GEOTECHNICAL INVESTIGATION, PROPOSED 12-ACRE RESIDENTIAL DEVELOPMENT, ±2500 GARRETSON AVENUE, CITY OF CORONA, CALIFORNIA

Prepared for:

TRG LAND, INC.
898 Production Place
Newport Beach, California 92663

Project No. 022607-001

September 17, 2012

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To: TRG Land, Inc.
898 Production Place
Newport Beach, California 92663

Attention: Mr. Mark Rogers

Subject: Preliminary Geotechnical Investigation, Proposed 12-Acre Residential Development, ±2500 Garretson Avenue, City of Corona, California

In accordance with your authorization, Leighton and Associates, Inc. has conducted this preliminary geotechnical investigation for the proposed 12-acre residential development located at ±2500 Garretson Avenue in the city of Corona, California. The purpose of this study has been to evaluate the general geotechnical conditions at the site with respect to the proposed improvements and to provide preliminary geotechnical recommendations for design and construction.

Based on this investigation, construction of the proposed residential development is feasible from a geotechnical standpoint. The most significant geotechnical issues at the site are those related to the potential for strong seismic shaking, the potential for moderate seismic settlement, and potentially compressible soils. Good planning and design of the project can limit the impact of these constraints. This report presents our preliminary findings, conclusions, and geotechnical recommendations for the project.

We appreciate the opportunity to work with you on the development of this project. If you have any questions, or if we can be of further service, please call us at your convenience at (909) 484-2205.

Respectfully submitted,

LEIGHTON AND ASSOCIATES, INC.

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1.0 INTRODUCTION

1.1 Site Location and Description

The proposed residential development site is located at ±2500 Garretson Avenue in the city of Corona. The site is roughly L-shaped in plan view with a total area of approximately 12 acres. The site is bounded by Garretson Avenue to the northwest, a row of single-family homes north of C L Fleming Circle to the southwest, a row of single-family homes west of Twinleaf Lane to the southeast, and the Islamic Society of Corona to the northeast. The site is bounded by a private residence to the north (see Figure 1).

The site has present and historical use as a citrus orchard. Storage and rubbish areas are located in the central and northern portions of the site.

1.2 Proposed Improvements

Our understanding of this project is based in part on our correspondence with you and on the 80-scale Site Plan, Corona-Norco Unified School District Roosevelt School, prepared by you and dated July 30, 2012. We understand that the site is in preliminary design, but is expected to be graded for residential lots and associated streets, drainage improvements and underground improvements, including storm drain, sewer, and wet and dry utilities.

1.3 Purpose of Investigation

The purpose of this study has been to evaluate the general geotechnical conditions at the site with respect to the proposed improvements and to provide preliminary geotechnical recommendations for design and construction.

Our geotechnical exploration included hollow-stem auger soil borings, laboratory testing and geotechnical analysis to evaluate existing conditions and develop the recommendations contained in this report.

1.4 Scope

The scope of our geotechnical investigation has included the following tasks:

- Geologic Hazards Review - We reviewed pertinent, readily available geologic and geotechnical literature covering the site. Our review included regional geologic maps and reports available from our library covering the site. Documents reviewed are listed in Appendix A, References.

- Pre-field Investigation Activities - We contacted Underground Services Alert (USA) prior to excavating borings so that utility companies/providers could mark utilities onsite.

- Field Exploration - A total of 8 exploratory soil borings were logged and sampled to evaluate subsurface conditions. The borings were drilled to depths ranging from 21.5 to 51.5 feet below the existing ground surface (bgs). Relatively undisturbed soil samples were obtained at selected intervals within the borings using a California Ring Sampler. Standard Penetration Tests (SPT) were conducted at selected depths and samples were obtained. Representative bulk soil samples were also collected at shallow depths from the borings. Logs of the geotechnical borings are presented in Appendix B. Approximate boring locations are shown on the accompanying Boring Location Map, Figure 2.

- Laboratory Tests - Laboratory tests were conducted on selected relatively undisturbed and bulk soil samples obtained during our field investigation. The laboratory testing program was designed to evaluate engineering characteristics of the onsite soil. Laboratory tests conducted during this investigation include:

    - In situ moisture content and dry density
    - Sieve analysis for grain-size distribution
    - Collapse potential
    - Expansion index
    - Maximum dry density and optimum moisture content
    - Water-soluble sulfate concentration in the soil
    - Resistivity, chloride content and pH

Results of the in situ dry density and moisture content tests are shown on the boring logs (Appendix B). Results of the remaining laboratory tests are provided in Appendix C.

