APPENDIX GEO
GEOTECHNICAL INVESTIGATION
April 6, 2016

Mr. Peter Baldacci  
St. Peter Construction Company, Inc.  
12885 Alcosta Blvd., Suite A  
San Ramon, CA  94583

Re: Geotechnical Investigation  
2150 & 2198 Pleasant Hill Road, Pleasant Hill, California  
SFB Project No.:  254-53

Mr. Baldacci:

As requested, Stevens, Ferrone & Bailey Engineering Company, Inc. has performed a geotechnical investigation for the proposed 2150 & 2198 Pleasant Hill Road residential development project in Pleasant Hill, California. The accompanying report presents the results of our field investigation, laboratory tests, and engineering analysis. The geotechnical conditions are discussed, and recommendations for the geotechnical engineering aspects of the project are presented. Conclusions and recommendations contained herein are based upon applicable standards of our profession at the time this report has been prepared. Should you have any questions or require additional information, please do not hesitate to contact me.

Sincerely,

Stevens, Ferrone & Bailey  
Engineering Company, Inc.

Ken Ferrone  
President

TC/KCF:le\encl.  
Copies:  Addressee (1 by email)
April 6, 2016

GEOTECHNICAL INVESTIGATION
2150 & 2198 PLEASANT HILL ROAD RESIDENTIAL DEVELOPMENT
PLEASANT HILL, CALIFORNIA
SFB PROJECT NO. 254-53

Prepared For:

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# TABLE OF CONTENTS

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

2.0 SCOPE OF WORK............................................................................................................2

3.0 SITE INVESTIGATION...................................................................................................3

3.1 Surface ....................................................................................................................3

3.2 Subsurface ..............................................................................................................3

3.3 Groundwater ..........................................................................................................4

3.4 Hydrologic Soil Group...........................................................................................4

3.5 Geology and Seismicity ..........................................................................................5

3.6 Slope Stability .........................................................................................................5

3.7 Liquefaction...........................................................................................................6

4.0 CONCLUSIONS AND RECOMMENDATIONS...........................................................7

4.1 Earthwork...............................................................................................................9

4.1.1 Clearing and Site Preparation ...........................................................................9

4.1.2 Existing Weak Soil and Fill Re-Compaction ...............................................10

4.1.3 Building Pads and Retaining Walls ...............................................................11

4.1.4 Subgrade Preparation .....................................................................................11

4.1.5 Fill Material .....................................................................................................11

4.1.6 Compaction .....................................................................................................11

4.1.7 Utility Trench Backfill ..................................................................................12

4.1.8 Exterior Flatwork ............................................................................................12

4.1.9 Construction during Wet Weather Conditions ...........................................13

4.1.10 Surface Drainage, Irrigation, and Landscaping ........................................13

4.1.11 Storm Water Runoff Structures .................................................................14

4.1.12 Setbacks .......................................................................................................16

4.1.13 Future Maintenance .....................................................................................16

4.1.14 Additional Recommendations .......................................................................17

4.2 Foundation Support................................................................................................17

4.2.1 Post-Tensioned Slabs ....................................................................................17

4.2.2 Retaining Walls and Soundwalls ................................................................19

4.2.3 Seismic Design Criteria ...............................................................................21

4.3 Pavements ..............................................................................................................22

5.0 CONDITIONS AND LIMITATIONS.........................................................................24
TABLE OF CONTENTS
(Continued)

FIGURES

1 Site Plan

APPENDICES

A Field Investigation
Figure A-1, Key to Exploratory Boring Logs
Exploratory Boring Logs (SFB-1 through SFB-3)

B Laboratory Investigation

C ASFE Guidelines
1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed residential development to be located at 2150 & 2198 Pleasant Hill Road in Pleasant Hill, California as shown on the Site Plan, Figure 1. The purpose of our investigation was to evaluate the geotechnical conditions at the site and provide recommendations regarding the geotechnical engineering aspects of the project.

Based on the information indicated on the Site Plan, as well as information provided by Mr. Peter Baldacci of St. Peter Construction Company, Inc., it is our understanding that the project will consist of developing approximately 5 acres for approximately 15 single-family, detached homes and lots. Associated underground utilities and driveways will be provided. The existing structures and facilities at the site will be demolished prior to new construction.

The conclusions and recommendations provided in this report are based upon the information presented above; Stevens, Ferrone & Bailey Engineering Company, Inc. (SFB) should be consulted if any changes to the project occur to assess if the changes affect the validity of this report.
2.0 SCOPE OF WORK

This investigation included the following scope of work:

- Reviewing published and unpublished geotechnical and geological literature relevant to the site;
- Performing reconnaissance of the site and surrounding area;
- Performing a subsurface exploration program, including drilling three exploratory borings to a maximum depth of about 21-1/2 feet;
- Performing laboratory testing of samples retrieved from the borings;
- Performing engineering analysis of the field and laboratory data; and
- Preparing this report.

The data obtained and the analyses performed were for the purpose of providing geotechnical design and construction criteria for site earthwork, underground utilities, drainage, building foundations, retaining walls and soundwalls, and pavements. Toxicity potential assessment of onsite materials or groundwater (including mold) was beyond our scope of work.
3.0 SITE INVESTIGATION

Reconnaissance of the site and surrounding area was performed on March 22, 2016. Subsurface exploration was performed using a truck-mounted drill rig equipped with 4-inch diameter, continuous flight, solid stem augers. Three exploratory borings were drilled on March 22, 2016 to a maximum depth of about 21-1/2 feet. The approximate locations of SFB’s borings are shown on the Site Plan, Figure 1. Logs of SFB’s borings and details regarding SFB’s field investigation are included in Appendix A. The results of SFB’s laboratory tests are discussed in Appendix B. It should be noted that changes in the surface and subsurface conditions can occur over time as a result of either natural processes or human activity and may affect the validity of the conclusions and recommendations in this report.

3.1 Surface

At the time of our investigation and as shown on Figure 1, the site was bounded by Pleasant Hill Road on the west, Boyd Road on the north, the Contra Costa Canal on the east, and existing residential developments on the south. The canal embankment slope along the eastern site boundary was about 5 to 20 feet high with slope inclinations varying from about 2:1 (horizontal to vertical) to 1:1. Slope erosion was observed in some areas.

The site was irregular in shape and had a plan area of about 5 acres with maximum dimensions of about 650 feet by 390 feet. The site surface grades generally sloped gently from the southwest downward toward the north, northeast, and east. The site was occupied by several wood-frame residential buildings with associated facilities that included asphalt concrete and concrete paved driveways, several sheds and outbuildings, retaining walls, and a swimming pool. Large and small diameter trees were generally located throughout the site. Surface vegetation within vacant areas generally consisted of a heavy growth of grasses and weeds.

Based on our review of historical topographic maps and aerial photographs of the site, it is our estimation that a buried drainage swale may exist within the northern portion of the site and that the drainage swale generally flowed from the southwest toward the northeast. The estimated alignment of the buried swale is shown on Figure 1.

3.2 Subsurface

The near-surface soil materials encountered by our borings at the site generally consisted of interbedded stiff to hard clays and silts and medium dense to very dense sands that extended to the maximum depth explored of about 21-1/2 feet. Boring SFB-2 also encountered approximately 3 feet of clayey fills immediately below the ground surface. Boring SFB-3,
located within the possible alignment of the buried swale, encountered gravelly and clayey fills that extended to a depth of about 8 feet. These undocumented fills are heterogeneous, potentially weak, and compressible.

According to the results of our laboratory testing, the near-surface more clayey soils have high plasticity and high expansion potential whereas more silty and sandy soils have a relatively low plasticity and low expansion potential.

Detailed descriptions of the materials encountered in our exploratory borings are presented on the boring logs in Appendix A. Our attached boring logs and related information depict location specific subsurface conditions encountered during our field investigation. The approximate locations of our borings were determined using pacing or landmark references and should be considered accurate only to the degree implied by the method used.

3.3 Groundwater

Groundwater was initially encountered in Boring SFB-3 at a depth of about 11-1/2 feet and rose to a depth of about 9 feet at the end of drilling. No groundwater was encountered in the other borings to a maximum depth explored in these borings of about 21-1/2 feet. The borings were backfilled with lean cement grout in accordance with Contra Costa County Environmental Health requirements prior to leaving the site. It should be noted that the borings might not have been left open for a sufficient period of time to establish equilibrium groundwater conditions. In addition, fluctuations in the groundwater level could occur due to change in seasons, variations in rainfall, and other factors.

3.4 Hydrologic Soil Group

The surface soils of the site have been mapped as Positas loam (2 to 9 percent slopes) and were assigned to Hydrologic Soil Groups D by USDA Natural Resources Conservation Service (NRCS); the soils have been categorized as having very low to moderately low transmission rates (approximately 0 to 0.06 inches per hour). The Group D soil is defined as having a very slow infiltration rate (high runoff potential) when thoroughly wet and may consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.
3.5 Geology and Seismicity

According to Helley and Graymer (1997), the site (below surficial fills if any) is underlain by Plio-Pleistocene undifferentiated continental gravel deposits that had been described as semi-consolidated to unconsolidated poorly sorted gravel, sand, silt and clay distributed in isolated patches throughout the country.

The project site is located in the San Francisco Bay Area that is considered one of the most seismically active regions in the United States. Significant earthquakes have occurred in the San Francisco Bay Area and are believed to be associated with crustal movements along a system of sub-parallel fault zones that generally trend in a northwesterly direction. According to the Alquist-Priolo Earthquake Fault Zones Map of the Walnut Creek Quadrangle (1993), the site is not located in an earthquake fault zone as designated by the State of California.

Earthquake intensities will vary throughout the San Francisco Bay Area, depending upon numerous factors including the magnitude of earthquake, the distance of the site from the causative fault, and the type of materials underlying the site. The U.S. Geological Survey (2008) indicated that there is a 63 percent chance of at least one magnitude 6.7 or greater earthquake striking the San Francisco Bay region between 2008 and 2037. Therefore, the site will probably be subjected to at least one moderate to severe earthquake that will cause strong ground shaking.

