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

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GEOTECHNICAL INVESTIGATION
20540 BROADWAY
SONOMA, CALIFORNIA
SFB PROJECT NO. 1066-2

Prepared For:

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1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed new residential development to be located at 20540 Broadway in Sonoma, California. The project site location is shown on the **Vicinity Map, Figure 1**, and **Site Plan, Figure 2**. The purpose of our investigation was to evaluate the geological and geotechnical conditions at the site and provide recommendations regarding the geological and geotechnical engineering aspects of the project.

Based on the information provided by Red Tail Multifamily Land Development and the project conceptual site plan prepared by Architects Orange and dated September 11, 2024, we understand that the project will consist of developing a suburban parcel (APN 128-321-007) of about 5 acres for a multi-family apartment complex. The new development will include about six, 2- to 3-story apartment buildings with about 120 dwelling units, a leasing office building, and associated facilities and underground utilities. Open space areas, access roads, parking lots, and stormwater treatment facilities are also anticipated. Actual details of the new development are to be determined and may differ from the conceptual plan. Nominal grading is anticipated.

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- Reviewing published and unpublished geotechnical and geological literature relevant to the site;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 photographs and topographic maps of the site and surrounding area;
- Performing reconnaissance of the site and surrounding area;
- Performing four exploratory borings to a maximum depth of about 38 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 wall/soundwall, and pavement. 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 four exploratory borings (B-1 to B-4) to a maximum depth of about 38 feet on December 23, 2024. The approximate locations of the borings are shown on the **Site Plan, Figure 2**. The borings were performed by West Coast Exploration of Escalon, California by 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.

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 D2488). Logs of the explorations as well as keys for the classification of the soil (**Figure A-1**) are included in **Appendix A.** Upon completion of our field explorations, the borings were backfilled with cement grout in accordance with Sonoma County Well & Septic Division permit requirements.

The approximate locations of our borings were determined by pacing, tape measurements, and/or alignment from landmark references. Latitude and longitude of exploration locations shown on the exploration logs are estimated from online map data from Microsoft; actual locations were not surveyed.

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. 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 exploration 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 fall with rope and cathead. The sampler was driven 18 inches and the number of blows were recorded as field blow counts for each 6 inches of penetration. The SPT N-value shown 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. 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, our 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:

- Seven moisture content and dry unit weight determinations per ASTM D2937;
- Two Atterberg Limits determinations (plastic and liquid limits) per ASTM D4318;
- Four sieve and hydrometer tests per ASTM D422 or D1140; and
- Five 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 of concrete and buried metal such as utilities and reinforcing steel are included in **Appendix B**. We recommend these corrosion test results be forwarded to the project's underground contractors, pipeline designers, concrete contractors, and foundation designers and contractors.

3.3 Surface Conditions and Site Development History

As shown on **Figure 2**, the site is bounded by Broadway on the west, a senior housing development on the northeast, and rural residential properties and undeveloped land on the other sides. The southerly flowing Nathanson Creek of about 6 to 10 feet deep is located at about 170 feet east of the site eastern boundary. The site is rectangular in shape and has a plan area of about 5 acres with maximum dimensions of about 790 feet by 280 feet. The site surface grade slopes very slightly downward toward the southeast.

At the time of our field explorations, the northwestern portion of the site was occupied by several residential buildings, outbuildings, asphalt concrete paved driveways, and associated facilities; the rest of the site was occupied by vineyards that appeared to be abandoned. Large and small diameter trees were generally located around the existing buildings as well as along portions of the site boundaries. Due to the recent storm in the area, the ground surface within the vineyards was very soft, wet, and inaccessible to vehicles.

Based on our review of historical topographic maps and aerial photographs, most of buildings within the site were built before the 1940s. The vineyards were likely planted in the 1970s.

3.4 Subsurface Conditions

The subsurface soils encountered in our borings generally consisted of interbedded firm to hard clays and silts and loose to very dense sands and gravels that extended to the maximum depth explored of about 38 feet. Drilling refusal in very dense gravels was encountered by the Mobile B-24 drill rig at the bottom of Boring B-2. The upper 2 to 3 feet of the surficial soils were saturated and weak due to surface water infiltration after the recent storm in the area.

According to the results of laboratory testing, the near-surface more clayey or silty soils have a medium plasticity and moderate expansion and shrinkage potential. However, the sandy or gravelly soils are non-plastic and have a low expansion and shrinkage potential. Detailed descriptions of soils encountered in our exploratory borings are presented on the exploration logs in **Appendix A**. Results of laboratory testing of retrieved onsite soils are included in **Appendix B**.

3.5 Groundwater

Groundwater was measured in Borings B-1 to B-3 at depths of about 4 to 5-1/2 feet at the end of drilling. However, no groundwater was encountered in Boring B-4 to the maximum depth explored of about 21-1/2. The high groundwater level in Borings B-1 to B-3 were likely caused by surface water infiltration after the recent storm in the area. It should be noted that our borings might not have been left open for a sufficient period of time to establish equilibrium groundwater conditions.

According to a previous environmental groundwater monitoring report (2018)[1] for the neighboring properties to the north (report downloaded from the California State GeoTracker website[2]), groundwater levels in an about 25 feet deep monitoring well (MW-12) located near the

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[1]Edd Clark & Associates, Inc., 2018, February 2018 Groundwater Monitoring Report, Four Corners Service Station, 20500 Broadway, Sonoma, California 95476, Job No. 0407,002.01, June 12.

[2]California State Water Resources Control Board GeoTracker Website, https:// https://geotracker.waterboards.ca.gov/profile_report.asp?global_id=T0609701018, accessed 1/13/2025.

northwestern corner of the project site were reported to fluctuate between about 7 and 23 feet deep from 2003 to 2018.

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. The depths to groundwater below existing ground surface may vary greatly across the 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)[3] and categorized as Wright Loam, 0 to 9 percent slopes (Unit WgC). This unit has been assigned by USDA to Hydrologic Soil Group D, and is characterized as having a very low to moderately low water transmission rate (0 to 0.06 inch per hour). Group D soils are defined as having a very slow infiltration rate when thoroughly wet and may consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan.

However, the results of our field explorations indicate sand and gravel layers with higher infiltration rates exist below the site at shallow depths in some areas (such as at the locations of Borings B-1 and B-3). If needed, we recommend field Double Ring Infiltrometer Tests (ASTM D3385) be performed at the planned stormwater treatment facility locations to evaluate the field infiltration rates at the potential infiltration depths.

Actual field infiltration rates will depend on the in-situ soil type, moisture, relative density, gradation, and fines content of soils, whether any water impeding clay layers exist at shallow depth, and proper and regular maintenance of the infiltration facilities. Compacted/engineered fills should be considered as having limited infiltration ability.

3.7 Geology and Seismicity

According to Wagner and Gutierrez (2010)[4], the site and the surrounding areas are mapped as being underlain by late Pleistocene to Holocene alluvial fan deposits. These soil deposits generally consist of sand, gravel, silt, and clays on gently sloping, fan-shaped, relatively undissected alluvial surfaces. The subsurface soil conditions encountered by our borings within the site are consistent with the regional geologic mapping.

The project site is in the San Francisco Bay Area, which is considered one of the most seismically active regions in the United States. Significant earthquakes have occurred in the region and are

[3]USDA NRCS, https://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx, accessed 12/16/2024.

[4]Wagner & 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 California[5]. 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)[6] 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 2022 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 0.87g 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)[7], the resulting deaggregation calculations indicate that the site has a 10% probability of exceeding a peak ground acceleration of about 0.55g 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.

[5] California Department of Conservation, Earthquake Fault Zones, CGS Special Publication 42, Revised 2018.
[6] Aagaard, 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).
[7] USGS Unified Hazard Tool, https://earthquake.usgs.gov/hazards/interactive/, accessed 1/13/2025.

As of the date of this report, the liquefaction potential of the site and surrounding area has not been evaluated by the State of California[8]. The site and surrounding areas are mapped by Witter et al. (2006)[9] as being within an area having a moderate susceptibility to liquefaction hazard.

To evaluate the soil liquefaction potential of the site, we performed SPT-based seismic soil liquefaction triggering and post-liquefaction deformation analyses in general accordance with the guidelines in CGS Special Publication 117A (2008)[10] and the recommended procedures by Southern California Earthquake Center (SCEC, 1999)[11]. The analyses were performed by using methodologies developed by Idriss and Boulanger (2008 and 2014)[12,13]. Subsurface soil data from our borings and an assumed high groundwater level at 5 feet deep were used in our liquefaction analyses.

A site modified peak ground acceleration (PGA_M) of 0.87g from a Maximum Considered Earthquake (MCE) event (per 2022 California Building Code and based on Site Class D stiff soil) and a modal earthquake magnitude (Mw) of 7.5 (based on the USGS Unified Hazard Tool deaggregation model with a return period of 2,475 years) were used in our analyses. Factor of safety against liquefaction triggering is defined as Cyclic Resistance Ratio (CRR) divided by the Cyclic Stress Ratio (CSR). Saturated soils are considered to be potentially liquefiable if the calculated factor of safety against liquefaction triggering is less than about 1.3 (per CGS Special Publication 117A). Post-liquefaction residual volumetric strains and deformations were estimated for the potentially liquefiable soils based on the methodologies used.

The results of our analyses indicate that some loose to medium dense sands and gravels encountered in our borings below the site have a moderate to high potential for liquefying when they are subjected to an MCE earthquake event. These sands and gravels appear to be in the form of randomly distributed pockets and generally locate within about 20 feet deep below ground surface. We estimate the MCE earthquake induced liquefaction in these sand and gravel pockets could result in residual volumetric strains varying from about 1 to 3 percent and may cause total ground surface settlements up to about 2-1/2 inches at the site.

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[8] Seismic Hazards Mapping Act, 1990.

[9] Witter, Knudsen, Sowers, Wentworth, Koehler, and Randolph, 2006, Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, California, U.S. Geological Survey Open File Report 2006-1037.

[10] California Geological Survey, 2008, Guidelines for Evaluating and Mitigating Seismic Hazards in California, CGS Special Publication 117A.

[11] Southern California Earthquake Center, 2002, Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California, June.

[12] Idriss and Boulanger, 2008, Soil Liquefaction During Earthquakes, EERI Monograph MNO-12.

[13] Boulanger and Idriss, 2014, CPT and SPT Based Liquefaction Triggering Procedures, Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California, Davis, CA, Report No. UCD/CGM-14/01, April.

