# GEOTECHNICAL INVESTIGATION PROPOSED RETAIL/COMMERCIAL DEVELOPMENT

NWC Euclid Avenue and Schaefer Avenue
Chino, California
For
Orbis Real Estate Partners

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

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December 12, 2022

Orbis Real Estate Partners
280 Newport Center Drive, Suite 240
Newport Beach, California 92660

Attention: Mr. Grant Ross

Project No.: **22G118-3**

Subject: **Geotechnical Investigation**
Proposed Retail/Commercial Development
NWC Euclid Avenue and Schaefer Avenue
Chino, California

Reference: Geotechnical Feasibility Study, Proposed Retail/Commercial Development, NWC Euclid Avenue and Schaefer Avenue, Chino, California, prepared for Orbis Real Estate Partners (OREP), by Southern California Geotechnical, Inc. (SCG), SCG Project No. 22G118-1, dated March 29, 2022.

Mr. Ross:

In accordance with your request, we have conducted a geotechnical investigation at the subject site. We are pleased to present this report summarizing the conclusions and recommendations developed from our investigation.

We sincerely appreciate the opportunity to be of service on this project. We look forward to providing additional consulting services during the course of the project. If we may be of further assistance in any manner, please contact our office.

Respectfully Submitted,

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# 1.0 EXECUTIVE SUMMARY

Presented below is a brief summary of the conclusions and recommendations of this investigation. Since this summary is not all inclusive, it should be read in complete context with the entire report.

**Geotechnical Design Considerations**

- The proposed development will consist of a 4-story self-storage building, a 4-story residence and parking structure building, and three single-story restaurant and retail buildings. The
- Undocumented fill soils were encountered at all of the boring locations, extending to depths of 2½ to 6½± feet below the existing site grades. These soils are underlain by low to moderate strength native alluvial soils, extending to depths of about 12± feet. These soils are underlain by moderate to high strength native alluvial soils, extending to at least the maximum depth explored of 80± feet.
- The results of laboratory testing indicate that some of the near-surface soils present within the upper 12± feet possess variable strengths and are moderately compressible. Some of the soils extending to depths of up to 25± possess a minor potential for consolidation settlement when loaded.
- The proposed multi-story structures are expected to possess relatively large design foundation loads, and based on the presence of low to moderate strength alluvium and fill soils at this site, these foundations would cause excessive settlements if supported on the presently existing soils. Based on economic and construction considerations, ground improvement consisting of rammed aggregate piers (RAPs) is considered to be the most feasible alternative to support the proposed buildings. RAPs consist of pre-augured cavities that are backfilled with compacted aggregate that creates relatively stiff columns of compacted stone surrounded by a stiffened soil matrix. Installation of the RAPs can significantly reduce settlements as well as increase the allowable bearing capacity of the soils.
- Single-story buildings and non-building structures such as retaining walls, site walls, trash enclosures, etc., may be supported on conventional shallow spread footings underlain by a newly placed layer of compacted structural fill.

**Site Preparation**

- Initial site stripping should include removal of any surficial vegetation from the site. Stripping should include any weeds, grasses, and any organic topsoils. Demolition of the fruit stand structures and any related improvements will also be necessary in the southeast corner of the site.
- Installation of the RAP system in the multi-story building pad areas will improve the soils beneath the foundations. However, it will be necessary to improve the soils that will support the new ground level floor slab. The proposed building areas should be overexcavated to a depth of at least 3 feet below existing grade and to a depth of 3 feet below proposed building pad subgrade elevation. The overexcavation should extend horizontally at least 5 feet beyond the building perimeter.
- New single-story retail and restaurant buildings may be supported on conventional shallow foundations underlain by newly placed structural fill soils. The single-story building pad areas should be overexcavated to at least 3 feet below the existing site grades and to a depth of at least 3 feet below the proposed building pad grade, whichever is greater. The overexcavation

should also extend to a sufficient depth to remove the existing undocumented fill soils, which extend to depths of at 2½ to 6½± feet below existing grades at the boring locations. Conventional shallow foundation systems should be overexcavated to a depth of at least 3 feet below proposed foundation bearing grade. These excavations should then be backfilled with structural fill soils, and compacted to at least 90 percent relative compaction.

- After the overexcavations have been completed, the resulting subgrade soils should be evaluated by the geotechnical engineer to identify any additional soils that should be removed. The resulting subgrade should then be scarified to a depth of 12 inches and moisture conditioned to 2 to 4 percent above optimum. The previously excavated soils may then be replaced as compacted structural fill. All structural fill soils placed within the proposed building areas should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density (95 percent relative compaction in the multi-story building areas).
- RAPs should be installed within the area of the proposed building foundations. The RAPs will be designed and constructed by an independent design-build firm. Installation of the RAPs should be monitored by a representative of the geotechnical engineer.
- The new pavement and flatwork subgrade soils are recommended to be scarified to a depth of 12± inches, thoroughly moisture conditioned and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density.

**Multi-Story Building Foundations**

- The new building foundations can be supported on the RAPs that will be installed at the foundation locations.
- 9,000 lbs/ft$^2$ maximum allowable soil bearing pressure.
- Reinforcement consisting of at least four (4) No. 5 rebars (2 top and 2 bottom) in strip footings. Additional reinforcement may be necessary for structural considerations.

**Single-Story Retail/Restaurant Building Foundations**

- The single-story buildings as well as accessory structures can be supported on conventional shallow foundations underlain by newly placed structural fill soils.
- 2,500 lbs/ft$^2$ maximum allowable soil bearing pressure.
- Reinforcement consisting of at least four (4) No. 5 rebars (2 top and 2 bottom) in strip footings. Additional reinforcement may be necessary for structural considerations.

**Building Floor Slabs**

- Conventional slab-on-grade, minimum 5 inches thick (single-story retail/restaurant buildings).
- Conventional slab-on-grade, minimum 6 inches thick (multi-story).
- Modulus of Subgrade Reaction: k = 125 psi/in.
- Minimum slab reinforcement: No. 3 bars at 18 inches on-center in both directions due to the presence of low expansive soils. The actual thickness and reinforcement of the floor slabs should be determined by the structural engineer.

**Pavement Design Recommendations**

| ASPHALT PAVEMENTS (R = 30) | | | |
|---|---|---|---|
| **Materials** | **Thickness (inches)** | | |
| | Auto Parking (TI = 4.0) | Auto Drive Lanes (TI = 5.0) | Light Truck Traffic (TI = 6.0) |
| Asphalt Concrete | 3 | 3 | 3½ |
| Aggregate Base | 4 | 6 | 8 |
| Compacted Subgrade | 12 | 12 | 12 |

| PORTLAND CEMENT CONCRETE PAVEMENTS (R = 30) | | |
|---|---|---|
| **Materials** | **Thickness (inches)** | |
| | Automobile Parking and Drive Areas (TI = 5.0) | Light Truck Traffic Areas (TI =6.0) |
| PCC | 5 | 5½ |
| Compacted Subgrade (95% minimum compaction) | 12 | 12 |

# 2.0 SCOPE OF SERVICES

The scope of services performed for this project was in general accordance with our Proposal No. 22P112-2, dated October 18, 2022. The scope of services included a visual site reconnaissance, subsurface exploration, field and laboratory testing, and geotechnical engineering analysis to provide criteria for preparing the design of the building foundations, building floor slabs, and parking lot pavements along with site preparation recommendations and construction considerations for the proposed development. The evaluation of the environmental aspects of this site was beyond the scope of services for this geotechnical investigation.

# 3.0 SITE AND PROJECT DESCRIPTION

## 3.1 Site Conditions

The site is located at the northwest corner of Schaefer Avenue and South Euclid Avenue (Highway 83) in Chino, California. The site is bounded to the north by a generally vacant lot that appears to have been used for farming, to the east by South Euclid Avenue, to the south by Schaefer Avenue and to the west by Fern Avenue. The general location of the site is illustrated on the Site Location Map, enclosed as Plate 1 of this report.

The site consists of a rectangular-shaped parcel, 9.5± acres in size. At the time of the referenced feasibility study, the majority of the site was covered with row crops. However, at the time of our recent subsurface investigation for this report, row crops were only present in the southeastern portion of the site. With the exception of the area in the southeast portion of the site that is presently planted, the ground surface cover generally consists of exposed soil and generally remains furrowed in the areas where crops were formerly planted. Two (2) fruit stand structures, 250 and 600± $ft^2$ in size are present near the southeast corner of the site.

