Stevens Ferrone Bailey Engineering

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

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GEOTECHNICAL INVESTIGATION
831 WATER STREET
SANTA CRUZ, CALIFORNIA
SFB PROJECT NO. 940-1

Prepared For:

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1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed mixed-use, multi-level development to be located at 831 Water Street in Santa Cruz, 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 proposed development information provided by Novin Development Corporation, it is our understanding that the project will consist of constructing two buildings with associated facilities. The buildings will include an underground parking garage with utility rooms, a ground level of mixed-use, and three and four overlying levels of residences with roof access. The gross site area is approximately 0.9 acres. It is estimated that the bottom of the proposed garage excavations will be about 15 feet below surface grade. Other than excavating for the below-grade parking garage, nominal grading is anticipated. Associated underground utilities, pavements, and access driveways to the garage will be constructed. The existing buildings and improvements at the site will be demolished prior to new construction.

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

2.0 SCOPE OF WORK

Our investigation of the site included the following scope of work:

- Reviewing published and unpublished geotechnical and geological literature relevant to the site;
- Reviewing historical aerial images and topographic maps of the site and surrounding area;
- Performing a reconnaissance of the site and surrounding area;
- Performing a subsurface exploration program to log and sample two exploratory borings to a maximum depth of about 26-1/2 feet;
- Performing laboratory testing of samples retrieved from the borings;
- Performing engineering analysis of field and laboratory data; and
- Preparing this report.

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

3.0 SITE INVESTIGATION

Reconnaissance of the site and surrounding area was performed on May 18 and May 25, 2021. A subsurface exploration program was performed on May 25, 2021 using a truck-mounted drill rig equipped with 6-inch diameter, continuous flight, solid stem augers. Two exploratory borings were drilled to a maximum depth of about 26-1/2 feet below existing grade. The approximate locations of the borings are shown on the Site Plan, Figure 1. Logs of the borings and details regarding our field investigation are included in Appendix A. The results of our laboratory tests are discussed in Appendix B. It should be noted that changes in the surface and subsurface conditions can occur over time as a result of either natural processes or human activity and may affect the validity of the conclusions and recommendations in this report.

3.1 Site History and Surface Description

At the time of our investigation and as shown on Figure 1, the site was bounded by Water Street on the south, residential developments on the west and north, and North Branciforte Avenue on the east. The site was roughly rectangular in shape, relatively level, and had a plan area of about 0.9-acres. A retaining wall, with fencing and bollards, was present along the southern side of the site, with a short section of retaining wall along the western side of the site by the southwest corner. Fill had been placed behind the walls in order to create the relatively level site. Water Street, located at a lower elevation than the site, sloped downward toward the west. The retaining walls appeared to be as high as 10 feet in the southwest corner of the site.

At the time of our field exploration, the site was occupied by a strip mall and a car wash with vacuuming bays. Asphalt concrete pavement existed in the areas beyond the buildings. Along the perimeter of the site, concrete curb bounded landscaping, and trees and utility poles were observed.

Based on our review of historical aerial photographs and topographic maps of the site and vicinity, it is our understanding that the site was vacant with minor structures until the shopping center was constructed in about 1967. The photos indicate the car wash was added to the development in about 1982. Since 1982, not much changed a the site except for the occasional re-surfacing of the pavement.

3.2 Subsurface Description

Below the pavement sections (approximately 4 inches of asphalt concrete overlying 8 inches of baserock), our borings encountered loose to medium dense sands to depths of about 10 to 13 feet below existing grade. The sands had varying amounts of silt and nominal amounts of clay. Underlying the sands, our borings encountered friable, weathered siltstone. The Simco 2400 SK-1 drill rig was able to auger through the siltstone as much as about 14 feet.

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

3.3 Groundwater

Groundwater was encountered in both borings and appeared to be perched on top of the siltstone layer. The borings encountered groundwater at depths of about 8 to 11 feet at the time of drilling, and groundwater was measured at a depth of about 9 feet after completing the drilling. Our borings were backfilled with lean cement grout in accordance with Santa Cruz County requirements prior to leaving the site. It should be noted that the borings might not have been left open for a sufficient period of time to establish equilibrium groundwater conditions. In addition, fluctuations in the groundwater level can occur due to seasonal changes, including variations in rainfall, and other factors.

3.4 Hydrologic Soil Group

The surface soils at the site have been mapped as Elder sandy loam, Pinto loam, Soquel loam, and Watsonville loam by the USDA Web Soil Survey (WSS).[1] These soils were assigned to Hydrologic Soil Groups A, C, C, and D, respectively, by the USDA Natural Resources Conservation Service (NRCS) and have been categorized as having moderately low to high rates of water transmission (0.06 to 9.92 inches per hour).

3.5 Geology and Seismicity

According to Brabb, et al (1997)[2], the site (below pavement sections) is underlain by Pleistocene, well-sorted sand with a few thin, continuous layers of gravel. Underlying the sands, Brabb maps the bedrock as Pliocene and Upper Miocene Purisima Formation consisting of very thickly bedded tuffaceous and diatomaceous siltstone containing interbeds of bluish-gray, semi-friable, fine grained andesitic sandstone. Our borings encountered siltstone to the maximum depth explored of 26-1/2 feet.