The actual ground surface damage or manifestation depends on characteristics of the triggering earthquakes and thicknesses of overlying non-liquefiable soil crust and underlying liquefiable soils as indicated by the empirical criteria developed by Ishihara (1985)[14]. According to Bowen and Jacka (2013)[15], the observations after the 2010-2011 Canterbury earthquakes (Mw 7.1, 6.2, 5.9 and 6.0) suggest liquefaction induced ground damage likely did not occur in locations where the non-liquefiable soil crust thickness is greater than 4 meter thick (or about 13 feet thick). It is our opinion the potential for ground surface damage or manifestation due to liquefaction occurring at the site is moderate to high if the groundwater level at 5 feet deep during an MCE event.

It is our understanding that the current building codes and the California statewide safety standard as outlined in California Geologic Survey (CGS) Special Publication 117A are only intended to protect life-safety and prevent collapse of non-essential buildings but not to prevent structural damage during a major liquefication event. To reduce the potential soil liquefaction impacts, the building foundations and superstructures should be designed to tolerate differential settlements caused by liquefaction without loss of the ability to support gravity loads. We recommend the building foundations and superstructures be designed to resist 2-1/2 inches of total settlement and 1-1/4 inch of differential settlement. The magnitude of differential settlement could occur directly below the center of a building shallow foundation system (or over a distance of about 30 feet) as a result of a “cupping” shape of the underlying supporting subgrade after soil liquefaction.

Underground pipelines (such as gas lines, sanitary sewers, storm drains, and water services) should be properly designed to compensate for the settlement caused by the liquefaction of the underlying supporting soils. It should be noted that after a liquefaction event, phenomena such as sand boils, ground cracking, and differential movement of overlying improvements such as structures, driveways, roadways, and utilities can occur and may require repair.

3.9 Lateral Spreading

As part of our soil liquefaction analyses, we 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 to unconfined spaces (such as the nearby Nathanson Creek located to the east of the site), which causes significant horizontal ground displacements.

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[14]Ishihara, 1985, Stability of Natural Deposits During Earthquakes, Proceedings of the Eleventh International Conference on Soil Mechanics and Foundation Engineering, San Francisco, CA Volume 1, p. 321-376.

[15]Bowen and Jacka, 2013, Liquefaction Induced Ground Damage in the Canterbury Earthquakes: Predictions vs. Reality, Proceedings 19th NZGS Geotechnical Symposium, Queenstown.

The potentially liquefiable sand lenses below the site appear to be in the form of randomly distributed pockets and have an equivalent SPT-$N_{1,60}$ value greater than 15, which is considered by Youd et al. (2002)[16] as unlikely to cause significant lateral spreading for an earthquake event of smaller than moment magnitude 8. In addition, the site eastern boundary is located at least 170 feet from the bank of the relatively shallow Nathanson Creek (about 6 to 10 feet deep).

Based on our review of available literature, the results of the field explorations, and results of our analyses, it is our opinion that the potential for lateral spreading adjacent to the nearby Nathanson Creek adversely impacting the site development is low.

[16]Youd, Hansen, and Bartlett, 2002, Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement, Journal of Geotechnical and Geoenvironmental Engineering, p. 1007-1017, December.

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: As described in **Section 3.4**, at the time of our field explorations, the upper 2 to 3 feet of the surficial soils were saturated and weak due to surface water infiltration after the recent storm in the area. We also anticipate that the demolition of the existing structures (and their associated foundations) and improvements, and the removal of existing vineyards will disturb and weaken the upper 2 to 3 feet of surface soils within the site.

In order to reduce the potential for damaging differential settlement of overlying improvements (such as structural fill, 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 depths of about 2 feet below the existing ground surface, scarifying and compacting the bottom 12 inches before placing any new fill, and placing well-blended, moisture conditioned, and properly compacted fill over the properly prepared subgrade. Deeper removal will be needed in areas if thicker weak soils are encountered during grading. The over-excavation should extend to depths where competent soils are encountered. The cuts for new roads will most likely remove most of these weak soils.

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 planned open space areas where ground instability or settlement is less a concern. 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 properties are 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 reused as new engineered fills at the site.

SHALLOW GROUNDWATER: As described in **Section 3.5**, groundwater was measured in Borings B-1 to B-3 at depths of about 4 to 5-1/2 feet at the end of drilling. However, no groundwater was encountered in Boring B-4 to the maximum depth explored of about 21-1/2 feet. The high groundwater level in Borings B-1 to B-3 were likely caused by surface water infiltration after the recent storm in the area. Groundwater levels in a nearby monitoring well located near the northwestern corner of the project site were reported to fluctuate between about 7 and 23 feet deep from 2003 to 2018. 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. The depths to groundwater below existing ground surface may vary greatly across the site.

Dewatering of excavations that extend below the groundwater level may be necessary, such as during underground utility installations. Installing shoring and/or dewatering wells may also be necessary to aid in the stabilization of excavation walls and bottoms.

LIQUEFACTION SETTLEMENT: As described in **Section 3.8** of this report, the results of our analyses indicate that some loose to medium dense sands and gravels encountered in our borings below the site have a moderate to high potential for liquefying when they are subjected to an MCE earthquake event. These sands and gravels appear to be in the form of randomly distributed pockets and generally locate within about 20 feet deep below ground surface. We estimate the MCE earthquake induced liquefaction in these sand and gravel pockets could result in residual volumetric strains varying from about 1 to 3 percent and may cause total ground surface settlements up to about 2-1/2 inches at the site.

To reduce the potential soil liquefaction impacts, the building foundations and superstructures should be designed to tolerate differential settlements caused by liquefaction without loss of the ability to support gravity loads. We recommend the building foundations and superstructures be designed to resist 2-1/2 inches of total settlement and 1-1/4 inch of differential settlement. The magnitude of differential settlement could occur directly below the center of a building shallow foundation system (or over a distance of about 30 feet) as a result of a “cupping” shape of the underlying supporting subgrade after soil liquefaction.

Underground pipelines (such as gas lines, sanitary sewers, storm drains, and water services) should be properly designed to compensate for the settlement caused by the liquefaction of the underlying supporting soils. It should be noted that after a liquefaction event, phenomena such as sand boils, ground cracking, and differential movement of overlying improvements such as structures, driveways, roadways, and utilities can occur and may require repair

DISSIMILAR SUBGRADE SOILS AND SHRINKAGE/EXPANSION POTENTIAL: As described in **Section 3.4**, the near-surface more clayey or silty soils have a medium plasticity and moderate expansion and shrinkage potential. However, the sandy or gravelly soils are non-plastic and have a low expansion and shrinkage potential.

To reduce the potential for damaging differential movement of the proposed structures, we recommend the proposed grading be performed so that the buildings and surrounding flatwork can be supported on fills with similar expansion potential. In no case should the buildings be underlain by subgrade (and within 3 feet of subgrade) consisting of both expansive clayey/silty soils or fills and relatively non-expansive sandy/gravelly soils or fills.

We recommend at least 3 feet thick of well-blended, moisture conditioned, engineered fill be provided below the buildings and surrounding flatwork located. The compacted, engineered fill layers should extend at least 5 feet beyond building footprints and at least 3 feet beyond surrounding exterior flatwork, whichever is greater.

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 brief evaluation 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, building foundation, retaining wall/soundwall, and pavement recommendations for use in design and construction of the project are presented below. We recommend SFB review the design and specifications to verify that the recommendations presented in this report have been properly interpreted and implemented in the design, plans, and specifications. We also recommend SFB be retained to provide consulting services and to perform construction observation and testing services during the construction phase of the project to observe and test the implementation of our recommendations, and to provide supplemental or revised recommendations in the event conditions different than those described in this report are encountered. We 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 driveways, pipelines and their backfill, designated trees and their associated entire root systems, vineyards, and debris. Holes resulting from the removal of underground obstructions extending below the proposed finish grade should be cleared and backfilled with fill materials as specified in **Section 4.1.5, *Fill Material***, and compacted to the requirements in **Section 4.1.6, *Compaction***. Tree roots may extend to depths of about 3 to 4 feet. Wells and septic systems, if they exist, should be abandoned in accordance with 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 4.1.5, *Fill Material***. We recommend fill materials composed of broken up concrete or asphalt concrete not be located within 3 feet of the ground surface in yard areas. Consideration should be given to placing these materials below pavements, directly under building footprints, or in deeper excavations. We recommend backfilling operations for any excavations be performed under the observation and testing of SFB. Crushed concrete materials from concrete demolition 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 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

As described previously, at the time of our field explorations, the upper 2 to 3 feet of the surficial soils were saturated and weak due to surface water infiltration after the recent storm in the area.

We also anticipate that the demolition of the existing structures (and their associated foundations) and improvements, and the removal of existing vineyards will disturb and weaken the upper 2 to 3 feet of surface soils within the site.

In order to reduce the potential for damaging differential settlement of overlying improvements (such as structural fill, 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 depths of about 2 feet below the existing ground surface, scarifying and compacting the bottom 12 inches before placing any new fill, and placing well-blended, moisture conditioned, and properly compacted fill over the properly prepared subgrade. Deeper removal will be needed in areas if thicker weak soils are encountered during grading. The over-excavation should extend to depths where competent soils are encountered. The cuts for new roads will most likely remove most of these weak soils.

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 planned open space areas where ground instability or settlement is less a concern. 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 properties are 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 4.1.5, *Fill Material***. Compaction should be performed in accordance with the recommendations in **Section 4.1.6, *Compaction***.

4.1.3 Building Pad

To reduce the potential for damaging differential movement of the proposed structures, we recommend the proposed grading be performed so that the buildings and surrounding flatwork can be supported on fills with similar expansion potential. In no case should the buildings be underlain by subgrade (and within 3 feet of subgrade) consisting of both expansive clayey/silty soils or fills and relatively non-expansive sandy/gravelly soils or fills.

We recommend at least 3 feet thick of well-blended, moisture conditioned, engineered fill be provided below the buildings and surrounding flatwork. The compacted, engineered fill layers should extend at least 5 feet beyond building footprints and at least 3 feet beyond surrounding exterior flatwork, whichever is greater.

4.1.4 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 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 experience vegetation growth, the exposed pads and subgrades 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 pad and subgrade reconditioning.

4.1.5 Fill Material

From a geotechnical and mechanical standpoint, onsite soil and fill 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 with not 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. 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.

4.1.6 Compaction

Within the upper 5 feet of the finished ground surface, we recommend structural fill be compacted at least 90 percent relative compaction, and structural fill below a depth of 5 feet be compacted to at least 95 percent relative compaction, as determined by ASTM D1557 (latest edition). We recommend the new fill be moisture conditioned approximately 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 un-compacted thickness.

4.1.7 Utility Trench Backfill

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

Onsite trench backfill should be compacted to at least 90 percent relative compaction. Imported sand trench backfill should be compacted to at least 95 percent relative compaction 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 crushed rock bedding and 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 crushed rock bedding and backfill.