Detailed topographic information was not available at the time of this report. Based on elevations obtained from Google Earth and visual observations, the overall site is generally flat with a slight slope to the southwest. The slope has an overall average gradient of about ½ percent and an elevation differential of approximately 4± feet.

## 3.2 Proposed Development

SCG was provided with a site plan prepared by AO Architecture. Based on this plan, the site will be developed with the following:

| Structure Type | Footprint ($ft^2$) | Location |
|---|---|---|
| Multi-Family Residential with Parking Structure (4 Stories) | 135,000 | West |
| 4-Level Self-Storage | 150,000 | Northeast Corner |
| Retail/Food Shops | 15,000 | East-Central |
| Food Shops | 4,500 | Southeast |
| Fast Food Restaurant | 2,500 | South-Central |

We expect that the buildings will be surrounded by asphaltic concrete pavements in the parking and drive areas, and limited areas of concrete flatwork and landscape planters throughout.

Detailed structural information has not been provided. We assume that the smaller retail and restaurant buildings will be of wood frame or metal stud construction, typically supported on conventional shallow foundations with concrete slab-on-grade floors. Based on the assumed construction, maximum column and wall loads for these buildings are expected to be on the order of 30 kips and 1 to 2 kips per linear foot, respectively.

The proposed self-storage facility will be 4 stories in height. At this time, no structural plans for the self-storage building have been provided to our office. Based on previous experience with similar structures, we assume that the proposed structure will be a reinforced CMU structure incorporating several interior columns and bearing walls. The interior partition walls will be supported directly on a thickened structural slab. Average column and wall loads are expected to be on the order of 400 kips and 10 kips per linear foot, respectively. We expect that maximum column loads may be on the order of 800 kips.

The proposed multi-family residential building and parking structure, located in the western portion of the site, will be 4 stories in height. Detailed structural information is not currently available. However, it is assumed that column and wall loads for the new multi-family residential building and parking structure will be on the order of 500 kips and 10 kips per linear foot, respectively. We expect that maximum column loads could be on the order of 1,000 kips in the proposed parking structure area.

Grading plans for the proposed development were not available at the time of this report. The proposed development is not expected to include any significant amounts of below-grade construction such as basements, subterranean levels, or crawl spaces. Based on the existing topography, and assuming a relatively balanced site, cuts and fills of less than 5± feet are expected to be necessary to achieve the proposed site grades.

## 3.3 Previous Study

Southern California Geotechnical, Inc. (SCG) previously performed a geotechnical feasibility study at the subject site. At the time of the referenced feasibility study, the proposed development was expected to consist of nine buildings including restaurants, retail shops, a grocery market, a fitness gym, a gas station. The proposed buildings were planned to possess footprint areas ranging between 3,000 and 23,000 $ft^2$. These buildings were generally planned to be single-story structures of wood frame or masonry block construction.

The subsurface investigation for this feasibility study included nine (9) borings, advanced to depths of 20 to 25± feet below the previously existing site grades. Artificial fill soils were encountered at the ground surface at all of the boring locations, extending to depths of 2½ to 6½± feet below ground surface. The fill soils generally consisted of loose to medium dense silty fine sands. Occasional layers of loose fine sands and fine sandy silts were also encountered. Native alluvium was encountered beneath the fill at all of the boring locations, extending to at least the maximum depth explored of 25± feet below ground surface. The alluvial soils generally consisted of loose to dense silty fine sands, fine sandy silts, fine sands and clayey fine sands, as well as stiff to very stiff clayey silts, silty clays and fine sandy clays. The alluvial soils were observed to possess trace to some iron oxide staining and calcareous nodules and veining. Free water was not encountered during drilling of any of the borings. Groundwater was not encountered at any of the boring locations.

# 4.0 SUBSURFACE EXPLORATION

## 4.1 Scope of Exploration/Sampling Methods

The subsurface exploration conducted for this geotechnical investigation consisted of eight (8) borings, identified as Boring Nos. B-10 through B-17, advanced to depths of 20 to 80± feet below the existing site grades. These borings were performed in addition to the nine borings previously performed for the referenced feasibility study (Boring Nos. B-1 through B-9, inclusive). All of the borings were logged during drilling by a member of our staff.

All of the borings were advanced with hollow-stem augers, by a conventional truck-mounted drilling rig. Representative bulk and relatively undisturbed soil samples were taken during drilling. Relatively undisturbed soil samples were taken with a split barrel "California Sampler" containing a series of one inch long, 2.416± inch diameter brass rings. This sampling method is described in ASTM Test Method D-3550. In-situ samples were also taken using a 1.4± inch inside diameter split spoon sampler, in general accordance with ASTM D-1586. Both of these samplers are driven into the ground with successive blows of a 140-pound weight falling 30 inches. The blow counts obtained during driving are recorded for further analysis. Bulk samples were collected in plastic bags to retain their original moisture content. The relatively undisturbed ring samples were placed in molded plastic sleeves that were then sealed and transported to our laboratory.

The approximate locations of the borings are indicated on the Boring Location Plan, included as Plate 2 in Appendix A of this report. The Boring Logs, which illustrate the conditions encountered at the boring locations, as well as the results of some of the laboratory testing, are included in Appendix B. The Boring Logs prepared at the time of the referenced feasibility study are included in Appendix F of this report.

## 4.2 Geotechnical Conditions

Artificial Fill

Artificial fill soils were encountered at the ground surface at all of the boring locations, extending to depths of 2½ to 6½± feet below ground surface. The fill soils generally consist of loose to medium dense silty fine sands. Occasional layers of loose fine sands and fine sandy silts were also encountered. The fill soils possess a disturbed and mottled appearance, resulting in their classification as artificial fill.

Alluvium

Native alluvium was encountered beneath the fill at all of the boring locations, extending to at least the maximum depth explored of 25± feet below ground surface. The alluvial soils encountered between depths of 2½ to 12± feet generally consist of loose to medium dense silty fine sands, fine sandy silts, fine sands and clayey fine sands, as well as stiff to very stiff clayey silts, silty clays and fine sandy clays. At greater depths, the native alluvial soils generally consist of medium dense to very dense silty sands and sandy silts with occasional well graded sand layers and some interbedded very stiff silty clay, sandy clay, and clayey silt layers. The alluvial soils generally possess trace to some iron oxide staining and calcareous nodules and veining.

Groundwater

Free water was not encountered during drilling of any of our infiltration borings. Based on the lack of water within the borings, the groundwater was considered to have existed at a depth in excess of 80± feet at the time of our subsurface exploration.

As part of our research, we reviewed available groundwater data in order to determine historic high groundwater levels in the vicinity of the site. The primary reference used to determine the groundwater depths in this area is the California Department of Water Resources website, http://www.water.ca.gov/waterdatalibrary/. The nearest monitoring well is located approximately 0.6 mile to the east from the site. Water level readings within this monitoring well indicates a high groundwater levels of about 138 feet below the ground surface in October 2021.

# 5.0 LABORATORY TESTING

The soil samples recovered from the subsurface exploration were returned to our laboratory for further testing to determine selected physical and engineering properties of the soils. The tests are briefly discussed below. It should be noted that the test results are specific to the actual samples tested, and variations could be expected at other locations and depths.

Classification

All recovered soil samples were classified using the Unified Soil Classification System (USCS), in accordance with ASTM D-2488. The field identifications were then supplemented with additional visual classifications and/or by laboratory testing. The USCS classifications are shown on the Boring Logs and are periodically referenced throughout this report.

Dry Density and Moisture Content

The density has been determined for selected relatively undisturbed ring samples. These densities were determined in general accordance with the method presented in ASTM D-2937. The results are recorded as dry unit weight in pounds per cubic foot. The moisture contents are determined in accordance with ASTM D-2216, and are expressed as a percentage of the dry weight. These test results are presented on the Boring Logs.