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[1]USDA NRCS, https://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx, accessed 06/2/2021.
[2]Brabb, et al, 1997, *Geologic Map of Santa Cruz County, California: A Digital Database*, USGS Open-File Report 97-489.

According to U.S. Geological Survey Open-File Report 97-745 C and E (Summary Distribution of Slides and Earth Flows in Santa Cruz County), the site is mapped as flat land with little or no potential for landslides or earth flows and is not located within an area having debris flow source potential. In addition, Cooper-Clark & Associates (1975; Preliminary Map of Landslide Deposits in Santa Cruz County, California) do not map any landslide deposits at the site or in the vicinity of the site. It is our opinion that the potential for landslides, earth flows, or debris flows to develop at the site is very low, especially given the relatively level topography of the site and surrounding areas.

The project site is in the Santa Cruz area which is considered one of the most seismically active regions in the United States. Significant earthquakes have occurred in the Santa Cruz area and are associated with crustal movements along a system of sub-parallel fault zones that generally trend in a northwesterly direction. The site is not located within or adjacent an Alquist-Priolo Earthquake Fault Zone.[3] During our field investigation, we did not observe evidence of active earthquake faulting crossing the site or in the vicinity of the site. Based on the results of our review and subsurface exploration, it is our opinion that the potential for faulting to cause surface rupture at the site is very low.

Earthquake intensities will vary throughout the Santa Cruz area depending upon numerous factors, including the magnitude of earthquake, the distance of the site from the causative fault, and the type of materials underlying the site. The U.S. Geological Survey (2016)[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 (including the Santa Cruz area) between 2014 and 2043. Therefore, the site will be subjected to strong ground shaking as is common for developments throughout the area.

According to the U.S. Geological Survey's Unified Hazard Tool and applying the Dynamic: Conterminous U.S. 2014 model (v4.2.0, accessed 06/10/2021), the resulting deaggregation calculations indicate that the site has a 10% probability of exceeding a peak ground acceleration of about 0.45g in 50 years (design basis ground motion based on soft rock site condition; mean return time of 475 years). The actual ground surface acceleration might vary depending upon the local seismic characteristics of the underlying bedrock and overlying unconsolidated soils.

3.6 Liquefaction

Soil liquefaction is a phenomenon primarily associated with saturated, relatively 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

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[3]Bryant and Hart, *Fault-Rupture Hazard Zones in California*, CGS Special Publication 42, Interim Revision 2007.
[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).

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.

The site has not been mapped by State of California, ABAG, or the U.S. Geological Survey for liquefaction hazard potential. According to the Santa Cruz County GIS for Liquefaction Susceptibility (accessed June 2, 2021), the site is located in an area designated as having low liquefaction susceptibility.

It is our opinion that the potential for liquefaction causing damage to the proposed structure is low due to the proposed excavation of the onsite liquefiable sands during the basement construction, and the basement construction extending into the underlying siltstone bedrock.

4.0 CONCLUSIONS AND RECOMMENDATIONS

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

WEAK SURFACE SOILS AND EXISTING FILLS: The removal of the existing structures and improvements at the site will likely result in loosening and weakening of the surface soils in the upper 2 to 3 feet. Wherever the weakened soils and existing fill materials are not removed as part of the parking garage excavation, we recommend they be over-excavated. Where the over-excavation limits abut adjacent property, SFB should be consulted to determine the actual vertical and lateral extent of over-excavation so that adjacent property is not adversely impacted. The removed soil and 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.

PARKING GARAGE EXCAVATIONS: If temporary construction slopes are to be used at the perimeter of the parking garage excavation, we recommend the slopes be no steeper than 1-1/2 horizontal to 1 vertical. The temporary slope inclinations may need to be adjusted at the time of construction; we recommend SFB monitor the excavations in order to provide additional recommendations at the time of construction. The top of the slopes should be appropriately setback from existing improvements, such as adjacent streets and buildings. All temporary construction slopes and existing improvements should be monitored during the construction process and appropriate remedial measures should be immediately installed if detrimental movements are observed or measured. Where construction slopes cannot be used due to space constraints, temporary shoring should be installed. We recommend current OSHA standards be followed during the design and construction of any temporary construction slopes and/or shoring.

The base of the garage excavation may be wet and unstable. If necessary, the subgrade soils can be mixed with 4 to 5 percent cement/lime mix by weight and compacted to at least 90 percent relative compaction to aid in base stabilization prior to foundation construction. SFB should be consulted at the time of construction to confirm these recommendations. We also recommend the subgrade stabilization be designed and performed by a specialty contractor.

CORROSION POTENTIAL: Two onsite soil samples were tested for pH (ASTM D4972), chlorides (ASTM D4327), sulfates (ASTM D4327), sulfides (ASTM D4658M), resistivity at 100% saturation (ASTM G57), and Redox potential (ASTM D1498) for use in evaluating the potential for corrosion on concrete and buried metal, such as utilities and reinforcing steel. The results of these tests and a brief summary of the results are included in Appendix B. We recommend these test results and brief summary be forwarded to your underground contractors, pipeline designers, and foundation designers and contractors so they can design and install corrosion protection measures. Please be aware that we are not corrosion protection experts; we recommend corrosion protection measures be designed and constructed so that all concrete and metal, including foundation reinforcement, are protected against corrosion. We also recommend additional testing be performed if the test results are deemed insufficient by the designers and installers of the corrosion protection. Landscaping soils typically contain fertilizers and other chemicals that can be highly corrosive to metals and concrete; landscaping soils commonly are in contact with foundations. Consideration should be given to testing the corrosion potential characteristics of proposed landscaping soils and other types of imported or modified soils in order to design and provide protection against corrosion for the foundation and pipelines.

