APPENDIX B

GEOTEchnical InVESTIGATION
GEOTECHNICAL INVESTIGATION

36-Unit Residential Development
18840 Saratoga Los Gatos Road
Los Gatos, California

PREPARED FOR:
THE STANLEY GROUP
18840 SARATOGA LOS GATOS ROAD
LOS GATOS, CALIFORNIA  95030

PREPARED BY:
GEOCON CONSULTANTS, INC.
6671 BRISA STREET
LIVERMORE, CALIFORNIA  94550

GEOCON PROJECT NO. E9051-04-01
APRIL 2018
Project No. E9051-04-01
April 24, 2018

The Stanley Group
18840 Saratoga Los Gatos Road
Los Gatos, California 95030

Attention: Mr. Russ Stanley

Subject: PROPOSED 36-UNIT RESIDENTIAL DEVELOPMENT
18840 SARATOGA LOS GATOS ROAD
LOS GATOS, CALIFORNIA
GEOTECHNICAL INVESTIGATION

Dear Mr. Stanley:

In accordance with your authorization of our proposal dated February 6, 2018, we have performed a geotechnical investigation for the subject project in Los Gatos, California. Our investigation was performed to observe the soil and geologic conditions that may impact site development for the project as presently planned. The accompanying report presents the results of our investigation and conclusions and recommendations pertaining to the geotechnical aspects of the proposed project. The findings of this study indicate the site is suitable for development as planned provided the recommendations of this report are implemented during design and construction. We understand the project site will be annexed into the City of Monte Sereno.

If you have any questions regarding this report, or if we may be of further service, please contact the undersigned at your convenience.

Sincerely,

GEOCON CONSULTANTS, INC.

Shane Rodacker, PE, GE
Senior Engineer

Jacob Bishop-Moser, EIT
Senior Staff Engineer
## TABLE OF CONTENTS

1. PURPOSE AND SCOPE ..................................................................................................................................................... 1
2. SITE CONDITIONS AND PROJECT DESCRIPTION ......................................................................................................... 1
3. GEOLOGIC SETTING ......................................................................................................................................................... 2
4. GEOLOGIC HAZARDS ........................................................................................................................................................... 2
   4.1 Surface Fault Rupture .................................................................................................................................................... 3
   4.2 Ground Shaking .............................................................................................................................................................. 3
   4.3 Liquefaction ................................................................................................................................................................... 4
   4.4 Landslides ......................................................................................................................................................................... 4
   4.5 Tsunamis and Seiches .................................................................................................................................................... 5
5. SOIL AND GROUNDWATER CONDITIONS ......................................................................................................................... 5
   5.1 Alluvium ........................................................................................................................................................................... 5
   5.2 Santa Clara Formation .................................................................................................................................................... 5
   5.3 Groundwater ................................................................................................................................................................. 5
6. CONCLUSIONS AND RECOMMENDATIONS .................................................................................................................... 6
   6.1 General ............................................................................................................................................................................. 6
   6.2 Seismic Design Criteria ................................................................................................................................................ 6
   6.3 Soil and Excavation Characteristics ............................................................................................................................ 7
   6.4 Materials for Fill .............................................................................................................................................................. 8
   6.5 Grading .............................................................................................................................................................................. 8
   6.6 Shallow Foundation Recommendations ..................................................................................................................... 9
   6.7 Post-Tensioned Foundation Recommendations ....................................................................................................... 10
   6.8 Temporary Excavations ................................................................................................................................................ 11
   6.9 Underground Utilities .................................................................................................................................................. 11
   6.10 Concrete Slabs-on-Grade ............................................................................................................................................ 12
   6.11 Moisture Protection Considerations ........................................................................................................................ 12
   6.12 Pavement Recommendations ..................................................................................................................................... 13
   6.13 Retaining Wall Design ................................................................................................................................................ 15
   6.14 Surface Drainage ....................................................................................................................................................... 16
7. FURTHER GEOTECHNICAL SERVICES ............................................................................................................................ 17
   7.1 Plan and Specification Review ..................................................................................................................................... 17
   7.2 Testing and Observation Services ................................................................................................................................ 17

## LIMITATIONS AND UNIFORMITY OF CONDITIONS

### FIGURES

- Figure 1, Vicinity Map
- Figure 2, Site Plan
- Figure 3, Geologic Cross Section A–A’
- Figure 4, Geologic Cross Section B–B’

### EXHIBITS

- Exhibit 1, Excerpted Information from County of Santa Clara GIS
- Exhibit 2, Excerpted Information from County of Santa Clara GIS

### APPENDIX A – FIELD INVESTIGATION

- Figure A1, Key to Boring Logs
- Figures A2 through A5, Logs of Exploratory Borings B1 through B4
- Figures A6 through A8, Cone Penetrometer Test Data, CPT1 through CPT3
TABLE OF CONTENTS (cont.)

APPENDIX B – LABORATORY TESTING
   Table B-I, Summary of Laboratory Atterberg Limits Test Results
   Table B-II, Summary of Laboratory No. 200 Wash Test Results
   Table B-III, Summary of Laboratory Expansion Index Test Results
   Table B-IV, Summary of Laboratory Direct Shear Test Results
   Figures B1 and B2, Summary of Laboratory Particle Size Analyses
   Figure B3, Summary of Laboratory Unconfined Compressive Strength Test Results

APPENDIX C – LIQUEFACTION ANALYSIS
   Selected CLiq Output

APPENDIX D – SLOPE STABILITY ANALYSIS
   Figure D1, SLOPE/W Output – Section B-B’ Existing Slope Condition
   Figure D2, SLOPE/W Output – Section B-B’ Temporary Cut

LIST OF REFERENCES
GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for a 36-unit residential subdivision proposed at 18840 Saratoga Los Gatos Road in Los Gatos, California (see Vicinity Map, Figure 1). The purpose of this investigation was to evaluate the subsurface soil and geologic conditions in the area of planned development and provide conclusions and recommendations pertaining to the geotechnical aspects of project design and construction, based on the conditions encountered during our study.

The scope of this investigation included field exploration, laboratory testing, engineering analysis and the preparation of this report. Our field exploration was performed on March 12 and March 16, 2018 and included the advancement of three Cone Penetrometer Test (CPT) soundings to maximum depths of approximately 51 feet or refusal and drilling four exploratory borings to approximately 26 ½ feet below existing grade at the site, or refusal. The locations of the CPT soundings and soil borings are depicted on the Site Plan, Figure 2. A detailed discussion of our field investigation and boring logs and CPT profiles are presented in Appendix A.

Laboratory tests were performed on selected soil samples obtained during the investigation to evaluate pertinent geotechnical parameters. Appendix B presents the laboratory test results in tabular format and graphical format. Appendix C presents selected output from our liquefaction analysis. Slope stability analyses are included in Appendix D.

The opinions expressed herein are based on analysis of the data obtained during the investigation and our experience with similar soil and geologic conditions. References reviewed to prepare this report are provided in the List of References section.

If project details vary significantly from those described herein, Geocon should be contacted to determine the necessity for review and possible revision of this report.

2. SITE CONDITIONS AND PROJECT DESCRIPTION

The approximately 4 ½ acre site is comprised of two adjacent parcels (Santa Clara County APN 510-08-018 and 510-08-019) on the southern side of Saratoga Los Gatos Road. The site is occupied by a motel and restaurant/bar businesses. Areas of asphalt parking, driveways and landscaping are also present. All existing improvements will be razed to accommodate the proposed development. Topographic information provided by the project civil engineer indicates the lower portion of the site slopes moderately to the northwest with high elevations on the order 480 feet MSL at the southeastern corner to elevations of approximately 460 to 465 feet MSL along Saratoga Los Gatos Road. A generally undeveloped parcel exists to the east. An approximately 40-foot-high slope ascends from the southern margin of the site with maximum inclinations on the order of 2 ½:1 (horizontal:vertical). Quito Fire Station borders the northwest corner of the site. Existing single family residential exists uphill to the south.

Based on the information provided by The Stanley Group, the proposed project will include the construction of a 36 residential units comprised of 21 single-family residences and 15 multi-family residences (see Figure 2). The buildings will be wood-framed and two stories in height (excluding basement/cellar parking) with concrete slab-on-grade and shallow footings or post-tensioned slabs for foundation support. Cuts and fills to reach pad grade for the proposed residential buildings will be on the approximate order of 4 feet or less for most of the development. Cuts up to approximately 8 feet will be performed at the rear of the site to established pad grade for the seven townhomes with tuck-under garages.
3. GEOLOGIC SETTING

Los Gatos is located within the Coast Ranges Geomorphic Province of California, which is characterized by a series of northwest trending mountains and valleys along the north and central coast of California. Topography is controlled by the predominant geological structural trends within the Coast Range that generally consist of northwest trending synclines, anticlines and faulted blocks. The dominant structure is a result of both active northwest trending strike-slip faulting, associated with the San Andreas Fault system, and east-west compression within the province.

The San Andreas Fault (SAF) is a major right-lateral strike-slip fault that extends from the Gulf of California in Mexico to Cape Mendocino in northern California. The SAF forms a portion of the boundary between two tectonic plates on the surface of the earth. To the west of the SAF is the Pacific Plate, which moves north relative to the North American Plate, located east of the fault. In the San Francisco Bay Area, movement across this plate boundary is concentrated on the SAF but also distributed, to a lesser extent, across several other faults including the Hayward and Calaveras faults, among others. Together, these faults are referred to as the SAF system.