According to the Probabilistic Seismic Hazard Analysis (NSHMP PSHA) interactive deaggregation model developed by U.S. Geological Survey (2008), the site has a 10% probability of exceeding a peak ground acceleration of about 0.5g in 50 years (design basis ground motion based on stiff soil site condition; mean return time of 475 years). The actual ground surface acceleration might vary depending upon the local seismic characteristics of the underlying bedrock and the overlying unconsolidated soils.

3.6 Slope Stability

According to U.S. Geological Survey Open-File Report 97-745 (landslide folio of the San Francisco Bay Area), the site is not mapped as having previously identified landslides or

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earthflows nor is it located within an area having debris flow source potential. Based on the results of our reconnaissance, field exploration, and review of documents, we did not observe evidence of adverse slope stability, erosion, or drainage conditions at the site, but observed erosion and creep of near-surface soils along the adjacent canal embankment slopes. We did not observe evidence of active, deep seated slope movement onsite or in the vicinity of the site. It is not uncommon, however, for relatively shallow slope movements to occur on embankment slopes along the canal. These movements may include downslope creep, erosion, and slumping. It is our opinion that there is a low potential for slope instability occurring at the site provided the recommendations in this report are followed.

Our scope of work did not include performing a comprehensive stability evaluation of the canal slopes under both static and earthquake conditions. The stability of the embankments is affected by numerous factors including the underlying soil and rock characteristics, rainfall, irrigation, earthquake shaking, and changes to the topography. Therefore, the stability of the canal slopes can change over time.

### 3.7 Liquefaction

Soil liquefaction is a phenomenon primarily associated with saturated, cohesionless, soil layers located close to the ground surface. These soils lose strength during cyclic loading, such as imposed by earthquakes. During the loss of strength, the soil acquires mobility sufficient to permit both horizontal and vertical movements. Soils that are most susceptible to liquefaction are clean, loose, uniformly graded, saturated, fine-grained sands that lie close to the ground surface.

According to the Association of Bay Area Governments (ABAG) and the U.S. Geological Survey, the site is located in an area that has been characterized as having very low liquefaction susceptibility\(^4\)^\(^5\). As of the date of this report, the liquefaction potential of the site has not been evaluated by the State of California\(^6\).

Based on our review of available literature and the results of exploratory borings at the site, it is our opinion that the potential for ground surface damage at the site resulting from liquefaction is low due to the lack of saturated liquefiable soils and presence of hard clayey and silty soils at a relatively shallow depth.

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\(^6\)Seismic Hazards Mapping Act, 1990.
4.0 CONCLUSIONS AND RECOMMENDATIONS

It is our opinion that the site is suitable for the proposed project from a geotechnical engineering standpoint. The conclusions and recommendations presented in this report should be incorporated in the design and construction of the project to reduce soil or foundation related issues. The following are the primary geotechnical considerations for development of the site.

EXISTING WEAK SOIL AND FILL MATERIALS: As described previously, heterogeneous, potentially weak and compressible soils and fills exist at the site and extend to depths of about 1 to 3 feet below the existing ground surface. In addition, weak fills exist within the alignment of the pre-existing drainage swale and extend to a depth of about 8 feet in the vicinity of Boring SFB-3. The estimated alignment is shown on Figure 1. The removal of the existing structures, improvements, and trees at the site will result in loosening of the surface soils in the upper 2 to 3 feet. In order to reduce the potential for damaging differential settlement of overlying improvements (such as new fills, building foundations, driveways, exterior flatwork, and pavements), we recommend that these weak fills and soils be completely removed and re-compacted.

We estimate the process can consist of over-excavating 2 feet, scarifying and re-compacting the bottom 12 inches in-place, and replacing the excavation with compacted fill materials. Deeper removal will be necessary within the pre-existing drainage swale; the actual depth of fills within the swale alignment area should be determined at the time of clearing using a backhoe or excavator. There would be no need to over-excavate fills and soils within areas that do not support improvements, such as within planned open spaces. The over-excavations should extend to depths where competent soil is encountered. The over-excavation and re-compaction should also extend at least 5 feet beyond building footprints and at least 3 feet beyond exterior flatwork (including driveways) and pavement wherever possible. Where the over-excavation limits abut adjacent property, SFB should be consulted to determine the actual vertical and lateral extent of over-excavation so that adjacent property is not adversely impacted. Over-excavations should be performed so that no more than 5 feet of differential fill thickness exists below the proposed building foundations. The removed fill and soil materials can be used as new fill provided it is placed and compacted in accordance with the recommendations presented in this report. The extent of the removal and re-compaction will vary across the site and should be determined in the field by SFB at the time of the earthwork operations.

DIFFERENTIAL EXPANSION POTENTIAL: The onsite more clayey soils have high expansion potential whereas the more silty and sandy soils have relatively low expansion potential. In order to provide a more uniform subgrade and reduce the potential for damaging
differential movement of building foundations, we recommend the proposed grading be performed so that each building foundation and surrounding flatwork be supported on fills with similar expansion potential. In no case should a building be underlain by subgrade (and within 3 feet of subgrade) consisting of both expansive clayey soils or fills and relatively low expansion potential silty and sandy soils and fills. We recommend a layer least 3 feet thick of well-mixed, moisture conditioned, and well blended engineered fill be provided below all building foundations and surrounding flatwork. The compacted, engineered fill layers should extend at least 5 feet beyond building footprints and at least 3 feet beyond exterior flatwork, including driveways. Our representative should be onsite during over-excavation and replacement to observe and test fill placement operations. The actual depth and lateral extent of removal and replacement should be determined in the field by SFB at the time of the earthwork operations.

The more clayey, highly expansive surface soil materials will be subjected to volume changes during seasonal fluctuations in moisture content. To reduce the potential for post-construction distress to the proposed structures resulting from swelling and shrinkage of these materials, we recommend that the proposed residences be supported on a post-tensioned slab foundation system that is designed to reduce the impact of the expansive soils. It should be noted that special design considerations will be required for exterior slabs.

SETBACKS FROM CANAL EMBANKMENT SLOPE: In order to reduce the potential for property damage caused by canal embankment slope erosion, soil creep, and slumping, canal embankment setbacks should be used. We recommend setbacks be established by projecting a 3:1 (horizontal to vertical) line from the nearest toe of the existing canal embankment upward toward the development. Where the projected line intersects the finished ground surface, we recommend improvements (including residential lots) be setback at least 10 feet from the intersection or top of the canal embankment, whichever is greater. If it is impractical to setback improvements, retaining systems (walls and/or geogrid reinforced earth) can be used that are designed and constructed to resist downslope ground movements.

During a major earthquake, it should be anticipated that some ground movement toward the toe of the embankments will occur. We recommend the project’s Civil Engineer determine the actual property, building, and improvement setbacks based upon the recommendations provided in this report, the final grading plans, California Building Code and local ordinances, and any other restrictions such as environmental regulations. Property located between the setback line and the embankment may experience movement as a result of embankment slope erosion, localized slumping, and other factors.

CORROSION POTENTIAL: Two onsite soil samples were tested for pH (ASTM D4972), chlorides (ASTM D4327), sulfates (ASTM D4327), sulfides (ASTM D4658M), resistivity at 100% saturation (ASTM G57), and Redox potential (ASTM D1498) for use in evaluating the
potential for corrosion on concrete and buried metal such as utilities and reinforcing steel. The results of these tests are included in Appendix B. We recommend these test results be forwarded to your underground contractors, pipeline designers, and foundation designers and contractors.

**ADDITIONAL RECOMMENDATIONS:** Detailed drainage, earthwork, foundation, retaining wall and soundwall, and pavement recommendations for use in design and construction of the project are presented below. We recommend SFB review the design and specifications to verify that the recommendations presented in this report have been properly interpreted and implemented in the design, plans, and specifications. We also recommend SFB be retained to provide consulting services and to perform construction observation and testing services during the construction phase of the project to observe and test the implementation of our recommendations, and to provide supplemental or revised recommendations in the event conditions different than those described in this report are encountered. We are not responsible for misinterpretation of our recommendations.

### 4.1 Earthwork

#### 4.1.1 Clearing and Site Preparation

The site should be cleared of all obstructions including any existing structures and their entire foundation systems, driveways, existing utilities and pipelines and their associated backfill, designated trees and their associated entire root systems, fencing, and debris. Holes resulting from the removal of underground obstructions extending below the proposed finish grade should be cleared and backfilled with fill materials as specified in **Section 4.1.5, Fill Material**, and compacted to the requirements in **Section 4.1.6, Compaction**. Tree roots may extend to depths of about 3 to 4 feet. Wells and septic systems, if they exist, should be abandoned in accordance with Contra Costa County standards.

From a geotechnical standpoint, any existing trench backfill materials, clay or concrete pipes, pavements, and concrete that are removed can be used as new fill onsite provided debris is removed and it is broken up to meet the size requirement for fill material in **Section 4.1.5, Fill Material**. We recommend fill materials composed of broken up concrete or asphalt concrete not be located within 3 feet of the ground surface in yard areas. Consideration should be given to placing these materials below pavements, directly under building footprints, or in deeper excavations. We recommend backfilling operations for any excavations be performed under the observation and testing of SFB.

At least two weeks prior to grading, areas containing surface vegetation should be mowed and the cut grasses and weeds removed from the site or stockpiled for use in landscaping. After mowing, the site should be disced or stripped. Portions of the site containing heavy surface
vegetation that is not removed by discing should be stripped to an appropriate depth to remove these materials. The amount of actual stripping should be determined in the field by SFB at the time of construction. Stripped materials should be removed from the site or stockpiled for later use in landscaping, if desired.

