Appendix B.
Preliminary Geotechnical Investigation
Appendix

This page intentionally left blank.
September 22, 2015

Mr. Mike Woods  
**TEMECULA HOTEL DEVELOPMENT LLC**  
28220 Jefferson, Suite 214  
Temecula, California 92590

**Subject:** Preliminary Geotechnical Investigation for the Proposed Hampton Inn Hotel Project (Phase 1), Located at 721 South Indian Hill Boulevard in the City of Claremont, Los Angeles County, California.

LGC Geo-Environmental, Inc. (LGC) is pleased to submit herewith our preliminary geotechnical investigation report for the proposed Hampton Inn Hotel (Phase 1) project (APN 831-601-901), located at 721 South Indian Hill Boulevard in the City of Claremont, Los Angeles County, California. This report presents the results of our research of published geologic/geotechnical reports and/or maps, review of aerial photographs, field exploration, geologic mapping, and laboratory testing; in addition to our geotechnical and geologic judgment, opinions, conclusions and preliminary recommendations associated with the proposed hospitality development.

Based on the results of our field exploration, geologic mapping, laboratory testing, geologic and geotechnical engineering evaluations, along with review of published literature and the 20-scale preliminary grading plan, it is our opinion that the subject site is suitable for the proposed hospitality development, provided that the recommendations presented herein are utilized during the design, grading, and construction. LGC should review grading plans, as well as any foundation/structural plans when they become available, and revise the recommendations presented herein, if necessary.

It has been a pleasure to be of service to you during the design stages of this project. Should you have any questions regarding the contents of this report or should you require additional information, please do not hesitate to contact us.

Respectfully submitted,

LGC Geo-Environmental, Inc.

[Signatures]

Robert L. Gregorek II, CEG 1257  
Engineering Geologist

ARP/RLG/LDC

Distribution: (4) Addressee
## TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0 INTRODUCTION</td>
<td>1</td>
</tr>
<tr>
<td>1.1 Proposed Construction and Grading</td>
<td>1</td>
</tr>
<tr>
<td>1.2 Location and Site Description</td>
<td>1</td>
</tr>
<tr>
<td>1.3 Topography and Drainage</td>
<td>2</td>
</tr>
<tr>
<td>1.4 Existing Improvements and Vegetation</td>
<td>2</td>
</tr>
<tr>
<td>1.5 Research of Previous Geological and Geotechnical Data</td>
<td>2</td>
</tr>
<tr>
<td>1.6 Aerial Photograph Analysis</td>
<td>2</td>
</tr>
<tr>
<td>2.0 FIELD INVESTIGATION</td>
<td>2</td>
</tr>
<tr>
<td>2.1 Geologic Mapping</td>
<td>2</td>
</tr>
<tr>
<td>2.2 Field Exploration</td>
<td>2</td>
</tr>
<tr>
<td>2.3 Laboratory Testing</td>
<td>4</td>
</tr>
<tr>
<td>3.0 FINDINGS</td>
<td>4</td>
</tr>
<tr>
<td>3.1 Regional Geologic Setting</td>
<td>4</td>
</tr>
<tr>
<td>3.2 Local Geology and Soil Conditions</td>
<td>5</td>
</tr>
<tr>
<td>3.3 Landslides</td>
<td>5</td>
</tr>
<tr>
<td>3.4 Groundwater</td>
<td>6</td>
</tr>
<tr>
<td>3.5 Caving</td>
<td>6</td>
</tr>
<tr>
<td>3.6 Surface Water</td>
<td>6</td>
</tr>
<tr>
<td>3.7 Faulting</td>
<td>6</td>
</tr>
<tr>
<td>3.8 Seismicity</td>
<td>6</td>
</tr>
<tr>
<td>3.9 Settlement-Analysis</td>
<td>7</td>
</tr>
<tr>
<td>4.0 CONCLUSIONS AND RECOMMENDATIONS</td>
<td>7</td>
</tr>
<tr>
<td>4.1 General</td>
<td>7</td>
</tr>
<tr>
<td>5.0 GEOLOGIC CONSIDERATIONS</td>
<td>8</td>
</tr>
<tr>
<td>5.1 Slopes</td>
<td>8</td>
</tr>
<tr>
<td>5.2 Faulting</td>
<td>8</td>
</tr>
<tr>
<td>5.3 Groundwater</td>
<td>8</td>
</tr>
<tr>
<td>5.4 Subsidence</td>
<td>8</td>
</tr>
<tr>
<td>5.5 Landsliding</td>
<td>8</td>
</tr>
<tr>
<td>5.6 Ground Rupture</td>
<td>9</td>
</tr>
<tr>
<td>5.7 Tsunamis and Seiches</td>
<td>9</td>
</tr>
<tr>
<td>6.0 SEISMIC-DESIGN CONSIDERATIONS</td>
<td>9</td>
</tr>
<tr>
<td>6.1 Ground Motions</td>
<td>9</td>
</tr>
<tr>
<td>6.2 Secondary Seismic Hazards</td>
<td>10</td>
</tr>
<tr>
<td>7.0 GEOTECHNICAL-DESIGN PARAMETERS</td>
<td>10</td>
</tr>
<tr>
<td>7.1 Shrinkage/Bulking and Subsidence</td>
<td>10</td>
</tr>
<tr>
<td>7.2 Excavation Characteristics</td>
<td>11</td>
</tr>
<tr>
<td>7.3 Compressible/Collapsible Soils</td>
<td>11</td>
</tr>
<tr>
<td>8.0 SITE EARTHWORK</td>
<td>11</td>
</tr>
<tr>
<td>8.1 General Earthwork and Grading Specifications</td>
<td>11</td>
</tr>
<tr>
<td>8.2 Geotechnical Observations and Testing</td>
<td>11</td>
</tr>
<tr>
<td>8.3 Clearing and Grubbing</td>
<td>12</td>
</tr>
<tr>
<td>8.4 Private Sewage System Abandonment</td>
<td>12</td>
</tr>
<tr>
<td>8.5 Water-Well Capping</td>
<td>12</td>
</tr>
</tbody>
</table>
8.6 Overexcavation and Ground Preparation ............................................. 12
8.7 Fill Suitability .................................................................................. 13
8.8 Oversized Material ........................................................................ 13
8.9 Cut/Fill Transitions and Differential Fill Thicknesses ..................... 13
8.10 Benching ...................................................................................... 13
8.11 Fill Placement ............................................................................ 14
8.12 Inclement Weather ...................................................................... 14

9.0 SLOPE CONSTRUCTION ................................................................ 14
9.1 Slope Stability .............................................................................. 14
9.2 Temporary Excavations ............................................................... 14

10.0 POST-GRADING CONSIDERATIONS ............................................. 15
10.1 Control of Surface Water and Drainage Control ......................... 15
10.2 Utility Trenches .......................................................................... 15

11.0 PRELIMINARY FOUNDATION DESIGN RECOMMENDATIONS ........ 16
11.1 General ...................................................................................... 16
11.2 Allowable-Bearing Values .......................................................... 16
11.3 Settlement .................................................................................. 16
11.4 Lateral Resistance .................................................................... 16
11.5 Footing Setbacks from Descending Slopes .................................. 17
11.6 Building Clearances from Ascending Slopes ................................ 17
11.7 Footing Observations ................................................................ 17
11.8 Expansive Soil Considerations .................................................... 17
11.9 Footing/Floor Slabs - Very Low Expansion Potential .................. 17

12.0 RETAINING WALLS ..................................................................... 18
12.1 Lateral Earth Pressures and Retaining Wall Design Parameters .... 18
12.2 Footing Embedments .................................................................. 19
12.3 Drainage .................................................................................... 20
12.4 Temporary Excavations ............................................................... 21
12.5 Retaining Wall Backfill ............................................................... 21

13.0 MASONRY GARDEN WALLS ....................................................... 21
13.1 Construction on Level Ground .................................................... 21
13.2 Construction Joints .................................................................... 21

14.0 CONCRETE FLATWORK ............................................................... 21
14.1 Nonstructural Concrete Flatwork ................................................ 21
14.2 Joint Spacing ............................................................................ 22
14.3 Subgrade Preparation ................................................................ 22

15.0 PLANTERS .................................................................................. 23

16.0 SOIL CORROSIVITY ................................................................... 23
16.1 Corrosivity to Concrete and Metal ............................................. 23

17.0 PRELIMINARY PAVEMENT DESIGN ........................................... 24

18.0 ALTERNATIVE PRELIMINARY PAVEMENT DESIGNS ............. 24

19.0 PLAN REVIEWS AND CONSTRUCTION SERVICES ................... 25

20.0 LIMITATIONS .............................................................................. 26
LIST OF TABLES, APPENDICES AND ILLUSTRATIONS

Tables

Table 1 – Seismic Design Soil Parameters (Page 9)
Table 2 – Estimated Shrinkage/Bulking (Page 10)
Table 3 – Excavation Characteristics (Page 11)
Table 4 – Lateral Earth Pressures (Page 19)
Table 5 – Nonstructural Concrete Flatwork for Very Low Expansive Soils (Page 22)
Table 6 – Preliminary Pavement Design (Page 24)
Table 7 – Preliminary Concrete Pavement Design (Page 25)
Table 8 – Preliminary Interlocking Concrete Paver Pavement Design (Page 25)

Figures & Plates

Figure 1 – Site Location Map (Page 3)
Plate 1 – Geotechnical Map (Rear of Text)

Appendices

Appendix A – References and Aerial Photographs (Rear of Text)
Appendix B – Field Exploration and Boring Logs (Rear of Text)
Appendix C – Laboratory Testing Procedures and Test Results (Rear of Text)
Appendix D – General Earthwork and Grading Specifications (Rear of Text)
1.0 **INTRODUCTION**

This report presents the results of LGC Geo-Environmental, Inc.’s (LGC) geotechnical investigation report for the proposed Hampton Inn Hotel (Phase 1) project (APN 831-601-901), located at 721 South Indian Hill Boulevard in the City of Claremont, Los Angeles County, California. The purposes of this geotechnical investigation was to determine the nature of surface and subsurface soil and bedrock conditions, evaluate their characteristics, and provide geotechnical recommendations with respect to grading, construction, foundation design, and other relevant aspects to the proposed hospitality development. The referenced 20-scale preliminary grading plan, which was provided, was utilized as the base map for our Geotechnical Map (Plate 1) of the site.

Our scope of services included:

- Review of available previous geologic/geotechnical literature, geologic maps, and aerial photographs pertinent to the site (Appendix A).
- Geologic mapping of the site.
- Subsurface exploration consisting of the excavation, sampling, and logging of five (5) borings, to depths ranging from 5.5 feet to 20.0 feet utilizing a hollow-stem auger drill rig. Logs of the borings are presented in Appendix B, with the approximate location depicted on the Geotechnical Map (Plate 1). The borings were excavated to evaluate the general characteristics of the subsurface geologic/geotechnical conditions on the subject project site including classification of site soils and bedrock, determination of depth to groundwater (if present), and to obtain representative soil and bedrock samples.
- Laboratory testing of representative soil samples obtained during our current subsurface exploration (Appendix C).
- Geotechnical engineering and geologic analysis of the data with respect to the proposed hotel development.
- Preparation of General Earthwork and Grading Specifications (Appendix D).
- Preparation of this report presenting our findings, conclusions and preliminary geotechnical design recommendations for the proposed development.

1.1 **Proposed Construction and Grading**

The referenced 20-scale preliminary grading plan prepared by Kimley-Horn, indicates that the proposed development will consist of a single hotel structure with associated roadways, parking areas, a swimming pool, walk ways, and hardscape and landscape areas. It is anticipated that the structure will be up to four-stories, with wood/steel frame and masonry wall construction and some masonry block walls. For this type of construction, relatively moderate to heavy loads will likely be imposed on the underlying foundation soils and bedrock. The referenced preliminary grading plan also indicates that maximum cuts and fills, ranging to approximately 2 feet and 5 feet respectively, are proposed. Proposed maximum cut and fill slope heights are about 2 feet and 5 feet respectively, at slope ratios of 2:1 (h:v) or flatter. In addition, retaining walls are also proposed at the various locations, up to approximately 2.5 feet to 6.0 feet high. In addition retaining walls are not proposed at this time.