Consolidation

Selected soil samples have been tested to determine their consolidation potential, in accordance with ASTM D-2435. The testing apparatus is designed to accept either natural or remolded samples in a one-inch-high ring, approximately 2.416 inches in diameter. Each sample is then loaded incrementally in a geometric progression and the resulting deflection is recorded at selected time intervals. Porous stones are in contact with the top and bottom of the sample to permit the addition or release of pore water. The samples are typically inundated with water at an intermediate load to determine their potential for collapse or heave. The results of the consolidation testing are plotted on Plates C-1 through C-8 in Appendix C of this report. The results of consolidation testing performed during the referenced feasibility study are included in Appendix F of this report.

Grain Size Analysis

Limited grain size analyses have been performed on several selected samples, in accordance with ASTM D-1140. These samples were washed over a #200 sieve to determine the percentage of fine-grained material in each sample, which is defined as the material which passes the #200 sieve. The weight of the portion of the sample retained on each screen is recorded and the percentage finer or coarser of the total weight is calculated. The results of these laboratory tests are shown on the attached boring logs.

Maximum Dry Density and Optimum Moisture Content

Representative bulk samples were tested for their maximum dry density and optimum moisture contents at the time of the referenced feasibility study. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557. These tests are generally used to compare the in-situ densities of undisturbed field samples, and for later compaction testing. Additional testing of other soil type or soil mixes may be necessary at a later date. The results of the testing are included in Appendix F of this report.

Direct Shear

Direct shear tests were performed on selected soil samples to determine their shear strength parameters. The tests were performed in accordance with ASTM D-3080. The testing apparatus is designed to accept either natural or remolded samples in a one-inch-high ring, approximately 2.416 inches in diameter. Three samples of the same soil are prepared by remolding them to 90± percent compaction and near optimum moisture. Each of the three samples are then loaded with different normal loads and the resulting shear strength is determined for that particular normal load. The shearing of the samples is performed at a rate slow enough to permit the dissipation of excess pore water pressure. Porous stones are in contact with the top and bottom of the sample to permit the addition or release of pore water. The results of the direct shear tests are presented on Plates C-13 through C-16.

Soluble Sulfates

Representative samples of the near-surface soil were submitted to a subcontracted analytical laboratory for determination of soluble sulfate content. Soluble sulfates are naturally present in soils, and if the concentration is high enough, can result in degradation of concrete which comes into contact with these soils. The results of the soluble sulfate testing are presented below, and are discussed further in a subsequent section of this report.

| **Sample Identification** | **Soluble Sulfates (%)** | **Sulfate Classification** |
|---|---|---|
| B-4 @ 0 to 5 feet (from previous study) | 0.032 | Not Applicable (S0) |
| B-6 @ 0 to 5 feet (from previous study) | 0.008 | Not Applicable (S0) |
| B-10 @ 0 to 5 feet | 0.027 | Not Applicable (S0) |
| B-17 @ 0 to 5 feet | 0.008 | Not Applicable (S0) |

Corrosivity Testing

Representative bulk samples of the near-surface soils were submitted to a subcontracted corrosion engineering laboratory to identify potentially corrosive characteristics with respect to common construction materials. The corrosivity testing included a determination of the electrical resistivity, pH, and chloride and nitrate concentrations of the soils, as well as other tests. The results of the corrosivity testing are presented below, and are discussed further in a subsequent section of this report.

| Sample Identification | Saturated Resistivity (ohm-cm) | pH | Chlorides (mg/kg) | Nitrates (mg/kg) |
|---|---|---|---|---|
| B-4 @ 0 to 5 feet (from previous study) | 4,891 | 8.7 | 1.5 | 67.0 |
| B-6 @ 0 to 5 feet (from previous study) | 1,943 | 8.2 | 10.0 | 275.6 |
| B-10 @ 0 to 5 feet | 3,621 | 8.4 | 5.2 | 98.2 |
| B-17 @ 0 to 5 feet | 1,920 | 7.2 | 25.3 | 310.4 |

Expansion Index

The expansion potential of the on-site soils was determined in general accordance with ASTM D-4829. The testing apparatus is designed to accept a 4-inch diameter, 1-in high, remolded sample. The sample is initially remolded to 50± 1 percent saturation and then loaded with a surcharge equivalent to 144 pounds per square foot. The sample is then inundated with water and allowed to swell against the surcharge. The resultant swell or consolidation is recorded after a 24-hour period. The results of the EI testing are as follows:

| Sample Identification | Expansion Index | Expansive Potential |
|---|---|---|
| B-4 @ 0 to 5 feet (from previous study) | 7 | Very Low |
| B-9 @ 0 to 5 feet (from previous study) | 41 | Low |
| B-13 @ 0 to 5 feet | 31 | Low |

Organic Content Testing

Several samples of the near surface soils were tested to determine their organic contents, in accordance with ASTM Test Method D-2974. The results of the testing are presented on the boring logs in Appendix F of this report. The tests indicated organic contents of 1.4 to 3.5 percent.

# 6.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our review, field exploration, laboratory testing and geotechnical analysis, the proposed development is considered feasible from a geotechnical standpoint. At the time of this report, multiple development schemes were being considered. Once the site plan and foundation loading configuration proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. It is expected that additional subsurface exploration, laboratory testing, and engineering analysis will be required at that time to finalize the geotechnical design recommendations. Any changes in the design, location or elevation of any structure, as outlined in this report, should be reviewed by this office.

The recommendations contained in this report should be taken into the design, construction, and grading considerations. The recommendations are contingent upon all grading and foundation construction activities being monitored by the geotechnical engineer of record.

The Grading Guide Specifications, included as Appendix D, should be considered part of this report, and should be incorporated into the project specifications. The contractor and/or owner of the development should bring to the attention of the geotechnical engineer any conditions that differ from those stated in this report, or which may be detrimental for the development.

## 6.1 Seismic Design Considerations

The subject site is located in an area which is subject to strong ground motions due to earthquakes. The performance of a site-specific seismic hazards analysis was beyond the scope of this investigation. However, numerous faults capable of producing significant ground motions are located near the subject site. Due to economic considerations, it is not generally considered reasonable to design a structure that is not susceptible to earthquake damage. Therefore, significant damage to structures may be unavoidable during large earthquakes. The proposed structures should, however, be designed to resist structural collapse and thereby provide reasonable protection from serious injury, catastrophic property damage and loss of life.

### Faulting and Seismicity

Research of available maps indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Furthermore, Southern California Geotechnical (SCG) did not identify any evidence of faulting during the geotechnical investigation. Therefore, the possibility of significant fault rupture on the site is considered to be low.

The potential for other geologic hazards such as seismically induced settlement, lateral spreading, tsunamis, inundation, seiches, flooding, and subsidence affecting the site is considered low.

### Seismic Design Parameters

Based on the anticipated adoption of the 2022 edition of the California Building Code (CBC) on January 1, 2023, we expect that the proposed development will be designed in accordance with the requirements of the 2022 edition CBC. Section 1613.1

Section 11.4.8 of ASCE 7-16 states that "it shall be permitted to perform a site response analysis or in Accordance with Section 21.1 and/or a ground motion hazard analysis (GMHA) in accordance with Section 21.2." Therefore, a site-specific GMHA was performed in accordance with Section 21.2 of ASCE 7-16 to determine the seismic design parameters for the new building at this site. Details regarding the performance of the GMHA are presented in the report prepared by Terra Geosciences, in Appendix E of this report. The seismic design parameters computed during this study are tabulated below.

The site classification was determined using shear wave velocity measurements for the soils present within the upper 100± feet at the subject site. The parameter $V_{100}$ is defined as the shear-wave velocity of the soil or bedrock material present within the upper 100 feet at the site. The shear-wave velocity was determined by a seismic shear wave survey performed by a licensed geophysicist. The results of the shear-wave survey are included in a report prepared by Terra Geosciences, included in Appendix E of this report. Based on the shear-wave survey performed by Terra Geosciences, the $V_{100}$ for the site is 982.1 feet per second. Table 20.3-1 of ASCE 7-16 indicates that an average shear velocity ranging between 600 and 1,200 feet per second corresponds to Site Class D.