### 4.1.2 Existing Weak Soil and Fill Re-Compaction

As described previously, heterogeneous, potentially weak and compressible fills exist at the site and extend to depths of about 1 to 3 feet below the existing ground surface. In addition, deeper weak soils and fills are likely to exist within the alignment of pre-existing drainage swale and may extend to a depth of about 8 feet. The estimated alignment is shown on Figure 1. The removal of the existing structures, improvements, and trees at the site will likely result in loosening of the surface soils in the upper 2 to 3 feet.

Where these weak fills and soils will not be removed during grading and where proposed improvements including new fills, building foundations, driveways, exterior flatwork, and pavements will be constructed, we recommend these weak fills and soils be completely removed and re-compacted.

We estimate the process can consist of over-excavating 2 feet, scarifying and re-compacting the bottom 12 inches in-place, and replacing the excavation with compacted fill materials. Deeper removal will be required within the pre-existing drainage swale; the actual depth of fills within the swale alignment area should be determined at the time of clearing. There would be no need to over-excavate the soils within areas that do not support improvements, such as within planned open spaces. The over-excavation should extend to depths where competent soil is encountered. The over-excavation and re-compaction should also extend at least 5 feet beyond building footprints and at least 3 feet beyond exterior flatwork (including driveways) and pavement wherever possible. Where the over-excavation limits abut adjacent property, SFB should be consulted to determine the actual vertical and lateral extent of over-excavation so that adjacent property is not adversely impacted. Over-excavations should be performed so that no more than 5 feet of differential fill thickness exists below the proposed building foundations. The extent of the removal and re-compaction will vary across the site and should be determined in the field by SFB at the time of the earthwork operations.

Removed fill and soil materials may be used as new fill onsite provided it satisfies the recommendations provided in Section 4.1.5, Fill Material. Compaction should be performed in accordance with the recommendations in Section 4.1.6, Compaction.
4.1.3 Building Pads and Retaining Walls

The proposed grading should be designed and constructed so that no more than 5 feet of differential fill thickness will exist below foundations or retaining walls supported on shallow foundations. Deeper over-excavation may be necessary to reduce fill differential thickness to 5 feet or less. We recommend a layer at least 3 feet thick of well-mixed, moisture conditioned and well blended engineered fill be provided below the building foundations and surrounding flatwork to reduce the potential for damaging differential movement.

4.1.4 Subgrade Preparation

If building pads, or exterior slab or driveway subgrade, are allowed to remain exposed to sun, wind or rain for an extended period of time, or are disturbed by animals, equipment, or vehicles, the exposed pads or subgrade may need to be reconditioned (moisture conditioned and/or scarified and recompacted) prior to foundation, exterior slab, or pavement construction. SFB should be consulted on the need for subgrade reconditioning when the subgrade is left exposed for extended periods of time.

The soil exposed in areas to receive improvements (such as structural fill, building and retaining foundations, driveways, and exterior flatwork) should be scarified to a depth of about 12 inches, moisture conditioned to approximately 3 to 5 percent over optimum water content, and compacted to the requirements for structural fill.

4.1.5 Fill Material

From a geotechnical and mechanical standpoint, onsite soils having an organic content of less than 3 percent by volume can be used as fill. Fill should not contain rocks or lumps larger than 6 inches in greatest dimension with not more than 15 percent larger than 2.5 inches. If required, imported fill should have a plasticity index of 25 or less and have a significant amount of cohesive fines.

In addition to the mechanical properties specifications, all imported fill material should have a resistivity (100% saturated) no less than the resistivity for the onsite soils, a pH of between approximately 6.0 and 8.5, a total water soluble chloride concentration less than 300 ppm, and a total water soluble sulfate concentration less than 500 ppm. We recommend import samples be submitted for corrosion and geotechnical testing at least two weeks prior to being brought onsite.

4.1.6 Compaction

Within the upper 5 feet of the finished ground surface, we recommend structural fill be compacted between 88 and 92 percent relative compaction, and structural fill below a depth of 5
feet be compacted to at least 90 percent relative compaction, as determined by ASTM D1557 (latest edition). We recommend the new fill be moisture conditioned approximately 3 to 5 percent over optimum water content. The upper 6 inches of subgrade soils beneath pavements should be compacted to at least 95 percent relative compaction. Fill material should be spread and compacted in lifts not exceeding approximately 8 to 12 inches in uncompacted thickness.

4.1.7 Utility Trench Backfill

Pipeline trenches should be backfilled with fill placed in lifts of approximately 8 inches in uncompacted thickness. Thicker lifts can be used provided the method of compaction is approved by SFB and the required minimum degree of compaction is achieved. Backfill should be placed by mechanical means only. Jetting is not permitted.

Onsite trench backfill should be compacted to at least 90 percent relative compaction. Imported sand trench backfill should be compacted to at least 95 percent relative compaction and sufficient water is added during backfilling operations to prevent the soil from "bulking" during compaction. The upper 3 feet of trench backfill in foundation, slab, and pavement areas should be entirely compacted to at least 95 percent relative compaction. To reduce piping and settlement of overlying improvements, we recommend rock bedding and rock backfill (if used) be completely surrounded by a filter fabric such as Mirafi 140N (or equivalent); alternatively, filter fabric would not be necessary if Caltrans Class 2 permeable material is used in lieu of rock bedding and rock backfill.

Sand or gravel backfilled trench laterals that extend toward driveways, exterior slabs-on-grade, or under the building foundations, and are located below irrigated landscaped areas such as lawns or planting strips, should be plugged with onsite clays, low strength concrete, or sand/cement slurry. The plug for the trench lateral should be located below the edge of pavement or slabs, and under the perimeter of the foundation. The plug should be at least 24 inches thick, extend the entire width of the trench, and extend from the bottom of the trench to the top of the sand or gravel backfill.

4.1.8 Exterior Flatwork

We recommend that exterior slabs (including patios, sidewalks, and driveways) be placed directly on the properly compacted fills. We do not recommend using aggregate base, gravel, or crushed rock below these improvements. If imported granular materials are placed below these elements, subsurface water can seep through the granular materials and cause the underlying soils to saturate or pipe. Prior to placing concrete, subgrade soils should be moisture conditioned to increase their moisture content to approximately 3 to 5 percent above laboratory optimum moisture (ASTM D-1557).
The more expansive clayey soils at the site could be subjected to volume changes during fluctuations in moisture content. As a result of these volume changes, some vertical movement of exterior slabs (such as driveways, sidewalks, patios, exterior flatwork, etc.) should be anticipated. This movement could result in damage to the exterior slabs and might require periodic maintenance or replacement. Adequate clearance should be provided between the exterior slabs and building elements that overhang these slabs, such as window sills or doors that open outward.

Consideration should be given to reinforcing exterior slabs with steel bars in lieu of wire mesh. To reduce potential crack formation, the installation of #4 bars spaced at approximately 18 inches on center in both directions should be considered. Score joints and expansion joints should be used to control cracking and allow for expansion and contraction of the concrete slabs. We recommend appropriate flexible, relatively impermeable fillers be used at all cold/expansion joints. The installation of dowels at all expansion and cold joints will reduce differential slab movements; if used, the dowels should be at least 30 inches long and should be spaced at a maximum lateral spacing of 18 inches. Although exterior slabs that are adequately reinforced will still crack, trip hazards requiring replacement of the slabs will be reduced if the slabs are properly reinforced.

4.1.9 Construction during Wet Weather Conditions

If construction proceeds during or shortly after wet weather conditions, the moisture content of the onsite soils could be significantly above optimum. Consequently, subgrade preparation, placement and/or reworking of onsite soil or fills as structural fill might not be possible. Alternative wet weather construction recommendations can be provided by our representative in the field at the time of construction, if appropriate. All the drainage measures recommended in this report should be implemented and maintained during and after construction, especially during wet weather conditions.

4.1.10 Surface Drainage, Irrigation, and Landscaping

Ponding of surface water must not be allowed on pavements, adjacent to foundations, at the top or bottom of slopes, and at the top or adjacent to retaining walls. Ponding of water should also not be allowed on the ground surface adjacent to or near exterior slabs, including driveways, walkways, and patios. Surface water should not be allowed to flow over the top of slopes, down slope faces, or over retaining walls.

We recommend positive surface gradients of at least 2 percent be provided adjacent to foundations to direct surface water away from the foundations and toward suitable discharge facilities. Roof downspouts and landscaping drainage inlets should be connected to solid pipes that discharge the collected water into appropriate water collection facilities. We recommend the
surface drainage be designed in accordance with the latest edition of the California Building Code.

In order to reduce differential foundation movements, landscaping (where used) should be placed uniformly adjacent to the foundation and exterior slabs. We recommend trees be no closer to the structure or exterior slabs than half the mature height of the tree; in no case should tree roots be allowed to extend near or below the foundations or exterior slabs.

Drainage inlets should be provided within enclosed planter areas and the collected water should be discharged onto pavement, into drainage swales, or into storm water collection systems. In order to reduce the potential for heaving, consideration should be given to lining planting areas and collecting the accumulated surface water in subdrain pipes that discharge to appropriate collection facilities. The drainage should be designed and constructed so that the moisture content of the soils surrounding the foundations do not become elevated and no ponding of water occurs. The inlets should be kept free of debris and be lower in elevation than the adjacent ground surface.

We recommend regular maintenance of the drainage systems be performed, including maintenance prior to rainstorms. The inspection should include checking drainage patterns to make sure they are performing properly, making sure drainage systems and inlets are functional and not clogged, and checking that erosion control measures are adequate for anticipated storm events. Immediate repairs should be performed if any of these measures appears to be inadequate.

Irrigation should be performed in a uniform, systematic manner as equally as possible on all sides of the foundations and exterior slabs to maintain moist soil conditions. Over-watering must be avoided. To reduce moisture changes in the natural soils and fills in landscaped areas, we recommend that drought resistant plants and low flow watering systems be used. All irrigation systems should be regularly inspected for leakage.

