GEOTECHNICAL INVESTIGATION
PROPOSED SEISMIC RETROFIT AND ADDITION
GUARDIAN COMMUNITY CENTER
2148 BRUSH STREET
Oakland, California

UNAUTHORIZED USE OR COPYING OF THIS DOCUMENT IS STRICTLY PROHIBITED BY ANYONE OTHER THAN THE CLIENT FOR THE SPECIFIC PROJECT

February 1, 2018
Project No. 17-1421
February 1, 2018
Project No. 17-1421

Mr. Joel Lunenfeld
12 Via Paraiso W
Tiburon, California 94920

Subject: Geotechnical Investigation Report
Proposed Seismic Retrofit and Potential Addition
The Guardian Community Center
2148 Brush Street
Oakland, California

Dear Mr. Lunenfeld,

We are pleased to present our geotechnical investigation report for the seismic retrofit and potential addition of the existing Guardian Community Center building at 2148 Brush Street in Oakland, California. Our investigation was performed in accordance with our proposal dated November 28, 2017.

The subject building is a one-story at-grade masonry building with plan dimensions of approximately 87 by 100 feet. There are no original foundation plans available for the structure. Cracking in the floor slab suggests the building has settled approximately 3/4 inch. A single test pit was excavated in the southwestern corner of the building by others to expose foundation conditions. Where exposed in the test pit, the foundation consists of a continuous concrete perimeter footing that is bottomed approximately 16 inches below top of slab (bts) and is approximately 14 inches wide.

We understand plans are to seismically retrofit the building. It is anticipated the seismic retrofit will consist of constructing concrete shear walls inside the building. Further, we understand two additional stories may be added to the building to provide additional space.

On the basis of our investigation, we conclude the proposed seismic retrofit and addition may be constructed as planned, provided the recommendations presented in the attached report are incorporated into the project plans and specification. Our investigation indicates the existing building footings are underlain by about two feet of fill, which includes soft to medium stiff sandy clay. Observations of floor-slab cracking around the building perimeter indicate the perimeter walls have settled, even under relatively light loads. Therefore, we recommend that no new loads be imposed on the existing footings.
We conclude the proposed seismic strengthening elements should be supported on shallow hand-dug piers which underpin the existing footings and bear on firm native soils below the fill, estimated to be 3 to 4 feet bts. Similarly, if new columns are installed as a part of a proposed addition, we conclude the columns should be supported on conventional spread footings bearing on firm native soil below the fill. Helical Mini-Piles may be used, if desired, to resist low to moderate uplift loads.

The recommendations contained in our report are based on a limited subsurface exploration. Consequently, variations between expected and actual subsurface conditions may be found in localized areas during construction. Therefore, we should be engaged to observe foundation installation 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.

DRAFT

Clayton J. Proto, P.E.
Project Engineer

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

Enclosure
1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by Rockridge Geotechnical, Inc. for the seismic retrofit and potential addition of the existing Guardian Community Center building at 2148 Brush Street in Oakland, California. The trapezoidal-shaped site is on the eastern side of Brush Street between 21st and 22nd Street, as shown on Figure 1, Site Location Map. It is bordered by a commercial building to the north, Interstate 980 to the east, an at-grade parking lot to the south, and Brush Street to the west.

The site is currently occupied by a one-story at-grade masonry building with plan dimensions of approximately 87 by 100 feet and a small asphalt-paved parking lot. We understand plans are to seismically retrofit the building. It is anticipated the seismic retrofit will consist of constructing concrete shear walls inside the building. Further, we understand two additional stories may be added to the building to provide additional space.

There are no original foundation plans available for the structure. Cracking in the floor slab suggests the building has settled approximately 3/4 inch. A single test pit was excavated in the southwestern corner of the building by others to expose foundation conditions. Where exposed in the test pit, the foundation consists of a continuous concrete perimeter footing that is bottomed approximately 16 inches below top of slab (bts) and is approximately 14 inches wide.

2.0 SCOPE OF SERVICES

Our geotechnical investigation was performed in accordance with our proposal dated November 28, 2017. Our scope of services consisted of exploring subsurface conditions at the site by advancing two test borings, performing two dynamic penetrometer tests (DPTs), and performing engineering analyses to develop conclusions and recommendations regarding:
• site hazards, including the potential for liquefaction and lateral spreading
• allowable bearing capacity, friction factor and passive resistance for existing footings
• the most appropriate foundation type(s) for the proposed addition
• design criteria for the recommended foundation type, including vertical and lateral capacities
• foundation settlement estimate for new addition
• subgrade preparation for new slab-on-grade areas
• site grading and excavation, including criteria for fill quality and compaction
• 2016 California Building Code site class and design spectral response acceleration parameters
• construction considerations.

3.0 FIELD INVESTIGATION

Prior to performing the subsurface field investigation, we obtained a permit from Alameda County Public Works Agency (ACPWA) and contacted Underground Service Alert (USA) to notify them of our work, as required by law. Details of the field exploration are described below.

3.1 Test Borings

Two borings, designated B-1 and B-2, were drilled on December 22, 2017 by Benevent Building of Concord, California at the approximate locations shown on the Site Plan, Figure 2. The borings were each drilled to a depth of 31.5 feet using a limited-access drill rig equipped with solid flight augers. During drilling, our field engineer logged the soil encountered and obtained representative samples for visual classification and laboratory testing. The logs of the borings are presented on Figures A-1 and A-2 in Appendix A. The soil encountered in the borings was classified in accordance with the classification chart shown on Figure A-3.

