimension. Oversize rock generated during grading operations should be removed from the site or placed in the lower portions of the deeper fills utilizing the typical detail shown on Plate SG-4, Appendix D. Any oversize materials buried on site should be placed individually or in windrows, and in a manner to avoid nesting, and then completely covered with granular on-site earth materials. The in-fill materials should be thoroughly watered and rolled to ensure closure of all voids. Oversize rock should not be placed within the upper 10 feet of finish grade within building sites or street areas where they may interfere with footing and utility trenches, or in areas where they may interfere with the future construction of swimming pools and/or spas. In areas where the in-situ quantity of cobble- and boulder-sized rock makes this 10-foot limit
unfeasible, the limit and rock size limitations may be modified at the discretion of the geotechnical engineer. Please note that where rock size limitations are increased, the associated cost of utility and foundation construction will increase accordingly.

**Processing of Cut Areas**

In shallow cut areas where unsuitable surficial materials are not removed in their entirety, these materials should be overexcavated to underlying competent materials and then brought back to grade with properly compacted fill. In deep cut areas where competent materials are exposed at grade, no special remedial work such as scarification or re-compaction will generally be required, other than processing the upper 1 foot of the building pads.

**Cut/Fill Transitions**

To mitigate distress to building structures related to the potential adverse effects of excessive differential settlement, cut/fill transitions should be eliminated from all building sites where the depth of fill placed within the "fill" portion exceeds proposed footing depths (e.g., 12 inches and 18 inches for one-story and two-story structures, respectively). This should be accomplished by overexcavating the "cut" portions and replacing the excavated materials as properly compacted fill. Recommended depths of overexcavation will depend on maximum depths of compacted fill placed on the "fill" portions, but will generally follow the guidelines provided below. Horizontal limits of overexcavation should extend beyond the perimeter building lines to a distance of 5 feet or to a distance equal to the required depth of overexcavation, whichever is greater. It is anticipated that finalized building locations will be unknown at the time the initial mass grading is performed to create the super pads. Therefore, elimination of cut/fill transitions will likely have to be performed when final grading operations are performed to develop individual building sites. If this is the case, cut/fill transition lines should be accurately shown on the as-built mass grading plans.

**Table 5**

**Overexcavation Depths**

<table>
<thead>
<tr>
<th>Depth of Fill</th>
<th>Depth of Overexcavation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Up to 3 feet</td>
<td>Equal depth</td>
</tr>
<tr>
<td>3 to 6 feet</td>
<td>3 feet</td>
</tr>
<tr>
<td>Greater than 6 feet</td>
<td>One-half of greatest fill depth placed on the &quot;fill&quot; portion; to 15 feet maximum</td>
</tr>
</tbody>
</table>
Capped Building Pads

To facilitate excavation of footing and plumbing trenches and minimize the need for forming of the footing trenches, all building pads should be capped with a minimum of 2 to 3 feet of soil that is free of cobbles exceeding a maximum dimension of 3 inches.

Deep Fill/Shallow Fill Transitions

To mitigate the detrimental effects of excessive differential settlement, deep fill/shallow fill transitions should also be eliminated from all building areas. This should be accomplished by overexcavating the "shallow" fill portions of each building area and replacing the excavated materials as properly compacted fill. Generally, the depths of overexcavation within the shallow fill area should equal one-half the thickness of the maximum depth of fill underlying the building area to a maximum depth of 15 feet.

Haul Roads

Haul roads should be selected to avoid disturbing terrain which is to remain in a natural state. Also, haul roads traversing compacted fill areas should be coordinated and planned to avoid or minimize generation of loose spill fills thereon. When this condition is unavoidable, close coordination with the project soils engineer and his representative will be required to eliminate intermingling of engineered and non-engineered fill.

During grading, special care should be exercised to avoid spilling and depositing of loose soil or debris onto slope areas and into areas programmed to remain in a natural state. Any loose slough, debris or other deleterious materials deposited or accumulated on natural areas will have to be removed by the contractor upon completion of grading.

Shrinkage/Bulking

Volumetric changes in earth quantities will occur when excavated on-site soils are replaced as properly compacted fill. Estimated shrinkage and subsidence factors were provided in a previous section. The estimates are exclusive of any oversize rock that is removed from the site and not buried within designated rock disposal areas.

Geotechnical Observations and Testing During Grading

Observations of the clearing operations, removal of low density surficial soils, keyway excavations, observation of cut slopes, and general grading procedures should be performed by a representative of the
project geotechnical consultant. It should be the grading contractor's responsibility to notify the project geotechnical consultant when fill areas and fill keys are ready for observation. A representative of the project geotechnical consultant should be present on site during all major grading operations to verify proper placement and adequate compaction of all fills, as well as to verify compliance with the other recommendations presented herein.

Slope Construction

Slope Drainage and Terracing

Cut and fill slopes exceeding a height of 30 feet should be provided with terraces and interceptor drains at vertical intervals specified in Section J109, Chapter 33 Appendix J of the 2013 CBC. Top-of-slope interceptor drains should be provided along the tops of cut slopes as specified in Section J109.3, Chapter 33 Appendix J of the 2013 CBC.

Cut Slopes

Cut slopes should be planned at a maximum slope ratio of 2:1 (h:v). A majority of cut slopes may expose cobbles and boulders that will result in an uneven slope face as well as create a condition for possible rock fall or toppling. Where this condition occurs, the affected cut slopes should be over-excavated and reconstructed with compacted fill.

Fill Slopes

Fill slopes should be planned at a maximum slope ratio of 2:1 (h:v). Additional recommendations for construction of fill slopes are provided below.

Fill Keys - Fill keys excavated into competent native soils will be required at the base of all proposed fill slopes to be constructed on slope surfaces inclining at 5:1 (H:V) or steeper. The fill keys should be excavated to a minimum depth of 2 feet into competent materials and have a minimum width of 15 feet or a width equal to one-half of the slope height, whichever is greater. The bottoms of the fill keys should be tilted back at a minimum of 2 percent towards the heel of the key. Internal backdrains may be required in the keyways at the discretion of the engineering geologist to prevent entrapment of irrigation water and rainwater in the key bottoms. Typical details for construction of the backdrains are shown on plates SG-2 and SG-3, Appendix C.

Surface Compaction - The finish surfaces of all fill slopes should be compacted to a minimum relative compaction of 90 percent. Final surface compaction should be achieved by overfilling the slopes during construction and then trimming the slopes back to the compacted inner core. As a secondary alternative, surface compaction should be obtained by back-rolling during construction to achieve at least 90 percent relative compaction within 6 to 8 inches of the finish surfaces. This initial back-rolling should be performed at vertical intervals not exceeding 4 to 5 feet. Final surface compaction should then be achieved by rolling
the slope surface with a cable-lowered sheepsfoot and then re-rolling with a grid roller. During final surface compaction, it is critical that the moisture content of the surface soils be maintained at near optimum moisture content or slightly higher.

**Fill-Over-Cut Slopes** - Where fill-over-cut transition slopes are proposed, a keyway excavated into competent bedrock or competent native soil should be provided at the contact. The keyway should be at least 15 feet wide and tilted back into the slope at a minimum gradient of 2 percent.

**Post-Grading Considerations**

**Utility Trenches**

All applicable requirements of the California Construction and General Industry Safety Orders, the Occupational Safety and Health Act of 1970, and the Construction Safety Act should be followed with respect to excavation of trenches for subsurface utilities. In general, the majority of the on-site soils consist of relatively clean, cohesionless sands and are classified as Type C soil in accordance with CAL/OSHA regulations. Accordingly, the sidewalls of open trenches should be sloped back at a ratio of 1.5:1 (h:v) or flatter. Trench shields should also be considered as added protection for workers entering the trenches.

All utility trench backfill should be compacted to a minimum relative compaction of 90 percent. Where on-site soils are utilized as backfill, mechanical compaction will be required. Density testing, along with probing, should be performed by a representative of the project geotechnical consultant to verify proper compaction.

For deep trenches with vertical walls, backfill should be placed in approximately 2-foot-thick maximum lifts, moisture conditioned to establish optimum or slightly above optimum moisture content, and then mechanically compacted with a hydra-hammer, pneumatic tampers, or similar equipment that can achieve the desired compaction. For deep trenches with sloped walls, backfill materials should be placed in approximately 8- to 12-inch-thick maximum lifts, and then compacted by rolling with a sheepsfoot tamper or similar equipment.

For shallow trenches where pipe may be damaged by mechanical compaction equipment, such as under building floor slabs, clean on-site sands having a sand equivalent (SE) value of 30 or greater should be utilized for backfill that is jetted or flooded into place, and then tamped into place. No specific relative compaction will be required; however, observation, probing, and if deemed necessary, testing should be performed by the project geotechnical consultant to verify that an adequate degree of compaction is achieved.
To avoid point loads and subsequent distress to asbestos, clay, cement, or plastic pipe, clean sand bedding should be placed at least 1-foot above all pipe in areas where excavated trench materials contain oversize rock. Sand bedding materials should be thoroughly jetted prior to placement of backfill.

Where utility trenches are proposed parallel to any building footing (interior and/or exterior trenches), the bottom of the trench should not extend below a 1:1 plane projected downward from the outside bottom edge of the adjacent footing.

**Pad Drainage**

Positive surface drainage systems consisting of a combination of sloped concrete flatwork, sheet flow gradients and earth swales, and surface area drains (where needed) should be provided around each building and within yard areas to collect and direct all surface waters to the adjacent streets. Sheet-flow-graded ground surfaces should be inclined at a minimum gradient of 2 percent away from building foundations and similar structures. Surface waters should not be allowed to collect or pond against building foundations and within the level areas of the lots, or to flow onto adjacent slopes. Roof gutters with downspouts should be used on the sides of houses where there is insufficient area to construct effective yard drainage devices and/or where roof drainage is directed onto adjacent slopes.

For unimproved graded lots to remain idle for a long period of time, pad drainage should be designed for a minimum gradient of 1 percent toward the adjacent streets.

**Slope Landscaping and Maintenance**

Proper slope and pad drainage are essential in the design of grading for the subject property. The overall stability of the graded slopes should not be adversely affected provided all drainage provisions are properly constructed and maintained thereafter, and provided all engineered slopes are landscaped with a deep-rooted, drought-resistant, and relatively maintenance-free plant species. Additional comments and recommendations are presented below with respect to slope drainage, landscaping and irrigation.

1. Proper drainage provisions for engineered slopes should consist of concrete terrace drains, downdrains and energy dissipaters (where required) constructed in accordance with County of Riverside grading codes. Provisions should also be made for construction of compacted earth berms along the tops of all engineered slopes.

2. All engineered slopes should be landscaped as soon as practical at the completion of grading. As noted, the landscaping should consist of a deep-rooted, drought-resistant, and maintenance-free plant species. If landscaping cannot be provided within a reasonable period of time, jute matting or equivalent, or a spray-on product designed to seal slope surfaces should be considered as a temporary measure to inhibit surface erosion.
3. Irrigation systems should be considered on the engineered slopes and a watering program then implemented which maintains a uniform, near optimum moisture condition in the soils. Overwatering and subsequent saturation of the slope soils should be avoided.