4.1.11 Storm Water Runoff Structures

To satisfy local and state permit requirements, most new development projects must control pollutant sources and reduce, detain, retain, and treat specified amounts of storm water runoff. The types of improvements that are designed to accomplish these goals are known as Post-Construction Requirements (PCR’s) and/or Low Impact Development (LID’s). The intent of these types of improvements is to conserve and incorporate on-site natural features, together with constructed hydrologic controls, to more closely mimic pre-development hydrology and watershed processes.
To aid in the Civil Engineering design and analyses of appropriate treatment facilities, we recommend the onsite soils be categorized as Hydrologic Soil Group D. Groundwater was initially encountered in Boring SFB-3 at a depth of about 11-1/2 feet and rose to a depth of about 9 feet at the end of drilling. No groundwater was encountered in the other borings to a maximum depth explored in these borings of about 21-1/2 feet.

We recommend PCR/LID improvements that are designed to detain or retain water such as bio-swales, porous pavement structures, and water detention basins, be lined with a relatively impermeable membrane in order to reduce water seepage and the potential for damage to other infrastructure improvements (such as pavements, foundations, and walkways). We recommend a relatively impermeable membrane such as STEGO Wrap 15-mil or equivalent be installed below and along the sides of these facilities that direct the collected water into subdrain pipes. The membrane should be lapped and sealed in accordance with the manufacture’s specifications, including taping joints where pipes penetrate the membrane.

The soil filter materials within basins and swales will consolidate over time causing long-term ground surface settlement. Additional filling within the basins and swales over time may be needed to maintain design surface elevations. The soil filter materials and associated compaction requirements should be specified by the Civil Engineer and shown in detail on the grading and improvement plans.

Sidewalls of earthen swales and basins steeper than 2:1 (horizontal to vertical) will experience downward and lateral movements that can cause significant ground surface movements. The magnitude and rate of movement depends upon the swale and basin backfill material type and compaction. To reduce the potential for damaging movements, we recommend 2:1 sidewall slopes be used or the slopes be appropriately restrained. SFB should be consulted to evaluate the need for sidewall restraint when swales or basins are planned.

Where swales and basins are located adjacent to improvements (such as foundations, pavements, curbs, driveways, and sidewalks), the improvements will be susceptible to settlement and lateral movements. To reduce the potential for vertical and lateral movement of the improvements, we recommend either the improvements be setback beyond a 1:1 (horizontal to vertical) plane projected upward from the bottom of the swale/basin or lateral restraint (such as deepened curbs or walls) be provided that is designed to resist the soil’s lateral pressures. In order to resist the lateral pressures, the lateral restraint will need to extend below the bottom of the swale/basin and should be engineered. Where foundations are located near swales or basins, we recommend the

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foundations be extended below the projected 1:1 plane projected upward from the bottom of the swale or basin.

4.1.12 Setbacks

In order to reduce the potential for property damage caused by canal embankment slope erosion, soil creep, and slumping, canal embankment setbacks should be used. We recommend setbacks be established by projecting a 3:1 (horizontal to vertical) line from the nearest toe of the existing canal embankment upward toward the development. Where the projected line intersects the finished ground surface, we recommend improvements (including residential lots) be setback at least 10 feet from the intersection or top of the canal embankment, whichever is greater. If it is impractical to setback improvements, retaining systems (walls and/or geogrid reinforced earth) can be used that are designed and constructed to resist downslope ground movements.

During a major earthquake, it should be anticipated that some ground movement toward the toe of the embankments will occur. We recommend the project’s Civil Engineer determine the actual property, building, and improvement setbacks based upon the recommendations provided in this report, the final grading plans, California Building Code and local ordinances, and any other restrictions such as environmental regulations. Property located between the setback line and the embankment may experience movement as a result of embankment slope erosion, localized slumping, and other factors.

4.1.13 Future Maintenance

In order to reduce water created issues, we recommend regular maintenance of the site and each lot be performed, including maintenance prior to rainstorms. Maintenance should include the recompaction of loosened soils, collapsing and infilling holes with compacted soils or low strength sand/cement grout, removal and control of digging animals, modifying storm water drainage patterns to allow for sheet flow into drainage inlets or ditches rather than concentrated flow or ponding, removal of debris within drainage ditches and inlets, and immediately repairing any erosion or soil flow. The inspection should include checking drainage patterns, making sure drainage systems are functional and not clogged, and erosion control measures are adequate for anticipated storm events. Immediate repair should be performed if any of these measures appears to be inadequate. Temporary and permanent erosion and sediment control measures should be installed over any exposed soils immediately after repairs are made.

Differential movement of exterior slabs can occur over time as a result of numerous factors. We recommend homeowners perform inspections and maintenance of the slabs, including infilling significant cracks, providing fillers at slab offsets, and replacing slabs if severely damaged.
4.1.14 Additional Recommendations

We recommend the drainage, irrigation, landscaping, and maintenance recommendations provided in this report be forwarded to your designers and contractors, and we recommend they be included in disclosure statements given to homeowners.

4.2 Foundation Support

4.2.1 Post-Tensioned Slabs

The residences located on level building pads can be supported on a post-tensioned slab foundation that is designed for the expansion potential of the onsite materials. The slab foundation should bear entirely on the properly prepared building pad. Recommendations for building pad preparation are described previously in Section 4.1.3, Building Pads and Retaining Walls, and Section 4.1.4, Subgrade Preparation, of this report. Prior to the concrete pour, we recommend the moisture content of the subgrade materials be approximately 3 to 5 percent above laboratory optimum moisture. If the building pads are left exposed for an extended period of time prior to constructing foundations, we recommend SFB be contacted for recommendations to re-condition the pads in order provide adequate building support.

The post-tensioned slab thickness should be determined by the Structural Engineer, however we recommend the post-tensioned slabs be at least 10 inches thick. An allowable bearing pressure of 1,500 pounds per square foot can be used for localized point and line loads. Deflection of the slab foundations should not exceed the values recommended in the most recent PTI Manual. Lateral loads, such as derived from earthquakes and wind, can be resisted by friction between the post-tensioned slab foundation bottom and the supporting subgrade. A friction coefficient of 0.25 is considered applicable.

At least 10 feet of cover should be provided between the outer face of slabs and un-retained slope faces, as measured laterally between slope faces and the slabs. Where less than 10 feet of cover exists, deepening of the edge of slabs may be necessary in order to achieve 10 feet of cover for buildings located near tops of slopes. Where slabs are located adjacent to utility trenches, the slab bearing surface should bear below an imaginary 1 horizontal to 1 vertical plane extending upward from the bottom edge of the adjacent utility trench. Alternatively, the slab reinforcing could be increased to span the area defined above assuming no soil support is provided.

A vapor retarder must be placed between the subgrade soils and the bottom of the slabs-on-grade. We recommend the vapor retarder consist of a single layer of Stego Wrap Vapor Barrier 15 mil or equivalent provided the equivalent satisfies the following criteria: a permeance as
tested before and after mandatory conditioning of less than 0.01 Perms and strength of Class A as determined by ASTM E 1745 (latest edition), and a thickness of at least 15 mils. Installation of the vapor retarder should conform to the latest edition of ASTM E 1643 (latest edition) and the manufacturers requirements, including all joints should be lapped at least 6 inches and sealed with Stego Tape or equal in accordance with the manufacturer’s specifications. Protrusions where pipes or conduit penetrate the membranes should be sealed with either one or a combination of Stego Tape, Stego Mastic, Stego Pipe Boots, or a product of equal quality as determined by the manufacturer’s instructions and ASTM E 1643. Care must be taken to protect the membrane from tears and punctures during construction. We do not recommend placing sand or gravel over the membrane.

Concrete slabs retain moisture and often take many months to dry; construction water added during the concrete pour further increases the curing time. If the slabs are not allowed to completely cure prior to constructing the super-structure, the concrete slabs will expel water vapor and the vapor will be trapped under impermeable flooring. The concrete mix design for the slabs should have a maximum water/cement ratio of 0.45; the actual water/cement ratio may need to be reduced if the concentration of soluble sulfates or chlorides in the supporting subgrade is detrimental to the concrete. The results of sulfate and chloride testing of onsite soil samples are included under separate cover for reference. We recommend you consult with your concrete slab designers and concrete contractors regarding methods to reduce the potential for differential concrete curing.

An experienced Structural Engineer should design the post-tensioned slabs to resist the differential soil movement. The preliminary soil design parameters presented below were generated using the procedures presented in the 3rd edition of the PTI design manual, PTI standard requirements, and a PTI preferred computer program, VOLFLO (Version 1.5 Build 120704), was employed to simulate the wetting and drying scenarios of the soils beneath the post-tensioned slabs.

The values provided below are based upon the post-tensioned slab foundations being entirely surrounded by uniform, properly drained, moderately irrigated landscaping; if differing conditions will exist that will cause differential soil moisture adjacent or below the slabs, or if portions of the foundations will be located adjacent to relatively dry or wet soils, then we should be consulted and modifications to the values below would need to be modified in writing. Please refer to Section 4.1.10, Surface Drainage, Irrigation, and Landscaping, for additional recommendations. We recommend the slab-subgrade friction values provided in the most recent

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9Post-Tensioning Institute, 2012, Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soils (PTI DC10.5-12).
PTI Manual be used in order to determine the friction that might be expected to exist during tendon stressing.

**SWELLING MODE**

<table>
<thead>
<tr>
<th></th>
<th>Center Lift</th>
<th>Edge Lift</th>
</tr>
</thead>
<tbody>
<tr>
<td>Edge Moisture Variation Distance ((e_m))</td>
<td>9.0 feet</td>
<td>5.0 feet</td>
</tr>
<tr>
<td>Differential Soil Movement ((y_m))</td>
<td>1.0 inch</td>
<td>1.3 inch</td>
</tr>
</tbody>
</table>

We recommend SFB review the foundation drawings and specifications prior to submittal to verify that the recommendations provided in this report have been used and properly interpreted in the design of the slabs.

**4.2.2 Retaining Walls and Soundwalls**

If segmental block walls with geogrid will be used at the site, SFB should be contacted to provide block wall and geogrid designs and specifications.

