Appendix C – Preliminary Geotechnical Investigation

Prepared for **Kimco Westlake L.P.**

PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
WESTLAKE CENTER – BURLINGTON SITE
99 SOUTHGATE AVENUE
DALY CITY, CALIFORNIA

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September 16, 2020
Project No. 20-1906

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September 16, 2020
Project No. 20-1906

Mr. Sam Knutson
Associate Director of Development
Kimco Westlake L.P.
15 Southgate Ave, Suite 201
Daly City, California 94015

Subject: Preliminary Geotechnical Investigation Report
Proposed Mixed-Use Development
Westlake Center – Burlington Site
99 Southgate Avenue
Daly City, California

Dear Mr. Knutson:

We are pleased to present our preliminary geotechnical report for the proposed mixed-use building to be constructed at the Westlake Center – Burlington site, located at 99 Southgate Avenue in Daly City, California. Our preliminary investigation was performed in accordance with our proposal dated August 14, 2020.

The project site consists of one parcel bordered to the north by Southgate Avenue, to the east by Palmcrest Drive, to the south by a two-story residential building, and to the west by Lake Merced Boulevard. The parcel encompasses 1.93 acres and is currently occupied by an existing single-story building with asphalt-paved parking areas around the western, northern, and eastern perimeter.

We understand the development currently envisioned for the site consists of demolishing the existing building and constructing a mixed-use residential building consisting of two stories of Type IA with five-stories of Type IIIA wood-frame construction over the concrete podium. The proposed project will consist of approximately 220 residential units and 10,000 square feet of street front retail and parking within the podium levels. The total building height will be approximately 76 feet.

Based on the results of our preliminary geotechnical investigation, we conclude there are no major geotechnical issues that would preclude development of the site as proposed. The primary geotechnical issue affecting the proposed development is the potential for as much as several inches of differential settlement due to cyclic densification within the medium dense native soil and undocumented fills of highly varying thickness across the site.

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On the basis of our experience, we judge the anticipated settlements due to cyclic densification exceed the typical tolerance of a conventional shallow foundation system. Therefore, we preliminarily conclude the proposed building could be supported on spread footings supported on a ground improvement system designed to reduce differential settlement to tolerable levels. Viable options for ground improvement include drilled displacement sand-cement (DDSC) columns, rapid impact compaction (RIC), or rammed aggregate piers (RAPs).

Our preliminary geotechnical investigation consisted of a limited subsurface exploration program. Prior to final design, a final geotechnical investigation should be performed to fill in data gaps of subsurface conditions and provide final conclusions and recommendations regarding the geotechnical aspects of the project.

We appreciate the opportunity to provide our services to you on this project. If you have any questions, please call.

Sincerely yours,
ROCKRIDGE GEOTECHNICAL, INC.

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Darcie Maffioli, P.E., G.E.
Senior Project Engineer

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

Enclosure

ROCKRIDGE GEOTECHNICAL

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PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
WESTLAKE CENTER – BURLINGTON SITE
99 SOUTHGATE AVENUE
Daly City, California

1.0 INTRODUCTION

This report presents the results of the preliminary geotechnical investigation performed by Rockridge Geotechnical, Inc. for the proposed mixed-use building to be constructed at the Westlake Center – Burlington site, located at 99 Southgate Avenue in Daly City, California. The site is located at the southwestern corner of the intersection of Southgate Avenue and Palmcrest Drive, as shown on the Site Location Map, Figure 1.

The site is bordered to the north by Southgate Avenue, to the east by Palmcrest Drive, to the south by a two-story residential building, and to the west by Lake Merced Boulevard. The parcel encompasses 1.93 acres and is currently occupied by an existing single-story building with asphalt-paved parking areas around the western, northern, and eastern perimeter.

Current plans are to demolishing the existing building and construct a mixed-use residential building consisting of two stories of Type IA with five-stories of Type IIIA wood-frame construction over the concrete podium. The proposed building is planned to be constructed at-grade. The proposed project will consist of approximately 220 residential units and 10,000 square feet of street-front retail and parking within the podium levels. The total building height will be approximately 76 feet.

2.0 SCOPE OF SERVICES

Our investigation was performed in accordance with our proposal dated August 14, 2020. Our scope of work consisted of exploring subsurface conditions at the site by performing four cone penetration tests (CPTs), one of which included seismic shear wave measurements, reviewing existing subsurface data available in the site vicinity, and performing engineering analyses to develop conclusions and recommendations regarding:

- the most appropriate foundation type(s) for the proposed building
- preliminary design criteria for the recommended foundation type(s), including vertical and lateral capacities
- estimates of foundation settlement
- site seismicity and seismic hazards, including the potential for liquefaction and liquefaction-induced ground failure
- design high groundwater level
- 2019 California Building Code (CBC) site class and mapped design spectral response acceleration parameters
- construction considerations.

3.0 FIELD INVESTIGATION

Our field investigation consisted of performing four CPTs, designated as CPT-1 through CPT-4, at the approximate locations shown on the Site Plan, Figure 2. Prior to performing our field investigation, we obtained a drilling permit from the City of Daly City Department of Water and Wastewater Resources and contacted Underground Service Alert (USA) to notify them of our work, as required by law. We also retained Precision Locating LLC, a private utility locator, to check that the CPT locations were clear of buried utilities.

The CPTs were advanced by ConeTec, Inc. of San Leandro, California on September 3, 2020. CPT-1 and CPT-3 were advanced to a target depth of approximately 50 feet below the existing ground surface (bgs). CPT-2 was advanced to refusal in very dense soil at a depth of 24 feet bgs. CPT-4 was planned to be advanced to a depth of 100 feet bgs, but practical refusal was encountered at approximately 75 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, frictional resistance, and pore water pressure, were recorded by a computer while the test was conducted. Accumulated data were processed by a

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computer to provide engineering information such as the soil behavior types, approximate strength characteristics, and the liquefaction potential of the soil encountered. The CPT logs showing tip resistance, friction ratio, and pore pressure, as well as correlated soil behavior type, are presented in Appendix A on Figures A-1 and A-4a. Shear wave velocities of the soil were measured while advancing CPT-4. Plots of the measured shear wave velocity at each interval are presented on Figure A-4b.

Upon completion, the CPTs were backfilled with cement grout and the pavement was patched with quick-set concrete.

4.0 SUBSURFACE CONDITIONS

The Regional Geologic Map (Figure 3) for the site vicinity indicates the majority of the site is underlain by artificial fill (af) and the east end of the site is underlain by Colma formation (Qc). Colma formation is Pleistocene-aged and generally consists of dense to very dense sands with varying silt and clay content and sandy clay (Bonilla, 1998). The site is located in a former tributary area of Lake Merced which is about 0.8 miles to the north of the site, as shown on Figure 3.

The results of our field investigation indicate that very dense sand, silty sand, and sandy silt of the Colma formation are near the surface in the northeastern corner of the site (CPT-2). In CPT-3 and CPT-4, these very dense sands of the Colma formation were encountered at depths of about 18 and 32 feet bgs, respectively. CPT-1 did not appear to encounter Colma formation, which may indicate these deposits previously eroded away within the former drainage associated with the Lake Merced tributary shown on the Regional Geologic Map, Figure 3.

