PRELIMINARY GEOTECHNICAL INVESTIGATION
SAN LEANDRO TECH CAMPUS
SAN LEANDRO CROSSINGS
San Leandro, California

1.0 INTRODUCTION

This report presents the results of the preliminary geotechnical investigation performed by Rockridge Geotechnical, Inc. for the proposed San Leandro Tech Campus development to be constructed in San Leandro, California. The site is located on the east side of Alvarado Street, southeast of its intersection with Davis Street, between West Estudillo Avenue and Thornton Street, as shown on the Site Location Map and Site Plan, Figures 1 and 2, respectively.

The site is a relatively level, trapezoidal-shaped lot with plan dimensions of about 1,500 feet in the north-south direction by 200 to 340 feet in the east-west direction. The site is currently vacant. Plans are to develop the site by constructing three six-story office buildings with 20,000- to 22,000-square-foot floor plates. The buildings will be constructed at-grade. The development will also include surface parking at the southern portion of the site.

According to Structural Engineers, Inc. (SEI), the project structural engineer, interior and exterior column dead plus live loads will be on the order of 715 kips and 410 kips, respectively.

2.0 SCOPE OF WORK

During our previous investigation in 2012 for the San Leandro Crossings project, we drilled four borings and performed five cone penetration tests (CPTs) in the northeastern portion of the site; two borings and three CPTs were advanced in or adjacent to the footprints of the current proposed buildings. The approximate locations of the previous borings and CPTs are shown on Figure 2. Logs of the previous borings and CPTs are presented in Appendix A of this report. The results of laboratory tests performed on selected soil samples from the 2012 investigation are presented in Appendix B.

Our preliminary geotechnical investigation for this project was performed in accordance with our proposal dated October 16, 2013. We reviewed and relied upon information from our 2012 field investigation at the site and performed engineering analyses to develop preliminary conclusions and recommendations regarding:

- soil and groundwater conditions at the site
- site seismicity and seismic hazards, including the potential for liquefaction and liquefaction-induced ground failure
- the most appropriate foundation type for the proposed structures
- design criteria for the recommended foundation type, including vertical and lateral capacities
- estimates of foundation settlement
- subgrade preparation for slab-on-grade floors and exterior flatwork
- site grading and excavation, including criteria for the fill quality and compaction
- flexible (asphalt concrete), rigid (Portland cement concrete), and porous pavement design sections
- 2013 California Building Code (CBC) site class and design spectral response acceleration parameters
- construction considerations.

The California Geological Survey (CGS) has prepared a map titled *State of California Seismic Hazard Zones, San Leandro Quadrangle*, dated 14 February 2003. This map was prepared in accordance with the Seismic Hazards Mapping Act of 1990 and is presented as Figure 3. The site is within a designated liquefaction hazard zone. The CGS has also recommended the content for site investigation reports within seismic hazard zones in the State of California Special Publication (SP) 117, titled *Guidelines for Evaluating and Mitigating Seismic Hazard Zones in California* (2008). The liquefaction evaluation performed for this study was prepared in general accordance with the recommendations presented in SP 117.

3.0 PREVIOUS FIELD INVESTIGATION AND LABORATORY TESTING

Our 2012 field investigation consisted of drilling four test borings, performing five CPTs, and performing laboratory testing on selected soil samples. Prior to advancing the test borings and CPTs, we obtained a drilling permit from Alameda County Public Works Agency (ACPWA) and contacted Underground Service Alert (USA) to notify them of our work, as required by law. Details of the field investigation and laboratory testing are described below.

3.1 Test Borings

Four test borings, designated as Borings B-1 through B-4, were drilled on October 10 and November 13, 2012 by Exploration Geoservices of San Jose, California at the approximate locations shown on Figure 2. The borings were drilled using a truck-mounted drill rig equipped with eight-inch-diameter hollow-stem augers. The four borings were drilled to depths between 30 and 31.5 feet below the ground surface (bgs).

During drilling, our field engineer logged the soil encountered and obtained representative samples for visual classification and laboratory testing. The logs of borings are presented in Appendix A. The soil encountered in the borings was classified in accordance with the classification chart presented in Appendix A.

