APPENDIX E

Geotechnical Investigation Report

ROCKRIDGE
GEOTECHNICAL

Prepared for **Los Altos Holdings LLC**

GEOTECHNICAL INVESTIGATION
FIRST STREET GREEN DEVELOPMENT
101 TO 151 FIRST STREET
Los Altos, California

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June 21, 2017
Project No. 17-1300

ROCKRIDGE
GEOTECHNICAL

June 21, 2017
Project No. 17-1300

Los Altos Holdings LLC
c/o Sares Regis Group of Northern California
901 Mariners Island Boulevard, Suite 700
San Mateo, California 94404
Attention: Ms. Meris Ota

Subject: Geotechnical Investigation Report
First Street Green Development
101 to 151 First Street
Los Altos, California

Dear Ms. Ota:

We are pleased to present our geotechnical investigation report for the proposed First Street Green office building to be constructed at 101 to 151 First Street in Los Altos, California. Our geotechnical investigation was performed in accordance with the Contract for Consultant Services between Los Altos Holdings LLC and Rockridge Geotechnical, Inc. dated April 10, 2017.

The site for the proposed office building encompasses eight adjoining parcels, from 101 through 151 First Street, and has plan dimensions of about 165 feet by 360 feet. Currently, the site is occupied by one- to two-story commercial buildings. Asphalt-paved parking lots and driveways are also present in various locations around the existing buildings. Plans are to demolish the existing improvements to construct a building consisting of three levels of office space over 1 to 3 levels of subterranean parking.

On the basis of the results of our geotechnical investigation, we conclude the site can be developed as planned, provided the recommendations presented in this report are incorporated into the project plans and specifications and implemented during construction. The primary geotechnical issues affecting the proposed development include: 1) providing adequate foundation support for the proposed building, and 2) providing suitable lateral support for the proposed excavation while minimizing the impact on the surrounding improvements. We conclude the building may be supported on conventional spread footings bearing on firm native alluvium. We also conclude the most appropriate temporary shoring system for the project is soldier pile and lagging; internal bracing or tiebacks will likely be needed for additional lateral support of excavations that exceed about 12 feet in depth.

The recommendations contained in our report are based on a limited subsurface investigation. Consequently, variations between expected and actual subsurface conditions may be found in localized areas during construction. Therefore, we should be engaged to observe site grading and shoring and foundation installations, during which time we may make changes in our recommendations, if deemed necessary.

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.

Darcie Maffioli

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GEOTECHNICAL INVESTIGATION
FIRST STREET GREEN DEVELOPMENT
101 TO 151 FIRST STREET
Los Altos, California

1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by Rockridge Geotechnical, Inc. for the proposed First Street Green office building to be constructed at 101 to 151 First Street in Los Altos, California. The site is located on the northeastern side of First Street, between Shasta and State streets, as shown on the Site Location Map, Figure 1.

The site for the proposed office building encompasses eight adjoining parcels, from 101 through 151 First Street, and has plan dimensions of about 165 feet by 360 feet. Currently, the site is occupied by one- to two-story commercial buildings. Asphalt-paved parking lots and driveways are also present in various locations around the existing buildings as shown on the Site Plan, Figure 2. The site is relatively level with existing site grades varying from about Elevation[1] 184 feet at the northeastern corner of the site to Elevation 190 feet at the southwestern corner of the site.

Plans are to demolish the existing improvements to construct a building consisting of three levels of office space over 1 to 3 levels of subterranean parking.

2.0 SCOPE OF SERVICES

Our geotechnical investigation was performed in accordance with the Contract for Consultant Services between Los Altos Holdings LLC and Rockridge Geotechnical, Inc. dated April 10, 2017. Our scope of services consisted of exploring subsurface conditions at the site by drilling six test borings, performing laboratory testing on selected soil samples, and performing engineering analyses to develop conclusions and recommendations regarding:

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[1] Unless otherwise noted, all elevations are grades in this report are based on topographic information shown on the drawing titled *Site Plan*, dated September 24, 2014, prepared by Kier & Wright Civil Engineers & Surveyors, Inc. (elevation datum is NAVD88).

- site seismicity and seismic hazards
- the most appropriate foundation type(s) for the proposed building
- design criteria for the recommended foundation type(s), including vertical and lateral capacities for each of the foundation type(s)
- estimates of foundation settlement
- lateral earth pressures for basement walls
- temporary cut slopes and shoring
- construction dewatering, as appropriate
- site preparation and grading, including criteria for fill quality and compaction
- subgrade preparation for slab-on-grade floors
- pavement design
- 2016 California Building Code (CBC) site class and design spectral response acceleration parameters
- corrosivity of the near-surface soil and the potential effects on buried concrete and metal structures and foundations
- construction considerations.

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

Subsurface conditions at the site were investigated by drilling six borings and performing laboratory testing on selected soil samples. Prior to performing our field investigation, we contacted Underground Service Alert (USA) to notify them of our work, as required by law. We also retained Precision Locating, a private utility locator, to check that the boring locations were clear of existing utilities. Upon completion, the borings were backfilled with cement grout. Details of the field investigation and laboratory testing are described below.

3.1 Test Borings

Six borings, designated as B-1 through B-6, were drilled between May 2 and 5, 2017 by Exploration Geoservices of San Jose, California. The borings were each drilled to a depth of 45 feet below the ground surface (bgs) using a drill rig equipped with approximately four-inch-inside-diameter and eight-inch-outside-diameter hollow-stem augers. The approximate locations of the borings are shown on Figure 2.

During drilling, our field engineer logged the soil encountered and obtained representative samples for visual classification and laboratory testing. The boring logs were developed based on laboratory test data, review of soil samples in the office, and the conditions recorded on the field logs. The boring logs are presented on Figures A-1 through A-6 in Appendix A. The soil encountered in the borings was classified in accordance with the classification chart shown on Figure A-7.

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 stainless steel or brass tubes.
- Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside and 1.5-inch inside diameter, without liners.

The S&H and SPT samplers were driven with a 140-pound, downhole, safety hammer falling about 30 inches per drop. The samplers were driven up to 18 inches and the hammer blows required to drive the sampler 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 used for this conversion were: (1) the last two blow counts if the sampler was driven more than 12 inches, (2) the last one blow count if the sampler was driven more than six inches but less than 12 inches, and (3) the only blow count if the sampler was driven six inches or less. The blow counts required to drive the S&H and SPT samplers were converted to approximate SPT N-values using factors of 0.7 and 1.2, respectively, to account for sampler type, approximate hammer energy, and the fact that the SPT sampler was designed to accommodate liners, but liners were not used. The converted SPT N-values are presented on the boring logs.

Upon completion of drilling the boreholes were backfilled with cement grout. Soil cuttings generated from the borings were removed from the site by the drilling subcontractor to be disposed of offsite.

3.2 Laboratory Testing

We re-examined each soil sample obtained from our borings to confirm the field classifications and selected representative samples for laboratory testing. Geotechnical laboratory tests were performed on selected soil samples to assess their engineering properties and physical characteristics. Soil samples were tested by B. Hillebrandt Soils Testing, Inc. of Alamo, California to measure moisture content, dry density, plasticity (Atterberg limits), and particle size distribution. Corrosivity testing of two soil samples was performed by Project X Corrosion Engineering of Murrieta, California. The results of the geotechnical laboratory tests are presented on the boring logs and in Appendix B.

4.0 SUBSURFACE CONDITIONS

The regional geology map prepared by Graymer (2000), a portion of which is presented on Figure 3, indicates the site is underlain by Pleistocene-age alluvial deposits (Qpa). The results of the borings indicate the site is underlain by alluvium to the maximum depth explored of 45 feet bgs.

Where explored, the upper 12 feet of soil consists of medium dense to dense clayey sand with variable amounts gravel, hard silt, and very stiff to hard clay with varying amounts of sand. The results of Atterberg limits test indicate the near-surface clay obtained from Borings B-2, B-3, and B-6 have a plasticity index ranging from 12 to 18 and, therefore, has a low expansion potential. Below a depth of 12 feet bgs, the alluvium generally consists of interbedded layers of hard clay with varying amounts of sand and gravel and dense to very dense sand with variable amounts of clay and gravel to the maximum depth explored of 45 feet bgs.

Groundwater was not encountered in the borings during our subsurface investigation. The report prepared by the California Geological Survey titled *Seismic Hazard Zone Report for the*

Mountain View Quadrangle, dated October 18, 2006, indicates the historic high groundwater level at the site is greater than 50 feet bgs.

5.0 SEISMIC CONSIDERATIONS

The results of our evaluation regarding seismic considerations for the project site are presented in the following sections.

5.1 Regional Seismicity and Faulting

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 Monte-Vista Shannon, San Andreas, and Hayward faults. These and other active 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 mean Characteristic Moment magnitude[2] [Working Group on California Earthquake Probabilities (WGCEP) (2008) and Cao et al. (2003)] are summarized in Table 1.

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[2] 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
Monte Vista-Shannon	3.0	Southwest	6.50
N. San Andreas - Peninsula	7.2	Southwest	7.23
N. San Andreas (1906 event)	7.2	Southwest	8.05
Total Hayward	24	Northeast	7.00
Total Hayward-Rodgers Creek	24	Northeast	7.33
N. San Andreas - Santa Cruz	25	Southeast	7.12
San Gregorio Connected	26	West	7.50
Total Calaveras	29	East	7.03
Zayante-Vergeles	35	Southeast	7.00
Mount Diablo Thrust	47	Northeast	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 a 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), a 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 October 17, 1989, with a M_w of 6.9. This earthquake occurred in the Santa Cruz Mountains about 44 kilometers southwest 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 a 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 2014 Working Group on California Earthquake Probabilities 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 Region during the next 30 years (starting from 2014) is 72 percent. The highest probabilities are assigned to the Hayward Fault, Calaveras Fault, and the northern segment of the San Andreas Fault. These probabilities are 14.3, 7.4, and 6.4 percent, respectively.

5.2 Seismic Hazards

During a major earthquake on a segment of one of the nearby faults, strong to very strong shaking is expected to occur at the project site. Strong shaking during an earthquake can result in ground failure such as that associated with liquefaction[3], lateral spreading[4] and cyclic densification.[5] We used the results of the borings to evaluate the potential for these phenomena to occur at the project site. The results of our analyses and evaluation are presented in the following sections.

5.2.1 Ground Shaking

The ground shaking intensity felt at the project site will depend on: 1) the size of the earthquake (magnitude), 2) the distance from the site to the fault source, 3) the directivity (focusing of

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

[4] 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.

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

earthquake energy along the fault in the direction of the rupture), and 4) site-specific soil conditions. The site is about three kilometers from the Monte-Vista Shannon Fault and less than eight kilometers from the San Andreas 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 Liquefaction and Associated Hazards

When a saturated, cohesionless soil liquefies, it experiences a temporary loss of shear strength created by a transient rise in excess pore pressure generated by strong ground motion. 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.

The site has **not** been mapped within a zone of liquefaction potential as shown on the map titled *State of California, Seismic Hazard Zones, Mountain View Quadrangle, Official Map,* prepared by the California Geological Survey (CGS), dated October 18, 2006 (Figure 5).