4. Irrigation systems should be constructed at the surface only. Construction of sprinkler lines in trenches should not be allowed without prior approval from the soils engineer and engineering geologist.

5. During construction of terrace drains and downdrains, care must be taken to avoid placement of loose soil on the slope surfaces.

6. A permanent slope maintenance program should be initiated. Proper slope maintenance must include the care of drainage and erosion control provisions, rodent control, and repair of leaking irrigation systems.

7. Provided the above recommendations are followed with respect to slope drainage, maintenance and landscaping, the potential for deep saturation of slope soils is considered very low.

**Preliminary Foundation Design Recommendations**

**General**

For planning purposes, we provide the following preliminary foundation design recommendations based on anticipated conditions at the completion of rough grading. Final design recommendations should be provided by the project geotechnical consultant based on final as-graded soil conditions existing within the building sites. The following recommendations are based on the 2013 California Building Code (CBC).

**Seismic Design Parameters**

Earthquake loads on earthen structures and buildings are a function of ground acceleration which may be determined from the site-specific ground motion analysis. Alternatively, a design response spectrum can be developed for the site based on the code guidelines. To provide the design team with the parameters necessary to construct the design acceleration response spectrum for this project, we used the computer applications that are available on the United States Geological Survey (USGS) website, [http://geohazards.usgs.gov/](http://geohazards.usgs.gov/). Specifically, the Design Maps website [http://geohazards.usgs.gov/designmaps/us/application.php](http://geohazards.usgs.gov/designmaps/us/application.php) was used to calculate the ground motion parameters.

To run the above computer applications, site latitude, longitude, risk category and knowledge of “Site Class” are required. The site class definition depends on the average shear wave velocity, $V_{S30}$, within the upper 30 meters (approximately 100 feet) of site soils. A shear wave velocity of 900 feet per second for the upper 100 feet was used for the site based on engineering experience and judgment.
The following table, Table 6, provides parameters required to construct the site-specific acceleration response spectrum based 2013 CBC guidelines.

### Table 6

SEISMIC DESIGN PARAMETERS

<table>
<thead>
<tr>
<th>Ground Motion Parameters</th>
<th>Reference</th>
<th>Parameter Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Latitude (North)</td>
<td>-</td>
<td>33.66</td>
<td>°</td>
</tr>
<tr>
<td>Longitude (West)</td>
<td>-</td>
<td>-115.91</td>
<td>°</td>
</tr>
<tr>
<td>Site Class Definition</td>
<td>Table 20.3-1, ASCE 7-10</td>
<td>D</td>
<td>-</td>
</tr>
<tr>
<td>Assumed Risk Category</td>
<td>Table 1604.5, CBC 2013</td>
<td>II</td>
<td>-</td>
</tr>
<tr>
<td>Mw - Earthquake Magnitude</td>
<td>Section 1803.5.12.2, CBC 2013</td>
<td>7.8</td>
<td>-</td>
</tr>
<tr>
<td>S1 - Mapped Spectral Response Acceleration</td>
<td>Figure 1613.3.1(1), CBC 2013</td>
<td>1.5</td>
<td>g</td>
</tr>
<tr>
<td>S1 - Mapped Spectral Response Acceleration</td>
<td>Figure 1613.3.1(2), CBC 2013</td>
<td>0.653</td>
<td>g</td>
</tr>
<tr>
<td>Fa - Site Coefficient</td>
<td>Table 1613.3.3(1), CBC 2013</td>
<td>1.0</td>
<td>-</td>
</tr>
<tr>
<td>Fv - Site Coefficient</td>
<td>Table 1613.3.3(2), CBC 2013</td>
<td>1.5</td>
<td>-</td>
</tr>
<tr>
<td>SMS - Adjusted Maximum Considered Earthquake</td>
<td>Equation 16-37, CBC 2013</td>
<td>1.5</td>
<td>g</td>
</tr>
<tr>
<td>SMS - Adjusted Maximum Considered Earthquake</td>
<td>Equation 16-38, CBC 2013</td>
<td>0.98</td>
<td>g</td>
</tr>
<tr>
<td>SDS - Design Spectral Response Acceleration</td>
<td>Equation 16-39, CBC 2013</td>
<td>1.0</td>
<td>g</td>
</tr>
<tr>
<td>SD1 - Design Spectral Response Acceleration</td>
<td>Equation 16-40, CBC 2013</td>
<td>0.653</td>
<td>g</td>
</tr>
<tr>
<td>To - (0.2 SD1/ SDS)</td>
<td>Section 11.3, ASCE 7-10</td>
<td>0.131</td>
<td>s</td>
</tr>
<tr>
<td>Tg - (SD1/ SDS)</td>
<td>Section 11.3, ASCE 7-10</td>
<td>0.653</td>
<td>s</td>
</tr>
<tr>
<td>TL - Long Period Transition Period</td>
<td>Figure 22-12, ASCE 7-10</td>
<td>8</td>
<td>s</td>
</tr>
<tr>
<td>Fpga - Site Coefficient</td>
<td>Figure 22-7, ASCE 7-10</td>
<td>1.0</td>
<td>-</td>
</tr>
<tr>
<td>PGA M - Peak Ground Acceleration at MCE 1</td>
<td>Equation 11.8-1, ASCE 7-10</td>
<td>0.568</td>
<td>g</td>
</tr>
<tr>
<td>Design PGA (0.4 SDS) – Short Retaining Walls</td>
<td>Equation 11.4-5, ASCE 7-10</td>
<td>0.4</td>
<td>g</td>
</tr>
<tr>
<td>Design PGA 2 (2/3 PGA M) - Slope Stability</td>
<td>Similar to Equations 16-39 &amp; 16-40</td>
<td>0.38</td>
<td>g</td>
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<tr>
<td>Crs - Short Period Risk Coefficient</td>
<td>Figure 22-17, ASCE 7-10</td>
<td>1.004</td>
<td>-</td>
</tr>
<tr>
<td>CR1 - Long Period Risk Coefficient</td>
<td>Figure 22-18, ASCE 7-10</td>
<td>0.979</td>
<td>-</td>
</tr>
<tr>
<td>Seismic Design Category 3</td>
<td>Section 1613.3.5, CBC 2013</td>
<td>D</td>
<td>-</td>
</tr>
</tbody>
</table>

1 PGA Calculated at the MCE return period of 2475 years (2 percent chance of exceedance in 50 years).
2 PGA Calculated at the Design Level of 2/3 of MCE which is approximately equivalent to a return period of 475 years (10 percent chance of exceedance in 50 years).
3 Seismic Design Category may be calculated by the structural engineer in accordance with the alternate design procedures of Section 1613.3.5.1 based on structural characteristics in addition to the ground motion parameters, this may supersede the category listed herein.

References:
- California Building Code (CBC), 2013, California Code of Regulations, Title 24, Part 2, Volume I and II
- American Society of Civil Engineers (ASCE/SEI), 2010, Minimum Design Load for Buildings and Other Structures, Standards 7-10
Building Clearances from Ascending Slopes

To conform with Section 1808.7.1 and Figure 1808.7.1 of the 2013 CBC, minimum clearances of H/2 (one-half of the total slope height) to a maximum of 15 feet should be maintained between buildings and the toe of any adjacent ascending slope. Retaining walls may be constructed at the base of the slopes to achieve the required building clearances.

Footing Setbacks from Descending Slopes

To conform with Section 1808.7.2 and Figure 1808.7.1 of the 2013 CBC, building footings to be constructed on or near descending slopes should be deepened, as necessary, to provide a minimum footing setback of H/3 (one-third of the total slope height). The footing setbacks should be 5 feet minimum where the slope height is 15 feet or less, and vary up to 40 feet maximum where slope heights exceed 15 feet. The footing setbacks should be measured along a horizontal line projected from the lower outside bottom edges of the footings to the face of the adjacent descending slope.

Several narrow and deeply incised natural drainages with near-vertical side walls traverse the subject site. If these drainages remain ungraded, footing setbacks for structures proposed near the drainages should be measured along a horizontal line to the intersection with an imaginary 1:1 line projected upward from the toe of the steep side wall.

Allowable Soil Bearing Capacities

Provided that site grading is performed in accordance with the “Earthwork” recommendations presented in this report, an allowable bearing value of 2,000 pounds per square foot is recommended for preliminary design of 24-inch-square pad footings and 12-inch-wide continuous footings founded at a minimum depth of 12 inches below the lowest adjacent final grade. This value may be increased by 20 percent for each additional one foot of width and/or depth, to a maximum value of 3,000 pounds per square foot. Recommended allowable bearing values include both dead and live loads, and may be increased by one-third when designing for short duration wind and seismic forces.

Footing Settlements

On-site native soil materials are not subject to hydro-consolidation (collapse) or significant consolidation under anticipated maximum overburden pressures. Under the above bearing values, maximum total static settlements of footings is expected to be approximately ¾ inches with a differential settlement of approximately ½ of an inch over a span of 40 feet (1:480). The majority of this estimated footing settlement will occur during building construction or shortly thereafter as the loads are applied.
Lateral Resistance

Provided that site grading is performed in accordance with our “Earthwork” recommendations, a passive earth pressure increasing at a rate of 250 pounds per square foot per foot of depth, to a maximum value of 2500 pounds per square foot, may be used to determine lateral bearing resistance for building foundations. In addition, a coefficient of friction of 0.35 times the dead load forces may be used between concrete and the supporting soil to determine lateral sliding resistance. An increase of one-third of the above values may also be used when designing for short duration wind and seismic forces.

The above values are based on footings placed directly against compacted fill or competent native soil. In the case where footing sides are formed, all backfill placed against the footings should be compacted to at a minimum relative compaction of 90 percent of maximum dry density.

Preliminary Guidelines for Footings and Slabs on-Grade Design and Construction

Results of the laboratory expansion index tests performed in accordance with ASTM D 4829 indicate a majority of the on-site soil and bedrock materials exhibit a Very Low expansion potential. However, a sample of clayey sand was found to exhibit a Low expansion potential. Therefore, based on the distribution of the various geologic units within the site and the anticipated grading, it is expected that upon the completion of rough grading that the majority of the soils underlying the building sites will exhibit a Very Low expansion potential.

Recommendations for Very Low Expansion Potential (EI of 20 or Less)

The results of our laboratory tests performed on representative samples of near-surface soils within the site during our investigation indicate that these materials exhibit expansion potentials that are within the Very Low range (Expansion Index from 0 to 20). As such, the design of slabs on-grade is considered to be exempt from the procedures outlined in Sections 1803.5.3 and 1808.6.2 of the 2013 CBC and may be performed using any method deemed rational and appropriate by the project structural engineer. However, the following minimum recommendations are presented herein for conditions where the project design team may require geotechnical engineering guidelines for design and construction of footings and slabs on-grade at the project site.