Soil samples were obtained using the following samplers:

- Sprague and Henwood (S&H) split-barrel sampler with a 3.0-inch outside diameter and 2.5-inch inside diameter, lined with 2.43-inch inside diameter brass tubes.
- Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside and 1.5-inch inside diameter, without liners.
- Shelby Tube (ST) sampler with a 3.0-inch outside diameter and a 2.875-inch inside diameter.

The samplers were driven with a 140-pound, down-hole, wireline hammer falling about 30 inches per drop. The samplers were driven up to 18 inches and the hammer blows required to drive the samplers were recorded every six inches and are presented on the boring logs. A "blow count" is defined as the number of hammer blows per six inches of penetration or 50 blows for six inches or less of penetration. The blow counts required to drive the S&H and SPT samplers were converted to approximate SPT N-values using factors of 0.6 and 1.0, respectively, to account for sampler type and approximate hammer energy. The blow counts used for this conversion were the last two blow counts. The converted SPT N-values are presented on the boring logs.

The ST sampler was pushed hydraulically into the soil; the pressure required to advance the sampler is shown on the boring log, measured in pounds per square inch (psi).

Upon completion of drilling, the boreholes were backfilled with cement grout under the observation of the ACPWA inspector. The soil cuttings generated by the borings were spread on the ground surface near the boreholes.

3.2 Cone Penetration Tests

Five CPTs, designated as CPT-1 through CPT-5, were performed to provide in-situ soil data at the approximate locations shown on Figure 2. John Sarmiento & Associates of Orinda, California advanced the CPTs on September 25, 2012, each to a depth of about 50 feet bgs.

The CPTs were performed by hydraulically pushing a 1.4-inch-diameter cone-tipped probe with a projected area of 10 square centimeters into the ground. The cone-tipped probe measured tip resistance and the friction sleeve behind the cone tip measured frictional resistance. Electrical strain gauges within the cone continuously measured soil parameters for the entire depth advanced. Soil data, including tip resistance and frictional resistance, were recorded by a computer while the test was conducted. Accumulated data were processed by computer to provide engineering information such as the types and approximate strength characteristics of the soil encountered. The CPT logs, showing tip resistance and friction ratio by depth, as well as interpreted SPT N-values, soil shear strength parameters, and soil classifications, are presented in Appendix A. The classification chart for the CPT logs is also presented in Appendix A. Upon completion, the CPTs were backfilled with cement grout.

3.3 Laboratory Testing

We re-examined the soil samples obtained from our borings to confirm the field classifications and select representative samples for laboratory testing. Soil samples were tested to measure moisture content, dry density, plasticity index, strength, compressibility, and resistance value (R-value. The results of the laboratory tests are presented on the boring logs and in Appendix B.

4.0 SUBSURFACE CONDITIONS

The results of our 2012 field investigation indicate the site is blanketed by up to 5-1/2 feet of undocumented fill consisting of medium dense to very dense gravel and gravel with clay and sand. The fill is underlain by alluvium consisting of interbedded clay, sand, and gravel. The upper alluvium, extending to depths of 23 to 29 feet bgs, generally consists of clay with varying sand content. Atterberg limits tests indicate the near-surface clay is moderately expansive. The consistency of the clay is generally stiff to very stiff across the site to depths of about 20 to 25 feet bgs. At depths between 20 and 29 feet bgs, we encountered 3- to 5-foot-thick layers of medium stiff clay in the borings and CPTs.

Beneath the upper clay layers are interbedded layers of alluvial soil that extended to the maximum depth explored of 50 feet bgs. The alluvial soils generally consist of medium dense to dense sand and gravel with varying clay content, and stiff to very stiff clay with varying sand and gravel contents.

Groundwater was measured at depths ranging from 23-1/2 to 26 feet bgs in the borings and CPTs. Considering the borings and CPTs encountered predominantly granular soils below a depth of 23 to 29 feet, we anticipate the stabilized groundwater level is close to the measured depths. We anticipate the depth to groundwater will vary several feet seasonally.

5.0 SEISMIC CONSIDERATIONS

5.1 Regional Seismicity

The site is located in the Coast Ranges geomorphic province of California that is characterized by northwest-trending valleys and ridges. These topographic features are controlled by folds and faults that resulted from the collision of the Farallon plate and North American plate and subsequent strike-slip faulting along the San Andreas Fault system. The San Andreas Fault is more than 600 miles long from Point Arena in the north to the Gulf of California in the south. The Coast Ranges province is bounded on the east by the Great Valley and on the west by the Pacific Ocean.