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

Where the bottoms of trenches are sloped steeper than 5 percent, we recommend a low permeability plug composed of low strength concrete, sand/cement slurry, or onsite clays be installed in the utility trenches every 50 feet on-center. The plug will reduce piping/consolidation from water seepage that may cause roadway and trench surface settlement. The plug should be at least 12 inches thick and extend to within 1 foot of the finished ground surface or to the base of the pavement section.

4.1.8 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 (such as driveways, sidewalks, patios, exterior flatwork, etc.) should be anticipated. This movement could result in damage to the exterior slabs and might require periodic maintenance or replacement. Adequate clearance should be provided between the exterior slabs and building elements that overhang these slabs, such as window sills or doors that open outward.

We recommend reinforcing exterior slabs with steel bars in lieu of wire mesh. To reduce potential crack formation, the installation of #4 bars spaced at approximately 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 soil 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.

4.1.9 Construction During Wet Weather Conditions

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

4.1.10 Surface Drainage, Irrigation, and Landscaping

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

Surface drainage should be designed in accordance with the latest edition of the California Building Code 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 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 foundations or exterior slabs. Installation of root barriers may be necessary to restrict root development below structures.

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

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

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

4.1.11 Storm Water 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 predevelopment 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, considerations 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.

4.1.12 Future Maintenance

In order to reduce water related issues, we recommend regular inspection and maintenance of the site 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 development owners perform inspections and maintenance of the slabs, including infilling significant cracks, providing fillers at slab offsets, and replacing slabs if severely damaged.

4.1.13 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 development owners and their maintenance associations.

4.2 Foundation Support

4.2.1 Post-Tensioned Slab

The proposed residential buildings can be supported on a post-tensioned slab foundation that is designed for the expansion potential of onsite soils and fills. The building foundations and superstructures should also be designed to tolerate differential settlements caused by liquefaction without loss of the ability to support gravity loads. We recommend the building foundations and superstructures be designed to resist 2-1/2 inches of total settlement and 1-1/2 inch of differential settlement. The magnitude of differential settlement could occur directly below the center of the building's shallow foundation system (or over a distance of about 30 feet) as a result of a "cupping" shape of the underlying supporting subgrade after soil liquefaction.

The slab foundation should bear entirely on properly prepared and compacted, well-blended structural fill. In no case should a slab foundation bear upon fills with differential expansion characteristics. We recommend at least 3 feet thick of well-blended, moisture conditioned, engineered fill be provided below the buildings and surrounding flatwork. The compacted, engineered fill layers should extend at least 5 feet beyond building footprints and at least 3 feet beyond surrounding exterior flatwork, whichever is greater.

Recommendations for building pad preparation are described previously in **Sections 4.1.3, *Building Pad***, and **4.1.4, *Subgrade Preparation***, of this report. Prior to the concrete pour, we recommend the moisture content of subgrade materials be conditioned to 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. We also 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.

The thickness of post-tensioned slab should be determined by the Structural Engineer; however, we recommend the post-tensioned slabs be at least 10 inches thick. In addition, deflection of the slab foundations should be designed not to exceed the values recommended in the most recent PTI design manual. An allowable bearing pressure of 1,500 pounds per square foot can be used for localized point and line loads. We estimate the maximum total static settlement of PT-slab foundations under the recommended allowable bearing pressure to be on the order of 1 inch or less. Differential static settlement is estimated to be approximately 1/2 inch or less.

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)[17], PTI standard requirements (2019)[18], 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.

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

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

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 4.1.10,** ***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

4.2.2 Retaining Walls

If segmental block walls with geogrid will be used at the site, SFB should be contacted to provide block wall and geogrid designs and specifications. 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. The global stability of the walls should also be evaluated where the walls will be located on slopes or where multi-tiered walls will be used.

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	85	40
Restrained (At-Rest Pressure)		60	95	80*

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

For retaining walls that need to resist earthquake induced lateral loads from nearby foundations, walls that retain buildings, walls that are to be designed to resist earthquake loads, and any retaining walls that are higher than 6 feet (as required by the 2022 CBC), we recommend the walls be designed to also resist an incremental seismic lateral earth pressure listed in the above table, using a triangular fluid pressure distribution (not inverted). This seismic induced earth pressure is in addition to the active pressures listed above. The seismic lateral earth pressure increments for unrestrained and restrained walls were estimated, respectively, based on 50% and 100% of the peak ground acceleration (PGA_M) from a Maximum Considered Earthquake (MCE) earthquake per ASCE 7-16/2022 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 4.1.5, *Fill Material***, and **Section 4.1.6, *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 bottoms of 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 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. Please contact SFB for footing foundation recommendations if footings will be used to support the walls.

4.3 Seismic Design Criteria

As described in **Section 3.8** of this report, some of the soils underlying the site are susceptible to soil liquefaction when subjected to an MCE event. These liquefiable soils are classified by the 2022 California Building Code (CBC) and ASCE 7-16 as Site Class “F” since they are vulnerable to potential failure or collapse under seismic loading. However, it is our understanding that the proposed project structures will have fundamental periods of vibration equal to or less than 0.5 second. Therefore, a site-specific response analysis is not required for liquefiable soils per Section 20.3.1 of ASCE 7-16, and the site class and seismic design parameters can be determined in accordance with ASCE 7-16 and its supplements.

Based on the site geology and subsurface soil conditions encountered at the site, we recommend the site be characterized as Site Class “D”, a “stiff soil” profile. For seismic designs using the 2022 CBC and ASCE 7-16, we recommend the following parameters be used. These parameters were calculated using the ASCE 7 Hazard Tool online program[19], and are based on the site being located at approximate latitude 38.274236°N and longitude 122.459087°W.

We assumed the proposed project structures are categorized as Risk Category II, and the exceptions of ASCE 7-16 and Supplement 3 Section 11.4.8 (Site-Specific Ground Motion Procedures) will be taken by the Structural Engineer. 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	1.898
S_1	0.716
F_a	1.000
F_v	See Section 11.4.8 of ASCE 7-16*
S_{MS}	1.898
S_{M1}	See Section 11.4.8 of ASCE 7-16*
S_{DS}	1.265
S_{D1}	See Section 11.4.8 of ASCE 7-16*
SDC	See Section 11.4.8 of ASCE 7-16*
PGA	0.794
F_{PGA}	1.1
PGA_M	0.873
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 the ASCE 7-16 Section 11.4.8 requirements and any applicable supplements.

4.4 Pavements

4.4.1 Asphalt Concrete Pavement

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

[19] ASCE, https://asce7hazardtool.online/, accessed 12/16/2024.

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

PRELIMINARY PAVEMENT DESIGN ALTERNATIVES SUBGRADE R-VALUE = 5			
Location	Pavement Components		Total Thickness (inches)
	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	
T.I. = 4.5 (auto & light truck parking)	2.5	9.0	11.5
T.I. = 5.0 (access ways/courts)	3.0	10.0	13.0

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 outriggers), 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.

4.4.2 Rigid Concrete Pavement

The analytical procedure used in our design of the rigid vehicular concrete pavement, such as for driveway and apron, garbage truck access, and trash enclosure, was based on the guide and method published by the American Concrete Institute (ACI 330R-1)[20]. A modulus of subgrade reaction of 100 pounds per square inch per inch was assigned to represent the engineered fill subgrade overlain by 12 inches of Class 2 aggregate base. The concrete was assumed to have a modulus of rupture of 550 pounds per square inch or have a compressive strength of 4,000 pounds per square inch.

Based on our analysis, we recommend the concrete pavement consist of 6 inches of concrete slab overlying 12 inches of Caltrans Class 2 aggregate base. We recommend the slabs be reinforced with a minimum of #4 bars spaced at approximately 18 inches on center in both directions. The actual thickness and reinforcing of the slabs should be designed based on the anticipated traffic loads. The concrete and aggregate base should be constructed in accordance with the appropriate specifications for pavements.

[20]American Concrete Institute, 2001, Guide for Design and Construction of Concrete Parking Lots, ACI 330R-01.

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 Red Tail Multifamily Land Development LLC and their consultants for specific application to the proposed residential development at 20540 Broadway in Sonoma, California, and is intended to represent our design recommendations to Red Tail Multifamily Land Development LLC. for specific application to the proposed project. The conclusions and recommendations contained in this report are solely professional opinions. It is the responsibility of Red Tail Multifamily Land Development LLC 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.

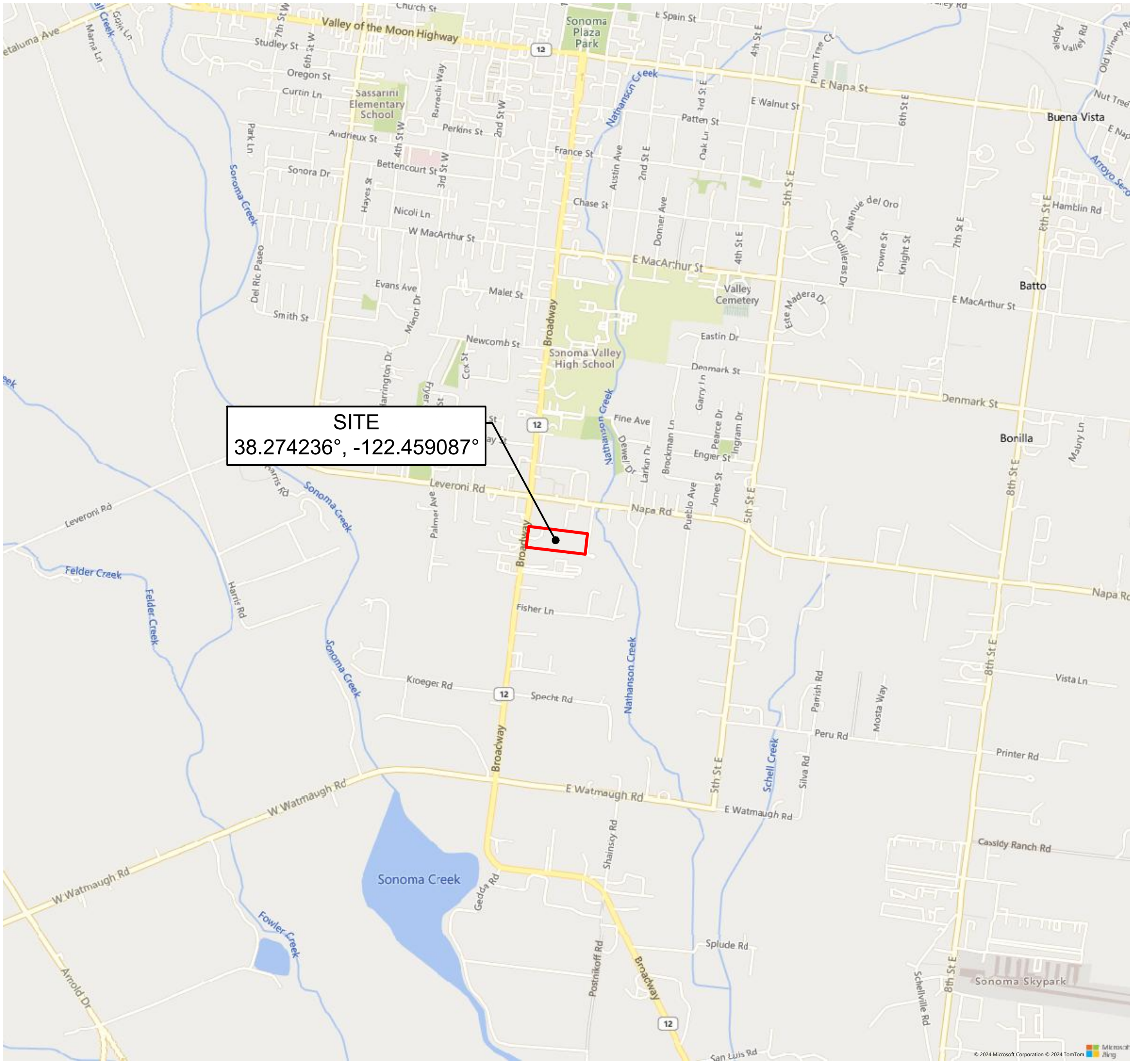
It should be understood that advancements in the practice of geotechnical engineering and engineering geology, or discovery of differing surface or subsurface conditions, may affect the validity of this report and are not uncommon. SFB strives to perform its services in a proper and professional manner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

