August 12, 2020
Project No.: 20-1864

Mr. Peter Beritzhoff
Caracol Property Owner, LLC
1725 South Bascom Avenue, Suite 1050
Campbell, California 95008

Subject: Preliminary Geotechnical Paper Study
Proposed Office Development
550 E. Brokaw Road
San Jose, California

Dear Mr. Beritzhoff:

This letter presents the results of the preliminary geotechnical study performed by Rockridge Geotechnical to support preliminary project planning for the proposed office development to be constructed at 550 E. Brokaw Road in San Jose, California. The site is located on the southeast side of E. Brokaw Road, west of its intersection with Highway 880, as shown on the attached Site Location Map (Figure 1).

The project site has been mapped within a zone of liquefaction potential on the *Earthquake Zones of Required Investigation* digital mapping provided by the California Geological Survey (CGS). Special Publication 117A by CGS (2008) recommends subsurface investigations in mapped liquefaction potential areas be performed using rotary-wash borings and/or cone penetration tests (CPTs) that extend at least 50 feet below the ground surface.

SCOPE OF SERVICES

The objective of our preliminary geotechnical study was to evaluate whether there are any adverse geotechnical or geological conditions that may significantly affect site development to support the preliminary planning process. Our study was performed in accordance with our proposal dated August 11, 2020. Our proposed scope of services consisted of researching and reviewing published geologic literature and nearby geotechnical reports to summarize anticipated subsurface conditions and identify any adverse geotechnical or geological conditions that may significantly affect the feasibility and costs associated with site development. Based on our document review, we are providing preliminary information regarding likely foundation type(s) and design for the proposed building. No site-specific borings or cone penetration tests (CPTs) were performed as part of this preliminary consultation scope.

SITE AND PROJECT DESCRIPTION

The subject site bounded by E. Brokaw Road to the northwest, the Mission Valley Ford Truck dealership and Highway 880 to the northeast, a self-storage facility to the southeast, and Junction Avenue to the southwest, as shown on the attached Site Plan (Figure 2). The site is made up of a single, irregularly shaped parcel, totaling about 19.7 acres, and is currently occupied by a Fry's Electronics commercial building surrounded by asphalt-paved parking areas.

Based on our review of the SDP Submission drawings, titled *Caracol Property Owner, LLC – 550 East Brokaw*, dated July 24, 2020 and prepared by Gensler, the project architects, we understand the proposed development being considered for the site consists of demolishing the existing improvements and constructing seven 8-story office buildings and two 9- to 10-level parking structures. All structures will be constructed at grade. The project will also include construction of internal streets, an Emergency Vehicle Access road through the central portion of the site, and extensive landscape improvements. The approximate layout of the proposed improvements, as currently envisioned, is presented on the attached Site Plan (Figure 2).

EXISTING GEOTECHNICAL INFORMATION

We reviewed the following geotechnical documents for previous projects in the general site vicinity:

- *Preliminary Geotechnical Investigation, Proposed Coyote Creek Trail, Montague Expressway to Watson Park, San Jose California*, prepared by Pacific Geotechnical Engineering, dated March 29, 2010 (closest boring located approximately 1,100 feet north-northeast of the subject site)

- *Geotechnical Investigation, 101 / Orchard Parkway, San Jose, California,* prepared by Treadwell & Rollo, Inc, dated November 7, 2007 (located approximately 6,000 feet west of the subject site)

- *Geotechnical Investigation, Proposed Office Buildings and Parking Structure, APN 237-16-072 – Phase 2, 60 East Brokaw Road, San Jose, California*, prepared by Silicon Valley Soil Engineering, dated October 26, 2018 (located approximately 2,600 feet southwest of the subject site)

- *Geotechnical Engineering Investigation, Coyote Creek Bridge (Replace) (Bridge No. 37-0636) and MSE Retaining Wall, County of Santa Clara, California,* prepared by Parikh Consultants, Inc., dated April 2, 2001 (located approximately 800 feet northeast of the subject site)

In addition to the geotechnical data discussed above, we also reviewed groundwater data available on the State of California Water Resources Control Board GeoTracker website (http://geotracker.swrcb.ca.gov) to help characterize potential groundwater levels at the project site. This data set included groundwater monitoring well readings from 1992 to 2010 at 524 E. Brokaw Road, located west, across Junction Avenue, from the subject site.

