Stevens Ferrone Bailey Engineering

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

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GEOTECHNICAL INVESTIGATION
19910 5TH STREET WEST
SONOMA, CALIFORNIA
SFB PROJECT NO. 155-107

Prepared For:

DeNova Homes
1500 Willow Pass Court
Concord, CA 94520

Prepared By:

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1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed residential development to be located at 19910 5th Street West in Sonoma, California as shown on the Site Plan, Figure 1. The purpose of our investigation was to evaluate the geotechnical conditions at the site and provide recommendations regarding the geotechnical engineering aspects of the project.

Based on the information indicated on the Site Plan, as well as information provided by DeNova Homes, it is our understanding that the project will consist of developing approximately 1.5 acres of land for a new residential development that includes approximately 15 single-family, detached homes. The homes will likely be wood-framed and maximum two-stories in height. Nominal grading is anticipated. Associated underground utilities and roadways will be constructed. The existing structures and facilities at the site will be demolished prior to new construction.

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

2.0 SCOPE OF WORK

Our investigation of the site 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- Reviewing published and unpublished geotechnical and geological literature relevant to the site;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 a reconnaissance of the site and surrounding area;
- Performing a subsurface exploration program to log and sample three exploratory borings to a maximum depth of about 16-1/2 feet;
- Performing laboratory testing of samples retrieved from the borings;
- Performing engineering analysis of field and laboratory data; and
- Preparing this report.

The data obtained and the analyses performed were for the purpose of providing geotechnical design and construction criteria for site earthwork, underground utilities, surface and subsurface drainage, building foundations, retaining walls and soundwalls, flatwork, and pavements. Evaluating the potential for toxicity of onsite materials or groundwater (including mold) and flooding were beyond our scope of work.

3.0 SITE INVESTIGATION

Reconnaissance of the site and surrounding area was performed on March 16 and April 6, 2021. A subsurface exploration program was performed on April 6, 2021 using a truck-mounted drill rig equipped with 4.5-inch diameter, continuous flight, solid stem augers. Three exploratory borings were drilled to a maximum depth of about 16-1/2 feet below existing grade. The approximate locations of the borings are shown on the Site Plan, Figure 1. Logs of the borings and details regarding our field investigation are included in Appendix A. The results of our laboratory tests are discussed in Appendix B. It should be noted that changes in the surface and subsurface conditions can occur over time as a result of either natural processes or human activity and may affect the validity of the conclusions and recommendations in this report.

3.1 Site History and Surface Description

At the time of our investigation and as shown on Figure 1, the site was bounded by 5th Street West on the west, West Macarthur Street on the north, and residential developments on the east and south. Roadway improvements were observed along West Macarthur Street. The site was relatively level. A vacant residential structure and shed were located in the north-central portion of the site. Wood fencing bounded the site on the east and south side. A drainage ditch was observed within the site along the east side. A storm drain culvert was observed in the southeast corner of the site. Vegetation consisted of a heavy growth of grasses and weeds along with small and large diameter trees and shrubs.

Based on our review of historical aerial photographs and topographic maps of the site and vicinity, it is our understanding that the site was previously used as an orchard. It is possible the root systems of the orchard remain below the existing grade which will require removal if encountered during grading and excavating. Over-excavation of tree root systems may extend about 3 to 4 feet below grade. From the air photo images, the existing vacant residence was built in about 1949.

3.2 Subsurface Description

Our borings encountered very stiff, silty clays with sand to depths of about 2-1/2 to 3 feet. Below this surficial clay layer, we encountered dense to very dense sands and gravels to depths of about 12 to 13 feet. Below the sands and gravels, the borings encountered hard, silty clays to the maximum depth explored of about 16-1/2 feet.

According to the results of our laboratory testing, the near-surface more clayey soils have a medium plasticity and moderate expansion potential. Detailed descriptions of the soils encountered in our exploratory borings are presented on the boring logs in Appendix A. Our attached boring logs and related information depict location-specific subsurface conditions encountered during our field investigation. The approximate locations of our borings were determined using pacing, measurements, and/or alignment from landmark references, and should be considered accurate only to the degree implied by the method used.

3.3 Groundwater

Groundwater was measured in our borings at depths of about 8 to 15 feet at the end of drilling. Our borings were backfilled with lean cement grout in accordance with Sonoma County requirements prior to leaving the site. It should be noted that the borings might not have been left open for a sufficient period of time to establish equilibrium groundwater conditions. In addition, fluctuations in the groundwater level can occur due to seasonal changes, including variations in rainfall, and other factors.

3.4 Hydrologic Soil Group

The surface soils at the site have been mapped as Huichica loam (2 to 6 percent slopes) in the western half of the site and Tuscan cobbly clay loam (0 to 9 percent slopes) in the eastern half of the site by the USDA Web Soil Survey (WSS).[1] These soils were assigned to Hydrologic Soil Group C and D, respectively, by the USDA Natural Resources Conservation Service (NRCS) and have been categorized as having very low to moderately low rates of water transmission (0.00 to 0.06 inches per hour).

Based on the soils encountered in our borings and the results of our laboratory testing, we recommend the near-surface soils be categorized as Hydrologic Soil Group C. Group C soils are defined as having a slow infiltration rate when thoroughly wet (high runoff potential) and may consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine to fine texture.

3.5 Geology and Seismicity

According to Sowers, et al, (1998)[2], the site (below surficial fills) is underlain by early or middle Pleistocene fan or terrace deposits that were described as moderately to deeply dissected alluvial deposits capped by alfisols, ultisols, or soils containing a silica or calcic hardpan. Wagner and Gutierrez (2010)[3] mapped the site as being underlain by early to late Pleistocene alluvial fan deposits that that composed of deeply dissected sand, gravel, silt, and clay.

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[1]USDA NRCS, https://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx, accessed 04/01/2021.

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

[3]Wagner & Gutierrez, 2010, *Geologic Map of the Napa 30' X 60' Quadrangle, California,* CGS.

The project site is located in the San Francisco Bay Area that is considered one of the most seismically active regions in the United States. Significant earthquakes have occurred in the San Francisco Bay Area and are believed to be associated with crustal movements along a system of sub-parallel fault zones that generally trend in a northwesterly direction. The site is not located within an Alquist-Priolo Earthquake Fault Zone as designated by the State of California[4].

According to U.S. Geological Survey Open-File Report 97-745 (San Francisco Bay Area Landslide Folio), the site is mapped as flat land with no potential for landslides or earth flows and is not located within an area having debris flow source potential. It is our opinion that the potential for landslides, earth flows, or debris flows to develop at the site is very low, especially given the relatively level topography of the site and surrounding areas.

Earthquake intensities will vary throughout the San Francisco Bay Area depending upon numerous factors, including the magnitude of earthquake, the distance of the site from the causative fault, and the type of materials underlying the site. The U.S. Geological Survey (2016)[5] has stated that there is a 72 percent chance of at least one magnitude 6.7 or greater earthquake striking the San Francisco Bay Region between 2014 and 2043. Therefore, the site will be subjected to strong ground shaking as is common for developments throughout the San Francisco Bay Area.