**SITE-SPECIFIC SEISMIC DESIGN PARAMETERS BASED ON ASCE 7-16 SECTION 21.2**

| Parameter | | Value |
|---|---|---|
| Mapped Spectral Acceleration at 0.2 sec Period | $S_S$ | 1.575 |
| Mapped Spectral Acceleration at 1.0 sec Period | $S_1$ | 0.583 |
| Site Class | --- | D |
| Site Modified Spectral Acceleration at 0.2 sec Period | $S_{MS}$ | 1.631 |
| Site Modified Spectral Acceleration at 1.0 sec Period | $S_{M1}$ | 1.177 |
| Design Spectral Acceleration at 0.2 sec Period | $S_{DS}$ | 1.090 |
| Design Spectral Acceleration at 1.0 sec Period | $S_{D1}$ | 0.780 |

Liquefaction

Liquefaction is the loss of strength in generally cohesionless, saturated soils when the pore-water pressure induced in the soil by a seismic event becomes equal to or exceeds the overburden pressure. The primary factors which influence the potential for liquefaction include groundwater table elevation, soil type and plasticity characteristics, relative density of the soil, initial confining pressure, and intensity and duration of ground shaking. The depth within which the occurrence of liquefaction may impact surface improvements is generally identified as the upper 50 feet below the existing ground surface. Liquefaction potential is greater in saturated, loose, poorly graded fine sands with a mean ($d_{50}$) grain size in the range of 0.075 to 0.2 mm (Seed and Idriss, 1971). Non-sensitive clayey (cohesive) soils which possess a plasticity index of at least 18 (Bray and Sancio, 2006) are generally not considered to be susceptible to liquefaction, nor are those soils which are above the historic static groundwater table.

The California Geological Survey (CGS) has not yet conducted detailed seismic hazards mapping in the area of the subject site. The general liquefaction susceptibility of the site was determined by research of the San Bernardino County Land Use Plan, General Plan, Geologic Hazard Overlays. The site is located within the Ontario Quadrangle (Map FH27C). This map indicates that the subject site is not located within an area of liquefaction susceptibility. Furthermore, our historic high groundwater research performed for this site indicates that the long-term groundwater table is considered to exist at a depth in excess of 100± feet. Based on the mapping performed by the county of San Bernardino and the subsurface conditions encountered at the boring locations, liquefaction is not considered to be a design concern for this project.

## 6.2 Geotechnical Design Considerations

### General

The new self-storage and residential structures at this site are expected to possess heights of 4 stories and will possess relatively high foundation loads. The proposed development also includes three (3) single-story restaurant/retail buildings that will possess relatively low foundation loads.

At the time of our field investigation, a portion of the subject site was actively being farmed and during our investigation for the referenced feasibility study, nearly the entire site was planted with row crops. All of the borings encountered fill soils in the upper 2½ to 6½± feet. These soils are considered to be undocumented fill, based on the previous and current site use. The fill soils are underlain by native alluvial soils at all of the boring locations.

Generally, the near-surface alluvium within the upper 2½ to 12± feet at the boring locations are loose to medium dense or (stiff to very stiff where clayey) and based on the results of consolidation/collapse testing, some of the alluvial soils possess a moderate potential for consolidation when loaded and collapse when inundated with water. The underlying native alluvium generally consists of medium dense to dense silty sands, sands and sandy silts and stiff to very stiff silty clay layers, sandy clay and clayey silt layers. The results of laboratory testing indicate that some of the soils between depths of 15 and 25± feet possess minor consolidation potentials. Based on these conditions, the artificial fill materials and the upper portion of the near-surface alluvium, in their present condition, are not considered suitable for the support of new foundations and floor slabs of the proposed structures.

Based on previous experience with similar projects, we expect that the multi-story structures may be supported on spread footings, a mat foundation, or a deep foundation system such as cast-in-drilled hole (CIDH) piles. The use of shallow foundations is typically the most cost-effective method of development. However, due to the presence of the undocumented fill soils and low to moderate strength alluvium within the upper 12 ± feet, and the high foundation loads of the proposed structures, shallow foundations (either spread footings or mat foundations) supported on the existing soils would experience significant settlements.

In formulating our recommendations, we have considered several options, including deep overexcavation of the existing soils followed by replacement with compacted structural fill, deep foundations (drilled piers or driven piles), and the use of rammed aggregate piers (RAPs). Our analysis indicates that the RAP solution will provide the greatest economic benefit in conjunction with acceptable levels of static settlement. The RAPs will be installed beneath the proposed building foundations.

This report provides recommendations for the use of a rammed aggregate pier (RAP) foundation system. The benefits of a RAP system include a significant reduction in the extent of remedial grading, an increased bearing capacity of the resulting soils, high coefficient of friction, and reduced static and seismic settlements. Further details regarding the RAP system are presented in a subsequent section of this report.

Conventional foundations and grading techniques are recommended for the new single-story buildings and other structures located outside the proposed building areas, such as retaining walls, trash enclosures, property line walls, etc.

If the RAP system is only used in foundation areas, some limited remedial grading is recommended in the proposed multi-story building areas in order to remove undocumented fill materials below the floor slab areas that will not be supported by the RAPs.

Settlement

Installation of the proposed RAP system in the multi-story building areas will result in a significant decrease in the static and seismic settlements. The RAP system should be designed to reduce the static settlements in the residential and self-storage building areas to approximately 1 inch and differential settlements will be less than 0.5 inch over a 30-foot span. Based on our experience with similar projects, the RAP system should be designed to reduce the static settlements in the parking structure area to approximately 1½ inch with differential settlements less than 0.75 inch over a 30-foot span. These settlements are considered to be within the settlement tolerances of the proposed multi-story structures, but this assumption should be verified by the project structural engineer.

The recommended remedial grading in the single-story building areas will remove a portion of the lower strength, potentially collapsible alluvial soils (and all of the undocumented fill materials) and replace these materials as compacted structural fill. The native soils that will remain in place below the recommended depth of overexcavation will not be subject to significant stress increases from the foundations of the new structures. Therefore, following completion of the recommended grading, post-construction settlements are expected to be within tolerable limits (approximately 1 inch and differential settlements will be less than 0.5 inch over a 30-foot span).

Soluble Sulfates

The results of the soluble sulfate testing indicate that the selected samples of the on-site soils possess sulfate concentrations that correspond to Class S0 with respect to the American Concrete Institute (ACI) Publication 318-14 Building Code Requirements for Structural Concrete and Commentary, Section 4.3. Therefore, specialized concrete mix designs are not considered to be necessary, with regard to sulfate protection purposes. It is, however, recommended that additional soluble sulfate testing be conducted during the future design-level geotechnical investigation as well as at the completion of rough grading to verify the soluble sulfate concentrations of the on-site soils.

Corrosion Potential

The results of laboratory testing indicate that samples of the on-site soils possess saturated resistivity values of 1,920 and 4,891 ohm-cm, and pH values of 7.2 and 8.7. These test results have been evaluated in accordance with guidelines published by the Ductile Iron Pipe Research Association (DIPRA). The DIPRA guidelines consist of a point system by which characteristics of the soils are used to quantify the corrosivity characteristics of the site. Redox potential, relative soil moisture content and sulfides are also included. Although sulfide testing and redox potential were not part of the scope of services for this project, we have evaluated the corrosivity characteristics of the on-site soils using resistivity, pH and moisture content. **Based on these factors, and utilizing the DIPRA procedure, some of the on-site soils are considered to be moderately corrosive to ductile iron pipe. Therefore, some type of protection is expected to be required for cast iron or ductile iron pipes.**

Relatively low concentrations (1.5 to 25.3 mg/kg) of chlorides were detected in the samples submitted for corrosivity testing. In general, soils possessing chloride concentrations in excess of 500 parts per million (ppm) are considered to be corrosive with respect to steel reinforcement within reinforced concrete. Based on the lack of any significant chlorides in the tested sample, the site is considered to have a C1 chloride exposure in accordance with the American Concrete Institute (ACI) Publication 318 Building Code Requirements for Structural Concrete and Commentary. Therefore, a specialized concrete mix design for reinforced concrete for protection against chloride exposure is not considered warranted.

Nitrates present in soil can be corrosive to copper tubing at concentrations greater than 50 mg/kg. The tested samples possess nitrate concentrations of 67.0 to 310.4 mg/kg. **Based on these test results, the on-site soils are considered to be corrosive to copper pipe with respect to their nitrate concentration.**

**SCG does not practice in the area of corrosion engineering. Therefore, the client may also wish to contact a corrosion engineer to provide a more thorough evaluation.**

Expansion

Laboratory testing performed on representative samples of the near-surface soils indicates that these materials possess a very low to low expansion potential (EI = 7 and 41). Based on the presence of potentially expansive soils at this site, care should be given to proper moisture conditioning the building pad subgrade soils to a moisture content of 2 to 4 percent above the ASTM D-1557 optimum during site grading. It is recommended that additional expansion index testing be conducted at the completion of rough grading to verify the expansion potential of the as-graded building pads.

