Stevens Ferrone & Bailey Engineering Company, Inc

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

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GEOTECHNICAL INVESTIGATION
RESIDENTIAL DEVELOPMENT
139 MILES LANE
WATSONVILLE, CALIFORNIA
SFB PROJECT NO. 824-2

Prepared For:

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Table of Contents

TABLE OF CONTENTS
(Continued)

FIGURES

APPENDICES

1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed new residential development to be located at 139 Miles Lane in Watsonville, California as shown on the Site Plan, Figure 1. The purpose of our investigation was to evaluate the geotechnical conditions at the site and provide recommendations regarding the geotechnical engineering aspects of the project.

Based on the information indicated on the Site Plan, as well as information provided by Mr. Carlos Jurado of MidPen Housing, it is our understanding that the project will consist of constructing approximately 20 new residential units in three multi-family residential buildings consisting of two-stories of Type V wood frame construction above carports. A wetland habitat will cross the lower portion of the site. Associated underground utilities, parking, and access driveways are planned. Cuts and fills on the order of 5 to 10 feet is expected for the planned development.

Elevations described in this report and shown on the boring logs are based upon elevations (datum unknown) shown on the conceptual site plan prepared by Wald Ruhnke & Dost Associates (WR&D). Figure 1 shows the WR&D conceptual plan and elevations.

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 should any changes to the project occur to assess if the changes affect the validity of this report.

2.0 SCOPE OF WORK

This investigation included the following scope of work:

- Reviewing published and unpublished geotechnical and geological literature relevant to the site;
- Performing reconnaissance of the site and surrounding area;
- Performing a subsurface exploration program, including drilling four exploratory borings to a maximum depth of about 41 ½ feet;
- Performing laboratory testing of samples retrieved from the borings;
- Performing engineering analysis of field and laboratory data; and
- Preparation of this report.

The data obtained and the analyses performed were for the purpose of providing geotechnical design and construction criteria for site earthwork, underground utilities, building foundations, retaining and basement walls, and pavements. Toxicity potential assessment of onsite materials, soils, or groundwater (including mold) and flooding evaluations were beyond our scope of work.

3.0 SITE INVESTIGATION

Reconnaissance of the site and surrounding area and subsurface exploration was performed on October 9, 2018. Subsurface exploration consisted of four exploratory borings drilled with a track-mounted Geoprobe 7822DT drill rig equipped with 7-inch diameter continuous flight hollow-stem augers. The borings were drilled to depths of 21½ to 41½ feet below the existing site surface. The approximate locations of our borings are shown on the Site Plan, Figure 1. Logs of our borings and details regarding our field investigation are included in Appendix A. The results of our laboratory tests are discussed in Appendix B. The borings were backfilled with soil cuttings in accordance with Santa Cruz County Environmental Health requirements prior to leaving the site. It should be noted that changes in the surface and subsurface conditions can occur over time as a result of either natural processes or human activity and may affect the validity of the conclusions and recommendations in this report.

3.1 Surface

At the time of our investigation and as shown on Figure 1, the site was bounded by single family residential buildings and a vacant lot to the south, undeveloped land and single-family residential buildings to the west, Miles Lane to the northwest, a vacant parcel of land to the north, and commercial development and parking lots to the east. The site was L shaped, and had a plan area of about 2.2 acres with maximum dimensions of about 533 feet by 262 feet. From the high point (about elevation 61 feet) at the eastern corner of the site, the surface of site sloped gently downwards at a rough grade of 9:1 (horizontal:vertical) to the west to the bottom of the wetland at approximate elevation 23 feet, before increasing to the west at a maximum grade of 14:1 (H:V) to approximate elevation 30 feet at the site boundary. The northwest edge of the site climbs relatively steeply at approximately 3:1 towards Miles lane. Surface soils observed on the site were weak and desiccated with cracks up to 5 inches wide.

3.2 Subsurface

Borings SFB-1 and SFB-2 encountered between 8 and 15 feet of firm to very stiff high plasticity clay with sand. Underlying this clay, stiff to very stiff lean clay with varying percentages of silt and sand, and interbedded lenses of medium dense to dense silty and clayey sand up to four feet thick, were encountered to the maximum depth explored of 41½ feet.

Borings SFB-3 and SFB-4 predominantly encountered stiff to very stiff lean clay with sand to the maximum depths explored of 21½ feet below the ground surface at these boring locations.

According to the results of laboratory testing, near-surface clayey materials in SFB-1 and SFB-2 have a high plasticity and high to critical expansion potential whereas near-surface clayey soils encountered elsewhere on the site have medium plasticity and moderate expansion potential.

Detailed descriptions of materials encountered in our exploratory borings are presented on the boring logs in Appendix A. Our attached boring logs and related information depict location specific subsurface conditions encountered during our field investigation. The approximate locations of our borings were determined using pacing or landmark references and should be considered accurate only to the degree implied by the method used.

3.3 Groundwater

Groundwater was encountered in Borings SFB-1 and SFB-2 at depths of 13 to 15 feet, corresponding to approximate elevations 10 to 13 feet. No groundwater was encountered in the other borings drilled to maximum depths of about 21½ feet. Our borings were backfilled with grout upon completion. It should be noted that our borings may not have been left open for a sufficient period of time to establish equilibrium groundwater conditions. Fluctuations in groundwater elevations could occur due to seasonal variations including rainfall, and other factors. It is likely that surface water levels within the wetlands area will rise during rainy months and during periods of flooding. Water levels within the drainage channel may affect the groundwater levels at the site.

3.4 Hydrologic Soil Group

Surficial soils are mapped as Tierra-Watsonville Complex (15 to 30 percent slopes) by USDA Web Soil Survey (WSS)[1]. The Tierra-Watsonville Complex is assigned to Hydrologic Soil Group D by the USDA Natural Resources Conservation Service (NRCS). Group D soils have very slow infiltration rates (approximately 0.0 to 0.06 inches per hour), and high runoff potential during winter rains. Type D soils predominantly consist of clays that have a high shrink-swell potential, soils that have a high-water tables, have a claypan or clay layer at or near the surface, or shallow soils above bedrock or other impermeable soils.

3.5 Geology and Seismicity

According to Brabb (1997)[2], except for the wetland area which crosses the site at lower elevations, the site is underlain by Pleistocene fluvial facies (Qwf) which consists of semi-consolidated, moderately to poorly sorted silt, sand, silty clay, and gravel. According to Brabb, the wetland area of the project is mapped as being underlain by Holocene age basin deposits (Qb) which consist of

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[1] https://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx (accessed 10/23/2018)
[2] Brabb, 1997, *Geologic Map of Santa Cruz County, California*, U.S. Geological Survey Open-File Report 97-489.

unconsolidated, plastic, silty clay and clay rich in organic material and locally containing interbedded thin layers of silt and silty sand.

The project site is located in an area which is considered one of the most seismically active regions in the United States. Significant earthquakes that have occurred in the area are believed to be associated with crustal movements along a system of sub-parallel fault zones that generally trend in a northwesterly direction. According to the Alquist-Priolo Special Studies Zones Map of the Watsonville West Quadrangle, the site is not located in an earthquake fault zone as designated by the State of California[3].

Earthquake intensities will vary throughout the San Francisco Bay Area, depending upon numerous factors including the magnitude of earthquake, the distance of the site to the earthquake epicenter, and the type of underlying materials. The U.S. Geological Survey (2016)[4] has stated that there is a 72 percent chance of at least one magnitude 6.7 or greater earthquake striking the San Francisco Bay region between 2014 and 2043. Therefore, the site will probably be subjected to at least one moderate to severe earthquake that will cause strong ground shaking.

According to the U.S. Geological Survey's Unified Hazard Tool and using the Dynamic: Conterminous U.S. 2008 (v3.3.1) model (accessed 10/30/2018), the resulting deaggregation calculations indicate there is a 10% probability that the site will experience peak ground acceleration exceeding 0.51g in 50 years (design basis ground motion based on stiff soil site condition; mean return time of 475 years). The actual ground surface acceleration may vary depending upon the local seismic characteristics of the underlying bedrock and the overlying unconsolidated soils.

3.4 Liquefaction

Soil liquefaction is a phenomenon primarily associated with saturated, cohesionless, soil layers located close to the ground surface. These soils lose strength during cyclic loading, such as imposed by earthquakes. During the loss of strength, the soil acquires mobility sufficient to permit both horizontal and vertical movements. Soils that are most susceptible to liquefaction are clean, loose, uniformly graded, saturated, fine-grained sands that lie close to the ground surface. According to ABAG, the site is located in a region that has not been mapped for liquefaction

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[3]State of California, *Special Studies Zones, Watsonville West Quadrangle*, Official Map, Effective: January 1,1976.
[4]Aagaard, Blair, Boatwright, Garcia, Harris, Michael, Schwartz, and DiLeo, *Earthquake Outlook for the San Francisco Bay Region 2014–2043,* USGS Fact Sheet 2016–3020, Revised August 2016 (ver. 1.1).

potential[5,6]. According to the U.S. Geological Survey, the site is predominantly underlain by Pleistocene sediments having a low liquefaction potential[7].