According to the U.S. Geological Survey's Unified Hazard Tool and applying the Dynamic: Conterminous U.S. 2014 model (v4.2.0, accessed 04/14/2021), 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 (design basis ground motion based on a dense soil site condition; mean return time of 475 years). The actual ground surface acceleration might vary depending upon the local seismic characteristics of the underlying bedrock and overlying unconsolidated soils.

3.6 Liquefaction

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

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[4]Hart and Bryant, *Fault-Rupture Hazard Zones in California*, CDMG Special Publication 42, Interim Revision 2007.
[5]Aagaard, Blair, Boatwright, Garcia, Harris, Michael, Schwartz, and DiLeo, *Earthquake Outlook for the San Francisco Bay Region 2014–2043,* USGS Fact Sheet 2016-3020, Revised August 2016 (ver. 1.1).

characterized as having very low to low liquefaction susceptibility[6,7]. Sower, et al, (1998) also mapped the site as having very low liquefaction susceptibility[8]. As of the date of this report, the liquefaction potential of the site has not been evaluated by the State of California[9].

Based on our review of available literature and the results of exploratory borings at the site, it is our opinion that the potential for ground surface damage caused by liquefaction at the site is very low.

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[6]Witter, Knudsen, Sowers, Wentworth, Koehler, and Randolph, 2006, *Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, California*", USGS Open File Report 2006-1037.
[7]Knudsen, Sowers, Witter, Wentworth, and Helly, 2000, "*Preliminary Maps of Quaternary Deposits and Liquefaction Susceptibility, Nine-County San Francisco Bay Region, California*", USGS Open File Report 00-444.
[8]Sowers, Noller, and Lettis, 1998, "*Liquefaction Susceptibility Map, Napa, California, 1: 100,000 Quadrangle: A Digital Database"*, USGS Open File Report 98-460.
[9]Seismic Hazards Mapping Act, 1990.

4.0 CONCLUSIONS AND RECOMMENDATIONS

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

SURFACE SOILS: The removal of the existing structures, existing improvements, and the tree roots (including former orchard tree roots) at the site will likely result in loosening and weakening of the surface soils in the upper 2 to 4 feet. In addition, the clayey surficial soils are relatively dry and will expand and heave causing damage to overlying improvements if exposed to water such as stormwater and irrigation water.

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

CORROSION POTENTIAL: Two onsite soil samples were tested for pH (ASTM D4972), chlorides (ASTM D4327), sulfates (ASTM D4327), sulfides (ASTM D4658M), resistivity at 100% saturation (ASTM G57), and Redox potential (ASTM D1498) for use in evaluating the potential for corrosion on concrete and buried metal, such as utilities and reinforcing steel. The results of these tests and a brief summary of the results are included under separate cover. We recommend these test results and brief summary be forwarded to your 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 drainage, earthwork, foundation, retaining walls, exterior slabs, and pavement recommendations for use in design and construction of the project are presented below. We recommend SFB review the design and specifications to verify that the recommendations presented in this report have been properly interpreted and implemented in the design, plans, and specifications. We also recommend SFB be retained to provide consulting services and to perform construction observation and testing services during the construction phase of the project to observe and test the implementation of our recommendations, and to provide supplemental or revised recommendations in the event conditions different than those described in this report are encountered. We assume no responsibility for misinterpretation of our recommendations.

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 and their underlying baserock, gravel, designated trees and landscaping and their associated root systems, and debris. Holes resulting from the removal of underground obstructions extending below the proposed finish grade should be cleared and backfilled with fill materials as specified in **Section 4.1.4, *Fill Material***, and compacted to the requirements in **Section 4.1.5, *Compaction***. The site was previously used as an orchard and tree roots may remain requiring removal. Tree roots may extend to depths of about 3 to 4 feet.

From a geotechnical standpoint, any existing trench backfill materials, clay or concrete pipes, gravel, pavements, and concrete that are removed can be used as new fill onsite provided debris is removed and it is broken up to meet the size requirement for fill material in **Section 4.1.4, *Fill Material***. We recommend fill materials composed of broken up concrete or asphalt concrete not be located within 2 feet of unpaved ground surfaces. Consideration should be given to placing these materials below pavements or in deeper excavations. We recommend backfilling operations for any excavations be performed under the observation and testing of SFB.

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 or stripped. Portions of the site containing heavy surface vegetation that is not removed by discing should be stripped to an appropriate depth to remove these materials. The amount of actual stripping should be determined in the field by SFB at the time of construction. Stripped materials should be removed from the site or stockpiled for later use in landscaping, if desired.

4.1.2 Weak Soil Re-Compaction

We estimate that the removal of the existing structures, existing improvements, and tree root systems will likely result in weakening and loosening of the surface soils in the upper 2 to 4 feet. We recommend these weakened soils be over-excavated and re-compacted. We estimate that the process can consist of over-excavating 2 feet, scarifying and re-compacting the bottom 12 inches in-place, and replacing the excavation with compacted fill materials. There would be no need to over-excavate the soils within areas that do not support improvements, such as within planned open spaces that do not support improvements. The over-excavation and re-compaction should also extend at least 5 feet beyond building footprints and at least 3 feet beyond exterior flatwork (including driveways) and pavement wherever possible.

Where the over-excavation limits abut adjacent property, SFB should be consulted to determine the actual vertical and lateral extent of over-excavation so that adjacent property is not adversely impacted. Over-excavations should be performed so that no more than 5 feet of differential fill thickness exists below the proposed building foundations. The removed soil materials can be used as new fill provided it is placed and compacted in accordance with the recommendations presented in this report. The extent of the removal and re-compaction will vary across the site and should be determined in the field by SFB at the time of the earthwork operations.

Removed soil materials may be used as new fill onsite provided it satisfies the recommendations provided in **Section 4.1.4, *Fill Material***. Compaction should be performed in accordance with the recommendations in **Section 4.1.5, *Compaction***.

4.1.3 Subgrade Preparation

After the completion of clearing, site preparation, excavation, and weak soil/fill removal, soils exposed in areas to receive improvements (such as new fill, building foundations, exterior flatwork, driveways, and pavements) should be scarified to a depth of about 12 inches, moisture conditioned to approximately 3 to 5 percent over optimum water content, and compacted to the requirements for structural fill. Subgrade preparation would not be necessary in areas where over-excavation and re-compaction of the surface soils has occurred.

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

4.1.4 Fill Material

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

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

4.1.5 Compaction

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

4.1.6 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 the driveway, exterior slab-on-grade, or under the building foundations, and are located below irrigated landscaped areas such as lawns or planting strips, should be plugged with onsite clays, low strength concrete or sand/cement slurry. The plug for the trench lateral should be located below the edge of pavement or 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.

4.1.7 Exterior Flatwork

We recommend that exterior slabs (including patios, sidewalks, and driveways) be placed directly on the properly compacted fills. We do not recommend using aggregate base, gravel, or crushed rock below these improvements. If imported granular materials are placed below these elements, subsurface water can seep through the granular materials and cause the underlying soils to saturate, pipe, and/or heave upward. Prior to placing concrete, subgrade soils should be moisture conditioned to increase their moisture content to approximately 3 to 5 percent above laboratory optimum moisture (ASTM D-1557).