Where walls retain soil, they must be designed to resist both lateral earth pressures and any additional lateral loads caused by surcharging such as building and roadway loads. The recommendations provided below are for retaining walls that are located at least 1.5\(H\) feet away from a building, where \(H\) is the height of the retaining portion of the walls. Where concrete or masonry walls are used to retain soil, we recommend unrestrained walls (walls free to deflect and disconnected from other structures) be designed to resist an equivalent fluid pressure of 50 pounds per cubic foot. This assumes a level backfill. Restrained walls (walls restrained from deflection) should be designed to resist an equivalent fluid pressure of 50 pounds per cubic foot plus a uniform pressure of 10\(H\) pounds per square foot, where \(H\) is the height of the wall in feet. Walls with inclined backfill should be designed for an additional equivalent fluid pressure of 1 pound per cubic foot for every 1 degree of slope inclination. Walls subjected to surcharge loads should be designed for an additional uniform lateral pressure equal to one-third and one-half the anticipated surcharge load for unrestrained and restrained walls, respectively. These lateral pressures depend upon the moisture content of the retained soils to be constant over time; if the moisture content of the retained soils will fluctuate or increase compared to the moisture content at time of construction, then SFB should be consulted and provide written modifications to this design criteria.

For retaining walls that need to resist earthquake induced lateral loads from nearby foundations, walls that are to be designed to resist earthquake loads, and any retaining walls that are higher
than 6 feet, we recommend the walls also be designed to resist a triangular pressure distribution equal to an equivalent fluid pressure of 50 pounds per cubic foot based on the ground acceleration from a design basis earthquake. This seismic induced earth pressure is in addition to the pressures noted above. Due to the transient nature of the seismic loading, a factor of safety of at least 1.1 can be used in the design of the walls when they resist seismic lateral loads. Some movement of the walls may occur during moderate to strong earthquake shaking and may result in distress as is typical for all structures subjected to earthquake shaking.

The recommended lateral pressures assume walls are fully-back drained to prevent the build-up of hydrostatic pressures. This can be accomplished by using ½ to ¾ inch crushed, uniformly graded gravel entirely wrapped in filter fabric such as Mirafi 140N or equal (an overlap of at least 12 inches should be provided at all fabric joints). The gravel and fabric should be at least 8 inches wide and extend from the base of the wall to within 12 inches of the finished grade at the top (Caltrans Class 2 permeable material (Section 68) may be used in lieu of gravel and filter fabric). A 4-inch diameter, perforated pipe should be installed at the base and centered within the gravel. The perforated pipe should be connected to a solid collector pipe that transmits the water directly to a storm drain, drainage inlet, or onto pavement. If weep holes are used in the wall, the perforated pipe within the gravel is not necessary provided the weep holes are kept free of animals and debris, are located no higher than approximately 6 inches from the lowest adjacent grade, and are able to function properly. As an alternative to using gravel, drainage panels (such as AWD SITEDRAIN Sheet 94 for walls or equal) may be used behind the walls in conjunction with perforated pipe (connected to solid collector pipe), weep holes, or strip drains (such as SITEDRAIN Strip 6000 or equal). If used, the drainage panels can be spaced on-center at approximately 2 times the panel width.

If heavy compaction equipment is used behind the walls, the walls should be appropriately designed to withstand loads exerted by the heavy equipment and/or temporarily braced. Fill placed behind walls should conform to the recommendations provided in Section 4.1.5, Fill Material, and Section 4.1.6, Compaction.

Retaining walls and soundwalls can be supported on drilled, cast-in-place, straight shaft friction piers that develop their load carrying capacity in the materials underlying the site. The piers should have a minimum diameter of 12 inches and a center-to-center spacing of at least three times the shaft diameter. We recommend that piers be at least 6 feet long. The pier reinforcing should be based on structural requirements but in no case should less than two #4 bars for the entire length of the pier be used.

The actual design depth of the piers should be determined using an allowable skin friction of 500 pounds per square foot (psf) for dead plus live loads, with a one-third increase for all loads including wind or seismic. Seventy percent of the skin friction value can be used to resist uplift.
Lateral load resistance can be developed in passive resistance for pier foundations. A passive resistance equal to an equivalent fluid weighing 350 pounds per cubic foot acting against twice the projected diameter of pier shafts can be used. The upper two feet of pier embedment should be neglected in the vertical and passive resistance design as measured from finished grade. The portion of the pier shaft located within 10 feet (as measured laterally) of the nearest slope face should also be ignored in the design.

We recommend the pier foundations be located outside of (or beyond) a 1:1 (horizontal to vertical) plane projected upward from the base of any wall or utility trench, or the portion of a pier located within this zone should be ignored in the design of the pier.

The bottoms of the pier excavations should be relatively dry and free of all loose cuttings or slough prior to placing reinforcing steel and concrete. Any accumulated water in pier excavations should be removed prior to placing concrete. We recommend that the excavation of all piers be performed under the direct observation of SFB to confirm that the pier foundations are founded in suitable materials and constructed in accordance with the recommendations presented herein. Preliminarily, we recommend concrete pours of pier excavations be performed within 24 hours of excavation and prior to any rainstorms. Where caving or high groundwater conditions exist, additional measures such as using casing, tremie methods, and pouring concrete immediately after excavating may be necessary. SFB should be consulted on the need for additional measures for pier construction as needed during construction.

### 4.2.3 Seismic Design Criteria

The following parameters were calculated using the U.S. Seismic Design Map program\textsuperscript{10}, and were based on the site being located at approximate latitude 37.942°N and longitude 122.081°W. For seismic design using the 2013 California Building Code (CBC), we recommend the following tabulated seismic design values be used.

<table>
<thead>
<tr>
<th>Seismic Parameter</th>
<th>Design Value</th>
<th>CBC Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site Class</td>
<td>D</td>
<td>Section 1613.3.2</td>
</tr>
<tr>
<td>S(_8)</td>
<td>1.61</td>
<td>Figure 1613.3.1(1)</td>
</tr>
<tr>
<td>S(_1)</td>
<td>0.60</td>
<td>Figure 1613.3.1(2)</td>
</tr>
<tr>
<td>F(_a)</td>
<td>1.0</td>
<td>Table 1613.3.3(1)</td>
</tr>
<tr>
<td>F(_v)</td>
<td>1.5</td>
<td>Table 1613.3.3(2)</td>
</tr>
</tbody>
</table>

4.3 Pavements

Based on the results of laboratory testing of onsite materials, we recommend that an R-value of 5 be used in preliminary asphalt concrete pavement design. We recommend R-value tests be performed once the pavement subgrade is established to confirm the R-value used in the design. Pavement subgrade composed of sandy and gravelly fills will result in higher R-values and thinner pavement sections.

We developed the following alternative preliminary pavement sections using Topic 608 of the State of California Department of Transportation Highway Design Manual, the recommended R-value, and typical traffic indices for residential developments. The project’s Civil Engineer or appropriate public agency should determine actual traffic indices. The pavement thicknesses shown below are SFB’s recommended minimum values; governing agencies may require pavement thicknesses greater than those shown.

<table>
<thead>
<tr>
<th>Location</th>
<th>Pavement Components</th>
<th>Total Thickness (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Asphalt Concrete (inches)</td>
<td>Class 2 Aggregate Base (inches)</td>
</tr>
<tr>
<td>T.I. = 4.5 (auto &amp; light truck parking)</td>
<td>3.0</td>
<td>9.0</td>
</tr>
<tr>
<td>T.I. = 5.0 (access ways/courts)</td>
<td>3.0</td>
<td>11.0</td>
</tr>
<tr>
<td>T.I. = 6.0 (primary roadways)</td>
<td>3.0</td>
<td>14.0</td>
</tr>
</tbody>
</table>

If the pavements are planned to be placed prior to or during construction, the traffic indices and pavement sections may not be adequate for support of what is typically more frequent and heavier construction traffic. If the pavement sections will be used for construction access by heavy trucks or construction equipment (especially fork lifts with support footings), SFB should be consulted to provide recommendations for alternative pavement sections capable of supporting the heavier use and heavier loads. If requested, SFB can provide recommendations for a phased placement of the asphalt concrete to reduce the potential for mechanical scars caused by construction traffic in the finished grade. Preliminary pavement sections should be revised, if necessary, when actual traffic indices are known and pavement subgrade elevations are determined.
Pavement baserock and asphalt concrete should be compacted to at least 95 percent relative compaction. The asphalt concrete compacted unit weight should be determined using Caltrans Test Method 308-A or ASTM Test Method D1188. Asphalt concrete should also satisfy the S-value requirements by Caltrans.

We recommend regular maintenance of the asphalt concrete be performed at approximately five year intervals. Maintenance may include sand slurry sealing, crack filling, and chip seals as necessary. If regular maintenance is not performed, the asphalt concrete layer could experience premature degradation requiring more extensive repairs.
5.0 CONDITIONS AND LIMITATIONS

SFB is not responsible for the validity or accuracy of information, analyses, test results, or designs provided to SFB by others or prepared by others. The analysis, designs, opinions, and recommendations submitted in this report are based in part upon the data obtained from our field work and upon information provided by others. Site exploration and testing characterizes subsurface conditions only at the locations where the explorations or tests are performed; actual subsurface conditions between explorations or tests may be different than those described in this report. Variations of subsurface conditions from those analyzed or characterized in this report are not uncommon and may become evident during construction. In addition, changes in the condition of the site can occur over time as a result of either natural processes (such as earthquakes, flooding, or changes in ground water levels) or human activity (such as construction adjacent to the site, dumping of fill, or excavating). If changes to the site’s surface or subsurface conditions occur since the performance of the field work described in this report, or if differing subsurface conditions are encountered, we should be contacted immediately to evaluate the differing conditions to assess if the opinions, conclusions, and recommendations provided in this report are still applicable or should be amended.

We recommend SFB be retained to provide geotechnical services during design, reviews, earthwork operations, paving operations, and foundation installation to confirm and observe compliance with the design concepts, specifications and recommendations presented in this report. Our presence will also allow us to modify design if unanticipated subsurface conditions are encountered or if changes to the scope of the project, as defined in this report, are made.