The major active faults in the area are the Hayward, San Andreas, Calaveras, and San Gregorio faults. These and other faults in the region are shown on Figure 4. For these and other active faults within a 50-kilometer radius of the site, the distance from the site and estimated mean characteristic Moment magnitude[1] [2007 Working Group on California Earthquake Probabilities (WGCEP) (USGS 2008) and Cao et al. (2003)] are summarized in Table 1.

[1] Moment magnitude is an energy-based scale and provides a physically meaningful measure of the size of a faulting event. Moment magnitude is directly related to average slip and fault rupture area.

TABLE 1
Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Mean Characteristic Moment Magnitude
Total Hayward	3	Northeast	7.00
Total Hayward-Rodgers Creek	3	Northeast	7.33
Total Calaveras	17	East	7.03
Mount Diablo Thrust	20	Northeast	6.70
Green Valley Connected	25	Northeast	6.80
N. San Andreas - Peninsula	26	West	7.23
N. San Andreas (1906 event)	26	West	8.05
Monte Vista-Shannon	33	South	6.50
Greenville Connected	34	East	7.00
San Gregorio Connected	36	West	7.50
N. San Andreas - North Coast	37	West	7.51
Great Valley 5, Pittsburg Kirby Hills	43	Northeast	6.70
Rodgers Creek	47	Northwest	7.07
West Napa	50	North	6.70

Since 1800, four major earthquakes have been recorded on the San Andreas Fault. In 1836, an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale occurred east of Monterey Bay on the San Andreas Fault (Toppozada and Borchardt 1998). The estimated Moment magnitude, M_w, for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to an M_w of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas Fault from Shelter Cove to San Juan Bautista approximately 470 kilometers in length. It had a maximum intensity of XI (MM), an M_w of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The most recent earthquake to affect the Bay Area was the Loma Prieta Earthquake of 17 October 1989 with an M_w of 6.9. This earthquake occurred in the Santa Cruz Mountains about 80 kilometers south of the site.

In 1868, an earthquake with an estimated maximum intensity of X on the MM scale occurred on the southern segment (between San Leandro and Fremont) of the Hayward Fault. The estimated M_w for the earthquake is 7.0. In 1861, an earthquake of unknown magnitude (probably an M_w of about 6.5) was reported on the Calaveras Fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill earthquake ($M_w = 6.2$).

The U.S. Geological Survey's (USGS) 2007 WGCEP has compiled the earthquake fault research for the San Francisco Bay area in order to estimate the probability of fault segment rupture. They have determined that the overall probability of moment magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Region during the next thirty years is 63 percent. The highest probabilities are assigned to the Hayward/Rodgers Creek Fault and the northern segment of the San Andreas Fault. These probabilities are 31 and 21 percent, respectively (USGS 2008).

5.2 Geologic Hazards

Because the project site is in a seismically active region, we evaluated the potential for earthquake-induced geologic hazards including ground shaking, ground surface rupture, liquefaction,[2] lateral spreading,[3] and cyclic densification[4]. We used the results of our 2012 field investigation to evaluate the potential of these phenomena occurring at the project site. Our evaluation of site seismic hazards was performed in general accordance with the guidelines presented in SP 117.

5.2.1 Ground Shaking

The ground shaking intensity felt at the project site will depend on: the size of the earthquake (magnitude), the distance from the site to the fault source, the directivity (focusing of earthquake energy along the fault in the direction of the rupture), and subsurface conditions. The site is about 3 kilometers from the Hayward Fault. Therefore, the potential exists for a large earthquake to induce strong to very strong ground shaking at the site during the life of the project.

5.2.2 Ground Surface Rupture

Historically, ground surface displacements closely follow the trace of geologically young faults. The site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act, and no known active or potentially active faults exist on the site. We therefore conclude the risk of fault offset at the site from a known active fault is very low. In a seismically active area, the remote possibility exists for future faulting in areas where no faults previously

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[2] Liquefaction is a phenomenon where loose, saturated, cohesionless soil experiences temporary reduction in strength during cyclic loading such as that produced by earthquakes.

[3] Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported downslope or in the direction of a free face by earthquake and gravitational forces.

