

November 10, 2022

**GEOTECHNICAL INVESTIGATION
19320 SONOMA HIGHWAY
SONOMA, CALIFORNIA
SFB PROJECT NO. 155-108**

Prepared For:

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- B Laboratory Testing
- C GBA Guidelines for Geotechnical Report

1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed residential development to be located at 19320 Sonoma Highway in Sonoma, California as shown on the **Vicinity Map, Figure 1**, and **Site Plan, Figure 2**. 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 project conceptual plan prepared by CBG and dated March 8, 2021, it is our understanding that the proposed project will consist of developing an about 2.2-acre site for about 7 multi-family residential buildings with a total of about 50 dwelling units. Other new improvements will include underground utilities and paved roadways. Nominal grading is anticipated. The existing residence, and associated improvements and facilities at the site, will be demolished prior to the 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- Reviewing published and unpublished geotechnical and geological literature relevant to the site;
- Reviewing historical aerial images and topographic maps of the site and surrounding area;
- Performing reconnaissance of the site and surrounding area;
- Performing five exploratory borings to a maximum depth of about 21 feet;
- Performing laboratory testing of soil 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 utility, drainage, building foundation, retaining walls, and pavements. Evaluating the potential for flooding and toxicity potential assessment of onsite materials or groundwater (including mold) were beyond our scope of work.

3.0 SITE INVESTIGATION

3.1 Field Exploration

Our geotechnical field exploration program for the project consisted of performing five exploratory borings to a maximum depth of about 21 feet on October 19, 2022. The borings were performed by West Coast Exploration, Inc. of Escalon, California, using a truck-mounted Mobile B-24 drill rig equipped with 4-inch diameter, continuous flight, solid stem augers and a 140-pound safety hammer. The approximate locations of the borings are shown on the **Site Plan, Figure 2**.

Our field engineer continuously logged the soils encountered in the borings. The soils are classified in general accordance with the Unified Soil Classification System (ASTM D2487 and ASTM D2488). Logs of the borings as well as a key for the classification of the soil (**Figure A-1**) are included in **Appendix A**. Upon completion of our field exploration, the borings were backfilled with lean cement grout in accordance with Sonoma County Resource Management Department requirements.

The approximate locations of the borings were determined by pacing, measurements, and/or alignment from landmark references, and should be considered accurate only to the degree implied by the method used. Latitude and longitude of boring locations shown on the boring logs were estimated from online map data from Microsoft; actual locations were not surveyed.

Representative samples were obtained from the borings at selected depths appropriate to the investigation. Relatively undisturbed samples were obtained using a 3-inch O.D. Modified California split barrel sampler with liners, and disturbed samples were obtained using a 2-inch O.D. Standard Penetration Test (SPT) split spoon sampler without liners. All samples were transported to our geotechnical laboratory 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 (N-value) were obtained in the borings with the samplers by dropping a 140-pound safety hammer through a 30-inch fall with a rope and cathead. 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 blows that were required to drive the last 12 inches, or the number of inches indicated where hard resistance was encountered. Blow counts recorded on the boring logs have been converted to equivalent SPT field blow counts. A sampler barrel size correction factor of 0.6 was applied to the blow counts from the Modified

California sampler. The recorded blow counts have not been corrected for other factors, such as hammer efficiency, borehole diameter, rod length, overburden pressure, and fines content.

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. In addition, the attached exploration 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.

3.2 Laboratory Testing

Our laboratory testing program for the project was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site. This program included the following testing:

- Four moisture content and dry unit weight determinations per ASTM D2937.
- Two Atterberg Limits determinations (plastic and liquid limits) per ASTM D4318.
- Two sieve and hydrometer tests per ASTM D422.
- Three unconfined compressive strength tests per ASTM D2166.

All tests were performed by our geotechnical laboratory in Concord, California. The results of the testing are included on the exploration logs and plotted laboratory results are also included in **Appendix B**.

Two selected onsite soil samples were tested by CERCO Analytical, Inc. in Concord, California for pH (ASTM D4972), chlorides (ASTM D4327), sulfates (ASTM D4327), sulfides (ASTM D4658M), resistivity at 100% saturation (ASTM G57), and Redox potential (ASTM D1498). The test results and a brief evaluation summary report prepared by CERCO regarding the onsite soils' potential for corrosion on concrete and buried metal such as utilities and reinforcing steel are included in **Appendix B**. We recommend these corrosion test results be forwarded to your underground contractors, pipeline designers, concrete contractors, and foundation designers and contractors

3.3 Site History and Surface Conditions

As shown on **Figure 2** and at the time of our investigation, the site was bounded by Sonoma Highway (Highway 12) on the west and existing commercial and residential developments on the

other sides. The site was located at about 440 feet east of the Sonoma Creek. The site was approximately rectangular in shape (excluded the southwestern corner), generally level, and had a plan area of about 2.2 acres with maximum dimensions of about 540 feet by 200 feet.

At the time of our field exploration, most of the site was vacant except for the northwestern corner that was occupied by a one-story, wood-frame residence built in about 1932. Concrete paved driveway and patios were located to the south and east of the residence. Large and small diameter trees, and shrubs were generally located within the western half of the site as well as along the site perimeters. The ground surface of the vacant area consisted of heavy growths of weeds and grasses.

Based on our review of historical aerial photographs of the site and vicinity, most of existing developments surrounding the site (except for the older structures along Sonoma Highway) were built after the 1960s'. In addition, it appears there was no previous development within the vacant area of the site in the past.

3.4 Subsurface Conditions

The subsurface soils encountered in our borings generally consisted of medium dense to very dense sands with interbeds of very stiff to hard sandy clays that extended to the maximum depth explored of about 21 feet. Some of the encountered sand layers were lightly cemented. In addition, drilling refusal was encountered by the Mobile B-24 drill rig at the bottom of Borings B-1, B-4, and B-5 at depths varying from about 4 to 14 feet. The upper about 2 feet of the surficial soils were generally loose, dry, and desiccated.

According to the results of laboratory testing, the onsite near-surface soils have a low to medium plasticity and low to moderate volumetric shrinkage and expansion potential. Detailed descriptions of soils encountered in our borings are presented on the exploration logs in **Appendix A**. Results of laboratory testing of retrieved onsite soils are also included in the exploration logs as well as in **Appendix B**.

3.5 Groundwater

No groundwater was encountered in our borings to the maximum depth of about 21 feet explored at 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. Fluctuations in the groundwater level could occur due to change in seasons, variations in rainfall, pumping of water wells in the surrounding area, water recharging from the nearby creek, and other factors.

According to the available groundwater level data from environmental reports downloaded from the State Water Resources Control Board GeoTracker website¹, groundwater levels reportedly fluctuated at about 3 to 20 feet deep between 1987 and 2007 at the nearby 19295 Sonoma Highway property (across the street to the northwest of the project site).

3.6 Hydrologic Soil Group

The surface soils of the site have been mapped by the USDA Natural Resource Conservation Services (NRCS) Web Soil Survey (WSS)² and categorized as Tuscan cobbly clay loam, 0 to 9 percent slopes (Unit TuC). This map unit had been assigned to Hydrologic Soil Group D by NRCS and is characterized as having very low water transmission rates. Group D soils are defined as having a very slow infiltration rate (high runoff potential) when thoroughly wet. These 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.

Based on the results of our field borings and laboratory testing of retrieved soil samples, we recommend that the onsite near-surface soils be assigned as Hydrologic Soil Group D due to the presence of dense to very dense, lightly cemented sands at shallow depths.

Actual field infiltration rates will depend on the in-situ soil type, moisture, relative density, gradation, and fines content of soils, and whether any water impeding clay layers exist at shallow depth. If needed, we recommend Double Ring Infiltrometer Tests (ASTM D3385) be performed to evaluate the actual field infiltration rates.

3.7 Geology and Seismicity

According to Wagner and Gutierrez (2010)³, the site and the surrounding areas are mapped as being underlain by early to late Pleistocene alluvial fan deposits. These soil deposits generally consist of deeply dissected sand, gravel, silt, and clay.

The project is located in the Sonoma Area that is considered to be one of the most seismically active regions in the United States. Significant earthquakes that have occurred in the region are

¹State Water Resources Control Board GeoTracker website, <https://geotracker.waterboards.ca.gov/>, accessed 11/7/2022.

²USDA NRCS, <https://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx>, accessed 11/7/2022.

³Wagner & Gutierrez, 2010, *Preliminary Geologic Map of the Napa 30'x 60' Quadrangle, California*, California Geological Survey.

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 California⁴. Therefore, it is our opinion that the potential for ground surface rupture due to a fault crossing the site is low.

Earthquake intensities will vary throughout the region, 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 indicated 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 be subjected to earthquakes that cause strong ground shaking.

According to 2019 CBC/ASCE 7-16, the site modified geometric mean peak ground acceleration (PGA_M) from a Maximum Considered Earthquake (MCE) event is estimated to be about 0.85g based on a stiff soil condition (Site Class D). The MCE peak ground acceleration generally has a 2% probability of being exceeded in 50 years (a mean return period of 2,475 years) except where deterministically capped along highly active faults.

According to the U.S. Geological Survey's Unified Hazard Tool and applying the Dynamic: Conterminous U.S. 2014 model (v4.2.0)⁶, the resulting deaggregation calculations indicate that the site has a 10% probability of exceeding a peak ground acceleration of about 0.54g in 50 years (a design earthquake ground motion based on a Site Class D with a mean return period of 475 years).

The actual ground surface acceleration might vary depending upon the local seismic characteristics of the underlying bedrock and the overlying soils.

3.8 Liquefaction

Soil liquefaction is a phenomenon primarily associated with saturated cohesionless soil layers. These soils can dramatically lose strength due to increased pore water pressure during cyclic loading, such as imposed by earthquakes. During the loss of strength, the soils acquire mobility sufficient to permit both horizontal and vertical movements. Soils that are most susceptible to liquefaction are clean, loose, uniformly graded, saturated sands that lie close to the ground surface; although, liquefaction can also occur in fine-grained soils, such as low-plasticity silts.

