Stevens Ferrone & Bailey
Engineering Company, Inc

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

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GEOTECHNICAL INVESTIGATION
2840 PARK AVENUE
SOQUEL, CALIFORNIA
SFB PROJECT NO. 940-2

Prepared For:

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1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed residential development to be located at 2840 Park Avenue in Soquel, California as shown on the Vicinity Map, Figure 1, and Site Plan, Figure 2. The purpose of our investigation was to evaluate the geotechnical conditions at the site and provide recommendations regarding the geotechnical engineering aspects of the project.

Based on the project conceptual site plan (Option 3) prepared by Ifland Engineers and dated November 9, 2021, we understand the project consists of developing a vacant land parcel of about 3/4 acre for a new at-grade multi-family residential building of about 60 feet by 140 feet in footprint area. The rear half of the building (east side) at the lower elevation will include a tuck under parking garage. Several asphalt concrete paved parking stalls are also proposed to be located to the southwest of the new building at the higher elevation. Cut and fill grading of up to about 5 feet is anticipated for the planned project development. A new retaining wall up to about 5 feet high will be constructed around the eastern and southern development boundaries to retain the new building pad and new paved sloped access driveway. Underground utilities will also be installed.

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

2.0 SCOPE OF WORK

This investigation included the following scope of work:

- Reviewing published and unpublished geotechnical and geological literature relevant to the site;
- Reviewing historical aerial images and topographic maps of the site and surrounding area;
- Performing reconnaissance of the site and surrounding area;
- Performing five exploratory borings to a maximum depth of about 21-1/2 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, 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 five exploratory borings on December 9, 2021, to a maximum depth of about 21-1/2 feet. The approximate locations of the borings are shown on the Site Plan, Figure 2. The borings were performed by Cenozoic Exploration of Aptos, California, using a truck-mounted Mobile B-24 drill rig (equipped with 4-inch diameter, continuous flight, solid stem augers and a 140-pound safety hammer) and a track-mounted Geoprobe 7822DT drill rig (equipped with 7-inch diameter, continuous flight, hollow stem augers and a 140-pound automatic trip hammer).

Our representative 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 borings as well as a key for the classification of the soil (Figure A-1) are included in Appendix A. Upon completion of our field exploration, the borings were backfilled in accordance with Santa Cruz County requirements.

The approximate locations of our borings were determined by pacing, measurements, and/or alignment from landmark references, and should be considered accurate only to the degree implied by the method used. Latitude and longitude of boring locations shown on the boring logs are estimated from online map data from Microsoft; actual locations were not surveyed. Elevations shown on the exploration logs were estimated from the project conceptual site plan (Option 3) prepared by Ifland Engineers and dated November 9, 2021 (datum unknown).

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. Sampler types are indicated in the "Sampler" column of the boring logs as designated in Figure A-1. All samples were transported to our geotechnical laboratory for evaluation and appropriate testing.

Resistance blow counts (N-value) were obtained in our borings with the samplers by dropping either a 140-pound safety hammer through a 30-inch fall with rope and cathead (assumed a hammer efficiency of about 60%) or a 140-pound automatic trip hammer through a 30-inch fall (assumed a hammer efficiency of about 84%). The SPT and Modified California samplers were driven 18 inches and the number of blows were recorded for each 6 inches of penetration. The blows per foot recorded on the boring logs represent the accumulated number of blows that were required to drive the last 12 inches, or the number of inches indicated where hard resistance was encountered. A sampler barrel size correction factor of 0.6 was applied to the blow counts from the Modified California sampler. In addition, the recorded blow counts (N_{60}) shown on our boring logs had been corrected to a standard 140-pound hammer that delivers 60% of energy. An energy ratio correction factor of 1.4 was used for the automatic trip hammer. The recorded blow counts have not been corrected for other factors, such as 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 (plastic and liquid limits) determination per ASTM D4318.
- Two sieve and hydrometer tests per ASTM D422.
- Five unconfined compressive strength test per ASTM D2166.

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

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

3.3 Site History and Surface Description

At the time of our field exploration and as shown on Figure 2, the site was bounded by Park Avenue and an existing office development with associated parking lot on the northwest, an existing residential townhome development to the northeast, an unnamed southwesterly flowing small creek on the southeast, and Cabrillo College Drive and open space slopes on the southwest.

The site was vacant, irregular in shape, and had a plan area of about 3/4 acre. The general site grade sloped downward toward the creek at the southeast with slope inclinations varying from 2:1 (horizontal to vertical) to 10:1. The average slope inclination adjacent to the creek was about 5:1. An incised drainage swale was also located along the southwestern site boundary, which discharged southeasterly to the creek. The site ground surface was wet and soft at the time of our field exploration, and covered with dense vegetation that included grasses, weeds, and trees. Large mature trees were generally located on slopes along the site eastern and southern boundaries. Wood and concrete debris was also observed within the site.

Based on our review of historical topographical maps and aerial photographs of the site and vicinity, a small unknown structure appeared to exist within the planned development area until around the 1950's. The existing office buildings to the northwest were built in the 1950's and 1980's. Cut and fill grading likely has been performed in the past to create the existing level office development pad with fills being placed along the eastern edge of the pad (where the existing gravel covered parking area and the steeper site slopes at the higher elevations were located). The placement and compaction of the previous fills most likely do not meet the current accepted geotechnical engineering standards (which also generally require keying, benching, and subdrainage for fills placed on slopes) and should not be considered as engineered fills. The residential townhome development to the north was built in the 1980's.

3.4 Subsurface

Based on the results of field exploration, our Borings B-1, B-2, and B-5 (located at the higher elevations of the site) encountered an about 3-1/2 to 8-foot thick layer of sandy and clayey fills. These undocumented fills were heterogenous, weak and loose and did not appear to be properly compacted and engineered. Borings B-3 and B-4 (located at the lower elevations of the site) encountered an about 3- to 5-foot layer of loose, weak, slope wash sands. Below these surficial loose and weak fill and soil layers, medium dense to very dense sands, stiff clays, and hard silts (or completely weathered siltstone) were encountered that extended to the maximum depth explored of about 21-1/2 feet.

The results of laboratory testing indicate that the surficial more clayey fills and soils have a moderate plasticity and a moderate expansion potential. However, the sandy fills and soils have a low plasticity and a low expansion potential. Detailed descriptions of soils encountered in our exploratory borings are presented on the boring logs in Appendix A. Results of laboratory testing of retrieved onsite soils are included in Appendix B.

3.5 Groundwater

Groundwater was encountered in Borings B-3 and B-4 at a depth of about 15 feet (or elevations of about 101 to 105 feet, datum unknown) which appeared to be perched on top of the underlying hard silt or completely weathered siltstone layer. No groundwater was encountered in the other borings to the maximum depth explored in these borings of about 21-1/2 feet. It should be noted that our borings might not have been left open for a sufficient period of time to establish equilibrium groundwater conditions.

In addition, fluctuations in the groundwater level could occur due to change in seasons, variations in rainfall, hillside seepage, water flow in the adjacent creek, and other factors. It is likely that during rainfall events, localized groundwater or seepage may develop within the fills and soils below the site and on the hillside slopes, and will seep toward lower elevations.

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)[1] and categorized into the following three map units:

a) Tierra-Watsonville complex, 15 to 30 percent slopes (Unit 174);
b) Watsonville loam, thick surface, 0 to 2 percent slopes (Unit 178); and
c) Watsonville loam, thick surface, 2 to 15 percent slopes (Unit 179).

All three units have been assigned to Hydrologic Soil Group D and were estimated to have very low to moderately low transmission rates (approximately 0 to 0.06 inches 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 or clay layer at or near the surface, and soils that are shallow over nearly impervious material.

According to the results of field borings and laboratory testing, the site is underlain by various clay, sand, and silt (or completely weathered siltstone) layers that are expected to have a wide

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[1]USDA NRCS, https://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx, accessed 12/23/2021.

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

3.7 Geology and Seismicity

According to Brabb, et al. (1997)[2], the site (below surficial fills and slope wash soils) is underlain by Pleistocene lowest emergent coastal terrace deposits that consist of semi-consolidated, generally well-sorted sand with a few thin, relatively continuous layers of gravel. These deposits are likely underlain by Pliocene and upper Miocene Purisima formation bedrock that consists of very thick bedded yellowish-gray tuffaceous and diatomaceous siltstone containing thick interbeds of bluish-gray, semi-friable, fine-grained andesitic sandstone.

According to U.S. Geological Survey Open-File Report 97-745 (1997)[3], the site is mapped as flat land with little or no potential for landslides or earth flows, and is not located within an area having debris flow source potential. In addition, Cooper-Clark & Associates (1975)[4] do not map any landslide deposits at the site or in the vicinity of the site. In addition, during our field reconnaissance, we did not observe evidence of deep landsliding and adverse drainage conditions within the site. However, the site slope surface is generally blanketed by loose and weak fills and slope wash soils that are prone to surface erosion and slumping.

It is our opinion that, based on the results of geologic literature review, field reconnaissance, and exploratory borings, the potential for landsliding at the planned development is low provided the recommendations contained in this report (which include removal and re-compaction of the existing fills and slope wash soils with appropriate keying, benching, and subdrainage, and setting back improvements from slopes) are implemented in the design and construction of the project.

The project site is located 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 San Francisco Bay Area which are associated with crustal movements along a system of sub-parallel

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[2]Brabb, Graham, Wentworth, Knifong, Graymer, and Blissenbach, 1997, Geology Map of Santa Cruz County, California, U.S. Geological Survey Open-File Report 97-489.

[3]Ellen, Mark, Wieczorek, Wentworth, Ramsey, and May, 1997, San Francisco Bay Region Landslide Folio, Part C (Summary Distribution of Slides and Earth Flows) and Part E (Debris Flow Source Maps), U.S. Geological Survey Open-File Report 97-745.

[4]Cooper-Clark & Associates, 1975, Preliminary Map of Landslide Deposits in Santa Cruz County, California, U.S. Geological Survey Open-File Report 98-792.

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].

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 2019 CBC/ASCE 7-16, the site geometric mean peak ground acceleration (PGA_M) from a Maximum Considered Earthquake (MCE) event is estimated to be about 0.90g based on a stiff soil condition (Site Class D). The MCE peak ground acceleration 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 0.53g in 50 years (a ground motion based on a stiff soil condition, Site Class D, with a mean return time of 475 years).

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

3.8 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, Earthquake Outlook for the San Francisco Bay Region 2014–2043, USGS Fact Sheet 2016–3020, Revised August 2016 (ver. 1.1).
[7] USGS Unified Hazard Tool, https://earthquake.usgs.gov/hazards/interactive/, accessed 12/23/2021.

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 planned development area is mapped by Dupre (1975)[9] as being within an area having a low liquefaction potential. The area within and immediately adjacent to the creek is mapped as having a high liquefaction potential.

Based on our review of available literature and the results of field exploration at the site, it is our opinion that the potential for ground surface damage at the planned development resulting from liquefaction is low since stiff clays, dense to very dense sands, and hard silts (completely weathered siltstone) exist below the site at shallow depths; soils and bedrock that is resistant to soil liquefaction.

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[8]Seismic Hazards Mapping Act, 1990.

[9]Dupré, 1975, Maps Showing Geology and Liquefaction Potential of Quaternary Deposits in Santa Cruz County, California, USGS Miscellaneous Field Studies Map MF-648, Sheet 2.

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 AND FILL MATERIALS: As described in Section 3.4, the results of our field exploration indicate the site is blanketed by about 3-1/2 to 8 feet of sandy and clayey undocumented fills at the higher elevations and about 3 to 5 feet of slope wash sands at the lower elevations. These undocumented fills and slope wash sands are loose, weak, potentially compressible, and prone to surface erosion and slumping.

In order to reduce the potential for damaging differential settlement of overlying improvements (such as new fills, building foundations, retaining walls, driveways, exterior flatwork, and pavements) and to improve site slope stability, we recommend that these weak fills and soils be completely over-excavated and re-compacted within the planned development area. The over-excavation should extend to depths where competent soils are encountered.

Over-excavation and re-compaction should extend at least 5 feet beyond building and retaining wall footprints and at least 3 feet beyond exterior flatwork and pavement wherever possible. There would be no need to over-excavate and re-compact the soils and fills within areas that do not support improvements, such as within open spaces. Where the over-excavation limits abut adjacent property, SFB should be consulted to determine the actual vertical and lateral extent of over-excavation so that adjacent property is not adversely impacted. Over-excavations should be performed so that no more than 5 feet of differential fill thickness exists below proposed building foundations. The actual 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 operations.

