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GEOTECHNICAL

Prepared for **180 Jones Associates, L.P.**

GEOTECHNICAL INVESTIGATION REPORT
PROPOSED AFFORDABLE HOUSING BUILDING
180 JONES STREET
SAN FRANCISCO, CALIFORNIA

April 8, 2020
Project No. 20-1805

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Mr. Jacob Goldstein
Tenderloin Neighborhood Development Corporation
201 Eddy Street
San Francisco, California 94102

Subject: Geotechnical Investigation Report
Proposed Affordable Housing Building
180 Jones Street
San Francisco, California

Dear Mr. Goldstein,

This report presents the results of our geotechnical investigation for the proposed affordable housing building to be constructed at 180 Jones Street in San Francisco, California. Our geotechnical investigation was performed in accordance with the Professional Services Agreement with 180 Jones Associates, L.P., dated January 26, 2020.

The project site is on the southeastern corner of the intersection of Turk and Jones streets. The subject property is a relatively level, rectangular-shaped, asphalt-paved parking lot with plan dimensions of 57.5 by 82.5 feet. The site is bordered by a six-story building to the south, a six-story parking garage to the east, Turk Street to the north and Jones Street to the west. The six-story building to the south has one level of basement and the six-story parking garage to the east has 2 to 3 levels of basement.

Plans are to construct an at-grade affordable housing building that will occupy the entire site. The proposed building will be nine stories high with a lobby, common areas, management offices, and utility rooms on the ground floor.

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 concerns at the project site are:

- the presence of loose to medium dense fill beneath the site that is susceptible to excessive static settlement under new building loads;
- providing adequate vertical and lateral support for the proposed building; and
- the presence of neighboring structure(s) with basement levels bordering the eastern and southern property lines.

We judge the proposed building may be supported on a mat foundation bearing on soil improved with drilled displacement sand-cement (DDSC) columns. DDSC column ground improvement serves to stiffen the overall soil matrix by densifying loose soil layers and/or transferring the foundation loads to more competent material below the loose to medium dense fill, thus reducing settlements and providing increased bearing capacity beneath a mat. In addition, properly designed and constructed DDSC columns can transfer mat foundation loads to depths below the zone-of-influence (ZOI) of neighboring basements.

The recommendations contained in our report are based on a limited subsurface exploration and laboratory testing program. Consequently, variations between expected and actual subsurface conditions may be found in localized areas during construction. Therefore, we should be engaged to observe excavation, grading, and installation of temporary shoring, ground improvement elements and foundations, 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,
ROCKRIDGE GEOTECHNICAL, INC.

Krystian Samlik, P.E.
Project Engineer

Linda H.J. Liang, G.E.
Associate Engineer

Enclosure

QUALITY CONTROL REVIEWER:

Craig S. Shields, P.E., G.E.
Principal Engineer

TABLE OF CONTENTS

LIST OF FIGURES

APPENDIX A

APPENDIX D

GEOTECHNICAL INVESTIGATION REPORT
PROPOSED AFFORDABLE HOUSING BUILDING
180 JONES STREET
San Francisco, California

1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by Rockridge Geotechnical, Inc. for the proposed affordable housing building to be constructed at 180 Jones Street in San Francisco, California. The project site is on the southeastern corner of the intersection of Turk and Jones streets, as shown on the Site Location Map, Figure 1.

The subject property is a relatively level, rectangular-shaped, asphalt-paved parking lot with plan dimensions of 57.5 by 82.5 feet, as shown on the Site Plan, Figure 2. The site is bordered by a six-story building to the south, a six-story parking garage to the east, Turk Street to the north and Jones Street to the west. Available plans indicate the six-story building to the south has one level of basement that bottoms about 8-1/2 feet below the ground surface (bgs) and its foundation bottoms about 11 feet bgs. We understand the six-story parking garage to the east has 2 to 3 levels of basement.

Plans are to construct an at-grade affordable housing building that will occupy the entire site. The proposed building will be nine stories high with a lobby, common areas, management offices, and utility rooms on the ground floor.

2.0 SCOPE OF SERVICES

Our geotechnical investigation was performed in accordance with the Professional Services Agreement with 180 Jones Associates, L.P., dated January 26, 2020. Our scope of services consisted of reviewing available subsurface information for the site and vicinity, exploring subsurface conditions at the site by performing two cone penetration tests (CPTs), advancing two hand-auger borings, performing a geophysical survey, performing laboratory testing on selected soil samples, and performing engineering analyses to develop conclusions and recommendations regarding:

- subsurface conditions
- site seismicity and seismic hazards, including the potential for liquefaction and lateral spreading, and total and differential settlement resulting from liquefaction and/or cyclic densification
- 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 for foundation settlement
- floor slabs
- design lateral earth pressures for permanent below-grade walls, such as elevator pit walls
- site preparation and grading, including criteria for fill quality and compaction
- temporary cut slopes and shoring
- soil corrosivity
- construction considerations

We also performed a site-specific ground motion hazard analysis to develop design response spectra in accordance with the 2019 San Francisco Building Code (SFBC).

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

Previously, Earth Mechanics Consulting Engineers (EMCE) performed a geotechnical investigation at the site and presented the findings in a report titled *Geotechnical Investigation, Planned Development, 181-189 Turk Street, San Francisco, California,* dated January 15, 2005. For this investigation, EMCE drilled one exploratory boring at the site to a depth of 12 feet bgs; the approximate location of this boring, labeled as EM-B-1, is shown on Figure 2.

To explore the subsurface conditions at the site, we performed two CPTs and advanced two hand-auger borings. Prior to performing the investigation, we obtained a drilling permit from San Francisco Public Health Department (SFDPH) and contacted Underground Service Alert (USA) to notify them of our work, as required by law. We also retained 1st Call Utility Locating, a private utility locator, to check for buried utilities at the CPT locations. We also performed a geophysical survey on February 6, 2020. Details of our field investigation and laboratory testing are presented below.

3.1 Cone Penetration Tests

Two CPTs, designated as CPT-1 and CPT-2, were performed to obtain in-situ soil data at the approximate locations shown on Figure 2. Middle Earth Geo Testing, Inc. of Orange, California performed the CPTs on February 18, 2020. CPT-1 was advanced to a depth of 50 feet bgs. CPT-2 encountered practical refusal at a depth of 34 feet bgs.

The CPTs were performed by hydraulically pushing a 1.7-inch-diameter cone-tipped probe with a projected area of 15 square centimeters into the ground. The cone-tipped probe measured tip resistance and the friction sleeve behind the cone tip measured frictional resistance. Electrical strain gauges within the cone continuously measured soil parameters for the entire depth advanced. Soil data, including tip resistance and frictional resistance, were recorded by a computer while the test was conducted. A special cone was also used to measure the in-situ soil shear wave velocity in approximately three-foot intervals in CPT-1.

Accumulated data were processed by computer to provide engineering such as the types and approximate strength characteristics of the soil encountered. The CPT logs showing tip resistance and friction ratio, as well as interpreted soil behavior type and shear wave velocities, are presented in Appendix A on Figures A-1a, A-1b and A-2. Upon completion, the CPT holes were backfilled with cement grout and topped with concrete.

3.2 Hand-Auger Borings

Two borings, designated as HA-1 and HA-2, were advanced using a three-inch-diameter hand auger on February 18, 2020 at the approximate locations shown on Figure 2. Our field engineer advanced the borings to obtain samples of the near-surface soil for visual classification and laboratory testing. HA-1 and HA-2 encountered refusal in rubble/debris at depths of 1-1/2 and 3-1/2 feet bgs, respectively. Upon completion, the boreholes were backfilled with the soil cuttings and patched with concrete. Logs of the borings are presented on Figures A-3 and A-4. The soil encountered in our borings was classified in accordance with the classification chart presented on Figure A-5.

3.3 Laboratory Testing

We re-examined the soil samples obtained from our borings to confirm the field classifications and selected samples for laboratory testing. Soil samples were tested for corrosion potential. The results of the laboratory tests are presented in Appendix B.

3.4 Seismic Surface-Wave Survey

To obtain shear wave velocity measurements for the upper 100 feet of soil for developing the site-specific response spectra, Advanced Geological Services (AGS) performed a geophysical survey at the site on February 6, 2020 using the seismic surface wave (MASW) survey method. The MASW survey consisted of collecting seismic surface-wave data along a survey line (seismic line) at the approximate location shown on Figure 2. To perform the MASW survey, a Rayleigh-wave is generated at certain locations (shot points) along a survey line by striking the ground with a sledgehammer. Rayleigh-wave energy at the ground surface is detected by vibration-sensitive devices called geophones. The geophone data are fed to a seismograph, where they are recorded, and then to a computer, where they are analyzed to interpret geologic features, such as shear wave velocities.

Details of the geophysical survey and the survey results are presented in the report prepared by AGS and attached in Appendix C.

4.0 SUBSURFACE CONDITIONS

A regional geologic map prepared by Graymer, et al. (2006), a portion of which is presented on Figure 3, indicates the site is underlain by Quaternary-age beach and Dune sand (Qs) deposits. Based on existing data (EMCE 2005) and the results of our CPTs and hand-auger borings, we conclude the site is underlain by 12 to 15 feet of fill. The fill generally consists of loose to medium dense sand with variable amounts of silt, clay, gravel, and debris (brick and concrete rubble). The fill is underlain by interbedded layers of medium dense to dense sand with variable amounts of silt and clay and stiff sandy clay and silty clay to depths of 28 to 32 feet bgs. Below depths of 28 and 32 feet, we encountered Colma formation that extends to the maximum depth explored of 50 feet bgs. Where explored the Colma formation consists of very stiff to hard clay with variable sand content and dense to very dense sand with variable clay content.

4.1 Groundwater

Middle Earth Geo Testing attempted to perform pore pressure dissipation tests in the CPTs to evaluate the depth to groundwater; however, they were unable to obtain stabilized groundwater readings from the pore pressure dissipation tests. Available historic groundwater information presented in the Seismic Hazard Zone Report for the City and County of San Francisco Quadrangle indicates the historic high groundwater at the site is about 20 to 25 feet bgs. Available subsurface information from geotechnical investigations performed in the site vicinity, including soil borings drilled at 145 Leavenworth Street and 361 Turk Street by Rockridge Geotechnical in 2017, indicates groundwater at the site vicinity to be about 23 to 27 feet bgs.

The depth to groundwater is expected to vary several feet annually, depending on rainfall amounts. Based on the available groundwater information, we conclude a design groundwater depth of 20 feet bgs should be used for this project.

5.0 SEISMIC CONSIDERATIONS

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 San Andreas, Hayward, and San Gregorio faults. These and other faults in the region are shown on 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)] 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 North San Andreas (SAO+SAN+SAP+SAS)	13	Southwest	8.04
North San Andreas (Peninsula, SAP)	13	Southwest	7.38
Total Hayward + Rodgers Creek (RC+HN+HS+HE)	17	East	7.58
Hayward (North, HN)	17	East	6.90
San Gregorio (North)	18	West	7.44
Hayward (South, HS)	21	East	7.00
North San Andreas (North Coast, SAN)	26	West	7.52
Total Calaveras (CN+CC+CS+CE)	33	East	7.43
Calaveras (North, CN)	33	East	6.86
Mount Diablo Thrust North CFM	34	East	6.72
Mount Diablo Thrust	34	East	6.67
Monte Vista - Shannon	35	Southeast	7.14
Concord	39	East	6.45
Green Valley	41	Northeast	6.30
Rodgers Creek - Healdsburg	43	North	7.19
Mount Diablo Thrust South	45	East	6.50
West Napa	45	Northeast	6.97
Clayton	45	East	6.57
Greenville (North)	48	East	6.86

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.

Since 1800, four major earthquakes have been recorded on the San Andreas Fault. In 1836, an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale occurred east of Monterey Bay on the San Andreas Fault (Toppozada and Borchardt 1998). The estimated Moment magnitude, M_w, for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to an M_w of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas Fault from Shelter Cove to San Juan Bautista approximately 470 kilometers in length. It had a maximum intensity of XI (MM), an M_w of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The Loma Prieta Earthquake of October 17, 1989 had an M_w of 6.9 and occurred about 95 kilometers south of the site.

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

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.

5.2 Seismic Hazards

Because the project is in a seismically active region, we evaluated the potential for earthquake-induced geologic hazards including ground shaking, ground surface rupture, liquefaction[2], lateral spreading[3] and cyclic densification.[4] We used the results of our investigation to evaluate the potential of these phenomena occurring at the project site. The results of our analyses and evaluation are presented in the following sections.

5.2.1 Ground Shaking

The seismicity of the site is governed by the activity of the San Andreas Fault, although ground shaking from future earthquakes on other faults, including the Hayward and San Gregorio 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.

5.2.2 Ground Surface Rupture

Historically, ground surface displacements closely follow the trace of geologically young faults. The site is within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act, but 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.

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

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

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

5.2.3 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 been mapped just inside a zone of liquefaction potential as shown on the map titled *State of California Seismic Hazard Zones, City and County of San Francisco, Official Map*, dated November 17, 2000 (Figure 5). We evaluated liquefaction potential at the site using the data collected from our CPTs.

Liquefaction susceptibility was assessed using the software CLiq v3.0 (GeoLogismiki, 2020). CLiq uses measured CPT data and assesses liquefaction susceptibility and post-earthquake vertical settlement, given a user-defined earthquake magnitude and peak ground acceleration (PGA). Our liquefaction analyses were performed using the methodology proposed by Boulanger & Idriss (2014). We also used the relationship proposed by Zhang, Robertson, and Brachman (2002) to estimate post-liquefaction volumetric strains and corresponding ground surface settlement; a relationship that is an extension of the work by Ishihara and Yoshimine (1992). Our analysis was performed using a high groundwater depth of 20 feet bgs. In accordance with the 2019 SFBC, we used a peak ground acceleration of 0.60 times gravity (g) in our liquefaction evaluation; this peak ground acceleration is consistent with the Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration adjusted for site effects (PGA_M). We also used a Moment magnitude 8.04 earthquake, which is consistent with the mean characteristic Moment magnitude for the San Andreas Fault, as presented in Table 1.

Our liquefaction analysis indicates there are thin (less than two feet thick), discontinuous layers of medium dense sand and silty sand below the groundwater that are susceptible to liquefaction. We estimate total settlement associated with liquefaction (referred to as post-liquefaction reconsolidation) after a major event on a nearby fault will be less than about 1/4 inch and differential settlement will be about 1/4 inches over a horizontal distance of 30 feet.

Considering the depth, thickness, and relative density of the potentially liquefiable layers, we conclude the potential for liquefaction-induced ground settlement, surface manifestations (i.e. sand boils) and lateral spreading is nil.

5.2.4 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 site is underlain by loose to medium dense sandy fill above the groundwater table that is susceptible to cyclic densification. We used a PGA_M of 0.60g and Moment magnitude 8.05 earthquake in our cyclic densification evaluation. We estimate total and differential ground settlement as a result of cyclic densification at the site will be up to about 3/4 inch and 1/2 inch across a horizontal distance of 30 feet, respectively.

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 concerns at the project site are:

- the presence of loose to medium dense fill beneath the site that is susceptible to excessive static settlement under new building loads;
- providing adequate vertical and lateral support for the proposed building; and
- the presence of neighboring structures with basement level(s) bordering the eastern and southern property lines.

These and other geotechnical issues are discussed in this section.

6.1 Foundation Support and Settlement

The factors influencing the selection of a safe, economical foundation system are providing an adequate factor of safety against bearing capacity failure, limiting differential settlement to an amount that can be tolerated by the structure above, constructability, and cost. The proposed building will be constructed at-grade and will be underlain by loose to medium dense sandy fill to depths of 12 to 15 feet bgs. Shallow foundations, such as spread footings or a mat, bearing on these soils will experience erratic and excessive settlement caused by compression of the sand under new building loads and seismically induced cyclic densification. In addition, shallow foundations will surcharge adjacent basement walls and foundations. Therefore, we judge the proposed building should not be supported on shallow foundations bearing on unimproved soil.

We judge the building may be supported on a mat foundation bearing on improved soil. Soil improvement serves to stiffen the overall soil matrix by densifying loose soil layers and/or transferring the foundation loads to more competent material below the loose to medium dense fill, thus reducing settlements and providing increased bearing capacity beneath a mat. There are several types of ground improvement that may be utilized to reduce differential settlement of the proposed building to an acceptable value. Based on our experience, we conclude drilled displacement sand-cement (DDSC) columns would be the most appropriate ground improvement method for this project. This system results in low vibrations during installation and is appropriate for use near adjacent structures. In addition, properly designed and constructed DDSC columns can effectively transfer mat foundation loads to depths below the zone-of-influence (ZOI) of neighboring basements.

DDSC columns are installed by advancing a 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. As a result, the DDSC columns densify the surrounding soil. Because of the displacement drilling method, fewer drilling spoils are generated for off-haul. DDSC columns are installed under design-build contracts by specialty contractors.