The design and construction guidelines that follow are based on the above soil conditions and may be considered for reducing the effects of variability in fabric, composition and, therefore, the detrimental behavior of the site soils such as excessive short- and long-term total and differential settlements. These guidelines have been developed on the basis of the previous experience of this
firm on projects with similar soil conditions. Although construction performed in accordance with these guidelines has been found to reduce post-construction movement and/or distress, they generally do not positively eliminate all potential effects of variability in soils characteristics and future settlement.

It should also be noted that the suggestions for dimension and reinforcement provided herein are performance-based and intended only as preliminary guidelines to achieve adequate performance under the anticipated soil conditions. However, they should not be construed as replacement for structural engineering analyses, experience and judgment. The project structural engineer, architect and/or civil engineer should make appropriate adjustments to slab and footing dimensions, and reinforcement type, size and spacing to account for internal concrete forces (e.g., thermal, shrinkage and expansion), as well as external forces (e.g., applied loads) as deemed necessary. Consideration should also be given to minimum design criteria as dictated by local building code requirements.

**Conventional Slabs on-Grade System**

Given the very low expansion potential exhibited by onsite soils, we recommend that footings and floor slabs be designed and constructed in accordance with the following minimum criteria.

**Footings**

1. Exterior continuous footings supporting one- and two-story structures should be founded at a minimum depth of 12 inches below the lowest adjacent final grade, respectively. Interior continuous footings may be founded at a minimum depth of 10 inches below the top of the adjacent finish floor slabs.

2. All continuous footings should have minimum widths of 12 and 15 inches for one- and two-story construction, respectively. All continuous footings should be reinforced with a minimum of two No. 4 bars, one top and one bottom.

3. A minimum 12-inch-wide grade beam founded at the same depth as adjacent footings should be provided across garage entrances or similar openings (such as large doors or bay windows). The grade beam should be reinforced with a similar manner as provided above.

4. Interior isolated pad footings, if required, should be a minimum of 24 inches square and founded at a minimum depth of 12 inches below the bottoms of the adjacent floor slabs. Pad footings should be reinforced with No. 4 bars spaced a maximum of 18 inches on centers, both ways, placed near the bottoms of the footings.

5. Exterior isolated pad footings intended for support of roof overhangs such as second-story decks, patio covers and similar construction should be a minimum of 24 inches square and founded at a minimum depth of 18 inches below the lowest adjacent final grade. The pad footings should be
reinforced with No. 4 bars spaced a maximum of 18 inches on centers, both ways, placed near the bottoms of the footings. Exterior isolated pad footings may need to be connected to adjacent pad and/or continuous footings via tie beams at the discretion of the project structural engineer.

6. The minimum footing dimensions and reinforcement recommended herein may be modified (increased or decreased subject to the constraints of Chapter 18 of the 2013 CBC) by the structural engineer responsible for foundation design based on his/her calculations, engineering experience and judgment.

Building Floor Slabs

1. Concrete floor slabs should be a minimum 4 inches thick and reinforced with No. 3 bars spaced a maximum of 24 inches on centers, both ways. Alternatively, the structural engineer may recommend the use of prefabricated welded wire mesh for slab reinforcement. For this condition, the welded wire mesh should be of sheet type (not rolled) and should consist of 6x6/W2.9xW2.9 (per the Wire Reinforcement Institute, WRI, designation) or stronger. All slab reinforcement should be supported on concrete chairs or brick to ensure the desired placement near mid-depth. Care should be exercised to prevent warping of the welded wire mesh between the chairs in order to ensure its placement at the desired mid-slab position.

2. Living area concrete floor slabs and areas to receive moisture sensitive floor covering should be underlain with a moisture vapor retarder consisting of a minimum 10-mil-thick polyethylene or polyolefin membrane that meets the minimum requirements of ASTM E96 and ASTM E1745 for vapor retarders (such as Husky Yellow Guard®, Stego® Wrap, or equivalent). All laps within the membrane should be sealed, and at least 2 inches of clean sand should be placed over the membrane to promote uniform curing of the concrete. To reduce the potential for punctures, the membrane should be placed on a pad surface that has been graded smooth without any sharp protrusions. If a smooth surface cannot be achieved by grading, consideration should be given to lowering the pad finished grade an additional inch and then placing a 1-inch-thick leveling course of sand across the pad surface prior to the placement of the membrane.

At the present time, some slab designers, geotechnical professionals and concrete experts view the sand layer below the slab (blotting sand) as a place for entrapment of excess moisture that could adversely impact moisture-sensitive floor coverings. As a preventive measure, the potential for moisture intrusion into the concrete slab could be reduced if the concrete is placed directly on the vapor retarder. However, if this sand layer is omitted, appropriate curing methods must be implemented to ensure that the concrete slab cures uniformly. A qualified materials engineer with experience in slab design and construction should provide recommendations for alternative methods of curing and supervise the construction process to ensure uniform slab curing. Additional steps would also need to be taken to prevent puncturing of the vapor retarder during concrete placement.

3. Garage floor slabs should be a minimum 4 inches thick and reinforced in a similar manner as living area floor slabs. Garage slabs should also be poured separately from adjacent wall footings with a positive separation maintained using ¾-inch-minimum felt expansion joint material. To control the propagation of shrinkage cracks, garage floor slabs should be quartered with weakened plane joints. Consideration should be given to placement of a moisture vapor retarder below the garage slab, similar to that provided in Item 2 above, should the garage slab be overlain with moisture sensitive floor covering.
4. Presaturation of the subgrade below floor slabs will not be required; however, prior to placing concrete, the subgrade below all dwelling and garage floor slab areas should be thoroughly moistened to achieve a moisture content that is at least equal to or slightly greater than optimum moisture content. This moisture content should penetrate to a minimum depth of 12 inches below the bottoms of the slabs.

5. The minimum dimensions and reinforcement recommended herein for building floor slabs may be modified (increased or decreased) by the structural engineer responsible for foundation design based on his/her calculations, engineering experience and judgment.

**Preliminary Retaining Wall Design Recommendations**

**Temporary Backcut Slopes**

To comply with CAL/OSHA regulations, temporary backcut slopes associated with construction of retaining walls should be excavated at a ratio of 1.5:1 (h:v). Steeper backcuts will require review by the engineering geologist.

**Allowable Bearing Values and Lateral Resistance**

Retaining wall footings may be designed using the allowable soil bearing and lateral resistance values recommended previously for design of building footings. However, when calculating passive resistance, the upper 6 inches of the soils should be ignored in areas where the footings will not be covered with concrete flatwork.

**On-Site Soils Used for Backfill**

On-site soils derived from the Fan deposits and Ocotillo Conglomerate consist predominately of clean sands exhibiting a very low expansion potential and sand equivalent (SE) values exceeding 30. Therefore, these soil materials are considered well-suited for use as backfill behind retaining walls provided they are cleared of cobbles exceeding a maximum dimension of 3 inches. Active earth pressures equivalent to fluids having densities of 35 and 51 pounds per cubic foot may be used for design of cantilevered walls retaining a level backfill and ascending 2:1 backfill, respectively. For walls that are restrained at the top, at-rest earth pressures of 53 and 78 pounds per cubic foot (equivalent fluid pressures) should be used. The above values are for retaining walls that have been supplied with a proper subdrain system. All walls should be designed to support any adjacent structural surcharge loads imposed by other nearby walls or footings in addition to the above-recommended active and at-rest earth pressures.
**Drainage and Waterproofing**

Perforated pipe and gravel subdrains should be installed behind all retaining walls to prevent entrapment of water in the backfill. Perforated pipe should consist of 4-inch-minimum diameter PVC Schedule 40, or ABS SDR-35, with the perforations lain down. The pipe should be encased in a 1-foot-wide column of 3/4-inch to 1½-inch open-graded gravel. If on-site sandy soils are used as backfill, the open-graded gravel should extend above the wall footings to a minimum height of 1-foot above the footing. The open-graded gravel should be completely wrapped in filter fabric consisting of Mirafi 140N, or equivalent. Solid outlet pipes should be connected to the subdrains and then routed to a suitable area for discharge of accumulated water. The backfilled sides of retaining walls should be coated with an approved waterproofing compound or covered with a similar material to inhibit migration of moisture through the walls.

**Wall Backfill**

Where on-site soils are used for backfill, they should be placed in approximately 6- to 8-inch-thick maximum lifts, watered as necessary to achieve near optimum moisture conditions, and then mechanically compacted in place to a minimum relative compaction of 90 percent. Flooding or jetting of the backfill materials should be avoided. A representative of the project geotechnical consultant should observe the backfill procedures and test the wall backfill to verify adequate compaction.

**Preliminary Recommendations for Masonry Block Walls**

**Construction near the Tops of Descending Slopes**

Continuous footings for masonry screen walls proposed on or within 5 feet from the top of a descending slope should be deepened such that a horizontal clearance of 5 feet or more is maintained between the outside bottom edge of the footings and the slope face. The footings should be reinforced with two No. 4 bars, one top and one bottom, or as recommended by the structural engineer.

**Construction on Level Ground**

Where masonry screen walls are proposed on level ground and 5 or more feet from the tops of descending slopes, the footings for these walls may be founded at a depth of 12 inches or more below the lowest adjacent final grade. These footings should also be reinforced with two No. 4 bars, one top and one bottom, or as recommended by the structural engineer.
**Construction Joints**

In order to mitigate the potential for unsightly cracking related to the effects of differential settlement, positive separations (construction joints) should be provided in the walls at horizontal intervals of approximately 25 feet and at each corner. The separations should be provided in the blocks only and not extend though the footings. The footings should be placed monolithically with continuous rebar to serve as effective grade beams along the full lengths of the walls.

**EXTERIOR CONCRETE FLATWORK**

**Very Low and Low Expansion Potential (Expansion Index 0 – 50)**

**General**

It is expected that a majority of the as-graded building pads will be underlain with subgrade soils exhibiting a very low expansion potential. However there were a few pockets of clayey sand noted in our investigation. Therefore for preliminary design purposes near-surface compacted fill soils within the site should be considered too be variable in expansion behavior and are expected to exhibit very low to low expansion potential. Due to typical project scheduling constraints, it may not be feasible to collect additional samples of subgrade soils for testing to verify their expansion potential immediately prior to pouring concrete. For this reason, we recommend that all exterior concrete flatwork such as sidewalks, patio slabs, large decorative slabs, concrete subslabs that will be covered with decorative pavers, private and/or public vehicular driveways and/or access roads within and adjacent to the site be designed by the project architect and/or structural engineer with consideration given to mitigating the potential cracking and uplift that can develop in soils exhibiting expansion index values that fall in the low category.

The guidelines that follow should be considered as minimums and are subject to review and revision by the project architect, structural engineer and/or landscape consultant as deemed appropriate. If sufficient time will be allowed in the project schedule for verification sampling and testing prior to the concrete pour, the test results generated may dictate that a somewhat less conservative design could be used.