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

CUT/FILL TRANSITIONS AND DIFFERENTIAL FILL THICKNESS: Proposed grading and the recommended weak soil and fill removal may result in cut/fill transitions across the building pad and differential fill thickness greater than 5 feet below building foundations. In order to reduce the potential for excessive differential movement across the proposed building foundations, we recommend that foundations bear entirely on an engineered fill layer of at least 3 feet thick and that no more the 5 feet of differential fill thickness exist below foundations. Over-excavation and re-compaction below foundations in will likely be necessary in some areas to satisfy this criterion.

DIFFERENTIAL EXPANSION POTENTIAL: The results of laboratory testing indicate that the surficial more clayey fills and soils have a moderate plasticity and a moderate expansion potential. However, the sandy fills and soils have a low plasticity and a low expansion potential. In order to provide a more uniform subgrade and reduce the potential for damaging differential movement of building foundations, we recommend the proposed grading be performed so that the building foundation and surrounding flatwork be supported on fills with similar expansion potential. In no case should a building be underlain by subgrade (and within 3 feet of subgrade) consisting of both expansive clayey soils or fills and relative non-expansive sandy soils or fills.

We recommend a layer least 3 feet thick of well-blended, moisture conditioned, engineered fill be provided below the building foundations and surrounding flatwork. The compacted, engineered fill layers should extend at least 5 feet beyond building footprint and at least 3 feet beyond exterior flatwork, including driveways. In addition, we recommend fill slopes be also built using well-blended, moisture conditioned, engineered fill to reduce the potential for slope expansion and creeping.

Our representative should be onsite during over-excavation and replacement to observe and test fill placement operations. The actual depth and lateral extent of removal and replacement should be determined in the field by SFB at the time of the earthwork operations; we recommend SFB prepare a geotechnical improvement plan showing the approximate lateral extent and depth of the recommended over-excavations.

The more clayey, expansive, onsite soil materials will be subjected to volume changes during seasonal fluctuations in moisture content. To reduce the potential for post-construction distress to the proposed building resulting from swelling and shrinkage of these materials, we recommend that the proposed building be supported on a foundation system that is designed to reduce the impact of the expansive soils. It should be noted that special design considerations will be required for exterior slabs.

SETBACKS FROM SLOPES: In order to reduce damage of improvements caused by potential slope erosion and slumping, appropriate slope setbacks should be used for the project. We recommend setbacks be established by projecting a 3:1 (horizontal to vertical) line from toe of slopes upward toward the improvements. Where the projected line intersects the finished ground surface, we recommend improvements be setback at least 5 feet from the intersection or at least 5 feet from top of the slope, whichever is greater. Buildings and structures should be setback at least 10 feet from the intersection or at least 10 feet from top of the slope, whichever is greater.

We should be further consulted to provide design alternatives if it is impractical to setback improvements, such as using deepened edges for the improvements or retaining walls. We recommend the project Civil Engineer determine the actual improvement and building setback based upon the recommendations provided in this report, California Building Code and local ordinances, and any other restrictions. Improvements located between the setback line and the slope may experience movement as a result of slope erosion, localized slumping, earthquake shaking, and other factors.

CORROSION POTENTIAL: Two onsite soil samples were tested for pH (ASTM D4972), chlorides (ASTM D4327), sulfates (ASTM D4327), sulfides (ASTM D4658M), resistivity at 100% saturation (ASTM G57), and Redox potential (ASTM D1498) for use in evaluating the potential for corrosion on concrete and buried metal, such as utilities and reinforcing steel. The results of these tests and 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.

SEEPAGE, SURFACE, AND SUBSURFACE WATER: Water seepage will occur during and after periods of rainfall and as a result of irrigation by "upstream" neighbors. To reduce the potential for seepage below and within planned improvements, we recommend installing subdrains where surface and seepage water is directed toward planned improvements such along the upslope sides of roadways when roadways are located on or near hillsides. After construction is complete, seepage may occur below the ground surface resulting from irrigation and storm water flow develop over time. Surface water should not be allowed to flow over the top of slopes and retaining walls. The actual location and extent of subdrains should be assessed by SFB during the development of the grading and improvement plans, and determined in the field by SFB at the time of construction.

EROSION AND SLOPE MAINTENANCE: Drainage and erosion control measures should be maintained during and after construction. Short-term and long-term erosion control are critical for the stability of any exposed cut and fill slopes, and may be necessary for the natural slopes in order to reduce sediment accumulation in the drainage systems. We recommend all exposed cut and fill slopes be seeded or planted with appropriately designed erosion resistant vegetation and fertilizer. The vegetation should be appropriately irrigated in order to establish and maintain growth. Over-watering must be avoided in order to reduce surficial instability and erosion. Vegetation should be deeply rooted to aid in the interlocking of the near-surface soils. Additional seeding and planting may be necessary in localized areas if the initial seeding or planting is unsuccessful. After seeding, fertilizing, and planting, staked erosion control blankets might be necessary to further stabilize the surficial soils.

Additional erosion control measures will need to be designed and implemented prior to the rainy season based upon the site's configuration. The measures could include straw wattles, silt fencing, hay bales, sediment collection basins, and filtration systems. Silt fencing should be designed for the site's soil type. Storm water discharge and release points from silt fencing should be designed to reduce erosion. In areas exposed to winter rains, we recommend an erosion control plan be prepared and implemented at least one month prior to the beginning of the rainy season. The erosion control measures will require inspection, modification, and re-mediation during the rainy season in order to comply with regulatory requirements.

ADDITIONAL RECOMMENDATIONS: Detailed earthwork, underground utility, drainage, building foundation, retaining wall, and pavement recommendations for use in design and construction of the project are presented below. We recommend SFB review the design and specifications to verify that the recommendations presented in this report have been properly interpreted and implemented in the design, plans, and specifications. We also recommend SFB be retained to provide consulting services and to perform construction observation and testing services during the construction phase of the project to observe and test the implementation of our recommendations, and to provide supplemental or revised recommendations in the event conditions different than those described in this report are encountered. We assume no responsibility for misinterpretation of our recommendations.

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

4.1 Earthwork

4.1.1 Clearing and Site Preparation

The site should be cleared of all obstructions including any designated trees and their associated entire root systems, and debris. Holes resulting from the removal of underground obstructions extending below the proposed finish grade should be cleared and backfilled with fill materials as specified in **Section 4.1.5,** ***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 Santa Cruz 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 building 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.

After clearing, areas containing heavy surface vegetation should be stripped to an appropriate depth to remove these materials. Stripped materials should be removed from the site or stockpiled for later use in landscaping, if desired.

4.1.2 Weak Soil and Fill Re-Compaction

As described in Section 3.4, the results of our field exploration indicate the site is blanketed by about 3-1/2 to 8 feet of sandy and clayey undocumented fills at the higher elevations and about 3 to 5 feet of slope wash sands at the lower elevations. These undocumented fills and slope wash sands are loose, weak, potentially compressible, and prone to surface erosion and slumping.

In order to reduce the potential for damaging differential settlement of overlying improvements (such as new fills, building foundations, retaining walls, driveways, exterior flatwork, and pavements) and to improve site slope stability, we recommend that these weak fills and soils be completely over-excavated and re-compacted within the planned development area. The over-excavation should extend to depths where competent soils are encountered.

Over-excavation and re-compaction should extend at least 5 feet beyond building and retaining wall footprints and at least 3 feet beyond exterior flatwork and pavement wherever possible. There would be no need to over-excavate and re-compact the soils within areas that do not support improvements, such as within open spaces. Where the over-excavation limits abut adjacent property, SFB should be consulted to determine the actual vertical and lateral extent of over-excavation so that adjacent property is not adversely impacted. Over-excavations should be performed so that no more than 5 feet of differential fill thickness exists below proposed building foundations. The actual 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 operations.

Removed soil and fill materials may be used as new fills 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

The proposed grading should be designed and constructed so that no more than 5 feet of differential fill thickness will exist below building supported on shallow foundations. Deeper over-excavation may be necessary to reduce fill differential thickness to 5 feet or less. We recommend a layer at least 3 feet thick of well-blended, moisture conditioned, engineered fill be provided below the building pad area to reduce the potential for damaging differential movement. The compacted, engineered fill layers should extend at least 5 feet beyond building footprint and at least 3 feet beyond exterior flatwork.

4.1.4 Subgrade Preparation

After the completion of clearing, site preparation, and weak soil and fill re-compaction, soils exposed in areas to receive improvements (such as new fills, building foundations, retaining walls, 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 the building pad, driveway subgrade, and/or pavement subgrade are exposed to sun, wind or rain for an extended period of time, or are heavily disturbed by vehicle traffic or animal borrowing, the exposed building pad, driveway subgrade, and pavement subgrade may need to be reconditioned (moisture conditioned and/or scarified and recompacted). 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.

If required, imported fill for general use should have a plasticity index of 15 or less. Imported non-expansive fill should be predominantly granular, have a plasticity index not exceeding 12, and have a significant fines content.

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 to at least 90 percent relative compaction, and structural fill below a depth of 5 feet be compacted to at least 95 percent relative compaction, as determined by ASTM D1557 (latest edition). We recommend the new fill be moisture conditioned approximately 2 to 3 percent over optimum water content. The upper 6 inches of subgrade soils beneath pavements should be compacted to at least 95 percent relative compaction. Fill material should be spread and compacted in lifts not exceeding approximately 8 to 12 inches in uncompacted thickness.

4.1.7 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 rock bedding and rock backfill (if used) be completely surrounded by a filter fabric such as Mirafi 140N (or equivalent); alternatively, filter fabric would not be necessary if Caltrans Class 2 permeable material is used in lieu of rock bedding and rock backfill.

Sand or gravel backfilled trench laterals that extend toward driveways, exterior slabs-on-grade, or under the building foundations, and are located below irrigated landscaped areas such as lawns or planting strips, should be plugged with onsite clays, low strength concrete, or sand/cement slurry. The plug for the trench 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.

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

4.1.8 Exterior Flatwork

We recommend that exterior slabs (including patios, sidewalks, and driveways) 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. 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 onsite expansive clayey soils 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.

Consideration should be given to reinforcing exterior slabs (including concrete trash enclosure 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 considered. Score joints and expansion joints should be used to control cracking and allow for expansion and contraction of the concrete slabs. We recommend appropriate flexible, relatively impermeable fillers be used at all cold/expansion joints. The installation of dowels at all expansion and cold joints will reduce differential slab movements; if used, the dowels should be at least 30 inches long and should be spaced at a maximum lateral spacing of 24 inches. Although exterior slabs that are adequately reinforced will still crack, trip hazards requiring replacement of the slabs will be reduced if the slabs are properly reinforced.

We do not recommend the use of flatwork having permeable joints (such as pavers or tiles with sand or gravel infilled joints) unless the underlying clayey subgrade is protected against water seepage or ponding. If not protected, the underlying subgrade will heave and/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 driveways, walkways, and patios. Surface water should not be allowed to flow over the top of slopes, down slope faces, or over retaining walls.

We recommend positive surface gradients of at least 2 percent be provided adjacent to foundations to direct surface water away from the foundations and toward suitable discharge facilities. Roof downspouts and landscaping drainage inlets should be connected to solid pipes that discharge the collected water into appropriate water collection facilities. We recommend the surface drainage be designed in accordance with the latest edition of the California Building Code.

In order to reduce differential foundation movements, landscaping (where used) should be placed uniformly adjacent to 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.

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 Subsurface Drainage

In order to reduce the potential for subsurface water related issues, we recommend subdrains be installed below engineered fill placed on slopes, at the toe of slopes, where open space areas direct water toward improvements, and also along the upslope sides of driveways and roadways where located on or adjacent a hillside. During the earthwork operations, additional subdrains may be necessary in areas of encountered or anticipated seepage. We recommend a subdrain be located below lined ditches or earthen swales. The location and extent of subdrains should be assessed by SFB during the development of the grading and improvement plans, and determined in the field by SFB at the time of construction.

Where used, subdrains should consist of a 4-inch diameter, rigid perforated pipe (perforations down) surrounded by free draining, uniformly graded, 1/2- to 3/4-inch crushed gravel wrapped in filter fabric such as Mirafi 140N or equivalent. The pipe should be underlain by about 1/2 to 1 inch of gravel, and on the sides by at least 4 inches of gravel. The filter fabric should overlap approximately 12 inches or more at joints. Subdrains should be connected to a solid, rigid, collector pipe with a minimum diameter of 4 inches. Subdrain pipes should consist of rigid PVC SDR-35 or PVC A-2000 (or equal) for fills less than 20 feet in height or thickness. Collector pipes should be connected to appropriate discharge facilities such as storm drains, drainage inlets, or storm drain manholes. Subdrain clean-outs should be provided. The clean-out locations should be based upon the reach of the rotary cleaning systems and the restrictions of pipe bends. Caltrans Class 2 permeable material may be used in lieu of gravel and filter fabric.

Where used, subdrain trenches should be at least 12 inches wide and about 4 feet deep below adjacent ground surface. If a subdrain trench extends to the ground surface and is not covered with concrete lined ditch or concrete flatwork, we recommend the subdrain trench be covered with a 12-inch thick cap consisting of native soil compacted to at least 90 percent relative compaction.