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

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

4.1 Earthwork

4.1.1 Clearing and Site Preparation

The site should be cleared of all obstructions including existing structures and their entire foundation systems, existing utilities and pipelines and their associated backfill, pavements and their underlying baserock, gravel, designated trees and landscaping and their associated root systems, and debris. Holes resulting from the removal of underground obstructions extending below the proposed finish grade should be cleared and backfilled with fill materials as specified in **Section 4.1.4, *Fill Material***, and compacted to the requirements in **Section 4.1.5, *Compaction***. Tree roots may extend to depths of about 3 to 4 feet.

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

4.1.2 Weak Soil and Existing Fill Removal

Wherever the loosened and weakened soils and existing fill materials are not removed as part of the parking garage excavations, we recommend they be over-excavated and recompacted as engineered fill. 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. We estimate that the removal of the existing structures and improvements at the site will likely result in loosening of the surface soils in the upper 2 to 3 feet. Removed soil materials may be used as new fill onsite provided it satisfies the recommendations provided in **Section 4.1.4, *Fill Material***. Compaction should be performed in accordance with the recommendations in **Section 4.1.5, *Compaction***.

4.1.3 Subgrade Preparation

After the completion of clearing, site preparation, excavation, and weak soil/fill removal, soils exposed in areas to receive improvements (such as new fill, building foundations, exterior flatwork, driveways, and pavements) should be scarified to a depth of about 12 inches, moisture conditioned to approximately 2 to 3 percent over optimum water content, and compacted to the requirements for structural fill.

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

4.1.4 Fill Material

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

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

4.1.5 Compaction

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

4.1.6 Utility Trench Backfill

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

Onsite trench backfill should be compacted to at least 90 percent relative compaction. Imported sand trench backfill should be compacted to at least 95 percent relative compaction and sufficient water is added during backfilling operations to prevent the soil from “bulking” during compaction. The upper 3 feet of trench backfill in foundation, slab, and pavement areas should be entirely compacted to at least 95 percent relative compaction. To reduce piping and settlement of overlying improvements, we recommend rock bedding and rock backfill (if used) be completely surrounded by a filter fabric such as Mirafi 140N (or equivalent); alternatively, filter fabric would not be necessary if Caltrans Class 2 permeable material is used in lieu of rock bedding and rock backfill.

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

Where utility trenches are located on slopes steeper than 10 horizontal to 1 vertical (10:1), or where the bottom of trenches are sloped steeper than 10:1, we recommend a low permeability plug composed of low strength concrete, sand/cement slurry, or onsite clays be installed in the utility trenches every 50 feet on-center. The plug will reduce piping from water seepage that may cause roadway and trench 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.

4.1.7 Exterior Flatwork

We recommend that exterior slabs (including patios, sidewalks, and driveways) be placed directly on the properly compacted fills. We do not recommend using aggregate base, gravel, or crushed rock below these improvements. If imported granular materials are placed below these elements, subsurface water can seep through the granular materials and cause the underlying soils to saturate, pipe, and/or heave upward. Prior to placing concrete, subgrade soils should be moisture conditioned to increase their moisture content to approximately 2 to 3 percent above laboratory optimum moisture (ASTM D-1557).

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

We recommend reinforcing exterior slabs with steel bars in lieu of wire mesh. To reduce potential crack formation, the installation of #4 bars spaced at approximately 24 inches on center in both directions should be installed. Score joints and expansion joints should be used to control cracking and allow for expansion and contraction of the concrete slab. We recommend appropriate flexible, relatively impermeable fillers be used at all cold/expansion joints. The installation of dowels at all expansion and cold joints will reduce differential slab movements; the dowels should be at least 30 inches long and should be spaced at a maximum lateral spacing of 124inches. Although exterior slabs that are adequately reinforced will still crack, trip hazards requiring replacement of the slabs will be reduced if the slab are properly reinforced.

4.1.8 Construction During Wet Weather Conditions

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

4.1.9 Surface Drainage, Irrigation, and Landscaping

Ponding and infiltration of water must not be allowed on or adjacent to pavements (including landscaping strips) and foundations. Ponding of water should also not be allowed on the ground surface adjacent to or near exterior slabs, including driveways, walkways, and patios. Surface grades should be sloped so that water sheet flows away from these improvements or sheet flows onto impermeable surfaces that directs the water into appropriate collection systems.

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

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

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

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

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

4.1.10 Storm Water Runoff Structures

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

We recommend storm water collection improvements that are designed to detain, retain, and/or treat water such as bio-swales, porous pavement structures, and water detention basins, be lined in order to reduce water seepage and the potential for damage and distress to other infrastructure improvements (such as pavements, foundations, and walkways) which can occur as a result of volumetric soil/fill changes (heaving and shrinking of the surrounding soil/fill). 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 or discharge location.

