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Engineering Geology
Storm Water Management
Construction Observation & Testing Services

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GEOTECHNICAL INVESTIGATION
1710 MOORPARK AVENUE
SAN JOSE, CALIFORNIA
SFB PROJECT NO. 674-6

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1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed residential development to be located at 1710 Moorpark Avenue in San Jose, 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 Ms. Helen Tong-Ishikawa of MidPen Housing Corporation, we understand that the project will consist of demolishing several existing buildings and an existing parking lot, and constructing a new building with 4-stories over a podium level. The new building will be supported on either a mat slab or footing foundation. Nominal grading is anticipated. Associated underground utilities and roadways will be constructed.

The preliminary 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- Reviewing published and unpublished geotechnical and geological literature relevant to the site;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 a reconnaissance of the site and surrounding area;
- Reviewing our previous feasibility report for the project dated January 3, 2019;
- Performing a supplemental subsurface exploration program, including drilling two exploratory borings to a maximum depth of about 21-1/2 feet;
- Performing laboratory testing of samples retrieved from the borings;
- Performing geotechnical engineering analysis of the field and laboratory data; and
- Preparing this design-level 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 pavements. Toxicity potential assessment of onsite materials, soils, 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 December 5, 2018 and September 12, 2019. Subsurface exploration was performed on December 13, 2018 and September 25, 2019 and included drilling four exploratory borings to a maximum depth of about 40 feet using a truck-mounted Mobile B-24 drill rig equipped with 4-1/2 inch diameter, continuous flight, solid stem augers. The approximate locations of the four borings are shown on the Site Plan, Figure 1. Boring logs and details regarding the 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 Description

At the time of our investigation and as shown on Figure 1, the site was bounded by Moorpark Avenue to the north, Richmond Avenue to the east, Leigh Avenue to the west, and a church sanctuary and chapel to the south. The site had a plan area of approximately 0.7 acres, with dimensions of about 200 feet by 150 feet, and was occupied by several buildings and a parking lot. The existing site grades were relatively level. The parking lot surface was covered with asphalt concrete pavement. Landscaping included grass lawns, shrubs, and small to large diameter trees. Removal of the below ground obstructions will vary in depth across the site.

3.2 Subsurface

Borings SFB-1 and SFB-4 encountered about 5 to 8 inches of asphalt concrete pavement. Below the pavement and in open space areas, we encountered stiff to very stiff silty clays and a layer of loose to medium dense sands to the maximum depth explored of 41-1/2 feet. The loose to medium dense sand layer was generally encountered between depths of 5 to 15 feet. According to the results of laboratory testing, the more surficial clayey materials have a medium plasticity and moderate expansion potential. In addition, the near-surface more clayey soils have variable expansion potential due to a variability in soil density.

Detailed descriptions of soils 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 by pacing and measurements from landmark references, and should be considered accurate only to the degree implied by the method used.

3.3 Groundwater

Groundwater was not encountered in any of the borings. Our borings were backfilled with lean cement grout in accordance with Santa Clara Valley Water District requirements prior to leaving the site. It should be noted that 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 Urban land-Elpaloalto complex (0 to 2 percent slopes) by USDA Web Soil Survey (WSS)[1], and was assigned to Hydrologic Soil Group C by the USDA Natural Resources Conservation Service (NRCS); Urban land-Elpaloalto complex has been categorized as having moderately high capacity to transmit water (K_{sat}, approximately 0.20 to 0.57 inches per hour).

The Group C soil is defined as having a slow infiltration rate when thoroughly wet and consists chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.

3.5 Geology and Seismicity

According to Helley et al, (1994)[2], the site is underlain by Holocene alluvial fan deposits consisting of medium dense to dense gravelly sand or sandy gravel that grades upward to sandy or silty clay.

The project site is 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. The site is not located within an Alquist-Priolo Earthquake Fault Zone as designated by the State of California[3].

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)[4] has stated

[1] http://websoilsurvey.sc.egov.usda.gov/App/HomePage.htm

[2] E.J. Helley, R.W. Graymer. G.A. Phelps, P.K. Showalter, and C.M. Wentworth, May 1994, *Quaternary Geology of Santa Clara Valley, Santa Clara, Alameda, and San Mateo Counties, California: A digital database.* United States Geological Survey, Open-File Report 94-231.

[3] Hart and Bryant, *Fault-Rupture Hazard Zones in California*, CDMG Special Publication 42. Interim Revision 2007.

[4] 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).

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 U.S. Geological Survey's Unified Hazard Tool and applying the Dynamic: Conterminous U.S. 2014 (v4.2.0) model (accessed 10/2/2019), the resulting deaggregation calculations indicate that the site has a 10% probability of exceeding a peak ground acceleration of about 0.56g 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 soils.

3.6 Liquefaction & Lateral Spreading

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 ABAG and the U.S. Geological Survey, site is in an area that has been characterized as having moderate liquefaction susceptibility[5,6]. The site is not located within a seismic hazard zone designated for liquefaction[7]. Our borings did not encounter liquefaction susceptible soils. Based on our review of available literature and the results of field explorations at the site, it is our opinion that the potential for ground surface damage at the site resulting from liquefaction or lateral spreading is low.

[5]Association of Bay Area Governments, 1980, *Liquefaction Susceptibility, San Francisco Bay Region.*
[6]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.
[7]State of California, *San Jose West Quadrangle, Seismic Hazard Zones, Official Map*, February 7, 2002.

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.

VARIABLE EXPANSION POTENTIAL AND REMOVAL OF OBSTRUCTIONS: The existing near-surface soils have a moderate to high potential for expansion and heaving when exposed to water and contracting (shrinking) when drying. In addition, the density of these near-surface more clayey soils vary across the site which results in variable expansion potential. Removal of below grade obstructions during the demolition phase of the project will also result in variable near-surface soil densities.

In order to reduce the potential for post-construction distress to proposed improvements, we recommend the near-surface soils (and fills if encountered during grading) be over-excavated and re-compacted. We recommend all existing fill materials be removed to a depth where competent native soils are encountered. We estimate the process can consist of removing the upper 2 feet of soils/fills, scarifying and re-compacting the bottom 12 inches in-place, and placing moisture-conditioned, compacted engineered fill over the properly prepared subgrade so that the proposed building is underlain by at least 3 feet of moisture conditioned, engineered fill. The compacted, engineered fill layer should extend at least 5 feet beyond the building footprint and at least 3 feet beyond exterior flatwork. 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.

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, along with a brief summary of the results, are included in Appendix B. We recommend these test results and summary be forwarded to your underground contractors, pipeline designers, and foundation designers and contractors. 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 for the life of the project. We also recommend additional testing be performed if the corrosion test results are deemed insufficient by the designers of the corrosion protection measures. Also, landscaping soils typically contain fertilizers and other materials than can be highly corrosive to metals and concrete; landscaping soils commonly are in contact with foundations. Consideration should be given to testing the corrosion potential characteristics of proposed landscaping soils and other types of imported or modified soils and forwarding the results to your corrosion protection designers and installers.

ADDITIONAL RECOMMENDATIONS: Detailed drainage, earthwork, foundation, retaining wall, and pavement recommendations for use in design and construction of the project are presented below. We recommend SFB review the project 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.

It is always the responsibility of all contractors to provide safe working conditions at the site. We recommend all OSHA regulations be followed, and excavation safety be always ensured. It is beyond our scope of work to provide excavation safety designs.

4.1 Earthwork

4.1.1 Clearing and Site Preparation

The site should be cleared of all obstructions including structures and their associated foundation systems, pavements, underground utilities and their associated backfill, any existing wells, fills, trees, shrubs, and grasses and their associated 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***. Wells, if present, should be abandoned in accordance with Valley Water District standards.

From a geotechnical standpoint, any existing fill materials, gravels, trench backfill materials, clay or concrete pipes, pavements, baserock, and concrete that are removed can be used as new fill onsite provided debris is removed, the materials are well mixed, and the materials are broken up to meet the size requirement for fill material in **Section 4.1.4, *Fill Material***. We recommend fill materials composed of broken concrete or asphalt concrete not be located within 3 feet of the ground surface in open space areas. Consideration should be given to placing these materials below pavements, directly under the building footprint, or in deeper excavations. We recommend backfilling operations for any excavations be performed under the observation and testing of SFB.

Portions of the site containing heavy surface vegetation 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 Subgrade Preparation

After completion of clearing and site preparation, 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 3 to 5 percent over optimum water content, and compacted to the requirements for structural fill.

If the building pad 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, the exposed subgrade or pavement subgrade may need to be reconditioned (moisture conditioned and/or scarified and re-compacted) 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.