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

FIGURES



NOTE: Basemap from Microsoft/TomTom 2024.

SCALE: 1" = 2,000'

0 2,000' 4,000'

DATE	Stevens Ferrone & Bailey Engineering Company, Inc	VICINITY MAP	FIGURE
January 2025	1600 Willow Pass Court Concord, CA 94520 Tel 925.688.1001 Fax 925.688.1005 www.SFandB.com	20540 BROADWAY Sonoma, California	1
PROJECT NO.			
1066-2			

1066-2, 20540 Broadway, Sonoma - December 2024.dwg
Printed: 12/4/2024

KEY

- Exploratory Boring by SFB (12/23/2024)
- Approximate Project Limit

NOTE: All locations shown are approximate.

NOTE: Basemaps from the project conceptual plan (prepared by Architects Orange and dated 9/11/2024) and Sonoma County Assessor's Parcel Map Book 128 Page 32. Aerial photo imagery from Microsoft 2024.

SCALE: 1" = 100' (0, 100', 200')	DATE: January 2025	Stevens Ferrone & Bailey Engineering Company, Inc	1600 Willow Pass Court, Concord, CA 94520, Tel 925.688.1001, Fax 925.688.1005, www.SFandB.com	SITE PLAN: 20540 BROADWAY, Sonoma, California	FIGURE 2
	PROJECT NO.: 1066-2				

1066-2, 20540 Broadway, Sonoma - December 2024.dwg
Printed: 1/8/2025

APPENDIX A
Field Exploration

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

KEY TO FIELD EXPLORATION LOGS

PROJECT: **20540 BROADWAY**
Sonoma, California

PROJECT NO: **1066-2**

FIGURE NO: **A-1**

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D2487 & D2488)

MAJOR DIVISIONS		GRAPHIC LOG	GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE-GRAINED SOILS (More than 50% retained on #200 sieve)	CLEAN GRAVELS (Less than 5% fines)		GW	Well-graded gravels, gravel-sand mixtures, trace or no fines
			GP	Poorly-graded gravels, gravel-sand mixtures, trace or no fines
	GRAVELS WITH FINES (More than 12% fines)		GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	CLEAN SANDS (Less than 5% fines)		SW	Well-graded sands, gravelly sands, trace or no fines
			SP	Poorly-graded sands, gravelly sands, trace or no fines
	SANDS WITH FINES (More than 12% fines)		SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS (More than 50% passes #200 sieve)	SILTS AND CLAYS (Liquid Limit less than 50)		ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands, 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 more)		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 SIEVE SIZE

	12"		3"		3/4"		#4		#10		#40		#200	

BOULDERS	COBBLES	GRAVELS		SANDS			SILTS AND CLAYS
		Coarse	Fine	Coarse	Medium	Fine	

304.8 mm | 76.2 mm | 19.0 mm | 4.75 mm | 2.00 mm | 0.425 mm | 0.075 mm

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.
**UCS: Unconfined Compressive Strength.

SYMBOLS AND NOTES

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

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

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

Shelby Tube

Bulk Sample

Core Barrel (See Log Notes)

Groundwater Level During Drilling

Groundwater Level at End of Drilling

INCREASING VISUAL MOISTURE CONTENT

Wet
Moist
Dry

CONSTITUENT PERCENTAGE

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

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

EXPLORATORY BORING B-1

PROJECT:
20540 BROADWAY
Sonoma, California

PROJECT NO: 1066-2	SURFACE ELEVATION: --
LOGGED / CHECKED BY: R. Ceraolo / T. Chen	DATE STARTED: 12/23/2024
DRILLER: West Coast Exploration	DATE FINISHED: 12/23/2024
DRILL RIG: Mobile B-24	DEPTH TO INITIAL WATER: 3 Feet
DRILLING METHOD: 4-inch Solid Flight Auger	DEPTH TO FINAL WATER: 4 Feet
HAMMER TYPE / WEIGHT / DROP: Safety Hammer with Rope & Cathead / 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 2 (38.274163°; -122.459623°)	

SUBSURFACE MATERIAL CLASSIFICATION

DESCRIPTION	CONSIST	GRAPHIC LOG	DEPTH (FEET)	SAMPLER	FIELD BLOW COUNT	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS NOTES AND REMARKS
CLAY (CL)/SAND (SC), brown, silty, sandy (fine- to coarse-grained), few to with gravel (fine, subangular to subrounded), wet.	firm to stiff		0		7 7 7	8	14.4	113.6	0.2	
Gravelly (fine to coarse, subangular to subrounded), moist.										
GRAVEL (GW-GM)/SAND (SW-SM), mottled gray brown, fine, subangular to subrounded, sandy (fine- to coarse-grained), few silt and clay, moist.	medium dense				8 8 8	16				
			5		8 10 10	12	15.5	108.6		At 6 Feet: Coarse Gravel = 8% Fine Gravel = 38% Coarse Sand = 18% Medium Sand = 18% Fine Sand = 9% Fines = 9%
No recovery.			10		8 10 14	14				
CLAY (CL), brown, silty, few sand (fine-grained), moist.	stiff									
			15		3 7 8	15				
	hard		20		14 15 22	37				
Bottom of Boring = 21.5 feet Notes: Stratification is approximate; variations must be expected. See report for additional details.			25							
			30							

EXPLORATORY BORING B-2

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

PROJECT NO: 1066-2	SURFACE ELEVATION: --
LOGGED / CHECKED BY: R. Ceraolo / T. Chen	DATE STARTED: 12/23/2024
DRILLER: West Coast Exploration	DATE FINISHED: 12/23/2024
DRILL RIG: Mobile B-24	DEPTH TO INITIAL WATER: 4 Feet
DRILLING METHOD: 4-inch Solid Flight Auger	DEPTH TO FINAL WATER: 4.5 Feet
HAMMER TYPE / WEIGHT / DROP: Safety Hammer with Rope & Cathead / 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 2 (38.274433°; -122.458703°)	

PROJECT:

20540 BROADWAY

Sonoma, California

SUBSURFACE MATERIAL CLASSIFICATION

DESCRIPTION	CONSIST	GRAPHIC LOG	DEPTH (FEET)	SAMPLER	FIELD BLOW COUNT	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS NOTES AND REMARKS
CLAY (CL)/SAND (SC), brown, silty, sandy (fine- to coarse-grained), few gravel (fine, subangular to subrounded), moist to wet.	firm		0		4 4 4	5	16.0	107.0	0.5	At 2 Feet: Non-plastic. Fine Gravel = 5% Coarse Sand = 7% Medium Sand = 11% Fine Sand = 40%. Fines = 37%
SAND (SM), brown, fine- to medium-grained, silty, few to with gravel (fine, subangular to subrounded), trace clay, moist to wet.	loose				3 3 3	6				At 3.5 Feet: Corrosion Tests.
SAND (SC), brown, fine- to coarse-grained, clayey, few gravel (fine, subangular to subrounded), wet.	medium dense		5		4 6 7	8				
GRAVEL (GC), mottled gray brown, fine to coarse, subangular to subrounded, clayey, sandy (fine- to coarse-grained), moist to wet.	loose / medium dense		10							
CLAY (CL), mottled light gray brown, silty, trace sand (fine-grained), moist.	stiff				11 7 7	14				
GRAVEL (GM)/SAND (SM), mottled gray brown, fine to coarse, subangular to subrounded, sandy (fine- to coarse-grained), with silt & clay, moist to wet. Hole caved to 20'.	medium dense / dense		15 / 20		10 12 25 / 12 15 32	22 / 47				
CLAY (CL), brown, sandy (fine- to medium-grained, few coarse-grained), silty, few gravel (fine, subangular to subrounded), moist.	very stiff		25		9 9 9	18				
GRAVEL (GM)/SAND (SM), mottled gray brown, fine to coarse, subangular to subrounded, sandy (fine- to coarse-grained), silty, trace clay, moist.	very dense		30							

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

EXPLORATORY BORING B-2

PROJECT NO: 1066-2	SURFACE ELEVATION: --
LOGGED / CHECKED BY: R. Ceraolo / T. Chen	DATE STARTED: 12/23/2024
DRILLER: West Coast Exploration	DATE FINISHED: 12/23/2024
DRILL RIG: Mobile B-24	DEPTH TO INITIAL WATER: 4 Feet
DRILLING METHOD: 4-inch Solid Flight Auger	DEPTH TO FINAL WATER: 4.5 Feet
HAMMER TYPE / WEIGHT / DROP: Safety Hammer with Rope & Cathead / 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 2 (38.274433°; -122.458703°)	

PROJECT:
20540 BROADWAY
Sonoma, California

SUBSURFACE MATERIAL CLASSIFICATION: DESCRIPTION	CONSIST	GRAPHIC LOG	DEPTH (FEET)	SAMPLER	FIELD BLOW COUNT	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS NOTES AND REMARKS
GRAVEL (GM)/SAND (SM), continued.	very dense				20 25 26	51				
			35		50/6"	50/6"				
Drilling refusal, clayey.					50/6"	50/6"				
Bottom of Boring = 38 feet Notes: Stratification is approximate; variations must be expected. See report for additional details.			40							
			45							
			50							
			55							
			60							

