ROCKRIDGE GEOTECHNICAL

Prepared for **Hall Equities Group**

GEOTECHNICAL INVESTIGATION
SARANAP MIXED-USE DEVELOPMENT
BOULEVARD WAY AND SARANAP AVENUE
CONTRA COSTA COUNTY, CALIFORNIA

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October 30, 2013
Project No. 12-477

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ROCKRIDGE GEOTECHNICAL

October 30, 2013
Project No. 12-477

Mr. Michael Smith
Vice President/Forward Planner
Hall Equities Group
1855 Olympic Boulevard, Suite 250
Walnut Creek, California 94596

Subject: Geotechnical Investigation
Saranap Mixed-Use Development
Boulevard Way and Saranap Avenue
Contra Costa County, California

Dear Mr. Smith:

We are pleased to present our geotechnical report for the proposed Saranap mixed-use development to be constructed in Contra Costa County, California. Our investigation was performed in accordance with our proposal dated October 26, 2012.

The site is located at the intersection of Boulevard Way and Saranap Avenue. The subject property is comprised of three "sites", referred to as Sites A, B, and C. Site A is at the northeast corner of Boulevard Way and Saranap Avenue and is currently occupied by a single-family residence, an apartment building, a vacant lot, two commercial buildings and asphalt-paved parking lots. Site B is located at the southeast corner of Boulevard Way and Saranap Avenue. The front portion of Site B is relatively flat and is occupied by an asphalt-paved parking lot and a two-story sanctuary building. The southern end of Site B slopes up gently to the south and is undeveloped. Site C is located at the southwest corner of Boulevard Way and Saranap Avenue. The northeast portion of Site C is occupied by a one-story retail building. The remainder of Site C consists of an asphalt-paved parking lot and landscaped areas.

Plans are to construct a mixed-use development that will include: (1) a seven-story mixed-use building with a basement at Site A, (2) a seven-story mixed-use building where the lower level is below-grade at Site B, and (3) a four-story residential building where the lower level is partially below-grade at Site C. Other project plans include improvements to the existing streets, sidewalks, and/or intersection within the project limits.

On the basis of our investigation, we conclude the proposed improvements may be constructed as planned, provided the recommendations presented in the attached report are incorporated into the project plans and specifications. The primary geotechnical concerns at the site are: (1) the presence of moderately to highly expansive near-surface clay underlying the site; (2) relatively shallow groundwater for construction of proposed buildings with one basement level; (3) providing adequate foundation support for the proposed buildings; and (4) providing adequate lateral support for adjacent improvements during excavation and construction of basement level.

We conclude the proposed buildings at Sites A and B may be supported on spread footings bearing on native soil or bedrock; provided permanent dewatering system will be installed to reduce hydrostatic pressures on the building slabs and foundations. The proposed building at Site C may be supported on isolated spread footings at interior column locations and continuous, deepened perimeter footings.

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

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

Sincerely yours,
ROCKRIDGE GEOTECHNICAL, INC.

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Enclosure

TABLE OF CONTENTS

1.0 INTRODUCTION 1

2.0 SCOPE OF WORK 2

3.0 FIELD INVESTIGATION AND LABORATORY TESTING 3
- 3.1 Test Borings 3
- 3.2 Hand-Auger Boring and DPT 4
- 3.3 Laboratory Testing 5

4.0 SITE AND SUBSURFACE CONDITIONS 5
- 4.1 Site A 5
- 4.2 Site B 6
- 4.3 Site C 7

5.0 SEISMIC CONSIDERATIONS 7
- 5.1 Regional Seismicity and Faulting 7
- 5.2 Seismic Hazards 11
  - 5.2.1 Ground Shaking 11
  - 5.2.2 Fault Rupture 11
  - 5.2.3 Liquefaction and Associated Hazards 12
  - 5.2.4 Cyclic Densification 12

6.0 DISCUSSIONS AND CONCLUSIONS 12
- 6.1 Foundation Support and Settlement 13
- 6.2 Groundwater 14
- 6.3 Dewatering 14
  - 6.3.1 Temporary Dewatering 15
  - 6.3.2 Permanent Dewatering 16
- 6.4 Temporary Cut Slopes and Shoring 17
- 6.5 Soil Corrosivity 17
- 6.6 Excavation, Monitoring, and Construction Considerations 18

7.0 RECOMMENDATIONS 18
- 7.1 Site Preparation and Grading 19
  - 7.1.1 Subgrade Preparation 19
  - 7.1.2 Fill Quality and Compaction 21
  - 7.1.3 Lime-Treatment 22
  - 7.1.4 Soil Subgrade Stabilization 22
  - 7.1.5 Exterior Flatwork Subgrade Preparation 24
  - 7.1.6 Utility Trench Backfill 25
  - 7.1.7 Geotechnical Services during Grading 26
- 7.2 Spread Footings 26

7.2.1 Buildings at Sites A and B 26
7.2.2 Building at Site C 27
7.3 Permanent Dewatering System 28
7.4 Floor Slabs 29
7.5 Permanent Retaining and Basement Walls 30
7.6 Temporary Cut Slopes and Shoring 31
7.6.1 Cantilevered Soldier Pile and Lagging Shoring System 32
7.6.2 Soldier Pile and Lagging Shoring System with Tiebacks 33
7.6.3 Tieback Testing 34
7.7 Pavement Design 35
7.7.1 Flexible (Asphalt Concrete) Pavement Design 35
7.7.2 Rigid (Portland-Cement Concrete) Pavement 36
7.8 Drainage and Landscaping 37
7.8.1 Bio-retention 38
7.9 Seismic Design 39

8.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION 39

9.0 LIMITATIONS 39

REFERENCES

FIGURES

APPENDIX A – Boring Logs and DPT Results

APPENDIX B – Laboratory Test Results

LIST OF FIGURES

Figure 1	Site Location Map
Figure 2	Site Plan
Figure 3	Regional Geologic Map
Figure 4	Regional Fault Map
Figure 5	Design Parameters for Soldier-Pile-and-Lagging with Tiebacks Temporary Shoring System

APPENDIX A

Figures A-1 to A-12	Borings B-1 through B-12
Figure A-13	Classification Chart
Figure A-14	Physical Properties Criteria for Rock Descriptions
Figure A-15	Dynamic Penetrometer Test Results

APPENDIX B

Figure B-1	Plasticity Chart
Figures B-2 and B-3	Particle Size Distribution Reports
Figures B-4 and B-5	R-value Test Reports
Corrosion Test Report	

GEOTECHNICAL INVESTIGATION
SARANAP MIXED-USE DEVELOPMENT
BOULEVARD WAY AND SARANAP AVENUE
Contra Costa County, California

1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by Rockridge Geotechnical, Inc. for the proposed Saranap mixed-use development to be constructed in Contra Costa County, California. The site is located at the intersection of Boulevard Way and Saranap Avenue, as shown on the Site Location Map, Figure 1.

The subject property is comprised of three "sites", referred to as Sites A, B, and C, as shown on the Site Plan, Figure 2. Site A is at the northeast corner of Boulevard Way and Saranap Avenue and is currently occupied by a single-family residence, an apartment building, a vacant lot, and two commercial buildings and paved parking lots. Site B is located at the southeast corner of Boulevard Way and Saranap Avenue. The front portion of Site B is relatively flat and is occupied by an asphalt-paved parking lot and a two-story sanctuary building. The southern end of Site B slopes up gently to the south and is undeveloped. Site C is located at the southwest corner of Boulevard Way and Saranap Avenue. The northeast portion of Site C is occupied by a one-story multi-tenant retail building. The remainder of Site C consists of an asphalt-paved parking lot and landscaped areas.

Plans are to construct new mixed-use buildings at Sites A, B, and C. The schematic plan for Site A is to construct a seven-story building with a basement. The basement will be used for parking and will include deepened parking pits. The first and second levels will consist of parking and retail space and the upper five levels will consist of dwelling units. At Site B, the plan is to construct a seven-story building with four levels of wood-frame residential units above three levels of concrete podium. The lower podium level will be constructed below-grade and the middle podium level will be partially below-grade. The podium levels will consist of parking and retail space. At Site C, the plan is to construct a four-story residential building: the first story will be partially below-grade.

Construction of the basements at Sites A, and B will require excavating up to about 20 feet below the ground surface (bgs); construction of the partial basement at Site C will require excavating up to about 10 feet bgs. Other project plans include improvements to the existing streets, sidewalks, and/or intersection within the project limits.

2.0 SCOPE OF WORK

Our investigation was performed in accordance with our proposal dated October 26, 2012. Our scope of work consisted of exploring the subsurface conditions at Sites A, B, and C and performing engineering analyses to develop conclusions and recommendations regarding:

- soil and groundwater conditions at the site, including a design groundwater level
- site seismicity and seismic hazards
- 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 foundation settlements
- corrosivity of the near-surface soil and the potential effects on buried concrete and metal structures and foundations
- site grading and excavation, including criteria for fill quality and compaction
- subgrade preparation for interior and exterior concrete slabs-on-grade
- temporary cut slopes and shoring
- temporary and permanent dewatering
- lateral earth pressures for design of permanent retaining and basement walls
- pavement design
- 2013 California Building Code (CBC) site class and design spectral response acceleration parameters
- construction considerations.

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

Our field investigation consisted of drilling 11 test borings, advancing one hand-auger boring, and performing a dynamic penetrometer test (DPT). Details of our field investigation are presented in this section.

3.1 Test Borings

Prior to drilling the test borings, we obtained drilling permits from the Contra Costa County Environmental Health Division (CCCEHD) and contacted Underground Service Alert (USA) to notify them of our work, as required by law. We also retained Precision Locating, LLC, a private utility locator, to check that the boring locations were clear of buried utilities.

On May 9, 10, and 13, 2013, 11 test borings, designated B-1 through B-8 and B-10 through B-12 were drilled by Exploration Geoservices of San Jose, California, using a truck-mounted drill rig equipped with six-inch-diameter hollow-stem augers at the approximate locations shown on Figure 2. These borings were drilled to depths between 24 and 25 feet bgs. During drilling, our field engineer logged the soil and rock encountered and obtained representative samples for visual classification and laboratory testing. The logs of the test borings are presented on Figures A-1 through A-8 and A-10 through A-12 in Appendix A. The soil and rock encountered in the borings were classified in accordance with the descriptions shown on Figures A-13 and A-14.

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

The samplers were driven with 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 and SPT samplers were converted to approximate SPT N-values using factors of 0.6 and 1.0, respectively, to account for sampler type and approximate hammer energy. The blow counts used for this conversion were: (1) the last two blow counts if the sampler was driven more than 12 inches, (2) the last one blow count if the sampler was driven more than six inches but less than 12 inches, and (3) the only blow count if the sampler was driven six inches or less. The converted SPT N-values are presented on the boring logs.

Upon completion of drilling, the boreholes were backfilled with cement grout under the observation of the CCCEHD inspector. The pavement at the borehole locations was patched with asphalt. The soil cuttings generated by the borings were placed in landscape areas at the site.

3.2 Hand-Auger Boring and DPT

Boring B-9 was located in a landscaped area behind the existing sanctuary building at Site B and was not accessible by a truck-mounted drill rig. Therefore, we advanced a hand-auger boring at this location on May 13, 2013 to a depth of 4 feet bgs to obtain soil samples for visual classification. The approximate location for Boring B-9 is shown on Figure 2. The log of Boring B-9 is shown on Figure A-9 in Appendix A.

On May 13, 2013, we also performed a DPT, designated DPT-1, adjacent to Boring B-9 to investigate the in-situ strength of the underlying soil. The DPT was performed by driving a 1.4-inch-diameter, cone-tipped probe into the ground with a 35-pound hammer falling about 15 inches. The blows required to drive the probe were recorded at 10-centimeter intervals and converted to SPT N-values for use in our engineering analyses. The DPT was advanced to practical refusal, defined as more than 50 blows to advance the probe 10 centimeters, at a depth of approximately 15 feet bgs. The DPT results are presented on Figure A-15 in Appendix A.

3.3 Laboratory Testing

We re-examined the soil samples obtained from our borings to confirm the field classifications and select representative samples for laboratory testing. Soil samples were tested to measure moisture content, dry density, Atterberg limits, gradation, resistance value (R-value), and corrosivity. The results of the laboratory tests are presented on the boring logs and in Appendix B.

4.0 SITE AND SUBSURFACE CONDITIONS

A regional geologic map of the area (Figure 3) indicates the site is underlain by sediments (QTs) overlying sedimentary rocks (Tms). Site and subsurface conditions for Sites A, B, and C are described below.

4.1 Site A

Site A is located at the northeast corner of Boulevard Way and Saranap Avenue and encompasses an area of 65,674 square feet. Site A slopes gently up to the north with ground surface elevations varying from Elevation 214 to 228 feet[1]. Site A is currently occupied by a single-family residence, an apartment building, a vacant lot, and two commercial buildings and paved parking lots. At Borings B-1 and B-2, the pavement section consists of 1 to 2 inches of asphalt concrete and 5 to 6 inches of aggregate base.

Borings B-1 through B-5 drilled in Site A indicate it is underlain by 8 to 22 feet of sediments consisting of interbedded layers of stiff to hard clay with variable amounts of sand and gravel, and medium dense to dense sand and gravel with variable amounts of clay. Atterberg limits tests performed on the near-surface clay obtained from Borings B-1 and B-4 indicate the clay is highly expansive[2]. The sediments are underlain by sedimentary rocks consisting of siltstones and mudstones that are soft to low hardness, plastic to friable in strength, and deeply weathered.

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[1] Elevations referenced in this report are based on Alta/ACSM Land Title Survey for the project dated June, 2012 and prepared by Kier & Wright Civil Engineers & Surveyors, Inc.

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

Approximate elevations of the top of bedrock encountered at boring locations are shown on Figure 2.

Groundwater was encountered at a depth of 15 feet bgs (Elevation 202 feet) in Boring B-1 during drilling. Groundwater was not encountered during drilling in Borings B-2, B-3, B-4, and B-5. The borehole for Boring B-3 was left open overnight before grouting and groundwater was measured at a depth of 9 feet bgs (Elevation 210 feet) in the borehole the next morning.

4.2 Site B

Site B is located at the southeast corner of Boulevard Way and Saranap Avenue. Site B is comprised of two parcels encompassing a total area of 57,546 square feet. The front (north) portion of Site B is relatively flat with the ground surface at approximately Elevation 215 feet, and is occupied by an asphalt-paved parking lot and a two-story sanctuary building. The southern end of Site B slopes up gently to the south with ground surface elevations varying from Elevations 215 to 230 feet, and is undeveloped.

Borings B-6, B-7, and B-9 and DPT-1 performed at the front (north) portion of Site B indicate it is underlain by 12 to 15 feet of sediments overlying sedimentary rock. Boring B-8, located at the south end of Site B encountered sediments to the maximum depth explored of 24.5 feet bgs. Where explored, sediments consist of interbedded layers of stiff to hard clay with variable amounts of sand, and medium dense to very dense sand and gravel with variable amounts of clay and silt. Atterberg limits performed on the near-surface clay obtained from Boring B-7 indicate the clay has a low expansion potential. Sedimentary rocks encountered consist of mudstone and claystone that are soft to low hardness, plastic to friable strength, and deeply weathered.

Groundwater was not encountered in Borings B-6 and B-9 and DPT-1 during drilling. Groundwater was encountered in Boring B-7 at 13 feet bgs (Elevation 199.5 feet) and Boring B-8 at 17.5 feet bgs (Elevation 204.5 feet) during drilling.

4.3 Site C

Site C is located at the southwest corner of Boulevard Way and Saranap Avenue. Site C is comprised of four parcels encompassing an area of 25,640 square feet. The northeast portion of the site is occupied by a one-story multi-tenant retail building. The remainder of the site is occupied by an asphalt-paved parking lot and landscaped areas. The ground surface at Site C varies from Elevations 225 feet at the north to about 232 feet at the south.

Borings B-10, B-11, and B-12 performed at Site C indicate it is underlain by sediments to depths of 21 feet bgs at Boring B-10 and to the maximum depths explored of 25 feet at Borings B-11 and B-12. The sediments consist of interbedded layers of stiff to very stiff clay with variable amounts of sand, and medium dense to dense sand with variable amounts of clay. Atterberg limits performed on the near-surface clay obtained from Boring B-11 indicate the clay is moderately to highly non-expansive. Bedrock encountered near the bottom of Boring B-10 consists of claystone that is soft to low hardness, plastic to friable strength, and deeply weathered.

Groundwater was encountered in Boring B-10 at a depth of 18.5 feet bgs (Elevation 208.5 feet) at the end of drilling, and at a depth of 20.3 feet bgs (Elevation 206.5 feet) 2-1/2 hours later, just prior to grouting. Groundwater was not encountered in Borings B-11 and B-12 during drilling.