4.1.3 Building Pad

We recommend the building and surrounding flatwork be underlain by layer at least 3 feet thick of well-mixed, moisture conditioned, and well blended engineered fill. The compacted, engineered fill layer should extend at least 5 feet beyond the building footprint and at least 3 feet beyond exterior flatwork. 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.

4.1.4 Fill Material

From a geotechnical and mechanical standpoint, onsite soils and fills 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 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

Within the upper 5 feet of the finished ground surface, we recommend structural fill be compacted to at least 90 percent relative compaction, and structural fill below a depth of 5 feet be compacted to at least 95 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.

4.1.6 Utility Trench Backfill

Pipeline trenches should be backfilled with fill placed in lifts of approximately 8 inches in un-compacted 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 with sufficient water 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). 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 across 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 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).

Expansive clayey soils at the site will be subjected to volume changes during fluctuations in moisture content. As a result of these volume changes, 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, we recommend the installation of #4 bars spaced at approximately 18 inches on center in both directions. Score joints and expansion joints should be used to control cracking and allow for expansion and contraction of 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; 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, moisture contents of 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 drainage measures recommended in this report should be implemented and maintained during and after construction, especially during wet weather conditions.

If lime treating the subgrade is desired for stabilization purposes, we preliminarily recommend the subgrade be stabilized to a depth of at least 12 inches using quicklime spread at a rate of at least 4 pounds per square foot. After the soil-lime has been mixed and re-mixed, we recommend the treated subgrade be compacted to not less than 92 percent relative compaction at a uniform moisture content of two percent above optimum moisture content (per ASTM D1557). SFB should be consulted at the time of stabilization to confirm if these recommendations are applicable based on the site 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 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 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 located adjacent to improvements, collecting accumulated surface water in subdrain pipes that discharge to appropriate collection facilities. Drainage should be designed and constructed so that the moisture content of 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 appear to be inadequate.

Irrigation should be performed in a uniform, systematic manner as equally as possible on all sides of 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/or treat specified amounts of storm water runoff. 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.

We recommend storm water collection improvements that are designed to detain, retain, and/or treat 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 and distress to other infrastructure improvements (such as pavements, foundations, and walkways) which can occur as a result of volumetric soil/fill changes (heaving and shrinking of the surrounding soil/fill). 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. A subdrain pipe should be used at the base of the infiltration materials to collect accumulated water and transmit the water to an appropriate facility.

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 will be needed to maintain design surface elevations. The soil filter materials, infiltration testing and procedures, and associated compaction requirements should be specified by the Civil Engineer and shown in detail on the grading and improvement plans.

Sidewalls of excavations for earthen swales and basins steeper than 2:1 (horizontal to vertical) will experience downward and lateral movements that can cause significant ground surface movements, including movement of adjacent improvements such as foundations, utilities, pavements, driveways, walkways, and curbs and gutters. 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 for earthen swales and basins, sidewalks be setback at least 3 feet from the top of the slope, creep sensitive improvements (such as roadway curbs) be setback at least 5 feet from the top of the slopes, or the slopes/sidewalls be appropriately restrained using an engineered retaining system, such as deepened curbs and foundations that are designed to resist lateral earth pressures and act as a retaining wall.

SFB should be consulted regarding the use, locations, and design of storm water detention and filtration facilities. We also recommend SFB observe and document the installation of liners, subdrain pipes, and soil filter materials during construction for conformance to the recommendations in this report and the development's plans and specifications.

4.1.11 Future Maintenance

In order to reduce water related issues, we recommend regular maintenance of the site be performed, including maintenance prior to rainstorms. Maintenance should include the re-compaction 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 appear 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 the development owner perform inspections and maintenance of slabs, including infilling significant cracks, providing fillers at slab offsets, and replacing slabs if severely damaged.

4.1.12 Additional Recommendations

We recommend that 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, development owners, and their maintenance associations.

4.2 Foundation Support

4.2.1 General

The podium structure can be supported by either footings with interior slabs-on-grade or a thickened mat slab. We expect total foundation settlements on the order of about 1 inch, with differential settlements of about ½ inch between columns, could occur. We recommend you consult with a structural engineer to determine the best foundation system for the structure.

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 onsite soil samples 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.

4.2.2 Footings and Slabs-on-Grade

The proposed new building can be supported on spreading footing foundations bearing on properly compacted engineered fills or native soils. Prior to the concrete pour, we recommend the subgrade materials be moisture conditioned to approximate 3 to 5 percent above laboratory optimum moisture. If the building pad is 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.

Footings should be at least 12 inches wide and should be founded at least 24 inches below lowest adjacent finished grade. The embedment depth of the exterior/perimeter footings can be measured from the lowest adjacent exterior grade (the ground surface grade that will exist adjacent the exterior of the building). The embedment depth of interior footings can be measured from the top of the interior slab-on-grade. Continuous footings should be designed with steel reinforcing, both top and bottom, to provide structural continuity and permit spanning of local irregularities.

The footings should be designed for an allowable bearing pressure of 2,000 pounds per square foot due to dead loads, 3,000 pounds per square foot due to dead plus live loads, and 4,000 pounds per square foot for all loads, including wind or seismic. These allowable bearing pressures are net values; therefore, the weight of the footing can be neglected for design purposes.

Lateral load resistance can be developed by friction between the footing foundation bottom and the supporting subgrade. A friction coefficient of 0.3 is considered applicable. As an alternative, a passive resistance equal to an equivalent fluid weighing 300 pcf acting against the vertical face of the foundations can be used; however the upper 24 inches should be ignored in the passive resistance design. If foundations are poured neat against the subgrade, the friction and passive resistance can be used in combination.

At least 10 feet of soil cover must be provided between the face of the footings and the face of slopes, as measured horizontally. The portion of the footing located closer than 10 feet from the face of slopes should be ignored in both the vertical and lateral load design.

Where foundations are located adjacent to utility trenches or bio-swales, the foundation bearing surface should bear below an imaginary 1 horizontal to 1 vertical plane extending upward from the bottom edge of the adjacent utility trench or bio-swale. Alternatively, the foundation reinforcing could be increased to span the area defined above assuming no soil support is provided.

Wetting prior to construction of the foundations should close any visible cracks in the bottoms of the footing excavations. We recommend that we observe the footing excavations prior to placing reinforcing steel or concrete to check that footings are founded on appropriate material.

Settlement of spread footing foundations under the proposed building loads is anticipated to be within tolerable limits for the proposed structure.

We recommend that new interior slabs-on-grade be at least 5 inches thick and be supported on at least 12 inches of imported, non-expansive fill. We recommend vehicular interior slabs-on-grade be at least 6 inches thick and that the upper 6 inches of non-expansive fill consist of Caltrans Class 2 aggregate base that is compacted to at least 95 percent relative compaction. The actual thickness of the slabs should be based upon the actual use and loading of the slabs. All slabs should be reinforced with at least #4 bars on 18-inch centers, both ways; however, the actual reinforcing should be provided with the anticipated use and loading of the slab.

We are not waterproofing experts. Any below grade slabs-on-grade should be protected against water intrusion in accordance with the California Building Code. The recommendations provided below are only meant to reduce the potential for water vapor from soil to permeate the interior concrete slabs-on-grade. We recommend a vapor retarder be placed between the bottom of the interior slabs-on-grade and the underlying aggregate base. 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 and the manufacturers requirements, including the requirements that all joints 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.

We recommend that 4 inches of ½ to ¾ inch drain rock be placed below the vapor retarder where interior slabs-on-grade are used (the 4 inches of drain rock can be considered part of the non-expansive layer), except where the vehicular slabs are underlain by the 6 inches of baserock. Prior to placement of the vapor retarder, the subgrade surfaces should be proof-rolled to provide a smooth, unyielding surface for slab support. The edges of the vapor retarder membrane should be draped over the interior side of the footing excavations and at least 12 inches below the pad grade prior to pouring the concrete. We recommend that the interior slabs-on-grade be poured monolithically with the footings except for vehicular slabs. The edges of vehicular slabs should be structurally separated from surrounding foundations; a relatively impermeable and flexible filler should be used in the joint between the vehicular slabs and the footing foundation. If a garage door is used, both the driveway and vehicular slabs should be connected to the perimeter footing below the garage door opening with dowels to reduce the potential for differential movements.