To evaluate the liquefaction potential of the medium dense sand lenses encountered in Boring SFB-1, we performed SPT-based liquefaction analyses based on procedures described by the Southern California Earthquake Center (SCEC, Martin and Lew, 1999), EERI Monograph 12 (2008)[8], updated SPT based liquefaction triggering procedures (2014)[9], and in accordance with the 2008 California Geological Survey's (CGS) Special Publication 117A guidelines. We also evaluated the liquefaction potential of silty soils encountered in our borings using criteria published by Andrews and Martin (2000)[10]. As required by the 2016 California Building Code (CBC), a peak ground acceleration from a Maximum Considered Earthquake (MCE) was used in our analyses; the MCE peak ground acceleration has a 2% probability of being exceeded in a 50-year period (mean return time of 2,475 years). Using the U.S. Geological Survey's 2008 hazard data model and applying the ASCE 7-10 Standard for risk category I/II/III (accessed 10/30/2018)[11], the Maximum Considered Earthquake geometric mean peak ground acceleration (PGA_m) for the site is 0.793g, with a mean earthquake magnitude of 7.25.

The results of our liquefaction analyses indicate that sand lenses encountered in SFB-1 below the design groundwater elevation of approximate 10 feet (datum unknown) have a high potential to liquefy. Earthquake induced liquefaction of these sand lenses could result in residual volumetric strains on the order of 2 to 2½ percent. We estimate that the liquefaction of these soils when subjected to an MCE event may cause total aerial ground surface settlements of about 1½ inches. Actual liquefaction induced ground surface damage will vary depending on the thickness of the overlying non-liquefiable soils and the underlying liquefiable soils[12].

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[5]Witter, Knudsen, Sowers, Wentworth, Koehler, and Randolph, 2006, *Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, California*", USGS Open File Report 2006-1037.

[6]Knudsen, Sowers, Witter, Wentworth, and Helly, 2000, "*Preliminary Maps of Quaternary Deposits and Liquefaction Susceptibility, Nine-County San Francisco Bay Region, California*", USGS Open File Report 00-444.

[7] Dupre, William, 1975, *Maps Showing Geology and Liquefaction Potential of Quaternary Deposits in Santa Cruz County, California*, USGS Misc. Field Studies Map MF-648.

[8] Idriss & Boulanger, 2008, *Soil Liquefaction During Earthquakes*, Earthquake Engineering Research Institute, MNO-12.

[9] Boulanger & Idriss, 2014, *CPT and SPT Based Liquefaction Triggering Procedures*, Center for Geotechnical Modeling, Report No. UCD/CGM-14/01, April 2014.

[10] Andrews and Martin, 2000, *Criteria for Liquefaction of Silty Soils*, paper presented during the 12th World Conference on Earthquake Engineering.

[11] https://earthquake.usgs.gov/designmaps/us/application.php?

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

We did not encounter liquefiable soils in the other borings performed onsite, therefore it is likely that the potential for liquefaction induced settlement is limited to the lower portion of the site around the wetland corridor.

Loss of localized foundation bearing support (such as when footing foundations are used) can occur as a result of liquefaction created sand boils, ground cracks, and similar phenomena. In addition, underground pipelines (gas lines, sanitary sewers, water services, etc.) should be properly designed to compensate for the settlement caused by the liquefaction of the underlying supporting soils. It should be noted that after a major liquefaction event, phenomena such as sand boils, ground cracking, and differential movement of overlying improvements such as roadways and utilities will be observed.

As our borings encountered firm to very stiff clay in the upper 10 feet of the borings, and liquefiable soils were only encountered below a depth of 30 feet, it is very unlikely that lateral spreading will affect the site.

4.0 CONCLUSION 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.

EXISTING WEAK SURFACE SOILS AND FILLS: Although undocumented fill was not encountered in our borings, the actual extent of undocumented fill on the site is unknown and areas of fill may be encountered during grading. As there no records of fill placement or compaction, the relative compaction and strength of any existing fill is unknown. In order to reduce the potential for damaging differential settlement of overlying improvements (such as new fills, building foundations, driveways, exterior flatwork, and pavements), we recommend that any fills encountered on the site be completely removed and re-compacted. The over-excavation should extend to depths where competent soil is encountered. The over-excavation and re-compaction of fill materials should also extend at least 5 feet beyond building footprints and at least 3 feet beyond exterior flatwork (including driveways) and pavement wherever possible.

Over-excavations should be performed so that no more than 5 feet of differential fill thickness exists below proposed building foundations. Removed fill materials can be used as new fill provided it is placed and compacted in accordance with the recommendations presented in this report. The extent of the removal and re-compaction will vary across the site and should be determined in the field by SFB at the time of the earthwork operations.

Weak desiccated soils mantle the site and extend to depths of about 3 feet. In order to reduce the potential for damaging differential settlement of overlying improvements including new fill materials, we recommend that these weak soils, if not being removed by the proposed grading, be over-excavated and re-compacted. We estimate the process can consist of removing the upper 2 foot of weak soils, scarifying and re-compacting the bottom 12 inches in-place, and placing moisture-conditioned, compacted engineered fill over the properly prepared subgrade. 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.

DIFFERENTIAL EXPANSION POTENTIAL: Our borings encountered both moderately and critically expansive clayey soils near the surface. In order to provide a more uniform subgrade and reduce the potential for damaging differential movement of building foundations and flatwork, we recommend the proposed grading be performed so that each building foundation and surrounding flatwork be supported on fills with similar expansion potential. We recommend a layer at least 3 feet thick of well-mixed, moisture conditioned, and well blended engineered fill be provided below all building foundations and surrounding flatwork. The compacted, engineered fill layer should extend at least 5 feet beyond building footprints and at least 3 feet beyond exterior flatwork, including driveways. 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.

Clayey fill and native soils will be subjected to volume changes during seasonal fluctuations in moisture content. To reduce the potential for post-construction distress to the proposed structures resulting from swelling and shrinkage of these materials, we recommend that the proposed residential structures be supported on post-tensioned slab foundations.

LIQUEFACTION AND LATERAL SPREADING: The results of our liquefaction analyses indicate that the medium dense saturated sands layers encountered in Boring SFB-1 have a high potential to liquefy. We estimate that the liquefaction of these soils when subjected to an MCE event may cause total aerial ground surface settlements of about 1½ inches. As no liquefiable soils were encountered in the other borings performed onsite, it is likely that liquefiable soils are limited to the lower portion of the site. As we did not encounter liquefiable soils within the upper 10 feet of our borings near the wetland area, it unlikely that lateral spreading will impact the site.

CORROSION POTENTIAL: Two samples retrieved from the borings 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 of concrete and buried metal such as utilities and reinforcing steel. The results of these tests, including a summary of the results, are included in Appendix B. We recommend these test results and summary be forwarded to your designers and contractors. Please be aware that we are not corrosion protection experts; we recommend corrosion protection measures be designed and constructed so that all concrete and metal is protected against corrosion for the life of the project. We also recommend additional testing be performed if the corrosion test results are deemed insufficient by the designers of the corrosion protection measures. Landscaping soils typically contain fertilizers and other materials than can be highly corrosive to metals and concrete; landscaping soils commonly are in contact with foundations. Consideration should be given to testing the corrosion potential characteristics of proposed landscaping soils and other types of imported or modified soils and forwarding the results to your corrosion protection designers and installers.

ADDITIONAL RECOMMENDATIONS: Detailed earthwork, foundation, retaining wall, and pavement recommendations for use in the 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.

4.1 Earthwork

4.1.1 Clearing and Site Preparation

The site should be cleared of all obstructions including any existing utility pipes and their backfill, existing structures and their foundations, designated trees and shrubs and their associated root systems, pavements, concrete, 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, if any, should be abandoned in accordance with Santa Cruz County standards.

From a geotechnical standpoint, any existing fill materials, 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.

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

4.1.2 Weak Soil and Fill Re-Compaction

As described above, weak and desiccated soils mantle the site, extending to depths of about 3 feet. In order to reduce the potential for damaging differential settlement of overlying improvements including new fill materials, we recommend that these weak soils, if not being removed by the proposed grading, be over-excavated and re-compacted. We estimate the process can consist of removing the upper 2 feet of weak soils, scarifying and re-compacting the bottom 12 inches in-place, and placing moisture-conditioned, compacted engineered fill over the properly prepared subgrade. 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.