Considering groundwater was not encountered in our borings that bottomed in very stiff to hard clay and dense to very dense sand, and available historic groundwater information of the site and vicinity indicate historic high groundwater to be greater than 50 feet bgs, we conclude the potential for liquefaction to occur at the site is nil.

5.2.3 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. The borings indicate there is a layer of medium dense clayey sand in Boring B-1 in the upper 12 feet bgs. Laboratory test results indicate this soil contains about 18 percent fines. Therefore, we conclude this clayey sand has a low susceptibility to cyclic densification because of its relative density and relatively high fines content; this soil will also be removed during excavation for the below-grade parking garage. In other areas, our borings indicate the soil at the site generally consists of cohesive soil and relatively dense granular soil, which are not susceptible to cyclic densification due to their relatively high fines content and/or high relative density. Therefore, we conclude the potential for ground settlement beneath the proposed office building with 1 to 3 subterranean parking levels resulting from cyclic densification is nil.

5.2.4 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 very 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 very low.

6.0 DISCUSSIONS AND CONCLUSIONS

From a geotechnical standpoint, we conclude the site can be developed as planned, provided the recommendations presented in this report are incorporated into the project plans and specifications and implemented during construction. The primary geotechnical issues affecting the proposed development are providing adequate foundation support for the proposed building and providing lateral support during excavation for the below-grade level(s). These and other geotechnical issues as they pertain to the proposed development are discussed in the remainder of this section.

6.1 Foundations and Settlement

The foundation level of the proposed building with 1 to 3 levels of subterranean parking will be underlain by alluvium which has moderate to high strength and relatively low compressibility. Therefore, we conclude the proposed building may be supported on a shallow foundation system, such as conventional spread footings.

Our settlement analyses indicate total settlement of conventional spread footings designed using the allowable bearing pressures presented in Section 7.2 of this report will be on the order of 3/4 inch and differential settlement will be on the order of 1/2 inch over a 30-foot horizontal distance.

6.2 Excavation Support

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). We anticipate an excavation extending to depths of about 12 to 32 feet bgs will be needed to construct the 1 to 3 subterranean levels and foundations. Considering the footprint of the proposed subterranean levels will likely occupy the entire site, temporary shoring will be required to retain the sides of the excavation. We judge that a soldier pile and lagging shoring system would be the most appropriate shoring system.

A soldier pile and lagging shoring system usually consists of steel H-beams and concrete placed in predrilled holes extending below the bottom of the excavation. It will be necessary to provide about 14 to 18 inches of space between the property line and the face of the shoring where the basement walls will be constructed adjacent to private property. Wood lagging is placed between the soldier piles as the excavation proceeds from the top down, in maximum five-foot-thick lifts. Where the required cut is less than about 12 feet, a soldier pile and lagging system can typically provide economical shoring without tiebacks and, therefore, will not encroach beyond the property line. Where cuts exceed about 12 feet in height, soldier pile and lagging systems are typically more economical if they include tieback anchors.

Tiebacks consist of post-tensioned steel strands or bars that are grouted into predrilled holes through the excavation face. Where tiebacks will extend beneath the streets and sidewalks and adjacent properties, an encroachment agreement will be required with the City of Los Altos and adjacent property owners. If permission from the City of Los Altos or adjacent property owners (as needed) cannot be obtained to install tiebacks beneath their properties, then internal bracing will be required.

The selection, design, construction, and performance of the shoring system should be the responsibility of the contractor. A structural engineer knowledgeable in this type of construction should design the shoring. Recommendations for the design and construction of soldier pile and lagging shoring system, without and with tiebacks, are presented in Section 7.5.

6.3 Excavation, Monitoring, and Construction Considerations

The soil to be excavated for the proposed building and utilities is expected to consist primarily of sand and clay with variable amounts of gravel which can be excavated with conventional earth-moving equipment such as backhoes.

During excavation, the shoring system may yield and deform, which could cause surrounding improvements to settle and move laterally. The magnitude of shoring movements and resulting ground deformations are difficult to estimate because they depend on many factors, including the retained height, the contractor's skill and quality control in the shoring installation. For a properly designed and constructed shoring system, we judge vertical and lateral movements behind the shoring system will be within ordinarily accepted limits of about one inch where there are no improvements within a horizontal distance equal to 1.5 times the height of the shoring and 1/2 inch where there are improvements within that horizontal distance.

The contractor should establish survey points on the shoring and on the ground surface at critical locations behind the shoring prior to the start of excavation. These survey points should be used to monitor the vertical and horizontal movements of the shoring and the ground behind the shoring during construction. The monitoring program should provide timely data which can be used to modify the shoring system if needed. In addition to monitoring of survey points, a pre-construction photographic survey should be performed for buildings within a horizontal distance equal to 1.5 times the maximum excavation depth to document the condition of the buildings prior to construction.

6.4 Soil Corrosivity

Corrosivity testing was performed by Project X Corrosion Engineer of Murrieta, California on samples of soil obtained during our field investigation from Boring B-3 at a depth of 9-1/2 feet bgs and from Boring B-5 at a depth of 2 feet bgs. The results of the test are presented in Appendix B of this report.

Based on the results of the corrosivity test, we conclude the soil is considered "mildly to moderately corrosive" with respect to resistivity. Accordingly, all buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric-coated steel or iron may need to be protected against corrosion depending upon the critical nature of the structure. If it is necessary to have metal in contact with soil, a corrosion engineer should be consulted to provide recommendations for corrosion protection. The results indicate that sulfate ion concentrations are sufficiently low such that they do not to pose a threat to buried concrete. In addition, the chloride ion concentrations are insufficient to impact steel reinforcement in concrete structures below ground adversely. The results of the pH test indicate the near-surface soil has a pH of about 7 and has "negligible" impact to buried metal and concrete.

7.0 RECOMMENDATIONS

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

7.1 Site Preparation and Grading

Site clearing should include removal of all existing pavements, former foundation elements, and underground utilities. Any vegetation and organic topsoil (if present) should be stripped in areas to receive improvements (i.e., building, pavement, or flatwork). Tree roots with a diameter greater than 1/2 inch within three feet of subgrade should be removed. Demolished asphalt concrete should be taken to an asphalt recycling facility.

Abandoned underground utilities should be removed to the property line or service connections and properly capped or plugged with concrete. Where existing utility lines are outside of the proposed building footprint and will not interfere with the proposed construction, they may be abandoned in place provided the lines are filled with CLSM or cement grout to the property line. Any excavations created during demolition should be properly backfilled with compacted fill or control low strength material (CLSM) under the direction of our field engineer. Overexcavations extending below the foundation subgrade should be backfilled with CLSM.

7.1.1 Subgrade Preparation

In areas to receive improvements (i.e. building pads or pavements) or fill, the soil subgrade should be scarified to a depth of at least eight inches, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent relative compaction[6]. Subgrade soil consisting of clean sand or gravel (defined as soil with less than 10 percent fines by weight) should be compacted to at least 95 percent relative compaction. The upper eight inches of soil subgrade for vehicular pavement should be compacted to at least 95 percent relative compaction and be non-yielding.

7.1.2 Fill Quality and Compaction

Fill should consist of on-site soil or imported soil (select fill) that is free of organic matter, contains no rocks or lumps larger than three inches in greatest dimension, has a liquid limit of less than 40 and a plasticity index lower than 12, and is approved by the Geotechnical Engineer. Samples of proposed imported 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.

Fill should be placed in horizontal lifts not exceeding eight inches in uncompacted thickness, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent relative compaction. Fill material consisting of clean sand or gravel (defined as soil with less than 10 percent fines by weight) should be compacted to at least 95 percent relative compaction. Fill greater than five feet in thickness or fill placed within the upper six inches of vehicular

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[6] 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.

pavement soil subgrade should also be compacted to at least 95 percent relative compaction, and be non-yielding.

Where the recommended compaction requirements are in conflict with the City of Los Altos standard details for pavements, sidewalks, or trenches within the public right-of-way, the City Engineer or inspector should determine which compaction requirements should take precedence.

Controlled Low Strength Material

Controlled low strength material (CLSM) may be considered as an alternative to fill beneath the building, concrete flatwork, or pavement. CLSM should meet the requirements in the 2015 Caltrans Standard Specifications. It is an ideal backfill material when adequate room is limited or not available for conventional compaction equipment, or when settlement of the backfill must be minimized. No compaction is required to place CLSM. CLSM should have a minimum 28-day unconfined strength of 50 pounds per square inch (psi); where CLSM will be placed beneath foundations, it should have a minimum 28-day unconfined strength of 100 psi.

7.1.3 Utility Trench Backfill

Excavations for utility trenches can be readily made with a backhoe. All trenches should conform to the current CAL-OSHA requirements. 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.

Backfill for utility trenches and other excavations is also considered fill, and should be placed and compacted in accordance with the recommendations previously presented. If imported clean sand or gravel (defined as soil with less than 10 percent fines) is used as backfill, it should be compacted to at least 95 percent relative compaction. 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 improvements above the fill.

Foundations for the proposed structures should be bottomed below an imaginary line extending up at a 1.5:1 (horizontal to vertical) inclination from the base of the utility trenches running parallel to the foundation. Alternatively, the portion of the utility trench (excluding bedding) that is below the 1.5:1 line can be backfilled with CLSM or Class 2 aggregate base compacted to at least 95 percent relative compaction. If utility trenches are to be excavated below this zone-of-influence line after construction of the building foundations, the trench walls need to be fully supported with shoring until CLSM is placed.

7.1.4 Exterior Concrete Flatwork

We recommend a minimum of four inches of Class 2 aggregate base be placed below exterior concrete flatwork, including patio slabs and sidewalks; the aggregate base should extend at least six inches beyond the slab edges where adjacent to landscaping. Class 2 aggregate base beneath exterior slabs-on-grade, such as patios and sidewalks, should be moisture-conditioned and compacted in accordance with the requirements provided above in Section 7.1.2.

7.2 Spread Footings

Provided the estimated total and differential settlements presented in Section 6.1 are acceptable, the proposed building may be supported on conventional spread footings bearing on firm native soil (alluvium). Continuous footings should be at least 18 inches wide and isolated spread footings should be at least 24 inches wide. Footings should be founded at least 24 inches below the lowest adjacent soil subgrade.

The footings may be designed using allowable bearing pressures of 6,000 pounds per square foot (psf) for dead-plus-live loads; this value may be increased by one-third for total design loads, which include wind or seismic forces. These allowable bearing capacities include factors of safety of at least 2.0 and 1.5 for dead-plus-live loads and total loads, respectively.

Lateral loads may be resisted by a combination of passive pressure on the vertical faces of the footings and friction between the bottoms of the footings and the supporting soil. To compute frictional resistance, we recommend using a friction coefficient of 0.35. To calculate the passive resistance for both sustained and transient loading, we recommend using an equivalent fluid weight of 330 pounds per cubic foot (pcf). Passive resistance for the upper foot of soil should be ignored unless it is confined by a pavement or slab. The values for the friction coefficient and passive pressure include a factor of safety of 1.5 and may be used in combination without further reduction.