**Thickness and Joint Spacing**

To reduce the potential of unsightly cracking, concrete walkways, patio-type slabs, large decorative slabs and concrete subslabs to be covered with decorative pavers should be at least 4 inches thick and provided with construction joints or expansion joints every 6 feet or less. Private driveways that will be designed for the use of passenger cars for access to private garages should also be at least 4 inches thick and provided with construction joints or expansion joints every 10 feet or less. Concrete pavement that will be designed based on an unlimited number of applications of an 18-kip single-axle load in public access areas, segments
of road that will be paved with concrete (such as bus stops and cross-walks) or access roads that will be subject to heavy truck loadings should have a minimum thickness of 5 inches and be provided with control joints spaced at maximum 10-foot intervals. A modulus of subgrade reaction of 125 pounds per cubic foot may be used for design of the public and access roads.

**Reinforcement**

All concrete flatwork having their largest plan-view panel dimension exceeding 10 feet should be reinforced with a minimum of No. 3 bars spaced 24 inches on centers, both ways. Alternatively, the slab reinforcement may consist of welded wire mesh of the sheet type (not rolled) with 6x6/W1.4xW1.4 designation in accordance with the Wire Reinforcement Institute (WRI). The reinforcement should be properly positioned near the middle of the slabs.

*The reinforcement recommendations provided herein are intended as guidelines to achieve adequate performance for anticipated soil conditions. The project architect, civil and/or structural engineer should make appropriate adjustments in reinforcement type, size and spacing to account for concrete internal (e.g., shrinkage and thermal) and external (e.g., applied loads) forces as deemed necessary.*

**Edge Beams (Optional)**

Where the outer edges of concrete flatwork are to be bordered by landscaping, it is recommended that consideration be given to the use of edge beams (thickened edges) to prevent excessive infiltration and accumulation of water under the slabs. Edge beams, if used, should be 6 to 8 inches wide, extend 8 inches below the tops of the finish slab surfaces. Edge beams are not mandatory; however, their inclusion in flatwork construction adjacent to landscaped areas is intended to reduce the potential for vertical and horizontal movement and subsequent cracking of the flatwork related to uplift forces that can develop in expansive soils.

**Subgrade Preparation**

**Compaction**

To reduce the potential for distress to concrete flatwork, the subgrade soils below concrete flatwork areas to a minimum depth of 12 inches (or deeper, as either prescribed elsewhere in this report or determined in the field) should be moisture conditioned to at least equal to, or slightly greater than, the optimum moisture content and then compacted to a minimum relative compaction of 90 percent. Where concrete public roads, concrete segments of roads and/or concrete access driveways are proposed, the upper 6 inches of subgrade soil should be compacted to a minimum 95 percent relative compaction.
Pre-Moistening

As a further measure to reduce the potential for concrete flatwork cracking, subgrade soils should be thoroughly moistened prior to placing concrete. The moisture content of the soils should be at least 1.2 times the optimum moisture content and penetrate to a minimum depth of 12 inches into the subgrade. Flooding or ponding of the subgrade is not considered feasible to achieve the above moisture conditions since this method would likely require construction of numerous earth berms to contain the water. Therefore, moisture conditioning should be achieved with sprinklers or a light spray applied to the subgrade over a period of few to several days just prior to pouring concrete. Pre-watering of the soils is intended to promote uniform curing of the concrete, reduce the development of shrinkage cracks and reduce the potential for differential expansion pressure on freshly poured flatwork. A representative of the project geotechnical consultant should observe and verify the density and moisture content of the soils, and the depth of moisture penetration prior to pouring concrete.

Drainage

Drainage from patios and other flatwork areas should be directed to local area drains and/or graded earth swales designed to carry runoff water to the adjacent streets or other approved drainage structures. The concrete flatwork should be sloped at a minimum gradient of one percent, or as prescribed by project civil engineer or local codes, away from building foundations, retaining walls, masonry garden walls and slope areas.

Tree Wells

Tree wells are not recommended in concrete flatwork areas since they introduce excessive water into the subgrade soils and allow root invasion, both of which can cause heaving and cracking of the flatwork.

Corrosivity Screening

As a screening level study, limited chemical and electrical tests were performed on representative samples of onsite soils to identify potential corrosive characteristics of these soils. The following sections present the test results and an interpretation of current codes and guidelines that are commonly used in our industry as they relate to the adverse impact of chemical contents of the site soils and their associated moisture on various components of the proposed structures in contact with site soils.

A variety of test methods are available to quantify corrosive potential of soils for various elements of construction materials. Depending on the test procedures adopted, characteristics of the leachate that is used to extract the target chemicals from the soils and the test equipment; the results can vary appreciably...
for different test methods in addition to those caused by variability in soil composition. The testing procedures referred to herein are considered to be typical for our industry and have been adopted and/or approved by many public or private agencies. In drawing conclusions from the results of our chemical and electrical laboratory testing and providing mitigation guidelines to reduce the detrimental impact of corrosive site soils on various components of the structure in contact with site soils, heavy references were made to 2013 CBC and American Concrete Institute, 2011 Structural Concrete Building Code (ACI 318-11). Where relevant information was not available in these codes, references were made to guidelines developed by California Department of Transportation (Caltrans), mainly because their risk tolerance for highway bridges are considered comparable to those for residential or commercial structures and that Post Tensioning Institute (PTI), in part, accepts and uses Caltrans’ relevant corrosivity criteria for post-tensioned slabs on-grade.

It should be noted that Petra does not practice corrosion engineering; therefore, the test results, opinion and engineering judgment provided herein should be considered as general guidelines only. Additional analyses would be warranted, especially, for cases where buried metallic building materials (such as copper and cast or ductile iron) in contact with site soils are planned for the project. In many cases, the project geotechnical engineer is not informed of these choices. Therefore, for conditions where such elements are considered, we recommend that the project design professionals (i.e., the architect and/or structural engineer) consider recommending a qualified corrosion engineer to conduct additional sampling and testing of near-surface soils during the final stages of site grading to provide a complete assessment of soil corrosivity. Recommendations to mitigate the detrimental effects of corrosive soils on buried metallic and other building materials that may be exposed to corrosive soils should be provided by the corrosion engineer as deemed appropriate.

**Concrete in Contact with Site Soils**

Soils containing soluble sulfates beyond certain threshold levels as well as acidic soils are considered to be detrimental to integrity of concrete placed in contact with such soils. For the purpose of this study, soluble sulfate concentrations in soils were determined in accordance with California Test Method No. 417. Soil acidity, as indicated by hydrogen-ion concentration (pH), was determined in accordance with California Test Method No. 643. The soil soluble sulfate severity rating is adopted from ACI 318 publication. The soil acid severity rating is adopted from The United States Department of Agriculture, Natural Resources Conservation Service classification.
The results of the laboratory tests indicate that a majority of the on-site soils contain a water soluble sulfate content of less than 0.10 percent. However, one sample of the younger alluvium was found to contain a soluble sulfate content of 0.42 percent.

**Soluble Sulfate of Less Than 0.10 Percent**

Based on Article 1904.1 of Section 1904 of the 2013 CBC, concrete that will be exposed to sulfates in site soil should be assigned exposure classes in accordance with the durability requirements of ACI 318.

Based on the test results and in reference to Table 4.2.1 of ACI 318-11, an exposure class of S0 is appropriate for onsite soils containing a soluble sulfate content of less than 0.10 percent (majority of on-site soils excluding the Palm Spring Formation). Accordingly, a severity level of **Not Applicable** for exposure to sulfate may be expected for concrete placed in contact with the onsite soil materials. As such, Table 4.3.1 of ACI 318-11 provides that no restriction for cement type or maximum water-cement ratio for the fresh concrete would be required. However, this table indicates that the concrete minimum unconfined compressive strength should not be less than 2,500 psi.

Further, the results of limited in-house testing of representative samples indicate that soils within the subject site are slightly to moderately alkaline with respect to pH (a pH of 7.4 to 7.9). Based on this finding and according to Section 8.22.2 of Caltrans’ 2003 Bridge Design Specifications (2003 BDS) requirements (which consider the combined effects of soluble sulfates and soil pH), a commercially available Type II Modified cement may be used.

The guidelines provided herein should be evaluated and confirmed, or modified, in its entirety by the project structural engineer and the contractor responsible for concrete placement for concrete used in exterior and interior footings, interior slabs on-ground, garage slabs, walls foundation and concrete exposed to weather such as driveways, patios, porches, walkways, ramps, steps, curbs, etc.

**Soluble Sulfate of Greater Than 0.10 Percent**

Samples of the Palm Spring Formation were found to contain soluble sulfate contents of 0.146 and 0.414 percent.

Based on Article 1904.1 of Section 1904 of the 2013 CBC, concrete that will be exposed to sulfates in water or soil should be assigned exposure classes in accordance with the durability requirements of ACI 318.

Based on the test results and in reference to Table 4.2.1 of ACI 318-11, an exposure class of S2 is appropriate for onsite soils. Accordingly, a severity level of **Severe** for exposure to sulfate may be expected.
for concrete placed in contact with the onsite soil materials. Further, Article 1904.2 of Section 1904 of the 2013 CBS requires that concrete mixtures conform to the most restrictive maximum water-cementitious material ratios, maximum cementitious admixture, minimum air-entrainment and minimum specified concrete compressive strength requirements of ACI 318. Table 4.3.1 of ACI 318-11 indicates that Type V cement (in accordance with ASTM C150) would be required for this condition. In addition, the maximum water/cement ratio of the fresh concrete should not exceed 0.45, and concrete minimum unconfined compressive strength, $f'_c$, should not be less than 4,500 psi. However, Post Tensioning Institute recommends a minimum $f'_c$ of 3,000 psi for post-tensioned slabs on-grade where water-soluble sulfate is greater than 0.2 percent by weight.

It should be noted that for occupancies and appurtenances thereto in Group R occupancies that are in buildings less than four stories above grade plane the 2013 CBC allows for an exception to the above requirements. That is, in lieu of the above requirements, Article 1904.2 of Section 1904 of the 2013 CBC provides that normal weight aggregate concrete is permitted to comply with the requirements of Table 1904.2 (in conjunction with Figure 1904.2), which appears to suggest that the minimum unconfined compressive strength, $f'_c$, may be reduced to 2,500 psi. It is our understanding that this recommendation may not apply to post-tensioned slabs on-grade as Post Tensioning Institute requirements is a minimum $f'_c$ of 3,000 psi for this condition.

The results of limited in-house testing of representative samples indicate that soils within the subject site are slightly to moderately alkaline with respect to pH (a pH of 7.4 to 7.9). Based on this finding and according to Section 8.22.2 of Caltrans’ 2003 Bridge Design Specifications (2003 BDS) requirements (which consider the combined effects of soluble sulfates and soil pH), a commercially available Type V or Type II Modified cement may be used.

The guidelines provided herein should be evaluated and confirmed, or modified, in its entirety by the project structural engineer and the contractor responsible for concrete placement for concrete used in exterior and interior footings, interior slabs on-ground, garage slabs, walls foundation and concrete exposed to weather such as driveways, patios, porches, walkways, ramps, steps, curbs, etc.