4.1.12 Storm Water Treatment Facilities

To satisfy local and state permit requirements, most new development projects must control pollutant sources and reduce, detain, retain, and/or treat specified amounts of storm water runoff. The intent of these types of storm water treatment facilities is to conserve and incorporate on-site natural features, together with constructed hydrologic controls, to more closely mimic pre-development hydrology and watershed processes. These facilities include bio-retention swales and basins, porous paver and pavement, water detention basins, and any proprietary underground storage and treatment systems.

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

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

Soil filter/bio-mix materials provide little to no lateral restraint of excavation side walls. Sidewalls of bio-retention swale and basin excavations (excavations made prior to the installation of the soil filter/bio-mix) steeper than 2:1 (horizontal to vertical) will experience downward and lateral movements that can cause distresses to adjacent improvements such as foundations, utilities, pavements, driveways, walkways, and curbs and gutters. The magnitude and rate of movement depend upon the swale and basin backfill material type and compaction. To reduce the potential for damaging movements, we recommend 2:1 or flatter excavation sidewall slopes be used for bio-retention swales and basins, sidewalks be setback at least 3 feet from the top of slopes, and creep sensitive improvements (such as roadway curbs) be setback at least 5 feet from the top of slopes. If the above sidewall slope and setback distance cannot be met, 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.

4.1.13 Engineered Slopes

4.1.13.1 General

We recommend proposed cut and non-reinforced fill slopes not exceed an inclination of 2:1 (horizontal to vertical) when they are no more than 10 feet high. Slopes higher than 10 feet should not exceed an inclination of 3:1 unless we are further consulted to evaluate the slope stability. Steeper fill slopes are feasible provided they are mechanically reinforced with geogrid; if requested, SFB can provide detailed designs of slope reinforcing if needed.

We recommend all cut and fill slopes be constructed with surface drainage collection and discharge facilities. Shallow slope movements such as surficial sloughing, toppling, and flows, could still occur as a result of erosion and unanticipated water infiltration. To decrease the potential for shallow slope movement, the drainage and erosion control recommendations presented in this report should be implemented in the design and construction of the site. The implemented drainage and erosion control measures should be maintained during and after construction. Slope benches should be constructed in accordance with the latest edition of the California Building Code. Slope maintenance may include re-establishing drainage patterns, controlling water infiltration, and repairing shallow slope movements.

4.1.13.2 Fill Slopes

We recommend proposed fill slopes be built using well blended, moisture conditioned, engineered fill to reduce the potential for slope expansion and creeping. We also recommend that fill slopes be over-built approximately 2 feet horizontally and then trimmed back to finished grades.

Where fills are placed on slopes steeper than 10:1 (horizontal to vertical), fills should be keyed at least 5 feet into competent native soils or at least 3 feet into competent bedrock. Keyways should be at least 10 feet wide and a subdrain should be placed at the bottom and to the rear of each keyway. The keyway should be sloped toward the back of the key at 2 percent or steeper. A subgrade bench and subdrain should be provided for approximately every 10 feet of vertical elevation gain, and the bench should extend at least one foot into competent soils. Subdrain construction is described in **Section 4.1.11, *Subsurface Drainage***.

If requested, SFB can prepare a geotechnical improvement plan to indicate the estimated locations of keyways and subdrains once the project grading plans are developed. The actual extent of the keying, benching, and subdrainage should be verified by SFB during earthwork operations.

4.1.13.3 Unstable Cut Slopes

Where cut slopes expose unstable soils, the unstable materials should be removed in accordance with the recommendations provided in **Section 4.1.2, *Existing Weak Soil and Fill Re-Compaction***. Cut slopes may need to be buttressed with engineered fill. Cut slopes should be observed by SFB at the time of grading to determine the actual extent of over-excavation and to assess the need for any additional remedial work.

4.1.14 Setbacks

In order to reduce damage of improvements caused by potential slope erosion and slumping, appropriate slope setbacks should be used for the project. We recommend setbacks be established by projecting a 3:1 (horizontal to vertical) line from toe of slopes upward toward the improvements. Where the projected line intersects the finished ground surface, we recommend improvements be setback at least 5 feet from the intersection or at least 5 feet from top of the slope, whichever is greater. The building should be setback at least 10 feet from the intersection or at least 10 feet from top of the slope, whichever is greater.

We should be further consulted to provide design alternatives if it is impractical to setback improvements, such as using deepened edges for the improvements or retaining walls. We recommend the project Civil Engineer determine the actual improvement and building setback based upon the recommendations provided in this report, California Building Code and local ordinances, and any other restrictions. Improvements located between the setback line and the slope may experience movement as a result of slope erosion, localized slumping, earthquake shaking, and other factors.

4.1.15 Future Maintenance

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

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

4.1.16 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 owners, HOAs, development owners, and their maintenance associations.

4.2 Foundation Support

4.2.1 Footing Foundations

The proposed residential building can be supported on conventional continuous and isolated spread footings that bear on engineered fills. Recommendations for building pad preparation are described previously in **Sections 4.1.2, *Weak Soil and Fill Re-Compaction***, **Section 4.1.3, *Building Pad***, and **4.1.4, *Subgrade Preparation***. Prior to the concrete pour, we recommend the moisture content of subgrade materials be approximately 2 to 3 percent above laboratory optimum moisture. If the building pads are left exposed for an extended period of time prior to constructing foundations, we recommend SFB be contacted for recommendations to re-condition the pads in order provide adequate building support.

Footings should be embedded at least 24 inches below the lowest adjacent finished grade. The footing dimension and reinforcement should be designed by the Structural Engineer; however, continuous and isolated spread footings should have minimum widths of 12 and 18 inches, respectively. The portion of the foundations located within 10 feet (as measured laterally) of the nearest slope face should be neglected in the vertical bearing and lateral resistance analyses. Also, the portions of the foundations located above an imaginary 1:1 (horizontal to vertical) plane extending upward from the bottom edges of any adjacent footings and utility trenches should also be neglected in the vertical bearing and lateral resistance analyses.

Alternatively, the foundation reinforcing could be increased to span the area defined above assuming no soil support is provided or the bottom of foundation could be deepened to bear below the area defined above. Our recommended allowable spread footing bearing pressures are provided below. These allowable bearing pressures are net values; therefore, the weight of the footing can be neglected for design purposes.

ALLOWABLE SPREAD FOOTING BEARING PRESSURES		
Load Condition	Allowable Bearing Pressures (psf)	Factor of Safety
Dead Load	2,000	3.0
Dead plus Live Loads	3,000	2.0
Total Loads (including Wind or Seismic)	4,000	1.5

We estimate maximum total settlement of foundations under the above recommended allowable bearing pressures to be on the order of 1 inch or less. Differential static settlement between similarly loaded footings is estimated to be approximately 1/2 inch.

Lateral loads may be resisted by a combination of friction between the foundation bottoms and the supporting subgrade and by passive resistance acting against the vertical faces of the foundations. A coefficient of sliding friction of 0.3 is considered applicable. In addition, an equivalent fluid weight of 300 pounds per cubic foot (pcf) acting against the side of the foundation may be used where the foundation concrete is poured neat against undisturbed subgrade. This value is based on a safety factor of at least 1.5 and generally corresponds to a lateral deflection of less than 1/2 inch. Passive resistance in the upper 12 inches of soil should be neglected unless the area in front of the footing is protected by concrete or pavement from disturbance. The allowable friction coefficient and passive resistance may be used concurrently without reduction.

Any visible cracks in the bottoms of the footing excavations should be closed by wetting prior to construction of the foundations. We should observe the footing excavations prior to placing reinforcing steel or concrete to check that footings are founded on appropriate materials. All foundation excavations should be cleaned of loose materials and should be free of water. The footing excavations should be kept moist prior to concrete placement.

If alternative foundation systems are being considered, SFB should be consulted to provide geotechnical design and construction criteria for the alternative foundation systems.

4.2.2 Interior Slabs-On-Grade with Footings

Where interior slabs-on-grade will be used in conjunction with footings, we recommend the interior slabs be at least 5 inches thick, reinforced with a minimum of #4 bars on 18-inch centers (both ways), and supported on an at least 12-inch thick layer of imported, predominantly granular, "non-expansive" engineered fills that meet the requirements presented in this report. The onsite sandy soils and fills can used if they meet the criteria of "non-expansive" fills.

Slab-on-grade subgrade surfaces should be proof-rolled to provide a smooth, unyielding surface for slab support. Floor slab control joints can be used to reduce damage due to shrinkage cracking. The actual thickness, reinforcing, jointing of the slabs should be designed by the project Structural Engineer based upon the actual use and loading of the slabs.

We recommend a vapor retarder and an underlying 4-inch layer of 3/4-inch, clean, crushed, uniformly graded gravel/drain rock be placed between the bottom of the slab and the recommended non-expansive engineered fill layer. The gravel/drain rock layer can be considered as part of the non-expansive engineered fill layer. We recommend the vapor retarder consist of a single layer of Stego Wrap Vapor Barrier 15 mil or equivalent provided the equivalent satisfies the following criteria: a permeance less than 0.01 perms as guided by ACI 302.2R, Class A strength as determined by ASTM E1745, 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 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 membrane 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. The edges of the vapor retarder membrane should be draped over the interior side of the perimeter footing and should extend at least 12 inches below the pad grade. We do not recommend placing sand or gravel over the membrane. Prior to placement of the vapor retarder, the subgrade surfaces should be proof-rolled to provide a smooth, unyielding surface for slab support.

We recommend that the interior slabs-on-grade (other than garage slabs) be poured monolithically with the footings. The edge of the garage slabs should be structurally separated (disconnected) from the surrounding footings/grade beams; a relatively impermeable and flexible filler should be used in the joint between the garage slab and the surrounding grade beams. We recommend a grade beam be provided directly below the garage door opening. Both the driveway and garage slabs should be doweled to the grade beam below the door opening with rebars to reduce the potential for differential movements.

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 installation of floor coverings.

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. All concrete placement and curing operations should be performed in accordance with the American Concrete Institute (ACI) manuals.

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.

4.2.3 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. We recommend all below-grade floors and walls (if used) be appropriately waterproofed; we recommend a waterproofing specialist be consulted for the waterproofing design.

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	42
Restrained (At-Rest Pressure)		60	95	N/A*

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

For retaining walls that need to resist earthquake induced lateral loads from nearby foundations, walls that are to be designed to resist earthquake loads, and any retaining walls that are higher than 6 feet (as required by the 2019 CBC), we recommend the walls be designed to also resist 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 was estimated based on the half of the peak ground acceleration from a Maximum Considered Earthquake (MCE) earthquake per ASCE 7-16/2019 CBC (0.5 x PGA_M). 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.

Where the walls will be retaining native soils (not engineered fill) and/or will be constructed on slopes, we recommend a global slope stability assessment be performed by SFB.

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. Soil creep forces equal to an equivalent fluid pressure of 125 pcf should also be applied to the upper 3 feet of the wall height where native slopes will exist above the walls. 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 4.1.6, *Fill Material***, and **Section 4.1.7, *Compaction***.

Retaining walls can be supported on spread footings as recommended in **Section 4.2.1, *Footing Foundations***. Alternatively, 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 3,500 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 two feet of pier embedment should be neglected in the vertical and passive resistance design as measured from finished grade unless it is confined by a pavement or concrete slab. The portion of the pier shaft located within 10 feet (as measured laterally) of the nearest slope face or above an imaginary 1:1 (horizontal to vertical) plane extending upward from the bottom of any adjacent walls or utility trenches should also be ignored in both the vertical bearing and passive resistance designs.

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

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

4.2.4 Seismic Design Criteria

For seismic design using the 2019 California Building Code (CBC), we recommend the following seismic design parameters be used. These parameters were calculated using the U.S. Seismic Design Map program[10], and are based on the site being located at approximate latitude 37.996777 °N and longitude 122.300945°W. These values are based on applying the ASCE 7-16 model, assuming the structure is categorized as Risk Category II, and assuming that *Exception Number (2) of ASCE 7-16 Section 11.4.8 – Site Specific Ground Procedure* applies. We should be contacted if any of these assumptions are incorrect or a site-specific ground motion hazard analysis is required.

SEISMIC PARAMETER	DESIGN VALUE
Site Class	D
S_S	1.954
S_1	0.754
S_{MS}	1.954
S_{M1}	Null – Section 11.4.8 ASCE 7-16
S_{DS}	1.303
S_{D1}	Null – Section 11.4.8 ASCE 7-16
SDC	Null – Section 11.4.8 ASCE 7-16
F_a	1
F_v	Null – Section 11.4.8 ASCE 7-16
PGA_M	0.903
T_L	12

[10]SEAONC/OSHPD, https://seismicmaps.org/, accessed 12/23/2021.