1.2 **Location and Site Description**

The subject site is roughly rectangular in shape, comprises about 3.5 acres, and is located at 721 South Indian Hill Boulevard in the city of Claremont, Los Angeles County, California.
The site is bounded on the north by residential development, on the south by the I-10 Highway, on the east by South Indian Hill Boulevard, and on the west by commercial and residential development. The general location and configuration of the site is shown on the Site Location Map (Figure 1).

1.3 **Topography and Drainage**

Elevations varied from approximately 1,036 feet above mean sea level (msl) within the southwestern portion of the site to approximately 1,043 feet above msl within the northern portion of the site. Drainage appears to be generally towards the south and southwest.

1.4 **Existing Improvements and Vegetation**

The subject site has been previously developed and includes an existing motel with associated paved parking areas, walkways, hardscape and landscape areas, and pool and recreation areas. All existing vegetation on the subject site is associated with the existing motel and recreation areas and is mainly comprised of local shrubs, trees, and grasses.

1.5 **Research of Previous Geological and Geotechnical Data**

This firm researched and reviewed available previous plans, as well as published and unpublished geologic data. Based on this firm's research, no previous geotechnical reports for the site are available. Pertinent information was incorporated into the conclusions and recommendations presented in our report.

1.6 **Aerial Photograph Analysis**

Paired stereo aerial photographs of the site and vicinity from 1949 through 1998 were reviewed and evaluated by this firm. The photographs were obtained from Continental Aerial Photo, Inc. Scales of the photographs reviewed (where available) were approximately 1" = 1,667' to 1" = 2,000'. A summary table of the photos reviewed is presented in Appendix A.

Our review of the aerial photographs indicates that the existing motel development appears to have existed since some time after mid-1965 and prior to early 1970. Prior to that time, the site appears to have been a citrus grove.

2.0 **FIELD INVESTIGATION**

2.1 **Geologic Mapping**

Surface geologic mapping of the site and accessible surrounding areas was accomplished by a geologist from this firm in August, 2015, utilizing the referenced 20-scale preliminary grading plan for plotting geologic units. This information has been plotted on the enclosed 20-scale Geotechnical Map (Plate 1).

2.2 **Field Exploration**

The subsurface exploration was performed on August 17, 2015, and involved the excavation of five (5) exploratory borings (Borings B-1 through B-5) to depths ranging from 5.5 feet to 20.0 feet utilizing a hollow-stem auger drill rig.
Prior to our subsurface work, an underground utilities clearance was obtained from Underground Services Alert of Southern California. At the conclusion of the subsurface exploration, all the borings were backfilled with on-site materials with some compactive effort. Minor settlement of the backfill soils may occur over time.

Earth materials encountered within the exploratory borings were classified and logged by a geologist from LGC in accordance with the visual-manual procedures of the Unified Soil Classification System. The approximate locations of the exploratory borings are shown on the Geotechnical Map (Plate 1) and descriptive logs are presented in Appendix B.

Associated with the subsurface exploration was the collection of bulk and relatively undisturbed samples of soil for laboratory testing. Bulk samples consisted of selected soil and bedrock materials obtained at various depth intervals from the exploratory borings. Undisturbed samples were obtained using a 3-inch outside diameter, modified California split-spoon soil sampler lined with brass rings. The soil sampler was driven with successive 30-inch drops of a mechanically driven, 140-pound automatic-trip hammer on the hollow-stem auger drill rig. The central portions of the driven samples were placed in sealed containers and transported to our laboratory for testing. The number of blows required to drive the split-spoon sampler 18 inches for the hollow-stem auger drill rig was recorded in 6-inch increments.

Standard Penetration Tests were also performed in accordance with the American Society for Testing Materials Standard Procedure (ASTM) D1586. This method consisted of driving an unlined standard split-barrel sampler 18 inches into the soil with successive 30-inch drops of the 140-pound automatic trip hammer. Blow counts were recorded for each 6-inch driving increment; however, the number of blows required to drive the split-spoon sampler and the standard split-barrel sampler for the last 12 of the 18 inches was identified as the standard penetration resistance of N-count and recorded in the boring logs. Disturbed soil samples from the unlined standard split-barrel sampler were placed in plastic bags and transported to our laboratory for testing.

2.3 **Laboratory Testing**

During our subsurface exploration, relatively undisturbed and bulk samples were retained for laboratory testing. Laboratory testing was performed on selected representative samples of onsite soil materials and included in-situ dry density and moisture content, maximum dry density and optimum moisture content, expansion index, sulfate content, chloride content, pH, resistivity, grain size analysis, direct shear, R-Value and consolidation tests. A brief description of the laboratory test criteria and test data are presented in Appendix C. In-situ moisture content and dry density are included in the exploration boring logs (Appendix B).

3.0 **FINDINGS**

3.1 **Regional Geologic Setting**

The subject site is located in the Peninsular Ranges Geomorphic Province in California. The site is situated in the northern portion of the Perris Block, which is a relatively stable structural block that is located between the San Jacinto Fault Zone to the east, the Elsinore Fault Zone to the west and the Cucamonga Fault Zone to the north. In general, the Perris Block consists of a Quaternary to Pleistocene-aged apron that overlies Paleozoic metamorphic bedrock and Cretaceous granitic bedrock. The apron is composed of Quaternary alluvium and alluvial fan
deposits that extend to a depth of roughly 850 feet to 900 feet. A map of the regional geology is presented on the Regional Geologic Map, Figure 2.

The proposed hotel site is located approximately 3.0 miles east of the San Jose Hills, approximately 3.4 miles south the San Gabriel Mountains and approximately 5.0 miles northeast of the Chino Hills.

3.2 **Local Geology and Soil Conditions**

Based on our review of available geological and geotechnical literature, current field mapping, and exploratory borings conducted at the site, it is our understanding that the site is primarily underlain by artificial fills (undocumented), topsoil, alluvial fan deposits, and older alluvium. Each unit is described in greater detail below and presented within the exploratory borings logs (Appendix B). The approximate locations of the observed geologic units are depicted on the Geotechnical Map (Plate 1).

- **Artificial Fill (map symbol Afu)** - Undocumented surficial artificial fill, associated with previous grading, was observed in the central and northeastern portions of the site, particularly in Borings B-4 and B-5. The fill material, based on our mapping, exploration, and observations, was comprised of sand, silty sand, and sandy silt, which was various shades of brown, orange, yellow, and gray, fine to coarse-grained, dry to damp, and loose to dense/stiff. The fills are estimated to be up to approximately 4 feet to 7 feet in thickness within the areas explored. Based on our observations and testing, the artificial fill is considered non-engineered and unsuitable for support of additional fill, structures, walls and improvements.

- **Topsoil (no map symbol)** – Topsoil existed in Borings B-1 and B-3. Based on exploration, mapping and topography, the topsoil is estimated to be about 1.0 foot to 3.5 feet thick and consists generally of silty sand which was dark brown, fine to coarse-grained, dry to damp and loose to medium dense, with some cobbles up to 5 inches in diameter. Topsoil is not present in some areas apparently due to previous grading.

- **Alluvial Fan Deposits (map symbol Of)** – Quaternary-aged alluvial fan deposits were observed within all portions of the site below the artificial fill and topsoil and consisted generally of sandy gravel, gravelly sand and, silty sand. Based on exploration and geologic mapping, these materials were observed to be generally and up to about 12.0 feet to 13.0 feet thick to the depths explored. These materials were various shades of brown, gray, yellow and orange, generally fine to very coarse-grained, dry to damp, medium dense to very dense, friable, and micaceous, and with some cobbles up to 6 inches in diameter.

- **Older Alluvium (Qoal)** – Pleistocene age older alluvium was present below the alluvial fan deposits, at depths of about 10 feet, within the areas explored. These materials consisted of silty sand and sandy silt, which are generally fine to coarse grained, various shades of orange, olive and brown, damp and medium dense to very dense and stiff to very stiff, with occasional gravel and a trace of pinhole pores.

3.3 **Landslides**

Review of geologic literature does not indicate the presence of landslides on or directly adjacent to the site.
3.4 **Groundwater**

Groundwater was not encountered during the current subsurface exploration up to 20 feet below the existing surface. A review of the Chino Basin Watermaster “Depth to Groundwater Contour Map, Fall 2006” indicates groundwater in the general site area varies from about 400 feet or more below the existing ground surface.

3.5 **Caving**

Caving was not encountered in the exploratory boring. However, caving may occur within excavations made into sandier and friable portions of the alluvial fan deposits.

3.6 **Surface Water**

Surface water runoff relative to project design is the purview of the project civil engineer and should be designed to be directed away from all structures and walls.

3.7 **Faulting**

The geologic structure of the Southern California area is dominated mainly by northwest-trending faults associated with the San Andreas system. Faults, such as the Whittier, Elsinore, San Jacinto and San Andreas, are major faults in this system and are known to be active and may produce moderate to strong ground shaking during an earthquake. In addition, the San Andreas, Elsinore and San Jacinto faults are known to have ruptured the ground surface in historic times.

Based on our review of published and unpublished geologic/geotechnical maps and literature pertaining to the site and regional geology, the closest active faults are the San Jose Fault located approximately 2.1 miles from the site and the Elsinore-Chino Central Avenue Fault, located approximately 4.8 miles from the site. Other active faults, within about 20 miles of the subject site, are the Sierra Madre Fault, approximately 5.0 miles; the Cucamonga Fault, approximately 5.2 miles; the Whittier Fault, approximately 12.9 miles; the Elysian Park Thrust Fault, approximately 14.5 miles; the Clamshell-Sawpit Fault, approximately 15.0 miles; the Elsinore-Glen Ivy Fault, approximately 16.6 miles; the San Jacinto-San Bernardino Fault, approximately 16.7 miles; the Raymond Fault, approximately 17.8 miles and the San Andreas-Southern Fault approximately 19.1 miles.

No faults (active, potentially active, or inactive) are known to project through the site. The site does not lie within an Alquist-Priolo Earthquake Fault Hazard Zone as defined by the State of California in the Alquist-Priolo Earthquake Fault Hazard Zoning Act or a Los Angeles County Fault Zone. The possibility of damage due to ground rupture is considered negligible since active faults are not known to cross the site.

3.8 **Seismicity**

Secondary effects of seismic shaking resulting from large earthquakes on the major faults in the southern California region, which may affect the site, include soil liquefaction and dynamic settlement. Liquefaction is a seismic phenomenon in which loose, saturated, granular soils behave similarly to a fluid when subject to high-intensity ground shaking. Liquefaction occurs when three general conditions exist: 1) shallow groundwater; 2) low density non-cohesive
(granular) soils; and 3) high-intensity ground motion. Studies indicate that saturated, loose to medium dense, near surface cohesionless soils exhibit the highest liquefaction potential, while dry, dense, cohesionless soils and cohesive soils exhibit low to negligible liquefaction potential.

Therefore the potential for liquefaction is considered nil based on laboratory testing, field exploration, the absence of groundwater to the depths explored, and nearby groundwater levels in excess of 50 feet below the ground surface.

Other secondary seismic effects include shallow ground rupture, seiches, and tsunamis. In general, these secondary effects of seismic shaking are a possibility throughout the Southern California region and are dependent on the distance between the site and causative fault and the onsite geology. A risk assessment of these secondary effects is provided in the following sections.

3.9 Settlement Analysis

The results of our subsurface exploration and laboratory testing indicate the site is underlain by approximately 2 feet to 7 feet of potentially compressible and/or hydro-collapsible soils, consisting of non-engineered artificial fill and topsoil. These materials exhibit the potential to settle or hydro-consolidate under the surcharge of proposed fill loads up to approximately 5 feet in depth and anticipated future structural loads. A portion of the total settlement is due to the potential hydro-consolidation.

In areas where overexcavation of non-engineered artificial fill to competent underlying alluvial fan deposits is accomplished, total settlement of about 0.50-inch, and a differential settlement of about 0.25-inch over a distance of about 40 feet could be anticipated.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 General

Based on the results of our current geotechnical investigation, it is our opinion that the proposed hotel development, as indicated on the referenced 20-scale preliminary grading plan, is feasible from a geotechnical and geologic standpoint, provided the following recommendations are incorporated into the design criteria and project specifications. When actual grading plans for the site and foundation/structural plans for the proposed development are available, a comprehensive plan review should be performed by this firm. Depending on the results, additional recommendations may be necessary for geotechnical design parameters for both earthwork and foundations. Grading should be conducted in accordance with local codes, the recommendations within this report, and future plan reviews. It is also our opinion that the proposed construction and grading will not adversely impact the geologic stability of adjoining properties.