EXPLORATORY BORING LOG 1066-2 B-2.ldat8 STEVENS FERRONE & BAILEY 1/13/2025

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

EXPLORATORY BORING B-3

PROJECT NO: 1066-2	SURFACE ELEVATION: --
LOGGED / CHECKED BY: R. Ceraolo / T. Chen	DATE STARTED: 12/23/2024
DRILLER: West Coast Exploration	DATE FINISHED: 12/23/2024
DRILL RIG: Mobile B-24	DEPTH TO INITIAL WATER: 3 Feet
DRILLING METHOD: 4-inch Solid Flight Auger	DEPTH TO FINAL WATER: 5.5 Feet
HAMMER TYPE / WEIGHT / DROP: Safety Hammer with Rope & Cathead / 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 2 (38.274617°; -122.459683°)	

PROJECT:

20540 BROADWAY

Sonoma, California

SUBSURFACE MATERIAL CLASSIFICATION: DESCRIPTION	CONSIST	DEPTH (FEET)	FIELD BLOW COUNT	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS NOTES AND REMARKS
CLAY (CL)/SAND (SC), brown, silty, sandy (fine- to coarse-grained), few gravel (fine, subangular to subrounded), moist to wet.	firm to stiff	0						
			8 10 9	11	20.0	101.8	1.2	
With sand (fine- to medium-grained).								
SAND (SW-SM), mottled gray brown, fine- to coarse-grained, gravelly (fine, subangular to subrounded), few silt and clay, moist to wet.	medium dense		5 7 9	16				
		5	7 14 8	13	17.1	97.5		At 6 Feet: Coarse Gravel = 1% Fine Gravel = 40% Coarse Sand = 21% Medium Sand = 20% Fine Sand = 10% Fines = 8%
Wet.	dense	10	25 28 34	37				
GRAVEL (GP-GM), mottled gray brown, fine to coarse, subangular to rounded, sandy (fine- to coarse-grained), few silt, wet.	dense	15	15 18 19	37				
		20	9 12 10	22				
CLAY (CL), grayish brown, silty, sandy (fine- to medium-grained), moist.	very stiff							
Bottom of Boring = 21.5 feet Notes: Stratification is approximate; variations must be expected. See report for additional details.		25						
		30						

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

EXPLORATORY BORING B-4

PROJECT NO: 1066-2	SURFACE ELEVATION: --
LOGGED / CHECKED BY: R. Ceraolo / T. Chen	DATE STARTED: 12/23/2024
DRILLER: West Coast Exploration	DATE FINISHED: 12/23/2024
DRILL RIG: Mobile B-24	DEPTH TO INITIAL WATER: Not Encountered
DRILLING METHOD: 4-inch Solid Flight Auger	DEPTH TO FINAL WATER: Not Encountered
HAMMER TYPE / WEIGHT / DROP: Safety Hammer with Rope & Cathead / 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 2 (38.274272°; -122.460311°)	

PROJECT:

20540 BROADWAY

Sonoma, California

SUBSURFACE MATERIAL CLASSIFICATION

DESCRIPTION	CONSIST	GRAPHIC LOG
CLAY (CL), brown, silty, sandy (fine- to coarse-grained), few gravel (fine, subangular to subrounded), moist.	firm to hard	
SILT (ML), light grayish brown, sandy (fine- to medium-grained), with clay, dry.	hard	
CLAY (CL), brown, silty, trace sand (fine-grained), moist.	stiff	
	very stiff	
Interbedded with thin silty sand lens (fine- to medium-grained).		
Bottom of Boring = 21.5 feet Notes: Stratification is approximate; variations must be expected. See report for additional details.		

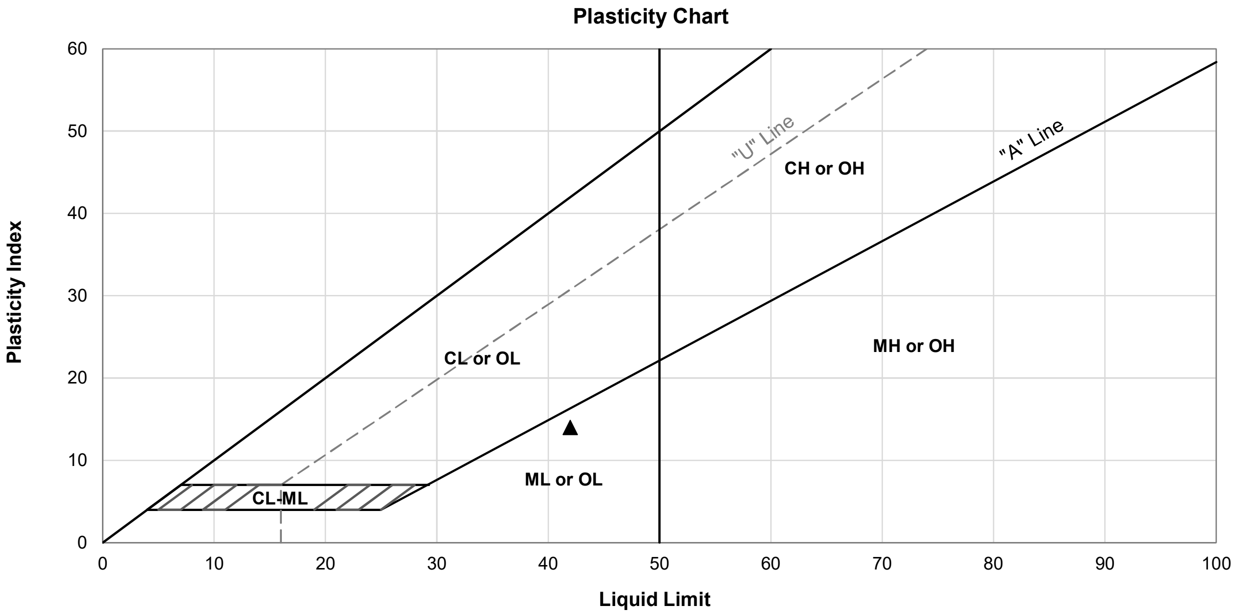
DEPTH (FEET)	SAMPLER	FIELD BLOW COUNT	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS NOTES AND REMARKS
0							
		9 30 50/6"	48	25.4	92.2	3.7	At 2 Feet: Liquid Limit = 42 Plasticity Index = 14 Fine Gravel = 2% Coarse Sand = 4% Medium Sand = 20% Fine Sand = 22% Fines = 52% Corrosion Tests.
		50/6"	50/6"				
5		50/6"	30/6"	25.0	92.1	7.5	
10		8 10 14	14				
15		10 21 22	26				
20		12 14 17	21				
25							
30							

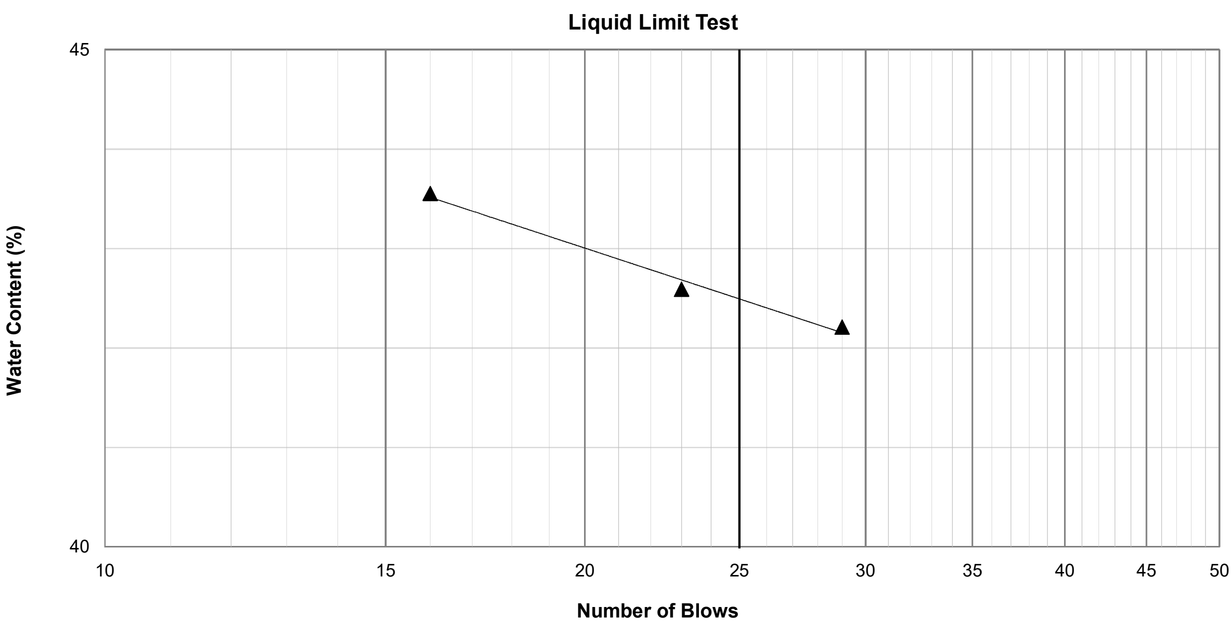
APPENDIX B

Laboratory Testing

ATTERBERG LIMITS

ASTM D4318





Test Results

Symbol	Sample Source	Sample Description	LL	PL	PI	%≤ #200	USCS
■	Boring B-2 at 2 ft.	Brown silty SAND trace gravel	-	-	NP*	36.8	SM
▲	Boring B-4 at 2 ft.	Brown sandy clayey SILT	42	28	14	52.5	ML