⁴California Department of Conservation, Earthquake Fault Zones, CGS Special Publication 42, Revised 2018.

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

⁶USGS Unified Hazard Tool, <https://earthquake.usgs.gov/hazards/interactive/>, accessed 11/7/2022.

As of the date of this report, the liquefaction potential of the site and surrounding area has not been evaluated by the State of California⁷. According to Witter et al. (2006)⁸, the site is located in an area that has been characterized as having low liquefaction susceptibility.

Based on our review of available geologic literature, and the results of exploratory borings and laboratory testing, it is our opinion that the potential for ground surface damage at the site resulting from liquefaction is low since the sandy soils encountered in our borings are generally dense to very dense in consistency. Due to the high density of the sandy soils, drilling refusal was encountered by the Mobile B-24 drill rig at the bottom of Borings B-1, B-4, and B-5 at depths varying from about 4 to 14 feet.

3.9 Lateral Spreading

As part of our soil liquefaction evaluation, we also evaluated the potential for lateral spreading impacting the site. Lateral spreading occurs when soils liquefy during an earthquake event, and the liquefied soils along with the overlying soils move laterally toward unconfined spaces (such as creek channels), which causes significant horizontal ground displacements.

Since the site is located at least 440 feet east from the nearby Sonoma Creek (of about 40 to 50 feet deep), it is our opinion that there is a low potential for lateral spreading adjacent to the creek (if it occurs) adversely impacting the site development.

⁷Seismic Hazards Mapping Act, 1990.

⁸Witter, 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.

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.

WEAK SOIL MATERIALS: The upper about 2 feet of the surficial soils were generally loose, dry, and desiccated at the time of our site field exploration. In addition, we estimate that demolition of the existing structures, and associated foundations and improvements will disturb and weaken the upper 2 feet of soils at the site. In order to reduce the potential for damaging differential settlement of overlying improvements (such as new fills, building foundations, driveways, exterior flatwork, and pavements), we recommend these weak soils be over-excavated and re-compacted. The process can consist of over-excavating to about 1 foot below the existing ground surface, scarifying and re-compacting the bottom 12 inches in-place, and placing well-blended, moisture conditioned, and properly compacted fill over the properly prepared subgrade. The over-excavation should extend to depths where competent soils are encountered. Deeper removal may be needed in areas where thicker weak soils are encountered during grading.

Over-excavation and re-compaction should extend at least 5 feet beyond building footprints and at least 3 feet beyond exterior flatwork (including driveways) and pavement wherever possible. There would be no need to over-excavate and re-compact the soils within areas that do not support improvements, such as within open spaces. 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 proposed building foundations. The extent of the removal and re-compaction may vary across the site and should be determined in the field by SFB at the time of the earthwork operation.

The removed soil materials can be used as new fills provided they are placed and compacted in accordance with the recommendations presented in this report.

EXPANSION POTENTIAL: The onsite more clayey, moderately expansive 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 buildings be supported on post-

tensioned slab foundations that are designed to reduce the impact of the onsite expansive soils. It should be noted that special design considerations will be required for exterior slabs.

CORROSION POTENTIAL: Two selected 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 **Appendix B**. We recommend these test results and brief evaluation summary be forwarded to your concrete contractors, underground contractors, pipeline designers, and foundation designers and contractors so 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, including foundation reinforcement, are protected against corrosion. We also recommend additional testing be performed if the test results are deemed insufficient by the designers and installers of the corrosion protection. Landscaping soils typically contain fertilizers and other chemicals that 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 in order to design and provide protection against corrosion for the foundation and pipelines.

ADDITIONAL RECOMMENDATIONS: Detailed earthwork, underground utility, drainage, foundation, retaining wall, 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.

It is the responsibility of the contractors to provide safe working conditions at the site at all times. We recommend all OSHA regulations be followed, and excavation safety be ensured at all times. 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 existing structures and their entire foundation systems, existing utilities and pipelines and their associated backfill, pavements, 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. Septic tank systems and water wells (if any) should be abandoned in accordance with Sonoma County standards.

From a geotechnical standpoint, any existing 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 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 landscaped 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. Crushed asphalt concrete and concrete materials can be re-used onsite as aggregate base or subbase if they meet current Caltrans specifications for aggregate base or subbase based on laboratory testing results.

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

4.1.2 Weak Soil Re-Compaction

As described previously, in order to reduce the potential for damaging differential settlement of overlying improvements (such as new fills, building foundations, driveways, exterior flatwork, and pavements), we recommend the weak surface soils be over-excavated and re-compacted. The process can consist of over-excavating to about 1 foot below the existing ground surface, scarifying

and re-compacting the bottom 12 inches in-place, and placing well-blended, moisture conditioned, and properly compacted fill over the properly prepared subgrade. The over-excavation should extend to depths where competent soils are encountered. Deeper removal may be needed in areas where thicker weak soils are encountered during grading.

Over-excavation and re-compaction should extend at least 5 feet beyond building footprints and at least 3 feet beyond exterior flatwork (including driveways) and pavement wherever possible. There would be no need to over-excavate and re-compact the soils within areas that do not support improvements, such as within open spaces. 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 proposed building foundations. The extent of the removal and re-compaction may vary across the site and should be determined in the field by SFB at the time of the earthwork operation.

The removed soil materials may be used as new fill onsite provided they satisfy 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 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 2 to 3 percent over optimum water content, and compacted to the requirements for structural fill. Subgrade preparation would not be necessary in areas where over-excavation and re-compaction of the surface soils and fills have occurred.

If completed building pads, driveway and pavement subgrades are allowed to remain exposed to sun, wind or rain for an extended period of time, are heavily disturbed by vehicle traffic or animal borrowing, or have vegetation growth, the exposed pads and subgrades 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 pad and subgrade reconditioning.

4.1.4 Fill Material

From a geotechnical and mechanical standpoint, onsite soils materials 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, and should have no more than 15 percent larger than 2.5 inches. Larger sized rock may be used as fill onsite provided it is closely monitored, placed properly to achieve compaction, and are located at depths below anticipated, future excavations; SFB should be consulted regarding the use of larger rock pieces in fill materials. If required, imported fill should have a plasticity index of 15 or less and have a significant amount of cohesive fines.

In addition to the mechanical property 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 at least 90 percent relative compaction as determined by ASTM D1557 (latest edition). We recommend the new fill be moisture conditioned approximately 2 to 3 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 driveway, exterior slab, or under a building foundation, and are located below irrigated landscaped areas such as lawn or planting strips, should be plugged with low strength concrete or sand/cement slurry. The plug for the trench lateral should be located below the edge of pavement or slab, 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.

We also recommend installing the plugs every 50 feet on center along any utility trenches that are sloped 5 percent or steeper to reduce soil piping from water seepage that may cause trench surface settlement. Where used, these plugs should extend to within 1 foot of the finished ground surface or to the base of the pavement section.

4.1.7 Exterior Flatwork

We recommend that exterior slabs (including driveways, patios, and walkways) be placed directly on the properly compacted fills. If imported granular materials are placed below these elements, subsurface water can seep through the granular materials and cause the underlying soils to saturate, pipe, and/or heave upward. Prior to placing concrete, subgrade soils should be moisture conditioned to increase their moisture content to approximately 2 to 3 percent above laboratory optimum moisture (ASTM D-1557).

The 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 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 24 inches on center in both directions should be installed. Score joints and expansion joints should be used to control cracking and allow for expansion and contraction of the concrete slab. 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 24 inches. Although exterior slabs that are adequately reinforced will still crack, trip hazards requiring replacement of the slabs will be reduced if the slab are properly reinforced.

We do not recommend the use of flatwork having permeable joints (such as pavers or tiles with sand or gravel infilled joints) unless the underlying clayey subgrade is protected against water seepage or ponding. If not protected, the underlying subgrade will heave, settle, and/or pipe and cause damage to the overlying improvements.

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, near the foundations, at the top or bottom of slopes, and at the top or near 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 and maintained adjacent to structure 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 should be placed uniformly adjacent to structure foundations and exterior slabs. We recommend trees be no closer to structures 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.

Landscaping drainage inlets and/or drainage swales must be provided and maintained around structures at all times that adequately collect irrigation and storm water and direct the water onto pavement or into storm water collection systems. Drainage inlets should be provided within

enclosed planter areas and the collected water should be discharged onto pavement, into drainage swales, or into an enclosed storm drain system. The drainage inlets and associated swales should be designed and constructed so that the moisture content of the soils surrounding structure 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 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 structure foundations and exterior slabs to maintain moist soil conditions. Over-watering must be avoided. To reduce moisture changes in the soils and fills in landscaped areas, we recommend that drought resistant plants and low flow watering systems be used. All irrigation systems should be inspected for leakage regularly.

4.1.10 Storm Water Treatment Facilities

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 storm water treatment facilities is to conserve and incorporate on-site natural features, together with constructed hydrologic controls, to more closely mimic pre-development hydrology and watershed processes. These facilities include bio-retention swales and basins, porous paver and pavement, water detention basins, and any proprietary underground storage and treatment systems.

In general, we recommend the portion of the storm water treatment facilities that are within 10 feet of structure foundations and improvements (such as building foundations, exterior flatwork, and pavements) be lined with a relatively impermeable membrane to reduce water seepage and the potential for damage and distress to the adjacent structures and improvements. The lining can consist of a relatively impermeable membrane such as STEGO Wrap 15-mil or equivalent. The membrane should be lapped and sealed in accordance with the manufacturer's specifications, including taping joints where pipes penetrate the membrane.

Soil filter/bio-mix 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/bio-mix 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.