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

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

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

4.1.11 Future Maintenance

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

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

4.1.12 Additional Recommendations

We recommend that the drainage, irrigation, landscaping, and maintenance recommendations provided in this report be forwarded to your designers and contractors, and we recommend they be provided to the development owners and HOAs.

4.2 Foundation Support

4.2.1 General Recommendations

We recommend the planned below-grade parking garages be supported on a structural mat foundation. We recommend appropriate water-proofing be applied to the below-grade structure to reduce the potential for seepage and moisture migration through the basement walls and mat slab, and also to reduce efflorescence buildup. Keys should also be provided at the wall construction joints and infilled with a waterstop type product. SFB is not a waterproofing design expert; we recommend the appropriate waterproofing expert be consulted.

Uplift resistance can be provided by the weight of the structure and the skin friction between the garage wall faces and adjacent soils/backfill. An uplift skin friction resistance of 100 pound per square foot is considered applicable against the walls.

In order to generate full vertical and passive resistance, at least 10 feet of soil cover must be provided between the face of the foundations or walls and the face of slopes, as measured horizontally. The portion of the foundation or wall located closer than 10 feet from the face of slopes should be ignored in both the vertical and lateral load design. Where foundations are located adjacent to utility trenches, the foundation 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 foundation reinforcing could be increased to span the area defined above assuming no soil support is provided.

Approach slabs to the garage should be connected to garage slabs using dowels to reduce the potential for differential movements at the joint between the adjoining slabs. To control concrete shrinkage cracking, the garage slabs should have deep score joints that are spaced at approximately 10-feet on center in both directions.

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 foundations.

4.2.2 Structural Mat Slab

We recommend a structural mat slab foundation be used to support the building and the parking garage structures. The subgrade materials beneath the mat should be considered to have an effective Plasticity Index of 15 percent and a coefficient of subgrade reaction of a 1-foot by 1-foot plate of 150 ksf/ft. The thickness of the foundation slab should be determined by the Structural Engineer. We recommend appropriate water-proofing be applied to the below-grade structure. There would be no need to place crushed gravel or baserock below the mat slab. If needed, concrete rat slabs can be used below the vapor retarder to aid in construction.

The mat slab foundation should be designed for an allowable dead plus live load bearing pressure of 2,000 pounds per square foot (psf). Areas of the slab which support point or line loads that cannot be adequately resisted by the mat slab foundation should be thickened to a minimum of 12 inches and supported directly on the subgrade. An allowable bearing pressure of 4,000 psf can be used for localized areas of the slab that are supported directly on the subgrade. The slab should be designed in accordance with the 2019 California Building Code (CBC) requirements.

4.2.3 Concrete

We recommend a concrete mix design with low water/cement ratio, such as a 0.45, be used for interior slabs-on-grade and structural mat slabs. 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. We recommend all concrete and steel be protected against corrosion. The results of corrosion testing on onsite soil samples are included separately; the foundation designer should determine if additional testing is needed.

Concrete slabs retain moisture and often take many months to dry; construction water added during the concrete pour further increases the drying time. If the slabs are not allowed to completely dry prior to constructing the super-structure, the concrete slabs will expel water vapor into the super-structure and the vapor will be trapped under impermeable flooring. Slabs must not be poured during or immediately after rainstorms. Any free water trapped on the membrane must be removed prior to the concrete pour. To reduce the potential for differential curing, we recommend you consult with your concrete specialists.

4.2.4 Retaining/Basement Walls

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

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

Wherever portions of the garage walls will not be back-drained, the walls should be designed to resist an equivalent fluid pressure of 83 pounds per cubic foot plus a uniform pressure of 8H 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 2 degrees of slope inclination. Walls subjected to surcharge loads should be designed for an additional uniform lateral pressure equal to one-third and one-half the anticipated surcharge load for unrestrained and restrained walls, respectively.

For retaining walls that need to resist seismic lateral forces from the retained soils, we recommend the walls be designed to also resist a triangular pressure distribution equal to an equivalent fluid pressure of 24 pounds per cubic foot based on the ground acceleration from a design basis earthquake.[5,6] This seismic pressure is in addition to the pressures noted above. Due to the transient nature of the seismic loading, a factor of safety of at least 1.1 can be used in the design

[5]Seed and Whitman, 1970, *Design of Earth Retaining Structures for Dynamic Loads.*

[6]Atik and Sitar, 2007, *Development of Improved Procedures for Seismic Design of Buried and Partially Buried Structures*, Pacific Earthquake Engineering Research Center.

of the walls when they resist seismic lateral loads. Some movement of the walls may occur during moderate to strong earthquake shaking and may result in distress as is typical for all structures within the Santa Cruz area subjected to earthquake shaking.