Soil samples were obtained using the following samplers:

• Sprague and Henwood (S&H) split-barrel sampler with a 3.0-inch outside diameter and 2.5-inch inside diameter, lined with 2.43-inch inside diameter tubes.
• Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside and 1.5-inch inside diameter, without liners.
The samplers were driven with an above-ground, 140-pound safety hammer falling 30 inches per drop utilizing a rope-and-cathead system. The samplers were driven up to 18 inches and the hammer blows required to drive the samplers were recorded every six inches and are presented on the boring logs. A “blow count” is defined as the number of hammer blows per six inches of penetration or 50 blows for six inches or less of penetration. The blow counts required to drive the S&H and SPT samplers were converted to approximate SPT N-values using factors of 0.7 and 1.2, respectively, to account for sampler type, approximate hammer energy, and the fact that the SPT sampler was driven without liners, but was sized to accommodate them. The blow counts used for this conversion were: (1) the last two blow counts if the sampler was driven more than 12 inches, (2) the last one blow count if the sampler was driven more than six inches but less than 12 inches, and (3) the only blow count if the sampler was driven six inches or less. The converted SPT N-values are presented on the boring logs.

Upon completion of drilling, the boreholes were backfilled with cement grout in accordance with ACPWA requirements. The surface of the boreholes was patched with quick-set concrete. The soil cuttings generated by the borings were left onsite.

3.2 Dynamic Penetrometer Test

We performed two dynamic penetrometer tests (DPTs), designated as DPT-1 and DPT-2, on December 22, 2017, at the approximate locations shown on Figure 2. The DPTs were performed by driving a 1.4-inch-diameter, cone-tipped probe into the ground with a 35-pound hammer falling about 15 inches. The blows required to drive the probe were recorded at 4-inch intervals and converted to approximate equivalent SPT N-values for use in our engineering analyses. DPT-1 and DPT-2 were advanced until practical refusal (defined at 50 blows for a 10-centimeter interval), which occurred at depths of 7.5 and 8 feet bgs, respectively. Plots of the DPT results are presented on Figure A-4 in Appendix A.

3.3 Test Pits

Prior to performing our subsurface investigation, one test pit was excavated by others to expose the existing building foundations. The locations of the test pit, designated as TP-1, is shown on
the attached Site Plan, Figure 2. The test pit was excavated to a depth of about 16 inches, at which point the bottom of the perimeter footing was encountered.

4.0 SUBSURFACE CONDITIONS

The geologic map prepared by Graymer et al. (2000), a portion of which is presented on Figure 3, indicates the site is underlain by Merritt sand (Holocene and Pleistocene, Qms), but is mapped near a geologic contact with Holocene-age alluvial fan deposits (Qhaf). Based on the results of our field investigation, we conclude the building basement slab is underlain by about 2 to 3.5 feet of fill consisting of brick debris and soft to medium stiff sandy clay. The fill is underlain by stiff to hard native clay with varying amounts of sand and medium dense to dense sand layers with varying amounts of silt and clay.

Groundwater was encountered at depths of 13 and 14 feet bgs in borings B-1 and B-2, respectively. The groundwater level is expected to fluctuate several feet seasonally, depending on amounts of rainfall. Regional groundwater maps suggest a historic high groundwater level of approximately 12 feet bgs.

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

\textbf{TABLE 1}
Regional Faults and Seismicity

<table>
<thead>
<tr>
<th>Fault Segment</th>
<th>Approximate Distance from Site (km)</th>
<th>Direction from Site</th>
<th>Maximum Magnitude</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Hayward</td>
<td>5.5</td>
<td>Northeast</td>
<td>7.00</td>
</tr>
<tr>
<td>Total Hayward-Rodgers Creek</td>
<td>5.5</td>
<td>Northeast</td>
<td>7.33</td>
</tr>
<tr>
<td>Mount Diablo Thrust</td>
<td>22</td>
<td>East</td>
<td>6.70</td>
</tr>
<tr>
<td>Total Calaveras</td>
<td>23</td>
<td>East</td>
<td>7.03</td>
</tr>
<tr>
<td>N. San Andreas - Peninsula</td>
<td>24</td>
<td>West</td>
<td>7.2</td>
</tr>
<tr>
<td>N. San Andreas (1906 event)</td>
<td>24</td>
<td>West</td>
<td>8.05</td>
</tr>
<tr>
<td>N. San Andreas - North Coast</td>
<td>26</td>
<td>West</td>
<td>7.5</td>
</tr>
<tr>
<td>Green Valley Connected</td>
<td>27</td>
<td>East</td>
<td>6.80</td>
</tr>
<tr>
<td>San Gregorio Connected</td>
<td>30</td>
<td>West</td>
<td>7.50</td>
</tr>
<tr>
<td>Rodgers Creek</td>
<td>34</td>
<td>Northwest</td>
<td>7.07</td>
</tr>
<tr>
<td>West Napa</td>
<td>39</td>
<td>North</td>
<td>6.70</td>
</tr>
<tr>
<td>Greenville Connected</td>
<td>40</td>
<td>East</td>
<td>7.00</td>
</tr>
<tr>
<td>Monte Vista-Shannon</td>
<td>42</td>
<td>South</td>
<td>6.50</td>
</tr>
<tr>
<td>Great Valley 5, Pittsburg Kirby Hills</td>
<td>44</td>
<td>East</td>
<td>6.70</td>
</tr>
</tbody>
</table>