5.0 SEISMIC CONSIDERATIONS

5.1 Regional Seismicity and Faulting

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 Calaveras, Hayward, San Andreas, and Green Valley and Mount Diablo faults. These and other faults in the region are shown on Figure 4. For these and other active faults within a 50-kilometer radius of the site, the distance from the site and estimated mean characteristic Moment magnitude[3] [2007 Working Group on California Earthquake Probabilities (WGCEP) (USGS 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
Mount Diablo Thrust	4	East	6.70
Green Valley Connected	7	East	6.80
Total Calaveras	10	Southeast	7.03
Total Hayward	14	Southwest	7.00
Total Hayward-Rodgers Creek	14	Southwest	7.33
Greenville Connected	22	East	7.00
Great Valley 5, Pittsburg Kirby Hills	25	East	6.70
West Napa	34	Northwest	6.70
Rodgers Creek	38	Northwest	7.07
N. San Andreas - Peninsula	43	West	7.23
N. San Andreas (1906 event)	43	West	8.05
N. San Andreas - North Coast	45	West	7.51
Great Valley 4b, Gordon Valley	45	Northeast	6.80
San Gregorio Connected	49	West	7.50

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, Mw, for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to an Mw 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 Mw of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The most recent earthquake to affect the Bay Area was the Loma Prieta Earthquake of 17 October 1989 with an Mw of 6.9. This earthquake occurred in the Santa Cruz Mountains approximately 97 kilometers southwest of the site.

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

The U.S. Geological Survey's (USGS) 2007 WGCEP has compiled the earthquake fault research for the San Francisco Bay area in order to estimate the probability of fault segment rupture. They have determined that the overall probability of moment magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Region during the next thirty years is 63 percent. The highest probabilities are assigned to the Hayward/Rodgers Creek Fault and the northern segment of the San Andreas Fault: these probabilities are 31 and 21 percent, respectively (USGS 2008). The probabilities assigned to Calaveras, Concord-Green Valley, and Mount Diablo Thrust faults are 7, 3, and 1 percent, respectively (USGS 2008).

5.2 Seismic Hazards

During a major earthquake on a segment of one of the nearby faults, strong to very strong ground shaking is expected to occur at the project site. Strong shaking during an earthquake can result in ground failure such as that associated with soil liquefaction[4], lateral spreading[5], and cyclic densification[6]. The results of our evaluation regarding seismic considerations for the project site are presented in the following sections.

5.2.1 Ground Shaking

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

5.2.2 Fault Rupture

Historically, ground surface displacements closely follow the trace of geologically young faults. The site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act, and no known active or potentially active faults exist on the site. We therefore conclude the risk of fault offset at the site from a known active fault is very low. In a seismically active area, the remote possibility exists for future faulting in areas where no faults previously

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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.

existed; however, we conclude the risk of surface faulting and consequent secondary ground failure from previously unknown faults is also very low.

5.2.3 Liquefaction and Associated Hazards

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

The site is generally underlain by stiff clay and interbedded clayey sand and clayey gravel layers which are not susceptible to liquefaction because of their cohesion and/or density. Therefore, we conclude the potential for liquefaction to occur at the site is very low.

5.2.4 Cyclic Densification

Cyclic densification (also referred to as differential compaction) of non-saturated sand (sand above groundwater table) can occur during an earthquake, resulting in settlement of the ground surface and overlying improvements. The soil encountered above the ground water table is not susceptible to cyclic densification because of it cohesion and/or density. Therefore, we conclude the potential for cyclic densification to occur at the site is very low.

6.0 DISCUSSIONS AND CONCLUSIONS

From a geotechnical standpoint, we conclude the site can be developed as planned, provided the recommendations presented in this report are incorporated into the project plans and specifications and implemented during construction. The primary geotechnical concerns associated with the proposed project are: (1) the presence of moderately to highly expansive near-surface clay underlying the site; (2) relatively shallow groundwater for construction of proposed buildings with basement levels; (3) providing adequate foundation support for the

proposed buildings; and (4) providing adequate lateral support for adjacent improvements during excavation and construction of basement levels. These and other geotechnical issues as they pertain to the proposed development are discussed in this section.

6.1 Foundation Support and Settlement

The moderately to highly expansive near-surface clay at the site is subject to volume changes during seasonal fluctuations in moisture content. These volume changes can cause cracking of foundations and floor slabs. Therefore, foundation and slabs should be designed and constructed to resist the effects of the expansive clay. These effects can be mitigated by moisture conditioning the expansive clay, providing non-expansive soil below interior and exterior slabs, and supporting foundations below the zone of severe moisture change.

The proposed buildings at Sites A and B will have a basement and the finished floor for these buildings will be located below the potential zone of moisture change. Therefore, there should be no volumetric changes in the soil and mitigation of the expansion potential of the soil is not required for proposed buildings at Sites A and B. We anticipate the foundation level of proposed buildings with a basement will be underlain by relatively strong sediments or bedrock that can support moderate building loads. Therefore, we conclude proposed buildings at Sites A and B can be supported on conventional spread footings.

The proposed building at Site C will be at-grade along Boulevard Way (western property line) and partially below-grade at the eastern portion of the site. We anticipate the proposed building at Site C will be underlain by relatively strong sediments that can support moderate building loads. We conclude the proposed building at Site C may be supported on individual spread footings at interior column locations and continuous, deepened perimeter footings. The perimeter footings should be deepened (see Section 7.2.2) to act as a barrier to reduce the potential for moisture change beneath the slab-on-grade floors.

We estimate total and differential settlement of properly constructed spread footings designed using the recommendations presented in Section 7.2 of this report will be less than 1 inch and 1/2 inch over a 30-foot horizontal distance, respectively.

6.2 Groundwater

As discussed in Section 4.0, groundwater was encountered in Borings B-1, B-3, B-7, B-8, B-10, and B-12 during drilling and/or prior to grouting at depths between 9 and 20.3 feet bgs (between Elevations 199.5 and 210 feet). Groundwater was not observed in the other borings during drilling. Because of the relatively low permeability of the clay and bedrock at these depths, the groundwater levels encountered in the borings are not necessarily the stabilized groundwater level.

Based on our experience with projects on sloping terrain and relatively shallow bedrock similar to the subject site, groundwater may be encountered where it has been trapped by impervious soil/rock or is flowing through pervious zones in the underlying bedrock. The presence of groundwater at the site will vary depending on localized subsurface conditions, the presence of damaged utility lines, rainfall amounts and other seasonal conditions, and irrigation practices. Based on the results of this investigation, we recommend proposed buildings at Sites A, B, and C be designed using design groundwater depths of 8 feet bgs, 10 feet bgs, and 15 feet bgs, respectively.

6.3 Dewatering

Because of the relatively high groundwater table and the anticipated depth of excavation, temporary dewatering will likely be required during construction of basement levels at Sites A and B. Based on our experience, we believe that providing an underslab drainage system to reduce hydrostatic pressures will result in a more economic foundation design than a fully waterproofed building restrained with hydrostatic uplift anchors.

6.3.1 Temporary Dewatering

Based on our experience on projects situated at sites with sloping terrain and shallow bedrock similar to the subject site, some seepage of groundwater into the basement excavations should be expected, especially if the excavations are open during wet weather conditions. In most cases, we anticipate groundwater seepage, if any, would have a relatively low flow rate. However, there may be bedrock underlying the site that may be highly permeable at some locations; particularly where groundwater can flow through poorly cemented sandstone and siltstone deposits or in fractures in the bedrock. Flow of groundwater into the excavation during construction could result in sloughing, slumping, or caving of the sides of the excavation and/or wet, difficult working conditions. Therefore, we anticipate it will be necessary to temporarily dewater excavations during construction.

Temporary dewatering is typically performed by installing a series of wells around the perimeter of the building, with interior wells also used for larger footprints. However, based on our investigation, we conclude the effectiveness of temporary dewatering wells will be limited at the site due to the relatively low permeability of bedrock encountered at the site. Therefore, we believe a passive system, in which water is collected from the perimeter of the site using gravel-filled trenches, will be more appropriate. Subdrains will also likely be required across the base of the excavation to provide a firm, relatively dry base from which to construct the foundation system.

The flow of water in the bedrock is controlled by the number, size, and connectivity of fractures. Therefore, the flow may be too large for a passive system in localized portions of the site, particularly where large fractures or poorly cemented sandstone are encountered. If these conditions are encountered, it may be necessary to install a limited number of dewatering wells behind the excavation, or to install trench drains and drum sumps in front of the temporary shoring system.

The dewatering system should be designed and installed by an experienced contractor. Groundwater removed during dewatering should be disposed of in accordance with applicable state and local regulations.

Typically, dewatering can induce settlement of adjacent structures and improvements when the water level is lowered around the perimeter of the site. However, because the site is generally underlain by interbedded deposits of stiff clays and dense sands and gravels overlying bedrock, we conclude the potential for dewatering-induced ground settlement is low.

6.3.2 Permanent Dewatering

We believe permanent dewatering to reduce hydrostatic pressures may result in a more economical foundation design than a fully waterproofed building with uplift anchors to resist hydrostatic uplift forces for buildings at Sites A and B. The permanent dewatering system should consist of an at least 12-inch-thick layer of Class 2 permeable material containing parallel collection pipes spaced approximately every 20 feet, center-to-center. The collection pipes should consist of four-inch-diameter, Schedule 40, perforated PVC pipes. The collection pipes should be connected to at least one sump pit. The groundwater seeping beneath the building should be continuously collected in the sump and then pumped into the storm drain or sanitary sewer system for appropriate discharge.

In addition to the underslab drainage system, wall backdrains should be installed along the perimeter walls to reduce the hydrostatic pressures acting on walls and to reduce the amount of water collected by the underslab drain. The wall backdrains may be connected to the underslab drain system.

The design of the permanent dewatering system should consider the possibility that a portion of the drain may fail, resulting in an unwanted build-up of hydrostatic pressures which could damage he foundation or basement walls. Inspection and maintenance programs should be implemented to verify that the system is performing as expected during the life of the structure. These programs should ensure that replacement parts, including pumps, are readily available in case of mechanical failure. In addition, it may be desirable to install “pop-off” valves in the

garage floor slab to allow water to drain into the garage in case hydrostatic pressure build up beneath the floor slab.

6.4 Temporary Cut Slopes and Shoring

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 shoring engineer should be responsible for shoring design. The contractor should be responsible for the construction and safety of temporary slopes.

We anticipate excavations for proposed buildings at Sites A, B, and C will extend to depths up to about 20, 20, and 10 feet bgs, respectively. Where space permits, the sides of the temporary excavation can be sloped. Where space does not permit sloping of the excavation perimeter, temporary shoring systems will be required to support the sides of the proposed excavations. We judge that a cantilevered soldier pile and lagging shoring system is appropriate for support of excavations that are less than 12 feet deep. Where the excavation depth exceeds 12 feet, a cantilevered system may be uneconomical and tiebacks may be required. Where it is not feasible to install tiebacks, then internal bracing of the excavation will be required.

6.5 Soil Corrosivity

Corrosivity testing was performed by Sunland Analytical of Rancho Cordova, California on soil samples obtained from Borings B-4 and B-11 at depths of 1.5 and 2 feet bgs, respectively. The results of the corrosivity tests are presented in Appendix B of this report. The resistivity test results indicate the near-surface soil is at the site highly corrosive to buried steel. Accordingly, buried iron, steel, cast iron, galvanized steel, and dielectric-coated steel or iron should be properly protected against corrosion. The chloride ion concentrations and pH of the soil samples are determined to be mildly to moderately corrosive to buried metallic and concrete structures. The sulfide and sulfate ion concentrations of the soil do not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

6.6 Excavation, Monitoring, and Construction Considerations

Some excavation of bedrock may be required for construction of proposed basements. The bedrock is soft to low hardness and deeply weathered near the surface and grades harder and less weathered with depth. We anticipate the weathered rock within the proposed excavation can be excavated with conventional grading equipment (excavators and bull dozers). However, because the bedrock was only investigated in widely spaced borings during our investigation, it is possible that harder rock and difficult drilling or excavation may be encountered in other parts of the site. Therefore, the contractors involved in shoring installation and excavation for basements should be prepared to excavate hard rock, including the possible use of hydraulic breaking equipment, and should bid the project accordingly.

The contractor should establish survey points on the shoring and on adjacent buildings and streets within 30 feet of the excavation perimeter prior to the start of excavation. These survey points should be used to monitor the vertical and horizontal movements of the shoring and surrounding structures and streets during construction. The contractor should also survey and take photographs of any adjacent buildings prior to the start of excavation and immediately after its completion.

During excavation, the shoring system may deform laterally, which could cause the ground surface adjacent to the shoring wall to settle. The magnitudes of shoring movements and the resulting settlements are difficult to estimate because they depend on many factors, including the method of installation and the contractor's skill in the shoring installation. Ground movements due to a properly designed and constructed shoring system should be within ordinary accepted limits of about one inch. A monitoring program should be established to evaluate the effects of the excavation on the adjacent buildings and surrounding ground.

7.0 RECOMMENDATIONS

Our recommendations for site preparation and grading, design of foundations and basement walls, temporary cut slopes and shoring, permanent dewatering, and other geotechnical aspects of the project are presented in this section.

7.1 Site Preparation and Grading

In areas to receive improvements (including buildings, pavements, and exterior concrete slabs), site preparation should include stripping existing vegetation and the upper soil containing over three percent organic matter. Site demolition should include the removal of 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 footprint and will not interfere with the proposed construction, they may be abandoned in-place as allowed by utility company. Voids resulting from demolition activities should be properly backfilled with compacted fill following the recommendations provided later in this section.

Demolished asphalt concrete should be taken to an asphalt recycling facility. The aggregate base underlying the asphalt pavement may be reused as select fill, provided it meets the requirements presented in Section 7.1.2.

7.1.1 Subgrade Preparation

To mitigate the detrimental effects of expansive near-surface clay, slabs-on-grade constructed at-grade or less than three feet bgs should be underlain by at least 12 inches (measured below the bottom of capillary moisture break) of non-expansive soil consisting of select fill or lime-treated onsite soil. Where the upper 12 inches of the at-grade building pad subgrade consists of low-plasticity soil, replacement with select fill or lime-treatment will not be required. The non-expansive soil should extend at least five feet beyond the perimeter of the proposed buildings where concrete flatwork or pavements will be constructed, except where constrained by the property line. The subgrade for proposed concrete flatwork and pavers should also be overexcavated to accommodate at least 12 inches of non-expansive soil.

Where the finished floor will be located below a depth of three feet bgs, the floor slab will be located below the potential zone of moisture change. Therefore, there should be no volumetric changes in the soil and mitigation of the expansion potential of the soil is not required beneath floor slabs founded at least three feet bgs.

The soil subgrade beneath proposed improvements or areas to be filled should be scarified to a depth of at least eight inches, and moisture-conditioned and compacted to the requirements presented in Table 2.

TABLE 2
Summary of Compaction Requirements

Location	Required Relative Compaction[7] (percent)	Moisture Requirement
Building pad – expansive clay	87 – 92	4+% above optimum
Building pad – low-plasticity soil	90+	Above optimum
Exterior slabs – expansive clay	87 – 92	4+% above optimum
Exterior slabs – low-plasticity soil	90+	Above optimum
Pavements – expansive clay	90+	2+% above optimum
Pavements – low-plasticity soil	95+	Above optimum
Pavements - aggregate base	95+	Near optimum
General fill – expansive clay	87 – 92	4+% above optimum
General fill – low-plasticity soil	90+	Above optimum
General fill – granular soil	95+	Near optimum
Utility trench backfill – expansive clay	87 – 92	4+% above optimum
Utility trench backfill – low-plasticity	90+	Above optimum
Utility trench – granular soil	95+	Near optimum

*Low-plasticity soil includes select fill and lime-treated onsite soil.
*Granular soil includes sand and gravel with less than 10 percent fines content.

Scarification and recompaction will not be required for basement floor subgrade where bedrock is exposed, provided the subgrade is maintained in a moist condition. It is also important that the

[7] 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.

moisture content of soil subgrade is sufficiently high to reduce the expansion potential. If the grading work is performed during the dry season, moisture-conditioning will be required.

If grading work is performed during the rainy season, the contractor may find the subgrade material too wet to compact to the recommended relative compaction and will have to be scarified and aerated to lower its moisture content so the specified compaction can be achieved. Material to be dried by aeration should be scarified to a depth of at least 12 inches; the scarified soil should be turned at least twice a day to promote uniform drying. Once the moisture content of the aerated soil has been reduced to acceptable levels, the soil should be compacted in accordance with our recommendations. Aeration typically is the least costly method used to stabilize the subgrade soil; however, it generally requires the most time to complete. Other soil stabilization alternatives include overexcavating and placing drier material, and lime-treatment. Recommendations for various subgrade stabilization options are presented below in Section 7.1.4.

If soft or poorly cemented sedimentary rock is encountered at the base of the excavation, the subgrade should be cut with a smooth bucket and hand-cleaned. Moist or wet siltstone, mudstone, or claystone bedrock may be susceptible to softening or disturbance by construction vehicles, particularly if there is standing water on the subgrade. If extensive portions of the subgrade are saturated at the time of grading, the subgrade may be disturbed. If the subgrade is disturbed, the soft, disturbed material should be overexcavated to expose firm material. The overexcavated areas should be backfilled with compacted aggregate base.

7.1.2 Fill Quality and Compaction

On-site soil may be used as general fill, provided the material is free of organic matter, contain no rocks or lumps larger than three inches in greatest dimension, and be approved by the Geotechnical Engineer. Select fill should consist of on-site or imported soil that is free of organic matter, contains no rocks or lumps larger than four inches in greatest dimension, has a liquid limit of less than 40 and a plasticity index lower than 12, and is approved by the Geotechnical Engineer. Samples of proposed fill material should be submitted to the

Geotechnical Engineer at least three business days prior to use at the site. The grading contractor should provide analytical test results or other suitable environmental documentation indicating the imported fill is free of hazardous materials at least three days before use at the site. If this data is not available, up to two weeks should be allowed to perform analytical testing on the proposed imported material. Fill should be placed in horizontal lifts not exceeding eight inches in uncompacted thickness, and moisture-conditioned and compacted to the requirements presented in Table 2.