The following is a summary of the primary geotechnical factors determined from our geotechnical investigation.
Based on our current subsurface exploration and review of pertinent geological maps and reports, the site is underlain by undocumented artificial fills including topsoil and alluvial fan deposits.

- There are no known landslides impacting the site.
- Groundwater is not considered a constraint for the proposed development.
- The potential for liquefaction is considered remote.
- Active or potentially active faults are not known to exist on the site.
- Laboratory test results of the upper soils (artificial fill, topsoil, and alluvial fan deposits) indicate a very low to low expansion potential and negligible potential for soluble sulfate effects on normal concrete and chloride effects on reinforcing steel.
- The majority of the site is underlain by approximately 2 feet to 7 feet of potentially compressible undocumented artificial fill and topsoil which may be prone to potential intolerable post-grading settlement and/or hydroconsolidation, under the surcharge of the future proposed structural loads and/or fill loads. These materials should be overexcavated to underlying competent alluvial fan deposits.
- From a geotechnical perspective, the existing onsite soils appear to be suitable material for use as fill, provided they are relatively free from rocks (larger than 6 inches in maximum dimension), construction debris, and organic material. It is anticipated that the onsite soils may be excavated with conventional heavy-duty construction equipment.

5.0 GEOLOGIC CONSIDERATIONS

5.1 Slopes
No natural slopes or existing cut/fill slopes with adverse conditions are anticipated.

5.2 Faulting
Geologic hazards due to fault rupture are not known to be present at the subject site.

5.3 Groundwater
Adverse effects on the proposed development resulting from groundwater are not anticipated.

5.4 Subsidence
In consideration of the anticipated grading, recommended overexcavations, proposed structures and improvements, and subsurface material types and their conditions, unfavorable ground subsidence is not anticipated.

5.5 Landsliding
Landslides or surface failures were not observed at or directly adjacent to the site. As a result, the possibility of the site being affected by landsliding is not anticipated.
5.6 *Ground Rupture*

Ground rupture due to active faulting is not likely to occur on site due to the absence of known active fault traces. Cracking due to shaking from distant seismic events is not considered a significant hazard, although it is a possibility at any site.

5.7 *Tsunamis and Seiches*

Based on the elevation of the proposed development at the site with respect to sea level and its distance from large open bodies of water, the potential of seiche and/or tsunami is considered to be negligible.

6.0 **SEISMIC-DESIGN CONSIDERATIONS**

6.1 *Ground Motions*

The site will probably experience ground shaking from moderate to large size earthquakes during the life of the proposed development. Furthermore, it should be recognized that the Southern California region is an area of high seismic risk, and that it is not considered feasible to make structures totally resistant to seismic-related hazards.

Structures within the site should be designed and constructed to resist the effects of seismic ground motions as provided in the 2013 CBC, Section 1613. The method of design is dependent on the seismic zoning, site characterizations, occupancy category, building configuration, type of structural system, and building height.

The following seismic design parameters, presented in Table 1, were developed based on the CBC 2013 and should be used for the proposed structures. A site coordinate of 34.0825° N, 117.7208° W was used to derive the seismic parameters presented below:

**TABLE 1**

<table>
<thead>
<tr>
<th>Seismic Design Soil Parameters (2013 CBC Section 1613)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site Class Definition ASCE 7; Chapter 20 (Table 20.3-1)</td>
</tr>
<tr>
<td>Mapped Spectral Response Acceleration Parameter Sₐ (for 0.2 second) (Figure 1613.5.3.(1))</td>
</tr>
<tr>
<td>Mapped Spectral Response Acceleration Parameter, S₁ (for 1.0 second) (Figure 1613.5.3.(2))</td>
</tr>
<tr>
<td>Site Coefficient Fₐ (short period) [Table 1613.3.3.(1)]</td>
</tr>
<tr>
<td>Site Coefficient Fᵥ (1-second period) [Table 1613.3.3.(2)]</td>
</tr>
<tr>
<td>Adjusted Maximum Considered Earthquake (MCE) Spectral Response Acceleration Parameter Sₘₛ (short period) (Eq. 16-37)</td>
</tr>
<tr>
<td>Adjusted Maximum Considered Earthquake (MCE) Spectral Response Acceleration Parameter Sᵥ₁ (1-second period) (Eq. 16-38)</td>
</tr>
<tr>
<td>Design Spectral Response Acceleration Parameter, Sₘₛ (short period) (Eq. 16-39)</td>
</tr>
<tr>
<td>Design Spectral Response Acceleration Parameter, Sᵥ₁ (1-second period) (Eq. 16-40)</td>
</tr>
</tbody>
</table>
6.2 **Secondary Seismic Hazards**

Secondary effects of seismic activity normally considered as possible hazards to a site include several types of ground failure, as well as induced flooding. Various general types of ground failures which might occur as a consequence of severe ground shaking of the site include liquefaction, landsliding, ground subsidence, ground lurching, and shallow ground rupture. 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 proposed grading and recommended overexcavation of potentially compressible materials within areas of proposed development, the secondary effects of liquefaction and other seismic activity noted above are considered unlikely at the site.

Seismically induced flooding, which might be considered a potential hazard to a site, normally includes flooding due to a tsunami (seismic sea wave), a 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. Since the site is located several miles inland from the nearest coastline of the Pacific Ocean at an elevation in excess of approximately 1,036 feet above msl, the potential for seismically induced flooding due to tsunami inundation is considered nonexistent. Since no enclosed bodies of water lie adjacent to the site, the potential for induced flooding at the site due to a seiche is also considered nonexistent.

7.0 **GEOTECHNICAL DESIGN PARAMETERS**

7.1 **Shrinkage/Bulking and Subsidence**

Volumetric changes in earth quantities will occur when excavated onsite soils are replaced as properly compacted fill. The following table, Table 2, is an estimate of the shrinkage and bulking factors for the various geologic units present onsite. These estimates are based on in-place densities of the various materials and on the estimated average degree of relative compaction that will be achieved during grading.

### Table 2

<table>
<thead>
<tr>
<th>GEOLOGIC UNIT</th>
<th>SHRINKAGE PERCENT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Artificial Fill, Undocumented (Afu)</td>
<td>5 to 10</td>
</tr>
<tr>
<td>Topsoil (No map symbol)</td>
<td>10 to 15</td>
</tr>
<tr>
<td>Alluvial Fan Deposits (Qf)</td>
<td>0 to 3</td>
</tr>
<tr>
<td>Older Alluvium (Qoa)</td>
<td>0 to 5</td>
</tr>
</tbody>
</table>

Subsidence of the alluvial fan deposits, because of recompression of exposed soils prior to fill placement, and placement of proposed fills, is estimated to be about 0.05 to 0.10 feet.

The above estimates of shrinkage are intended as an aid for project engineers in determining earthwork quantities. However, these estimates should be used with some caution since they are not absolute values. These are preliminary rough estimates which may vary with...
depth of removal, stripping losses, field conditions at the time of grading, etc. Handling losses, and reduction in volume due to removal of oversized material, are not included in the estimates.

7.2 Excavation Characteristics

The following excavation characteristics of the various material types at the site have been developed based on LGC’s geologic mapping and experience with these materials in the area and are presented in Table 3 below:

<table>
<thead>
<tr>
<th>GEOLOGIC UNIT</th>
<th>Easy* Ripping</th>
<th>Moderately** Difficult Ripping</th>
<th>Oversized Material (&gt;6 inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Artificial Fill, Undocumented (Af)</td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Topsoil</td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Alluvial Fan Deposits (Qf)</td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Older Alluvium (Qoal)</td>
<td>X</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*D-8 with double rippers  
**D-9 with single ripper

7.3 Compressible/Collapsible Soils

The results of our laboratory testing indicate that the existing, undocumented artificial fill, topsoil, and bedrock are susceptible to varying degrees of intolerable settlement and/or hydroconsolidation (collapse) when a load is applied or the soil is saturated. Consequently, these materials should be collectively overexcavated to underlying competent alluvial fan deposits (Qf) or older alluvium (Qoal) and replaced as engineered compacted fill.

8.0 SITE EARTHWORK

8.1 General Earthwork and Grading Specifications

Earthwork and grading should be performed in accordance with applicable requirements of the grading code of the City of Claremont and in accordance with the following recommendations prepared by this firm. Grading should also be performed in accordance with the applicable provisions of the attached “Standard Grading Specifications” prepared by LGC (Appendix D), unless specifically revised or amended herein. In case of conflict, the following recommendations shall supersede those included in as part of LGC’s General Earthwork and Grading Specifications (Appendix D).

8.2 Geotechnical Observations and Testing

Prior to the start of grading, a meeting should be held at the site with the owner, developer, grading contractor, civil engineer and geotechnical consultant to discuss the work schedule and geotechnical aspects of the grading. Rough grading, which includes clearing, overexcavation, scarification/processing and fill placement, should be accomplished under the full-time
observation and testing of the geotechnical consultant. Fills should not be placed without prior approval from the geotechnical consultant.

A representative of the project geotechnical consultant should also be present onsite during grading operations to document proper placement and compaction offills, as well as to document excavations and compliance with the other recommendations presented herein.

8.3 Clearing and Grubbing

Weeds, grasses, and trees in areas to be graded should be stripped and hauled offsite. Trees to be removed should be grubbed so that their stumps and major-root systems are also removed and the organic materials hauled offsite. During site grading, roots, tree branches and other deleterious materials missed during clearing and grubbing operations should be removed from fill prior to placement.

The project geotechnical consultant or his qualified representative should be notified at the appropriate times to provide observation and testing services during clearing and grubbing operations to observe and document compliance with the above recommendations. In addition, buried structures, unusual or adverse soil conditions encountered that are not described or anticipated herein should be brought to the immediate attention of the geotechnical consultant.

8.4 Private Sewage System Abandonment

Private sewage systems and/or other subsurface structures that may be encountered should be located, removed and/or properly abandoned. Abandonment and/or removal of septic systems that may exist should be in accordance with local codes. Seepage pits, if abandoned in-place, should be pumped clean, backfilled with gravel or clean sand jetted into place, and then capped with 2 feet or more of at least a 2-sack slurry for a minimum distance of 2 feet outside the edge of the seepage pit. The top of the slurry cap should be at least 10 feet below proposed grade.

8.5 Water-Well Capping

Unknown water wells that are encountered within the site, which are to be abandoned, should be abandoned and capped under permit by the appropriate governmental agency from Los Angeles County. In addition, a minimum 10-foot thick compacted fill blanket, below proposed grade, should be placed above the previously or newly-capped water wells.

8.6 Overexcavation and Ground Preparation

The site is underlain by approximately 2 feet to 7 feet of potentially compressible soils. Therefore, existing, undocumented artificial fill and topsoil are considered unsuitable for support of proposed fills, structures, and/or improvements, and should be overexcavated to expose underlying competent alluvial fan deposits or older alluvium. Where overexcavation and grading do not provide 5 feet or more of fill below finished grade within areas for proposed structures or walls, the area should be overexcavated to 5 feet or more below proposed grade or 3 feet or more below bottoms of footings for structures and 2 feet or more below bottoms of footings for walls, whichever is deeper. Actual depths of overexcavation should be evaluated upon review of final grading and foundation plans as well as during grading on the basis of observations and testing during grading by the project geotechnical consultant.
Prior to placing engineered fill, exposed bottom surfaces in each overexcavated area should first be scarified to a depth of approximately 6 inches, watered or air-dried as necessary to achieve a uniform moisture content of optimum or higher and then compacted in place to a relative compaction of 90 percent or more (based on American Standard of Testing and Materials [ASTM] Test Method D1557).

The estimated locations, extent, and approximate depths for overexcavation of unsuitable materials are indicated on the enclosed Geotechnical Map (Plate 1). The geotechnical consultant should be provided with appropriate survey staking during grading to document that depths and/or locations of recommended overexcavation are adequate.

Sidewalls for overexcavations greater than 5 feet in height should be no steeper than 1:1 horizontal to vertical (h:v) and should be periodically slope-boarded during their excavation to remove loose surficial debris and facilitate mapping. Flatter excavations may be necessary for stability.

The grading contractor will need to consider appropriate measures necessary to excavate existing improvements adjacent to the site without endangering them due to caving or sloughing.

8.7 **Fill Suitability**

Soil materials excavated during grading are generally considered suitable for use as compacted fill provided that they do not contain significant amounts of trash, vegetation, organic material, construction debris, and oversize material.