This report is a design document that has been prepared in accordance with generally accepted geological and geotechnical engineering practices for the exclusive use of St. Peter Construction Company, Inc. and their consultants for specific application to the proposed residential development project located at 2150 & 2198 Pleasant Hill Road in Pleasant Hill, California, and is intended to represent our design recommendations to St. Peter Construction Company, Inc. for specific application to the 2150 & 2198 Pleasant Hill Road project. The conclusions and recommendations contained in this report are solely professional opinions. It is the responsibility of St. Peter Construction Company, Inc. to transmit the information and recommendations of this report to those designing and constructing the project. We will not be responsible for the misinterpretation of the information provided in this report. We recommend SFB be retained to review geological and geotechnical aspects of the construction calculations, specifications, and plans; we should also be retained to participate in prebid and preconstruction conferences to clarify the opinions, conclusions, and recommendations contained in this report.
It should be understood that advancements in the practice of geotechnical engineering and engineering geology, or discovery of differing surface or subsurface conditions, may affect the validity of this report and are not uncommon. SFB strives to perform its services in a proper and professional manner with reasonable care and competence but we are not infallible. Geological engineering and geotechnical engineering are disciplines that are far less exact than other engineering disciplines; therefore we should be consulted if it is not completely understood what the limitations to using this report are.

In the event that there are any changes in the nature, design or location of the project, as described in this report, or if any future additions are planned, the conclusions and recommendations contained in this report shall not be considered valid unless we are contacted in writing, the project changes are reviewed by us, and the conclusions and recommendations presented in this report are modified or verified in writing. The opinions, conclusions, and recommendations contained in this report are based upon the description of the project as presented in the introduction section of this report.

This report does not necessarily represent all of the information that has been communicated by us to St. Peter Construction Company, Inc. and their consultants during the course of this engagement and our rendering of professional services to St. Peter Construction Company, Inc. Reliance on this report by parties other than those described above must be at their own risk unless we are first consulted as to the parties’ intended use of this report and only after we obtain the written consent of St. Peter Construction Company, Inc. to divulge information that may have been communicated to St. Peter Construction Company, Inc. We cannot accept consequences for use of segregated portions of this report.

Please refer to Appendix C for additional guidelines regarding use of this report.
SFB-1

Estimated Location of Possible Buried Drainage Swale

SFB-2

Approximate Location of SFB Exploratory Boring (3/22/16)

Approximate Project Limit

NOTE: Base map was created by overlaying the project preliminary site plan prepared by DK Consulting and dated 10/2/15 on Google Earth image dated 10/30/15.

APPROXIMATE SCALE: 1" = 120'

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2150 & 2198 Pleasant Hill Road
Pleasant Hill, California
APPENDIX A
Field Investigation
APPENDIX A
Field Investigation

Our field investigation for the proposed 2150 & 2198 Pleasant Hill Road residential development project in Pleasant Hill, California, consisted of surface reconnaissance and a subsurface exploration program. Geotechnical reconnaissance of the site and surrounding area was performed on March 22, 2016. Subsurface exploration was performed using a truck-mounted drill rig equipped with 4-inch diameter, continuous flight, solid stem augers. Three exploratory borings were drilled on March 22, 2016. Our representative continuously logged the soils encountered in the borings in the field. The soils are described in general accordance with the Unified Soil Classification System (ASTM D2487). The logs of the borings as well as a key for the classification of the soil (Figure A-1) are included as part of this appendix.

Representative samples were obtained from our exploratory borings at selected depths appropriate to the investigation. Relatively undisturbed samples were obtained using a 3-inch O.D. split barrel sampler with liners, and disturbed samples were obtained using the 2-inch O.D. split spoon sampler. All samples were transmitted to our offices for evaluation and appropriate testing. Both sampler types are indicated in the “Sampler” column of the boring logs as designated in Figure A-1.

Resistance blow counts were obtained in our boring with the samplers by dropping a 140-pound safety hammer through a 30-inch free fall. The sampler was driven 18 inches and the number of blows were recorded for each 6 inches of penetration. The blows per foot recorded on the boring logs represent the accumulated number of converted blows that were required to drive the last 12 inches, or the number of inches indicated where hard resistance was encountered. The blow counts recorded on the boring logs have been converted to equivalent SPT field blowcounts, but have not been corrected for overburden, silt content, or other factors.

The attached boring logs and related information show our interpretation of the subsurface conditions at the dates and locations indicated, and it is not warranted that they are representative of subsurface conditions at other locations and times.
### UNIFIED SOIL CLASSIFICATION SYSTEM

<table>
<thead>
<tr>
<th>Major Divisions</th>
<th>Description</th>
<th>Major Divisions</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravel</td>
<td>Well-graded gravels or gravel sand mixtures, little or no fines</td>
<td>( \text{PI} )</td>
<td>Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity</td>
</tr>
<tr>
<td>Gravely Soils</td>
<td>Poorly-graded gravels or gravel sand mixture, little or no fines</td>
<td>( \text{LL} ) &amp; ( \text{CL} )</td>
<td>Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays</td>
</tr>
<tr>
<td>Coarse Grained Soils</td>
<td>Silty gravels, gravel-sand-silt mixtures</td>
<td>( \text{OL} )</td>
<td>Organic silts and organic silt-clays of low plasticity</td>
</tr>
<tr>
<td></td>
<td>Clayey gravels, gravel-sand-clay mixtures</td>
<td>( \text{MH} )</td>
<td>Inorganic silts, micaceous or diatomaceous fine or silty soils, elastic silts</td>
</tr>
<tr>
<td></td>
<td>Well-graded sands or gravelly sands, little or no fines</td>
<td>( \text{CH} )</td>
<td>Inorganic clays of high plasticity, fat clays</td>
</tr>
<tr>
<td></td>
<td>Poorly-graded sands or gravelly sands, little or no fines</td>
<td>( \text{OH} )</td>
<td>Organic clays of medium to high plasticity</td>
</tr>
<tr>
<td></td>
<td>Silty sands, sand-silt mixtures</td>
<td>( \text{PT} )</td>
<td>Peat and other highly organic soils</td>
</tr>
</tbody>
</table>

### GRAIN SIZES

<table>
<thead>
<tr>
<th>U.S. STANDARD SERIES SIEVE</th>
<th>CLEAR SQUARE SIEVE OPENINGS</th>
</tr>
</thead>
<tbody>
<tr>
<td>200</td>
<td>3/4&quot;</td>
</tr>
<tr>
<td>40</td>
<td>3&quot;</td>
</tr>
<tr>
<td>10</td>
<td>12&quot;</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Sands and Clays</th>
<th>Sieve Material</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fine</td>
<td>Medium</td>
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<tr>
<td>Gravel</td>
<td>Fine</td>
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<tr>
<td>Sand</td>
<td>Fine</td>
</tr>
<tr>
<td>Soils</td>
<td>Fine</td>
</tr>
</tbody>
</table>

### RELATIVE DENSITY

<table>
<thead>
<tr>
<th>Sands and Gravels</th>
<th>Blows/Foot*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Loose</td>
<td>0 - 4</td>
</tr>
<tr>
<td>Loose</td>
<td>4 - 10</td>
</tr>
<tr>
<td>Medium Dense</td>
<td>10 - 30</td>
</tr>
<tr>
<td>Dense</td>
<td>30 - 50</td>
</tr>
<tr>
<td>Very Dense</td>
<td>Over 50</td>
</tr>
</tbody>
</table>

### CONSISTENCY

<table>
<thead>
<tr>
<th>Sands and Clays</th>
<th>Blows/Foot*</th>
<th>Strength (tsf)**</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Soft</td>
<td>0 - 2</td>
<td>0 - 1/4</td>
</tr>
<tr>
<td>Soft</td>
<td>2 - 4</td>
<td>1/4 - 1/2</td>
</tr>
<tr>
<td>Firm</td>
<td>4 - 8</td>
<td>1/2 - 1</td>
</tr>
<tr>
<td>Stiff</td>
<td>8 - 16</td>
<td>1 - 2</td>
</tr>
<tr>
<td>Very Stiff</td>
<td>16 - 32</td>
<td>2 - 4</td>
</tr>
<tr>
<td>Hard</td>
<td>Over 32</td>
<td>Over 4</td>
</tr>
</tbody>
</table>

### SYMBOLS & NOTES

- Standard Penetration sampler (2" OD Split Barrel)
- Modified California sampler (3" OD Split Barrel)
- California Sampler (2.5" OD Split Barrel)
- Shelby Tube
- Pitcher Barrel
- HQ Core
- Ground Water level initially encountered
- Ground Water level at end of drilling

### PROJECT NO.

254-53

### DATE

April 2016

### FIGURE NO.

A-1
### DESCRIPTION AND CLASSIFICATION

<table>
<thead>
<tr>
<th>DESCRIPTION AND REMARKS</th>
<th>CONSIST</th>
<th>SOIL TYPE</th>
<th>DEPTH (FEET)</th>
<th>SPT N-VALUE</th>
<th>WATER CONTENT (%)</th>
<th>DRY DENSITY (PCF)</th>
<th>UNC. COMP. (KSF)</th>
<th>OTHER TESTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>CLAY (CL), grayish brown, silty, some sand (fine-grained), trace rootlets, damp.</td>
<td>stiff</td>
<td>0</td>
<td>13</td>
<td>23</td>
<td></td>
<td></td>
<td></td>
<td>At 2’s: Liquid Limit = 45 Plasticity Index = 31 Medium Sand = 2% Fine Sand = 20% Silt = 39% Clay = 39%</td>
</tr>
<tr>
<td>Change color to mottled gray brown, some to with sand (fine-to-medium-grained).</td>
<td>very stiff</td>
<td>5</td>
<td>21</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SILT (ML), mottled grayish yellowish brown, sandy (fine-to-medium-grained), trace clay, dry to damp.</td>
<td>medium dense</td>
<td>10</td>
<td>26</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SAND (SC), yellowish brown, fine- to medium-grained, with clay &amp; silt, dry to damp.</td>
<td>medium dense</td>
<td>15</td>
<td>47</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CLAY (CL), olive brown, silty, with sand (fine-to-medium-grained), dry.</td>
<td>very stiff</td>
<td>20</td>
<td>52</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CLAY (CL/CH), olive gray brown, silty, with sand (fine-grained), dry.</td>
<td>hard</td>
<td>25</td>
<td>49</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Interbeded with sand lense (fine- to medium-grained).</td>
<td>hard</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Change color to mottled olive brownish gray, some to with sand (fine-to-medium-grained), dry to damp.</td>
<td></td>
<td>30</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Interbeded sand with lense (fine- to medium-grained).</td>
<td></td>
<td>35</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bottom of Boring = 21.5 feet</td>
<td>Notes: Stratification is approximate. Variations must be expected. Blowcounts converted to SPT N-values. See Report for additional details.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
**EXPLORATORY BORING LOG**