The material above the very dense sands of the Colma formation is generally medium dense to very dense granular soil with varying fines content. It is difficult to definitively characterize the thickness of undocumented fill using CPT data; however, based on the regional geologic setting and documentation of the former drainage beneath much of the site (Figure 3), we conclude that much of the soil encountered above the Colma formation is likely undocumented fill of highly varying thickness across the site. In CPT-1, a layer of clay and silty clay was encountered

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between depths of 25 and 28 feet bgs. Material descriptions are based on the methodology by Robertson (2010) to describe Soil Behavior Type (SBT). Logs of the soil behavior type for each CPT are presented on Figures A-1 through A-4a in Appendix A.

The groundwater level was indirectly measured in CPT-4 by performing a pore-pressure dissipation test. During our investigation, groundwater was measured at a depth of 41 feet bgs. To further evaluate the depth to groundwater at the site, we reviewed groundwater data on the State of California Water Resources Control Board GeoTracker website (https://geotracker.waterboards.ca.gov/). There is one monitoring well (MW-8) on the subject site as well as many other wells across Lake Merced Boulevard to the west of the site, at 151 Southgate Avenue. Readings taken at monitoring well MW-8 between October 2008 and January 2020 showed the groundwater fluctuated about 1.75 feet over the 12-year monitoring period with the shallowest groundwater measurement at 51.75 feet bgs taken on February 18, 2018.

5.0 SEISMIC CONSIDERATIONS

The San Francisco Bay Area is considered to be one of the most seismically active regions in the world. The results of our evaluation regarding seismic considerations for the project site are presented in the following sections.

5.1 Regional Seismicity and Faulting

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

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The major active faults in the area are the San Andreas, San Gregorio, and Hayward faults. These and other faults in the region are shown on Figure 4. For these and other active faults within a 50-kilometer radius of the site, the distance from the site and estimated characteristic moment magnitude[1] [Peterson 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)	2.7	Southwest	8.04
North San Andreas (Peninsula, SAP)	2.7	Southwest	7.38
San Gregorio (North)	8.8	West	7.44
North San Andreas (North Coast, SAN)	27	Northwest	7.52
Total Hayward + Rodgers Creek (RC+HN+HS+HE)	28	East	7.58
Hayward (North, HN)	28	East	6.90
Hayward (South, HS)	28	East	7.00
Monte Vista - Shannon	29	Southeast	7.14
Total Calaveras (CN+CC+CS+CE)	42	East	7.43
Calaveras (North, CN)	42	East	6.86
Mount Diablo Thrust	44	East	6.67
Mount Diablo Thrust North CFM	44	East	6.72
Butano	44	South	6.93
Concord	49	East	6.45

Since 1800, four major earthquakes have been recorded on the North San Andreas Fault. In 1836, an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale occurred east of Monterey Bay on the San Andreas Fault (Toppozada and Borchardt 1998). The estimated Moment magnitude, M_w, for this earthquake is about 6.25. In 1838, an

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

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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 91 kilometers south of the site. On August 24, 2014, an earthquake with an estimated maximum intensity of VIII (severe) on the MM scale occurred on the West Napa fault. This earthquake was the largest earthquake event in the San Francisco Bay Area since the Loma Prieta Earthquake. The M_w of the 2014 South Napa Earthquake was 6.0.

In 1868, an earthquake with an estimated maximum intensity of X on the MM scale occurred on the southern segment (between San Leandro and Fremont) of the Hayward Fault. The estimated M_w for the earthquake is 7.0. In 1861, an earthquake of unknown magnitude (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 part of the UCERF3 project, researchers estimated that the probability of at least one $M_W \geq$ 6.7 earthquake occurring the greater San Francisco Bay Area during a 30-year period (starting in 2014) is 72 percent. The highest probabilities are assigned to the 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 Geologic Hazards

Because the project site in in a seismically active region, we evaluated the potential for earthquake-induced geologic hazards, including ground shaking, ground surface rupture,

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liquefaction[2], lateral spreading[3] and cyclic densification.[4] We used the results of our field investigation to evaluate the potential of these phenomena occurring at the project site.

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 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 not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act, and no known active or potentially active faults exist on the site. We, therefore, conclude the risk of fault offset at the site from a known active fault is very low. In a seismically active area, the remote possibility exists for future faulting in areas where no faults previously existed; however, we conclude the risk of surface faulting and consequent secondary ground failure from previously unknown faults is also very low.

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,

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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.

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loss of bearing strength, ground fissures, and sand boils are evidence of excess pore pressure generation and liquefaction.

The CPTs encountered dense to very dense sand and silty sand below the groundwater, which are not susceptible to liquefaction due to their cohesion and/or relatively density. The historic groundwater measurements in the monitoring well on site indicate the groundwater table at the site is deeper than 50 feet bgs. Therefore, we conclude the potential for liquefaction and associated liquefaction-induced hazards to occur at the site is very low.

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 medium dense to very dense sand above the groundwater table, some of which is susceptible to cyclic densification. We evaluated the cyclic densification potential of soil encountered at the site using data collected in CPTs using the software CLiq v3.0 (GeoLogismiki, 2019) and the methodology by Robertson and Shao (2010).

In accordance with the 2019 CBC, we used a peak ground acceleration of 1.04 times gravity (g) in our cyclic densification 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 characteristic moment magnitude for the Total North San Andreas Fault, as presented in Table 1.

At the northeastern corner of the site, where the top of Colma formation is very shallow, we estimate that cyclic densification will be nil. However, in the remaining areas, we estimate the site could experience several inches of settlement due to cyclic densification during a major earthquake if the density of the medium dense sand is not improved. With the potential for

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

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cyclic densification of the northeastern corner of the site to be negligible and the remaining areas to have several inches of cyclic densification settlement, it will be critical to manage the potential for differential settlement across the proposed building. During our final investigation, we should further evaluate the magnitude of cyclic densification with borings and laboratory testing.

6.0 PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our engineering analyses using the data from the CPTs, we conclude there are no major geotechnical or geological issues that would preclude development of the site as proposed. The primary geotechnical issue affecting the proposed development is managing the potential for several inches of differential settlement due to cyclic densification across the site. These issues, construction considerations, and seismic design are discussed in more detail in the following sections.

6.1 Foundations and Settlement

The proposed building is planned to be constructed at-grade. Based on our preliminary assessment of the subsurface soils, there is the potential for as much as several inches of differential settlement due to cyclic densification across the site following a major earthquake. More accurate estimates of the total and differential settlement will be provided once the structural loading has been determined, the foundation type has been determined, and after we complete the final subsurface exploration and laboratory testing programs for the project site.

On the basis of our experience, we judge the anticipated settlements due to cyclic densification exceed the typical tolerance of a shallow foundation system. Therefore, we conclude the proposed building could be supported on spread footings supported on a ground improvement system designed to reduce differential settlement to tolerable levels. 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 layers subject to cyclic densification, thus reducing settlements and providing increased bearing capacity beneath footings. Several types of ground improvement may be utilized to mitigate differential settlements of the proposed building. Viable options for ground improvement are discussed in the following sections.