**Metals Encased in Concrete**

Soils containing a soluble chloride concentration beyond a certain threshold level are considered corrosive to metallic elements such as reinforcement bars, tendons, cables, bolts, etc. that are encased in concrete that, in turn, is in contact with such soils. For the purpose of this study, soluble chlorides (Cl) in soils were determined in accordance with California Test Method No. 422.
Based on Article 1904.1 of Section 1904 of the 2013 CBC, concrete that will be exposed to chlorides from “deicing chemicals, salt, saltwater, brackish water, seawater or spray from these sources, where concrete has steel reinforcement” should be assigned exposure classes in accordance with the durability requirements of ACI 318. According to Table 4.2.1 of ACI 318-11, an exposure class of C0 with a severity designation of Not Applicable is appropriate for reinforced concrete that remains dry or protected from moisture. Similarly, an exposure class of C1 with a severity designation of Moderate is appropriate for reinforced concrete that is exposed to moisture but not to external sources of chlorides. And, lastly, an exposure class of C2 with a severity designation of Severe is appropriate for reinforced concrete that is exposed to moisture and external sources of chlorides as enumerated above.

Based on our understanding of the project, it is our professional opinion that an exposure class of C1 with a severity designation of Moderate is appropriate for a majority of reinforced concrete, to be placed at the site, that are in contact with site soils. It should be noted, however, that an exposure class of C2 with a severity designation of Severe is more appropriate for reinforced concrete that is planned for pool walls and decking, should such features be considered for the project.

The results of our limited laboratory tests performed indicate that onsite soils contain a water-soluble chloride concentration of 73 to 185 parts per million (ppm). Article 1904.2 of Section 1904 of the 2013 CBC requires that concrete mixtures conform to the most restrictive maximum water-cementitious material ratios, maximum cementitious admixture, minimum air-entrainment and minimum specified concrete compressive strength requirements of ACI 318 based on the exposure classes assigned in Article 1904.1. No maximum water/cement ratio for the fresh concrete is prescribed by ACI 318 for class C1 (or Moderate severity) exposure condition. However, Table 4.3.1 of ACI 318-11 indicates that concrete minimum unconfined compressive strength, $f'_c$, should not be less than 2,500 psi. For class C2 (or Severe) exposure condition, Table 4.3.1 of ACI 318-11 requires that the maximum water/cement ratio of the fresh concrete should not exceed 0.40 and concrete minimum unconfined compressive strength, $f'_c$, should not be less than 5,000 psi.

The guidelines provided herein should be evaluated and confirmed, or modified, in its entirety by the project structural engineer for reinforced concrete placement for concrete used in exterior and interior footings, interior slabs on-ground, garage slabs walls foundation and concrete exposed to weather such as driveways, patios, porches, walkways, ramps, steps, curbs, etc.

**Metallic Elements in Contact with Site Soils**

Elevated concentrations of soluble salts in soils tend to induce low level electrical currents in metallic objects in contact with such soils. This process promotes metal corrosion and can lead to distress to building
metallic components that are in contact with site soils. The minimum electrical resistivity measurement provides a simple indication of relative concentration of soluble salts in the soil and, therefore, is widely used to estimate soil corrosivity with regard to metals. For the purpose of this investigation, the minimum resistivity in soils is measured in accordance with California Test Method No. 643. The soil corrosion severity rating is adopted from the Handbook of Corrosion Engineering by Pierre R. Roberge.

The minimum electrical resistivity for onsite soils was found to be less than 620 ohm-cm based on limited testing. The result indicates that on-site soils are Extremely Corrosive to ferrous metals and copper. As such, any ferrous metal or copper components of the subject buildings (such as cast iron or ductile iron piping, copper tubing, etc.) that are expected to be placed in direct contact with site soils should be protected against detrimental effects of extremely corrosive soils based on recommendations provided by a qualified corrosion engineer.

REPORT LIMITATIONS

This report is based solely on the results of our site reconnaissance, and our review of the referenced reports and literature. The conclusions and recommendations contained in this report are presented on that basis.

This report has been prepared consistent with the level of care being provided by other professionals providing similar services at the same locale and in the same time period. This report provides our professional opinions and, as such, they are not to be considered a guaranty or warranty.

This report has not been prepared for use by parties or projects other than those named or described herein. This report may not contain sufficient information for other parties or other purposes.

We sincerely appreciate this opportunity to be of service. Please do not hesitate to call the undersigned if you have any questions regarding this report.

Respectfully submitted,

PETRA GEOSCIENCES, INC.

Alan Pace  
Senior Associate Geologist  
CEG 1952  
AP/JMS/Im

J. Montgomery Schultz  
Senior Project Engineer  
GE 2941  
2/15/17
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REFERENCES


Aerial Photographs Reviewed

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Approximate location of Project Boundary
Well Location (DWR Record Available)
Approximate location of Project Footprint
Approximate location of exploratory test pit

Approx. 1 Mile

Base map: USGS Eagle Mountains and Palm Springs, California, 30X60 minute quadrangles
Base map: U.S. Geological Survey Cottonwood Basin 7.5 minute quadrangle, 1984

EXPLANATION

Approximate location of Project Boundary
Approximate location of Project Footprint
Approximate location of exploratory test pit
Approximate location of buried basement fault, from previous geophysical study by GeoVision (2005)
Approximate location of geologic contact
Quaternary younger alluvium
Quaternary older alluvium-unit 1
Quaternary older alluvium-unit 2
Quaternary Ocotillo Formation
Pre-Tertiary granitic and gneissic rocks

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GEOTECHNICAL MAP
PARADISE VALLEY
RIVERSIDE COUNTY, CALIFORNIA

DATE: February 2017
J.N.: 346-11
DWG BY: DB/AWAC
SCALE: 1" = 2,000'
**EXPLANATION**

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<th>Geologic Time Scale</th>
<th>Years Before Present (Approx.)</th>
<th>Fault Symbol</th>
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<td>Displacement during historical time (e.g., San Andreas fault 1906). Includes areas of known fault creep.</td>
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<td>11,700</td>
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<td>Displacement during Holocene time.</td>
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<td>700,000</td>
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<td>Faults showing evidence of displacement during late Quaternary time.</td>
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<td>1,600,000</td>
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<td>Undifferentiated Quaternary faults. Most faults in this category show evidence of displacement during the last 1,600,000 years. Possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.</td>
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<td>Fault cuts strata of Pliocene or older age.</td>
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* Quaternary now recognized as extending to 2.6 Ma (Walker and Geissman, 2009). Quaternary faults in this map were established using the previous 1.6 Ma criterion.

Source: Jennings and Bryant, 2010
Off-Site Improvements/Alternatives and Geology

EXPLANATION

- Substation Avenue 52 ROW
- At&T Fiber Optic Line Splice and Replacement
- MWD Turnout, Waterline, and Access Road
- Subject Property
- On-Site Development Footprint

Geological Layers:
- KTo: Jurassic & Cretaceous, mainly granitoid rocks
- Qal: Quaternary Alluvium
- Tsu: Late Cenozoic - Pleistocene sedimentary rocks
- KJp: Jurassic & Cretaceous, mainly granitoid rocks
- PC: Proterozoic plutonic & metamorphic rocks

MapLegend:
- Powerline Alignment
- Alternative Powerline Alignment
- At&T Fiber Optic Line Splice and Replacement
- MWD Turnout, Waterline, and Access Road

Date: February 2017

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APPENDIX A

EXPLORATION LOGS
### Logs of Test Pits

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<tr>
<th>Test Pit Number</th>
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<th>Description</th>
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<td><strong>Older Alluvium (Ooa)</strong>&lt;br&gt;Poorly-graded GRAVEL (GP): yellowish brown, dry, loose; fine to coarse sand (30%), fine to coarse gravel (40%), boulders (15%), interfingered with Poorly-graded SAND with gravel.</td>
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<tr>
<td></td>
<td>2.0 - 6.0'</td>
<td>Silty SAND (SM): yellowish brown, dry, medium dense; fine to coarse sand (15%), fine to coarse gravel (15%), cobbles (40%), boulders (15%).</td>
</tr>
<tr>
<td></td>
<td></td>
<td>TOTAL DEPTH = 6 feet&lt;br&gt;NO GROUNDWATER ENCOUNTERED, NO CAVING&lt;br&gt;Backfilled w/spoils 9/18/13</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test Pit Number</th>
<th>Depth</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP-4</td>
<td>0.0 - 7.0'</td>
<td><strong>Older Alluvium (Ooa)</strong>&lt;br&gt;Silty GRAVEL (GM): yellowish brown, dry, loose to medium dense; fine to coarse sand (30%), fine to coarse gravel (50%), cobbles (10%), boulders (10%), moderate porosity in the upper foot, interfingered with Poorly-graded SAND, interfingered with Poorly-graded SAND, slight porosity at 6 feet, fine rootlets in upper four feet.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>TOTAL DEPTH = 7 feet&lt;br&gt;NO GROUNDWATER ENCOUNTERED, NO CAVING&lt;br&gt;Backfilled w/spoils 9/18/13</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test Pit Number</th>
<th>Depth</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP-5</td>
<td>3.0 - 7.0'</td>
<td><strong>Older Alluvium (Ooa)</strong>&lt;br&gt;Poorly-graded GRAVEL (GP): yellowish brown, dry, medium dense; fine to coarse sand (30%), fine to coarse gravel (40%), cobbles (20%), boulders (10%).</td>
</tr>
<tr>
<td></td>
<td>7.0 - 14.0'</td>
<td>Poorly-graded SAND (SP): yellowish brown, dry, medium dense; fine to coarse sand (70%), fine to coarse gravel (30%), trace silt, interfingered with 6-inch to 1-foot lenses of Poorly-graded GRAVEL.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>TOTAL DEPTH = 14 feet&lt;br&gt;NO GROUNDWATER ENCOUNTERED, NO CAVING&lt;br&gt;Backfilled w/spoils 9/18/13</td>
</tr>
</tbody>
</table>
# Logs of Test Pits