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

\textsuperscript{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.
rupture along the San Andreas fault from Shelter Cove to San Juan Bautista approximately 470 kilometers in length. It had a maximum intensity of XI (MM), an $M_w$ of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The most recent earthquake to affect the Bay Area was the Loma Prieta Earthquake of 17 October 1989 with an $M_w$ of 6.9. This earthquake occurred in the Santa Cruz Mountains about 93 kilometers south of the site.

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

The U.S. Geological Survey's 2014 Working Group on California Earthquake Probabilities has compiled the earthquake fault research for the San Francisco Bay area in order to estimate the probability of fault segment rupture. They have determined that the overall probability of moment magnitude 6.7 or greater earthquake occurring in the San Francisco Region during the next 30 years (starting from 2014) is 72 percent. The highest probabilities are assigned to the Hayward fault, Calaveras fault, and the northern segment of the San Andreas fault. These probabilities are 14.3, 7.4, and 6.4 percent, respectively.

### 5.2 Geologic Hazards

Because the project site is in a seismically active region, we evaluated the potential for earthquake-induced geologic hazards including ground shaking, ground surface rupture, liquefaction, lateral spreading, and cyclic densification. We used the results of our field investigation to evaluate the potential of these phenomena occurring at the project site.

---

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.
5.2.1 Ground Shaking

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

5.2.2 Liquefaction and Associated Hazards

When a saturated, cohesionless soil liquefies, it experiences a temporary loss of shear strength created by a transient rise in excess pore pressure generated by strong ground motion. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits. Flow failure, lateral spreading, differential settlement, loss of bearing strength, ground fissures and sand boils are evidence of excess pore pressure generation and liquefaction.

The site is partially mapped within a zone of liquefaction potential as shown on the map titled *State of California Seismic Hazard Zones, Oakland West Quadrangle, Official Map*, dated February 14, 2003 (see Figure 5). We evaluated the liquefaction potential of soil encountered below groundwater at the site using data collected in our borings using the methodology outlined by Youd et al. (2001). Our analyses were performed using an assumed high “during earthquake” groundwater depth of 12 feet bgs. In accordance with the 2016 California Building Code (CBC), we used a peak ground acceleration of 0.67 times gravity (g) in our liquefaction evaluation; this peak ground acceleration is consistent with the Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration adjusted for site effects (PGA_M). We also used a moment magnitude 7.33 earthquake, which is consistent with the mean characteristic moment magnitude for the Hayward fault, as presented in Table 2.

---

4 Cyclic densification is a phenomenon in which non-saturated, cohesionless soil is compacted by earthquake vibrations, causing ground-surface settlement.
Our analysis indicates that the granular soil below the groundwater table is sufficiently dense to resist liquefaction. Therefore, we conclude the potential for liquefaction at the subject property is low.

Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Because we conclude the potential for liquefaction to occur at the site is low, we also conclude the potential for lateral spreading at the site is low.

5.2.3 Cyclic Densification

Seismically induced compaction (also referred to as cyclic densification) of non-saturated sand (sand above groundwater table) can occur during an earthquake, resulting in settlement of the ground surface and overlying improvements. Based on the results of our borings, DPTs, and the potentially highly variable nature of the near-surface fill, we conclude there may up to three feet of relatively loose granular that may be susceptible to cyclic densification in localized areas. We estimate cyclic densification during a major earthquake may cause up to 1/2 inch of settlement of the existing footings and differential settlement on the order of 1/2 inch over a horizontal distance of 30 feet. Foundation elements which gain capacity below the fill layer would not be susceptible to settlement from cyclic densification.

5.2.4 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.
6.0 CONCLUSIONS AND RECOMMENDATIONS

From a geotechnical standpoint, we conclude the building can be renovated and seismically upgraded as planned. Our recommendations for evaluation of existing footings and design of new foundations, as well as fill placement and seismic design are presented below.

6.1 Foundation Support and Settlement

Based on the results of our investigation, we conclude the existing building footings are immediately underlain by about two feet of fill, which includes soft to medium stiff sandy clay. Observations of floor-slab cracking around the building perimeter indicate the perimeter walls have settled, even under relatively light loads. Therefore, we recommend that no new loads be imposed on the existing footings.

We conclude the proposed seismic strengthening elements should be supported on shallow hand-dug piers which underpin the existing footings and bear on firm native soils below the fill, estimated to be 3 to 4 feet bts. Similarly, if new columns are installed as a part of a proposed addition, we conclude the columns should be supported on conventional spread footings bearing on firm native soil below the fill.

Footing and pier excavations should be free of standing water, debris, and loose or disturbed materials prior to placing concrete. We should check footing and pier excavations prior to placement of reinforcing steel. The existing fill below the floor slab may not stand vertically in cuts and, therefore, the foundation subcontractor should plan on forming the sides of the excavations with Stayform or equivalent.