Shrinkage/Subsidence

Removal and recompaction of the near-surface fill and alluvial soils is estimated to result in an average shrinkage of 5 to 13 percent. This assumes average compaction of 92 percent within the new engineered fill soils. It should be noted that these shrinkage and bulking estimates are based on dry density testing performed on small-diameter samples taken at the boring locations. If a more accurate and precise shrinkage estimate is desired, SCG can perform a shrinkage study involving several excavated test pits where in-place densities are determined using in-situ testing methods instead of laboratory density testing on small-diameter samples. Please contact SCG for details and a cost estimate regarding a shrinkage study, if desired.

Minor ground subsidence is expected to occur in the soils below the zone of removal, due to settlement and machinery working. The subsidence is estimated to be 0.15 feet.

These estimates are based on previous experience and the subsurface conditions encountered at the boring locations. The actual amount of subsidence is expected to be variable and will be dependent on the type of machinery used, repetitions of use, and dynamic effects, all of which are difficult to assess precisely. These estimates should be reviewed and revised as necessary based on the additional subsurface exploration that is expected to occur after the project plans have been finalized.

Grading and Foundation Plan Review

Detailed foundation plans and grading plans were not available at the time of this report. It is therefore recommended that we be provided with copies of the plans, when they become available, for review with regard to the conclusions, recommendations, and assumptions contained within this report. If the proposed building loads exceed those discussed herein, the potential for settlement should be re-evaluated by the geotechnical engineer. Following our review of the updated plans, it is expected that additional geotechnical investigation will be required.

## 6.3 Site Grading Recommendations

The grading recommendations presented below are based on the subsurface conditions encountered at the boring locations and our understanding of the proposed development. We recommend that all grading activities be completed in accordance with the Grading Guide Specifications included as Appendix D of this report, unless superseded by site-specific recommendations presented below.

Site Stripping and Demolition

Initial site stripping should include removal of any surficial vegetation and topsoil. This should include any weeds and grass. Removal of any trees should also include any associated root masses. The actual extent of site stripping should be determined in the field by the geotechnical engineer, based on the organic content and stability of the materials encountered.

The proposed development will require demolition of the existing buildings and other improvements including pavements. Any existing improvements that will not remain in place for use with the new development should be removed in their entirety. This should include all foundations, floor slabs, utilities, trees and associated root masses, and any other above-ground and subsurface improvements associated with the existing structures. The existing pavements are not expected to be reused with the new development. Debris resultant from demolition should be disposed of off-site. Asphaltic concrete and Portland cement concrete debris may be crushed and made into miscellaneous base for use in the proposed pavement areas or crushed to a particle size less than 2 inches and blended with the on-site sandy soils or imported sandy soils for use in structural fills.

Detailed structural information regarding the existing buildings has not been provided to our office. Therefore, the foundation systems supporting the existing buildings are presently unknown by SCG. If any of the existing buildings are supported on deep foundation systems, the deep foundation elements located within the proposed structure areas should be cut off at a depth of at least 3 feet below the bottom of the planned overexcavation. Where deep foundations are encountered within proposed pavement areas, they should be cut off at a depth of at least 2 feet below the proposed pavement subgrade or at a depth of at least 1 foot below the bottom of any planned utilities.

Treatment of Existing Soils:  Multi-story Building Pads

Remedial grading will be necessary within the proposed building and parking structure pad areas to remove all of the undocumented fill soils and a portion of the existing variable strength and variable density near-surface alluvial soils and to provide more uniform support conditions for the new floor slabs of the new structures. Based on conditions encountered at the boring locations, undocumented fill soils extend to depths of 2½ to 6½± feet below existing grade.

In addition to removing all of the undocumented fill soils, it is recommended that the overexcavation extend to a depth of at least 3 feet below existing grade and to a depth of at least 3 feet below proposed grade, whichever is greater.  The overexcavation areas should extend at least 5 feet beyond the building perimeters. If the proposed structures incorporate any exterior columns (such as for a canopy or overhang) the area of overexcavation should also encompass these areas.

Following completion of the overexcavation, the subgrade soils within the overexcavation areas should be evaluated by the geotechnical engineer to verify their suitability to serve as the structural fill subgrade. This evaluation should include proofrolling and probing to identify any soft, loose or otherwise unstable soils that must be removed. Some localized areas of deeper excavation may be required if additional fill materials or loose, porous, overly moist, or low density native soils are encountered at the base of the overexcavation.

After a suitable overexcavation subgrade has been achieved, the exposed soils should be scarified to a depth of at least 12 inches and moisture conditioned or air dried to achieve a moisture content of 2 to 4 percent above optimum moisture content. The subgrade soils should then be recompacted to at least 95 percent of the ASTM D-1557 maximum dry density. The building pad areas may then be raised to grade with previously excavated soils or imported, structural fill. **All structural fill soils within the proposed multi-story building areas should be compacted to at least 95 percent of the ASTM D-1557 maximum dry density.**

Treatment of Existing Soils: New Foundation Areas (Multi-Story Buildings)

It is recommended that the existing soils in the areas of the building multi-story building foundations be improved through the installation of RAPs.  The RAPs will be installed throughout the foundation areas. Based on the existing conditions, the RAPs will extend to depths of approximately 10 to 25± feet below foundation bearing grade.

The RAP construction process consists of utilizing pre-augured holes that are backfilled with aggregate that is compacted in place using static crowd pressure augmented with a high frequency, low amplitude, vibratory hammer. The impact hammer densifies the aggregate vertically while the tamper foot forces aggregate laterally into the cavity sidewalls, resulting in stiff RAP elements and a stiffened matrix soil between the RAPs. The actual diameter of the RAPs will be determined by WGI, but typically range from 24 to 30 inches.

The RAP installation process should be observed and documented by a representative of the geotechnical engineer. This documentation should include RAP spacing, diameter, and depth.

Treatment of Existing Soils: Single-Story Retail and Restaurant Building Pads

Remedial grading should be performed within the proposed single-story building pad areas in order to remove all disturbed alluvium and undocumented fill soils that may be encountered. Based on conditions encountered at the boring locations, this generally will require excavation to depths of 2½ to 6½± feet, although localized areas of deeper undocumented fill soils may exist at locations not explored by the borings. The existing soils within the proposed building pad areas are also recommended to be overexcavated to a depth of at least 3 feet below proposed building pad subgrade elevation and to a depth of at least 3 feet below existing grade, whichever is greater. Within the influence zones of the new foundations, the overexcavation should extend to a depth of at least 3 feet below proposed foundation bearing grade.

Following completion of the overexcavation, the subgrade soils within the overexcavation areas should be evaluated by the geotechnical engineer to verify their suitability to serve as the structural fill subgrade. This evaluation should include proofrolling and probing to identify any soft, loose or otherwise unstable soils that must be removed. Some localized areas of deeper excavation may be required if additional fill materials or loose, porous, overly moist, or low-density native soils are encountered at the base of the overexcavation.

After a suitable overexcavation subgrade has been achieved, the exposed soils should be scarified to a depth of at least 12 inches and moisture conditioned or air dried to achieve a moisture content of 2 to 4 percent above optimum moisture content. The subgrade soils should then be recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The building pad areas may then be raised to grade with previously excavated soils or imported, structural fill. All structural fill soils within the proposed single-story building pad areas should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density.

Treatment of Existing Soils: Retaining Walls and Site Walls

It is expected that shallow foundations will also be required for support of appurtenances located outside the building areas, such as retaining walls, site walls, trash enclosures, etc. The existing soils within the foundation areas of these accessory structures should be overexcavated to a depth of at least 2 feet below foundation bearing grade and replaced as compacted structural fill. Any undocumented fill soils should also be removed from the retaining wall areas. The overexcavation subgrade soils should be evaluated by the geotechnical engineer prior to scarifying, moisture conditioning and recompacting the upper 12 inches of exposed subgrade soils. The previously excavated soils may then be replaced as compacted structural fill.