Basement rock west of the SAF is generally granitic, while to the east it consists of a chaotic mixture of highly deformed marine sedimentary, submarine volcanic and metamorphic rocks of the Franciscan Complex. Both are typically Jurassic to Cretaceous in age (205 to 65 million years old). Overlying the basement rocks are Cretaceous (about 140 to 65 million years old) marine, as well as Tertiary (about 65 to 1.6 million years old) marine and non-marine sedimentary rocks with some continental volcanic rock. These Cretaceous and Tertiary rocks have typically been extensively folded and faulted largely because of movement along the SAF system, which has been ongoing for about the last 25 million years, and regional compression during the last about 4 million years. The inland valleys, as well as the structural depression within which San Francisco Bay is located, are filled with unconsolidated to semi-consolidated deposits of Quaternary age (about the last 1.6 million years). Continental deposits (alluvium) consist of unconsolidated to semi-consolidated sand, silt, clay and gravel, while the bay deposits typically consist of soft organic-rich silt and clay (bay mud) or sand.

Web-based geologic mapping by the United States Geological Survey indicates the site is underlain by late Pliocene to Pleistocene-age Santa Clara Formation. Younger alluvial deposits overlie the formational materials throughout much of the site. Artificial fills from past episodes of site development may also be present.

4. GEOLOGIC HAZARDS

Geologists and seismologists recognize the San Francisco Bay Area as one of the most seismically-active regions in the United States. The significant earthquakes that occur in the Bay Area are associated with crustal movements along well-defined active fault zones that generally trend in a northwesterly direction.

The table below presents approximate distances to active faults in the site vicinity based on mapping by CGS, as presented in an online fault database maintained by Caltrans. Site coordinates are N 37.2408°, W 122.0016°.
TABLE 4.1  
REGIONAL FAULT SUMMARY

<table>
<thead>
<tr>
<th>Fault Name</th>
<th>Approximate Distance to Site (miles)</th>
<th>Maximum Earthquake Magnitude, $M_w$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Monte Vista Shannon</td>
<td>1½</td>
<td>6.4</td>
</tr>
<tr>
<td>San Andreas (Santa Cruz Mountains)</td>
<td>3</td>
<td>8.0</td>
</tr>
<tr>
<td>San Andreas (Peninsula)</td>
<td>6½</td>
<td>8.0</td>
</tr>
<tr>
<td>Sargent</td>
<td>8</td>
<td>7.0</td>
</tr>
<tr>
<td>Zayante-Vergeles (Upper)</td>
<td>9½</td>
<td>7.0</td>
</tr>
<tr>
<td>Silver Creek</td>
<td>10</td>
<td>6.9</td>
</tr>
<tr>
<td>Zayante-Vergeles (Lower)</td>
<td>12</td>
<td>7.0</td>
</tr>
<tr>
<td>Hayward (Southern Extension)</td>
<td>14½</td>
<td>6.7</td>
</tr>
<tr>
<td>Calaveras (Central)</td>
<td>17</td>
<td>6.9</td>
</tr>
<tr>
<td>Hayward (South)</td>
<td>17½</td>
<td>7.3</td>
</tr>
<tr>
<td>Calaveras (North)</td>
<td>18½</td>
<td>6.9</td>
</tr>
<tr>
<td>San Gregorio</td>
<td>19</td>
<td>7.4</td>
</tr>
<tr>
<td>Monterey Bay – Tularcitos</td>
<td>23½</td>
<td>7.2</td>
</tr>
</tbody>
</table>

Faults tabulated above and many others in the Bay Area are sources of potential ground motion. However, earthquakes that might occur on other faults within the northern California area are also potential generators of significant ground motion and could cause ground shaking at the site.

4.1 Surface Fault Rupture

The site is not within a currently established State of California Earthquake Fault Zone for surface fault rupture hazards. No active faults are known to pass directly beneath the site. By CGS definition, an active fault is one with surface displacement within the last 11,000 years. A potentially-active fault has demonstrated evidence of surface displacement with the past 1.6 million years. Faults that have not moved in the last 1.6 million years are typically considered inactive.

The site is mapped within a Santa Clara County-designated Fault Rupture Hazard Zone due to a suspected fault trace originally mapped by the California Department of Water Resources in 1975 on the basis of displacement found in buried stream channels offsite. However, subsequent geologic mapping by others did not observe evidence of faulting at the ground surface (Terratech, 1985). In addition, excerpted GIS information provided by the County of Santa Clara does not indicate the presence of active or potentially-active faults at the site (see Exhibits 1 and 2).

4.2 Ground Shaking

We used the USGS web-based *Unified Hazard Tool* to estimate the peak ground acceleration (PGA) and mean magnitude associated with a 2,475-year return period that corresponds to an event with 2 percent
chance of exceedance in 50 years. The USGS estimated PGA is 1.41g and the modal magnitude is 7.9 for borderline Seismic Site Class C/D (Vs30 = 360 m/sec) based on a recent 2014 model within the application. While listing PGA is useful for comparison of potential effects of fault activity in a region, other considerations are important in seismic design, including frequency and duration of motion and soil conditions underlying the site.

4.3 Liquefaction

The site is located within a State of California Seismic Hazard Zone for liquefaction. Web-based mapping by the USGS indicates a “high” susceptibility to liquefaction for most of the site. Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary loss of shear strength due to pore pressure buildup under the cyclic shear stresses associated with intense earthquakes. Primary factors that trigger liquefaction are: moderate to strong ground shaking (seismic source), relatively clean, loose granular soils (primarily poorly graded sands and silty sands), and saturated soil conditions (shallow groundwater). Due to the increasing overburden pressure with depth, liquefaction of granular soils is generally limited to the upper 50 feet of a soil profile.

We assessed the potential for liquefaction using the computer software program CLiq (Version 2.0, Geologismiki) and the in-situ soil parameters measured in the CPT soundings. The software applied the methodology of Boulanger and Idriss (2014) to the CPT data to evaluate liquefaction potential and estimate resultant settlements. Our analysis also considered the potential for cyclic softening in clayey soils. Our evaluation incorporated an earthquake moment magnitude (Mw) of 7.9. Groundwater was not encountered in our soil borings at the site. However, we used a groundwater depth of 15 feet in our analysis as a conservative assumption. We used a ground motion/Peak Ground Acceleration (PGA) of 1.02 g for our analysis based on the USGS US Seismic Design Maps application.

Our liquefaction analysis identified potentially liquefiable layers at each CPT location. In general, these layers are relatively thin and located more than 15 feet below existing grade at the site. Consequences of liquefaction can include ground surface settlement, ground loss (sand boils) and lateral slope displacements (lateral spreading). For liquefaction-induced sand boils or fissures to occur, pore water pressure induced within liquefied strata must exert enough force to break through overlying, non-liquefiable layers. Based on methodology recommended by Youd and Garris (1995), which modified and advanced original research by Ishihara (1985), a capping layer of non-liquefiable soil can prevent the occurrence of sand boils and fissures. The presence of the non-liquefiable layer that mantles the site (which was observed to be at least 15 feet thick in our soil borings) and the depth to significant liquefiable layers, the potential for ground loss due to sand boils or fissures in a seismic event is considered low. Based on the limited potential for liquefaction and the distance to significant slope faces, the potential for lateral spreading to affect site improvements is low.

The likely consequence of potential liquefaction at the site is settlement. Our analysis indicates that total ground surface settlements on the order of \( \frac{1}{2} \) inch or less may result from liquefaction and/or cyclic softening after a seismic event. Selected output from our liquefaction analysis is presented in Appendix C.

4.4 Landslides

There are no known landslides near the site, nor is the site in the path of any known landslides. Based on the soils conditions encountered in our exploratory borings and observed in the hillside at the southern margin of the site, and our slope stability analyses, we do not consider the potential for a landslide to be a
significant hazard to this project. Our slope stability analyses considered the proposed cuts at the toe of the ascending slope at the southern margin of the site. The results of our analysis indicate acceptable factor of safety against deep-seated instability. A detailed discussion and output from our slope stability analyses are presented in Appendix D.

4.5 Tsunamis and Seiches

The site is not located within a coastal area. Therefore, tsunamis (seismic sea waves) are not considered a significant hazard at the site.

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. No major water-retaining structures are located immediately up gradient from the project site. Flooding from a seismically-induced seiche is considered unlikely.

5. SOIL AND GROUNDWATER CONDITIONS

5.1 Alluvium

Each of our soil borings encountered alluvial deposits. The alluvial materials were observed as stiff to hard clays with variable amounts of silt, sand and gravels and medium dense gravelly sands with clay and silt. The alluvium extended to depth of approximately 13 ½ feet or less at our soil boring locations.

5.2 Santa Clara Formation

Geologic references map late Pliocene to Pleistocene-age Santa Clara Formation at the site. Based our exploratory borings and site observations, Santa Clara Formation comprises the hillside at the southern margin of the site and underlies the alluvium in the lower, flatter portion of the site. As encountered in our borings, the formational materials were observed as dense to very dense, gravelly sands with variable amounts of clay and hard, sandy to gravelly clays. Our soil borings encountered alluvium to the maximum depth explored – approximately 26 ½ feet below existing grade. The sandstone encountered at the bottom of our Boring B4 may be an older formational unit but the distinction has not been made for the purposes of this study.

5.3 Groundwater

Groundwater was not encountered in our soil borings. Historic high groundwater levels in the general site vicinity are less than 20 feet below existing grade based on CGS mapping. Actual groundwater levels will fluctuate seasonally and with variations in rainfall, temperature and other factors and may be higher or lower than observed during our study.
6. CONCLUSIONS AND RECOMMENDATIONS

6.1 General

6.1.1 No overriding geotechnical constraints were encountered during our investigation that would preclude the project as presently proposed. Primary geotechnical considerations are the potential for liquefaction-related settlements, site seismicity and the stability of the hillside that ascends the southern margin of the site.

6.1.2 The proposed project redevelops a site with past episodes of grading and construction. As such, unknown underground improvements and areas of undocumented fill materials (not discussed herein) may be present. If encountered, supplemental recommendations will be provided during site development.

6.1.3 As discussed in Section 4.3, the site is susceptible to liquefaction. Our analysis indicates that, if liquefaction and/or cyclic softening were to occur, total ground surface settlements on the order of ½ inch may result. In our experience, this amount of settlement should not require special structural design considerations or special mitigation measures. In addition to settlements from structural loading, the design of structures and site improvements should accommodate seismically-induced differential settlements of ½ inch over a horizontal distance of 50 feet.