8.8 **Oversized Material**

Oversized material that may be encountered during grading, greater than 6 inches, should be reduced in size or removed from the site

8.9 **Cut/Fill Transitions and Differential Fill Thicknesses**

To mitigate distress to structures and walls related to the detrimental effect of differential settlement, the cut portions should be eliminated from cut/fill transition areas in order that the entire structure or wall be founded on a uniform bearing material. This should be accomplished by overexcavating the “cut” portions and shallow fill portions 5 feet or more below proposed pad grade or 3 feet below proposed footings for structures and 2 feet or more below bottoms of footings for walls, whichever is deeper and replacing the excavated materials as properly compacted fill. Recommended depths of overexcavation are provided in the following table:

<table>
<thead>
<tr>
<th>DEPTH OF FILL (&quot;fill&quot; portion)</th>
<th>DEPTH OF OVEREXCAVATION (&quot;cut&quot; portion)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Up to 15 feet</td>
<td>5 feet</td>
</tr>
<tr>
<td>Greater than 15 feet</td>
<td>One-third the maximum thickness of fill placed on the &quot;fill&quot; portion (10 feet maximum)</td>
</tr>
</tbody>
</table>

8.10 **Benching**

Where compacted fills are to be placed on natural slope surfaces inclining at 5:1 (h:v) or greater, the ground should be excavated to create a series of level benches, which are at least a...
minimum height of 4 feet, excavated into competent bedrock or existing compacted engineered materials. Typical benching details are described in the attached LGC “Standard Grading Specifications” (Appendix D).

8.11 Fill Placement

Fills should be placed in lifts no greater than 6 inches in uncompacted thickness, watered or air-dried as necessary to achieve a uniform moisture content of at least optimum moisture content, and then compacted in place to relative compaction of 90 percent or more. Fills should be maintained in a relatively level condition. The laboratory maximum dry density and optimum moisture content for each change in soil type should be determined in accordance with ASTM Test Method D1557.

8.12 Inclement Weather

Inclement weather may cause rapid erosion during mass grading and/or construction. Proper erosion and drainage control measures should be taken during periods of inclement weather in accordance with City of Claremont, Los Angeles County and California State requirements.

9.0 SLOPE CONSTRUCTION

9.1 Slope Stability

Proposed cut and fill slopes which are indicated on the referenced 20-scale preliminary grading plan, constructed at a 2:1 horizontal to vertical (h:v) or flatter should be grossly stable.

Portions of any proposed cut slopes may expose low-density, undocumented artificial fill or topsoil as well as significant layers of relatively non-cohesive alluvial fan deposits which will likely require stabilization by overexcavation and replacement with compacted fill. During detailed grading plan review stages a detailed slope stability analyses may be warranted.

9.2 Temporary Excavations

Temporary excavations varying up to a height of approximately 5 feet to 6 feet below existing grades will be necessary to accommodate the recommended overexcavation of the unsuitable soil or bedrock materials. Based on the physical properties of the onsite soils, temporary excavations exceeding 5 feet in height should be cut back at a ratio of 1:1 (h:v) or flatter, for the duration of the overexcavation and recomposition of unsuitable soil material. Temporary slopes excavated at the above slope configurations are expected to remain stable during grading operations. However, the temporary excavations should be observed by a representative of the project geotechnical consultant for any evidence of potential instability. Depending on the results of these observations, revised slope configurations may be necessary.

Other factors which should be considered with respect to the stability of the temporary slopes include construction traffic and storage of materials on or near the tops of the slopes, construction scheduling, presence of nearby walls or structures on adjacent properties, and weather conditions at the time of construction. 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 also be followed.
10.0 POST-GRADING CONSIDERATIONS

10.1 Control of Surface Water and Drainage Control

Positive-drainage devices such as sloping sidewalks, graded-swales, and/or area drains, should be provided to collect and direct water away from the structure and slopes. Neither rain nor excess irrigation water should be allowed to collect or pond against building foundations. Drainage should be directed to adjacent driveways, adjacent streets or storm-drain facilities and maintained at all times. Since the site is in a semi-arid climate area, from a geotechnical standpoint, the ground surface adjacent to the structures should be sloped at a gradient of at least 2 percent for a distance of at least 10 feet. Each graded lot should be further maintained by a swale or drainage path at a gradient of at least 1 percent. Where necessary, drainage paths may be shortened by use of area drains and collector pipes.

Planters with open bottoms adjacent to buildings should be avoided. Planters should not be designed adjacent to buildings unless provisions for drainage are made, such as catch basins, liners, and/or area drains. Over watering must be avoided.

10.2 Utility Trenches

Utility-trench backfill within roadways, utility easements, under walls, sidewalks, driveways, floor slabs and any other structures or improvements should be compacted. The onsite soils should generally be suitable as trench backfill provided they are screened of rocks and other material over 3 inches in diameter and organic matter. Trench backfill should be compacted in uniform lifts (generally not exceeding 6 inches to 8 inches in uncompacted thickness) by mechanical means to at least 90 percent relative density (per ASTM Test Method D1557).

Where onsite soils are utilized as backfill, mechanical compaction should be used. Density testing, along with probing, should be performed by the project geotechnical consultant or his representative, to document proper compaction.

If trenches are shallow, the use of conventional equipment may result in damage to the utilities. Clean sand, having a sand equivalent (SE) of 30 or greater should be used to bed and shade the utilities. Sand backfill should be densified. The densification may be accomplished by jetting or flooding and then tamping to ensure adequate compaction. A representative from LGC should observe, probe, and test the backfill to verify compliance with the project specifications.

Utility-trench sidewalls deeper than 5 feet should be laid back at a ratio of 1:1 (h:v) or flatter or braced. A trench box may be used in lieu of shoring. If shoring is anticipated, LGC should be contacted to provide design parameters.

To avoid point-loads and subsequent distress to clay, cement or plastic pipe, imported sand bedding should be placed 1-foot or more above pipe in areas where excavated trench materials contain significant cobbles. Sand-bedding materials should be compacted and tested prior to placement of backfill.

Where utility trenches are proposed parallel to building footings (interior and/or exterior trenches), the bottom of the trench should not be located within a 1:1 (h:v) plane projected downward from the outside bottom edge of the adjacent footing.
11.0 PRELIMINARY FOUNDATION DESIGN RECOMMENDATIONS

11.1 General

Provided that site grading is performed in accordance with the recommendations of this report, conventional shallow foundations are considered feasible for support of the proposed hotel structures. Tentative foundation recommendations are provided herein. However, these recommendations may require modification depending on as-graded conditions existing within the building sites upon completion of grading.

11.2 Allowable-Bearing Values

An allowable-bearing value of 2,500 pounds per square foot (psf) may be used for 12-inch wide or greater continuous footings or 24-inch square pad footings, founded completely within in competent compacted fill at a depth of 12-inches or more below the lowest adjacent final grade. This value may be increased by 20 percent for each additional foot of width and depth, to a value no greater than 3,500 psf. The recommended allowable-bearing value includes both dead and live loads and may be increased by one-third for short-duration wind and seismic forces. The bearing capacities should be re-evaluated when loads and footing sizes have been finalized.

11.3 Settlement

Based on the general settlement characteristics of compacted fill, the previous overexcavation recommendations in this report and anticipated loading, it is estimated the site would be subjected to a total settlement about 0.50-inch, and a differential settlement of about 0.25-inch over a distance of about 40 feet. It is anticipated that the majority of the settlement will occur during construction or shortly thereafter as building loads are applied.

The above settlement estimates are based on the assumption that the proposed precise grading will be performed in accordance with the grading recommendations presented in this report and that the project geotechnical consultant will observe and/or test the soil conditions in the footing excavations.

11.4 Lateral Resistance

Lateral forces on footings should be resisted by passive earth resistance and friction at the bottom of the footing. Foundations should be designed for a passive earth pressure of 280 psf per foot of depth to a maximum value of 2,800 psf and a coefficient of friction of 0.35. The passive earth pressure incorporates a minimum factor of safety of 1.5. When combining passive and friction forces, passive resistance should be reduced by 1/3. The above values may be increased by 1/3 when designing for short-duration wind or seismic forces.

The above values are based on footings placed directly against compacted fill or undisturbed native soil. In the case where footing sides are formed, backfill placed against the footings should be compacted to 90 percent or more of maximum dry density as determined by ASTM D1557.
11.5 **Footing Setbacks from Descending Slopes**

Where structures are proposed near the tops of descending graded or natural slopes, the footing setbacks from the slope face should conform to the 2013 CBC, Figure 1808.7.1. The required setback is H/3 (one-third the slope height) measured along a horizontal line projected from the lower outside face of the footing to the slope face. The footing setbacks should be 5 feet or more where the slope height is 15 feet or less and vary up to 40 feet where the slope height exceeds 15 feet.

11.6 **Building Clearances from Ascending Slopes**

Building setbacks from ascending graded or natural slopes should conform with the 2013 CBC, Figure 1808.7.1, which requires a building clearance of H/2 (one-half the slope height) varying from 5 to 15 feet. The building clearance is measured along a horizontal line projected from the toe of the slope to the face of the building. A retaining wall may be constructed at the base of the slope to achieve the required building clearance.

11.7 **Footing Observations**

Footing excavations should be observed by the project geotechnical consultant to document that they have been excavated into competent bearing soils. The foundation excavations should be observed prior to the placement of forms, reinforcement or concrete. The excavations should be trimmed neat, level and square. Loose, sloughed or moisture-softened soil should be removed prior to concrete placement.

Excavated materials from footing excavations should not be placed in slab-on-ground areas unless the soils are compacted to 90 percent or more of maximum dry density as determined by ASTM D1557.

11.8 **Expansive Soil Considerations**

Results of preliminary laboratory tests by LGC indicate onsite soil materials exhibit expansion potentials of **VERY LOW** in accordance with 2013 CBC, Chapter 18. Given that generally the expansion index of the onsite soils is **VERY LOW**, recommendations to mitigate the effects of expansive soils may not be required. However, expansive soil conditions of the near surface finish grade soils should be evaluated and tested for individual building pads on a pad-by-pad basis during and at the completion of rough grading to verify and/or modify the anticipated conditions. The design and construction details presented herein are intended to provide recommendations for the levels of expansion potential which may be evident at the completion of rough grading. Furthermore, it should be noted that additional slab thickness, footing sizes and/or reinforcement more stringent than the recommendations that follow should be provided as recommended by the project structural engineer.

11.9 **Footing/Floor Slabs - Very Low Expansion Potential**

The following are our recommendations where foundation soils exhibit **VERY LOW** expansion potential as classified in accordance with 2013 CBC. For this condition, it is recommended that footings and floors be constructed and reinforced in accordance with the following criteria. However, additional slab thickness, footing sizes and/or reinforcement may be required by the project architect or structural engineer.
• Footings
  – Exterior continuous footings should be founded into compacted engineered fill below the lowest adjacent final grade at minimum depths of 18 inches deep for one-story to two-story construction and 24 inches deep for three-story and four-story construction. Interior continuous footings may be founded at a depth of 12 inches or greater for one-story and two-story structures and 18-inches or greater for three-story and four-story structures into compacted engineered fill below the lowest adjacent final grade. Continuous footings should have a minimum width of 15 inches or more for one-story and two-story structures and 18-inches or more for three-story and four-story structures.
  – Continuous footings should be reinforced with a minimum of two (2) No. 4 bars, one near the top and one near the bottom.
  – Interior isolated pad footings should be 24 inches or more square and founded at a depth of 12 inches or more for one-story and two-story structures and 18-inches or more for three-story and four-story structures, below the lowest adjacent grade. Footings should be reinforced in accordance with the structural engineer’s recommendation.
  – Exterior pad footings should be 24 inches or more square and founded at a depth of 18 inches or more below the lowest adjacent grade. Footings should be reinforced in accordance with the structural engineer’s recommendations.

• Floor Slabs
  – Concrete floor slabs should be 5 inches or more thick and reinforced with No. 3 bars spaced 24 inches or less on-centers, both ways. Slab reinforcement should be supported on concrete chairs or bricks so that the desired placement is near mid-depth.
  – Concrete floors should be underlain with a moisture-vapor retarder consisting of 15-mil thick vapor barrier. Laps within the membrane should be sealed and overlapped 12 inches. Two inches or more of clean sand should be placed above and below the membrane to promote uniform curing of the concrete.
  – Prior to placing concrete, subgrade soils should be thoroughly moistened to approximately 100% of optimum moisture content to promote uniform curing of the concrete and reduce the development of shrinkage cracks. The moisture content should penetrate to a minimum depth of 12 inches.