Subsurface Conditions

The regional geologic map of the site vicinity indicates the site is located in an area mapped as Holocene-age alluvium (Qha), as shown on the attached Regional Geologic Map (Figure 3). Coyote Creek runs approximately southeast-northwest approximately 800 feet northeast of the site.

Based on our review of available subsurface information from other investigations in the site vicinity, we anticipate the project site is underlain by alluvial deposits consisting of interbedded low- to high-plasticity, medium stiff to very stiff clays and silts and loose to dense sands and gravels with varying fines content. We anticipate there are likely some thin layers of fine-grained deposits that are only lightly over-consolidated in the soil profile, especially near the groundwater table, and therefore are moderately to highly compressible under new loads. Where/if clay soils are present near the ground surface, we anticipate they may be moderately expansive. Expansive soil undergoes volume changes with changes in moisture content.

Based on our review of available historic groundwater data in the near vicinity of the site, we anticipate the depth to groundwater fluctuates from roughly 7 to 12 feet bgs. The groundwater level at the site is expected to fluctuate several feet seasonally, as well as between wet and dry years, depending on the amount of annual rainfall.

SEISMIC CONSIDERATIONS

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, Calaveras, and San Andreas faults. These and other faults in the region are shown on the attached Regional Fault Map (Figure 4). Numerous damaging earthquakes have occurred along these faults in recorded time. For these and other active faults within a 50-kilometer radius of the site, the distance from the site and estimated characteristic moment magnitude[1] [Petersen et al. (2014) & Thompson et al. (2016)]

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1 Moment magnitude (M_w) 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.

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are summarized in Table 1. These references are based on the Third Uniform California Earthquake Rupture Forecast (UCERF3), prepared by Field et al. (2013).

TABLE 1
Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Characteristic Moment Magnitude
Total Hayward + Rodgers Creek (RC+HN+HS+HE)	7.8	Northeast	7.58
Hayward (South, HS)	7.8	Northeast	7.00
Hayward (Extension, HE)	9.4	East	6.18
Total Calaveras (CN+CC+CS+CE)	11	East	7.43
Calaveras (Central, CC)	11	East	6.85
Calaveras (North, CN)	12	Northeast	6.86
Monte Vista - Shannon	15	Southwest	7.14
Total North San Andreas (SAO+SAN+SAP+SAS)	21	Southwest	8.04
North San Andreas (Peninsula, SAP)	21	Southwest	7.38
North San Andreas (Santa Cruz Mts, SAS)	24	Southwest	7.15
Butano	25	Southwest	6.93
Las Positas	27	Northeast	6.50
Sargent	27	South	6.71
Zayante-Vergeles (2011 CFM)	32	South	7.48
Zayante-Vergeles	33	South	7.00
Greenville (North)	35	East	6.86
Greenville (South)	35	East	6.64
Mount Diablo Thrust	40	North	6.67
Mount Diablo Thrust South	40	Northeast	6.50
San Gregorio (North)	43	West	7.44
Mount Diablo Thrust North CFM	44	North	6.72
Calaveras (South, CS)	50	Southeast	6.38

Since 1800, four major earthquakes have been recorded on the North 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 Loma Prieta Earthquake of October 17, 1989 had an M_w of 6.9 and occurred about 38 kilometers south of the site. On August 24, 2014, an earthquake with an estimated maximum intensity of VIII (severe) on the MM scale occurred on the West Napa fault. This earthquake was the largest earthquake event in the San Francisco Bay Area since the Loma Prieta Earthquake. The M_w of the 2014 South Napa Earthquake was 6.0.

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 (estimated 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 ($M_w = 6.2$).

As a part of the UCERF3 project, researchers estimated that the probability of at least one $M_w \geq 6.7$ earthquake occurring in the greater San Francisco Bay Area during a 30-year period (starting in 2014) is 72 percent. The highest probabilities are assigned to sections of the Hayward (South), Calaveras (Central), and the North San Andreas (Santa Cruz Mountains) faults. The respective probabilities are approximately 25, 21 , and 17 percent.

Geologic Hazards

Because the project site is in a seismically active region, we preliminarily evaluated, at a high level, the potential for earthquake-induced geologic hazards including ground shaking, ground surface rupture, liquefaction,[2] lateral spreading,[3] and cyclic densification[4] based on available subsurface data in the site vicinity. 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.

