Stevens Ferrone & Bailey Engineering Company, Inc

Geotechnical Engineering
Engineering Geology
Storm Water Management
Construction Observation & Testing Services

October 5, 2017

Mr. David Agee
Warmington Residential
2400 Camino Ramon, Suite 234
San Ramon, CA 94583

Re: Geotechnical Investigation
Acacia Village, Santa Rosa CA
SFB Project No.: 552-6

Mr. Agee:

As requested, Stevens, Ferrone & Bailey Engineering Company, Inc. has performed a geotechnical investigation for the proposed Acacia Village residential development to be located at 746 Acacia Lane in Santa Rosa, 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

HP/KCF:lc\encl.
Copies: Addressee (1 by email)

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October 5, 2017

GEOTECHNICAL INVESTIGATION
ACACIA VILLAGE
SANTA ROSA, CALIFORNIA
SFB PROJECT NO. 552-6

Prepared For:

Warmington Residential
2400 Camino Ramon, Suite 234
San Ramon, CA 94583

Prepared By:

Stevens, Ferrone & Bailey Engineering Company, Inc.

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1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed Acacia Village residential development to be located at 746 Acacia Lane in Santa Rosa, 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. David Agee of Warmington Residential, it is our understanding that the project will consist of developing approximately 2.5 acres of land for 19 cottages with remote garages, and 6 two-story single-family homes. Associated underground utilities, access ways, a village green, and a community facility will be provided. Nominal grading is anticipated. 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 31-1/2 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, and pavements. Toxicity potential assessment of onsite materials or groundwater (including mold) and flooding evaluations were beyond our scope of work.

3.0 SITE INVESTIGATION

Reconnaissance of the site and surrounding area was performed on September 27, 2017. 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 September 27, 2017 to a maximum depth of about 31-1/2 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 Acacia Lane on the southwest, and existing residential developments on the other sides. The site was rectangular in shape, relatively level, and had a plan area of about 2.5 acres with maximum dimensions of about 430 feet by 255 feet.

At the time of our field exploration, an existing single-story, wood-frame residence with associated facilities built in approximately 1953, a storage shed, and parking cover were located in the middle of the site. An abandoned septic tank was located on the northwest side of the residence and a water well was located behind the residence. Two chicken and turkey coops were observed behind the existing residence. The side portions of the site were vacant (except for several grazing goats) and the surface vegetation generally consisted of a heavy growth of weeds and grasses. Large and small diameter trees were generally surrounding the existing residence and along the site boundary. Fencing and debris were observed throughout the site.

Based on our review of historical aerial photographs and topographic maps of the site and vicinity, it is our understanding the site was previously occupied by an orchard that had possibly been removed in the 1950's when the house was built.

3.2 Subsurface

The near-surface soil materials encountered by our borings at the site generally consisted of interbedded stiff to hard silty clays and medium dense to very dense sandy gravels that extended to the maximum depth explored of about 31-1/2 feet. According to the results of laboratory testing, the onsite more clayey soil materials have a high plasticity and high 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 measured in our borings at depths of about 10-1/2 to 14 feet at the end of drilling. Our borings were backfilled with lean cement grout in accordance with Sonoma County requirements prior to leaving the site. It should be noted that our 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 at the site have been mapped as Pleasanton-Haire complex (0 to 9 percent slopes) in the southern corner of the site and Positas gravelly loam (0 to 9 percent slopes) in the other portions of the site by USDA Web Soil Survey (WSS)[1]. The soils were assigned to Hydrologic Soil Groups C and D by USDA Natural Resources Conservation Service (NRCS); the soils have been categorized as having very low to moderately high transmission rates (approximately 0 to 0.57 inches per hour).

The Group C soil is defined as having a slow infiltration rate when thoroughly wet and may consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. Group D soil is defined as having a very slow infiltration rate when thoroughly wet (high runoff potential) 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.

3.5 Geology and Seismicity

According to Sowers, et al, (1998)[2], the site is underlain by Late Pleistocene to Holocene fan deposits that are generally composed of sand, gravel, silt, and clay that are moderately to poorly sorted, and moderately to poorly bedded. Wagner and Gutierrez (2010)[3] also mapped the site as

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[1]http://websoilsurvey.sc.egov.usda.gov/App/HomePage.htm
[2]Sowers, Noller, and Lettis, 1998, "*Map Showing Quaternary Geology and Liquefaction Susceptibility, Napa, California, 1: 100,000 Quadrangle: A Digital Database*", USGS Open File Report 98-460.
[3]Wagner & Gutierrez, 2010, "Preliminary *Geologic Map of the Napa 30' X 60' Quadrangle, California*", USGS.

being underlain by Holocene alluvial fan deposits that are generally composed of sand, gravel, silt, and clay that are moderately to poorly sorted, and moderately to poorly bedded.

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 Santa Rosa Quadrangle[4], 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 (2016)[5] has stated that there is a 72 percent chance of at least one magnitude 6.7 or greater earthquake striking the San Francisco Bay region between 2014 and 2043. Therefore, the site will probably be subjected to at least one moderate to severe earthquake that will cause strong ground shaking.

According to the Unified Hazard Tool (Dynamic: Conterminous U.S. 2014 v4.1.1) deaggregation model developed by U.S. Geological Survey (2014)[6], the site has a 10% probability of exceeding a peak ground acceleration of about 0.7g in 50 years (design basis ground motion based on stiff to hard 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 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

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[4]State of California, "*Special Studies Zones, Santa Rosa Quadrangle*", Revised Official Map, Effective: July 1, 1983.

[5]Aagaard, Blair, Boatwright, Garcia, Harris, Michael, Schwartz, and DiLeo, "*Earthquake Outlook for the San Francisco Bay Region 2014–2043*", USGS Fact Sheet 2016–3020, Revised August 2016 (ver. 1.1).

[6]https://earthquake.usgs.gov/hazards/interactive/

surface. According to ABAG and the U.S. Geological Survey[7,8], the site is located in an area that has been characterized as having low to moderate liquefaction susceptibility. Sowers, et al, (1998)[9] also mapped the site as having low to moderate liquefaction susceptibility. As of the date of this report, the liquefaction potential of the site has not been evaluated by the State of California[10].

SFB performed SPT-based liquefaction analyses based on procedures described by the Southern California Earthquake Center (SCEC, Martin and Lew, 1999), research papers by Seed (2001)[11], and EERI Monograph 12 (2008)[12]. Peak ground accelerations from Maximum Considered Earthquake (MCE) with a modal earthquake magnitude of 7.5 were used in our analyses. The MCE peak ground motion has a 2% probability of being exceeded in a 50-year period (mean return time of 2,475 years), which results in an onsite peak ground acceleration of 0.81g per ASCE 7.

The results of our analyses indicate that the saturated, medium dense, thin sand lens of about 3 feet thick encountered by our Boring SFB-2 at a depth of about 15 feet has a low potential for liquefying when subjected to MCE earthquake events. Therefore, it is our opinion that the potential for ground surface damage at the site because of liquefaction is low.

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[7]Witter, Knudsen, Sowers, Wentworth, Koehler, and Randolph, 2006, "*Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, California*", USGS Open File Report 2006-1037.

[8]Knudsen, Sowers, Witter, Wentworth, and Helly, 2000, "*Preliminary Maps of Quaternary Deposits and Liquefaction Susceptibility, Nine-County San Francisco Bay Region, California*", USGS Open File Report 00-444.

[9]Sowers, Noller, and Lettis, 1998, "*Liquefaction Susceptibility Map, Napa, California, 1: 100,000 Quadrangle: A Digital Database*", USGS Open File Report 98-460.

[10]Seismic Hazards Mapping Act, 1990.

[11]Seed et al., 2001, "*Recent Advances in Soil Liquefaction Engineering and Seismic Site Response Evaluation*", Fourth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics and Symposium in Honor of Professor W.D. Liam Finn, San Diego, California.

[12]Idriss & Boulanger, 2008, "*Soil Liquefaction During Earthquakes*", Earthquake Engineering Research Institute, MNO-12.

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.