If footings do not have sufficient uplift capacity for the design seismic loads, ground anchors can be used in conjunction with the footings to resist uplift forces. Considering the stiff to hard fine-grained soils underlying the site, helical mini-piles (HMPs) are also a viable option to provide additional uplift resistance to footings.

Foundation-specific recommendations are presented in the following subsections.
6.1.1 New Spread Footings

Spread footings which bear on firm native soil (estimated to be 3 to 4 feet bts) should be designed using an allowable bearing pressure of 4,000 pounds per square foot (psf) for dead-plus-live loads; this value may be increased by one-third for total loads, including wind and seismic loads. Continuous footings should be at least 18 inches wide and isolated footings should be at least 24 inches square. The recommended allowable bearing pressures for dead-plus-live and total load conditions include factors of safety of at least 2.0 and 1.5, respectively. We estimate settlement of footings properly designed and constructed footings will be 1/2 inch or less.

Lateral loads may be resisted by a combination of passive pressure on the vertical faces of the footings and friction between the bottoms of the footings and the supporting soil. To compute lateral resistance, we recommend using an equivalent fluid weight of 200 pounds per cubic foot (pcf); the upper foot of soil should be ignored unless confined by a slab. Frictional resistance should be computed using a base friction coefficient of 0.25. The passive pressure and frictional resistance values include a factor of safety of at least 1.5.

6.1.2 Underpinning Piers

Where loads on the existing perimeter footings will be increased, or where the existing footings will experience additional seismic loading, we recommend the existing footing be underpinned to bear on firm native soil. Underpinning piers may be designed using the allowable pressures for new footings described in Section 6.1.1. The width, number, and location of the underpinning piers should be determined by the structural engineer based on the required capacity and the ability of the existing foundation to span an area of non-support. Prior to construction, the depth and location of footings for the adjacent building to the north should be determined.

6.1.3 Helical Mini-Piles

Helical mini-piles (HMPs), also known as helical anchors and helical foundation piers, may be used to resist low to moderate uplift forces. An HMP is a segmented deep foundation system
with helical bearing plates welded to a central steel shaft. Segments are joined with bolted couplings. HMPs are screwed into the ground using a hydraulic motor attached to a backhoe or skid-steer loader to a specified depth or refusal. Corrosion protection for HMPs typically consists of a galvanized coating.

The design of HMPs, which includes the number, size, and depth of helices required to attain an ultimate tension capacity, as well as the bar size required for the anticipated installation torque, is typically determined by the design-build contractor that will be installing the system. During construction, a minimum of one HMP should be load tested in tension to verify the contractor’s pile design and installation methods achieve the required capacity. Based on our experience on previous projects in similar soil conditions, we anticipate that an allowable HMP uplift capacity of 10 to 15 kips is feasible for the stiff to hard clays underlying this site. We recommend the HMP subcontractor determine the number, size, and depth of helices required to attain an ultimate tension capacity of at least 1.5 times the design tension load. In addition, the upward deflection of the HMP should be limited to a tolerable level at the design tension load, as specified by the structural engineer. In our experience maximum allowable deflection at the design load is typically about 3/4 inch.

Prior to installing production HMPs, at least one HMP should be load tested in tension to 1.5 times the design tension load (DL) for each anchor length and helix configuration installed at the site. The HMPs to be tested should be located in close proximity to the production HMPs. During testing, the deflection of each HMP should be monitored with a free-standing, tripod-mounted dial gauge accurate to at least 0.001 inches. We recommend deflection of the HMPs be measured at load increments equal to 25 percent DL. The maximum test load should be held for a minimum of 10 minutes, with readings taken at 0, 1, 3, 6, and 10 minutes. If the difference between the 1- and 10-minute readings is more than 0.08 inches, the load should be held for an additional 50 minutes, with additional readings taken at 15, 20, 30, 45 and 60 minutes. If the deflection is more than 0.08 inches between the 10- and 60-minute readings or the total deflection is greater than the maximum tolerable deflection at the design load, the HMP design should be revised. Additional load tests should be performed to confirm the revised design
meets the above criteria. At least two percent of all production piles shall be proof-tested to 1.5 times DL.

The lateral capacities of HMPs will be negligible because of the small shaft cross section and the soil disturbance caused by installation of the helices. Where HMPs are installed, frictional resistance along the base of the footing should be ignored.

HMPs should be protected against corrosion in accordance with the requirements of ICC-ES AC 358. In addition, in accordance with CBC 2016 Section 1810.3.1.5.1, all helical pile materials that are subject to corrosion shall include at least 1/16 inch of corrosion allowance.

6.2 Fill Quality and Compaction

In areas that will receive fill, the soil subgrade exposed should be scarified to a depth of at least eight inches, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent relative compaction\(^5\) (95 percent relative compaction for granular soil containing less than 10 percent fines content).

Material excavated at the site will primarily consist of artificial clayey fill with varying sand content that may be reused as backfill provided it is at a moisture content that compaction can be achieved and free of debris. If imported fill (select fill) is required, it should be free of organic matter, contain no rocks or lumps larger than three inches in greatest dimension, have a liquid limit less than 40 and plasticity index less than 12, and be approved by the Geotechnical Engineer. Samples of proposed select fill material should be submitted to the Geotechnical Engineer at least three business days prior to use at the site. The grading contractor should provide analytical test results or other suitable environmental documentation indicating the imported fill is free of hazardous materials at least three days before use at the site. If this data is not available, up to two weeks should be allowed to perform analytical testing on the proposed imported material.