7.1.3 Lime-Treatment

If the non-expansive soil to be placed beneath the at-grade building pads and exterior concrete flatwork and pavement is to consist of lime-treated on-site soil, the upper 12 inches of the soil subgrade should be treated in place with about four percent dolomitic quicklime by dry weight of soil. The dry weight of soil should be assumed to be 110 pounds per cubic foot (pcf) for calculating lime quantities. A specialty subcontractor should perform the lime treatment. Prior to lime treatment, we recommend the site be graded to a level pad elevation in accordance with our previous recommendations and all below-grade obstructions removed. The soil treated with lime should be mixed and compacted in one lift. The lime should be thoroughly blended with the soil and allowed to set for 24 hours prior to remixing and compaction. The lime-treated soil should be moisture-conditioned to above optimum moisture content and compacted to at least 90 percent relative compaction.

It should be noted that disposal of lime-treated soil is typically expensive because of the high pH of the treated soil. In addition, lime-treated soil should be completely removed from landscaping areas.

7.1.4 Soil Subgrade Stabilization

In some areas, soft, wet soil may be exposed during grading, causing the subgrade to deflect and rut under the weight of grading equipment. Although, the majority of the soil beneath the site consists of stiff to hard clay, if heavy wheeled equipment is used close to the water table, these

materials may become disturbed and soften. In these areas, some form of subgrade stabilization may be required. Several options for stabilizing subgrade are presented below.

Aeration

Aeration consists of mixing and turning the soil to naturally lower the moisture content to an acceptable level. Aeration typically requires several days to a week of warm, dry weather to effectively dry the material. Material to be dried by aeration should be scarified to a depth of at least 12 inches; the scarified material should be turned at least twice a day to promote uniform drying. Once the moisture content of the aerated soil has been reduced to acceptable levels, the soil should be compacted in accordance with our previous recommendations. Aeration is typically the least costly subgrade stabilization alternative; however, it generally requires the most time to complete and may not be effective if the soft material extends to great depths. Aeration will likely not be effective where the podium subgrade extends below or near the groundwater table; however, it depends on the time of year construction is performed.

Overexcavation

Another method of achieving suitable subgrade in areas where soft, wet soil is exposed is to overexcavate the soft subgrade soil and replace it with drier, granular material. If the soft material extends to great depths, the upper 18 to 24 inches of soft material may be overexcavated and a geotextile tensile fabric (Mirafi 500X or equivalent) placed beneath the granular backfill to help span over the weaker material. The fabric should be pulled tight and placed at the base of the overexcavation, extending at least two feet laterally beyond the limits of the overexcavation in all directions. The fabric should be overlapped by at least two feet at all seams. Granular material such as Class 2 aggregate base should then be placed and compacted over the geotextile tensile fabric.

Where very soft subgrade conditions are encountered, a bi-directional geogrid, such as Tensar TriAx TX-140 or equivalent, may be required in lieu of tensile fabric. Where geogrids are used the depth of overexcavation will likely be on the order of 12 to 18 inches. The geogrids should be overlapped by at least two feet and tied with hog rings or nylon ties at a spacing not to exceed

10 feet. The geogrids should be covered with a well-graded granular fill such as Class 2 aggregate base; open-graded rock should not be used. All backfill placed over the geogrid should be compacted in accordance with our previous recommendations.

Chemical Treatment

Lime and/or cement have been successfully used to dry and stabilize fine-grained soils with varying degrees of success. Lime- and/or cement-treatment will generally decrease soil density, change its plasticity properties, and increase its strength. The degree to which lime will react with soil depends on such variables as type of soil, mineralogy, quantity of lime, and length of time the lime-soil mixture is cured. Cement is generally used when a significant amount of granular material or low-plasticity silt is present in the soil. The quantity of lime and/or cement added generally ranges from 3 to 7 percent by weight and should be determined by laboratory testing. The specialty contractor performing the chemical treatment should select the most appropriate additive and quantity for the soil conditions encountered.

If chemical treatment is used to stabilize soft subgrade, a treatment depth of about 18 inches below the final soil subgrade will likely be required. The soil being treated should be scarified and thoroughly broken up to full depth and width. The treated soil should not contain rocks or soil clods larger than three inches in greatest dimension. Treated soil should be compacted to at least 90 percent relative compaction, and at least 95 percent relative compaction in the upper six inches of vehicular pavement subgrade.

7.1.5 Exterior Flatwork Subgrade Preparation

We recommend exterior concrete flatwork, including patio slabs, pavers, and sidewalks, be underlain by at least 12 inches of non-expansive soil consisting of on-site low-plasticity soil, select fill, or lime-treated on-site soil. Even with 12 inches of non-expansive soil, 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.

7.1.6 Utility Trench Backfill

Excavations for utility trenches can be readily made with a backhoe. All trenches should conform to the current CAL-OSHA requirements. To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of sand or fine gravel. After the pipes and conduits are tested, inspected (if required) and approved, they should be covered to a depth of six inches with sand or fine gravel, which should be mechanically tamped. The pipe bedding and cover should be eliminated where an impermeable plug is required as described below. Backfill for utility trenches and other excavations is also considered fill, and should be placed and compacted as according to the recommendations previously presented. If imported clean sand or gravel (defined as soil with less than 10 percent fines) is used as backfill, it should be compacted to at least 95 percent relative compaction. Jetting of trench backfill should not be permitted. Special care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to the pavement section.

Foundations for the proposed building should be bottomed below an imaginary line extending up at a 1.5:1 (horizontal to vertical) inclination from the base of utility trenches. Alternatively, the portion of the utility trench (excluding bedding) that is below the 1.5:1 line can be backfilled with controlled low-strength material (CLSM) with a 28-day unconfined compressive strength of at least 100 pounds per square inch (psi).

Where utility trenches enter at-grade building pads, an impermeable plug consisting of CLSM, at least three feet in length, should be installed where the trenches enter the building footprint. Furthermore, where sand- or gravel-backfilled trenches cross planter areas and pass below asphalt or concrete pavements, a similar plug should be placed at the edge of the pavement. The purpose of these recommendations is to reduce the potential for water to become trapped in trenches beneath the building or pavements. This trapped water can cause heaving of soils beneath slabs and softening of subgrade soil beneath pavements.

7.1.7 Geotechnical Services during Grading

All grading and site development plans should be reviewed by the Geotechnical Engineer to verify that they conform to the intent of our recommendations. The Geotechnical Engineer should be present and onsite to observe demolition and related fill placement, site grading, subgrade preparation, and fill and subdrain placement. In addition, the Geotechnical Engineer should perform nuclear field density tests to check the moisture content and relative compaction of soil subgrade, engineered fill, and utility trench backfill. These observations and tests will allow us to compare actual with anticipated soil conditions and to check that the contractor's work conforms to the geotechnical aspects of the plans and specifications. If any variations or undesirable conditions are encountered during construction, we should be notified so that additional recommendations can be made.

7.2 Spread Footings

7.2.1 Buildings at Sites A and B

The proposed buildings at Sites A and B will have basements and may be supported on continuous and/or individual spread footings bearing on firm native soil or bedrock. Footings should be bottomed at least 12 inches below the adjacent interior soil subgrade (measured from the base of the underslab drain rock. Continuous footings should be at least 18 inches wide and isolated spread footings should be at least 24 inches wide. We recommend footings founded on firm native soil be designed using allowable bearing pressures of 4,000 pounds per square foot (psf) for dead-plus-live loads and 5,300 psf for total design loads, which include wind or seismic forces; these values include factors of safety of at least 2.0 and 1.5, respectively. Footings founded on bedrock may be designed using allowable bearing pressures of 10,000 psf for dead-plus-live loads and 13,000 psf for total design loads, which include wind or seismic forces; these values include factors of safety of at least 2.0 and 1.5, respectively. The approximate elevations of the top of bedrock encountered at boring locations are shown on Figure 2.

Lateral loads may be resisted by a combination of passive pressure on the vertical faces of the footings and friction between the bottoms of the footings and the supporting soil/bedrock. To

compute lateral resistance for footings excavated into soil, we recommend using an equivalent fluid weight of 270 pounds per cubic foot (pcf); and frictional resistance should be computed using a base friction coefficient of 0.35. To compute lateral resistance for footings excavated into bedrock, we recommend using an equivalent fluid weight of 500 pcf; and frictional resistance should be computed using a base friction coefficient of 0.5. The upper foot of soil/bedrock should be ignored for passive resistance unless confined by a slab or pavement. 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.

Footing excavations should be free of standing water, debris, and disturbed materials prior to placing concrete. The bottoms and sides of the footing excavations should be moistened following excavation and maintained in a moist condition until concrete is placed. If the foundation soil dries during construction, the footing will eventually heave, which may result in cracking and distress. Where water infiltration is encountered during footing excavations, we recommend protecting the footing subgrades from softening due to water infiltration by placing a two-inch-thick mud slab at the bottom of the excavations immediately after they are excavated. We should check footing excavations prior to placement of reinforcing steel and concrete.

7.2.2 Building at Site C

We recommend the proposed building at Site C be supported on individual spread footings at interior column locations and continuous, deepened perimeter footings. Continuous footings should be at least 18 inches wide and isolated spread footings should be at least 24 inches wide. Perimeter footings should be bottomed at least 36 inches below the lowest adjacent outside grade. The perimeter footing embedment depth may be decreased by six inches where pavement or concrete flatwork is adjacent to the building. Interior footings should extend at least 24 inches below the bottom of the capillary moisture break. Spread footings for proposed at-grade buildings should bear on firm native soil and 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 may be resisted by a combination of passive pressure on the vertical faces of the footings and friction between the bottoms of the footings and the supporting soil. To compute lateral resistance, we recommend using an equivalent fluid weight of 270 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 0.35. 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.

Footing excavations should be free of standing water, debris, and disturbed materials prior to placing concrete. The bottoms and sides of the footing excavations should be moistened following excavation and maintained in a moist condition until concrete is placed. If the foundation soil dries during construction, the footing will eventually heave, which may result in cracking and distress. We should check footing excavations prior to placement of reinforcing steel and concrete.

7.3 Permanent Dewatering System

For proposed buildings at Sites A and B, a permanent dewatering system will be required beneath the basement slab-on-grade floors to prevent hydrostatic uplift pressure on the slabs. We recommend the permanent dewatering system consist of an at least 12-inch-thick layer of Class 2 permeable material containing parallel collection pipes spaced approximately every 20 feet, center-to-center. The Class 2 permeable material should meet the requirements of Caltrans Standard Specifications 68-1.025 most recent edition. The collection pipes should consist of four-inch-diameter, Schedule 40, perforated PVC pipe (perforations down). The pipes should be installed such that they are surrounded on all sides by at least four inches of permeable material. The pipes should drain at a gradient of at least one percent to at least one sump pit location in the garage level. If manifold pipes are required to collect the water from individual collector pipes, they should consist of solid PVC pipes of appropriate diameter to collect the required volume of water. The sump system should be designed to discharge water directly to the storm drain system without allowing the water to build up beneath the slab-on-grade floor.

7.4 Floor Slabs

Where water vapor transmission through the floor slab is undesirable, we recommend installing a water vapor retarder beneath the floor. A vapor retarder is 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 be installed below the slab-on-grade in utility rooms and any areas in or adjacent to the parking garage that will be used for storage and/or will receive a floor covering or coating.

A capillary moisture break consists of at least four inches of clean, free-draining gravel or crushed rock. The vapor retarders for Sites A and B should meet the requirements for Class A and the vapor retarder for Site C 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. *If required by the structural engineer*, the vapor retarder may be covered with two inches of sand to aid in curing the concrete and to protect the vapor retarder during slab construction. The sand overlying the vapor retarder should be moist at the time concrete is placed. However, excess water trapped in the sand could eventually be transmitted as vapor through the slab. Therefore, if rain is forecast prior to pouring the slab, the sand should be covered with plastic sheeting to avoid wetting. If the sand becomes wet, concrete should not be placed until the sand has been dried or replaced.

The particle size of the capillary break material and sand (if used) should meet the gradation requirements presented in Table 3.

TABLE 3
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
Sand	
No. 4	100
No. 200	0 – 5

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 0.50. If the concrete is poured directly over the vapor retarder (no sand layer), we recommend the w/c ratio of the concrete not exceed 0.45 and water not be added in the field. If necessary, workability should be increased by adding plasticizers. In addition, the slabs should be properly cured. Before floor coverings, if any, are placed, the contractor should check that the concrete surface and the moisture emission levels (if emission testing is required) meet the manufacturer's requirements.

7.5 Permanent Retaining and Basement Walls

Permanent retaining and basement walls should be designed to resist the lateral earth pressure imposed by the retained soil and bedrock, as well as a surcharge pressure from nearby vehicles and structures, where appropriate. We recommend restrained permanent walls at the site be designed using an at-rest equivalent fluid weight of 60 pcf and unrestrained permanent walls be designed using an active equivalent fluid weight of 40 pcf. To evaluate restrained and unrestrained permanent walls for seismic loading, we recommend using an active equivalent fluid weight of 40 pcf plus a seismic increment of 5 pcf.

Where traffic loads are expected within 10 feet of the walls, an additional design load of 100 psf should be applied to the upper ten feet of the wall. Where neighboring and existing foundations are supported above a "zone-of-influence" line extending up from the bottom of a permanent wall at an inclination of 1.5:1 (horizontal: vertical), the wall should be designed for a surcharge pressure. We can provide the surcharge pressure once the dimensions and depth of the neighboring foundations are known.

The design pressure recommended above is based on fully drained walls. One acceptable method for backdraining a wall is to place a prefabricated drainage panel against the back of the wall. The drainage panel should extend down to a perforated PVC collector pipe at the base of the wall. The pipe should be surrounded on all sides by at least four inches of Caltrans Class 2 permeable material or 3/4-inch drain rock wrapped in filter fabric (Mirafi NC or equivalent). AdvanEdge pipe or equivalent may be used in lieu of the pipe and permeable material. The pipe should be sloped to drain to a sump or another suitable outlet.

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

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

7.6 Temporary Cut Slopes and Shoring

The safety of workers and equipment in or near the excavation is the responsibility of the contractor. The selection, design, construction, and performance of the shoring system should be the responsibility of the contractor. A structural engineer knowledgeable in this type of construction should design the shoring. We should review the geotechnical aspects of the proposed shoring system to ensure that it meets our requirements. During construction, we

should observe the installation of the shoring system and check the condition of the soil encountered during excavation.

We judge that temporary cuts which are less than 15 feet high, above groundwater, and inclined no steeper than 1.5:1 (horizontal:vertical) in sediments and 3/4:1 in weathered rock, will be stable provided that they are not surcharged by equipment or building material. Temporary shoring will be required where temporary slopes are not possible because of space constraints.

A structural/civil engineer knowledgeable in soldier-pile-and-lagging temporary shoring systems should be retained to design the shoring. The shoring designer should design the shoring system for lateral deformation of less than one inch at any location on the shoring. We should review the final shoring plans and calculations to check that they are consistent with the recommendations presented in this report.

7.6.1 Cantilevered Soldier Pile and Lagging Shoring System

We recommend a cantilevered soldier pile-and-lagging shoring system be designed to resist an active equivalent fluid weight of 40 pcf. In locations where minimizing lateral deflections is critical, such as near adjacent buildings or near sensitive underground utilities, the shoring system should be designed to resist an at-rest equivalent fluid weight of 60 pcf, plus any foundation surcharge loads. Where traffic loads are expected within 10 feet of the shoring walls, an additional design load of 100 psf should be applied to the upper ten feet of the wall. Where construction equipment will be working behind the walls within a horizontal distance equal to the wall height, the design should include a surcharge pressure of 250 psf. The above pressures should be assumed to act over the entire width of the lagging installed above the excavation. The active pressure need only be assumed to act over one pile width below the bottom of the excavation.

Passive resistance at the toe of the soldier pile should be computed using an equivalent fluid weights of 270 and 500 pcf in soil and bedrock, respectively, with a maximum passive earth pressure of 2,500 psf and 4,000 psf in soil and bedrock, respectively. The upper foot of soil should be ignored when computing passive resistance. Passive pressure can be assumed to act

over an area of three soldier pile widths assuming the toe of the soldier pile is filled with structural concrete. If lean concrete is placed in the soldier pile shaft, the passive pressure can be assumed to act over two pile diameters. These passive pressure values include a factor of safety of at least 1.5.

Soldier piles should be placed in pre-drilled holes backfilled with concrete. Where granular layers are present below the groundwater, installation of the soldier piles may require casing or use of drilling slurry to reduce caving of the holes. Installing soldier piles by driving or using vibratory methods is acceptable, but should not be permitted within 25 feet of existing structures.

7.6.2 Soldier Pile and Lagging Shoring System with Tiebacks

Recommended lateral pressures for the design of soldier pile and lagging shoring with tiebacks are presented on Figure 5. Where it is not feasible to install tiebacks, then internal bracing of the excavation will be required. Internal bracing should be preloaded to limit movement of the shoring. The penetration of the soldier piles must be sufficient to ensure stability and resist the downward loading of tiebacks.