- Engineering Analysis - Data obtained from our background review and field exploration was evaluated and analyzed to provide geotechnical conclusions and preliminary recommendations presented in the following sections.

- Report Preparation - Results of our geologic hazards review and geotechnical investigation have been summarized in this report, presenting our findings, conclusions and preliminary recommendations for the project.

2.0 FINDINGS

2.1 Regional Geologic Setting

The site is located within the Chino Basin in the northern portion of the Peninsular Ranges geomorphic province of California. Major structural features surround this region, including the Cucamonga fault and the San Gabriel/San Bernardino Mountains to the north, the Chino-Central Avenue fault and Puente/Chino Hills to the west and the San Jacinto fault to the east. This is an area of large-scale crustal disturbance as the relatively northwestward moving Peninsular Ranges Province collides with the Transverse Ranges Province (San Gabriel and San Bernardino Mountains) to the north. Several active or potentially active faults have been mapped in the region and are believed to accommodate compression associated with this tectonic collision. The site is located approximately 2.1 kilometers northeast of the Chino-Elsinore fault. This is a major fault zone marking the eastern flank of the Santa Ana Mountains.

2.2 Subsurface Soil Conditions

Based upon our review of pertinent geotechnical literature and our current subsurface exploration, the site is underlain by alluvial fan deposits. The alluvial soil encountered within our excavations generally consisted of sand and gravel with layers of lean clay interspersed. The soil was generally dry to moist and medium dense to dense. The in-situ moisture content within the upper approximately 20 feet generally ranged from 2 to 11 percent. More detailed descriptions of the subsurface conditions are presented on the boring logs in Appendix B.

2.2.1 Compressible and Collapsible Soil

Soil compressibility refers to a soil's potential for settlement when subjected to increased loads as from a new structure or fill surcharge. Based on our investigation, the alluvial soils encountered are considered slightly to moderately compressible. Partial removal and recompaction of this soil will be necessary to reduce the potential for adverse differential settlement of the proposed improvements.

Collapse potential (moisture sensitivity, sometimes referred to as 'hydrocollapse') refers to the potential settlement of a soil under existing stresses upon being wetted. Based on observations of the soil during our exploration and the results of laboratory testing, the onsite near-surface soils are expected to have a moderate collapse potential.

2.2.2 Expansive Soils

Expansive soils contain significant amounts of clay particles that swell considerably when wetted and shrink when dried. Structures constructed on these soils are subjected to large uplifting forces caused by the swelling. Without proper measures taken, heaving and cracking of both building foundations and slabs-on-grade could result.

A representative sample of the subsurface soil was tested for expansion potential and was found to have an expansion index of 6. The onsite soils are expected to have a very low to low expansion potential.

2.2.3 Sulfate Content

Water-soluble sulfates in soil can react adversely with concrete. However, concrete in contact with soil containing sulfate concentrations of less than 0.1 percent by weight is considered to have negligible sulfate exposure based on the American Concrete Institute (ACI) publication 318-08, Section 4.2 (ACI, 2008), adopted by the 2010 CBC (Section 1904A.2).

A near-surface soil sample was tested during this investigation for soluble sulfate content. The results of this test indicated a sulfate content of less than 0.02 percent by weight, indicating negligible sulfate exposure. As such, the soils exposed at pad grade are not expected to pose a significant potential for sulfate reaction with concrete.

2.2.4 Resistivity, Chloride and pH

Soil corrosivity to ferrous metals can be estimated by the soil's electrical resistivity, chloride content and pH. In general, soil having a minimum resistivity less than 2,000 ohm-cm is considered corrosive. Soil with a chloride content of 500 parts-per-million (ppm) or more is considered corrosive to ferrous metals.

As a screening for potentially corrosive soil, a representative soil sample was tested during this investigation to determine its minimum resistivity, chloride content, and pH. These tests indicated a minimum resistivity of approximately 4,750 ohm-cm, a chloride content of 626 ppm, and pH of 7.2. Based on the chloride content, the onsite soil is considered corrosive.

2.3 Groundwater

Regional groundwater data prepared by the United State Geological Survey indicates groundwater is present in the area at elevations of approximately 540 to 560 feet above mean sea level, or approximately 440 to 460 feet below the ground surface (USGS, 2012). Groundwater was not encountered in our 8 hollow-stem auger borings to a maximum depth of 51.5 feet below ground surface.