4.3 Pavements

Based on the results of borings and laboratory testing, we recommend that an R-value of 10 be used in preliminary asphalt concrete pavement design. We recommend additional R-value tests be performed once the pavement subgrade is established to confirm the R-value used in the design. Pavement subgrade completely composed of sandy and gravelly fills will result in higher R-values and thinner pavement sections.

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

PRELIMINARY PAVEMENT DESIGN ALTERNATIVES SUBGRADE R-VALUE = 10			
Location	Pavement Components		Total Thickness (inches)
	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	
T.I. = 4.5 (auto & light truck parking)	3.0	8.0	11.0
T.I. = 5.0 (access ways)	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.

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 Novin Development and their consultants for specific application to the proposed residential project at 2840 Park Avenue in Soquel, California, and is intended to represent our design recommendations to Novin Development for specific application to the 2840 Park Avenue project. The conclusions and recommendations contained in this report are solely professional opinions. It is the responsibility of Novin Development 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.

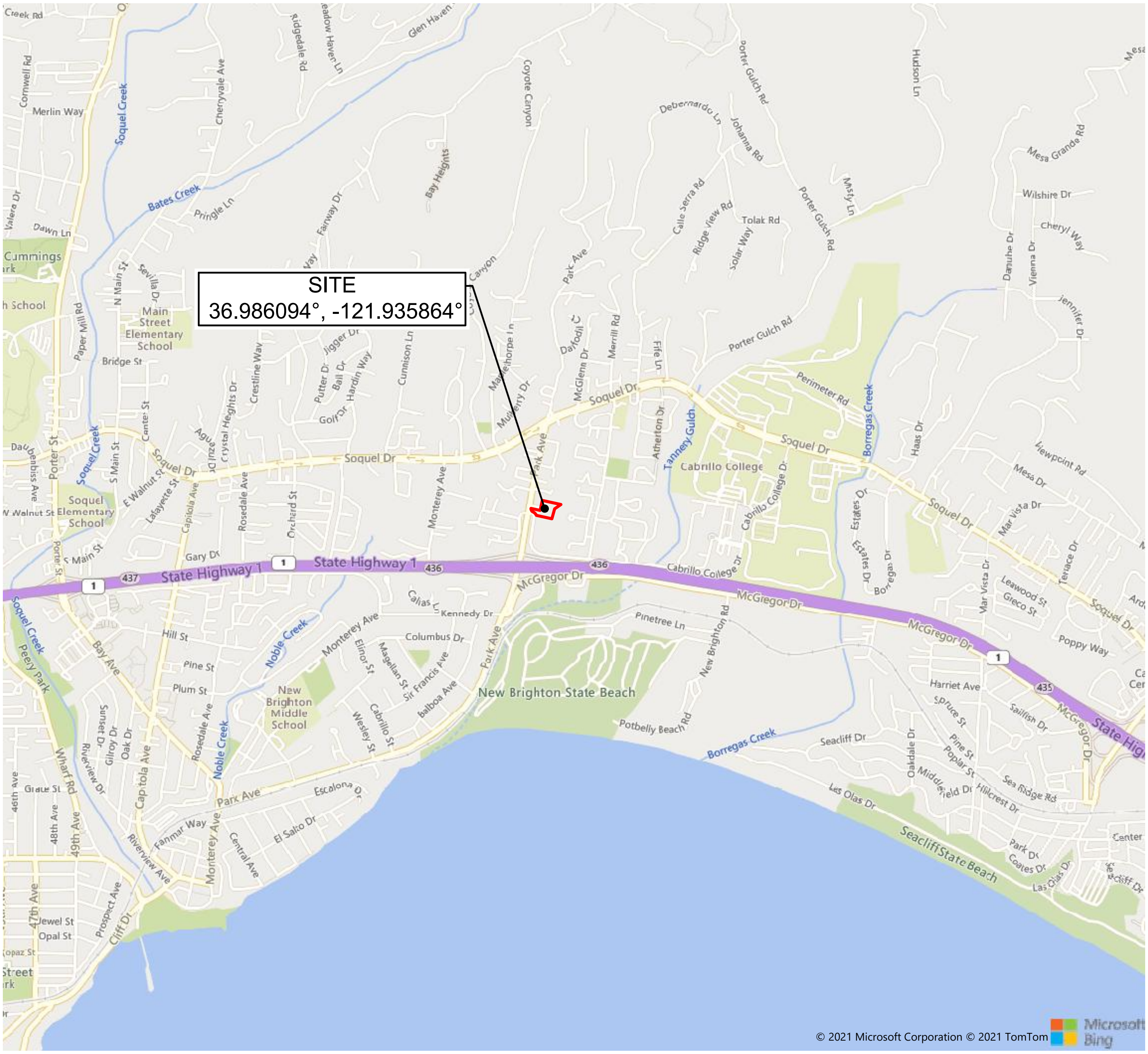
It should be understood that advancements in the practice of geotechnical engineering and engineering geology, or discovery of differing surface or subsurface conditions, may affect the validity of this report and are not uncommon. SFB strives to perform its services in a proper and professional manner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

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

FIGURES



N

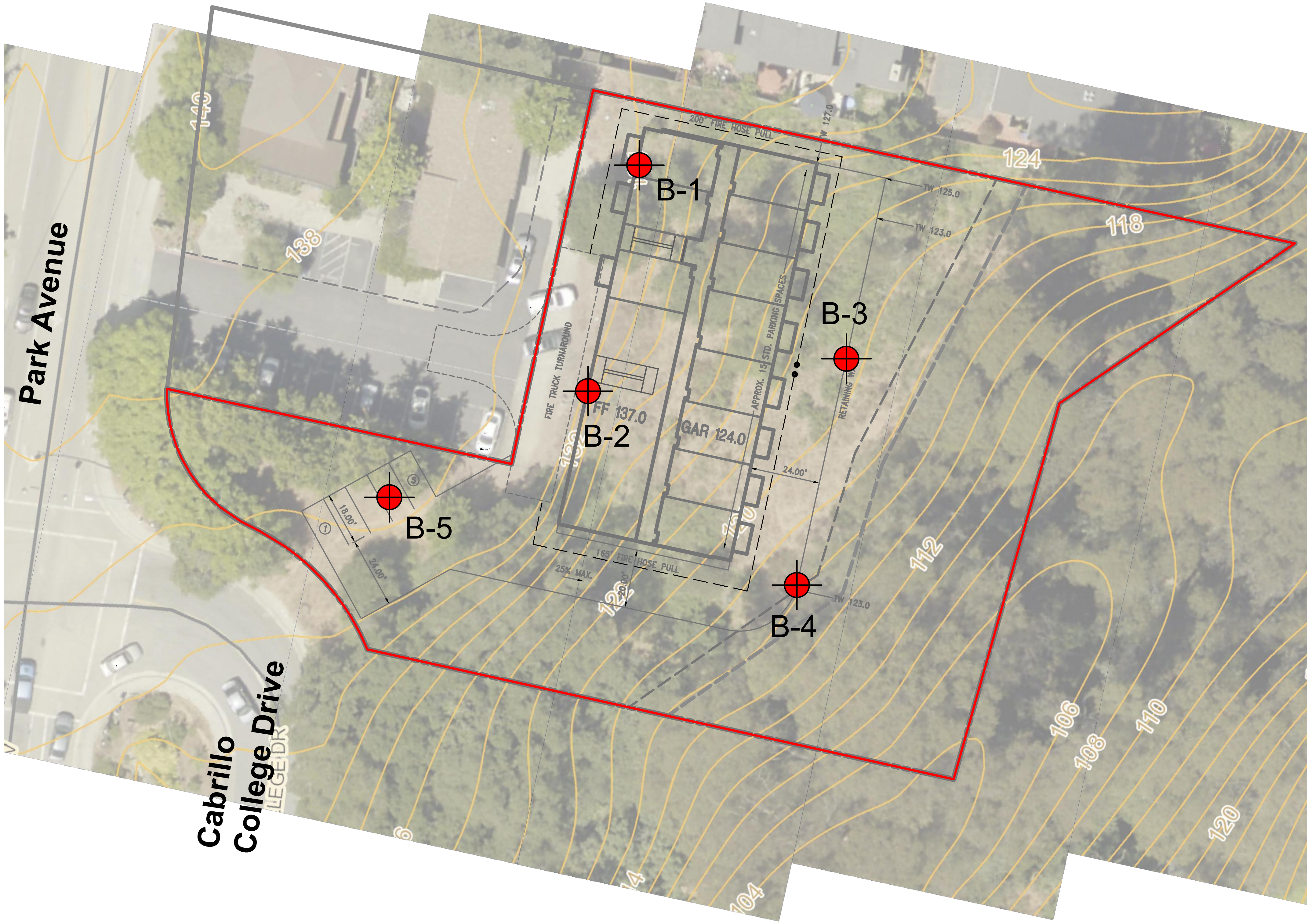
APPROXIMATE SCALE: 1" = 2,000'

0 2,000' 4,000'

NOTE: Basemap from Microsoft/TomTom 2021.

DATE	Stevens Ferrone & Bailey Engineering Company, Inc	VICINITY MAP	FIGURE
December 2021	1600 Willow Pass Court Concord, CA 94520 Tel 925.688.1001 Fax 925.688.1005 www.SFandB.com	2840 PARK AVENUE Soquel, California	1
PROJECT NO.			
940-2			

F:\PROJECT DOCUMENTS\CAD\GEOTECHNICAL INVESTIGATION\940-2\940-2, 2840 Park Ave Soquel - December 2021.dwg
Printed: 12/23/2021



KEY

Exploratory Boring by SFB (12/9/21)

Approximate Project Limit

NOTE: All locations shown are approximate.

NOTE: Base map from the project Conceptual Site Plan - Option 3 prepared by Ifland Engineers and dated 11/9/2021.

APPROXIMATE SCALE: 1" = 40'	DATE			SITE PLAN	FIGURE
0 40' 80'	December 2021	Stevens Ferrone & Bailey Engineering Company, Inc	1600 Willow Pass Court Concord, CA 94520 Tel 925.688.1001 Fax 925.688.1005 www.SFandB.com	2840 PARK AVENUE Soquel, California	2
	PROJECT NO.				
	940-2				

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Printed: 12/23/2021

APPENDIX A

Field Exploration

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

KEY TO EXPLORATORY BORING LOGS

PROJECT: **2840 PARK AVENUE**
Soquel, California

PROJECT NO: **940-2**

FIGURE NO: **A-1**

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS		GRAPHIC LOG	GROUP SYMBOL	DESCRIPTION
COARSE-GRAINED SOILS (More than 50% of material is larger than #200 sieve)	CLEAN GRAVELS (Less than 5% fines)		GW	Well-graded gravels or gravel-sand mixtures, little or no fines
			GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
	GRAVELS WITH FINES (More than 12% fines)		GM	Silty gravels or gravel-sand-silt mixtures
			GC	Clayey gravels or gravel-sand-clay mixtures
	CLEAN SANDS (Less than 5% fines)		SW	Well-graded sands or gravelly sands, little or no fines
			SP	Poorly-graded sands or gravelly sands, little or no fines
	SANDS WITH FINES (More than 12% fines)		SM	Silty sands or sand-silt mixtures
			SC	Clayey sands or sand-clay mixtures

MAJOR DIVISIONS		GRAPHIC LOG	GROUP SYMBOL	DESCRIPTION
FINE-GRAINED SOILS (More than 50% of material is smaller than #200 sieve)	SILTS AND CLAYS (Liquid Limit less than 50%)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts of low to medium plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and clays of low plasticity
	SILTS AND CLAYS (Liquid Limit 50% or greater)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts of high plasticity
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic silts and clays of medium to high plasticity
HIGHLY ORGANIC SOILS			PT	Peat and other highly organic soils

GRAIN SIZES

U.S. STANDARD SERIES SIEVE — CLEAR SQUARE SIEVE OPENINGS

	#200	#40	#10	#4	3/4"	3"	12"	
SILTS AND CLAYS	SANDS			GRAVELS		COBBLES	BOULDERS	
	Fine	Medium	Coarse	Fine	Coarse			

RELATIVE DENSITY

SANDS AND GRAVELS	BLOWS/FOOT*
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	Over 50

CONSISTENCY

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

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

SYMBOLS AND NOTES

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

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

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

Shelby Tube

Pitcher Barrel

HQ Core

Groundwater Level During Drilling

Groundwater Level at End of Drilling

INCREASING VISUAL MOISTURE CONTENT

Saturated
Wet
Moist
Damp
Dry

CONSTITUENT PERCENTAGE

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

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EXPLORATORY BORING B-1

PROJECT:
2840 PARK AVENUE
Soquel, CA

PROJECT NO: 940-2	SURFACE ELEVATION: 134 feet
LOGGED BY: M. Mendoza	DATE STARTED: 12/09/21
DRILL RIG: Mobile B-24	DATE FINISHED: 12/09/21
DRILLING METHOD: 4-inch Solid Stem Auger	DEPTH TO INITIAL WATER: Not Encountered
HAMMER METHOD: Rope and Cathead	DEPTH TO FINAL WATER: Not Encountered
HAMMER WEIGHT / DROP: 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 2 (36.986312°, -121.935917°)	