<table>
<thead>
<tr>
<th>Test Pit Number</th>
<th>Depth</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Older Alluvium (Qoa₁)</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TP-6A</td>
<td>0.0 - 6.0'</td>
<td>Poorly-graded GRAVEL (GP): yellowish brown, dry, loose; fine to coarse sand (20%), fine to coarse gravel (40%), cobbles (30%), boulders (10%), interlayered with Poorly-graded SAND (SP): yellowish brown, dry, loose; fine to coarse sand (70%), fine to coarse gravel (30%), trace silt (1&quot; to 1' lenses), voids down to 5 feet, fine rootlets down to 2 feet. Bulk sample at 0 to 4 feet. TOTAL DEPTH = 6 feet</td>
</tr>
<tr>
<td><strong>Wash Deposits (Qw)</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TP-7</td>
<td>0.0 - 1.0'</td>
<td>Silty SAND (SM): yellowish brown, dry, loose; fine to coarse sand (40%), fine to coarse gravel (20%), cobbles (10%), boulders (10%), porous, fine rootlets interfingered with Silty GRAVEL.</td>
</tr>
<tr>
<td>1.0 - 6.0'</td>
<td>Older Alluvium (Qoa₁)</td>
<td>Poorly-graded GRAVEL (GP): yellowish brown, dry, loose; fine to coarse sand (20%), fine to coarse gravel (40%), cobbles (30%), boulders (10%), interlayered with Poorly-graded SAND (SP) lenses with moderate porosity</td>
</tr>
<tr>
<td><strong>Older Alluvium (Qoa₂)</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TP-8</td>
<td>0.0 – 2.6'</td>
<td>Clayey SAND (SC): reddish brown, dry, dense; fine to coarse sand (30%), fine to coarse gravel (30%), cobbles (15%), boulders (10%), moderate porosity, well developed desert varnish at surface. Bulk sample at 0 to 2.6 feet.</td>
</tr>
<tr>
<td>2.6 – 4.2'</td>
<td>Poorly-graded GRAVEL (GP): yellowish brown, dry, loose; fine to coarse sand (20%), fine to coarse gravel (40%), cobbles (30%), boulders (10%).</td>
<td></td>
</tr>
<tr>
<td>4.2 – 6.0'</td>
<td>Poorly-graded SAND (SP): white/yellowish brown, dry, very dense; fine to coarse sand (70%), fine to coarse gravel (30%), cobbles (30%), boulders (10%), abundant caliche cementation.</td>
<td>TOTAL DEPTH = 6 feet NO GROUNDWATER ENCOUNTERED, NO CAVING Backfilled w/spoils 9/18/13</td>
</tr>
</tbody>
</table>
# Logs of Test Pits

<table>
<thead>
<tr>
<th>Test Pit Number</th>
<th>Depth</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP-9A</td>
<td>0.0 – 3.0”</td>
<td><strong>Topsoil</strong>&lt;br&gt;Silty GRAVEL (GM): yellowish brown, dry, loose; fine sand (30%), fine to coarse gravel (40%), cobbles (30%), boulders (10%), porous.</td>
</tr>
<tr>
<td></td>
<td>3.0” - 2.0’</td>
<td><strong>Older Alluvium (Qoa₁)</strong>&lt;br&gt;Silty GRAVEL (GM): reddish brown, dry, dense; fine to coarse sand (30%), fine to coarse gravel (55%).&lt;br&gt;Bulk sample at 2 to 5 feet.</td>
</tr>
<tr>
<td></td>
<td>2.0 – 2.5’</td>
<td>Poorly-graded GRAVEL (GP): gray, dry, very dense; fine to coarse sand (30%), fine to coarse gravel (60%), cobbles (10%), trace silt.</td>
</tr>
<tr>
<td></td>
<td>2.5 – 3.0’</td>
<td>Poorly-graded SAND (SP): gray, dry, very dense; fine to coarse sand (70%), fine to coarse gravel (30%).</td>
</tr>
<tr>
<td></td>
<td>3.0 – 6.0’</td>
<td>Silty GRAVEL (GM): gray, dry, very dense; fine to coarse sand (20%), fine to coarse gravel (20%), cobbles (30%), boulders (15%).</td>
</tr>
<tr>
<td></td>
<td></td>
<td>TOTAL DEPTH = 6 feet</td>
</tr>
<tr>
<td></td>
<td></td>
<td>NO GROUNDWATER ENCOUNTERED, NO CAVING</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Backfilled w/spoils 9/18/13</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test Pit Number</th>
<th>Depth</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP-10</td>
<td>0.0 – 2.6’</td>
<td><strong>Older Alluvium (Qoa₁)</strong>&lt;br&gt;Poorly-graded GRAVEL (GP): yellowish brown, dry, loose; fine to coarse sand (30%), fine to coarse gravel (40%), cobbles (20%), boulders (10%).</td>
</tr>
<tr>
<td></td>
<td>1.0 – 6.0’</td>
<td>Poorly-graded GRAVEL (GP): yellowish brown, dry, loose; fine to coarse sand (20%), fine to coarse gravel (50%), cobbles (20%), boulders (10%), interbedded with Poorly-graded SAND (SP): yellowish brown, dry, loose to medium dense, fine to coarse sand (60%), fine to coarse gravel (40%).</td>
</tr>
<tr>
<td></td>
<td></td>
<td>TOTAL DEPTH = 6 feet</td>
</tr>
<tr>
<td></td>
<td></td>
<td>NO GROUNDWATER ENCOUNTERED, NO CAVING</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Backfilled w/spoils 9/18/13</td>
</tr>
</tbody>
</table>
# Logs of Test Pits

<table>
<thead>
<tr>
<th>Test Pit Number</th>
<th>Depth</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP-11</td>
<td>0.0 - 4.0’</td>
<td><strong>Younger Alluvium (Qal)</strong> Poorly-graded GRAVEL with silt (GP-GM): yellowish brown, dry, loose; fine to coarse sand (30%), fine to coarse gravel (40%), cobbles (10%), boulders (10%), moderate porosity.</td>
</tr>
<tr>
<td></td>
<td>4.0 - 7.0’</td>
<td>Silty GRAVEL (GM): yellowish brown, dry, loose to medium dense; fine to coarse sand (20%), fine to coarse gravel (45%), cobbles (10%), boulders (10%).</td>
</tr>
<tr>
<td></td>
<td></td>
<td>TOTAL DEPTH = 7 feet NO GROUNDWATER ENCOUNTERED, NO CAVING Backfilled w/spoils 9/18/13</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test Pit Number</th>
<th>Depth</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP-12</td>
<td>0.0 – 2.5’</td>
<td><strong>Wash Deposits (Qw)</strong> Silty GRAVEL (GM): yellowish brown, dry, loose to medium dense; fine to coarse sand (20%), fine to coarse gravel (30%), cobbles (20%), boulders (15%), porous, fine rootlets.</td>
</tr>
<tr>
<td></td>
<td>2.5 - 5.0’</td>
<td><strong>Younger Alluvium (Qal)</strong> Silty GRAVEL (GM): yellowish brown, dry, medium dense; fine to coarse sand (20%), fine to coarse gravel (30%), cobbles (15%), boulders (20%).</td>
</tr>
<tr>
<td></td>
<td>5.0 - 6.0’</td>
<td>Clayey SAND (SC): reddish brown, dry, medium dense to dense, fine to coarse sand (80%). Ring sample at 5 feet. Bulk sample at 5 to 9 feet.</td>
</tr>
<tr>
<td></td>
<td>6.0 - 9.0’</td>
<td>Silty SAND (SM): reddish brown, dry, medium dense; fine to coarse sand (55%), fine to coarse gravel (15%), cobbles (10%).</td>
</tr>
<tr>
<td></td>
<td></td>
<td>TOTAL DEPTH = 9 feet NO GROUNDWATER ENCOUNTERED, NO CAVING Backfilled w/spoils 9/18/13</td>
</tr>
</tbody>
</table>
## Logs of Test Pits

<table>
<thead>
<tr>
<th>Test Pit Number</th>
<th>Depth</th>
<th>Description</th>
</tr>
</thead>
</table>
| TP-13           | 0.0 – 3.5’ | Wash Deposits (Qw)  
Poorly-graded SAND (SP): yellowish brown, dry, loose; fine to coarse sand (20%), fine to coarse gravel (20%), cobbles (10%), interbedded with Silty GRAVEL and GRAVEL, moderate porosity throughout.  
Younger Alluvium (Qal)  
Poorly-graded GRAVEL (GP): yellowish brown, dry, loose to medium dense (at 6 feet); fine to coarse sand (30%), fine to coarse gravel (40%), cobbles (20%), boulders (10%).  
TOTAL DEPTH = 7 feet  
NO GROUNDWATER ENCOUNTERED, NO CAVING  
Backfilled w/spoils 9/18/13 |

<table>
<thead>
<tr>
<th>TEST PIT NUMBER</th>
<th>DEPTH</th>
<th>DESCRIPTION</th>
</tr>
</thead>
</table>
| TP-15           | 0.0 – 2.5’ | Wash Deposits (Qw)  
Poorly-graded SAND (SP): yellowish brown, dry, loose; fine to coarse sand (70%), fine to coarse gravel (30%) interfingered with Silty GRAVEL and Silty SAND.  
Bag at 1.0’  
2.5 – 3.5’ | Poorly-graded GRAVEL (GP): yellowish brown, dry, loose; fine to coarse sand (30%), fine to coarse gravel (60%), cobbles (10%).  
3.5 – 7.0’ | Younger Alluvium (Qal)  
Poorly-graded SAND (SP): yellowish brown, dry, loose; fine to coarse sand (80%), fine to coarse gravel (20%), interfingered with lenses of Silty GRAVEL.  
medium dense at 5.5 feet.  
boulders at 6.5 feet.  
TOTAL DEPTH = 7 feet  
NO GROUNDWATER ENCOUNTERED, NO CAVING  
Backfilled w/spoils 9/18/13 |
APPENDIX B

LABORATORY TEST PROCEDURES

LABORATORY DATA SUMMARY
LABORATORY TEST PROCEDURES

Soil Classification

Soils encountered within the property were classified and described using the visual-manual procedures of the Unified Soil Classification System in general accordance with Test Method ASTM D 2488. The assigned group symbols are presented in the Exploration Log, Appendix A.

Laboratory Maximum Dry Density/Optimum Moisture

The maximum dry density and optimum moisture content of the on-site soils were determined for selected samples in accordance with ASTM D 1557. The results of these tests are presented on Plate B-1.

Expansion Potential

Expansion index tests were performed on a selected soil sample in accordance with ASTM 4829. The results of this test is presented on Plate B-1.

Soluble Sulfates and Chlorides

Chemical analyses were performed on selected samples of the onsite soils to determine soluble sulfate and chloride contents in accordance with California Test Method Nos. 417 and 422, respectively. Test results are presented on Plate B-1.

pH and Minimum Resistivity

pH and minimum resistivity tests were performed on selected samples of the onsite soils to provide a preliminary evaluation of their corrosive potential to concrete and metal construction materials. These tests were performed in accordance with California Test Method No. 643. The results of these tests are included in Plate B-1.

Grain Size Analysis

Grain size analyses were performed on selected samples in accordance with ASTM C136. The test results are graphically presented on Plates B-2 through B-5.

Direct Shear

The Coulomb shear strength parameters (angle of internal friction and cohesion) were determined for a selected undisturbed sample of the in situ soils located at the approximate proposed basement level. This test was performed in general accordance with ASTM D 3080. Three specimens were prepared for the test. The test specimens were artificially saturated, and then sheared under varying normal loads at a maximum constant rate of strain of 0.05 inches per minute. The test results are graphically presented on Plates B-6 and B-7.