Soil filter/bio-mix materials provide little to no lateral restraint of excavation side walls. Sidewalls of bio-retention swale and basin excavations (excavations made prior to the installation of the soil filter/bio-mix) steeper than 2:1 (horizontal to vertical) will experience downward and lateral movements that can cause distresses to adjacent improvements such as foundations, utilities, pavements, driveways, walkways, and curbs and gutters. The magnitude and rate of movement depend upon the swale and basin backfill material type and compaction. To reduce the potential for damaging movements, we recommend 2:1 or flatter excavation sidewall slopes be used for bio-retention swales and basins, sidewalks be setback at least 3 feet from the top of slopes, and creep sensitive improvements (such as roadway curbs) be setback at least 5 feet from the top of slopes. If the above sidewall slope and setback distance cannot be met, consideration should be given to using below-grade concrete sidewalls that are designed and constructed as retaining walls. Alternatively, deepened sidewalk slab edge or roadway curbs can be used and designed to resist lateral earth pressures and act as a retaining wall. SFB should be consulted to evaluate the need for sidewall restraint when swales or basins are planned. We also recommend SFB observe and document the installation of liners, subdrain pipes, and soil filter/bio-mix materials during construction for conformance to the recommendations in this report and the development's plans and specifications.

Where used, proprietary underground storage and treatment systems should be installed and maintained in accordance with the manufacturer's specifications. In addition, the manufacturer should be consulted for vertical and lateral bearing capacities and anticipated deformations of these systems if they will also support exterior slabs and pavements that are subjected to vehicular traffic.

4.1.11 Future Maintenance

In order to reduce water related issues, we recommend regular inspection and maintenance of the site and development be performed, including maintenance prior to rainstorms. Inspections 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. 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.

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

4.1.12 Additional Recommendations

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

4.2 Foundation Support

4.2.1 Post-Tensioned Slabs

The proposed residential buildings can be supported on a post-tensioned slab foundation that is designed for the expansion potential of onsite soils. The slab foundation should bear entirely on properly prepared and 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, *Weak Soil Re-Compaction***, and **4.1.3, *Subgrade Preparation***. Prior to the concrete pour, we recommend the moisture content of subgrade materials be approximately 2 to 3 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 design 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 1:1 (horizontal to 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 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 Class A 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 lapping and all joints at least 6 inches and sealing 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. Any 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 which will be trapped under impermeable flooring. The concrete mix design for 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. If a higher water/cement ratio is being considered, we recommend higher vapor transmission be taken into account in the design and construction of the buildings. We recommend the foundation designer determine if corrosion protection is needed for the foundation concrete and reinforcing steel. The results of sulfate and chloride testing of 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.

During the curing process, concrete slabs will shrink in volume resulting in cracks developing in the slab. Curing of concrete can take many months (or possibly longer) to complete. These concrete cracks may be visible on the surface of the slab during and after the curing process. In order to

reduce the potential for crack propagation through overlying brittle surfaces such as tile or stone flooring, we recommend appropriate crack isolation measures be used between the concrete slab and flooring to reduce the potential for slab cracks to propagate into these brittle flooring surfaces.

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 (2008)⁹, PTI standard requirements (2019)¹⁰, and a PTI preferred computer program, VOLFLO (Version 1.5 Build 120704), was employed to simulate the wetting and drying scenarios of the soils beneath the post-tensioned slabs.

The values provided below are based upon the post-tensioned slab foundations being entirely surrounded by uniform, properly drained, moderately irrigated landscaping; if differing conditions will exist that will cause differential soil moisture adjacent or below the slabs, or if portions of the foundations will be located adjacent to relatively dry or wet soils, then we should be consulted. Modifications to the design values below would need to be made 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

	<u>Center Lift</u>	<u>Edge Lift</u>
Edge Moisture Variation Distance (e_m)	9.0 feet	5.0 feet
Differential Soil Movement (y_m)	0.5 inch	1.0 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.

⁹Post-Tensioning Institute, 2008, Design of Post-Tensioned Slabs-On-Ground (PTI DC10.1-08), Third Edition.

¹⁰Post-Tensioning Institute, 2019, Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soils (PTI DC10.5-19).

4.2.2 Retaining Walls

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. Any walls that retain soils should be designed to resist both lateral earth pressures and any additional lateral loads caused by roadway surcharging, earthquake loading, and hydrostatic pressure if wall back-drainage is not provided.

If walls are allowed to deflect or rotate (unrestrained walls), they can be designed to resist active pressures. If no movement is allowed at the top of walls (restrained walls), at-rest pressures should be used in wall design. The recommended active and at-rest lateral earth pressures under both drained and undrained conditions are provided in the table below.

LATERAL EARTH PRESSURES FOR RETAINING STRUCTURES				
Wall Condition	Backfill Condition	Drained Equivalent Fluid Pressure (pcf)	Undrained Equivalent Fluid Pressure (pcf)	Incremental Seismic Pressure (pcf)
Unrestrained (Active Pressure)	Level	40	80	40
Restrained (At-rest Pressure)		60	90	80*

*Note: For restrained walls, use the static active pressure plus the seismic increment in the seismic design.

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 (as required by the 2019 CBC), we recommend the walls be designed to also resist incremental seismic lateral earth pressures listed in the above table, using a triangular fluid pressure distribution (not inverted). The seismic induced earth pressures are in addition to the static active pressure listed above (for both unrestrained and restrained walls). The seismic lateral earth pressure increments for unrestrained and restrained walls were estimated, respectively, based on 50% and 100% of the peak ground acceleration (PGAM) from a Maximum Considered Earthquake (MCE) earthquake per ASCE 7-16/2019 CBC. 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.

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. Any surcharge loads located within an imaginary 1:1 (horizontal to vertical) plane projected upward from the base of the walls will increase the lateral earth pressures on the wall. Walls subjected to surcharge loads should be designed for an additional uniform lateral pressure (rectangular distribution) equal to one-third (0.33) and one-half (0.5) the anticipated surcharge load for unrestrained and restrained walls, respectively. Walls adjacent to areas subject to vehicular traffic should be designed for a 2-foot equivalent soil surcharge (250 psf). We should be consulted to provide load contributions from other particular surcharges located behind walls if needed.

It should be noted the lateral earth 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.

The above recommended drained lateral earth pressures assume walls are fully back drained to prevent the build-up of hydrostatic pressures. If drainage behind the wall is omitted, the wall should be designed for undrained condition. Wall back-drainage can be accomplished by using 1/2- to 3/4-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 12 inches wide and extend from the base of the wall to within about 1 foot of the finished grade at the top (Class 2 permeable material per Caltrans Specification Section 68 may be used in lieu of gravel and filter fabric). The upper 1 foot of cover backfill should consist of relatively impervious material.

Where wall back-drainage is used, a 4-inch diameter, perforated, PVC SDR-35 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 suitable discharge facilities. 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. Weepholes can be spaced at about 10 to 15 feet apart. As an alternative to using gravel, pre-fabricated 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 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 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. 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. Eighty percent of the skin friction value can be used to resist uplift. Lateral load resistance can be developed in passive resistance for pier foundations. We recommend an allowable soil passive resistance (which includes a factor of safety of 1.5) equal to an equivalent fluid weighing 300 pounds per cubic foot be used for pier foundations. This value can be used up to a maximum value of 3,600 psf. The passive resistance can be applied against twice the projected diameter of pier shaft if the piers are spaced center-on-center at least 3 times of the pier shaft diameter.

The upper 2 feet of pier embedment should be neglected in the vertical and passive resistance design as measured from finished grade unless it is confined by a pavement or concrete slab. The portion of the pier shaft located within 10 feet (as measured laterally) of the nearest slope face or above an imaginary 1:1 (horizontal to vertical) plane extending upward from the bottom of any adjacent walls or utility trenches should also be ignored in both the vertical bearing and passive resistance designs.

The bottom of pier excavation should be relatively dry and free of all loose cuttings or slough prior to placing reinforcing steel and concrete. Any accumulated water in pier excavation 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 pour 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 dewatering, casing, slurry, tremie methods, and/or pouring

concrete immediately after excavating may be necessary. SFB should be consulted for additional measures for pier construction as needed during construction.

As an alternative to using pier foundations to support the walls, footings may be used. SFB should be consulted to provide footing foundation recommendations for retaining walls where used.

4.2.3 Seismic Design Criteria

The following parameters were calculated using the U.S. Seismic Design Map program¹¹, and are based on the site being located at approximate latitude 38.297523°N and longitude 122.474271°W. For seismic design using the 2019 California Building Code (CBC), we recommend the following seismic design parameters be used for the project. These values are based on applying the ASCE 7-16 model, assuming the structure is categorized as Risk Category II, and assuming that *Exception Number (2) of ASCE 7-16 Section 11.4.8 – Site Specific Ground Procedure* applies. We should be contacted if any of these assumptions are incorrect or a site-specific ground motion hazard analysis is required.

SEISMIC PARAMETER	DESIGN VALUE
Site Class	D
S _s	1.838
S ₁	0.693
S _{MS}	1.838
S _{M1}	See Section 11.4.8 of ASCE 7-16*
S _{DS}	1.225
S _{D1}	See Section 11.4.8 of ASCE 7-16*
SDC	See Section 11.4.8 of ASCE 7-16*
F _a	1.000
F _v	See Section 11.4.8 of ASCE 7-16*
PGAM	0.846
T _L	8

*Note: The values of F_v, S_{M1}, S_{D1}, and Seismic Design Category (SDC) should be determined by the Structural Engineer based on ASCE 7-16 Section 11.4.8 requirements.

¹¹SEAONC/OSHPD, <https://seismicmaps.org/>, accessed 11/7/2022.

4.3 Pavements

Based on the soils encountered in our borings and results of laboratory testing, we recommend that an R-value of 15 be used in preliminary asphalt concrete pavement design. We recommend R-value tests be performed once the pavement subgrade is established to confirm the R-value used in the design. Pavement subgrade 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 = 15			
Location	Pavement Components		Total Thickness (inches)
	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	
T.I. = 4.5 (auto & light truck parking)	2.5	7.5	10.0
T.I. = 5.0 (access ways/courts)	3.0	8.0	11.0

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.