\(^5\) Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557 laboratory compaction procedure.
Fill should be placed in lifts not exceeding eight inches in loose thickness, moisture-conditioned to near optimum moisture content, and compacted to at least 90 percent relative compaction (95 percent relative compaction for granular soil containing less than 10 percent fines content).

6.3 Seismic Design

For design in accordance with the 2016 CBC, we recommend Site Class D be used. The latitude and longitude of the site are 37.8122° and -122.2748° respectively. Hence, in accordance with the 2016 CBC, we recommend the following:

- $S_S = 1.725g$, $S_1 = 0.683g$
- $S_MS = 1.725g$, $S_M1 = 1.025g$
- $S_DS = 1.15g$, $S_D1 = 0.683g$
- Seismic Design Category D for Risk Categories I, II, and III

6.4 Concrete Slab-on-Grade Floor

Areas to receive new floor slabs may be designed as slab-on-grade. The upper eight inches of slab subgrade should be scarified, moisture conditioned, and recompacted in accordance with the recommendations in Section 6.2.

Where water vapor transmission through the floor slab is not desirable, we recommend installing a capillary moisture break and water vapor retarder beneath the floor slab. A capillary moisture break consists of at least four inches of clean, free-draining gravel or crushed rock. The vapor retarder should meet the requirements for Class 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. The particle size of the capillary break material should meet the gradation requirements presented in Table 2.
TABLE 2
Gradation Requirements for Capillary Moisture Break

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Percentage Passing Sieve</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 inch</td>
<td>90 – 100</td>
</tr>
<tr>
<td>3/4 inch</td>
<td>30 – 100</td>
</tr>
<tr>
<td>1/2 inch</td>
<td>5 – 25</td>
</tr>
<tr>
<td>3/8 inch</td>
<td>0 – 6</td>
</tr>
</tbody>
</table>

The slab should be properly cured. Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab. Therefore, concrete for the floor slab should have a low w/c ratio - less than 0.45. Water should not be added to the concrete mix in the field. If necessary, workability should be increased by adding plasticizers. Before the floor covering is placed, the contractor should check that the concrete surface and the moisture emission levels (if emission testing is required) meet the manufacturer’s requirements.

6.5 Soil Corrosivity

Laboratory testing was performed by Project X Corrosion Engineering to evaluate the corrosivity of near surface clay samples obtained from borings B-1 at a depth of 3 feet bgs. The results of this corrosivity testing are presented at the end of Appendix B.

Based on the results of the resistivity test, we conclude the soil at this site is “highly corrosive” to buried metal. Accordingly, all buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric-coated steel or iron should be protected against corrosion depending upon the critical nature of the structure. If it is necessary to have metal in contact with soil, a corrosion engineer should be consulted to provide detailed recommendations for corrosion protection. In addition, the results indicate the chloride ion concentration is considered “corrosive” and intrusion of
chloride ions into the concrete may lead to corrosion of the embedded steel reinforcement (Caltrans, 2015). To help prevent the diffusion of chloride ions into the concrete, supplementary cementitious materials such as fly-ash, silica fume, slag, etc. should be used in the concrete design to reduce the permeability of the concrete. A low water-cement ratio leads to a denser concrete, which also leads to lower permeability concrete (Caltrans, 2010).

The pH of the soil is “mildly corrosive” to buried steel and concrete structures. The results indicate that sulfate ion concentrations are insufficient to damage concrete structures below ground.

6.6 Construction Considerations

The soil to be excavated for the new footings is expected to consist mostly of clay with varying sand content. Due to overhead restrictions and proximity to existing foundations, limited access or/ hand excavation equipment should be used. Excavations that will be deeper than five feet and will be entered by workers should be sloped or shored in accordance with CAL-OSHA standards (29 CFR Part 1926). The contractor should be responsible for the construction and safety of temporary slopes.

Heavy equipment should not be used within existing building footprint and within 10 horizontal feet from adjacent shallow foundations. Jumping jack or hand-operated vibratory plate compactors should be used for compacting fill within this zone.

7.0 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 installation of new foundations and fill placement and compaction. 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.
8.0 LIMITATIONS

This geotechnical study 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 exploratory borings and DPTs. If any variations or undesirable conditions are encountered during construction, we should be notified so that additional recommendations can be made. The foundation recommendations presented in this report are developed exclusively for the proposed development described in this report and are not valid for other locations and construction in the project vicinity.
REFERENCES


California Department of Transportation (2010). Memo to Designers 10-5, Protection of Reinforcement Against Corrosion Due to Chlorides, Acids, and Sulfates, June.


California Geologic Survey (2008), Fault Rupture Hazard Zones in California, Special Publication 42.


Jennings, C.W., 1994, Fault activity map of California and adjacent areas with locations and ages of recent volcanic eruptions: California Division of Mines and Geology Geologic Data Map No. 6, scale 1: 750,000.