The capacities and lengths of the ground improvement elements should be determined by the design-build contractor that installs the system; however for planning purposes, it may be assumed that DDSC columns will extend 35 to 40 feet bgs; where DDSC columns are near adjacent basements, the DDSC columns may need to extend deeper to avoid surcharging neighboring basement walls.

We estimate total combined static and seismically induced settlement of a mat foundation supported on ground improved with DDSCs will be on the order of one inch, with less than 1/2 inch of differential settlement across a horizontal distance of 30 feet. The actual allowable bearing pressures and estimated settlements should be provided by the design-build ground improvement contractor.

6.2 Excavation and Temporary Shoring

Excavations that will be entered by workers should be sloped or shored in accordance with the Occupational Safety and Health Administration (OSHA) standards (29 CFR Part 1926). The contractor should be responsible for the construction and safety of temporary slopes. The shoring designer should be responsible for the shoring design.

Where space permits, the sides of the temporary excavation can be sloped. Where space does not permit sloping of the excavation perimeter, a shoring system will be required to support the sides of the proposed excavation. We judge that a cantilevered soldier pile and lagging shoring system is appropriate for support of excavations that are less than 12 feet deep.

6.3 Construction Considerations

The soil to be excavated consists predominately of sandy fill, which can be excavated with conventional earth-moving equipment such as loaders and backhoes. Removal of existing on-site improvements, including pavements and buried foundation, if any, will require equipment capable of breaking concrete.

There are existing buildings adjacent to the site. Heavy equipment should not be used within 10 horizontal feet from adjacent shallow foundations and basement walls. Jumping jack or hand-operated vibratory plate compactors should be used for compacting fill within this zone.

6.4 Soil Corrosivity

Corrosivity testing was performed by Project X Corrosion Testing of Murrieta, California on soil samples obtained from Borings HA-1 and HA-2 at depths of 1.0 and 3.0 feet bgs, respectively. The results of the corrosivity tests are presented in Appendix B. Based on the resistivity test results, the sample is classified as negligibly corrosive to buried steel, which is typical for relatively clean sands. The chloride, sulfide, and sulfate ion concentrations and pH of the soil do not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

7.0 RECOMMENDATIONS

Our recommendations for site preparation and fill placement, design of foundations and below-grade walls, temporary shoring, and other geotechnical aspects of the project are presented in this section.

7.1 Site Preparation and Fill Placement

Site demolition should include the removal of existing underground utilities and foundations that will interfere with the construction of the proposed building. In general, abandoned underground utilities should be removed to the property line or service connections and properly capped or plugged with concrete. Where existing utility lines will not interfere with the proposed construction, they may be abandoned in-place provided the lines are filled with lean concrete or cement grout to the property line. Voids resulting from demolition activities that extend below finished subgrade should be properly backfilled with compacted fill following the recommendations provided later in this section and under the observation of the Geotechnical Engineer.

In areas to receive fill or improvements (i.e. building pad subgrade), the soil subgrade should be scarified to a depth of eight inches, moisture-conditioned to near optimum moisture content, and compacted to at least 95 percent relative compaction[5].

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 placed below foundations, fill greater than five feet in thickness, and any fill material consisting of clean sand or gravel (defined as poorly-graded soil with less than five percent fines by weight) should be compacted to at least 95 percent relative compaction. Fill placed within the upper foot of pavement soil subgrade should also be compacted to at least 95 percent relative compaction and be non-yielding.

7.1.1 Utility Trenches

The thickness and type of bedding material required for utilities will depend on the soil conditions at the utility trench bottom. As a minimum, bedding should extend at least D/4 (with D equal to the outside pipe diameter) below the bottom of the pipe. However, the bedding should be at least four inches thick. This minimum bedding thickness and either clean sand, rod mill or pea gravel bedding material is adequate for shallow trenches above the groundwater level.

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

Backfill for utility trenches should be compacted according to the recommendations presented for general site fill. Jetting of trench backfill should not be permitted. If sand or gravel with less than five percent fines (particles passing the No. 200 sieve) is used, it should be compacted to at least 95 percent relative compaction. Pea gravel, drain rock, and rod mill should be mechanically tamped in 12-inch lifts where placed beneath pavements. Special care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to the pavement section.

7.1.2 Exterior Concrete Slabs

For all concrete flatwork, exterior slabs, and pavers, the upper eight inches of soil should be scarified and compacted in accordance with the compaction requirements presented above in Section 7.1. Exterior slabs and ramps attached to the building should be hinged to accommodate differential settlement between the building and outside ground due to seismically induced cyclic densification.

7.2 Mat Foundation on Ground Improvement

As discussed in Section 6.1, we conclude the proposed building may be supported on a mat foundation bearing on ground improved with DDSC columns. For preliminary design of a mat foundation bearing on improved ground, we recommend ground improvement elements extend to depths of about 35 to 40 feet bgs. We anticipate the ground improvement systems described in Section 7.3, if properly designed and constructed, should be capable of increasing the allowable bearing pressure to 4,000 to 6,000 pounds per square foot (psf) for dead-plus-live loads with a one-third increase for total loads; the allowable bearing pressure will depend on the strength, diameter, length and spacing of the DDSC columns. For design of the mat bearing on improved ground, we recommend using a preliminary modulus of vertical subgrade reaction of 35 to 55 pounds per cubic inch (pci) where the allowable bearing pressures are 4,000 to 6,000 psf, respectively, for dead-plus-live loads; these values may be increased by one-third for total loads. The vertical subgrade reaction values have been reduced to account for the size of the

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compaction procedure.

mat. Once the structural engineer estimates the distribution of bearing stress on the bottom of the mat, we should review the distribution and revise the modulus of vertical subgrade reaction, if appropriate.

The final design allowable bearing pressures, estimated settlements, and modulus of vertical subgrade reaction should be determined by the design-build ground improvement contractor, as these values will be based on the diameter, depth, and spacing of the ground improvement elements.

Lateral loads may be resisted by a combination of passive pressure on the vertical faces of the mat and friction between the bottoms of the mat and the supporting soil. To compute passive resistance, we recommend using an allowable equivalent fluid weight of 240 pounds per cubic foot (pcf); the upper foot of soil should be ignored unless confined by a slab or pavement. The allowable friction factor will depend on whether a vapor retarder is used at the base of the mat. If no membrane is used, an allowable base friction coefficient of 0.35 may be used in design. Where a vapor retarder is placed beneath the mat, a base friction coefficient of 0.20 should be used. The passive pressure and frictional resistance values include a factor of safety of at least 1.5 and may be used in combination without further reduction.

The mat subgrade should be free of standing water, debris, and disturbed materials prior to placing concrete. The mat subgrade should be compacted with a large vibratory plate compactor or a small smooth-drum roller to provide a firm surface prior to placement of the vapor retarder (if used) and reinforcing steel.

7.3 Ground Improvement

DDSC column ground improvement systems are installed under design-build contracts by specialty contractors. The required size, spacing, length, and strength of the DDSC column ground improvement elements should be determined by the design-build contractor based on the proposed structural loads and the desired level of improvement (tolerable settlement and/or desired bearing capacity). The length and spacing of the DDSC columns should be sufficient to limit total combined static and seismic settlement to one inch and differential static settlement to

1/2 inch across a horizontal distance of 30 feet, and to prevent surcharge from the mat foundation onto adjacent basements. For planning purposes, we recommend the ground improvement elements extend to depths of about 35 to 40 feet bgs. Where the DDSC columns are adjacent to neighboring basements, the DDSC columns should be set back at least four feet horizontally from the basement wall and be deepened such that the “equivalent footing” depth is below the basement ZOI. The basement ZOI is an imaginary line projected up from the bottom of the basement at an inclination of 1.5:1 (horizontal to vertical).

We estimate the ground improvement systems previously described, if properly designed, should be capable of increasing the allowable bearing pressures to about 4,000 to 6,000 psf for dead-plus-live loads with a one-third increase for total loads. These allowable bearing pressures may be higher or lower, depending on the size, spacing, depth, strength, and construction methods of the ground improvement elements selected by the design-build contractor.

We recommend the interface between the ground improvement elements and bottoms of mat foundation be separated by a minimum 12-inch-thick compacted aggregate cushion consisting of Class 2 aggregate base or crushed rock. The purpose of the aggregate cushion is to provide some degree of isolation between the two elements, which will help prevent excessive moments from being induced in the ground improvement columns during lateral loading.

We recommend the ground improvement design be verified in the field by performing at least one full-scale load test in compression and one load test in tension (if DDSCs will be used to resist uplift loads). The load tests should be performed by the design-build contractor under our observation. Details regarding the proposed load testing program should be included in the design-build submittal for our review prior to mobilization to the site. The load tests should be performed on non-production elements constructed using the same equipment, means-and-methods, area replacement ratio, and grout factor proposed for the production elements. The results of the load testing program should be evaluated by the design-build contractor’s engineer, as well as our engineer, to confirm the columns provide an adequate factor of safety with respect to axial load failure and an acceptable axial deflection at the design load prior to commencing with production installation.

7.4 Vapor Retarder

If water vapor moving through the mat foundation is considered detrimental, we recommend installing a water vapor retarder beneath the mat. The vapor retarder can be placed directly on the soil subgrade. The vapor retarder should meet the requirements for Class A 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.

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 mat foundation should have a w/c ratio less than 0.45. If necessary, workability should be increased by adding plasticizers. In addition, the mat should be properly cured. Before floor coverings, if any, are placed, the contractor should check that the concrete surface and the moisture emission levels (if emission testing is required) meet the manufacturer's requirements.

7.5 Permanent Below-Grade Walls

Permanent below-grade walls (i.e., elevator pit walls) should be designed to resist lateral earth pressure imposed by the retained soil, as well as a surcharge pressure from nearby vehicles, where appropriate. In addition, because the site is in a seismically active area, below-grade walls should be designed to resist pressures associated with seismic forces.

We recommend unrestrained walls be designed for active pressure of 35 pcf (triangular distribution) plus a seismic increment of 12 pcf (triangular distribution). We recommend restrained below-grade walls at the site be designed for the more critical of:

- at-rest pressure using an equivalent fluid weight of 55 pcf (triangular distribution); or
- active pressure using an equivalent fluid weight of 35 pcf (triangular distribution) plus a seismic increment of 26 pcf (triangular distribution)

Where traffic loads are expected within 10 feet of the walls, an additional design load of 50 psf should be applied to the upper 10 feet of the wall. Below-grade walls adjacent to existing structures should be designed for surcharge pressures if the foundations supporting the adjacent buildings are founded above the zone-of-influence for the below-grade walls. This zone is defined as an imaginary line extending up from the bottom of the wall at an inclination of 1.5:1 (horizontal to vertical). The influence on a wall from a foundation that is founded within this zone-of-influence should be analyzed on an individual basis after the geometry has been determined.

Considering the soil adjacent to the walls and immediately below the walls will consist of sand, which has a relatively high permeability, it will not be necessary to install a groundwater collection system behind the walls because subsurface water from surface water infiltration or other sources will seep downward rather than collect behind the walls.

To protect against moisture migration, below-grade walls should be waterproofed and water stops should be placed at all construction joints. If backfill is required behind below-grade walls, the walls should be braced, or hand compaction equipment used, to prevent unacceptable surcharges on walls (as determined by the Structural Engineer).

7.6 Temporary Cut Slopes and Shoring

Excavations that will be entered by workers should be shored or sloped in accordance with the Occupational Safety and Health Administration (OSHA) standards (29 CFR Part 1926). The contractor should be responsible for the construction and safety of temporary slopes. The shoring designer should be responsible for the shoring design.

Where space permits, the sides of the temporary excavation can be sloped. We recommend temporary slopes not exceed an inclination of 1.5:1 (horizontal to vertical) in sandy soil (OSHA Type C Soil). Where space does not permit sloping of the excavation perimeter, a shoring system will be required to support the sides of the proposed excavation. We judge that a cantilevered soldier pile and lagging shoring system is appropriate for support of excavations that are less than 12 feet deep.

A structural/civil engineer knowledgeable in this type of construction should be retained to design the shoring. The shoring designer should design the shoring system for lateral deformation of less than one inch (1/2 inch if neighboring structures are within a horizontal distance equal to two times the height of the shoring) at any location on the shoring. We should review the final shoring plans and calculations to check that they are consistent with the recommendations presented in this report.

7.6.1 Cantilevered Soldier Pile and Timber Lagging Shoring System

For design of a cantilevered shoring system, we recommend using an at-rest earth pressure equivalent to a fluid weight of 55 pcf where there is a structure within a horizontal distance equal to two times the retained soil height and using an active earth pressure equivalent to a fluid weight of 35 pcf where there are no structures within that horizontal distance. 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. Shoring should be designed for surcharge loads where there will be construction equipment and/or stockpiled soil within a horizontal distance of 10 feet from the edge of excavation. We can provide recommendations for surcharge pressures once surcharge loads are known.

Passive resistance at the toe of the soldier piles should be computed using an equivalent fluid weight of 240 pcf up to a maximum passive earth pressure of 2,000 psf. 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. If lean concrete is placed in the soldier pile shaft, the passive pressure can be assumed to act over two pile diameters. These passive pressure values include a factor of safety of at least 1.5.

Soldier piles should be placed in pre-drilled holes backfilled with concrete. Drilling of the shafts for the soldier piles will likely require casing and/or use of drilling mud to prevent caving. Installing soldier piles by driving or using vibratory methods is not acceptable for this site due to the close proximity to existing structures.

Relatively clean sandy soil will likely be encountered at the excavation. Clean sand is prone to caving. Therefore, we recommend the bottom of excavation not extend more than one foot below the last row of lagging when excavating in clean sand. If voids are created behind lagging boards due to localized caving or overcutting, they should be filled with cement slurry or hand-packed soil prior to proceeding with excavation.

7.7 Seismic Design

Based on the results of our field investigation and engineering analysis, we conclude a designation of Site Class D (V_{s30} = 340 m/s) is appropriate and consistent with the 2019 SFBC.

To estimate the magnitudes of potential ground shaking for the seismic design of the proposed building, we performed a site-specific ground motion hazard analysis. Specifically, we performed a probabilistic seismic hazard analysis (PSHA) and a deterministic seismic hazard analysis to develop site-specific horizontal response spectra for the MCE_R and design (DE) levels of shaking consistent with the definitions presented in the 2019 SFBC and Chapter 21 of ASCE 7-16. Details regarding our site-specific ground motion hazard analysis are presented in Appendix D. The recommended spectra are presented on Figure D-6 and digitized values of the recommended MCE_R and DE spectrum are presented below in Table 2 for a damping ratio of 5 percent.

TABLE 2
Recommended Spectral Accelerations (g)
Damping Ratio of 5 percent

Period (seconds)	MCE_R	DE
0.01	0.712	0.475
0.02	0.715	0.476
0.03	0.739	0.492
0.05	0.840	0.560
0.08	1.009	0.673
0.10	1.162	0.774
0.15	1.404	0.936
0.20	1.558	1.038
0.25	1.668	1.112
0.30	1.731	1.154
0.40	1.738	1.159
0.50	1.648	1.099
0.75	1.302	0.868
1.0	1.200	0.800
1.5	0.800	0.533
2.0	0.600	0.400
3.0	0.400	0.267
4.0	0.300	0.200
5.0	0.240	0.160
7.5	0.160	0.107
10	0.120	0.080

Because the site-specific procedure was used to develop the MCE_R and DE response spectra, the seismic design parameters presented below in Table 3, which were determined in accordance with Section 21.4 of ASCE 7-16, should be used.

TABLE 3
Design Spectral Acceleration Values

Parameter	Spectral Acceleration Value (g)
S_{MS}	1.564
S_{M1}	1.200
S_{DS}	1.043
S_{D1}	0.800

8.0 ADDITIONAL GEOTECHNICAL SERVICES

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 building foundations. These observations will allow us to compare actual with anticipated subsurface 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 borings and CPTs. If any variations or undesirable conditions are encountered during construction, we should be notified so that additional recommendations can be made. The foundation recommendations presented in this report are developed exclusively for the proposed development described in this report and are not valid for other locations and construction in the project vicinity.