Passive resistance at the toe of the soldier piles should be computed using an equivalent fluid weights of 270 and 500 pcf in soil and bedrock, respectively, with a maximum passive earth pressure of 2,500 psf and 4,000 psf in soil and bedrock, respectively. These values include factors of safety of at least 1.5. Passive pressure can be assumed to act over an area of three soldier pile widths provided the toe of the soldier pile hole is filled with structural concrete. The upper foot of soil should be ignored when computing passive resistance. Recommendations for the installation of soldier piles and lagging are presented in the previous Section 7.6.1.

Vertical loads can be resisted by skin friction along the portion of the soldier piles below the excavation. We recommend using an allowable skin friction value of 1,000 psf to compute the required soldier pile embedment. End bearing should be neglected.

Design criteria for tiebacks are also presented on Figure 5. As shown, tiebacks should derive their load-bearing capacity from the soil behind an imaginary line sloping upward from a point

H/5 feet away from the bottom of the excavation at an angle of 60 degrees from horizontal, where H is the wall height in feet. The minimum stressing lengths for strand and bar tendons should be 15 and 10 feet, respectively. The minimum bond length for strand and bar tendons should both be 15 feet.

Allowable capacities of the tiebacks will depend upon the drilling method, hole diameter, grout pressure, and workmanship. The shoring contractor should be prepared to use smooth-cased method (such as a Klemm rig) to install the tiebacks where caving soil is encountered.

The shoring designer should be responsible for determining the actual length of tieback required. The determination should be based on the designer's familiarity with the installation method to be used.

7.6.3 Tieback Testing

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

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

minutes to 60 minutes, and the movements should be recorded at 15, 20, 25, 30, 45, and 60 minutes.

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

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

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

7.7 Pavement Design

Design recommendations for asphalt concrete and Portland cement concrete pavements and concrete pavers are presented in the following sections.

7.7.1 Flexible (Asphalt Concrete) Pavement Design

The State of California flexible pavement design method was used to develop the recommended asphalt concrete (AC) pavement sections. Resistance value (R-value) tests performed on near-surface soil samples collected from Borings B-1 and B-10 indicate the material has R-values of 2 and 4, respectively. For AC pavement design, we selected a minimum recommended R-value of

5. Recommended pavement sections for traffic indices ranging from 4.5 to 8.0 are presented in Table 4.

TABLE 4
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
6.0	4.0	11.5
7.0	4.5	14.5
8.0	5.0	17.5

The soil subgrade beneath AC pavements should be prepared and compacted in accordance with the recommendations presented in Section 7.1. In addition, the subgrade should be a firm and non-yielding surface. The subgrade should be proof-rolled to confirm it is non-yielding prior to placing the aggregate base. The Class 2 aggregate base should be moisture-conditioned to near optimum moisture content and compacted to at least 95 percent relative compaction.

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

7.7.2 Rigid (Portland-Cement Concrete) Pavement

Concrete pavement design is based on a maximum single-axle load of 20,000 pounds and a maximum tandem axle load of 32,000 pounds and light truck traffic (i.e., a few trucks per week). The recommended rigid pavement section for these axle loads is six inches of Portland cement concrete over six inches of Class 2 aggregate base. Where fire truck traffic is expected, the

pavement section should consist of 6-1/2 inches of Portland cement concrete over six inches of Class 2 aggregate base.

The modulus of rupture of the concrete should be at least 500 psi at 28 days. Contraction joints should be constructed at 15-foot spacing. Where the outer edge of a concrete pavement meets asphalt concrete pavement, the concrete slab should be thickened by 50 percent at a taper not to exceed a slope of 1 in 10. For areas that will receive weekly garbage truck traffic, we recommend the slab be reinforced with a minimum of No. 4 bars at 16-inch spacing in both directions. Recommendations for subgrade preparation and aggregate base compaction for concrete pavement are the same as those described above for AC pavement.

7.8 Drainage and Landscaping

Positive surface drainage should be provided around the buildings 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 at-grade buildings should be avoided to reduce the amount of water introduced to the expansive clay subgrade.

Prior experience and industry literature indicate that some species of high water-demand[8] trees can induce ground-surface settlement by drawing water from the expansive clay, causing it to shrink. Where these types of trees are planted near buildings, the ground-surface settlement may result in damage to structure. This problem usually occurs 10 or more years after planting, as the trees reach mature height. To reduce the risk of tree-induced, building settlement, we recommend trees of the following genera are not planted within 25 feet of the proposed building: *Eucalyptus, Populus, Quercus, Crataegus, Salix, Sorbus* (simple-leafed), *Ulmus, Cupressus,*

[8] "Water-demand" refers to the ability of the tree to withdraw large amounts of water from the soil subgrade, rather than soil suction exerted by the root system.

Chamaecyparis, and *Cupressocyparis.* Because this is a limited list and does not include all genera that may induce ground-surface settlement, a tree specialist should be consulted prior to selection of trees to be planted at the site.

Care should be taken to minimize the potential for subsurface water to collect beneath pavements and pedestrian walkways. Where landscape beds and tree wells are immediately adjacent to pavements and flatwork that 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 aggregate base. These barriers may consist of either flexible impermeable membranes or deepened concrete curbs.

7.8.1 Bio-retention

We anticipate proposed bio-retention areas, if any, at the site will be constructed on predominantly clayey soil. The rate of infiltration for the clayey soil that will underlie the majority of the proposed bio-retention areas is expected to be very low; and the near-surface clay at the site is moderately to highly expansive. The primary concerns with the proposed bio-retention areas are: (1) providing suitable support for foundations and curbs constructed near the bio-retention areas, and (2) potential for subsurface water from the bio-retention areas to migrate (and possibly build up) beneath pavements and the proposed buildings. Therefore, we recommend the following for design of bioswales:

- We recommend bio-retention areas constructed at the site should be provided with underdrains and/or drain inlets because of the low permeability of the near-surface soil.
- Bio-retentions areas should be constructed no closer than 5 feet from the new buildings and pavements.
- The sides of bio-retention areas should be sloped 1:1 (horizontal:vertical).
- Where a vertical curb or foundation is constructed near a bio-retention area, the curb and the edge of the foundation should be founded below an imaginary line extending up at an inclination of 1.5:1 (horizontal: vertical) from the base of the bio-retention area.
- Subdrain pipes should be placed at the base of bio-retention areas. The subdrain pipes should be installed 8 inches above the bottom of the infiltration area for treatment areas that are at least 5 feet away from the new buildings and pavements. The intent of this

recommendation is to allow infiltration into the underlying soil, but to reduce the potential for bio-retention areas to flood during periods of heavy rainfall.

Where the proposed bio-retention areas will be located less than 5 feet from the new buildings and pavements due to space constraints, a 4-inch-diameter perforated subdrain pipe should be placed 4 inches above the base of the treatment area. The intent is to allow infiltration into the underlying soil, but to reduce potential for groundwater build-up in the treatment area.

7.9 Seismic Design

For design in accordance with the 2013 CBC, we recommend Site Class C be used for the site. The latitude and longitude of the site are 37.89319 and -122.07771, respectively. Hence, in accordance with the 2013 CBC, we recommend the following:

- S_S = 1.849g, S_1 = 0.603g
- S_{MS} = 1.849g, S_{M1} = 0.784g
- S_{DS} = 1.233g, S_{D1} = 0.523g
- PGA_M = 0.692g
- Seismic Design Category D for Risk Categories I, II and III.

8.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

Prior to construction, Rockridge Geotechnical should review the project plans and specifications to verify that they conform to the intent of our recommendations. During construction, our field engineer should provide on-site observation and testing during site preparation, placement and compaction of fill, and installation of temporary shoring and building 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 study has been conducted in accordance with the standard of care commonly used as state-of-practice in the profession. No other warranties are either expressed or implied.

The recommendations made in this report are based on the assumption that the subsurface conditions do not deviate appreciably from those disclosed in the exploratory borings. 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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Cao, T., Bryant, W. A., Rowshandel, B., Branum D. and Wills, C. J. (2003). The Revised 2002 California Probabilistic Seismic Hazard Maps.

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Jennings, C.W. (1994). Fault Activity Map of California and Adjacent Areas with Locations and Ages of Recent Volcanic Eruptions: California Division of Mines and Geology Geologic Data Map No. 6, scale 1: 750,000.

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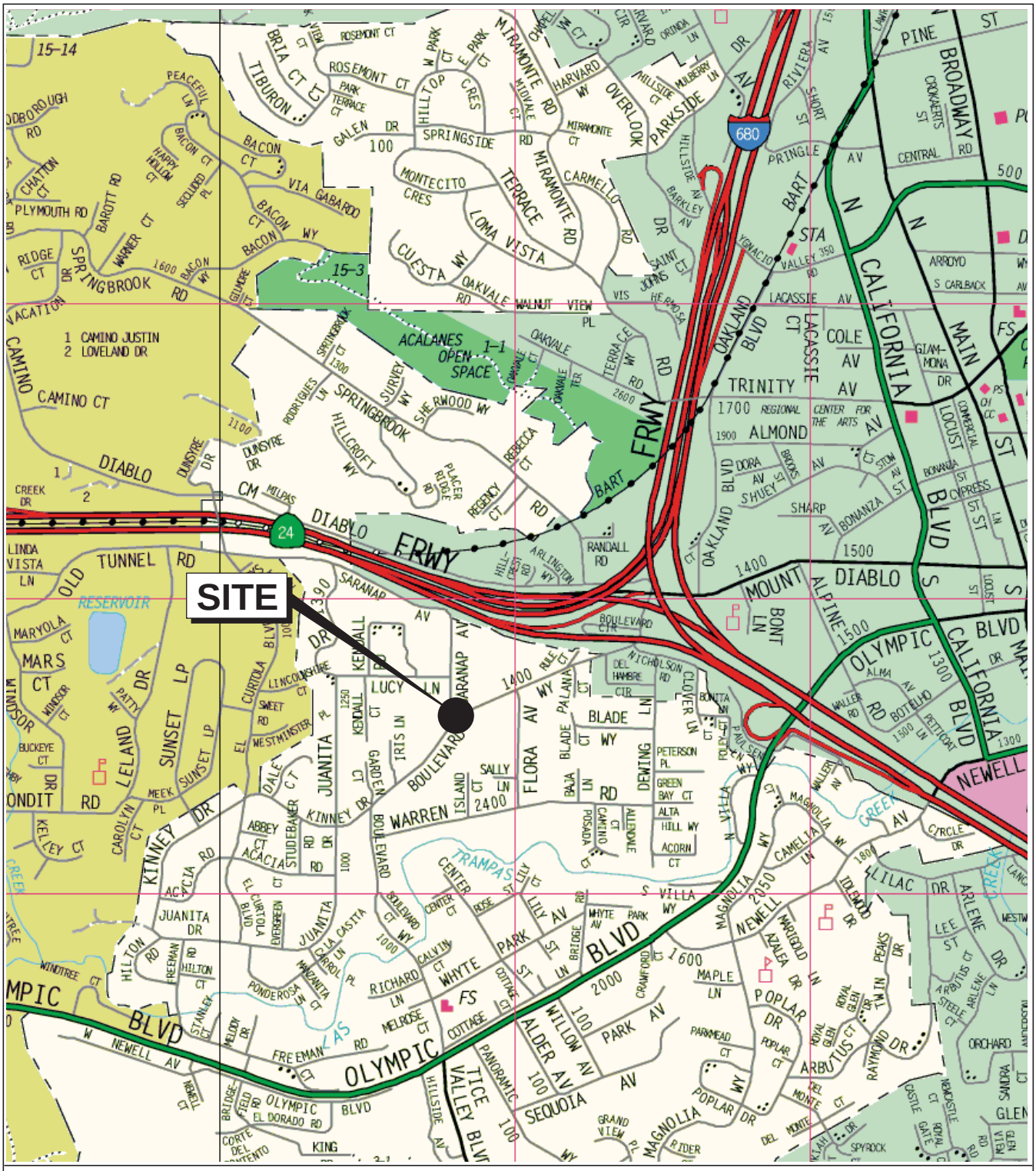
FIGURES



Base map: The Thomas Guide
Contra Costa County
1999

0 1/4 1/2 Mile

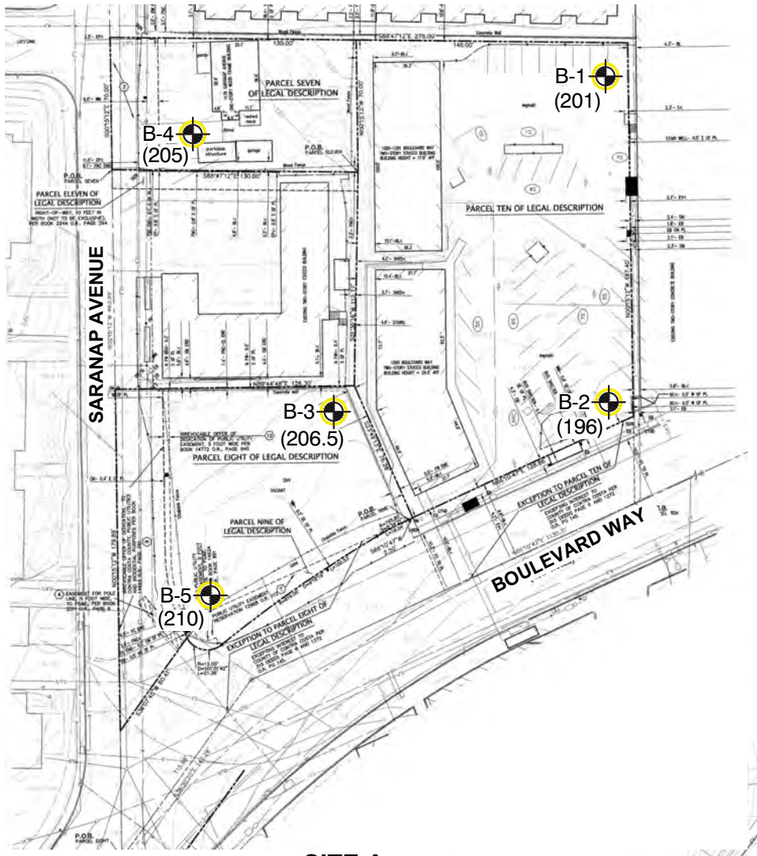
Approximate scale

SARANAP MIXED-USE DEVELOPMENT
BOULEVARD WAY AND SARANAP AVENUE
Contra Costa County, California

ROCKRIDGE GEOTECHNICAL

SITE LOCATION MAP

Date 10/29/13	Project No. 12-477	Figure 1



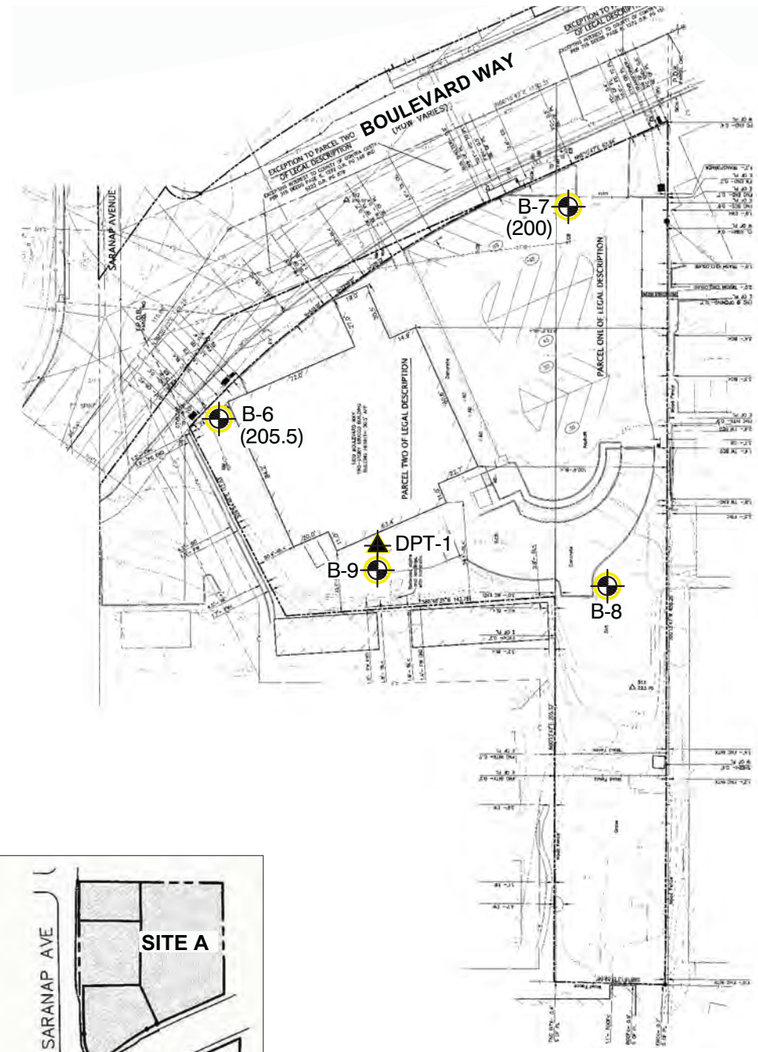
SITE A

EXPLANATION

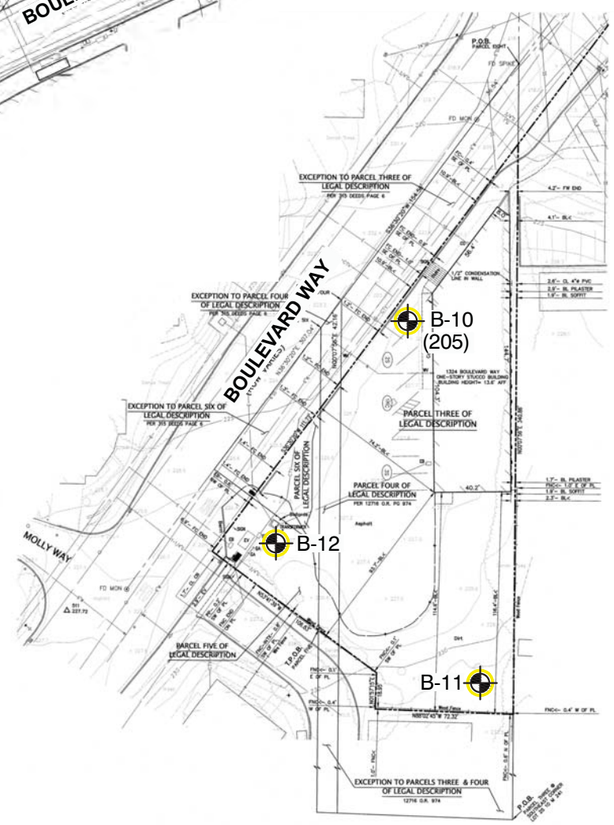
B-1 — Approximate location of test boring by Rockridge Geotechnical, Inc., May 2013