12.0 RETAINING WALLS

12.1 Lateral Earth Pressures and Retaining Wall Design Parameters

Conventional foundations for retaining walls within properly compacted fill within competent bedrock should be embedded at least 18 inches below lowest adjacent grade. At this depth, an allowable bearing capacity of 2,500 psf may be assumed for retaining walls founded in competent compacted fill.

The following are lateral earth pressures are recommended for retaining walls up to 10 feet high that may be proposed. The recommended lateral pressures for approved on-site or import soils (with an expansion index of 20 or less and an angle of internal friction (phi) of at
least 32 degrees) for level or sloping backfill are presented in Table 4. Onsite soil should be screened of rocks and other material over 3 inches in diameter.

The following lateral earth pressures are recommended for retaining walls that may be proposed up to 6 feet high and for dynamic conditions for retaining walls with heights ranging from 6 feet up to 10 feet. The recommended lateral pressures for approved on-site soils (with an expansion index of 20 or less) for level or sloping backfill are presented in Table 4. Onsite fill soil should be screened of rocks and other material over 3 inches in diameter.

**TABLE 4**

*Lateral Earth Pressures*

<table>
<thead>
<tr>
<th>CONDITIONS</th>
<th>Level Backfill (up to 6 feet)</th>
<th>Level Backfill Dynamic (&gt;6 feet to 10 feet)</th>
<th>2:1 Backfill Ascending (up to 6 feet)</th>
<th>2:1 Backfill Ascending-Dynamic (&gt;6 feet to 10 feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Active</td>
<td>45</td>
<td>75</td>
<td>65</td>
<td>95</td>
</tr>
<tr>
<td>At-Rest</td>
<td>65</td>
<td>95</td>
<td>90</td>
<td>125</td>
</tr>
<tr>
<td>Passive</td>
<td>280</td>
<td>280</td>
<td>150</td>
<td>150</td>
</tr>
</tbody>
</table>

For sliding resistance, the friction coefficient of 0.40 may be used at the concrete and soil interface. Wall footings should be designed in accordance with structural considerations. The passive resistance value may be increased by one-third when considering loads of short duration such as wind or seismic loads.

Embedded structural walls should be designed for lateral earth pressures exerted on them. Restrained structural walls should be designed for at rest conditions. The magnitude of those pressures depends on the amount of deformation that the wall can yield under load. If the wall can yield enough to mobilize the full shear strength of the soil, it can be designed for “active” pressure. If the wall cannot yield under the applied load, the shear strength of the retained soil cannot be mobilized and the earth pressure will be higher. Such walls should be designed for “at-rest” conditions. If a structure moves toward the soils, the resulting resistance developed by the soil is the “passive” resistance.

The equivalent fluid pressure values assume free-draining conditions and a soil expansion index of 20 or less. If conditions other than those assumed above are anticipated, revised equivalent fluid pressure values should be provided on an individual-case basis by the geotechnical engineer.

Surcharge loading effects from the adjacent structures should be evaluated by the geotechnical and structural engineers.

**12.2 Footing Embedments**

The base of retaining wall footings constructed on level ground may be founded at a depth of 18 inches or more below the lowest adjacent final grade. Where retaining walls are proposed on or within 15 feet from the top of an adjacent descending fill slopes, the footings should be deepened such that a horizontal clearance of H/3 or more (one-third the slope height) is
maintained between the outside bottom edges of the footings and the face of the slope but not to exceed 15 feet nor be less than 5 feet. The above recommended footing setbacks are preliminary and may be revised based on site specific soil conditions. Footing or pier excavations should be observed by the project geotechnical representative to document that the footing trenches have been excavated into competent bearing soils and to the embeddings recommended above. These observations should be performed prior to placing forms or reinforcing steel.

12.3 Drainage

Surcharge loading effects from the adjacent structures should be evaluated by the geotechnical and structural engineers. All retaining wall structures should be provided with appropriate drainage and appropriately waterproofed. The outlet pipe should be sloped to drain to a suitable outlet. It should be noted that that recommended subdrains does not provide protection against seepage through the face of the wall and/or efflorescence. If such seepage or efflorescence is undesirable, retaining walls should be waterproofed to reduce this potential.

Weep holes or open vertical masonry joints should be provided in retaining walls 3 feet or less in height to reduce the likelihood of entrapment of water in the backfill. Weep holes, if used, should be 3 inches or more in diameter and provided at intervals of 6 feet or less along the wall. Open vertical masonry joints, if used, should be provided at 32-inch or less intervals. A continuous gravel fill, 12 inches by 12 inches, should be placed behind the weep holes or open masonry joints. The gravel should be wrapped in filter fabric to reduce infiltration of fines and subsequent clogging of the gravel. Filter fabric may consist of Mirafi 140N or equivalent.

In lieu of weep holes or open joints, for retaining walls less than 3 feet, a perforated pipe and gravel subdrain may be used. Perforated pipe should consist of 4-inch or more diameter PVC Schedule 40 or ABS SDR-35, with the perforations laid down. The pipe should be embedded in 1.5 cubic feet per foot of 0.75 or 1.5-inch open graded gravel wrapped in filter fabric. Filter fabric may consist of Mirafi 140N equivalent.

Retaining walls greater than 3 feet high should be provided with a continuous backdrain for the full height of the wall. This drain could consist of geosynthetic drainage composite, such as Miradrain 6000 or equivalent, or a permeable drain material, placed against the entire backside of the wall. If a permeable drain material is used, the backdrain should be 1 or more feet thick. Caltrans Class II permeable material or open graded gravel or crushed stone may be used as permeable drain material. If gravel or crushed stone is used, it should have less than 5 percent material passing the No. 200 sieve. The drain should be separated from the backfill with a geofabric. The upper 1-foot of the backdrain should be covered with compacted fill. A drainage pipe consisting of 4-inch diameter perforated pipe (described above) surrounded by 1 cubic foot per foot of gravel or crushed rock wrapped in a filter fabric should be provided along the back of the wall. The pipe should be placed with perforations down, sloped at 2 percent or more and discharge to an appropriate outlet through a solid pipe. The pipe should outlet away from structures and slopes. The outside portions of retaining walls supporting backfill should be coated with an approved waterproofing compound to inhibit infiltration of moisture through the walls.
12.4 Temporary Excavations

The retaining walls should be constructed and backfilled as soon as possible after backcut excavations are constructed. Prolonged exposure of backcut slopes may result in some localized slope instability. To facilitate retaining wall construction, the lower 5 feet of temporary slopes may be cut vertical and the upper portions exceeding a height of 5 feet should be cut back at a gradient of 1:1 (h:v) or flatter for the duration of construction. However, temporary slopes should be observed by the project geotechnical consultant for evidence of potential instability. Depending on the results of these observations, flatter slopes may be necessary. The potential effects of various parameters such as weather, heavy equipment travel, storage near the tops of the temporary excavations and construction scheduling should also be considered in the stability of temporary slopes. Water should not be permitted to drain away from the slope. Surcharges due to equipment, spoil piles, etc., should not be allowed within 10 feet of the top of the slope.

All excavations should be made in accordance with Cal/OSHA. Excavation safety is the sole responsibility of the contractor.

12.5 Retaining Wall Backfill

The retaining wall backfill soils (with an expansion index of 20 or less) should be placed in 6 to 8 inch loose lifts, watered or air-dried as necessary to achieve near optimum moisture conditions, and compacted to at least 90 percent relative density (based on ASTM Test Methods D2922 and D3017).

13.0 MASONRY GARDEN WALLS

13.1 Construction on Level Ground

Where masonry screen walls or garden walls are proposed on level ground and 5 feet or more from the tops of descending slopes, the footings for these walls may be founded at a depth of 18 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 and in accordance with the structural engineer's recommendations.

13.2 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 through the footings. The footings should be placed monolithically with continuous rebar to serve as effective "grade beams" along the full lengths of the walls.

14.0 CONCRETE FLATWORK

14.1 Nonstructural Concrete Flatwork

Concrete flatwork (such as walkways, driveways, patios, bicycle trails, etc.) has a high potential for cracking due to changes in soil volume related to soil-moisture fluctuations. To
reduce the potential for excessive cracking and lifting, concrete should be designed in accordance with the minimum guidelines outlined in Table 5. These guidelines will reduce the potential for irregular cracking and promote cracking along construction joints, but will not eliminate all cracking or lifting. Thickening the concrete and/or adding additional reinforcement will further reduce cosmetic distress.

**TABLE 5**

*Nonstructural Concrete Flatwork for Very Low Expansive Soils*

<table>
<thead>
<tr>
<th></th>
<th><strong>Private Sidewalks</strong></th>
<th><strong>Private Drives</strong></th>
<th><strong>Patio/Entryways</strong></th>
<th><strong>City Sidewalk Curb and Gutters</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum Thickness (in.)</td>
<td>4 (nominal)</td>
<td>4(full)</td>
<td>4 (full)</td>
<td>City/Agency Standard</td>
</tr>
<tr>
<td>Presaturation</td>
<td>Presoak to 12 inches</td>
<td>Presoak to 12 inches</td>
<td>Presoak to 12 inches</td>
<td>City/Agency Standard</td>
</tr>
<tr>
<td>Reinforcement</td>
<td>—</td>
<td>No. 3 at 24 inches on centers</td>
<td>No. 3 at 24 inches on centers</td>
<td>City/Agency Standard</td>
</tr>
<tr>
<td>Thickened Edge</td>
<td>—</td>
<td>8” X 8”</td>
<td>8” X 8”</td>
<td>City/Agency Standard</td>
</tr>
<tr>
<td>Crack Control</td>
<td>Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness</td>
<td>Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness</td>
<td>Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness</td>
<td>City/Agency Standard</td>
</tr>
<tr>
<td>Maximum Joint Spacing</td>
<td>5 feet</td>
<td>10 feet or quarter cut whichever is closer</td>
<td>6 feet</td>
<td>City/Agency Standard</td>
</tr>
</tbody>
</table>

14.2 **Joint Spacing**

To reduce the potential for unsightly cracking, concrete sidewalks and patio type slabs should be provided with construction or expansion joints every 6 feet or less. Concrete driveway slabs should be provided with construction or expansion joints every 10 feet or less.

14.3 **Subgrade Preparation**

As a further measure to reduce cracking of concrete flatwork, the upper 12 inches of subgrade soils below concrete-flatwork areas should first be compacted to a relative density of 90 percent of more and then thoroughly watered to achieve a moisture content that is equal to or slightly greater than optimum moisture content. This moisture should extend to a depth of 12 inches or more below subgrade and maintained in the soils during placement of concrete. Pre-watering of the soils will promote uniform curing of the concrete and reduce the potential for the development of shrinkage cracks. A representative of the project geotechnical consultant should observe and document the density and moisture content of the soils and depth of moisture penetration prior to placing concrete.
15.0 PLANTERS

Area drains should be extended into planters that are located within 5 feet of building walls, foundations, retaining walls and masonry garden walls to reduce excessive infiltration of water into the adjacent foundation soils. The surface of the ground in these areas should also be sloped at a gradient of 2 percent or more away from the walls and foundations. Drip-irrigation systems are also recommended to reduce overwatering and subsequent saturation of the adjacent foundation soils.

16.0 SOIL CORROSIVITY

16.1 Corrosivity to Concrete and Metal

The National Association of Corrosion Engineers (NACE) defines corrosion as “a deterioration of a substance or its properties because of a reaction with its environment”. From a geotechnical viewpoint, the “environment” is the prevailing foundation soils and the “substances” are the reinforced concrete foundations or various buried metallic elements such as rebar, piles, pipes, etc., which are in direct contact with or within close vicinity of the foundation soil.

In general, soil environments that are detrimental to concrete have high concentrations of soluble sulfates. ACI 318R-05, Table 4.3.1 provides specific guidelines for the concrete mix design based on different amount of soluble sulfate content. The minimum amount of chloride ions in the soil environment that are corrosive to steel, either in the form of reinforcement protected by concrete cover, or plain steel substructures such as steel pipes or piles, is 500 ppm per California Test 532 and ACI 318R-05, Table 4.4.1.