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6.1.1 Drilled Displacement Sand-Cement Columns

We consider drilled displacement sand-cement (DDSC) columns to be an appropriate ground improvement method for this project. 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. This system results in low vibration during installation and generates fewer drilling spoils for off-haul, compared to conventional drilled piers. DDSC columns are installed under design-build contracts by specialty contractors. The required size, spacing, length, and strength of columns should be determined by the design-build contractor, based on the desired level of improvement (i.e. the tolerable settlement and desired allowable bearing pressure), as determined by the structural engineer. We recommend a preliminary design, including calculations of static and seismic settlements, be prepared by the ground improvement contractor and submitted for review by us, as well as the structural engineer.

The capacities and lengths of the ground improvement elements should be determined by the design-build contractor that installs the system; however, for preliminary planning purposes, it may be assumed that DDSCs as much as 35 feet long in the western portion of the site and may not be needed in the northeastern corner of the site, where the foundations will bear directly on the very dense sands of the Colma formation.

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6.1.2 Rapid Impact Compaction

We judge it is feasible to densify the upper medium dense soil layers in-situ by using Rapid Impact Compaction (RIC). RIC is generally suitable for densifying granular soils within 10 to 15 feet of the ground surface, although, marginal improvement may be achieved to a depth of 20 feet. Because of existing improvements surrounding the site, vibrations should be monitored, and setbacks from improvements should be established.

Prior to performing the production RIC at the site, a pilot-testing program should be performed to confirm the effectiveness of the proposed RIC spacing and the number of passes. We should choose the locations of the test sections and review the ground improvement contractor's submittal for the proposed test sections. CPTs should be performed before and after the testing program to quantitatively evaluate the improvement. In addition, we recommend time be included in the construction schedule for a second round of pilot-testing for a different spacing or pattern in the event the first round does not show adequate improvement.

Where RIC is performed, it will cause uneven settlement of the ground surface. Consequently, fill will be required to raise site grades in these locations.

6.1.3 Rammed Aggregate Piers

RAPs are typically constructed by drilling a 30-inch-diameter shaft and replacing the excavated soil with compacted aggregate. The aggregate generally consists of clean, open-graded crushed rock below the water table and Class 2 aggregate base above the water table. The aggregate is compacted in approximately 12-inch-thick lifts using a modified hydraulic hammer mounted on an excavator. RAPs develop vertical support through a combination of frictional resistance along the shaft of the pier and improvement of the surrounding soil matrix, allowing use of significantly larger bearing capacities than feasible in unimproved soil. RAPs can also be designed to resist transient uplift loads by installing steel rods in the pier; the rods are attached to a flat steel plate at the base of the of the footings. Lateral loads are resisted through a combination of passive pressure on the face of the footings and friction along the base of the footings. The frictional resistance is larger for a RAP-supported footing than for a footing supported on unimproved ground because of the presence of the compacted aggregate.

The required size, spacing, and lengths of the RAP elements should be determined by the design-build contractor, based on the desired level of improvement (i.e. the tolerable settlement and desired allowable bearing pressure), as determined by the structural engineer.

6.2 Construction Considerations

The soil to be excavated consists primarily of sand, which can be excavated with conventional earth-moving equipment such as loaders and backhoes. The majority of the site is currently covered with asphalt and a commercial building. The foundation types, sizes, and depths are not currently known. Site clearing should include the removal of all existing pavements, slabs,

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former foundations, and underground utilities. If concrete debris or former foundation elements are encountered during grading, removal will require equipment capable of breaking concrete, such as a hoe-ram.

Excavations that will be entered by workers should be sloped or shored in accordance with CAL-OSHA standards (29 CFR Part 1926). The contractor should be responsible for the construction and safety of temporary slopes. We judge temporary slopes above the groundwater table with a maximum inclination of 1.5:1 (horizontal to vertical) should be stable, provided the slope is not surcharged by adjacent structures, construction equipment, or stockpiled soil.

6.3 Seismic Design

The latitude and longitude of the site are 37.6974° and -122.4822°, respectively. For design in accordance with 2019 CBC, we preliminarily recommend the following:

- Site Class D
- $S_S = 2.216$, $S_1 = 0.923g$

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The 2019 CBC is based on the guidelines contained within ASCE 7-16 which stipulate that where S_1 is greater than 0.2 times gravity (g) for Site Class D, a ground motion hazard analysis is needed unless the seismic response coefficient (C_s) value will be calculated as outlined in Section 11.4.8, Exception 2. Assuming the C_s value will be calculated as outlined in Section 11.4.8, Exception 2, we recommend the following seismic design parameters:

- $F_a = 1.0$, $F_v = 1.7$
- $S_{MS} = 2.216g$, $S_{M1} = 1.569g$
- $S_{DS} = 1.477$, $S_{D1} = 1.046g$
- Seismic Design Category E for Risk Factors I, II, and III

Depending on the structural design methodology and fundamental period of the proposed building, it may be advantageous to perform a site-specific ground motion hazard analysis (the project structural engineer should confirm). We can perform a ground motion hazard analysis upon request.

7.0 ADDITIONAL GEOTECHNICAL SERVICES

The preliminary conclusions and recommendations presented within are based on a preliminary field investigation and not intended for final design. Prior to final design, we should be retained to provide a final geotechnical report based on a supplemental field investigation and the final proposed development. Additional borings and CPTs will be required to further evaluate the subsurface conditions beneath the site. Once our final report has been completed, the design team has selected a foundation system, and prior to construction, we should review the project plans and specifications to check their conformance with the intent of our final recommendations. During construction, we should observe site preparation, foundation installation, ground improvement installation and load testing, and the placement and compaction of fill. These observations will allow us to compare the actual with the anticipated soil conditions and to check if the contractor's work conforms with the geotechnical aspects of the plans and specifications.