DPT-1 — Approximate location of dynamic penetrometer test by Rockridge Geotechnical, May 2013

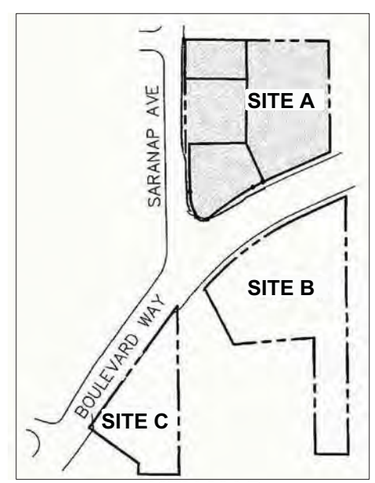
(201) — Approximate top of bedrock elevation in feet, where bedrock was encountered in borings



SITE B



SITE C



KEY MAP

0 80 Feet

Approximate scale

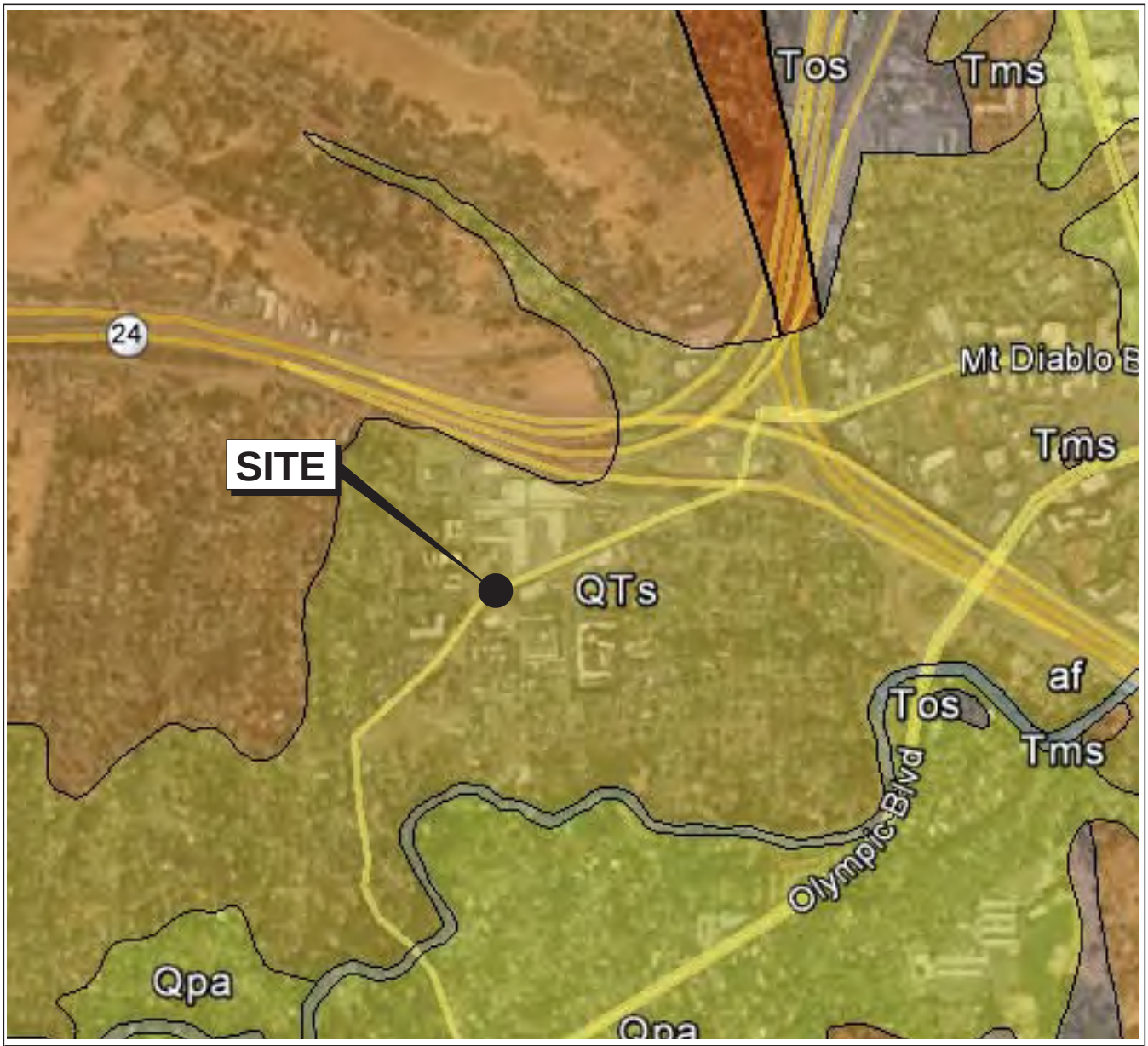
Reference: Base map from a drawing titled "ALTA/ACSM Land Title Survey", by Kier & Wright Surveyors, Inc., dated June 2012.

SARANAP MIXED-USE DEVELOPMENT
BOULEVARD WAY AND SARANAP AVENUE
Contra Costa County, California

SITE PLAN

Date 10/28/13	Project No. 12-477	Figure 2

ROCKRIDGE GEOTECHNICAL



Base map: Google Earth with U.S. Geological Survey (USGS), Contra Costa County, 2012.

EXPLANATION

af	**Artificial Fill**
Qpa	**Alluvium (Pleistocene)**
QTs	**Sediments (early Pleistocene and (or Pilocene)**
Tpms	**Sedimentary rocks (Pliocene and early Miocene)**
Tms	**Sedimentary rocks (Miocene)**
Tos	**Sedimentary rocks (Oligocene)**

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

0 1000 Feet

Approximate scale

SARANAP MIXED-USE DEVELOPMENT
BOULEVARD WAY AND SARANAP AVENUE
Contra Costa County, California

ROCKRIDGE GEOTECHNICAL

REGIONAL GEOLOGIC MAP

Date 10/29/13	Project No. 12-477	Figure 3

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

EXPLANATION

Strike slip

Thrust (Reverse)

Normal

N

0 5 10 Miles

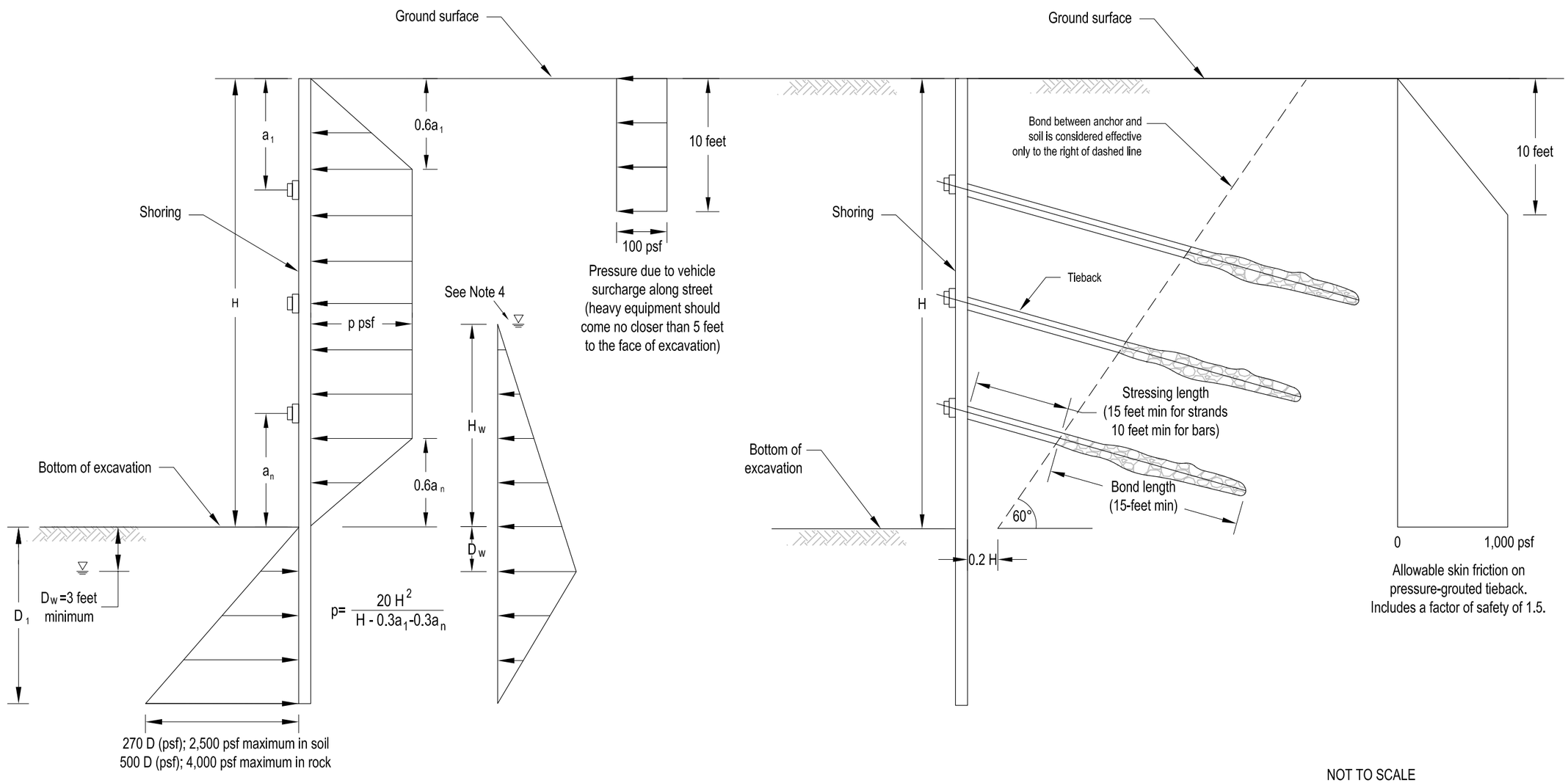
Approximate scale

SARANAP MIXED-USE DEVELOPMENT
BOULEVARD WAY AND SARANAP AVENUE
Contra Costa County, California

ROCKRIDGE GEOTECHNICAL

REGIONAL FAULT MAP

Date 10/29/13	Project No. 12-477	Figure 4



NOT TO SCALE

Notes:

1. Passive pressures include a factor of safety of about 1.5.
2. For soldier piles spaced at more than three times the soldier pile diameter, the passive pressure should be assumed to act over three diameters.
3. Active pressure should be assumed to act over one pile diameter.
4. Design groundwater level at 8 and 10 feet below ground surface for Sites A and B, respectively.

SARANAP MIXED-USE DEVELOPMENT
BOULEVARD WAY AND SARANAP AVENUE
Contra Costa County, California

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

Date 10/29/13	Project No. 12-477	Figure 5

ROCKRIDGE GEOTECHNICAL

APPENDIX A

Boring Logs and DPT Results

PROJECT: **SARANAP MIXED-USE DEVELOPMENT BOULEVARD WAY AND SARANAP AVENUE**
Contra Costa County, California

Log of Boring B-1

Boring location:	See Site Plan, Figure 2
Date started:	5/9/13
Date finished:	5/9/13
Drilling method:	Hollow Stem Auger
Hammer weight/drop:	140 lbs./30 inches
Hammer type:	Downhole Hammer
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by:	R. Nelson

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
					Approximate Ground Surface Elevation: 217.2 feet[2]						
					2 inches asphalt concrete						
					5 inches aggregate base						
1–2	S&H	4 8 12	12	CL	CLAY with SAND (CL) mottled light brown and dark brown, stiff, moist, fine to medium sand, trace fine gravel LL = 44, PI = 30; see Figure B-1 R-value, see Figure B-4					16.6	106
4–5	S&H	12 27 29	34	GC	CLAYEY GRAVEL with SAND (GC) mottled yellow-brown and dark brown, dense, moist, fine gravel, trace sand						
7–8				SC	CLAYEY SAND with GRAVEL (SC) light brown, medium dense, moist, fine to coarse sand, fine gravel						
9	S&H	9 10 13	14		Sieve Analysis, see Figure B-2				30.6	14.7	111
10–11				GC	CLAYEY GRAVEL with SAND (GC) olive-gray, medium dense, moist						
13				CL	CLAY (CL) mottled yellow-brown and brown, very stiff, moist						
14–15	S&H	13 15 20	21		during drilling						
16–25					SILTSTONE gray, low hardness, friable, deep weathering, moist — BEDROCK						
19	S&H	35 50/5"	30/5"								
24–25	SPT	12 28 34	62								

Boring terminated at a depth of 25 feet below ground surface.
Boring backfilled with cement grout under observation of Contra Costa County.
Groundwater encountered at a depth of 15 feet during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively, to account for sampler type and hammer energy.
[2] Elevations based on ALTA/ACSU Land Title Survey for project, dated June 2012, by Kier & Wright Civil Engineers & Surveyors, Inc.

ROCKRIDGE GEOTECHNICAL

Project No.: 12-477 | Figure: A-1

PROJECT: **SARANAP MIXED-USE DEVELOPMENT BOULEVARD WAY AND SARANAP AVENUE**
Contra Costa County, California

Log of Boring B-2

Boring location:	See Site Plan, Figure 2
Date started:	5/9/13
Date finished:	5/9/13
Drilling method:	Hollow Stem Auger
Hammer weight/drop:	140 lbs./30 inches
Hammer type:	Downhole Hammer
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by:	R. Nelson

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION (Approximate Ground Surface Elevation: 214.6 feet[2])	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
					1 inch asphalt concrete						
					6 inches aggregate base						
1–2	S&H	7, 20, 21	25	CL	SANDY CLAY with GRAVEL (CL) dark brown, very stiff, moist, fine sand, fine gravel						
4–5	S&H	13, 40, 50/5"	54/ 11"	CL	olive-gray, hard, trace gravel						
9–10	SPT	4, 5, 8	13	SC	CLAYEY SAND (SC) grades gray to brown, medium dense, wet, fine to coarse grained; Sieve Analysis, see Figure B-2				38.7		
14–15	SPT	13, 15, 17	32	SC	yellow-brown, dense, wet, trace fine gravel, increased coarse sand						
19–20	S&H	17, 20, 50/5"	42		SILTSTONE gray, low hardness, friable, deep weathering, moist (BEDROCK)						
24	SPT	25, 50/6"	50/6"								

Boring terminated at a depth of 25 feet below ground surface.
Boring backfilled with cement grout under observation of Contra Costa County.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively, to account for sampler type and hammer energy.
[2] Elevations based on ALTA/ACSU Land Title Survey for project, dated June 2012, by Kier & Wright Civil Engineers & Surveyors, Inc.

ROCKRIDGE GEOTECHNICAL

Project No.: 12-477 | Figure: A-2

PROJECT: **SARANAP MIXED-USE DEVELOPMENT BOULEVARD WAY AND SARANAP AVENUE**
Contra Costa County, California

Log of Boring B-3

Boring location:	See Site Plan, Figure 2
Date started:	5/9/13
Date finished:	5/9/13
Drilling method:	Hollow Stem Auger
Hammer weight/drop:	140 lbs./30 inches
Hammer type:	Downhole Hammer
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by:	R. Nelson

LABORATORY TEST DATA

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION (Approximate Ground Surface Elevation: 219.0 feet[2])	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
1–2	S&H		13 26 27	32	SP-SC	SAND with CLAY and GRAVEL (SP-SC) mottled yellow-brown and brown, dense, moist, fine to coarse sand, fine gravel						
4–5	S&H		17 32 41	44								
9–10	SPT		11 13 14	27		groundwater observed 5/10/13 prior to grouting medium dense						
12–13						SILTSTONE yellow-brown to brown, low hardness, friable, deep weathering, moist						
14	S&H		23 50/5"	30/5"								
19	SPT		26 50/6"	50/6"		gray						
24–25	SPT		25 26 28	54								

BEDROCK

Boring terminated at a depth of 25 feet below ground surface.
Boring backfilled with cement grout under observation of Contra Costa County.
Groundwater not encountered during drilling. Groundwater observed in borehole prior to grouting on 5/10/13 at 9.1 feet below ground surface.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively, to account for sampler type and hammer energy.
[2] Elevations based on ALTA/ACSU Land Title Survey for project, dated June 2012, by Kier & Wright Civil Engineers & Surveyors, Inc.