PETRA GEOTECHNICAL, INC.
J.N. 11-346
<table>
<thead>
<tr>
<th>Test Pit Number</th>
<th>Sample Depth (ft)</th>
<th>Soil Description</th>
<th>Max. Dry Density (pcf)</th>
<th>Optimum Moisture (%)</th>
<th>Expansion Index</th>
<th>Expansion Potential</th>
<th>Atterberg Limits</th>
<th>Sulfate Content (%)</th>
<th>Chloride Content (ppm)</th>
<th>pH</th>
<th>Minimum Resistivity (Ohm-cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP-6A</td>
<td>0-4</td>
<td>Gravelly SAND (SP)</td>
<td>145.0</td>
<td>6.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TP-8</td>
<td>0-2.5</td>
<td>Clayey SAND (SC)</td>
<td>137.0</td>
<td>7.5</td>
<td>22</td>
<td>Low</td>
<td></td>
<td>0.03</td>
<td>73</td>
<td>7.9</td>
<td>2000</td>
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<tr>
<td>TP-9A</td>
<td>2-5</td>
<td>Gravelly SAND (SP)</td>
<td>142.0</td>
<td>5.5</td>
<td></td>
<td></td>
<td></td>
<td>0.06</td>
<td>80</td>
<td>7.4</td>
<td>1200</td>
</tr>
<tr>
<td>TP-12</td>
<td>5-9</td>
<td>Silty SAND (SM)</td>
<td>136.0</td>
<td>8.0</td>
<td></td>
<td></td>
<td></td>
<td>0.42</td>
<td>185</td>
<td>7.5</td>
<td>620</td>
</tr>
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</table>

Test Procedures:  
1. Per ASTM Test Method D 1557  
2. Per ASTM Test Method D 4829  
3. Per ASTM Test Method D 4318  
4. Per Caltrans Test Method 417  
5. Per Caltrans Test Method 422  
6. Per Caltrans Test Method 643
## GRAIN SIZE ANALYSIS

**Specimen Identification**: TP-9A  
**Classification**: POORLY GRADED SAND with GRAVEL (SP)  
**MC%**: 0.47  
**LL**: 28.9

<table>
<thead>
<tr>
<th>COBBLES</th>
<th>GRAVEL</th>
<th>SAND</th>
<th>SILT OR CLAY</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>coarse</td>
<td>fine</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>Specimen Identification</th>
<th>D100</th>
<th>D60</th>
<th>D30</th>
<th>D50</th>
<th>%Gravel</th>
<th>%Sand</th>
<th>%Silt</th>
<th>%Clay</th>
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</thead>
<tbody>
<tr>
<td>TP-9A</td>
<td>2.0</td>
<td>75.00</td>
<td>9.56</td>
<td>1.218</td>
<td>6.0842</td>
<td>41.2</td>
<td>42.4</td>
<td>2.1</td>
</tr>
</tbody>
</table>

**J.N. 346-11**  
**PETRA GEOTECHNICAL, INC.**  
**GRAIN SIZE ANALYSIS**  
**December, 2013**  
**PLATE B-3**
SAMPLE LOCATION | DESCRIPTION            | FRICTION ANGLE (°) | COHESION (PSF) |
----------------|-------------------------|--------------------|-----------------|
● TP- 9A @ 2.0  | SAND (SP) - Peak        | 37                 | 460             |
● TP- 9A @ 2.0  | SAND (SP) @ 0.25" Displ.| 36                 | 0               |

NOTES:
Samples Remolded to 90% of Maximum Dry Density
All Samples Were Presoaked Prior to Shearing
### Notes:
Samples Remolded to 90% of Maximum Dry Density
All Samples Were Presoaked Prior to Shearing

### Sample Data

<table>
<thead>
<tr>
<th>Sample Location</th>
<th>Description</th>
<th>Friction Angle (°)</th>
<th>Cohesion (PSF)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP-12 @ 6.0</td>
<td>SILTY SAND (SM) - Peak</td>
<td>34</td>
<td>450</td>
</tr>
<tr>
<td>TP-12 @ 6.0</td>
<td>SILTY SAND (SM) @ 0.25° Displ.</td>
<td>30</td>
<td>275</td>
</tr>
</tbody>
</table>

---

**DIRECT SHEAR TEST DATA**

J.N. 346-11

PETRA GEOTECHNICAL, INC.

December, 2013

PLATE B-7
STANDARD GRADING SPECIFICATIONS

These specifications present the usual and minimum requirements for projects on which Petra Geotechnical, Inc. is the geotechnical consultant. No deviation from these specifications will be allowed, except where specifically superseded in the preliminary geology and soils report, or in other written communication signed by the Soils Engineer and Engineering Geologist of record (Geotechnical Consultant).

I. GENERAL

A. The Geotechnical Consultant is the Owner's or Builder's representative on the project. For the purpose of these specifications, participation by the Geotechnical Consultant includes that observation performed by any person or persons employed by, and responsible to, the licensed Soils Engineer and Engineering Geologist signing the soils report.

B. The contractor should prepare and submit to the Owner and Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "spreads" and the estimated quantities of daily earthwork to be performed prior to the commencement of grading. This work plan should be reviewed by the Geotechnical Consultant to schedule personnel to perform the appropriate level of observation, mapping, and compaction testing as necessary.

C. All clearing, site preparation, or earthwork performed on the project shall be conducted by the Contractor in accordance with the recommendations presented in the geotechnical report and under the observation of the Geotechnical Consultant.

D. It is the Contractor's responsibility to prepare the ground surface to receive the fills to the satisfaction of the Geotechnical Consultant and to place, spread, mix, water, and compact the fill in accordance with the specifications of the Geotechnical Consultant. The Contractor shall also remove all material considered unsatisfactory by the Geotechnical Consultant.

E. It is the Contractor's responsibility to have suitable and sufficient compaction equipment on the job site to handle the amount of fill being placed. If necessary, excavation equipment will be shut down to permit completion of compaction to project specifications. Sufficient watering apparatus will also be provided by the Contractor, with due consideration for the fill material, rate of placement, and time of year.

F. After completion of grading a report will be submitted by the Geotechnical Consultant.

II. SITE PREPARATION

A. Clearing and Grubbing

1. All vegetation such as trees, brush, grass, roots, and deleterious material shall be disposed of offsite. This removal shall be concluded prior to placing fill.

2. Any underground structures such as cesspools, cisterns, mining shafts, tunnels, septic tanks, wells, pipe lines, etc., are to be removed or treated in a manner prescribed by the Geotechnical Consultant.
STANDARD GRADING SPECIFICATIONS

III. FILL AREA PREPARATION

A. Remedial Removals/Overexcavations

1. Remedial removals, as well as overexcavation for remedial purposes, shall be evaluated by the Geotechnical Consultant. Remedial removal depths presented in the geotechnical report and shown on the geotechnical plans are estimates only. The actual extent of removal should be determined by the Geotechnical Consultant based on the conditions exposed during grading. All soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as determined by the Geotechnical Consultant.

2. Soil, alluvium, or bedrock materials determined by the Soils Engineer as being unsuitable for placement in compacted fills shall be removed from the site. Any material incorporated as a part of a compacted fill must be approved by the Geotechnical Consultant.

3. Should potentially hazardous materials be encountered, the Contractor should stop work in the affected area. An environmental consultant specializing in hazardous materials should be notified immediately for evaluation and handling of these materials prior to continuing work in the affected area.

B. Evaluation/Acceptance of Fill Areas

All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide sufficient survey control for determining locations and elevations of processed areas, keys, and benches.

C. Processing

After the ground surface to receive fill has been declared satisfactory for support of fill by the Geotechnical Consultant, it shall be scarified to a minimum depth of 6 inches and until the ground surface is uniform and free from ruts, hollows, hummocks, or other uneven features which may prevent uniform compaction.

The scarified ground surface shall then be brought to optimum moisture, mixed as required, and compacted to a minimum relative compaction of 90 percent.

D. Subdrains

Subdrainage devices shall be constructed in compliance with the ordinances of the controlling governmental agency, and/or with the recommendations of the Geotechnical Consultant. (Typical Canyon Subdrain details are given on Plate SG-1).
STANDARD GRADING SPECIFICATIONS

E. Cut/Fill & Deep Fill/Shallow Fill Transitions

In order to provide uniform bearing conditions in cut/fill and deep fill/shallow fill transition lots, the cut and shallow fill portions of the lot should be overexcavated to the depths and the horizontal limits discussed in the approved geotechnical report and replaced with compacted fill. (Typical details are given on Plate SG-7.)

IV. COMPACTED FILL MATERIAL

A. General

Materials excavated on the property may be utilized in the fill, provided each material has been determined to be suitable by the Geotechnical Consultant. Material to be used for fill shall be essentially free of organic material and other deleterious substances. Roots, tree branches, and other matter missed during clearing shall be removed from the fill as recommended by the Geotechnical Consultant. Material that is spongy, subject to decay, or otherwise considered unsuitable shall not be used in the compacted fill.

Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

B. Oversize Materials

Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 12 inches in diameter, shall be taken offsite or placed in accordance with the recommendations of the Geotechnical Consultant in areas designated as suitable for rock disposal (Typical details for Rock Disposal are given on Plate SG-4).

Rock fragments less than 12 inches in diameter may be utilized in the fill provided, they are not nested or placed in concentrated pockets; they are surrounded by compacted fine grained soil material and the distribution of rocks is approved by the Geotechnical Consultant.

C. Laboratory Testing

Representative samples of materials to be utilized as compacted fill shall be analyzed by the laboratory of the Geotechnical Consultant to determine their physical properties. If any material other than that previously tested is encountered during grading, the appropriate analysis of this material shall be conducted by the Geotechnical Consultant as soon as possible.
STANDARD GRADING SPECIFICATIONS

D. Import

If importing of fill material is required for grading, proposed import material should meet the requirements of the previous section. The import source shall be given to the Geotechnical Consultant at least 2 working days prior to importing so that appropriate tests can be performed and its suitability determined.

V. FILL PLACEMENT AND COMPACTION

A. Fill Layers

Material used in the compacting process shall be evenly spread, watered, processed, and compacted in thin lifts not to exceed 6 inches in thickness to obtain a uniformly dense layer. The fill shall be placed and compacted on a horizontal plane, unless otherwise approved by the Geotechnical Consultant.

B. Moisture Conditioning

Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly above optimum moisture content.

C. Compaction

Each layer shall be compacted to 90 percent of the maximum density in compliance with the testing method specified by the controlling governmental agency. (In general, ASTMD 1557-02, will be used.)

If compaction to a lesser percentage is authorized by the controlling governmental agency because of a specific land use or expansive soils condition, the area to received fill compacted to less than 90 percent shall either be delineated on the grading plan or appropriate reference made to the area in the soils report.

D. Failing Areas

If the moisture content or relative density varies from that required by the Geotechnical Consultant, the Contractor shall rework the fill until it is approved by the Geotechnical Consultant.

E. Benching

All fills shall be keyed and benched through all topsoil, colluvium, alluvium or creep material, into sound bedrock or firm material where the slope receiving fill exceeds a ratio of 5 horizontal to 1 vertical, in accordance with the recommendations of the Geotechnical Consultant.
VI. SLOPES

A. Fill Slopes

The contractor will be required to obtain a minimum relative compaction of 90 percent out to the finish slope face of fill slopes, buttresses, and stabilization fills. This may be achieved by either overbuilding the slope and cutting back to the compacted core, or by direct compaction of the slope face with suitable equipment, or by any other procedure that produces the required compaction.