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FIGURES

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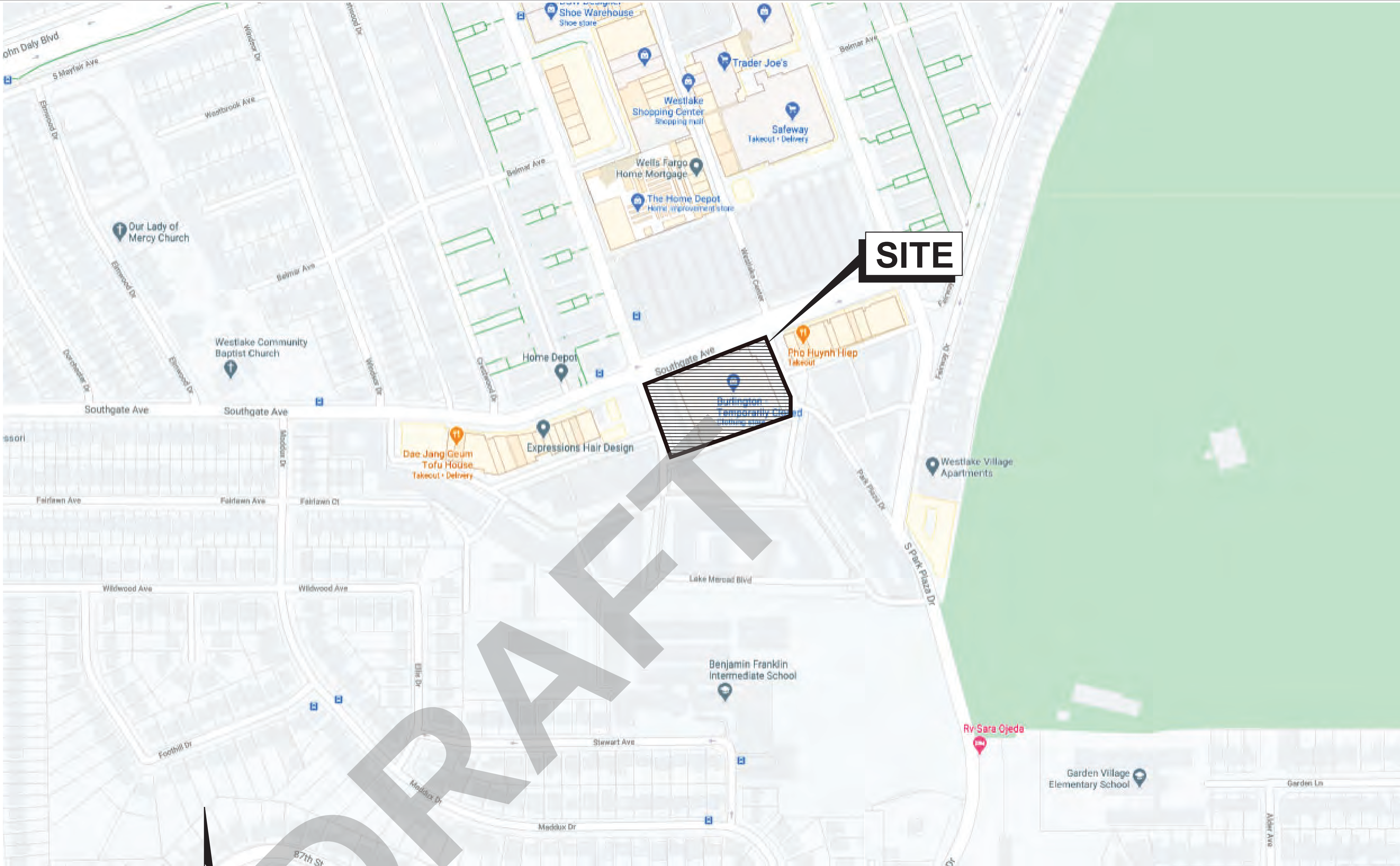
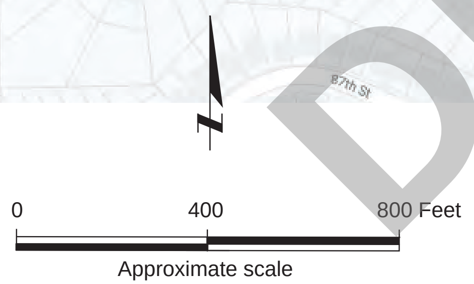


Base map: Google Maps, 2020.



WESTLAKE CENTER
99 SOUTHGATE AVENUE
Daly City, California

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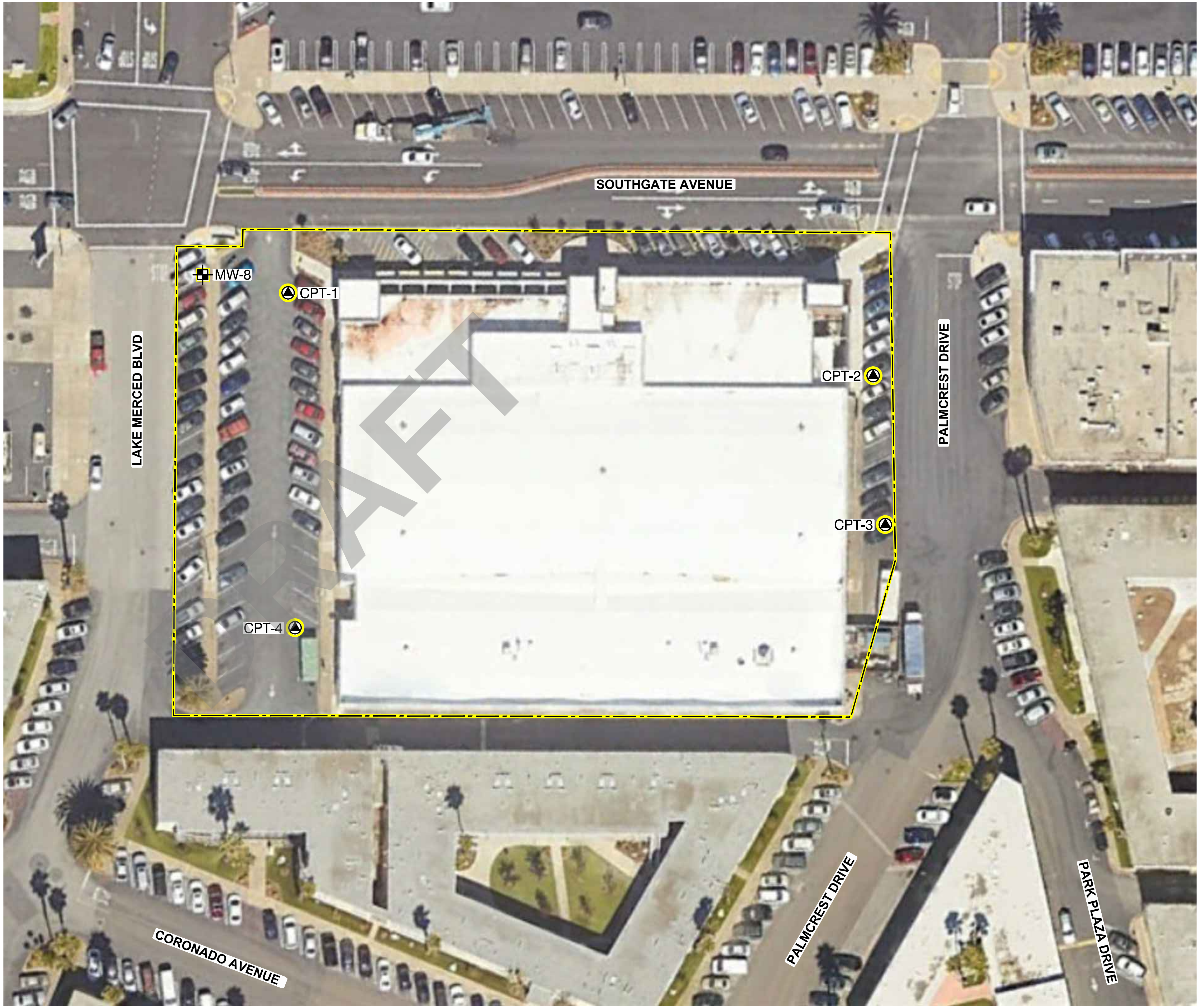
SITE LOCATION MAP

Date 09/04/20	Project No. 20-1906	Figure 1



EXPLANATION

CPT-1 Approximate location of cone penetration test by Rockridge Geotechnical Inc., September 3, 2020

MW-8 Approximate location of existing monitoring well, Stratus Environmental, Inc., August 29, 2008

Project limits

0 50 Feet

Approximate scale

Base map: Google Earth, 2020.