DESCRIPTION AND CLASSIFICATION

DESCRIPTION AND REMARKS	CONSIST	DEPTH (FEET)	SPT N_{60}-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS AND NOTES
FILL: SAND (SC)/CLAY (CL), brown, sandy (fine- to coarse-grained), trace gravel (fine to coarse, subangular), damp to moist.	loose	0	4	13.9	107.6	1.6	
Change color to grayish brown.			3				
SAND (SM), mottled light gray yellowish brown, fine- to medium-grained, lightly cemented, dry.	very dense	5	44/11"				
Change color to mottled gray brown, trace clay, dry to damp.	dense	10	43				
SAND (SP-SM), mottled gray brown, fine- to medium-grained, some coarse-grained, with gravel (fine to coarse, subangular to subrounded), dry.	very dense	15	30/6"				
SAND (SP-SM), grayish brown, fine- to medium-grained, trace coarse-grained, dry.	very dense	20	39/11"				
Bottom of Boring = 23.4 feet Groundwater was not encountered during drilling. Notes: Stratification is approximate, variations must be expected. Blow counts converted to SPT N-values. See report for additional details.		25					
		30					

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Tel: (925) 688-1001

EXPLORATORY BORING B-2

PROJECT NO: 940-2	SURFACE ELEVATION: 134 feet
LOGGED BY: M. Mendoza	DATE STARTED: 12/09/21
DRILL RIG: Mobile B-24	DATE FINISHED: 12/09/21
DRILLING METHOD: 4-inch Solid Stem Auger	DEPTH TO INITIAL WATER: Not Encountered
HAMMER METHOD: Rope and Cathead	DEPTH TO FINAL WATER: Not Encountered
HAMMER WEIGHT / DROP: 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 2 (36.986100°, -121.935977°)	

PROJECT:

2840 PARK AVENUE

Soquel, CA

DESCRIPTION AND CLASSIFICATION

DESCRIPTION AND REMARKS	CONSIST	GRAPHIC LOG	DEPTH (FEET) ELEVATION	SAMPLER	SPT N_{60}-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS AND NOTES
FILL: SAND (SC)/CLAY (CL), dark brown, fine- to medium-grained, some coarse-grained, clayey, some gravel (fine, subangular to subrounded), damp.	loose		0						
					5	14.0	107.1	1.4	
A 2 inch concrete fragment at 2 feet.					5				
Change color to mottled gray brown, damp to moist.									
			5						
Trace gravel (fine, subangular).					5	15.9	111.2	0.8	At 6 feet: Liquid Limit = 25, Plasticity Index = 10, Coarse Sand = 4%, Medium Sand = 26%, Fine Sand = 32%, Silt = 17%, Clay = 21%, Corrosion Test, See Appendix B.
CLAY (CL)/SAND (SC), mottled brown bluish gray, sandy (fine-grained), with organic or hydrocarbon odor, damp.	stiff								
			10						
Change color to mottled gray brown, some gravel (fine, angular to subangular) at 11 feet.					14	17.7	111.7	3.2	At 11 feet: Liquid Limit = 35, Plasticity Index = 18, Fine Gravel = 1%, Coarse Sand = 1%, Medium Sand = 15%, Fine Sand = 48%, Silt = 10%, Clay = 25%
SAND (SM), mottled gray yellowish brown, fine- to medium-grained, silty, damp.	dense								
			15		43				
	very dense		20		43/11"				
Bottom of Boring = 21.4 feet Groundwater was not encountered during drilling. Notes: Stratification is approximate, variations must be expected. Blow counts converted to SPT N-values. See report for additional details.									
			25						
			30						

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EXPLORATORY BORING B-3

PROJECT:
2840 PARK AVENUE
Soquel, CA

PROJECT NO: 940-2	SURFACE ELEVATION: 120 feet
LOGGED BY: M. Mendoza	DATE STARTED: 12/09/21
DRILL RIG: Geoprobe 7822 DT	DATE FINISHED: 12/09/21
DRILLING METHOD: 7-inch Hollow Stem Auger	DEPTH TO INITIAL WATER: 15 feet
HAMMER METHOD: Automatic Trip	DEPTH TO FINAL WATER: 15 feet
HAMMER WEIGHT / DROP: 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 2 (36.986131°, -121.935675°)	

DESCRIPTION AND REMARKS	CONSIST	GRAPHIC LOG	DEPTH (FEET) ELEVATION	SAMPLER	SPT N_{60}-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS AND NOTES
SLOPE WASH: SAND (SM), dark brown, fine- to medium-grained, silty, trace roots, dry. Change color to yellowish brown, some clay, dry to damp.	loose		0		7 6	12.2	101.4		At 2 feet: Corrosion Test, See Appendix B.
SAND (SM), yellowish brown, fine- to medium-grained, some coarse-grained, with to silty, some gravel (fine, subangular to subrounded), trace clay, dry.	dense		5		33				
SAND (SM), brown, fine- to medium-grained, trace coarse-grained, silty, dry. Change color to brownish gray at 15 feet, wet.	medium dense dense		10 15		21 39				
SILT (ML), bluish gray, some sand (fine-grained), dry. (COMPLETELY WEATHERED SILTSTONE).	hard		20		65/11"				
Bottom of Boring = 21.4 feet Notes: Stratification is approximate, variations must be expected. Blow counts converted to SPT N-values. See report for additional details.			25 30						

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Concord, CA 94520
Tel: (925) 688-1001

EXPLORATORY BORING B-4

PROJECT NO: 940-2	SURFACE ELEVATION: 116 feet
LOGGED BY: M. Mendoza	DATE STARTED: 12/09/21
DRILL RIG: Geoprobe 7822 DT	DATE FINISHED: 12/09/21
DRILLING METHOD: 7-inch Hollow Stem Auger	DEPTH TO INITIAL WATER: 15 feet
HAMMER METHOD: Automatic Trip	DEPTH TO FINAL WATER: 15 feet
HAMMER WEIGHT / DROP: 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 2 (36.985918°, -121.935733°)	

PROJECT:

2840 PARK AVENUE

Soquel, CA

DESCRIPTION AND REMARKS	CONSIST	GRAPHIC LOG	DEPTH (FEET) ELEVATION	SAMPLER	SPT N_{60}-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS AND NOTES
SLOPE WASH: SAND (SM), dark brown, fine- to medium-grained, silty, dry to damp.	loose		0		7	11.4	96.2		
SAND (SM), yellowish brown, fine- to medium-grained, trace coarse-grained, with silt, dry.	medium dense				17				
Fine- to coarse-grained, some gravel (fine, subangular to subrounded), lightly cemented, trace roots, dry. A 1.5 inch chert fragment at 6 feet.	very dense		5		54				
With chert fragments.	medium dense		10		18				
SILT (ML), bluish gray, some sand (fine-grained), dry. (COMPLETELY WEATHERED SILTSTONE)	hard		15		65				
Sandy (fine- to medium-grained).			20		63				
Bottom of Boring = 21.5 feet Notes: Stratification is approximate, variations must be expected. Blow counts converted to SPT N-values. See report for additional details.			25						
			30						

EXPLORATORY BORING LOG 940-2 B-4.ldat8 STEVENS FERRONE & BAILEY 12/22/2021

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Concord, CA 94520
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EXPLORATORY BORING B-5

PROJECT NO: 940-2	SURFACE ELEVATION: 136 feet
LOGGED BY: M. Mendoza	DATE STARTED: 12/09/21
DRILL RIG: Geoprobe 7822 DT	DATE FINISHED: 12/09/21
DRILLING METHOD: 7-inch Hollow Stem Auger	DEPTH TO INITIAL WATER: Not Encountered
HAMMER METHOD: Automatic Trip	DEPTH TO FINAL WATER: Not Encountered
HAMMER WEIGHT / DROP: 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 2 (36.986001°, -121.936209°)	

PROJECT:

2840 PARK AVENUE

Soquel, CA

DESCRIPTION AND CLASSIFICATION: DESCRIPTION AND REMARKS	CONSIST	GRAPHIC LOG	DEPTH (FEET) ELEVATION	SAMPLER	SPT N_{60}-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS AND NOTES
FILL: SAND (SM)/CLAY (CL), dark brown, fine- to coarse-grained, with to silty, with gravel (fine to coarse, subangular to subrounded), trace clay, dry.	loose		0		13	8.3	117.5	1.6	
					8				
CLAY (CL), grayish brown, silty, some sand (fine-grained), dry to damp.	stiff								
SAND (SM), yellowish brown, fine- to medium-grained, trace coarse-grained, with to silty, trace gravel (fine, subangular), dry.	medium dense		5		15				
Bottom of Boring = 6.5 feet Groundwater was not encountered during drilling. Notes: Stratification is approximate, variations must be expected. Blow counts converted to SPT N-values. See report for additional details.			10						
			15						
			20						
			25						
			30						

EXPLORATORY BORING LOG 940-2 B-5.ldat8 STEVENS FERRONE & BAILEY 12/28/2021

APPENDIX B

Laboratory Testing

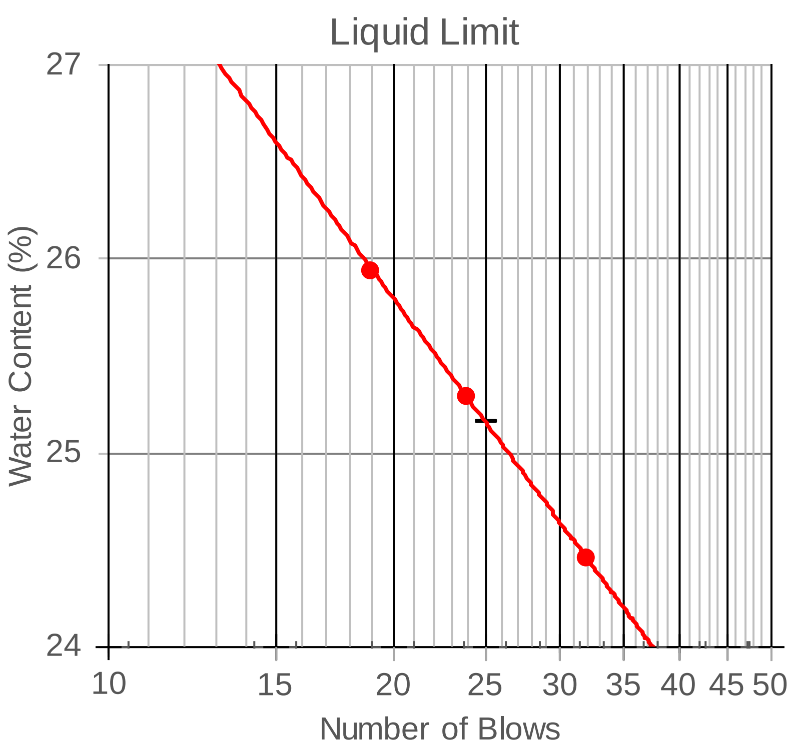
Stevens Ferrone & Bailey Engineering Company, Inc