SURFACE SOIL RE-COMPACATION: The removal of the existing structures and improvements at the site will likely result in loosening of the surface soils in the upper 2 to 3 feet. In addition, the removal of the previous orchard (and existing trees) loosen the soils in the upper 2 to 3 feet. To reduce the potential for damaging differential settlement of overlying improvements (such as new fills, building foundations, driveways, exterior flatwork, and pavements), we recommend these weak soils be completely removed and re-compacted.

We estimate the process consist of over-excavating to 2 feet, scarifying and re-compacting the bottom 12 inches in-place, and replacing the excavation with compacted fill materials. 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 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.

SHALLOW GROUNDWATER: Groundwater was measured in our borings at depths of about 10-1/2 to 14 feet at the end of drilling. Temporary dewatering of excavations that extend below the groundwater may be necessary such as during underground utility installations.

EXPANSION POTENTIAL: The more clayey, 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.

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 under separate cover. We recommend these test results be forwarded to your underground contractors, pipeline designers, and foundation designers and contractors so that they can design and install corrosion protection measures. Please be aware that we are not corrosion protection experts; we recommend corrosion protection measures be designed and constructed so that all concrete and metal is protected against corrosion. We also recommend additional testing be performed if the test results in Appendix B are deemed insufficient by the designers of the corrosion protection measures.

ADDITIONAL RECOMMENDATIONS: Detailed drainage, earthwork, foundation, retaining wall, garage wall, exterior slabs, 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 assume no responsibility for misinterpretation of our recommendations

4.1 Earthwork

4.1.1 Clearing and Site Preparation

The site should be cleared of all obstructions any existing structures and their entire foundation systems, wells and septic systems, existing driveway, existing utilities and pipelines and their associated backfill, designated trees and their associated entire root systems, 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 4.1.4, *Fill Material***, and compacted to the requirements in **Section 4.1.5, *Compaction***. Tree roots may extend to depths of about 3 to 4 feet. Wells and septic systems should be abandoned in accordance with Sonoma 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 4.1.4, *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.

4.1.2 Soil Re-Compaction

As described previously, the removal of the existing structures and improvements at the site (including the previous orchard and the existing trees) will result in loosening of the surface soils in the upper 2 to 3 feet. We recommend these soils be completely removed and re-compacted.

We estimate the process consist of over-excavating 2 feet, scarifying and re-compacting the bottom 12 inches in-place, and replacing the excavation with compacted fill materials. 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 soil materials may be used as new fill onsite provided it satisfies the recommendations provided in **Section 4.1.4, *Fill Material***. Compaction should be performed in accordance with the recommendations in **Section 4.1.5, *Compaction***.

4.1.3 Subgrade Preparation

After the completion of clearing, site preparation, and weak fill and soil re-compaction, soil exposed in areas to receive improvements (such as structural fill, building foundations, driveways, exterior flatwork, and pavements) should be scarified to a depth of about 12 inches, moisture conditioned to approximately 5 percent over optimum water content, and compacted to the requirements for structural fill. If building pads or pavement subgrade are allowed to remain exposed to sun, wind, or rain for an extended period of time, or are disturbed by borrowing animals or vehicles, the exposed subgrade or pavement subgrade may need to be reconditioned (moisture conditioned and/or scarified and recompacted) prior to foundation or pavement construction. SFB should be consulted on the need for subgrade reconditioning when the subgrade is left exposed for extended periods of time.

4.1.4 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.

4.1.5 Compaction

We recommend structural fill be compacted to between 88 and 92 percent relative compaction, as determined by ASTM D1557 (latest edition). We recommend the new fill be moisture conditioned approximately 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.

4.1.6 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.

4.1.7 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 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.

We recommend 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.

4.1.8 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.

4.1.9 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.

4.1.10 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[13]. Groundwater was encountered in our borings at depths as shallow as 10-1/2 feet deep.

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

[13]U.S. Department of Agriculture, Natural Resources Conservation Service, *National Engineering Handbook Part 630, Chapter 7, Hydrologic Soil Groups*, updated January 2009.

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

4.1.11 Setbacks

We recommend the residences be setback from the site and property boundaries in accordance with the California Building Code and local ordinances. Where driveways or improvements are located adjacent to cut, fill or native slopes, we recommend the driveway pavement section or improvements be setback at least 5 feet from the top or toe of the adjacent slope for slopes less than 10 feet high, and setback at least 10 feet where the slopes are greater than 10 feet in height. If setting back pavements or improvements is not feasible, we recommend SFB be consulted for alternative recommendations.

4.1.12 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.

4.1.13 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 and HOA's.

4.2 Foundation Support

4.2.1 Post-Tensioned Slabs

The proposed residential buildings can be supported on post-tensioned slab foundations that are designed for the expansion potential of the onsite soils. The slab foundation should bear entirely on properly prepared, compacted structural fill. In no case should a slab foundation bear upon fills with differential expansion characteristics. Recommendations for building pad preparation are described previously in **Sections 4.1.2, *Soil Re-Compaction***, and **Sections 4.1.3, *Subgrade Preparation.*** Prior to the concrete pour, we recommend the moisture content of the subgrade materials be approximately 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. We recommend the foundation designer determine if corrosion protection is needed for the foundation concrete and reinforcing steel. The results of corrosion testing on one onsite soil sample are included in Appendix B; the foundation designer should determine if additional testing is needed. In addition, 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[14], PTI standard requirements[15], and a PTI preferred computer program, VOLFLO (Version 1.5 Build

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[14]Post-Tensioning Institute, 2008, *Design of Post-Tensioned Slabs-On-Ground (PTI DC10.1-08),* Third Edition.

[15]Post-Tensioning Institute, 2012, *Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soils (PTI DC10.5-12).*

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 4.1.9, *Surface Drainage, Irrigation, and Landscaping***, for additional recommendations. We recommend the slab-subgrade friction values provided in the most recent PTI Manual be used in order to determine the friction that might be expected to exist during tendon stressing.

SWELLING MODE

	Center Lift	Edge Lift
Edge Moisture Variation Distance (e_m)	9.0 feet	5.0 feet
Differential Soil Movement (y_m)	0.5 inch	1.5 inch

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 (MSE walls) 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 1.5H 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 10H 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 65 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 4.1.4,** ***Fill Material***, and **Section 4.1.5,** ***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 three 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[16], and were based on the site being located at approximate latitude 38.465°N and longitude 122.671°W. For seismic design using the 2016 California Building Code (CBC), we recommend the following tabulated seismic design values be used.

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[16]USGS Website, *http://earthquake.usgs.gov/hazards/designmaps/usdesign.php*, last updated 6/23/14.

2016 CBC SEISMIC PARAMETERS		
Seismic Parameter	Design Value	CBC Reference
Site Class	D	Section 1613.3.2
S_S	2.098	Figure 1613.3.1(1)
S_1	0.861	Figure 1613.3.1(2)
F_a	1.0	Table 1613.3.3(1)
F_v	1.5	Table 1613.3.3(2)

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 additional R-value tests be performed once the pavement subgrade is established to confirm the R-value used in the design. Pavement subgrade completely 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.

PRELIMINARY PAVEMENT DESIGN ALTERNATIVES SUBGRADE R-VALUE = 5			
Location	Pavement Components		Total Thickness (inches)
	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	
T.I. = 4.5 (auto & light truck parking)	3.0	9.0	12.0
T.I. = 5.0 (access ways/courts)	3.0	11.0	14.0
T.I. = 6.0 (primary roadways)	3.0	14.0	17.0

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 Warmington Residential and their consultants for specific application to the proposed Acacia Village residential development at 746 Acacia Lane in Santa Rosa, California, and is intended to represent our design recommendations to Warmington Residential for specific application to the Acacia Village project. The conclusions and recommendations contained in this report are solely professional opinions. It is the responsibility of Warmington Residential 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 Warmington Residential and their consultants during the course of this engagement and our rendering of professional services to Warmington Residential. 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 Warmington Residential to divulge information that may have been communicated to Warmington Residential. 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.

FIGURE

KEY

SFB-3 — APPROXIMATE LOCATION OF SFB EXPLORATORY BORING (9/27/17)

APPROXIMATE PROJECT LIMIT

N

APPROXIMATE SCALE: 1" = 80'

0 80' 160'

NOTE: Base map was created by overlaying the preliminary master site and planning plan Sheet L1.0 prepared by Firma Design Group and dated 12/20/16 on Google Earth image dated 6/16/17.