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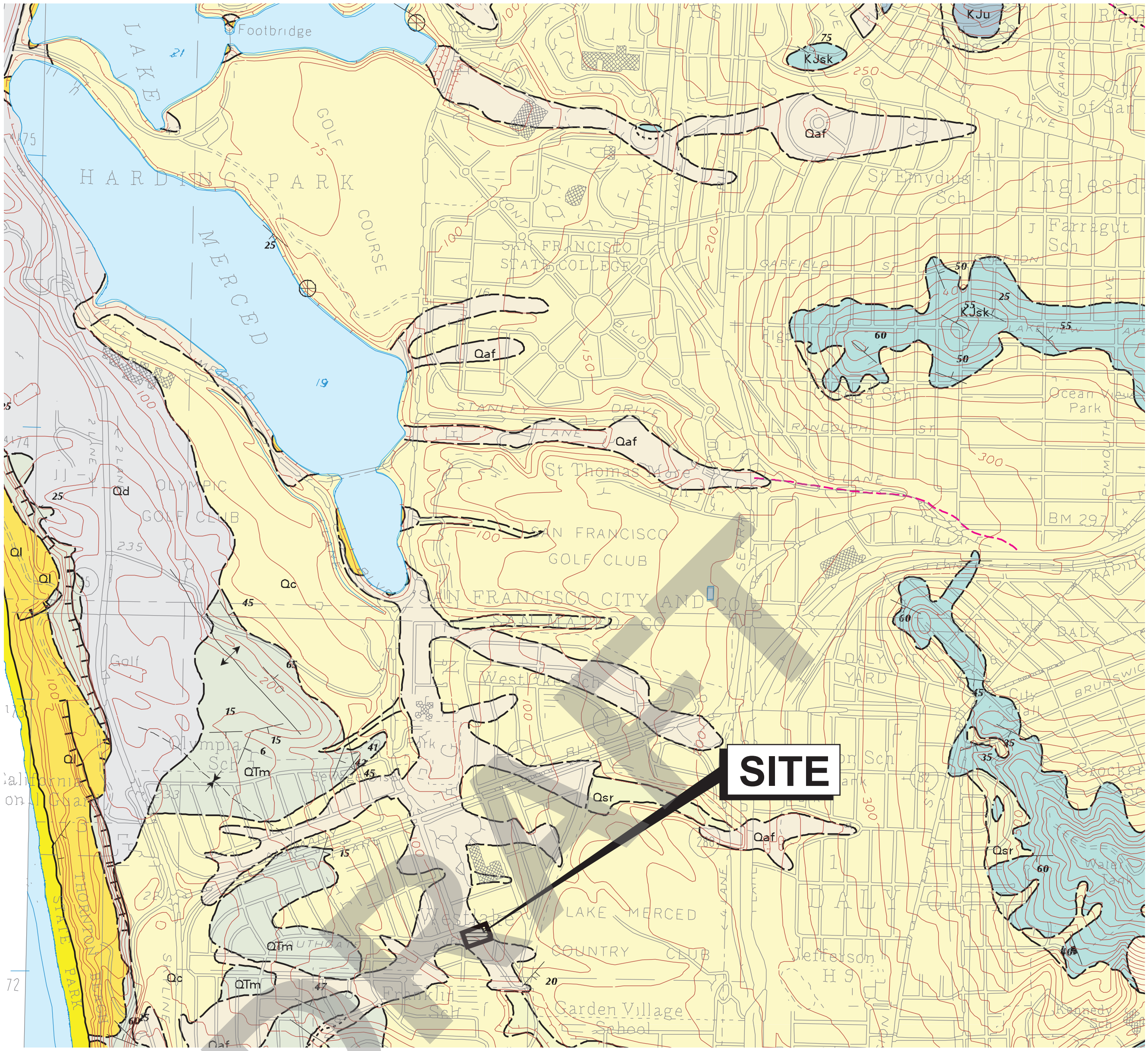
SITE PLAN

Date 09/09/20 | Project No. 20-1906 | Figure 2

ROCKRIDGE GEOTECHNICAL



DRAFT

	water
Qaf	Artificial fill
Qc	Colma Formation
Ql	Landslide deposits
Qd	Dune sand
Qsr	Slope debris and ravine fill
QTm	Merced Formation
KJs	Sandstone and shale

Base map: Preliminary Geologic Map of the SF South 7.5' Quadrangle and part of the Hunters Points 7.5' Quadrangle, San Francisco Bay Area, California, 1998

0 2,000 4,000 Feet

Approximate scale

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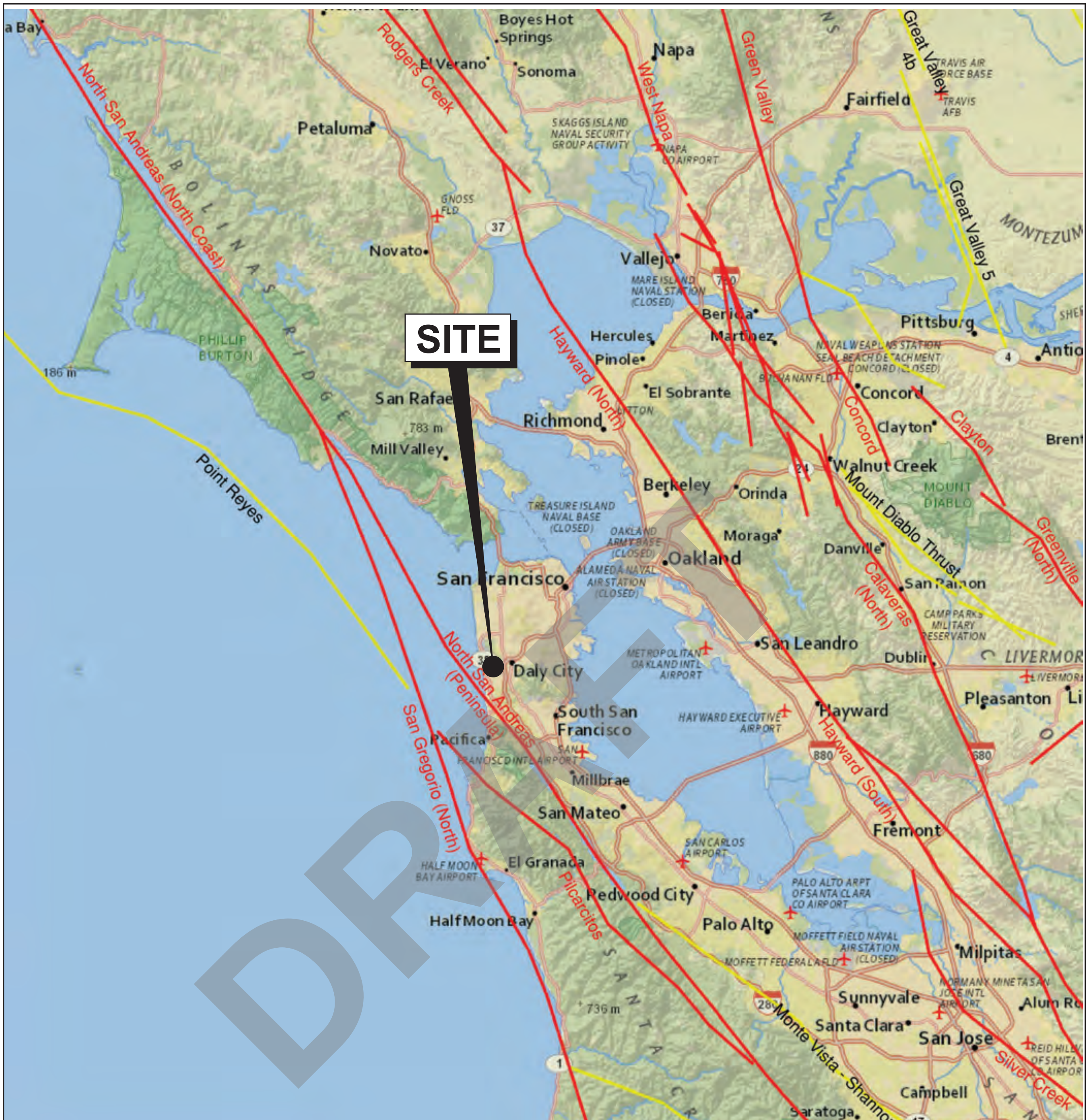
REGIONAL GEOLOGIC MAP