Atterberg Limits Test – ASTM D4318

Project Number: 940-2 **Boring/Sample No:** B-2 **Depth:** 6

Project Name: 2840 Park Avenue **Test Date:** 12-21-21

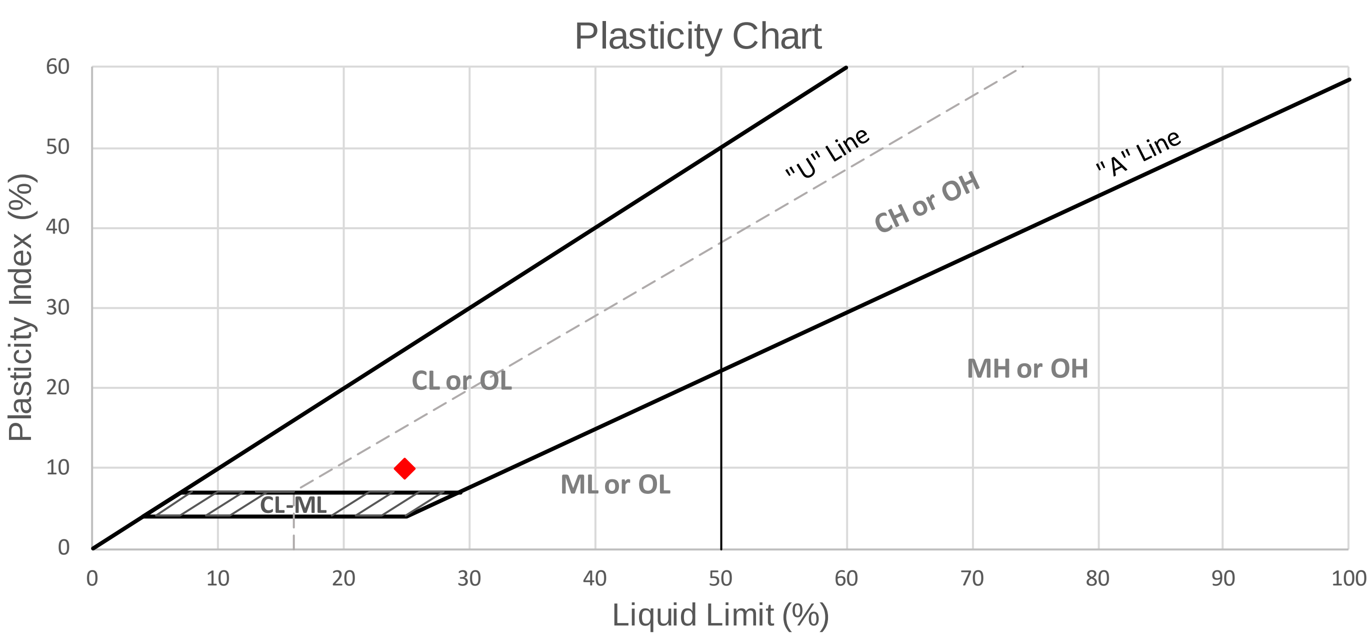
Description: Dark brown silty clayey SAND (SC) **Tested By:** R



Plastic Limit Data

Trial	1	2	Ave
Water Content (%)	14.9	14.3	14.6

Data Summary	
Liquid Limit	**25**
Plastic Limit	**15**
Plasticity Index	**10**
Natural Water Content	**15.7**
Liquidity Index	**0.070**
% Passing #200 Sieve	**38.5**

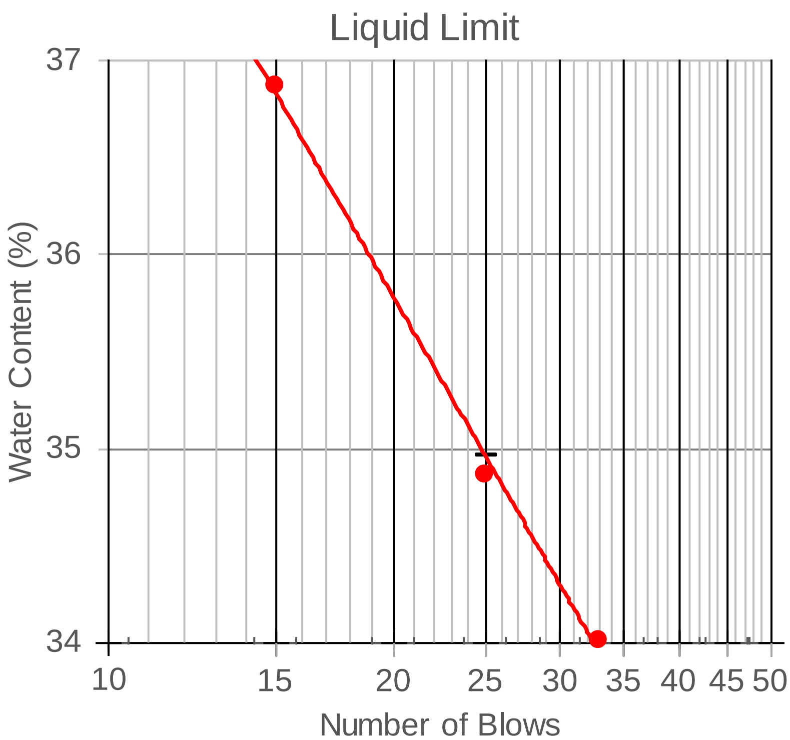


Atterberg Limits Test – ASTM D4318

Project Number: 940-2 **Boring/Sample No:** B-2 **Depth:** 11

Project Name: 2840 Park Avenue **Test Date:** 12-21-21

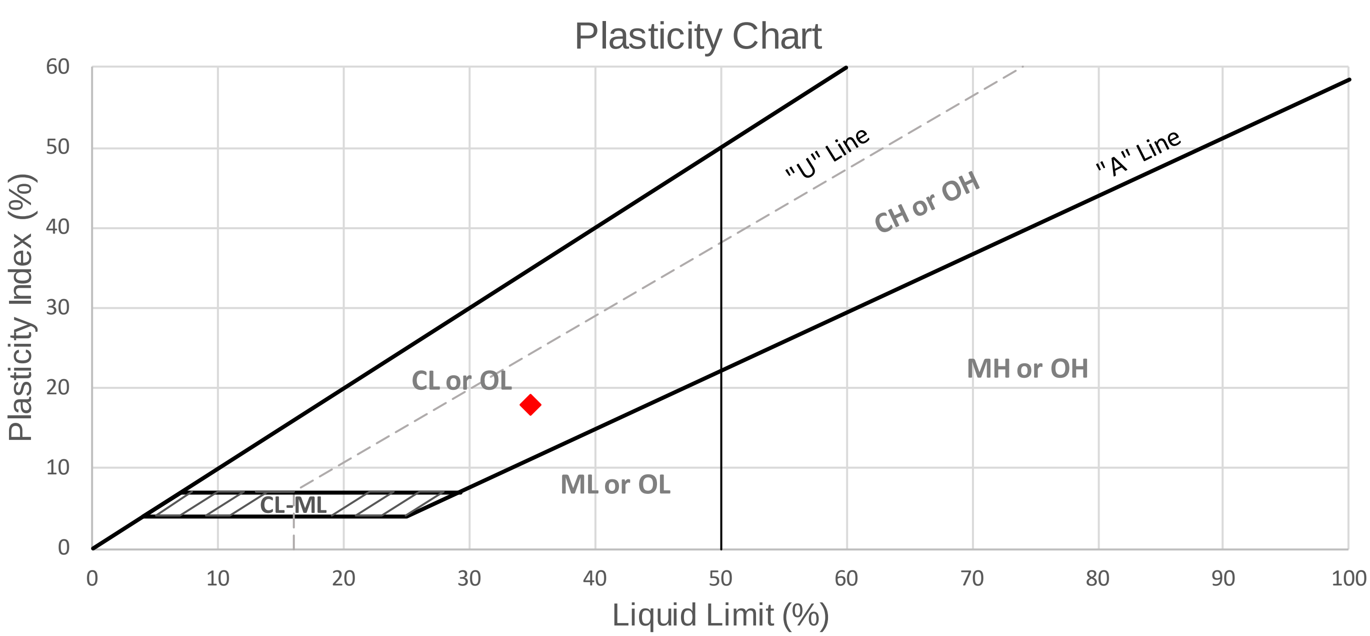
Description: Light rust brown silty clayey SAND (SC) **Tested By:** R



Plastic Limit Data

Trial	1	2	Ave
Water Content (%)	17.2	15.9	16.6

Data Summary	
Liquid Limit	**35**
Plastic Limit	**17**
Plasticity Index	**18**
Natural Water Content	**17.7**
Liquidity Index	**0.039**
% Passing #200 Sieve	**35.4**



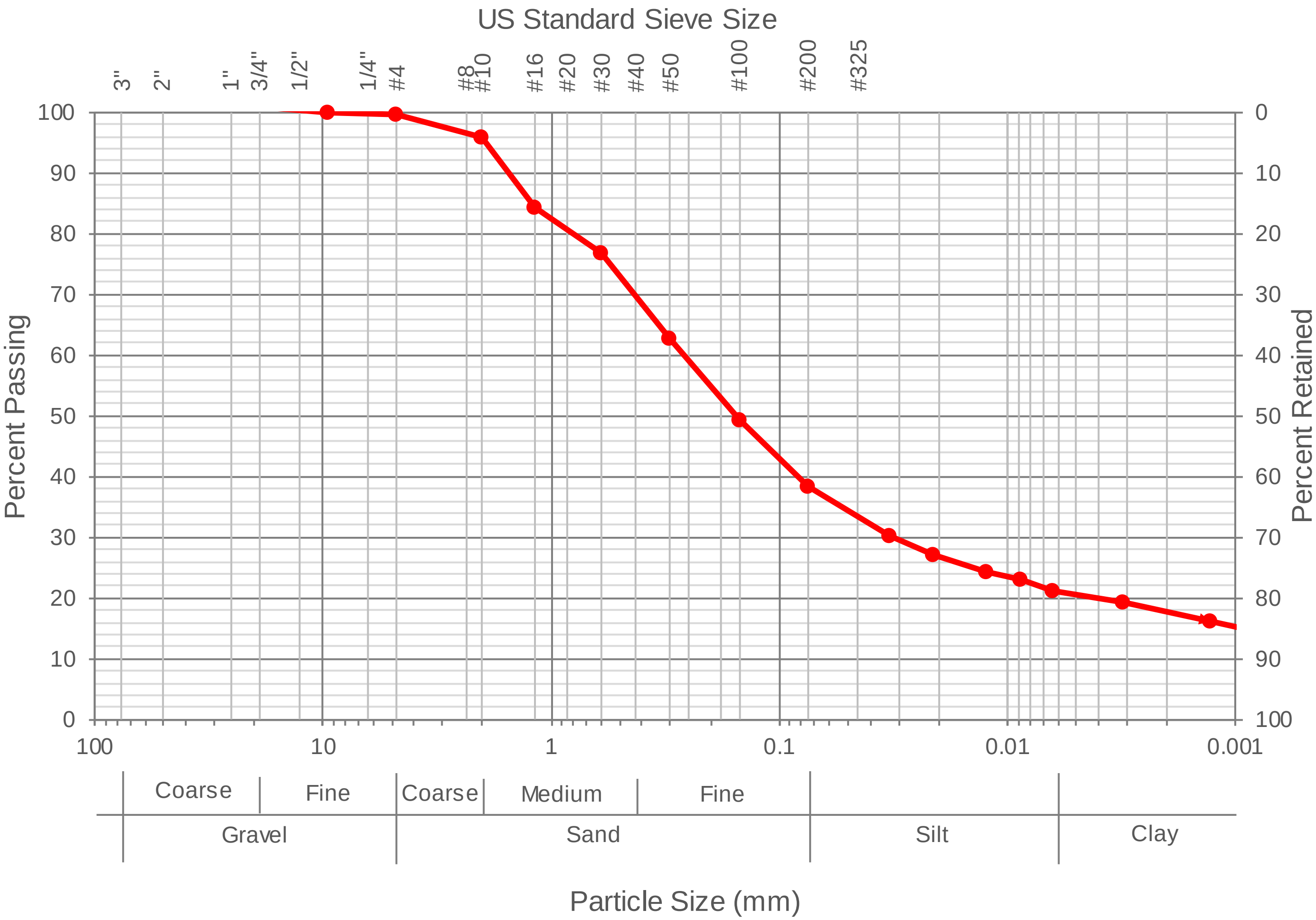
Stevens Ferrone & Bailey Engineering Company, Inc

Hydrometer Analysis – ASTM D422

Project Number: 940-2 **Boring/Sample No:** B-2 **Depth:** 6

Project Name: 2840 Park Avenue **Test Date:** 12-22-21

Description: Dark brown silty clayey SAND (SC) **Tested By:** R



Composite Sieve Data

Standard Sieve Size	Percent Passing
3”	
1.5”	
3/4”	
3/8”	100.0
#4	99.5
#10	95.9
#16	84.2
#30	77.0
#50	62.6
#100	49.3
#200	38.5

Particle Diameter (mm)	Percent Soil in Suspension
0.0326	30.3
0.0211	27.2
0.0124	24.2
0.0088	23.2
0.0063	21.2
0.0031	19.2
0.0013	16.1