FIGURES
APPENDIX A
Logs of Borings and Dynamic Penetrometer Test Results
APPENDIX B
Laboratory Test Results
# TABLE OF CONTENTS

1.0 INTRODUCTION .................................................................................................................. 1

2.0 SCOPE OF SERVICES ......................................................................................................... 1

3.0 FIELD INVESTIGATION .................................................................................................... 2
   3.1 Test Borings ...................................................................................................................... 2
   3.2 Dynamic Penetrometer Test ......................................................................................... 3
   3.3 Test Pits .......................................................................................................................... 3

4.0 SUBSURFACE CONDITIONS ............................................................................................ 4

5.0 SEISMIC CONSIDERATIONS .......................................................................................... 4
   5.1 Regional Seismicity and Faulting ................................................................................ 4
   5.2 Geologic Hazards ......................................................................................................... 6
      5.2.1 Ground Shaking ..................................................................................................... 7
   5.2.2 Liquefaction and Associated Hazards ..................................................................... 7
   5.2.3 Cyclic Densification ................................................................................................. 8
   5.2.4 Ground Surface Rupture ......................................................................................... 8

6.0 CONCLUSIONS AND RECOMMENDATIONS .................................................................. 9
   6.1 Foundation Support and Settlement ............................................................................. 9
      6.1.1 New Spread Footings ............................................................................................ 10
      6.1.2 Underpinning Piers ............................................................................................... 10
      6.1.3 Helical Mini-Piles ................................................................................................ 10
      6.2 Fill Quality and Compaction .................................................................................... 12
   6.3 Seismic Design ............................................................................................................ 13
   6.4 Concrete Slab-on-Grade Floor ..................................................................................... 13
   6.5 Soil Corrosivity ........................................................................................................... 14
   6.6 Construction Considerations ....................................................................................... 15

7.0 ADDITIONAL GEOTECHNICAL SERVICES .................................................................. 15

8.0 LIMITATIONS .................................................................................................................. 16

REFERENCES

FIGURES

APPENDIX A – Logs of Borings and Dynamic Penetrometer Test Results

APPENDIX B – Laboratory Test Results
LIST OF FIGURES

Figure 1      Site Location Map
Figure 2      Site Plan
Figure 3      Regional Geologic Map
Figure 4      Regional Fault Map
Figure 5      Seismic Hazards Zone Map

APPENDIX A

Figures A-1 and A-2  Logs of Boring B-1 and B-2
Figure A-3      Boring Classification Chart
Figure A-4      Dynamic Penetrometer Test Results, DPT-1 and DPT-2

APPENDIX B

Figure B-1      Unconsolidated-Undrained Triaxial Compression Test
Figure B-2      Plasticity Chart
Corrosivity Test Results
SITE LOCATION MAP

2148 BRUSH STREET
Oakland, California

Date 01/04/18  Project No. 17-1421  Figure 1
EXPLANATION

DPT-1 ◀️ Approximate location of dynamic penetrometer test by Rockridge Geotechnical Inc., December 22, 2017

B-1 ○ Approximate location of boring by Rockridge Geotechnical Inc., December 22, 2017

TP-1 ▶️ Approximate location of test pit by others, December 2017

Project limits

2148 BRUSH STREET
Oakland, California

ROCKRIDGE GEOTECHNICAL

Date 01/30/18 Project No. 17-1421 Figure 2

Base map: Google Earth, 2017.
EXPLANATION

- **af**: Artificial fill (Historic)
- **Qhaf**: Alluvial fan and fluvial deposits (Holocene)
- **Qms**: Merritt sand (Holocene and Pleistocene)
- **Qpaf**: Alluvial fan and fluvial deposits (Pleistocene)
- **Qmt**: Marine terrace deposits (Pleistocene)

Base map: USGS MF 2342, Geologic Map and Map Database of the Oakland Metropolitan Area, Alameda, Contra Costa, and San Francisco Counties, California (Graymer, 2000).
EXPLANATION

- **Strike slip**
- **Thrust (Reverse)**
- **Normal**

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

**SITE**

2148 BRUSH STREET
Oakland, California

**REGIONAL FAULT MAP**

Date 01/04/18 | Project No. 17-1421 | Figure 4
EXPLANATION

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

**Earthquake-Induced Landslides;** Areas where previous occurrence 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" Oakland West Quadrangle.
Released on February 14, 2003
4 inches of concrete
brick debris

SANDY CLAY (CL)
dark gray-brown, soft, moist to wet, fine-grained sand, low plasticity

CLAY (CL)
dark olive-brown, stiff, moist, fine-grained sand, trace gravel
LL = 38, PI = 25; see Appendix B
olive, very stiff, no gravel

CLAYEY SAND (SC)
olive-gray, medium dense, moist, fine-grained sand

Silty SAND (SM)
olive-gray, dense, moist, fine-grained sand, trace clay

SANDY CLAY (CL)
olive-gray with trace brown mottling, very stiff, moist, fine-grained sand, trace fine gravel

SAND with SILT and GRAVEL (SP-SM)
olive-gray, medium dense, moist, fine- to coarse-grained sand, fine to coarse, angular to subangular gravel, weak hydrocarbon odor
Silty SAND (SM)
olive brown, medium dense, wet, fine-grained sand, trace clay

SANDY CLAY with GRAVEL (CL)
olive-brown, very stiff, wet, fine- to coarse-grained sand, angular to subangular gravel