As previously discussed, although undocumented fill was not encountered in any of our borings, it is possible that areas of fill may be encountered during grading. We recommend existing fills be completely removed and re-compacted. The over-excavation should extend to depths where competent soil is encountered. The over-excavation and re-compaction should also extend at least 5 feet beyond building footprints and at least 3 feet beyond exterior flatwork (including driveways) and pavement wherever possible.

Where 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. Removed fill materials can be used as new fill provided it is placed and compacted in accordance with the recommendations presented in this report. The extent of the removal and re-compaction will vary across the site and should be determined in the field by SFB at the time of the earthwork operations.

Removed fill and desiccated surface soil materials may be used as new fill onsite provided it satisfies 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 Pads

After all grading is completed, the resulting compacted fill below building pads should not exceed 5 feet in differential fill thickness. Over-excavation of native soils during grading may need to be performed in order to satisfy this recommendation. The lateral extent of over-excavations should be determined based upon a 1:1 plane projected downward from the outermost edge of the planned foundation.

4.1.4 Subgrade Preparation

After the completion of clearing and site preparation, soil exposed in areas to receive improvements such as structural fill, slabs-on-grade, and new pavement should be scarified to a depth of about 12 inches, moisture conditioned approximately 3 to 5 percent over optimum water content and compacted to the requirements for structural fill.

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

4.1.5 Fill Material

From a geotechnical and mechanical standpoint, onsite soils having an organic content of less than 3 percent by volume can be used as fill. Fill should not contain rocks or lumps larger than 6 inches in greatest dimension with not more than 15 percent larger than 2.5 inches. If required, imported fill should have a plasticity index of 25 or less and have a significant amount of cohesive fines.

In addition to the mechanical properties specifications, all imported fill material should have a resistivity (100% saturated) no less than the resistivity for 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

We recommend general engineered fill be compacted to at least 90 percent relative compaction, as determined by ASTM D1557 (latest edition). We recommend the new fill be moisture conditioned approximately 3 to 5 percent over optimum water content. The upper 6 inches of subgrade soils beneath pavements should be compacted to at least 95 percent relative compaction. Fill materials below a depth of 10 feet should also be compacted to at least 95 percent relative compaction. Fill material should be spread and compacted in lifts not exceeding approximately 8 to 12 inches in un-compacted thickness.

4.1.7 Utility Trench Backfill

Pipeline trenches should be backfilled with fill placed in lifts of approximately 8 to 12 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 should be added during backfilling operations to prevent the soil from "bulking" during compaction. The upper 3 feet of trench backfill in 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 utility trenches that extend toward slabs-on-grade should be plugged with onsite clays, low strength concrete, or sand/cement slurry. The plug for the trenches should be located below the edge of slabs. The plug should be at least 24 inches thick, extend across the entire width of the trench, and extend from the bottom of the trench to the top of the sand or gravel backfill.

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

4.1.8 Exterior Flatwork

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

Expansive clayey soils at the site will be subjected to volume changes during fluctuations in moisture content. As a result of these volume changes, some vertical movement of exterior slabs should be anticipated. This movement could result in damage to the exterior slabs and might require periodic maintenance or replacement. Adequate clearance should be provided between the exterior slabs and structure elements that overhang these slabs.

We recommend reinforcing exterior slabs with steel bars in lieu of wire mesh. To reduce potential crack formation, #4 bars spaced at approximately 18 inches on center in both directions should be installed. Score joints and expansion joints should be used to control cracking and allow for expansion and contraction of the concrete slabs. We recommend appropriate flexible, relatively impermeable fillers be used at all cold/expansion joints. The installation of dowels at all expansion and cold joints will reduce differential slab movements; the dowels should be at least 30 inches long and should be spaced at a maximum lateral spacing of 18 inches. Although exterior slabs that are adequately reinforced will still crack, trip hazards requiring replacement of the slabs will be reduced if the slabs are properly reinforced.

4.1.9 Construction During Wet Weather Conditions

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

4.1.10 Surface Drainage, Landscaping, and Irrigation

Ponding of surface water must not be allowed on pavements, adjacent to foundations, or 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 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 the structure or exterior slabs than half the mature height of the tree; in no case should tree roots be allowed to extend near or below foundations or exterior slabs.

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

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

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

4.1.11 Storm Water Runoff Structures

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

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

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

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

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

4.1.12 Future Maintenance

We recommend regular inspection and maintenance of the site be performed, including maintenance prior to rainstorms. Inspection should include checking drainage patterns, making sure drainage systems are functional and not clogged, and erosion control measures are adequate for anticipated storm events. Immediate repair should be performed if any of these measures appears to be inadequate. 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. Temporary and permanent erosion and sediment control measures should be installed over any exposed soils immediately after repairs are made.

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

4.1.13 Additional Recommendations

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

4.2 Foundation Support

4.2.1 Buildings

The proposed buildings can be supported on post-tensioned slab foundations that are designed for the expansion potential of the onsite soils. The post-tensioned slab foundations should bear entirely on properly prepared, compacted structural fill. In no case should a slab foundation bear upon fills with differential expansion characteristics. Recommendations for building pad preparation are described previously in **Sections 4.1.2, 4.1.3**, and **4.1.4**. Prior to the concrete pour, we recommend the moisture content of the subgrade materials be at least 3 percent above laboratory optimum moisture. If the building pads are left exposed for an extended period of time prior to constructing foundations, we recommend SFB be contacted for recommendations to re-condition the pads in order provide adequate building support.

The post-tensioned slab thickness should be determined by the Structural Engineer, however we recommend the post-tensioned slabs be at least 10 inches thick. An allowable bearing pressure of 1,500 pounds per square foot can be used to evaluate column load support provided the slab is thickened, the thickened area is at least 12 inches wide, and the thickened slab section is supported directly on the underlying, properly prepared building pad subgrade. Thickening (turning down) of slab edge may be necessary to adequately provide support at the perimeter. Slab thickening will also be necessary at hold-down anchor locations.

Deflection of the slab foundations should not exceed the values recommended in the most recent PTI Manual. Lateral loads, such as derived from earthquakes and wind, can be resisted by friction between the post-tensioned slab foundation bottom and the supporting subgrade. A friction coefficient of 0.25 is considered applicable.

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

A vapor retarder must be placed between subgrade soils and the bottom of the slabs-on-grade. We recommend the vapor retarder consist of a single layer of Stego Wrap Vapor Barrier 15 mil Class A or equivalent provided the equivalent satisfies the following criteria: a permeance as tested before and after mandatory conditioning of less than 0.01 Perms and strength of Class A as determined by ASTM E 1745 (latest edition), and a thickness of at least 15 mils. Installation of the vapor retarder should conform to the latest edition of ASTM E 1643 (latest edition) and the manufacturers requirements, including lapping all joints least 6 inches and sealing with Stego Tape or equal in accordance with the manufacturer's specifications. Protrusions where pipes or conduit penetrate the membranes should be sealed with either one or a combination of Stego Tape, Stego Mastic, Stego Pipe Boots, or a product of equal quality as determined by the manufacturer's instructions and ASTM E 1643. Care must be taken to protect the membrane from tears and punctures during construction. We do not recommend placing sand or gravel over the membrane.

Concrete slabs retain moisture and often take many months to dry; construction water added during the concrete pour further increases the curing time. If the slabs are not allowed to completely cure prior to constructing the super-structure, the concrete slabs will expel water vapor and the vapor will be trapped under impermeable flooring. The concrete mix design for the slabs should have a maximum water/cement ratio of 0.45; the actual water/cement ratio may need to be reduced if the concentration of soluble sulfates or chlorides in the supporting subgrade is detrimental to the concrete and reinforcing steel. The results of sulfate and chloride testing of two onsite soil samples, including a brief summary of the results, are included in Appendix B. We recommend you consult with your concrete slab designers and concrete contractors regarding methods to reduce the potential for differential concrete curing.

An experienced Structural Engineer should design the post-tensioned slabs to resist the differential soil movement. The soil design parameters presented below were generated using the procedures presented in the Post-Tensioning Institute (PTI) design manual and PTI published specifications, and the PTI preferred computer program VOLFLO was employed to simulate the wetting and drying scenarios of the soils beneath the post-tensioned slabs.

The values provided below are based upon the post-tensioned slab foundations being entirely surrounded by uniform, properly drained, moderately irrigated landscaping; if differing conditions will exist that will cause differential soil moisture adjacent or below the slabs, or if portions of the foundations will be located adjacent to relatively dry or wet soils, then we should be consulted and modifications to the values below would need to be modified in writing. Please refer to **Section 4.1.10,** ***Surface Drainage, Irrigation, and Landscaping***, for additional recommendations. We recommend the slab-subgrade friction values provided in the most recent PTI Manual be used in order to determine the friction that might be expected to exist during tendon stressing.