We recommend the pavement materials and construction conform to Caltrans Standard Specifications. Pavement aggregate base and asphalt concrete should be compacted to at least 95 percent relative compaction as determined by ASTM D1557 or Caltrans Test Method 375. 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 characterize 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 DeNova Homes and their consultants for specific application to the proposed residential development project at 19320 Sonoma Highway in Sonoma, California, and is intended to represent our design recommendations to DeNova Homes for specific application to 19320 Sonoma Highway project. The conclusions and recommendations contained in this report are solely professional opinions. It is the responsibility of DeNova Homes 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 construction calculations, specifications, and plans; we should also be retained to participate in pre-bid and pre-construction conferences to clarify the opinions, conclusions, and recommendations contained in this report.

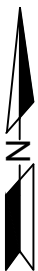
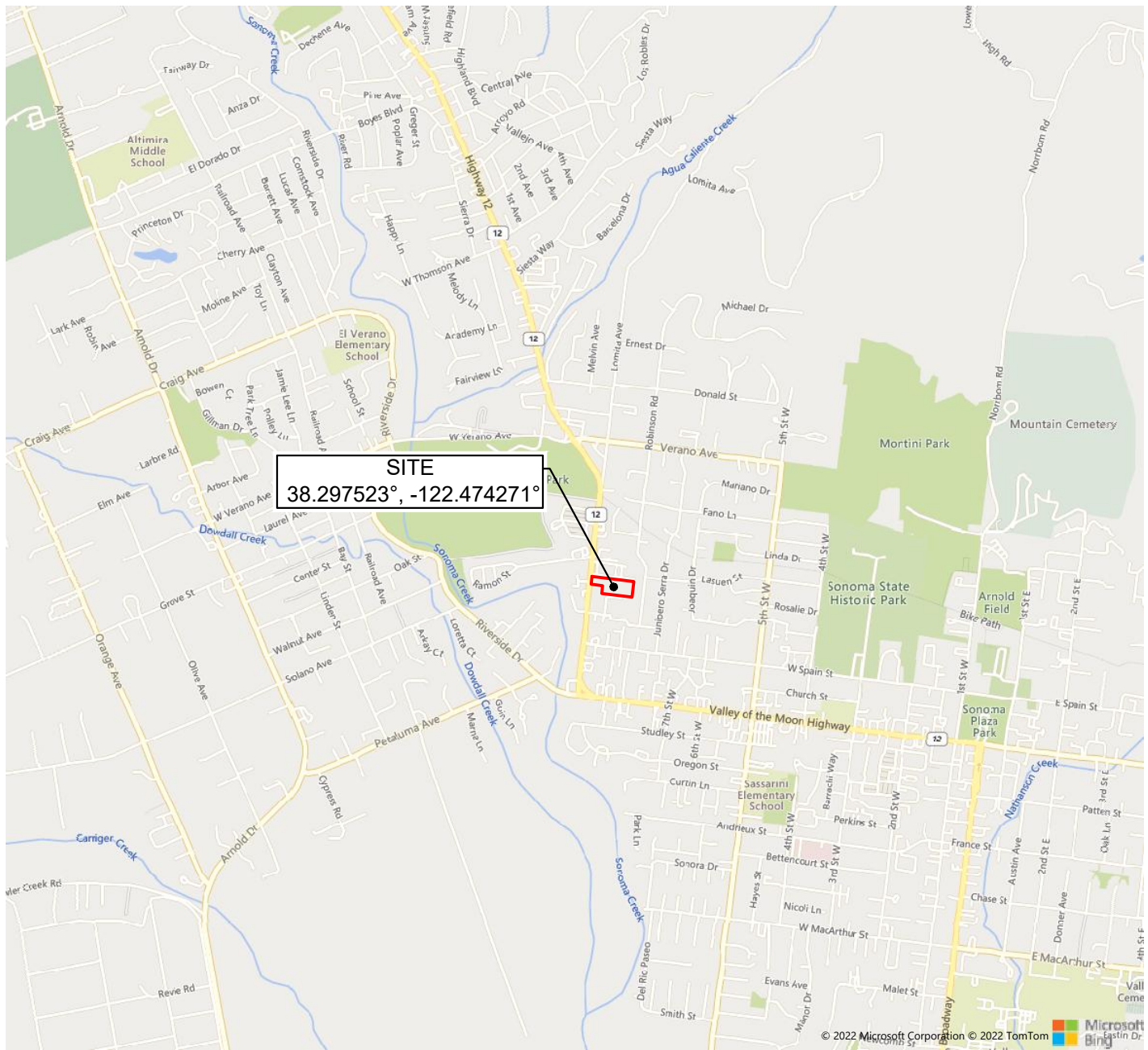
It should be understood that advancements in the practice of geotechnical engineering and engineering geology, or discovery of differing surface or subsurface conditions, may affect the validity of this report and are not uncommon. SFB strives to perform its services in a proper and professional manner 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 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 DeNova Homes and their consultants during the course of this engagement and our rendering of professional services to DeNova Homes. 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 DeNova Homes to divulge information that may have been communicated to the DeNova Homes. We cannot accept consequences for use of segregated portions of this report.

Please refer to Appendix C for Geoprofessional Business Association (GBA) guidelines regarding use of this report.

FIGURES



APPROXIMATE SCALE: 1" = 2,000'



NOTE: Basemap from Microsoft/TomTom 2022.

DATE		1600 Willow Pass Court Concord, CA 94520 Tel 925.688.1001 Fax 925.688.1005 www.SFandB.com	VICINITY MAP	FIGURE
November 2022			19320 SONOMA HIGHWAY Sonoma, California	1
PROJECT NO.				
155-108				



KEY

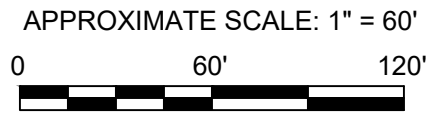
 Exploratory Boring by SFB (10/19/22)

 Approximate Project Limit

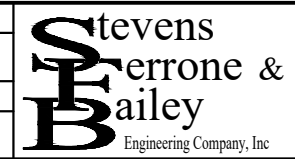
NOTE: All locations shown are approximate.



NOTE: Base map from the project conceptual plan prepared by CBG and dated 3/8/2021. Aerial photo imagery from Microsoft 2022.



DATE	November 2022
PROJECT NO.	155-108



1600 Willow Pass Court
 Concord, CA 94520
 Tel 925.688.1001
 Fax 925.688.1005
 www.SFandB.com

SITE PLAN
 19320 SONOMA HIGHWAY
 Sonoma, California

FIGURE
 2

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APPENDIX A
Field Exploration

KEY TO EXPLORATORY BORING LOGS

PROJECT:

19320 SONOMA HIGHWAY
Sonoma, California

PROJECT NO: **155-108**

FIGURE NO: **A-1**

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS		GRAPHIC LOG	GROUP SYMBOL	DESCRIPTION	MAJOR DIVISIONS	GRAPHIC LOG	GROUP SYMBOL	DESCRIPTION	
COARSE-GRAINED SOILS (More than 50% of material is larger than #200 sieve)	CLEAN GRAVELS (Less than 5% fines)		GW	Well-graded gravels or gravel-sand mixtures, little or no fines	FINE-GRAINED SOILS (More than 50% of material is smaller than #200 sieve)	SILTS AND CLAYS (Liquid Limit less than 50%)	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts of low to medium plasticity	
			GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	
	GRAVELS WITH FINES (More than 12% fines)		GM	Silty gravels or gravel-sand-silt mixtures			OL	Organic silts and clays of low plasticity	
			GC	Clayey gravels or gravel-sand-clay mixtures			SILTS AND CLAYS (Liquid Limit 50% or greater)	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts of high plasticity
	CLEAN SANDS (Less than 5% fines)		SW	Well-graded sands or gravelly sands, little or no fines		CH		Inorganic clays of high plasticity, fat clays	
			SP	Poorly-graded sands or gravelly sands, little or no fines		OH		Organic silts and clays of medium to high plasticity	
	SANDS WITH FINES (More than 12% fines)		SM	Silty sands or sand-silt mixtures		HIGHLY ORGANIC SOILS		PT	Peat and other highly organic soils
			SC	Clayey sands or sand-clay mixtures					

GRAIN SIZES

U.S. STANDARD SERIES SIEVE

CLEAR SQUARE SIEVE OPENINGS

#200

#40

#10

#4

3/4"

3"

12"

SILTS AND CLAYS	SANDS			GRAVELS		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*	UCS (KSF)**
Very Soft	0 - 2	0 - 1/2
Soft	2 - 4	1/2 - 1
Firm	4 - 8	1 - 2
Stiff	8 - 16	2 - 4
Very Stiff	16 - 32	4 - 8
Hard	Over 32	Over 8

*Number of blows for a 140-pound hammer falling 30 inches to drive a 2" O.D. (1-3/8" I.D.) split spoon sampler.