Treatment of Existing Soils: Parking and Drive Areas

Based on economic considerations, overexcavation of the existing undocumented fill soils in the new flatwork, parking and drive areas is not considered warranted, with the exception of areas where lower strength or unstable soils are identified by the geotechnical engineer during grading. Subgrade preparation in the new parking and drive areas should initially consist of removal of all soils disturbed during stripping and demolition operations.

The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. Any such materials should be removed to a level of firm and unyielding soil. The exposed subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to 2 to 4 percent above the optimum moisture content, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. Based on the presence of variable strength surficial soils throughout the site, it is expected that some isolated areas of additional overexcavation may be required to remove zones of lower strength, unsuitable soils.

The grading recommendations presented above for the proposed flatwork, parking and drive areas assume that the owner and/or developer can tolerate minor amounts of settlement within these areas. The grading recommendations presented above do not mitigate the extent of undocumented fill or compressible/collapsible native alluvium in the flatwork, parking and drive areas. As such, some settlement and associated pavement distress could occur. Typically, repair of such distressed areas involves significantly lower costs than completely mitigating these soils at the time of construction. If the owner cannot tolerate the risk of such settlements, the flatwork, parking and drive areas should be overexcavated to a depth of 2 feet below proposed pavement subgrade elevation, with the resulting soils replaced as compacted structural fill.

Treatment of Existing Soils: Flatwork Areas

Subgrade preparation in the new flatwork areas should initially consist of removal of all soils disturbed during stripping and demolition operations. The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. The subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to 2 to 4 percent above optimum, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. Based on the presence of variable strength alluvial soils throughout the site, it is expected that some isolated areas of additional overexcavation may be required to remove zones of lower strength, unsuitable soils.

Fill Placement

- Fill soils should be placed in thin (6± inches), near-horizontal lifts, moisture conditioned to 2 to 4 percent above the optimum moisture content, and compacted. Drying of the on-site soils may be required before placement and compaction of structural fill.
- On-site soils may be used for fill provided they are cleaned of any debris to the satisfaction of the geotechnical engineer.
- All grading and fill placement activities should be completed in accordance with the requirements of the 2022 CBC and the grading code of the city of Chino.
- All fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. We recommend that fill soils placed within the new multi-story building areas be

compacted to at least 95 percent of the ASTM D-1557 maximum dry density. Fill soils should be well mixed.

- Compaction tests should be performed periodically by the geotechnical engineer as random verification of compaction and moisture content. These tests are intended to aid the contractor. Since the tests are taken at discrete locations and depths, they may not be indicative of the entire fill and therefore should not relieve the contractor of his responsibility to meet the job specifications.

Imported Structural Fill

All imported structural fill should consist of very low expansive (EI < 20), well graded soils possessing at least 10 percent fines (that portion of the sample passing the No. 200 sieve). Additional specifications for structural fill are presented in the Grading Guide Specifications, included as Appendix D.

Utility Trench Backfill

In general, all utility trench backfill should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Compacted trench backfill should conform to the requirements of the local grading code, and more restrictive requirements may be indicated by the city of Chino. All utility trench backfills should be witnessed by the geotechnical engineer. The trench backfill soils should be compaction tested where possible; probed and visually evaluated elsewhere.

Utility trenches which parallel a footing, and extending below a 1h:1v plane projected from the outside edge of the footing should be backfilled with structural fill soils, compacted to at least 90 percent of the ASTM D-1557 standard. Pea gravel backfill should not be used for these trenches.

## 6.4 Construction Considerations

Excavation Considerations

The near-surface soils generally consist of silty sands and sandy silts and silty clays and clayey silts. Some of these materials may be subject to caving within shallow excavations. Where caving occurs within shallow excavations, flattened excavation slopes may be sufficient to provide excavation stability. On a preliminary basis, temporary excavation slopes consisting of sands, silty sands and sandy silts should be made no steeper than 2h:1v. The contractor should take all necessary precautions during grading and foundation construction to prevent damage to structures and improvements which are adjacent to the proposed development. Deeper excavations may require some form of external stabilization such as shoring or bracing. Maintaining adequate moisture content within the near-surface soils will improve excavation stability. All excavation activities on this site should be conducted in accordance with Cal-OSHA regulations.

Moisture Sensitive Subgrade Soils

The near-surface soils generally consist of moist silty sands, and sandy silts, as well as silty clays and clayey silts and will become unstable if exposed to significant moisture infiltration or disturbance by construction traffic. If grading occurs during a period of relatively wet weather, an increase in subgrade instability should also be expected. The site should, therefore, be graded to prevent ponding of surface water and to prevent water from running into excavations.

If the construction schedule dictates that site grading will occur during a period of wet weather, allowances should be made for costs and delays associated with drying the on-site soils or import of a drier, less moisture sensitive fill material. Grading during wet or cool weather may also increase the depth of overexcavation in the pad areas as well as the need for subgrade stabilization.

<u>Groundwater</u>

Groundwater was not encountered at any of the borings, which extended to depths as great as 80± feet below the existing site grades. Therefore, groundwater is not expected to impact the grading or foundation construction activities.

## 6.5 Foundation Design and Construction

Based on the preceding grading recommendations, it is assumed that the new multi-story building foundations will be underlain by existing soils which have been improved by the placement of a system of RAPs. The foundations for the single-story buildings, as well as new appurtenances, such as retaining walls, site walls, trash enclosures, etc., will be underlain by newly placed structural fill soils, extending to depths of at least 3 feet below proposed foundation bearing grades. Based on these subsurface profiles, the proposed structures may be designed as follows:

<u>Multi-story Building Foundation Design Parameters (New Building Foundations)</u>

New square and rectangular footings may be designed as follows:

- Maximum, net allowable soil bearing pressure: 9,000 lbs/ft$^2$.
- Minimum wall/column footing width: 14 inches/24 inches.
- Minimum longitudinal steel reinforcement within strip footings: Four (4) No. 5 rebars (2 top and 2 bottom).
- Minimum foundation embedment: 12 inches into suitable structural fill soils, and at least 24 inches below adjacent exterior grade. Interior column footings may be placed immediately beneath the floor slab. The building foundations should be directly supported on the RAPs.
- It is recommended that the perimeter building foundations be continuous across all exterior doorways. Any flatwork adjacent to the exterior doors should be doweled into the perimeter foundations in a manner determined by the structural engineer.
- The current site plan indicates that the residences will be constructed immediately adjacent to the central parking structure. These structures will have significantly different loading conditions, and possibly different settlement tolerances. Therefore, it is

recommended that the residences be structurally isolated from the heavily loaded central structure.

The allowable bearing pressures presented above may be increased by 1/3 when considering short duration wind or seismic loads. The minimum steel reinforcement recommended above is based on standard geotechnical practice. Additional rigidity may be necessary for structural considerations. The actual design of the foundations should be determined by the structural engineer. The maximum allowable bearing pressure presented above is preliminary; the actual bearing pressure should be determined by the RAP designer based on both bearing capacity and settlement considerations. The bearing pressure is also contingent upon our review of the final site plan and completion of any necessary supplemental geotechnical investigation at the site.

Foundation Design Parameters (Single-Story Buildings and Non-Building Foundations)

New square and rectangular footings used to support the single-story retail and restaurant buildings, as well as any accessory structures such as retaining walls, screen walls, and trash enclosures may be designed as follows:

- Maximum, net allowable soil bearing pressure: 2,500 lbs/ft$^2$.
- Minimum wall/column footing width: 14 inches/24 inches.
- Minimum longitudinal steel reinforcement within strip footings: Four (4) No. 5 rebars (2 top and 2 bottom).
- Minimum foundation embedment: 12 inches into suitable structural fill soils, and at least 18 inches below adjacent exterior grade.

The allowable bearing pressures presented above may be increased by 1/3 when considering short duration wind or seismic loads. The minimum steel reinforcement recommended above is based on standard geotechnical practice. Additional rigidity may be necessary for structural considerations. The actual design of the foundations should be determined by the structural engineer.

Foundation Construction

The foundation subgrade soils should be evaluated at the time of site grading, as discussed in Section 6.3 of this report. It is further recommended that the foundation subgrade soils be evaluated by the geotechnical engineer immediately prior to steel or concrete placement. Soils suitable for support of the new building foundations should consist of existing soils that have been improved through the placement of the RAPs. Soils suitable for direct foundation support in the accessory structure areas should consist of newly placed structural fill, compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Any unsuitable materials should be removed to a depth of suitable bearing compacted structural fill, with the resulting excavations backfilled with compacted fill soils. As an alternative, lean concrete slurry (500 to 1,500 psi) may be used to backfill such isolated overexcavations.