Footing excavations should be free of standing water, debris, and disturbed materials prior to placing concrete. If footings are excavated during the rainy season they should incorporate a mud slab to protect the footing subgrade. This will involve over-excavating the footing by about two inches and placing lean concrete or sand-cement slurry in the bottom (following inspection by our engineer). A mud slab will help protect the footing subgrade during placement of reinforcing steel. Water can then be pumped from the footing excavations prior to placement of structural concrete, if present. The bottoms and sides of the footing excavations should be moistened following excavation and maintained in a moist condition until concrete is placed. We should check footing excavations prior to placement of reinforcing steel.

7.3 Floor Slab

Where water vapor transmission through the floor slab is undesirable, we recommend installing a capillary moisture break and a water vapor retarder beneath the slab-on-grade. A vapor retarder and capillary moisture break are often not required beneath parking garage floor slabs because there is sufficient air circulation to allow evaporation of moisture that is transmitted through the slab; however, we recommend the vapor retarder and capillary break be installed below the slab-on-grade in utility rooms and any areas in or adjacent to the parking garage that will be used for storage and/or will receive a floor covering or coating.

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. The particle size of the capillary break material should meet the gradation requirements presented in Table 2.

TABLE 2
Gradation Requirements for Capillary Moisture Break

Sieve Size	Percentage Passing Sieve
1 inch	90 – 100
3/4 inch	30 – 100
1/2 inch	5 – 25
3/8 inch	0 – 6

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 the concrete is poured directly over the vapor retarder, we recommend the w/c ratio of the concrete not exceed 0.45. In either case, water should not be added to the concrete mix in the field. If necessary, workability should be increased by adding plasticizers. In addition, the slab should be properly cured. Before the floor covering is 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 Permanent Below-Grade Walls

Permanent below-grade walls should be designed to resist static lateral earth pressures, lateral pressures caused by earthquakes, and traffic loads (if vehicular traffic is expected within 10 feet of the wall). We recommend restrained below-grade walls be designed for the more critical of the following criteria:

- At-rest equivalent fluid weight of 60 pcf, plus a traffic increment where the wall will be within 10 feet of adjacent streets, or
- Active pressure of 40 pcf plus a seismic increment of 32 pcf (triangular distribution).

The recommended lateral earth pressures above are based on a level backfill condition with no additional surcharge loads from vehicles or adjacent building foundations. Where the below-grade wall is subject to vehicular loading within 10 feet of the wall, an additional uniform lateral pressure of 50 psf applied to the upper 10 feet of the wall.

The lateral earth pressures recommended are applicable to walls that are backdrained to prevent the buildup of hydrostatic pressure. One acceptable method for backdraining the wall is to place a prefabricated drainage panel against the back of the wall. The drainage panel should extend down to a perforated PVC collector pipe at the base of the walls. The pipe should be surrounded on all sides by at least four inches of Caltrans Class 2 permeable material (see Caltrans Standard Specifications Section 68-1.025) or 3/4-inch drain rock wrapped in filter fabric (Mirafi NC or equivalent). The collector pipe should be sloped to drain to a sump or another suitable outlet. Where shoring is installed and there is insufficient room to install a perforated pipe between the shoring and the back of the basement wall, the drainage panel should extend down to a proprietary, prefabricated collector drain system, such as Tremdrain Total Drain or Hydroduct Coil (or equivalent), designed to work in conjunction with the drainage panel. The pipe should be connected to a suitable discharge point inside or outside the basement. We should check the manufacturer’s specifications regarding the proposed prefabricated drainage panel material to verify it is appropriate for its intended use.

To protect against moisture migration, below-grade basement walls should be waterproofed and water stops should be placed at all construction joints. In recent years, we have observed numerous leaks in below-grade portions of buildings constructed with waterproofed, shotcrete walls. In areas where there is a high sensitivity to leaks, we recommend cast-in-place concrete basement walls be considered.

If backfill is required behind basement walls, the walls should be braced, or hand compaction equipment used, to prevent unacceptable surcharges on walls (as determined by the structural engineer).

7.5 Temporary Cut Slopes and Shoring

We anticipate an excavation from 12 to 32 feet bgs will be needed to construct the proposed one- to three-level basement parking garage and foundations. We judge that temporary slope cuts in sandy soils, corresponding to CAL-OSHA Types C soil, above the groundwater table inclined no steeper than 1.5:1 (horizontal:vertical), will be stable provided that they are not surcharged by equipment or building material.

Temporary shoring will be required where temporary slopes are not possible because of space constraints. As discussed in Section 6.2, we conclude soldier pile and lagging shoring systems with or without tiebacks are appropriate for support of excavations for this project and are presented in this section.

7.5.1 Cantilevered Soldier Pile and Lagging Shoring System

We recommend a cantilevered soldier pile-and-lagging shoring system be designed to resist an active equivalent fluid weight of 40 pcf. In locations where minimizing lateral deflections is critical, such as near adjacent buildings or near sensitive underground utilities, the shoring system should be designed to resist an at-rest equivalent fluid weight of 60 pcf. Where traffic loads are expected within 10 feet of the shoring walls, an additional design load of 50 psf should be applied to the upper 10 feet of the wall. Where construction equipment will be working behind the walls within a horizontal distance equal to the wall height, the design should include a surcharge pressure of 250 psf.

The above pressures should be assumed to act over the entire width of the lagging installed above the excavation. Passive resistance at the toe of the soldier pile should be computed using an equivalent fluid weight 330 pcf with a maximum passive earth pressure of 4,000 psf. These passive pressure values include a factor of safety of at least 1.5. The upper foot of soil should be ignored when computing passive resistance. Passive pressure can be assumed to act over an area of three soldier pile widths assuming the toe of the soldier pile is filled with structural concrete. The shoring designer should check that the specified minimum concrete strength is sufficient to spread the anticipated loads to three soldier pile widths.

Soldier piles should be placed in pre-drilled holes backfilled with concrete or installed in soil-mix columns. The subsurface soils predominately consist of sands and gravels, therefore, the shoring contractor should be prepared to use casing or drilling slurry to reduce caving of holes, where necessary.

The penetration of the soldier piles must be sufficient to ensure stability and resist the downward loading of tiebacks. Vertical loads can be resisted by skin friction along the portion of the soldier piles below the excavation. We recommend using an allowable skin friction value of 1,000 psf to compute the required soldier pile embedment. End bearing should be neglected.

7.5.2 Soldier Pile and Lagging Shoring System with Tiebacks

Recommended lateral pressures for the design of soldier pile and lagging shoring with tiebacks are presented on Figure 6. Where it is not feasible to install tiebacks, then internal bracing of the excavation will be required. Internal bracing should be preloaded to limit movement of the shoring. In calculating these design pressures, we assumed drained conditions with no hydrostatic pressure acting on the shoring. Where traffic loads are expected within 10 feet of the shoring walls, an additional design load of 50 psf should be applied to the upper 10 feet of the wall. Where construction equipment will be working behind the walls within a horizontal distance of 10 feet, the design should include a surcharge pressure of 250 psf acting over the upper 10 feet of the wall. The above pressures should be assumed to act over the entire width of the lagging installed above the excavation.

Passive resistance at the toe of the soldier pile should be computed using an equivalent fluid weight 330 pcf with a maximum passive earth pressure of 4,000 psf. These passive pressure values include a factor of safety of at least 1.5. The upper foot of soil should be ignored when computing passive resistance. Passive pressure can be assumed to act over an area of three soldier pile widths, or pile-to-pile spacing, whichever is less, assuming the toe of the soldier pile is filled with concrete or lean concrete that is sufficiently strong to accommodate the corresponding stresses.

Soldier piles should be placed in pre-drilled holes backfilled with concrete or installed in soil-mix columns. The subsurface soils consist of interbedded clay and sand layers with variable gravel content, therefore, the shoring contractor should be prepared to use casing or drilling slurry to reduce caving of holes, where necessary.

The penetration of the soldier piles must be sufficient to ensure stability and resist the downward loading of tiebacks. Vertical loads can be resisted by skin friction along the portion of the soldier piles below the excavation. We recommend using an allowable skin friction value of 1,000 psf to compute the required soldier pile embedment. End bearing should be neglected.

Design criteria for tiebacks are also presented on Figure 6. As shown, tiebacks should derive their load-bearing capacity from the soil behind an imaginary line sloping upward from a point H/5 feet away from the bottom of the excavation at an angle of 60 degrees from horizontal, where H is the wall height in feet. The minimum stressing lengths for strand and bar tendons should be 15 and 10 feet, respectively. The minimum bond length for strand and bar tendons should both be 15 feet.

Allowable capacities of the tiebacks will depend upon the drilling method, hole diameter, grout pressure, and workmanship. The shoring contractor should use a smooth-cased method (such as a Klemm rig) to install the tiebacks beneath adjacent buildings, where applicable. The shoring designer should be responsible for determining the actual length of tiebacks required to resist the design loads. The determination should be based on the designer's familiarity with the installation method to be used.

Tieback Testing

The computed bond length of tiebacks should be confirmed by a performance- and proof-testing program under the observation of our field engineer. The first two production tiebacks and two percent of the remaining tiebacks should be performance tested to 1.5 times the design load. The remaining tiebacks should be confirmed by a proof-test to 1.25 times the design load. The movement of each tieback should be monitored with a free-standing, tripod-mounted dial gauge during performance and proof testing. The bottom of excavation should not extend more than two feet below a row of unsecured tiebacks.

The performance test is used to verify the capacity and the load-deformation behavior of the tiebacks. It is also used to separate and identify the causes of tieback movement, and to check that the designed unbonded length has been established. In the performance test, the load is applied to the tieback in several cycles of incremental loading and unloading. During the test, the tieback load and movement are measured. The maximum test load should be held for a minimum of 10 minutes, with readings taken at 0, 1, 3, 6, and 10 minutes. If the difference between the 1- and 10-minute reading is less than 0.04 inch during the loading, the test is discontinued. If the difference is more than 0.04 inch, the holding period is extended by 50 minutes to 60 minutes, and the movements should be recorded at 15, 20, 25, 30, 45, and 60 minutes.

A proof test is a simple test used to measure the total movement of the tieback during one cycle of incremental loading. The maximum test load should be held for a minimum of 10 minutes, with readings taken at 0, 1, 2, 3, 6, and 10 minutes. If the difference between the 1- and 10-minute reading is less than 0.04 inch, the test is discontinued. If the difference is more than 0.04 inch, the holding period is extended by 50 minutes to 60 minutes, and the movements should be recorded at 15, 20, 25, 30, 45, and 60 minutes.

We should evaluate the tieback test results and determine whether the tiebacks are acceptable. A performance- or proof-tested tieback with a 10-minute hold is acceptable if the tieback carries the maximum test load with less than 0.04 inch movement between 1 and 10 minutes, and total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length.

A performance- or proof-tested tieback with a 60-minute hold is acceptable if the tieback carries the maximum test load with less than 0.08 inch movement between 6 and 60 minutes, and total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length. Tiebacks that failed to meet the first criterion will be assigned a reduced capacity.

7.6 Pavement Design

Design recommendations for flexible (asphalt concrete) and rigid (Portland-cement concrete) pavements are presented in the following sections.