Project Info

Project Number: 1066-2

Project Name: 20540 Broadway

Project Location: Sonoma, CA

Tested by: R. Tuazon

Checked by: T. Chen

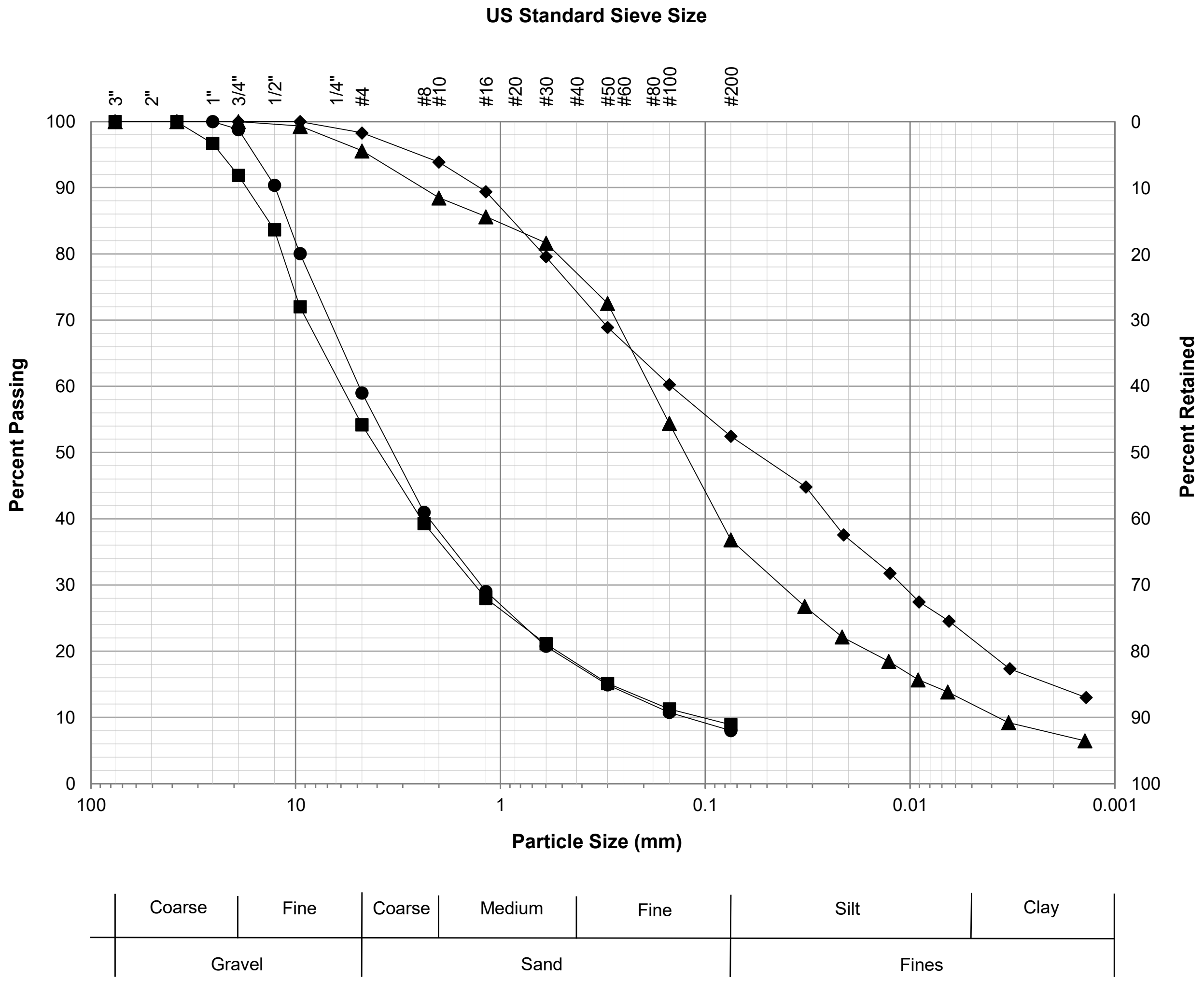
Test Report Date: 1/8/2025

*Note: NP = Non-plastic

Stevens Ferrone & Bailey Engineering Company, Inc

PARTICLE SIZE DISTRIBUTION

ASTM C136, D422 & D1140



Test Results

Symbol	% >3"	% Gravel		% Sand			% Fines		D_{60}	D_{30}	D_{10}	C_u	C_c
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay					
■	0.0	8.2	37.7	17.6	18.4	9.3	8.9		5.962	1.336	0.103	57.628	2.892
▲	0.0	0.0	4.4	7.1	11.4	40.3	36.8		0.186	0.043	0.004	50.085	2.650
●	0.0	1.2	39.8	20.9	20.3	9.8	8.0		4.909	1.247	0.123	39.791	2.569
♦	0.0	0.0	1.7	4.4	19.6	21.8	52.4		0.147	0.011			

Sample Info

Symbol	Sample Source	Sample Description	USCS
■	Boring B-1 at 6 ft.	Red brown sandy GRAVEL some silt	GW-GM
▲	Boring B-2 at 2 ft.	Brown silty SAND trace gravel	SM
●	Boring B-3 at 6 ft.	Red brown gravelly SAND some silt	SW-SM
♦	Boring B-4 at 2 ft.	Brown sandy clayey SILT	ML

Project Info

Project Number: 1066-2 Test Report Date: 1/6/2025

Project Name: 20540 Broadway

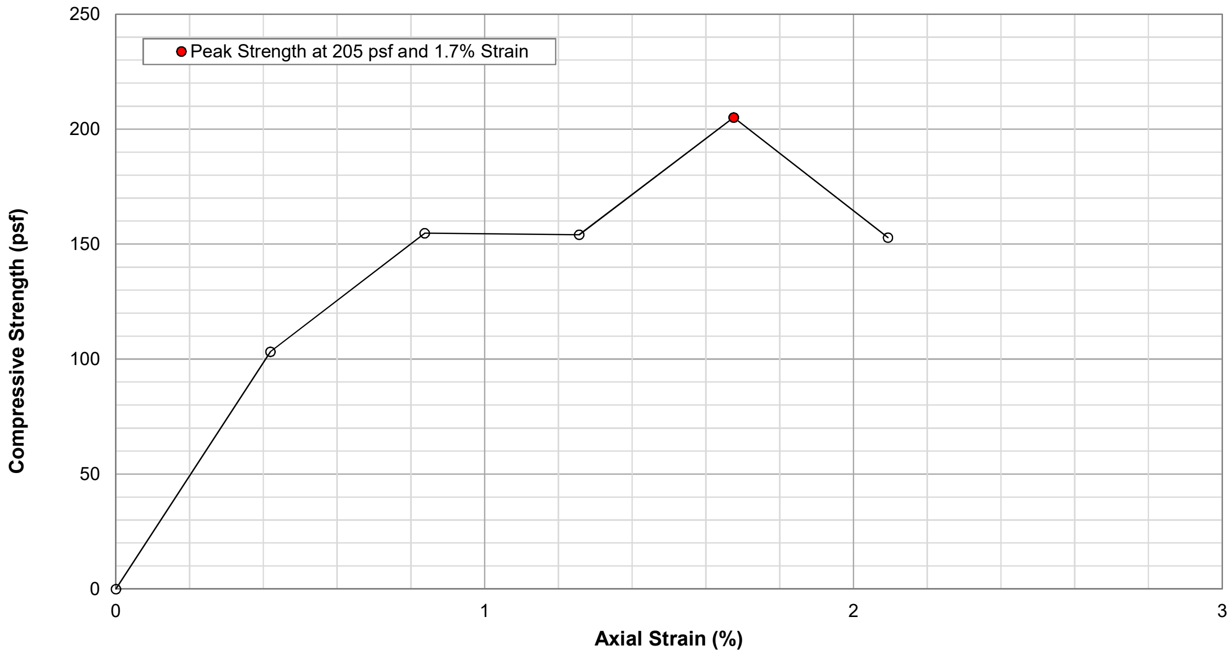
Project Location: Sonoma, CA

Tested by: R. Tuazon

Checked by: T. Chen

UNCONFINED COMPRESSIVE STRENGTH

ASTM D2166



Sample Images

Before Test	After Test	After Test
		

Sample Properties

Sample Description:	Brown sandy GRAVEL some clay (GC/SC)	
Diameter:	2.42	in
Height:	5.97	in
Height/Diameter:	2.47	
Wet Unit Weight:	129.9	pcf
Water Content:	14.4	%
Dry Unit Weight:	113.6	pcf

Test Results

Test Date:	1/2/2025	
Compressive Strength:	205	psf
Axial Strain at Failure:	1.7	%
Test Strain Rate:	0.05	in/min
Test Time to Failure:	2	min
Remarks:		

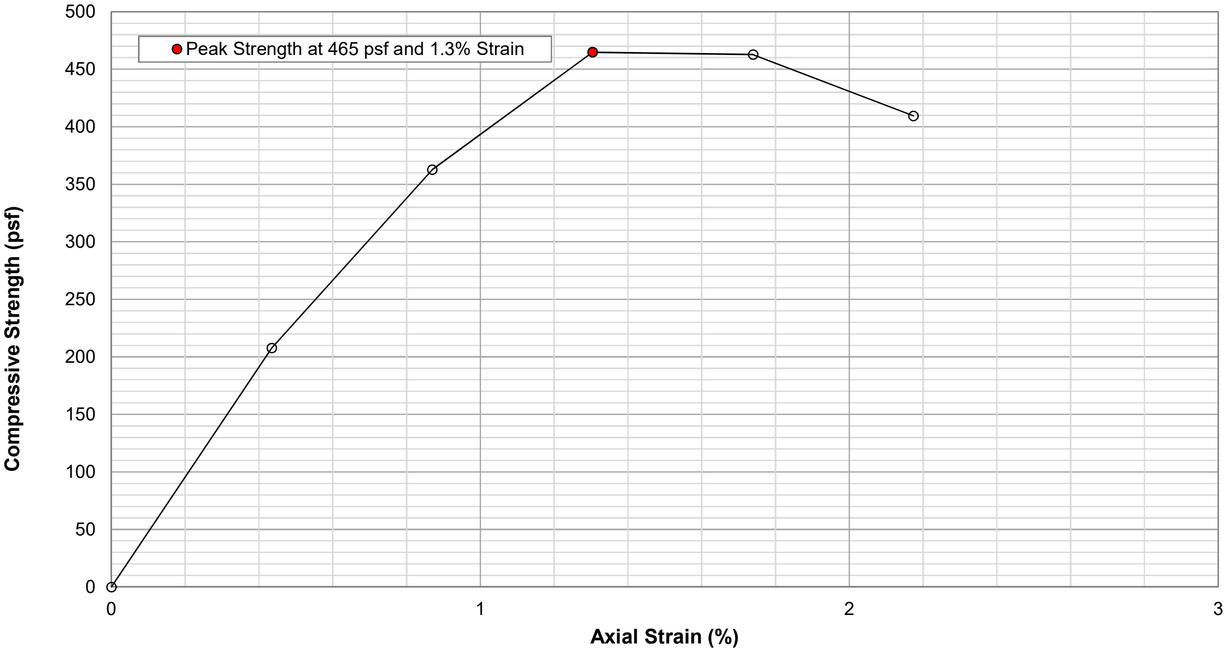
Project Info

Project Number:	1066-2
Project Name:	20540 Broadway
Project Location:	Sonoma, CA
Sample Source/No.:	Boring B-1
Sample Depth:	2 ft.
Tested by:	R. Tuazon
Checked by:	T. Chen