DATE	Stevens Ferrone & Bailey Engineering Company, Inc	1600 Willow Pass Court Concord, CA 94520 Tel 925.688.1001 Fax 925.688.1005 www.SFandB.com	SITE PLAN	FIGURE
October 2017			ACACIA VILLAGE Santa Rosa, California	1
PROJECT NO.				
552-6				

APPENDIX A
Field Investigation

APPENDIX A

Field Investigation

Our field investigation for the proposed Acacia Village residential development to be located at 746 Acacia Lane in Santa Rosa, California, consisted of surface reconnaissance and a subsurface exploration program. Geotechnical reconnaissance of the site and surrounding area was performed on September 27, 2017. 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 September 27, 2017. 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 boring 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 blow counts, 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

Major Divisions		grf	ltr	Description
Coarse Grained Soils	Gravel Gravelly Soils		GW	Well-graded gravels or gravel sand mixtures, little or no fines
			GP	Poorly-graded gravels or gravel sand mixture, little or no fines
			GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sand And Sandy Soils		SW	Well-graded sands or gravelly sands, little or no fines
			SP	Poorly-graded sands or gravelly sands, little or no fines
			SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, and-clay mixtures

Major Divisions		grf	ltr	Description
Soils	Silts And Clays LL < 50		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silt-clays of low plasticity
	Silts And Clays LL > 50		MH	Inorganic silts, micaceous or diatomaceous fine or silty soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
Highly Organic Soils			PT	Peat and other highly organic soils

GRAIN SIZES

U.S. STANDARD SERIES SIEVE: 200, 40, 10, 4
CLEAR SQUARE SIEVE OPENINGS: 3/4", 3", 12"

Silts and Clays	Sand			Gravel		Cobbles	Boulders
	Fine	Medium	Coarse	Fine	Coarse		

RELATIVE DENSITY

Sands and Gravels	Blows/Foot*
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	Over 50

CONSISTENCY

Silts and Clays	Blows/Foot*	Strength (tsf)**
Very Soft	0 - 2	0 - 1/4
Soft	2 - 4	1/4 - 1/2
Firm	4 - 8	1/2 - 1
Stiff	8 - 16	1 - 2
Very Stiff	16 - 32	2 - 4
Hard	Over 32	Over 4

*Number of Blows for a 140-pound hammer falling 30 inches, driving a 2-inch O.D. (1-3/8" I.D.) split spoon sampler.
**Unconfined compressive strength.

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

PI = Plasticity Index
LL = Liquid Limit
R = R-Value

Increasing Visual Moisture Content

Saturated
Wet
Moist
Damp
Dry

Constituent Percentage

trace	<5%
some	5-15%
with	16-30%
-y	31-49%

KEY 552-6.GPJ STEVENS FERRONE BAILEY.GDT 10/5/17

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KEY TO EXPLORATORY BORING LOGS

ACACIA VILLAGE
Santa Rosa, California

PROJECT NO.	DATE	FIGURE NO.
552-6	October 2017	A-1

DRILL RIG Mobile B-24 CFA	SURFACE ELEVATION ---	LOGGED BY HP
DEPTH TO GROUND WATER 12.5 feet	BORING DIAMETER 4-inch	DATE DRILLED 09/27/17

DESCRIPTION AND CLASSIFICATION: DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UNC. COMP. (KSF)	OTHER TESTS
CLAY (CH), dark brown, with silt, with sand(fine- to coarse-grained), trace gravel(fine, subangular to subrounded), with carbonates, dry.	very stiff - hard		0		23	17	109	20.2	
CLAY (CL), mottled light brown, silty, some sand(fine- to coarse-grained), trace carbonates, dry.	hard				33				
			5		33	14	107	11.9	
Change color to brown, trace gravel(fine, subangular to subrounded), with sand clasts.									
CLAY (CL), olive brown, silty, some sand(fine-grained), trace gravel(fine, subangular to subrounded), dry to damp.	hard								
			10		54/9"	20	106	2.8	
Sandy gravel lense at 11 feet.									
			15		30				
Change color to light brown with black mottles, sandy(fine- to medium-grained), damp.	very stiff								
			20		36				
Sandy lense at 20 feet.	hard								
			25		30				
	very stiff								
			30		36				
Change color to gray brown, damp to moist.	hard								
Bottom of Boring = 31.5 feet Notes: Stratification is approximate, variations must be expected. Blowcounts converted to SPT N-values. See Report for additional details.									

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EXPLORATORY BORING LOG

ACACIA VILLAGE
Santa Rosa, California

PROJECT NO.	DATE	BORING NO.
552-6	**October 2017**	**SFB-1**

DRILL RIG Mobile B-24 CFA	SURFACE ELEVATION ---	LOGGED BY HP
DEPTH TO GROUND WATER 10.5 feet	BORING DIAMETER 4-inch	DATE DRILLED 09/27/17

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UNC. COMP. (KSF)	OTHER TESTS
CLAY (CH), dark brown, with silt, with sand(fine- to coarse-grained), trace gravel(fine, subangular to subrounded), with carbonates, dry.	hard		0		32	19	104	14.2	At 2' : Liquid Limit = 56, Plasticity Index = 38, Fine Gravel = 2%, Coarse Sand = 5%, Medium Sand = 8%, Fine Sand = 16%, Silt = 26%, Clay = 43%
					35				
			5		22				
CLAY (CL), light brown, silty, sandy(fine- to medium-grained), trace gravel(fine, subangular to subrounded), dry to damp.	very stiff								
SAND (SC), brown, fine- to coarse-grained, some gravel(fine, subangular to subrounded), with clay and silt, damp to moist.	very dense		10		50/6"				
Moist to wet.	medium dense		15		22				At 16' : Fine Gravel = 8%, Coarse Sand = 16%, Medium Sand = 31%, Fine Sand = 26%, Fines = 19%
	very dense		20		67				
			25		48				
CLAY (CL), mottled light olive brown, silty, with sand(fine- to medium-grained), trace gravel(fine, subangular to subrounded), damp to moist.	hard								
Bottom of Boring = 26.5 feet Notes: Stratification is approximate, variations must be expected. Blowcounts converted to SPT N-values. See Report for additional details.			30						

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EXPLORATORY BORING LOG

ACACIA VILLAGE
Santa Rosa, California

PROJECT NO.	DATE	BORING NO.
552-6	October 2017	SFB-2

DRILL RIG Mobile B-24 CFA	SURFACE ELEVATION ---	LOGGED BY HP
DEPTH TO GROUND WATER 14 feet	BORING DIAMETER 4-inch	DATE DRILLED 09/27/17

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UNC. COMP. (KSF)	OTHER TESTS
CLAY (CL), motttled dark brown, with sand, sandy(fine- to coarse-grained), trace carbonates, dry.	stiff to very stiff		0		16	19			At 2' : Liquid Limit = 43 Plasticity Index = 28 Coarse Sand = 3% Medium Sand = 6% Fine Sand = 25% Silt = 26% Clay = 40%
CLAY (CL), brown with black mottles, silty, some sand(fine- to medium-grained), trace gravel(fine, subangular to subrounded), dry to damp. Change color to light brown, sandy(fine- to medium-grained).	hard		5		33 60				
GRAVEL (GC), brown, fine to coarse, subangular to subrounded, sandy(fine- to coarse-grained), with clay and silt, damp.	very dense		10		60				
CLAY (CL), mottled black olive light brown, silty, sandy(fine-grained), damp.	very stiff		15		26				
Bottom of Boring = 16.5 feet Notes: Stratification is approximate, variations must be expected. Blowcounts converted to SPT N-values. See Report for additional details.			20 25 30						

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EXPLORATORY BORING LOG

ACACIA VILLAGE
Santa Rosa, California

PROJECT NO.	DATE	BORING NO.
552-6	October 2017	SFB-3

APPENDIX B

Laboratory Investigation

APPENDIX B

Laboratory Investigation

Our laboratory testing program for the proposed Acacia Village residential development to be located at 746 Acacia Lane in Santa Rosa, California was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site.