The more expansive clayey soils at the site could be subjected to volume changes during seasonal fluctuations in moisture content. As a result of these volume changes, some vertical movement of exterior slabs (such as patios, sidewalks, concrete driveways, etc.) should be anticipated. This movement could result in damage to the exterior slabs and might require periodic maintenance or replacement. Adequate clearance should be provided between the exterior slabs and building elements that overhang these slabs, such as window sills or doors that open outward.

We recommend reinforcing exterior slabs with steel bars in lieu of wire mesh. To reduce potential crack formation, the installation of #4 bars spaced at approximately 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 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, or if soils/fills with high water contents are encountered, 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 construction recommendations, such as using lime to stabilize the soils/fills, 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 and infiltration of water must not be allowed on or adjacent to pavements (including landscaping strips) and foundations. Ponding of water should also not be allowed on the ground surface adjacent to or near exterior slabs, including driveways, walkways, and patios. Surface grades should be sloped so that water sheet flows away from these improvements or sheet flows onto impermeable surfaces that directs the water into appropriate collection systems.

We recommend positive surface gradients of at least 2 percent be maintained adjacent to the 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.

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

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

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

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

4.1.10 Storm Water Runoff Structures

To satisfy local and state permit requirements, most new development projects must control pollutant sources and reduce, detain, retain, and/or treat specified amounts of storm water runoff. The intent of these types of improvements is to conserve and incorporate onsite natural features, together with constructed hydrologic controls, to more closely mimic pre-development hydrology and watershed processes.

We recommend storm water collection improvements that are designed to detain, retain, and/or treat water such as bio-swales, porous pavement structures, and water detention basins, be lined with a relatively impermeable membrane in order to reduce water seepage and the potential for damage and distress to other infrastructure improvements (such as pavements, foundations, and walkways) which can occur as a result of volumetric soil/fill changes (heaving and shrinking of the surrounding soil/fill). We recommend a relatively impermeable membrane such as STEGO Wrap 15-mil or equivalent be installed below and along the sides of these facilities that direct the collected water into subdrain pipes. The membrane should be lapped and sealed in accordance with the manufacture’s specifications, including taping joints where pipes penetrate the membrane. A subdrain pipe should be used at the base of the infiltration materials to collect accumulated water and transmit the water to an appropriate facility.

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

Excavated trench walls and slopes of earthen swales and basins steeper than 2:1 (horizontal to vertical) will experience downward and lateral movements that can cause significant ground surface movements, including movement of adjacent improvements such as foundations, utilities, pavements, walkways, and curbs and gutters. The magnitude and rate of movement depends upon the swale and basin backfill material type and compaction. To reduce the potential for damaging movements, we recommend 2:1 sidewall and trench wall slopes be used for earthen swales and basins, sidewalks be setback at least 3 feet from the top of the slope, creep sensitive improvements (such as roadway curbs) be setback at least 5 feet from the top of the slopes, or the slopes/sidewalls be appropriately restrained using an engineered retaining system, such as deepened curbs and foundations that are designed to resist lateral earth pressures and act as a retaining wall.

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

4.1.11 Future Maintenance

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

Differential movement of exterior slabs can occur over time as a result of numerous factors. We recommend regular inspections and maintenance of slabs be performed, including infilling significant cracks, providing fillers at slab offsets, and replacing slab 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 provided to the lot owners and HOAs.

4.2 Foundation Support

4.2.1 Post-Tensioned Slab Foundations

We recommend the proposed residential buildings be supported on a post-tensioned slab foundation that is designed for the expansion potential of the onsite soils. The slab foundation should bear entirely on properly prepared, compacted structural fill. In no case should a slab foundation bear upon fills with differential expansion characteristics. Recommendations for building pad preparation are described previously in **Sections 4.1.2 and 4.1.3** of this report.

Prior to the concrete pour, we recommend the moisture content of the subgrade materials be approximately 3 to 5 percent above laboratory optimum moisture. If the building pads are left exposed for an extended period of time prior to constructing foundations, we recommend SFB be contacted for recommendations to re-condition the pads in order provide adequate building support.

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

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

A vapor retarder must be placed between the subgrade soils and the bottom of the slabs-on-grade. We recommend the vapor retarder consist of a single layer of Stego Wrap Vapor Barrier 15 mil 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 all joints should be lapped at least 6 inches and sealed with Stego Tape or equal in accordance with the manufacturer's specifications. Protrusions where pipes or conduit penetrate the membranes should be sealed with either one or a combination of Stego Tape, Stego Mastic, Stego Pipe Boots, or a product of equal quality as determined by the manufacturer's instructions and ASTM E 1643. Care must be taken to protect the membrane from tears and punctures during construction. We do not recommend placing sand or gravel over the membrane.

Concrete slabs retain moisture and often take many months to dry; construction water added during the concrete pour further increases the curing time. If the slabs are not allowed to completely cure prior to constructing the super-structure, the concrete slabs will expel water vapor and the vapor will be trapped under impermeable flooring. All concrete shrinks during curing which results in cracks. We recommend you consult with your foundation designers and contractors to prepare procedures and methods to control the cracking.

In order to reduce the potential for vapor transmission through the concrete slab, we recommend the concrete mix design for the slabs have a maximum water/cement ratio of 0.45. 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 homes. 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 and/or reinforcing steel. The results of sulfate and chloride testing of onsite soil samples are included under separate cover. We recommend you consult with your concrete slab designers and concrete contractors regarding methods to reduce the potential for differential concrete curing and cracking.

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 Post-Tensioning Institute (PTI) design manual and PTI published specifications, and the PTI preferred computer program VOLFLO was employed to simulate the wetting and drying scenarios of the soils beneath the post-tensioned slabs.

The values provided below are based upon the post-tensioned slab foundations being entirely surrounded by uniform, properly drained, moderately irrigated landscaping; if differing conditions will exist that will cause differential soil moisture adjacent or below the slabs, or if portions of the foundations will be located adjacent to relatively dry or wet soils, then we should be consulted and modifications to the values below would need to be modified in writing. Please refer to **Section 4.1.9, *Surface Drainage, Irrigation, and Landscaping***, for additional recommendations. We recommend the slab-subgrade friction values provided in the most recent PTI Manual be used in order to determine the friction that might be expected to exist during tendon stressing.

SWELLING MODE

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

4.2.2 Retaining Walls

Where walls retain soil, they must be designed to resist both lateral earth pressures and any additional lateral loads caused by surcharging such as building and roadway loads.