The corrosion potential of the onsite materials was evaluated for its effect on steel and concrete. The corrosion potential was evaluated using the results of laboratory tests on representative samples obtained during our field exploration. Laboratory testing was performed to evaluate pH, minimum electrical resistivity and chloride and soluble sulfate content. Based on testing performed during this investigation within the project site, the onsite soils are classified as having a negligible sulfate exposure condition in accordance with ACI 318R-05, Table 4.3.1, and negligible chloride exposure condition in accordance with ACI 318R-05, Table 4.4.1. Based on laboratory testing of on-site soils it is also our opinion that onsite soil should be considered mildly to moderately corrosive to buried metals due to the moderate to high resistivity. Metal piping should be corrosion-protected or consideration should be given to using plastic piping instead of metal.

Despite the minimum recommendation above, LGC is not a corrosion-engineering firm. Therefore, we recommend that you consult with a competent corrosion engineer and conduct additional testing (if required) to evaluate the actual corrosion potential of the site and to provide recommendations to reduce the corrosion potential with respect to the proposed improvements. The recommendations of the corrosion engineer may supersede the above requirements.

These recommendations are based on the current and previous samples of the subsurface soils or bedrock. The initiation of grading at the site could blend various soil types and import soils may be used locally. These changes made to the foundation soils could alter sulfate-content.
levels. Accordingly, it is recommended that additional testing be performed at the completion of grading.

17.0 PRELIMINARY PAVEMENT DESIGN

Structural pavement section design recommendations presented herein are based on a soil samples recovered during our preliminary geotechnical investigation. However, it should be understood that the soil material exposed during grading may differ from the materials sampled and tested during this investigation. Therefore, preliminary pavement recommendations are subject to verification and possible revision based on any revised traffic indices as well as sampling and testing of subgrade soils that exist after rough grading.

For planning and design purposes, we have prepared the following preliminary pavement sections based on an R-value testing results on a near surface soil sample collected. R-value testing indicated an R-value of 77. Based on utilizing a maximum design R-Value of 50 and the assumed Traffic Indices (T.I.’s) of 5.0 and 6.0, Table 6 presents recommended preliminary pavement designs for a range of assumed traffic conditions. City of Claremont minimum pavement sections were also considered in our pavement design.

TABLE 6
Preliminary Pavement Design

<table>
<thead>
<tr>
<th>AREA</th>
<th>ASSUMED TRAFFIC INDEX</th>
<th>DESIGN R-VALUE</th>
<th>ASPHALTIC CONCRETE (AC) (feet)</th>
<th>AGGREGATE BASE (AB) (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parking Lot and Auto Drive Areas</td>
<td>5.0</td>
<td>50</td>
<td>0.25</td>
<td>0.34</td>
</tr>
<tr>
<td>Entrance Apron and Heavy Traffic Areas</td>
<td>6.0</td>
<td>50</td>
<td>0.25</td>
<td>0.42</td>
</tr>
</tbody>
</table>

Subgrade soil immediately below the aggregate base (base) should be compacted to a minimum of 95 percent relative compaction based on ASTM Test Method D1557 to a minimum depth of 12 inches. Final subgrade compaction should be performed prior to placing base or asphaltic concrete and after all utility trench backfills have been compacted and tested.

Base materials should consist of crushed aggregate base conforming to Section 200-2 of Greenbook. The upper 12 inches of all aggregate base materials should be compacted to at least 95 percent of the laboratory maximum dry density determined in accordance with ASTM D1557.

Our preliminary pavement recommendations should be considered as minimum and can be revised once actual Traffic Indices are known or superseded by the City of Claremont.

18.0 ALTERNATIVE PRELIMINARY PAVEMENT DESIGNS

For planning and design purposes, we have prepared the following alternative preliminary concrete pavement and preliminary interlocking concrete paver pavement sections based on the previous R-value testing results of a near surface soil sample. Based on utilizing a maximum design R-Value of 50
and the assumed Traffic Indices (T.I.'s) of 5.0 and 6.0, Tables 7 and 8 present recommended alternative preliminary concrete and interlocking concrete paver pavement designs for a range of assumed traffic conditions:

**TABLE 7**

**Preliminary Concrete Pavement Design**

<table>
<thead>
<tr>
<th>AREA</th>
<th>ASSUMED TRAFFIC INDEX</th>
<th>DESIGN R-VALUE</th>
<th>REINFORCED CONCRETE (feet)</th>
<th>AGGREGATE BASE (AB) (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parking Lot and Auto Drive Areas</td>
<td>5.0</td>
<td>50</td>
<td>0.50</td>
<td>0.34</td>
</tr>
<tr>
<td>Entrance Apron and Heavy Traffic Areas</td>
<td>6.0</td>
<td>50</td>
<td>0.50</td>
<td>0.42</td>
</tr>
</tbody>
</table>

Concrete pavement should be reinforced with No. 3 bars spaced 24 inches or less on-centers, both ways. Slab reinforcement should be supported on concrete chairs or bricks so that the desired placement is near mid-depth.

**TABLE 8**

**Preliminary Interlocking Concrete Paver Pavement Design**

<table>
<thead>
<tr>
<th>AREA</th>
<th>ASSUMED TRAFFIC INDEX</th>
<th>DESIGN R-VALUE</th>
<th>INTERLOCKING CONCRETE PAVER THICKNESS (feet)</th>
<th>AGGREGATE BASE (AB) (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parking Lot and Auto Drive Areas</td>
<td>5.0</td>
<td>50</td>
<td>0.26</td>
<td>0.34</td>
</tr>
<tr>
<td>Entrance Apron and Heavy Traffic Areas</td>
<td>6.0</td>
<td>50</td>
<td>0.26</td>
<td>0.42</td>
</tr>
</tbody>
</table>

Subgrade soil immediately below the aggregate base (base) should be compacted to a minimum of 95 percent relative compaction based on ASTM Test Method D1557 to a minimum depth of 12 inches. Final subgrade compaction should be performed prior to placing base or asphaltic concrete and after all utility trench backfills have been compacted and tested.

Base materials should consist of crushed aggregate base conforming to Section 200-2 of Greenbook. The upper 12 inches of all aggregate base materials should be compacted to at least 95 percent of the laboratory maximum dry density determined in accordance with ASTM D1557.

Our alternative preliminary pavement recommendations should be considered as minimum and can be revised once actual Traffic Indices are known or superseded by the City of Claremont.

**19.0 PLAN REVIEWS AND CONSTRUCTION SERVICES**

This report has been prepared for the exclusive use of Temecula Hotel Development LLC to assist the project engineer and architect in the design of the proposed development. It is recommended that LGC be engaged to review the rough grading plans, structural plans and the final design drawings and
specifications prior to construction. This is to document that the recommendations contained in this report have been properly interpreted and incorporated into the project specifications. LGC's review of the rough grading plans may indicate that additional subsurface exploration, laboratory testing and analysis should be performed to address areas of concern. If LGC is not accorded the opportunity to review these documents, we can take no responsibility for misinterpretation of our recommendations.

We recommend that LGC be retained to provide geotechnical engineering services during both the rough grading and construction phases of the work. This is to document compliance with the design, specifications or recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated prior to start of construction.

If the project plans change significantly (e.g., building loads or type of structures), we should be retained to review our original design recommendations and their applicability to the revised construction. If conditions are encountered during construction that appear to be different than those indicated in this report, this office should be notified immediately. Design and construction revisions may be required.

20.0 LIMITATIONS

Our services were performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable engineers and geologists practicing in this or similar localities. The professional opinions contained herein have been derived in accordance with current standards of practice. No other warranty, expressed or implied, is made as to the conclusions and professional advice included in this report. The samples taken and submitted for laboratory testing, the observations made and the in-situ field testing performed are believed representative of the entire project; however, soil and geologic conditions can vary in characteristics between excavations, both laterally and vertically and may be different than our preliminary findings.

If this occurs, the changed conditions must be evaluated by the project geotechnical engineer and engineering geologist and design(s) adjusted as required or alternate design(s) recommended.

This report is issued with the understanding that it is the responsibility of the owner, or of his/her representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and/or project engineer and incorporated into the plans, and the necessary steps are taken to see that the contractor and/or subcontractor properly implements the recommendations in the field. The contractor and/or subcontractor should notify the owner if they consider any of the recommendations presented herein to be unsafe.

The conclusions and opinions contained in this report are based on the results of the described geotechnical evaluations and represent our professional judgment. The findings, conclusions and recommendations contained in this report are to be considered tentative only and subject to confirmation by the undersigned during the construction process. Without this confirmation, this report is to be considered incomplete and LGC or the undersigned professionals assume no responsibility for its use.

The conclusions and opinions contained in this report are valid up to a period of 2 years from the date of this report. Changes in the conditions of a property can and do occur with the passage of time, whether they be because of natural processes or the works of man on this or adjacent properties. In
addition, changes in applicable or appropriate codes or standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore if any of the above mentioned situations occur, an update of this report should be completed.

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

The opportunity to be of service is appreciated. Should you have any questions regarding the content of this report, or should you require additional information, please do not hesitate to contact this office at your earliest convenience.
APPENDIX A

REFERENCES AND AERIAL PHOTOGRAPHS
APPENDIX A

References Reviewed


Greensfelder, R.W., 1974, Maximum Credible Rock Accelerations from Earthquakes in California, CDMG, MS-23.


Jennings, Charles W., 1994, Fault Activity Map of California and Adjacent areas, Map No. 6, California Division of Mines and Geology.


Southern California Earthquake Center, University of Southern California, Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines For Analyzing and Mitigating Liquefaction Hazards in California, March 1999.

APPENDIX A

Aerial Photographs Reviewed

<table>
<thead>
<tr>
<th>SOURCE</th>
<th>FLIGHT</th>
<th>FRAME(S)</th>
<th>FLIGHT DATE</th>
<th>SCALE</th>
</tr>
</thead>
<tbody>
<tr>
<td>USDA</td>
<td>AXL-7F</td>
<td>12-13</td>
<td>5/21/49</td>
<td>1&quot;=1.667'</td>
</tr>
<tr>
<td>USDA</td>
<td>AXJ-9K</td>
<td>162-163</td>
<td>1/2/53</td>
<td>1&quot;=1.667'</td>
</tr>
<tr>
<td>USDA</td>
<td>AXL-17W</td>
<td>33-34</td>
<td>10/16/59</td>
<td>1&quot;=1.667'</td>
</tr>
<tr>
<td>USDA</td>
<td>AXL-18W</td>
<td>99-100</td>
<td>11/6/59</td>
<td>1&quot;=1.667'</td>
</tr>
<tr>
<td>Aerial Mapping Co.</td>
<td>311-1</td>
<td>5-7</td>
<td>3/29/60</td>
<td>N/A</td>
</tr>
<tr>
<td>Continental</td>
<td>65200</td>
<td>164-166</td>
<td>5/8/65</td>
<td>N/A</td>
</tr>
<tr>
<td>Continental</td>
<td>60-2</td>
<td>40-41</td>
<td>1-30-70</td>
<td>N/A</td>
</tr>
<tr>
<td>Continental</td>
<td>60-1</td>
<td>25-26</td>
<td>1/30/70</td>
<td>N/A</td>
</tr>
<tr>
<td>Continental</td>
<td>71000</td>
<td>110-111</td>
<td>3/3/71</td>
<td>N/A</td>
</tr>
<tr>
<td>Continental</td>
<td>75000</td>
<td>152-153</td>
<td>10/24/75</td>
<td>N/A</td>
</tr>
<tr>
<td>Continental</td>
<td>PC-C11</td>
<td>61-62</td>
<td>1/15/76</td>
<td>N/A</td>
</tr>
<tr>
<td>Continental</td>
<td>76162</td>
<td>170-171</td>
<td>11/7/76</td>
<td>N/A</td>
</tr>
<tr>
<td>Continental</td>
<td>FC-LA</td>
<td>181-182</td>
<td>5/12/79</td>
<td>N/A</td>
</tr>
<tr>
<td>Continental</td>
<td>SBD</td>
<td>14,16</td>
<td>1/1/80</td>
<td>N/A</td>
</tr>
<tr>
<td>Continental</td>
<td>83001</td>
<td>155-156</td>
<td>12/2/83</td>
<td>N/A</td>
</tr>
<tr>
<td>Continental</td>
<td>F</td>
<td>48-49</td>
<td>12/30/86</td>
<td>N/A</td>
</tr>
<tr>
<td>Continental</td>
<td>C81-9</td>
<td>26-27</td>
<td>5/25/90</td>
<td>N/A</td>
</tr>
<tr>
<td>Continental</td>
<td>C89-21</td>
<td>243-244</td>
<td>5/13/93</td>
<td>1&quot;=2,000'</td>
</tr>
<tr>
<td>Continental</td>
<td>C115-26</td>
<td>59-60</td>
<td>7/11/95</td>
<td>1&quot;=2,000'</td>
</tr>
<tr>
<td>Continental</td>
<td>C127-26</td>
<td>230-231</td>
<td>10/18/98</td>
<td>1&quot;=2,000'</td>
</tr>
</tbody>
</table>
APPENDIX B

FIELD EXPLORATION AND BORING LOGS
APPENDIX B

Field Exploration

B-1 General

Geologic mapping of the site was carried out by LGC’s personnel. The locations of the exploratory excavations were chosen to obtain subsurface information needed to achieve the objective for this investigation.