<table>
<thead>
<tr>
<th>BORING NO.</th>
<th>DATE</th>
<th>BORING NO.</th>
</tr>
</thead>
<tbody>
<tr>
<td>254-53</td>
<td>April 2016</td>
<td>SFB-2</td>
</tr>
</tbody>
</table>

**2150 & 2198 PLEASANT HILL ROAD**
Pleasant Hill, California

---

**DESCRIPTION AND CLASSIFICATION**

<table>
<thead>
<tr>
<th>DESCRIPTION AND REMARKS</th>
<th>CONSIST</th>
<th>SOIL TYPE</th>
<th>DEPTH (FEET)</th>
<th>SAMPLER</th>
<th>SPT N-VALUE</th>
<th>WATER CONTENT (%)</th>
<th>DRY DENSITY (pcf)</th>
<th>UNC. COMP. (KSF)</th>
<th>OTHER TESTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>FILL: CLAY (CL), mottled dark gray brown, silty, some sand(fine- to medium-grained), trace gravel(fine, subrounded to subangular), moist to wet.</td>
<td>stiff</td>
<td></td>
<td>0</td>
<td></td>
<td>9</td>
<td>16</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CLAY (CL), mottled brown, silty, with sand(fine- to medium-grained, trace coarse-grained), trace gravel(fine, subrounded), dry to damp. Some gravel(fine, subrounded), dry to damp.</td>
<td>hard</td>
<td></td>
<td>5</td>
<td></td>
<td>52</td>
<td>17</td>
<td>112</td>
<td>11.3</td>
<td></td>
</tr>
<tr>
<td>SILT (ML)/SAND (SM), olive gray brown, sandy(fine- to medium-grained), dry to damp.</td>
<td>hard</td>
<td></td>
<td>10</td>
<td></td>
<td>58/10&quot;</td>
<td>22</td>
<td>101</td>
<td>4.7</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>15</td>
<td></td>
<td>30/5&quot;</td>
<td>21</td>
<td>102</td>
<td>7.3</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Change color to mottled orangish olive gray brown.

Bottom of Boring = 21.5 feet

Notes: Stratification is approximate, variations must be expected. Blowcounts converted to SPT N-values. See Report for additional details.
<table>
<thead>
<tr>
<th>DESCRIPTION AND REMARKS</th>
<th>CONSIST</th>
<th>SOIL TYPE</th>
<th>DEPTH (FEET)</th>
<th>SPT N-VALUE</th>
<th>WATER CONTENT (%)</th>
<th>DRY DENSITY (PCF)</th>
<th>UNCONS. COMP. (KSF)</th>
<th>OTHER TESTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>FILL: GRAVEL (GC), mottled dark grayish brown, fine to coarse, angular to subrounded, sandy (fine- to coarse-grained), with clay &amp; silt, some asphalt concrete fragments, damp. Clayey, damp.</td>
<td>medium dense</td>
<td>loose</td>
<td>0</td>
<td>11</td>
<td>16</td>
<td>94</td>
<td>1.6</td>
<td>At 11&quot;: Liquid Limit = 49 Plasticity Index = 35</td>
</tr>
<tr>
<td>FILL: CLAY (CL), mottled dark gray yellowish brown, silty, some sand (fine- to coarse-grained), trace gravel (fine, subrounded to subangular), damp.</td>
<td>stiff</td>
<td></td>
<td>5</td>
<td>8</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CLAY (CL), dark brownish gray, silty, some sand (fine- to medium-grained), trace gravel (fine, subrounded to subangular), trace rootlets, moist.</td>
<td>stiff</td>
<td></td>
<td>10</td>
<td>26</td>
<td>94</td>
<td>1.6</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Some gravel (fine, angular to subrounded), wet.</td>
<td></td>
<td></td>
<td>15</td>
<td>22</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CLAY (CL), grayish brown, silty, some sand (fine- to medium-grained, trace coarse-grained), trace gravel (fine, subangular to rounded), dry to damp.</td>
<td>stiff</td>
<td></td>
<td>20</td>
<td>28</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CLAY (CL/CH), olive gray brown, silty, some sand (fine-grained), dry to damp.</td>
<td>very stiff</td>
<td></td>
<td>25</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bottom of Boring = 21.5 feet</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes: Stratification is approximate, variations must be expected. Blowcounts converted to SPT N-values. See Report for additional details.
APPENDIX B
Laboratory Investigation
APPENDIX B
Laboratory Investigation

Our laboratory testing program for the proposed 2150 & 2198 Pleasant Hill Road residential
development project in Pleasant Hill, California was directed toward a quantitative and
qualitative evaluation of the physical and mechanical properties of the soils underlying the site.

The natural water content was determined on seven samples of the subsurface soils. The water
contents are recorded on the boring logs at the appropriate sample depths.

Dry density determination was performed on four samples of the subsurface soils to evaluate
their physical properties. The results of the tests are shown on the boring logs at the appropriate
sample depths.

Atterberg Limit determinations were performed on two samples of the subsurface soils to
determine the range of water content over which these materials exhibit plasticity. These values
are used to classify the soil in accordance with the Unified Soil Classification System and to
indicate the soil's compressibility and expansion potentials. The results of the tests are presented
on the boring log at the appropriate sample depths and are also attached to this appendix.

Unconfined compression test was performed on four relatively undisturbed samples of the
subsurface soils to evaluate the undrained shear strengths of these materials. Failure was taken
as the peak normal stress. The results of the tests are presented on the boring logs at the
appropriate sample depths and are also attached to this appendix.

Gradation and hydrometer tests were performed on one sample of the subsurface soils. These
tests were performed to assist in the classification of the soils and to determine their grain size
distribution. The results of the tests are presented on the boring logs at the appropriate sample
depth and are also attached to this appendix.

Two onsite soil samples were tested for pH (ASTM D4972), chlorides (ASTM D4327), sulfates
(ASTM D4327), sulfides (ASTM D4658M), resistivity at 100% saturation (ASTM G57), and
Redox potential (ASTM D1498) for use in evaluating the potential for corrosion on concrete and
buried metal such as utilities and reinforcing steel. The results of these tests are included in this
Appendix. We recommend these test results be forwarded to your underground contractors,
pipeline designers, and foundation designers and contractors.
Atterberg Limits Test – ASTM D4318

Project Number: 254-53  Project Name: 2150 & 2198 Pleasant Hill Road
Boring/Sample Number: B-1  Depth: 2  Date: 03-25-16
Description of Sample: Brown silty CLAY with sand (CL)  Tested By: R

Data Summary

- Liquid Limit: 45
- Plastic Limit: 14
- Plasticity Index: 31
- Natural Water Content: 23.3
- Liquidity Index: 0.300
- % Passing #200: 78.3

Plastic Limit Data

<table>
<thead>
<tr>
<th>Trial</th>
<th>1</th>
<th>2</th>
<th>Ave</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water Content (%)</td>
<td>14.2</td>
<td>13.9</td>
<td>14</td>
</tr>
</tbody>
</table>

Plasticity Chart

- "U" Line
- "A" Line
- CL or OL
- ML or OL
- MH or OH
- CH or OH
- CL-ML

Number of Blows: 10 to 50
Water Content (%): 43 to 48
Liquid Limit (%): 43 to 48
Plasticity Index (%): 10 to 60
Liquid Limit (%): 0 to 100
Hydrometer Analysis – ASTM D422

Project Number: 254-53  Project Name: 2150 & 2198 Pleasant Hill Road
Sample Number: B-1  Description: Brown silty CLAY with sand (CL)
Depth: 2  Test Date: 03-28-16  Tested By: R

Composite Sieve Data

<table>
<thead>
<tr>
<th>US Standard Sieve Size</th>
<th>Percent Retained</th>
</tr>
</thead>
<tbody>
<tr>
<td>3&quot;</td>
<td>100</td>
</tr>
<tr>
<td>1.5&quot;</td>
<td>99.8</td>
</tr>
<tr>
<td>1&quot;</td>
<td>99.7</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>98.5</td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>93.8</td>
</tr>
<tr>
<td>#4</td>
<td>78.3</td>
</tr>
<tr>
<td>#10</td>
<td>100</td>
</tr>
<tr>
<td>#16</td>
<td>99.8</td>
</tr>
<tr>
<td>#30</td>
<td>99.7</td>
</tr>
<tr>
<td>#50</td>
<td>98.5</td>
</tr>
<tr>
<td>#100</td>
<td>93.8</td>
</tr>
<tr>
<td>#200</td>
<td>78.3</td>
</tr>
</tbody>
</table>

Particle Size (mm)  Percent Soil in Suspension

<table>
<thead>
<tr>
<th>Particle Diameter (mm)</th>
<th>Percent Soil in Suspension</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0319</td>
<td>58.9</td>
</tr>
<tr>
<td>0.0206</td>
<td>53.5</td>
</tr>
<tr>
<td>0.0122</td>
<td>46.4</td>
</tr>
<tr>
<td>0.0088</td>
<td>42.8</td>
</tr>
<tr>
<td>0.0062</td>
<td>41.0</td>
</tr>
<tr>
<td>0.0031</td>
<td>37.5</td>
</tr>
<tr>
<td>0.0013</td>
<td>32.1</td>
</tr>
</tbody>
</table>
Atterberg Limits Test – ASTM D4318