As shown on the attached Figure 5, the project site has been mapped within a zone of liquefaction potential on *Earthquake Zones of Required Investigation* digital mapping provided by the California Geological Survey (CGS). CGS has provided recommendations for the content

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, also referred to as differential compaction, is a phenomenon in which non-saturated, cohesionless soil is compacted by earthquake vibrations, causing ground-surface settlement.

of site investigation reports within seismic hazard zones in Special Publication 117 (SP-117) titled *Guidelines for Evaluating and Mitigating Seismic Hazard Zones in California*, dated September 11, 2008. SP-117 recommends subsurface investigations in mapped liquefaction hazard zones be performed using at least one rotary-wash boring and/or cone penetration test (CPT) that extend to a depth of at least 50 feet. While the current study presented herein is not based site-specific borings or CPTs, we conclude the final geotechnical investigation for the project should include rotary wash borings and/or CPTs that extend to at least 50 feet bgs, in accordance with SP-117.

Based on our review of the available subsurface data in the general site vicinity, we preliminarily conclude there is a high likelihood that thin potentially liquefiable soil layers are present below the groundwater at the site, and therefore, a detailed liquefaction evaluation should be performed based on the results of a final geotechnical investigation for the project, which should include CPTs and laboratory testing on select layers of potentially liquefiable deposits. In our experience, the potentially liquefiable deposits in this area are generally thin, discontinuous, and sporadic in nature, due to the depositional setting. We preliminarily conclude there is the potential for as much as several inches of liquefaction-induced settlement in portions of the site.

Ground Shaking

The seismicity of the site is governed by the activity of the Hayward, Calaveras, and San Andreas Faults, although ground shaking from future earthquakes on other faults will also be felt at the site. The intensity of earthquake ground motion at the site will depend upon the characteristics of the generating fault, distance to the earthquake epicenter, and magnitude and duration of the earthquake. We judge that strong to very strong ground shaking could occur at the site during a large earthquake on one of the nearby faults.

Fault 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 low. In a seismically active area, the remote possibility exists for future faulting in areas where no faults previously existed; however, we conclude the risk of surface faulting and consequent secondary ground failure from previously unknown faults is also low.

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Cyclic Densification

Cyclic densification (also referred to as differential compaction) of non-saturated sand (sand above groundwater table) can occur during an earthquake, resulting in settlement of the ground surface and overlying improvements. In general, the available subsurface information for the site vicinity indicates that there is the potential for loose to medium dense granular deposits above the groundwater table at the site, which may result in settlement of shallow foundations. The potential for cyclic densification should be further evaluated during the final geotechnical investigation for the project.

PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

Based on our review of the existing geotechnical data in the vicinity of the subject site and the results of our preliminary engineering analyses, we conclude there are no major geotechnical or geological issues that would preclude development of the site as proposed. The primary geotechnical issues affecting the proposed development include:

- the potential presence of interbedded thin, moderately weak and compressible fine-grained layers within the generally stiff soil profile, which may result in excessive settlement under static building loads;
- the potential presence of liquefiable soil layers (anticipated to be relatively thin) within the soil profile, which could result in as much as several inches of ground settlement following a major earthquake;
- the potential for moderately expansive near-surface soil that is susceptible to volume changes with changes in moisture content.

Our preliminary conclusions and recommendations regarding these issues are presented in the following sections.

Foundations

Based on our review of the available subsurface information and our experience on past similar projects in the site vicinity, we anticipate there may be thin, moderately weak and compressible fine-grained layers within the generally stiff soil profile that will result in foundation settlement under static load conditions. Furthermore, as previously discussed, we preliminarily conclude there is the potential for as much as several inches of liquefaction-induced settlement, pending the results of a final site-specific investigation and liquefaction evaluation. While it may be possible to support the proposed office buildings and parking structures on conventional spread footings without excessive differential settlements, we preliminarily conclude the proposed office buildings and parking structures will likely need to be supported on footings underlain by a ground improvement system that is designed to control static and seismically induced settlements to tolerable levels. Potential ground improvement systems are discussed in more detail below.