[4] Cyclic densification is a phenomenon in which non-saturated, cohesionless soil is compacted by earthquake vibrations, causing ground-surface settlement.

existed; however, we conclude the risk of surface faulting and consequent secondary ground failure from previously unknown faults is also very low.

5.2.3 Cyclic Densification

Seismically induced compaction or cyclic densification of non-saturated sand (sand above the groundwater table) caused by earthquake vibrations may result in differential settlement. Based on the subsurface data from our 2012 field investigations, we conclude the soil above the groundwater table contains sufficient clay that the risk of cyclic densification is very low.

5.2.4 Liquefaction and Associated Hazards

Liquefaction is a phenomenon in which saturated soil temporarily loses strength from the build-up of excess pore water pressure, especially during earthquake-induced cyclic loading. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits. Flow failure, lateral spreading, differential settlement, loss of bearing strength, ground fissures and sand boils are evidence of excess pore pressure generation and liquefaction. We evaluated liquefaction potential at the site in accordance with SP 117, as described below.

SP 117 states that liquefaction analyses should be performed using subsurface information from rotary-wash borings and/or CPTs. We used the results of the CPTs to evaluate the potential for liquefaction to occur at the site.

Our analyses were performed using a high groundwater depth of 20 feet bgs. In accordance with the 2013 CBC, we used a peak ground acceleration of 0.803 times gravity (g) in our liquefaction evaluation; this peak ground acceleration is consistent with the Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration adjusted for site effects (PGA_M). We also used a Moment magnitude 7.33 earthquake, which is consistent with the mean characteristic Moment magnitude for the Hayward Fault, as presented in Table 1.

Thin potentially liquefiable soil layers or lenses were encountered at depths ranging from 23 to 50 feet bgs at the site. These layers are typically 1 to 4 feet thick and are generally at transition zones between fine-grained (i.e., silt and clay) and granular soil layers. Our engineering analyses indicate that settlement resulting from post-liquefaction reconsolidation at the site could be up to 2 inches and differential settlement may be up to 1 inch over a horizontal distance of 30 feet.

Our analysis indicates the non-liquefiable soil overlying the potentially liquefiable soil layers is sufficiently thick and the potentially liquefiable layers are sufficiently thin such that the potential for surface manifestations from liquefaction, such as sand boils, and loss of bearing capacity for shallow foundations are low.

Lateral spreading occurs when a continuous layer of soil liquefies at depth and the soil layers above move toward an unsupported face, such as a shoreline slope, or in the direction of a regional slope or gradient. Based on the discontinuous nature and depth of the potentially liquefiable layers and the lack of controlling boundary conditions, we believe the potential for lateral spreading to occur at the project site is very low.

6.0 DISCUSSIONS AND PRELIMINARY CONCLUSIONS

Based on the results of our previous field investigation and laboratory testing, and our engineering analyses, we conclude that the proposed project can be developed as planned. The primary geotechnical concerns are:

- geologic hazards associated with strong shaking on a nearby fault, including settlement resulting from post-liquefaction reconsolidation following a major earthquake
- the presence of up to 5-1/2 feet of undocumented fill across the site
- the presence of compressible clay zones below the site.

These geotechnical concerns and their impact on the proposed foundation design and construction are discussed in the following sections.

6.1 Foundation Support and Settlement

The presence of up to 5-1/2 feet of undocumented fill and zones of relatively weak and compressible clay, the potential for up to 2 inches of ground-surface settlement due to post-liquefaction reconsolidation during a Maximum Considered Earthquake (MCE) event, and the relatively high column loads are the primary considerations in selecting a suitable foundation system for the proposed buildings. In developing our conclusions below, we have assumed the undocumented fill beneath the proposed buildings will overexcavated and recompacted in accordance with the recommendations presented in Section 7.1 of this report.

Based on our engineering analyses using the results of our previous field investigation, we conclude the proposed six-story buildings may be supported on mat foundations that bear on engineered fill and/or native soil. According to the structural engineer, the dead plus live loads for the proposed buildings are on the order of 715 kips and 410 kips for interior and exterior columns, respectively. We anticipate total static settlement of the mat foundation will be up to 1-1/2 inches and differential static settlement will be about 3/4 inch over a horizontal distance of 30 feet. We anticipate about 75 percent of the total static settlement will be complete by the end of construction. As previously discussed, there is the potential for up to 2 inches of liquefaction-induced settlement during a MCE event. The mat foundations would be able to span areas of non-support resulting from localize liquefaction-induced settlement, settle somewhat uniformly because of its rigidity and reduce post-earthquake differential settlement to tolerable levels.