Where back-drainage will be used behind retaining walls, the back-drainage system can consist of 1/2 to 3/4 inch crushed, uniformly graded gravel entirely wrapped in filter fabric such as Mirafi 140N or equal (an overlap of at least 12 inches should be provided at all fabric joints). The gravel and fabric should be at least 8 inches wide and extend from the base of the wall to within 12 inches of the finished grade at the top (Caltrans Class 2 permeable material (Section 68) may be used in lieu of gravel and filter fabric). A 4-inch diameter, perforated pipe should be installed at the base and centered within the gravel. The perforated pipe should be connected to a solid collector pipe that transmits the water directly to a storm drain, sump pump, drainage inlet, or onto pavement. If weep holes are used in the wall, the perforated pipe within the gravel is not necessary provided the weep holes are kept free of animals and debris, are located no higher than approximately 6 inches from the lowest adjacent grade, and are able to function properly. As an alternative to using gravel, drainage panels (such as AWD SITEDRAIN Sheet 94 for walls or equal) may be used behind the walls in conjunction with perforated pipe (connected to solid collector pipe), weep holes, or strip drains (such as SITEDRAIN Strip 6000 or equal). If used, the drainage panels can be spaced on-center at approximately 2 times the panel width. All wall subdrains should be connected to a solid pipe that discharges to an appropriate drainage facility.

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

4.2.5 Lateral Load Resistance

For the slab foundation, lateral loads, such as derived from earthquakes and wind, can be resisted by friction between the slab foundation bottom and the supporting subgrade. A friction coefficient of 0.25 is considered applicable.

The basement walls, if appropriately reinforced to withstand the passive pressures, may be used to resist lateral loads, including those imposed by earthquakes. Approximately 0.02H (where H is the height of wall) of wall deflection will need to occur before generating the full passive resistance against the retained soils. A passive resistance equal to an equivalent fluid weighing 350 pounds per cubic foot acting against the vertical face of the walls can be used. The portion of the basement wall within 2 feet of the exterior finished grade should be ignored in the passive resistance calculation.

4.2.6 Seismic Design Criteria

The following parameters were calculated using the U.S. Seismic Design Map program,[7] and are based on the site being located at approximate latitude 36.98181°N and longitude 122.0144°W. For seismic design using the 2019 California Building Code (CBC), we recommend the following seismic design parameters be used. These values are based on applying the ASCE 7-16 model, assuming the structure is categorized as Risk Category II, and assuming that *Exception Number (2) of ASCE 7-16 Section 11.4.8 – Site Specific Ground Procedure* applies. We should be contacted if any of these assumptions are incorrect or a site-specific ground motion hazard analysis is required.

SEISMIC PARAMETER	DESIGN VALUE
Site Class	C
S_S	1.678
S_1	0.644
S_{MS}	2.013
S_{M1}	0.902
S_{DS}	1.342
S_{D1}	0.601
SDC	D
F_a	1.2
F_v	1.4
PGA_M	0.844

4.3 Pavements

4.3.1 Asphalt Concrete

Based on the results of laboratory testing of onsite materials and our borings, we recommend that an R-value of 30 be used in asphalt concrete pavement design. We developed the following alternative preliminary pavement sections using Topic 608 of the State of California Department of Transportation Highway Design Manual, the recommended R-value, and typical traffic indices for multi-use developments. The pavement thicknesses shown below are SFB's recommended minimum values; governing agencies may require pavement thicknesses greater than those shown.

[7] SEAONC/OSHPD, https://seismicmaps.org/, accessed 06/10/2021.

PRELIMINARY PAVEMENT DESIGN ALTERNATIVES SUBGRADE R-VALUE = 30			
Location	Pavement Components		Total Thickness (inches)
	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	
T.I. = 4.5 (auto & light truck parking)	3.0	6.0	9.0
T.I. = 5.0 (access ways)	3.0	7.0	10.0

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

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

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

4.3.2 Concrete Slab for Trash Enclosures

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 75 pounds per square inch per inch was assigned to represent a reworked, onsite subgrade underlain 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 CONDITIONS AND LIMITATIONS

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

We recommend SFB be retained to provide geotechnical services during design, reviews, earthwork and paving operations, and foundation installation to confirm and observe compliance with the design concepts, specifications and recommendations presented in this report. Our presence will also allow us to modify design if unanticipated subsurface conditions are encountered or if changes to the scope of the project, as defined in this report, are made.

This report is a design document that has been prepared in accordance with generally accepted geological and geotechnical engineering practices for the exclusive use of Novin Development Corporation and their consultants for specific application to the proposed new mixed-use buildings to be located at 831 Water Street, Santa Cruz, California, and is intended to represent our design recommendations to Novin Development Corporation for specific application to the 831 Water Street project. The conclusions and recommendations contained in this report are solely professional opinions. It is the responsibility of Novin Development 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 pre-construction conferences to clarify the opinions, conclusions, and recommendations contained in this report.