Deep foundations, such as driven piles, torque-down piles, or auger cast-in-place piles, are viable alternatives to footings on a ground improvement system, however, for the office buildings, we anticipate deep foundations will be less cost effective. Due to the high loads associated with the proposed parking structures, deep foundations may be more cost-effective than ground improvement, as higher vertical and lateral capacities can be achieved.

Ground Improvement Systems

As discussed above, if the proposed office buildings and parking structures are supported on shallow foundations, we preliminarily conclude a ground improvement system will likely be needed below the shallow foundations to control settlements to tolerable levels. In addition to reducing the potential for damaging static and seismically induced differential settlements, a ground improvement system can be designed to increase the allowable bearing pressures, which would reduce the required size of footings. In some cases, ground improvement can also be implemented to reduce or mitigate liquefaction potential. Therefore, the costs associated with the ground improvement system may be partially offset by savings in structural design.

There are several types of ground improvement that may be utilized to mitigate differential settlements of the proposed structures. We consider soil-cement mix (SMX) columns, drilled displacement sand-cement (DDSC) columns, or displacement compacted aggregate piers / vibro-replacement stone columns (CAPs / VSCs) to be potentially viable ground improvement methods for this project. The techniques vary in their mechanism of improving the soil, but essentially serve to (a) stiffen the soil matrix, and (b) transfer building loads to deeper, stronger strata. The ground improvement types are discussed further below. The required size, spacing, length, and strength of ground improvement should be determined by the design-build contractor, based on the desired level of improvement (i.e. the tolerable settlement and desired allowable bearing pressure), as coordinated with the structural engineer. The suitability of each ground improvement system should be further evaluated based on the results of a site-specific geotechnical investigation prior to final design. Each system is described below in more detail.

<u>Drilled Displacement Sand Cement (DDSC) Columns</u>

DDSC columns are installed by advancing a continuous flight, hollow-stem auger that mostly displaces the soil and then pumping a sand-cement mixture into the hole under pressure as the auger is withdrawn. DDSC columns result in low vibration during installation and generate little to no drilling spoils for off-haul. DDSC columns are installed under design-build contracts by specialty contractors.

Due to the relative size and stiffness of the DDSC columns compared to the surrounding soil, a reinforced engineered fill pad (“rock cushion”) is typically installed between the foundation and the underlying DDSC columns.

<u>Soil Cement Mix (SMX) Columns</u>

SMX columns are installed by injecting and blending cement into the soil using a drill rig equipped with augers or specialty cutting heads. SMX columns are installed under design-build contracts by specialty contractors. SMX columns result in low vibration during installation and generate some spoils for off-haul.

<u>Displacement Compacted Aggregate Piers or Vibro-Replacement Stone Columns</u>

Conventional CAPs are typically constructed by drilling a 30-inch-diameter shaft and replacing the excavated soil with compacted aggregate in lifts. Because conventional CAPs rely on drilling an open hole prior to placing the aggregate in compacted lifts, the holes are prone to caving, especially in granular materials below the groundwater. Therefore, we recommend a displacement-type CAP (i.e. Impact Rammed Aggregate Pier) or vibro-replacement stone column (VSC) system for this project, which do not rely on drilling open holes, but rather mostly displace the in-situ soil by placing and compacting the aggregate in lifts as the tooling is withdrawn from the ground. Displacement CAPs result in moderate to high amounts of vibration during installation. Displacement CAPs and VSCs can be challenging to install through dense and/or very stiff soils without pre-drilling, which can generate additional spoils.

Expansive Soil

We anticipate that if clay soils are present near the surface at the subject site, there is a high likelihood they are moderately expansive. Expansive near-surface soil is subject to volume changes during seasonal fluctuations in moisture content. These volume changes can cause movement and cracking of foundations, pavements, slabs, and below-grade walls. Therefore, foundations, pavements, slabs, and below-grade walls may need to be designed and constructed to resist the effects of the expansive soil. In general, the effects of expansive soil can be mitigated by moisture-conditioning the expansive soil, providing select, non-expansive fill or lime-treated soil below interior and exterior slabs and behind retaining walls, and either supporting foundations below the zone of severe moisture change or by providing a stiff, shallow foundation that can limit deformation of the structure as the underlying soil shrinks and swells. For planning and budgeting purposes, we preliminarily recommend a minimum of 6 inches of select fill or 12 inches of lime-treated on-site soil be placed beneath the concrete slabs-on-grade for the office buildings and parking structures. In addition, 6 inches of select fill or lime-treated on-site soil should be placed beneath proposed exterior concrete flatwork, including patio slabs and sidewalks.