Although deep foundations, such as driven piles, are feasible, we believe they would be more costly than mat foundations.

6.2 Areal Settlement Due to the Placement of New Fill

Current plans do not indicate any significant re-grading of the site. If new fill is placed, some settlement will occur. The amount of settlement will depend on the lateral extent and the new fill thickness. We should evaluate settlement further once plans have been finalized.

6.3 Construction Considerations

The soil to be excavated for the new foundations and underground utilities is expected to include the undocumented predominantly granular fill, as well as the underlying native clay. If site grading is performed during the rainy season, the clay beneath the fill will likely be wet and will have to be dried before compaction can be achieved. Heavy rubber-tired equipment could cause excessive deflection (pumping) of the wet clay and therefore should be avoided.

Excavations that will be deeper than five feet and will be entered by workers should be sloped or shored in accordance with CAL-OSHA standards (29 CFR Part 1926). The contractor should be responsible for the construction and safety of temporary slopes.

7.0 PRELIMINARY RECOMMENDATIONS

Our preliminary recommendations for site preparation and grading, design of foundations, and other geotechnical aspects of the project are presented in this section.

7.1 Site Preparation and Grading

All areas to receive improvements should be stripped of vegetation and organic topsoil. From a geotechnical standpoint, the stripped organic topsoil may be stockpiled for later use in landscaped areas; however, organic topsoil should not be used as compacted fill.

The existing undocumented fill should be overexcavated and recompacted in areas that will receive improvements (i.e. buildings, pavements, and hardscapes). Rocks or chunks of concrete or bricks larger than four inches in greatest dimension and all organic material should be removed from the fill during the overexcavation. The native soil subgrade exposed at the base of the overexcavation should be scarified to a minimum of eight inches, moisture-conditioned, and compacted per the requirements in Table 2. The overexcavated fill should then be placed in lifts not exceeding eight inches in uncompacted thickness, moisture-conditioned to above optimum moisture, and compacted to at least 95 percent relative compaction[5] below foundation level, within six inches of pavement subgrade, and where the fill thickness is greater than five feet. Elsewhere, the overexcavated fill should be compacted to at least 90 percent relative compaction.

If the on-site moderately expansive clay is to be used as general site fill, it should be moisture-conditioned to at least two percent above optimum moisture content, placed in lifts not exceeding eight inches in uncompacted thickness, and compacted to at least 90 percent relative compaction. Select fill should consist of imported or on-site soil that is free of organic matter, contain no rocks or lumps larger than four inches in greatest dimension, have a liquid limit less than 40 and plasticity index less than 12, and be approved by the Geotechnical Engineer. Select fill should be placed in lifts not exceeding eight inches in uncompacted thickness, moisture-conditioned, and compacted per the requirements of Table 2. Samples of proposed select fill material should be submitted to the geotechnical engineer at least three business days prior to use at the site. The grading contractor should provide analytical test results or other suitable environmental documentation indicating the imported fill is free of hazardous materials at least three days before use at the site. If this data is not available, up to two weeks should be allowed to perform analytical testing on the proposed imported material.

In areas where wet and/or weak subgrade soils are encountered during subgrade preparation or other grading activities, the weak soil should be removed and replaced with select fill. The compaction requirements, including those for trenches and pavements, are summarized in Table 2 below.

5 Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557 laboratory compaction procedure.

TABLE 2
Summary of Compaction Requirements

Location	Required Relative Compaction (percent)	Moisture Requirement
Building pad subgrade – native clay	90+	2+% above optimum
Below building foundations – low-plasticity on-site and select fill	95+	Above optimum
General fill – low-plasticity soil	90+	Above optimum
General fill – native clay	90+	2+% above optimum
Utility trench backfill - native soil	90+	2+% above optimum
Utility trench backfill – low-plasticity	90+	Above optimum
Utility trench - clean sand or gravel	95+	Near optimum
Pavement subgrade	95+	Near optimum
Pavement - aggregate base	95+	Near optimum
Exterior slabs – native soil subgrade	90+	2+% above optimum
Exterior slabs – low-plasticity soil	90+	Above optimum

7.1.1 Utility Trenches

Excavations for utility trenches can be readily made with a backhoe. Despite careful site preparation, unexpected obstructions may make some of the trenching operations difficult. All trenches should conform to the current CAL-OSHA requirements.