Groundwater levels at the site were approximately 300 feet below the ground surface in 1933 (CDWR, 1970); this depth is considered the historically highest groundwater level for the site.

2.4 Faulting and Seismicity

In general, the primary seismic hazards for sites in the region could include strong ground shaking and fault rupture. The potential for fault rupture and seismic shaking are discussed below.

2.4.1 Surface Faulting

Our review of available in-house literature indicates that there are no known active or potentially active faults that traverse the site, and the site is not located within a currently designated Alquist-Priolo (A-P) Earthquake Fault Zone (CGS, 2000) or County of Riverside Establish Fault Zone (Riverside County, 2012). The closest active fault to the site is the Chino-Elsinore fault, located approximately 2.1 km from the site.

The site is not known to be located on or near a pressure ridge. Based on our understanding of the current geologic framework, the potential for future surface rupture of active faults onsite is considered very low.

2.4.2 Seismic Design Parameters

The principal seismic hazard that could affect the site is ground shaking resulting from an earthquake occurring along several major active or potentially active faults in southern California. Design of the proposed improvements in accordance with current California Building Code (CBC) requirements is intended to reduce the impact of seismic shaking on the proposed improvements.

Recommended seismic design acceleration parameters in accordance with the 2010 CBC are presented in Table 1 below.

Table 1 - 2010 CBC Seismic Design Parameters

CBC Categorization/Coefficient	Design Value
Site Longitude (decimal degrees)	-117.5649
Site Latitude (decimal degrees)	33.8502
Site Class Definition (Table 1613.5.2)	D
Mapped Spectral Response Acceleration at 0.2s Period, $\mathbf{S_s}$ (Figure 1613.5(3))	1.82g
Mapped Spectral Response Acceleration at 1s Period, $\mathbf{S_1}$ (Figure 1613.5(4))	0.62g
Short Period Site Coefficient at 0.2s Period, $\mathbf{F_a}$ (Table 1613.5.3(1))	1.0
Long Period Site Coefficient at 1s Period, $\mathbf{F_v}$ (Table 1613.5.3(2)	1.5
Adjusted Spectral Response Acceleration at 0.2s Period, $\mathbf{S_{MS}}$ (Eq. 16-37)	1.82g
Adjusted Spectral Response Acceleration at 1s Period, $\mathbf{S_{M1}}$ (Eq. 16-38)	0.94g
Design Spectral Response Acceleration at 0.2s Period, $\mathbf{S_{DS}}$ (Eq. 16-39)	1.21g
Design Spectral Response Acceleration at 1s Period, $\mathbf{S_{D1}}$ (Eq. 16-40)	0.62g

2.4.3 Seismic Parameters for Geotechnical Evaluation

Peak Horizontal Ground Acceleration (PHGA) for the site was estimated using a probabilistic seismic hazard analysis, based on currently available earthquake and fault information. This analysis considers fault geometry, earthquake magnitude, and attenuation relationships. The site-specific seismic hazard contribution is based on the 2008 USGS national probabilistic seismic hazard model. The probabilistic seismic hazard analysis indicates that there is a 2 percent probability of exceeding a 0.98g PGA in a 50-year time period (recurrence interval of 2,475 years) for a site with alluvial soil conditions. Two-thirds of this value (or 0.65g) was used for liquefaction and seismic settlement analysis, based on the 2010 California Building Code. Hazard deaggregation was performed using the USGS Interactive Deaggregation Web Page, which indicates that the modal earthquake magnitude is approximately 6.8 (MW) at a distance on the order of 3.2 kilometers.

2.4.4 Historical Seismicity

We performed an evaluation of site historical seismicity with respect to significant past earthquakes (those recorded from the mid 1800's to 2012 with magnitudes 5 or greater) using the EQSEARCH computer program (Blake, 2000). This is a relatively simple analysis, based on epicenters, and does not include more complex characteristics of earthquakes, such as rupture length and direction; however, it gives an idea of past seismicity at the site. This analysis suggests that the largest ground acceleration at the site from historical earthquakes is estimated to have been roughly 0.29g.

2.5 Secondary Seismic Hazards

In general, secondary seismic hazards for sites in the region could include soil liquefaction, earthquake-induced settlement, lateral displacement, landsliding, and earthquake induced flooding. The potential for secondary seismic hazards at the site is discussed below.

2.5.1 Liquefaction Potential

Liquefaction is the loss of soil strength or stiffness due to a buildup of pore-water pressure during severe ground shaking. Liquefaction is associated primarily with loose (low density), saturated, fine- to medium-grained, cohesionless soils. Effects of liquefaction can include sand boils, settlement, and bearing capacity failures below structural foundations.