4.2.3 Mat Slab

If a thickened, steel reinforced, mat slab is used to support the structure, we anticipate the slab will be at least 18 inches thick; the actual thickness of the mat slab should be determined by the Structural Engineer. The slab will need to be designed to resist the differential soil movements that will occur due to the expansion and contraction of the near-surface soils that provide support for the slab. The subgrade materials beneath the slabs should be considered to have an effective Plasticity Index of 25 percent and a vertical modulus of subgrade reaction of 14 pounds/cubic inch (pci) based on estimated column spacing of about 30 feet. An allowable dead plus live load bearing pressure of 1,500 pounds per square foot (psf) could be used for the mat slab foundation. An allowable bearing pressure of about 2,000 psf can be used for localized areas of the slab that are supported directly on the subgrade. The slab should be designed in accordance with California Building Code requirements; it is our understanding that a finite element analysis will be performed to design the mat foundation. We recommend a vapor retarder be installed below the mat slab in accordance with our recommendations above in **Section 4.2.2**. We are not waterproofing experts. Any below grade slabs-on-grade should be protected against water intrusion in accordance with the California Building Code.

4.2.4 Retaining Walls

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. Where concrete or masonry walls are used to retain soil, we recommend drained, unrestrained walls (walls free to deflect and disconnected from other structures) be designed to resist an equivalent fluid pressure of 45 pounds per cubic foot. This assumes a level backfill. Drained, restrained walls (walls restrained from deflection) should be designed to resist an equivalent fluid pressure of 45 pounds per cubic foot plus a uniform pressure of 10H pounds per square foot, where H is the height of the wall in feet.

Wherever portions of below-grade garage retaining walls will not be backdrained, the walls should be designed to resist an equivalent fluid pressure of 85 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 2 degrees 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 walls that retain soils and/or fills higher than 6 feet as measured from top of footing to top of backfill, we recommend the walls also be designed to resist a triangular pressure distribution equal to an equivalent fluid pressure of 24 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.

Retaining wall backdrainage 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 can be supported on footing foundations in accordance with the recommendations in Section 4.2.2 of this report.

4.2.5 Seismic Design Criteria

The following parameters were calculated using U.S. Seismic Design Maps program[8], and were based on the site being located at approximate latitude 37.3159°N and longitude 121.923°W and Risk Category II. For seismic design using the 2016 California Building Code (CBC), we recommend the following seismic design parameters be used.

Seismic Parameter	Design Value
Risk Category	II
Site Class	D
S_S	1.5
S_1	0.6
F_a	1.0
F_v	1.5

4.3 Pavement

Based on the results of the exploratory borings and 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

[8]https://seismicmaps.org, accessed 10/03/2019.

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 pavements are planned to be placed prior to or during construction, 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.

This report is a document that has been prepared in accordance with generally accepted geological and geotechnical engineering practices for the exclusive use of MidPen Housing Corporation and its consultants for specific application to the proposed 1710 Moorpark Avenue development project in San Jose, California, and is intended to represent our preliminary recommendations to MidPen Housing Corporation for specific application to the 1710 Moorpark Avenue project.

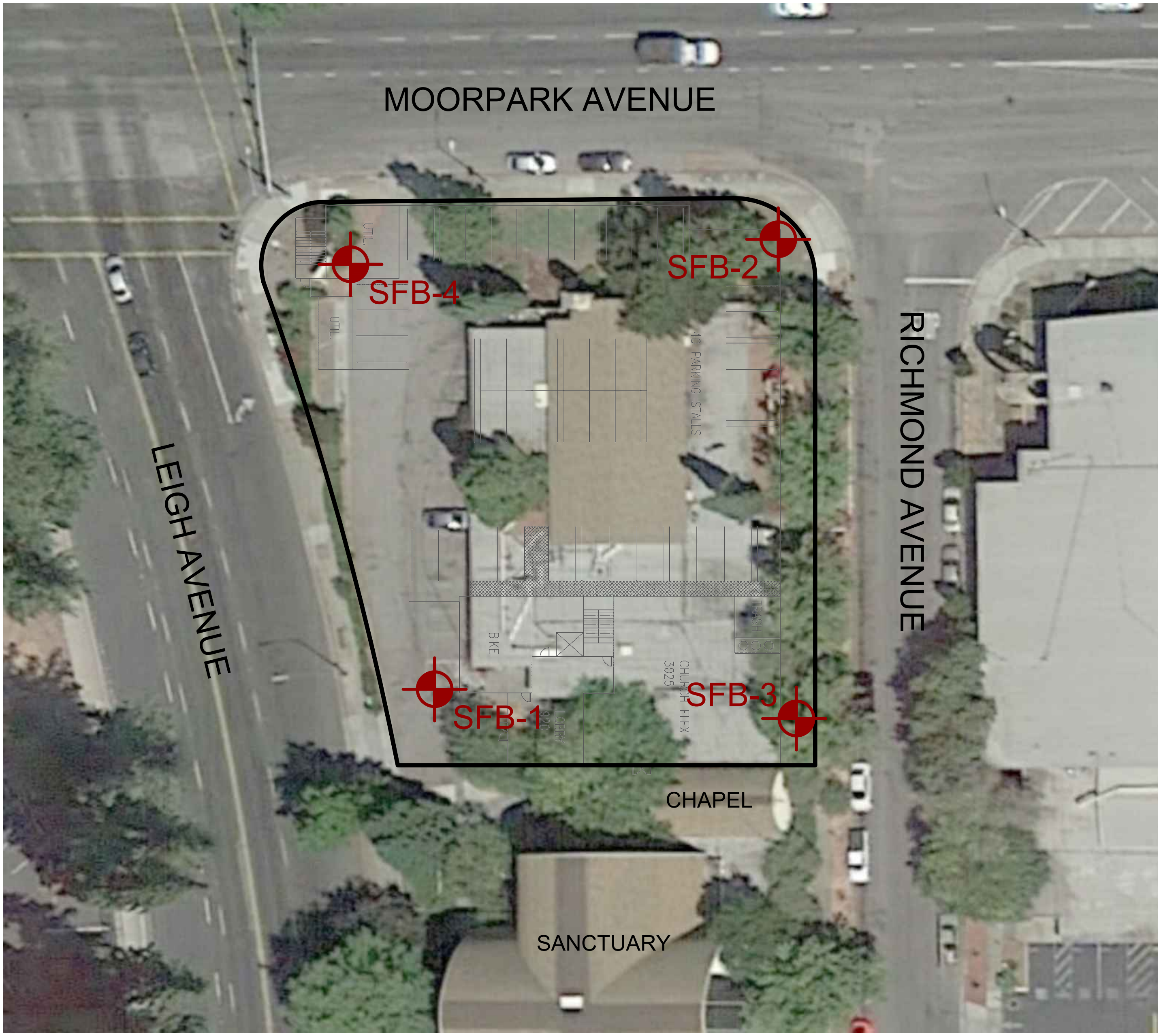
It should be understood that advancements in the practice of geotechnical engineering and engineering geology, or discovery of differing surface or subsurface conditions, may affect the validity of this report and are not uncommon. SFB strives to perform its services in a proper and professional manner 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 the limitations to using this are not completely understood.

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 MidPen Housing Corporation and its consultants during the course of this engagement and our rendering of professional services to MidPen Housing Corporation. 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 MidPen Housing Corporation to divulge information that may have been communicated to MidPen Housing Corporation. 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-4 — APPROXIMATE LOCATION OF SFB EXPLORATORY BORING (12/13/18, 09/25/19)

APPROXIMATE PROJECT LIMIT

N

APPROXIMATE SCALE: 1" = 50'

0 50' 100'

NOTE: Basemap was created by overlaying the project site plan (west facing terrace option) prepared by HKIT Architects and dated 11/29/18 on Google Earth image dated 8/9/18.