A visual survey was conducted to verify that the proposed excavations would not encounter any subsurface utility lines. No underground lines were encountered during the field exploratory program.

B-2 Excavation, Drilling and Sampling

Our subsurface exploration was performed on August 17, 2015, which included drilling, logging and sampling five (5) hollow-stem auger borings, to depths ranging from 5.5 feet to 20.0 feet. Logs of the borings are presented in Appendix B, and their approximate locations are depicted on the Geotechnical Map (Plate 1). Prior to the subsurface work, an underground utilities clearance was obtained from Underground Service Alert of Southern California. At the conclusion of the subsurface investigation, all test pits were backfilled with native materials. Minor settlement of the backfill soils may occur over time.

During our subsurface investigation, representative bulk and relatively undisturbed samples were retained for laboratory testing. Laboratory testing was performed on selected representative samples of onsite soil and/or bedrock materials soil samples and included in-situ dry density and moisture content, maximum dry density and optimum moisture content, expansion index, sulfate content, chloride content, pH, resistivity, grain size analysis, direct shear and consolidation tests. A discussion of the tests performed and a summary of the results are presented in Appendix C. The moisture and density test results are presented on the boring logs which are presented on the following pages.

B-3 Miscellaneous

The boring logs describe the earth materials encountered, sampling method used, and field and laboratory tests performed. The logs also show the test pit number, date of completion, and the name of the logger. A geologist logged the borings in accordance with the Standard Practice for Description and Identification of Soils (Visual-Manual Procedure) ASTM D2488-93. The boundaries between soil types shown on the logs are approximate and the transition between different soil layers may be gradual. The logs of the borings are presented on the following pages.
## Geotechnical Boring Log B-1

**Date:** 8/17/15  
**Project Name:** 721 S. Indian Hill  
**Logged By:** KRM  
**Drilling Company:** 2R  
**Type of Rig:** Hollow Stem Auger  
**Drive Weight (lbs.):** 140  
**Drop (in.):** 30  
**Hole Diameter (in.):** 8"  
**Top of Hole Elevation (ft):** Hole Location: See Geotechnical Map

<table>
<thead>
<tr>
<th>Elevation (MSL)</th>
<th>Blow Count / 12&quot;</th>
<th>Sample No.</th>
<th>Soil Graphic / Group</th>
<th>Geologic / Group Symbol</th>
<th>In-Situ Moist. (%)</th>
<th>Dry Density (pcf)</th>
<th>Standard Penetration Test</th>
<th>CURVE</th>
<th>Type of Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>8 11</td>
<td>R1</td>
<td>SM</td>
<td></td>
<td>6.6</td>
<td>107.2</td>
<td>10.20</td>
<td>12.7</td>
<td>MAX Density Expansion Sulfae, Chloride, Resistivity, Shear, R-Val. Sample @ 0.5' 2.0'</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>4.9</td>
<td>40</td>
<td>33.5</td>
<td></td>
<td>Bulk Sample @ 0.5' 2.0'</td>
</tr>
</tbody>
</table>

**DESCRIPTION**
- **ASPHALT:** 5" THICK
  - Silty SAND: dark brown, dry to damp, loose to medium dense, fine to coarse grained, some fine to coarse gravel, traces of small cobbles up to 5" diameter.
- **ALLUVIAL FAN DEPOSITS:**
  - Sandy GRAVEL/Gravelly SAND: orange brown, loose, medium dense, fine to coarse grained, damp, some silt, friable, abundant fine to coarse gravel, trace of cobbles, difficult drilling.

**Total Depth:** 5.5'  
**Practical Refusal @ 5.5'**  
**No Groundwater**  
**No Caving**

---

**Sample Legend**
- X Bag Sample  
- Bag Sample  
- SPT Sample  
- Ring Sample (CA modified)
# Geotechnical Boring Log B-2

**Date:** 8/17/15  
**Project Name:** 721 S. Indian Hill  
**Logged By:** KRM  
**Drilling Company:** 2R  
**Type of Rig:** Hollow Stem Auger  
**Drive Weight (lbs.):** 140  
**Drop (in.):** 30  
**Hole Diameter (in.):** 8"  
**Top of Hole Elevation (ft.):**  
**Hole Location:** See Geotechnical Map

<table>
<thead>
<tr>
<th>Elevation (M.S.L.) and Depth (ft.)</th>
<th>Blow Count / 12&quot;</th>
<th>Sample No.</th>
<th>Soil Graphic / Group Symbol</th>
<th>DESCRIPTION</th>
<th>In-Situ Moist. (%)</th>
<th>Dry Density (pcf)</th>
<th>Standard Penetration Test Depth</th>
<th>N</th>
<th>CURVE</th>
<th>Type of Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
<td>Q1 GP-SP</td>
<td></td>
<td>ASPHALT: 5.5&quot; THICK</td>
<td>4.2</td>
<td>20.35</td>
<td>10 20 30 50</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>14 16</td>
<td>R1</td>
<td>SP</td>
<td>ALLUVIAL FAN DEPOSITS:</td>
<td>1.9</td>
<td>177.5</td>
<td>25</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Sandy GRAVEL/Gravely SAND: dark brown, damp, loose to medium dense, fine to coarse grained, abundant fine gravel, friable, slightly micaceous. @2.0', dense.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>16 16</td>
<td>R2</td>
<td></td>
<td>SAND: dark brown and yellow brown, dry to damp, dense, fine to very coarse grained, friable, abundant fine to coarse gravel, some cobbles, slightly micaceous. @8.0', light orange brown, very dense, gravelly mostly of granitic composition. difficult drilling.</td>
<td>1.7</td>
<td>127.9</td>
<td>45</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
| 15                                |                   |            |                             | Total Depth: 13'  
Practical Refusal @ 13'  
No Groundwater  
No Caving | | | | | | |

**Sample Legend:**  
- Bag Sample  
- SPT Sample  
- Ring Sample (CA modified)  

---

**Geotechnical Consulting**
Geotechnical Boring Log B-3

Date: 8/17/15  Project Name: 721 S. Indian Hill  Page 1 of 1
Project Number: G151321-10  Logged By: KRM
Drilling Company: 2R  Type of Rig: Hollow Stem Auger
Drive Weight (lbs.): 140  Hole Diameter (in.): 8"
Drop (in.): 30
Top of Hole Elevation (ft.):  Hole Location: See Geotechnical Map

Elevation (MSL) and Depth (ft.)  Blow Count / 12'  Sample No.  Soil Graphic  Geologic / Group Symbol  DESCRIPTION

0  R1  SM  ASPHALT

ASPHALT

Silty SAND; dark brown, dry to damp, loose to medium dense, fine to medium grained, some fine to coarse gravel, slightly micaceous.

0.4  19.5  1.0  2.0  28

5  R2  Qaf

ALLUVIAL FAN DEPOSITS:

Sandy GRAVEL: orange brown and yellow brown, dry to damp, medium dense to dense, fine to coarse grained, abundant fine to coarse gravel (granitic composition), very friable.

@3.0', light yellow brown, damp, trace of silt, difficult drilling.

3.2  120.7  3.0  4.0  33.5

10  S1  SP

SAND: light orange brown and yellow gray, dry, very dense, fine to coarse grained, fine to coarse gravel, some cobbles up to 6", very friable, trace of silt.

Total Depth: 9'
Practical Refusal @ 9.0'
No Groundwater
No Caving

Sample Legend
X Bag Sample
S SPT Sample
R Ring Sample (CA modified)

Geotechnical Consulting

LGC
## Geotechnical Boring Log B-4

**Date:** 8/17/15  
**Project Name:** 721 S. Indian Hill  
**Logged By:** KRM  
**Project Number:** G151321-10  
**Drilling Company:** 2R  
**Type of Rig:** Hollow Stem Auger  
**Drop (in.):** 30"  
**Hole Diameter (in.):** 8"  
**Top of Hole Elevation (ft):** Hole Location: See Geotechnical Map

### Description

<table>
<thead>
<tr>
<th>Elevation (M.S.L.) and Depth (ft.)</th>
<th>Bow Count / 12th</th>
<th>Sample No.</th>
<th>Soil Graphic / Group Symbol</th>
<th>Geologic / Group Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>21</td>
<td>R1</td>
<td>Afu SM SP</td>
<td></td>
<td><strong>ASPHALT: 3&quot; THICK</strong></td>
</tr>
<tr>
<td>1.5</td>
<td>26</td>
<td>R2</td>
<td>Qf GP</td>
<td></td>
<td><strong>ARTIFICIAL FILL, UNDOCUMENTED:</strong></td>
</tr>
<tr>
<td>1.5</td>
<td>26</td>
<td>R2</td>
<td>Qf GP</td>
<td></td>
<td>Silty SAND: olive brown and light gray brown, dry, medium dense, fine to medium grained, some fine to coarse gravel.</td>
</tr>
<tr>
<td>1.5</td>
<td>26</td>
<td>R2</td>
<td>Qf GP</td>
<td></td>
<td>SAND: light gray brown and light yellow brown, damp, medium dense to dense, fine to coarse grained, abundant fine to coarse gravel with some cobbles up to 5&quot;, very friable.</td>
</tr>
<tr>
<td>2.2</td>
<td>33</td>
<td>R3</td>
<td>SP</td>
<td>Qf GP</td>
<td><strong>ALLUVIAL FAN DEPOSITS:</strong></td>
</tr>
<tr>
<td>2.2</td>
<td>33</td>
<td>R3</td>
<td>SP</td>
<td>Qf GP</td>
<td>Sandy GRAVEL: light brown gray and gray brown, dry, very dense, fine to medium grained, abundant fine to coarse gravel, cobbles up to 6&quot;.</td>
</tr>
<tr>
<td>2.2</td>
<td>33</td>
<td>R3</td>
<td>SP</td>
<td>Qf GP</td>
<td>SAND: light orange brown, dry to damp, dense to very dense, very fine to medium grained with occasional coarse grains, some fine gravel, very friable.</td>
</tr>
<tr>
<td>1.6</td>
<td>36</td>
<td>R4</td>
<td>SP</td>
<td>Qf GP</td>
<td>@13.0&quot;, olive-brown, damp, very dense, difficult drilling.</td>
</tr>
</tbody>
</table>

**Total Depth:** 17"  
**Practical Refusal @ 17.0"**  
**No Groundwater**  
**No Caving**

---

**Geotechnical Consulting**  
**LGC**

---

### Standard Penetration Test

<table>
<thead>
<tr>
<th>Depth</th>
<th>N</th>
<th>SPT</th>
<th>CURVE</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td></td>
<td>136.4</td>
<td>5.1</td>
</tr>
<tr>
<td>30</td>
<td></td>
<td>136.9</td>
<td>1.5</td>
</tr>
<tr>
<td>50</td>
<td></td>
<td>136.0</td>
<td>1.5</td>
</tr>
</tbody>
</table>

---

**Sample Legend**

- Bag Sample
- SPT Sample
- Ring Sample (CA modified)
**Geotechnical Boring Log B-5**

**Date:** 8/17/15  
**Project Name:** 721 S. Indian Hill  
**Logged By:** KRM  
**Drilling Company:** 2R  
**Type of Rig:** Hollow-Stem Auger  
**Drive Weight (lbs.):** 140  
**Drop (in.):** 30  
**Hole Diameter (in.):** 8"  
**Top of Hole Elevation (ft.):**  
**Hole Location:** See Geotechnical Map