Date 09/14/20 | Project No. 20-1906 | Figure 3



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

EXPLANATION

Strike slip

Thrust (Reverse)

0 5 10 Miles

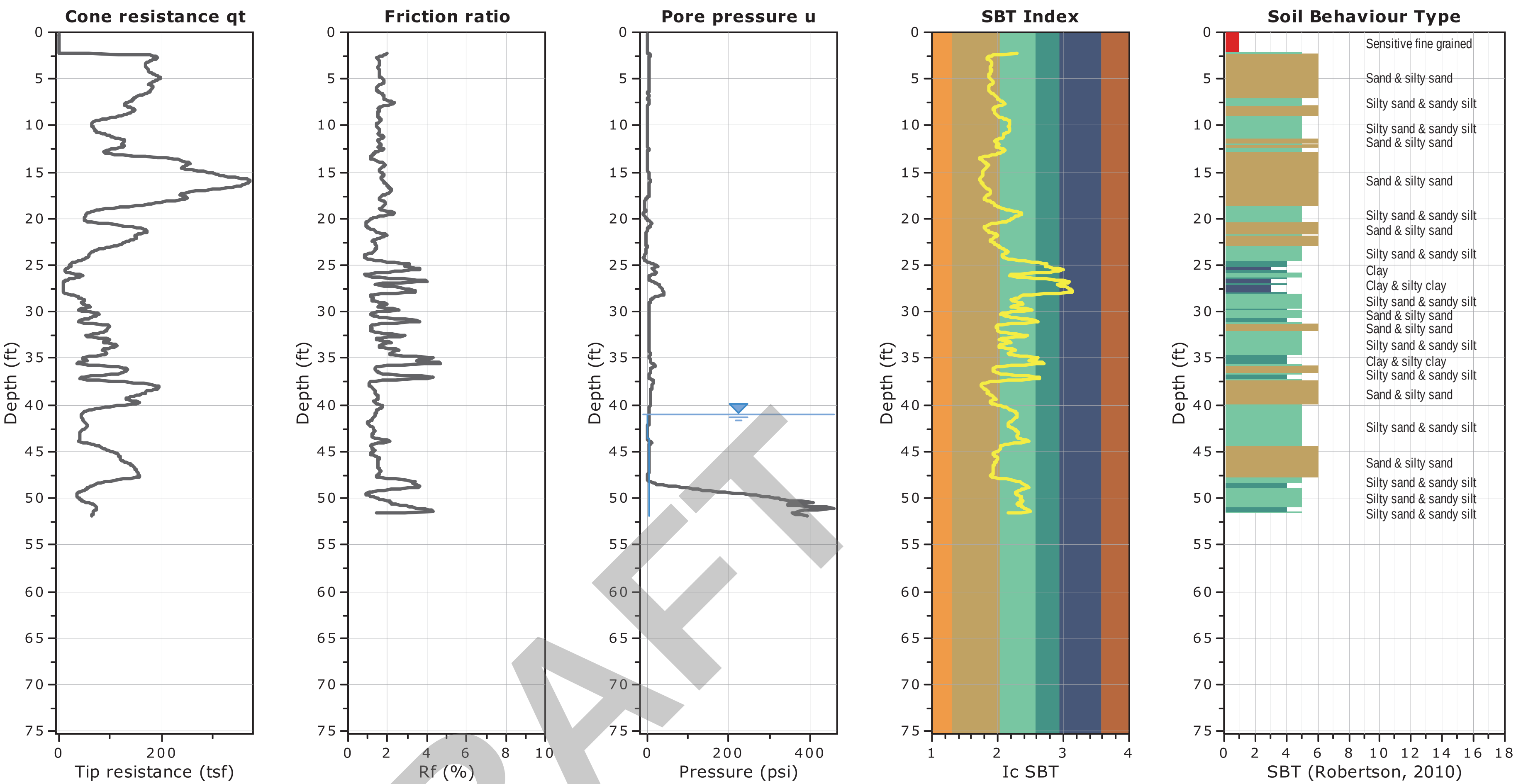
Approximate scale

WESTLAKE CENTER 99 SOUTHGATE AVENUE Daly City, California	REGIONAL FAULT MAP		
ROCKRIDGE GEOTECHNICAL	Date 09/03/20	Project No. 20-1906	Figure 4

ROCKRIDGE GEOTECHNICAL

APPENDIX A
Cone Penetration Test Results

DRAFT



Total depth: 51.84 ft, Date: September 3, 2020
Assumed Groundwater Depth: 41 feet
Cone Operator: Cone Tec

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

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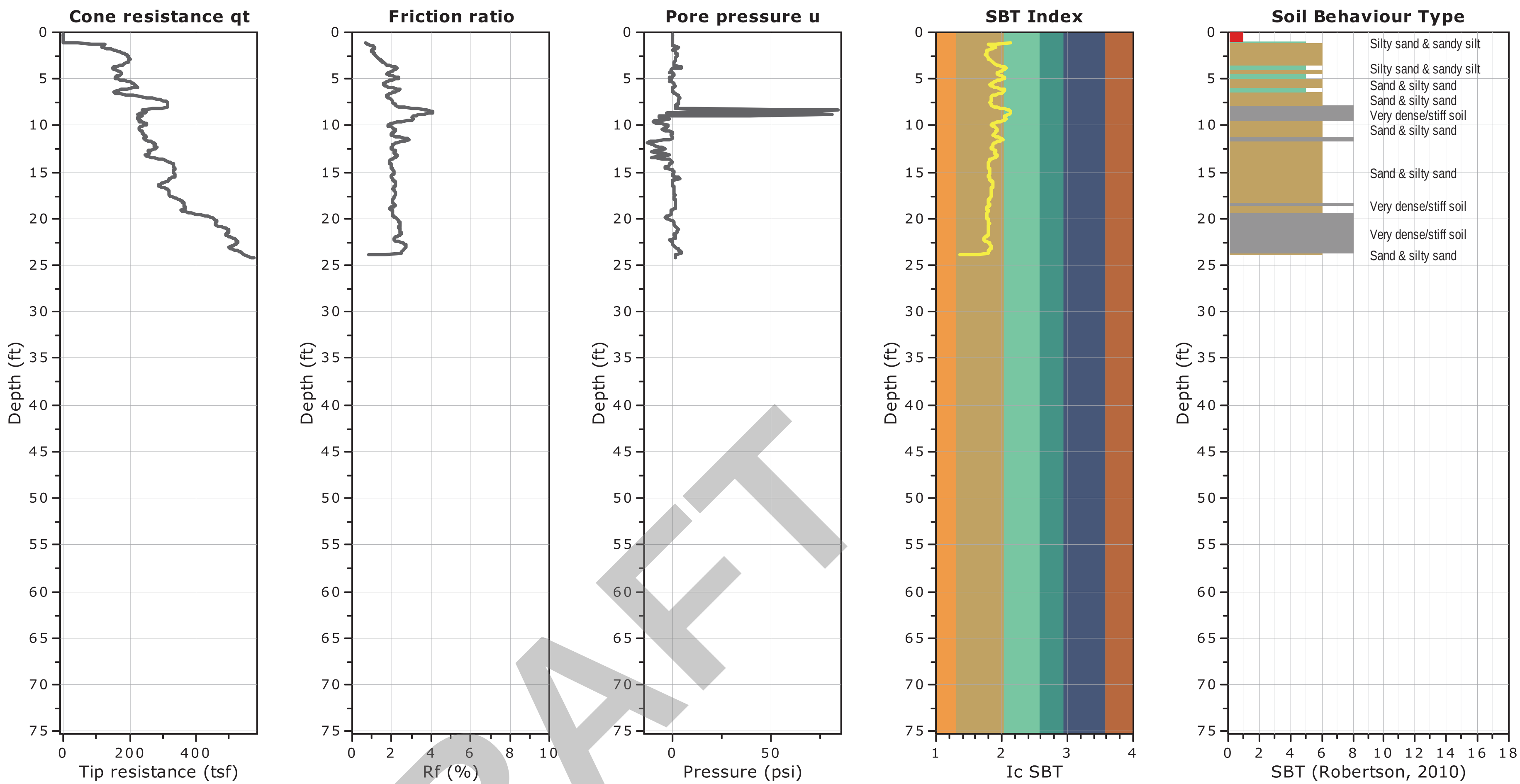
CONE PENETRATION TEST RESULTS
CPT-1