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 2019			1710 MOORPARK AVENUE San Jose, California	1
PROJECT NO.				
674-6				

APPENDIX A

Field Investigation

APPENDIX A
Field Investigation

Reconnaissance of the site and surrounding area was performed on December 5, 2018 and September 12, 2019. Subsurface exploration was performed on December 13, 2018 and September 25, 2019 and included drilling four exploratory borings to a maximum depth of about 40 feet using a truck-mounted drill B-24 rig equipped with 4.5-inch diameter, continuous flight, solid stem augers. The approximate locations of the four borings are shown on the Site Plan, Figure 1. 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 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%

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KEY TO EXPLORATORY BORING LOGS

1710 MOORPARK AVENUE
San Jose, California

PROJECT NO.	DATE	FIGURE NO.
674-6	OCTOBER 2019	A-1

DRILL RIG Mobile B-24, CFA	SURFACE ELEVATION ---	LOGGED BY NS
DEPTH TO GROUND WATER Not Encountered	BORING DIAMETER 4.5-inch	DATE DRILLED 12/13/18

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
ASPHALT, 8" thick.			0						
CLAY (CL), brown, silty, some sand(fine- to coarse-grained), damp.	very stiff				21	15	93	7.3	At 2': Liquid Limit = 46% Plasticity Index = 24
					30	11			
			5						
CLAY (CL), brown, sandy(fine- to coarse-grained), silty, damp.	stiff to very stiff				16	8	103	2.4	
With gravel at 8'.									
			10						
SAND (SP), brown, gravelly, trace clay, trace silt, damp.	medium dense				12				At 10.5': Percent Passing #200 Sieve = 4.4%
SAND (SP), brown, some clay, some silt, damp.	medium dense								
			15						
CLAY (CL), light brown, silty, with sand(fine- to coarse-grained), damp.	stiff to very stiff				16				
			20						
	very stiff				18				At 21': Percent Passing #200 Sieve = 66.6%
			25						
					23				

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EXPLORATORY BORING LOG

1710 MOORPARK AVENUE
San Jose, California

PROJECT NO.	DATE	BORING NO.
674-6	**OCTOBER 2019**	**SFB-1**

DRILL RIG Mobile B-24, CFA	SURFACE ELEVATION ---	LOGGED BY NS
DEPTH TO GROUND WATER Not Encountered	BORING DIAMETER 4.5-inch	DATE DRILLED 12/13/18

DESCRIPTION AND CLASSIFICATION			DEPTH (FEET)	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UNC. COMP. (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE							
			30						
SAND (SP), brown, gravelly, damp.	medium dense				29				
			35						
CLAY (CL), brown, sandy(fine- to coarse-grained), silty, damp.	very stiff				22				
			40		19				
Bottom of Boring = 41.5 feet Notes: Stratification is approximate, variations must be expected. Blowcounts converted to SPT N-values. See report for additional details.									
			45						
			50						
			55						

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EXPLORATORY BORING LOG

1710 MOORPARK AVENUE
San Jose, California

PROJECT NO.	DATE	BORING NO.
674-6	**OCTOBER 2019**	**SFB-1**

DRILL RIG Mobile B-24, CFA	SURFACE ELEVATION ---	LOGGED BY NS
DEPTH TO GROUND WATER Not Encountered	BORING DIAMETER 4.5-inch	DATE DRILLED 12/13/18

DESCRIPTION AND CLASSIFICATION			DEPTH (FEET)	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UNC. COMP. (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE							
CLAY (CL), brown, sandy(fine- to coarse-grained), silty, some roots, trace gravel, damp.	hard		0						
Some gravel at 2'.					36	7	104	3.6	
					30	6			
SAND (SC), brown mottled with gray, fine- to coarse-grained, clayey, silty, with gravel, trace roots, damp.	medium dense to dense		5						
SAND (SP), brown mottled with gray, fine- to coarse-grained, gravelly, some clay, some silt, damp.	dense				33				At 5.5': Percent Passing #200 Sieve = 28.3%
SAND (SP), brown, fine- to coarse-grained, gravelly, trace clay, trace silt, damp.	loose		10		6				
			15		20	15			
CLAY (CL), brown, sandy(fine- to coarse-grained), silty, damp.	very stiff		20						
With sand at 21'.					25				
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.			25						

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EXPLORATORY BORING LOG

1710 MOORPARK AVENUE
San Jose, California

PROJECT NO.	DATE	BORING NO.
674-6	**OCTOBER 2019**	**SFB-2**

DRILL RIG Mobile B-24, CFA	SURFACE ELEVATION ---	LOGGED BY NS
DEPTH TO GROUND WATER Not Encountered	BORING DIAMETER 4.5-inch	DATE DRILLED 09/25/19

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 (CL), brown, silty, with sand(fine- to coarse-grained), trace roots, damp. Trace gravel at 2'.	very stiff		0		17	12	89	2.4	At 1.5': Liquid Limit = 35% Plasticity Index = 16
CLAY (CL), brown, silty, with sand(fine- to coarse-grained), trace gravel, damp.	stiff				15	13			At 3': Percent Passing #200 Sieve = 71.8%
Bottom of Boring = 3.5 feet Notes: Stratification is approximate, variations must be expected. Blowcounts converted to SPT N-values. See report for additional details.			5 10 15 20 25						

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EXPLORATORY BORING LOG

1710 MOORPARK AVENUE
San Jose, California

PROJECT NO.	DATE	BORING NO.
674-6	**OCTOBER 2019**	**SFB-3**

DRILL RIG Mobile B-24, CFA	SURFACE ELEVATION ---	LOGGED BY NS
DEPTH TO GROUND WATER Not Encountered	BORING DIAMETER 4.5-inch	DATE DRILLED 09/25/19

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UNC. COMP. (KSF)	OTHER TESTS
ASPHALT, 5" thick.			0						
CLAY (CL), dark brown, silty, some sand(fine- to coarse-grained), gravelly, damp.	stiff				16	14	103	3.3	At 2': Liquid Limit = 40% Plasticity Index = 20
					8				
			5		10	5	101		At 6': Percent Passing #200 Sieve = 7.2%
SAND (SM), dark brown, fine- to coarse-grained, gravelly, some silt, damp.	loose to medium dense								
	loose		10		9				At 11': Percent Passing #200 Sieve = 8.5%
CLAY (CL), brown, sandy(fine- to coarse-grained), silty, damp.	stiff								
			15		13	15	104		At 16': Percent Passing #200 Sieve = 58.3%
			20		16				
CLAY (CL), brown, silty, some sand(fine- to coarse-grained), damp.	stiff to very stiff								
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.									
			25						

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EXPLORATORY BORING LOG

1710 MOORPARK AVENUE
San Jose, California

PROJECT NO.	DATE	BORING NO.
674-6	**OCTOBER 2019**	**SFB-4**

APPENDIX B

Laboratory Investigation

APPENDIX B
Laboratory Investigation

Our laboratory testing program for the proposed residential development to be located at 1710 Moorpark Avenue in San Jose, 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 eleven 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 seven samples of the subsurface soils to evaluate their physical properties. The results of this test are shown on the boring log at the appropriate sample depths.

Atterberg Limit determinations were performed on three 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 test are presented on the boring log at the appropriate sample depth and are also attached to this appendix.

The percent passing the #200 sieve was determined on seven samples of the subsurface soils to aid in the classification of these soils. The results of the tests are shown on the boring logs at the appropriate sample depth.

Unconfined compression testing was performed on five 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 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 and a brief summary of the results are included in this appendix. We recommend these test results and summary be forwarded to your underground contractors, pipeline designers, and foundation designers and contractors.

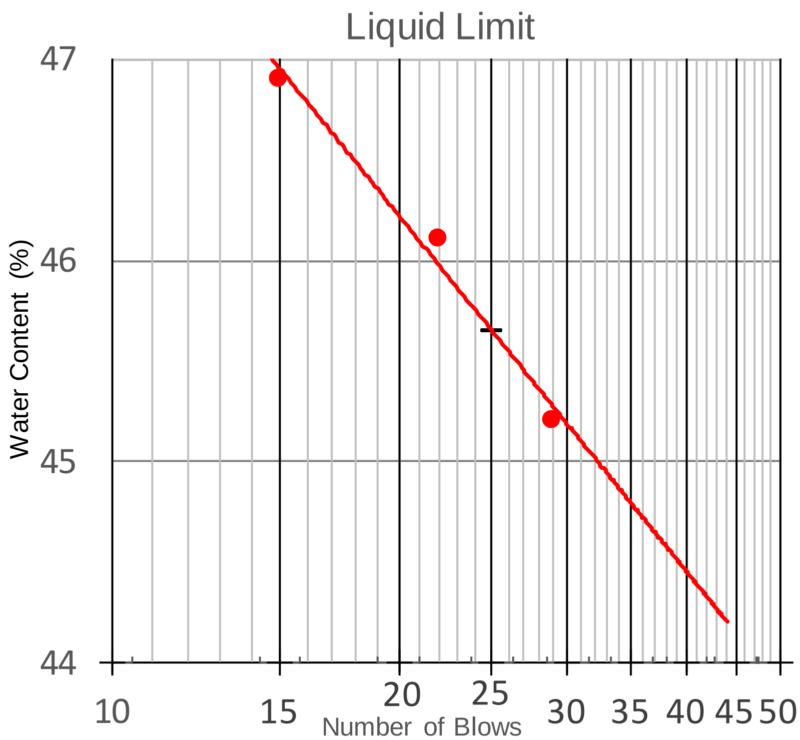
Stevens Ferrone & Bailey Engineering Company, Inc