**Unconfined Compressive Strength.

SYMBOLS AND NOTES

Standard Penetration Test Sampler (2" O.D. Split Barrel)

Shelby Tube

Groundwater Level During Drilling

Modified California Sampler (3" O.D. Split Barrel)

Pitcher Barrel

Groundwater Level at End of Drilling

California Sampler (2.5" O.D. Split Barrel)

HQ Core

INCREASING VISUAL MOISTURE CONTENT

↑ Saturated
Wet
Moist
Damp
Dry

CONSTITUENT PERCENTAGE

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



1600 Willow Pass Court
 Concord, CA 94520
 Tel: (925) 688-1001

EXPLORATORY BORING B-1

PROJECT NO: 155-108	SURFACE ELEVATION: --
LOGGED BY: R. Ceraolo	DATE STARTED: 10/19/22
DRILL RIG: Mobile B-24	DATE FINISHED: 10/19/22
DRILLING METHOD: 4-inch Solid Flight Auger	DEPTH TO INITIAL WATER: Not Encountered
HAMMER METHOD: Rope and Cathead	DEPTH TO FINAL WATER: Not Encountered
HAMMER WEIGHT / DROP: 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 2 (38.297728°, -122.475216°)	

PROJECT:
19320 SONOMA HIGHWAY
 Sonoma, CA

DESCRIPTION AND CLASSIFICATION			DEPTH (FEET)	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS AND NOTES
DESCRIPTION AND REMARKS	CONSIST	GRAPHIC LOG							
SAND (SM), light brown, fine- to medium-grained, some coarse-grained, silty, dry. Clayey at 2 feet	loose		0		43	11.6	99.1	7.6	At 2 Feet: Corrosion Tests
Change color to pale brown, lightly cemented, trace gravel (fine to coarse, subangular to subrounded).	dense		50/6"						
Change color to light brown.	very dense		30/6"						
Change color to mottled gray brown, dry.			50/6"						
Drilling refusal at 14 feet.			15		62				
CLAY (CL)/SAND (SC), light grayish brown, sandy (fine- to medium-grained), dry.	hard								
Bottom of Boring = 15.5 feet Notes: Stratification is approximate, variations must be expected. Blow counts converted to SPT N-values. See report for additional details.									

EXPLORATORY BORING LOG 155-108 B-1.dwg STEVENS FERRONE & BAILEY 11/10/2022



1600 Willow Pass Court
 Concord, CA 94520
 Tel: (925) 688-1001

EXPLORATORY BORING B-2

PROJECT NO: 155-108	SURFACE ELEVATION: --
LOGGED BY: R. Ceraolo	DATE STARTED: 10/19/22
DRILL RIG: Mobile B-24	DATE FINISHED: 10/19/22
DRILLING METHOD: 4-inch Solid Flight Auger	DEPTH TO INITIAL WATER: Not Encountered
HAMMER METHOD: Rope and Cathead	DEPTH TO FINAL WATER: Not Encountered
HAMMER WEIGHT / DROP: 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 2 (38.297661°, -122.474502°)	

PROJECT:
19320 SONOMA HIGHWAY
 Sonoma, CA

DESCRIPTION AND CLASSIFICATION			DEPTH (FEET)	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS AND NOTES
DESCRIPTION AND REMARKS	CONSIST	GRAPHIC LOG							
SAND (SM), yellowish brown, fine- to coarse-grained, silty, with gravel (fine to coarse, subangular to subrounded), some clay, trace roots, dry.	loose medium dense		0		28	15.0	80.6	2.0	
SAND (SC), pale brown, fine- to coarse-grained, with clay, with gravel (fine, subangular), dry.	very dense				70				
SAND (SM), light brown, fine- to medium-grained, some coarse-grained, some gravel (fine, subangular), lightly cemented, dry.	very dense		5		30/2" 50/3"				
CLAY (CL), mottled grayish brown, sandy (fine- to medium-grained), trace gravel (fine, subangular), damp. Damp to moist.	very stiff								
	hard		10		33				
	very stiff		15		18				
Change color to brown, dry.	hard		20		30/6" 50/6"				
Bottom of Boring = 21 feet Notes: Stratification is approximate, variations must be expected. Blow counts converted to SPT N-values. See report for additional details.			25						
			30						

EXPLORATORY BORING LOG 155-108 B-2.dwg STEVENS FERRONE & BAILEY 11/10/2022



1600 Willow Pass Court
 Concord, CA 94520
 Tel: (925) 688-1001

EXPLORATORY BORING B-3

PROJECT NO: 155-108	SURFACE ELEVATION: --
LOGGED BY: R. Ceraolo	DATE STARTED: 10/19/22
DRILL RIG: Mobile B-24	DATE FINISHED: 10/19/22
DRILLING METHOD: 4-inch Solid Flight Auger	DEPTH TO INITIAL WATER: Not Encountered
HAMMER METHOD: Rope and Cathead	DEPTH TO FINAL WATER: Not Encountered
HAMMER WEIGHT / DROP: 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 2 (38.297374°, -122.474706°)	

PROJECT:
19320 SONOMA HIGHWAY
 Sonoma, CA

DESCRIPTION AND CLASSIFICATION			DEPTH (FEET)	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS AND NOTES
DESCRIPTION AND REMARKS	CONSIST	GRAPHIC LOG							
SAND (SM), brown, fine- to coarse-grained, with to silty, some gravel (fine, subangular to subrounded), some clay, dry.	loose medium dense		0		18	14.4	83.8	2.1	At 2 Feet: Liquid Limit = 33 Plasticity Index = 12 Coarse Sand = 2% Medium Sand = 15% Fine Sand = 22% Fines = 61% At 3.5 Feet: Corrosion Tests
SAND (SC)/CLAY (CL), mottled white brown, fine- to medium-grained, trace coarse-grained, clayey, lightly cemented, dry.	medium dense dense				40				
GRAVEL (GP-GM), mottled gray light brown, fine to coarse, angular to subrounded, sandy (fine- to coarse-grained), some silt, dry.	very dense		5		30/6"				
SAND (SM), light brown, fine- to medium-grained, with to silty, dry.	very dense		10		50/5"				
Bottom of Boring = 10.4 feet Notes: Stratification is approximate, variations must be expected. Blow counts converted to SPT N-values. See report for additional details.									
			15						
			20						
			25						
			30						

EXPLORATORY BORING LOG 155-108 B-3.dwg STEVENS FERRONE & BAILEY 11/10/2022



1600 Willow Pass Court
 Concord, CA 94520
 Tel: (925) 688-1001

EXPLORATORY BORING B-4

PROJECT NO: 155-108	SURFACE ELEVATION: --
LOGGED BY: R. Ceraolo	DATE STARTED: 10/19/22
DRILL RIG: Mobile B-24	DATE FINISHED: 10/19/22
DRILLING METHOD: 4-inch Solid Flight Auger	DEPTH TO INITIAL WATER: Not Encountered
HAMMER METHOD: Rope and Cathead	DEPTH TO FINAL WATER: Not Encountered
HAMMER WEIGHT / DROP: 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 2 (38.297333°, -122.474029°)	

PROJECT:
19320 SONOMA HIGHWAY
 Sonoma, CA

DESCRIPTION AND CLASSIFICATION			DEPTH (FEET)	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS AND NOTES
DESCRIPTION AND REMARKS	CONSIST	GRAPHIC LOG							
SAND (SM), mottled red gray brown, fine- to coarse-grained, with silt and clay, some gravel (fine, subrounded to subangular), dry.	loose		0		30/3"				
	very dense		60/4"						
SAND (SM), gray, fine-grained, silty, trace gravel (fine to coarse, subangular), dry. Drilling refusal at 4 feet.	very dense		5		50/3"				
Bottom of Boring = 4.3 feet Notes: Stratification is approximate, variations must be expected. Blow counts converted to SPT N-values. See report for additional details.			30						