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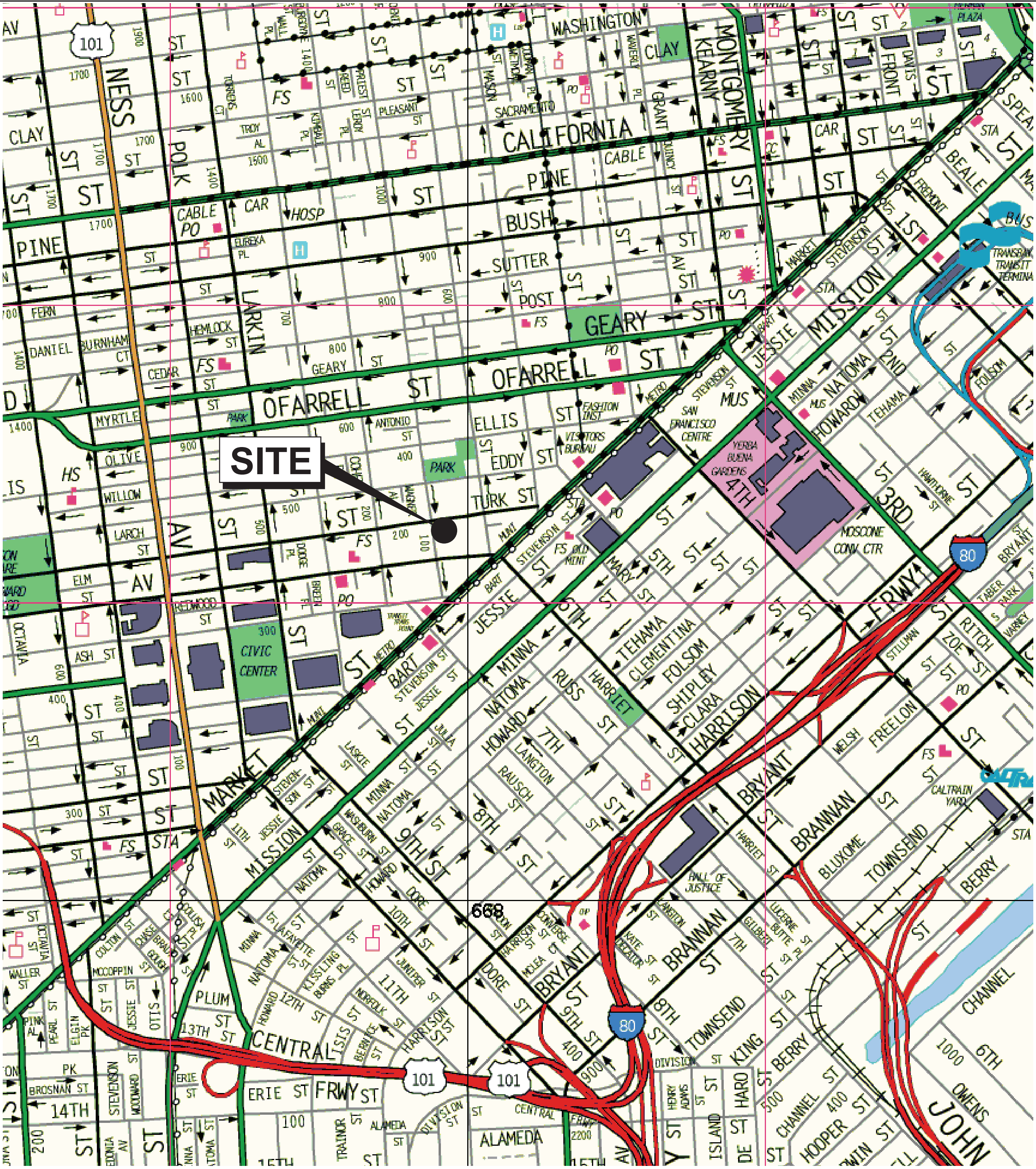
ROCKRIDGE GEOTECHNICAL

FIGURES



Base map: The Thomas Guide
San Francisco County
2002

0 1/4 1/2 Mile

Approximate scale

180 JONES STREET
San Francisco, California

ROCKRIDGE
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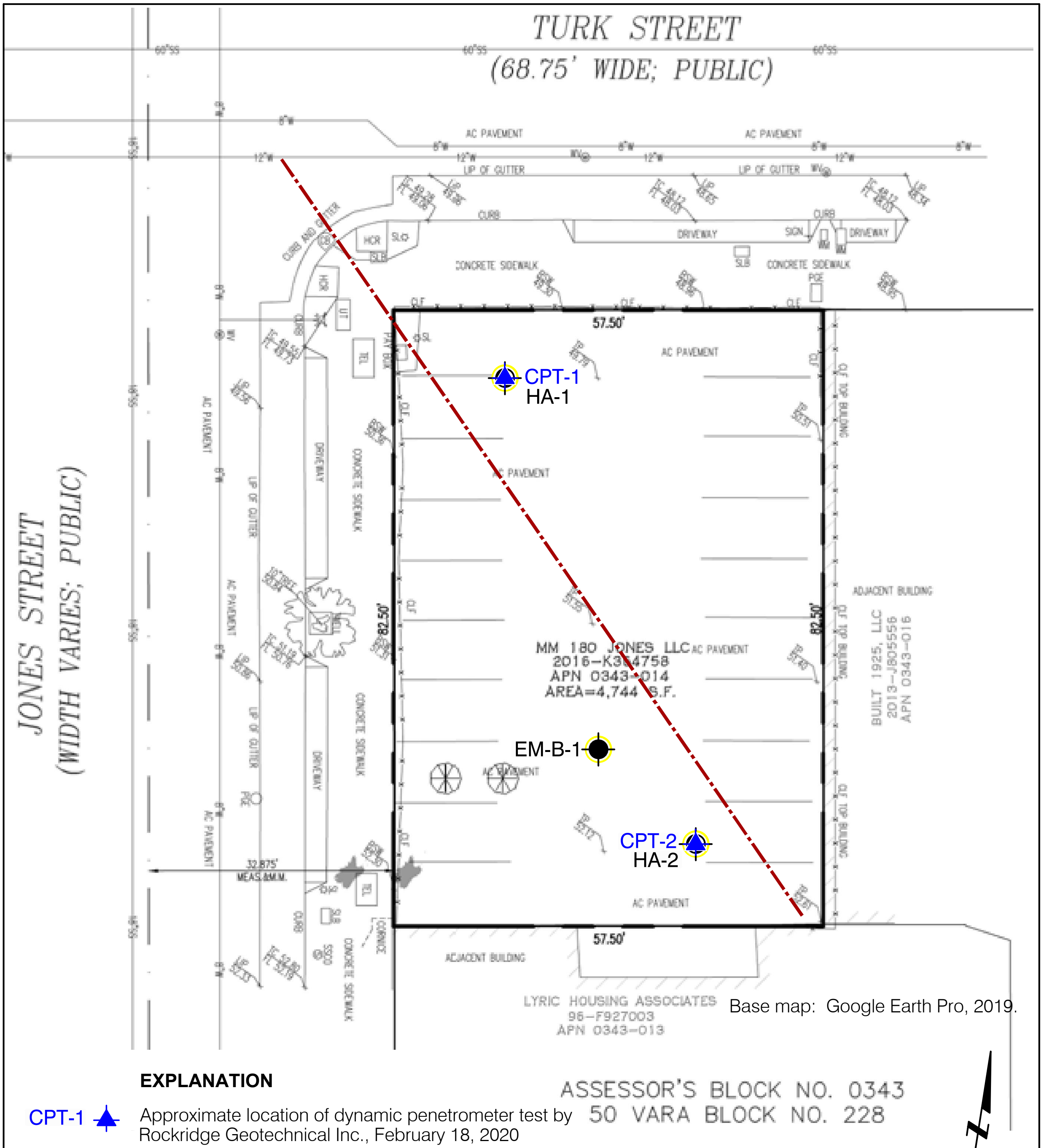
SITE LOCATION MAP

Date 03/12/20 | Project No. 20-1805 | Figure 1



EXPLANATION

CPT-1 — Approximate location of dynamic penetrometer test by Rockridge Geotechnical Inc., February 18, 2020

HA-1 — Approximate location of hand-auger boring by Rockridge Geotechnical Inc., February 18, 2020

EM-B-1 — Approximate location of boring by Earth Mechanics Consulting Engineers, December 5, 2004

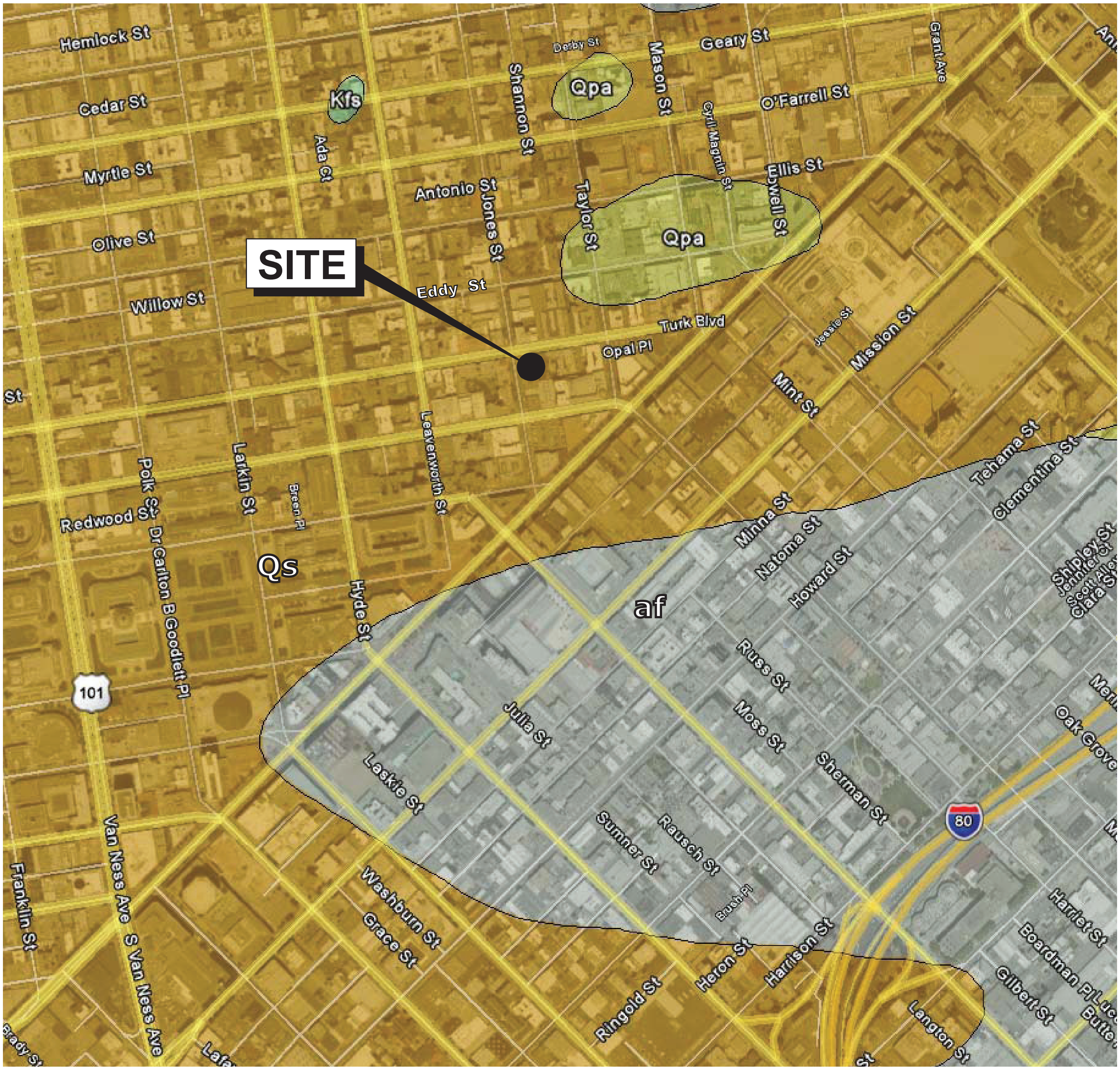
Approximate location of geophysical survey lines performed by Advanced Geological Services, February 6, 2020

0 20 Feet

Approximate scale

Reference: Base map from a drawing titled "A.L.T.A./N.S.P.S. Land Title Survey", by Luk and Associates, dated October 2019.

180 JONES STREET San Francisco, California	SITE PLAN
ROCKRIDGE GEOTECHNICAL	Date 03/13/20 | Project No. 20-1805 | Figure 2



Base map: Google Earth with U.S. Geological Survey (USGS), San Francisco County, 2019.

EXPLANATION

af	**Artificial Fill**
Qs	**Beach and Dune sand (Quaternary)**
Qpa	**Alluvium (Pleistocene)**
Kfs	**Franciscan Complex sedimentary rocks (Cretaceous)**

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

0 1/4 1/2 Mile

Approximate scale

180 JONES STREET
San Francisco, California

ROCKRIDGE GEOTECHNICAL

REGIONAL GEOLOGIC MAP

Date 03/12/20 | Project No. 20-1805 | Figure 3

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

EXPLANATION

Strike slip

Thrust (Reverse)

Normal

0 5 10 Miles

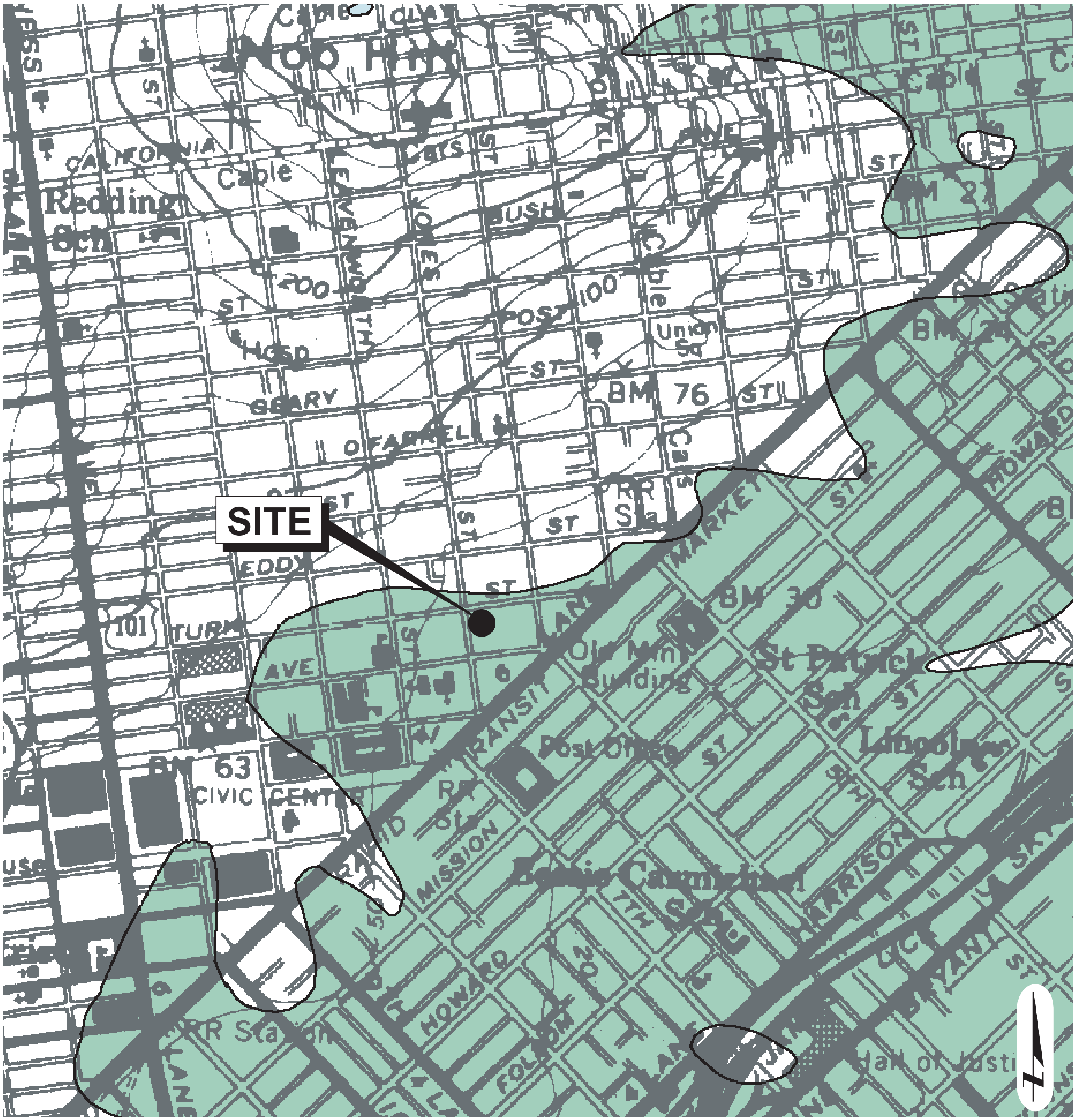
Approximate scale

180 JONES STREET
San Francisco, California

ROCKRIDGE GEOTECHNICAL

REGIONAL FAULT MAP

Date 03/12/20	Project No. 20-1805	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.