Seismic Design

We anticipate the proposed structures will be designed using the seismic provisions in the 2019 California Building Code. For design in accordance with the 2019 CBC, we preliminarily recommend Site Class D be used, assuming there are not thick, continuous layers of liquefiable soil underlying the site, which will need to be confirmed during final geotechnical investigation. The latitude and longitude of the site are 37.3799° and -121.9066°, respectively. Assuming the seismic response coefficient (C_s) value will be calculated as outlined in *ASCE 7-16, Section 11.4.8, Exception 2*, we preliminarily recommend the following seismic design parameters:

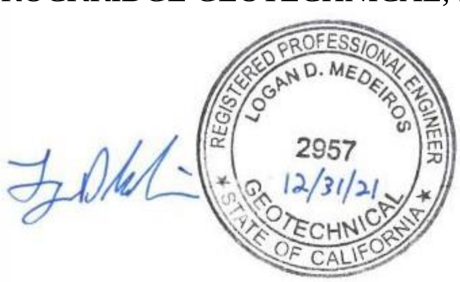
- S_S = 1.533g, S_1 = 0.6g
- S_{MS} = 1.533g, S_{M1} = 1.020g
- S_{DS} = 1.022g, S_{D1} = 0.680g
- Seismic Design Category D for Risk Categories I, II, and III.

ADDITIONAL GEOTECHNICAL SERVICES

The preliminary conclusions and recommendations presented herein are based on our review of existing subsurface data collected in the site vicinity and our preliminary engineering analyses; they are not intended for final design. Prior to final design, a final geotechnical report should be prepared based on a site-specific field investigation and the final proposed development plan. Exploratory borings and CPTs will be required to further evaluate the subsurface conditions at the site. Prior to construction, the geotechnical engineer of record should review the project plans and specifications to check their conformance with the intent of the final recommendations. During construction, the geotechnical engineer of record should observe site preparation, foundation installation, ground improvement installation and load testing, and the placement and compaction of fill. These observations will allow them to compare the actual with the anticipated soil conditions and to check if the contractor's work conforms with the geotechnical aspects of the plans and specifications.