SWELLING MODE

	Center Lift	Edge Lift
Edge Moisture Variation Distance (e_m)	7.0 feet	4.0 feet
Differential Soil Movement (y_m)	0.5 inch	1.5 inch

We recommend SFB review the foundation drawings and specifications prior to submittal to verify that the recommendations provided in this report have been used and properly interpreted in the design of the slabs.

4.2.2 Retaining Walls and Soundwalls

Where walls retain soil, they must be designed to resist both lateral earth pressures and any additional lateral loads caused by surcharging such as building and roadway loads. Where concrete or masonry walls are used to retain soil, we recommend unrestrained walls (walls free to deflect and disconnected from other structures) be designed to resist an equivalent fluid pressure of 50 pounds per cubic foot. This assumes a level backfill. Restrained walls (walls restrained from deflection) should be designed to resist an equivalent fluid pressure of 50 pounds per cubic foot plus a uniform pressure of 10H pounds per square foot, where H is the height of the wall in feet. Walls with inclined backfill should be designed for an additional equivalent fluid pressure of 1 pound per cubic foot for every 1 degree of slope inclination. Walls subjected to surcharge loads should be designed for an additional uniform lateral pressure equal to one-third and one-half the anticipated surcharge load for unrestrained and restrained walls, respectively. These lateral pressures depend upon the moisture content of the retained soils to be constant over time; if the moisture content of the retained soils will fluctuate or increase compared to the moisture content at time of construction, then SFB should be consulted and provide written modifications to this design criteria.

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.

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 2016 CBC, we recommend the walls also be designed to resist a triangular pressure distribution equal to an equivalent fluid pressure of 27 pounds per cubic foot. This seismic induced earth pressure is in addition to the static earth pressures noted above. Due to the transient nature of the seismic loading, a factor of safety of at least 1.1 can be used in the design of the walls when they resist seismic lateral loads. Some movement of the walls may occur during moderate to strong earthquake shaking and may result in distress as is typical for all structures subjected to earthquake shaking.

The recommended lateral pressures assume walls are fully-back drained to prevent the build-up of hydrostatic pressures. This can be accomplished by using ½ to ¾ inch crushed, uniformly graded gravel entirely wrapped in filter fabric such as Mirafi 140N or equal (an overlap of at least 12 inches should be provided at all fabric joints). The gravel and fabric should be at least 8 inches wide and extend from the base of the wall to within 12 inches of the finished grade at the top. Caltrans Class 2 permeable material (Section 68) may be used in lieu of gravel and filter fabric. A 4-inch diameter, perforated pipe should be installed at the base of the wall and centered within the gravel. Perforated pipe should be connected to a solid collector pipe that transmits water directly to a storm drain, drainage inlet, or onto pavement. If weep holes are used in the wall, the use of perforated pipe is not necessary provided the weep holes are kept free of animals and debris, are located no higher than approximately 6 inches from the lowest adjacent grade and are able to function properly. As an alternative to using gravel, drainage panels (such as AWD SITEDRAIN Sheet 94 for walls or equal) may be used behind the walls in conjunction with perforated pipe (connected to solid collector pipe), weep holes, or strip drains (such as SITEDRAIN Strip 6000 or equal). If used, the drainage panels can be spaced on-center at approximately 2 times the panel width.

If heavy compaction equipment is used behind the walls, the walls should be appropriately designed to withstand loads exerted by the heavy equipment and/or temporarily braced. Fill placed behind walls should conform to the recommendations provided in **Section 4.1.5, *Fill Material***, and **Section 4.1.6, *Compaction***.

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

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

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

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

4.2.3 Seismic Design Criteria

The following parameters were calculated using the U.S. Geological Survey's Seismic Design Map program[13], and were based on the site being located at approximate latitude 39.9219°N and longitude 121.7631°W. For seismic design using the 2016 California Building Code (CBC), we recommend the following seismic design parameters be used.

2016 CBC SEISMIC PARAMETERS		
Seismic Parameter	Design Value	CBC Reference
Site Class	D	Section 1613.3.2
S_S	2.070	Figure 1613.3.1(1)
S_1	0.805	Figure 1613.3.1(2)
F_a	1.0	Table 1613.3.3(1)
F_v	1.5	Table 1613.3.3(2)

[13]USGS Website, *http://earthquake.usgs.gov/hazards/designmaps/usdesign.php*, accessed 10/30/2018.

4.3 Pavements

4.3.1 Flexible Pavements

Based on the results of the exploratory borings and laboratory testing of onsite materials, we recommend that an R-value of 5 be used in asphalt concrete pavement design. We developed the following alternative preliminary pavement sections using Topic 608 of the State of California Department of Transportation Highway Design Manual, the recommended R-value, and typical traffic indices for 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.

PAVEMENT DESIGN ALTERNATIVES SUBGRADE R-VALUE = 5			
Location	Pavement Components		Total Thickness (inches)
	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	
T.I. = 4.5 (auto & light truck parking)	3.0	9.0	12.0
T.I. = 5.0 (access ways)	3.0	11.0	14.0
T.I. = 6.0 (primary roadways)	3.0	14.0	17.0

If the pavements are planned to be placed prior to or during construction, the traffic indices and pavement sections may not be adequate for support of what is typically more frequent and heavier construction traffic. If the pavement sections will be used for construction access by heavy trucks or construction equipment (especially fork lifts with support footings), SFB should be consulted to provide recommendations for alternative pavement sections capable of supporting the heavier use and heavier loads. If requested, SFB can provide recommendations for a phased placement of the asphalt concrete to reduce the potential for mechanical scars caused by construction traffic in the finished grade. Preliminary pavement sections should be revised, if necessary, when actual traffic indices are known and pavement subgrade elevations are determined.

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

4.3.2 Rigid Pavements

The analytical procedure used in our design of the rigid vehicular concrete pavement was the method published by the Portland Cement Association. A modulus of subgrade reaction of 85 pounds per square inch per inch was assigned to represent a reworked, onsite subgrade overlain by 6 inches of aggregate base. The modulus of rupture for concrete was assumed to be 550 pounds per square inch. Based on our analysis, we recommend the concrete slab for the trash enclosure consist of 6 inches of concrete overlying 6 inches of Caltrans Class 2 aggregate baserock. The Concrete and baserock should be constructed in accordance with the appropriate specifications for pavements.

5.0 CONCLUSION AND LIMITATIONS

SFB is not responsible for the validity or accuracy of information, analyses, test results, or designs provided to SFB by others or prepared by others. The analysis, designs, opinions, and recommendations submitted in this report are based in part upon the data obtained from our field work and upon information provided by others. Site exploration and testing characterizes subsurface conditions only at the locations where the explorations or tests are performed; actual subsurface conditions between explorations or tests may be different than those described in this report. Variations of subsurface conditions from those analyzed or characterized in this report are not uncommon and may become evident during construction. In addition, changes in the condition of the site can occur over time as a result of either natural processes (such as earthquakes, flooding, or changes in ground water levels) or human activity (such as construction adjacent to the site, dumping of fill, or excavating). If changes to the site 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 MidPen Housing Corporation and their consultants for specific application to the proposed new residential development located at 139 Miles Lane in Watsonville, California, and is intended to represent our design recommendations to MidPen Housing Corporation for specific application to the new residential development project. The conclusions and recommendations contained in this report are solely professional opinions. It is the responsibility of MidPen Housing Corporation to transmit the information and recommendations of this report to those designing and constructing the project. We will not be responsible for the misinterpretation of the information provided in this report. We recommend SFB be retained to review geological and geotechnical aspects of the construction calculations, specifications, and plans; we should also be retained to participate in pre-bid and preconstruction 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. SFB strives to perform its services in a proper and professional manner with reasonable care and competence but we are not infallible. Geological engineering and geotechnical engineering are disciplines that are far less exact than other engineering disciplines; therefore, we should be consulted if it is not completely understood what the limitations to using this report are.

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

This report does not necessarily represent all of the information that has been communicated by us to MidPen Housing Corporation and their consultants during the course of this engagement and our rendering of professional services to MidPen Housing Corporation. Reliance on this report by parties other than those described above must be at their own risk unless we are first consulted as to the parties' intended use of this report and only after we obtain the written consent of MidPen Housing Corporation to divulge information that may have been communicated to MidPen Housing Corporation. We cannot accept consequences for use of segregated portions of this report.

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

FIGURES

KEY

B-4 — APPROXIMATE LOCATION OF SFB EXPLORATORY BORING (10/9/18)

APPROXIMATE PROJECT LIMIT

NOTE: Base map prepared by Wald Ruhnke & Dost Architects LLP and received 9/6/18.