Project Number: 254-53  Project Name: 2150 & 2198 Pleasant Hill Road

Boring/Sample Number: B-3  Depth: 11  Date: 03-25-16

Description of Sample: Dark brown silty CLAY some sand trace gravel (CL)

Tested By: R

Data Summary

<table>
<thead>
<tr>
<th>Liquid Limit</th>
<th>Plastic Limit</th>
<th>Plasticity Index</th>
<th>Natural Water Content</th>
<th>Liquidity Index</th>
<th>% Passing #200</th>
</tr>
</thead>
<tbody>
<tr>
<td>49</td>
<td>14</td>
<td>35</td>
<td>25.8</td>
<td>0.337</td>
<td>--</td>
</tr>
</tbody>
</table>

Plastic Limit Data

<table>
<thead>
<tr>
<th>Trial</th>
<th>1</th>
<th>2</th>
<th>Ave</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water Content (%)</td>
<td>13.6</td>
<td>13.5</td>
<td>14</td>
</tr>
</tbody>
</table>

Plasticity Chart

- CL or OL
- ML or OL
- CL-ML
- CH or OH
- "A" Line
- "U" Line
- MH or OH

% Passing #200: --
UNCONFINED COMpressive STRENGTh – D2166

Project Number: 254-53  Boring #: B-2  Depth: 6
Project Name: 2150 & 2198 Pleasant Hill Road  Date: 3/24/2016
Description: Brown silty CLAY some sand (CL)  Tested By: R

Soil Specimen Initial Measurements

<table>
<thead>
<tr>
<th>Diameter</th>
<th>2.42 in</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Area</td>
<td>4.60 in²</td>
</tr>
<tr>
<td>Initial Length</td>
<td>5 in</td>
</tr>
<tr>
<td>Volume</td>
<td>0.01331 ft³</td>
</tr>
<tr>
<td>Water Content</td>
<td>16.8</td>
</tr>
<tr>
<td>Wet Density</td>
<td>130.2 pcf</td>
</tr>
<tr>
<td>Dry Density</td>
<td>111.5 pcf</td>
</tr>
</tbody>
</table>

Max Unconfined Compressive Strength

<table>
<thead>
<tr>
<th>Elapsed Time</th>
<th>6 min</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical Dial</td>
<td>0.3 in</td>
</tr>
<tr>
<td>Strain</td>
<td>6.0 %</td>
</tr>
<tr>
<td>Area</td>
<td>0.03398 ft²</td>
</tr>
<tr>
<td>Axial Load</td>
<td>382.6 lbs</td>
</tr>
<tr>
<td>Compressive Strength</td>
<td>11,258 psf</td>
</tr>
</tbody>
</table>
**UNCONFINED COMpressive STRENGTH – D2166**

**Project Number:** 254-53  **Boring #:** B-2  **Depth:** 11

**Project Name:** 2150 & 2198 Pleasant Hill Road  **Date:** 3/24/2016

**Description:** Light gray brown sandy SILT some clay (ML/SM)  **Tested By:** R

---

**Soil Specimen Initial Measurements**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter</td>
<td>2.42 in</td>
</tr>
<tr>
<td>Initial Area</td>
<td>4.60 in²</td>
</tr>
<tr>
<td>Initial Length</td>
<td>5 in</td>
</tr>
<tr>
<td>Volume</td>
<td>0.01331 ft³</td>
</tr>
<tr>
<td>Water Content</td>
<td>22.3</td>
</tr>
<tr>
<td>Wet Density</td>
<td>123.1 pcf</td>
</tr>
<tr>
<td>Dry Density</td>
<td>100.7 pcf</td>
</tr>
</tbody>
</table>

---

**Max Unconfined Compressive Strength**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elapsed Time</td>
<td>5.5 min</td>
</tr>
<tr>
<td>Vertical Dial</td>
<td>0.275 in</td>
</tr>
<tr>
<td>Strain</td>
<td>5.5 %</td>
</tr>
<tr>
<td>Area</td>
<td>0.03380 ft²</td>
</tr>
<tr>
<td>Axial Load</td>
<td>159.2 lbs</td>
</tr>
<tr>
<td>Compressive Strength</td>
<td>4,710 psf</td>
</tr>
</tbody>
</table>
**UNCONFINED COMPRESSIVE STRENGTH – D2166**

**Project Number:** 254-53  
**Boring #:** B-2  
**Depth:** 15.5

**Project Name:** 2150 & 2198 Pleasant Hill Road  
**Date:** 3/24/2016

**Description:** Light gray brown sandy SILT some clay (ML/SM)  
**Tested By:** R

---

### Soil Specimen Initial Measurements

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter</td>
<td>2.42 in</td>
</tr>
<tr>
<td>Initial Area</td>
<td>4.60 in²</td>
</tr>
<tr>
<td>Initial Length</td>
<td>5 in</td>
</tr>
<tr>
<td>Volume</td>
<td>0.01331 ft³</td>
</tr>
<tr>
<td>Water Content</td>
<td>20.9</td>
</tr>
<tr>
<td>Wet Density</td>
<td>123.5 pcf</td>
</tr>
<tr>
<td>Dry Density</td>
<td>102.2 pcf</td>
</tr>
</tbody>
</table>

---

### Max Unconfined Compressive Strength

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elapsed Time</td>
<td>2.5 min</td>
</tr>
<tr>
<td>Vertical Dial</td>
<td>0.125 in</td>
</tr>
<tr>
<td>Strain</td>
<td>2.5 %</td>
</tr>
<tr>
<td>Area</td>
<td>0.03276 ft²</td>
</tr>
<tr>
<td>Axial Load</td>
<td>239.3 lbs</td>
</tr>
<tr>
<td>Compressive Strength</td>
<td>7,304 psf</td>
</tr>
</tbody>
</table>
Project Number: 254-53  Boring #: B-3  Depth: 11
Project Name: 2150 & 2198 Pleasant Hill Road  Date: 3/24/2016
Description: Dark brown silty CLAY some sand trace gravel (CL)  Tested By: R

Soil Specimen Initial Measurements

<table>
<thead>
<tr>
<th>Measurement</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter</td>
<td>2.42 in</td>
</tr>
<tr>
<td>Initial Area</td>
<td>4.60 in²</td>
</tr>
<tr>
<td>Initial Length</td>
<td>5 in</td>
</tr>
<tr>
<td>Volume</td>
<td>0.01331 ft³</td>
</tr>
<tr>
<td>Water Content</td>
<td>25.8</td>
</tr>
<tr>
<td>Wet Density</td>
<td>118.5 pcf</td>
</tr>
<tr>
<td>Dry Density</td>
<td>94.2 pcf</td>
</tr>
</tbody>
</table>

Max Unconfined Compressive Strength

<table>
<thead>
<tr>
<th>Measurement</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elapsed Time</td>
<td>9.5 min</td>
</tr>
<tr>
<td>Vertical Dial</td>
<td>0.475 in</td>
</tr>
<tr>
<td>Strain</td>
<td>9.5 %</td>
</tr>
<tr>
<td>Area</td>
<td>0.03530 ft²</td>
</tr>
<tr>
<td>Axial Load</td>
<td>55.9 lbs</td>
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<tr>
<td>Compressive Strength</td>
<td>1,584 psf</td>
</tr>
<tr>
<td>Job/Sample No.</td>
<td>Sample I.D.</td>
</tr>
<tr>
<td>---------------</td>
<td>-------------</td>
</tr>
<tr>
<td>1603233-001</td>
<td>SFB-1 @ 1.5'</td>
</tr>
<tr>
<td>1603233-002</td>
<td>SFB-2 @ 5.5'</td>
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Method:

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<th>ASTM D1498</th>
<th>ASTM D4972</th>
<th>ASTM D1125M</th>
<th>ASTM G57</th>
<th>ASTM D4658M</th>
<th>ASTM D4327</th>
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Reporting Limit:

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<th>Redox</th>
<th>pH</th>
<th>Conductivity</th>
<th>Resistivity</th>
<th>Sulfide</th>
<th>Chloride</th>
<th>Sulfate</th>
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</thead>
<tbody>
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<td>-</td>
<td>10</td>
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<td>N.D.</td>
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Date Analyzed:

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<th>Conductivity</th>
<th>Resistivity</th>
<th>Sulfide</th>
<th>Chloride</th>
<th>Sulfate</th>
</tr>
</thead>
</table>

* Results Reported on "As Received" Basis
N.D. - None Detected

Cheryl McMillen
Laboratory Director

Quality Control Summary - All laboratory quality control parameters were found to be within established limits
APPENDIX C
ASFE Guidelines
Important Information about Your Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared solely for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. And no one—not even you—should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, always inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. Do not rely on a geotechnical engineering report whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. Always contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are Not Final

Do not overrely on the construction recommendations included in your report. These recommendations are not final, because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual
subsurface conditions revealed during construction. The geotechnical engineer who developed your report cannot assume responsibility or liability for the report’s recommendations if that engineer does not perform construction observation.

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members’ misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer’s Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should never be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, but recognize that separating logs from the report can elevate risk.

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, but preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report’s accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. Be sure contractors have sufficient time to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled “limitations,” many of these provisions indicate where geotechnical engineers’ responsibilities begin and end, to help others recognize their own responsibilities and risks. Read these provisions closely. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a geoenvironmental study differ significantly from those used to perform a geotechnical study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. Unanticipated environmental problems have led to numerous project failures. If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. Do not rely on an environmental report prepared for someone else.

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the express purpose of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant. none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.

Rely on Your ASFE-Member Geotechnical Engineer for Additional Assistance

Membership in ASFE/The Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

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