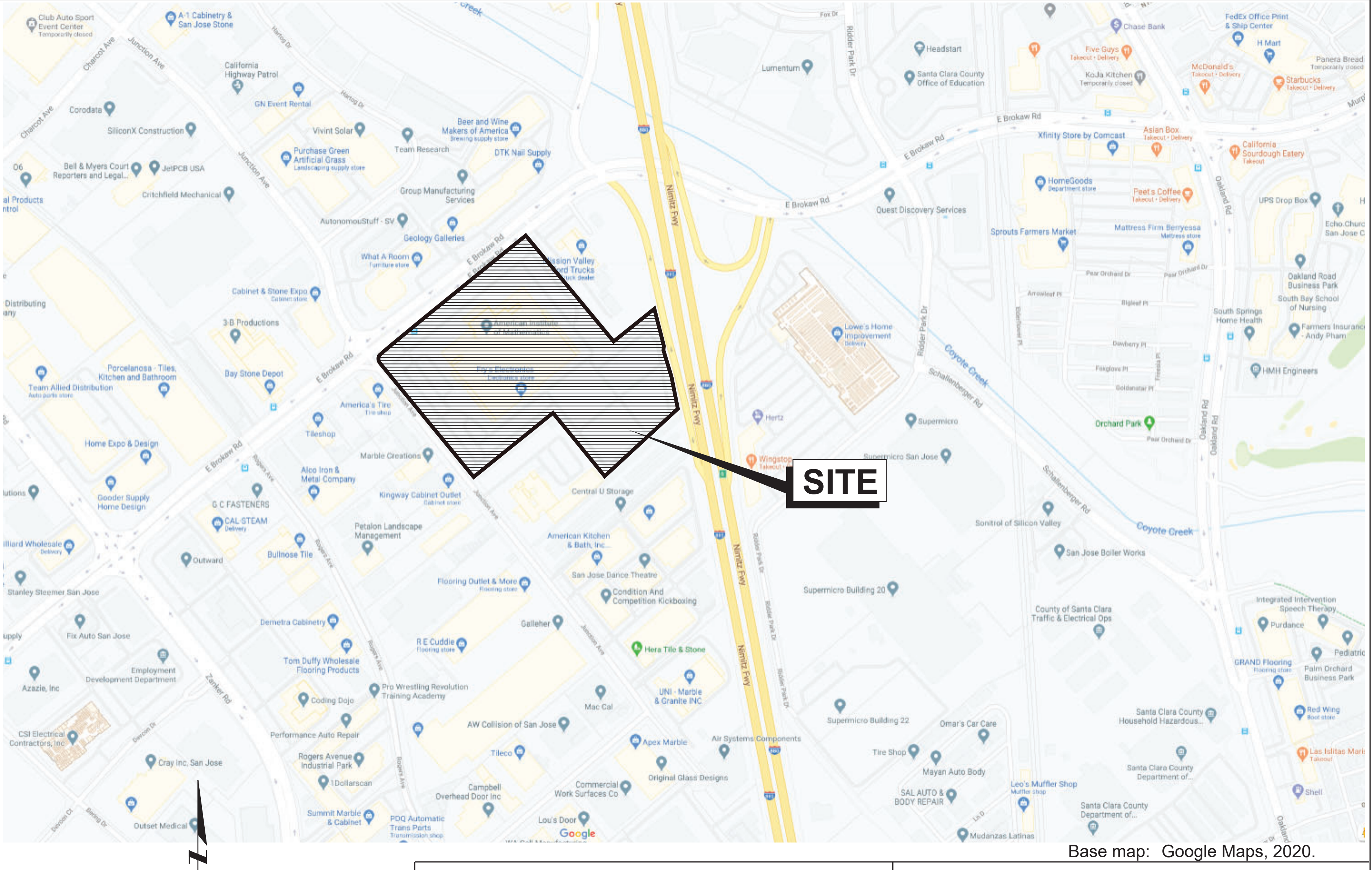
We trust this letter provides the information you need at this time. We appreciate the opportunity to provide our services to you on this project. If you have any questions, please call.

Sincerely yours,
ROCKRIDGE GEOTECHNICAL, INC.



Logan D. Medeiros, P.E., G.E.
Senior Engineer

Attachments: Figure 1 – Site Location map
Figure 2 – Site Plan
Figure 3 – Regional Geologic Map
Figure 4 – Regional Fault Map
Figure 5 – Earthquake Zones of Required Investigation Map



Base map: Google Maps, 2020.