APPROXIMATE SCALE: 1" = 50' (0, 50', 100')	DATE: November 2018; PROJECT NO.: 824-2	Stevens Ferrone & Bailey Engineering Company, Inc; 1600 Willow Pass Court, Concord, CA 94520; Tel 925.688.1001; Fax 925.688.1005; www.SFandB.com	SITE PLAN: 139 MILES LANE, Watsonville, California	FIGURE 1

APPENDIX A

Field Investigation

APPENDIX A

Field Investigation

Our field investigation for the proposed new residential development located at 139 Miles Lane in Watsonville, California consisted of surface reconnaissance and a subsurface exploration program performed on October 9, 2018. Subsurface exploration was performed using a track-mounted drill rig equipped with 7-inch diameter, continuous flight, hollow-stem augers. Four exploratory borings were drilled to maximum depth of about 41-1/2 feet using a Geoprobe 7822DT drill rig. Our representative continuously logged the soils encountered in the borings in the field. The soils are described in general accordance with the Unified Soil Classification System (ASTM D2487). Boring logs as well as a key for the classification of the soil (Figure A-1) are included as part of this appendix.

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

Resistance to sampler advancement was measured using blow counts, which were obtained by dropping a 140-pound auto-trip hammer through a 30-inch free fall. The samplers were driven 18 inches and the number of blows to advance the sampler each 6-inch interval was recorded. The blows per foot recorded on the boring logs represent the accumulated number of converted blows that were required to drive the last 12 inches, unless otherwise noted where greater resistance was encountered. Blow counts recorded on the boring logs have been converted to equivalent N_{60} SPT field blow counts, but have not been corrected for overburden, silt content, or other factors.

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

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions		grf	ltr	Description
Coarse Grained Soils	Gravel Gravelly Soils		GW	Well-graded gravels or gravel sand mixtures, little or no fines
			GP	Poorly-graded gravels or gravel sand mixture, little or no fines
			GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sand And Sandy Soils		SW	Well-graded sands or gravelly sands, little or no fines
			SP	Poorly-graded sands or gravelly sands, little or no fines
			SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, and-clay mixtures

Major Divisions		grf	ltr	Description
Soils	Silts And Clays LL < 50		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silt-clays of low plasticity
	Silts And Clays LL > 50		MH	Inorganic silts, micaceous or diatomaceous fine or silty soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
Highly Organic Soils			PT	Peat and other highly organic soils

GRAIN SIZES

U.S. STANDARD SERIES SIEVE — 200, 40, 10, 4
CLEAR SQUARE SIEVE OPENINGS — 3/4", 3", 12"

Silts and Clays	Sand			Gravel		Cobbles	Boulders
	Fine	Medium	Coarse	Fine	Coarse		

RELATIVE DENSITY

Sands and Gravels	Blows/Foot*
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	Over 50

CONSISTENCY

Silts and Clays	Blows/Foot*	Strength (tsf)**
Very Soft	0 - 2	0 - 1/4
Soft	2 - 4	1/4 - 1/2
Firm	4 - 8	1/2 - 1
Stiff	8 - 16	1 - 2
Very Stiff	16 - 32	2 - 4
Hard	Over 32	Over 4

*Number of Blows for a 140-pound hammer falling 30 inches, driving a 2-inch O.D. (1-3/8" I.D.) split spoon sampler.
**Unconfined compressive strength.

SYMBOLS & NOTES

- Standard Penetration sampler (2" OD Split Barrel)
- Modified California sampler (3" OD Split Barrel)
- California Sampler (2.5" OD Split Barrel)
- Shelby Tube
- Pitcher Barrel
- HQ Core
- Ground Water level initially encountered
- Ground Water level at end of drilling

PI = Plasticity Index
LL = Liquid Limit
R = R-Value

Increasing Visual Moisture Content

Saturated
Wet
Moist
Damp
Dry

Constituent Percentage

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

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KEY TO EXPLORATORY BORING LOGS

139 MILES LANE
Wastsonville, California

PROJECT NO.	DATE	FIGURE NO.
824-2	November 2018	A-1

DRILL RIG Geoprobe 7822DT, HSA	SURFACE ELEVATION 27.5 feet	LOGGED BY OL
DEPTH TO GROUND WATER 13 feet	BORING DIAMETER 8-inch	DATE DRILLED 10/09/18

DESCRIPTION AND CLASSIFICATION: DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UNC. COMP. (KSF)	OTHER TESTS
COLLUVIUM: CLAY (CH), gray-brown, sitly, some sand(fine- to medium-grained), damp to moist.	firm		11	13	84	0.6	
Change color to dark gray-brown, some sand(fine- to coarse-grained).			11				
CLAY (CH), bluish-gray with brown mottling, silty, some sand(fine- to medium-grained, trace coarse-grained), damp to moist.	firm		8	25	99	2.6	
CLAY (CL), bluish-gray with brown mottling, trace gravel(fine, subangular to angular), moist.	stiff		13	23	102		
Change color to olive, silty, some sand(fine- to medium-grained), wet.			14	30			
Change color to yellow-brown and olive-brown.	very stiff		20				
Change color to yellow-brown with iron staining, trace sand(fine- to medium-grained).	stiff		14				
Change color to light gray and red-brown, with sand(fine- to medium-grained).	firm		7				
SAND (SC), brown, fine- to medium-grained, some clay, with silt, wet.	dense						

DEPTH (FEET): 0, 5, 10, 15, 20, 25

ELEVATION: 25, 20, 15, 10, 5, 0

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EXPLORATORY BORING LOG

139 MILES LANE
Wastsonville, California

PROJECT NO.	DATE	BORING NO.
824-2	**November 2018**	**SFB-1**

DRILL RIG Geoprobe 7822DT, HSA	SURFACE ELEVATION 27.5 feet	LOGGED BY OL
DEPTH TO GROUND WATER 13 feet	BORING DIAMETER 8-inch	DATE DRILLED 10/09/18

DESCRIPTION AND CLASSIFICATION			DEPTH (FEET) ELEVATION	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UNC. COMP. (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE							
SAND (SC), continued.	dense		30		36				At 33': Percent Passing #200 Sieve = 47%
SILT (ML), brown and red-brown, sandy(fine- to medium-grained), with clay, wet.	stiff		-5						
SAND (SM), brown, fine- to medium-grained, some silt, wet.	medium dense				22				
SAND (SM), blue-gray, fine- to medium-grained, silty, wet.	medium dense		35		19				At 35': Percent Passing #200 Sieve = 44%
CLAY (CL), blue-gray, silty, trace sand(fine- to medium-grained), wet.	stiff		-10 40		14 16				
Bottom of Boring = 41.5 feet Notes: Stratification is approximate, variations must be expected. Blowcounts converted to SPT N-values. See Report for additional details.			-15 45 -20 50 -25 55 -30						

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EXPLORATORY BORING LOG

139 MILES LANE
Wastsonville, California

PROJECT NO.	DATE	BORING NO.
824-2	**November 2018**	**SFB-1**

DRILL RIG Geoprobe 7822DT, HSA	SURFACE ELEVATION 28 feet	LOGGED BY OL
DEPTH TO GROUND WATER 15 feet	BORING DIAMETER 8-inch	DATE DRILLED 10/09/18

DESCRIPTION AND CLASSIFICATION: DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	DEPTH (FEET)	ELEVATION	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UNC. COMP. (KSF)	OTHER TESTS
COLLIVIUM: CLAY (CH), dark gray-brown, silty, some sand(fine- to medium-grained), damp to moist.	very stiff		0				18	96		At 2': Liquid Limit = 59%; Plasticiy Index = 42; Medium Sand = 2%; Fine Sand = 8%; Silt = 21%; Clay = 69%
						18				
				25						
Change color to dark bluish-gray, trace gravel(fine, subangular to subrounded), moist.	stiff					14				
			5							
Change color to light grayish-blue with iron staining, trace sand(fine-grained).						10	27	96	3.8	
				20						
			10							
						11	33	86		
				15						
			15 ▼							
Change color to olive with iron staining.						9	37	84		
CLAY (CL), blue, silty, trace sand(fine-grained), moist.	stiff									
				10						
			20							
	firm					6				
Bottom of Boring = 21.5 feet Notes: Stratification is approximate, variations must be expected. Blowcounts converted to SPT N-values. See Report for additional details.										
				5						
			25							
				0						

Stevens, Ferrone & Bailey Engineering Company, Inc.
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Concord, CA 94520
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EXPLORATORY BORING LOG

139 MILES LANE
Wastsonville, California

PROJECT NO.	DATE	BORING NO.
824-2	**November 2018**	**SFB-2**

DRILL RIG Geoprobe 7822DT, HSA	SURFACE ELEVATION 33 feet	LOGGED BY OL
DEPTH TO GROUND WATER Not Encountered	BORING DIAMETER 8-inch	DATE DRILLED 10/09/18

DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UNC. COMP. (KSF)	OTHER TESTS
COLLIVIUM: CLAY (CL), grayish-brown, silty, with sand(fine- to coarse-grained), trace gravel(fine, subangular to subrounded), damp.	very stiff		15 16	12	92		
CLAY (CL), light brown, silty, some sand(fine- to medium-grained), moist.	very stiff		16	31	86		
SAND (SC), mottled light gray and red-brown, fine- to medium-grained, with clay, moist.	medium dense						
CLAY (CL), light brown, sandy(fine- to medium-grained), with silt, moist.	very stiff		17	31	91		
CLAY (CL), light brown, silty, trace sand(fine-grained), moist.	very stiff		16 17				
Bottom of Boring = 21.5 feet Notes: Stratification is approximate, variations must be expected. Blowcounts converted to SPT N-values. See Report for additional details.							