7.6.1 Flexible (Asphalt Concrete) Pavement Design

The State of California flexible pavement design method was used to develop the recommended asphalt concrete pavement sections. Based on the results of an R-value test performed on a sample of near-surface soil from the adjacent Western Plaza site (southeast of First Street Green office building site), we used an R-value of 35 for design.

Table 3 presents our pavement section recommendations for traffic indices (TIs) of 4.5 through 6.0. The Civil Engineer for the project should check that the TI's presented in this report are appropriate for the intended use depending on the amount of anticipated loading conditions, and traffic and City of Los Altos requirements. We can provide additional pavement sections for different TIs, if needed.

TABLE 3
Recommended Asphalt Pavement Sections

TI	Asphaltic Concrete (inches)	Class 2 Aggregate Base R = 78 (inches)
4.5	2.5	4.5
5.0	3.0	4.5
5.5	3.0	6.0
6.0	3.5	6.5

The upper six inches of the subgrade should be moisture-conditioned and compacted in accordance with requirements presented in Section 7.1.1. The subgrade should be proof-rolled under the observation of our field engineer to confirm it is non-yielding prior to placement of the aggregate base. The Class 2 aggregate base should be moisture-conditioned to near optimum moisture content and compacted to at least 95 percent relative compaction and be non-yielding. The aggregate base should also be proof-rolled under the observation of our field engineer to confirm it is non-yielding prior to paving.

If pavements slope down away from irrigated landscaped areas, curbs adjacent to those areas should extend through the aggregate base and at least three inches into the underlying soil to reduce the potential for irrigation water to infiltrate into the pavement section. If drip irrigation is used in the landscaping adjacent to the pavement, however, the deepened curb is not required.

7.6.2 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), which corresponds to a TI of 4.5 to 6.0. The recommended rigid pavement section for light axle loads (i.e. passenger cars and TI of 4.5 and 5.0) is five inches of Portland cement concrete over six inches of Class 2 aggregate base. Where light truck traffic (including garbage and fire trucks) is expected (i.e. TI of 5.5 to 6.0), the pavement section should consist of six inches of Portland cement concrete over six inches of Class 2 aggregate base.

The modulus of rupture and unconfined compressive strength of the concrete should be at least 500 and 3,000 pounds per square inch (psi) at 28 days, respectively. 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.

7.7 Seismic Design

We understand the proposed building will be designed using the seismic provisions in the 2016 CBC. For design in accordance with the 2016 CBC, we recommend Site Class C be used. The latitude and longitude of the site are 37.3799° and -122.1193°, respectively. Hence, in accordance with the 2016 CBC, we recommend the following:

- S_S = 1.900g, S_1 = 0.777g
- S_{MS} = 1.900g, S_{M1} = 1.010g
- S_{DS} = 1.267g, S_{D1} = 0.673g
- Seismic Design Category E for Risk Factors I, II, and III.

8.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

Prior to construction, Rockridge Geotechnical should review the project plans and specifications to verify that they conform to the intent of our recommendations. During construction, our field engineer should provide on-site observation and testing during site preparation, placement and compaction of fill, and installation of temporary shoring, and foundations. These observations will allow us to compare actual with anticipated soil conditions and to verify that the contractor's work conforms to the geotechnical aspects of the plans and specifications.

9.0 LIMITATIONS

This 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 recommendations made in this report are based on the assumption that the subsurface conditions do not deviate appreciably from those disclosed in the test borings. 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.

REFERENCES

2015 Caltrans Standard Specifications

2016 California Building Code

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California Department of Transportation, Division of New Technology, Materials and Research, Office of Geotechnical Engineering, (2014). *SNAIL Program, A User Manual*, updated December 2014, available from http://www.dot.ca.gov/hq/esc/geotech/software/geo_software.html.

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Graymer, R.W. (2000). Geologic map and map database of the Oakland metropolitan area, Alameda, Contra Costa, and San Francisco Counties, California: U.S. Geological Survey Miscellaneous Field Studies MF–2342, scale 1:50,000. (Available at http://pubs.usgs.gov/mf/2000/2342/.)

Toppozada, T.R. and Borchardt G. (1998). "Re-evaluation of the 1936 "Hayward Fault" and the 1838 San Andreas Fault Earthquakes." Bulletin of Seismological Society of America, 88(1), 140-159.

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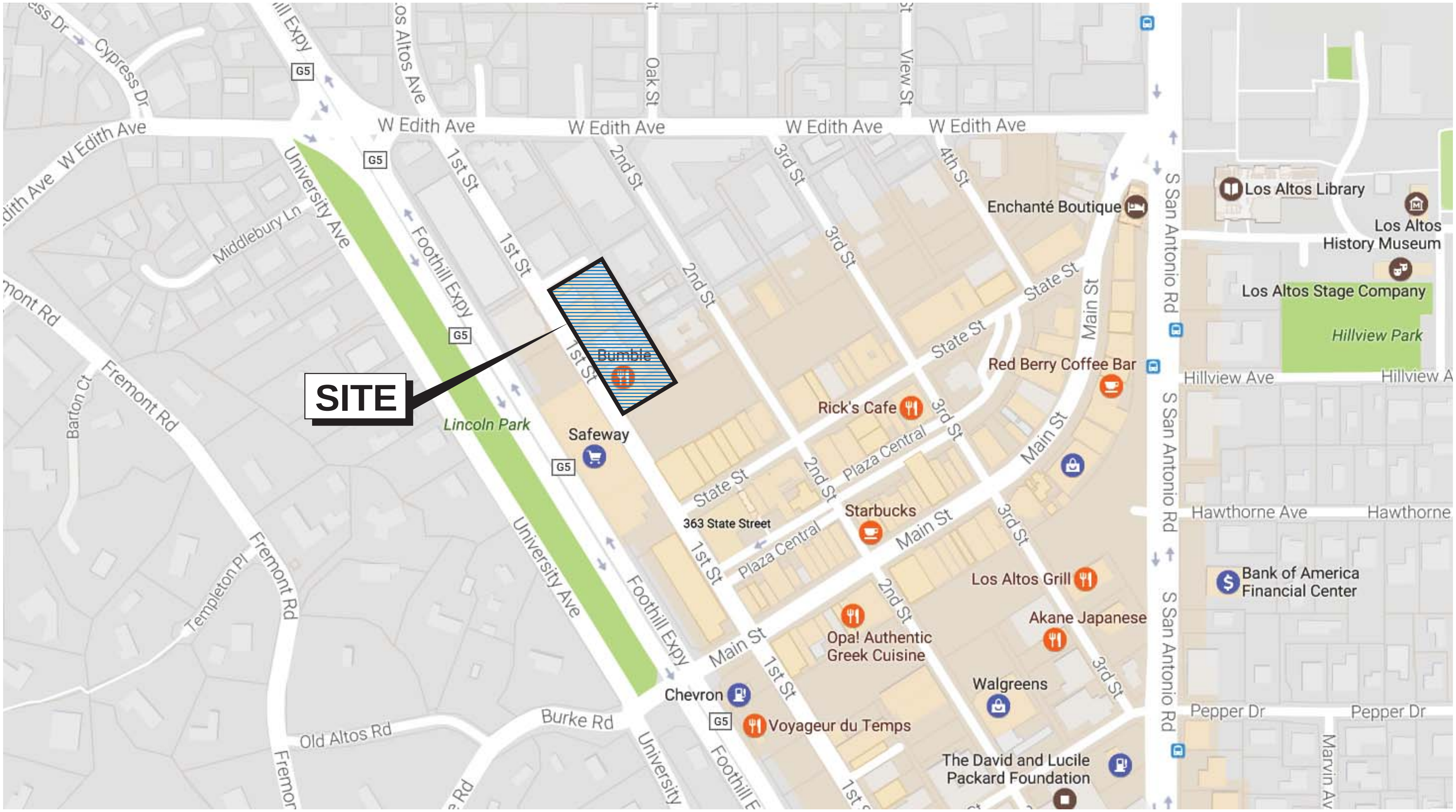
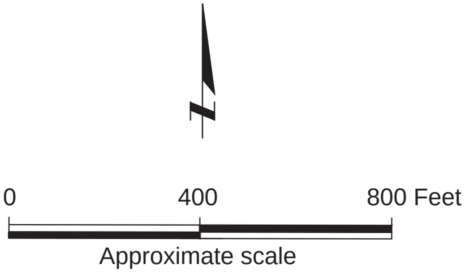
FIGURES



Base map: Google Map, 2017

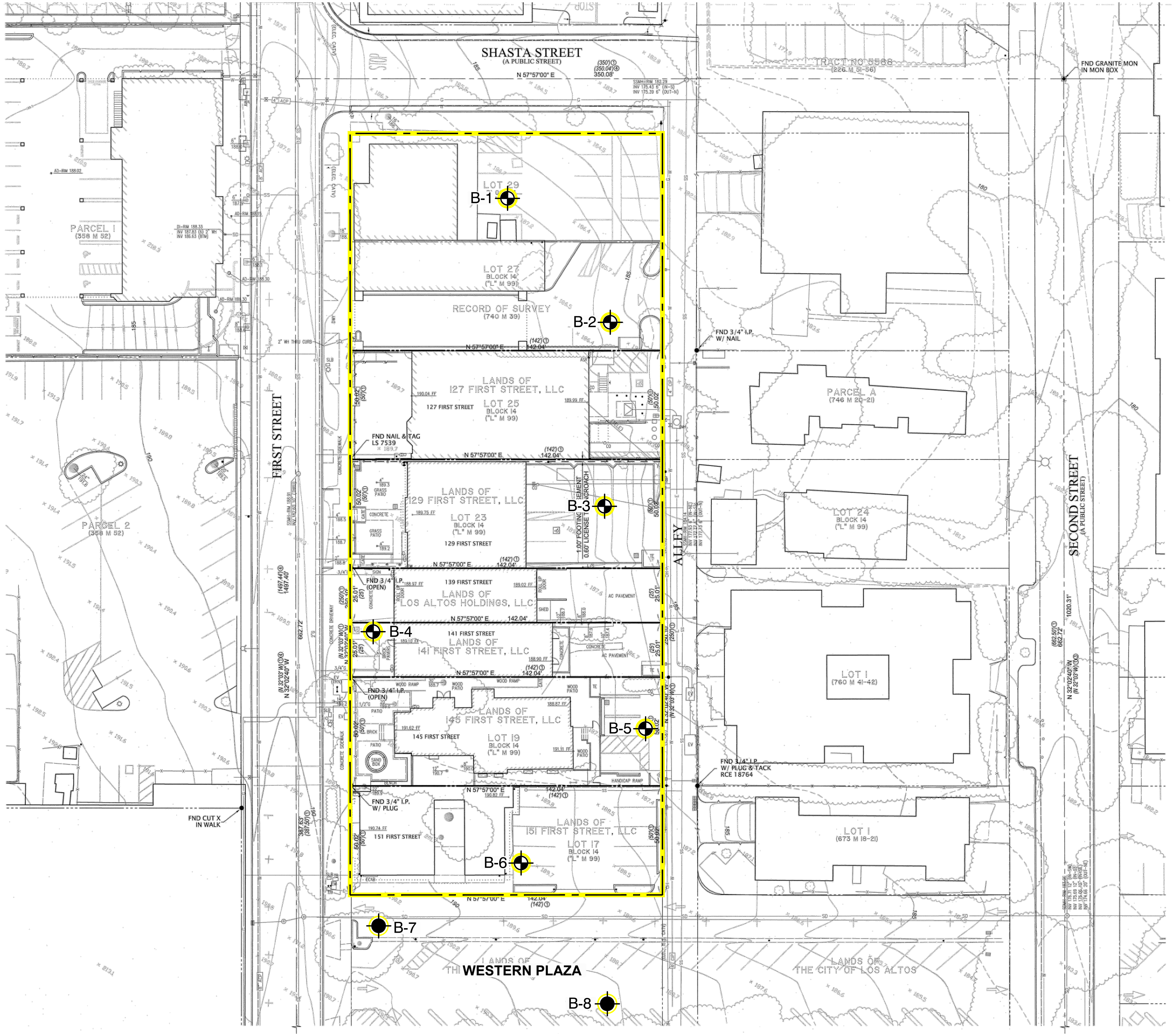


FIRST STREET GREEN DEVELOPMENT
101 TO 151 FIRST STREET
Los Altos, California

ROCKRIDGE
GEOTECHNICAL

SITE LOCATION MAP

Date 06/13/17	Project No. 17-1300	Figure 1



EXPLANATION

B-1 ⊕	Approximate location of boring by Rockridge Geotechnical Inc., May 2 through 5, 2017
B-7 ●	Approximate location of boring by Rockridge Geotechnical Inc., May 23, 2017
— · —	Project limits