0 600 1,200 Feet

Approximate scale

550 E. BROKAW ROAD
San Jose, California

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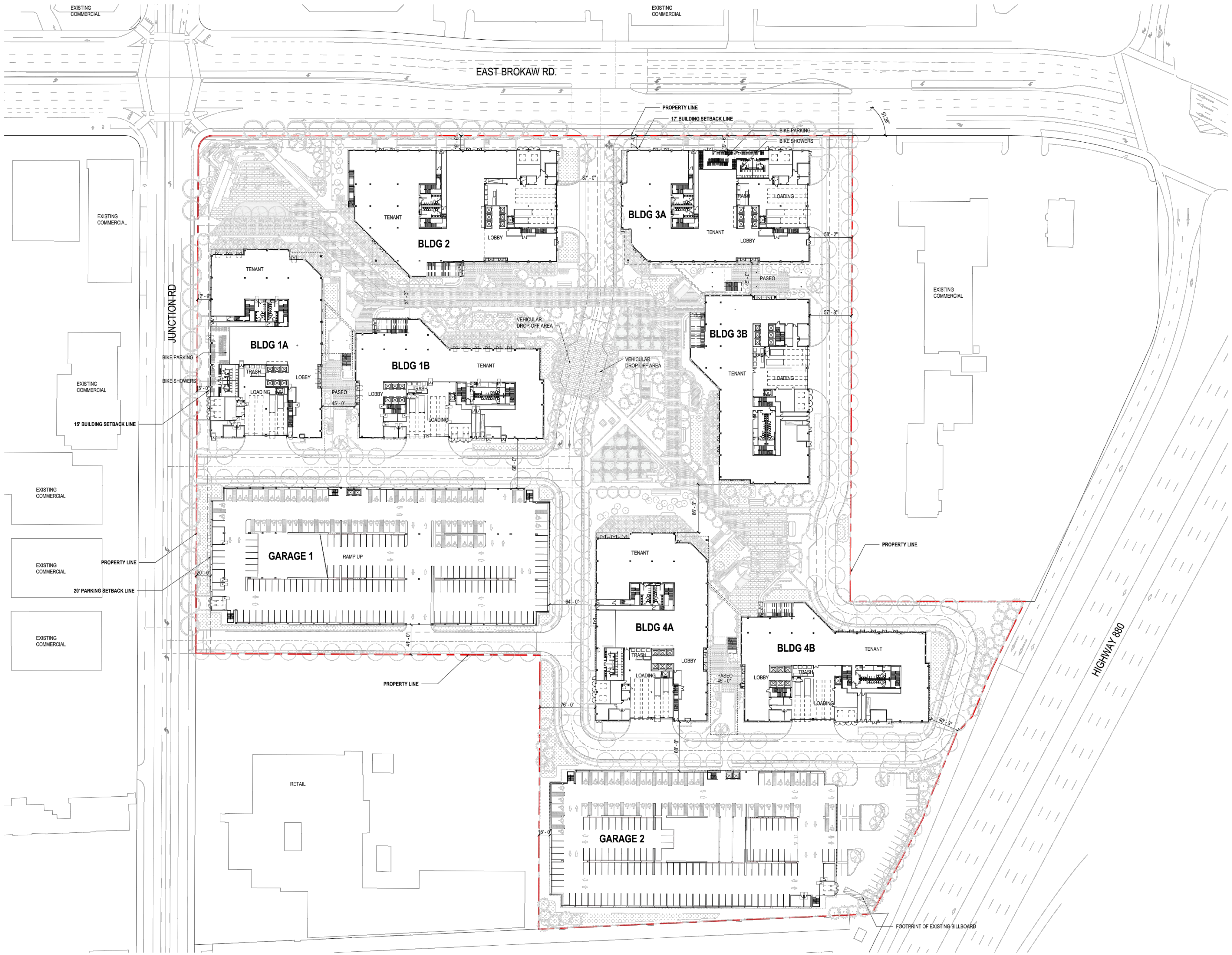
SITE LOCATION MAP

Date 08/05/20	Project No. 20-1864	Figure 1



0 160 Feet
Approximate scale

Reference: Base map from a drawing titled "Site Plan", by Gensler, dated July 27, 2020.

550 E. BROKAW ROAD
San Jose, California

SITE PLAN

Date 08/05/20	Project No. 20-1864	Figure 2

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Base map: Google Earth with U.S. Geological Survey (USGS), Santa Clara County, 2020.

EXPLANATION

af	Artificial Fill
Qha	Alluvium (Holocene)
Qsl	Hillslope Deposits (Quaternary)
Qpa	Alluvium (Pleistocene)
Tms	Sedimentary rocks (Miocene)
KJs	Great Valley complex sedimentary rocks (Early Cretaceous and (or) Late Jurassic)
Jv	Great Valley complex volcanic rocks (Jurassic)
Jsp	Great Valley complex serpentinite (Jurassic)

Geologic contact: dashed where approximate and dotted where concealed, queried where uncertain

0 1 2 Miles

Approximate scale

550 E. BROKAW ROAD
San Jose, California

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REGIONAL GEOLOGIC MAP

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Base Map: U.S. Geological Survey (USGS), National Seismic Hazards Maps - Fault Sources, 2014.

EXPLANATION

Strike slip

Thrust (Reverse)

0 4 8 Miles

Approximate scale

550 E. BROKAW ROAD
San Jose, California

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REGIONAL FAULT MAP

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Liquefaction Zones
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslide Zones
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Reference:
Earthquake Zones of Required Investigation
California Geological Survey
Milpitas Quadrangle; October 19, 2004
San Jose West Quadrangle; February 7, 2002

0 2,000 4,000 Feet
Approximate scale

550 E. BROKAW ROAD
San Jose, California

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EARTHQUAKE ZONES OF REQUIRED INVESTIGATION MAP

Date 08/05/20	Project No. 20-1864	Figure 5