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EXPLORATORY BORING LOG

139 MILES LANE
Wastsonville, California

PROJECT NO.	DATE	BORING NO.
824-2	**November 2018**	**SFB-3**

DRILL RIG Geoprobe 7822DT, HSA	SURFACE ELEVATION 52.5 feet	LOGGED BY OL
DEPTH TO GROUND WATER Not Encountered	BORING DIAMETER 8-inch	DATE DRILLED 10/09/18

DESCRIPTION AND CLASSIFICATION			DEPTH (FEET)	ELEVATION	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UNC. COMP. (KSF)	OTHER TESTS
DESCRIPTION AND REMARKS	CONSIST	SOIL TYPE								
CLAY (CL), brown, silty, with sand(fine- to medium-grained), damp.	stiff		0							
						15	8	94		
				50						
Change color to mottled olive and red-brown, trace sand(fine- to medium-grained), moist.						13				
			5							
	very stiff					22	26	96		
				45						
			10							
Change color to mottled light gray and red-brown.	stiff					16	37	83		
				40						
			15							
Change color to light brown, trace sand(fine-grained).						13	35	87		
				35						
			20							
						14				
Bottom of Boring = 21.5 feet Notes: Stratification is approximate, variations must be expected. Blowcounts converted to SPT N-values. See Report for additional details.										
				30						
			25							
				25						

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EXPLORATORY BORING LOG

139 MILES LANE
Wastsonville, California

PROJECT NO.	DATE	BORING NO.
824-2	**November 2018**	**SFB-4**

APPENDIX B

Laboratory Investigation

APPENDIX B

Laboratory Investigation

Our laboratory testing program for the proposed new residential development to be located at 139 Miles Lane in Watsonville, California, was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site.

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

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

Atterberg Limit determinations were performed on one subsurface soil sample to determine the range of water content over which the material exhibits plasticity. These values are used to classify the soil in accordance with the Unified Soil Classification System and to indicate the soil's compressibility and expansion potentials. The results of the tests are presented on the boring logs at the appropriate sample depth, and are also attached to this appendix.

The percent passing the #200 sieve was determined on three samples of subsurface soils. These tests were performed to assist in the classification of the soils. The results of the tests are presented on the boring logs at the appropriate sample depths.

Gradation and hydrometer tests were performed on one subsurface soil sample. These tests were performed to assist in the classification of the soils and to determine their grain size distributions. The results of the tests are presented on the boring log at the appropriate sample depth and are included in this appendix.

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

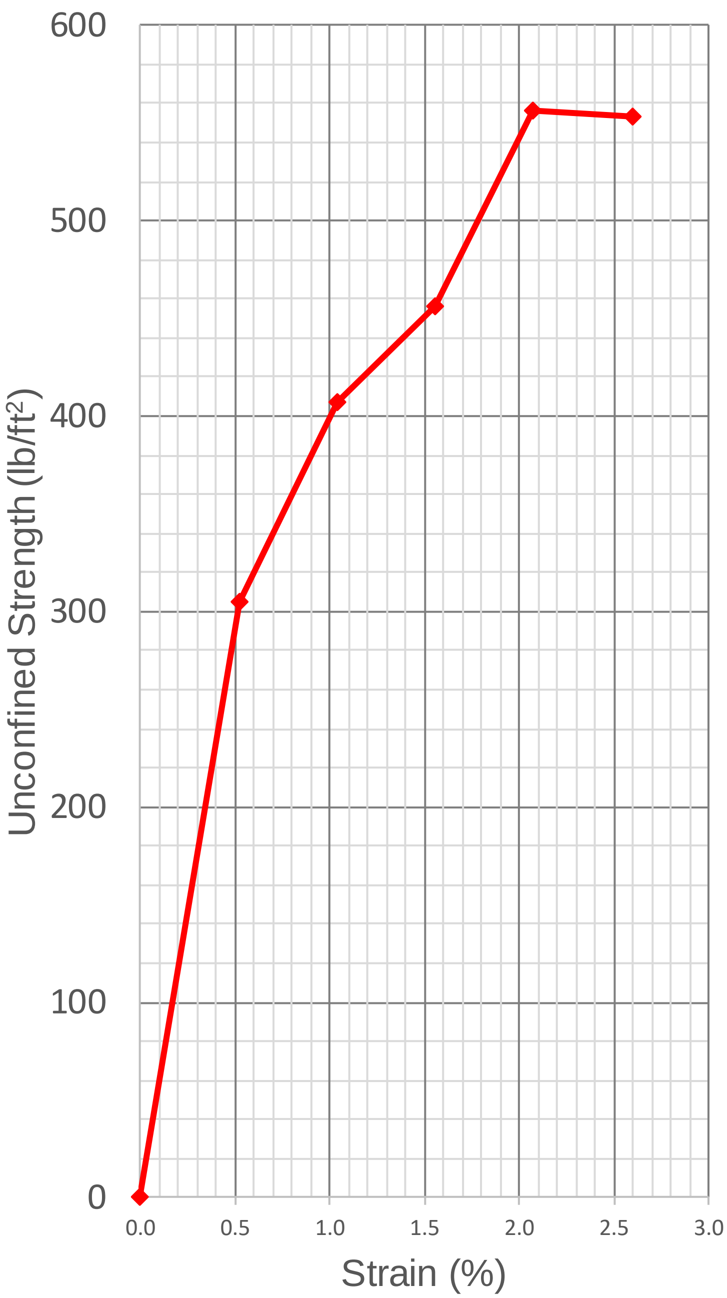
Two onsite soil samples were tested for pH (ASTM D4972), chlorides (ASTM D4327), sulfates (ASTM D4327), sulfides (ASTM D4658M), resistivity at 100% saturation (ASTM G57), and Redox potential (ASTM D1498) for use in evaluating the potential for corrosion on concrete and buried metal such as utilities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

UNCONFINED COMPRESSIVE STRENGTH – D2166

Project Number: 824-2 **Boring #:** B-1 **Depth:** 2

Project Name: 139 Miles Lane **Date:** 10/11/2018

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	4.82 in
Volume	0.01283 ft^3
Water Content	13.2
Wet Density	94.7 pcf
Dry Density	83.7 pcf

Max Unconfined Compressive Strength	
Elapsed Time	2 min
Vertical Dial	0.1 in
Strain	2.1 %
Area	0.03262 ft^2
Axial Load	18.1 lbs
Compressive Strength	555 psf

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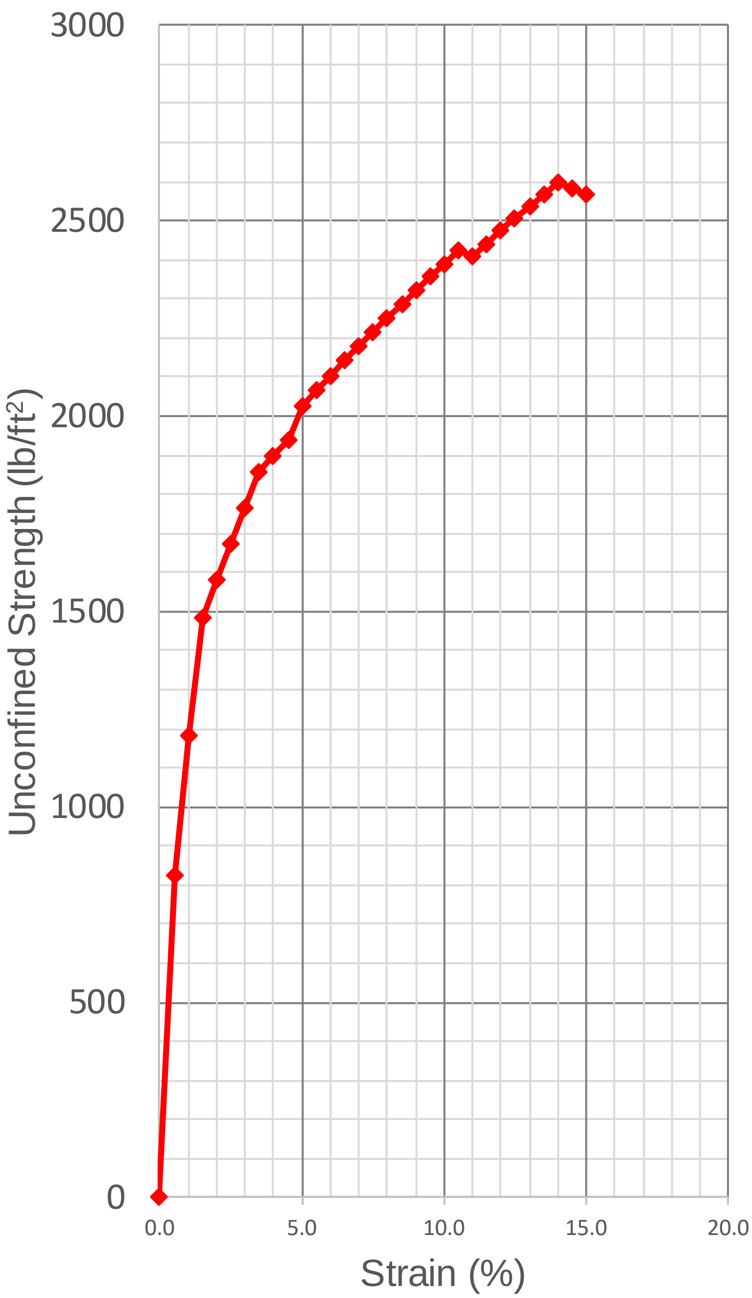
UNCONFINED COMPRESSIVE STRENGTH – D2166