Atterberg Limits Test – ASTM D4318

Project Number: 674-6 **Project Name:** 1710 Moorpark

Boring/Sample No: B-1 **Depth:** 2 **Date:** 12-20-18

Description of Sample: Brown silty CLAY some sand (CL) **Tested By** R

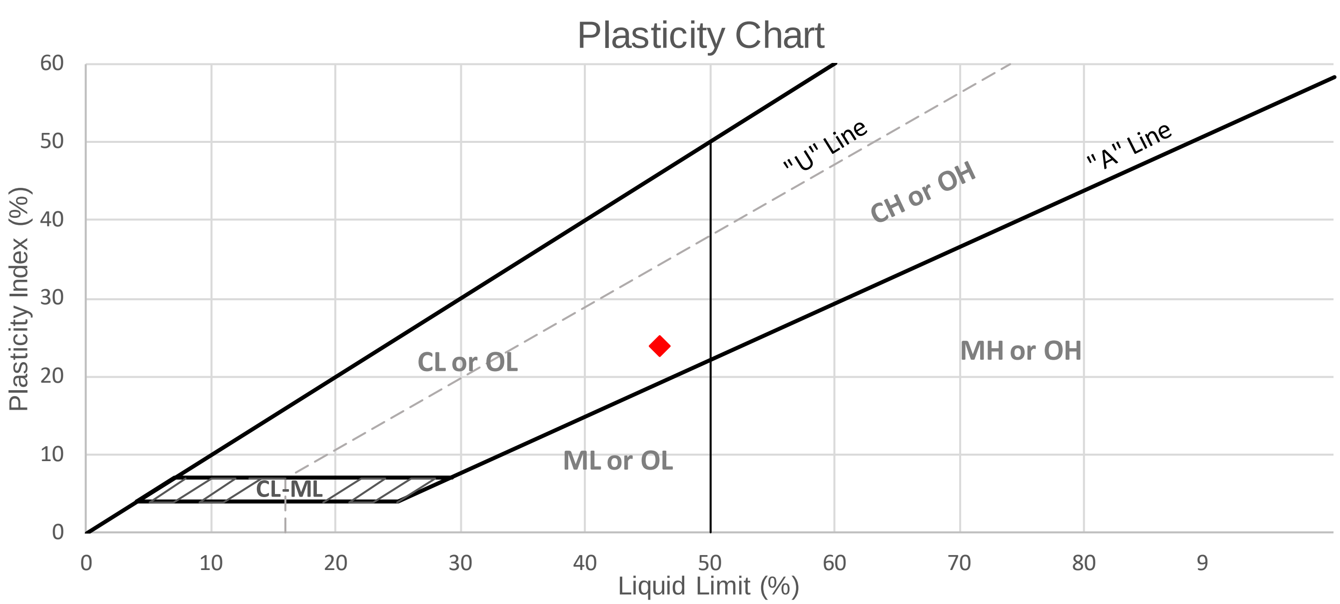


Plastic Limit Data

Trial	1	2	Ave
Water Content (%)	22.1	22.6	22

Data Summary

Liquid Limit	**46**
Plastic Limit	**22**
Plasticity Index	**24**
Natural Water Content	**15.1**
Liquidity Index	**-0.288**
% Passing #200 Sieve	**--**

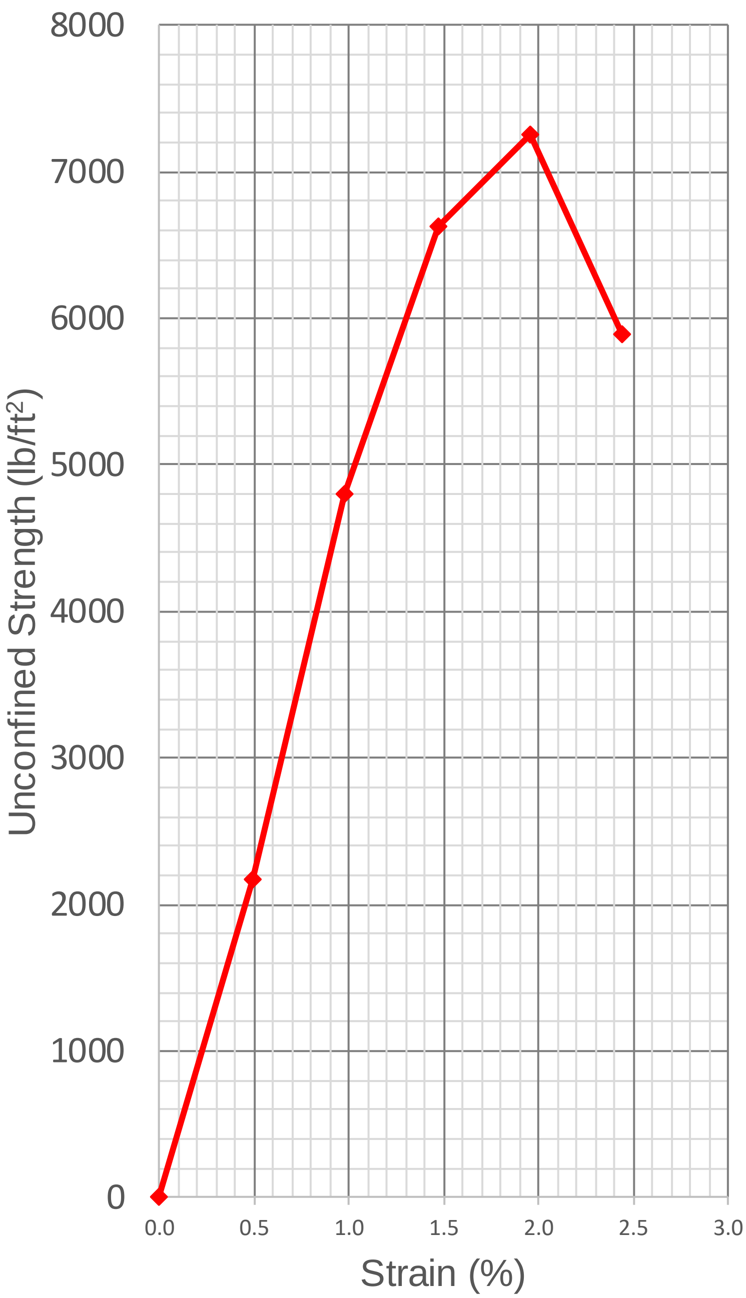


UNCONFINED COMPRESSIVE STRENGTH – D2166

Project Number: 674-6 **Boring #:** B-1 **Depth:** 2

Project Name: 1710 Moorpark **Date:** 12/19/2018

Description: 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	5.12 in
Volume	0.01363 ft^3
Water Content	15.1
Wet Density	107.4 pcf
Dry Density	93.3 pcf

Max Unconfined Compressive Strength	
Elapsed Time	2 min
Vertical Dial	0.1 in
Strain	2.0 %
Area	0.03258 ft^2
Axial Load	236.5 lbs
Compressive Strength	7,259 psf

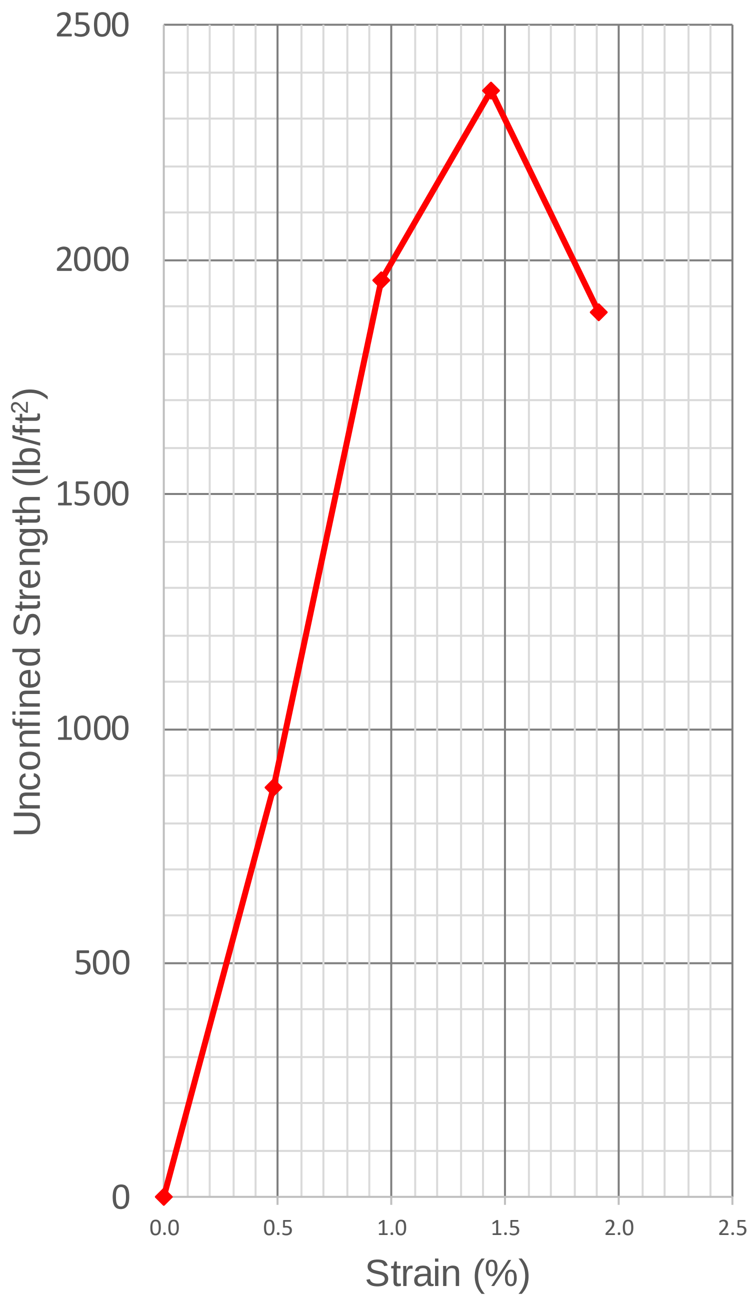
Stevens Ferrone & Bailey Engineering Company, Inc