CLAY (CL)
olive gray mottled with olive brown, stiff, wet, trace fine- to medium-grained sand

SANDY CLAY (CL)
gray, very stiff, wet, fine-grained sand

S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively, to account for sampler type and hammer energy.
PROJECT: 2148 BRUSH STREET
Oakland, California

Boring location: See Site Plan, Figure 2
Date started: 12/22/18     Date finished: 12/22/18
Drilling method: Solid Stem Auger
Hammer weight/drop: 140 lbs./30 inches     Hammer type: Rope & Cathead
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

<table>
<thead>
<tr>
<th>DEPTH (feet)</th>
<th>SAMPLES</th>
<th>MATERIAL DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>S&amp;H</td>
<td>8 inches of concrete</td>
</tr>
<tr>
<td>2</td>
<td>S&amp;H</td>
<td>4 inches of aggregate base</td>
</tr>
<tr>
<td>3</td>
<td>S&amp;H</td>
<td>SANDY CLAY (CL) dark brown to dark gray-brown, medium stiff, moist to wet, fine-grained sand, with roots and rootlets, low plasticity</td>
</tr>
<tr>
<td>4</td>
<td>S&amp;H</td>
<td>SANDY CLAY (CL) olive with gray-brown mottling, hard, moist, fine-grained sand</td>
</tr>
<tr>
<td>5</td>
<td>S&amp;H</td>
<td>CLAYEY SAND (SC) olive-brown, black and red-brown mottling, dense, moist, fine-grained sand, trace coarse-grained sand</td>
</tr>
<tr>
<td>6</td>
<td>S&amp;H</td>
<td>SILTY SAND (SM) brown to red-brown, dense, moist, fine-grained sand, trace fine gravel and clay</td>
</tr>
<tr>
<td>7</td>
<td>S&amp;H</td>
<td>SAND with SILT (SP-SM) brown to red-brown, dense, wet, fine-grained sand</td>
</tr>
<tr>
<td>8</td>
<td>SPT</td>
<td>SILTY SAND (SM) gray-brown, medium dense, wet, fine-grained sand</td>
</tr>
<tr>
<td>9</td>
<td>S&amp;H</td>
<td>CLAYEY SAND (SC) gray-brown, medium dense, wet, trace subrounded fine gravel</td>
</tr>
<tr>
<td>10</td>
<td>S&amp;H</td>
<td>CLAYEY SAND (SC) gray with gray-brown mottling, very stiff, wet, trace fine-grained sand</td>
</tr>
<tr>
<td>11</td>
<td>S&amp;H</td>
<td>SILTY SAND (SM) gray-brown, medium dense to dense, wet, fine-grained sand</td>
</tr>
</tbody>
</table>

LABORATORY TEST DATA

<table>
<thead>
<tr>
<th>Type of Strength Test</th>
<th>Compacting Pressure</th>
<th>Shear Strength</th>
<th>Fines</th>
<th>Natural Moisture Content</th>
<th>Dry Density</th>
</tr>
</thead>
<tbody>
<tr>
<td>Triaxial Test</td>
<td>4,500</td>
<td>22.0</td>
<td>105</td>
<td>500</td>
<td></td>
</tr>
</tbody>
</table>

S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively, to account for sampler type and hammer energy.

Boring terminated at a depth of 31.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 14 feet during drilling.
# Unified Soil Classification System

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

## Grain Size Chart

<table>
<thead>
<tr>
<th>Classification</th>
<th>U.S. Standard Sieve Size</th>
<th>Grain Size in Millimeters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Boulders</td>
<td>Above 12”</td>
<td>Above 305</td>
</tr>
<tr>
<td>Cobble</td>
<td>12” to 3”</td>
<td>305 to 76.2</td>
</tr>
<tr>
<td>Gravel coarse</td>
<td>3” to No. 4</td>
<td>76.2 to 4.76</td>
</tr>
<tr>
<td>Gravel fine</td>
<td>3” to 3/4”</td>
<td>76.2 to 19.1</td>
</tr>
<tr>
<td>Gravel fine</td>
<td>3/4” to No. 4</td>
<td>19.1 to 4.76</td>
</tr>
<tr>
<td>Sand coarse</td>
<td>No. 4 to No. 200</td>
<td>4.76 to 0.075</td>
</tr>
<tr>
<td>Sand medium</td>
<td>No. 4 to No. 10</td>
<td>4.76 to 2.00</td>
</tr>
<tr>
<td>Sand fine</td>
<td>No. 10 to No. 40</td>
<td>2.00 to 0.420</td>
</tr>
<tr>
<td>Silt and Clay</td>
<td>No. 40 to No. 200</td>
<td>0.420 to 0.075</td>
</tr>
<tr>
<td></td>
<td>Below No. 200</td>
<td>Below 0.075</td>
</tr>
</tbody>
</table>

- ▲ 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
DYNAMIC PENETROMETER TEST RESULTS

A-4

BLOWS PER 4 INCHES (10 centimeters)

DEPTH BELOW GROUND SURFACE (feet)

2148 BRUSH STREET
Oakland, California

ROCKRIDGE GEOTECHNICAL

Date 01/04/18 | Project No. 17-1421 | Figure A-4
Sampler Type: Sprague & Henwood  
Shear Strength: 4,500 psf