Project Number: 824-2 **Boring #:** B-1 **Depth:** 6

Project Name: 139 Miles Lane **Date:** 10/11/2018

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



Soil Specimen Initial Measurements	
Diameter	2.42 in
Initial Area	4.60 in^2
Initial Length	5 in
Volume	0.01331 ft^3
Water Content	24.7
Wet Density	123.9 pcf
Dry Density	99.4 pcf

Max Unconfined Compressive Strength	
Elapsed Time	14 min
Vertical Dial	0.7 in
Strain	14.0 %
Area	0.03714 ft^2
Axial Load	96.5 lbs
Compressive Strength	2,598 psf

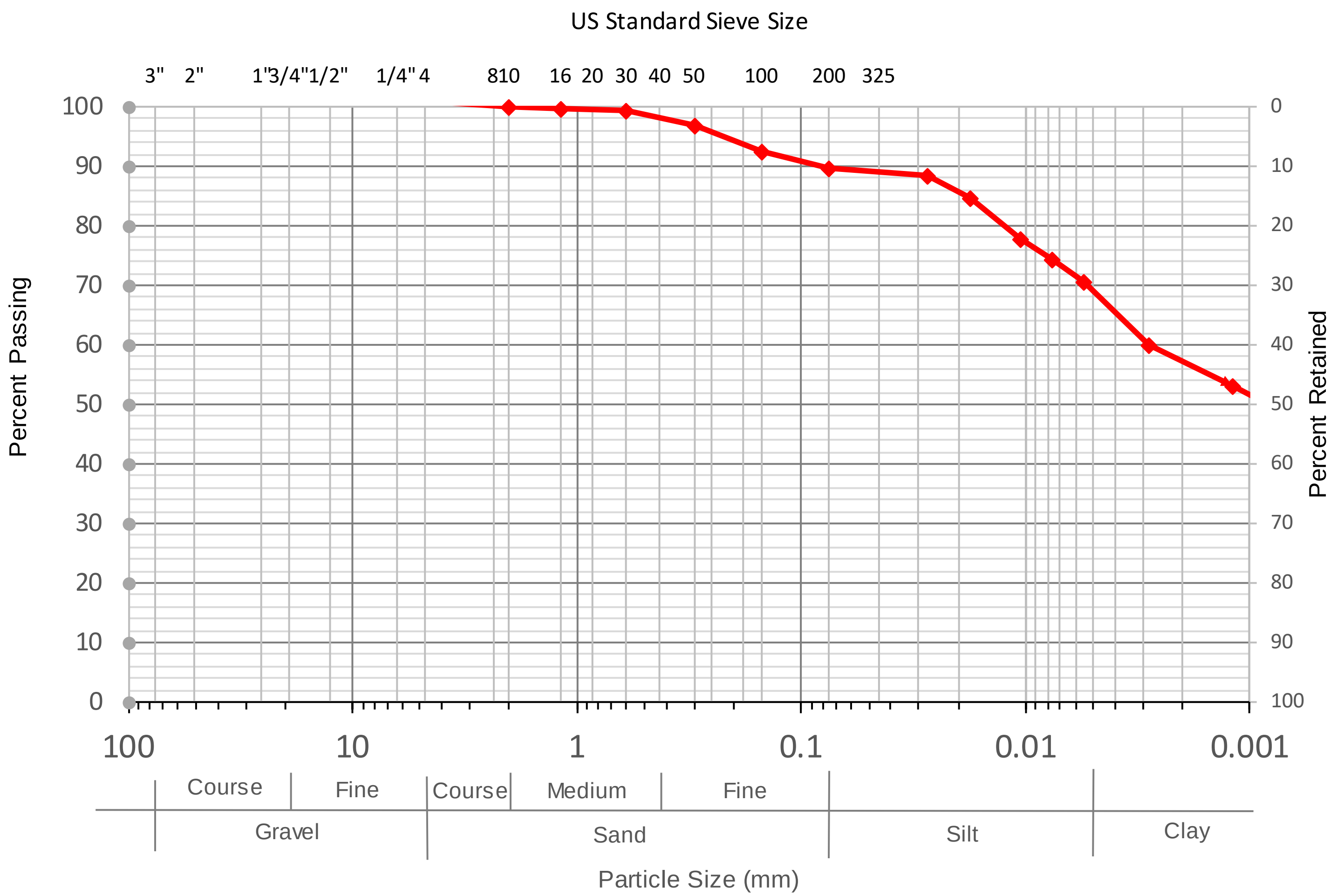
Stevens Ferrone & Bailey Engineering Company, Inc

Hydrometer Analysis – ASTM D422

Project Number: 824-2 **Project Name:** 139 Miles Lane

Sample Number: B-2 **Description:** Black silty CLAY some sand (CH)

Depth: 2.0 **Test Date:** 10-18-18 **Tested By:** R



Composite Sieve Data

Standard Sieve Size	Percent Passing
3”	
1.5”	
3/4”	
3/8”	
#4	
#10	100
#16	99.8
#30	99.5
#50	97.0
#100	92.6
#200	89.6

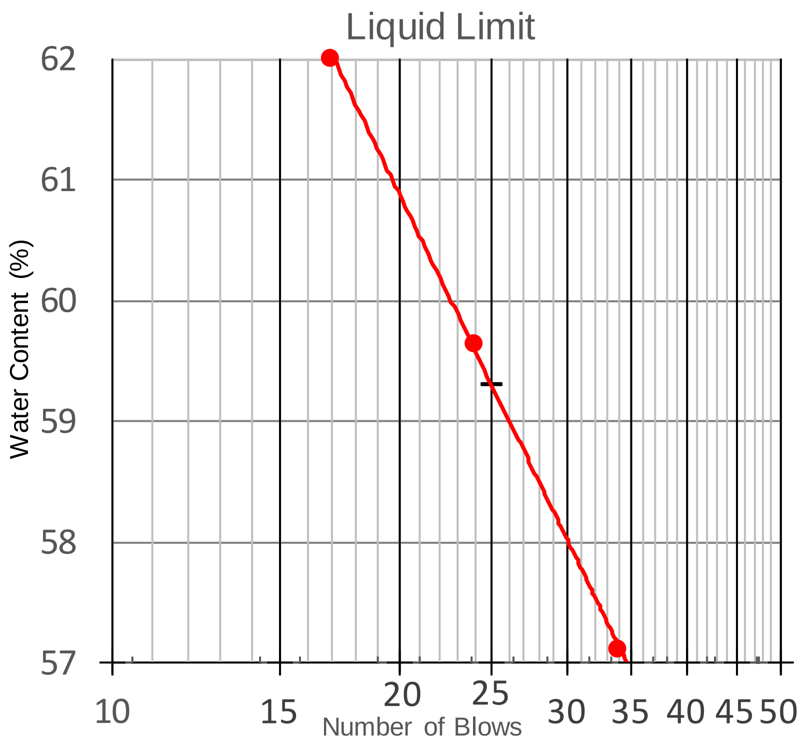
Particle Diameter (mm)	Percent Soil in Suspension
0.0275	88.3
0.0177	84.8
0.0106	77.7
0.0076	74.2
0.0055	70.7
0.0028	60.1
0.0013	53.0