EXPLORATORY BORING B-5

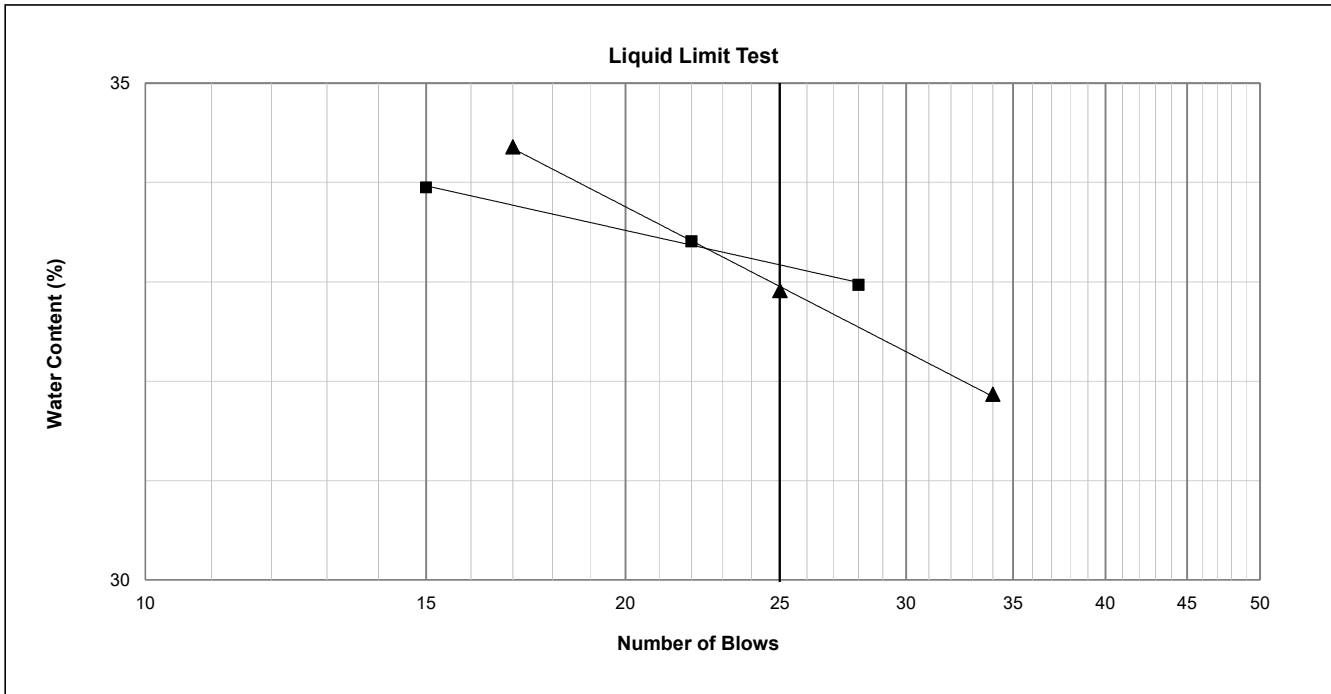
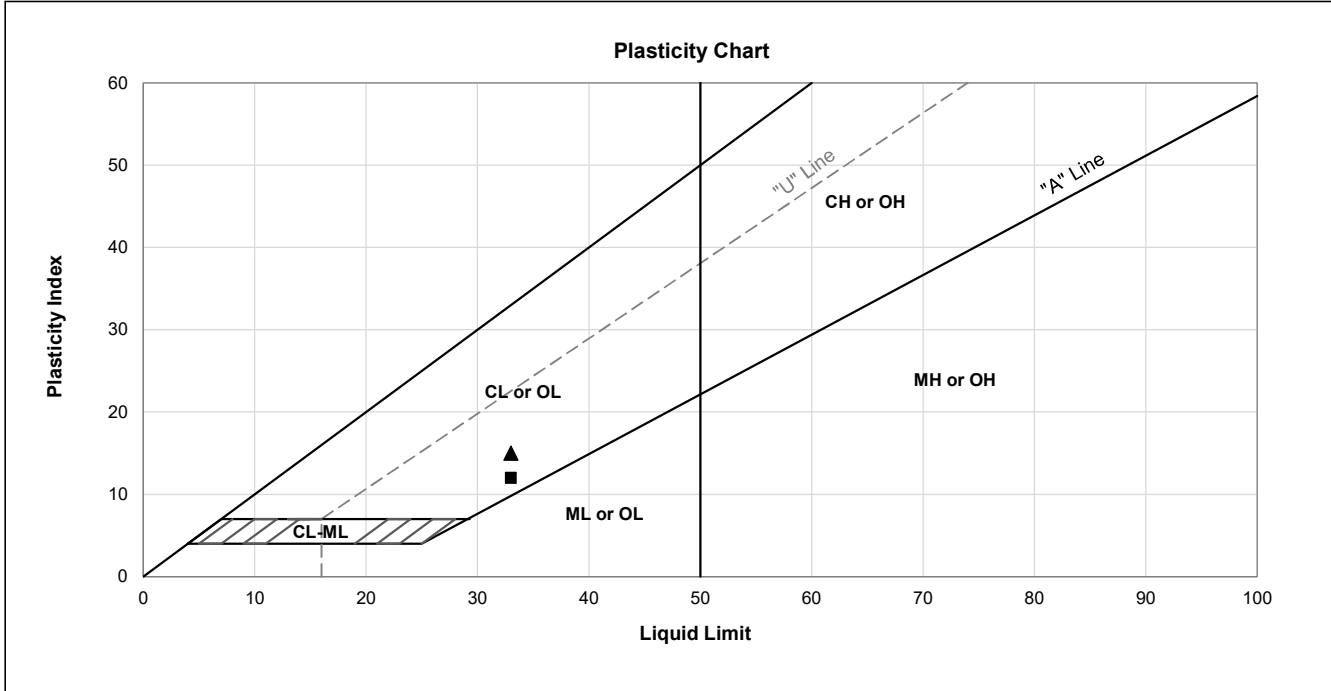
PROJECT NO: 155-108	SURFACE ELEVATION: --
LOGGED BY: R. Ceraolo	DATE STARTED: 10/19/22
DRILL RIG: Mobile B-24	DATE FINISHED: 10/19/22
DRILLING METHOD: 4-inch Solid Flight Auger	DEPTH TO INITIAL WATER: Not Encountered
HAMMER METHOD: Rope and Cathead	DEPTH TO FINAL WATER: Not Encountered
HAMMER WEIGHT / DROP: 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 2 (38.297636°, -122.473699°)	

PROJECT:
19320 SONOMA HIGHWAY
Sonoma, CA

DESCRIPTION AND CLASSIFICATION			DEPTH (FEET)	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS AND NOTES
DESCRIPTION AND REMARKS	CONSIST	GRAPHIC LOG							
SAND (SC)/CLAY (CL), mottled gray, fine- to coarse-grained, clayey, some gravel (fine, subrounded to subangular), dry.	loose		0		32	10.5			At 2 Feet: Liquid Limit = 33 Plasticity Index = 15 Fine Gravel = 8% Coarse Sand = 7% Medium Sand = 11% Fine Sand = 19% Fines = 55%
	dense		46						
SAND (SM), light brown, fine- to medium-grained, some coarse-grained, with to silty, trace gravel (fine, subangular), dry.	very dense		5		50/6"				
CLAY (CL), dark brown, gravelly (fine to coarse, angular to subangular), with sand (fine- to coarse-grained), with silt, dry. Drilling refusal at 7 feet.	hard				50/5"				
Bottom of Boring = 7.4 feet Notes: Stratification is approximate, variations must be expected. Blow counts converted to SPT N-values. See report for additional details.			10						
			15						
			20						
			25						
			30						