Date 09/04/20 | Project No. 20-1906 | Figure A-1



DRAFT

Total depth: 24.20 ft, Date: September 3, 2020
Groundwater not encountered
Cone Operator: Cone Tec

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

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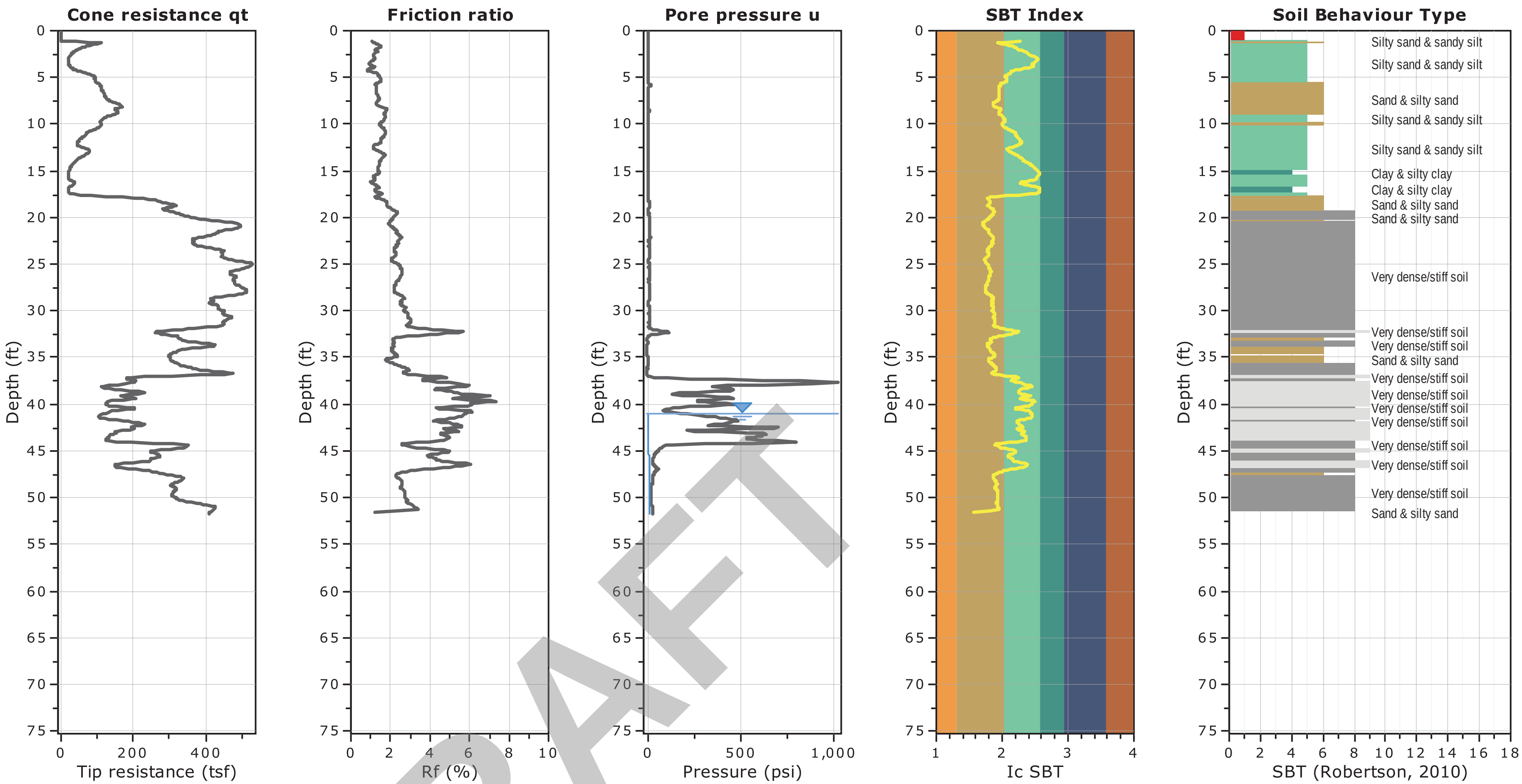
ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-2

Date 09/04/20 | Project No. 20-1906 | Figure A-2



Total depth: 51.76 ft, Date: September 3, 2020
Assumed Groundwater Depth: 41 feet
Cone Operator: Cone Tec