UNCONFINED COMPRESSIVE STRENGTH – D2166

Project Number: 674-6 **Boring #:** B-1 **Depth:** 5.5

Project Name: 1710 Moorpark **Date:** 12/19/2018

Description: Brown sandy silty CLAY (CL) **Tested By:** R



Soil Specimen Initial Measurements	
Diameter	2.42 in
Initial Area	4.60 in^2
Initial Length	5.24 in
Volume	0.01395 ft^3
Water Content	8.3
Wet Density	111.2 pcf
Dry Density	102.7 pcf

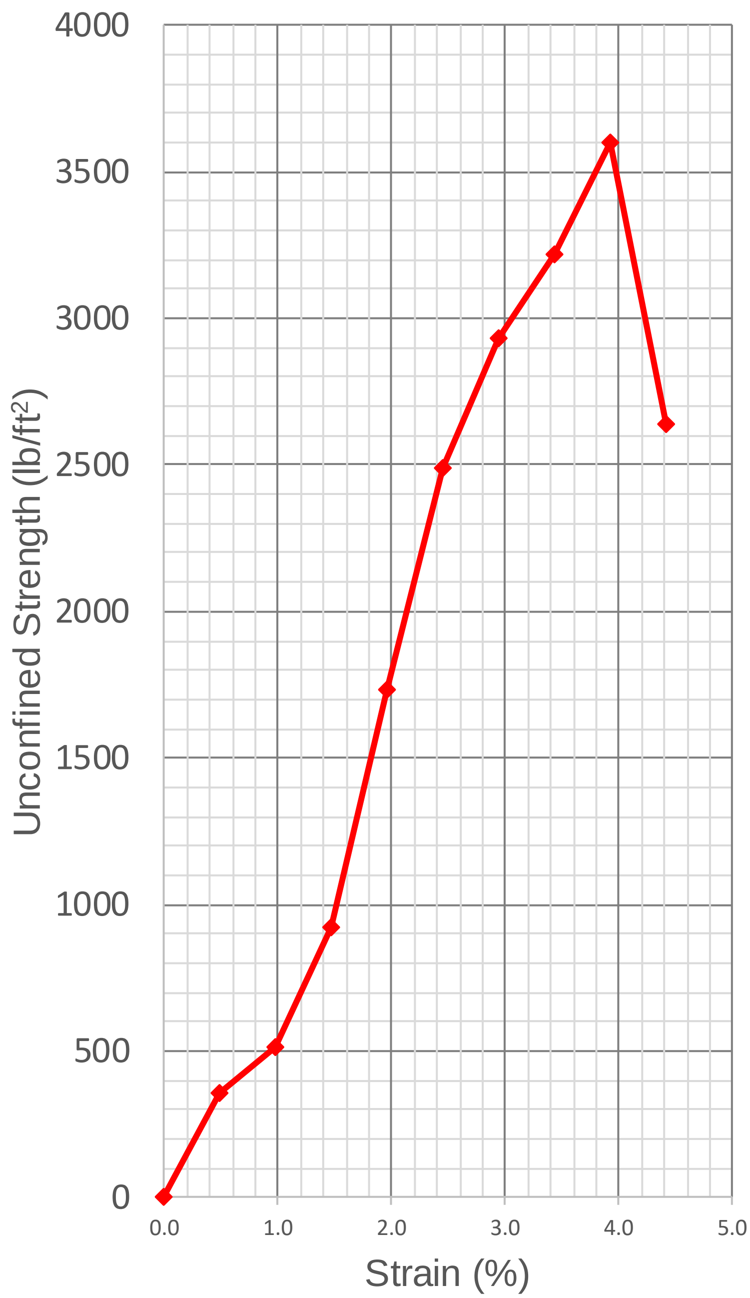
Max Unconfined Compressive Strength	
Elapsed Time	1.5 min
Vertical Dial	0.075 in
Strain	1.4 %
Area	0.03241 ft^2
Axial Load	76.5 lbs
Compressive Strength	2,361 psf

UNCONFINED COMPRESSIVE STRENGTH – D2166

Project Number: 674-6 **Boring #:** B-2 **Depth:** 2

Project Name: 1710 Moorpark **Date:** 12/19/2018

Description: Brown sandy silty CLAY some gravel (CL) **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	7.1
Wet Density	111.1 pcf
Dry Density	103.7 pcf

Max Unconfined Compressive Strength	
Elapsed Time	4 min
Vertical Dial	0.2 in
Strain	3.9 %
Area	0.03325 ft^2
Axial Load	119.8 lbs
Compressive Strength	3,603 psf

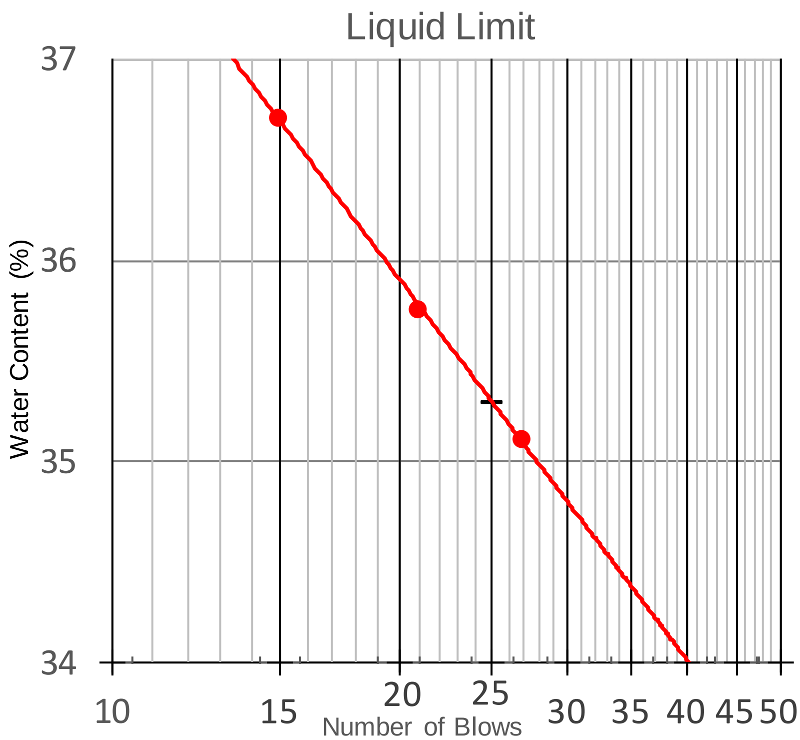
Stevens Ferrone & Bailey Engineering Company, Inc