The foundation subgrade soils should also be properly moisture conditioned to 2 to 4 percent above the Modified Proctor optimum, to a depth of at least 12 inches below bearing grade. Since it is typically not feasible to increase the moisture content of the floor slab and foundation subgrade soils once rough grading has been completed, care should be taken to maintain the moisture content of the building pad subgrade soils throughout the construction process.

Estimated Foundation Settlements

Post-construction total and differential static settlements of shallow foundations designed and constructed in accordance with the previously presented recommendations are estimated to be less than 1 and ½ inches, respectively. Differential movements are expected to occur over a 30-foot span, thereby resulting in an angular distortion of less than 0.002 inches per inch. \

Lateral Load Resistance

Lateral load resistance will be developed by a combination of friction acting at the base of foundations and slabs and the passive earth pressure developed by footings below grade. The following friction and passive pressure may be used to resist lateral forces:

- Passive Earth Pressure: 300 lbs/ft$^3$
- Friction Coefficient: 0.30
- Friction Coefficient: 0.4 for foundations supported directly on RAPs.

These are allowable values, and include a factor of safety. When combining friction and passive resistance, the passive pressure component should be reduced by one-third. These values assume that footings will be poured directly against compacted structural fill soils. The maximum allowable passive pressure is 2,500 lbs/ft$^2$.

## 6.6 Floor Slab Design and Construction

Subgrades which will support new floor slabs should be prepared in accordance with the recommendations contained in the ***Site Grading Recommendations*** section of this report. Based on the anticipated grading which will occur at this site, the floors of the proposed structures may be constructed as conventional slabs-on-grade supported on newly placed structural fill, extending to a depth of at least 3 feet below finished pad grade. Based on geotechnical considerations, the floor slabs may be designed as follows:

- Minimum slab thickness (small retail buildings/restaurants): 5 inches.
- Minimum slab thickness (multi-story residence and self-storage buildings): 6 inches

- Modulus of Subgrade Reaction: 125 psi/in.

- Minimum slab reinforcement: No. 3 bars at 18 inches on-center in both directions, based on the presence of low expansive soils. The actual floor slab reinforcement should be determined by the structural engineer, based upon the imposed loading.

- Slab underlayment: If moisture sensitive floor coverings will be used then minimum slab underlayment should consist of a moisture vapor barrier constructed below the entire slab area where such moisture sensitive floor coverings are expected. The moisture vapor barrier should meet or exceed the Class A rating as defined by ASTM E 1745-97 and have a permeance rating less than 0.01 perms as described in ASTM E 96-95 and ASTM E 154-88. A polyolefin material such as Stego® Wrap Vapor Barrier or equivalent will meet these specifications. The moisture vapor barrier should be properly constructed in accordance with all applicable manufacturer specifications. Given that a rock free subgrade is anticipated and that a capillary break is not required, sand below the barrier is not required. The need for sand and/or the amount of sand above the moisture vapor barrier should be specified by the structural engineer or concrete contractor. The selection of sand above the barrier is not a geotechnical engineering issue and hence outside our purview. Where moisture sensitive floor coverings are not anticipated, the vapor barrier may be eliminated.

- Moisture condition the floor slab subgrade soils to 2 to 4 percent above the Modified Proctor optimum moisture content, to a depth of 12 inches. The moisture content of the floor slab subgrade soils should be verified by the geotechnical engineer within 24 hours prior to concrete placement.
- Proper concrete curing techniques should be utilized to reduce the potential for slab curling or the formation of excessive shrinkage cracks.

The actual design of the floor slab should be completed by the structural engineer to verify adequate thickness and reinforcement.

## 6.7 Exterior Flatwork Design and Construction

Subgrades which will support new exterior slabs-on-grade for sidewalks, patios, and other concrete flatwork, should be prepared in accordance with the recommendations contained in the ***Grading Recommendations*** section of this report. Based on geotechnical considerations, exterior slabs on grade may be designed as follows:

- Minimum slab thickness: 4½ inches.
- Minimum slab reinforcement: No. 3 bars at 18 inches on center, in both directions.
- The flatwork at building entry areas should be structurally connected to the perimeter foundation that is recommended to span across the door opening. This recommendation is designed to reduce the potential for differential movement at this joint.
- Moisture condition the slab subgrade soils to at least 2 to 4 percent above the optimum moisture content, to a depth of at least 12 inches. Adequate moisture conditioning should be verified by the geotechnical engineer 24 hours prior to concrete placement.
- Proper concrete curing techniques should be utilized to reduce the potential for slab curling or the formation of excessive shrinkage cracks.
- Control joints should be provided at a maximum spacing of 8 feet on center in two directions for slabs and at 6 feet on center for sidewalks. Control joints are intended to direct cracking. Minor cracking of exterior concrete slabs on grade should be expected.

Expansion or felt joints should be used at the interface of exterior slabs on grade and any fixed structures to permit relative movement.

## 6.8 Retaining Wall Design and Construction

Although not indicated on the site plan, some small (less than 6 feet in height) retaining walls may be required to facilitate the new site grades. The parameters recommended for use in the design of these walls are presented below.

Retaining Wall Design Parameters

Based on the soil conditions encountered at the boring locations, the following parameters may be used for preliminary design of new retaining walls for this site. The following parameters assume that only the on-site sands, silty sands and sandy silts should be utilized for retaining wall backfill. Based on the results of our direct shear testing, the on-site soils consisting of sands, silty sands and sandy silts have been preliminarily assigned a conservatively estimated friction angle of 30 degrees when compacted to 90 percent of the ASTM-1557 maximum dry density. The on-site silty clays, fine sandy clays, and clayey silts are expansive likely possess lower strengths and therefore, should not be used as retaining wall backfill.

If desired, SCG could provide design parameters for an alternative select backfill material behind the retaining walls. The use of select backfill material could result in lower lateral earth pressures. In order to use the design parameters for the imported select fill, this material must be placed within the entire active failure wedge. This wedge is defined as extending from the heel of the retaining wall upwards at an angle of approximately 60° from horizontal. If select backfill material behind the retaining wall is desired, SCG should be contacted for supplementary recommendations.

**RETAINING WALL DESIGN PARAMETERS**

<table>
<tr><th colspan="2" rowspan="2">Design Parameter</th><th>Soil Type</th></tr>
<tr><th>On-Site Sands, Silty Sands and Sandy Silts</th></tr>
<tr><td colspan="2">Internal Friction Angle (ϕ)</td><td>30°</td></tr>
<tr><td colspan="2">Unit Weight</td><td>130 lbs/ft³</td></tr>
<tr><td rowspan="3">Equivalent Fluid Pressure:</td><td>Active Condition (level backfill)</td><td>43 lbs/ft³</td></tr>
<tr><td>Active Condition (2h:1v backfill)</td><td>70 lbs/ft³</td></tr>
<tr><td>At-Rest Condition (level backfill)</td><td>65 lbs/ft³</td></tr>
</table>

The walls should be designed using a soil-footing coefficient of friction of 0.30 and an equivalent passive pressure of 300 lbs/ft$^3$. The structural engineer should incorporate appropriate factors of safety in the design of the retaining walls.

The active earth pressure may be used for the design of retaining walls that do not directly support structures or support soils that in turn support structures and which will be allowed to deflect. The at-rest earth pressure should be used for walls that will not be allowed to deflect such as those which will support foundation bearing soils, or which will support foundation loads directly.

Where the soils on the toe side of the retaining wall are not covered by a "hard" surface such as a structure or pavement, the upper 1 foot of soil should be neglected when calculating passive resistance due to the potential for the material to become disturbed or degraded during the life of the structure.

Seismic Lateral Earth Pressures

In addition to the lateral earth pressures presented in the previous section, retaining walls which are more than 6 feet in height should be designed for a seismic lateral earth pressure, in accordance with the 2022 CBC. Based on the current site plan, it is not expected that any walls in excess of 6 feet in height will be required for this project. If any such walls are proposed, our office should be contacted for supplementary design recommendations.