6.1.4 Where shallow foundation systems are designed and constructed as recommended herein, post-construction settlement due to dead + live loads would be on the order of ¾ inch or less with differential settlements of approximately ½ inch across a horizontal distance of 50 feet.

6.1.5 Any changes in the design, location or elevation of the proposed improvements, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

6.1.6 All references to relative compaction and optimum moisture content in this report are based on ASTM D 1557 (latest edition).

6.2 Seismic Design Criteria

6.2.1 We understand that seismic structural design will be performed in accordance with the provisions of the 2016 CBC which is based on the American Society of Civil Engineers (ASCE) publication Minimum Design Loads for Buildings and Other Structures (ASCE 7-10). We used the USGS web-based application US Seismic Design Maps to evaluate site-specific seismic design parameters in accordance with the 2016 CBC and ASCE 7-10. Results are summarized in Table 6.2.1. The values presented are for the risk-targeted maximum considered earthquake (MCE_R).
TABLE 6.2.1
2016 CBC SEISMIC DESIGN PARAMETERS

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>2016 CBC / ASCE 7-10 Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site Class</td>
<td>D</td>
<td>Section 1613.3.2/Table 20.3-1</td>
</tr>
<tr>
<td>MCE&lt;sub&gt;d&lt;/sub&gt; Ground Motion Spectral Response Acceleration – Class B (short), S&lt;sub&gt;S&lt;/sub&gt;</td>
<td>2.706g</td>
<td>Figure 1613.3.1(1) / Figure 22-1</td>
</tr>
<tr>
<td>MCE&lt;sub&gt;d&lt;/sub&gt; Ground Motion Spectral Response Acceleration – Class B (1 sec), S&lt;sub&gt;1&lt;/sub&gt;</td>
<td>1.009g</td>
<td>Figure 1613.3.1(2) / Figure 22-2</td>
</tr>
<tr>
<td>Site Coefficient, F&lt;sub&gt;A&lt;/sub&gt;</td>
<td>1.0</td>
<td>Table 1613.3.3(1) / Table 11.4-1</td>
</tr>
<tr>
<td>Site Coefficient, F&lt;sub&gt;V&lt;/sub&gt;</td>
<td>1.5</td>
<td>Table 1613.3.3(2) / Table 11.4-2</td>
</tr>
<tr>
<td>Site Class Modified MCE&lt;sub&gt;d&lt;/sub&gt; Spectral Response Acceleration (short), S&lt;sub&gt;M&lt;/sub&gt;&lt;sub&gt;S&lt;/sub&gt;</td>
<td>2.706g</td>
<td>Eq. 16-37 / Eq. 11.4-1</td>
</tr>
<tr>
<td>Site Class Modified MCE&lt;sub&gt;d&lt;/sub&gt; Spectral Response Acceleration (1 sec), S&lt;sub&gt;M&lt;/sub&gt;&lt;sub&gt;1&lt;/sub&gt;</td>
<td>1.513g</td>
<td>Eq. 16-38 / Eq. 11.4-2</td>
</tr>
<tr>
<td>5% Damped Design Spectral Response Acceleration (short), S&lt;sub&gt;D&lt;/sub&gt;&lt;sub&gt;S&lt;/sub&gt;</td>
<td>1.804g</td>
<td>Eq. 16-39 / Eq. 11.4-3</td>
</tr>
<tr>
<td>5% Damped Design Spectral Response Acceleration (1 sec), S&lt;sub&gt;D&lt;/sub&gt;&lt;sub&gt;1&lt;/sub&gt;</td>
<td>1.009g</td>
<td>Eq. 16-40 / Eq. 11.4-4</td>
</tr>
</tbody>
</table>

6.2.2 Table 6.2.2 presents additional seismic design parameters for projects with Seismic Design Categories of D through F in accordance with ASCE 7-10 for the mapped maximum considered geometric mean (MCE<sub>G</sub>).

TABLE 6.2.2
2016 CBC SITE ACCELERATION DESIGN PARAMETERS

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>ASCE 7-10 Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mapped MCE&lt;sub&gt;d&lt;/sub&gt; Peak Ground Acceleration, PGA</td>
<td>1.024g</td>
<td>Figure 22-7</td>
</tr>
<tr>
<td>Site Coefficient, F&lt;sub&gt;PGA&lt;/sub&gt;</td>
<td>1.0</td>
<td>Table 11.8-1</td>
</tr>
<tr>
<td>Site Class Modified MCE&lt;sub&gt;d&lt;/sub&gt; Peak Ground Acceleration, PGA&lt;sub&gt;M&lt;/sub&gt;</td>
<td>1.024g</td>
<td>Section 11.8.3 (Eq. 11.8-1)</td>
</tr>
</tbody>
</table>

6.2.3 Conformance to the criteria presented in Tables 6.2.1 and 6.2.2 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a maximum level earthquake occurs. The primary goal of seismic design is to protect life and not to avoid structural damage, since such design may be economically prohibitive.

6.3 Soil and Excavation Characteristics

6.3.1 The onsite alluvial soils can be excavated with moderate effort using conventional excavation. We do not anticipate excavations in the native alluvium or Santa Clara Formation at the site will generate oversize material (greater than 6 inches in nominal dimension). However, unknown or
unanticipated constituents may exist, especially within areas of artificial fill. Any artificial fills at the site are undocumented and may contain constituents not reported herein. Below-grade improvements associated with prior site development may also be present.

6.3.2 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable Occupational Safety and Health Administration (OSHA) rules and regulations to maintain safety and maintain the stability of adjacent existing improvements.

6.3.3 The soils encountered at in our soil borings are not considered “expansive” as defined by 2016 CBC based on our laboratory testing. The recommendations of this report assume proposed foundation systems will derive support in engineered fills or competent alluvium or Santa Clara Formation.

6.4 Materials for Fill

6.4.1 Soils generated from cut operations or foundation excavations at the site are suitable for use as engineered fill in structural areas provided they do not contain deleterious matter, organic material, or cementations larger than 6 inches in maximum dimension. Excavated soils may be wet and require drying prior to use and engineered fill.

6.4.2 Import fill material should be primarily granular with a “low” expansion potential (Expansion Index less than 50), a Plasticity Index less than 15, be free of organic material and construction debris, and not contain rock larger than 6 inches in greatest dimension.

6.4.3 Environmental characteristics and corrosion potential of import soil materials may also be considered. Proposed import materials should be sampled, tested, and approved by Geocon prior to its transportation to the site.

6.5 Grading

6.5.1 All clearing operations and earthwork (including over-excavation, scarification, and recompaement) should be observed and all fills tested for recommended compaction and moisture content by representatives of Geocon.

6.5.2 Structural areas should be considered as areas extending a minimum of 5 feet horizontally from a foundation or beyond the outside dimensions of buildings, including footings and overhangs carrying structural loads, and where not restricted by property boundaries.

6.5.3 A preconstruction conference should be held at the site prior to the beginning of grading operations with the owner, contractor, civil engineer and geotechnical engineer in attendance. Special soil handling requirements can be discussed at that time.

6.5.4 After complete demolition and removal of existing structures, site preparation should commence with the removal of all existing improvements from the area to be developed/graded. All active or inactive utilities within the construction area should be protected, relocated, or abandoned. Any pipelines to be abandoned that are greater than 2 inches and less than 18 inches in diameter should be removed or filled with sand-cement slurry. Utilities larger than 18 inches in diameter should be removed. Excavations or depressions resulting from site clearing operations, or other
existing excavations or depressions, should be restored with engineered fill in accordance with the recommendations of this report.

6.5.5 Existing soils in areas to receive fill or remain near current grades should be over-excavated at least one foot below existing grade, not including the thickness of existing pavement section components or slabs. The resultant over-excavation bottoms should then be scarified to a depth of approximately 1 foot, moisture conditioned to near to slightly above optimum moisture and recompaクトed to at least 90% relative compaction. Subgrade for building pads or exterior slabs should be scarified to a depth of approximately 1 foot, moisture conditioned to near to slightly above optimum and recompaクトed to at least 90% relative compaction. Subgrade in pavement areas should be scarified to a depth of approximately 1 foot, moisture conditioned to near to slightly above optimum moisture and recompaクトed to at least 95% relative compaction.

6.5.6 In general, over-excavated materials may be used for new engineered fill provided they do not contain deleterious matter, organic material, or cementations larger than 6 inches in maximum dimension. Over-excavations and the exposed bottom surfaces and bottom processing should be observed by our representatives. Supplemental recommendations may be provided based on site conditions during grading. Areas of deeper over-excavation may be required.

6.5.7 All structural fill and backfill should be placed in layers no thicker than will allow for adequate bonding and compaction (typically 8 to 12 inches). Fill soils should be placed and compacted to at least 90% relative compaction near to slightly above optimum moisture. Fill areas with in-place density tests showing moisture contents less than those recommended will require additional moisture conditioning prior to placing additional fill.

6.6 Shallow Foundation Recommendations

6.6.1 The proposed residential structures and ancillary site structures such as short retaining walls, screen walls, or trash enclosures may utilize conventional foundations consisting of continuous strip footings founded in competent native alluvial materials or properly compacted fill. The following recommendations are based on the assumption that the soils within 5 feet of finish grade will consist of low expansive materials (Expansion Index less than 50). Over-excavations may be required if soft or loose soils are encountered in footing excavations.

6.6.2 It is recommended that conventional continuous footings have a minimum embedment depth of 18 inches below lowest adjacent pad grade. The footings should be at least 12 inches wide.

6.6.3 Footings proportioned as recommended may be designed for an allowable soil bearing pressure of 3,000 pounds per square foot (psf). The allowable bearing pressure is for dead + live loads may be increased by up to one-third for transient loads due to wind or seismic forces.