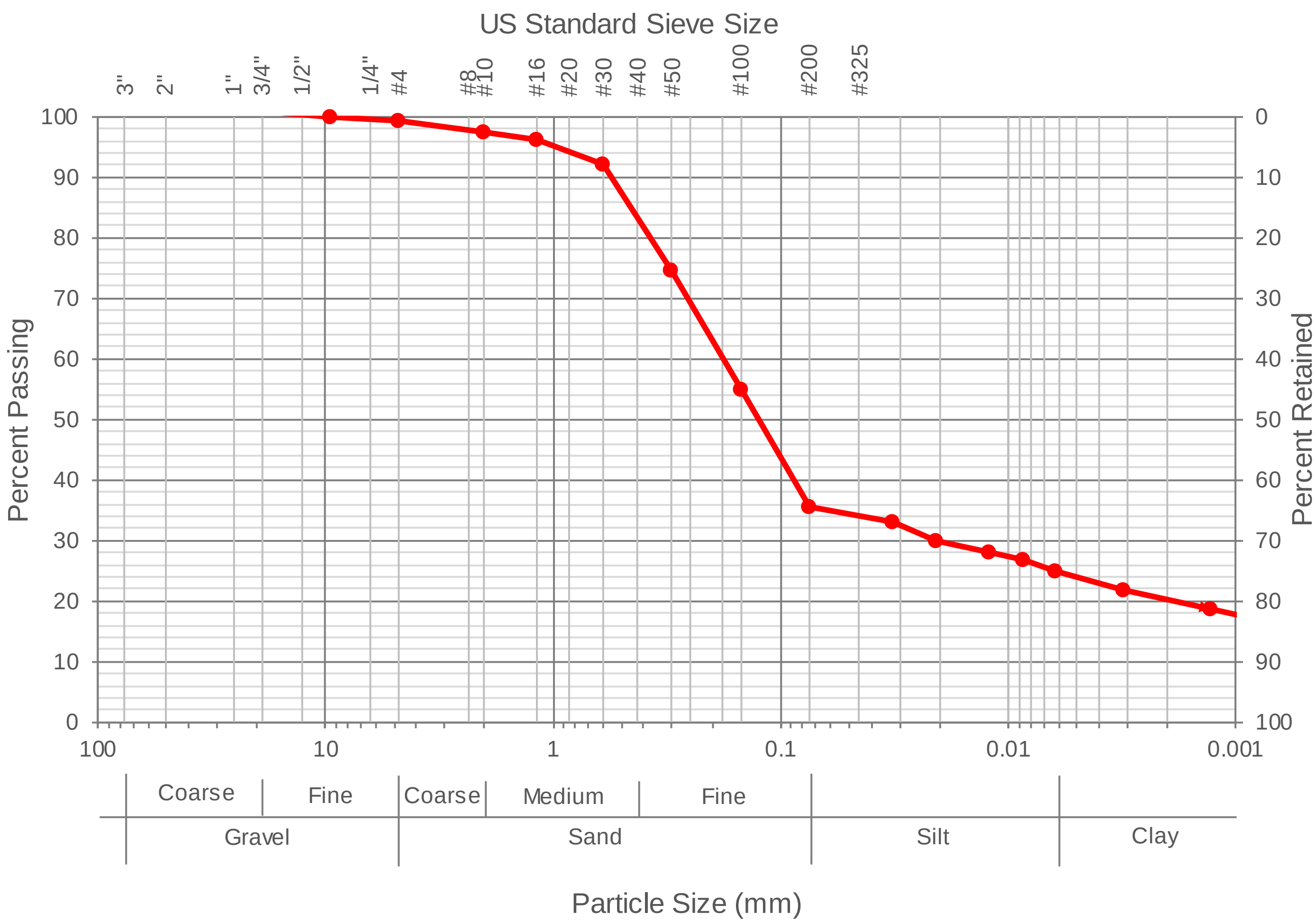
Stevens Ferrone & Bailey Engineering Company, Inc

Hydrometer Analysis – ASTM D422

Project Number: 940-2 **Boring/Sample No:** B-2 **Depth:** 11

Project Name: 2840 Park Avenue **Test Date:** 12-22-21

Description: Light rust brown silty clayey SAND (SC) **Tested By:** R



Composite Sieve Data

Standard Sieve Size	Percent Passing
3”	
1.5”	
3/4”	
3/8”	100.0
#4	99.2
#10	97.6
#16	96.3
#30	92.3
#50	74.7
#100	55.0
#200	35.4

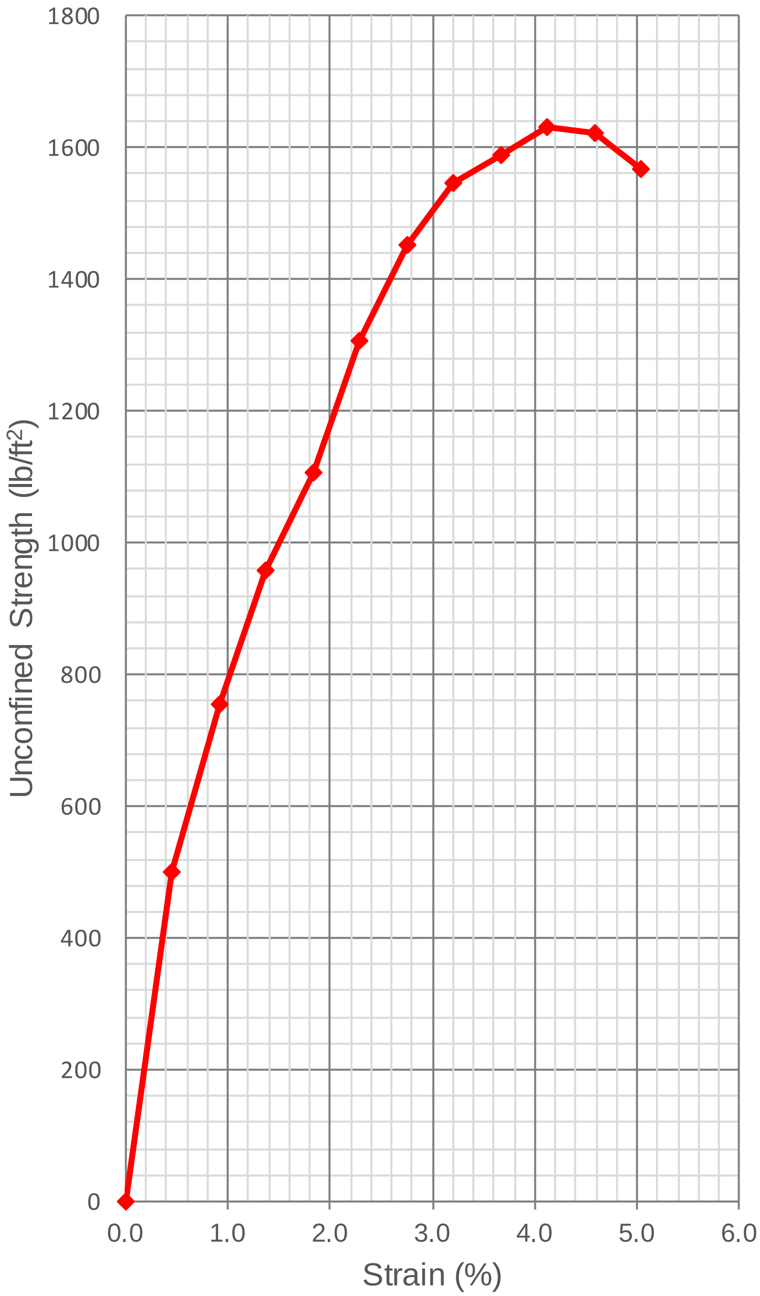
Particle Diameter (mm)	Percent Soil in Suspension
0.0322	33.2
0.0207	30.1
0.0122	28.0
0.0086	27.0
0.0062	24.9
0.0031	21.8
0.0013	18.7

UNCONFINED COMPRESSIVE STRENGTH – D2166

Project Number: 940-2 **Boring/Sample No:** B-1 **Depth :** 2

Project Name: 2840 Park Avenue **Date:** 12-20-21

Description: Dark brown sandy silty CLAY trace gravel (CL) **Tested By:** R



Soil Specimen Initial Measurements

Diameter	2.42 in
Initial Area	4.60 in^2
Initial Length	5.45 in
Volume	0.01451 ft^3
Water Content	13.9 %
Wet Density	122.5 pcf
Dry Density	107.6 pcf

Max Unconfined Compressive Strength

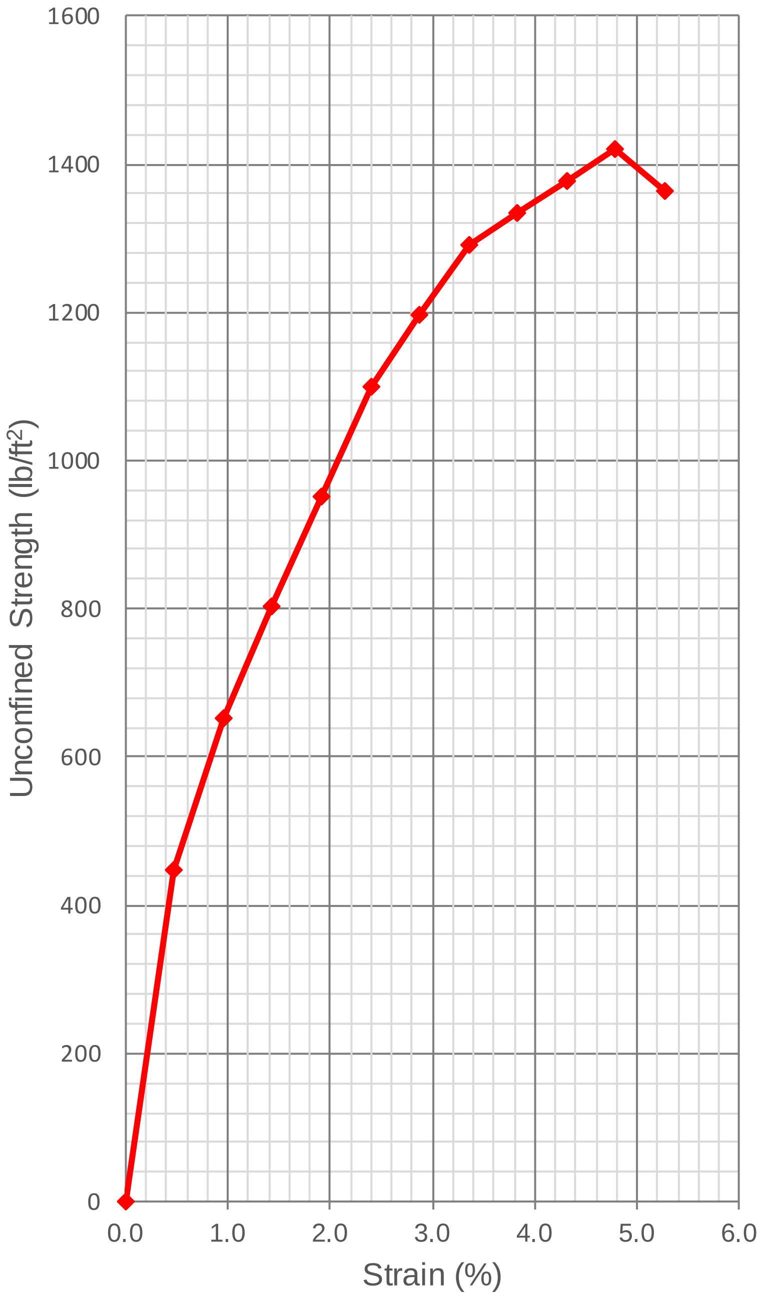
Elapsed Time	4.5 min
Vertical Dial	0.225 in
Strain	4.1 %
Area	0.03332 ft^2
Axial Load	54.3 lbs
Compressive Strength	1,630 psf

UNCONFINED COMPRESSIVE STRENGTH – D2166

Project Number: 940-2 **Boring/Sample No:** B-2 **Depth :** 2

Project Name: 2840 Park Avenue **Date:** 12-20-21

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



Soil Specimen Initial Measurements	
Diameter	2.42 in
Initial Area	4.60 in^2
Initial Length	5.22 in
Volume	0.01389 ft^3
Water Content	14.0 %
Wet Density	122.0 pcf
Dry Density	107.1 pcf

Max Unconfined Compressive Strength	
Elapsed Time	5.0 min
Vertical Dial	0.25 in
Strain	4.8 %
Area	0.03355 ft^2
Axial Load	47.7 lbs
Compressive Strength	1,422 psf