Current and historical groundwater levels at the site are greater than 300 feet below the ground surface. Based on this information, liquefaction is not a hazard at this site.

2.5.2 Seismically Induced Settlement

Seismically induced settlement consists of dry dynamic settlement (above groundwater) and liquefaction-induced settlement (below groundwater). During a strong seismic event, seismically induced settlement can occur within loose to moderately dense sandy soil due to reduction in volume during and shortly after an earthquake event. Settlement caused by ground shaking is often nonuniformly distributed, which can result in differential settlement.

We have performed analyses to estimate potential seismically induced settlement using the LiquefyPro computer program by CivilTech Software (2008). The results of our analyses suggest that the onsite soils are susceptible to as much as 2.5 inches of seismic settlement based on the design earthquake. Due to the relatively laterally uniform nature of soils, differential settlement due to seismic loading is assumed to be 1.25 inches over a horizontal distance of 40 feet. A summary of seismic settlement analysis is included in Appendix D.

2.6 Slope Stability and Landslides

Since significant slopes are not located on or near the site, slope instability and landslides are not an issue at the site. Therefore, the site is not considered susceptible to slope instability. As such, slope stability evaluation (including development of static and dynamic strength parameters, pseudostatic slope stability coefficients, dynamic site conditions evaluation, and slope stability mitigation) is not warranted for this project.

2.7 Flooding Potential

The site is not located within a 100-year flood zone or a 500-year flood zone on the Federal Emergency Management Agency (FEMA) Flood Map for the site.

3.0 CONCLUSIONS AND RECOMMENDATIONS

3.1 General Conclusions

Based on this investigation, construction of the proposed residential development is feasible from a geotechnical standpoint. No severe geologic or soils related issues were identified that would preclude development of the site for the proposed improvements. The most significant geotechnical issues at the site are those related to the potential for strong seismic shaking, the potential for moderate seismic settlement, and potentially compressible soils. Good planning and design of the project can limit the impact of these constraints. Remedial recommendations for these and other geotechnical issues are provided in the following sections.

3.2 Earthwork and Grading

Grading should be performed in accordance with the General Earthwork and Grading Specifications presented in Appendix E, unless specifically revised or amended below or by future recommendations based on final development plans.

3.2.1 Site Preparation

Prior to construction, the areas of the proposed improvements should be cleared of vegetation, trash, and debris. These stumps should be completely removed. Any underground obstructions onsite that interfere with the proposed foundations should be removed. Efforts should be made to locate any existing utility lines. Those lines should be removed or rerouted if they interfere with the proposed construction, and the resulting cavities should be backfilled and compacted as recommended in Sections 3.2.3 and 3.10.

3.2.2 Overexcavation and Recompaction

To reduce the potential for adverse total and differential settlement of the proposed structures, the underlying subgrade soil should be prepared in such a manner that a uniform response to the applied loads is achieved.

For the proposed buildings, we recommend that the onsite soils be removed to a minimum depth of 5 feet below the existing ground surface or 3 feet below the bottom of the proposed footings, whichever is deeper. The removal bottom should extend horizontally beyond the proposed structures a minimum of 5 feet from the outside edges of the footings, or distance equal to the depth of overexcavation below the footings, whichever is farther. During overexcavation, the soil conditions should be observed by Leighton to further evaluate these recommendations based on actual field conditions encountered. Deeper overexcavation and recompaction may be recommended based on the conditions exposed.

Areas outside of the proposed buildings planned for asphalt or concrete pavement (such as parking areas or fire lanes), flatwork (such as sidewalks), site walls and low retaining walls, and areas to receive fill should be overexcavated to a minimum depth of 24 inches below existing grade or 12 inches below proposed subgrade, whichever is deeper.

After completion of the overexcavation, and prior to fill placement, the exposed surfaces should be scarified to a minimum depth of 6 inches, moisture conditioned to or slightly above optimum moisture content, and recompacted to a minimum 90 percent relative compaction, relative to the ASTM D 1557 laboratory maximum density.

3.2.3 Fill Placement and Compaction

The onsite soil is suitable for use as compacted structural fill, provided it is free of debris and oversized material (greater than 8 inches in largest dimension). Any soil to be placed as fill, whether onsite or imported material, should be accepted by Leighton and Associates.