The natural water contents was determined on five 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 five 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 sample 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 depth and are also attached to this appendix.

Gradation and hydrometer tests were performed on tw0 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 log at the appropriate sample depth 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.

Two onsite soil sample 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 are included under separate cover. We recommend these test results be forwarded to your underground contractors, pipeline designers, and foundation designers and contractors.

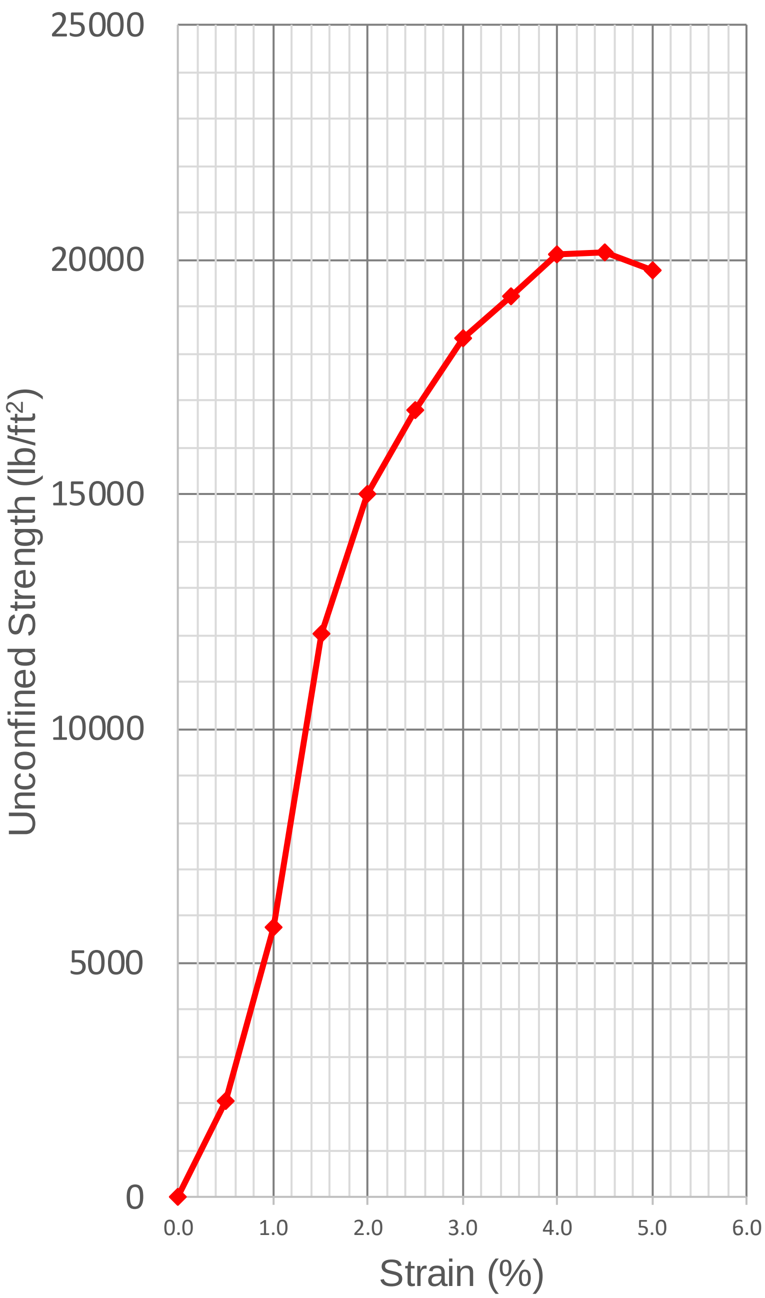
.

UNCONFINED COMPRESSIVE STRENGTH – D2166

Project Number: 552-6 **Boring #:** B-1 **Depth:** 2

Project Name: Acacia Village **Date:** 9/28/2017

Description: Brown sandy CLAY some gravel (CL/SC) **Tested By:** R



Soil Specimen Initial Measurements	
Diameter	2.42 in
Initial Area	4.60 in^2
Initial Length	5 in
Volume	0.01331 ft^3
Water Content	16.5
Wet Density	126.3 pcf
Dry Density	108.5 pcf

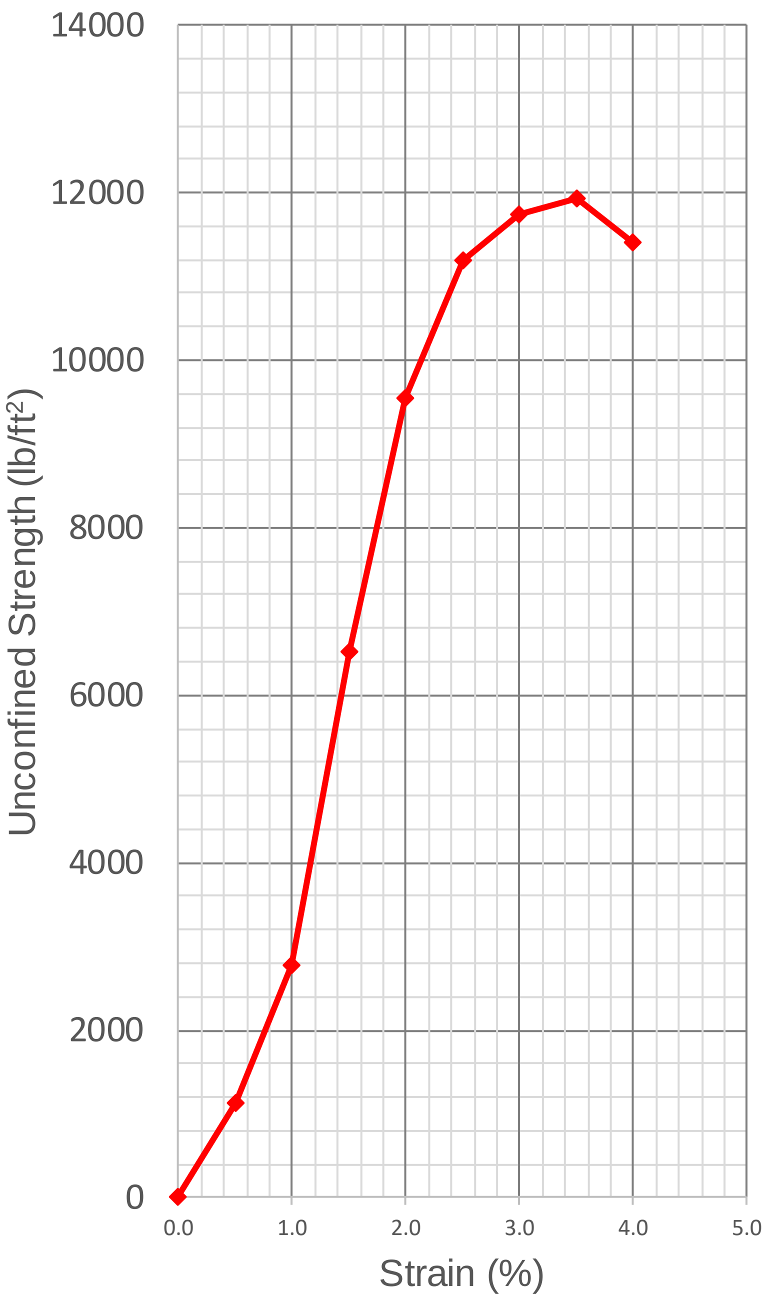
Max Unconfined Compressive Strength	
Elapsed Time	4.5 min
Vertical Dial	0.225 in
Strain	4.5 %
Area	0.03345 ft^2
Axial Load	674.7 lbs
Compressive Strength	20,171 psf

UNCONFINED COMPRESSIVE STRENGTH – D2166

Project Number: 552-6 **Boring #:** B-1 **Depth:** 6

Project Name: Acacia Village **Date:** 9/28/2017

Description: Dark brown silty CLAY some sand (CL) **Tested By:** R



Soil Specimen Initial Measurements	
Diameter	2.42 in
Initial Area	4.60 in^2
Initial Length	5 in
Volume	0.01331 ft^3
Water Content	14.2
Wet Density	121.6 pcf
Dry Density	106.5 pcf