Atterberg Limits Test – ASTM D4318

Project Number: 824-2 **Project Name:** 139 Miles Lane

Boring/Sample No: B-2 **Depth:** 2.0 **Date:** 10-17-18

Description of Sample: Black silty CLAY some sand (CH) **Tested By** R

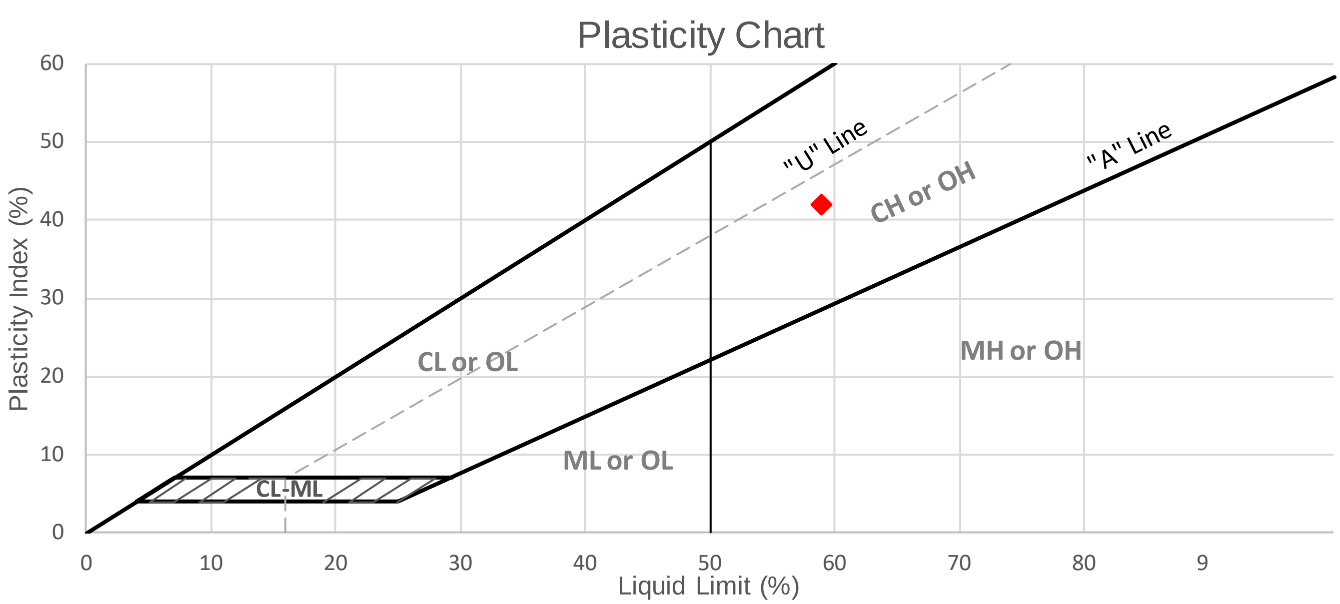


Plastic Limit Data

Trial	1	2	Ave
Water Content (%)	16.9	16.8	17

Data Summary

Liquid Limit	**59**
Plastic Limit	**17**
Plasticity Index	**42**
Natural Water Content	**18.2**
Liquidity Index	**0.029**
% Passing #200 Sieve	**89.6**



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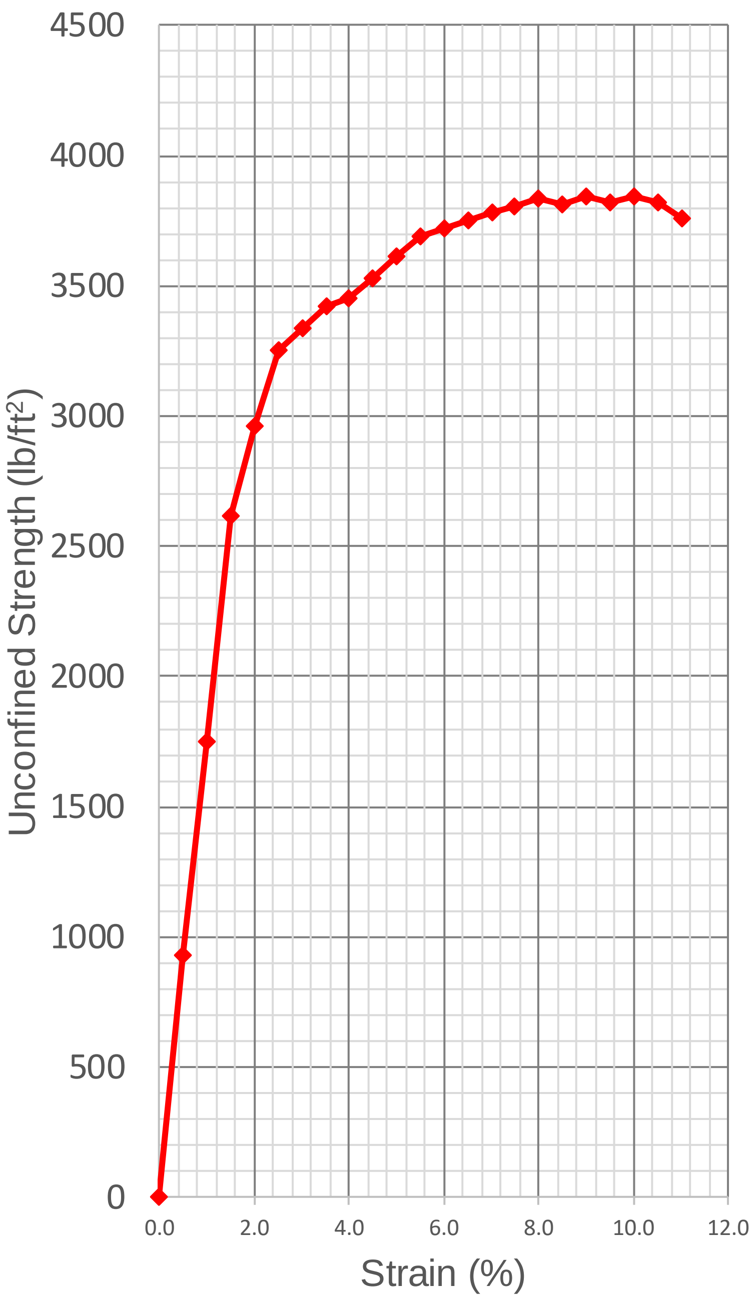
UNCONFINED COMPRESSIVE STRENGTH – D2166

Project Number: 824-2 **Boring #:** B-2 **Depth:** 5.5

Project Name: 139 Miles Lane **Date:** 10/11/2018

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



Soil Specimen Initial Measurements	
Diameter	2.42 in
Initial Area	4.60 in^2
Initial Length	5 in
Volume	0.01331 ft^3
Water Content	27.4
Wet Density	122.2 pcf
Dry Density	96.0 pcf

Max Unconfined Compressive Strength	
Elapsed Time	10 min
Vertical Dial	0.5 in
Strain	10.0 %
Area	0.03549 ft^2
Axial Load	136.5 lbs
Compressive Strength	3,846 psf

California State Certified Laboratory No. 2153

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

23 October, 2018

Job No.1810101
Cust. No.11486

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

Subject: Project No.: SFB 824-2
Project Name: 139 Miles Lane, Watsonville, CA
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Litzau:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on October 11, 2018. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, both samples are classified as "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 & 270 mg/kg and are determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentrations reflect none detected & 89 mg/kg and are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The pH of the soils are 7.20 & 7.24, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials are 330-mV & 390-mV and are indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions.

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

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

Very truly yours,
CERCO ANALYTICAL, INC.

Cheryl McMillen for

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

JDH/jdl
Enclosure

California State Certified Laboratory No. 2153

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www.cercoanalytical.com

Client:	Stevens, Ferrone & Bailey
Client's Project No.:	SFB 824-2
Client's Project Name:	139 Miles Lane, Watsonville, CA
Date Sampled:	9-Oct-18
Date Received:	11-Oct-18
Matrix:	Soil
Authorization:	Signed Chain of Custody

Date of Report: 23-Oct-2018

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)*
1810101-001	SFB-1 @ 3' (Bag shows @2.5')	390	7.20	-	1,600	-	N.D.	N.D.
1810101-002	SFB-2 @ 2.5' (Bag shows @ 3')	330	7.24	-	530	-	270	89

	Redox	pH	Conductivity	Resistivity	Sulfide	Chloride	Sulfate
Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	22-Oct-2018	22-Oct-2018	-	23-Oct-2018	-	22-Oct-2018	22-Oct-2018

* Results Reported on "As Received" Basis

N.D. - None Detected

Cheryl McMillen
Laboratory Director

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

APPENDIX C
ASFE Guidelines

Important Information about Your

Geotechnical Engineering Report

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

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

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely on Your ASFE-Member Geotechnical Engineer for Additional Assistance

Membership in ASFE/The Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

ASFE THE GEOPROFESSIONAL BUSINESS ASSOCIATION

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