We recommend that unrestrained walls (walls free to deflect and disconnected from other structures) be designed to resist an equivalent fluid pressure of 40 pounds per cubic foot. This assumes a level backfill. Restrained walls (walls restrained from deflection) should be designed to resist an equivalent fluid pressure of 40 pounds per cubic foot plus a uniform pressure of 8H pounds per square foot, where H is the height of the wall in feet. These pressures are applicable for retaining walls that are all fully back-drained. In addition, these lateral pressures depend upon the moisture content of the retained soils to be constant over time; if the moisture content of the retained soils will fluctuate or increase compared to the moisture content at time of construction, then SFB should be consulted and provide written modifications to this design criteria.

Walls with inclined backfill should be designed for an additional equivalent fluid pressure of 1 pound per cubic foot for every 2 degrees of slope inclination. Walls subjected to surcharge loads should be designed for an additional uniform lateral pressure equal to one-third and one-half the anticipated surcharge load for unrestrained and restrained walls, respectively.

For retaining walls that need to resist seismic lateral forces from the retained soils, we recommend the walls be designed to also resist a triangular pressure distribution equal to an equivalent fluid pressure of 28 pounds per cubic foot based on the ground acceleration from a design basis earthquake. This seismic pressure is in addition to the pressures noted above. Due to the transient nature of the seismic loading, a factor of safety of at least 1.1 can be used in the design of the walls when they resist seismic lateral loads. Some movement of the walls may occur during moderate to strong earthquake shaking and may result in distress as is typical for all structures within the San Francisco Bay Area subjected to earthquake shaking.

Where back-drainage will be used behind retaining walls, the back-drainage system can consist of 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 8 inches wide and extend from the base of the wall to within 12 inches of the finished grade at the top (Caltrans Class 2 permeable material (Section 68) may be used in lieu of gravel and filter fabric). A 4-inch diameter, perforated pipe should be installed at the base and centered within the gravel. The perforated pipe should be connected to a solid collector pipe that transmits the water directly to a storm drain, sump pump, drainage inlet, or onto pavement. If weep holes are used in the wall, the perforated pipe within the gravel is not necessary provided the weep holes are kept free of animals and debris, are located no higher than approximately 6 inches from the lowest adjacent grade, and are able to function properly. As an alternative to using gravel, drainage panels (such as AWD SITEDRAIN Sheet 94 for walls or equal) may be used behind the walls in conjunction with perforated pipe (connected to solid collector pipe), weep holes, or strip drains (such as SITEDRAIN Strip 6000 or equal). If used, the drainage panels can be spaced on-center at approximately 2 times the panel width. All wall subdrains should be connected to a solid pipe that discharges to an appropriate drainage facility.

If heavy compaction equipment is used behind the walls, the walls should be appropriately designed to withstand loads exerted by the heavy equipment and/or temporarily braced. Fill placed behind walls should conform to the recommendations provided in **Section 4.1.4, *Fill Material***, and **Section 4.1.5, *Compaction***.

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

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

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

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

As an alternative to using pier foundations to support the walls, footings may be used. Please contact SFB for footing foundation recommendations if footings will be used to support the walls.

4.2.3 Seismic Design Criteria

The following parameters were calculated using the U.S. Seismic Design Map program,[10] and are based on the site being located at approximate latitude 38.285°N and longitude 122.469°W. For seismic design using the 2019 California Building Code (CBC), we recommend the following seismic design parameters be used. 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.

[10]SEAONC/OSHPD, https://seismicmaps.org/, accessed 02/24/2021.

SEISMIC PARAMETER	DESIGN VALUE
Site Class	C
S_S	1.902
S_1	0.718
S_{MS}	2.282
S_{M1}	1.005
S_{DS}	1.521
S_{D1}	0.67
SDC	D
F_a	1.2
F_v	1.4
PGA_M	0.955

4.3 Pavements

4.3.1 Asphalt Concrete

Based on the results of laboratory testing of onsite materials and our borings, we recommend that an R-value of 10 be used in asphalt concrete pavement design. 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 multi-use developments. 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 = 10			
Location	Pavement Components		Total Thickness (inches)
	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	
T.I. = 4.5 (auto & light truck parking)	3.0	8.0	11.0
T.I. = 5.0 (access ways/courts)	3.0	9.0	12.0
T.I. = 6.0 (primary roadways)	3.0	13.0	16.0

If pavements are planned to be placed before 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), SFB should be consulted to provide recommendations for alternative pavement sections capable of supporting heavier loads and higher use. If requested, SFB can provide recommendations for a phased placement of asphalt concrete to reduce the potential for mechanical scars caused by construction traffic in the finished grade. Preliminary pavement sections should be revised, if necessary, when actual traffic indices are known and pavement subgrade elevations are determined.

Pavement subgrade, baserock and asphalt concrete should be compacted to at least 95 percent relative compaction. The asphalt concrete compacted unit weight should be determined using Caltrans Test Method 308-A or ASTM Test Method D1188. Asphalt concrete should also satisfy the S-value requirements by Caltrans.

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

5.0 CONDITIONS AND LIMITATIONS

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

We recommend SFB be retained to provide geotechnical services during design, reviews, earthwork and 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 to be located at 19910 5th Street, Sonoma, California, and is intended to represent our design recommendations to DeNova Homes for specific application to the 19910 5th Street 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 the 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.

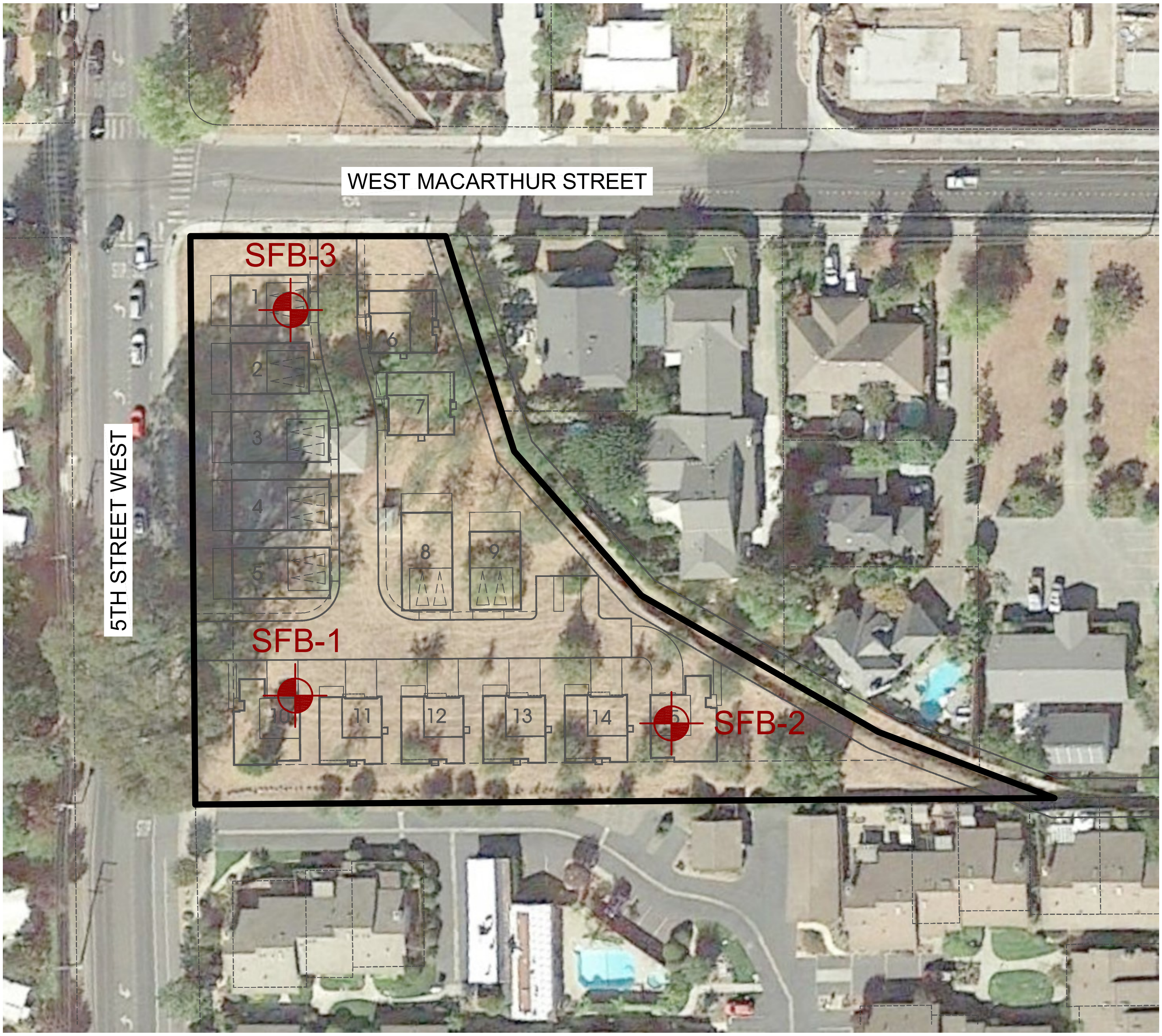
It should be understood that advancements in the practice of geotechnical engineering and engineering geology, or discovery of differing surface or subsurface conditions, may affect the validity of this report and are not uncommon. SFB strives to perform its services in a proper and professional manner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