Reference:
State of California "Seismic Hazard Zones"
City and County of San Francisco
Released on November 17, 2000

0 1,000 2,000 Feet
Approximate scale

180 JONES STREET
San Francisco, California

ROCKRIDGE GEOTECHNICAL

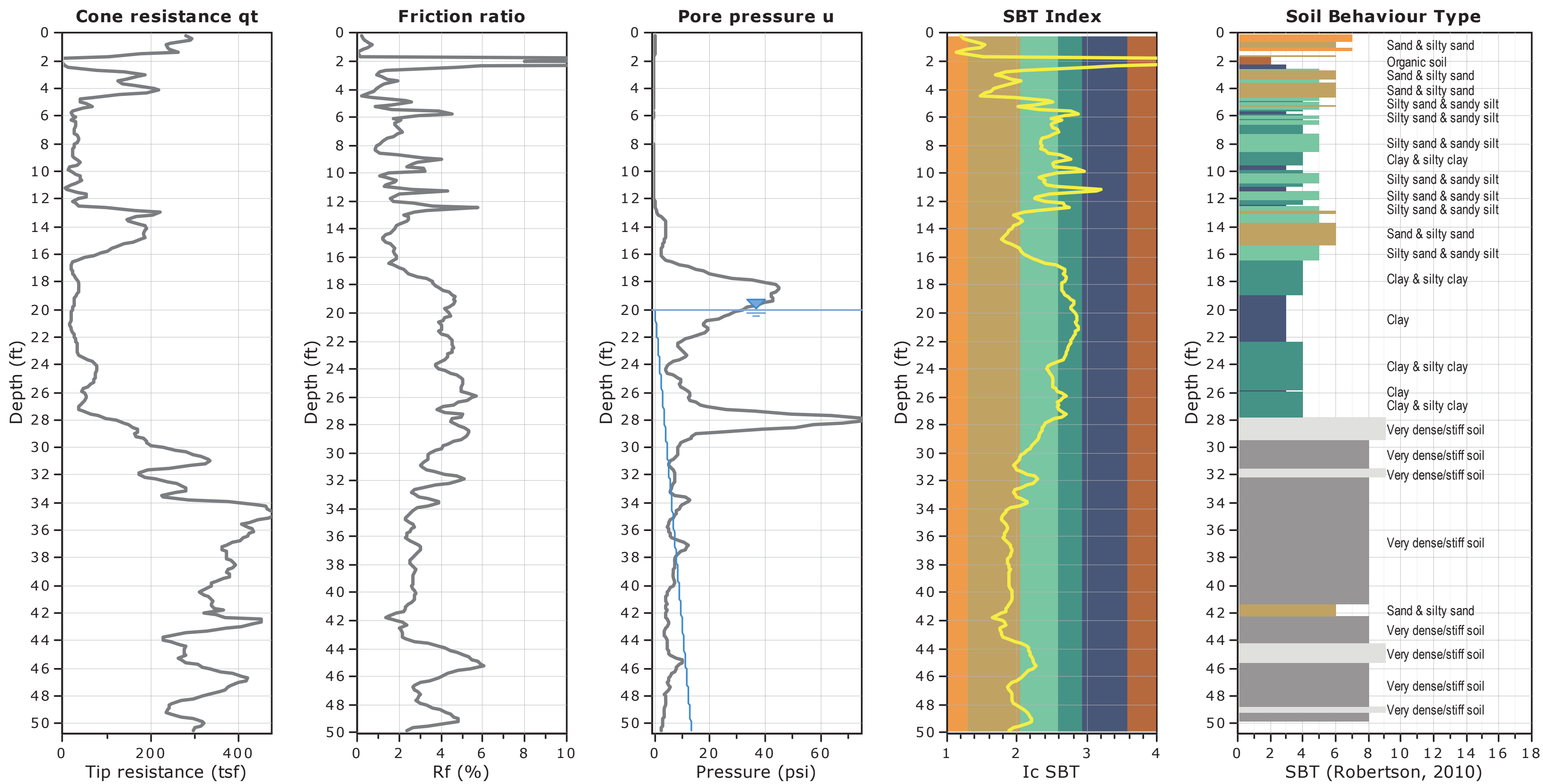
SEISMIC HAZARDS ZONE MAP

Date 03/12/20	Project No. 20-1805	Figure 5

ROCKRIDGE GEOTECHNICAL

APPENDIX A

Cone Penetration Test Results and Boring Logs



Total depth: 50.5 ft, Date: 2/18/2020
Depth to Groundwater: 20 feet (estimated from nearby data)
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

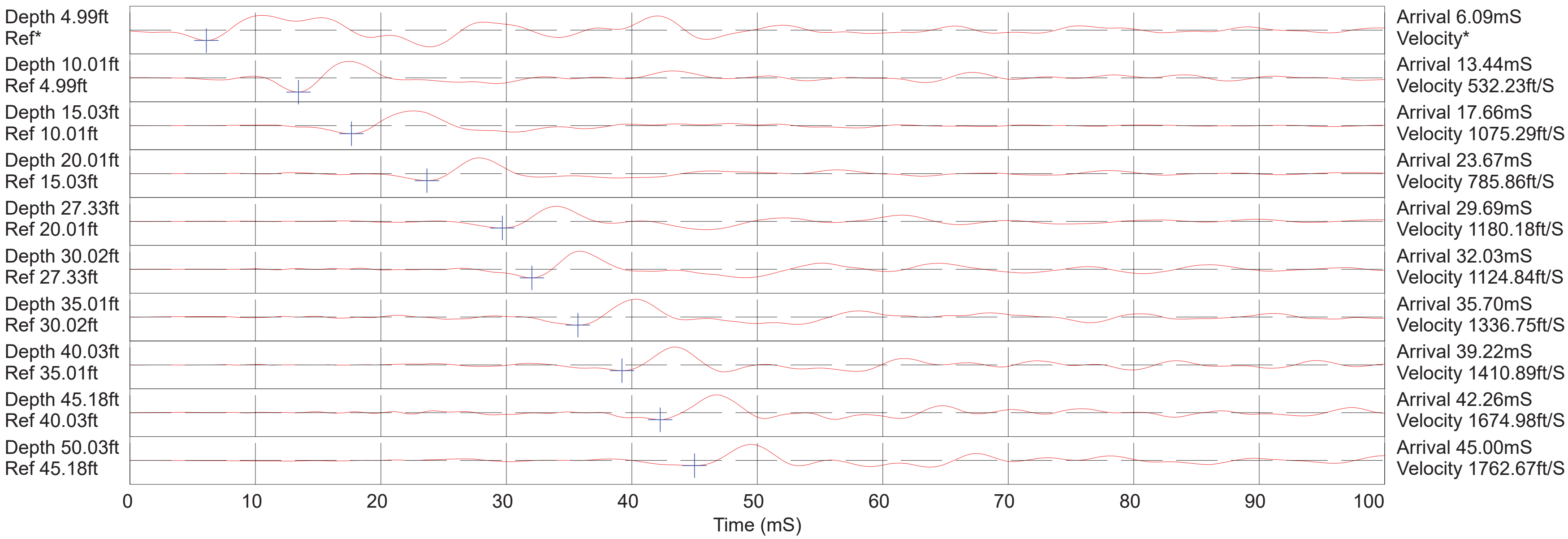
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

180 JONES STREET
San Francisco, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-1

Date 04/08/20 | Project No. 20-1805 | Figure A-1a



Hammer to Rod String Distance (ft): 5.83
* = Not Determined

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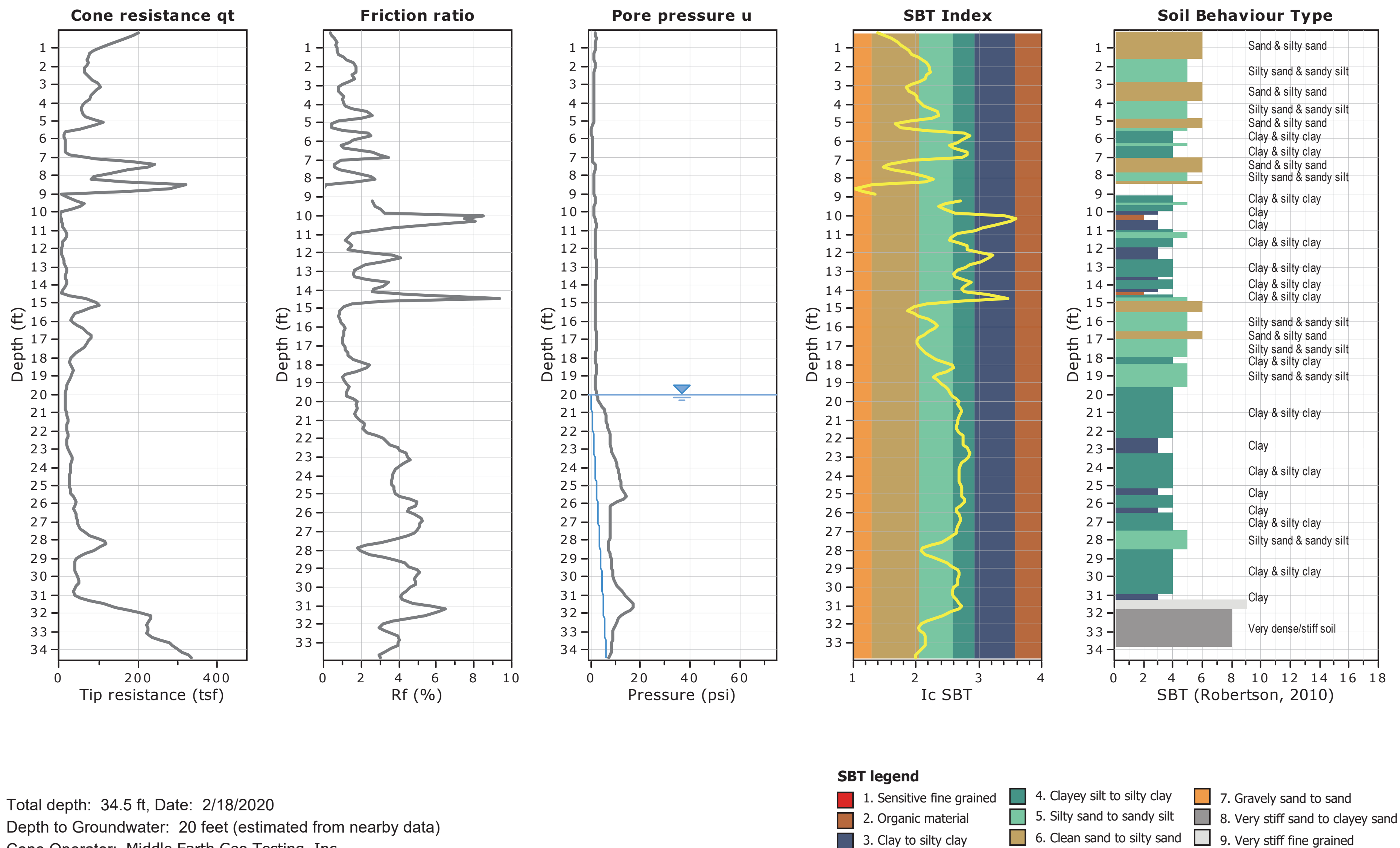
SHEAR WAVE VELOCITIES, CPT-1

Date 04/08/20	Project No. 20-1805	Figure A-1b



Total depth: 34.5 ft, Date: 2/18/2020
Depth to Groundwater: 20 feet (estimated from nearby data)
Cone Operator: Middle Earth Geo Testing, Inc.

180 JONES STREET San Francisco, California	**CONE PENETRATION TEST RESULTS** **CPT-2**
ROCKRIDGE GEOTECHNICAL	Date 03/12/20 · Project No. 20-1805 · Figure A-2

PROJECT:	180 JONES STREET San Francisco, California	Log of Boring HA-1 PAGE 1 OF 1

Boring location: See Site Plan, Figure 2		Logged by: K. Samlik	
Date started: 02/18/2020	Date finished: 02/18/2020		
Drilling method: Hand-Auger			
Hammer weight/drop: N/A	Hammer type: N/A	LABORATORY TEST DATA	
Sampler: Grab			

DEPTH (feet)	SAMPLES: 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
						2-3 inches of asphalt						
						3 inches of aggregate base						
	GRAB	☒			SP	SAND (SP) brown, loose, moist, with debris						
1	GRAB	☒				Corrosivity Test; see Appendix B						
2												
3												
4												
5												
6												
7												
8												
9												
10												

Boring terminated at a depth of 1.5 feet below ground surface.
Boring backfilled with soil cuttings.
Groundwater not encountered during hand-augering.

ROCKRIDGE GEOTECHNICAL

Project No.: 20-1805

Figure: A-3

PROJECT: **180 JONES STREET**
San Francisco, California

Log of Boring HA-2

Boring location:	See Site Plan, Figure 2		
Date started:	02/18/2020	Date finished:	02/18/2020
Drilling method:	Hand-Auger		
Hammer weight/drop:	N/A	Hammer type:	N/A
Sampler:	Grab		

Logged by: K. Samlik

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
						3-4 inches of asphalt						
						2-3 inches of aggregate base						
	GRAB	X			SP	SAND (SP) light brown, loose, moist, with brick rubble						
1	GRAB	X				gravel, debris, up to 2-inch-diameter debris larger than 2 inches in the ground, have to break it up with breaker bar brown						
2	GRAB	X										
3	GRAB	X				Corrosivity Test; see Appendix B						
4												
5												
6												
7												
8												
9												
10												

Boring terminated at a depth of 3.5 feet below ground surface.
Boring backfilled with soil cuttings.
Groundwater not encountered during hand-augering.

ROCKRIDGE GEOTECHNICAL

Project No.: 20-1805

Figure: A-4

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	Range of Grain Sizes: Grain Size in Millimeters
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel	3" to No. 4	76.2 to 4.76
coarse	3" to 3/4"	76.2 to 19.1
fine	3/4" to No. 4	19.1 to 4.76
Sand	No. 4 to No. 200	4.76 to 0.075
coarse	No. 4 to No. 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	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

180 JONES STREET
San Francisco, California

ROCKRIDGE GEOTECHNICAL

CLASSIFICATION CHART

Date 03/12/20 | Project No. 20-1805 | Figure A-5

ROCKRIDGE GEOTECHNICAL

APPENDIX B

Laboratory Test Results

Soil Analysis Lab Results

Client: Rockridge Geotechnical, Inc.
Job Name: 180 Jones
Client Job Number: 20-1805
Project X Job Number: S200303C
March 6, 2020

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51	ASTM G200	SM 4500-S2-D	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327
Bore# / Description	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity**		**pH**	**Redox**	**Sulfide** S^{2-}	**Nitrate** NO_3^-	**Ammonium** NH_4^+	**Lithium** Li^+	**Sodium** Na^+	**Potassium** K^+	**Magnesium** Mg^{2+}	**Calcium** Ca^{2+}	**Flouride** F_2^-	**Phosphate** PO_4^{3-}
						As Rec'd	Minimum												
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ohm-cm)	(Ohm-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
HA-1-2	1.0	27.3	0.0027	22.2	0.0022	34,840	5,427	7.9	133.0	0.5	22.1	8.4	ND	28.9	6.3	6.1	38.1	0.7	2.2
HA-2-4	3.0	73.5	0.0074	14.8	0.0015	28,140	5,092	10.1	95.0	0.3	20.8	1.7	ND	9.3	2.4	0.6	59.2	0.1	2.0

Cations and Anions, except Sulfide and Bicarbonate, tested with Ion Chromatography
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
ND = 0 = Not Detected | NT = Not Tested | Unk = Unknown
Chemical Analysis performed on 1:3 Soil-To-Water extract

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APPENDIX C

MASW Seismic Survey

ADVANCED GEOLOGICAL SERVICES

1605 School Street, #4
Moraga CA 94556
925 (808-8965)

February 11, 2020

Linda H.J. Liang, P.E., G.E.
Rockridge Geotechnical
270 Grand Avenue | Oakland, CA 94610

Subject: Report
MASW Seismic Survey
180 Jones Street
San Francisco, California

Dear Ms. Liang:

1.0 INTRODUCTION

This letter presents the results of Advanced Geological Services, Inc. (AGS) seismic surface-wave (MASW) survey at the 180 Jones Street site in San Francisco, California (Figure 1). The survey objective was to assess geologic layering and the average shear-wave velocity of the upper 100 feet of subsurface (Vs30) to aid Rockridge's geotechnical investigation of the site.

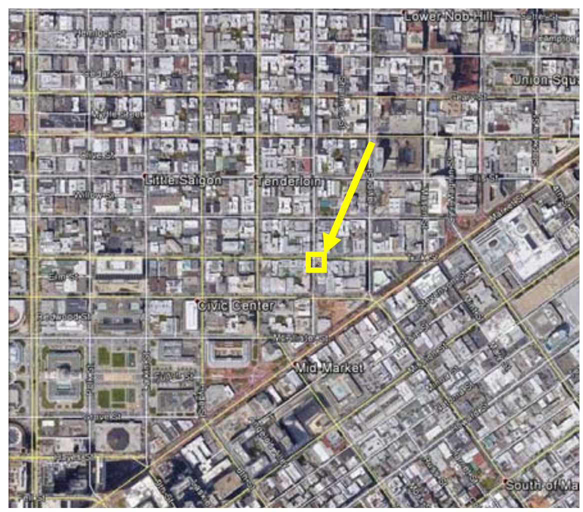


Figure 1 - MASW Seismic Survey Area, 180 Jones St. San Francisco CA

The survey was performed on February 6, 2020 by AGS senior geophysicist Roark W. Smith, with assistance from a Rockridge representative. In general, the survey entailed the collection of seismic surface-wave data along one seismic line positioned diagonally across the existing parking lot site. The surface-wave data were processed using the multi-channel analysis of surface waves (MASW) technique to delineate subsurface velocity layering, and assess the average shear-wave velocity of the upper 30 meters/100 feet (V_s30) to determine the site's IBC site classification.