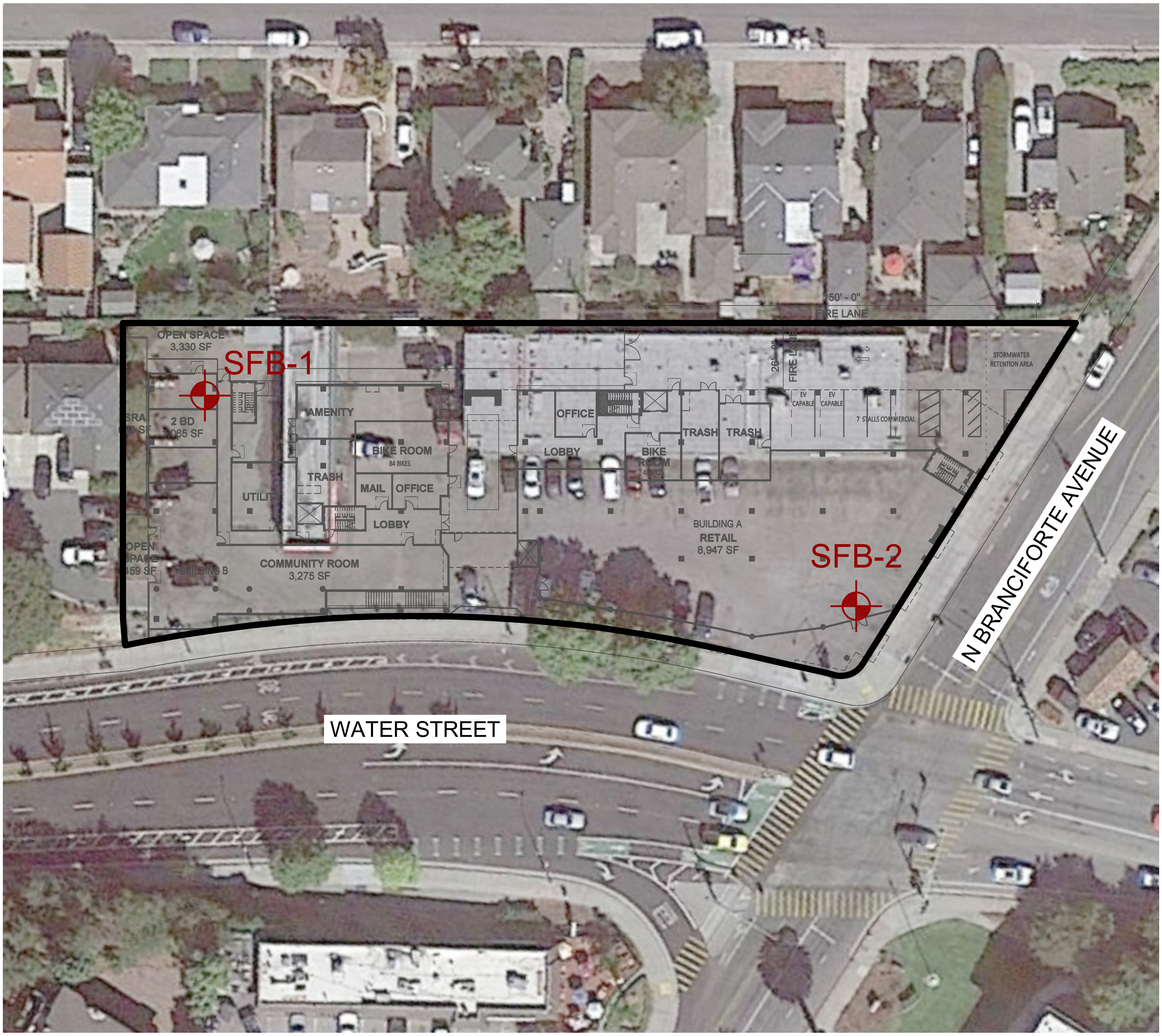
It should be understood that advancements in the practice of geotechnical engineering and engineering geology, or discovery of differing surface or subsurface conditions, may affect the validity of this report and are not uncommon. SFB strives to perform its services in a proper and professional manner with reasonable care and competence but we are not infallible. Geological engineering and geotechnical engineering are disciplines that are far less exact than other engineering disciplines; therefore, we should be consulted if it is not completely understood what the limitations to using this report are.

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

This report does not necessarily represent all of the information that has been communicated by us to Novin Development Corporation and their consultants during the course of this engagement and our rendering of professional services to Novin Development 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 Novin Development Corporation to divulge information that may have been communicated to Novin Development 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

ALL LOCATIONS ARE APPROXIMATE

SFB-2 — SFB EXPLORATORY BORING (05/25/2021)

PROJECT LIMIT

N

APPROXIMATE SCALE: 1" = 60'

0 60' 120'

BASE: Project Conceptual Design Set, Ground Floor Plan, prepared by HMH Engineers and dated 08/05/2020 overlayed on a Google Earth image dated 09/26/2020.

DATE	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	FIGURE
June 2021			831 WATER STREET Santa Cruz, California	1
PROJECT NO.				
940-1				

APPENDIX A
Field Investigation

APPENDIX A

Field Investigation

Our field investigation for the proposed new mixed-use building to be located at 831 Water Street in Santa Cruz, California, consisted of surface reconnaissance and a subsurface exploration program. Reconnaissance of the site and surrounding area was performed on May 18 and May 25, 2021. Subsurface exploration was performed using a truck-mounted drill rig equipped with 6-inch diameter, continuous flight, solid stem augers. Two exploratory borings were drilled on May 25, 2021 to a maximum depth of about 26-1/2 feet below existing grade. Our representative continuously logged the soils encountered in the borings during our field investigation. The soils are described in general accordance with the Unified Soil Classification System (ASTM D2487). The logs of the borings, as well as, a key for the classification of the soil (Figure A-1) and bedrock (Figure A-2) are included as part of this appendix.