ROCKRIDGE GEOTECHNICAL

Project No.: 12-477
Figure: A-3

PROJECT: **SARANAP MIXED-USE DEVELOPMENT BOULEVARD WAY AND SARANAP AVENUE**
Contra Costa County, California

Log of Boring B-4

Boring location:	See Site Plan, Figure 2
Date started:	5/13/13
Date finished:	5/13/13
Drilling method:	Hollow Stem Auger
Hammer weight/drop:	140 lbs./30 inches
Hammer type:	Downhole Hammer
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by:	R. Nelson

Approximate Ground Surface Elevation: 228.6 feet[2]

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
1–2	S&H	■	8 10 14	13	CL-CH	CLAY (CL-CH) mottled brown and light brown, stiff, moist Corrosion Test, see Appendix B						
4–5	S&H	■	21 36 50/5"	50/ 11"	CL-CH	hard LL = 50, PI = 32; see Figure B-1					14.3	106
9–10	SPT	◪	11 14 22	36	CL-CH	trace sand						
14–15	S&H	■	12 15 24	23	CL-CH	very stiff, increase in sand content, trace fine gravel						
17–20					SC	CLAYEY SAND with GRAVEL (SC) mottled brown and light brown, dense, moist						
19–20	SPT	◪	11 14 19	33	SC							
23–25	SPT	◪	14 21 27	48		MUDSTONE gray, soft to low hardness, friable, deeply weathered, moist — BEDROCK						

Boring terminated at a depth of 25 feet below ground surface.
Boring backfilled with cement grout under observation of Contra Costa County.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively, to account for sampler type and hammer energy.
[2] Elevations based on ALTA/ACSU Land Title Survey for project, dated June 2012, by Kier & Wright Civil Engineers & Surveyors, Inc.

ROCKRIDGE GEOTECHNICAL

Project No.: 12-477 | Figure: A-4

PROJECT: **SARANAP MIXED-USE DEVELOPMENT BOULEVARD WAY AND SARANAP AVENUE**
Contra Costa County, California

Log of Boring B-5

Boring location: See Site Plan, Figure 2
Logged by: R. Nelson
Date started: 5/9/13 | Date finished: 5/9/13
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches | Hammer type: Downhole Hammer
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

LABORATORY TEST DATA

Approximate Ground Surface Elevation: 218.0 feet[2]

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
1–2	S&H	8 12 16	17	CL	SANDY CLAY (CL) dark brown, very stiff, moist, fine to coarse sand						
4–5	S&H	17 28 46	44	CL	light-brown, hard						
5–8				SC	CLAYEY SAND (SC) mottled yellow-brown and brown, dense, moist						
9–10	S&H	23 33 38	43		SILTSTONE olive-gray, soft to low hardness, plastic to friable, deep weathering, moist						
14–15	SPT	7 13 21	34								
19	S&H	50/6"	30/6"		grades to low hardness, friable to weak						
24	SPT	30 50/6"	50/6"								

BEDROCK

Boring terminated at a depth of 25 feet below ground surface.
Boring backfilled with cement grout under observation of Contra Costa County.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively, to account for sampler type and hammer energy.
[2] Elevations based on ALTA/ACSU Land Title Survey for project, dated June 2012, by Kier & Wright Civil Engineers & Surveyors, Inc.

ROCKRIDGE GEOTECHNICAL

Project No.: 12-477 | Figure: A-5

PROJECT: **SARANAP MIXED-USE DEVELOPMENT BOULEVARD WAY AND SARANAP AVENUE**
Contra Costa County, California

Log of Boring B-6

Boring location:	See Site Plan, Figure 2
Date started:	5/13/13
Date finished:	5/13/13
Drilling method:	Hollow Stem Auger
Hammer weight/drop:	140 lbs./30 inches
Hammer type:	Downhole Hammer
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by:	R. Nelson

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION (Approximate Ground Surface Elevation: 217.5 feet[2])	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
					2 inches asphalt concrete						
					4 inches aggregate base						
1–2	S&H	13 15 19	20	GC	CLAYEY GRAVEL with SAND (GC) yellow-brown, medium dense, moist, fine to coarse sand						
4–5	S&H	24 26 31	34	GC	dense						
7–10				CL	CLAY (CL) mottled yellow-brown and brown, hard, moist						
9–10	S&H	16 21 34	33	CL							
12–24					MUDSTONE red-brown, soft to low hardness, plastic to friable, deep weathering, moist						
14–15	SPT	14 17 21	38								
19–20	S&H	41 50/5"	30/5"		gray, low hardness, weak						
24	SPT	29 50/6"	50/6"								

BEDROCK

Boring terminated at a depth of 25 feet below ground surface.
Boring backfilled with cement grout under observation of Contra Costa County.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively, to account for sampler type and hammer energy.
[2] Elevations based on ALTA/ACSU Land Title Survey for project, dated June 2012, by Kier & Wright Civil Engineers & Surveyors, Inc.

ROCKRIDGE GEOTECHNICAL

Project No.: 12-477
Figure: A-6

PROJECT: **SARANAP MIXED-USE DEVELOPMENT BOULEVARD WAY AND SARANAP AVENUE**
Contra Costa County, California

Log of Boring B-7

Boring location:	See Site Plan, Figure 2
Date started:	5/10/13
Date finished:	5/10/13
Drilling method:	Hollow Stem Auger
Hammer weight/drop:	140 lbs./30 inches
Hammer type:	Downhole Hammer
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by:	R. Nelson

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION (Approximate Ground Surface Elevation: 212.4 feet[2])	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
					2.5 inches asphalt concrete						
					5 inches aggregate base						
1–2	S&H	5 8 11	11	CL	CLAY with SAND (CL) dark brown, stiff, moist LL = 27, PI = 12; see Figure B-1					13.0	117
3–5	S&H	10 18 26	26	CL	very stiff, coarse sand, some fine gravel						
7–10	S&H	6 11 13	14	SC	CLAYEY SAND (SC) mottled olive and brown, medium dense, wet, fine grained sand; Sieve Analysis, see Figure B-2				34.6	20.3	103
13–15	SPT	5 13 19	32		∇ during drilling CLAYSTONE dark brown, soft to low hardness, plastic to friable, deep weathering, miost						
19	SPT	30 50/6"	50/6"		grades to low hardness, friable to weak						
24	SPT	50/6"	50/6"		BEDROCK						

Boring terminated at a depth of 24 feet below ground surface.
Boring backfilled with cement grout under observation of Contra Costa County.
Groundwater encountered at a depth of 13 feet during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively, to account for sampler type and hammer energy.
[2] Elevations based on ALTA/ACSU Land Title Survey for project, dated June 2012, by Kier & Wright Civil Engineers & Surveyors, Inc.

ROCKRIDGE GEOTECHNICAL

Project No.: 12-477 | Figure: A-7

PROJECT: **SARANAP MIXED-USE DEVELOPMENT BOULEVARD WAY AND SARANAP AVENUE**
Contra Costa County, California

Log of Boring B-8

Boring location: See Site Plan, Figure 2
Logged by: R. Nelson
Date started: 5/10/13 | Date finished: 5/10/13
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches | Hammer type: Downhole Hammer
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Approximate Ground Surface Elevation: 222.0 feet[2]

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
1–2	S&H		4 6 8	8	CL	CLAY with SAND (CL) dark brown, medium stiff to stiff, moist, trace fine to coarse sand						
4–5	S&H		12 19 23	25	CL	SANDY CLAY (CL) brown, very stiff, moist, fine to coarse sand Sieve Analysis, see Figure B-3				60.7	14.3	113
9–10	SPT		11 11 11	22		increase coarse sand						
14–15	S&H		9 30 27	34	SC	CLAYEY SAND (SC) brown, dense, moist, fine sand Sieve Analysis, see Figure B-3				34.9		
19–20	SPT		5 7 8	15	SM	during drilling SILTY SAND (SM) brown, medium dense, wet, fine sand Sieve Analysis, see Figure B-3				38.4		
24	S&H		19 50/6"	30/6"	GP	SANDY GRAVEL (GP) brown and dark brown, very dense, wet, fine gravel, fine to coarse sand						

Boring terminated at a depth of 24.5 feet below ground surface.
Boring backfilled with cement grout under observation of Contra Costa County.
Groundwater encountered at a depth of 17.5 feet during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively, to account for sampler type and hammer energy.
[2] Elevations based on ALTA/ACSU Land Title Survey for project, dated June 2012, by Kier & Wright Civil Engineers & Surveyors, Inc.

ROCKRIDGE GEOTECHNICAL

Project No.: 12-477 | Figure: A-8

PROJECT: **SARANAP MIXED-USE DEVELOPMENT**
BOULEVARD WAY AND SARANAP AVENUE
Contra Costa County, California

Log of Boring B-9

Boring location: See Site Plan, Figure 2
Logged by: T. Williams
Date started: 5/13/13
Date finished: 5/13/13
Drilling method: Hand-Augered
Hammer weight/drop: N/A
Hammer type: N/A
Sampler: Hand-augered (HA)

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION (Approximate Ground Surface Elevation: 218.5 feet[1])	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
1	HA	☒			CL	SANDY CLAY (CL) yellow-brown, moist						
2	HA	☒										
3	HA	☒			SC	CLAYEY SAND (SC) yellow-brown, dry to moist, with gravel						
4	HA	☒										
5												
6												
7												
8												
9												
10												
11												
12												
13												
14												
15												
16												
17												
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19												
20												
21												
22												
23												
24												
25												
26												
27												
28												
29												
30												

Boring terminated at a depth of 4 feet below ground surface.

[1] Elevations based on ALTA/ACSU Land Title Survey for project, dated June 2012, by Kier & Wright Civil Engineers & Surveyors, Inc.

ROCKRIDGE GEOTECHNICAL

Project No.: 12-477
Figure: A-9

PROJECT: **SARANAP MIXED-USE DEVELOPMENT BOULEVARD WAY AND SARANAP AVENUE**
Contra Costa County, California

Log of Boring B-10

Boring location: See Site Plan, Figure 2
Logged by: R. Nelson
Date started: 5/10/13 | Date finished: 5/10/13
Drilling method: Hollow Stem Auger
Hammer weight/drop: 140 lbs./30 inches | Hammer type: Cable-Downhole Hammer
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION (Approximate Ground Surface Elevation: 227.0 feet[2])	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						2.5 inches asphalt concrete 5 inches aggregate base						
1–3	S&H		7 9 13	13	SC	CLAYEY SAND (SC) dark brown, medium dense, moist, fine sand, trace gravel R-value, see Figure B-5						
4–5	S&H		10 11 18	17	SC	Sieve Analysis, see Figure B-3				28.8	12.3	112
6–10	SPT		7 8 8	16	SC	CLAYEY SAND (SC) brown, medium dense, moist						
11–15	S&H		15 25 50/6"	45	SP	SAND (SP) brown, dense, moist, fine to coarse sand						
16–20	SPT		9 12 14	26	SC	CLAYEY SAND (SC) mottled gray and olive-gray, medium dense, moist, with trace of gravel 8:25 AM on 05/10/13 11:00 AM on 05/10/13						
21–25	SPT		7 14 18	32		CLAYSTONE olive-gray, soft to low hardness, plastic to friable, deep weathering, wet BEDROCK						

Boring terminated at a depth of 25 feet below ground surface.
Boring backfilled with cement grout under observation of Contra Costa County.
Groundwater encountered at a depth of 18.5 and 20.3 feet during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively, to account for sampler type and hammer energy.
[2] Elevations based on ALTA/ACSU Land Title Survey for project, dated June 2012, by Kier & Wright Civil Engineers & Surveyors, Inc.

ROCKRIDGE GEOTECHNICAL

Project No.: 12-477 | Figure: A-10

PROJECT: **SARANAP MIXED-USE DEVELOPMENT BOULEVARD WAY AND SARANAP AVENUE**
Contra Costa County, California

Log of Boring B-11

Boring location:	See Site Plan, Figure 2
Date started:	5/13/13
Date finished:	5/13/13
Drilling method:	Hollow Stem Auger
Hammer weight/drop:	140 lbs./30 inches
Hammer type:	Downhole Hammer
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by:	R. Nelson

Approximate Ground Surface Elevation: 231.4 feet[2]

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
1–2	S&H	7 7 8	9	CL	CLAY (CL) brown, stiff, moist, trace sand and organics Corrosion Test, see Appendix B		
4–5	S&H	9 17 24	25	CL	LL = 43, PI = 28; see Figure B-1 yellow-brown, very stiff, increase fine sand content	15.6	104
9–10	S&H	24 50/6"	30/6"	SC	CLAYEY SAND with GRAVEL (SC) brown, very dense, moist, coarse sand, fine to coarse gravel		
14–15	SPT	12 11 12	23	SC	less clay, medium dense		
19–20	SPT	12 8 7	15	CL	CLAY (CL) light gray, stiff, moist		
22–23				CL	SANDY CLAY (CL) olive-gray, very stiff, moist, fine sand		
24–25	S&H	13 7 20	22	CL			

LABORATORY TEST DATA: Type of Strength Test; Confining Pressure Lbs/Sq Ft; Shear Strength Lbs/Sq Ft; Fines %

Boring terminated at a depth of 25 feet below ground surface.
Boring backfilled with cement grout under observation of Contra Costa County.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively, to account for sampler type and hammer energy.
[2] Elevations based on ALTA/ACSU Land Title Survey for project, dated June 2012, by Kier & Wright Civil Engineers & Surveyors, Inc.

ROCKRIDGE GEOTECHNICAL

Project No.: 12-477 | Figure: A-11

PROJECT: **SARANAP MIXED-USE DEVELOPMENT BOULEVARD WAY AND SARANAP AVENUE**
Contra Costa County, California

Log of Boring B-12

Boring location:	See Site Plan, Figure 2
Date started:	5/10/13
Date finished:	5/10/13
Drilling method:	Hollow Stem Auger
Hammer weight/drop:	140 lbs./30 inches
Hammer type:	Downhole Hammer
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by:	R. Nelson

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
					Approximate Ground Surface Elevation: 227.2 feet[2]						
					4 inches asphalt concrete						
					5 inches aggregate base						
1–2	S&H	7 11 13	14	CL	CLAY with SAND (CL) dark brown, stiff, moist, trace fine to coarse sand						
3					grades to brown						
4–5	S&H	6 8 12	12		mottled brown and yellow brown						
6–7											
8											
9–10	S&H	11 19 27	28		very stiff						
11–12											
13											
14–15	S&H	13 15 18	20		▽ during drilling						
16–17											
18											
19–20	SPT	5 7 9	16								
21				SP	SAND (SP) mottled olive and brown, dense, wet, fine to coarse grained sand, trace fine gravel, trace clay						
22–23											
24–25	S&H	10 21 33	32								
26–27											
28–29											
30											

Boring terminated at a depth of 25 feet below ground surface.
Boring backfilled with cement grout under observation of Contra Costa County.
Groundwater encountered at a depth of 14.8 feet during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively, to account for sampler type and hammer energy.
[2] Elevations based on ALTA/ACSU Land Title Survey for project, dated June 2012, by Kier & Wright Civil Engineers & Surveyors, Inc.

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Project No.: 12-477
Figure: A-12

UNIFIED SOIL CLASSIFICATION SYSTEM

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

GRAIN SIZE CHART

Classification	Range of Grain Sizes	
	U.S. Standard Sieve Size	Grain Size in Millimeters
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel	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

SARANAP MIXED-USE DEVELOPMENT
BOULEVARD WAY AND SARANAP AVENUE
Contra Costa County, California

ROCKRIDGE GEOTECHNICAL

CLASSIFICATION CHART

Date 10/29/13 | Project No. 12-477 | Figure A-13

I FRACTURING

Intensity	Size of Pieces in Feet
Very little fractured	Greater than 4.0
Occasionally fractured	1.0 to 4.0
Moderately fractured	0.5 to 1.0
Closely fractured	0.1 to 0.5
Intensely fractured	0.05 to 0.1
Crushed	Less than 0.05

II HARDNESS

1. **Soft** - reserved for plastic material alone.
2. **Low hardness** - can be gouged deeply or carved easily with a knife blade.
3. **Moderately hard** - can be readily scratched by a knife blade; scratch leaves a heavy trace of dust and is readily visible after the powder has been blown away.
4. **Hard** - can be scratched with difficulty; scratch produced a little powder and is often faintly visible.
5. **Very hard** - cannot be scratched with knife blade; leaves a metallic streak.

III STRENGTH

1. **Plastic** or very low strength.
2. **Friable** - crumbles easily by rubbing with fingers.
3. **Weak** - an unfractured specimen of such material will crumble under light hammer blows.
4. **Moderately strong** - specimen will withstand a few heavy hammer blows before breaking.
5. **Strong** - specimen will withstand a few heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.
6. **Very strong** - specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.