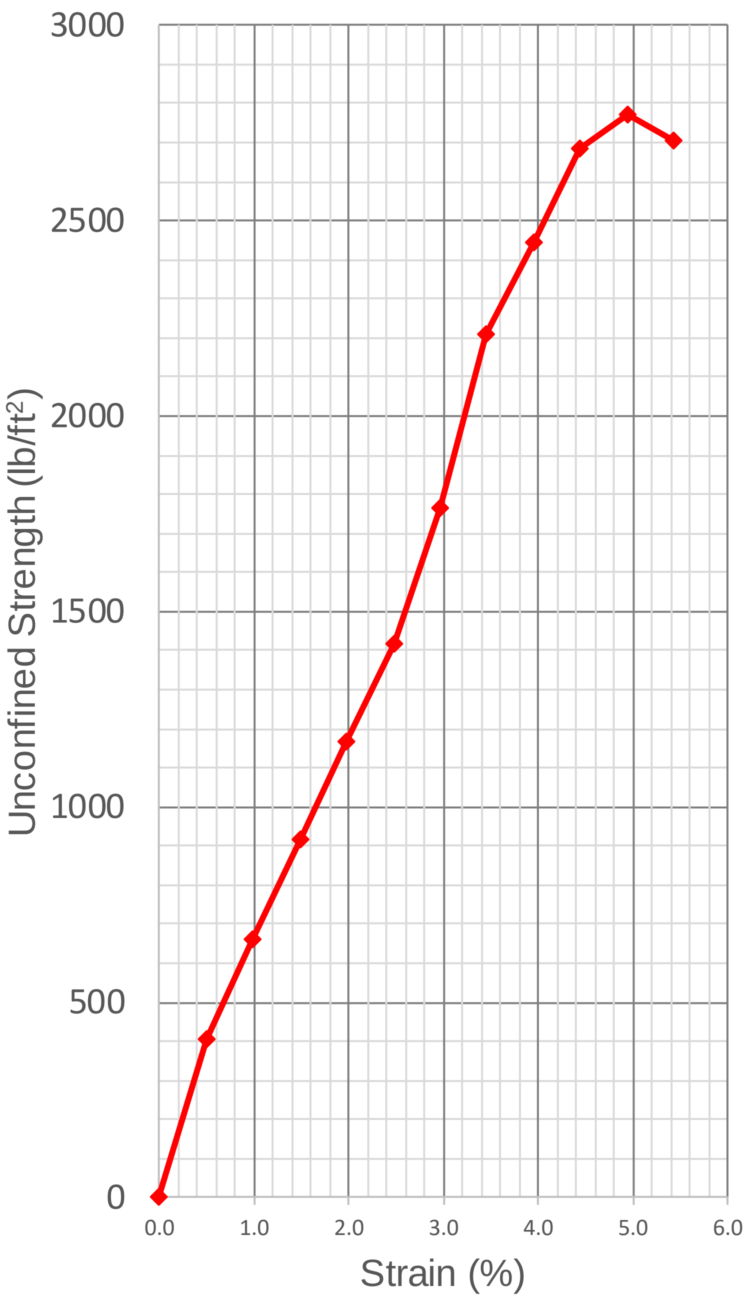
Max Unconfined Compressive Strength	
Elapsed Time	3.5 min
Vertical Dial	0.175 in
Strain	3.5 %
Area	0.03310 ft^2
Axial Load	394.7 lbs
Compressive Strength	11,923 psf

UNCONFINED COMPRESSIVE STRENGTH – D2166

Project Number: 552-6 **Boring #:** B-1 **Depth:** 11

Project Name: Acacia Village **Date:** 9/28/2017

Description: Brown sandy silty CLAY some gravel (CL/SC) **Tested By:** R



Soil Specimen Initial Measurements	
Diameter	2.42 in
Initial Area	4.60 in^2
Initial Length	5.07 in
Volume	0.01350 ft^3
Water Content	19.6
Wet Density	126.8 pcf
Dry Density	106.0 pcf

Max Unconfined Compressive Strength	
Elapsed Time	5 min
Vertical Dial	0.25 in
Strain	4.9 %
Area	0.03360 ft^2
Axial Load	93.0 lbs
Compressive Strength	2,768 psf

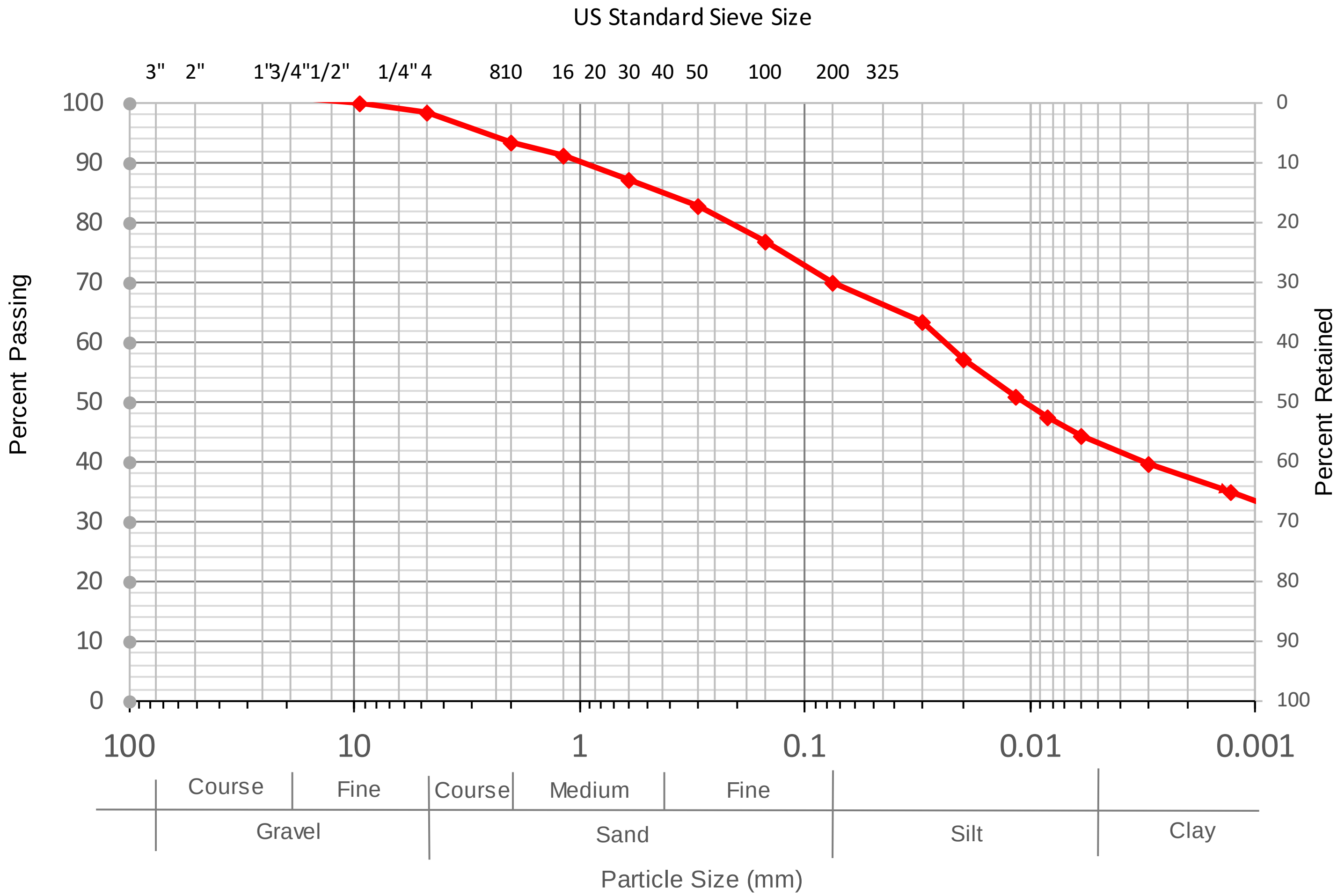
Stevens Ferrone & Bailey Engineering Company, Inc