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

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

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

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KEY TO EXPLORATORY BORING LOGS

PROJECT:

831 WATER STREET
Santa Cruz, California

PROJECT NO: **940-1**

FIGURE NO: **A-1**

UNIFIED SOIL CLASSIFICATION SYSTEM

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

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

GRAIN SIZES

	U.S. STANDARD SERIES SIEVE				CLEAR SQUARE SIEVE OPENINGS			
	#200	#40	#10	#4	3/4"	3"	12"	
SILTS AND CLAYS	SANDS			GRAVELS		COBBLES	BOULDERS	
	Fine	Medium	Coarse	Fine	Coarse			

RELATIVE DENSITY

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

CONSISTENCY

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

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

**Unconfined Compressive Strength.

SYMBOLS AND NOTES

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

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

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

Shelby Tube

Pitcher Barrel

HQ Core

Groundwater Level During Drilling

Groundwater Level at End of Drilling

INCREASING VISUAL MOISTURE CONTENT

Saturated
Wet
Moist
Damp
Dry

CONSTITUENT PERCENTAGE

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

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KEY TO ROCK CHARACTERISTICS

PROJECT: **831 WATER STREET**
Santa Cruz, California

PROJECT NO: **940-1**

FIGURE NO: **A-2**

STRENGTH/CONSISTENCY

PLASTIC - Very low strength; soil-like strength.

FRIABLE - Easily broken down with hand or finger pressure.

WEAK - Crumbles under light hammer blows.

MODERATELY STRONG- Withstands a few heavy hammer blows before breaking.

STRONG - Withstands a few heavy hammer blows while only yielding large fragments.

VERY STRONG - Resists breakage from heavy hammer blows while only yielding small chips and dust.

WEATHERING

COMPLETE - Rock reduced to soil-like structure; rock fabric not apparent or only in localized areas.

DEEP - Rock fabric apparent but extensive disintegration; moderate to complete mineral decomposition; fractures coated and/or filled with oxides, carbonates, and/or soil.

MODERATE - Rock fabric apparent; partial disintegration of minerals; all rock discolored or stained; moderately coated fractures.

SLIGHT - Rock generally fresh but fractures stained and may be slightly coated.

FRESH - No disintegration or discoloration; joints may show slight staining.

HARDNESS

SOFT - Easily scratched with fingernail; soil-like consistency.

LOW - Easily gouged or carved with knife blade.

MODERATE - Readily scratched with knife; scratch is readily visible and leaves heavy trace of dust.

HARD - Scratched with knife with difficulty; scatch is faintly visible and leaves little dust.

VERY HARD - Cannot be scratched with knife and leaves metallic streak.

FRACTURE SPACING

CRUSHED - Less than 1/4 inch

INTENSELY - 1/4 inch to 1 inch

CLOSELY - 1 inch to 6 inches

MODERATELY- 6 inches to 1 foot

OCCASIONALLY - 1 foot to 4 feet

LITTLE - Greater than 4 feet

BEDDING SPACING

THINLY LAMINATED- Less than 1/10 inch

LAMINATED - 1/10 inch to 1/2 inch

VERY THIN - 1/2 inch to 1 inch

THIN - 1 inch to 6 inches

THICK - 6 inches to 2 feet

VERY THICK - Greater than 2 feet

MASSIVE - No apparent bedding spacing

ROUGHNESS OF SURFACE

SMOOTH - Appears smooth and surface feels smooth to the touch; may be slickensided.

SLIGHTLY ROUGH - Asperities on surface are clearly visible.

MEDIUM ROUGH - Asperities are clearly visible and surface feels abrasive.

ROUGH - Large angular asperities are visible; some ridge and high side angle steps are evident.

VERY ROUGH - Near vertical steps and ridges are visible on surface.

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EXPLORATORY BORING SFB-1

PROJECT NO: 940-1	SURFACE ELEVATION: --
LOGGED BY: HP	DATE STARTED: 05/25/21
DRILL RIG: Simco 2400 SK-1	DATE FINISHED: 05/25/21
DRILLING METHOD: 6-inch Solid Stem Auger	DEPTH TO INITIAL WATER: 11 feet
HAMMER METHOD: Rope and Cathead	DEPTH TO FINAL WATER: 9 feet
HAMMER WEIGHT / DROP: 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 1	

PROJECT:
831 WATER STREET
Santa Cruz, California

DESCRIPTION AND CLASSIFICATION

DESCRIPTION AND REMARKS	CONSIST	GRAPHIC LOG	DEPTH (FEET) ELEVATION	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS AND NOTES
Asphalt Concrete (AC) about 4 inches thick.			0						
Aggregate Base (AB) about 8 inches thick.									
SAND (SC), dark grayish-brown, fine-grained, with clay, some silt, damp.	loose				4	11.6	106.3		At 1.5 feet: Percent Passing #200 Sieve = 36%
SAND (SC), olive-brown with yellowish-brown mottling, fine- to medium-grained, some clay, trace silt, damp to moist.	loose				7				At 3 feet: Percent Passing #200 Sieve = 46%
SAND (SM), olive-brown, fine- to medium-grained, trace gravel (fine, subangular to subrounded), some silt, trace clay, damp to moist.	medium dense		5		16	15.1	102.2		At 6 feet: Percent Passing #200 Sieve = 8%
SILTSTONE, greenish-gray, with clay, some sand (fine-grained), completely weathered, dry to damp.	friable		10		40				
Deeply weathered, dry.			15		50/3"				
			20		55/5.5"				
Bottom of Boring = 20.95 feet Notes: Stratification is approximate, variations must be expected. Blow counts converted to SPT N-values. See report for additional details.			25						
			30						