All fill soil should be placed in thin, loose lifts, moisture-conditioned, as necessary, with moisture contents of at least optimum, and compacted to a minimum 90 percent relative compaction as determined by ASTM Test Method D1557. Aggregate base for pavement should be compacted to a minimum of 95 percent relative compaction.

3.2.4 Import Fill Soil

Import soil to be placed as fill should be geotechnically accepted by Leighton. Preferably at least 3 working days prior to proposed import to the site, the contractor should provide Leighton pertinent information of the proposed import soil, such as location of the soil, whether stockpiled or native in place, and pertinent geotechnical reports if available. We recommend that a Leighton representative visit the proposed import site to observe the soil conditions and obtain representative soil samples. Potential issues may include soil that is more expansive than onsite soil, soil that is too wet, soil that is too rocky or too dissimilar to onsite soils, oversize material, organics, debris, etc.

3.2.5 Shrinkage and Subsidence

The change in volume of excavated and recompacted soil varies according to soil type and location. This volume change is represented as a percentage increase (bulking) or decrease (shrinkage) in volume of fill after removal and recompaction. Field and laboratory data used in our calculations included laboratory-measured maximum dry densities for soil types encountered at the subject site and the measured in-place densities of soils encountered. We preliminarily estimate the following earth volume changes will occur during grading. These are rough estimates:

Shrinkage (Approximate)	10 to 15 percent
Subsidence (Approximate)	0.1 foot

These shrinkage estimates do not include any loss due to rock removal. The level of fill compaction, variations in the dry density of the existing soils and other factors influence the amount of volume change.

It should be noted that subsidence, as referred to above, is settlement of in-place earth materials due to heavy equipment processing. It does not refer to potential settlement due to placement of additional loads from new fill (i.e., raising of grades).

These shrinkage values are general guide values. Actual values will vary, due to the varying soil conditions and varying construction techniques. It is not possible to estimate exact values. Therefore, as with any grading project, some earthwork volume adjustments should be anticipated during grading.

3.3 Foundations

Overexcavation and recompaction of the footing subgrade soil should be performed as detailed in Section 3.1. The following recommendations are based on the onsite soil conditions and soils with a low expansion potential.

Due to the potential for seismic settlement, we recommend that the proposed structures be constructed with a foundation that is designed to tolerate 1½ inches of differential settlement over a horizontal distance of 40 feet.

Differential settlement due to static loading is estimated at ½ inch over a horizontal distance of 30 feet between or along similarly loaded footings. Since settlement is a function of footing sustained load, size and contact bearing pressure, differential settlement can be expected between adjacent columns or walls where a large differential loading condition exists.

3.3.1 Minimum Embedment and Width

Based on our investigation, footings for conventional one- to two-story structures should have a minimum embedment of 18 inches, with a minimum width of 24 and 15 inches for isolated and continuous footings, respectively.

3.3.2 Allowable Bearing

An allowable bearing pressure of 1,800 pounds-per-square-foot (psf) may be used, based on the minimum embedment depth and width above. This allowable bearing value may be increased by 250 psf per foot increase in depth or width to a maximum allowable bearing pressure of 3,500 psf. These allowable bearing pressures are for total dead load and sustained live loads. Footing reinforcement should be designed by the structural engineer.

3.3.3 Lateral Load Resistance

Soil resistance available to withstand lateral loads on a shallow foundation is a function of the frictional resistance along the base of the footing and the passive resistance that may develop as the face of the structure tends to move into the soil. The frictional resistance between the base of the foundation and the subgrade soil may be computed using an allowable coefficient of friction of 0.35. The passive resistance may be computed using an allowable (factor of safety of 1.5 applied) equivalent fluid pressure of 250 pounds per cubic foot (pcf), assuming there is constant contact between the footing and undisturbed soil. Friction and passive pressure may be combined without reduction, provided the footings can move laterally sufficiently to develop passive pressure (approximately ¼ to ½ inch); otherwise, friction alone should be assumed.

3.3.4 Increase in Bearing and Friction – Short Duration Loads

For the case of short term loading (seismic and wind loading), an increase of 1/3 would apply to the bearing pressure and friction values. The ultimate bearing pressure is assumed to be roughly three times the allowable bearing pressure. However, this ultimate pressure only considers structural failure/collapse (life safety) and not structural damage or significant cosmetic damage. Excessive settlement may occur before the ultimate bearing pressure is attained.