Retaining Wall Foundation Design

The retaining wall foundations should be supported within newly placed compacted structural fill, extending to a depth of at least 3 feet below proposed foundation bearing grade. Foundations to support new retaining walls should be designed in accordance with the general Foundation Design Parameters presented in a previous section of this report.

Backfill Material

With the exception of fine sandy clays, clayey silts, and silty clays, the on-site soils may be used to backfill the retaining walls. However, all backfill material placed within 3 feet of the back wall-face should have a particle size no greater than 3 inches. The retaining wall backfill materials should be well graded.

It is recommended that a minimum 1 foot thick layer of free-draining granular material (less than 5 percent passing the No. 200 sieve) be placed against the face of the retaining walls. This material should extend from the top of the retaining wall footing to within 1 foot of the ground surface on the back side of the retaining wall. This material should be approved by the geotechnical engineer. In lieu of the 1-foot-thick layer of free-draining material, a properly installed prefabricated drainage composite such as the MiraDRAIN 6000XL (or approved equivalent), which is specifically designed for use behind retaining walls, may be used. If the layer of free-draining material is not covered by an impermeable surface, such as a structure or pavement, a 12-inch-thick layer of a low permeability soil should be placed over the backfill to reduce surface water migration to the underlying soils. The layer of free draining granular material should be separated from the backfill soils by a suitable geotextile, approved by the geotechnical engineer.

All retaining wall backfill should be placed and compacted under engineering-controlled conditions in the necessary layer thicknesses to ensure an in-place density between 90 and 93 percent of the maximum dry density as determined by the Modified Proctor test (ASTM D1557). Care should be taken to avoid over-compaction of the soils behind the retaining walls, and the use of heavy compaction equipment should be avoided.

Subsurface Drainage

As previously indicated, the retaining wall design parameters are based upon drained backfill conditions. Consequently, some form of permanent drainage system will be necessary in conjunction with the appropriate backfill material. Subsurface drainage may consist of either:

- A weep hole drainage system typically consisting of a series of 2-inch diameter holes in the wall situated slightly above the ground surface elevation on the exposed side of the wall and at an approximate 10-foot on-center spacing. Alternatively, 4-inch diameter holes at an approximate 20-foot on-center spacing can be used for this type of drainage system. In addition, the weep holes should include a 2 cubic foot pocket of open graded gravel, surrounded by an approved geotextile fabric, at each weep hole location.

- A 4-inch diameter perforated pipe surrounded by 2 cubic feet of gravel per linear foot of drain placed behind the wall, above the retaining wall footing. The gravel layer should be wrapped in a suitable geotextile fabric to reduce the potential for migration of fines. The footing drain should be extended to daylight or tied into a storm drainage system. The actual design of this type of system should be determined by the civil engineer to verify that the drainage system possesses the adequate capacity and slope for its intended use.

Weep holes or a footing drain will not be required on the inside of building stem walls.

## 6.9 Pavement Design Parameters

Site preparation in the pavement area should be completed as previously recommended in the ***Site Grading Recommendations*** section of this report. The subsequent pavement recommendations assume proper drainage and construction monitoring, and are based on either PCA or CALTRANS design parameters for a twenty (20) year design period. However, these designs also assume a routine pavement maintenance program to obtain the anticipated 20-year pavement service life.

Pavement Subgrades

It is anticipated that the new pavements will be primarily supported on a layer of compacted structural fill, consisting of scarified, thoroughly moisture conditioned and recompacted existing soils. The near-surface soils generally consist of silty sands, clayey sands, sandy clays and sandy silts. These soils are expected to provide fair to good pavement support characteristics. R-value testing was outside the scope of work for this project. Based on their classification, these soils are assumed to possess an R-value of at least 30. Any fill material imported to the site should have support characteristics equal to or greater than that of the on-site soils and be placed and compacted under engineering-controlled conditions. It is recommended that R-value testing be performed after completion of rough grading. Depending upon the results of the R-value testing, it may be feasible to use thinner pavement sections in some areas of the site.

Asphaltic Concrete

Presented below are the recommended thicknesses for new flexible pavement structures consisting of asphaltic concrete over a granular base. The pavement designs are based on the traffic indices (TI's) indicated. The client and/or civil engineer should verify that these TI's are representative of the anticipated traffic volumes. If the client and/or civil engineer determine that the expected traffic volume will exceed the applicable traffic index, we should be contacted for supplementary recommendations. The design traffic indices equate to the following approximate daily traffic volumes over a 20-year design life, assuming six operational traffic days per week.

| **Traffic Index** | **No. of Heavy Trucks per Day** |
|---|---|
| 4.0 | 0 |
| 5.0 | 1 |
| 6.0 | 3 |

For the purpose of the traffic volumes indicated above, a truck is defined as a 5-axle tractor trailer unit with one 8-kip axle and two 32-kip tandem axles. All of the traffic indices allow for 1,000 automobiles per day.

| **ASPHALT PAVEMENTS (R = 30)** | | | |
|---|---|---|---|
| **Materials** | **Thickness (inches)** | | |
| | Auto Parking (TI = 4.0) | Auto Drive Lanes (TI = 5.0) | Light Truck Traffic (TI = 6.0) |
| Asphalt Concrete | 3 | 3 | 3½ |
| Aggregate Base | 4 | 6 | 8 |
| Compacted Subgrade | 12 | 12 | 12 |

The aggregate base course should be compacted to at least 95 percent of the ASTM D-1557 maximum dry density. The asphaltic concrete should be compacted to at least 95 percent of the batch plant-reported maximum density. The aggregate base course may consist of crushed aggregate base (CAB) or crushed miscellaneous base (CMB), which is a recycled gravel, asphalt and concrete material. The gradation, R-Value, Sand Equivalent, and Percentage Wear of the CAB or CMB should comply with appropriate specifications contained in the current edition of the "Greenbook" Standard Specifications for Public Works Construction.

Portland Cement Concrete

The preparation of the subgrade soils within concrete pavement areas should be performed as previously described for proposed asphalt pavement areas. The minimum recommended thicknesses for the Portland Cement Concrete pavement sections are as follows:

| PORTLAND CEMENT CONCRETE PAVEMENTS (R = 30) | | |
|---|---|---|
| **Materials** | **Thickness (inches)** | |
| | Automobile Parking and Drive Areas (TI = 5.0) | Light Truck Traffic Areas (TI =6.0) |
| PCC | 5 | 5½ |
| Compacted Subgrade (95% minimum compaction) | 12 | 12 |

The concrete should have a 28-day compressive strength of at least 3,000 psi. Reinforcing within all pavements should be designed by the structural engineer. The maximum joint spacing within all of the PCC pavements is recommended to be equal to or less than 30 times the pavement thickness. The actual joint spacing and reinforcing of the Portland cement concrete pavements should be determined by the structural engineer.

## 7.0 GENERAL COMMENTS

This report has been prepared as an instrument of service for use by the client, in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. This report may be provided to the contractor(s) and other design consultants to disclose information relative to the project. However, this report is not intended to be utilized as a specification in and of itself, without appropriate interpretation by the project architect, civil engineer, and/or structural engineer. The reproduction and distribution of this report must be authorized by the client and Southern California Geotechnical, Inc. Furthermore, any reliance on this report by an unauthorized third party is at such party's sole risk, and we accept no responsibility for damage or loss which may occur. The client(s)' reliance upon this report is subject to the Engineering Services Agreement, incorporated into our proposal for this project.

The analysis of this site was based on a subsurface profile interpolated from limited discrete soil samples. While the materials encountered in the project area are considered to be representative of the total area, some variations should be expected between boring locations and sample depths. If the conditions encountered during construction vary significantly from those detailed herein, we should be contacted immediately to determine if the conditions alter the recommendations contained herein.

This report has been based on assumed or provided characteristics of the proposed development. It is recommended that the owner, client, architect, structural engineer, and civil engineer carefully review these assumptions to ensure that they are consistent with the characteristics of the proposed development. If discrepancies exist, they should be brought to our attention to verify that they do not affect the conclusions and recommendations contained herein. We also recommend that the project plans and specifications be submitted to our office for review to verify that our recommendations have been correctly interpreted.

The analysis, conclusions, and recommendations contained within this report have been promulgated in accordance with generally accepted professional geotechnical engineering practice. No other warranty is implied or expressed.