<table>
<thead>
<tr>
<th>Elevation (MSL) and Depth (ft.)</th>
<th>Bow Count / 12&quot;</th>
<th>Sample No.</th>
<th>Soil Graphic</th>
<th>Geologic Group Symbol</th>
<th>In-Situ Moist. (%)</th>
<th>Dry Density (pcf)</th>
<th>SPT Depth N</th>
<th>CURVE 10 30 50 Type of Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>9 19 15 15</td>
<td>R1</td>
<td>Alu SM</td>
<td>SM-ML</td>
<td>1.2</td>
<td>126.3</td>
<td>1.0/2.0</td>
<td>23  MAX Density</td>
</tr>
<tr>
<td>-5</td>
<td>12 24 26</td>
<td>R2</td>
<td>Silty SAND</td>
<td>Light brown, dry, loose to medium dense, very fine to medium grained, some fine gravel.</td>
<td>2.0</td>
<td>130.6</td>
<td>4.0/6.0</td>
<td>30  Consol.</td>
</tr>
<tr>
<td>-10</td>
<td>18 21 11</td>
<td>S1</td>
<td>Alluvial Fan Deposits</td>
<td>Silty SAND: light orange brown and light yellow gray, dry, dense, fine to medium grained, some fine to coarse gravel, occasional cobbles up to 4&quot;.</td>
<td>1.2</td>
<td>7.0/10.0</td>
<td>7.0/10.0</td>
<td>26  Consol.</td>
</tr>
<tr>
<td>-15</td>
<td>22 14 11</td>
<td>R3</td>
<td>Older Alluvium</td>
<td>Silty SAND/Sandy SILT: orange olive, light orange brown, damp, medium dense to dense/stiff to very stiff, fine to coarse grained, occasional fine to coarse gravel, trace of pinhole pores and rootcasts.</td>
<td>4.6</td>
<td>117.6</td>
<td>10/0-11.0</td>
<td>17  Bulk Sample 10.0 15.0</td>
</tr>
</tbody>
</table>

**Total Depth:** 20'  
Practical refusal at 20'  
No Groundwater  
No Caving

---

Sample Legend:
- [ ] Bag Sample
- [ ] SPT Sample
- [ ] Ring Sample (CA modified)

Geotechnical Consulting

---

B-41
APPENDIX C

LABORATORY TESTING PROCEDURES AND TEST RESULTS
APPENDIX C

Laboratory Testing Procedures and Test Results

The laboratory testing program was directed towards providing quantitative data relating to the relevant engineering properties of the soils. Samples considered representative of site conditions were tested in general accordance with American Society for Testing and Materials (ASTM) procedure and/or California Test Methods (CTM), where applicable. The following summary is a brief outline of the test type and a table summarizing the test results.

Soil Classification: Soils were classified according the Unified Soil Classification System (USCS) in accordance with ASTM Test Methods D2487 and D2488. The soil classifications (or group symbol) are shown on the laboratory test data, and boring logs.

Maximum Dry Density Tests: The maximum dry density and optimum moisture content of typical materials were determined in accordance with ASTM test method D1557. The test results are presented in the table below:

<table>
<thead>
<tr>
<th>SAMPLE LOCATION</th>
<th>SAMPLE DESCRIPTION (USCS)</th>
<th>MAXIMUM DRY DENSITY (% by weight)</th>
<th>OPTIMUM MOISTURE CONTENT (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1 @0.5'-2.0'</td>
<td>Silty SAND (SM)</td>
<td>133.5</td>
<td>7.0</td>
</tr>
<tr>
<td>B-5 @ 0.5'-5.0'</td>
<td>Silty SAND (SM)</td>
<td>132.7</td>
<td>7.0</td>
</tr>
</tbody>
</table>

Expansion Index: The expansion potential of a selected sample was evaluated by the Expansion Index Test, U.B.C. Standard No. 18-2 and/or ASTM test method D4829. Specimens are molded under a given compactive energy at or near the optimum moisture content and approximately 50 percent saturation or approximately 90 percent relative compaction. The prepared 1-inch thick by 4-inch diameter specimens are loaded to an equivalent 144 psf surcharge and are inundated with tap water until volumetric equilibrium is reached. The results of these tests are presented in the table below:

<table>
<thead>
<tr>
<th>SAMPLE LOCATION</th>
<th>SAMPLE DESCRIPTION (USCS)</th>
<th>EXPANSION INDEX</th>
<th>EXPANSION POTENTIAL*</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1 @0.5'-2.0'</td>
<td>Silty SAND (SM)</td>
<td>0</td>
<td>Very Low</td>
</tr>
</tbody>
</table>

*Per ASTM D4829

Soluble Sulfates: The soluble sulfate content of selected samples was determined by standard geotechnical methods (CTM 417). The soluble sulfate content is used to determine the appropriate cement type and maximum water-cement ratios. The test results are presented in the table below:

<table>
<thead>
<tr>
<th>SAMPLE LOCATION</th>
<th>SAMPLE DESCRIPTION (USCS)</th>
<th>SULFATE CONTENT (ppm)</th>
<th>SULFATE EXPOSURE*</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1 @ 0.5'-2.0'</td>
<td>Silty SAND (SM)</td>
<td>Non-Detect</td>
<td>Negligible</td>
</tr>
</tbody>
</table>

*Per ACI 318R-05 Table 4.3.1
**Chloride Content:** Chloride content was tested with CTM 422. The results are presented below:

<table>
<thead>
<tr>
<th>SAMPLE LOCATION</th>
<th>SAMPLE DESCRIPTION (USCS)</th>
<th>CHLORIDE CONTENT (ppm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1 @ 0.5'-2.0'</td>
<td>Silty SAND (SM)</td>
<td>32</td>
</tr>
</tbody>
</table>

**Minimum Resistivity and pH Tests:** Minimum resistivity and pH tests were performed with CTM 643. The results are presented in the table below:

<table>
<thead>
<tr>
<th>SAMPLE LOCATION</th>
<th>SAMPLE DESCRIPTION (USCS)</th>
<th>pH</th>
<th>MINIMUM RESISTIVITY (ohm-cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1 @ 0.5'-2.0'</td>
<td>Silty SAND (SM)</td>
<td>6.89</td>
<td>7,900</td>
</tr>
</tbody>
</table>

**Direct Shear:** Direct shear tests were performed on selected remolded samples, which were soaked for a minimum of 24 hours under a surcharge equal to the applied normal force during testing. After transfer of the sample to the shear box, and reloading the sample, pore pressures set up in the sample due to the transfer were allowed to dissipate for a period of approximately 1 hour prior to application of shearing force. The samples were tested under various normal loads, a motor-driven, strain-controlled, direct-shear testing apparatus at a strain rate of less than 0.001 to 0.5 inch per minute (depending upon the soil type). The graphical test results are presented in the table below:

<table>
<thead>
<tr>
<th>SAMPLE LOCATION</th>
<th>SAMPLE DESCRIPTION</th>
<th>ANGLE OF INTERNAL FRICTION (degrees)</th>
<th>COHESION (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1 @ 0.5'-2.0'</td>
<td>Silty SAND (SM)</td>
<td>32</td>
<td>320</td>
</tr>
</tbody>
</table>

**R-Value:** The resistance R-value was determined by the ASTM test method D2844 for base, sub-base, and basement soils. The samples were prepared and exudation pressure and R-value were determined. These results were used for pavement design:

<table>
<thead>
<tr>
<th>SAMPLE LOCATION</th>
<th>SAMPLE DESCRIPTION (USCS)</th>
<th>R-VALUE</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1 @ 0.5'-2.0'</td>
<td>Silty SAND (SM)</td>
<td>77</td>
</tr>
</tbody>
</table>

**Consolidation:** Consolidation tests were performed on undisturbed samples (Modified ASTM Test method D2435). The samples (2.42 inches in diameter and 1-inch in height) were placed in a consolidometer and increasing loads were applied. The sample was allowed to consolidate under “double drainage” and total deformation for each loading step was recorded. The percent consolidation for each load stamp was recorded as the ration of the amount of vertical compression to the original sample height. The graphical test results are presented on the following pages.
CONsolidation Test Results

Note: Filled circle denotes readings after sample was submerged in water

<table>
<thead>
<tr>
<th></th>
<th>In-place</th>
<th>Remolded</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry Density, (pcf):</td>
<td>117.6</td>
<td></td>
</tr>
<tr>
<td>Moisture (%):</td>
<td>4.6</td>
<td></td>
</tr>
<tr>
<td>Water Added @ ksf:</td>
<td></td>
<td>1.378</td>
</tr>
<tr>
<td>Maximum Load, ksf:</td>
<td></td>
<td>11.020</td>
</tr>
<tr>
<td>Soil Description:</td>
<td>Silty Sand/Sandy Silt</td>
<td></td>
</tr>
<tr>
<td>U.S.C.S.</td>
<td>SM/ML</td>
<td></td>
</tr>
<tr>
<td>% Collapse/Swell (-):</td>
<td></td>
<td>0.27</td>
</tr>
</tbody>
</table>

P.N. G151321-10 LOCATION: B-5 @ 10.0'
CLIENT: Temecula Hotel Development

LGC
APPENDIX D

GENERAL EARTHWORK AND GRADING SPECIFICATIONS
APPENDIX D

General Earthwork and Grading Specifications

1.0 General

1.1 Intent: These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

1.2 The Geotechnical Consultant of Record: Prior to commencement of work, the owner shall employ a qualified Geotechnical Consultant of Record (Geotechnical Consultant). The Geotechnical Consultant shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

Prior to commencement of grading, the Geotechnical Consultant shall review the "work plan" prepared by the Earthwork Contractor (Contractor) and schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing.

During the grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to confirm that the attained level of compaction is being accomplished as specified. The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

1.3 The Earthwork Contractor: The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing the grading in accordance with the project plans and specifications. The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "equipment" of work and the estimated quantities of daily earthwork contemplated for the site prior to commencement of grading.
The Contractor shall inform the owner and the Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate personnel will be available for observation and testing. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish the earthwork in accordance with the applicable grading codes and agency ordinances, these Specifications, and the recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of the Geotechnical Consultant, unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, insufficient buttress key size, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the Geotechnical Consultant shall reject the work and may recommend to the owner that construction be stopped until the conditions are rectified. It is the contractor’s sole responsibility to provide proper fill compaction.

2.0 Preparation of Areas to be Filled

2.1 Clearing and Grubbing: Vegetation, such as brush, grass, roots, and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies, and the Geotechnical Consultant.

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). No fill lift shall contain more than 10 percent of organic matter. Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed. The contractor is responsible for all hazardous waste relating to his work. The Geotechnical Consultant does not have expertise in this area. If hazardous waste is a concern, then the Client should acquire the services of a qualified environmental assessor.

2.2 Processing: Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be overexcavated as specified in the following section. Scarification shall continue until soils are broken down and free of oversize material and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

2.3 Overexcavation: In addition to removals and overexcavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy,
organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as evaluated by the Geotechnical Consultant during grading.

2.4 **Benching:** Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise overexcavated to provide a flat subgrade for the fill.

2.5 **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 the survey control for determining elevations of processed areas, keys, and benches.

3.0 **Fill Material**

3.1 **General:** Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by the Geotechnical Consultant prior to placement. 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.

3.2 **Oversize:** Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 8 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or underground construction.

3.3 **Import:** If importing of fill material is required for grading, proposed import material shall meet the requirements of Section 3.1. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

4.0 **Fill Placement and Compaction**

4.1 **Fill Layers:** Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

4.2 **Fill Moisture Conditioning:** Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed
in accordance with the American Society of Testing and Materials (ASTM Test Method D1557-91).

4.3 **Compaction of Fill:** After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557-91). Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

4.4 **Compaction of Fill Slopes:** In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557-91.

4.5 **Compaction Testing:** Field tests for moisture content and relative compaction of the fill soils shall be performed by the Geotechnical Consultant. Location and frequency of tests shall be at the Consultant's discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

4.6 **Frequency of Compaction Testing:** Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. In addition, as a guideline, at least one (1) test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.

4.7 **Compaction Test Locations:**

The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two (2) grade stakes within a horizontal distance of 100 feet and vertically less than 5 feet apart from potential test locations shall be provided.

5.0 **Subdrain Installation**

Subdrain systems shall be installed in accordance with the approved geotechnical report(s) and grading plan. The Geotechnical Consultant may recommend additional subdrain and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys.