ROCKRIDGE GEOTECHNICAL

Prepared for **Silicon Sage Builders**

**FINAL GEOTECHNICAL INVESTIGATION**
**PROPOSED APARTMENT BUILDINGS**
**OSGOOD II-IV**
**41911-42021 OSGOOD ROAD**
**FREMONT, CALIFORNIA**



January 7, 2020
Project No. 17-1297

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Ms. Shaivali Desai
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Silicon Sage Builders
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Sunnyvale, California 94086

Subject: Final Geotechnical Investigation Report
Proposed Apartment Buildings
Osgood II-IV
41911-42021 Osgood Road
Fremont, California

Dear Ms. Desai:

We are pleased to present the results of our geotechnical investigation for the proposed apartment buildings to be constructed at 41911-42021 Osgood Road in Fremont, California. Our services were performed in accordance with our proposals dated February 8, 2018 (Osgood II) and April 12, 2018 (Osgood III).

The site is located on the western side of Osgood Road between Washington Boulevard and Blacow Road and consists of three adjacent parcels with combined dimensions of about 380 by 420 feet. The site is currently occupied by two single-story commercial buildings, a single-family residence, asphalt-paved parking lots, and landscaping areas.

Based on our discussions with the project team, we understand the proposed development will consist of two apartment buildings, each with four levels of wood-framed construction over a one-level, at-grade concrete podium structure which will contain parking.

From a geotechnical standpoint, we conclude the site can be developed as planned, provided the recommendations presented in this report are incorporated into the project plans and specifications and implemented during construction. The primary geotechnical issues affecting the proposed development include the potential for strong ground shaking at the site, the presence of surficial undocumented fill in portions of the site, and providing adequate vertical and lateral foundation support.

Provided the estimated total and differential settlements presented in our report are acceptable, the buildings may be supported on conventional spread footings which are

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deepened to gain support below the surficial undocumented fill or, alternatively, the undocumented fill is over-excavated and replaced with engineered (compacted) fill.

Our report contains specific recommendations regarding earthwork and grading, foundation design, and other geotechnical issues. The recommendations contained in our report are based on limited subsurface exploration. Consequently, variations between expected and actual soil conditions may be found in localized areas during construction. Therefore, we should be engaged to observe foundation, grading, and fill placement, 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.

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Enclosure

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**FINAL GEOTECHNICAL INVESTIGATION**
**PROPOSED APARTMENT BUILDINGS**
**OSGOOD II-IV**
**41911-42021 OSGOOD ROAD**
**Fremont, California**

## 1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed apartment buildings to be constructed at 41911-42021 Osgood Road in Fremont, California. The subject site is located on the western side of Osgood Road between Washington Boulevard and Blacow Road, as shown on the Site Location Map, Figure 1. We previously performed a preliminary geotechnical study for the 42021 Osgood Road property and presented our results in a letter dated April 13, 2017.

The site consists of three parcels: Osgood II (42021 Osgood Road), Osgood III (41965 Osgood Road), and Osgood IV (41911 Osgood Road). The parcels are bounded by Osgood Road to the east, a single-family home to the north, an apartment building to the south, and Bay Area Rapid Transit (BART) tracks to the west. The combined parcels comprise an area that is approximately trapezoidal-shaped with maximum plan dimensions of about 380 by 420 feet. The site is currently occupied by two single-story commercials buildings, a single-family residence, asphalt-paved parking lots, and landscaping areas, as shown on the Site Plan, Figure 2.

Based on our discussions with the project team, we understand the proposed development will consist of two apartment buildings, each with four levels of wood-framed construction over a one-level, at-grade concrete podium structure which will contain parking. Finished floor elevations[1] of the buildings will be 67.2 feet and 67.5 feet. Structural loads are not currently available for the proposed buildings, however, based on our experience with similar structures, we estimate the proposed buildings will have maximum dead-plus-live column loads on the order of 500 kips.

[1] Elevations reference NGVD29 and are based on *Planning Submittal, Osgood II Residences* prepared by BKF Engineers, revision date 9/25/2019

## 2.0 SCOPE OF SERVICES

Our services were performed in accordance with our proposals dated February 8, 2018 (Osgood II) and April 12, 2018 (Osgood III). The objective of our investigation was to evaluate subsurface conditions at the site and develop conclusions and recommendations regarding the geotechnical aspects of the proposed project. Our scope of work consisted of reviewing existing subsurface data available for the subject site and site vicinity, further evaluating subsurface conditions at the site by drilling ten exploratory borings, advancing five hand auger borings, performing four dynamic penetration tests (DPTs), and performing engineering analyses to develop conclusions and recommendations regarding:

- soil and groundwater conditions beneath the site
- site seismicity and seismic hazards, including the potential for liquefaction and liquefaction-induced ground failure
- the most appropriate foundation type(s) for the proposed buildings
- design criteria for the recommended foundation type(s), including vertical and lateral capacities
- estimates of static and seismically induced foundation settlement
- subgrade preparation for pavements and exterior concrete flatwork
- site grading and excavation, including criteria for fill quality and compaction
- flexible and rigid pavement design
- soil corrosivity
- 2019 California Building Code (CBC) site class and design spectral response acceleration parameters
- construction considerations

## 3.0 FIELD INVESTIGATION

Subsurface conditions at the site were investigated by drilling ten exploratory borings, advancing five hand auger borings, performing four DPTs, and performing laboratory testing on select soil samples collected from the borings. Prior to drilling the borings, we obtained a drilling permits from Alameda County Water District (ACWD), contacted Underground Service Alert (USA) to notify them of our work, as required by law, and retained Precision Locating, LLC, a private utility locator, to check that the boring locations were clear of existing underground utilities. Upon completion, the borings were backfilled with cement grout in accordance with ACWD requirements and under the observation of their inspector. Details of the field investigation and laboratory testing are described in the following sections.

### 3.1 Exploratory Borings

The exploratory borings were drilled on March 22 and June 29, 2018 by Exploration GeoServices of San Jose, California. The borings, designated B-1 through B-10, were each drilled to depths between about 10 and 21-1/2 feet bgs using a truck-mounted drill rig equipped with eight-inch-diameter hollow-stem augers. During drilling, our field engineer logged the soil encountered and obtained representative samples for visual classification and laboratory testing. The approximate locations of the exploratory borings are shown on the Site Plan, Figure 2. The logs of the borings are presented on Figures A-1 through A-10. The soil encountered in the borings was classified in accordance with the Classification Chart shown on Figure A-16.

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

The type of sampler used was selected based on soil type and the desired sample quality for laboratory testing. In general, the S&H sampler was used to obtain samples in medium stiff to very stiff cohesive soil and the SPT sampler was used to evaluate the relative density of granular soils. The S&H and SPT samplers were driven with a 140-pound, downhole, wireline hammer falling about 30 inches per drop. 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 sampler were converted to approximate SPT N-values using factors of 0.6 (B-1 through

B-6) or 0.7 (B-7 though B10) to account for sampler type, approximate hammer energy, as previously measured by the drilling subcontractor. SPT blow counts were converted using a factor of 1.2 to account for the fact that the SPT sampler was designed to accommodate liners, but liners were not used. The blow counts used for this conversion were the last two blow counts. The converted SPT N-values are presented on the boring logs. The soil cuttings from the boring were spread in unpaved areas onsite (Osgood IV) or drummed (Osgood II). Analytical testing of the drummed cuttings indicated they were non-hazardous, and they were subsequently transported to an appropriate disposal facility.

### 3.2 Hand Auger Borings and Dynamic Penetration Tests (DPTs)

The Osgood III parcel was not accessible to a truck-mounted drill rig. Therefore, we explored the subsurface conditions at this location by advancing five hand-auger borings, designated as HA-1 through HA-5, and four DPTs, designated as DPT-2 through DPT-5. The approximate locations of the DPTs and hand-auger borings are shown on the Site Plan, Figure 2.

The hand-auger borings were advanced to supplement the DPT data and to obtain samples of the soil for visual classification and laboratory testing. The borings were each advanced using a three-inch-diameter hand auger to practical refusal at depths of 7 to 10 feet bgs. The subsurface conditions encountered in the hand auger borings are presented on Figures A-11 through A-15 in Appendix A. The soil encountered is classified in accordance with the charts presented on Figure A-16.

The DPTs consisted of manually driving a 1.4-inch-diameter cone-tipped probe with a 30-pound hammer falling 15 inches. The blow counts required to drive the probe were recorded at 10-centimeter intervals. The DPTs were advanced to until firm soil was encountered, defined as more than 30 blows per 4-inch interval, at depths of about 7-1/2 to 10 feet bgs. The DPT results are presented on Figure A-17.

### 3.3 Laboratory Testing

We re-examined each soil sample obtained from our borings to confirm the field classifications and selected representative samples for laboratory testing. Soil samples were tested to measure moisture content, dry density, Atterberg limits (plasticity index), percent passing the No. 200 sieve, and corrosivity. The Atterberg limits test is an indirect measurement of the expansion potential of soil. The results of the geotechnical laboratory tests are presented on the boring logs and on Figure A-18. The corrosivity test results are presented in Appendix B.

### 3.4 Previous Investigations

We reviewed three geotechnical reports for previous projects at the site or in the immediate vicinity:

- *Geotechnical Investigation and Pavement Design, Proposed Industrial Development, 42021 Osgood Road, Fremont, California*, prepared by United Soil Engineering Inc. (USE) and dated 9 February 1998
- *Preliminary Geotechnical Investigation for 41829 & 41875 Osgood Road*, prepared by Rockridge Geotechnical Inc. and dated 23 April 2015
- *Geotechnical Investigation, Proposed Condominiums, Osgood Residences, 42111 & 42183 Osgood Road, Fremont, California,* prepared by Silicon Valley Soil Engineering (SVSE) and dated 9 June 2014

The borings were drilled to depths of between about 20 and 44 feet bgs. We also reviewed the results of four cone penetrations tests (CPTs) and four borings performed on neighboring parcels which extended to depths between about 21 and 60 feet bgs. Approximate locations of the borings and CPTs are shown on the Site Plan, Figure 2.

## 4.0 SUBSURFACE CONDITIONS

The Regional Geologic Map of the site and vicinity (Figure 3) indicates the site is located on the margin of two distinct geologic units mapped as Pleistocene-aged (11,000 to 5 million years before present) alluvial deposits (Qpa) and Holocene-aged (less than 11,000 years before present) alluvial deposits (Qha).

The results of our investigations indicate the site is generally underlain by alluvium that extends to the maximum depth explored of 60 feet bgs. A layer of fill up to about 5 feet thick was encountered at some locations and may be associated with the previous property development. In general, the Osgood III parcel is underlain by approximately 5 feet of weak fill and alluvium consisting of medium stiff clay and medium dense sand. The upper 5 feet of material at the Osgood II and IV parcels generally consists of stiff to very stiff sandy clay and medium dense to dense clayey sand. Below a depth of about 5 feet, the alluvium consists of stiff to hard clay and silt with variable amounts of sand interbedded with layers of dense to very dense sand and gravel. In general, the sand and gravel layers are less than 5 feet thick, though they may be up to 10 feet thick in some locations. The results of Atterberg limits tests performed on near-surface soil samples (clayey sand and sandy clay) indicate the soil has low to high expansion potential[2].

We understand groundwater was encountered in the direct-push boring performed onsite by Arcadis in March, 2017 at a depth of about 40 feet, which corresponds to approximately Elevation 25 feet.

The groundwater level at the site is expected to fluctuate several feet seasonally with potentially larger fluctuations annually, depending on the amount of rainfall. To estimate the highest potential groundwater level at the site, we reviewed information on the State of California Water Resources Control Board GeoTracker website (http://geotracker.swrcb.ca.gov). The three closest sites with groundwater data on the GeoTracker website are at 41100 Roberts Avenue, which is approximately 2,000 feet northwest of the site, 41482 Fremont Boulevard, which is approximately 1,800 feet west of the site, and 41980 Fremont Boulevard, which is approximately 1,800 feet southwest of the site. The highest recorded groundwater levels at each site are approximately 16 feet, 19 feet, and 21 feet at 41100 Roberts Avenue, 41482 Fremont Boulevard, and 41980 Fremont Boulevard, respectively.

2 Expansive soil undergoes volume changes with changes in moisture content.

Per the 2004 document titled *Seismic Hazard Zone Report for the Niles 7.5-Minute Quadrangle, Alameda County, California*, prepared by the California Geological Survey (CGS), the historic high groundwater level at the site is estimated to be approximately 30 feet bgs.

## 5.0 SEISMIC CONSIDERATIONS

### 5.1 Regional Seismicity

The site is located in the Coast Ranges geomorphic province that is characterized by northwest-southeast trending valleys and ridges. These are controlled by folds and faults that resulted from the collision of the Farallon and North American plates and subsequent shearing along the San Andreas fault system. Movements along this plate boundary in the Northern California region occur along right-lateral strike-slip faults of the San Andreas fault system.

The major active faults in the area are the Hayward, Calaveras, and San Andreas faults. For these and other active faults within a 50-kilometer radius of the site, the distance from the site and mean characteristic Moment magnitude[3] [Working Group on California Earthquake Probabilities (WGCEP) (2008) and Cao et al. (2003)] are summarized in Table 1.

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

**TABLE 1**
**Regional Faults and Seismicity**

| Fault Segment | Approximate Distance from Site (km) | Direction from Site | Mean Characteristic Moment Magnitude |
|---|---|---|---|
| Total Hayward | <1 | East | 7.00 |
| Total Hayward-Rodgers Creek | <1 | East | 7.33 |
| Total Calaveras | 9 | East | 7.03 |
| Monte Vista-Shannon | 25 | Southwest | 6.50 |
| Mount Diablo Thrust | 25 | Northeast | 6.7 |
| N. San Andreas - Peninsula | 29 | Southwest | 7.23 |
| N. San Andreas (1906 event) | 29 | Southwest | 8.05 |
| Greenville Connected | 30 | East | 7.00 |
| N. San Andreas - Santa Cruz | 39 | South | 7.12 |
| Green Valley Connected | 42 | North | 6.80 |
| Great Valley 7 | 44 | East | 6.90 |
| San Gregorio Connected | 46 | West | 7.50 |
| Zayante-Vergeles | 49 | South | 7.00 |

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 55 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 ($M_w = 6.2$).

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 to estimate the probability of fault segment rupture. They have determined that the overall probability of moment magnitude 6.7 or greater earthquake occurring in the San Francisco Region during the next 30 years (starting from 2014) is 72 percent. The highest probabilities are assigned to the Hayward fault, Calaveras fault, and the northern segment of the San Andreas fault. These probabilities are 14.3, 7.4, and 6.4 percent, respectively.

## 5.2 Seismic Hazards

The site has been mapped outside a zone of liquefaction potential on the map titled *State of California, Seismic Hazard Zones, Niles* Quadrangle, *Official Map,* prepared by the California Geological Survey (CGS), dated October 19, 2004 (Figure 5). However, 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,[4] lateral spreading,[5] and cyclic densification[6] at the site and the immediate vicinity using the available subsurface information.

### 5.2.1 Ground Shaking

The seismicity of the site is governed by the activity of the Hayward fault, although ground shaking from future earthquakes on other faults, including the San Andreas and Calaveras faults, will also be felt at the site. These and other faults in the region are shown in relation to the site on Figure 4. 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. Given the site's very close proximity to the mapped trace of the Hayward fault, we judge that very strong to violent ground shaking could occur at the site during a large earthquake on this fault.

### 5.2.2 Fault Rupture

The subject site lies in close proximity to an Earthquake Fault Zone for the Hayward fault, as defined by the Alquist-Priolo (AP) Earthquake Fault Zoning Act. To address the threat to structures from fault rupture, the AP Earthquake Fault Zoning Act (formerly known as the Alquist-Priolo Special Studies Zone) was signed into law in California in 1972. The act requires that a detailed geologic study be performed for new structures (intended for human occupancy) within defined earthquake fault zones to ensure that the structures are not built over active fault traces. Because historic ground surface displacements closely follow the trace of geologically young faults, the earthquake fault zones have been established along potentially and recently active fault traces. To account for imprecise locations of faults and the possible existence of

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

5 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.

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

active branches, zone boundaries are generally positioned about 500 feet on either side of major active faults.

We evaluated the location of the nearby mapped AP Zone relative to the site using digitized maps[7] available on the CGS website. As shown on Figure 6, the site is located approximately 10 feet outside of the Hayward fault AP Zone; however, a setback of this size is smaller than our precision to locate the AP Zone relative to the site. We can provide the electronic AP Zone files to the project civil engineer to determine the exact zone boundary relative to the site. Provided that the site is confirmed to be outside of the AP zone, or if the building footprint is appropriately setback, we conclude the requirements for a detailed fault study, which generally include exploratory trenching to check for evidence of recent faulting, will not apply to the proposed development.

Several detailed studies of the Hayward fault have been performed in close proximity to the subject site. Further information regarding these studies and the potential for future ground ruptures can be found in the following report:

> *Final Environmental Impact Statement and 4(f)/6(f) Evaluation BART Warm Springs Extension, Volume I, California,* prepared by *Federal Transit Administration U.S Department of Transportation and San Francisco Bay Area Rapid Transit District* California, dated May 16, 2006.

Considering there have been several fault studies performed in the site vicinity, the main trace of the Hayward fault has been clearly delineated in this area, and no active splays from this section of the fault have been documented (to our knowledge), we conclude the potential for fault rupture at the site (i.e., outside of an AP Zone) is low.

### 5.2.3 Liquefaction and Associated Hazards

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

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[7] California State Department of Conservation, Division of Mines and Geology (2001). *GIS files of Official Maps of Alquist-Priolo Earthquake Fault Zones, Central Coastal Region, Earthquake Fault Zone Map of the Niles Quadrangle*. DMG CD 2001-04, File Name: NILES_AP

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

We evaluated the liquefaction potential of soil encountered below groundwater at the site and site vicinity using the available nearby CPT data and results of the exploratory boring. Our analyses indicate the soil below the groundwater table has sufficient cohesion and/or relative density to resist liquefaction and, therefore, we conclude the potential for liquefaction to occur at the site is very low. We also conclude lateral spreading resulting from liquefaction is also 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 soil encountered above the groundwater table is not susceptible to cyclic densification due to its cohesion and relative density. Therefore, we conclude the potential for cyclic densification to occur at the site is low.

## 6.0 DISCUSSIONS AND CONCLUSIONS

From a geotechnical standpoint, we conclude the site can be developed as planned, provided the recommendations presented in this report are incorporated into the project plans and specifications and implemented during construction. The primary geotechnical issues affecting the proposed development include the potential for strong ground shaking at the site, the presence of surficial undocumented fill in portions of the site, and providing adequate vertical and lateral foundation support. These and other geotechnical issues are addressed in the remainder of this section.

### 6.1 Foundation Support and Settlement

Based on the results of our investigation, we anticipate up to about 6 feet of undocumented fill and/or weak alluvial deposits may be present across the Osgood III parcel and in isolated locations in the Osgood II and IV parcels. Below this depth, the alluvial soil has moderate to high strength and low to moderate compressibility. We conclude the proposed structures can be supported on conventional spread footings bearing on firm native soil and/or engineered (recompacted) fill. Therefore, we conclude the proposed apartment buildings should include foundations that gain support on native, undisturbed soil below the fill. This can be achieved by deepening the structural footings, or alternatively, footing excavations may be over-excavated down to competent native material and backfilled with controlled density fill (CDF) or sand-cement slurry up to the design bottom-of-footing elevation. The CDF would serve to transfer footing loads to more firm soils and prevent the need for extending reinforced structural concrete down to the native material.

An additional alternative to the deepened footing and CDF options presented above, would be to over-excavate the undocumented fill across the Osgood III parcel (and portions of the Osgood II and IV parcels, where identified in the field) during mass grading of the building pads and recompact the material as an engineered fill. This would eliminate the need for deepening the foundations below the fill—however, the perimeter foundations will still need to be deepened to mitigate the effects of expansive soil, as discussed in more detail in the following section.

Our settlement analyses indicate total and differential settlement of conventional footings bearing on undisturbed native soil or properly engineered fill, designed using the allowable bearing pressures presented in Section 7.2 of this report, will be less than 1-1/2 inch and on the order of a 3/4 inch over a 30-foot horizontal distance, respectively.

### 6.2 Expansive Soil

Atterberg limits tests performed on samples of the near-surface clay indicate the surficial soil has expansion potential which varies from low to high. Expansive near-surface soil is subject to volume changes during fluctuations in moisture content. These volume changes can cause movement and cracking of foundations, pavements, slabs, and below-grade walls. Therefore, foundations, pavements, slabs, and below-grade walls should be designed and constructed to resist the effects of the expansive soil.

In general, the effects of expansive soil can be mitigated by moisture-conditioning the expansive soil, providing select, non-expansive fill or lime-treated soil below interior and exterior slabs and behind retaining walls, and either supporting foundations below the zone of severe moisture change or by providing a stiff, shallow foundation that can limit deformation of the superstructure as the underlying soil shrinks and swells. We conclude the proposed building should include a deepened continuous perimeter footing to help control the potential for long-term moisture change beneath the building.

To prevent the soil subgrade beneath the building and garage slabs-on-grade from drying during construction and to reduce the long-term effects of expansive subgrade soil, a minimum of 12 inches of non-expansive fill should be placed on the prepared subgrade. The non-expansive fill may consist of imported select fill material. Alternatively, the upper 12 inches of slab subgrade may be treated in place with lime and/or cement to reduce its expansion potential.

### 6.3 Construction Considerations

The soil to be excavated predominantly consists of sandy clay and clayey sand, which can be excavated with conventional earth-moving equipment such as loaders and backhoes. If the site grading is performed during the rainy season, the near-surface clay will likely be wet and will have to be dried before compaction can be achieved. Heavy rubber-tired equipment, such as haul trucks, scrapers, and vibratory rollers, could cause excessive deflection (pumping) of the wet clay and therefore should be avoided if this condition occurs. If the project schedule or weather conditions do not permit sufficient time for drying of the soil by aeration, the subgrade can be treated with lime and/or cement prior to compaction to create a stable "winterized" subgrade. It is also important that the moisture content of subgrade soil is sufficiently high to reduce the future expansion potential. If the grading work is performed during the dry season, moisture-conditioning may be required.

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.

### 6.4 Soil Corrosivity

Corrosivity testing was performed by Project X Corrosion Engineering of Murrieta, California on a sample of near-surface soil obtained during our field investigation from boring B-5 at a depth of about 3 feet bgs. The results of the test are presented in Appendix B of this report. The results of the corrosivity analyses indicate the sample is "moderately corrosive" with respect to resistivity. Accordingly, all buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric-coated steel or iron may need to be protected against corrosion, depending upon the critical nature of the structure. *If it is necessary to have metal in contact with soil, a corrosion engineer should be consulted to provide recommendations for corrosion protection.* The results indicate that sulfate ion concentrations are sufficiently low to not pose a threat to buried concrete. The chloride ion concentrations are considered "mildly corrosive" to steel reinforcement in concrete structures below ground.

## 7.0 RECOMMENDATIONS

In accordance with our scope of services, the remainder of this report presents our recommendations for site preparation and grading, foundation support, concrete flatwork, pavements, and seismic design.

### 7.1 Site Preparation and Grading

Site demolition should include the removal of all existing pavements, underground utilities, and foundations. 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 are outside of the proposed building footprints and 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 should be properly backfilled with compacted fill under our direction following the recommendations provided later in this section. Demolished asphalt concrete should be taken to an asphalt recycling facility. Aggregate base beneath existing pavements may be re-used as select fill if carefully segregated.

In areas that will receive fill or new improvements (i.e. building pads, concrete flatwork, pavements, etc.), the soil subgrade exposed should be scarified to a depth of at least 12 inches, moisture-conditioned, and compacted in accordance with the relative compaction[8] requirements presented in Table 2. Note that "moisture-conditioning" may require wetting or drying of the soil, depending on the particular conditions encountered at the time of construction. If zones of soft and/or loose undocumented fill are encountered during site grading, the fill should be over-excavated under the observation of our field engineer and replaced as a properly compacted fill. If the over-excavation and re-compaction option is selected to mitigate the undocumented fill blanketing the majority of the site, as discussed in Section 6.1, the material should be removed down to undisturbed native material under the direction of our field engineer, moisture-

[8] 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.

conditioned, placed in 8-inch-thick lifts, and compacted in accordance with the compaction requirements presented in Table 2. The building pads should be graded to accommodate a minimum 12 inches of select fill of lime-/cement-treated on-site soil. Areas outside the building footprints that will receive exterior concrete flatwork should be graded to accommodate a minimum of 6 inches of imported select fill.

All fill should be free of organic matter and contain no rocks or lumps larger than three inches in greatest dimension. 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.

All fill should be placed in horizontal lifts not exceeding eight inches in uncompacted thickness, moisture-conditioned, and compacted in accordance with the requirements provided below in Table 2.

**TABLE 2**
**Summary of Compaction Requirements**

| Location | Required Relative Compaction (percent) | Moisture Requirement |
|---|---|---|
| Garage slab subgrade – select fill (top 12 inches) | 95+ | Above optimum |
| General fill – expansive clay | 88 – 92 | 3+% above optimum |
| General fill – non-expansive (less than 5 feet thick) | 90+ | Above optimum |
| General fill – non-expansive (more than 5 feet thick) | 95+ | Above optimum |
| Utility trench backfill – expansive clay | 88 – 92 | 3+% above optimum |
| Utility trench backfill – non-expansive | 90+ | Above optimum |
| Utility trench - clean sand or gravel | 95+ | Near optimum |
| Exterior slabs – expansive clay | 88 – 92 | 3+% above optimum |
| Exterior slabs – low-plasticity | 90+ | Above optimum |
| Exterior slabs – select fill | 90+ | Above optimum |
| Pavement subgrade – expansive clay | 92+ | 2+% above optimum |
| Pavement subgrade – non-expansive | 95+ | Above optimum |
| Pavement - aggregate base | 95+ | Near optimum |

***Select Fill***

Select fill should consist of imported or on-site soil that is 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.

***Aggregate Base Material***

Imported aggregate base material may be used as select fill, trench backfill (above bedding materials), or as select fill beneath pavements, exterior concrete flatwork, or the garage slab.

Aggregate base beneath pavements should meet the requirements in the 2015 Caltrans Standard Specifications, Section 26, for Class 2 Aggregate Base (3/4 inch maximum).

***Controlled Low Strength Material***

Controlled low strength material (CLSM) may be considered as an alternative to fill beneath structures or pavement.  CLSM should meet the requirements in the 2015 Caltrans Standard Specifications.  It is an ideal backfill material when adequate room is limited or not available for conventional compaction equipment, or when settlement of the backfill must be minimized.  No compaction is required to place CLSM.  CLSM should have a minimum 28-day unconfined compressive strength of at least 100 pounds per square inch (psi).  CLSM should be used for the required impermeable trench plugs, as described in Section 7.1.2 and Figure 7.

### 7.1.1 Exterior Concrete Flatwork

Exterior concrete flatwork that will not receive vehicular traffic (i.e. sidewalks, patios, etc.) should be underlain by at least six inches of select fill compacted to at least 90 percent relative compaction.  Prior to placement of the select fill, the upper 8 inches of the subgrade soil should be scarified, moisture-conditioned to above optimum moisture content, and compacted in accordance with the requirements presented in Table 2.  This grading should be performed under the observation of our field engineer.  If zones of weak or loose soil that extend deeper than the upper 8 inches are encountered during grading, the material should be over-excavated down to firm material, as determined by our field engineer, and replaced with engineered fill.

Even with six inches of select fill, exterior slabs may experience some cracking due to shrinking and swelling of the underlying expansive soil.  Thickening the slab edges and adding additional reinforcement will control this cracking to some degree.  In addition, where slabs provide access to buildings, it would be prudent to dowel the entrance to the building to permit rotation of the slab as the exterior ground shrinks and swells and to prevent a vertical offset at the entries.

### 7.1.2 Utility Trench Backfill

Excavations for utility trenches can be readily made with a backhoe. All temporary excavations used in construction should be designed, planned, constructed, and maintained by the contractor and should conform to the current CAL-OSHA requirements. To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of clean sand or fine gravel. After the pipes and conduits are tested, inspected (if required) and approved, they should be covered to a depth of six inches with clean sand or fine gravel, which should be mechanically tamped. Backfill for utility trenches and other excavations is also considered fill, and should be placed and compacted according to the recommendations presented in Table 2. If imported clean sand or gravel (defined as poorly graded soil with less than 5 percent fines) is used as backfill, it should be compacted to at least 95 percent relative compaction. Jetting of trench backfill should not be permitted. Special care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to the pavement section.

Impermeable plugs consisting of sand-cement slurry, at least three feet in length, should be installed in lieu of sand or fine gravel pipe bedding where utility trenches enter the building footprint. A typical detail for the recommended utility trench low-permeability plug at building perimeters is presented on Figure 7. The purpose of this recommendation is to reduce the potential for water to become trapped in trenches beneath the building. This trapped water can cause heaving of soils beneath the building slab.

Underground utility trenches that are to be excavated within the zone-of-influence of the building foundation should be backfilled with CLSM (see Section 7.1 for material requirements) below a depth defined by an imaginary line extending down from the bottom of the foundation at an inclination of 1.5:1 (horizontal:vertical). We highly recommend installing these utilities and backfilling trenches prior to pouring the building foundations. If utility trenches are to be excavated below this zone-of-influence line after construction of the building foundations, the trench walls need to be fully supported with shoring until CLSM is placed.

### 7.1.3 Drainage and Landscaping

Positive surface drainage should be provided around the building to direct surface water away from the foundations. To reduce the potential for water ponding adjacent to the buildings, we recommend the ground surface within a horizontal distance of five feet from the building slope down away from the building with a surface gradient of at least two percent in unpaved areas and one percent in paved areas. In addition, roof downspouts should be discharged into controlled drainage facilities to keep the water away from the foundations. The use of water-intensive landscaping around the perimeter of the building should be avoided to reduce the amount of water introduced to the near-surface clay.

Care should be taken to minimize the potential for subsurface water to collect beneath non-permeable pavements and pedestrian walkways. Where landscape beds and tree wells are immediately adjacent to pavements and flatwork which are not designed as permeable systems, we recommend vertical cutoff barriers be incorporated into the design to prevent irrigation water from saturating the subgrade and AB. These barriers may consist of either flexible impermeable membranes or deepened concrete curbs.

Storm water treatment systems (infiltration basins, rain gardens, bio-retention systems, vegetated swales, flow-through planters, etc.) constructed at the site should be provided with underdrains, as well as impermeable liners where they will be within 10 feet of the building or site retaining walls. Due to the low permeability of the upper clay, these systems should not be designed for exfiltration in to the subgrade soil near structural improvements. The drainage layer beneath the "treatment" soil should consist of a minimum 12-inch-thick layer of Caltrans Class 2 Permeable drainage material and include a minimum 4-inch-diameter perforated drain pipe placed with the perforations facing downward. An impermeable liner consisting of a high density polyethylene liner (or equivalent) that is at least 15 mils thick should line the entire bottom and sides of the system, where required. In locations where treatment basins are not in close proximity to foundations, retaining walls, concrete flatwork, or pavements, the impermeable liner may be omitted; however, these should be assessed on a case-by-case basis.

### 7.1.4 Fill Criteria for Permeable Pavements

We understand that site grades will be raised an average of about 5 feet prior to paving. Therefore, any permeable/pervious pavements will discharge into the fill. Based on information provided by BKF, the subgrade below the pavement sections requires an infiltration rate of at least 0.013 inches per hour (unfactored) for the pervious pavement to perform as designed.

Estimating infiltration rates of soil types is an approximate exercise. For preliminary screening of potential imported fill materials, we recommend considering soil types with the United Soil Classification System (USCS) designations as outlined in Table 3.

**TABLE 3**
**Appropriate Fill Types USCS Designation**

| Soil Type (Abbreviation) | Fines Content (%) | Plasticity Index Range | Likelihood of Satisfying Infiltration Requirements |
|---|---|---|---|
| Clean Sand (SP, SW)<br>Clean Gravel (GP, GW) | 0-12 | NP | Very High |
| Silty Sand (SM)<br>Silty Gravel (GM) | 12-49 | NP-3 | High |
| Clayey Sand (SC)<br>Clayey Gravel (GC) | 12-49 | >7 | Moderate |
| Silt (ML)<br>Clay (CL) | >50 | 0-22<br>>7 | Low |
| High Plasticity Clay (CH)<br>High Plasticity Silt (MH)<br>Organic Soil (OH, OL, PT) | - | - | Not appropriate |

If the proposed fill has a fines content of 12% or more (i.e., not clean sand or gravel), laboratory permeability testing should be performed on a sample of the proposed material under our direction prior to importing to the site. We can perform additional field infiltration tests on the placed fill, as required by the permitting entity.

### 7.2 Foundations

Provided the estimated total and differential settlements presented in Section 6.1 are acceptable, the proposed apartment buildings may be supported on spread footings that derive support on native soil below the undocumented fill blanketing the site. This can be achieved by bottoming the footings below the existing fill, which we estimate extends up to 6 feet below existing grades (bgs) in some portions of the site. Alternatively, foundation support may be deepened to the native soil by over-excavating the undocumented fill and placing CDF up to the design bottom-of-footing elevation. An additional alternative would be to over-excavate the undocumented fill and replace it as an engineered fill under our engineer's observation during rough grading of the site.

The foundation system should include a continuous perimeter footing bottomed at least 30 inches below the outside grade. Interior isolated spread footings should be bottomed at least 24 inches below the adjacent soil subgrade (top of select fill and bottom of capillary break, where included). Footings to be constructed near underground utilities, underslab vaults, or stormwater treatment features should be bottomed below an imaginary line extending up at an inclination of 1.5:1 (horizontal:vertical) from the bottom of the utility trench or vault. The footings may be designed using allowable bearing pressures of 4,000 psf for dead-plus-live loads and 5,300 psf for total design loads, which include wind or seismic forces.

Lateral loads on shallow footings may be resisted by a combination of passive pressure acting on the vertical faces of the footings and friction between the bottoms of the footings and the supporting soil. To compute passive resistance for transient loading, we recommend using an allowable uniform pressure of 1,800 psf (rectangular distribution). To compute passive resistance for sustained lateral loads, we recommend using an equivalent fluid weight (triangular distribution) of 300 pounds per cubic foot (pcf). The upper foot of soil should be ignored unless confined by a slab or pavement. Frictional resistance should be computed using a base friction coefficient of 0.3. The passive pressure and frictional resistance values include a factor of safety of at least 1.5 and may be used in combination without reduction.

We should check footing excavations prior to placement of reinforcing steel. Foundation excavations should be free of standing water, debris, undocumented fill, and disturbed materials prior to placing concrete. If footings are excavated during the rainy season, they may need to incorporate a mud slab to protect the footing subgrade. This will involve over-excavating the footing by about 3 inches and placing lean concrete or sand-cement slurry in the bottom (following inspection by our engineer). A mud slab will help protect the footing subgrade during placement of reinforcing steel. Water can then be pumped from the excavations prior to placement of structural concrete, if present. The bottoms and sides of the footing excavations should be moistened following excavation and maintained in a moist condition until concrete is placed.

### 7.3 Floor Slabs

The proposed building floor slabs may consist of a conventional slabs-on-grade, provided they are underlain by a minimum 12-inch-thick layer of select fill, such as Caltrans Class 2 aggregate base (AB). Where water vapor transmission through the slabs is considered detrimental, we recommend installing a capillary moisture break and water vapor retarder beneath the floor slab (above the 12-inch-thick select fill layer). A vapor retarder and capillary moisture break are generally not required beneath parking garage floor slabs because there is sufficient air circulation to allow evaporation of moisture that is transmitted through the slab; however, we recommend the vapor retarder and capillary break be installed below the slabs-on-grade in finished spaces, utility rooms, and any areas in or adjacent to the parking garage that will be used for storage and/or will receive a floor covering or coating.

A capillary moisture break consists of at least four inches of clean, free-draining gravel or crushed rock. The vapor retarder should meet the requirements for Class B vapor retarders stated in ASTM E1745. The vapor retarder should be placed in accordance with the requirements of ASTM E1643. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder.

The particle size of the capillary break material should meet the gradation requirements presented in Table 4.

**TABLE 4**
**Gradation Requirements for Capillary Moisture Break**

| Sieve Size | Percentage Passing Sieve |
|---|---|
| *Gravel or Crushed Rock* | |
| 1 inch | 90 – 100 |
| 3/4 inch | 30 – 100 |
| 1/2 inch | 5 – 25 |
| 3/8 inch | 0 – 6 |

The slabs 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 slabs should have a low w/c ratio—less than 0.45—and water not be added in the field. If necessary, workability should be increased by adding plasticizers. 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.

### 7.4 Pavement Design

For pavement design, we assumed a resistance value (R-value) of 5, which is appropriate for the on-site expansive clay and a conservative design assumption for imported fill. We understand the City of Fremont requires pavement to have a minimum design life of 30 years. Based on information provided by BKF (project civil engineer), we understand that a TI of 5.5 is sufficient for a 30 year design life when considering the anticipated number of equivalent single axel loads (ESALs). This calculation is included in Appendix B. We can provide additional pavement sections for different TIs or R-values (once proposed fill has been sourced), if needed.

### 7.4.1 General Pavement Recommendations

The upper 8 inches of the subgrade should be moisture-conditioned and compacted in accordance with requirements presented in Table 2 in Section 7.1. The subgrade should be proof-rolled under the observation of our field engineer to confirm it is non-yielding prior to placement of the impermeable liner (where used) and/or base course(s).

The aggregate base and permeable base materials should be moisture-conditioned to near optimum moisture content and compacted to at least 90 and 95 percent relative compaction for pedestrian and vehicular applications, respectively. All base courses should also be proof-rolled under the observation of our field engineer to confirm they are non-yielding prior to paving.

If non-permeable pavements are adjacent to irrigated landscaped areas, bioswales, or permeable/pervious pavements, curbs adjacent to those areas should extend through the aggregate base and at least three inches into the underlying soil to reduce the potential for irrigation water to infiltrate into the non-permeable pavement sections. Further, deepened curbs near bioswales may require some type of lateral restraint. The need for lateral restraint of deepened curbs should be evaluated during final design of the biotreatment features.

### 7.4.2 Non-Permeable Flexible (Asphalt Concrete) Pavement Design

The State of California flexible pavement design method was used to develop the recommended asphalt concrete pavement sections. Recommended pavement sections for traffic indices (TIs) ranging from 4.5 to 6.5 are presented in Table 5.

**TABLE 5**
**Non-Permeable AC Pavement Sections**

| TI | Asphaltic Concrete (inches) | Class 2 Aggregate Base R = 78 (inches) |
|---|---|---|
| 4.5 | 2.5 | 9.5 |
| 5.0 | 3.0 | 10.0 |
| 5.5 | 3.0 | 12.0 |
| 6.0 | 3.5 | 13.0 |
| 6.5 | 4.0 | 13.5 |

#### 7.4.3 Non-Permeable Rigid (Portland Cement Concrete) Pavement

Concrete pavement design is based on a maximum single-axle load of 20,000 pounds and a maximum tandem axle load of 34,000 pounds and moderate truck traffic (i.e., several trucks per week). The recommended rigid pavement section for these axle loads is 6.5 inches of Portland cement concrete (PCC) over six inches of Class 2 aggregate base. For areas that will receive fire truck traffic, the PCC thickness should be increased to 7.0 inches. For areas that will experience only passenger vehicle traffic, the recommended pavement section is five inches of PCC over six inches of Class 2 aggregate base.

The modulus of rupture and unconfined compressive strength of the concrete should be at least 500 and 3,200 pounds per square inch (psi) at 28 days, respectively. Contraction joints should be placed at maximum spacing of 15 feet. Where the outer edge of a concrete pavement meets asphalt pavement, the concrete slab should be thickened by 50 percent at a taper not to exceed a slope of 1 in 10. The pavement should be reinforced with a minimum of No. 4 bars at 18 inches on center in both directions. Recommendations for subgrade preparation and aggregate base compaction for concrete pavement are the same as those we have described above for asphalt concrete pavement.

### 7.4.4 Permeable Interlocking Concrete Pavers

We recommend permeable interlocking concrete pavements (ICP) be designed in accordance with the guidelines presented by the Interlocking Concrete Pavement Institute (ICPI, 2005). These guidelines include specific recommendations for permeable aggregate subbase, base, and bedding courses to be placed beneath ICP pavements.

ICPI's guidelines call for 2 inches of bedding material consisting of ASTM No. 8 crushed aggregate directly below the pavers. This material is also recommended for fill material between the pavers. The ASTM No. 8 bedding should be underlain by a permeable base course of ASTM No. 57 crushed aggregate. The ASTM No. 57 permeable base course should be underlain by a permeable subbase course of ASTM No. 2 or 57 crushed aggregate. Gradation requirements for various ASTM aggregates are presented in Table 6.

**TABLE 6**
**Gradation Requirements for Various ASTM Crushed Aggregates Beneath Permeable Concrete Pavers**

| Sieve Size | Percentage Passing Sieve | | |
|---|---|---|---|
| | ASTM No. 8 Crushed Aggregate | ASTM No. 57 Crushed Aggregate | ASTM No. 2 Crushed Aggregate |
| 3 inch | - | - | 100 |
| 2-1/2 inch | - | - | 90-100 |
| 2 inch | - | - | 35-70 |
| 1-1/2 inch | - | 100 | 0-15 |
| 1 inch | - | 95 – 100 | - |
| 3/4 inch | - | - | 0-5 |
| 1/2 inch | 100 | 25 – 60 | - |
| 3/8 inch | 85 – 100 | - | - |
| No. 4 | 10 – 30 | 0 – 10 | - |
| No. 8 | 0 – 10 | 0 – 5 | - |
| No. 16 | 0 – 5 | - | - |

A geotextile filter fabric (Mirafi NC or equivalent) should be placed on the subgrade prior to placing the base course of aggregate. The No. 2 crushed aggregate subbase course should be placed in lifts not exceeding 6 inches in loose thickness and compacted using a smooth-drum roller that weighs a minimum of 10 tons, operated in static (non-vibratory) mode. The subsequent course of No. 57 crushed aggregate may be placed in one lift and should be compacted with a smooth-drum roller in vibratory mode with sufficient passes to create an unyielding surface. Placement and compaction of the permeable aggregate base and subbase should be performed under the observation of our field engineer. Following compaction of the No. 57 aggregate, the No. 8 bedding, not exceeding 2 inches in loose thickness, should be placed and screeded to a level, undisturbed surface immediately prior to paver installation.

The required thicknesses of the permeable aggregate base and subbase courses depends on the water storage design requirements, as well as the traffic loading demand. Our recommendations for the minimum permeable ICP pavement sections subject to the proposed vehicular traffic are presented in Table 7. Recommended section for permeable ICPs only subject to pedestrian traffic are also provided.

**TABLE 7**
**Recommended Pavement Sections for**
**Permeable Interlocking Concrete Pavers**

| Pavement Type | Leveling Course (ASTM No. 8) (inches) | Base Course (ASTM No. 57) (inches) | Reservoir / Sub-Base Course (ASTM No. 2) (inches) |
|---|---|---|---|
| Pedestrian | 2.0 | 4.0 | 6.0 |
| Vehicular | 2.0 | 4.0 | 10.0 |

The above recommended ICP pavement sections are based on the ICPI technical guidelines (ICPI, 2005). From a geotechnical standpoint, it is acceptable to design the pedestrian ICP section to exclude the No. 2 subbase course, in which case the No. 57 base course should be increased to 10 inches.

### 7.4.5 Permeable Asphalt Pavement

Our recommendations for the minimum permeable asphalt pavement sections subject to the proposed vehicular traffic are presented in Table 8.

**TABLE 8**
**Recommended Pavement Sections for**
**Permeable Asphalt Pavement**

| Pavement Loading | Porous Asphalt (inches) | Base Course (ASTM No. 57) (inches) | Reservoir / Sub-Base Course (ASTM No. 2 or No. 57) (inches) |
|---|---|---|---|
| Pedestrian | 4.0 | 4.0 | 8.0 |
| Vehicular (TI $\leq$ 6.5) | 4.0 | 4.0 | 11.0 |

The above recommended asphalt pavement sections are based on the guidelines prepared by the San Francisco Public Utility Commission (2016), which references the AASHTO "Guide for Design of Pavement Structures". A geotextile filter fabric (Mirafi NC or equivalent) should be placed on the subgrade prior to placing the base course of aggregate. The aggregate courses should be placed and compacted as outlined in Section 7.4.4.

### 7.5 Seismic Design

We anticipate that the proposed buildings will be designed under the 2019 version of the California Building Code (CBC), which is based on the guidelines contained within *ASCE 7-16*. The latitude and longitude of the site are 37.5284º and -121.9520º, respectively. Measured shear wave velocities at the site are consistent with Site Class D classification. Therefore, we recommend Site Class D be used for structural design of the buildings.

Assuming the seismic response coefficient ($C_s$) value will be calculated as outlined in *ASCE 7-16*, *Section 11.4.8*, *Exception 2*, we recommend the following seismic design parameters:

- $S_S$ = 2.209g, $S_1$ = 0.852g
- $F_a$ = 1.0, $F_v$ = 1.7
- $S_{MS}$ = 2.209g, $S_{M1}$ = 1.448g
- $S_{DS}$ = 1.473g, $S_{D1}$ = 0.966g
- Seismic Design Category E for Risk Factors I, II, and III

In lieu of adjusting the $C_s$ value, our scope can be revised to include a site-specific ground motion hazard analysis, if desired from a structural engineering standpoint.

## 8.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

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

## 9.0 LIMITATIONS

This geotechnical consultation 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 exploration locations. 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


0 1,000 2,000 Feet

Approximate scale

Base map: Google Map, 2016

| **OSGOOD II - IV**<br>Fremont, California | **SITE LOCATION MAP** | | |
|---|---|---|---|
| ROCKRIDGE GEOTECHNICAL | Date 06/12/18 | Project No. 17-1297 | Figure 1 |


Base map: Google Earth, 2016.

**EXPLANATION**

B-1 Approximate location of boring by Rockridge Geotechnical Inc., March and June, 2018

HA-2 DPT-2 Approximate location of hand auger boring and dynamic pentrometer test by Rockridge Geotechnical Inc., April, 2018

CPT-1 Approximate location of cone penetration test by Rockridge Geotechnical Inc., February 2015

S-2 Approximate location of borings by Silicon Valley Soil Engineering, June 2014

U-2 Approximate location of boring by United Soil Engineering, February 1998

Project limits

0 80 Feet

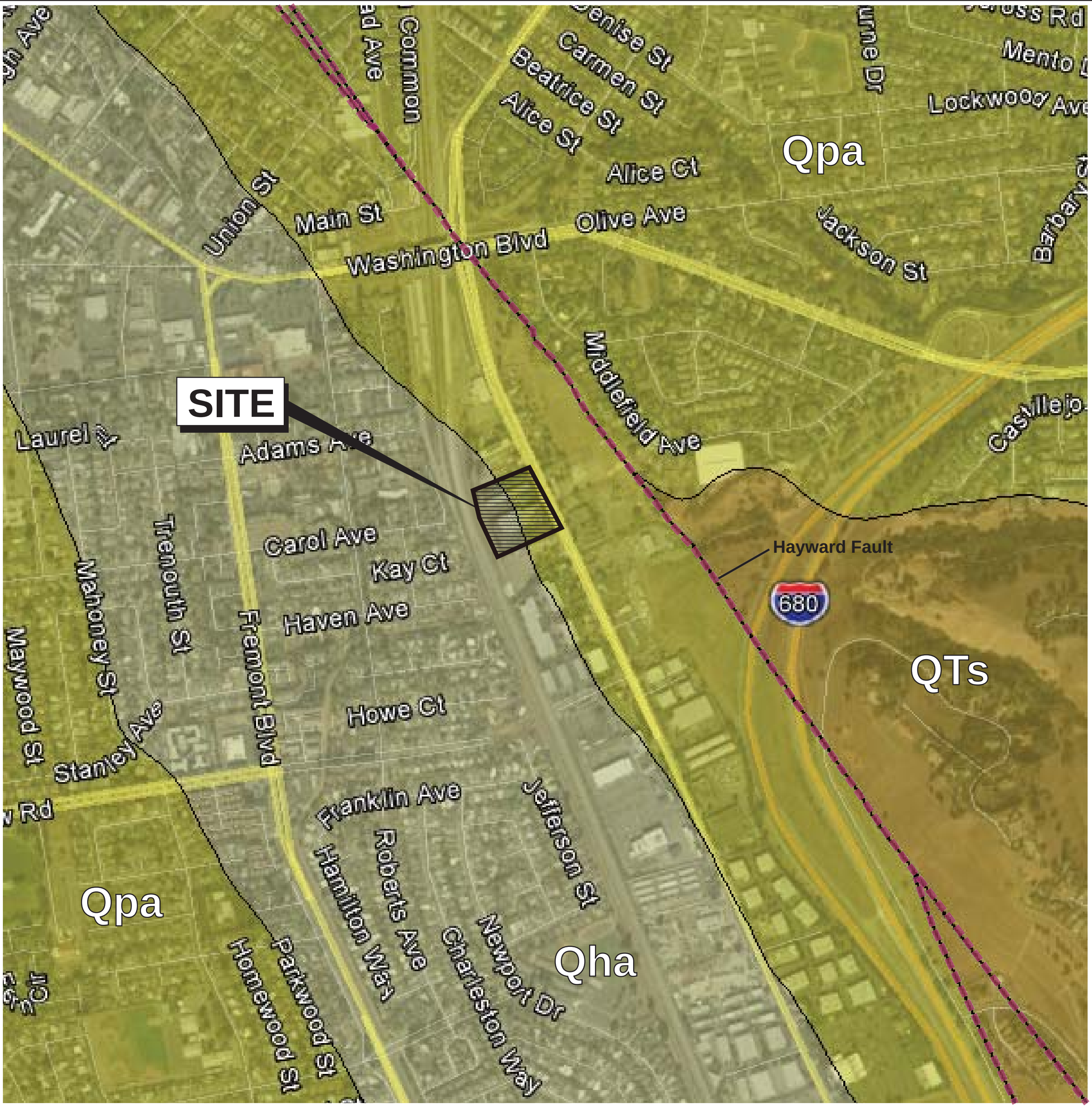
Approximate scale

**OSGOOD II - IV**
Fremont, California

**SITE PLAN**

| Date 06/12/18 | Project No. 17-1297 | Figure 2 |
|---|---|---|

ROCKRIDGE GEOTECHNICAL




**EXPLANATION**

| | |
|---|---|
| Qha | Alluvium (Holocene) |
| Qpa | Alluvium (Pleistocene) |
| QTs | Sediments (early Pleistocene and (or) Pliocene) |
| | Geologic contact |
| | Fault |

Base map: Google Earth, 2016
Geologic map: Graymer et al. 2006

N

0 1000 2000 Feet

Approximate scale

**OSGOOD II - IV**
Fremont, California

ROCKRIDGE GEOTECHNICAL

**REGIONAL GEOLOGIC MAP**

Date 06/12/18 | Project No. 17-1297 | Figure 3


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

**EXPLANATION**

Strike slip

Thrust (Reverse)

Normal

0 5 10 Miles

Approximate scale

**OSGOOD II - IV**
Fremont, California

ROCKRIDGE GEOTECHNICAL

**REGIONAL FAULT MAP**

| Date 06/11/18 | Project No. 17-1297 | Figure 4 |
|---|---|---|


**EXPLANATION**

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

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

0 1,500 3,000 Feet

Approximate scale

Reference:
State of California "Seismic Hazard Zones"
Niles Quadrangle
Released on October 19, 2004

**OSGOOD II - IV**
Fremont, California

**REGIONAL SEISMIC HAZARDS MAP**

ROCKRIDGE GEOTECHNICAL

Date 06/11/18 | Project No. 17-1297 | Figure 5


Reference: Division of Mines and Geology (2001).
Base map: ESRI Digitalglobe.

**EXPLANATION**

Approximate Site Limits

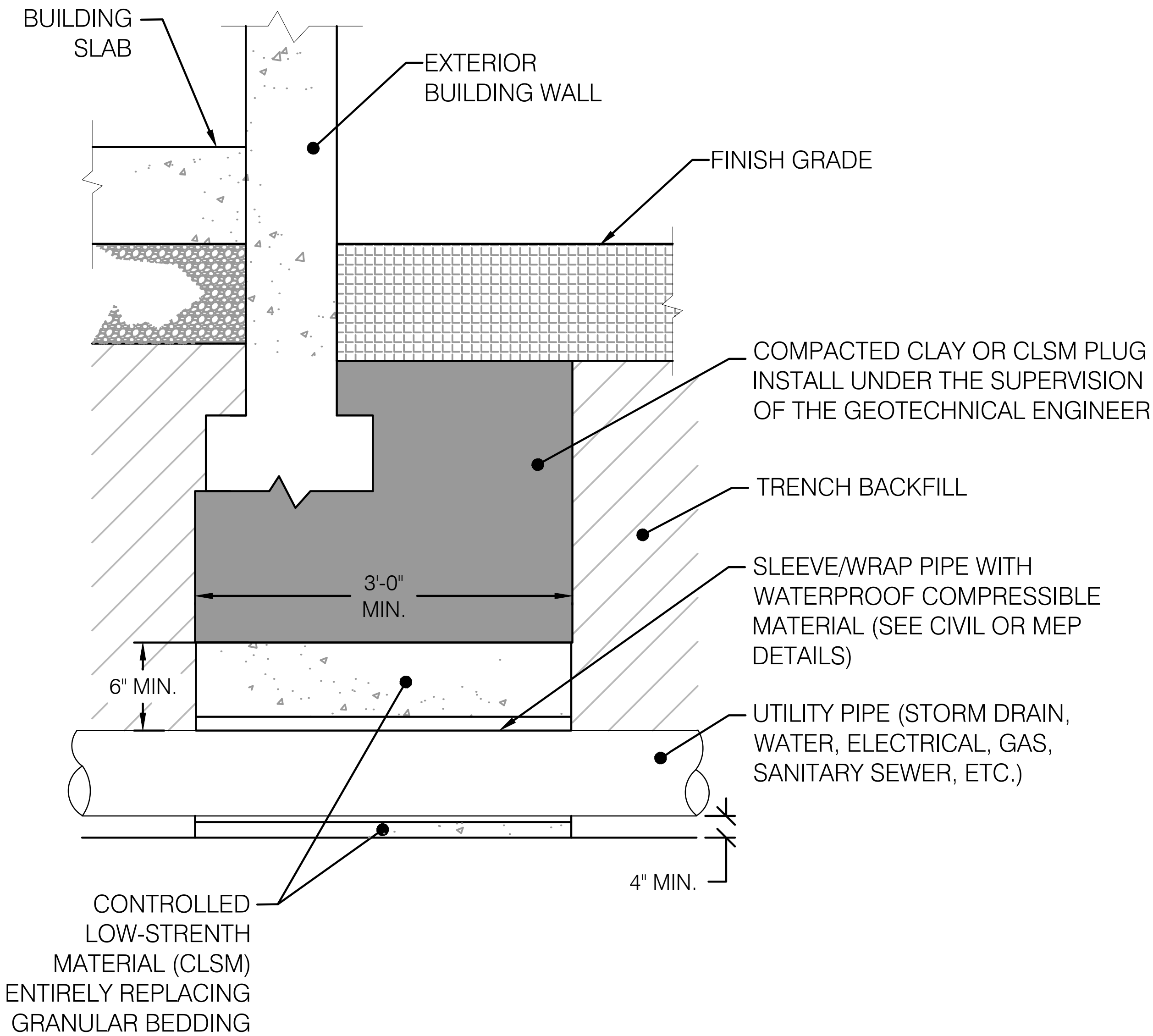
0 150 Feet
Approximate scale

**OSGOOD II - IV**
Fremont, California

ROCKRIDGE GEOTECHNICAL

**ALQUIST PRIOLO ZONE**

Date 07/31/18 | Project No. 17-1297 | Figure 6




Not to Scale

**OSGOOD II - IV**
Fremont, California

ROCKRIDGE GEOTECHNICAL

**UTILITY TRENCH LOW-PERMEABILITY PLUG AT BUILDING PERIMETER**

| Date 07/31/18 | Project No. 17-1297 | Figure 7 |
|---|---|---|

**APPENDIX A**

Logs of Borings, DPT Results, and Laboratory Test Results

| PROJECT: | OSGOOD II - IV<br>Fremont, California | **Log of Boring B-1**<br>PAGE 1 OF 1 |
|---|---|---|

| | |
|---|---|
| Boring location: | See Site Plan, Figure 2 |
| Date started: | 3/22/18 — Date finished: 3/22/18 |
| Drilling method: | Hollow Stem Auger |
| Hammer weight/drop: | 140 lbs./30 inches — Hammer type: Downhole Wireline |
| Sampler: | Sprague & Henwood (S&H) |
| Logged by: | W. Gozali |
| Drilled by: | Exploration Geoservices |
| Rig: | Mobile B-53 Red |

LABORATORY TEST DATA

| DEPTH (feet) | Sampler Type | 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 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | 4 inches of asphalt concrete | | | | | | |
| | | | | | 4 inches of aggregate base | | | | | | |
| 1–2 | S&H | 17 27 30 | 34 | SC | CLAYEY SAND with GRAVEL (SC) dark gray-brown, dense, moist, fine- to coarse-grained sand, fine gravel | | | | | 9.8 | 126 |
| 4–5 | S&H | 19 23 26 | 29 | SC | medium dense to dense | | | | | | |
| 6–7 | S&H | 6 7 10 | 10 | CL | SANDY CLAY (CL) dark brown, stiff, moist, fine-grained sand | PP | | 3,500 | | 13.8 | 111 |
| 9–10 | S&H | 12 12 17 | 17 | CL | brown, very stiff | PP | | >4,000 | | | |
| 12 | | | | CL | CLAY with SAND (CL) yellow-brown, hard, moist, fine-grained sand | | | | | | |
| 14–15 | S&H | 14 17 35 | 31 | CL | | | | | | | |

Boring terminated at a depth of 15 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H blow counts for the last two increments were converted to SPT N-Values using a factor of 0.6, to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 17-1297 Figure: A-1

# Log of Boring B-2

PROJECT: **OSGOOD II - IV**
Fremont, California

PAGE 1 OF 1

| | |
|---|---|
| Boring location: | See Site Plan, Figure 2 |
| Date started: | 3/22/18 |
| Date finished: | 3/22/18 |
| Drilling method: | Hollow Stem Auger |
| Hammer weight/drop: | 140 lbs./30 inches |
| Hammer type: | Downhole Wireline |
| Sampler: | Sprague & Henwood (S&H) |
| Logged by: | W. Gozali |
| Drilled by: | Exploration Geoservices |
| Rig: | Mobile B-53 Red |

LABORATORY TEST DATA

| DEPTH (feet) | Sampler Type | 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 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | asphalt concrete | | | | | | |
| | | | | | aggregate base | | | | | | |
| 1–2 | S&H | 19 11 19 | 18 | CL | SANDY CLAY (CL) gray-brown, very stiff, moist, trace fine-grained sand LL = 40, PI = 25; see Figure A-18 | | | | 61 | 17.2 | 116 |
| 3.5–5 | S&H | 12 14 19 | 20 | SC | CLAYEY SAND (SC) dark brown with light brown mottling, medium dense, moist, fine-gained sand | | | | | | |
| 6–7.5 | S&H | 12 15 18 | 20 | CL | SANDY CLAY (CL) brown with light brown mottling, very stiff, moist, fine-grained sand | | | | | 12.6 | 120 |
| 8.5–10 | S&H | 11 14 15 | 17 | CL | red mottling | | | | | | |
| 13.5–15 | S&H | 16 28 33 | 37 | CL | yellow-brown, hard | | | | | 14.3 | 121 |

Boring terminated at a depth of 15 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H blow counts for the last two increments were converted to SPT N-Values using a factor of 0.6, to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 17-1297

Figure: A-2

# Log of Boring B-3

PROJECT: **OSGOOD II - IV**
Fremont, California



Boring location: See Site Plan, Figure 2
Date started: 3/22/18 | Date finished: 3/22/18
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches | Hammer type: Downhole Wireline
Sampler: Sprague & Henwood (S&H)

Logged by: W. Gozali
Drilled by: Exploration Geoservices
Rig: Mobile B-53 Red

| DEPTH (feet) | Sampler Type | Sample | Blows/ 6" | SPT N-Value[1] | LITHOLOGY | MATERIAL DESCRIPTION | Type of Strength Test | Confining Pressure Lbs/Sq Ft | Shear Strength Lbs/Sq Ft | Fines % | Natural Moisture Content, % | Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | asphalt concrete | | | | | | |
| 0.5 | | | | | | aggregate base | | | | | | |
| 1–2 | S&H | ■ | 6 6 9 | 9 | SC | CLAYEY SAND (SC) dark brown, loose, moist, fine- to coarse-grained sand LL = 30, PI = 15; see Figure A-18 | | | | 47 | 14.5 | 107 |
| 4 | S&H | ■ | 7 9 10 | 11 | SC | medium dense | | | | | | |
| 6–7 | S&H | ■ | 10 13 18 | 19 | SC | red-gray, decrease in clay content | | | | | 12.3 | 123 |
| 8–9 | S&H | ■ | 6 12 15 | 16 | CL | CLAY with SAND (CL) red-gray, stiff to very stiff, moist, fine-grained sand | | | | | | |
| 12–14 | S&H | ■ | 13 22 43 | 39 | CL | CLAY (CL) red-gray, hard, moist | | | | | | |
| 15 | | | | | | | | | | | | |

Boring terminated at a depth of 15 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H blow counts for the last two increments were converted to SPT N-Values using a factor of 0.6, to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 17-1297 | Figure: A-3

PROJECT: **OSGOOD II - IV** Fremont, California

# Log of Boring B-4



Boring location: See Site Plan, Figure 2

Date started: 3/22/18 | Date finished: 3/22/18

Drilling method: Hollow Stem Auger

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

Sampler: Sprague & Henwood (S&H)

Logged by: W. Gozali
Drilled by: Exploration Geoservices
Rig: Mobile B-53 Red

LABORATORY TEST DATA

| DEPTH (feet) | Sampler Type | Sample | Blows/ 6" | SPT N-Value[1] | LITHOLOGY | MATERIAL DESCRIPTION | Type of Strength Test | Confining Pressure Lbs/Sq Ft | Shear Strength Lbs/Sq Ft | Fines % | Natural Moisture Content, % | Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | asphalt concrete | | | | | | |
| | | | | | | aggregate base | | | | | | |
| 1–2 | S&H | ■ | 11 8 8 | 10 | CL | SANDY CLAY with GRAVEL (CL) dark gray-brown, stiff, moist, fine-grained sand, fine gravel | | | | | | |
| 4 | S&H | ■ | 9 17 19 | 22 | CL | dark brown, very stiff | | | | | | |
| 6–7 | S&H | ■ | 7 12 18 | 18 | CL | SANDY CLAY (CL) red-gray, very stiff, moist, fine-grained sand | | | | 59 | 14.8 | 118 |
| 9 | S&H | ■ | 17 21 29 | 30 | CL | very stiff to hard | | | | | | |
| 10 | | | | | | | | | | | | |

Boring terminated at a depth of 10 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H blow counts for the last two increments were converted to SPT N-Values using a factor of 0.6, to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 17-1297 | Figure: A-4

PROJECT: **OSGOOD II - IV**
Fremont, California

# Log of Boring B-5



Boring location: See Site Plan, Figure 2
Date started: 3/22/18 Date finished: 3/22/18
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches Hammer type: Downhole Wireline
Sampler: Sprague & Henwood (S&H)

Logged by: W. Gozali
Drilled by: Exploration Geoservices
Rig: Mobile B-53 Red

LABORATORY TEST DATA

| DEPTH (feet) | Sampler Type | 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 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | 3 inches of asphalt concrete | | | | | | |
| | | | | | 3 inches of aggregate base | | | | | | |
| 1 | | | | CL | SANDY CLAY (CL) dark brown with trace yellow, stiff, moist, fine-grained sand | | | | | | |
| 2 | S&H | 10, 10, 8 | 11 | | | | | | | 13.0 | 112 |
| 3 | S&H | 8, 15, 15 | 18 | | dark gray-brown, very stiff | | | | | | |
| 5 | S&H | 7, 18, 28 | 28 | | LL = 30, PI = 16; see Figure A-18 | | | | | 16.7 | 111 |
| 7 | | | | SC | CLAYEY SAND (SC) yellow-brown, dense, moist, fine- to coarse-grained sand | | | | | | |
| 9 | S&H | 13, 26, 34 | 36 | | | | | | | | |
| 12.5 | | | | SC | CLAYEY SAND (SC) light brown, medium dense, moist, fine-grained sand | | | | | | |
| 14 | S&H | 12, 13, 23 | 22 | | | | | | | | |
| 15 | | | | | | | | | | | |

Boring terminated at a depth of 15 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H blow counts for the last two increments were converted to SPT N-Values using a factor of 0.6, to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 17-1297
Figure: A-5

PROJECT: **OSGOOD II - IV**
Fremont, California

# Log of Boring B-6



Boring location: See Site Plan, Figure 2

Date started: 3/22/18 | Date finished: 3/22/18

Drilling method: Hollow Stem Auger

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

Sampler: Sprague & Henwood (S&H)

Logged by: W. Gozali
Drilled by: Exploration Geoservices
Rig: Mobile B-53 Red

LABORATORY TEST DATA

| DEPTH (feet) | Sampler Type | Sample | Blows/ 6" | SPT N-Value[1] | LITHOLOGY | MATERIAL DESCRIPTION | Type of Strength Test | Confining Pressure Lbs/Sq Ft | Shear Strength Lbs/Sq Ft | Fines % | Natural Moisture Content, % | Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | 6 inches of asphalt concrete | | | | | | |
| | | | | | | 6 inches of aggregate base | | | | | | |
| 1–2 | S&H | | 20 15 12 | 16 | SC | CLAYEY SAND (SC) dark brown with light gray, medium dense, moist, fine- to coarse-grained sand | | | | | 8.8 | 109 |
| 3–5 | S&H | | 7 13 14 | 16 | CL | SANDY CLAY (CL) dark brown, stiff to very stiff, moist, fine-grained sand | | | | 58 | 13.9 | 118 |
| 5–8 | S&H | | 8 17 28 | 27 | SC | CLAYEY SAND with GRAVEL (SC) yellow to yellow-brown, medium dense, moist, fine- to coarse-grained sand, fine gravel | | | | | | |
| 8–10 | S&H | | 13 22 23 | 27 | SC | CLAYEY SAND (SC) yellow-brown, medium dense, moist, fine- to coarse-grained sand | | | | | | |

Depth scale: 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22

Boring terminated at a depth of 10 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H blow counts for the last two increments were converted to SPT N-Values using a factor of 0.6, to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 17-1297 | Figure: A-6

PROJECT: **OSGOOD II - IV**
Fremont, California

# Log of Boring B-7



| | |
|---|---|
| Boring location: | See Site Plan, Figure 2 |
| Date started: 6/29/18 | Date finished: 6/29/18 |
| Drilling method: | Hollow Stem Auger |
| Hammer weight/drop: 140 lbs./30 inches | Hammer type: Downhole Wireline |
| Sampler: | Sprague & Henwood (S&H) |
| Logged by: | J. Pisenti |
| Drilled by: | Exploration Geoservices |
| Rig: | Mobile B-56 |

| 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 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | 3 inches of asphalt concrete | | | | | | |
| | | | | | | 9 inches of aggregate base | | | | | | |
| 1–2 | S&H | | 8 10 12 | 15 | SM | SILTY SAND (SM) gray-brown, medium dense, moist, fine-grained sand, trace rootlets | | | | | | |
| 2–3 | | | | | CL | SANDY CLAY (CL) olive-brown and yellow-brown, hard, moist, trace organics | | | | | | |
| 4 | S&H | | 18 28 40 | 48 | | LL = 31, PI = 17; see Figure A-18 | | | | 56 | 13.8 | 121 |
| 5.5–7 | S&H | | 8 12 20 | 22 | SC | CLAYEY SAND (SC) light brown, medium dense, moist, fine-grained sand | | | | | | |
| 9 | S&H | | 12 24 26 | 35 | | yellow-brown, dense, increase in silt content | | | | | | |
| 11.5–12 | S&H | | 16 36 36 | 50 | SM | SILTY SAND (SM) light-brown, dense to very dense, moist, fine-grained sand | | | | | | |
| 16 | S&H | | 18 24 30 | 38 | | dense, decrease in silt content | | | | | | |

Boring terminated at a depth of 16.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H blow counts for the last two increments were converted to SPT N-Values using a factor of 0.7, to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 17-1297
Figure: A-7

PROJECT: **OSGOOD II - IV**
Fremont, California

# Log of Boring B-8



Boring location: See Site Plan, Figure 2
Date started: 6/29/18 Date finished: 6/29/18
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches Hammer type: Downhole Wireline
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Logged by: J. Pisenti
Drilled by: Exploration Geoservices
Rig: Mobile B-56

| DEPTH (feet) | SAMPLES: Sampler Type | Blows/ 6" | 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 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0–1 | | | | | 3 inches of asphalt concrete<br>9 inches of aggregate base | | | | | | |
| 1–3 | S&H | 14<br>12<br>18 | 21 | CL | CLAY with GRAVEL (CL)<br>gray-brown with olive-gray, very stiff, moist, fine subangular gravel | PP | | 1,500 | | 12.9 | 121 |
| 3–6 | S&H | 7<br>11<br>18 | 20 | SC | CLAYEY SAND (SC)<br>gray-brown, medium dense, moist to wet, fine- to coarse-grained sand | | | | | 15.1 | 120 |
| 6–8 | S&H | 13<br>29<br>50/5" | 55/ 11" | SC | CLAYEY SAND (SC)<br>orange-brown, very dense, moist | | | | | | |
| 8–11 | S&H | 13<br>26<br>38 | 45 | CL | SANDY CLAY (CL)<br>yellow-brown, hard, moist, fine-grained sand | | | | | | |
| 11–14 | S&H | 19<br>31<br>45 | 53 | SM | SILTY SAND (SM)<br>light brown, very dense, moist, fine-grained sand, lightly cemented | | | | | 8.0 | 125 |
| 14–17 | S&H | 17<br>17<br>17 | 24 | SM | yellow-brown, medium dense, fine- to coarse-grained sand | | | | | | |
| 17–20 | S&H | 17<br>21<br>33 | 38 | SW | SAND (SW)<br>yellow and gray-brown, dense, moist, well graded | | | | | | |
| 20–21.5 | SPT | 17<br>21<br>23 | 53 | SM | SILTY SAND (SM)<br>yellow-brown, very dense, moist, fine-grained sand | | | | | | |

Boring terminated at a depth of 21.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.
PP = Pocket Penetrometer

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

ROCKRIDGE GEOTECHNICAL

Project No.: 17-1297
Figure: A-8

| PROJECT: | OSGOOD II - IV<br>Fremont, California | **Log of Boring B-9**<br>PAGE 1 OF 1 |
|---|---|---|

| | |
|---|---|
| Boring location: | See Site Plan, Figure 2 |
| Date started: | 6/29/18 |
| Date finished: | 6/29/18 |
| Drilling method: | Hollow Stem Auger |
| Hammer weight/drop: | 140 lbs./30 inches |
| Hammer type: | Downhole Wireline |
| Sampler: | Sprague & Henwood (S&H) |
| Logged by: | J. Pisenti |
| Drilled by: | Exploration Geoservices |
| Rig: | Mobile B-56 |

LABORATORY TEST DATA

| DEPTH (feet) | Sampler Type | Sample | Blows/ 6" | SPT N-Value[1] | LITHOLOGY | MATERIAL DESCRIPTION | Type of Strength Test | Confining Pressure Lbs/Sq Ft | Shear Strength Lbs/Sq Ft | Fines % | Natural Moisture Content, % | Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | 3 inches of asphalt concrete | | | | | | |
| | | | | | | 9 inches of aggregate base | | | | | | |
| 1–3 | S&H | | 21, 24, 20 | 31 | SM | SILTY SAND with GRAVEL (SM)<br>dark gray, dense, moist, fine- to coarse-grained sand<br>fine subrounded gravel | | | | | | |
| 3–8.5 | S&H | | 6, 12, 13 | 18 | CL | SANDY CLAY (CL)<br>gray-brown, very stiff, moist, fine-grained sand, with silt | | | | | | |
| | S&H | | 11, 14, 17 | 22 | | moist to wet | | | | | | |
| 8.5–11.5 | S&H | | 26, 27, 31 | 41 | SM | SILTY SAND with GRAVEL (SM)<br>light brown and yellow, dense, moist, well graded | | | | | | |
| 11.5–13 | S&H | | 14, 28, 28 | 39 | SC | CLAYEY SAND (SC)<br>yellow-brown, dense, moist, fine-grained sand | | | | | | |
| 13–15 | S&H | | 21, 23, 40 | 44 | CL | CLAY with SAND (CL)<br>yellow-brown, hard, moist, fine-grained sand | | | | | | |

Boring terminated at a depth of 15 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H blow counts for the last two increments were converted to SPT N-Values using a factor of 0.7, to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 17-1297

Figure: A-9

| PROJECT: | OSGOOD II - IV<br>Fremont, California | **Log of Boring B-10**<br>PAGE 1 OF 1 |
|---|---|---|

Boring location: See Site Plan, Figure 2

Date started: 6/29/18 | Date finished: 6/29/18

Drilling method: Hollow Stem Auger

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

Sampler: Sprague & Henwood (S&H)

Logged by: J. Pisenti
Drilled by: Exploration Geoservices
Rig: Mobile B-56

| DEPTH (feet) | SAMPLES: Sampler Type | 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 inches of asphalt concrete | | | | | | |
| | | | | | 9 inches of aggregate base | | | | | | |
| 1 | | | | SC | CLAYEY SAND (SC)<br>dark gray-brown, medium dense, moist, trace fine gravel | | | | | | |
| 2 | S&H | 12<br>21<br>17 | 27 | | | | | | 37 | 7.7 | 124 |
| 4 | S&H | 7<br>4<br>4 | 6 | | loose, no gravel<br>LL = 27, PI = 12; see Figure A-18 | | | | 44 | 13.3 | 104 |
| 6 | | | | CH | CLAY (CH)<br>dark brown, hard, moist, trace fine-grained sand | | | | | | |
| 7 | S&H | 14<br>21<br>28 | 34 | | | PP | | 3,250 | | | |
| 9 | S&H | 16<br>21<br>28 | 34 | CL | SANDY CLAY (CL)<br>brown, hard, moist, fine-grained sand | | | | | | |
| 11 | | | | SC | CLAYEY SAND (SC)<br>yellow-brown, dense, moist, fine-grained sand | | | | | | |
| 12 | S&H | 16<br>24<br>34 | 41 | | | | | | | | |
| 14 | S&H | 15<br>28<br>40 | 48 | | olive-brown | | | | | | |
| 17 | | | | SM | SILTY SAND (SM)<br>yellow-brown, dense, moist, fine-grained sand | | | | | | |
| 19 | S&H | 16<br>26<br>40 | 46 | | | | | | | | |
| 20 | | | | | | | | | | | |

Boring terminated at a depth of 20 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.
PP = Pocket Penetrometer

[1] S&H blow counts for the last two increments were converted to SPT N-Values using a factor of 0.7, to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 17-1297 | Figure: A-10

| PROJECT: | OSGOOD II - IV<br>Fremont, California | **Log of Boring HA-1**<br>PAGE 1 OF 1 |
|---|---|---|

Boring location: See Site Plan, Figure 2

Date started: 4/17/18 | Date finished: 4/17/18

Drilling method: 3-inch Hand Auger

Hammer weight/drop: N/A | Hammer type: N/A

Sampler: GRAB

Logged by: Q. Flores/ J. Sarmiento

| DEPTH (feet) | SAMPLES: Sampler Type | Sample | Blows/ 6" | 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 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0–2.5 | | | | | SC | CLAYEY SAND (SC)<br>dark brown, moist, trace coarse to fine gravel<br>tree root at 1 foot | | | | | | |
| 2.5–4 | GRAB | X (~3.0–3.5) | | | CL | CLAY with SAND (CL)<br>brown with black mottling, stiff, moist<br>LL = 47, PI = 31; see Figure A-18 | | | | | 20.4 | |
| 4–6.25 | GRAB | X (~4.9–5.0) | | | SC | CLAYEY SAND (SC)<br>brown to red-brown, dry to moist, fine-grained sand | | | | | | |
| 6.25–8 | GRAB | X (~6.5–7.0) | | | SM | SILTY SAND (SM)<br>yellow-brown, moist, fine-grained sand, trace gravel | | | | | | |

Depth scale: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11

Boring terminated at a depth of 8.0 feet below ground surface.
Boring backfilled with soil cuttings.
Groundwater not encountered during hand augering.

ROCKRIDGE GEOTECHNICAL

Project No.: 17-1297 | Figure: A-11

PROJECT: **OSGOOD II - IV**
Fremont, California

# Log of Boring HA-2



| | | | |
|---|---|---|---|
| Boring location: | See Site Plan, Figure 2 | Logged by: | Q. Flores/ J. Sarmiento |
| Date started: | 4/17/18 | Date finished: | 4/17/18 |
| Drilling method: | 3-inch Hand Auger | | |
| Hammer weight/drop: | N/A | Hammer type: | N/A |
| Sampler: | GRAB | | |

| DEPTH (feet) | SAMPLES: Sampler Type | Sample | Blows/ 6" | 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 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0–2 | | | | | CL | SANDY CLAY (CL) dark gray-brown, moist, fine-grained sand, trace coarse-grained sand to fine gravel | | | | | | |
| 2 | GRAB | X | | | | LL = 27, PI = 11; see Figure A-18 increasing sand content | | | | | 11.5 | |
| 2–4.5 | | | | | SC | CLAYEY SAND (SC) gray-brown, moist, trace coarse-grained sand to fine gravel, increasing clay content | | | | | | |
| 4 | GRAB | X | | | | | | | | | | |
| 4.5–5.5 | | | | | CL | SANDY CLAY (CL) gray-brown, moist to wet, trace rootlets | | | | | | |
| 5.5–6.25 | | | | | SC | CLAYEY SAND (SC) yellow-brown, moist, fine-grained sand, trace medium-grained sand | | | | | | |
| 6.25–7 | GRAB | X | | | CL | SANDY CLAY (CL) yellow-brown, very stiff, moist, fine- to medium-grained sand brown with dark brown mottling | | | | | | |

Boring terminated at a depth of 7 feet below ground surface.
Boring backfilled with soil cuttings.
Groundwater not encountered during hand augering.

ROCKRIDGE GEOTECHNICAL

Project No.: 17-1297

Figure: A-12

| PROJECT: | **OSGOOD II - IV** Fremont, California | **Log of Boring HA-3** PAGE 1 OF 1 |
|---|---|---|

| | |
|---|---|
| Boring location: See Site Plan, Figure 2 | Logged by: Q. Flores/ J. Sarmiento |
| Date started: 4/17/18 — Date finished: 4/17/18 | |
| Drilling method: 3-inch Hand Auger | |
| Hammer weight/drop: N/A — Hammer type: N/A | |
| Sampler: GRAB | |

| DEPTH (feet) | Sampler Type | Sample | Blows/ 6" | SPT N-Value[1] | LITHOLOGY | MATERIAL DESCRIPTION | Type of Strength Test | Confining Pressure Lbs/Sq Ft | Shear Strength Lbs/Sq Ft | Fines % | Natural Moisture Content, % | Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | CL | SANDY CLAY (CL) dark brown, moist, trace fine gravel | | | | | | |
| 1 | | | | | | | | | | | | |
| | | | | | | increase in gravel content | | | | | | |
| 2 | | | | | | light brown to brown | | | | | | |
| 3 | | | | | | decrease in gravel content, dark brown | | | | | | |
| | GRAB | ☒ | | | CL | CLAY with SAND (CL) dark brown, moist, fine- to coarse-grained sand | | | | | 14.1 | |
| 4 | | | | | | | | | | | | |
| 5 | | | | | | trace fine gravel red-brown mottling | | | | | | |
| 6 | GRAB | ☒ | | | CL | SANDY CLAY (CL) gray-brown, moist, fine- to medium-grained sand, trace fine gravel, with silt | | | | | | |
| | | | | | | trace orange-brown mottling | | | | | | |
| 7 | | | | | SC | CLAYEY SAND (SC) yellow-brown, moist, fine-grained sand | | | | | | |
| 8 | GRAB | ☒ | | | CL | SANDY CLAY (CL) yellow-brown, moist, increase in stiffness, fine-grained sand | | | | | | |
| 9 | | | | | | | | | | | | |
| 10 | | | | | | | | | | | | |
| 11 | | | | | | | | | | | | |

Boring terminated at a depth of 8.5 feet below ground surface.
Boring backfilled with soil cuttings.
Groundwater not encountered during hand augering.

ROCKRIDGE GEOTECHNICAL

| Project No.: 17-1297 | Figure: A-13 |
|---|---|

PROJECT: **OSGOOD II - IV**
Fremont, California

# Log of Boring HA-4



Boring location: See Site Plan, Figure 2

Date started: 4/17/18 | Date finished: 4/17/18

Drilling method: 3-inch Hand Auger

Hammer weight/drop: N/A | Hammer type: N/A

Sampler: GRAB

Logged by: Q. Flores/ J. Sarmiento

LABORATORY TEST DATA

| DEPTH (feet) | Sampler Type | Sample | Blows/ 6" | SPT N-Value[1] | LITHOLOGY | MATERIAL DESCRIPTION | Type of Strength Test | Confining Pressure Lbs/Sq Ft | Shear Strength Lbs/Sq Ft | Fines % | Natural Moisture Content, % | Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0–1 | | | | | CL | SANDY CLAY (CL) dark brown, moist, fine sand, trace gravel | | | | | | |
| 1 | | | | | | trace rootlets | | | | | | |
| 2 | GRAB | X | | | SC | CLAYEY SAND (SC) dark gray-brown, moist, trace fine gravel | | | | | 13.8 | |
| 3 | GRAB | X | | | SM | SILTY SAND (SM) brown, moist, fine-grained sand, trace coarse-grained sand | | | | | | |
| 4 | | | | | | | | | | | | |
| 5 | | | | | CL | SANDY CLAY (CL) yellow-brown, moist, fine-grained sand | | | | | | |
| 6 | GRAB | X | | | | | | | | | | |
| 6.5 | GRAB | X | | | | | | | | | | |
| 7.5 | GRAB | X | | | | | | | | | | |
| 8 | | | | | | | | | | | | |
| 9 | | | | | | | | | | | | |
| 10 | | | | | | | | | | | | |
| 11 | | | | | | | | | | | | |

Boring terminated at a depth of 8.0 feet below ground surface.
Boring backfilled with soil cuttings.
Groundwater not encountered during hand augering.

ROCKRIDGE GEOTECHNICAL

Project No.: 17-1297 | Figure: A-14

| PROJECT: | OSGOOD II - IV<br>Fremont, California | Log of Boring HA-5<br>PAGE 1 OF 1 |
|---|---|---|

| | |
|---|---|
| Boring location: | See Site Plan, Figure 2 |
| Date started: 4/17/18 | Date finished: 4/17/18 |
| Drilling method: | 3-inch Hand Auger |
| Hammer weight/drop: N/A | Hammer type: N/A |
| Sampler: | GRAB |
| Logged by: | Q. Flores/ J. Sarmiento |

LABORATORY TEST DATA

| 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 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0–2 | | | | | CL | SANDY CLAY (CL)<br>dark brown, moist, trace subrounded gravel, fine- to medium-grained sand | | | | | | |
| 2–3.25 | | | | | SC | CLAYEY SAND (SC)<br>dark brown, moist, trace fine gravel | | | | | | |
| 3–3.5 | GRAB | X | | | | | | | | | 20.1 | |
| 3.25–4.5 | | | | | CL | CLAY with SAND (CL)<br>dark gray-brown, with trace brown mottling, moist, trace fine gravel<br>LL = 34, PI = 19; see Figure A-18 | | | | | | |
| 4.5–7.5 | | | | | CL | SANDY CLAY (CL)<br>gray-brown, very stiff, moist, fine-grained sand | | | | | | |
| 5–5.5 | GRAB | X | | | | | | | | | | |
| 7.5–9 | | | | | CL | CLAY with SAND (CL)<br>light brown, very stiff, moist, fine-grained sand | | | | | | |
| 8–8.5 | GRAB | X | | | | | | | | | | |
| 9–10.5 | | | | | CL | SANDY CLAY (CL)<br>yellow-brown, moist, fine- to medium-grained sand | | | | | | |
| 9.5–10 | GRAB | X | | | | | | | | | | |

Boring terminated at a depth of 10.5 feet below ground surface.
Boring backfilled with soil cuttings.
Groundwater not encountered during hand augering.

ROCKRIDGE GEOTECHNICAL

| Project No.: | Figure: |
|---|---|
| 17-1297 | A-15 |

ROCKRIDGE 17-1297 HAND AUGERS.GPJ TR.GDT 11/13/18

## 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

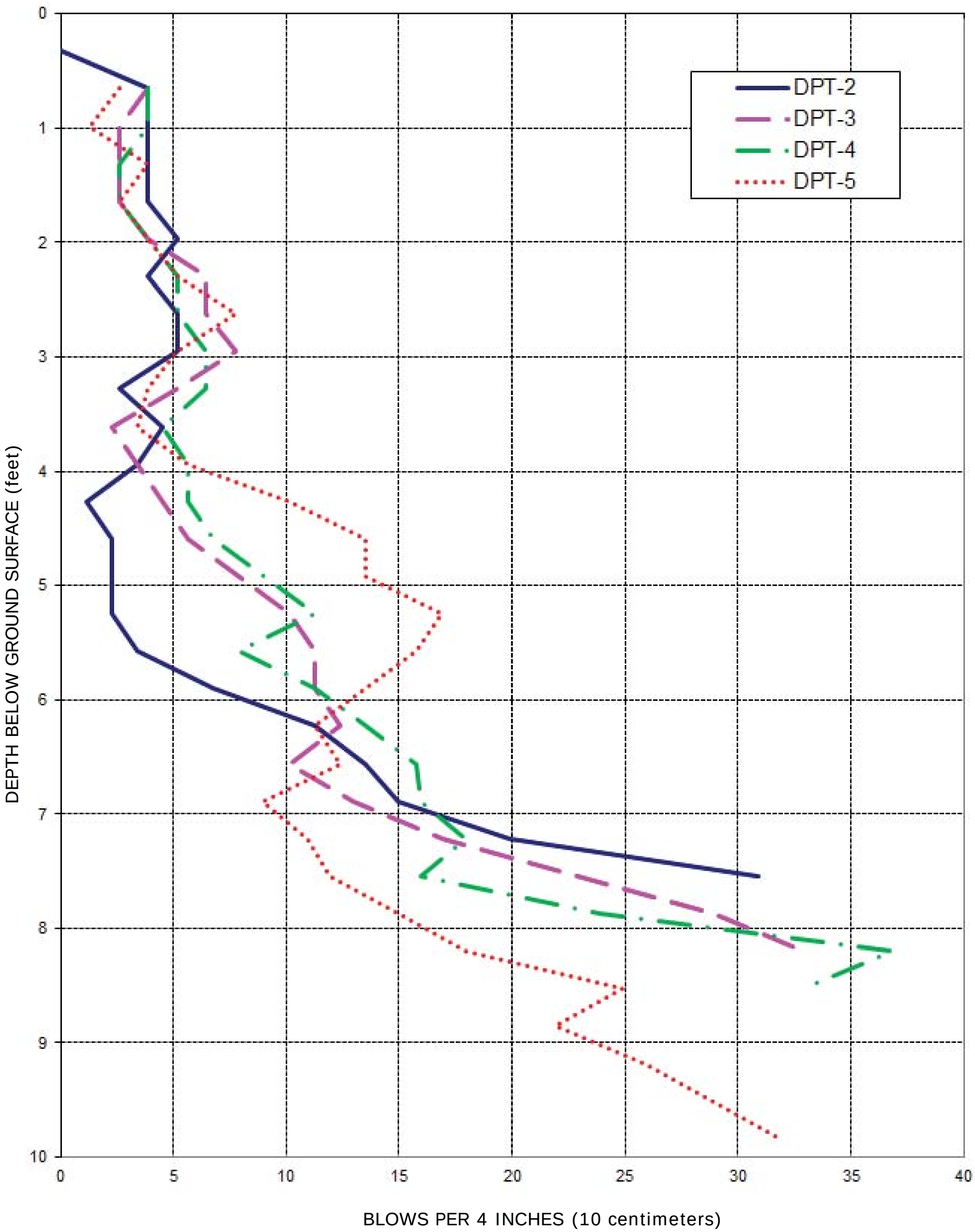
## SAMPLER TYPE

| | |
|---|---|
| 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 |

**OSGOOD II - IV**
Fremont, California

ROCKRIDGE GEOTECHNICAL

## CLASSIFICATION CHART

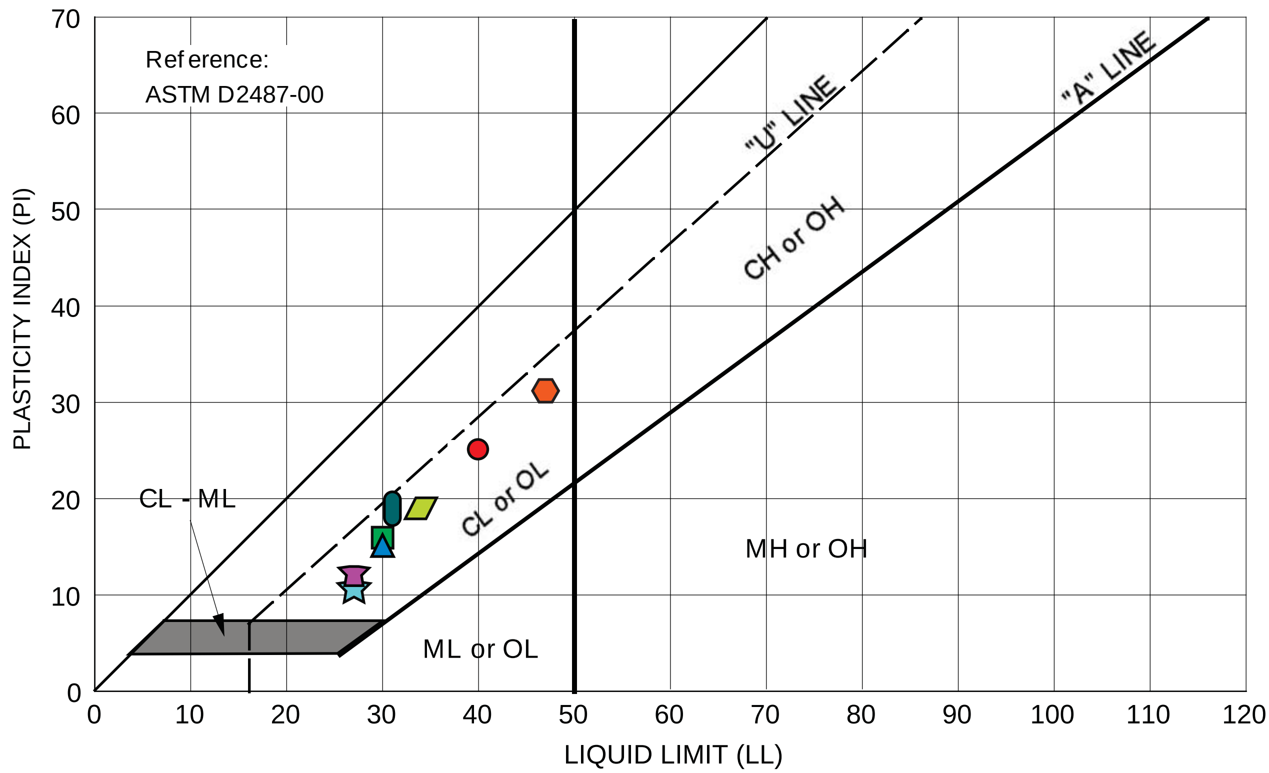



**OSGOOD II - IV**
Fremont, California

**DYNAMIC PENETROMETER TEST RESULTS**

ROCKRIDGE GEOTECHNICAL

| Date 07/31/18 | Project No. 17-1297 | Figure A-17 |
|---|---|---|




| Symbol | Source | Description and Classification | Natural M.C. (%) | Liquid Limit (%) | Plasticity Index (%) | % Passing #200 Sieve |
|---|---|---|---|---|---|---|
| ● | B-2 at 2.5 feet | SANDY CLAY (CL), gray-brown | 17.2 | 40 | 25 | 61 |
| ▲ | B-3 at 2.5 feet | CLAYEY SAND (SC), dark brown | 14.5 | 30 | 15 | 47 |
| ■ | B-5 at 4.5 feet | SANDY CLAY (CL), dark gray-brown | 16.7 | 30 | 16 | -- |
| ⬢ | HA-1 at 3.0 feet | CLAY with SAND (CL), brown with black mottling | 20.4 | 47 | 31 | -- |
| ★ | HA-2 at 1.5 feet | SANDY CLAY (CL), dark gray-brown | 11.5 | 27 | 11 | -- |
| ▱ | HA-5 at 3.0 feet | CLAY with SAND (CL), dark gray-brown | 20.1 | 34 | 19 | -- |
| ⬮ | B-7 at 4.5 feet | SANDY CLAY (CL), olive-brown and gray-brown | 13.8 | 31 | 17 | 56 |
| ⧗ | B-10 at 4.5 feet | CLAYEY SAND (SC), dark gray-brown | 13.3 | 27 | 12 | 44 |

**OSGOOD II - IV**
Fremont, California

ROCKRIDGE GEOTECHNICAL

**PLASTICITY CHART**

Date 11/13/18 | Project No. 17-1297 | Figure A-18

ROCKRIDGE GEOTECHNICAL

# APPENDIX B

# Soil Analysis Lab Results

Client: Rockridge Geotechnical
Job Name: Osgood II
Client Job Number: 17-1297
Project X Job Number: S180410D
April 16, 2018

| | Method | ASTM G187 | | ASTM D516 | | ASTM D512B | | SM 4500-NO3-E | SM 4500-NH3-C | SM 4500-S2-D | ASTM G200 | ASTM G51 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Bore# / Description** | **Depth** | **Resistivity** As Rec'd | Minimum | **Sulfates** | | **Chlorides** | | **Nitrate** | **Ammonia** | **Sulfide** | **Redox** | **pH** |
| | (ft) | (Ohm-cm) | (Ohm-cm) | (mg/kg) | (wt%) | (mg/kg) | (wt%) | (mg/kg) | (mg/kg) | (mg/kg) | (mV) | |
| B-5 # 2 | 3.0-3.5 | 1,206 | 1,206 | 48 | 0.0048 | 336 | 0.0336 | 30 | 78.0 | 1.20 | 145 | 8.29 |

Unk = Unknown
NT = Not Tested
ND = 0 = 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.
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**30 Year Life-Pavement Design for Osgood II Residences**

The proposed Traffic Index (TI) for Osgood II residences is 5.5.

Per the 2017 Caltrans Highway Design Manual table 613.3 C "Conversion of ESAL to Traffic Index", the Equivalent Single Axle Load (ESAL) for a TI of 5.5 is:

**TI of 5.5 = 23,500 ESALs**

The projected truck traffic volume for Osgood II residences assuming the type of trucks that will be on site are fully loaded delivery trucks (2 axles) and fully loaded garbage trucks (3 axles) is:

**DELIVERY TRUCKS (2 Axles)**

**North Side: 162 apartment units**

Assume the average resident moving in/moving out timeframe per unit = 2 years

162units /2 years = **81 trips per year**

**South Side: 122 Condominium units**

Assume the average resident moving in/moving out timeframe per unit = 5 years

122 units/5 years = 25 trips per year

Total number of trips per year = 81 + 25 = **106 trips per year**

**Total move in/move out trips per year = 81 trips + 106 trips = 187 trips per year**
**Total move in/move out trips for 2 axles for 30 years = 187 trips per year x 2 axles x 30 years = 11,220 trips per 30 years**

**GARBAGE TRUCKS (3 Axles)**

Assume 1 garbage truck per week per residential complex:

Total number of trips per 30 year:

2 garbage trucks (1 per residential complex) x 3 axles x 52 weeks per year x 30 years = **9,360 trips per 30 years**

**TOTAL NUMBER OF ESALs per 30 years = 11,220 + 9,360 = 20,580 ESAL's < 23,500**

## CONCLUSION

Based on the projected traffic volume and Equivalent Single Axle load for the proposed project, the TI of 5.5 for the pavement design will be adequate for a 30-year life