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

This report does not necessarily represent all of the information that has been communicated by us to 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 DeNova Homes. We cannot accept consequences for use of segregated portions of this report.

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

FIGURES



KEY

ALL LOCATIONS ARE APPROXIMATE

SFB-3 — SFB EXPLORATORY BORING (04/06/2021)

PROJECT LIMIT

N

APPROXIMATE SCALE: 1" = 80'

0 80' 160'

BASE: Conceptual site plan prepared by William Hezmalhalch Architects, Inc. and dated 01/21/2021 overlayed on a Google Earth image dated 10/21/2020.

DATE	Stevens Ferrone & Bailey Engineering Company, Inc	SITE PLAN	FIGURE
April 2021	1600 Willow Pass Court Concord, CA 94520 Tel 925.688.1001 Fax 925.688.1005 www.SFandB.com	19910 5TH STREET WEST Sonoma, California	1
PROJECT NO.			
155-107			

APPENDIX A
Field Investigation

APPENDIX A
Field Investigation

Our field investigation for the proposed residential development to be located at 19910 5th Street in Sonoma, California, consisted of surface reconnaissance and a subsurface exploration program. Reconnaissance of the site and surrounding area, and exploratory borings, were performed on April 6, 2021. Subsurface exploration was performed using a truck-mounted drill rig equipped with 4-inch diameter, continuous flight, solid stem augers. Three exploratory borings were drilled to a maximum depth of about 16-1/2 feet below existing grade. Our representative continuously logged the soils encountered in the borings during our field investigation. The soils are described in general accordance with the Unified Soil Classification System (ASTM D2487). The logs of the borings, as well as, a key for the classification of the soil (Figure A-1) are included as part of this appendix.

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

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

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

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

KEY TO EXPLORATORY BORING LOGS

PROJECT:

19910 5TH STREET WEST
Sonoma, California

PROJECT NO: **155-107**

FIGURE NO: **A-1**

UNIFIED SOIL CLASSIFICATION SYSTEM

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
			GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
	GRAVELS WITH FINES (More than 12% fines)		GM	Silty gravels or gravel-sand-silt mixtures
			GC	Clayey gravels or gravel-sand-clay mixtures
	CLEAN SANDS (Less than 5% fines)		SW	Well-graded sands or gravelly sands, little or no fines
			SP	Poorly-graded sands or gravelly sands, little or no fines
	SANDS WITH FINES (More than 12% fines)		SM	Silty sands or sand-silt mixtures
			SC	Clayey sands or sand-clay mixtures

MAJOR DIVISIONS		GRAPHIC LOG	GROUP SYMBOL	DESCRIPTION
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
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and clays of low plasticity
	SILTS AND CLAYS (Liquid Limit 50% or greater)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts of high plasticity
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic silts and clays of medium to high plasticity
HIGHLY ORGANIC SOILS			PT	Peat and other highly organic soils

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)

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

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

Shelby Tube

Pitcher Barrel

HQ Core

Groundwater Level During Drilling

Groundwater Level at End of Drilling

INCREASING VISUAL MOISTURE CONTENT

Saturated
Wet
Moist
Damp
Dry

CONSTITUENT PERCENTAGE

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

Stevens Ferrone Bailey Engineering

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EXPLORATORY BORING SFB-1

PROJECT NO: 155-107	SURFACE ELEVATION: --
LOGGED BY: HP	DATE STARTED: 04/06/21
DRILL RIG: Mobile B-24	DATE FINISHED: 04/06/21
DRILLING METHOD: 4.5-inch Solid Stem Auger	DEPTH TO INITIAL WATER: 9 feet
HAMMER METHOD: Rope and Cathead	DEPTH TO FINAL WATER: 8 feet
HAMMER WEIGHT / DROP: 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 1	

PROJECT:
19910 5TH STREET WEST
Sonoma, California

DESCRIPTION AND REMARKS	CONSIST	GRAPHIC LOG	DEPTH (FEET) ELEVATION	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS AND NOTES
CLAY (CL), dark brown, silty, with sand (fine-grained), trace gravel (fine, subangular to subrounded), with rootlets, dry to damp.	very stiff		0		26	12.8	100.1	1.7	At 2 feet: Liquid Limit = 34% Plasticity Index = 15 Percent Passing #200 Sieve = 50%
GRAVEL (GC), mottled brown, fine to coarse, angular to subrounded, sandy (fine- to coarse-grained), with clay, some silt, with rock fragments and cobbles, dry to damp.	very dense		5		70				
					31/6"				
Moist.	dense		10		39				
CLAY (CL), brown, silty, with sand (fine-grained), damp.	hard		15		55				
Bottom of Boring = 16.5 feet Notes: Stratification is approximate, variations must be expected. Blow counts converted to SPT N-values. See report for additional details.			20						
			25						

Stevens Ferrone Bailey Engineering

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Concord, CA 94520
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EXPLORATORY BORING SFB-2