Note: Elevation Contours shown are based on aerial photogrammetrix methods, see "Site Plan" by Kier & Wright Civil Engineers and Surveyors, Inc. for additional information. Contours shown are NAVD 88 datum.

0 50 Feet

Approximate scale

Reference: Base map from a drawing titled "Site Plan", by Kier & Wright Civil Engineers & Surveyors, Inc., dated September 24, 2014.

FIRST STREET GREEN DEVELOPMENT
101 TO 151 FIRST STREET
Los Altos, California

SITE PLAN

Date 06/13/17	Project No. 17-1300	Figure 2

ROCKRIDGE GEOTECHNICAL

Base map: Google Earth with U.S. Geological Survey (USGS), Santa Clara County, 2017.

EXPLANATION

af	Artificial Fill
Qha	Alluvium (Holocene)
Qpa	Alluvium (Pleistocene)
~	Geologic contact: dashed where approximate and dotted where concealed, queried where uncertin

N

0 1000 2000 Feet

Approximate scale

FIRST STREET GREEN DEVELOPMENT
101 TO 151 FIRST STREET
Los Altos, California

ROCKRIDGE GEOTECHNICAL

REGIONAL GEOLOGIC MAP

Date 06/13/17 | Project No. 17-1300 | Figure 3

Base Map: U.S. Geological Survey (USGS), National Seismic Hazards Maps - Fault Sources, 2008.

EXPLANATION

🟥	Strike slip
⬛	Thrust (Reverse)
🟦	Normal

0 5 10 Miles

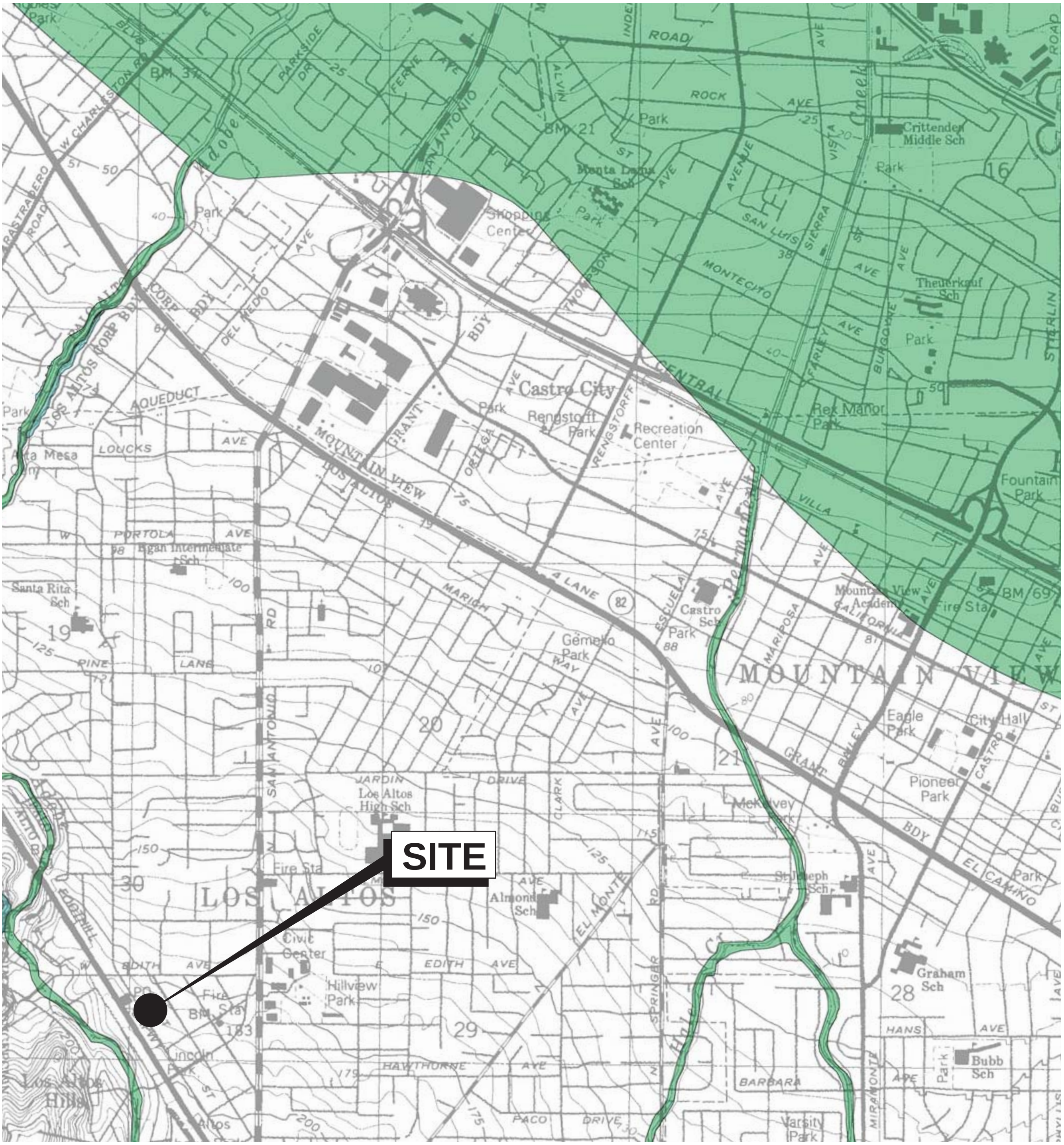
Approximate scale

FIRST STREET GREEN DEVELOPMENT
101 TO 151 FIRST STREET
Los Altos, California

ROCKRIDGE
GEOTECHNICAL

REGIONAL FAULT MAP

Date 06/13/17 | Project No. 17-1300 | Figure 4



EXPLANATION

🟩 **Liquefaction;** Areas where historic occurence of liquefaction, or local topographic, geological, geotechnical, and subsurface water conditions indicate a potential for permanent ground displacements.

🟦 **Earthquake-Induced Landslides;** Areas where previous occurence of landslide movement, or local topographic, geological, geotechnical, and subsurface water conditions indicate a potential for permanent ground displacements.

0 2,000 4,000 Feet

Approximate scale

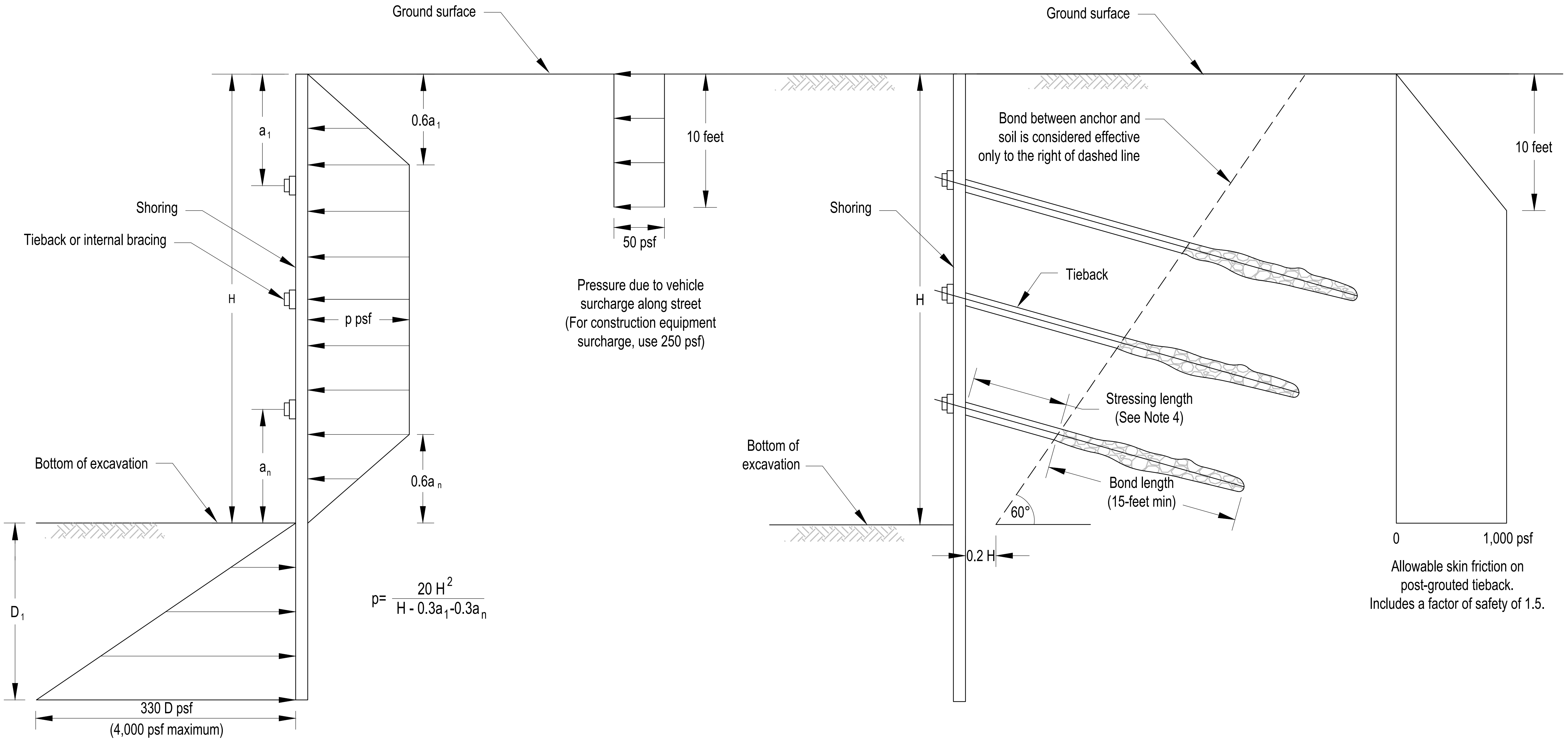
Reference:
State of California "Seismic Hazard Zones"
Mountain View Quadrangle.
Released on October 18, 2006

FIRST STREET GREEN DEVELOPMENT
101 TO 151 FIRST STREET
Los Altos, California

ROCKRIDGE
GEOTECHNICAL

SEISMIC HAZARDS ZONE MAP

Date 06/13/17 | Project No. 17-1300 | Figure 5



NOT TO SCALE

Notes:
1. Passive pressures include a factor of safety of about 1.5.
2. For soldier piles spaced at more than three times the soldier pile diameter, the passive pressure should be assumed to act over three diameters, provided the concrete or soil-cement mix is sufficiently strong to accommodate the corresponding stresses (shoring designer should confirm).
3. Stressing length: minimum 15 feet for strands, minimum 10 feet for bars.