Atterberg Limits Test – ASTM D4318

Project Number: 674-6 **Project Name:** 1710 Moorpark

Boring/Sample No: SFB-3 **Depth:** 1.5 **Date:** 09-27-19

Description of Sample: Brown silty CLAY with sand (CL) **Tested By** R

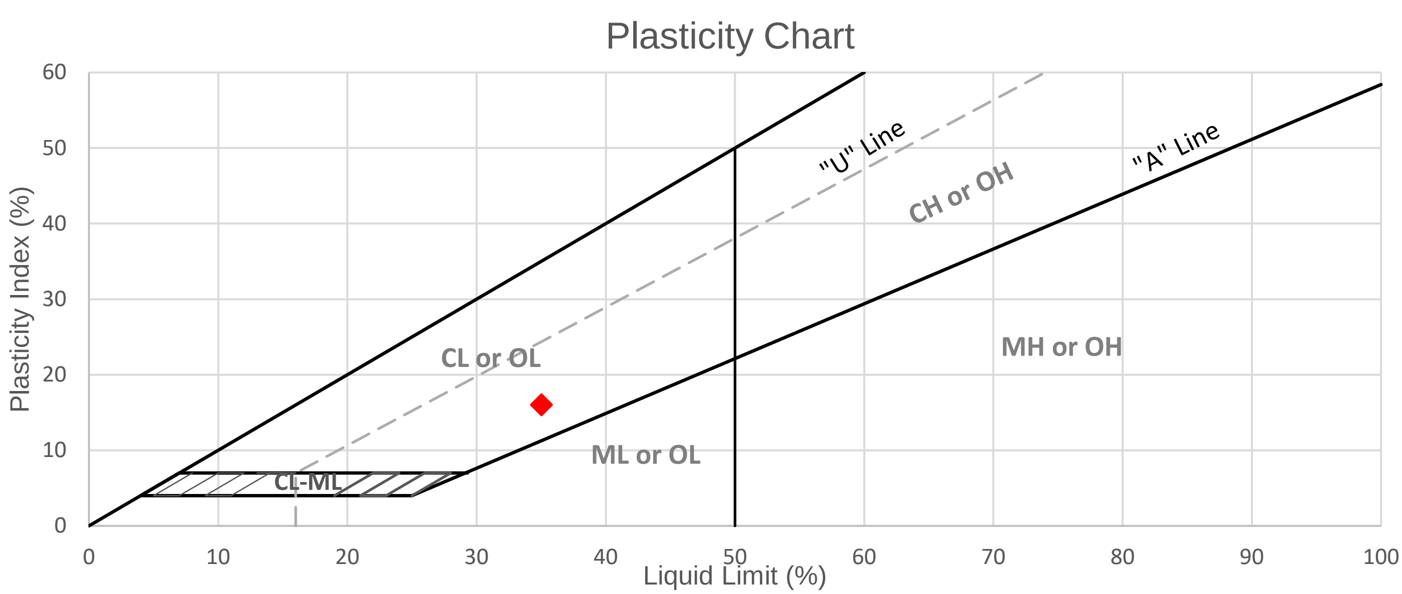


Plastic Limit Data

Trial	1	2	Ave
Water Content (%)	18.9	18.6	18.8

Data Summary

Liquid Limit	**35**
Plastic Limit	**19**
Plasticity Index	**16**
Natural Water Content	**11.7**
Liquidity Index	**-0.456**
% Passing #200 Sieve	**--**

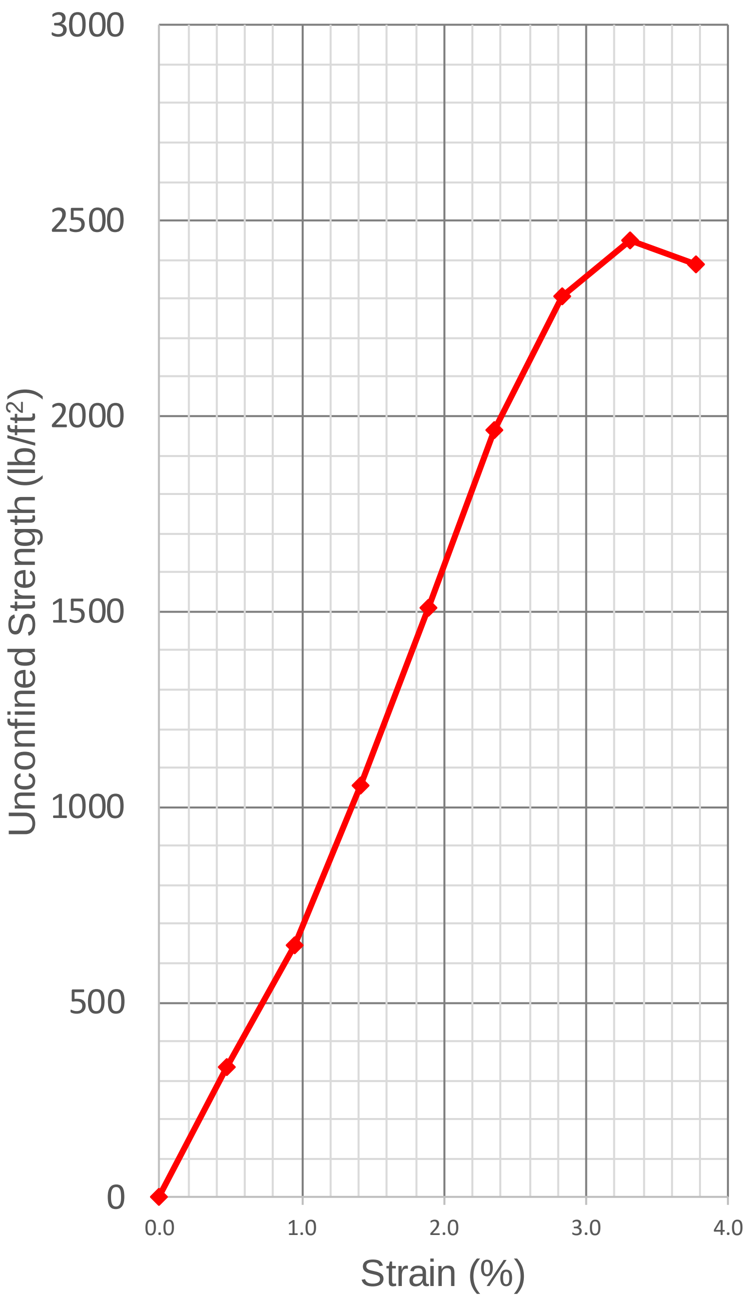


UNCONFINED COMPRESSIVE STRENGTH – D2166

Project Number: 674-6 **Boring #:** SFB-3 **Depth :** 1.5

Project Name: 1710 Moorpark **Date:** 9/26/2019

Description: Brown silty CLAY with sand (CL) **Tested By:** R



Soil Specimen Initial Measurements	
Diameter	2.42 in
Initial Area	4.60 in^2
Initial Length	5.3 in
Volume	0.01411 ft^3
Water Content	11.7 %
Wet Density	99.3 pcf
Dry Density	88.9 pcf

Max Unconfined Compressive Strength	
Elapsed Time	3.5 min
Vertical Dial	0.175 in
Strain	3.3 %
Area	0.03304 ft^2
Axial Load	80.9 lbs
Compressive Strength	2,449 psf

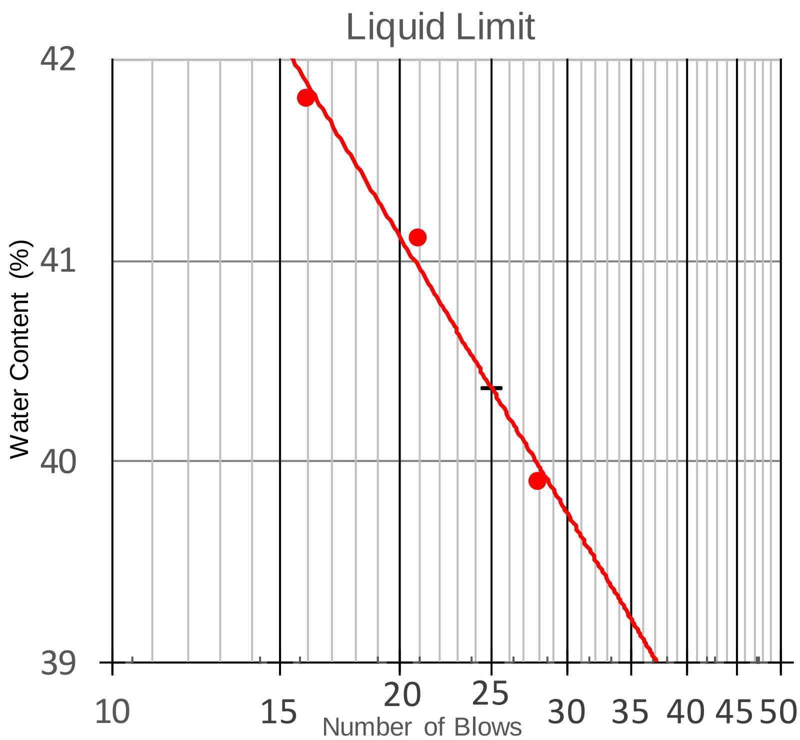
Stevens Ferrone & Bailey Engineering Company, Inc