3.3.5 Settlement Estimates

The recommended overexcavation, relative compaction and allowable bearing pressure are based on a total allowable, post construction settlement of 1 inch and potential seismic settlement of 3 inches. Differential settlement due to static loading is estimated at ½ inch over a horizontal distance of 40 feet between or along similarly loaded footings, and 1½ inches in 40 feet for seismic settlement. Since settlement is a function of footing sustained load, size and contact bearing pressure, differential settlement can be expected between adjacent columns or walls where a large differential loading condition exists.

3.4 Additional Recommendations for Slabs-On-Grade

Concrete slabs-on-grade should be designed by the structural engineer in accordance with the current CBC for a soil with a very low expansion potential and the settlement estimates listed in Section 3.3.5. Observation and possibly testing to confirm the expansion potential of the near surface soil should be conducted during site grading.

The following minimum slab recommendations should be used. More stringent requirements may be required by agencies, the structural engineer, the architect, or the CBC. Slabs-on-grade should have the following minimum recommended components:

Subgrade Moisture Conditioning: The subgrade soil should be moisture conditioned to at least 2 percentage points above optimum moisture content to a minimum depth of 12 inches prior to placing steel or concrete.

Concrete Thickness and Structural Design: Thickness of slabs-on-grade should be designed by the structural engineer, but should be at least 4½ inches thick (this is referring to the actual minimum thickness, not the nominal thickness). Reinforcing steel should be designed by the structural engineer, but as a minimum (for conventionally reinforced slabs) should be No. 4 rebar placed at 18 inches on center, each direction, mid-depth in the slab.

Minor cracking of the concrete as it cures, due to drying and shrinkage is normal and should be expected. However, cracking is often aggravated by a high water/cement ratio, high concrete temperature at the time of placement, small nominal aggregate size, aggregate that is not sufficiently clean, and rapid moisture loss due to hot, dry, and/or windy weather conditions during placement and curing. Cracking due to temperature and moisture fluctuations can also be expected. Low slump concrete can reduce the potential for shrinkage cracking. Additionally, reinforcement in slabs and foundations can generally reduce the potential for shrinkage cracking. The structural engineer should consider these and other pertinent concrete design and construction considerations in slab design and specifications.

3.4.1 Slab Underlayment for Moisture Vapor Retarding

Because moisture vapor from the underlying soils will be transmitted through slabs-on-grade without preventive measures, slab underlayment for moisture vapor retarding should be designed by qualified professionals (such as the structural engineer and/or architect) where control of moisture vapor transmission through slabs is considered important to this project (such as where moisture-sensitive floor coverings or equipment are planned). Slab underlayment typically includes a moisture vapor retarder membrane (such as 10-mil thick or greater), a capillary break (such as clean sand or crushed stone), and provisions for protection of the vapor retarder during construction (such as sand under and possibly over the vapor retarder). The structural engineer and/or architect should specify pertinent slab and concrete design parameters, such as whether a sand blotter/capillary break layer should be placed over the vapor retarder, and details of a capillary break system.

Moisture retarders can reduce, but not eliminate moisture vapor rise from the underlying soils up through the slab. Moisture retarders should be designed and constructed in accordance with applicable American

Concrete Institute, Portland Cement Association, Post-Tensioning Institute, ASTM International, and California Building Code requirements and guidelines.

Leighton does not practice in the field of moisture vapor transmission evaluation/mitigation, since this does not fall under the geotechnical discipline. Therefore, we recommend that a qualified person, such as the flooring subcontractor, structural engineer, and/or architect, be consulted to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. That person (or persons) should provide recommendations for mitigation of potential adverse impact of moisture vapor transmission on various components of the structures as deemed appropriate. In addition, the recommendations in this report and our services in general are not intended to address mold prevention, since we, along with geotechnical consultants in general, do not practice in the area of mold prevention. If specific recommendations are desired, a professional mold prevention consultant should be contacted.

3.5 Seismic Design Parameters

Seismic parameters presented in this report should be considered during project design. In order to reduce the effects of ground shaking produced by regional seismic events, seismic design should be performed in accordance with the 2010 CBC. The 2010 seismic design parameters listed in Table 1 of Section 2.4.2 of this report should be considered for the seismic analysis of the subject site.

3.6 Lateral Earth Pressures

We recommend that retaining walls be backfilled with very low expansive soil and constructed with a backdrain in accordance with the recommendations provided on Figure 3, *Retaining Wall Backfill and Subdrain Detail*. Using expansive soil as retaining wall backfill will result in higher lateral earth pressures exerted on the wall and are, therefore, not recommended. Based on these recommendations, the following parameters may be used for the design of conventional retaining walls.