PROJECT NO: 155-107	SURFACE ELEVATION: --
LOGGED BY: HP	DATE STARTED: 04/06/21
DRILL RIG: Mobile B-24	DATE FINISHED: 04/06/21
DRILLING METHOD: 4.5-inch Solid Stem Auger	DEPTH TO INITIAL WATER: 16.5 feet
HAMMER METHOD: Rope and Cathead	DEPTH TO FINAL WATER: 15 feet
HAMMER WEIGHT / DROP: 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 1	

PROJECT:
19910 5TH STREET WEST
Sonoma, California

DESCRIPTION AND REMARKS	CONSIST	GRAPHIC LOG	DEPTH (FEET) ELEVATION	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS AND NOTES
CLAY (CL), dark brown, silty, with sand (fine-grained), trace gravel (fine, subangular to subrounded), with rootlets, dry to damp.	very stiff		0		29	10.7	99.5	2.3	
GRAVEL (GC), mottled brown, fine to coarse, angular to subrounded, sandy (fine- to coarse-grained), with clay, some silt, dry to damp.	very dense				70				
Trace cobbles at 5 feet.			5		30/6"				
With rock fragments.			10		55/6"				
CLAY (CL), brown, silty, with sand (fine-grained), damp to moist.	hard		15		36				
Bottom of Boring = 16.5 feet Notes: Stratification is approximate, variations must be expected. Blow counts converted to SPT N-values. See report for additional details.			20 25						

EXPLORATORY BORING SFB-3

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

PROJECT: **19910 5TH STREET WEST**
Sonoma, California

PROJECT NO: 155-107	SURFACE ELEVATION: --
LOGGED BY: HP	DATE STARTED: 04/06/21
DRILL RIG: Mobile B-24	DATE FINISHED: 04/06/21
DRILLING METHOD: 4.5-inch Solid Stem Auger	DEPTH TO INITIAL WATER: 10 feet
HAMMER METHOD: Rope and Cathead	DEPTH TO FINAL WATER: 8 feet
HAMMER WEIGHT / DROP: 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 1	

DESCRIPTION AND REMARKS	CONSIST	GRAPHIC LOG	DEPTH (FEET) ELEVATION	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS AND NOTES
CLAY (CL), dark brown, silty, with sand (fine-grained), trace gravel (fine, subangular to subrounded), with rootlets, dry to damp.	very stiff		0		21	14.8	108.3	2.3	
SAND (SC), mottled brown, fine- to coarse-grained, with clay, some silt, trace gravel (fine, angular to subrounded), dry to damp.	dense				34				
			5		32				
GRAVEL (GC), mottled brown, fine to coarse, angular to subrounded, sandy (fine- to coarse-grained), with clay, some silt, damp.	dense								
Damp to moist.	very dense		10		30/6"				
CLAY (CL), brown, silty, with sand (fine-grained), damp.	hard		15		58				
Bottom of Boring = 16.5 feet Notes: Stratification is approximate, variations must be expected. Blow counts converted to SPT N-values. See report for additional details.			20						
			25						

EXPLORATORY BORING LOG 155-107 SFB-3.ldat8 STEVENS FERRONE & BAILEY 4/14/2021

APPENDIX B

Laboratory Investigation

APPENDIX B

Laboratory Investigation

Our laboratory testing program for the proposed residential development to be located at 19910 5th Street in Sonoma, California, was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site.

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

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

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

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

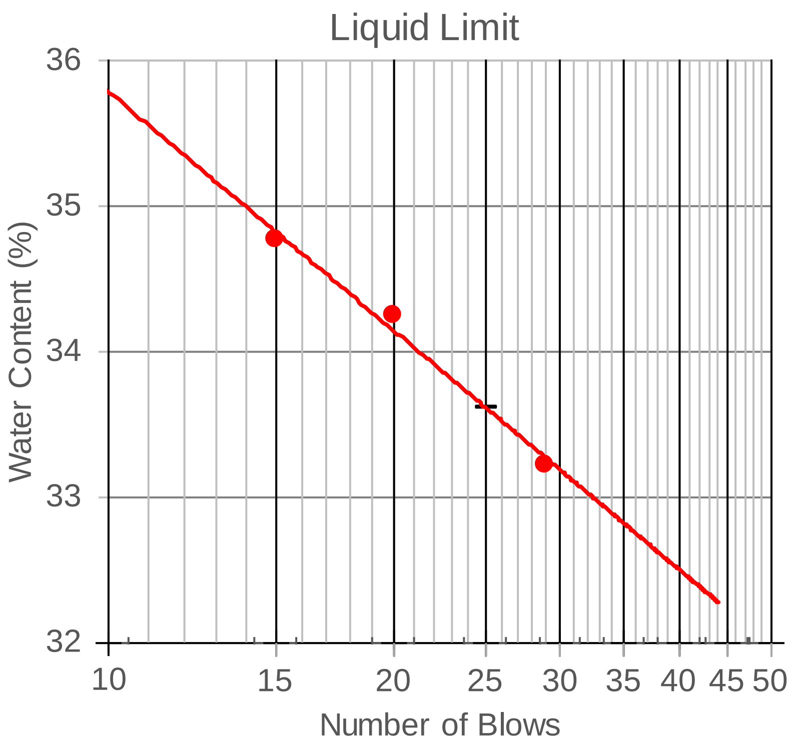
Two onsite soil samples were tested for pH (ASTM D4972), chlorides (ASTM D4327), sulfates (ASTM D4327), sulfides (ASTM D4658M), resistivity at 100% saturation (ASTM G57), and Redox potential (ASTM D1498) for use in evaluating the potential for corrosion on concrete and buried metal, such as utilities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