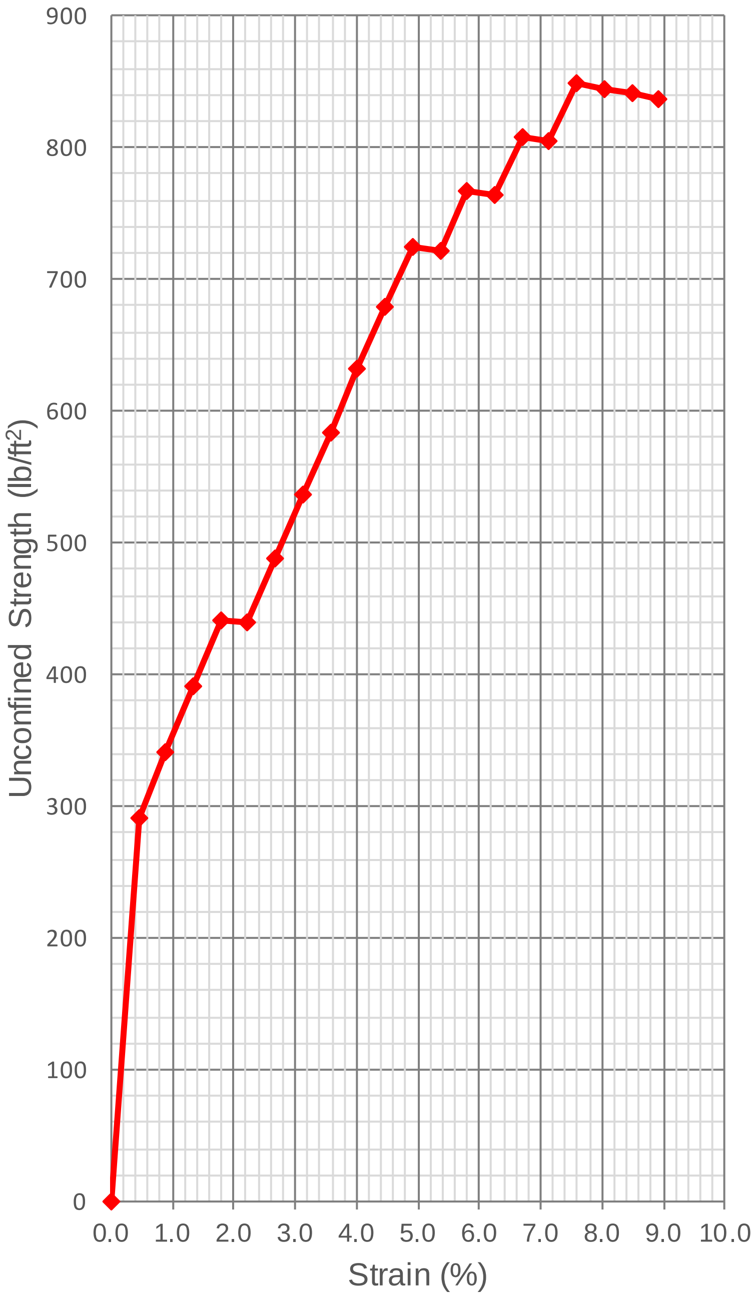
Stevens Ferrone & Bailey Engineering Company, Inc

UNCONFINED COMPRESSIVE STRENGTH – D2166

Project Number: 940-2 **Boring/Sample No:** B-2 **Depth :** 6

Project Name: 2840 Park Avenue **Date:** 12-20-21

Description: Dark brown silty clayey SAND (SC) **Tested By:** R



Soil Specimen Initial Measurements	
Diameter	2.42 in
Initial Area	4.60 in^2
Initial Length	5.6 in
Volume	0.01491 ft^3
Water Content	15.7 %
Wet Density	128.7 pcf
Dry Density	111.2 pcf

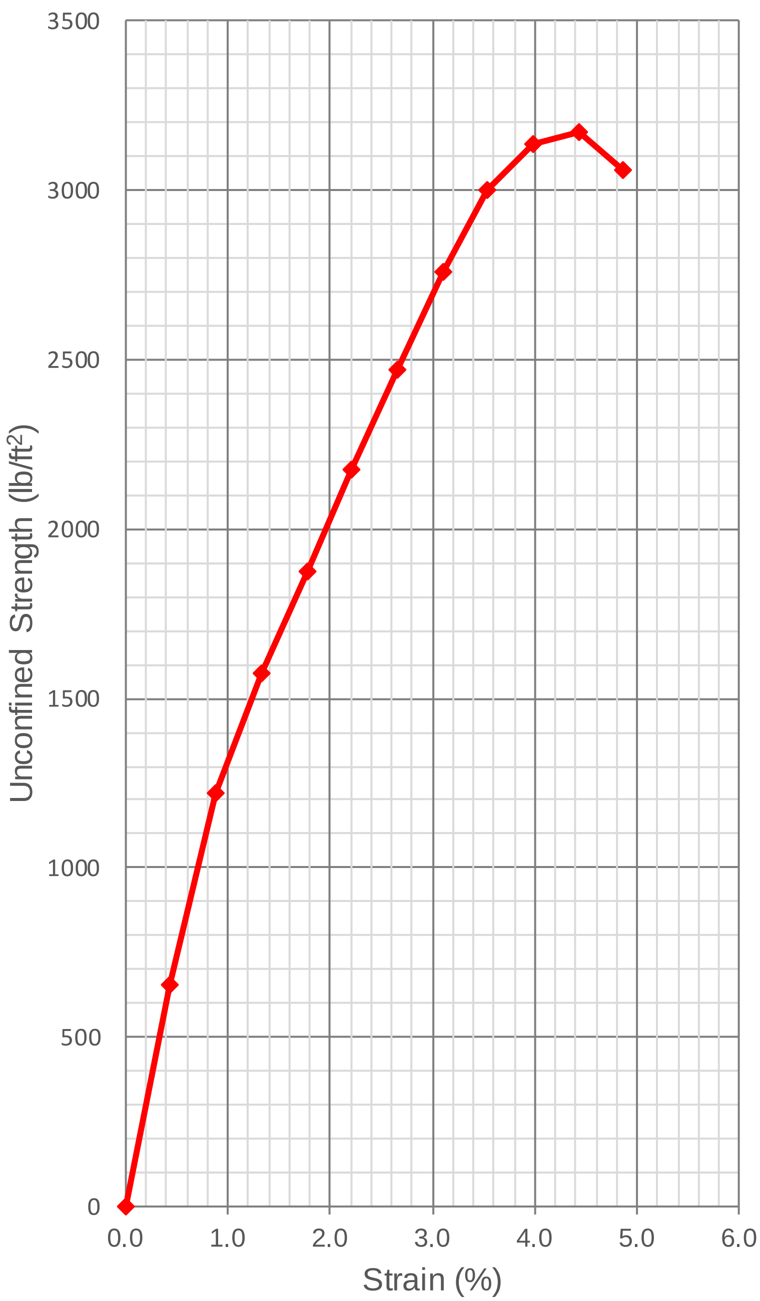
Max Unconfined Compressive Strength	
Elapsed Time	8.5 min
Vertical Dial	0.425 in
Strain	7.6 %
Area	0.03456 ft^2
Axial Load	29.3 lbs
Compressive Strength	848 psf

UNCONFINED COMPRESSIVE STRENGTH – D2166

Project Number: 940-2 **Boring/Sample No:** B-2 **Depth :** 11

Project Name: 2840 Park Avenue **Date:** 12-20-21

Description: Light rust brown silty clayey SAND (SC) **Tested By:** R



Soil Specimen Initial Measurements	
Diameter	2.42 in
Initial Area	4.60 in^2
Initial Length	5.65 in
Volume	0.01504 ft^3
Water Content	17.7 %
Wet Density	131.5 pcf
Dry Density	111.7 pcf

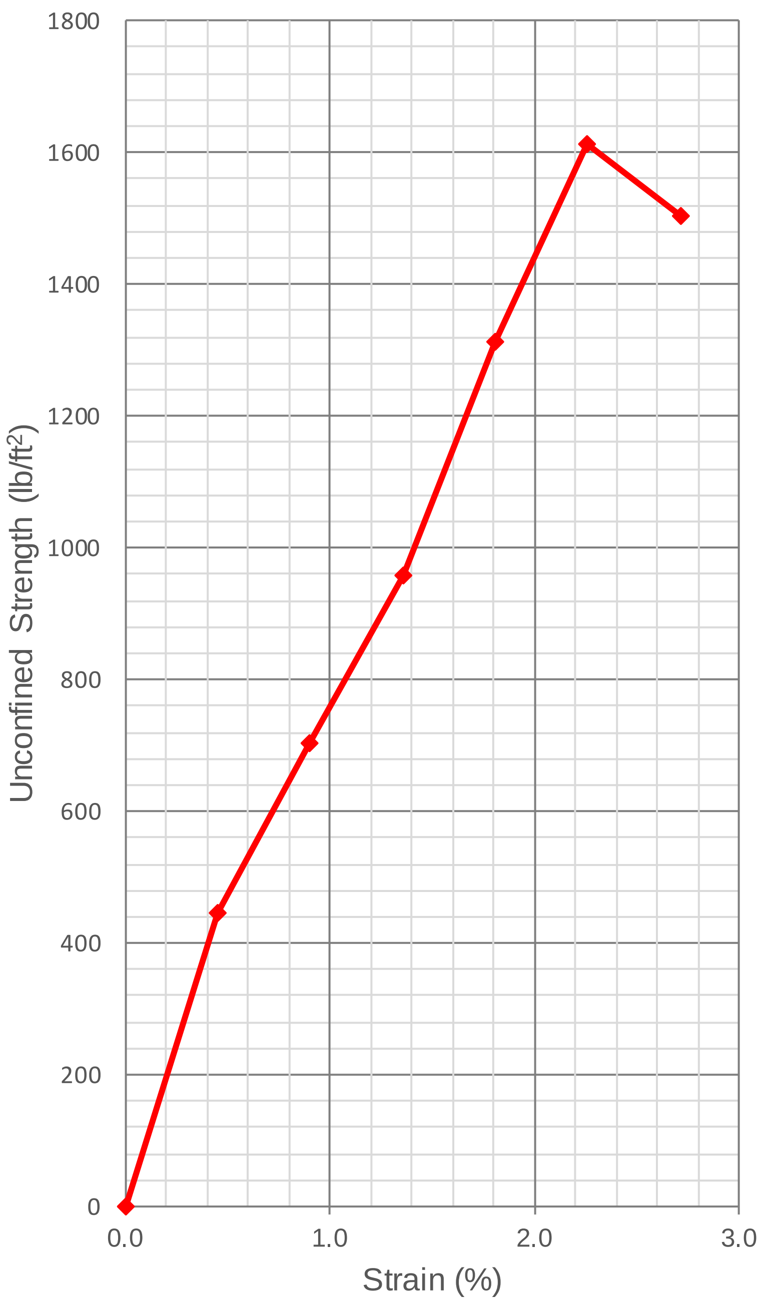
Max Unconfined Compressive Strength	
Elapsed Time	5.0 min
Vertical Dial	0.25 in
Strain	4.4 %
Area	0.03342 ft^2
Axial Load	106.0 lbs
Compressive Strength	3,171 psf

<u>UNCONFINED COMPRESSIVE STRENGTH – D2166</u>

Project Number: 940-2 **Boring/Sample No:** B-5 **Depth :** 2

Project Name: 2840 Park Avenue **Date:** 12-20-21

Description: Dark brown silty SAND with AC (SM) **Tested By:** R



Soil Specimen Initial Measurements	
Diameter	2.42 in
Initial Area	4.60 in^2
Initial Length	5.53 in
Volume	0.01472 ft^3
Water Content	8.3 %
Wet Density	127.2 pcf
Dry Density	117.5 pcf

Max Unconfined Compressive Strength	
Elapsed Time	2.5 min
Vertical Dial	0.125 in
Strain	2.3 %
Area	0.03268 ft^2
Axial Load	52.7 lbs
Compressive Strength	1,612 psf

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

23 December, 2021

Job No. 2112015
Cust. No. 11486

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

Subject: Project No.: SFB 940-2
Project Name: 2840 Park Avenue, Soquel, CA
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Chen:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on December 13, 2021. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, Sample No.001 is classified as "moderately corrosive" and Sample No.002 is classified as "mildly corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

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

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

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

The pH of the soils are 5.90 & 6.23 which does present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures. Soils with a pH of <6.0 is considered to be corrosive to buried iron, steel, mortar-coated steel and reinforced concrete structures. Therefore, corrosion prevention measures need to be considered for structures to be placed in this acidic soil.

The redox potentials are 420-mV & 430-mV. Both samples are indicative of aerobic soil conditions.

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

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

Very truly yours,
CERCO ANALYTICAL, INC.

Sherri Moore

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

JDH/jdl
Enclosure

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Client: Stevens, Ferrone & Bailey Engineering
Client's Project No.: 940-2
Client's Project Name: 2840 Park Avenue, Soquel, CA
Date Sampled: 9-Dec-21
Date Received: 13-Dec-21
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 23-Dec-2021

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)*
2112015-001	B-2 @ 6'	430	6.23	-	6,100	N.D.	N.D.	N.D.
2112015-002	B-3 @ 2'	420	5.90	-	16,000	N.D.	N.D.	N.D.

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Detection Limit:	-	-	10	-	50	15	15
Date Analyzed:	22-Dec-2021	22-Dec-2021	-	21-Dec-2021	23-Dec-2021	22-Dec-2021	22-Dec-2021

* Results Reported on "As Received" Basis

N.D. - None Detected

Sherri Moore
for Cheryl McMillen
Laboratory Director

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

APPENDIX C

GBA Guidelines for Geotechnical Report

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

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

Understand the Geotechnical-Engineering Services Provided for this Report

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

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

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

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

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

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

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

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

Read this Report in Full

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

You Need to Inform Your Geotechnical Engineer About Change

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

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

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept* *responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

Most of the "Findings" Related in This Report Are Professional Opinions

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

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are <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

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