Backfill for utility trenches and other excavations is also considered fill, and it should be compacted according to the recommendations presented for site preparation and fill placement. Jetting of trench backfill should not be permitted. Special care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to the pavement section. In the public right-of-way, backfill materials and compaction should comply with City of San Leandro specifications.

To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of sand or fine gravel. After the pipes and conduits are tested, inspected (if required) and approved, they should be covered to a depth of six inches with sand or fine gravel, which should be mechanically tamped.

7.2 Mat Foundations

The mat foundations should be designed to span a 10-foot-diameter unsupported area and to cantilever a minimum of three feet at the perimeter of the mat. Considering the large area of the mat, we expect the average bearing stress under the mat to be low; however, concentrated stresses will occur at column locations and at the edges of the mat. The mat should be designed to impose a maximum dead-plus-live bearing pressure of 4,000 psf on the foundation subgrade soil. This pressure may be increased by one-third for total load conditions.

To develop adequate mat rigidity, we recommend the mat be designed for dead-plus-live-load conditions using a modulus of vertical subgrade reaction of 15 pounds per cubic inch (pci). This value has been corrected to take into account the mat width and may be increased by 50 percent for total load conditions. Once the structural engineer estimates the distribution of bearing stress on the bottom of the mat, we should review the distribution and revise the modulus of subgrade reaction, if appropriate.

Assuming the mat is supported on a vapor retarder, a friction factor of 0.2 may be used to compute base friction. Where the mat foundation is supported directly on soil, a friction factor of 0.3 may be used. To calculate the passive resistance against the vertical faces of the mat, an equivalent fluid weight of 350 pcf should be used. The values for friction coefficient and passive pressure include a factor of safety of 1.5.

The mat subgrade should be kept moist prior to placement of the vapor retarder. We should check the mat subgrade prior to placing the vapor retarder to confirm it is free of standing water, debris, and disturbed materials.

7.3 Floor Slab

Where water vapor transmission through the mat foundation is undesirable, we recommend installing a capillary moisture break and a water vapor retarder beneath the mat. A capillary moisture break consists of at least four inches of clean, free-draining gravel or crushed rock. The vapor retarder should meet the requirements for Class B vapor retarders stated in ASTM E1745. The vapor retarder should be placed in accordance with the requirements of ASTM E1643. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder. For the mat foundation, the capillary moisture break can be eliminated if a Class A vapor retarder is used. If required by the structural engineer, the vapor retarder may be covered with two inches of sand to aid in curing the concrete and to protect the vapor retarder during slab construction. The sand overlying the vapor retarder should be moist at the time concrete is placed. However, excess water trapped in the sand could eventually be transmitted as vapor through the slab. Therefore, if rain is forecast prior to pouring the slab, the sand should be covered with plastic sheeting to avoid wetting. If the sand becomes wet, concrete should not be placed until the sand has been dried or replaced.

The particle size of the capillary break material and sand (if used) should meet the gradation requirements presented in Table 3.

TABLE 3
Gradation Requirements for Capillary Moisture Break

Sieve Size	Percentage Passing Sieve
Gravel or Crushed Rock	
1 inch	90 – 100
3/4 inch	30 – 100
1/2 inch	5 – 25
3/8 inch	0 – 6
Sand	
No. 4	100
No. 200	0 – 5

Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab. Therefore, concrete for the floor slab should have a low w/c ratio - less than 0.50. If necessary, workability should be increased by adding plasticizers. In addition, the slab should be properly cured. Before floor coverings, if any, are placed, the contractor should check that the concrete surface and the moisture emission levels (if emission testing is required) meet the manufacturer's requirements.

7.4 Pavement Design

7.4.1 Flexible (Asphalt Concrete) Pavement Design

The State of California flexible pavement design method was used to develop the recommended asphalt concrete pavement sections. The final soil subgrade in pavement areas will likely consist of gravel with clay and sand. We obtained a soil sample from the proposed parking area and performed laboratory tests to determine the R-value for pavement design. Laboratory test results indicate the soil tested has an R-value of 16. We used a reduced R-value of 10 for our pavement design to account for soil variability across the site.