6.6.4 The allowable passive pressure used to resist lateral movement of the footings may be assumed to be equal to a fluid weighing 300 pounds per cubic foot (pcf). Where not protected by pavement or flatwork, the upper one foot of soil should be ignored when calculating passive resistance. The allowable coefficient of friction to resist sliding is 0.30 for concrete against soil. Combined passive resistance and friction may be utilized for design provided that the frictional resistance is reduced by 50%.

6.6.5 Minimum reinforcement for continuous footings should consist of four No. 4 steel reinforcing bars; two placed near the top of the footing and two near the bottom.
6.6.6 The foundation dimensions and minimum reinforcement recommendations presented herein are based upon soil conditions only and are not intended to be used in lieu of those required for structural purposes.

6.6.7 Underground utilities running parallel to footings should not be constructed in the zone of influence of footings. The zone of influence may be taken to be the area beneath the footing and within a 1:1 plane extending out and down from the bottom edge of the footing.

6.6.8 The foundation subgrade should be sprinkled as necessary to maintain a moist condition without significant shrinkage cracks as would be expected in any concrete placement. Our representative should observe all footing excavations prior to placing reinforcing steel.

6.7 Post-Tensioned Foundation Recommendations

6.7.1 Post-tensioned foundations may be used to support the proposed residential structures and should be designed by a structural engineer experienced in post-tensioned slab design and design criteria of the Post-Tensioning Institute (PTI), Third Edition. The post-tensioned design should incorporate the geotechnical parameters presented on the table below. The parameters presented are based on the guidelines presented in the PTI, Third Edition design manual.

<table>
<thead>
<tr>
<th>TABLE 6.7</th>
<th>POST-TENSIONED FOUNDATION SYSTEM DESIGN PARAMETERS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Post-Tensioning Institute (PTI), Third Edition Design Parameters</td>
<td>Recommended Value</td>
</tr>
<tr>
<td>Equilibrium Suction</td>
<td>3.6</td>
</tr>
<tr>
<td>Edge Lift Moisture Variation Distance, eM (feet)</td>
<td>5.1</td>
</tr>
<tr>
<td>Edge Lift, yM (inches)</td>
<td>0.64</td>
</tr>
<tr>
<td>Center Lift Moisture Variation Distance, eM (feet)</td>
<td>9.0</td>
</tr>
<tr>
<td>Center Lift, yM (inches)</td>
<td>0.76</td>
</tr>
</tbody>
</table>

6.7.2 Post-tensioned foundations should be embedded in accordance with the recommendations of the structural engineer. If a post-tensioned mat foundation system is planned, the slab should possess a thickened edge with a minimum width of 12 inches. The thickened edge should extend below the crushed rock underlayment layer.

6.7.3 The thickness of post-tensioned foundation systems should be determined by the project structural engineer. Based on our experience with similar projects and soils conditions, we anticipate the post-tensioned slab thicknesses will be on the order of 10 to 12 inches.

6.7.4 Our experience indicates that post-tensioned slabs are susceptible to excessive edge lift, regardless of the underlying soil conditions. Placing reinforcing steel at the bottom of the perimeter footings and the interior stiffener beams may mitigate this potential. Current PTI design procedures primarily address the potential center lift of slabs but, because of the placement of the reinforcing tendons in the top of the slab, the resulting eccentricity after tensioning reduces the ability of the system to mitigate edge lift. The structural engineer should design the foundation system to reduce the potential of edge lift occurring for the proposed structures.
6.7.5 During the construction of the post-tension foundation system, the concrete should be placed monolithically. Under no circumstances should cold joints be allowed to form between the footings/grade beams and the slab during the construction of the post-tension foundation system.

6.7.6 The use of isolated footings, which are located beyond the perimeter of the building and support structural elements connected to the building, are not recommended. Where this condition cannot be avoided, the isolated footings should be connected and tied to the building foundation system with grade beams.

6.7.7 Consideration should be given to connecting patio slabs to the building foundation to reduce the potential for future separation to occur.

6.7.8 Post-tensioned slabs should be underlain by at least 3 inches of ½-inch or ¾-inch crushed rock with no more than 5 percent passing the No. 200 sieve to serve as a capillary break.

6.7.9 Subgrade for post-tensioned foundations should be tested immediately prior to placing underlayment materials (crushed rock and vapor barrier) to verify that subgrade moisture content is appropriate.

6.8 Temporary Excavations

6.8.1 The native alluvium and Santa Clara Formation can be considered a Type B soil in accordance with OSHA guidelines. If free water, sandy or cohesionless soils or undocumented fills are encountered the materials should be downgraded to Type C. The contractor should have a “competent person” as defined by OSHA evaluate all excavations. All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping and possibly shoring.

6.8.2 It is the contractor’s responsibility to provide sufficient and safe excavation support as well as protecting nearby utilities, structures, and other improvements which may be damaged by earth movements.

6.9 Underground Utilities

6.9.1 Underground utility trenches should be backfilled with properly compacted material. The material excavated from the trenches should be adequate for use as backfill provided it does not contain deleterious matter, vegetation or rock larger than six inches in maximum dimension. Trench backfill should be placed in loose lifts not exceeding eight inches and should be compacted to at least 90% relative compaction at least 2% above optimum moisture content (near optimum where backfill materials are predominantly sands and gravels).

6.9.2 Bedding and pipe zone backfill typically extends from the bottom of the trench excavations to a minimum of 6 inches above the crown of the pipe. Pipe bedding material should consist of crushed aggregate, clean sand or similar open-graded material. Proposed bedding and pipe zone materials should be reviewed by Geocon prior to construction; open-graded materials such as ⅛ inch drain rock may require wrapping with filter fabric to mitigate the potential for piping. Pipe bedding and backfill should also conform to the requirements of the governing utility agency.
6.10 Concrete Slabs-on-Grade

6.10.1 Exterior concrete slabs-on-grade subject to vehicle loading are considered pavements should be designed in accordance with the recommendations in Section 6.12 of this report.

6.10.2 Concrete slabs-on-grade for structures, not subject to vehicle loading, should be a minimum of 4 inches thick and minimum slab reinforcement should consist of No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Steel reinforcing should be positioned vertically near the slab midpoint.

6.10.3 Interior slabs or slabs in areas where moisture would be objectionable should be underlain by 4 inches of ½-inch or ¾-inch crushed rock with no more than 5% passing the No. 200 sieve to serve as a capillary break.

6.10.4 Exterior slabs, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Consideration may be given to providing at least four inches of Class 2 Aggregate Base (AB) compacted to at least 90% relative compaction below exterior concrete slabs to provide a more uniform support characteristic. Although site soils are generally not expansive, the provision of an underlying AB layer can reduce cosmetic cracking in the slabs. Prior to placing AB or rebar for the exterior slabs, the subgrade should be moisture conditioned to near optimum and properly compacted to at least 90% relative compaction.

6.10.5 In lieu of specific recommendations from the structural or civil engineer, we recommend that crack control joints be spaced at intervals not greater than 8 feet for 4-inch-thick slabs. Crack control joints should extend a minimum depth of one-fourth the slab thickness and should be constructed using saw-cuts or other methods as soon as practical after concrete placement. Construction joints should be designed by the project structural engineer.

6.10.6 The recommendations of this report are intended to reduce the potential for cracking of slabs due to soil movement. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade may exhibit some cracking due to soil movement. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

6.11 Moisture Protection Considerations

6.11.1 A vapor barrier is not required beneath interior slabs-on-grade for geotechnical purposes. Further, the migration of moisture through concrete slabs or moisture otherwise released from slabs is not a geotechnical issue. However, for the convenience of the owner, we are providing the following general suggestions for consideration by the owner, architect, structural engineer, and contractor. The suggested procedures may reduce the potential for moisture-related floor covering failures on concrete slabs-on-grade, but moisture problems may still occur even if the procedures are followed. If more detailed recommendations are desired, we recommend consulting a specialist in this field. If a vapor barrier is used beneath mat slab foundations, friction between the mat slab and underlying substrate when evaluating lateral loading resistance.
6.11.2 A vapor barrier meeting ASTM E 1745-09 Class C requirements may be placed directly below the slab, without a sand cushion. To reduce the potential for punctures, a higher quality vapor barrier (15 mil, Class A or B) should be used. The vapor barrier, if used, should extend to the edges of the slab, and should be sealed at all seams and penetrations.

6.11.3 The concrete water/cement ratio should be as low as possible. The water/cement ratio should not exceed 0.45 for concrete placed directly on the vapor barrier. Midrange plasticizers could be used to facilitate concrete placement and workability.

6.11.4 Proper finishing, curing, and moisture vapor emission testing should be performed in accordance with the latest guidelines provided by the American Concrete Institute, Portland Cement Association, and ASTM.

6.12 Pavement Recommendations

6.12.1 The upper 12 inches of pavement subgrade should be scarified, moisture conditioned to near to slightly above optimum and compacted to at least 95% relative compaction. Prior to placing aggregate base, the finished subgrade should be proof-rolled with a laden water truck (or similar equipment with high contact pressure) to verify stability.

6.12.2 Sidewalk, curb, gutter, and driveway encroachments should be designed and constructed in accordance with City of Monte Sereno requirements, as applicable.

6.12.3 We recommend the following asphalt concrete (AC) pavement sections for design to establish subgrade elevations in pavement areas. The project civil engineer should determine the appropriate Traffic Index (TI) based on anticipated traffic conditions. The flexible pavement sections below are based on estimated design TIs and an assumed R-Value of 5 for the subgrade soils. We can provide additional sections based on other TIs if necessary. The project civil engineer should confirm the TIs estimated herein. Supplemental soil sampling and laboratory R-value testing could allow reductions in the section thicknesses tabulated below.