FIRST STREET GREEN DEVELOPMENT
101 TO 151 FIRST STREET
Los Altos, California

DESIGN PARAMETERS FOR
SOLDIER-PILE-AND-LAGGING
TEMPORARY SHORING SYSTEM

Date 06/13/17	Project No. 17-1300	Figure 6

ROCKRIDGE
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APPENDIX A
Logs of Borings

PROJECT: **FIRST STREET GREEN DEVELOPMENT**
101 TO 151 FIRST STREET
Los Altos, California

Log of Boring B-1

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2

Date started: 5/2/17

Date finished: 5/2/17

Drilling method: Hollow Stem Auger

Hammer weight/drop: 140 lbs./30 inches

Hammer type: Downhole wireline

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Logged by: K. Samlik

Approximate Ground Surface Elevation: 187 feet[2]

DEPTH (feet)	SAMPLES Sampler Type	SAMPLES Sample	SAMPLES Blows/ 6"	SAMPLES SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						4 inches of asphalt						
1						CLAYEY SAND with GRAVEL (SC) brown, medium dense, moist, subrounded fine gravel, fine to medium sand, angular coarse gravel						
2	S&H	■	17 10 10	14								
3												
4	S&H	□	6 7 7	10		fine to coarse sand Particle Size Distribution; see Appendix B				18		
5												
6					SC							
7												
8												
9	S&H	■	6 6 9	11		light brown to red-brown						
10												
11												
12						SAND with CLAY and GRAVEL (SP-SC) brown, very dense, medium to coarse sand, fine gravel						
13												
14	SPT	◩	25 28 33	73	SP-SC	Particle Size Distribution; see Appendix B				12		
15												
16												
17						SANDY CLAY (CL) brown, hard, moist, fine sand, trace fine gravel						
18												
19	SPT	◩	26 36 50/4"	103/ 10"								
20												
21												
22												
23					CL							
24	S&H	■	15 26 40	46								
25												
26												
27												
28												
29	S&H	■	26 44 50/4"	66/10"		trace coarse sand						
30												

ROCKRIDGE GEOTECHNICAL

Project No.: 17-1300

Figure: A-1a

PROJECT: **FIRST STREET GREEN DEVELOPMENT**
101 TO 151 FIRST STREET
Los Altos, California

Log of Boring B-1

PAGE 2 OF 2

DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]			Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
					CL	SANDY CLAY (CL) (continued)						
31												
32						CLAYEY SAND with GRAVEL (SC) brown, very dense, moist, medium to coarse sand, subrounded fine gravel						
33												
34	SPT	◪	13 20 28	58		Particle Size Distribution; see Appendix B				17		
35												
36					SC							
37												
38												
39	S&H	■	40 26 50/4"	53/10"		fine to coarse sand						
40												
41						SANDY CLAY (CL) brown, hard, moist, fine to coarse sand, trace angular fine gavel						
42												
43					CL							
44	SPT	◪	30 34 50	101								
45												
46												
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 45 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively, to account for sampler type and hammer energy. SPT sampler used without liners.

[2] Ground surface elevation estimated using "Site Plan" prepared by Kier & Wright Civil Engineers & Surveyors, Inc., dated September 24, 2014. (Datum: NAVD88)

ROCKRIDGE GEOTECHNICAL

Project No.: 17-1300
Figure: A-1b

PROJECT: **FIRST STREET GREEN DEVELOPMENT**
101 TO 151 FIRST STREET
Los Altos, California

Log of Boring B-2

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2
Date started: 5/2/17 Date finished: 5/2/17
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches Hammer type: Downhole wireline
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Logged by: K. Samlik

LABORATORY TEST DATA

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Approximate Ground Surface Elevation: 186 feet[2]	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0						2 inches of asphalt						
1–2	S&H	■	14 14 15	20	CL	SANDY CLAY (CL) red-brown, very stiff, moist, fine to coarse sand, trace rootlets LL = 33, PI = 12; see Appendix B					14.3	115
4	S&H	■	9 24 33	40		hard, no rootlets						
6					SC	CLAYEY SAND with GRAVEL (SC) brown, dense, moist, fine to coarse sand, subangular fine to coarse gravel						
9	S&H	■	20 30 37	47		Particle Size Distribution; see Appendix B				14		
14	SPT	◪	23 28 27	66		very dense						
19	SPT	◪	14 37 35	86								
24	SPT	◪	36 50/6"	60/6"		coarse gravel in sampler shoe Particle Size Distribution; see Appendix B				15		
29	SPT	◪	20 50/6"	60/6"		coarse gravel in sampler shoe						
30												

ROCKRIDGE GEOTECHNICAL

Project No.: 17-1300
Figure: A-2a

PROJECT: **FIRST STREET GREEN DEVELOPMENT**
101 TO 151 FIRST STREET
Los Altos, California

Log of Boring B-2

PAGE 2 OF 2

DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]			Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						CLAYEY SAND with GRAVEL (SC) (continued)						
31												
32												
33												
34	SPT	◪	28 28 27	66		coarse gravel-sized fragments						
35												
36												
37					SC							
38												
39	SPT	◪	44 46 40	103								
40												
41												
42												
43												
44	SPT	◪	25 20 25	54								
45												
46												
47												
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60												

Boring terminated at a depth of 45 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively, to account for sampler type and hammer energy. SPT sampler used without liners.

[2] Ground surface elevation estimated using "Site Plan" prepared by Kier & Wright Civil Engineers & Surveyors, Inc., dated September 24, 2014. (Datum: NAVD88)

ROCKRIDGE GEOTECHNICAL

Project No.: 17-1300

Figure: A-2b

PROJECT: **FIRST STREET GREEN DEVELOPMENT**
101 TO 151 FIRST STREET
Los Altos, California

Log of Boring B-3

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2
Date started: 5/3/17
Date finished: 5/3/17
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Downhole wireline
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by: K. Samlik

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Approximate Ground Surface Elevation: 188 feet[2]	LABORATORY TEST DATA Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0						6 inches of concrete						
1–2.5	S&H	■	32 33 36	48	SC	CLAYEY SAND with GRAVEL (SC) light brown and olive with dark brown, dense, moist, angular fine gravel, fine to coarse sand						
3.5						medium dense						
4–5	S&H	■	16 18 21	27	CL	SANDY CLAY (CL) red-brown, very stiff, moist, fine sand LL = 33, PI = 14; see Appendix B					16.0	117
9–10	S&H	■	14 14 24	27		trace coarse sand Corrosion Test; see Appendix B						
14–15	S&H	■	26 36 50/4"	60/ 10"	SP-SC	SAND with CLAY and GRAVEL (SP-SC) brown, very dense, moist, angular fine gravel, fine and coarse sand						
19–20	SPT	◪	30 14 16	36	CL	SANDY CLAY (CL) brown, hard, moist, medium to coarse sand, trace subrounded fine gravel						
24–25	S&H	■	5 22 30	36		fine sand, no subrounded fine gravel						
27					CL	CLAY with SAND (CL) red-brown, hard, moist, fine sand						
28.5–30	S&H	■	22 32 42	52								
29.5					CL	SANDY CLAY (CL)						

PROJECT: **FIRST STREET GREEN DEVELOPMENT**
101 TO 151 FIRST STREET
Los Altos, California

Log of Boring B-3

PAGE 2 OF 2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
					CL	SANDY CLAY (CL) (continued) red-brown, hard, moist, fine to medium sand						
31												
32						CLAYEY SAND with GRAVEL (SC) brown with yellow and orange mottling, dense, moist, fine to coarse sand, subrounded fine gravel Particle Size Distribution; see Appendix B						
33												
34	SPT	◩	18 14 18	38						18		
35												
36												
37												
38					SC							
39	SPT	◩	22 34 50	101		red-brown, very dense, medium to coarse sand, fine angular gravel						
40												
41												
42												
43												
44	SPT	◩	36 38 33	85		red-brown with light brown and dark brown mottling, fine to coarse sand, subrounded fine gravel						
45												
46												
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 45 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively, to account for sampler type and hammer energy. SPT sampler used without liners.

[2] Ground surface elevation estimated using "Site Plan" prepared by Kier & Wright Civil Engineers & Surveyors, Inc., dated September 24, 2014. (Datum: NAVD88)

ROCKRIDGE GEOTECHNICAL

Project No.: 17-1300

Figure: A-3b

PROJECT: **FIRST STREET GREEN DEVELOPMENT**
101 TO 151 FIRST STREET
Los Altos, California

Log of Boring B-4

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2
Date started: 5/4/17
Date finished: 5/4/17
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Downhole wireline
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by: K. Samlik

DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]		Approximate Ground Surface Elevation: 189 feet[2]	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						4 inches of concrete						
1						SANDY CLAY (CL) red-brown, very stiff, moist, fine to medium sand, trace organics						
2	S&H	■	8 11 18	20								
3												
4	S&H	■	12 22 30	36	CL	brown, hard					20.5	109
5												
6												
7												
8												
9	S&H	■	20 34 50/5"	59/ 11"		SILT (ML) brown, hard, moist						
10					ML							
11												
12												
13						SAND with CLAY and GRAVEL (SP-SC) dark brown, very dense, moist, angular to subrounded fine gravel, fine to coarse sand Particle Size Distribution; see Appendix B						
14	S&H	■	40 46 35	57						6		
15												
16												
17					SP-SC							
18												
19	SPT	◩	22 36 29	78		brown						
20												
21												
22												
23						SANDY CLAY (CL) brown, hard, moist, fine sand						
24	SPT	◩	6 11 26	44								
25					CL							
26												
27						SANDY CLAY with GRAVEL (CL) red-brown, very stiff, moist, subangular to angular coarse gravel						
28					CL							
29	SPT	◩	9 11 13	29								
30												

ROCKRIDGE GEOTECHNICAL

Project No.: 17-1300
Figure: A-4a

PROJECT: **FIRST STREET GREEN DEVELOPMENT**
101 TO 151 FIRST STREET
Los Altos, California

Log of Boring B-4

PAGE 2 OF 2

	SAMPLES						LABORATORY TEST DATA					
DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						SANDY CLAY with GRAVEL (CL) (continued)						
31					CL							
32						CLAYEY SAND with GRAVEL (SC) red-brown, very dense, moist, medium to coarse sand, trace fine subrounded to angular gravel						
33												
34	SPT	◪	26 40 36	91								
35												
36					SC							
37												
38												
39	SPT	◪	12 24 10	41		dense						
40						SANDY CLAY (CL) red-brown, hard, moist, subrounded coarse sand, fine sand						
41												
42					CL							
43												
44	S&H	▨	4 14 40	38								
45												
46												
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 45 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

---

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively, to account for sampler type and hammer energy. SPT sampler used without liners.