IV WEATHERING - The physical and chemical disintegration and decomposition of rocks and minerals by natural processes such as oxidation, reduction, hydration, solution, carbonation, and freezing and thawing.

D. Deep - moderate to complete mineral decomposition; extensive disintegration; deep and thorough discoloration; many fractures, all extensively coated or filled with oxides, carbonates and/or clay or silt.

M. Moderate - slight change or partial decomposition of minerals; little disintegration; cementation little to unaffected. Moderate to occasionally intense discoloration. Moderately coated fractures.

L. Little - no megascopic decomposition of minerals; little of no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains on fracture surfaces.

F. Fresh - unaffected by weathering agents. No disintegration of discoloration. Fractures usually less numerous than joints.

ADDITIONAL COMMENTS:

V CONSOLIDATION OF SEDIMENTARY ROCKS: usually determined from unweathered samples. Largely dependent on cementation.

U = unconsolidated
P = poorly consolidated
M = moderately consolidated
W = well consolidated

VI BEDDING OF SEDIMENTARY ROCKS

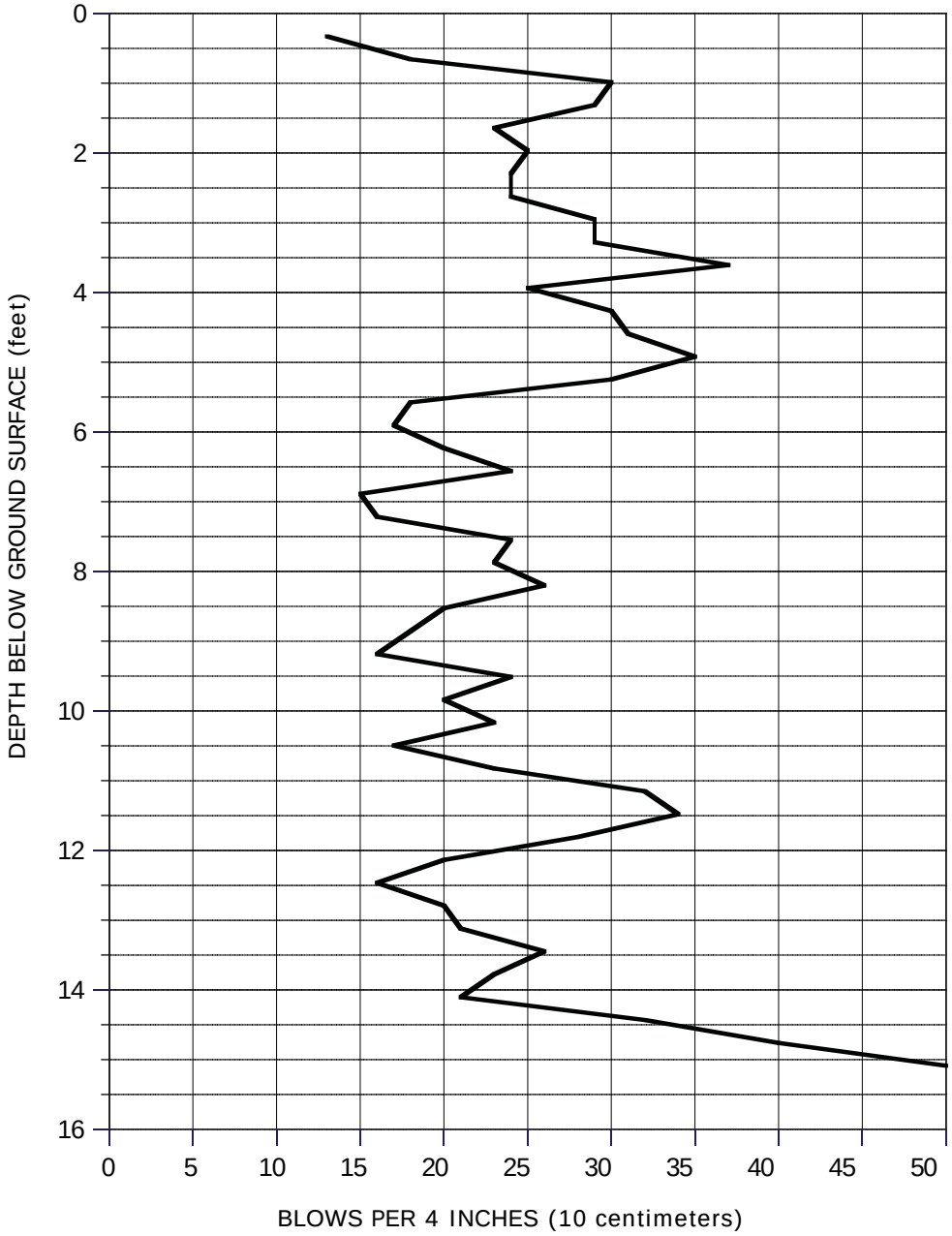
Splitting Property	Thickness	Stratification
Massive	Greater than 4.0 ft.	very thick-bedded
Blocky	2.0 to 4.0 ft.	thick bedded
Slabby	0.2 to 2.0 ft.	thin bedded
Flaggy	0.05 to 0.2 ft.	very thin-bedded
Shaly or platy	0.01 to 0.05 ft.	laminated
Papery	less than 0.01	thinly laminated

SARANAP MIXED-USE DEVELOPMENT
BOULEVARD WAY AND SARANAP AVENUE
Contra Costa County, California

ROCKRIDGE GEOTECHNICAL

PHYSICAL PROPERTIES CRITERIA FOR ROCK DESCRIPTIONS

Date 10/29/13	Project No. 12-477	Figure A-14



Note: Approximate ground surface elevation is 218.5 feet.

SARANAP MIXED-USE DEVELOPMENT
BOULEVARD WAY AND SARANAP AVENUE
Contra Costa County, California

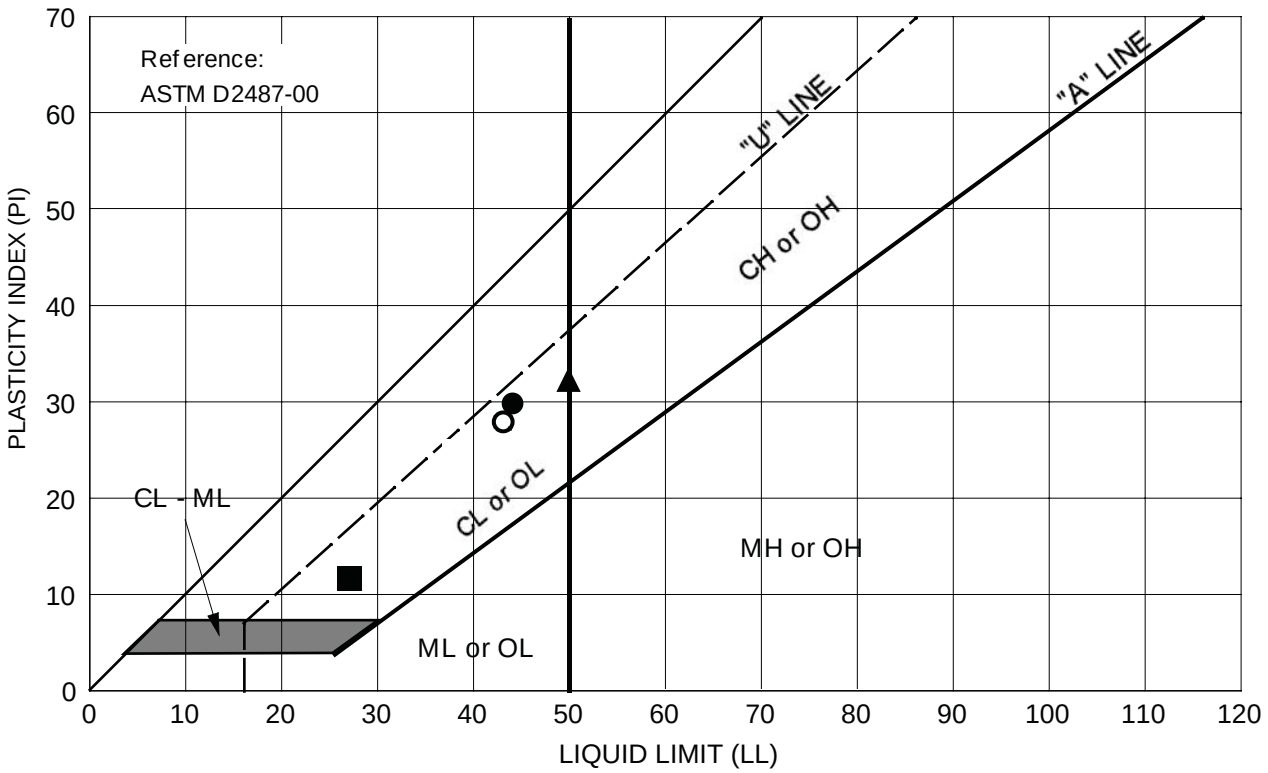
ROCKRIDGE GEOTECHNICAL

DYNAMIC PENETROMETER TEST RESULTS, DPT-1

Date 10/29/13	Project No. 12-477	Figure A-15

APPENDIX B

Laboratory Test Results



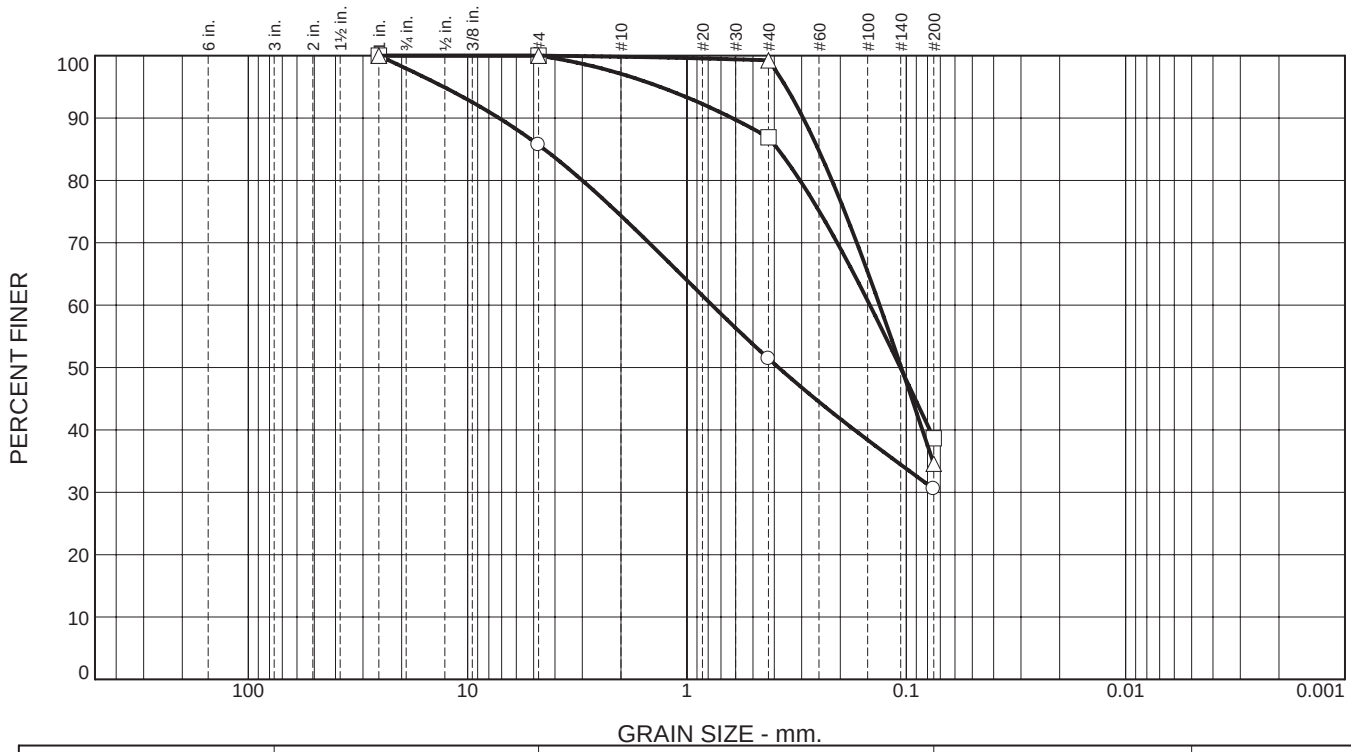
Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-1 at 2.0 feet	CLAY with SAND (CL), mottled light brown and dark brown	16.6	44	30	--
▲	B-4 at 4.0 feet	CLAY (CH), mottled brown and light brown	14.3	50	32	--
■	B-7 at 2.0 feet	CLAY with SAND (CL), dark brown	13.0	27	12	--
○	B-11 at 4.0 feet	CLAY (CL), yellow-brown	15.6	43	28	--

SARANAP MIXED-USE DEVELOPMENT
BOULEVARD WAY AND SARANAP AVENUE
Contra Costa County, California

ROCKRIDGE GEOTECHNICAL

PLASTICITY CHART

Date 10/29/13 | Project No. 12-477 | Figure B-1



	% +3"	% Gravel	% Sand	% Silt	% Clay
○	0.0	14.3	55.1	30.6	
□	0.0	0.0	61.3	38.7	
△	0.0	0.0	65.4	34.6	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			4.4729	0.7679	0.3826					
□			0.3846	0.1465	0.1064					
△			0.2521	0.1323	0.1053					

Material Description	USCS	AASHTO
○ Olive clayey SAND □ Olive clayey SAND △ Dark greenish gray clayey SAND		

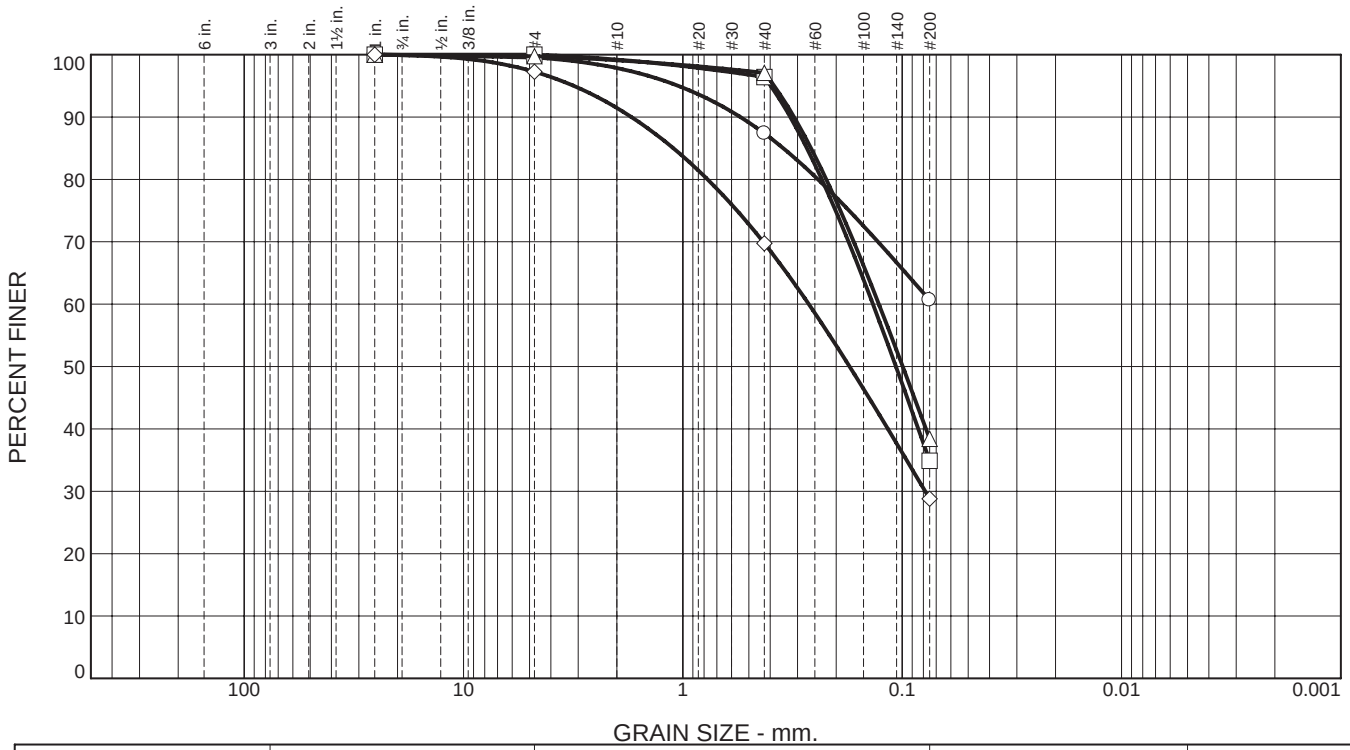
○ **Source of Sample:** B-1 **Depth:** 9.0'
□ **Source of Sample:** B-2 **Depth:** 8.5'
△ **Source of Sample:** B-7 **Depth:** 9.0'

SARANAP MIXED-USE DEVELOPMENT
BOULEVARD WAY AND SARANAP AVENUE
Contra Costa County, California