If the proposed pavement will experience little or no truck traffic (including garbage trucks), we recommend a traffic index (TI) of 4.5 be used for asphalt concrete pavement design. Pavement areas that will be subject to garbage truck traffic should be designed for a TI of 5.5. The project civil engineer should check that the TI's presented in this report are appropriate for the intended use. Recommended pavement sections for these traffic indices are presented in Table 4.

TABLE 4
Recommended AC Pavement Sections

TI	Asphaltic Concrete (inches)	Class 2 Aggregate Base R = 78 (inches)
4.5	2.5	8.5
5.0	3.0	9.0
5.5	3.0	11.0

The upper six inches of the subgrade should be moisture-conditioned and compacted in accordance with requirements presented in Section 7.1. The aggregate base should be moisture conditioned to near optimum and compacted to at least 95 percent relative compaction.

To prevent irrigation water from entering the pavement section, curbs adjacent to landscaped areas should extend through the aggregate base and at least three inches into the underlying soil subgrade.

7.4.2 Rigid (Portland Cement Concrete) Pavement

Concrete pavement design is based on a maximum single-axle load of 20,000 pounds and a maximum tandem axle load of 32,000 pounds and light truck traffic (i.e., a few trucks per week). The recommended rigid pavement section for these axle loads is six inches of Portland cement concrete over six inches of Class 2 aggregate base.

The modulus of rupture of the concrete should be at least 500 psi at 28 days. Contraction joints should be constructed at 15-foot spacing. Where the outer edge of a concrete pavement meets asphalt concrete pavement, the concrete slab should be thickened by 50 percent at a taper not to exceed a slope of 1 in 10. For areas that will receive weekly garbage truck traffic, we recommend the slab be reinforced with a minimum of No. 4 bars at 16-inch spacing in both directions. Recommendations for subgrade preparation and aggregate base compaction for concrete pavement are the same as those we have described above for asphalt concrete pavement.

7.4.3 Non-Vehicular Concrete Pavers

The upper 12 inches of soil subgrade for concrete pavers should consist of non-expansive soil prepared in accordance with the recommendations presented in Section 7.1.

<u>Non-Permeable Concrete Pavers</u>

We recommend non-permeable pedestrian pavers and sand bedding be underlain by at least six inches of Class 2 aggregate base compacted to at least 90 percent relative compaction.

<u>Permeable Interlocking Concrete Pavers</u>

We recommend permeable interlocking concrete pavements (ICP) be designed in accordance with the guidelines presented by the Interlocking Concrete Pavement Institute (ICPI 2005). These guidelines include specific recommendations for permeable aggregate subbase, base, and bedding courses to be placed beneath ICP pavements. We recommend non-vehicular permeable pavers be designed for partial exfiltration of water into the subgrade soil. This requires installing a subdrain system at the base of the pervious aggregate materials, which are underlain by an filter fabric. ICPI's generalized paver section partial exfiltration is presented on Figure 5.

The soil subgrade beneath ICP pavements should be prepared and compacted in accordance with the recommendations presented in Section 7.1. In addition, the subgrade should be a firm and non-yielding surface. The subgrade should be proof-rolled under the observation of our field engineer to confirm it is non-yielding prior to placing the filter fabric and aggregate base materials. The soil subgrade at the bottom of the permeable section should slope down toward the drain pipe trench at a gradient of at least two percent. The perforated pipe should slope down to a suitable outlet at a minimum gradient of one percent. The pipe should be placed with the perforations down on a minimum of two inches of permeable subbase.

ICPI's guidelines call for 1-1/2 to 2 inches of bedding material consisting of ASTM No. 8 aggregate directly below the pavers. This material is also recommended for fill material between the pavers. As shown in Table 5 below, this material consists of fine gravel with 10 to 30 percent sand.

TABLE 5
Gradation Requirements for ASTM No. 8 Aggregate

Sieve Size	Percentage Passing Sieve
1/2 inch	100
3/8 inch	85 – 100
No. 4	10 – 30
No. 8	0 – 10
No. 16	0 – 5

The ASTM No. 8 bedding should be underlain by a permeable base course of ASTM No. 57 crushed aggregate. As shown in Table 6, ASTM No. 57 aggregate consists of open-graded gravel with a gradation between that of the 3/4-inch drain rock and the ASTM No. 8 aggregate.