B. Side Hill Fills

The key for side hill fills shall be a minimum of 15 feet within bedrock or firm materials, unless otherwise specified in the soils report. (See detail on Plate SG-5.)

C. Fill-Over-Cut Slopes

Fill-over-cut slopes shall be properly keyed through topsoil, colluvium or creep material into rock or firm materials, and the transition shall be stripped of all soils prior to placing fill. (See detail on Plate SG-6.)

D. Landscaping

All fill slopes should be planted or protected from erosion by other methods specified in the soils report.

E. Cut Slopes

1. The Geotechnical Consultant should observe all cut slopes at vertical intervals not exceeding 10 feet.

2. If any conditions not anticipated in the preliminary report such as perched water, seepage, lenticular or confined strata of a potentially adverse nature, unfavorably inclined bedding, joints or fault planes are encountered during grading, these conditions shall be evaluated by the Geotechnical Consultant, and recommendations shall be made to treat these problems (Typical details for stabilization of a portion of a cut slope are given in Plates SG-2 and SG-3.).

3. Cut slopes that face in the same direction as the prevailing drainage shall be protected from slope wash by a non-erodible interceptor swale placed at the top of the slope.

4. Unless otherwise specified in the soils and geological report, no cut slopes shall be excavated higher or steeper than that allowed by the ordinances of controlling governmental agencies.

5. Drainage terraces shall be constructed in compliance with the ordinances of controlling governmental agencies, or with the recommendations of the Geotechnical Consultant.
STANDARD GRADING SPECIFICATIONS

VII. GRADING OBSERVATION

A. General

All cleanouts, processed ground to receive fill, key excavations, subdrains, and rock disposals must be observed and approved by the Geotechnical Consultant prior to placing any fill. It shall be the Contractor's responsibility to notify the Geotechnical Consultant when such areas are ready.

B. Compaction Testing

Observation of the fill placement shall be provided by the Geotechnical Consultant during the progress of grading. Location and frequency of tests shall be at the Consultants discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations may be selected to verify adequacy of compaction levels in areas that are judged to be susceptible to inadequate compaction.

C. Frequency of Compaction Testing

In general, density tests should be made at intervals not exceeding 2 feet of fill height or every 1000 cubic yards of fill placed. This criteria will vary depending on soil conditions and the size of the job. In any event, an adequate number of field density tests shall be made to verify that the required compaction is being achieved.

VIII. CONSTRUCTION CONSIDERATIONS

A. Erosion control measures, when necessary, shall be provided by the Contractor during grading and prior to the completion and construction of permanent drainage controls.

B. Upon completion of grading and termination of observations by the Geotechnical Consultant, no further filling or excavating, including that necessary for footings, foundations, large tree wells, retaining walls, or other features shall be performed without the approval of the Geotechnical Consultant.

C. Care shall be taken by the Contractor during final grading to preserve any berms, drainage terraces, interceptor swales, or other devices of permanent nature on or adjacent to the property.
SUBDRAIN SYSTEM:
9 CUBIC FEET PER LINEAL FOOT OF OPEN-GRADED GRAVEL ENCASED IN FILTER FABRIC. SEE PLATE SG-3 FOR OPEN-GRADED GRAVEL SPECIFICATIONS.

FILTER FABRIC SHALL CONSIST OF MIRAFL 140N OR APPROVED EQUIVALENT. FILTER FABRIC SHOULD BE LAPPED A MINIMUM OF 12 INCHES.

ALTERNATE SUBDRAIN SYSTEM:
MINIMUM OF 9 CUBIC FEET PER LINEAL FOOT OF CLASS 2 FILTER MATERIAL. SEE PLATE SG-3 FOR CLASS 2 FILTER MATERIAL SPECIFICATIONS. CLASS 2 MATERIAL DOES NOT NEED TO BE ENCASED IN FILTER FABRIC.

MINIMUM 6-INCH DIAMETER PVC SCHEDULE 40, OR ABS SDR-35 WITH A MINIMUM OF EIGHT 1/4-INCH DIAMETER PERFORATIONS PER LINEAL FOOT IN BOTTOM HALF OF PIPE. PIPE TO BE LAID WITH PERFORATIONS FACING DOWN.

NOTES:
1. FOR CONTINUOUS RUNS IN EXCESS OF 500 FEET USE 8-INCH DIAMETER PIPE.
2. FINAL 20 FEET OF PIPE AT OUTLET SHALL BE NON-PERFORATED AND BACKFILLED WITH FINE-GRAINED MATERIAL.
NOTES:
1. 30' maximum vertical spacing between subdrain systems.
2. 100' maximum horizontal distance between non-perforated outlet pipes. (See Below)
3. Minimum gradient of 2% for all perforated and non-perforated pipe.

SECTION A-A (PERFORATED PIPE PROFILE)
SECTION B-B (OUTLET PIPE)

PIPE SPECIFICATIONS:
1. 4-INCH MINIMUM DIAMETER, PVC SCHEDULE 40 OR ABS SDR-35.
2. FOR PERFORATED PIPE, MINIMUM 8 PERFORATIONS PER FOOT ON BOTTOM HALF OF PIPE.

FILTER MATERIAL/FABRIC SPECIFICATIONS:
OPEN-GRADED GRAVEL ENCASED IN FILTER FABRIC.
(MIRAFI 140N OR EQUIVALENT)

OPEN-GRADED GRAVEL

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<td>88 - 100</td>
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<tr>
<td>1-INCH</td>
<td>5 - 40</td>
</tr>
<tr>
<td>3/4-INCH</td>
<td>0 - 17</td>
</tr>
<tr>
<td>3/8-INCH</td>
<td>0 - 7</td>
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<tr>
<td>No. 200</td>
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ALTERNATE:
CLASS 2 PERMEABLE FILTER MATERIAL PER CALTRANS STANDARD SPECIFICATION 68-1.025.

CLASS 2 FILTER MATERIAL

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</tbody>
</table>

PETRA

BUTTRESS OR STABILIZATION FILL SUBDRAIN

PLATE SG-3
FINISHED GRADE

CLEAR AREA FOR FOUNDATIONS, UTILITIES AND SWIMMING POOLS

SLOPE FACE

STREET

WINDROW

COMPACTED FILL

10' MIN.

15' MIN.

15' MIN.

5' OR MIN. OF 2' BELOW DEPTH OF DEEPEST UTILITY TRENCH, WHICHEVER IS GREATER

TYPICAL WINROW DETAIL (END VIEW)

GRANULAR SOIL JETTED OR FLOODED TO FILL VOIDS

COMPACTED FILL

PLACED IN 3 TO 8 INCH THICK HORIZONTAL LIFTS

15' MIN.

TYPICAL WINROW DETAIL (PROFILE VIEW)

JETTED OR FLOODED GRANULAR SOIL

100' MAX.

NOTE: OVERSIZE ROCK IS DEFINED AS CLASTS HAVING A MAXIMUM DIMENSION OF 12" OR LARGER

PETRA

TYPICAL ROCK DISPOSAL DETAIL

PLATE SG-4
NOTES:
1. WHERE NATURAL SLOPE GRADIENT IS 5:1 OR LESS, BENCHING IS NOT NECESSARY; HOWEVER, FILL IS NOT TO BE PLACED ON COMPRESSIBLE OR UNSUITABLE MATERIAL.
2. SOILS ENGINEER TO DETERMINE IF SUBDRAIN IS REQUIRED.
PROPOSED GRADE

COMPACTED FILL

VARIABLE
10' TYPICAL

WEATHERED BEDROCK

COLLUVIUM

15' MINIMUM
KEY WIDTH

ALLUVIUM

TOPSOIL

CUT

NATURAL GROUND
SURFACE

REMOVE UNSUITABLE
MATERIAL

CUT / FILL CONTACT
SHOWN ON GRADING PLAN
SHOWN ON AS-BUILT

WEATHERED BEDROCK

COMPETENT BEDROCK OR SOIL MATERIALS
AS DETERMINED BY THE
GEOTECHNICAL CONSULTANT

MAINTAIN 15' MIN. HORIZONTAL WIDTH
FROM SLOPE FACE TO BENCH / BACKCUT

INSTALLATION OF SUBDRAIN TO BE DETERMINED
BY THE GEOTECHNICAL CONSULTANT.
IF REQUIRED, SEE PLATES SG-2 AND SG-3
FOR TYPICAL SUBDRAIN DETAILS.

THE CUT PORTION OF THE SLOPE SHOULD BE EXCAVATED
AND EVALUATED BY THE ENGINEERING GEOLOGIST PRIOR
TO CONSTRUCTING THE FILL PORTION OF THE SLOPE.
CUT LOT
UNSUITABLE MATERIAL EXPOSED IN PORTION OF CUT PAD

ORIGINAL GROUND SURFACE

PROPOSED GRADE
WEATHERED BEDROCK

ALLUVIUM

REMOVE UNSUITABLE MATERIAL

TOPSOIL

(D) or 5' MIN

COMPACTED FILL

OVEREXCAVATE AND RECOMPACT

COMPETENT BEDROCK OR SOIL MATERIALS AS DETERMINED BY THE GEOTECHNICAL CONSULTANT

TYPICAL BENCHING

CUT-FILL TRANSITION LOT

ORIGINAL GROUND SURFACE

PROPOSED GRADE
WEATHERED BEDROCK

COMPACTED FILL

TOPSOIL

(D) or 5' MIN

OVEREXCAVATE AND RECOMPACT

COMPETENT BEDROCK OR SOIL MATERIALS AS DETERMINED BY THE GEOTECHNICAL CONSULTANT

TYPICAL BENCHING

MAXIMUM FILL THICKNESS (F) DEPTH OF OVEREXCAVATION (D)
FOOTING DEPTH TO 3 FEET EQUAL DEPTH
3 TO 6 FEET 3 FEET
GREATER THAN 6 FEET 1/2 THE THICKNESS OF DEEPEST FILL PLACED WITHIN THE "FILL" PORTION (F) TO 15 FEET MAXIMUM
Proposed 2:1 Fill Slope

Existing Ground Surface

Desired Removal Limits Beyond Toe

Temporary Slope

Topsoil, Alluvium, Colluvium, Weathered Bedrock

Remove Unsuitable Material

Typical Benching Into Competent Bedrock or Soil Materials as Determined by the Geotechnical Consultant

.10' Minimum Key Width Embedded a Minimum of 2' Into Competent Bedrock or Soil Materials as Determined by the Geotechnical Consultant

D = Recommended Depth of Removal Per Geotechnical Report

Place Compacted Fill to Natural Slope Grade

Petra

Typical Removals Beyond Toe of Proposed Fill Slope

Plate SG-8
NOTE:
1. "D" SHALL BE 10 FEET MINIMUM OR AS DETERMINED BY SOILS ENGINEER.
NOTE:
1. "W" SHALL BE 10 FEET MINIMUM OR AS DETERMINED BY SOILS ENGINEER.