Atterberg Limits Test – ASTM D4318

Project Number: 674-6 **Project Name:** 1710 Moorpark

Boring/Sample No: SFB-4 **Depth:** 2 **Date:** 09-27-19

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

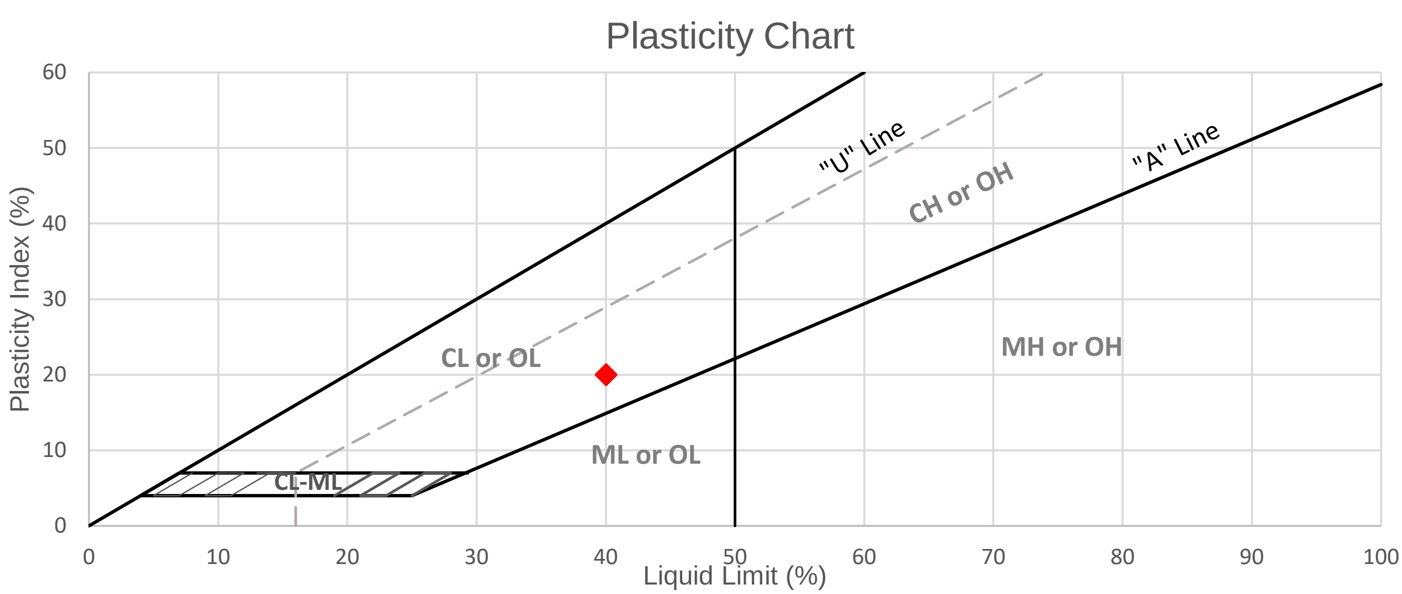


Plastic Limit Data

Trial	1	2	Ave
Water Content (%)	19.7	19.7	19.7

Data Summary

Liquid Limit	**40**
Plastic Limit	**20**
Plasticity Index	**20**
Natural Water Content	**13.6**
Liquidity Index	**-0.320**
% Passing #200 Sieve	**--**



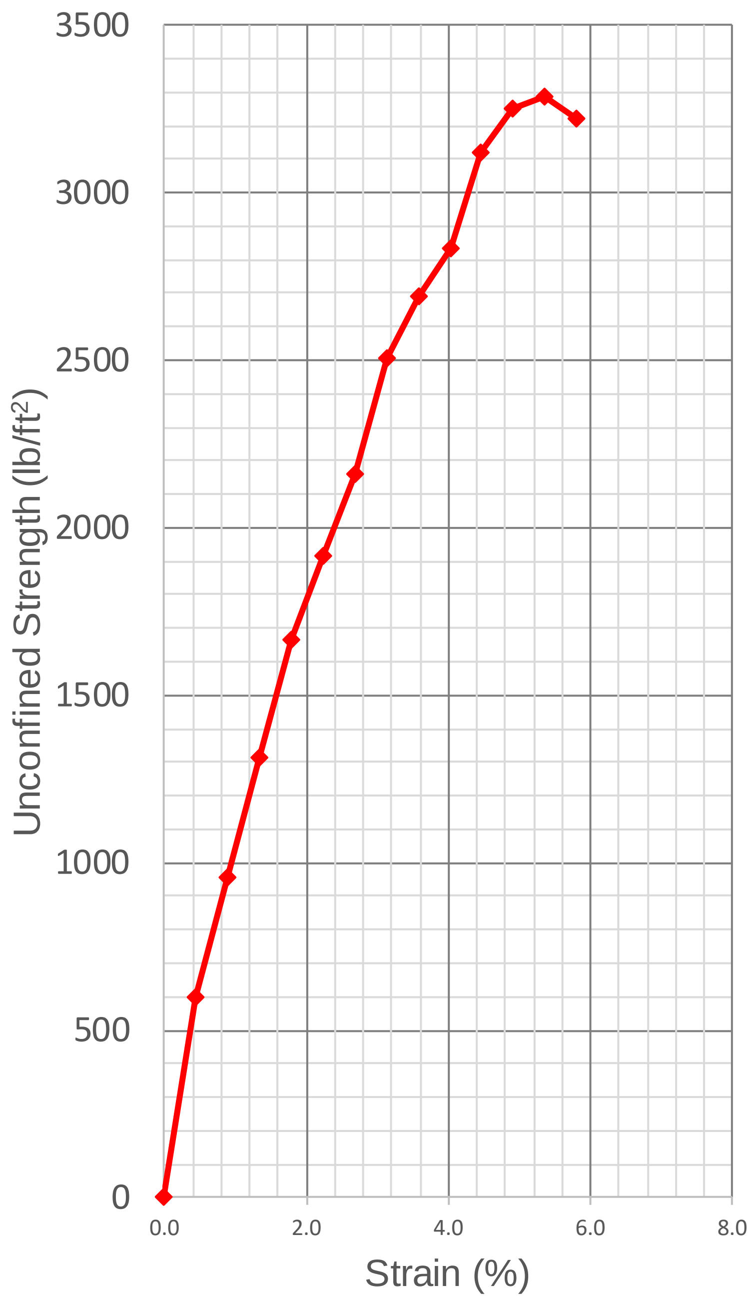
Stevens Ferrone & Bailey Engineering Company, Inc

UNCONFINED COMPRESSIVE STRENGTH – D2166

Project Number: 674-6 **Boring #:** SFB-4 **Depth :** 2

Project Name: 1710 Moorpark **Date:** 9/26/2019

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



Soil Specimen Initial Measurements	
Diameter	2.42 in
Initial Area	4.60 in^2
Initial Length	5.6 in
Volume	0.01491 ft^3
Water Content	13.6 %
Wet Density	117.1 pcf
Dry Density	103.1 pcf

Max Unconfined Compressive Strength	
Elapsed Time	6 min
Vertical Dial	0.3 in
Strain	5.4 %
Area	0.03375 ft^2
Axial Load	110.9 lbs
Compressive Strength	3,286 psf

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

11 October, 2019

Job No. 1910031
Cust. No. 11486

Mr. Ken Ferrone
Stevens, Ferrone & Bailey
1600 Willow Pass Court
Concord, CA 94520

Subject: Project No.: SFB 674-6
Project Name: 1710 Moorpark
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Ferrone

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on October 03, 2019. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, both samples are classified as "moderately corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentrations range from none detected to 35 mg/kg. Because the chloride ion concentrations are less than 300 mg/kg, they are determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentrations range from none detected to 27 mg/kg and are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The pH of the soils range from 7.42 to 7.53 which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials are both 430-mV which is indicative of aerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client: Stevens, Ferrone & Bailey
Client's Project No.: 674-6
Client's Project Name: 1710 Moorpark
Date Sampled: Not Indicated
Date Received: 3-Oct-19
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 11-Oct-2019

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1910031-001	SFB-3 @ 2.0'	430	7.42	-	3,000	-	35	27
1910031-002	SFB-4 @ 3.0'	430	7.53	-	2,600	-	N.D.	N.D.

	Redox	pH	Conductivity	Resistivity	Sulfide	Chloride	Sulfate
Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	10-Oct-2019	9-Oct-2019	-	7-Oct-2019	-	9-Oct-2019	9-Oct-2019

* 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. *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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