ROCKRIDGE GEOTECHNICAL

PARTICLE SIZE DISTRIBUTION REPORT

Date 10/29/13 | Project No. 12-477 | Figure B-2



	% +3"	% Gravel	% Sand	% Silt	% Clay
○	0.0	0.4	38.9	60.7	
□	0.0	0.0	65.1	34.9	
△	0.0	0.3	61.3	38.4	
◇	0.0	2.7	68.5	28.8	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			0.3486							
□			0.2719	0.1359	0.1067					
△			0.2604	0.1277	0.0995					
◇			1.1047	0.2666	0.1737	0.0786				

Material Description	USCS	AASHTO
○ Dark yellowish brown sandy CLAY		
□ Dark yellowish brown clayey SAND		
△ Olive brown clayey SAND		
◇ Dark yellowish brown clayey SAND		

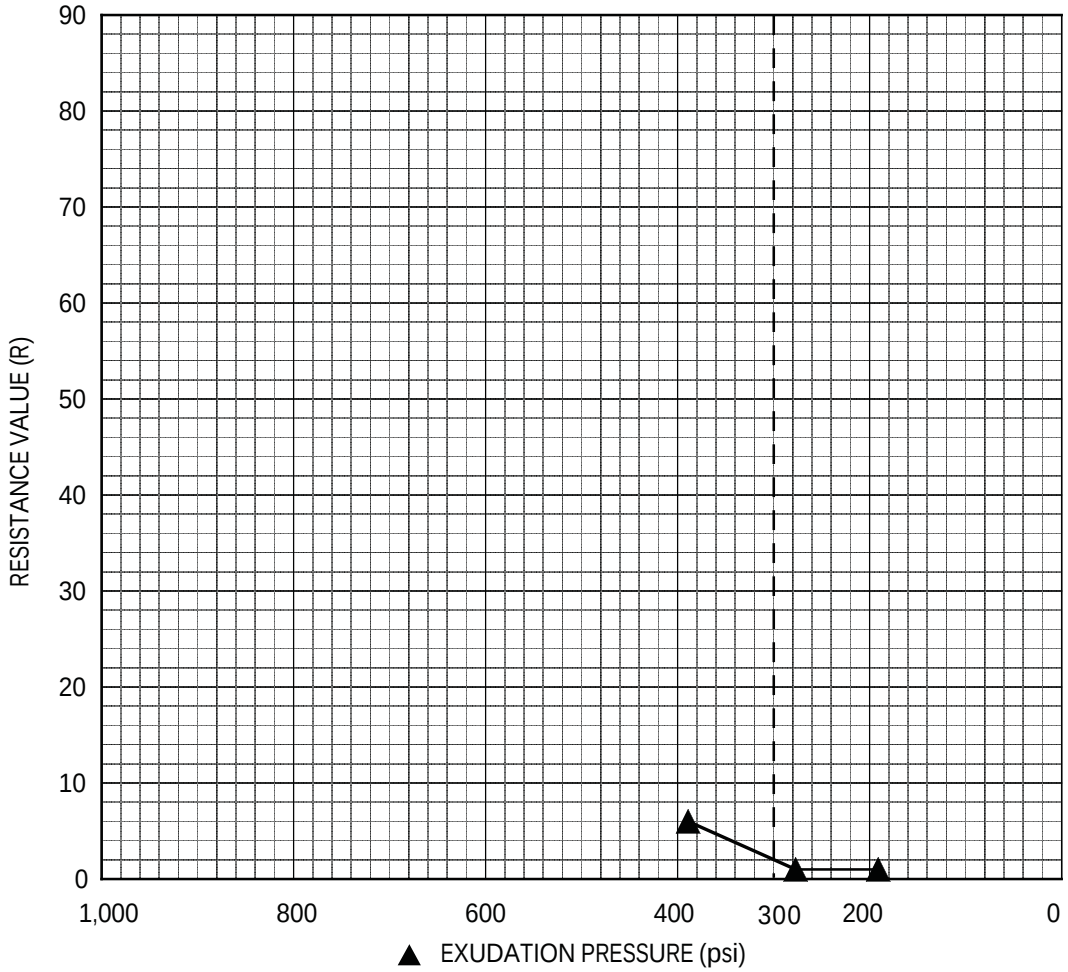
○ **Source of Sample:** B-8 **Depth:** 4.5'
□ **Source of Sample:** B-8 **Depth:** 14.5'
△ **Source of Sample:** B-8 **Depth:** 18.5'
◇ **Source of Sample:** B-10 **Depth:** 4.0'

SARANAP MIXED-USE DEVELOPMENT
BOULEVARD WAY AND SARANAP AVENUE
Contra Costa County, California

ROCKRIDGE GEOTECHNICAL

PARTICLE SIZE DISTRIBUTION REPORT

Date 10/29/13 | Project No. 12-477 | Figure B-3



Resistance R-Value and Expansion Pressure - Cal Test 301

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	40	104.9	22.7	0.00	156	2.64	191	1	1
2	40	95.2	22.0	0.00	156	2.65	277	1	1
3	40	100.0	21.2	0.00	145	2.54	389	6	6

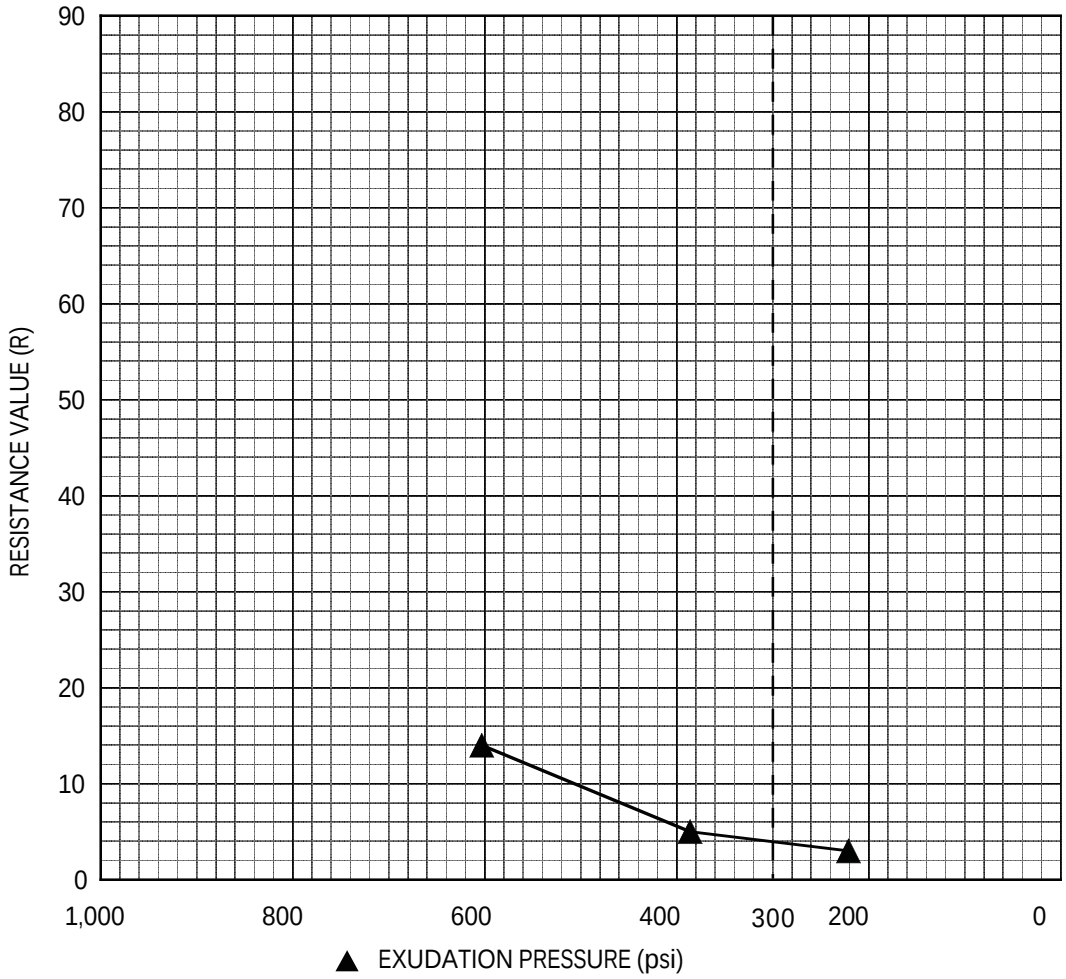
Test Results	Material Description
R-Value at 300 psi exudation pressure = 2	CLAY with SAND (CL), mottled light brown and dark brown
	Sample Source: B-1 at 1 feet

SARANAP MIXED-USE DEVELOPMENT
BOULEVARD WAY AND SARANAP AVENUE
Contra Costa County, California

ROCKRIDGE GEOTECHNICAL

R-VALUE TEST REPORT

Date 10/29/13	Project No. 12-477	Figure B-4



Resistance R-Value and Expansion Pressure - Cal Test 301

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	40	106.7	17.8	0.00	124	2.43	603	14	13
2	40	102.3	21.4	0.00	146	2.52	386	5	5
3	40	100.3	23.4	0.00	151	2.50	221	3	3

Test Results	Material Description
R-Value at 300 psi exudation pressure = 4	CLAYEY SAND (SC), dark brown
	Sample Source: B-10 at 1 feet

SARANAP MIXED-USE DEVELOPMENT
BOULEVARD WAY AND SARANAP AVENUE
Contra Costa County, California

ROCKRIDGE GEOTECHNICAL

R-VALUE TEST REPORT

Date 10/29/13	Project No. 12-477	Figure B-5

Sunland Analytical
11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

New Data Added

Date Reported 05/29/2013
Date Submitted 05/22/2013

To: Linda Liang
Rockridge Geotechnical, Inc.
4379 Piedmont Ave
Oakland, CA 94611

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 12-477-SARANAP Site ID : B-4-1 @ 1.5 FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 64616-133623.

EVALUATION FOR SOIL CORROSION

Soil pH	5.92	
Moisture	21.0 %	
Minimum Resistivity	0.27 ohm-cm (x1000)	
Chloride	259.9 ppm	00.02599 %
Sulfate	<.1 ppm	
Redox Potential	(+) 154 mv	
Sulfate Reducing Bacteria Presence - NEGATIVE		

METHODS
pH and Min.Resistivity CA DOT Test #643 Mod.(Sm.Cell)
Sulfate CA DOT Test #417, Chloride CA DOT Test #422
Redox Potential ASTM D1498m, Sulfate Reducing Bacteria AWWA C105-72

Sunland Analytical
11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 05/29/2013
Date Submitted 05/22/2013

To: Linda Liang
Rockridge Geotechnical, Inc.
4379 Piedmont Ave
Oakland, CA 94611

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following:
Location : 12-477-SARANAP Site ID : B-4-1 @ 1.5 FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 64616-133623.

Extractable Sulfide Analysis

TYPE OF TEST	RESULTS	UNITS
Sulfide	ND	mg/kg

DETECTION LIMITS
Sulfide 0.05

Method 9031m, ND = Detection Limits

Sunland Analytical
11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

New Data Added

Date Reported 05/29/2013
Date Submitted 05/22/2013

To: Linda Liang
Rockridge Geotechnical, Inc.
4379 Piedmont Ave
Oakland, CA 94611

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 12-477-SARANAP Site ID : B-11-1 @ 2.0 FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 64616-133624.

EVALUATION FOR SOIL CORROSION

Soil pH	5.56	
Moisture	19.2 %	
Minimum Resistivity	0.59 ohm-cm (x1000)	
Chloride	16.4 ppm	00.00164 %
Sulfate	<.1 ppm	
Redox Potential	(+) 163 mv	
Sulfate Reducing Bacteria Presence - NEGATIVE		

METHODS
pH and Min.Resistivity CA DOT Test #643 Mod.(Sm.Cell)
Sulfate CA DOT Test #417, Chloride CA DOT Test #422
Redox Potential ASTM D1498m, Sulfate Reducing Bacteria AWWA C105-72

Sunland Analytical
11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

New Data Added

Date Reported 05/29/2013
Date Submitted 05/22/2013

To: Linda Liang
Rockridge Geotechnical, Inc.
4379 Piedmont Ave
Oakland, CA 94611

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following:
Location : 12-477-SARANAP Site ID : B-11-1 @ 2.0 FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 64616-133624.

Extractable Sulfide Analysis

TYPE OF TEST	RESULTS	UNITS
Sulfide	ND	mg/kg

DETECTION LIMITS
Sulfide 0.05

Method 9031m, ND = Below Detection Limits

ROCKRIDGE GEOTECHNICAL

April 9, 2014
Project No. 12-477

Mr. Michael Smith
Vice President/Forward Planner
Hall Equities Group
1855 Olympic Boulevard, Suite 250
Walnut Creet, California 94596

Subject: Letter of Confirmation for Site B1
Saranap Mixed-Use Development
Contra Costa County, California

Dear Mr. Smith,

This letter presents our conclusions regarding geotechnical information available for adequate and safe development at Site B1 of the Saranap mixed-use project in Contra Costa County, California. We previously performed a geotechnical investigation for the Saranap project, the results of which were presented in our report dated October 30, 2013.

Site B1 is located between Sites B and C and bordered by Site B to the east, Boulevard Way to the north, and Site C to the west. As part of our 2013 geotechnical investigation for the Saranap project, we drilled two soil borings, designated Borings B-6 and B-10. Boring B-1 was located at the northwest corner of Site B and Boring B-1 was located at the northeast corner of Site C. These two borings are in close proximity to Site B1.

We conclude our 2013 geotechnical investigation for the Saranap project provided sufficient information to develop conclusions and recommendations for the geotechnical aspects of the proposed development at Site B1. We also conclude that implementation of the recommendations we prepared for Sites B and C in our October 30, 2013 report will assure adequate and safe development at Site B1.

We trust that this letter provides you with the information you requested. If you have any questions, please call.

Sincerely yours,
ROCKRIDGE GEOTECHNICAL, INC.

Linda H. J. Liang, P.E., G.E.
Senior Engineer

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

ROCKRIDGE GEOTECHNICAL

April 10, 2014
Project No. 12-477

Mr. Michael Smith
Vice President/Forward Planner
Hall Equities Group
1855 Olympic Boulevard, Suite 250
Walnut Creet, California 94596

Subject: Sites A and B – Groundwater Design Alternatives
Saranap Mixed-Use Development
Contra Costa County, California

Dear Mr. Smith,

This letter presents our conclusions regarding groundwater design alternatives for the proposed development at Sites A and B of the Saranap mixed-use project in Contra Costa County, California. We previously performed a geotechnical investigation for the Saranap project, the results of which were presented in our report dated October 30, 2013.

On the basis of our geotechnical investigation, we recommended proposed buildings at Sites A and B be designed using design groundwater depths of 8 and 10 feet below the ground surface (bgs), respectively. Because of the relatively high groundwater table and the anticipated depth of excavation, design measures will need to be implemented for construction of basement levels at Sites A and B. We conclude feasible design measures to address groundwater consist of the following three alternatives:

- Alternative 1: Fully waterproofed building with the basement walls and building foundation designed for hydrostatic pressure
- Alternative 2: Install underslab drainage system and discharge groundwater to sewer system
- Alternative 3: Install underslab drainage system and discharge groundwater to storm drain

For Alternative 1, we recommend the below-grade portion of the building be fully waterproofed and the building foundation and floor slab be designed to resist hydrostatic uplift force. Unbalanced hydrostatic uplift forces may be resisted by installing uplift anchors (tiedown anchors). Tiedown anchors will be installed within the footprint of the proposed basement and at the proposed basement elevation. There will be no substantial change in excavation associated with installation of tiedown anchors. Installation of tiedown anchors will generate drilling spoils that will need to be off-hauled; however, we conclude the amount of off-haul will not alter substantially the amount of off-haul for the entire project (off-haul for entire project estimated at 70,000 cubic yards).

Alternatives 2 and 3 consist of installing an underslab drainage system to eliminate hydrostatic pressure on the floor slab and foundations. The permanent dewatering system should consist of an at least 12-inch-thick layer of Class 2 permeable material containing parallel collection pipes spaced approximately every 20 feet, center-to-center. The collection pipes should be connected to at least one sump pit. The groundwater seeping beneath the building should be continuously collected in the sump and then pumped into the sanitary sewer system (Alternative 2) or storm drain (Alternative 3). The underslab system would be in lieu of grading and construction that would otherwise occur at the site, and would not result in a substantial difference in the amount of grading, excavation, off-haul, or material delivery to the site.

For both Alternatives 2 and 3, the sump pit will be located inside the basement footprint and extend slightly below the basement subgrade. For these alternatives, a back-up generator is generally needed to run the discharge pump inside the sump pit in case of a power failure. The emergency generators proposed for the project could be used for the sump pump, without any need to increase the size or number of generators.

Long-term discharge of water into the sanitary sewer or storm drain system will require a discharge permit from applicable governing agencies. The governing agencies may require laboratory testing of groundwater samples to evaluate the groundwater quality. Depending on the water quality test results, the governing agency may require the groundwater be treated (i.e. with carbon filter or other appropriate treatment systems) prior to discharge into the storm drain system. A groundwater treatment system, if required, may be located inside the footprint of the proposed basement.

We trust that this letter provides you with the information you requested. If you have any questions, please call.

Sincerely yours,
ROCKRIDGE GEOTECHNICAL, INC.

Linda H. J. Liang, P.E., G.E.
Senior Engineer

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