Diameter (in): 2.39  
Height (in): 5.4

Moisture Content: 22%  
Confining Pressure: 500 psf

Dry Density: 105 pcf  
Strain Rate: 1%/min

Source: B-2 at 4.0 feet

Description: SANDY CLAY (CL), olive with gray-brown mottling

2148 BRUSH STREET  
Oakland, California

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date: 01/05/18  Project No. 17-1421  Figure B-1
**PLASTICITY CHART**

Reference:
ASTM D2487-00

**Symbol** | **Source** | **Description and Classification** | **Natural M.C. (%)** | **Liquid Limit (%)** | **Plasticity Index (%)** | **% Passing #200 Sieve**
---|---|---|---|---|---|---
[•] | B-1 at 4 feet | SANDY CLAY (CL), dark olive-brown | 18.2 | 38 | 25 | --

**2148 BRUSH STREET**
Oakland, California

ROCKRIDGE GEOTECHNICAL

Date 01/10/18 | Project No. 17-1421 | Figure B-2
Results Only Soil Testing for 2148 Brush Street

January 9, 2018

Prepared for:
Clayton Proto
Rockridge Geotechnical
270 Grand Ave,
Oakland, CA 94610
cjproto@rockridgegeo.com

Project X Job#: S180108A
Client Job or PO#: 17-1421
Soil Analysis Lab Results

Client: Rockridge Geotechnical
Job Name: 2148 Brush Street
Client Job Number: 17-1421
Project X Job Number: S180108A
January 9, 2018

<table>
<thead>
<tr>
<th>Bore# / Description</th>
<th>Method</th>
<th>ASTM G187</th>
<th>ASTM D516</th>
<th>ASTM D512B</th>
<th>SM 4500-NO3-E</th>
<th>SM 4500-NH3-C</th>
<th>SM 4500-S2-D</th>
<th>ASTM G280</th>
<th>ASTM G51</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Depth</td>
<td>Resistivity</td>
<td>Sulfates</td>
<td>Chlorides</td>
<td>Nitrate</td>
<td>Ammonia</td>
<td>Sulfide</td>
<td>Redox</td>
<td>pH</td>
</tr>
<tr>
<td></td>
<td>(ft)</td>
<td>(Ohm-cm)</td>
<td>(mg/kg)</td>
<td>(mg/kg)</td>
<td>(mg/kg)</td>
<td>(mg/kg)</td>
<td>(mg/kg)</td>
<td>(mV)</td>
<td></td>
</tr>
<tr>
<td>B-1 #3</td>
<td>3.5</td>
<td>804</td>
<td>804</td>
<td>480</td>
<td>0.0480</td>
<td>1740</td>
<td>0.1740</td>
<td>132</td>
<td>258.0</td>
</tr>
</tbody>
</table>

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

Please call if you have any questions.

Prepared by,

Ernesto Padilla, BSME
Field Engineer

Respectfully Submitted,

Eddie Hernandez, M.Sc., P.E.
Sr. Corrosion Consultant
NACE Corrosion Technologist #16592
Professional Engineer
California No. M37102
ehernandez@projectxcorrosion.com
**Project X**

Corrosion Engineering

**Corrosion Control - Soil, Water, and Metallurgy Lab**

**IMPORTANT:** Please complete Project and Sample Identification Data as you would like it to appear in report & include this form with samples.

<table>
<thead>
<tr>
<th>Company Name:</th>
<th>Rockridge Geotechnical</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mailing Address:</td>
<td>270 Grand Avenue, Oakland California</td>
</tr>
<tr>
<td>Accounting Contact:</td>
<td>Kate Schenk</td>
</tr>
<tr>
<td>Project Name:</td>
<td>2148 Brush Street</td>
</tr>
<tr>
<td>Client Project No:</td>
<td>17-1421</td>
</tr>
</tbody>
</table>

**Project X Job #:**

<table>
<thead>
<tr>
<th>Date:</th>
<th></th>
</tr>
</thead>
</table>

**Contact Name:** Clayton Proto  
**Contact Email:** cjproto@rockridgegeo.com  
**Invoice Email:** kaschenk@rockridgegeo.com  
**Phone No.:** 510-420-5738 x 120

**Project Name:** 2148 Brush Street  
**Client Project No:** 17-1421  
**P.O. #:**

<table>
<thead>
<tr>
<th>Turn Around Time:</th>
<th>X</th>
</tr>
</thead>
</table>

**ANALYSIS REQUESTED (Please circle):**

- Soil Resistivity
- Chloride
- Sulfate
- Redox Potential
- pH
- Alkalinity
- Nitrate
- Ammonia
- Moisture Content
- Soil Corrosivity
- Evaluation Report
- Metallographic Analysis

**NOTES**

**SPECIAL INSTRUCTIONS:**

**SAMPLE ID - BORE #:**

<table>
<thead>
<tr>
<th>SAMPLE ID - BORE #</th>
<th>DESCRIPTION</th>
<th>DEPTH (ft)</th>
<th>DATE COLLECTED</th>
<th>CORROSION ANALYSIS</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1 #3</td>
<td>CLAY</td>
<td>3.5</td>
<td>12/22/17</td>
<td></td>
</tr>
</tbody>
</table>