[2] Ground surface elevation estimated using "Site Plan" prepared by Kier & Wright Civil Engineers & Surveyors, Inc., dated September 24, 2014. (Datum: NAVD88)

ROCKRIDGE GEOTECHNICAL

Project No.: 17-1300

Figure: A-4b

ROCKRIDGE 17-1300.GPJ TR.GDT 6/13/17

PROJECT: **FIRST STREET GREEN DEVELOPMENT**
101 TO 151 FIRST STREET
Los Altos, California

Log of Boring B-5

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2
Date started: 5/3/17
Date finished: 5/3/17
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Downhole wireline
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by: K. Samlik

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Approximate Ground Surface Elevation: 186 feet[2]	LABORATORY TEST DATA Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						2 inches of asphalt						
1–3	S&H	■	5 10 16	18	CL	SANDY CLAY (CL) dark brown, very stiff, moist, fine to medium sand Corrosion Test; see Appendix B						
4–7	S&H	■	16 16 35	36	CL	CLAY with SAND (CL) brown, hard, moist, fine sand					20.0	97
7–11	S&H	■	8 14 17	22	CL	SANDY CLAY (CL) brown, very stiff, moist, fine to medium sand, trace subangular fine gravel					15.5	111
12–16	S&H	■	40 50/6"	35/6"	SP-SC	SAND with CLAY and GRAVEL (SP-SC) brown, very dense, moist, fine to coarse sand, coarse gravel in sampler shoe						
17–22	SPT	◩	24 18 22	48	SC	CLAYEY SAND (SC) brown, dense, moist, fine to coarse sand						
22–28	SPT	◩	6 8 9	20	CL	SANDY CLAY (CL) light brown, very stiff, moist, fine sand						
29–30	S&H	■	12 18 34	36		dark brown, hard, trace medium sand						

ROCKRIDGE GEOTECHNICAL
Project No.: 17-1300
Figure: A-5a

PROJECT: **FIRST STREET GREEN DEVELOPMENT**
101 TO 151 FIRST STREET
Los Altos, California

Log of Boring B-5

PAGE 2 OF 2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
						SANDY CLAY (CL) (continued)						
31					CL							
32												
33						SANDY CLAY with GRAVEL (CL) dark brown, hard, moist, fine to coarse sand, angular fine gravel						
34	S&H	■	5 26 50	53	CL							
35												
36												
37						CLAYEY SAND with GRAVEL (SC) brown, very dense, moist, fine to coarse sand, subangular fine gravel						
38												
39	S&H	■	40 50/5"	35/5"								
40					SC							
41												
42												
43												
44	SPT	◪	42 50/6"	60/6"								
45												
46												
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 44.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively, to account for sampler type and hammer energy. SPT sampler used without liners.

[2] Ground surface elevation estimated using "Site Plan" prepared by Kier & Wright Civil Engineers & Surveyors, Inc., dated September 24, 2014. (Datum: NAVD88)

ROCKRIDGE GEOTECHNICAL

Project No.: 17-1300

Figure: A-5b

PROJECT:	**FIRST STREET GREEN DEVELOPMENT 101 TO 151 FIRST STREET** Los Altos, California

Log of Boring B-6

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2

Date started: 5/4/17 | Date finished: 5/4/17

Drilling method: Hollow Stem Auger

Hammer weight/drop: 140 lbs./30 inches | Hammer type: Downhole wireline

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Logged by: K. Samlik

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION — Approximate Ground Surface Elevation: 190 feet[2]	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						4 inches of concrete						
					CL	SANDY CLAY (CL) red-brown, hard, moist, fine to medium sand LL = 37, PI = 18; see Appendix B						
2	S&H		12 21 30	36							15.6	119
4	S&H		12 28 38	46		brown						
					SC	CLAYEY SAND (SC) yellow-brown, medium dense, moist, fine to medium sand Particle Size Distribution; see Appendix B						
9	S&H		8 12 22	24						28	13.1	120
					SP-SC	SAND with CLAY and GRAVEL (SP-SC) brown, dense, moist, medium to coarse sand, subangular fine to coarse gravel						
14	S&H		30 36 30	46								
19	SPT		38 28 20	58		very dense, angular fine gravel						
24	SPT		18 26 40	79		Particle Size Distribution; see Appendix B				12		
29	SPT		40 36 16	62								
30					CL	SANDY CLAY (CL)						

ROCKRIDGE GEOTECHNICAL

Project No.: 17-1300

Figure: A-6a

PROJECT: **FIRST STREET GREEN DEVELOPMENT**
101 TO 151 FIRST STREET
Los Altos, California

Log of Boring B-6

PAGE 2 OF 2

	SAMPLES						LABORATORY TEST DATA					
DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31					CL	SANDY CLAY (CL) (continued) brown, hard, moist, fine sand, trace coarse sand						
32						SANDY CLAY with GRAVEL (CL) brown, hard, moist, fine to medium sand, subangular coarse gravel						
34	S&H	▨	18 42 50/5"	64/4"	CL							
36						CLAYEY SAND with GRAVEL (SC) brown, very dense, moist, fine to medium sand, angular coarse gravel						
39	SPT	▨	50/5"	60/5"								
41					SC							
44	SPT	▨	30 35 30	78								
45												

Boring terminated at a depth of 45 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively, to account for sampler type and hammer energy. SPT sampler used without liners.

[2] Ground surface elevation estimated using "Site Plan" prepared by Kier & Wright Civil Engineers & Surveyors, Inc., dated September 24, 2014. (Datum: NAVD88)

ROCKRIDGE GEOTECHNICAL

Project No.: 17-1300

Figure: A-6b

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions		Symbols	Typical Names
Coarse-Grained Soils (more than half of soil > no. 200 sieve size)	Gravels (More than half of coarse fraction > no. 4 sieve size)	**GW**	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	Sands (More than half of coarse fraction < no. 4 sieve size)	**SW**	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
Fine -Grained Soils (more than half of soil < no. 200 sieve size)	Silts and Clays LL = < 50	**ML**	Inorganic silts and clayey silts of low plasticity, sandy silts, gravelly silts
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	Silts and Clays LL = > 50	**MH**	Inorganic silts of high plasticity
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic silts and clays of high plasticity
Highly Organic Soils		**PT**	Peat and other highly organic soils

GRAIN SIZE CHART

Classification	Range of Grain Sizes	
	U.S. Standard Sieve Size	Grain Size in Millimeters
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel coarse fine	3" to No. 4 3" to 3/4" 3/4" to No. 4	76.2 to 4.76 76.2 to 19.1 19.1 to 4.76
Sand coarse medium fine	No. 4 to No. 200 No. 4 to No. 10 No. 10 to No. 40 No. 40 to No. 200	4.76 to 0.075 4.76 to 2.00 2.00 to 0.420 0.420 to 0.075
Silt and Clay	Below No. 200	Below 0.075

▽ Unstabilized groundwater level

▼ Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

⬒	Sample taken with Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter. Darkened area indicates soil recovered
◩	Classification sample taken with Standard Penetration Test sampler
◨	Undisturbed sample taken with thin-walled tube
⊠	Disturbed sample
⊙	Sampling attempted with no recovery
◫	Core sample
◉	Analytical laboratory sample
▥	Sample taken with Direct Push sampler
▥	Sonic

SAMPLER TYPE

C	Core barrel
CA	California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter
D&M	Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube
O	Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube
PT	Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube
S&H	Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter
SPT	Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.5-inch inside diameter
ST	Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

FIRST STREET GREEN DEVELOPMENT
101 TO 151 FIRST STREET
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ROCKRIDGE
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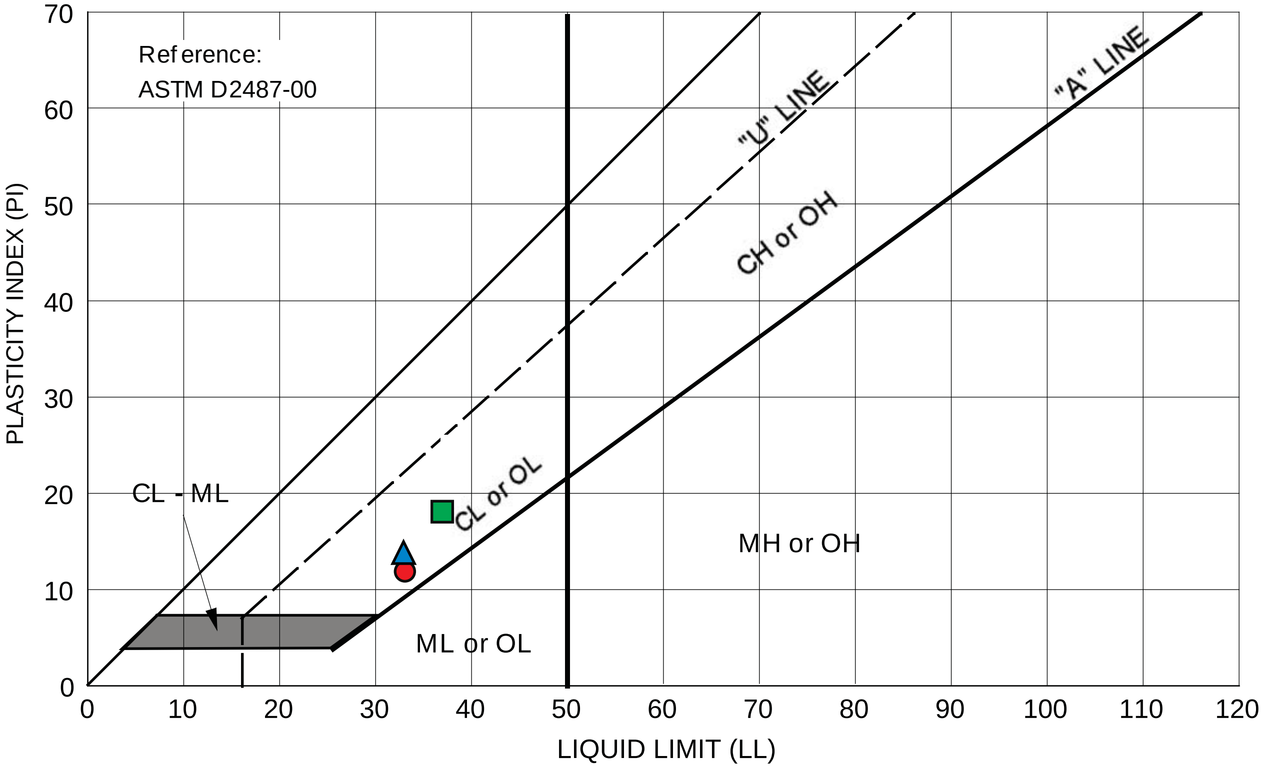
CLASSIFICATION CHART