<table>
<thead>
<tr>
<th>Location</th>
<th>Estimated Traffic Index (TI)</th>
<th>AC Thickness (inches)</th>
<th>Class 2 AB Thickness (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parking Stalls</td>
<td>4.5</td>
<td>3</td>
<td>8</td>
</tr>
<tr>
<td>Driveways</td>
<td>6.0</td>
<td>3 ½</td>
<td>12 ½</td>
</tr>
<tr>
<td>Heavy-Duty</td>
<td>7.0</td>
<td>4</td>
<td>15 ½</td>
</tr>
</tbody>
</table>

Note: The recommended flexible pavement sections are based on the following assumptions:

1. AB: Class 2 AB with a minimum R-Value of 78 and meeting the requirements of Section 26 of the latest Caltrans Standard Specifications.
2. AB is compacted to 95% or higher relative compaction at or near optimum moisture content. Prior to placing AB, the subgrade should be proof-rolled with a loaded water truck to verify stability.
3. AC: Asphalt concrete conforming to local agency standards or Section 39 of the latest Caltrans Standard Specifications.
6.12.4 The AC sections in Table 6.12.3 are final, minimum thicknesses. If staged-pavements are used, the construction bottom AC lift should be at least 2 inches thick. Following construction, the finish top AC lift should be at least 1½ inches thick.

6.12.5 We understand that pervious asphalt pavements will be used in some areas of the project. The following pervious pavement sections were determined in general accordance with Caltrans’ design methodology. The sections below are based on estimated design TIs and an assumed R-Value of 5 for the subgrade soils. The project civil engineer should confirm the TIs estimated herein. We can provide additional sections based on other TIs if necessary.

<table>
<thead>
<tr>
<th>Location</th>
<th>Estimated Traffic Index (TI)</th>
<th>OGFC Thickness (feet)</th>
<th>ATPB Thickness (feet)</th>
<th>Class 4 ASB Thickness (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parking Stalls</td>
<td>4.5</td>
<td>0.1</td>
<td>0.25</td>
<td>1.25</td>
</tr>
<tr>
<td>Driveways</td>
<td>6.0</td>
<td>0.1</td>
<td>0.40</td>
<td>1.50</td>
</tr>
</tbody>
</table>

Notes:
1. OGFC: Open Graded Friction Course per Section 39 of the latest Caltrans Standard Specifications.
2. ATPB: Asphalt Treated Permeable Base per Section 29 of the latest Caltrans Standard Specifications.
3. Class 4 ASB: Class 4 Aggregate Subbase per Section 25 of the latest Caltrans Standard Specifications. Class 4 ASB should be compacted to 95% or higher relative compaction at or near optimum moisture content.
4. The Class 4 ASB thicknesses in Table 6.12.5 are for structural support only and are not intended to represent minimum required thicknesses for reservoir storage, which should be determined by the project civil engineer.
5. Caltrans generally recommends that pervious pavements be used in cut areas, and not in areas where fills are required to attain subgrade elevation.
6. Pervious pavements should be designed and constructed per Caltrans requirements.

6.12.6 Unless specifically designed and evaluated by the project structural engineer, where concrete paving will be utilized for support of vehicles, we recommend the concrete be a minimum of 6 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. In addition, doweling, reinforcing steel or other load-transfer mechanism should be provided at joints if desired to reduce the potential for vertical offset. The concrete should have a minimum 28-day compressive strength of 3,500 psi. We should evaluate pavements to support heavy truck traffic on a case-by-case basis; supplemental recommendations may be provided.

6.12.7 We recommend that at least 6 inches of Class 2 Aggregate Base be used below rigid concrete pavements. The aggregate base should be compacted to at least 95% relative compaction near optimum moisture content.

6.12.8 In general, we recommend that concrete pavements be designed, constructed and maintained in accordance with industry standards such as those provided by the American Concrete Pavement Association.
6.12.9 Crack control joints should be spaced at intervals not greater than 12 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness and should be constructed using saw-cuts or other methods as soon as practical after concrete placement. Construction joints should be designed by the project structural engineer.

6.12.10 The performance of pavements is highly dependent upon providing positive surface drainage away from the edge of pavements. Ponding of water on or adjacent to the pavement will likely result in saturation of the subgrade materials and subsequent cracking, subsidence and pavement distress. If planters are planned adjacent to paving, it is recommended that the perimeter curb be extended at least 6 inches below the bottom of the aggregate base to minimize the introduction of water beneath the paving. Alternatives such as plastic moisture cut-offs or modified drop-inlets may also be considered in lieu of deepened curbs.

6.13 Retaining Wall Design

6.13.1 Lateral earth pressures may be used in the design of retaining walls and buried structures. Lateral earth pressures against these facilities may be assumed to be equal to the pressure exerted by an equivalent fluid. The unit weight of the equivalent fluid depends on the design conditions. Table 6.13 summarizes the weights of the equivalent fluid based on the different design conditions.

<table>
<thead>
<tr>
<th>Condition</th>
<th>Equivalent Fluid Density</th>
</tr>
</thead>
<tbody>
<tr>
<td>Active</td>
<td>40 pcf</td>
</tr>
<tr>
<td>At-Rest</td>
<td>60 pcf</td>
</tr>
</tbody>
</table>

6.13.2 Unrestrained walls should be designed using the active case. Unrestrained walls are those that are allowed to rotate more than 0.001H (where H is the height of the wall). Walls restrained from movement such as basement walls should be designed using the at-rest case. The above soil pressures assume level backfill under drained conditions within an area bounded by the wall and a 1:1 plane extending upward from the base of the wall and no surcharges within that same area. Where backfill surfaces will be inclined at 2:1 or flatter, an additional 15 pcf should be added to the equivalent fluid density values in Table 6.13.

6.13.3 Unless project-specific loading information is provided by the structural engineer, where vehicle loads are expected atop the wall backfill, an additional uniform surcharge pressure equivalent to 2 feet of backfill soil should be used for design. Where the vehicle loading will be limited to passenger cars, the additional uniform surcharge equivalent may be reduced to 1 foot of backfill soil.

6.13.4 Retaining walls greater than 2 feet tall (retained height) should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces and should be waterproofed as required by the project architect. Positive drainage for retaining walls should consist of a vertical layer of permeable material positioned between the retaining wall and the soil backfill. The permeable material may be composed of a composite drainage geosynthetic or a natural
permeable material such as crushed gravel at least 12 inches thick and capped with at least 12 inches of native soil. A geosynthetic filter fabric should be placed between the gravel and the soil backfill. Provisions for removal of collected water should be provided for either system by installing a perforated drainage pipe along the bottom of the permeable material which leads to suitable drainage facilities.

6.13.5 We recommend that all retaining wall designs be reviewed by Geocon to confirm the incorporation of the recommendations provided herein. In particular, potential surcharges from adjacent structures and other improvements should be reviewed by Geocon.

6.14 Surface Drainage

6.14.1 Proper surface drainage is critical to the future performance of the project. Uncontrolled infiltration of irrigation excess and storm runoff into the soils can adversely affect the performance of the planned improvements. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change to important engineering properties. Proper drainage should be maintained at all times.

6.14.2 All site drainage should be collected and transferred in accordance with an engineered storm water management plan, or to facilities designed by the project civil engineer, in non-erosive drainage devices. Drainage should not be allowed to pond near any foundations or retaining walls. Drainage should not be allowed to flow uncontrolled over any descending slope. The proposed structures should be provided with roof gutters. Discharge from downspouts, roof drains and scuppers not permitted onto unprotected soils within five feet of the building perimeter. Planters which are located adjacent to foundations should be sealed or properly drained to prevent moisture intrusion into the materials providing foundation support. Landscape irrigation within five feet of the building perimeter footings should be kept to a minimum to just support vegetative life.

6.14.3 Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures. The building pad and pavement areas should be fine graded such that water is not allowed to pond. Final soil grade should slope a minimum of 2% away from structures.

6.14.4 We recommend implemented measures to reduce infiltrating surface water near buildings and slabs-on-grade. Such measures may include:

- Selecting drought-tolerant plants that require little or no irrigation, especially within 5 feet of buildings, slabs-on-grade, or pavements.
- Using drip irrigation or low-output sprinklers.
- Using automatic timers for irrigation systems.
- Appropriately spaced area drains.
- Hard-piping roof downspouts to appropriate collection facilities.
7. FURTHER GEOTECHNICAL SERVICES

7.1 Plan and Specification Review

7.1.1 We should review project plans and specifications prior to final design submittal to assess whether our recommendations have been properly implemented and evaluate if additional analysis and/or recommendations are required.

7.2 Testing and Observation Services

7.2.1 The recommendations provided in this report are based on the assumption that we will continue as Geotechnical Engineer of Record throughout the construction phase and provide compaction testing and observation services and foundation observations throughout the project. It is important to maintain continuity of geotechnical interpretation and confirm that field conditions encountered are similar to those anticipated during design. If we are not retained for these services, we cannot assume any responsibility for others' interpretation of our recommendations, and therefore the future performance of the project.
The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon Consultants, Inc. should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the geotechnical scope of services provided by Geocon Consultants, Inc.

This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate 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, this report is subject to review and should not be relied upon after a period of three years.

Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices used in the site area at this time. No warranty is provided, express or implied.
LEGEND:

- **B4**: Approximate Boring Location
- **CPT3**: Approximate Cone Penetrometer Test (CPT) Location
- **A**: Approximate Cross-Section Location

Scale in Feet

0 80
APPENDIX A
FIELD EXPLORATION

Fieldwork for our investigation included a site visit, subsurface exploration, and soil sampling. The locations of our exploratory borings and CPTs are shown on the Site Plan, Figure 2. Soil boring logs and CPT profiles for our exploration are presented as figures following the text in this appendix. The borings and CPTs were located by pacing from existing reference points. Therefore, the exploration locations shown on Figure 2 are approximate.