Table 2 - Retaining Wall Design Parameters

Static Equivalent Fluid Pressure (pcf)	
Condition	Level Backfill
Active	35
At-Rest	55
Passive (ultimate)	375 (Maximum 3,500 psf)

The above values do not contain an appreciable factor of safety, so the structural engineer should apply the applicable factors of safety and/or load factors during design.

Cantilever walls that are designed to yield at least 0.001H, where H is equal to the wall height, may be designed using the active condition. Rigid walls and walls braced at the top should be designed using the at-rest condition.

Passive pressure is used to compute soil resistance to lateral structural movement. In addition, for sliding resistance, a frictional resistance coefficient of 0.35 may be used at the concrete and soil interface. The lateral passive resistance should be taken into account only if it is ensured that the soil providing passive resistance, embedded against the foundation elements, will remain intact with time. A soil unit weight of 120 pcf may be assumed for calculating the actual weight of the soil over the wall footing.

In addition to the above lateral forces due to retained earth, surcharge due to improvements, such as an adjacent structure or traffic loading, should be considered in the design of the retaining wall. Loads applied within a 1:1 projection from the surcharging structure on the stem of the wall should be considered in the design. A third of uniform vertical surcharge-loads should be applied at the surface as a horizontal pressure on cantilever (active) retaining walls, while half of uniform vertical surcharge-loads should be applied as a horizontal pressure on braced (at-rest) retaining walls. To account for automobile parking surcharge, we suggest that a uniform horizontal pressure of 100 psf (for restrained walls) or 70 psf (for cantilever walls) be added for design, where autos are parked within a horizontal distance behind the retaining wall less than the height of the retaining wall stem.

Conventional retaining wall footings should have a minimum width of 24 inches and a minimum embedment of 12 inches below the lowest adjacent grade. An allowable bearing pressure of 1,800 psf may be used for retaining wall footing design, based on the minimum footing width and depth. This bearing value may be increased by 250 psf per foot increase in width or depth to a maximum allowable bearing pressure of 3,500 psf.

3.7 Cement Type and Corrosion Protection

Based on the results of laboratory testing, concrete structures in contact with the onsite soil will have negligible exposure to water-soluble sulfates in the soil. Therefore, common Type II cement may be used for concrete construction. Concrete should be designed in accordance with ACI 318-08, Section 4.2 (ACI, 2008), adopted by the 2010 CBC (Section 1904.2).

Based on our laboratory testing, the onsite soil is considered corrosive to ferrous metals. Corrosion protection of underground metallic utilities will be required. A corrosion engineer can be consulted if specific recommendations are desired. Corrosion information presented in this report should be provided to your underground utility contractors.

3.8 Pavement Design

We have conducted pavement design as outlined in the current Caltrans Highway Design Manual, our geotechnical experience in the site vicinity, and assuming an R-value of 50. We also reviewed the City of Corona Street Design Table, Standard Plan No. 100. Based on our review, the city standard should be used for design. Preliminary flexible pavement sections may consist of the following for the street types/traffic Indices (TI) indicated. Final pavement design should be based on the Traffic Index determined by the project civil engineer and R-value testing provided near the end of grading.

Table 3 - Asphalt Pavement Section Thickness

Traffic Index	Asphaltic Concrete (AC) Thickness (inches)	Class 2 Aggregate Base (AB) Thickness (inches)
Low Volume, 5	4	6
Local, 5.5	4	6
Collector, 8	5	8

If asphalt pavement is to be constructed prior to construction, the full pavement thickness should be placed to support heavy construction traffic.

In areas where rigid concrete pavement is planned and trucks may drive on this pavement, we recommend 6½ inches of Portland Cement Concrete (PCC) with a 28-day compressive strength of 4,000 psi over 6 inches of aggregate base placed on prepared subgrade soil (see Section 3.2.2). Reinforcement should be specified by the structural engineer, but should be a minimum of #3 rebar at 18 inches on center each way. The PCC pavement sections should be provided with crack-control joints spaced no more than 14 feet on center each way. If sawcuts are used, they should have a minimum depth of ¼ of the slab thickness and made within 24 hours of concrete placement. We recommend that sections be as nearly square as possible.

PCC sidewalks should be at least 4 inches thick over prepared subgrade soil, with construction joints no more than 8 feet on center each way, with sections as nearly square as possible. Use of reinforcing, such as welded-wire mesh, will help reduce severity of cracking.