Hydrometer Analysis – ASTM D422

Project Number: 552-6 **Project Name:** Acacia Village

Sample Number: B-2 **Description:** Dark brown sandy silty CLAY trace gravel (CH)

Depth: 2 **Test Date:** 10-02-17 **Tested By:** R



Composite Sieve Data

Standard Sieve Size	Percent Passing
3”	
1.5”	
3/4”	
3/8”	100
#4	98.4
#10	93.5
#16	91.1
#30	87.3
#50	82.9
#100	77.0
#200	69.9

Particle Diameter (mm)	Percent Soil in Suspension
0.0301	63.5
0.0197	57.1
0.0117	50.8
0.0084	47.6
0.0060	44.4
0.0030	39.7
0.0013	34.9

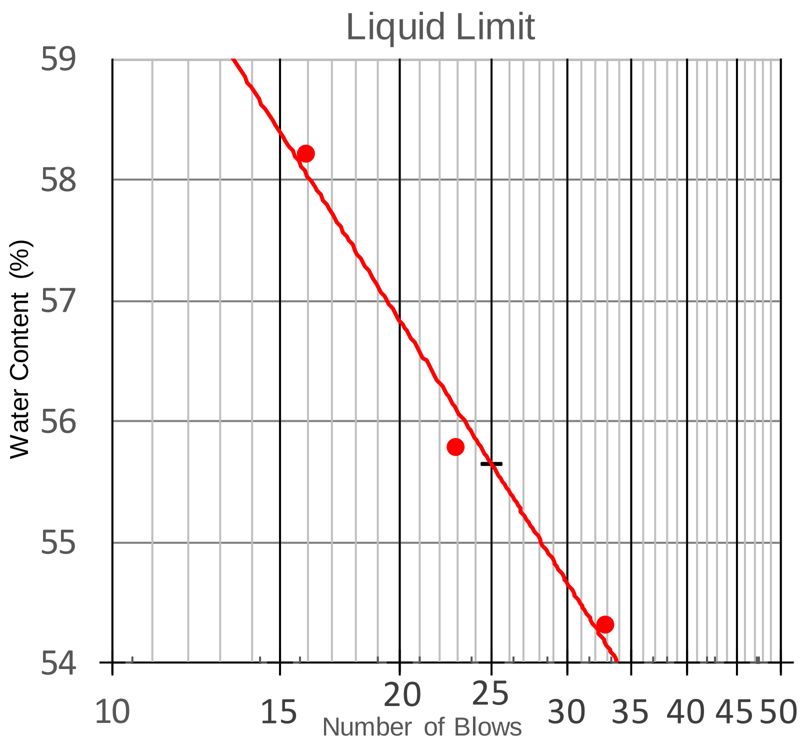
Stevens Ferrone & Bailey Engineering Company, Inc

Atterberg Limits Test – ASTM D4318

Project Number: 552-6 **Project Name:** Acacia Village

Boring/Sample No: B-2 **Depth:** 2 **Date:** 09-29-17

Description of Sample: Dark brown sandy silty CLAY trace gravel (CH) **Tested By** R

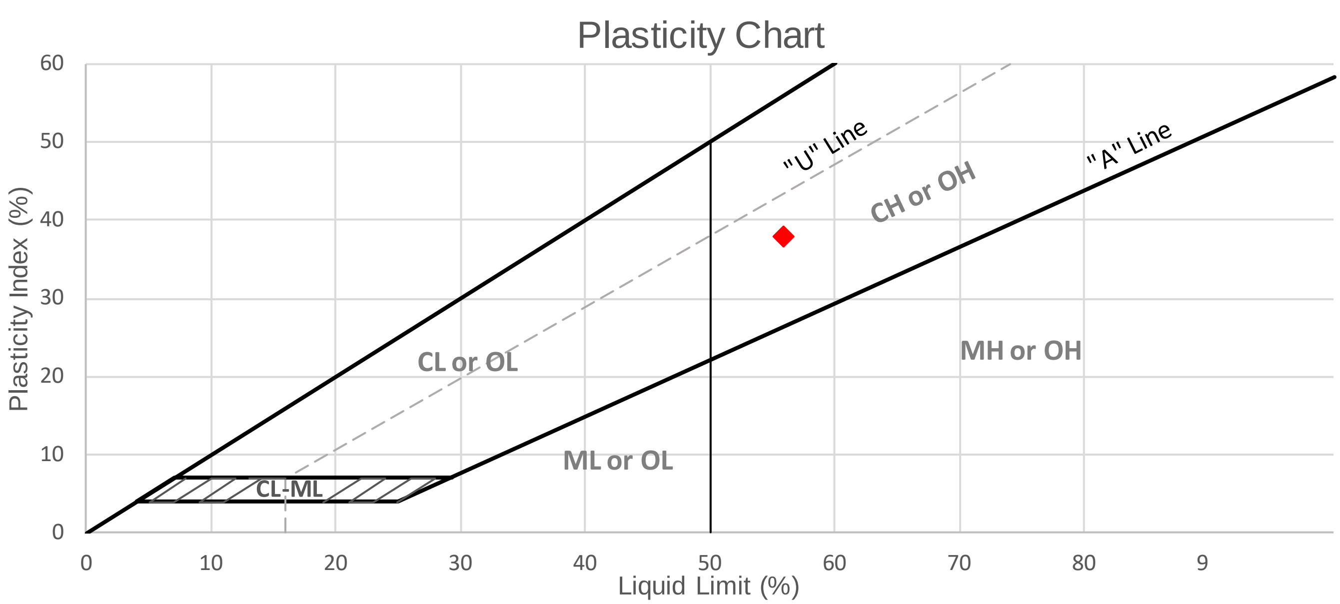


Plastic Limit Data

Trial	1	2	Ave
Water Content (%)	18.4	18.2	18

Data Summary

Liquid Limit	**56**
Plastic Limit	**18**
Plasticity Index	**38**
Natural Water Content	**19.3**
Liquidity Index	**0.034**
% Passing #200 Sieve	**69.9**