APPENDIX B
Laboratory Testing

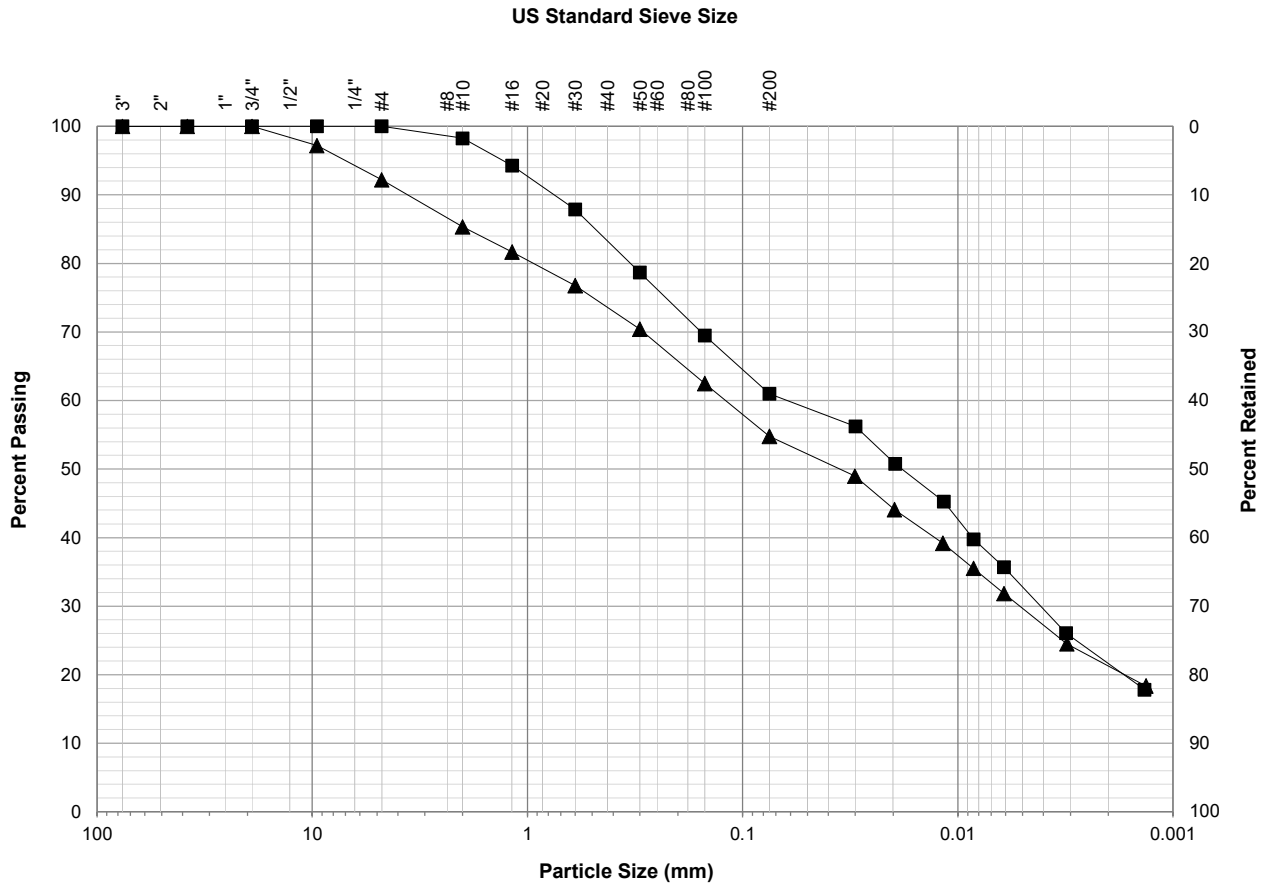
ATTERBERG LIMITS
 ASTM D4318



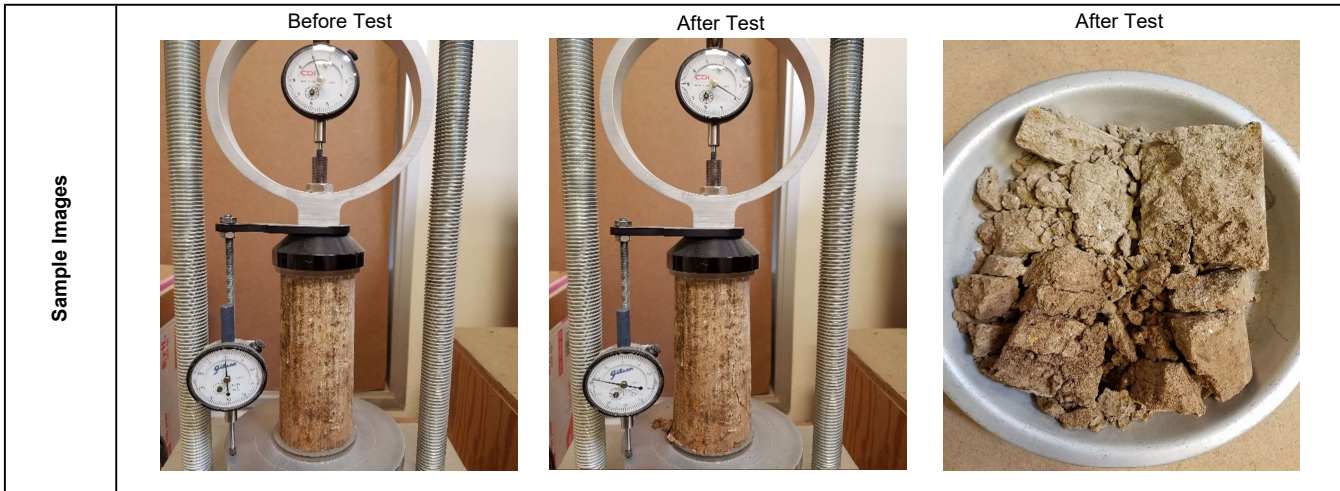
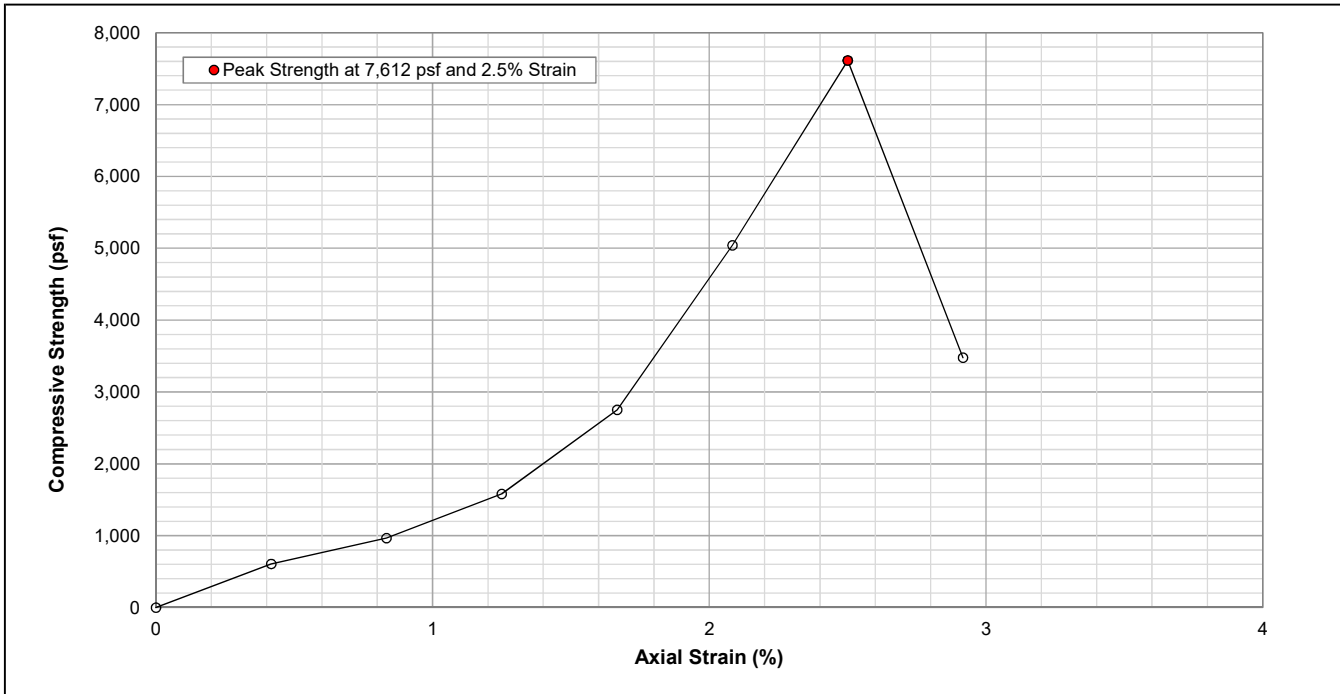
Test Results	Symbol	Sample Source	Sample Description	LL	PL	PI	% _{≤ #200}	USCS
	■	Boring B-3 at 2 ft	Red brown sandy silty CLAY	33	21	12	61.1	CL/ML
▲	Boring B-5 at 2 ft	Brown sandy silty CLAY some gravel	33	18	15	54.7	CL	
Project Info	Project Number:	155-108		Test Report Date:		10/31/2022		
	Project Name:	19320 Sonoma Highway						
	Project Location:	Sonoma, CA						
	Tested by:	R. Tuazon						
	Checked by:	T. Chen						

PARTICLE SIZE DISTRIBUTION

ASTM C136, D422 & D1140



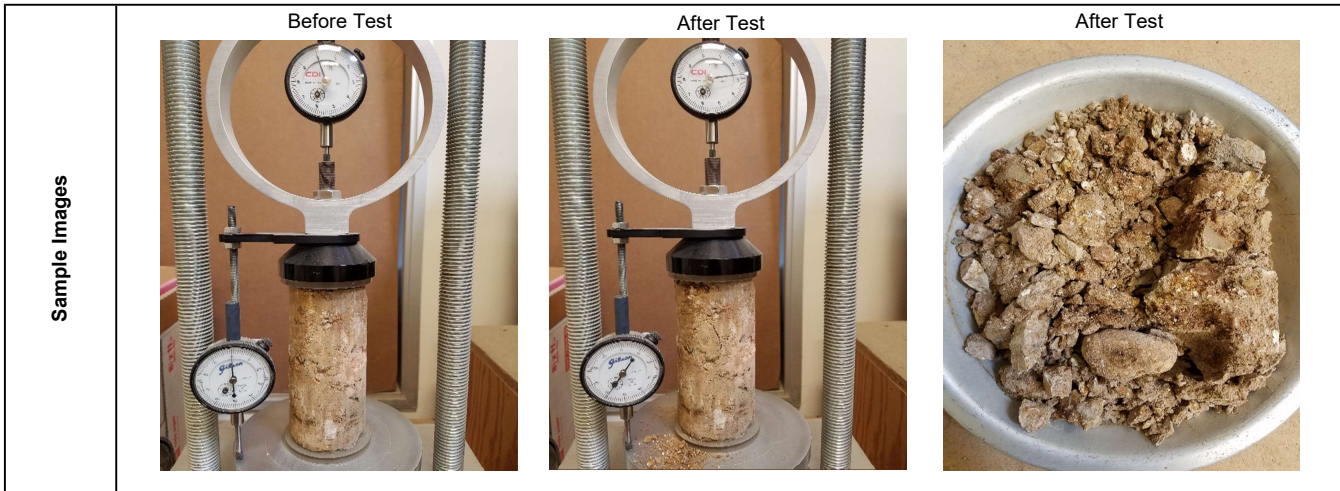
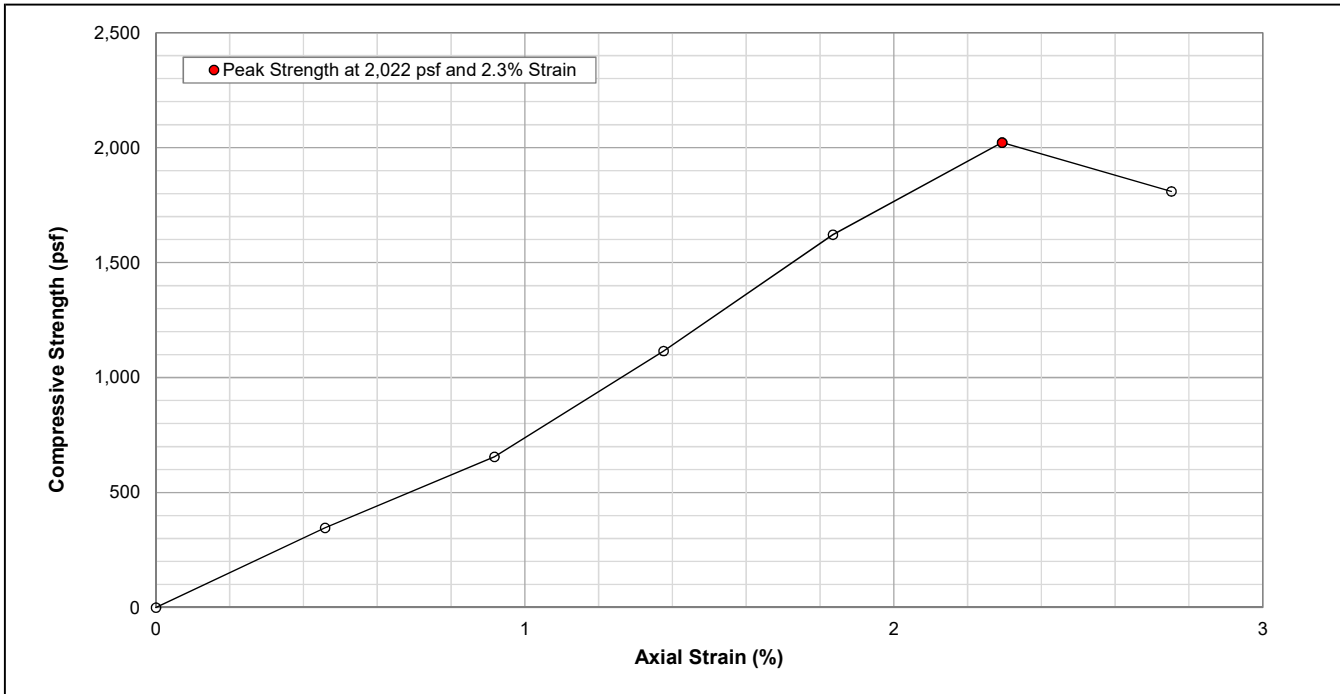
UNCONFINED COMPRESSIVE STRENGTH
 ASTM D2166



Sample Properties	Sample Description:	Light brown silty CLAY some sand (CL)	Test Results	Test Date:	10/28/2022
	Diameter:	2.42 in		Compressive Strength:	7,612 psf
	Height:	6 in		Axial Strain at Failure:	2.5 %
	Height/Diameter:	2.48		Test Strain Rate:	0.05 in/min
	Wet Unit Weight:	110.6 pcf		Test Time to Failure:	3 min
	Water Content:	11.6 %		Remarks:	
	Dry Unit Weight:	99.1 pcf			