EXPLORATORY BORING LOG 940-1 SFB-1.ldat8 STEVENS FERRONE & BAILEY 6/4/2021

Stevens Ferrone Bailey Engineering

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Tel: (925) 688-1001

EXPLORATORY BORING SFB-2

PROJECT:

831 WATER STREET
Santa Cruz, California

PROJECT NO: 940-1	SURFACE ELEVATION: --
LOGGED BY: HP	DATE STARTED: 05/25/21
DRILL RIG: Simco 2400 SK-1	DATE FINISHED: 05/25/21
DRILLING METHOD: 6-inch Solid Stem Auger	DEPTH TO INITIAL WATER: 8 feet
HAMMER METHOD: Rope and Cathead	DEPTH TO FINAL WATER: 9 feet
HAMMER WEIGHT / DROP: 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 1	

DESCRIPTION AND CLASSIFICATION: DESCRIPTION AND REMARKS	CONSIST	GRAPHIC LOG	DEPTH (FEET) ELEVATION	SAMPLER	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS AND NOTES
Asphalt Concrete (AC) about 4 inches thick.			0						
Aggregate Base (AB) about 8 inches thick.									
SAND (SC), dark grayish-brown, fine-grained, with clay, some silt, damp.	medium dense				13	9.3	126.6		
SAND (SC), olive-brown with yellowish-brown mottling, fine- to medium-grained, some clay, trace silt, damp to moist.	medium dense				12				At 3 feet: Percent Passing #200 Sieve = 32%
SAND (SC), olive-brown, fine- to medium-grained, trace gravel (fine, subangular to subrounded), with clay, some silt, moist.	loose		5		8	18.4	95.7		At 5.5 feet: Percent Passing #200 Sieve = 23%
SAND (SM), mottled olive, silty, fine-grained, some clay, moist to wet.	loose		10		9				At 11 feet: Percent Passing #200 Sieve = 43%
SILTSTONE, olive with iron staining, with clay, some sand (fine-grained), completely weathered, damp to moist.	friable		15		12				
Change color to greenish-gray.			20		43				
			25		44				
Bottom of Boring = 26.5 feet Notes: Stratification is approximate, variations must be expected. Blow counts converted to SPT N-values. See report for additional details.			30						

APPENDIX B

Laboratory Investigation

APPENDIX B
Laboratory Investigation

Our laboratory testing program for the proposed new mixed-use building to be located at 831 Water Street in Santa Cruz, 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 four samples of the subsurface soils. The water contents are recorded on the boring logs at the appropriate sample depths.

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

The passing the #200 sieve analysis was performed on six samples retrieved from the borings in order to assess grain sizes. The results of these tests are shown on the borings logs at the appropriate sample depths.

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

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

10 June, 2021

Job No. 2105190
Cust. No. 11486

Ms. Hayley Palilla
Stevens, Ferrone & Bailey
1600 Willow Pass Court
Concord, CA 94520

Subject: Project No.: SFB 940-1
Project Name: 831 Water St., Santa Cruz, CA
Corrosivity Analysis – ASTM Test Methods

Dear Ms. Palilla:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on May 26, 2021. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

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

The chloride ion concentrations are none detected with a detection limit of 15 mg/kg.

The sulfate ion concentrations range from 29 to 44 mg/kg and are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

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

The pH of the soils ranged from 7.26 to 7.54 which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials ranged from 300 to 370-mV which is 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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Client:	Stevens, Ferrone & Bailey
Client's Project No.:	SFB 940-1
Client's Project Name:	831 Water St., Santa Cruz, CA
Date Sampled:	25-May-21
Date Received:	26-May-21
Matrix:	Soil
Authorization:	Signed Chain of Custody

Date of Report: 10-Jun-2021

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
2105190-001	SFB-1 @ 1'	300	7.26	-	4,200	N.D.	N.D.	44
2105190-002	SFB-2 @ 3'	370	7.54	-	6,100	N.D.	N.D.	29

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	8-Jun-2021	7-Jun-2021	-	9-Jun-2021	3-Jun-2021	7-Jun-2021	7-Jun-2021

Cheryl McMillen
Laboratory Director

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

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

APPENDIX C
Report Guidelines

Important Information about This

Geotechnical-Engineering Report

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

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

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

Understand the Geotechnical-Engineering Services Provided for this Report

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

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

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

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

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

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

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

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

Read this Report in Full

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

You Need to Inform Your Geotechnical Engineer About Change

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

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

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

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

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

This Report's Recommendations Are Confirmation-Dependent

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

This Report Could Be Misinterpreted

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

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

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

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for information purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are <u>Not</u> Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

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

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