Atterberg Limits Test – ASTM D4318

Project Number: 155-107 **Boring/Sample No:** SFB-1 **Depth:** 2

Project Name: 19910 5th St. W **Test Date:** 04-09-21

Description: Red brown sandy silty CLAY some gravel (CL) **Tested By:** R

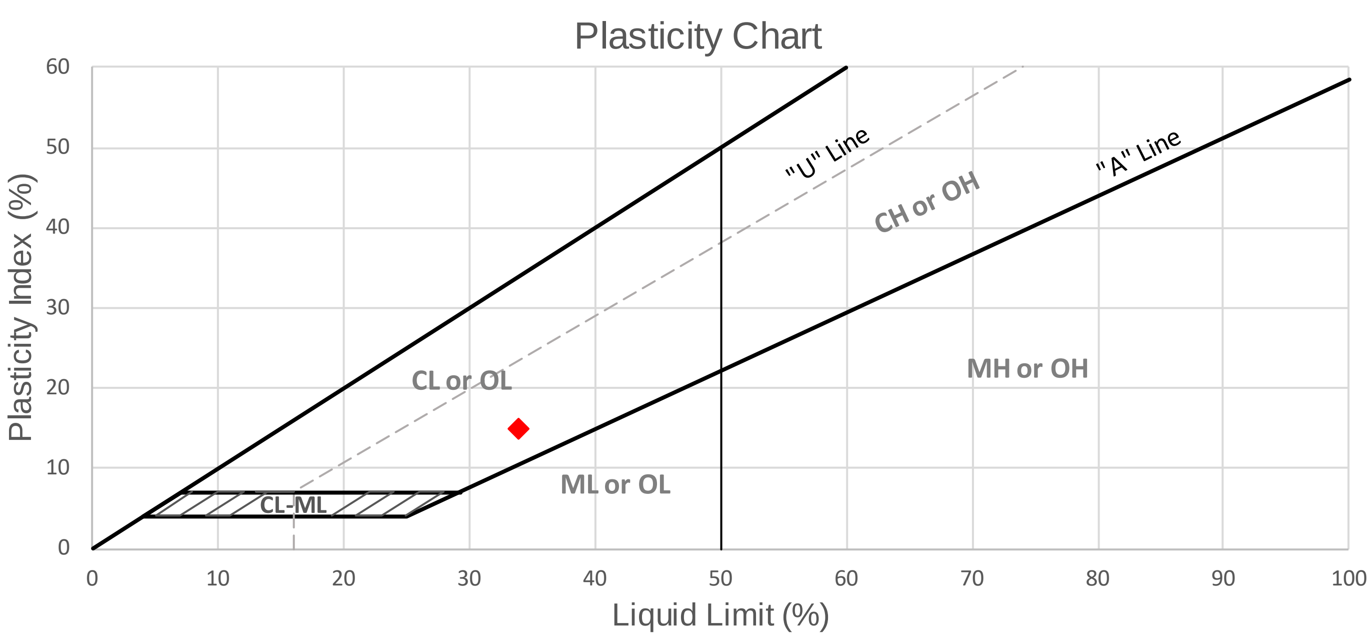


Plastic Limit Data

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

Data Summary

Liquid Limit	**34**
Plastic Limit	**19**
Plasticity Index	**15**
Natural Water Content	12.8
Liquidity Index	-0.413
% Passing #200 Sieve	50.4

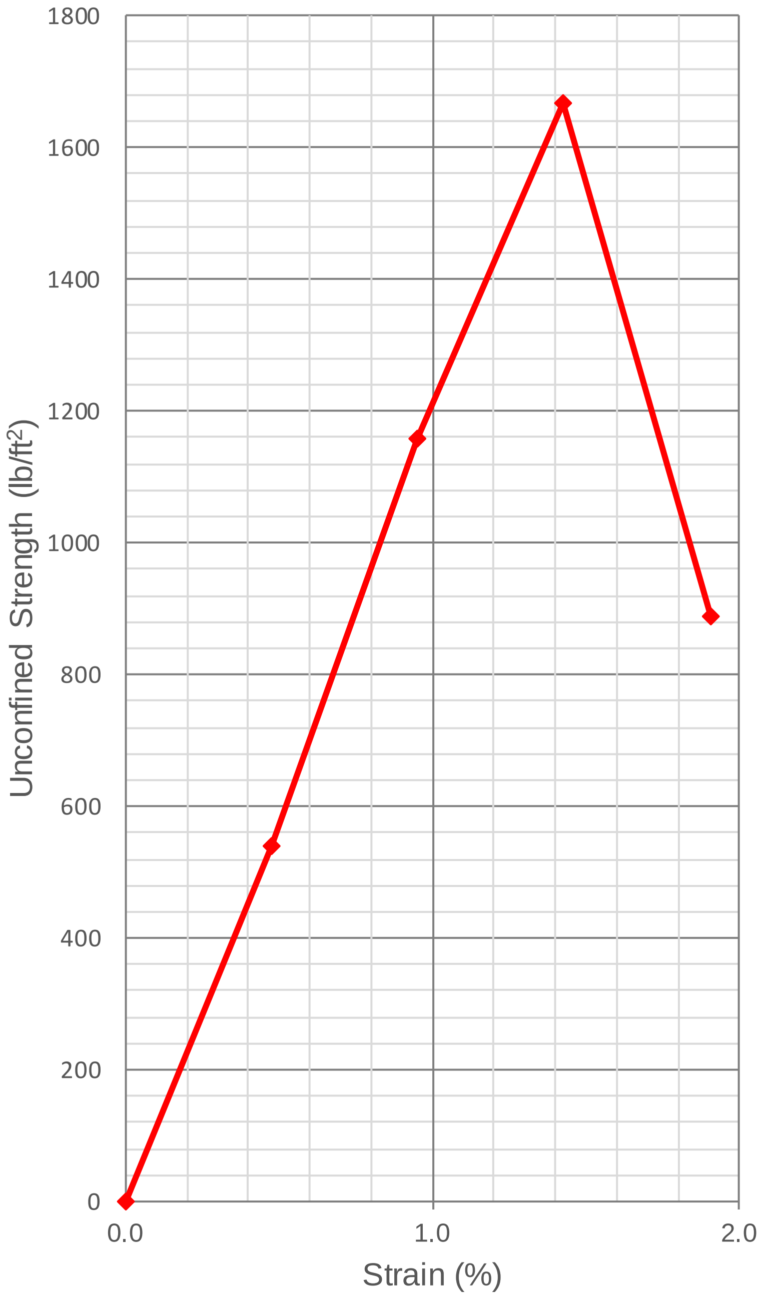


UNCONFINED COMPRESSIVE STRENGTH – D2166

Project Number: 155-107 **Boring/Sample No:** SFB-1 **Depth :** 2

Project Name: 19910 5th St. W **Date:** 04-08-21

Description: Red brown sandy silty CLAY some gravel (CL) **Tested By:** R



Soil Specimen Initial Measurements	
Diameter	2.42 in
Initial Area	4.60 in^2
Initial Length	5.25 in
Volume	0.01397 ft^3
Water Content	12.8 %
Wet Density	112.9 pcf
Dry Density	100.1 pcf