2.0 SUMMARY OF FINDINGS

- Vs30 at the 180 Jones Street Site is 1,126 feet per second (fps), which equates to IBC Site Class D ("stiff soil"); however, it is worth noting that the Vs30 value for this site places it very near the 1,200 fps boundary between Class D and Class C ("very dense soil and soft rock").

- The velocity layer model shows slightly lower velocity material in the shallower subsurface, with S-wave velocity increasing from about 900 fps to just under 1,200 fps at about 18 feet below ground surface (bgs).

3.0 SITE DESCRIPTION

The 180 Jones Street site is an active, asphalt-paved public parking lot, although parking spaces at the northwest and southeast corners of the lot were cordoned off to facilitate the seismic survey. The lot measures about 60 feet east-west by 75 feet north-south. AGS laid out the seismic line diagonally across the lot in order to maximize line length, hence, investigation depth. Accordingly, the northwest end of the line extended across the sidewalk and the seismic shotpoint was placed about 15 feet into Turk Street in order to accommodate a full 24-channel geophone array (Figure 2). It is worth noting that tent encampments of homeless people were present on the sidewalk along Jones Street at the time of the investigation and the sidewalk became quite busy with foot traffic as morning wore on, so Rockridge's assistance proved invaluable for keeping pedestrians clear of the seismic gear during the investigation.

4.0 SEISMIC SURFACE-WAVE METHOD OVERVIEW

The Seismic Surface-Wave method entails the use of data processing techniques known as Spectral Analysis of Surface Waves (SASW), Multi-channel Analysis of Surface Waves (MASW), and/or Refraction Micro-tremor (REMI). Surface-wave surveys use essentially the same field set-up as a conventional seismic refraction survey (i.e., a geophone array), but a different part of the recorded seismic signal— the Rayleigh (surface) wave— is analyzed instead of the P-wave. Briefly, a surface-wave survey entails measuring the velocity of surface waves using an array of motion detectors (geophones) placed on the ground surface. Because surface-wave velocity closely follows shear-wave velocity (90 to 95% of V_S), surface-wave velocity data can be used to estimate shear wave velocity (V_S).

Surface-Waves are seismic waves that travel along or near the surface of the earth; they can be "active-source" waves generated specifically for the seismic survey (e.g., with explosives or a hammer blow to the ground surface) or "passive-source" waves generated by ambient natural and cultural sources such as ocean waves and vibrations from vehicle traffic and factories. In general, active-source waves are of higher frequency and provide information about the shallower subsurface, while passive-source waves are of lower frequency and can provide deeper subsurface information, albeit with lower resolution. MASW or SASW surveys use active-source surface waves, usually generated with a sledgehammer. Refraction Micro-tremor, or REMI surveys use ambient surface waves.

Surface-Waves travel in assemblages of frequencies, with each frequency having a corresponding wavelength. Because surface-waves are influenced by subsurface material to a depth approximately equal to the surface-wave's wavelength, a velocity vs. depth profile can be generated by measuring the velocity of surface-waves of varying wavelengths. Surface waves with shorter wavelengths (higher frequencies) respond to the material properties (e.g., stiffness) of shallower materials while waves with longer wavelengths (lower frequency) respond to deeper materials.

Specialized computer software is used to identify surface-waves in the recorded data and prepare a 'velocity spectrum' image, which the geophysical analyst interprets to produce a 'dispersion curve' that depicts how velocity varies with frequency (hence, depth). The dispersion curve is then used to prepare a model depicting subsurface velocity layering at a point that is taken to be at the center of the geophone array. Surface-wave surveys produce a 1-dimensional (1-D) profile showing S-wave velocity variations with depth at a point that is taken to be at the center of the geophone array.

5.0 FIELD PROCEDURES

In order to maximize seismic array length and investigation depth at this relatively small site AGS positioned the seismic line diagonally, running at south-east to north-west, angle across the parking lot (Figure 2). The seismic line comprised an array of geophones spaced five (5) feet apart, and a shotpoint located in-line with the geophone array and 10 feet from the nearest geophone. Because the seismic line was placed in a paved area, AGS replaced the spikes on the bases of the geophones, which are normally used to couple the geophones to soil, with metal base plates, which were then anchored to the pavement surface with "spackling paste". Next, AGS generated seismic energy through multiple impacts with a 16-lb sledge hammer against the asphalt pavement on Turk Street. Ten hammer blows were struck at the shotpoint, a technique called "stacking," which is used to increase the signal-to-noise ratio and thus improve data quality.

The seismic waves were detected using 4.5-Hz geophones from GeoSpace Corp and recorded by a DAQLink II seismic system connected to a laptop computer. The data were recorded for 2 seconds using a 0.125 millisecond (ms) sample rate. After the seismic data were acquired AGS mapped the seismic line location by referencing it to the parking lot spaces and nearby streets and curblines, which are readily visible on Google Earth aerial imagery.

6.0 DATA PROCESSING AND ANALYSIS

Seismic data were transferred from the seismograph to a desktop computer where they were processed using the *SeisImager/SW* software package by Geometrics, Inc. In general, surface wave data processing entails first producing a velocity spectrum image, which shows the phase velocity for the various frequencies of surface waves detected. This image is used as the basis for interpreting ("picking") a dispersion curve, which is a graph that depicts how surface-wave velocity varies with frequency (hence, with depth). The dispersion curve is then used to prepare an initial 1-D model of surface-wave velocity versus depth using a one-third wavelength approximation (i.e., a given phase velocity is assigned to a depth that is one-third of the wavelength of the corresponding surface-wave). The initial velocity layer model is then adjusted using an inversion process ("inverted") until the corresponding synthetic dispersion curve achieves a "best-fit" match to the original dispersion curve (the one that was interpreted from the observed data— i.e., the velocity spectrum image). The degree or closeness of the fit between the interpreted and synthetic curves (expressed as a RMS percentage error) provides an indication of how well the model represents actual subsurface conditions.

The inversion process was used to produce a velocity layer model that depicts S-wave velocity variations with depth at a single point, which is taken to be at the center of the geophone array (Figure 3).

7.0 RESULTS

The investigation results are presented on Figures 2 and 3 summarized on Table 1, below. Figure 2 shows the seismic line location. Figure 3 shows the investigation results in the form of a 1-D velocity layer model depicting variations in shear- (S-) wave velocity with depth in the subsurface beneath the seismic line; Figure 3 also shows the associated velocity spectrum image with the interpreted dispersion curve, along with the calculated Vs30 and IBC site class.

Overall, Vs30 at the 180 Jones Street site is 1,126 feet per second (fps), which equates to IBC Site Class D ("stiff soil"), although it is worth noting that the Vs30 value for this site places it very near the 1,200 fps boundary between Class D and Class C ("very dense soil and soft rock"). The velocity layer model also shows slightly lower velocity material in the shallower subsurface, with S-wave velocity increasing from about 900 fps to just under 1,200 fps at about 18 feet below ground surface (bgs).

Table 1- Summary of Results from the 180 Jones Street MASW Seismic Survey

Seismic Line	Location	Vs30 (fps)	Site Class*	Survey Date	Site Conditions	Remarks
SL-1	Diagonally across the parking lot	1,126	D ("stiff soil")	2/6/20	Asphalt-paved parking lot and city sidewalk	Significant velocity inversion; high-velocity layer observed between 10 and 35 ft bgs
*Note: Class D ranges from 600 to 1,200 fps						

8.0 CLOSING

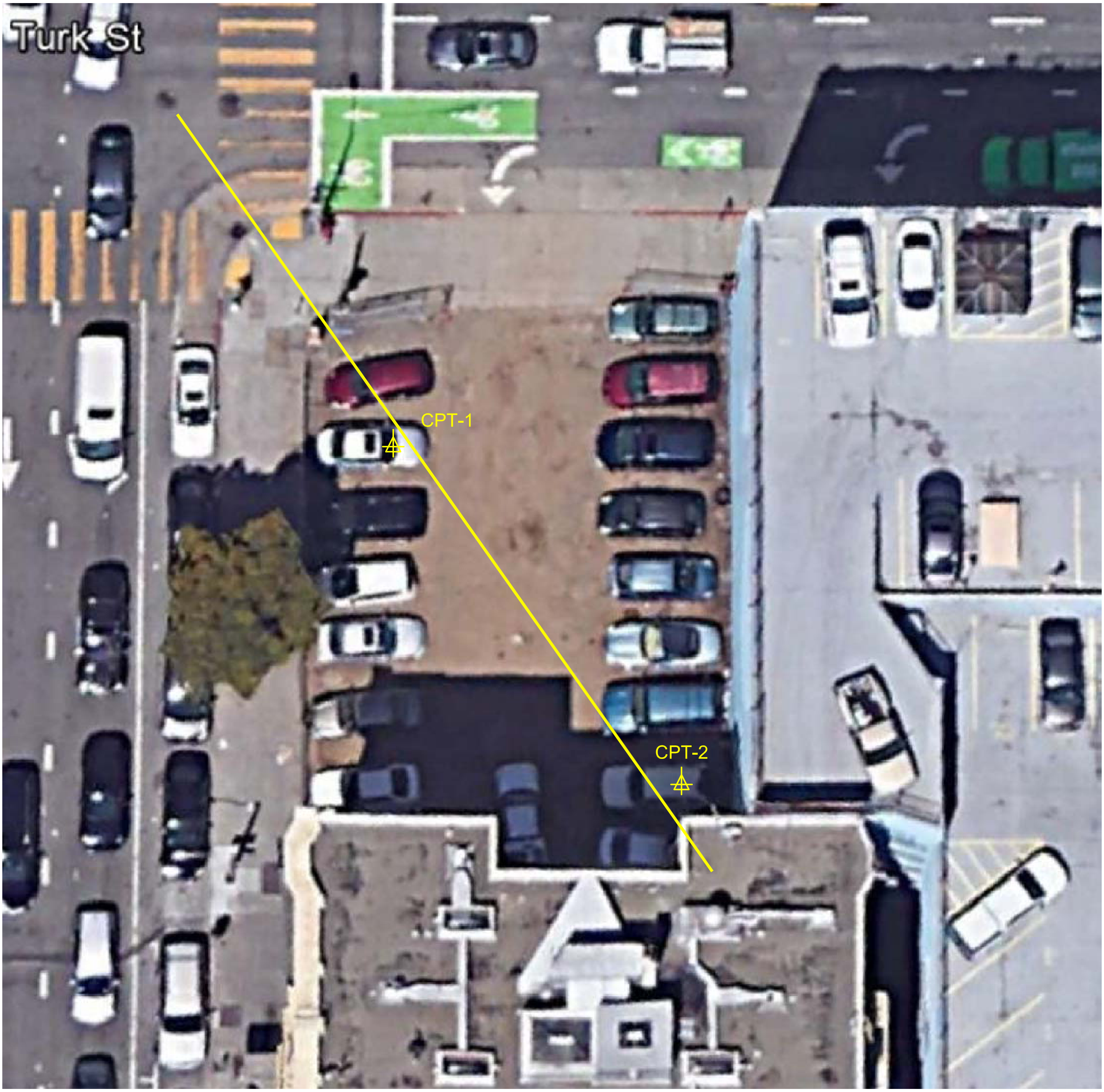
All geophysical data and field notes collected as a part of this investigation will be archived at the AGS office. The data collection and interpretation methods used in this investigation are consistent with standard practices applied to similar geophysical investigations. The correlation of geophysical responses with probable subsurface features is based on the past results of similar surveys although it is possible that some variation could exist at this site. Due to the nature of geophysical data, no guarantees can be made or implied regarding the targets identified or the presence or absence of additional objects or targets.

AGS appreciates working for you. We enjoyed this project and we look forward to working with you again.

Sincerely,

Roark W. Smith, GP 987
Senior Geophysicist
Advanced Geological Services, Inc.

Figures:
- Figure 1 Seismic Survey Area Location (imbedded in Report text, above)
- Figure 2 Seismic Line Location, 180 Jones Street, San Francisco, California
- Figure 3 MASW Seismic Survey Results, 180 Jones Street



EXPLANATION

AGS MASW SEISMIC LINE

CPT-2 ROCKRIDGE CPT LOCATION (PROPOSED)

0 20 ft 40

ADVANCED GEOLOGICAL SERVICES

1605 School Street
Suite 4
Moraga, CA 94556
(925) 808-8965

MASW Seismic Line Location
180 Jones Street

LOCATION: San Francisco, California

CLIENT: Rockridge Geotechnical

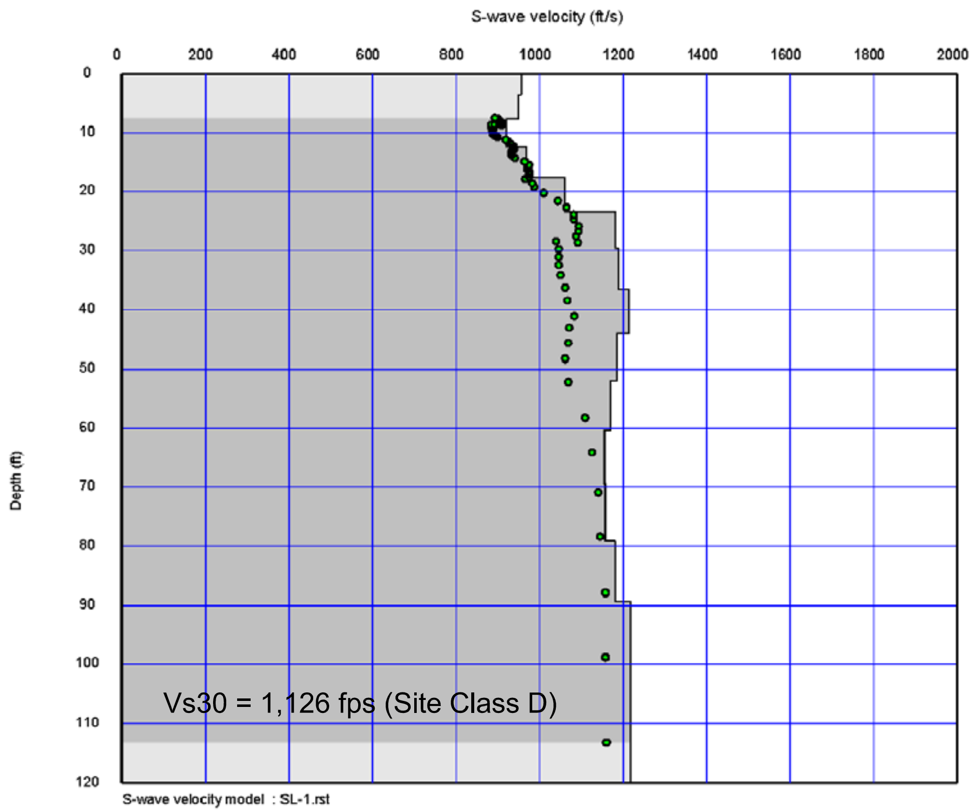
PROJECT #: 20-017-1CA

DATE: Feb 7, 2020

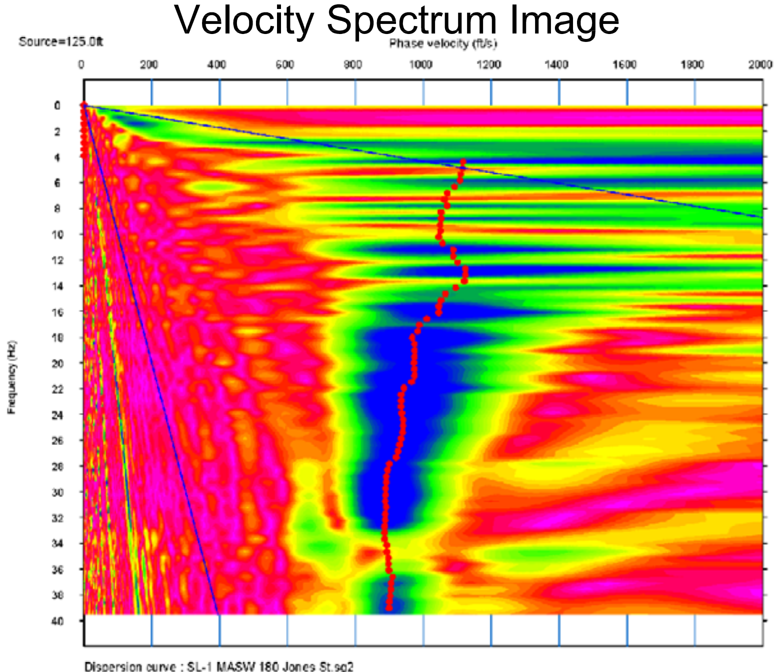
DRAWN BY: R. SMITH

FIGURE 2

Velocity Layer Model



Velocity Spectrum Image



ADVANCED GEOLOGICAL SERVICES

1605 School Street
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MASW Seismic Survey Results
180 Jones Street

LOCATION: San Francisco, California

CLIENT: Rockridge Geotechnical

PROJECT #: 20-017-1CA

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DRAWN BY: R. SMITH

FIGURE 3

ROCKRIDGE GEOTECHNICAL

APPENDIX D

Site-Specific Ground Motion Hazard Analysis

APPENDIX D
SITE-SPECIFIC GROUND MOTION HAZARD ANALYSIS

This appendix presents the results of our site-specific ground motion hazard analysis, which was conducted to estimate potential magnitudes of ground shaking at the site during future earthquakes. Our study consisted of developing horizontal response spectra for two levels of shaking corresponding to the Risk-Targeted Maximum Considered Earthquake (MCE_R) and the Design Earthquake (DE) in accordance with the 2019 San Francisco Building Code (SFBC).