Our field exploration included the advancement of three CPT soundings to maximum depths of approximately 60 feet below the existing ground surface utilizing a truck-mounted CPT rig with a down-pressure capacity of approximately 20 tons. The CPTs were performed on March 12, 2018 by Middle Earth Geo Testing of Fremont, California using an integrated electronic cone system. The cone has a tip area of 10 square centimeters, a friction sleeve area of 150 square centimeters, and a ratio of friction sleeve area to tip end area equal to 0.85. The cone bearing ($Q_c$) and sleeve friction ($F_s$) were measured and recorded during tests at approximately 2-inch depth intervals. The CPT data consisting of cone bearing, sleeve friction, friction ratio and equivalent standard penetration blow counts (N) versus penetration depth below the existing ground surface for each location has been recorded and is presented in this appendix.

Our borings were performed on March 16, 2018 using a truck-mounted Mobile B-61 drill rig equipped with 8-inch hollow-stem augers. Sampling in the borings was accomplished using a down-hole wire-line 140-pound hammer with a 30-inch drop. Samples were obtained with a 3-inch outside-diameter (OD), split spoon (California Modified) sampler, and a 2-inch OD, Standard Penetration Test (SPT) sampler. The number of blows required to drive the sampler the last 12 inches (or fraction thereof) of the 18-inch sampling interval were recorded on the boring logs. The blow counts shown on the boring logs should not be interpreted as standard SPT “N” values; corrections have not been applied.

Subsurface conditions encountered in the exploratory boring were visually examined, classified and logged in general accordance with the American Society for Testing and Materials (ASTM) Practice for Description and Identification of Soils (Visual-Manual Procedure D2488). This system uses the Unified Soil Classification System (USCS) for soil designations. The log depicts soil and geologic conditions encountered and depths at which samples were obtained. The log also includes our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, drill rig penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the field logs were revised based on subsequent laboratory testing.

Upon completion, our hollow-stem auger borings were backfilled with a mixture of lean cement grout and compacted soil cuttings. Our CPTs were backfilled in accordance with Santa Clara Valley Water District permit requirements.
### Unified Soil Classification

<table>
<thead>
<tr>
<th>Major Divisions</th>
<th>Typical Names</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravels</td>
<td>Gravel With or Without Sand</td>
<td>Well-graded gravel with or without sand</td>
</tr>
<tr>
<td>Sands</td>
<td>Sands With or Without Gravel</td>
<td>Well-graded sands with or without gravel</td>
</tr>
<tr>
<td>Silts and Clays</td>
<td>Silts and Clays</td>
<td>Silts and very fine clays</td>
</tr>
<tr>
<td>Highly Organic Soils</td>
<td>Peat and Other Highly Organic Soils</td>
<td>Organic material throughout</td>
</tr>
</tbody>
</table>

### Bedding Spacing Descriptions

<table>
<thead>
<tr>
<th>Thickness/Spacing</th>
<th>Descriptor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Greater Than 10 ft</td>
<td>Very thinly bedded</td>
</tr>
<tr>
<td>5 to 10 ft</td>
<td>Moderately bedded</td>
</tr>
<tr>
<td>1 to 5 ft</td>
<td>Thickly bedded</td>
</tr>
<tr>
<td>1 inch to 2 inches</td>
<td>Moderately bedded</td>
</tr>
<tr>
<td>1 inch to 2 inches</td>
<td>Thickly bedded</td>
</tr>
<tr>
<td>Less Than 1 inch</td>
<td>Very thickly bedded</td>
</tr>
</tbody>
</table>

### Structure Descriptions

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alternating Layers of Varying Material or Color with Layers at Least 1 inch Thick</td>
<td>Alternating Layers</td>
</tr>
<tr>
<td>Bends Along Definite Planes of Fracture with Little Resistance</td>
<td>Moderately bedded</td>
</tr>
<tr>
<td>Fracture Planes Appear Polished or Glossy, Sometimes Stratified</td>
<td>Bifurcated</td>
</tr>
<tr>
<td>Cohesive Soil that Can Be Broken Down into Smaller Angular Lumps Which Resist Further Breakdown</td>
<td>Buggy</td>
</tr>
<tr>
<td>Inclusion of Small Pockets of Different Soil, Such as Small Lenses of Sand Scattered Through a Mass of Clay</td>
<td>Laminated</td>
</tr>
<tr>
<td>Same Color and Material Throughout</td>
<td>Heterogeneous</td>
</tr>
</tbody>
</table>

### Cementation/Induration Descriptions

<table>
<thead>
<tr>
<th>Field Test</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crumbles or Breaks with Hammer or Little Finger Pressure</td>
<td>Weakly cemented/indurated</td>
</tr>
<tr>
<td>Crumbles or Breaks with Considerable Finger Pressure</td>
<td>Moderately cemented/indurated</td>
</tr>
<tr>
<td>Will Not Crumble or Break with Finger Pressure</td>
<td>Strongly cemented/indurated</td>
</tr>
</tbody>
</table>

### Igneous/Metamorphic Rock Strength Descriptions

<table>
<thead>
<tr>
<th>Degree of Decomposition</th>
<th>Field Recognition</th>
<th>Engineering Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>SOIL</td>
<td>DISCOLORED, CHANGED TO SOIL, FABRIC DESTROYED</td>
<td>EASY TO DIG</td>
</tr>
<tr>
<td>COMPLETELY WEATHERED</td>
<td>DISCOLORED, CHANGED TO SOIL, FABRIC PARTLY PRESERVED</td>
<td>EXCAVATED BY HAND OR SHOVELING</td>
</tr>
<tr>
<td>HIGHLY WEATHERED</td>
<td>DISCOLORED, HIGHLY FRAGMENTED, FABRIC ALTERED AROUND FRACtURES</td>
<td>EXCAVATED BY HAND OR HAMMER, HARD TO DIG</td>
</tr>
<tr>
<td>MODERATELY WEATHERED</td>
<td>DISCOLORED, FRACTURED, INTACT ROCK NOTICABLEY WEAKER THAN FRESH ROCK</td>
<td>EXCAVATED WITH DIGGING TOOLS AND CHISELS, FRAGMENTS WEAKENED</td>
</tr>
<tr>
<td>SLIGHTLY WEATHERED</td>
<td>MAY BE DISCOLORED, SOME FRACtURES, INTACT ROCK NOTICABLEY WEAKER THAN FRESH ROCK</td>
<td>EXPLOSIONS FOR EXCAVATION, WEAKENED JOINTS AND FRACtURES</td>
</tr>
<tr>
<td>FRESH</td>
<td>NO DISCOLORED, OR LOSS OF STRENGTH</td>
<td>REQUIRES EXPLOSIONS</td>
</tr>
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</table>

### Igneous/Metamorphic Rock Weathering Descriptions

<table>
<thead>
<tr>
<th>Field Test</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>No Observed Fractures</td>
<td>Unfractured/Unjointed</td>
</tr>
<tr>
<td>Majority of Joint/Fractures Spaced at 1 to 3 ft Intervals</td>
<td>Slightly Fractured/Jointed</td>
</tr>
<tr>
<td>Majority of Joint/Fractures Spaced at 4 to 8 ft Intervals</td>
<td>Moderately Fractured/Jointed</td>
</tr>
<tr>
<td>Majority of Joint/Fractures Spaced at 1 ft to 8 ft Intervals with Scattered Fragments</td>
<td>Intensely Fractured/Jointed</td>
</tr>
<tr>
<td>Majority of Structures Spaced at Less than 1 ft Intervals; Mostly Recovered as Chips and Fragments</td>
<td>Very Intensely Fractured/Jointed</td>
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</tbody>
</table>

### Gravel/Cobble/Boulder Descriptions

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pass a 3/4 inch sieve and be retained on a No. 4 sieve (4 to 57)</td>
<td>Gravel</td>
</tr>
<tr>
<td>Pass a 1/4-inch square opening and be retained on a 1/4-inch sieve (0.125)</td>
<td>Cobble</td>
</tr>
<tr>
<td>Will not pass a 1/2-inch square opening (0.125)</td>
<td>Boulder</td>
</tr>
<tr>
<td>DEPTH IN FEET</td>
<td>SAMPLE NO.</td>
</tr>
<tr>
<td>---------------</td>
<td>------------</td>
</tr>
<tr>
<td>0</td>
<td>B1-1-5</td>
</tr>
<tr>
<td>1</td>
<td>B1-2-5-3</td>
</tr>
<tr>
<td>2</td>
<td>B1-3</td>
</tr>
<tr>
<td>5</td>
<td>B1-5-5.5</td>
</tr>
<tr>
<td>6</td>
<td>B1-5.5</td>
</tr>
<tr>
<td>7</td>
<td>B1-8.5-9</td>
</tr>
<tr>
<td>9</td>
<td>B1-9</td>
</tr>
<tr>
<td>15</td>
<td>B1-14-15</td>
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</table>

END OF BORING AT APPROXIMATELY 15 FEET
NO FREE WATER ENCOUNTERED
BACKFILLED WITH COMPACTED CUTTINGS AND CEMENT AND CAPPED WITH CONCRETE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.
<table>
<thead>
<tr>
<th>DEPTH IN FEET</th>
<th>SAMPLE NO.</th>
<th>LITHOLOGY</th>
<th>SOIL CLASS (USCS)</th>
<th>GROUNDWATER</th>
<th>MATERIAL DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>B2-1-5</td>
<td>CL</td>
<td></td>
<td></td>
<td>Approximately 3½ inches AC</td>
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<td>B2-2.5-3</td>
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<td></td>
<td></td>
<td>ALLUVIUM</td>
</tr>
<tr>
<td></td>
<td>B2-3</td>
<td></td>
<td></td>
<td></td>
<td>Stiff, moist, gray, Silty (f) Sandy CLAY with trace (f) rounded gravels</td>
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<tr>
<td></td>
<td>B2-4-4.5</td>
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<td></td>
<td></td>
<td>-same</td>
</tr>
<tr>
<td></td>
<td>B2-4.5</td>
<td></td>
<td></td>
<td></td>
<td>-very stiff, brown streaked gray, less silt and sand</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>-pp=3-3½</td>
</tr>
<tr>
<td>9</td>
<td>B2-9</td>
<td>SC</td>
<td></td>
<td></td>
<td>SANTA CLARA FORMATION</td>
</tr>
<tr>
<td></td>
<td>B2-9.5</td>
<td></td>
<td></td>
<td></td>
<td>Medium dense, moist, orange-brown, brown, and red, Clayey (f-m) SAND with few rounded gravels</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>-dense, more gravels, less clay</td>
</tr>
<tr>
<td>19</td>
<td>B2-19</td>
<td>CL</td>
<td></td>
<td></td>
<td>Hard, damp, brown, gray, and orange-brown, sub-rounded to angular (f) Gravelly CLAY with (f-m) sand</td>
</tr>
<tr>
<td></td>
<td>B2-10.5</td>
<td></td>
<td></td>
<td></td>
<td>-pp&gt;4½</td>
</tr>
</tbody>
</table>

**Notes:**
- The log of subsurface conditions shown herein applies only at the specific boring or trench location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.