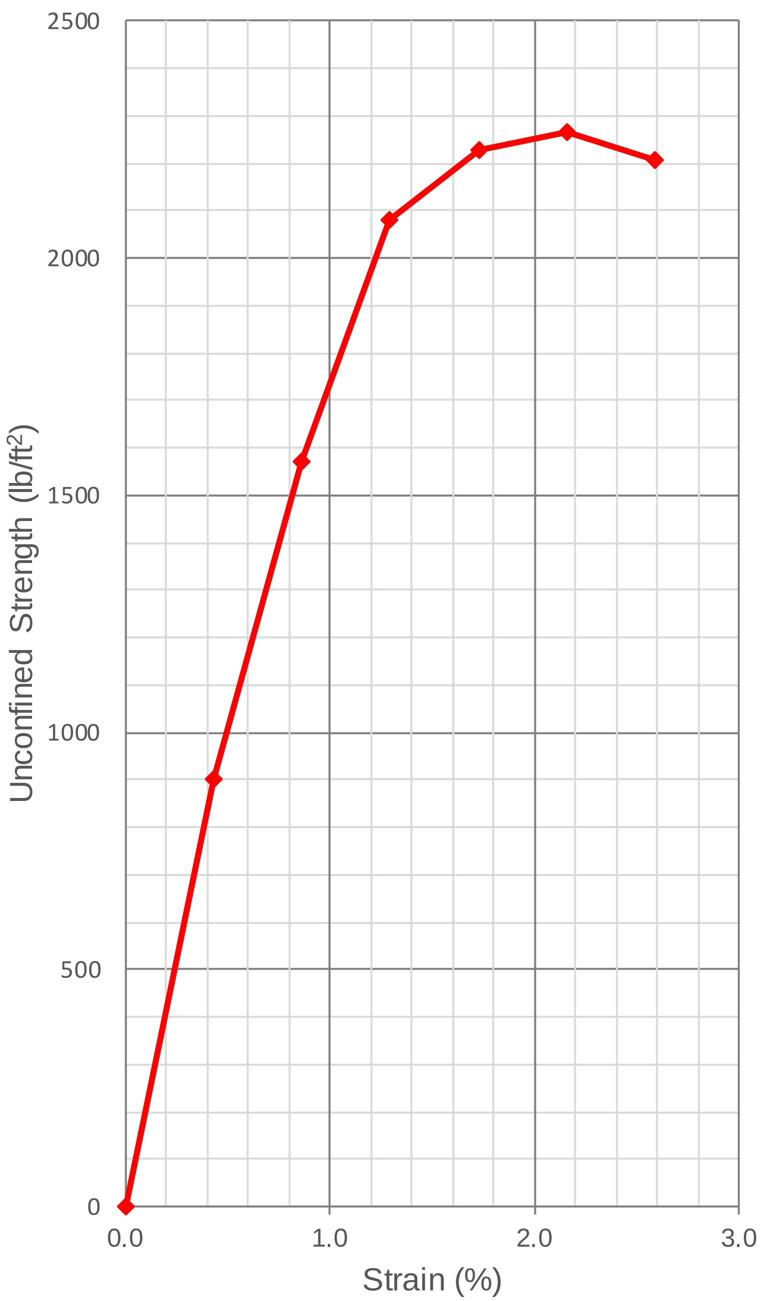
Max Unconfined Compressive Strength	
Elapsed Time	1.5 min
Vertical Dial	0.075 in
Strain	1.4 %
Area	0.03240 ft^2
Axial Load	54.0 lbs
Compressive Strength	1,666 psf

UNCONFINED COMPRESSIVE STRENGTH – D2166

Project Number: 155-107 **Boring/Sample No:** SFB-2 **Depth :** 2

Project Name: 19910 5th St. W **Date:** 04-08-21

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



Soil Specimen Initial Measurements	
Diameter	2.42 in
Initial Area	4.60 in^2
Initial Length	5.8 in
Volume	0.01544 ft^3
Water Content	10.7 %
Wet Density	110.2 pcf
Dry Density	99.5 pcf

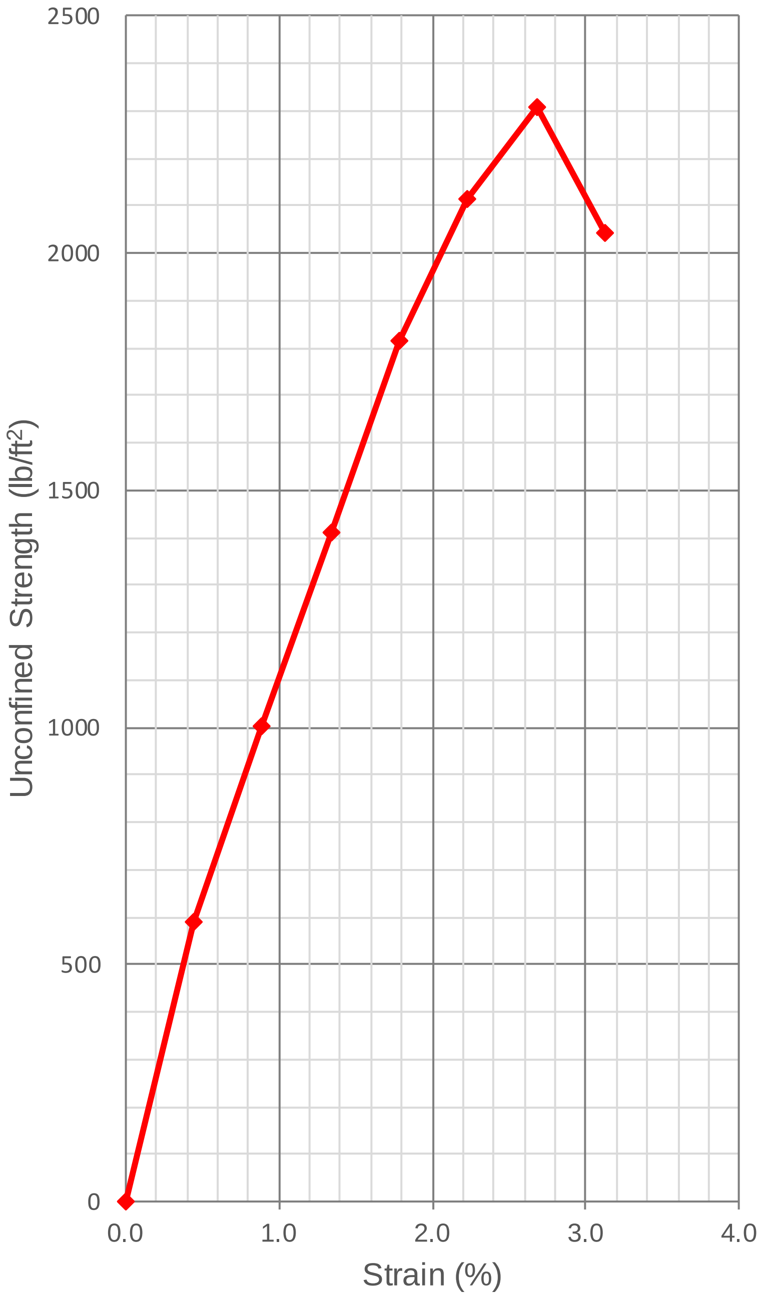
Max Unconfined Compressive Strength	
Elapsed Time	2.5 min
Vertical Dial	0.125 in
Strain	2.2 %
Area	0.03265 ft^2
Axial Load	74.0 lbs
Compressive Strength	2,267 psf

UNCONFINED COMPRESSIVE STRENGTH – D2166

Project Number: 155-107 **Boring/Sample No:** SFB-3 **Depth :** 2

Project Name: 19910 5th St. W **Date:** 04-08-21

Description: Red brown silty CLAY with sand some gravel (CL) **Tested By:** R



Soil Specimen Initial Measurements	
Diameter	2.42 in
Initial Area	4.60 in^2
Initial Length	5.6 in
Volume	0.01491 ft^3
Water Content	14.8 %
Wet Density	124.3 pcf
Dry Density	108.3 pcf

Max Unconfined Compressive Strength	
Elapsed Time	3.0 min
Vertical Dial	0.15 in
Strain	2.7 %
Area	0.03282 ft^2
Axial Load	75.7 lbs
Compressive Strength	2,306 psf

APPENDIX C

Report Guidelines

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.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

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