To develop site-specific response spectra for the project, we performed a Probabilistic Seismic Hazard Analysis (PSHA) and Deterministic Seismic Hazard Analysis (DSHA) to develop a site-specific horizontal MCE_R spectrum. The 2019 SFBC defines the site-specific MCE_R spectral response acceleration at any period as the lesser of the risk-targeted probabilistic ground motion having 2 percent probability of exceedance in 50 years or the 84th-percentile deterministic ground motion calculated for the characteristic earthquake on the governing fault(s) in the direction of maximum response (Rot_{D100}). The resulting spectrum is then checked against minimum values stipulated in ASCE 7-16 Chapter 21.2.2 (Supplement 1) and Chapter 21.3 (Supplement 1). Our ground motion hazard analysis is detailed in the following sections.

D1.0 Probabilistic Seismic Hazard Analysis

We performed a PSHA, which involves computing how often a specified level of ground motion will be exceeded at the site, considering potential earthquake scenarios from all relevant seismic sources. The magnitude, location, and recurrence interval of future earthquakes are uncertain and a PSHA systematically accounts for these uncertainties. A PSHA requires information regarding the seismicity, location, and geometry of each seismic source, along with empirical ground motion prediction equations (GMPEs) - also referred to as attenuation relationships. GMPEs describe the probability of a ground motion given the properties of the earthquake source (magnitude and style-of-faulting), distance, fault geometry, and shear wave velocity (V_s) profiles of the underlying soil and rock. In most cases, the ground motion is assumed to follow a lognormal distribution and the GMPEs provide the median response, as well as the standard deviation.

In developing the MCE_R spectrum, we performed a PSHA for a 2 percent probability of exceedance in 50 years, in accordance with ASCE 7-16. The response spectrum was developed using the computer code OpenSHA (Field et al., 2003), which is based on the probabilistic seismic hazard model developed by Cornell (1968) and McGuire (1976).

OpenSHA was run using the seismic sources as included in the Third Uniform California Earthquake Rupture Forecast (UCERF3), prepared by Field et al. (2013). The magnitudes of shaking at the site resulting from each earthquake scenario were then estimated using a suite of GMPEs.

D1.1 Probabilistic Model

The traditional equation for a seismic hazard analysis of a single point source is:

$$\nu_i(Sa>z) = N_i(M_{min}) \int\int f_{mi}(M)\, f_{ri}(r)\, P(Sa>z \mid M,r)\, dr\, dM$$

where:

- $\nu_i(Sa>z)$ is the annual rate of events resulting in a ground motion amplitude that exceeds a value z
- Sa represents a ground motion parameter (commonly spectral acceleration) for a given frequency of vibration. Sa is assumed to be log-normally distributed about the mean with a standard error that is dependent upon the magnitude and GMPE used.
- $N_i(M_{min})$ is the annual rate of earthquakes of magnitude greater than or equal to M_{min}
- $f_m(M)$ and $f_r(r)$ are probability density functions for magnitude and distance, which describe the relative rates of different earthquake scenarios
- $P(Sa>z \mid M,r)$ is the conditional probability of observing a ground motion parameter Sa greater than z for a given earthquake magnitude and distance

For multiple seismic sources, the total annual rate of events with ground motions that exceed z at the site is the sum of the annual rate of events from the individual sources (faults and areal sources):

$$\nu(Sa>z) = \sum \nu_i(Sa>z)$$

The purpose of summing the rates over all sources is to compute how often a certain level of shaking may occur at the site, regardless of what source causes the ground motion. To convert the annual rate of events to a probability, we compute the probability that the ground motion exceeds z at least once during a specified time interval. Assuming the occurrence of earthquakes follows a Poisson process (no "memory" of past earthquakes), the probability of exceeding ground motion level z at least once in T years is given by:

$$P(Sa>z \mid T) = 1 - e^{-\nu(Sa>z)T}$$

D1.2 Source Modeling and Characterization

We used the UCERF3 model (mean solution of Fault Model 3-1 and 3-2), including known faults within 200 km of the site. Table D-1 presents the distance, relative direction from the site, and characteristic moment magnitude of various faults. Due to the potential for variable rupture lengths—including multi-fault ruptures—the UCERF3 model assigns varying probabilities to a large number (>1.2 million) of a potential earthquake rupture "scenarios". We also included USGS 2008 gridded seismic sources, which are used by the USGS to represent background seismicity, special seismic zones and intra-slab events.

ROCKRIDGE GEOTECHNICAL

TABLE D-1
Fault Sources Considered in Hazard Model

Fault Segment	Approximate Distance from Site (km)	Direction (Source to Site)	Characteristic Moment Magnitude
Total North San Andreas (SAO+SAN+SAP+SAS)	13	Southwest	8.04
North San Andreas (Peninsula, SAP)	13	Southwest	7.38
Total Hayward + Rodgers Creek (RC+HN+HS+HE)	17	East	7.58
Hayward (North, HN)	17	East	6.90
San Gregorio (North)	18	West	7.44
Point Reyes (connector)	19	Southwest	-
Pilarcitos	20	Southwest	-
Hayward (South, HS)	21	East	7.00
North San Andreas (North Coast, SAN)	26	West	7.52
Contra Costa (Briones)	27	Northeast	-
Contra Costa (Lafayette)	29	East	-
Contra Costa Shear Zone (connector)	30	East	-
Contra Costa (Reliez Valley)	30	East	-
Franklin	30	Northeast	-
Contra Costa (Dillon Point)	32	Northeast	-
Contra Costa (Larkey)	33	East	-
Total Calaveras (CN+CC+CS+CE)	33	East	7.43
Calaveras (North, CN)	33	East	6.86
Contra Costa (Southampton)	33	Northeast	-
Contra Costa (Ozal - Columbus)	34	Northeast	-
Mount Diablo Thrust North CFM	34	East	6.72
Mount Diablo Thrust	34	East	6.67
Monte Vista - Shannon	35	Southeast	7.14
Mission (connected)	37	East	-
Concord	39	East	6.45
Contra Costa (Vallejo)	40	Northeast	-
Green Valley	41	Northeast	6.30
Point Reyes	41	West	-
Contra Costa (Lake Chabot)	41	Northeast	-
Rodgers Creek - Healdsburg	43	North	7.19
Los Medanos - Roe Island	44	Northeast	-
Mount Diablo Thrust South	45	East	6.50
West Napa	45	Northeast	6.97
Clayton	45	East	6.57
Greenville (North)	48	East	6.86
Bennett Valley	50	North	-
Silver Creek	51	Southeast	-
Great Valley 05 (Pittsburg - Kirby Hills alt1)	53	East	6.60
Butano	53	South	6.93
Great Valley 05 (Pittsburg - Kirby Hills alt2)	56	East	6.66
Las Positas	56	East	6.50

TABLE D-1 (continued)
Fault Sources Considered in Hazard Model

Fault Segment	Approximate Distance from Site (km)	Direction (Source to Site)	Characteristic Moment Magnitude
Calaveras (Central, CC)	64	Southeast	6.85
Zayante-Vergeles (2011 CFM)	67	South	7.48
Great Valley 06 (Midland alt1)	68	East	7.27
Great Valley 06 (Midland alt2)	69	East	7.12
Hayward (Extension, HE)	69	Southeast	6.18
Great Valley 04b (Gordon Valley)	69	Northeast	6.77
Hunting Creek (Berryessa)	72	North	6.69
North San Andreas (Santa Cruz Mts, SAS)	77	Southeast	7.15
Great Valley 07 (Orestimba)	78	East	6.82
Maacama	82	North	7.55
Sargent	82	Southeast	6.71
Greenville (South)	83	East	6.64
Zayante-Vergeles	86	Southeast	7.00
Reliz	90	South	7.44
Great Valley 04a (Trout Creek)	91	Northeast	6.60
Monterey Bay-Tularcitos	99	Southeast	7.26
Hunting Creek (Bartlett Springs connector)	100	North	6.79
Great Valley 03a (Dunnigan Hills)	105	Northeast	6.53
San Gregorio (South)	105	South	7.24
Collayami	106	North	6.70
Great Valley 03 (Mysterious Ridge)	106	Northeast	7.03
Ortigalita (North)	112	East	6.80
Calaveras (South, CS)	113	Southeast	6.38
Great Valley 08 (Quinto)	131	East	6.59
Quien Sabe	131	Southeast	5.61
Ortigalita (South)	135	East	7.01
Wight Way	135	North	-
Bartlett Springs	135	North	7.54
San Andreas (Creeping Section)	137	Southeast	6.70
Calaveras (Extension, CE)	138	Southeast	6.80
Great Valley 02	146	North	-
Great Valley 09 (Laguna Seca)	147	East	6.57
Swain Ravine - Spenceville	166	Northeast	-
Great Valley 01	168	North	-
Great Valley 10 (Panoche)	183	Southeast	6.49
Hosgri	192	Southeast	7.54
North San Andreas (Offshore, SAO)	194	Northwest	7.38

Note: Characteristic magnitude as defined by BSSC2014 catalog.

D1.3 Ground Motion Prediction Equations

We used the average response resulting from a suite of GMPEs developed as part of the Pacific Earthquake Engineering Research Center (PEER) Next Generation Attenuation of Ground Motion West2 (NGA-West2) Project. Specifically, we used the relationships by Abrahamson et al. (ASK; 2014), Boore et al. (BSSA; 2014), Campbell & Bozorgnia (CB; 2014), and Chiou & Youngs (CY; 2014). The NGA-West2 project expanded the original moderate-to-large magnitude NGA-West1 ground motion database with data from recent significant world-wide crustal earthquakes that occurred after 2003. Additionally, small-to-moderate magnitude California earthquakes were added to aid the "small magnitude" scaling of the NGA-West2 GMPEs (PEER, 2013/03).

The NGA-West2 models were developed to predict the median ground-motion response spectra of a ground motion when rotated over all horizontal orientations, referred to as Sa_{RotD50}. Per Chapter 21 of ASCE 7-16, spectral response accelerations in the "direction of maximum horizontal response" (referred to as maximum direction or $Sa_{RotD100}$) are required for development of the MCE_R response spectrum. We used the factors developed by Shahi and Baker (PEER 2013/10) to convert the Sa_{RotD50} values to $Sa_{RotD100}$ values. A summary of these factors is presented below in Table D-2.

TABLE D-2
Ratio of $Sa_{RotD100}$ to Sa_{RotD50} Spectral Demand
(Shahi and Baker, 2013)

Period (seconds)	Ratio
0.01	1.19
0.02	1.19
0.03	1.19
0.05	1.19
0.07	1.19
0.10	1.19
0.15	1.20
0.20	1.21
0.25	1.22
0.30	1.22
0.40	1.23
0.50	1.23
0.75	1.24
1.00	1.24
1.50	1.24
2.00	1.24
3.00	1.25
4.00	1.26
5.00	1.26
7.50	1.28
10.00	1.29

Our field investigation for the project included shear wave velocity measurements using seismic cone penetration tests (CPTs) to a depth of about 50 feet, as well as geophysical measurements of surface waves (MASW) used to estimate shear wave velocities to a depth of 100 feet. Based on the results of these methods, we selected a V_{S30} of 340 m/sec (approximately 1120 ft/sec), which is consistent with a Site Class D soil site.

D1.4 PSHA Results

Figure D-1 presents the results of the PSHA for the 2 percent probability of exceedance in 50 years hazard level (2,475 year return period), as well as the average of the GMPEs for both RotD50 and RotD100 orientations. The RotD100 spectral values were converted to risk-targeted values using the risk coefficients defined in Chapter 22.2.1 Method 1 of ASCE 7-16. The corresponding values for C_{RS} and C_{R1} are 0.921 and 0.904, respectively.

Figures D-2a and D-2b present the deaggregation plots of the PSHA results for the 2 percent probability of exceedance in 50 years hazard level. Based on our examination of the deaggregation results, we conclude that the hazard for PGA, a spectral period of 0.5 seconds, a spectral period of 1.0 second, and a spectral period of 4.0 seconds are all dominated by earthquake scenarios occurring on the San Andreas fault with a modal magnitude of 7.6 to 8.4 and a modal distance range of 10 to 15 kilometers. The Hayward fault is the second largest contributor of hazard, with a modal magnitude of 6.8 to 8.0 and a modal distance range of 15 to 20 kilometers. The deaggregation results for mean magnitude, distance, and epsilon are presented in Table D-3 below.

TABLE D-3
Deaggregation Results
Mean Magnitude, Distance, and Epsilon

Period (seconds)	Mean Magnitude	Mean Distance (km)	Mean Epsilon
0.01 (PGA)	7.3	16.5	1.95
0.50	7.5	17.3	1.90
1.0	7.6	17.2	1.82
2.0	7.7	16.5	1.70
4.0	7.8	15.8	1.40

D2.0 Deterministic Analysis

We performed a deterministic analysis in developing the MCE_R spectrum for the site. In a deterministic analysis, the magnitude and source distance of a single earthquake scenario is input into GMPE(s). The MCE for our deterministic analysis was defined as an event having a moment magnitude 8.04, which is consistent with the characteristic moment magnitude for the rupture of the Northern San Andreas fault (Table D-1), at about 13.4 kilometers from the site.

The GMPEs discussed in Section D1.3 were also used in our deterministic analysis. Per Chapter 21 of ASCE 7-16, the deterministic MCE_R spectral accelerations shall be taken as the 84th-percentile of the ground motion in the direction of maximum horizontal response. Figure D-3 presents the 84th-percentile spectrum for each GMPE (RotD50), as well as the average 84th-percentile spectrum for both RotD50 and RotD100 values.

D3.0 Recommended MCE_R and Design Spectra

The 2019 SFBC defines the site-specific MCE_R as the lesser of the risk-targeted probabilistic spectrum having 2 percent probability of exceedance in 50 years or the 84th-percentile deterministic event on the governing fault (both taken as maximum direction values) for each spectral period. Additionally, the resulting spectra needs to exceed 80% of the General Design Spectrum (mapped values), as defined in Chapter 21.3 of ASCE 7-16.

Figure D-4 and Table D-4 presents a comparison between the risk-targeted PSHA results for a 2 percent probability of exceedance in 50 years and 84th-percentile deterministic results. The 84th percentile DSHA results are less than the PSHA at all spectral periods (up to 10 seconds) and therefore controls the site-specific MCE_R spectrum.

TABLE D-4
Comparison of Site-Specific Spectra for Development of MCE_R Spectrum
Sa (g) for 5 percent damping

Period (seconds)	Risk-Targeted PSHA 2% probability of exceedance in 50 years (RotD100)	84th-Percentile Deterministic (RotD100)	Site-Specific MCE_R Spectrum
0.01	0.976	**0.712**	0.712
0.02	0.982	**0.715**	0.715
0.03	1.033	**0.739**	0.739
0.05	1.220	**0.840**	0.840
0.075	1.536	**1.009**	1.009
0.10	1.791	**1.162**	1.162
0.15	2.109	**1.404**	1.404
0.20	2.260	**1.558**	1.558
0.25	2.375	**1.668**	1.668
0.30	2.429	**1.731**	1.731
0.40	2.345	**1.738**	1.738
0.50	2.190	**1.648**	1.648
0.75	1.704	**1.302**	1.302
1.0	1.317	**1.027**	1.027
1.5	0.870	**0.704**	0.704
2.0	0.633	**0.516**	0.516
3.0	0.415	**0.345**	0.345
4.0	0.300	**0.254**	0.254
5.0	0.235	**0.198**	0.198
7.5	0.135	**0.113**	0.113
10	0.083	**0.069**	0.069

Figure D-5 and Table D-5 present the development of the recommended DE spectrum following the procedures outlined in Chapter 21.4 of ASCE 7-16. DE is defined as two-thirds of the MCE_R; however, the recommended DE may not be less than 80 percent of the General Design Spectrum (Site Class D) at any period. Figure D-5 and Table D-5 present a comparison of two-thirds of the site-specific MCE_R spectrum with 80 percent of the general design spectrum for Site Class D. The DE spectrum is controlled by two-thirds of the site-specific MCE_R at a period up to 0.75 seconds. The DE spectrum is controlled by 80% of the General Design Spectrum between 1.0 and 10 seconds.