TABLE 6
Gradation Requirements for ASTM No. 57 Aggregate

Sieve Size	Percentage Passing Sieve
1-1/2 inch	100
1 inch	95 – 100
1/2 inch	25 – 60
No. 4	0 – 10
No. 8	0 – 5

The ASTM No. 57 permeable base course should be underlain by a permeable subbase course of ASTM No. 2 crushed aggregate. The gradation requirements for ASTM No. 2 crushed aggregate subbase are presented in Table 7.

TABLE 7
Gradation Requirements for ASTM No. 2 Aggregate

Sieve Size	Percentage Passing Sieve
3 inch	100
2-1/2 inch	90-100
2 inch	35-70
1-1/2 inch	0-15
3/4 inch	0 -5

The No. 2 aggregate subbase course should be placed in lifts not exceeding 6 inches in loose thickness and compacted using a smooth-drum roller, operated in static (non-vibratory) mode. The subsequent course of No. 57 aggregate may be placed in one lift and should be compacted with a smooth-drum roller in vibratory mode with sufficient passes to create an unyielding surface. Placement and compaction of the permeable aggregate base and subbase should be performed under the observation of our field engineer. Following compaction of the No. 57

aggregate, the No. 8 bedding, not exceeding 2 inches in loose thickness, should be placed and screeded to a level, undisturbed surface immediately prior to paver installation.

The required thicknesses of the permeable aggregate base and subbase courses depends on the infiltration and water storage design requirements, as well as the traffic loading demand. Our recommendations for the minimum permeable ICP pavement section for pedestrian traffic are presented in Table 8.

TABLE 8
Recommended Pavement Sections for
Permeable Interlocking Concrete Pavers

TI	ASTM No. 8 Bedding Aggregate (inches)	ASTM No. 57 Stone Base (inches)	ASTM No. 2 Stone Subbase (inches)
Pedestrian	1.5-2.0	4.0 (10)	6.0 (0)

The above recommended ICP pavement section is based on the ICPI technical guidelines (ICPI 2005). From a geotechnical standpoint, it is acceptable to design the pedestrian ICP section to exclude the No. 2 subbase course, in which case the No. 57 base course should be increased to 10 inches. From a geotechnical standpoint, it is also acceptable to use compacted structural planting mix in lieu of the No. 57 and No. 2 base courses in locations where the pedestrian ICP section is adjacent to tree wells and is required for promoting root growth. If either of these approaches are used, the perforated pipe should include a filter fabric sleeve to prevent the finer aggregate or organic material from entering the perforations.

7.5 Seismic Design

We understand the proposed buildings will be designed using the 2013 CBC. Although the CBC calls for a Site Class F designation for sites underlain by potentially liquefiable soil, we conclude a Site Class D is more appropriate because the potentially liquefiable soil layers are thin and discontinuous.

For design in accordance with the 2013 CBC, we recommend Site Class D be used. The latitude and longitude of the site are 37.7218 and -122.1617, respectively. Hence, in accordance with the 2013 CBC, we recommend the following:

- S_S = 2.088 g, S_1 = 0.856g
- S_{MS} = 2.088g, S_{M1} = 1.285g
- S_{DS} = 1.392g, S_{D1} = 0.856g
- PGA_M = 0.803g
- Seismic Design Category E for Risk Categories I, II, and III.

8.0 ADDITIONAL GEOTECHNICAL SERVICES

Prior to final design, additional borings and/or CPTs should be performed within the proposed building footprints to supplement existing subsurface information and to develop final geotechnical conclusions and recommendations.

9.0 LIMITATIONS

This preliminary geotechnical investigation has been conducted in accordance with the standard of care commonly used as state-of-practice in the profession. No other warranties are either expressed or implied. The preliminary recommendations made in this report are based on the assumption that the subsurface conditions do not deviate appreciably from those disclosed in the exploratory borings and CPTs. If any variations or undesirable conditions are encountered during construction, we should be notified so that additional recommendations can be made. The foundation recommendations presented in this report are developed exclusively for the proposed development described in this report and are not valid for other locations and construction in the project vicinity.

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