Test Report Date: 1/3/2025

UNCONFINED COMPRESSIVE STRENGTH

ASTM D2166



Sample Images

Before Test

After Test

After Test

Sample Properties

Sample Description:	Brown silty SAND trace gravel (SM)	
Diameter:	2.42	in
Height:	5.75	in
Height/Diameter:	2.38	
Wet Unit Weight:	124.1	pcf
Water Content:	16.0	%
Dry Unit Weight:	107.0	pcf

Test Results

Test Date:	1/2/2025	
Compressive Strength:	465	psf
Axial Strain at Failure:	1.3	%
Test Strain Rate:	0.05	in/min
Test Time to Failure:	1.5	min
Remarks:		

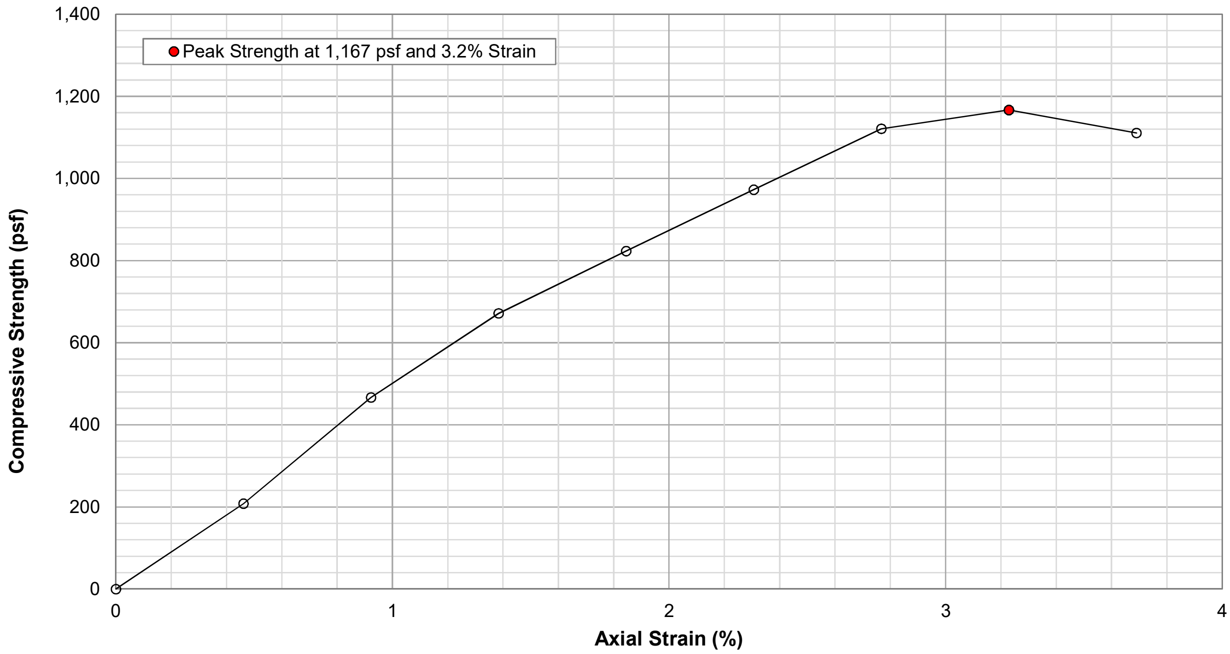
Project Info

Project Number:	1066-2
Test Report Date:	1/3/2025
Project Name:	20540 Broadway
Project Location:	Sonoma, CA
Sample Source/No.:	Boring B-2
Sample Depth:	2 ft.
Tested by:	R. Tuazon
Checked by:	T. Chen

Stevens Ferrone & Bailey Engineering Company, Inc

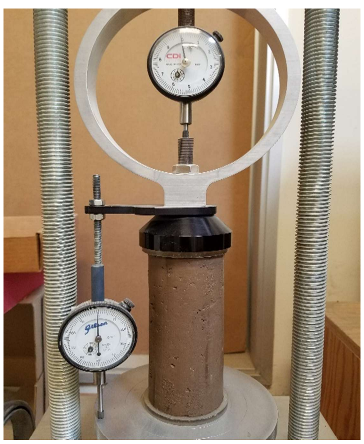
UNCONFINED COMPRESSIVE STRENGTH

ASTM D2166

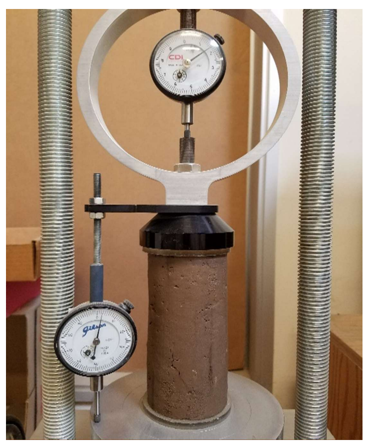


Sample Images

Before Test



After Test



After Test

Sample Properties

Sample Description:	Brown sandy silty CLAY trace gravel (CL/SC)	
Diameter:	2.42	in
Height:	5.42	in
Height/Diameter:	2.24	
Wet Unit Weight:	122.2	pcf
Water Content:	20.0	%
Dry Unit Weight:	101.8	pcf

Test Results

Test Date:	1/2/2025	
Compressive Strength:	1,167	psf
Axial Strain at Failure:	3.2	%
Test Strain Rate:	0.05	in/min
Test Time to Failure:	3.5	min
Remarks:		

Project Info

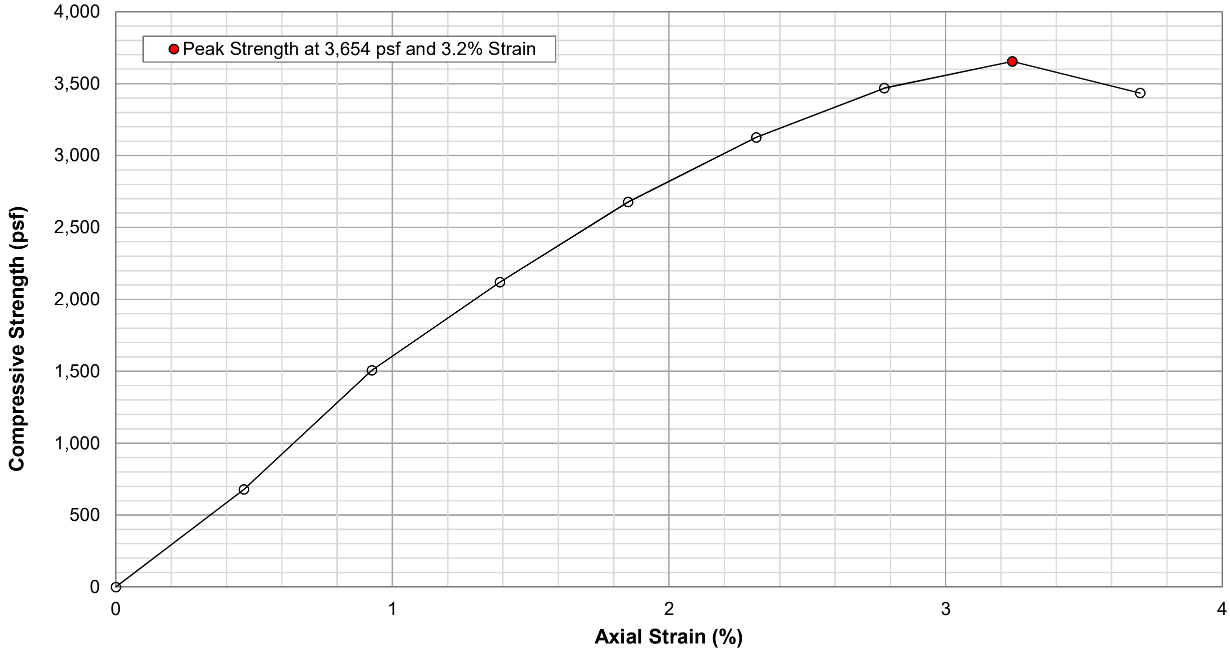
Project Number:	1066-2
Project Name:	20540 Broadway
Project Location:	Sonoma, CA
Sample Source/No.:	Boring B-3
Sample Depth:	2 ft.
Tested by:	R. Tuazon
Checked by:	T. Chen

Test Report Date: 1/3/2025

Stevens Ferrone & Bailey Engineering Company, Inc

UNCONFINED COMPRESSIVE STRENGTH

ASTM D2166



Sample Images

Before Test

After Test

After Test

Sample Properties

Sample Description:	Brown sandy clayey SILT (ML)	
Diameter:	2.42	in
Height:	5.4	in
Height/Diameter:	2.23	
Wet Unit Weight:	115.6	pcf
Water Content:	25.4	%
Dry Unit Weight:	92.2	pcf

Test Results

Test Date:	1/2/2025	
Compressive Strength:	3,654	psf
Axial Strain at Failure:	3.2	%
Test Strain Rate:	0.05	in/min
Test Time to Failure:	3.5	min
Remarks:		

Project Info

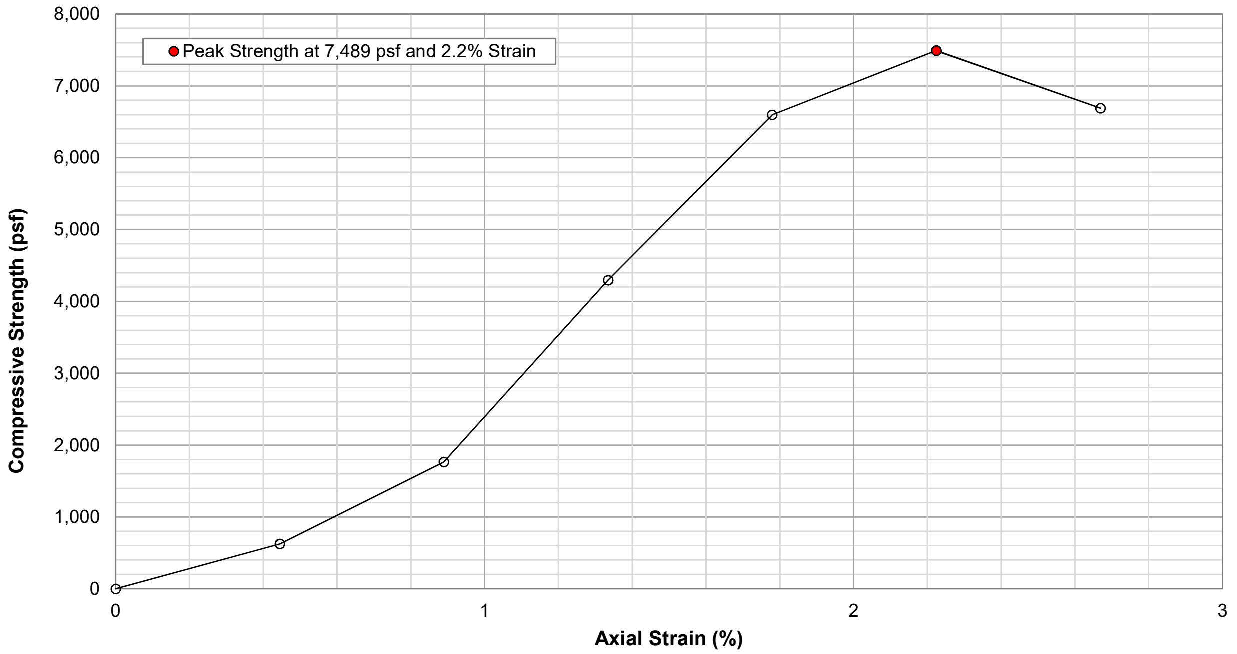
Project Number:	1066-2
Project Name:	20540 Broadway
Project Location:	Sonoma, CA
Sample Source/No.:	Boring B-4
Sample Depth:	2 ft.
Tested by:	R. Tuazon
Checked by:	T. Chen