TABLE D-5
Comparison of Site-Specific and Code Spectra
for Development of DE Spectrum
Sa (g) for 5 percent damping

Period (seconds)	Site Specific MCE_R	2/3 times MCE_R	80% of Site Class D General Design Spectrum	Recommended DE
0.01	0.712	**0.475**	0.344	0.475
0.02	0.715	**0.476**	0.368	0.476
0.03	0.739	**0.492**	0.392	0.492
0.05	0.840	**0.560**	0.440	0.560
0.075	1.009	**0.673**	0.500	0.673
0.10	1.162	**0.774**	0.560	0.774
0.15	1.404	**0.936**	0.800	0.936
0.20	1.558	**1.038**	0.800	1.038
0.25	1.668	**1.112**	0.800	1.112
0.30	1.731	**1.154**	0.800	1.154
0.40	1.738	**1.159**	0.800	1.159
0.50	1.648	**1.099**	0.800	1.099
0.75	1.302	**0.868**	0.800	0.868
1.0	1.027	0.685	**0.800**	0.800
1.5	0.704	0.469	**0.533**	0.533
2.0	0.516	0.344	**0.400**	0.400
3.0	0.345	0.230	**0.267**	0.267
4.0	0.254	0.170	**0.200**	0.200
5.0	0.198	0.132	**0.160**	0.160
7.5	0.113	0.075	**0.107**	0.107
10	0.069	0.046	**0.080**	0.080

The recommended MCE_R spectrum is then calculated as 1.5 times the recommended DE spectrum. The recommended DE and MCE_R spectra are shown on Figures D-5 and D-6. Figure D-6 also presents a comparison between the recommended DE and MCE_R spectra based on the site-specific analysis and the code based "mapped values" for Site Class D. Digitized values of the recommended MCE_R and DE spectra for a damping ratio of 5 percent are presented below in Table D-6.

TABLE D-6
Recommended Spectral Accelerations (g)
Damping Ratio of 5 percent

Period (seconds)	MCE_R	DE
0.01	0.712	0.475
0.02	0.715	0.476
0.03	0.739	0.492
0.05	0.840	0.560
0.08	1.009	0.673
0.10	1.162	0.774
0.15	1.404	0.936
0.20	1.558	1.038
0.25	1.668	1.112
0.30	1.731	1.154
0.40	1.738	1.159
0.50	1.648	1.099
0.75	1.302	0.868
1.0	1.200	0.800
1.5	0.800	0.533
2.0	0.600	0.400
3.0	0.400	0.267
4.0	0.300	0.200
5.0	0.240	0.160
7.5	0.160	0.107
10	0.120	0.080

Because the site-specific procedure was used to develop the MCE_R and DE response spectra, the seismic design parameters presented in Table D-7, which were determined in accordance with Section 21.4 of ASCE 7-16, should be used.

TABLE D-7
Design Spectral Acceleration Values

Parameter	Spectral Acceleration Value (g)
S_{MS}	1.564
S_{M1}	1.200
S_{DS}	1.043
S_{D1}	0.800

Where S_{DS} is 90% of the maximum Sa between 0.2 and 5.0 seconds and S_{D1} is the maximum value of T*Sa for periods from 1.0 to 5.0 seconds for sites with $V_{s,30}$ less than or equal to 1,200 feet per seconds.

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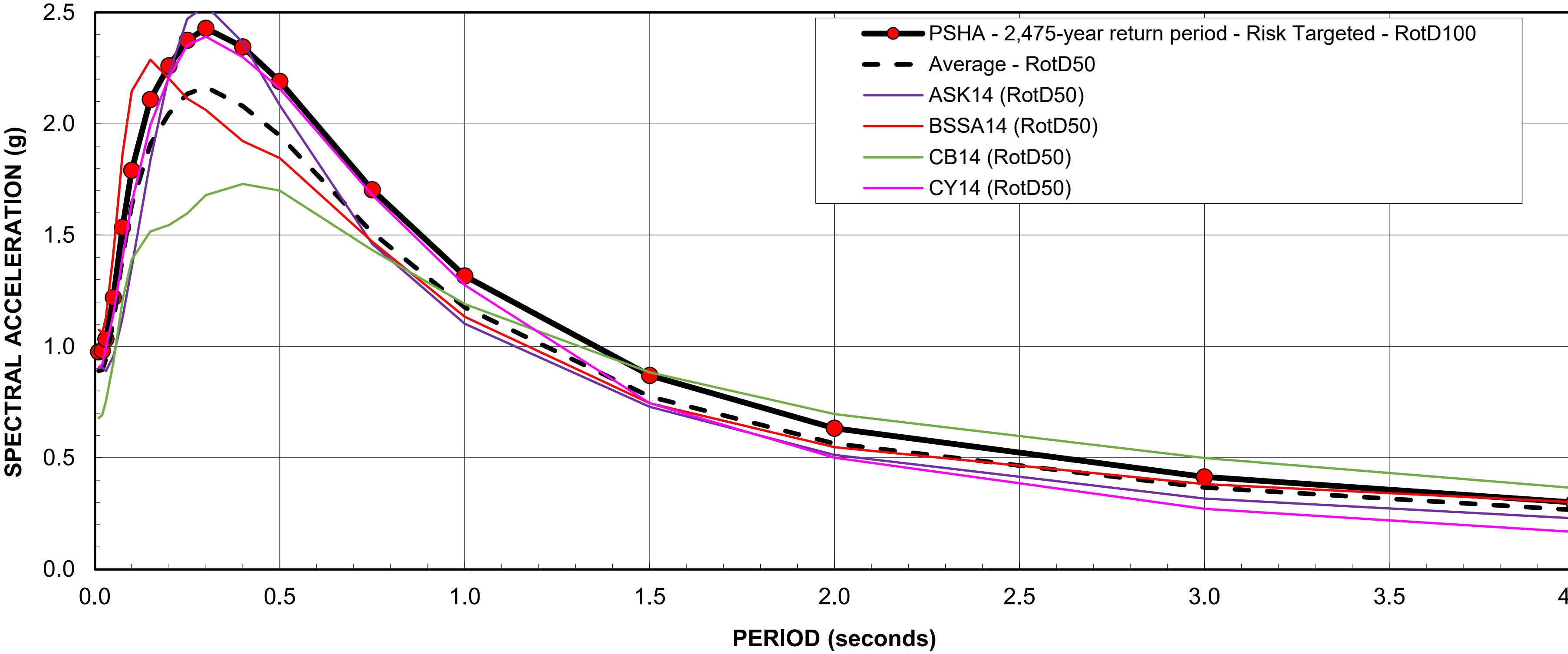


Damping Ratio = 5%

Notes: (1) V_{s30} = 340 m/s
(2) NGA West-2 Attenuation Relationships (2014), equal weighting: ASK14, BSSA14, CB14, CY14
(3) Max direction (RotD100) conversion per Shahi & Baker (2013)
(4) Risk factors per ASCE 7-16 Ch21.2.1 Method 1

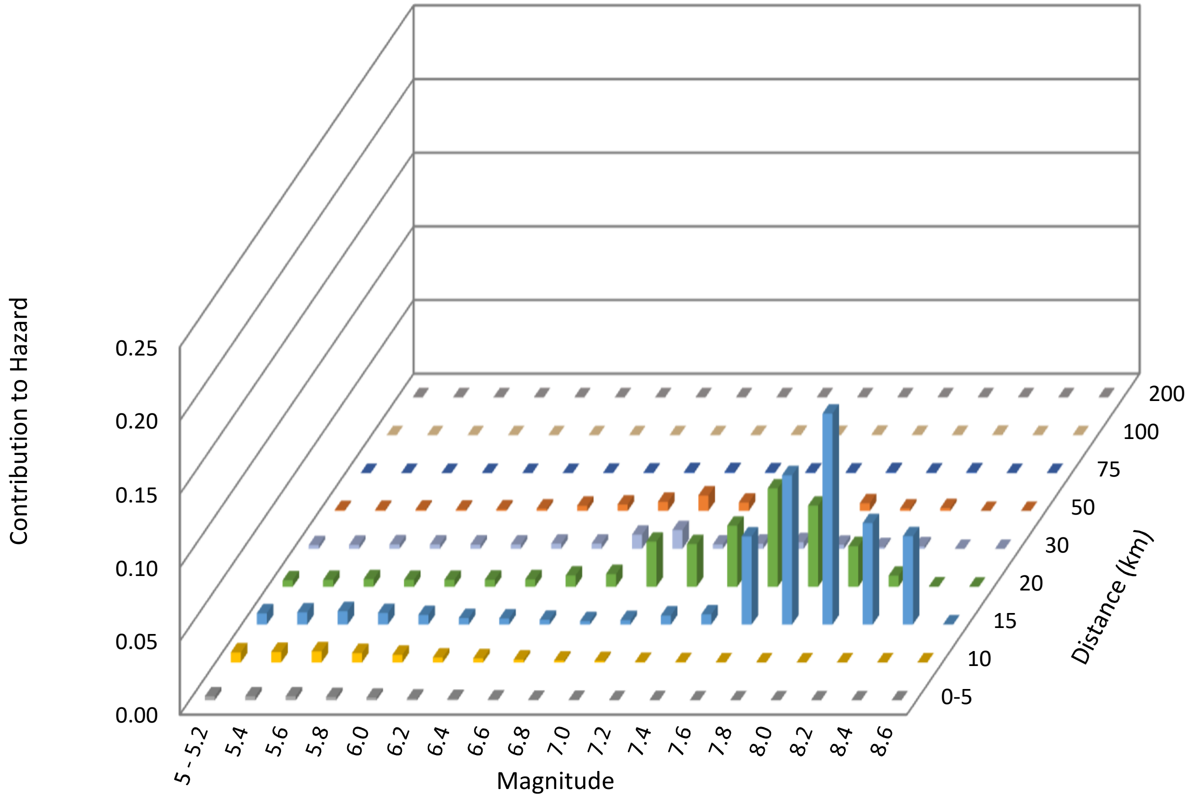
180 JONES STREET

San Francisco, California

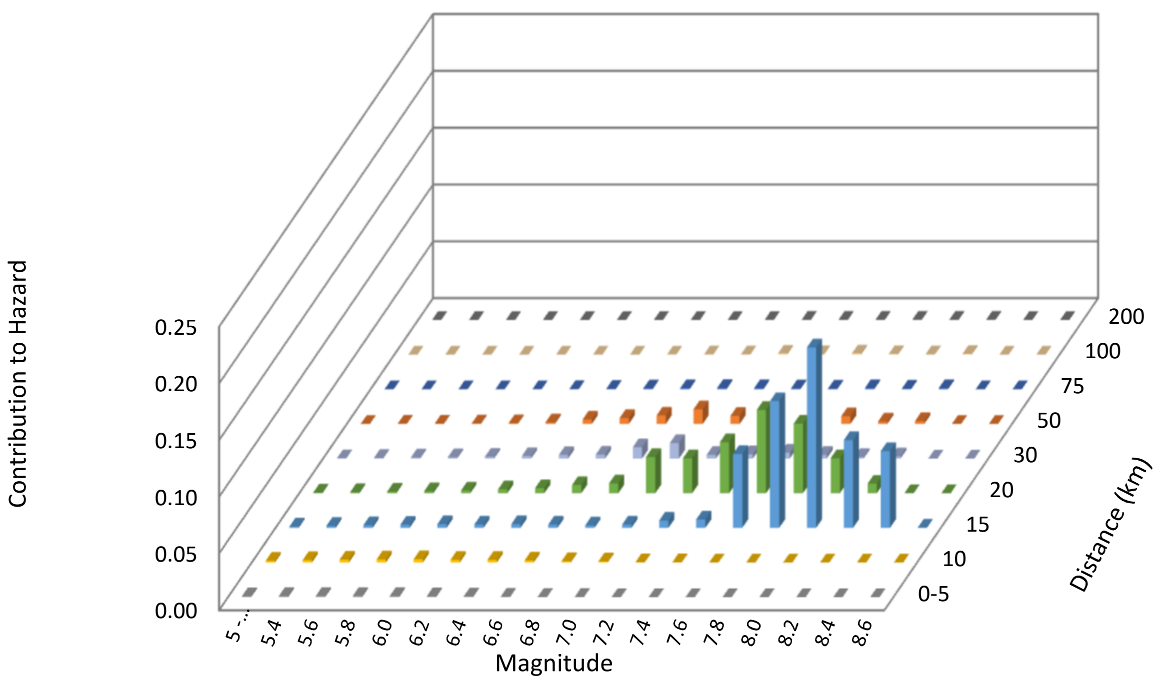
RESULTS OF PSHA, 2 PERCENT PROBABILITY OF EXCEEDANCE IN 50 YEARS

Date:03/12/20	Project No. 20-1805	Figure D-1

ROCKRIDGE GEOTECHNICAL



(a) PGA



(b) S_a, T = 0.5 seconds

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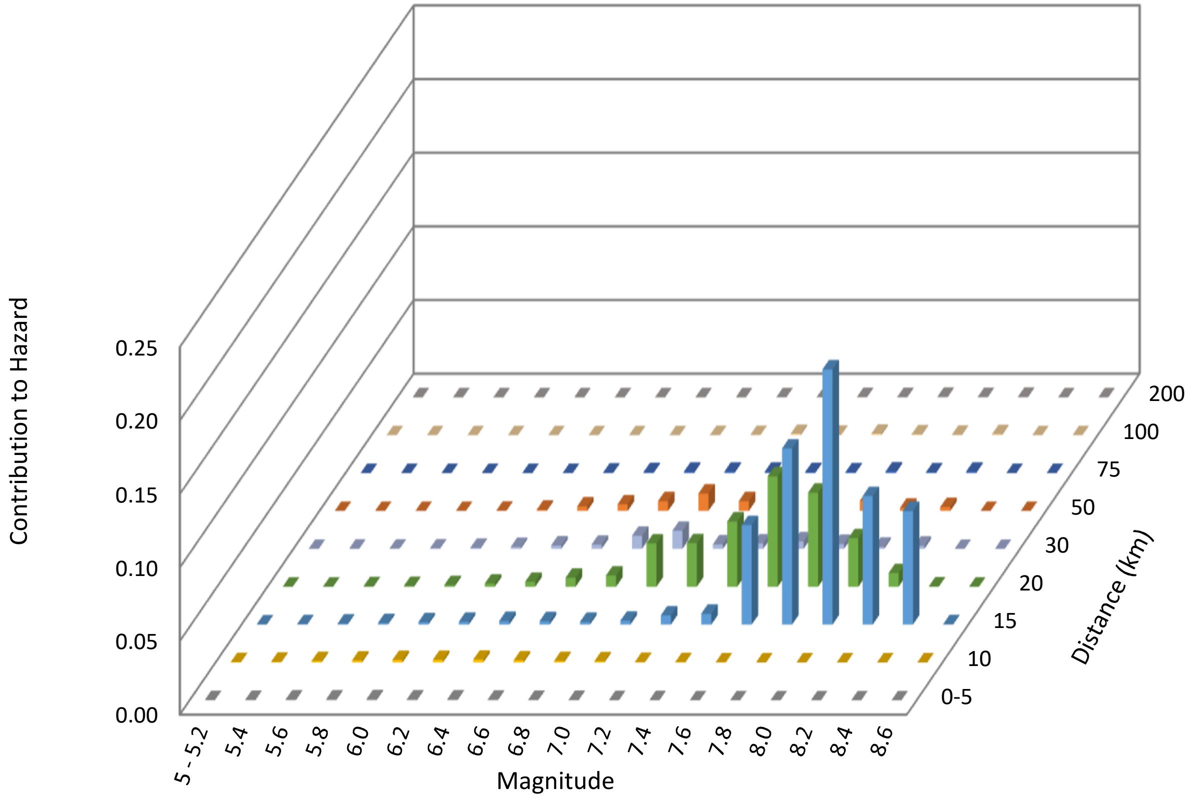
San Francisco, California