Date 06/13/17 | Project No. 17-1300 | Figure A-7

APPENDIX B
Laboratory Test Results



Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-2 at 2.0 feet	SANDY CLAY (CL), red-brown	14.3	33	12	--
▲	B-3 at 4.5 feet	SANDY CLAY (CL), red-brown	16.0	33	14	--
■	B-6 at 2.0 feet	SANDY CLAY (CL), red-brown	15.6	37	18	--

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101 TO 151 FIRST STREET
Los Altos, California

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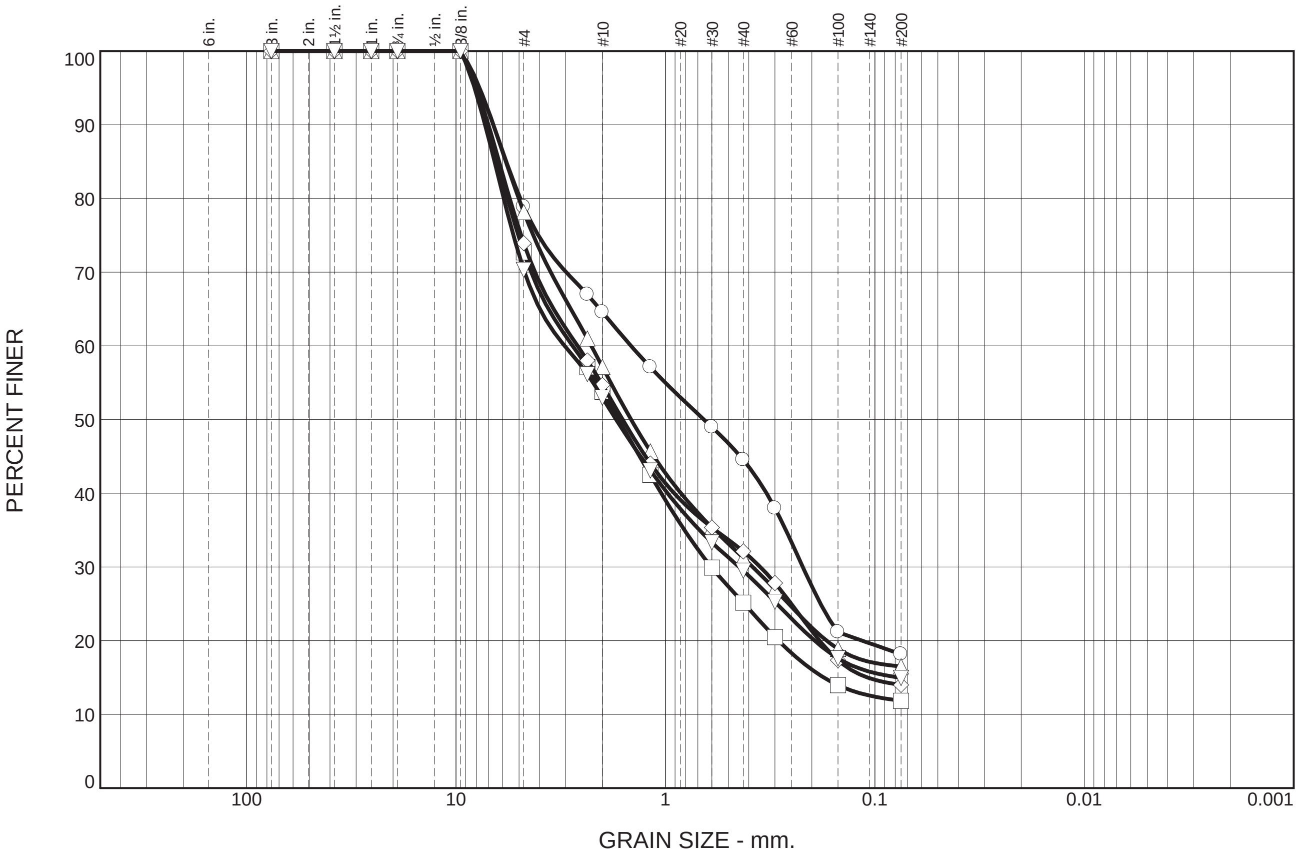
PLASTICITY CHART

Date 06/13/17 | Project No. 17-1300 | Figure B-1



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay

MATERIAL DATA

SYMBOL	SOURCE	DEPTH (ft.)	Material Description	USCS
○	B-1	4.5'	CLAYEY SAND with GRAVEL, brown	SC
□	B-1	13.5'	SAND with CLAY and GRAVEL, brown	SP-SC
△	B-1	33.5'	CLAYEY SAND with GRAVEL, brown	SC
◇	B-2	9.5'	CLAYEY SAND with GRAVEL, brown	SC
▽	B-2	23.5'	CLAYEY SAND with GRAVEL, brown	SC

FIRST STREET GREEN DEVELOPMENT
101 TO 151 FIRST STREET
Los Altos, California

ROCKRIDGE
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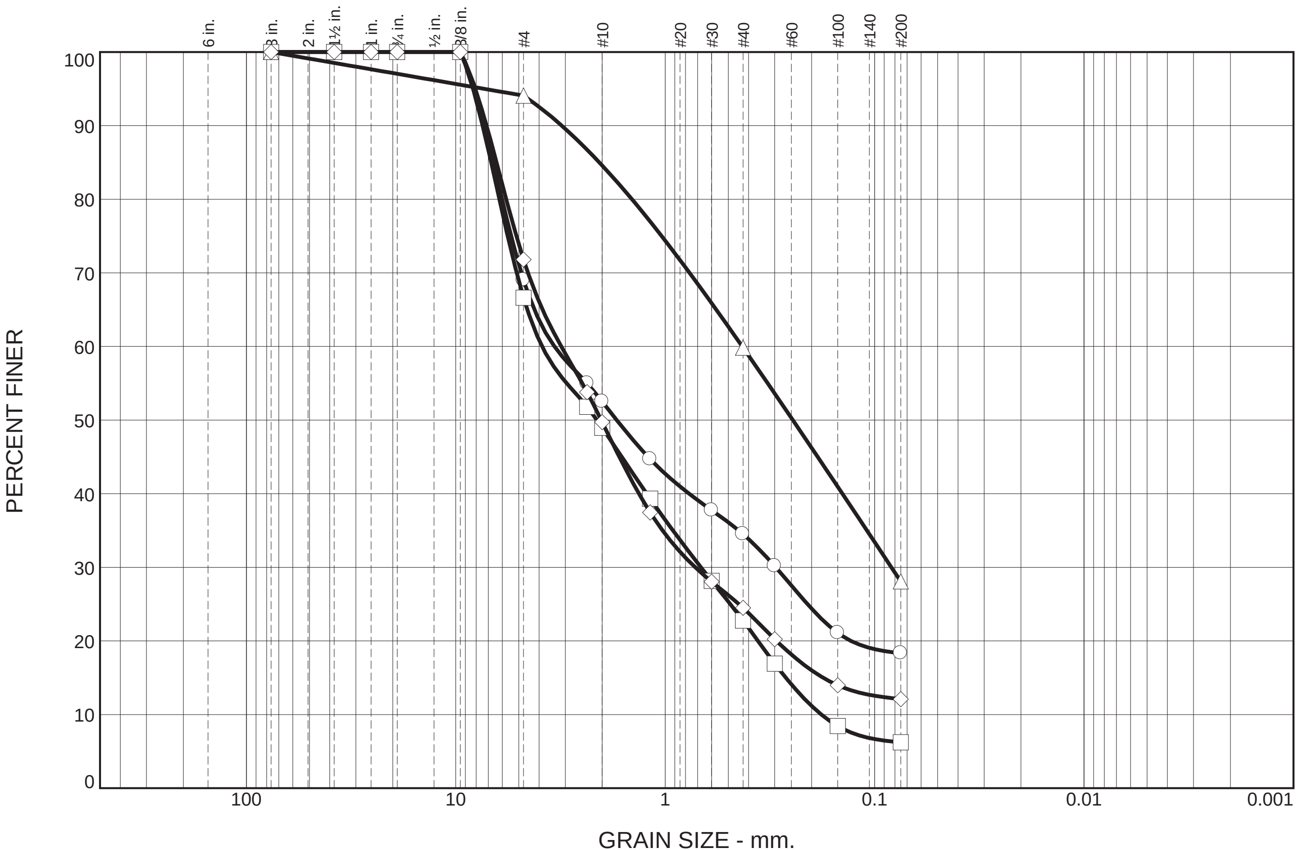
PARTICLE SIZE DISTRIBUTION REPORT

Date 06/13/17 | Project No. 17-1300 | Figure B-2



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay

MATERIAL DATA

SYMBOL	SOURCE	DEPTH (ft.)	Material Description	USCS
○	B-3	33.5'	CLAYEY SAND with GRAVEL, brown with yellow and orange mottling	SC
□	B-4	14.5'	SAND with CLAY and GRAVEL, dark brown	SP-SC
△	B-6	9.5'	CLAYEY SAND, yellow-brown	SC
◇	B-6	23.5'	SAND with CLAY and GRAVEL, brown	SP-SC

FIRST STREET GREEN DEVELOPMENT
101 TO 151 FIRST STREET
Los Altos, California

ROCKRIDGE
GEOTECHNICAL

PARTICLE SIZE DISTRIBUTION REPORT

Date 06/13/17	Project No. 17-1300	Figure B-3

Results Only Soil Testing
for
First Street Green - Office

May 16, 2017

Prepared for:
Darcie Moffioli
Rockridge Geotechnical
270 Grand Ave,
Oakland, CA 94610
damaffioli@rockridgegeo.com

Project X Job #: S170512C
Client Job or PO #: 17-1300

SOIL ANALYSIS LAB RESULTS

Client: Rockridge Geotechnical
Job Name: First Street Green - Office
Client Job Number: 17-1300
Project X Job Number: S170512C
May 16, 2017

	Method	ASTM G187	ASTM G187	ASTM D516		ASTM D512B		SM 4500-E	SM 4500-C	SM 4500-D	ASTM G200	ASTM G51
Bore# / Description	**Depth**	**As-Rec'd Resistivity**	**Min-Resistivity**	**Sulfates**		**Chlorides**		**Nitrate**	**Ammonia**	**Sulfide**	**Redox**	**pH**
	(ft)	**(Ohm-cm)**	**(Ohm-cm)**	**(mg/kg)**	**(wt%)**	**(mg/kg)**	**(wt%)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mV)**	
B - 3	9.5	3,082	1,742	150	0.0150	153	0.0153	51	17	1.17	154	7.12
B - 5	2.0	1,139	1,139	60	0.0060	135	0.0135	45	20	1.41	170	7.19

Unk = Unknown
NT = Not Tested
ND = 0 = Not Detected
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
mg/L - milligrams per liter of liquid volume
Chemical Analysis performed on 1:3 Soil-To-Water extract

Please call if you have any questions.

Prepared by,

Ernesto Padilla, BSME
Field Engineer

Respectfully Submitted,

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