**Sample Symbols:**
- Sampling Unsuccessful
- Standard Penetration Test
- Drive Sample (Undisturbed)
- Disturbed or Bag Sample
- Chunk Sample
- Water Table or Seepage
<table>
<thead>
<tr>
<th>DEPTH IN FEET</th>
<th>SAMPLE NO.</th>
<th>LITHOLOGY</th>
<th>SOIL CLASS (USCS)</th>
<th>GROUNDWATER</th>
<th>MATERIAL DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td></td>
<td>SC</td>
<td></td>
<td></td>
<td>Very dense, damp to moist, orange-brown, brown, red, and tan, sub-angular to sub-rounded (f-c) Gravelly Clayey (f-c) SAND</td>
</tr>
<tr>
<td>21</td>
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<td></td>
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<td>22</td>
<td></td>
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<td></td>
</tr>
<tr>
<td>24</td>
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<td></td>
<td></td>
<td></td>
<td>END OF BORING AT APPROXIMATELY 25 FEET</td>
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<tr>
<td></td>
<td>B2-24.5</td>
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<td></td>
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<td>NO FREE WATER ENCOUNTERED</td>
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<td></td>
<td></td>
<td></td>
<td>BACKFILLED WITH COMPACTED CUTTINGS AND CEMENT AND</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>CAPPED WITH CONCRETE</td>
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</tbody>
</table>

END OF BORING AT APPROXIMATELY 25 FEET
NO FREE WATER ENCOUNTERED
BACKFILLED WITH COMPACTED CUTTINGS AND CEMENT AND CAPPED WITH CONCRETE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.
## BORING B3

### MATERIAL DESCRIPTION

<table>
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<th>DEPTH IN FEET</th>
<th>SAMPLE NO.</th>
<th>LITHOLOGY</th>
<th>SOIL CLASS (USCS)</th>
<th>GROUNDWATER</th>
<th>ELEV. (MSL.)</th>
<th>DATE COMPLETED</th>
<th>PENETRATION RESISTANCE (BLOWS/FT.)</th>
<th>DRY DENSITY (P.C.F.)</th>
<th>MOISTURE CONTENT (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
<td>CL</td>
<td></td>
<td></td>
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<td>3/16/2018</td>
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</tr>
<tr>
<td>1</td>
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</tr>
<tr>
<td>2</td>
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</tr>
<tr>
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<td>B3-10.5</td>
<td>CL</td>
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</tr>
</tbody>
</table>

**Approximately 4½ inches AC**

ALLUVIUM  
Very stiff, damp to moist, orange-brown, CLAY with few (f-c) sands and sub-rounded to rounded (f) gravels  
-pp=2½-3

-hard, more sand

SANTA CLARA FORMATION  
Hard, damp, brown, orange-brown, red, and tan, (f-c) Sandy CLAY with few sub-rounded to sub-angular (f) gravels

-brown, orange-brown, red, and gray, more gravels, less sand  
-pp>4½

-less gravels (rounded to sub-rounded)

-orange-brown, tan, brown, and red, sand (f-c), gravels (f-c) (angular to sub-rounded)  
-some gravels appear to be claystone

---

**NOTE:**  
The log of subsurface conditions shown hereon applies only at the specific boring or trench location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.
**BORING B3**

<table>
<thead>
<tr>
<th>DEPTH</th>
<th>SAMPLE NO.</th>
<th>SOIL CLASS (USCS)</th>
<th>ELEV. (MSL.)</th>
<th>DATE COMPLETED</th>
<th>PENETRATION RESISTANCE (BLOWS/FT.)</th>
<th>DRY DENSITY (p/c.f.)</th>
<th>MOISTURE CONTENT (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td></td>
<td></td>
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<td>3/16/2018</td>
<td></td>
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</tr>
<tr>
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<td>23</td>
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<td></td>
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</tr>
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**MATERIAL DESCRIPTION**

-pp>4½
-turbulent structure to soil matrix

-same
-pp>4½

END OF BORING AT APPROXIMATELY 24½ FEET
NO FREE WATER ENCOUNTERED
BACKFILLED WITH COMPACTED CUTTINGS AND CEMENT AND CAPPED WITH CONCRETE

**NOTE:**

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**SAMPLE SYMBOLS**

- Sampling Unsuccessful
- Standard Penetration Test
- Drive Sample (Undisturbed)
- Disturbed or Bag Sample
- Chunk Sample
- Water Table or Seepage
### BORING B4

**ELEV. (MSL.)**

**ENG./GEO.**

**DATE COMPLETED**

**EQUIPMENT**

**HANGER TYPE**

<table>
<thead>
<tr>
<th>DEPTH IN FEET</th>
<th>SAMPLE NO.</th>
<th>LITHOLOGY</th>
<th>SOIL CLASS (USCS)</th>
<th>GROUNDWATER</th>
<th>PENETRATION RESISTANCE (BLOWS/FT.)</th>
<th>DRY DENSITY (P.C.F.)</th>
<th>MOISTURE CONTENT (%)</th>
</tr>
</thead>
<tbody>
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<td></td>
<td>20</td>
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</tr>
</tbody>
</table>

**NOTE:** THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

**SAMPLE SYMBOLS**

- **☐:** SAMPLING UNSUCCESSFUL
- **niej:** STANDARD PENETRATION TEST
- **☐:** DRIVE SAMPLE (UNDISTURBED)
- **☐:** DISTURBED OR BAG SAMPLE
- **☐:** CHUNK SAMPLE
- **☐:** WATER TABLE OR SEEPAGE

**Project Name:** Stanley Group Los Gatos

**Project No.:** E9051-04-01

**Location:** BORING LOGS GPJ

**Date:** 03/30/18

Figure, Log of Boring B4, page 1 of 2
<table>
<thead>
<tr>
<th>DEPTH</th>
<th>SAMPLE NO.</th>
<th>LITHOLOGY</th>
<th>SOIL CLASS (USCS)</th>
<th>DATE COMPLETED</th>
<th>PENETRATION RESISTANCE (BLOWS/FT.)</th>
<th>DRY DENSITY (pcf)</th>
<th>MOISTURE CONTENT (%)</th>
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</thead>
<tbody>
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</tr>
<tr>
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<td>SC</td>
<td>Very dense, moist, brown, angular (f-c) Gravelly Clayey (f-c) SAND</td>
<td>3/16/2018</td>
<td></td>
<td></td>
<td>88</td>
</tr>
<tr>
<td>24.5</td>
<td>B4-24.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>25</td>
<td>B4-26-26.5</td>
<td>SANDSTONE</td>
<td>Gray, fine-grained -very poor recovery</td>
<td></td>
<td></td>
<td></td>
<td>50/6&quot;</td>
</tr>
<tr>
<td>26</td>
<td>B4-26-26.5</td>
<td></td>
<td>END OF BORING AT APPROXIMATELY 26½ FEET NO FREE WATER ENCOUNTERED BACKFILLED WITH COMPACTED CUTTINGS AND CEMENT AND CAPPED WITH CONCRETE</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.
CPT DATA

<table>
<thead>
<tr>
<th>DEPTH (ft)</th>
<th>TIP TSB</th>
<th>FRICTION TSB</th>
<th>Fv/Qt (%)</th>
<th>SPT N</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>100</td>
<td>0</td>
</tr>
<tr>
<td>5</td>
<td>0</td>
<td>0</td>
<td>100</td>
<td>0</td>
</tr>
<tr>
<td>10</td>
<td>0</td>
<td>0</td>
<td>100</td>
<td>0</td>
</tr>
<tr>
<td>15</td>
<td>0</td>
<td>0</td>
<td>100</td>
<td>0</td>
</tr>
<tr>
<td>20</td>
<td>0</td>
<td>0</td>
<td>100</td>
<td>0</td>
</tr>
<tr>
<td>25</td>
<td>0</td>
<td>0</td>
<td>100</td>
<td>0</td>
</tr>
<tr>
<td>30</td>
<td>0</td>
<td>0</td>
<td>100</td>
<td>0</td>
</tr>
<tr>
<td>35</td>
<td>0</td>
<td>0</td>
<td>100</td>
<td>0</td>
</tr>
<tr>
<td>40</td>
<td>0</td>
<td>0</td>
<td>100</td>
<td>0</td>
</tr>
<tr>
<td>45</td>
<td>0</td>
<td>0</td>
<td>100</td>
<td>0</td>
</tr>
<tr>
<td>50</td>
<td>0</td>
<td>0</td>
<td>100</td>
<td>0</td>
</tr>
</tbody>
</table>

Cone Size 10cm squared

Soil Behavior Type based on data from UBC-1983

1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Figure A6

Project: 18840 Saratoga Los Gatos Road
Location: Los Gatos, California
Project No. E9051-04-01
Date: March 2018
**CONE PENETROMETER TEST PROFILE**

**CPT 2**

**Project:** 18840 Saratoga Los Gatos Road  
**Location:** Los Gatos, California  
**Project No.:** E9051-04-01  
**Date:** March 2018  