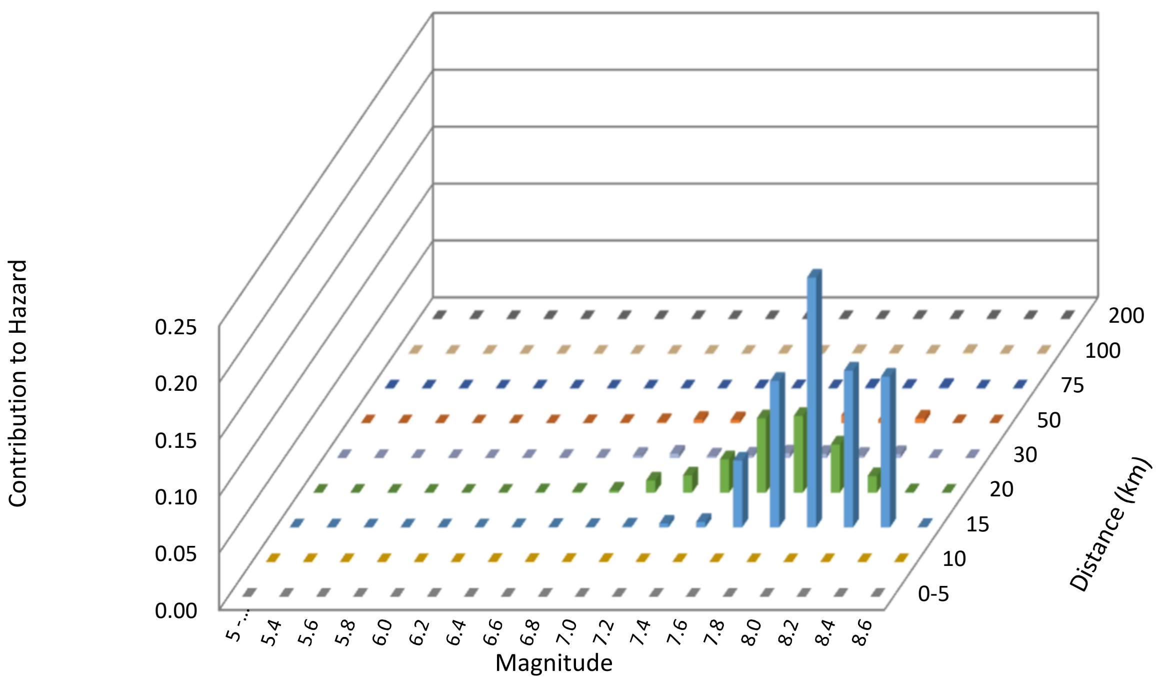
2 PERCENT PROBABILITY OF EXCEEDANCE IN 50 YEARS DEAGGREGATION

Date:03/12/20	Project No. 20-1805	Figure D-2a

ROCKRIDGE GEOTECHNICAL



(c) S_a, T = 1.0 seconds



(b) S_a, T = 4.0 seconds

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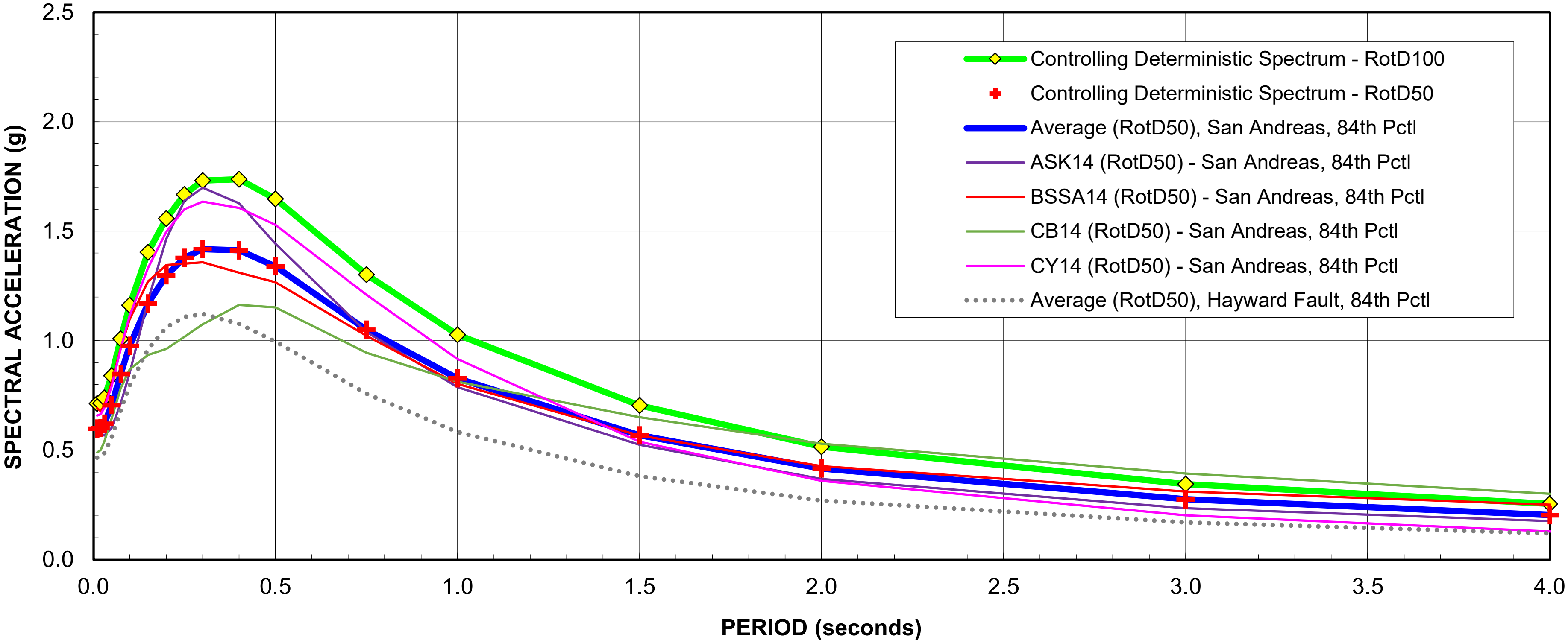
San Francisco, California

2 PERCENT PROBABILITY OF EXCEEDANCE IN 50 YEARS DEAGGREGATION

Date:03/12/20	Project No. 20-1805	Figure D-2b

ROCKRIDGE GEOTECHNICAL



Damping Ratio = 5%

Notes: (1) V_{s30} = 340 m/s

(2) NGA West-2 Attenuation Relationships (2014), equal weighting: ASK14, BSSA14, CB14, CY14

(3) Max direction (RotD100) conversion per Shahi & Baker (2013)

(4) Controlling deterministic event (all periods) is a M_w 8.04 EQ occuring on the San Andreas fault about 13.4 km from the site

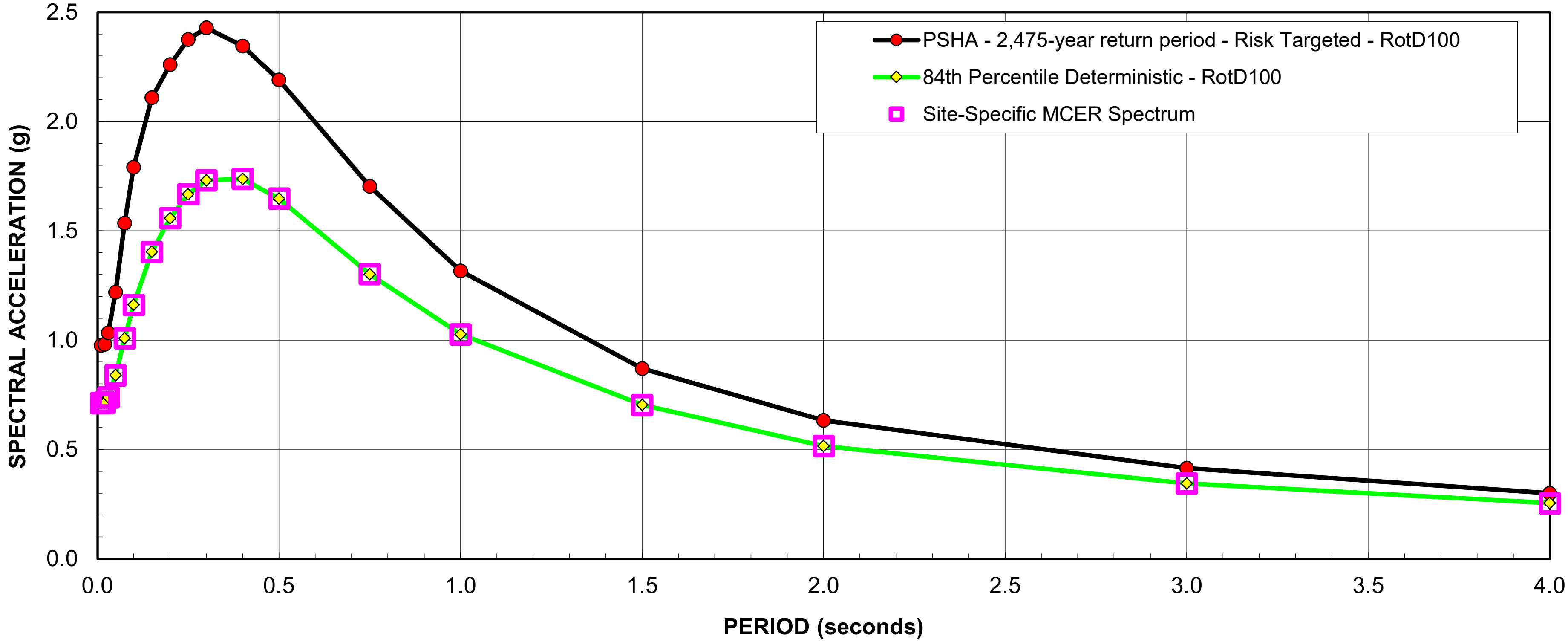
180 JONES STREET

San Francisco, California

RESULTS OF DETERMINISTIC ANALYSIS

Date:03/12/20	Project No. 20-1805	Figure D-3

ROCKRIDGE GEOTECHNICAL



Damping Ratio = 5%

Notes: (1) Site-Specific MCE_R defined at each period as the lesser of:

(a) Risk-Targetted PSHA (2,475 yr exceedence period) in direction of maximum horizontal response (RotD100)

(b) 84th Percentile Deterministic Spectrum in the direction of maximum horizontal response (RotD100)

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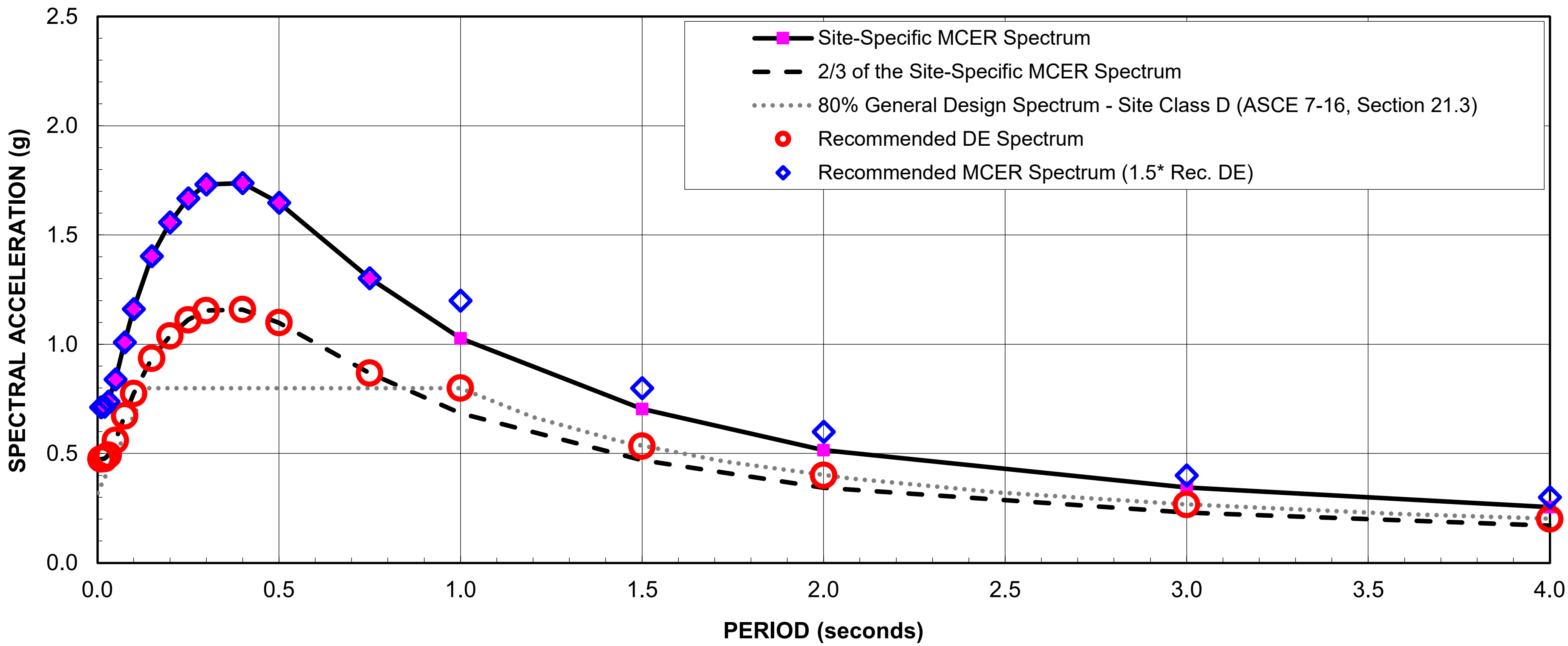
COMPARISON OF DETERMINISTIC AND PROBABILISTIC MCE_R SPECTRA		
Date:03/12/20	Project No. 20-1805	Figure D-4
ROCKRIDGE GEOTECHNICAL		



Damping Ratio = 5%

Notes: (1) Recommended DE spectrum must meet or exceed 80% of the General Design Spectrum (Site Class D, mapped values) at all periods

(2) Recommended MCE_R spectrum taken as 1.5 times recommended DE spectrum

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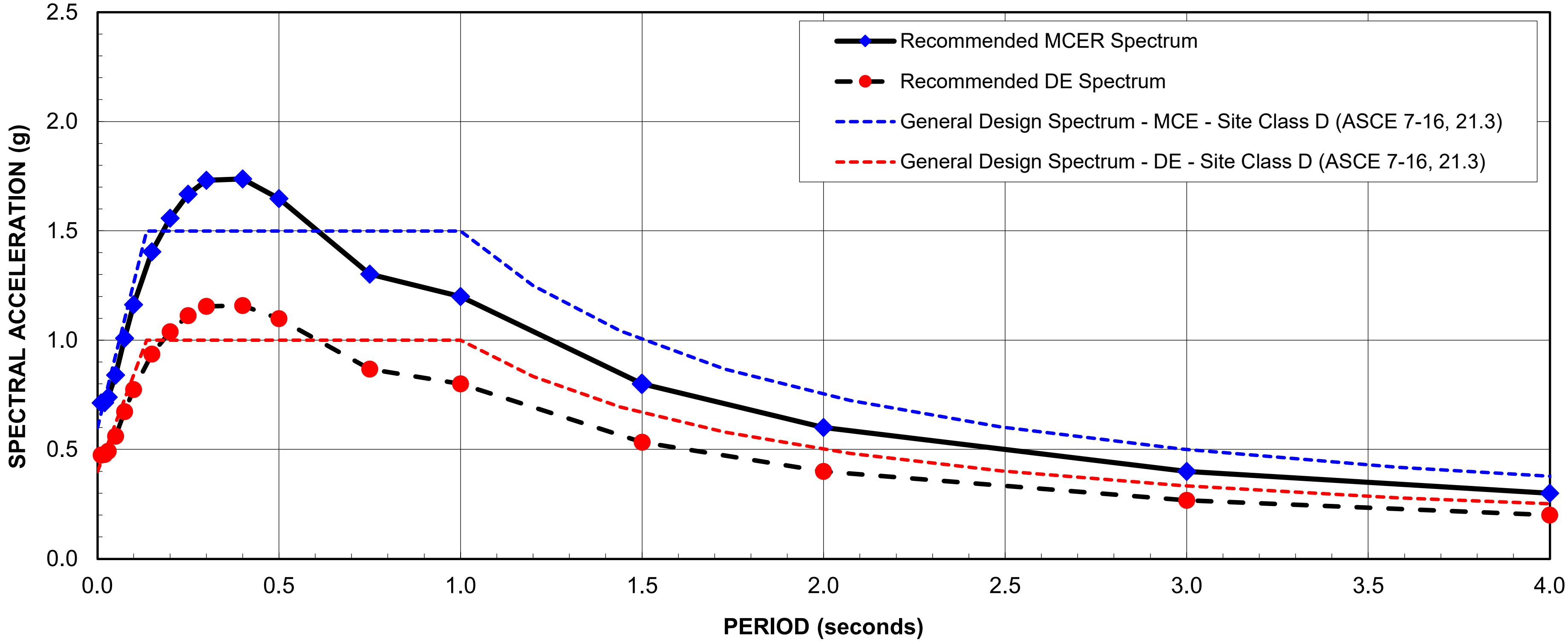
COMPARISON OF SITE-SPECIFIC AND CODE MINIMUM SPECTRA		
Date:03/12/20	Project No. 20-1805	Figure D-5
ROCKRIDGE GEOTECHNICAL		



Damping Ratio = 5%

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RECOMMENDED MCE$_R$ AND DE SPECTRA WITH CODE SPECTRA

Date:03/12/20	Project No. 20-1805	Figure D-6

ROCKRIDGE GEOTECHNICAL