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

DRAFT

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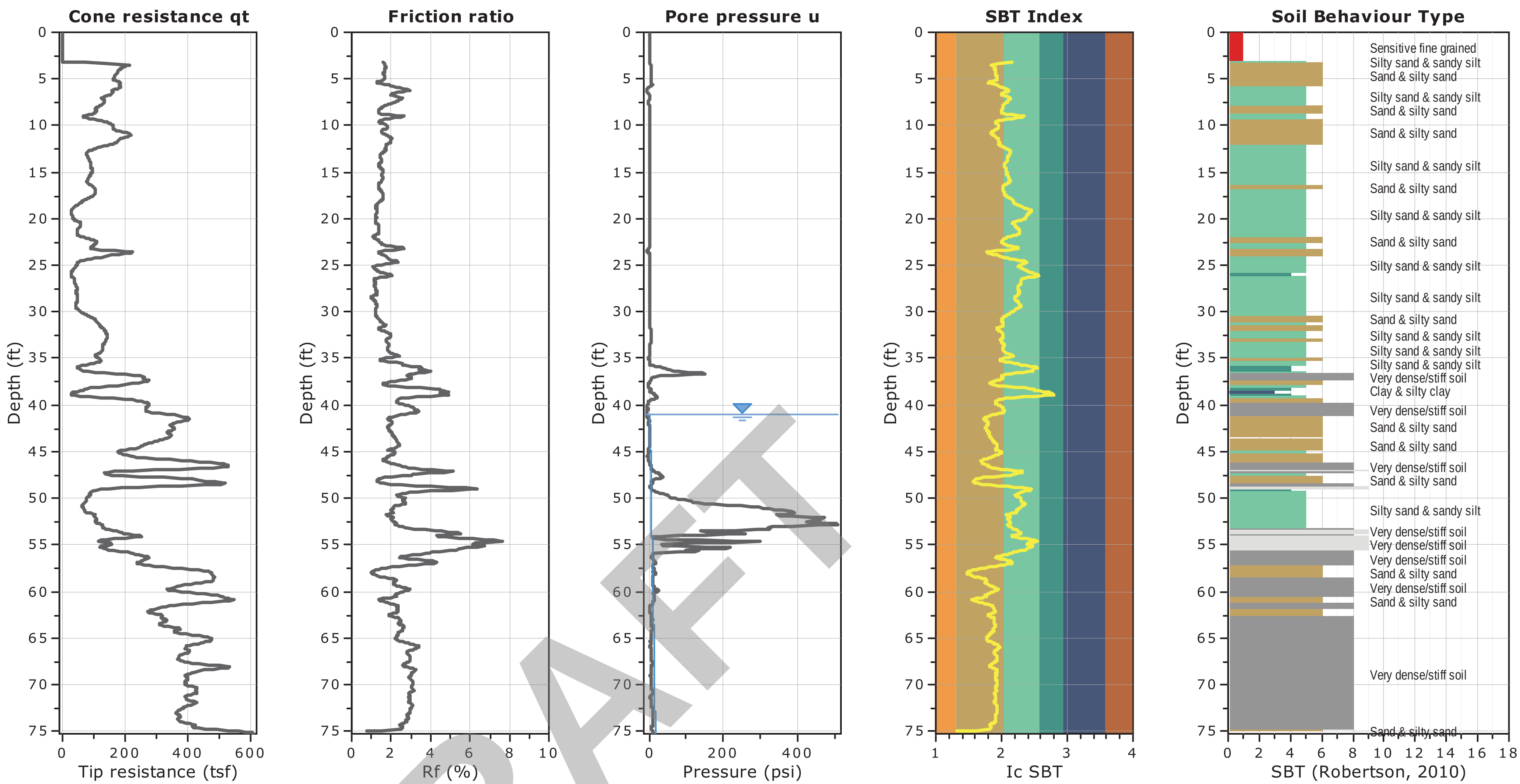
ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-3

Date 09/04/20 | Project No. 20-1906 | Figure A-3



DRAFT

Total depth: 75.3 ft, Date: September 3, 2020

Estimated Groundwater Depth: 41 feet (measured using a pore pressure dissipation test)

Cone Operator: Cone Tec

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

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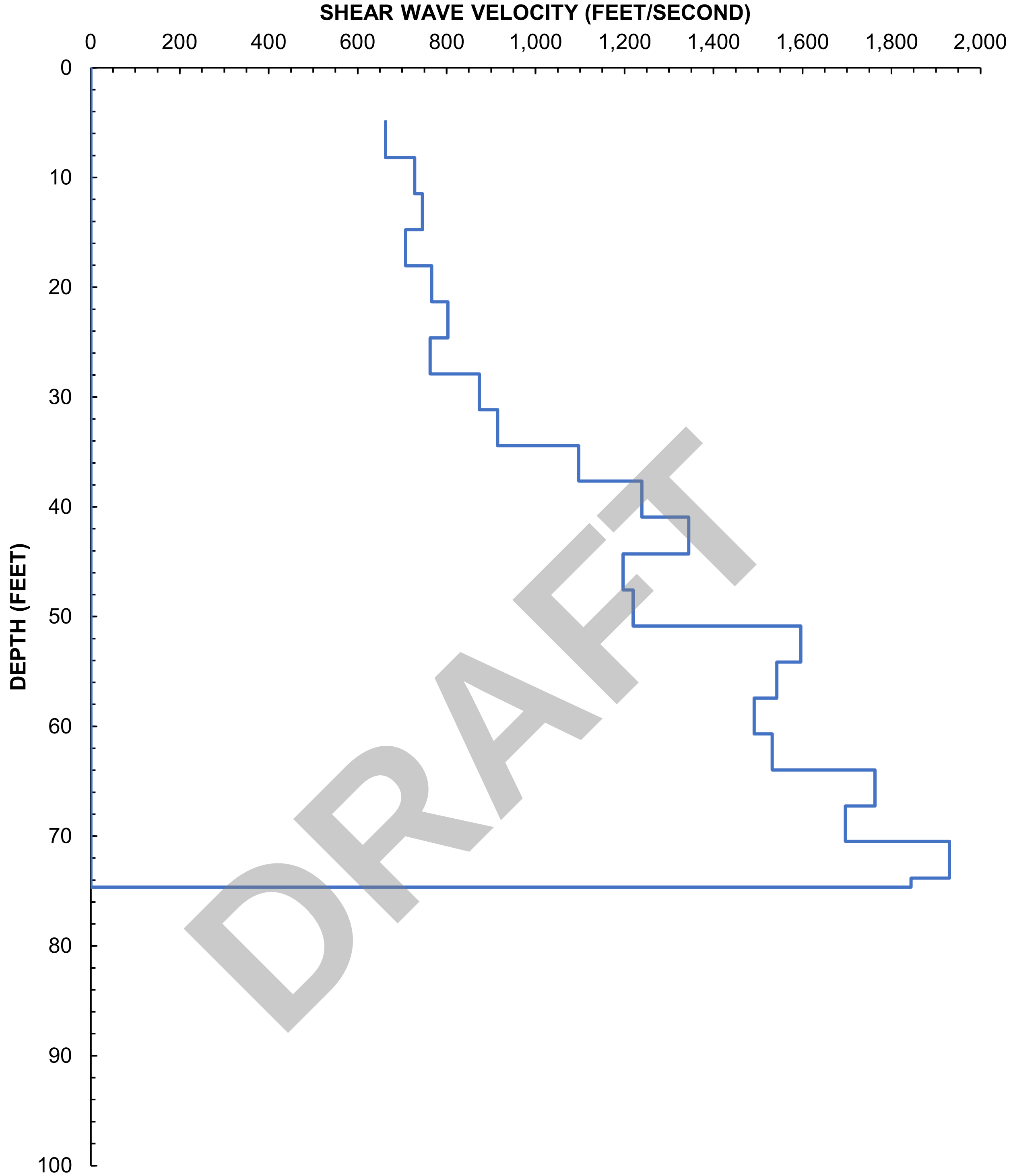
ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-4

Date 09/09/20	Project No. 20-1906	Figure A-4a



SHEAR WAVE VELOCITY CPT-4		
WESTLAKE CENTER **99 SOUTHGATE AVENUE** Daly City, California		
Date: 9/14/20	Project No. 20-1906	Figure A-4b
ROCKRIDGE GEOTECHNICAL		