**FIGURE A7**

---

**CPT DATA**

- **DEPTH (ft):** 0, 5, 10, 15, 20, 25, 30, 35, 40, 45, 50
- **TIP:** 0, 5, 10, 15, 20, 25, 30, 35, 40, 45
- **FRICTION:** 0, 5, 10, 15, 20, 25, 30, 35, 40, 45
- **FS/Qt %:** 0, 5, 10, 15, 20, 25, 30, 35, 40, 45
- **SPT N:** 0, 2, 4, 6, 8, 10, 12

**SOIL BEHAVIOR TYPE**

- **1 - sensitive fine grained**
- **2 - organic material**
- **3 - clay**
- **4 - silty clay to clay**
- **5 - clayey silt to silty clay**
- **6 - sandy silt to clayey silt**
- **7 - silty sand to sandy silt**
- **8 - sand to silty sand**
- **9 - sand**
- **10 - gravelly sand to sand**
- **11 - very stiff fine grained (*)**
- **12 - sand to clayey sand (*)**

*Soil behavior type and SPT based on data from UBC-1983*

---

**Net Area Ratio .8**

**Maximum Depth:** 24.77 ft

**EST GW Depth During Test:** 17.00 ft

**Geocon Inc**
**Project: Stanley Group Los Gatos**
**Operator: RB-BH**
**Filename: SDF(011).cpt**

---

**Job Number:** E9051-04-01

---

**Date and Time:** 3/12/2018 9:42:22 AM

---

**Cone Number:** DDG1350

---

**Cone Size:** 10cm squared
CPT DATA

DEPTH (ft)

SOIL BEHAVIOR TYPE

1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 10cm squared

'(*)Soil behavior type and SPT based on data from UBC-1983

Project: 18840 Saratoga Los Gatos Road
Location: Los Gatos, California
Project No. E9051-04-01
Date: March 2018

FIGURE A8
Laboratory tests were performed in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. Selected samples were tested for in-situ dry density and moisture content, grain size distribution, Atterberg Limits, expansion potential, unconfined compressive strength and shear strength. The results of our testing are summarized in tabular format below and the following figures. In-situ dry density and/or moisture content test results are included on the boring logs in Appendix A.

### TABLE B-I
**SUMMARY OF LABORATORY ATTERBERG LIMITS TEST RESULTS**  
ASTM D 4318

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Liquid Limit</th>
<th>Plastic Limit</th>
<th>Plasticity Index</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1-2.5-3</td>
<td>20</td>
<td>15</td>
<td>5</td>
</tr>
<tr>
<td>B4-2.5-3</td>
<td>23</td>
<td>15</td>
<td>8</td>
</tr>
</tbody>
</table>

### TABLE B-II
**SUMMARY OF LABORATORY GRAIN SIZE ANALYSIS – NO. 200 WASH**  
ASTM D1140

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Sample Depth (feet)</th>
<th>Fraction Passing No. 200 Sieve (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B2</td>
<td>4-4.5</td>
<td>55</td>
</tr>
<tr>
<td>B3</td>
<td>13.5</td>
<td>65</td>
</tr>
<tr>
<td>B4</td>
<td>4-4.5</td>
<td>21</td>
</tr>
<tr>
<td>B4-9.5</td>
<td>9.5</td>
<td>42</td>
</tr>
</tbody>
</table>

### TABLE B-III
**SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS**  
ASTM D 4829

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Moisture Content</th>
<th>Dry Density* (pcf)</th>
<th>Expansion Index</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Before Test (%)</td>
<td>After Test (%)</td>
<td></td>
</tr>
<tr>
<td>B1-1-5</td>
<td>10.4</td>
<td>17.3</td>
<td>109.3</td>
</tr>
<tr>
<td>B3-1-5</td>
<td>9.0</td>
<td>18.0</td>
<td>112.9</td>
</tr>
</tbody>
</table>

*Before saturation.
### APPENDIX B
LABORATORY TESTING (cont.)

#### TABLE B-IV
SUMMARY OF LABORATORY DIRECT SHEAR TEST RESULTS
ASTM D 3080

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Sample Depth (feet)</th>
<th>Initial Average Dry Density (pcf)</th>
<th>Initial Average Moisture Content (%)</th>
<th>Cohesion (psf)</th>
<th>Angle of Shear Resistance (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B2</td>
<td>9.5</td>
<td>109.1</td>
<td>21.1</td>
<td>400</td>
<td>26</td>
</tr>
<tr>
<td>B3</td>
<td>4.5</td>
<td>113.8</td>
<td>15.6</td>
<td>650</td>
<td>32</td>
</tr>
</tbody>
</table>
Boring: B1

Sieve Date: 3/23/18

Depth To Sample: 5-5.5'

Tested and Computed by: AC

Test Data

<table>
<thead>
<tr>
<th>Sieve Number</th>
<th>1 1/2&quot;</th>
<th>1&quot;</th>
<th>3/4&quot;</th>
<th>1/2&quot;</th>
<th>3/8&quot;</th>
<th>#4</th>
<th>#8</th>
<th>#16</th>
<th>#30</th>
<th>#50</th>
<th>#100</th>
<th>#200</th>
</tr>
</thead>
<tbody>
<tr>
<td>% Passing</td>
<td>100.0</td>
<td>91.2</td>
<td>87.5</td>
<td>76.5</td>
<td>72.3</td>
<td>57.4</td>
<td>46.6</td>
<td>37.7</td>
<td>27.7</td>
<td>18.2</td>
<td>13.8</td>
<td>11.2</td>
</tr>
</tbody>
</table>

Geocon Consultants, Inc.
6671 Brisa Street
Livermore, CA 94550
Telephone: (925) 371-5900
Fax: (925) 371-5915

Particle Size Analysis - ASTM D422
Project: 18840 Saratoga Los Gatos Road
Location: Los Gatos, California
Project No.: E9051-04-01

Figure B1
Boring: B2
Sieve Date: 3/23/18
Depth To Sample: 14'
Tested and Computed by: AC

Geocon Consultants, Inc.
6671 Brisa Street
Livermore, CA 94550
Telephone: (925) 371-5900
Fax: (925) 371-5915

Particle Size Analysis - ASTM D422
Project: 18840 Saratoga Los Gatos Road
Location: Los Gatos, California
Project No.: E9051-04-01

Figure B2
Sample Description

- Boring Number: B3
- Sample Depth (feet): 8.5'
- Material Description: Brown with light brown streaks, CLAY with sand

Initial Conditions at Start of Test

- Height (inch) average of 3: 4.67
- Diameter (inch) average of 3: 2.41
- Moisture Content (%): 12.3
- Dry Density (pcf): 117.5
- Estimated Specific Gravity: 2.7
- Saturation (%): 76.8

Shear Test Conditions

- Strain Rate (%/min): 0.7525
- Major Principal Stress at Failure (psf): 13350
- Strain at Failure (%): 4.1

Test Results

- Unconfined Compressive Strength (tons/ft²): 6.7
- Unconfined Compressive Strength (lbs/ft²): 13351
- Shear Strength (tons/ft²): 3.3
- Shear Strength (lbs/ft²): 6676
APPENDIX C
SELECTED OUTPUT – LIQUEFACTION ANALYSIS
Overlay Normalized Plots

Norm. cone resistance

Norm. friction ratio

Norm. pore pressure ratio

SBTn Plot
We evaluated the stability of the existing slope configuration and proposed temporary cut condition using the computer program SLOPE/W (Version 7.23 by Geo-Slope International ©2007). Our analysis considered a circular failure mode under static conditions. We used Spencer’s Method for our analyses. Spencer’s Method satisfies both force and moment equilibrium conditions and is generally accepted for failure surfaces of any shape.

Lithology at each the selected cross-section was modeled based on conditions encountered in our soil borings and observed on the existing slope at the site. Soil shear strength parameters for our analyses were developed through laboratory testing on soil samples obtained from our exploratory borings, published typical values for soil type and in-situ density or consistency, and engineering judgement.

Based on our analyses, the existing slope and proposed temporary cut possess factors of safety against deep-seated slope instability of 1.5 or greater for static conditions. Factor of safety is the ratio of the summation of driving forces divided by the summation of resisting forces. A factor of safety of 1.0 indicates that the driving and resisting forces are equal and the slope is at a state of impending failure/movement. A factor of safety greater than 1.0 indicates the presence of reserve strength; however, does not guarantee that failure will not occur. Rather, the probability of failure generally decreases as the factor of safety increases.
EXISTING SLOPE CONDITION

Santa Clara Formation

Alluvium

Soil Type: Alluvium
Unit Weight: 130 pcf
Cohesion: 400 psf
Phi: 28°

Soil Type: Santa Clara Formation
Unit Weight: 130 pcf
Cohesion: 400 psf
Phi: 30°

Distance (Feet)

0 20 40 60 80 100 120 140 160 180 200 220 240

Elevation (Feet MSL)

440
450
460
470
480
490
500
510
520
530

Project: 18840 Saratoga-Los Gatos Road
Location: Los Gatos, California
Project No. E9051-04-01
Date: April 2018

FIGURE D1
SECTION B-B'

PROPOSED CUT

Project: 18840 Saratoga-Los Gatos Road
Location: Los Gatos, California
Project No. E9051-04-01
Date: April 2018

Santa Clara Formation
Alluvium

Unit Weight: 130 pcf
Cohesion: 400 psf
Phi: 28°

Unit Weight: 130 pcf
Cohesion: 400 psf
Phi: 30°
LIST OF REFERENCES

American Concrete Institute, *Building Code Requirements for Structural Concrete and Commentary, ACI 318-14*, 2014.


Boulanger, R.W. and Idriss, I.M., CPT and SPT Based Liquefaction Triggering Procedures, UC Davis Center for Geotechnical Modeling Report No. UCD/CGM-14/01, April 2014.


Terratech, *Geologic Maps of the Lower Saratoga Hillside Area, Saratoga, California*, 1985, excerpted information provided by the County of Santa Clara.


Unpublished reports, aerial photographs and maps on file with Geocon.