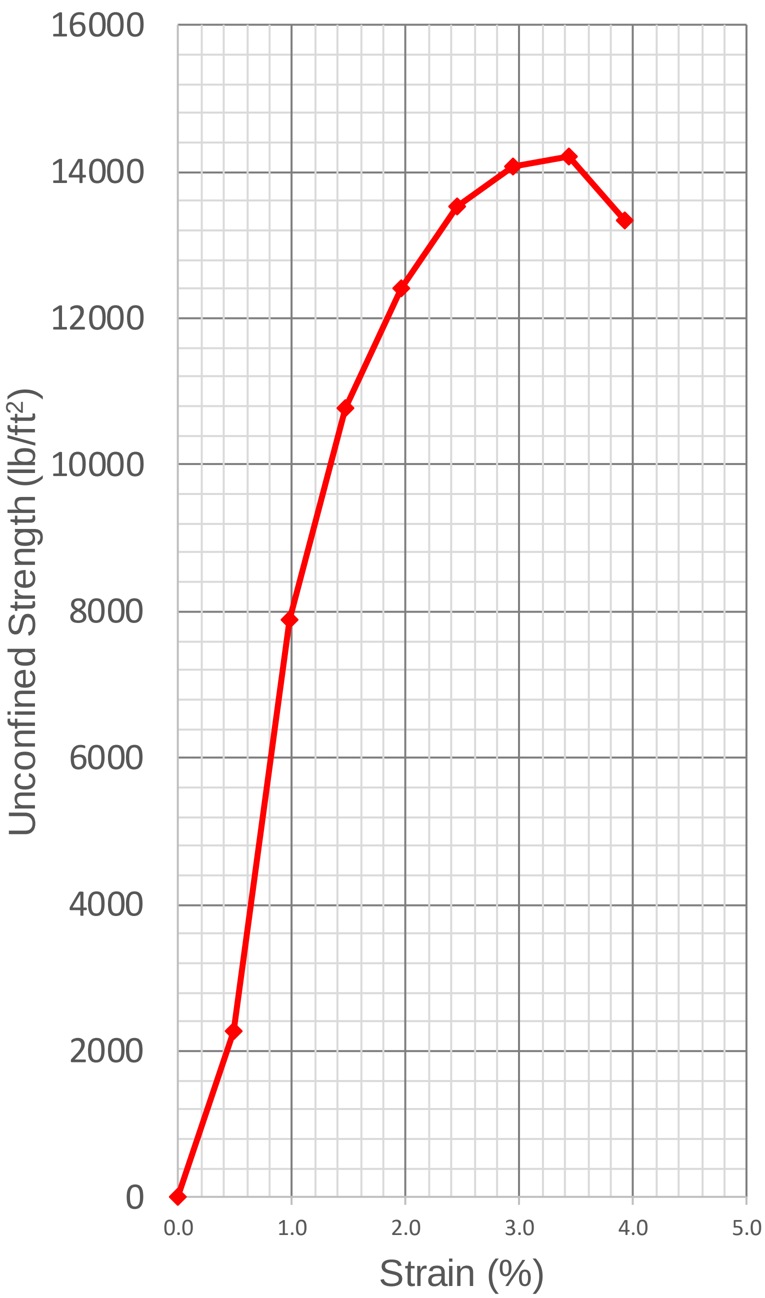


UNCONFINED COMPRESSIVE STRENGTH – D2166

Project Number: 552-6 **Boring #:** B-2 **Depth:** 2

Project Name: Acacia Village **Date:** 9/28/2017

Description: Dark brown sandy silty CLAY some gravel (CH) **Tested By:** R



Soil Specimen Initial Measurements	
Diameter	2.42 in
Initial Area	4.60 in^2
Initial Length	5.1 in
Volume	0.01358 ft^3
Water Content	19.3
Wet Density	124.1 pcf
Dry Density	104.0 pcf

Max Unconfined Compressive Strength	
Elapsed Time	3.5 min
Vertical Dial	0.175 in
Strain	3.4 %
Area	0.03308 ft^2
Axial Load	469.7 lbs
Compressive Strength	14,199 psf

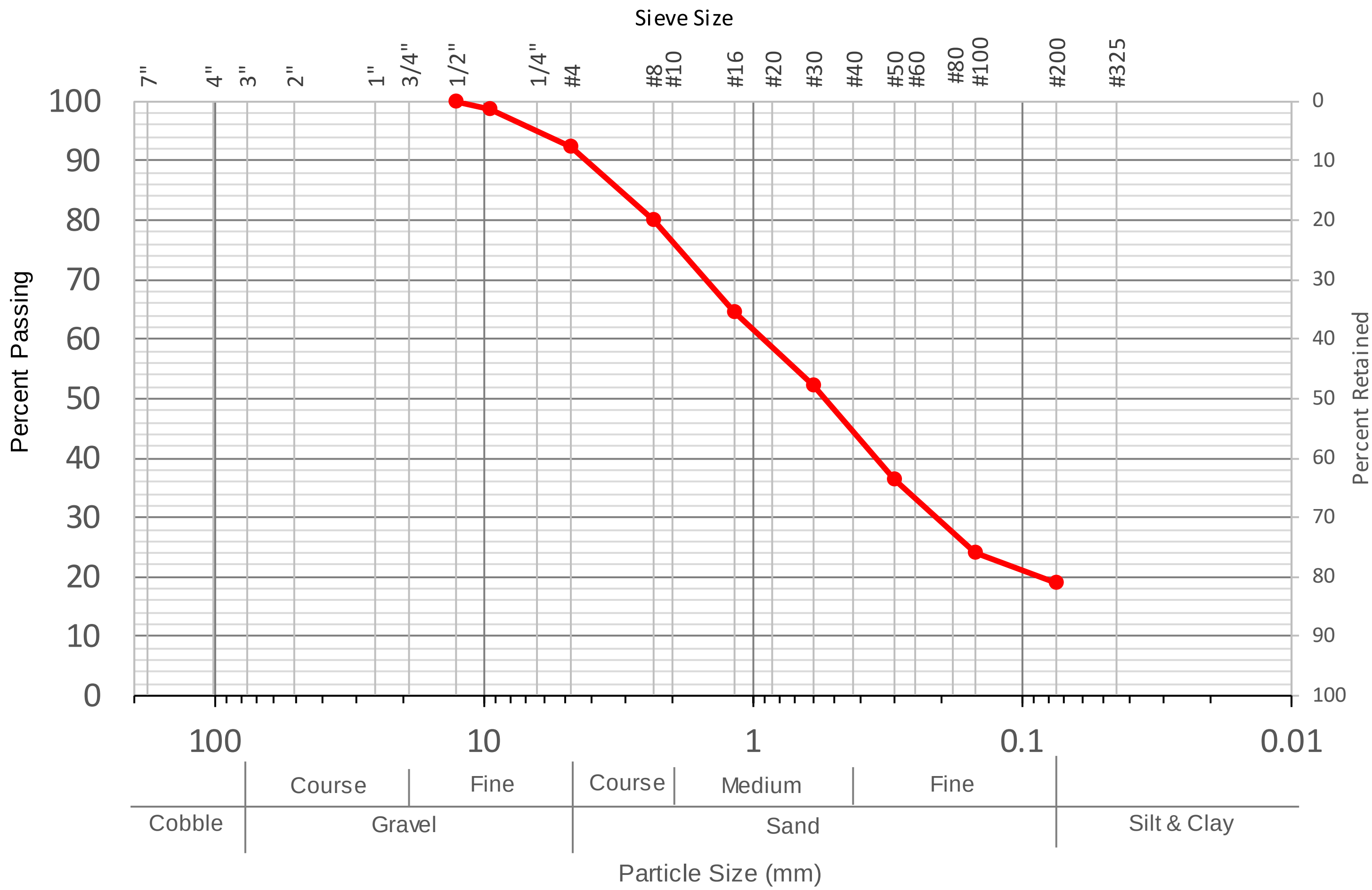
Stevens Ferrone & Bailey Engineering Company, Inc

Sieve Analysis – ASTM C136

Project Number: 552-6 **Project Name:** Acacia Village **Sampling Date:** 9/28/2017

Sample Number: B-2 @ 16' **Description:** Brown SAND with clay some gravel (SC) **Test Date:** 9/29/2017

Sampled By: HP **Source:** Onsite **Tested By:** R



Composite Sieve Data		
Standard Sieve Size	Percent Passing	Specs [1]
3"		
2.5"		
2"		
1.5"		
1"		
3/4"		
1/2"	100.0	
3/8"	98.7	
#4	92.5	
#8	79.9	
#16	64.6	
#30	52.1	
#50	36.3	
#100	24.1	
#200	19.1	