Test Report Date: 1/3/2025

Stevens Ferrone & Bailey Engineering Company, Inc

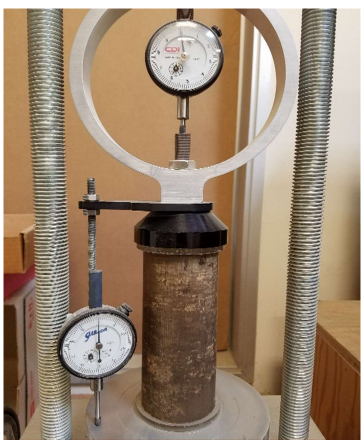
UNCONFINED COMPRESSIVE STRENGTH

ASTM D2166

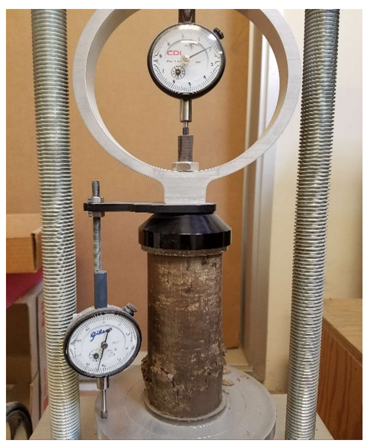


Sample Images

Before Test



After Test



After Test

Sample Properties

Sample Description:	Brown silty CLAY some sand (CL)	
Diameter:	2.42	in
Height:	5.62	in
Height/Diameter:	2.32	
Wet Unit Weight:	115.1	pcf
Water Content:	25.0	%
Dry Unit Weight:	92.1	pcf

Test Results

Test Date:	1/2/2025	
Compressive Strength:	7,489	psf
Axial Strain at Failure:	2.2	%
Test Strain Rate:	0.05	in/min
Test Time to Failure:	2.5	min
Remarks:		

Project Info

Project Number:	1066-2
Project Name:	20540 Broadway
Project Location:	Sonoma, CA
Sample Source/No.:	Boring B-4
Sample Depth:	5 ft.
Tested by:	R. Tuazon
Checked by:	T. Chen

Test Report Date: 1/3/2025

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

2 January, 2025

Job No. 2412053
Cust. No. 11486

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

Subject: Project No.: 1066-2
Project Name: 20540 Broadway, Sonoma, CA
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Chen:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on December 26, 2024. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

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

The chloride ion concentrations 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 soil is 6.50 and 6.55, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials are 270-mV and 280-mV. Both samples are indicative of potentially "slightly corrosive" soils.

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.

Julio Ch for

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

JDH/jdl
Enclosure

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

Client: Stevens, Ferrone & Bailey Engineering
Client's Project No.: 1066-2
Client's Project Name: 20540 Broadway, Sonoma, CA
Date Sampled: 23-Dec-24
Date Received: 26-Dec-24
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 2-Jan-2025

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)*
2412053-001	B-2 at 3.5'	270	6.55	-	6,700	N.D.	N.D.	N.D.
2412053-002	B-4 at 2'	280	6.50	-	2,800	N.D.	N.D.	N.D.

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reportring Limit:	-	-	10	-	50	15	15
Date Analyzed:	2-Jan-2025	27-Dec-2024	-	31-Dec-2024	2-Jan-2025	30-Dec-2024	30-Dec-2024

Julia Clauson

Julia Clauson
Chemist

* Results Reported on "As Received" Basis

N.D. - None Detected

<u>Quality Control Summary</u> - All laboratory quality control parameters were found to be within established limits

APPENDIX C

Soil Liquefaction Analyses

Stevens Ferrone & Bailey Engineering Company, Inc

LIQUEFACTION ANALYSES BASED ON SPT DATA
20540 BROADWAY, SONOMA, CALIFORNIA

SFB 1066-2
2025/1/8 TC

a_{max} =	0.87	g	ASCE 7-16	PGAm
Mw =	7.5			

B-1

Ground Elevation =				ft		
Depth to Ground Water Table =			5	ft	= EL	ft
γ =	120	pcf				
γ_{sat} =	125	pcf				
Boring Diameter =	5	inch =	127	mm		
Rod Length Above Ground =	3	ft =	0.9	m		

Elevation	Depth	Depth	N	σ_v	σ_v	σ_v'	σ_v'	C_R	Liner Correction	C_S	C_B	C_E	C_N	$N_{1,60}$	FC
ft	ft	m	blow/ft	psf	kPa	psf	kPa		Y/N						%
	6.0	1.8	12	725.0	34.8	662.6	31.8	0.80	N	1.00	1.05	1.0	1.70	17	9
	11.0	3.4	14	1,350.0	64.7	975.6	46.8	0.85	N	1.00	1.05	1.0	1.47	18	10

Depth	ΔN	$N_{1,60,cs}$	r_d	CSR	BI2014 MSF_{max}	MSF	K_σ	$CRR_{M=7.5,1 atm}$	CRR	FS	γlim	Yoshimine 2006 $F\alpha$	γmax	ε_v	ΔH (ft)	Δs (in)
ft		blow/ft												%	ft	in
6.0	1	18	0.99	0.61	1.41	1.00	1.10	0.18	0.20	0.3	0.202	0.626	0.202	2.5	3.5	1.1
11.0	1	19	0.98	0.77	1.47	1.00	1.10	0.20	0.22	0.3	0.169	0.546	0.169	2.4	5.0	1.4
													Total	2.4	8.5	2.5

LIQUEFACTION ANALYSES BASED ON SPT DATA
20540 BROADWAY, SONOMA, CALIFORNIA

Stevens Ferrone & Bailey Engineering Company, Inc

SFB 1066-2
2025/1/8 TC

a_{max} = 0.87 g ASCE 7-16 PGAm
Mw = 7.5

B-2
Ground Elevation = ft
Depth to Ground Water Table = 5 ft = EL ft
γ = 120 pcf
γ_{sat} = 125 pcf
Boring Diameter = 5 inch = 127 mm
Rod Length Above Ground = 3 ft = 0.9 m

Elevation	Depth	Depth	N	σ_v	σ_v	σ_v'	σ_v'	C_R	Liner Correction	C_S	C_B	C_E	C_N	$N_{1,60}$	FC	ΔN	$N_{1,60,cs}$	r_d	CSR	BI2014 MSF_{max}	MSF	K_σ	$CRR_{M=7.5,1\ atm}$	CRR	FS	γlim	Yoshimine 2006 $F\alpha$	γmax	ε_v	ΔH (ft)	Δs (in)
ft	ft	m	blow/ft	psf	kPa	psf	kPa		Y/N						%		blow/ft												%	ft	in
	6.0	1.8	8	725.0	34.8	662.6	31.8	0.80	N	1.00	1.05	1.0	1.70	11	30	5	17	0.99	0.61	1.37	1.00	1.10	0.17	0.19	0.3	0.227	0.677	0.227	2.6	1.5	0.5
	11.0	3.4	14	1,350.0	64.7	975.6	46.8	0.85	Y	1.14	1.05	1.0	1.47	21	30	5	26	0.98	0.77	1.78	1.00	1.10	0.32	0.35	0.5	0.076	0.155	0.076	1.7	4.5	0.9
	16.0	4.9	22	1,975.0	94.7	1,288.6	61.8	0.95	N	1.00	1.05	1.0	1.23	27	15	3	30	0.96	0.83	2.01	1.00	1.10	0.50	0.55	0.7	0.045	-0.108	0.045	0.9	4.0	0.4
																												Total	1.5	10.0	1.8

Stevens Ferrone & Bailey Engineering Company, Inc

LIQUEFACTION ANALYSES BASED ON SPT DATA
20540 BROADWAY, SONOMA, CALIFORNIA

SFB 1066-2
2025/1/8 TC

a_{max} =	0.87	g	ASCE 7-16	PGAm
Mw =	7.5			

B-3

Ground Elevation =				ft		
Depth to Ground Water Table =			5	ft	= EL	ft
γ =	120	pcf				
γ_{sat} =	125	pcf				
Boring Diameter =	5	inch =	127	mm		
Rod Length Above Ground =	3	ft =	0.9	m		

									Liner											BI2014							Yoshimine 2006				
Elevation	Depth	Depth	N	σ_v	σ_v	σ_v'	σ_v'	C_R	Correction	C_S	C_B	C_E	C_N	$N_{1,60}$	FC	ΔN	$N_{1,60,cs}$	r_d	CSR	MSF_{max}	MSF	K_σ	$CRR_{M=7.5,1\ atm}$	CRR	FS	γlim	$F\alpha$	γmax	ε_v	ΔH (ft)	Δs (in)
ft	ft	m	blow/ft	psf	kPa	psf	kPa		Y/N						%		blow/ft												%	ft	in
	6.0	1.8	13	725.0	34.8	662.6	31.8	0.80	N	1.00	1.05	1.0	1.70	19	8	0	19	0.99	0.61	1.45	1.00	1.10	0.19	0.21	0.3	0.179	0.573	0.179	2.4	3.0	0.9
	21.0	6.4	22	2,600.0	124.6	1,601.6	76.8	0.95	Y	1.22	1.05	1.0	1.12	30	10	1	31	0.94	0.87	2.07	1.00	1.06	0.57	0.61	0.7	0.039	-0.171	0.039	0.7	3.0	0.3
																												Total	1.6	6.0	1.1

APPENDIX D

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 <u>not</u> 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 <u>Not</u> 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 <u>not</u> 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 <u>not</u> building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org