All pavement construction should be performed in accordance with the Standard Specifications for Public Works Construction. Field observations and periodic testing, as needed during placement of the base course materials, should be undertaken to ensure that the requirements of the standard specifications are fulfilled. Prior to placement of aggregate base, the subgrade soil should be processed to a minimum depth of 6 inches, moisture-conditioned, as necessary, and recompacted to a minimum of 90 percent relative compaction. Aggregate base should be moisture conditioned, as necessary, and compacted to a minimum of 95 percent relative compaction.

3.9 Temporary Excavations

All temporary excavations, including utility trenches, retaining wall excavations and other excavations should be performed in accordance with project plans, specifications and all OSHA requirements, and the current edition of the California Construction Safety Orders, latest edition.

No surcharge loads should be permitted within a horizontal distance equal to the height of cut or 5 feet, whichever is greater from the top of the slope, unless the cut is shored appropriately. Excavations that extend below an imaginary plane inclined at 45 degrees below the edge of any adjacent existing site foundation should be properly shored to maintain support of the adjacent structures.

Cantilever shoring should be designed based on the active fluid pressure presented in the retaining wall section. If excavations are braced at the top and at specific design intervals, the active pressure may then be approximated by a rectangular soil pressure distribution with the pressure per foot of width equal to 22H, where H (feet) is equal to the depth of the excavation being shored.

During construction, the soil conditions should be regularly evaluated to verify that conditions are as anticipated. The contractor should be responsible for providing the "competent person" required by OSHA, standards to evaluate soil conditions. Close coordination between the competent person and Leighton and Associates should be maintained to facilitate construction while providing safe excavations.

3.10 Trench Backfill

Utility-type trenches onsite can be backfilled with onsite material, provided it is free of debris, significant organic material and oversized material (greater than 3 inches for trench backfill within 3 feet of a pipe, and 6 inches for trench backfill above). Prior to backfilling the trench, pipes should be bedded and shaded in a granular material that has a sand equivalent of 30 or greater. We recommend that open-graded crushed rock or similar material not be used as bedding material, unless special provisions are implemented to limit the migration of surrounding soil into the open-graded material, such as the use of filter fabric around the open-graded material. The bedding material should extend 12 inches above the top of the pipe. The bedding/shading sand should be densified in-place by mechanical means, or in areas where the trench walls and bottom soil have a minimum sand equivalent of 15, the bedding sand may be jetted. Bedding sand should be placed in accordance with the Standard Specifications for Public Works Construction – Greenbook (Public Works Standard, Inc., 2009), current edition. The native soil fill should be placed in loose layers, moisture conditioned, as necessary, and mechanically compacted using a minimum standard of 90 percent relative compaction based on ASTM D 1557. The thickness of layers should be based on the compaction equipment used in accordance with the current Greenbook.

3.11 Surface Drainage

Positive surface drainage should be provided to direct surface water away from structures and towards suitable collective drainage facilities. Surface drainage should be provided to prevent ponding of water adjacent to the structures. In general, the area around the buildings should slope away from the buildings. Care should be taken to avoid heavy irrigation, and under-irrigation should also be avoided.

3.12 Limitations and Additional Geotechnical Services

The geotechnical recommendations presented in this report are based on subsurface conditions as interpreted from limited subsurface explorations and limited laboratory testing. Our geotechnical recommendations provided in this report are based on information available at the time the report was prepared and may change as plans are developed. However, additional geotechnical study and analysis may be required based on final development plans. Leighton and Associates should review the site and grading plans when available and comment further on the geotechnical aspects of the project. Geotechnical observation and testing should be conducted during excavation and all phases of grading operations. Our conclusions and preliminary recommendations should be reviewed and verified by Leighton and Associates during construction and revised accordingly if geotechnical conditions encountered vary from our findings and interpretations. Changes in subsurface conditions can and do occur over time. Therefore, our findings, conclusions, and recommendations presented in this report are based on the assumption that Leighton and Associates will provide geotechnical observation and testing during construction. Please refer to the ASFE “Important Information about Your Geotechnical Engineering Report” presented at the end of this report.

Environmental services were not included as part of this study. This report was prepared for the sole use of TRG Land, Inc. for application to the design of the proposed project in accordance with generally accepted geotechnical engineering practices at this time in California.

Geotechnical observation and testing should be provided:

- After completion of site demo/clearing.
- During overexcavation of compressible soil.
- During compaction of all fill materials.
- After excavation of all footings and prior to placement of concrete.
- During utility trench backfilling and compaction.
- During pavement subgrade and base preparation.
- When any unusual conditions are encountered.