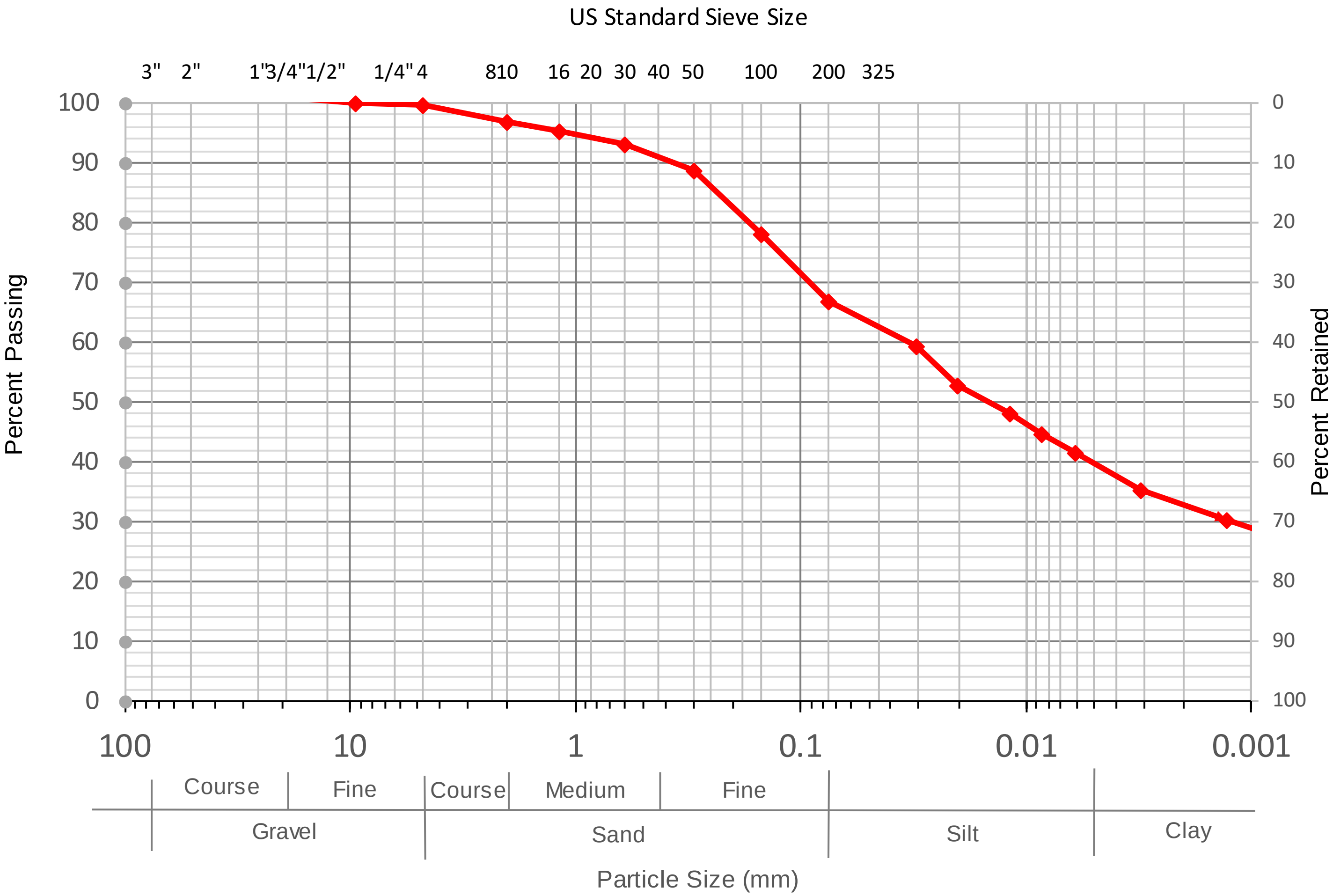
Stevens Ferrone & Bailey Engineering Company, Inc

Hydrometer Analysis – ASTM D422

Project Number: 552-6 **Project Name:** Acacia Village

Sample Number: B-3 **Description:** Dark brown sandy silty CLAY (CL)

Depth: 2 **Test Date:** 10-02-17 **Tested By:** R



Composite Sieve Data

Standard Sieve Size	Percent Passing
3”	
1.5”	
3/4”	
3/8”	100
#4	99.7
#10	96.8
#16	95.2
#30	93.0
#50	88.6
#100	78.2
#200	66.8

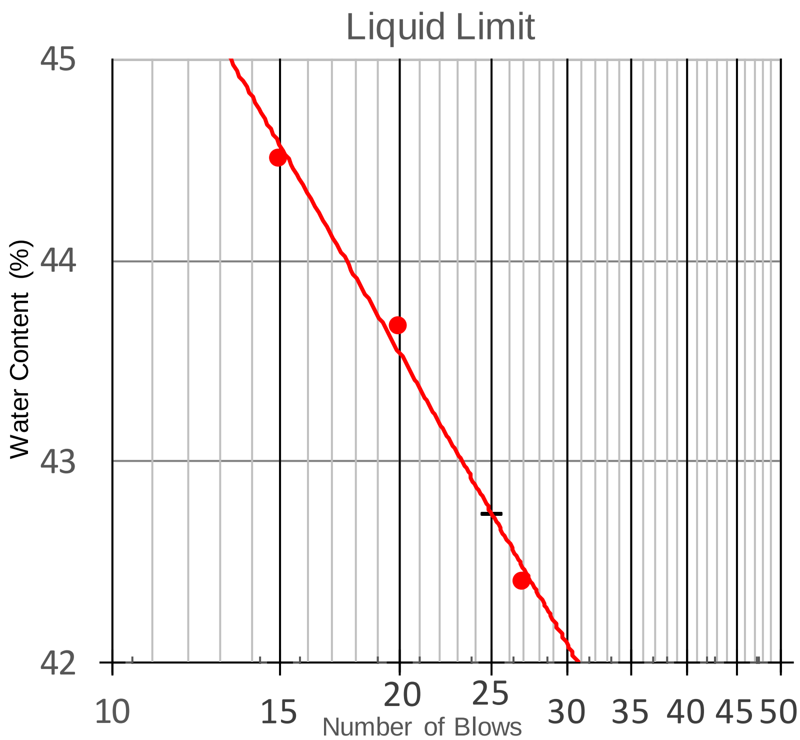
Particle Diameter (mm)	Percent Soil in Suspension
0.0308	59.2
0.0202	52.8
0.0119	48.0
0.0085	44.8
0.0061	41.6
0.0031	35.2
0.0013	30.4

Atterberg Limits Test – ASTM D4318

Project Number: 552-6 **Project Name:** Acacia Village

Boring/Sample No: B-3 **Depth:** 2 **Date:** 09-29-17

Description of Sample: Dark brown sandy silty CLAY (CL) **Tested By** R

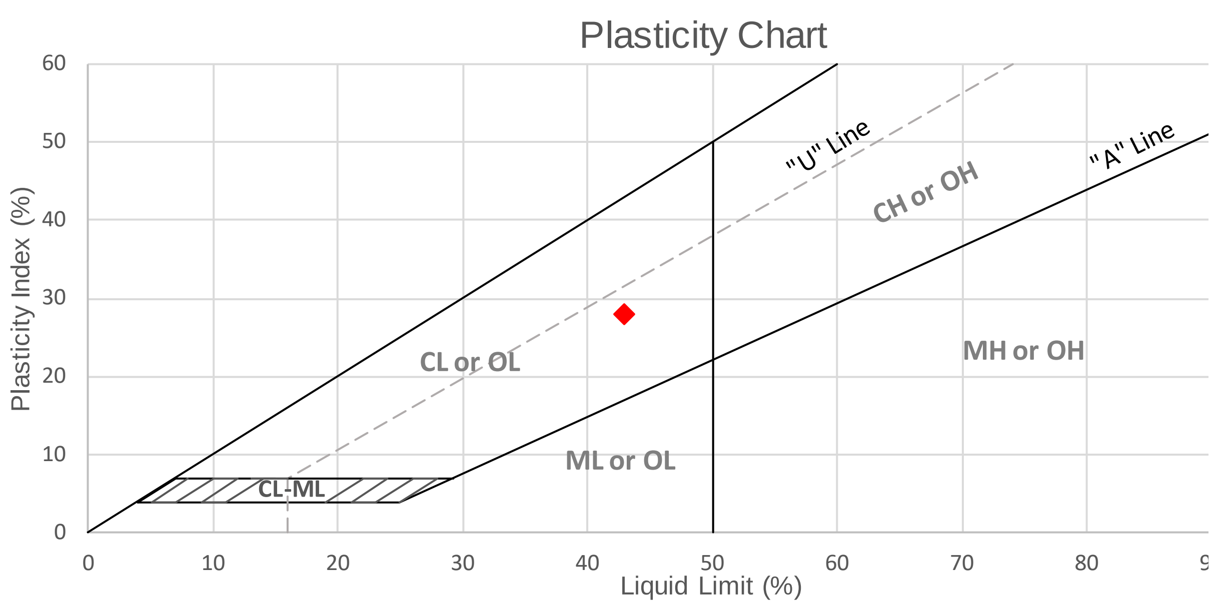


Plastic Limit Data

Trial	1	2	Ave
Water Content (%)	15.0	15.3	15

Data Summary

Liquid Limit	**43**
Plastic Limit	**15**
Plasticity Index	**28**
Natural Water Content	**19.1**
Liquidity Index	**0.146**
% Passing #200 Sieve	**66.8**



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. *Those 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.

ASFE THE GEOPROFESSIONAL BUSINESS ASSOCIATION

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