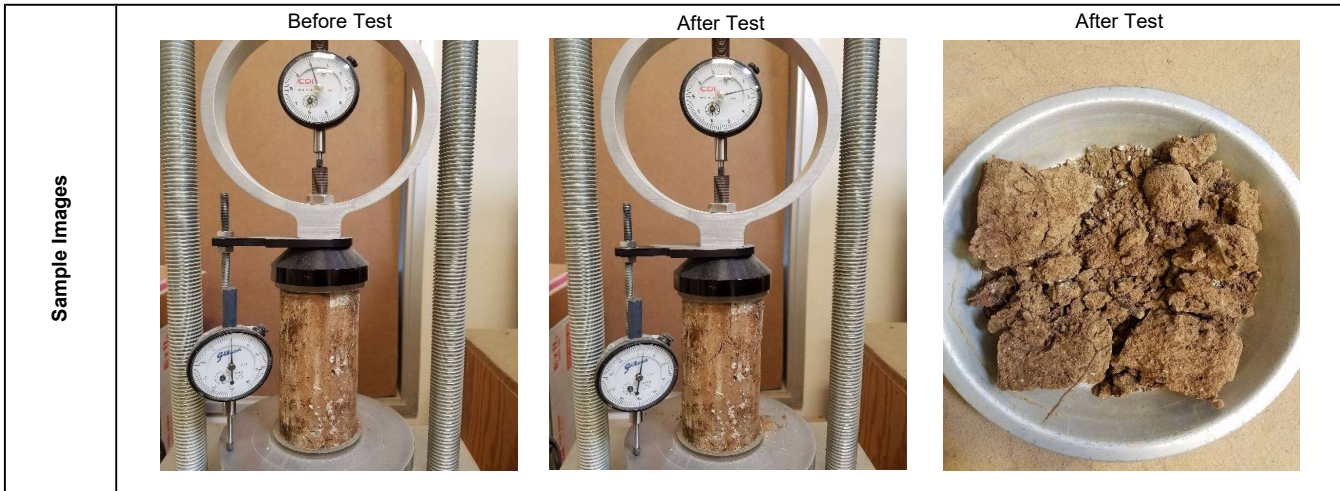
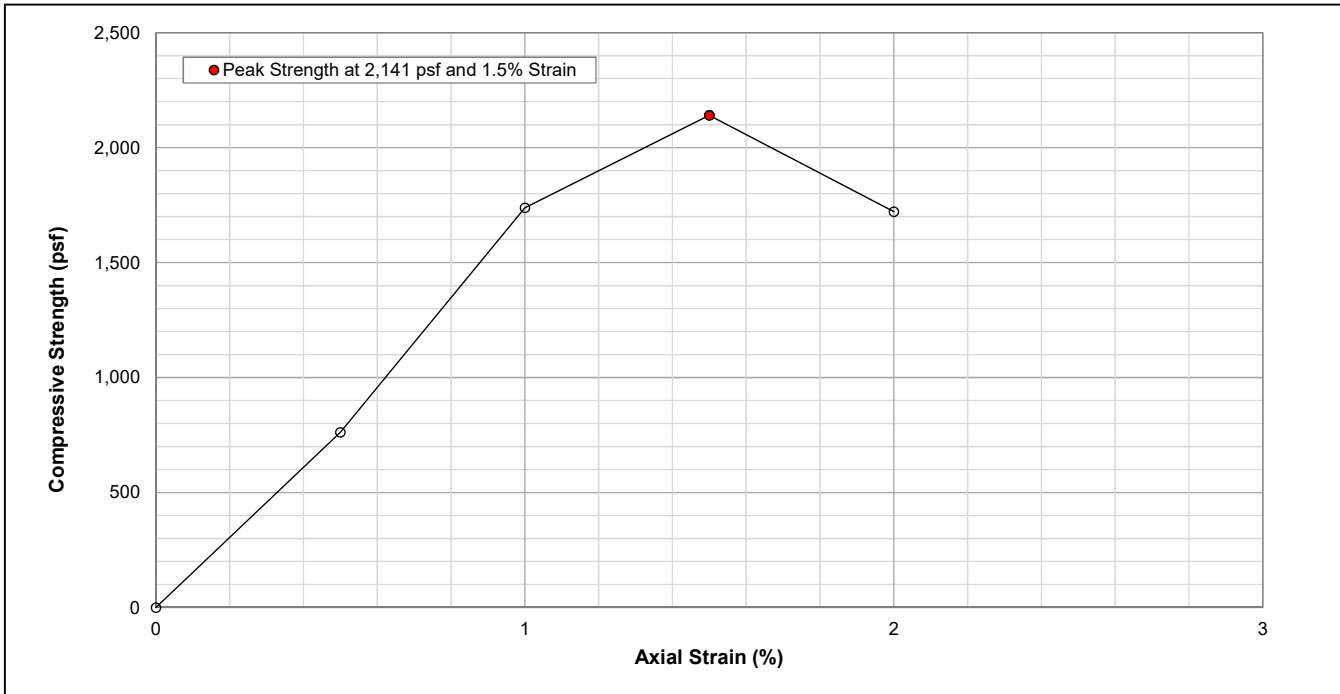
Project Info	Project Number:	155-108	Test Report Date:	10/28/2022
	Project Name:	19320 Sonoma Highway		
	Project Location:	Sonoma, CA		
	Sample Source/No.:	Boring B-1		
	Sample Depth:	2 ft		
	Tested by:	R. Tuazon		
	Checked by:	T. Chen		

UNCONFINED COMPRESSIVE STRENGTH
 ASTM D2166



Sample Properties	Sample Description:	Light red brown sandy SILT some clay (ML/SM)	Test Results	Test Date:	10/28/2022
	Diameter:	2.42 in		Compressive Strength:	2,022 psf
	Height:	5.45 in		Axial Strain at Failure:	2.3 %
	Height/Diameter:	2.25		Test Strain Rate:	0.05 in/min
	Wet Unit Weight:	92.7 pcf		Test Time to Failure:	2.5 min
	Water Content:	15.0 %		Remarks:	
	Dry Unit Weight:	80.6 pcf			
Project Info	Project Number:	155-108	Test Report Date: 10/28/2022		
	Project Name:	19320 Sonoma Highway			
	Project Location:	Sonoma, CA			
	Sample Source/No.:	Boring B-2			
	Sample Depth:	2 ft			
	Tested by:	R. Tuazon			
	Checked by:	T. Chen			

UNCONFINED COMPRESSIVE STRENGTH
 ASTM D2166



Sample Properties	Sample Description:	Red brown sandy silty CLAY (CL/ML)	Test Results	Test Date:	10/28/2022
	Diameter:	2.42 in		Compressive Strength:	2,141 psf
	Height:	5 in		Axial Strain at Failure:	1.5 %
	Height/Diameter:	2.07		Test Strain Rate:	0.05 in/min
	Wet Unit Weight:	95.9 pcf		Test Time to Failure:	1.5 min
	Water Content:	14.4 %		Remarks:	
	Dry Unit Weight:	83.8 pcf			
Project Info	Project Number:	155-108	Test Report Date: 10/28/2022		
	Project Name:	19320 Sonoma Highway			
	Project Location:	Sonoma, CA			
	Sample Source/No.:	Boring B-3			
	Sample Depth:	2 ft			
	Tested by:	R. Tuazon			
	Checked by:	T. Chen			



8 November, 2022

Job No. 2210042
Cust. No. 11486

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

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

Subject: Project No.: 155-108
Project Name: 19320 Sonoma Highway, Sonoma, CA
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Chen:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on October 21, 2022. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, both samples are classified as “mildly 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 reflect none detected with a reporting limit of 15 mg/kg.

The sulfate ion concentrations reflect none detected with a reporting limit of 15 mg/kg.

The sulfide ion concentrations reflect none detected with a reporting limit of 50 mg/kg.

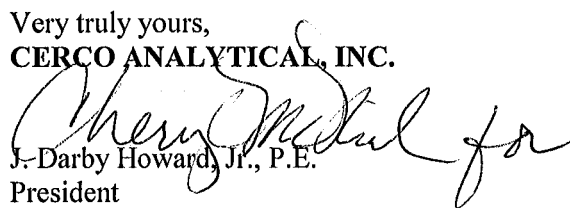
The pH of the soils are 7.03 and 7.30, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potential for both samples is 360-mV, and are indicative of potentially “slightly corrosive” soils resulting from anaerobic 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

Client: Stevens, Ferrone & Bailey Engineering
 Client's Project No.: 155-108
 Client's Project Name: 19320 Sonoma Highway, Sonoma, CA
 Date Sampled: 19-Oct-22
 Date Received: 21-Oct-22
 Matrix: Soil
 Authorization: Signed Chain of Custody

Date of Report: 8-Nov-2022

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)*
2210042-001	B-1 @ 2'	360	7.30	-	11,000	N.D.	N.D.	N.D.
2210042-002	B-3 @ 3.5'	360	7.03	-	9,600	N.D.	N.D.	N.D.

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	27-Oct-2022	28-Oct-2022	-	1-Nov-2022	8-Nov-2022	27-Oct-2022	27-Oct-2022

Sherry Moore
 Sherry Moore
 Chemist

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

APPENDIX C
GBA Guidelines for Geotechnical Report

Important Information about This

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.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer

will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it.* A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept*

responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.

Most of the “Findings” Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site’s subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report’s Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are not final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals’ misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals’ plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note*

conspicuously that you’ve included the material for information purposes only. To avoid misunderstanding, you may also want to note that “informational purposes” means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include 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 personnel, equipment, and techniques used to perform an environmental study – e.g., a “phase-one” or “phase-two” environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer’s services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, *proper implementation of the geotechnical engineer’s recommendations will not of itself be sufficient to prevent moisture infiltration.* **Confront the risk of moisture infiltration** by including building-envelope or mold specialists on the design team. **Geotechnical engineers are not building-envelope or mold specialists.**



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