HERZOG
GEOTECHNICAL
CONSULTING ENGINEERS

July 11, 2022
Project Number 4167-01-22

Andrew McCune
49 Sunnyside Avenue
San Anselmo, California 94960

RE: Report
Geotechnical Investigation
49 Sunnyside Avenue
San Anselmo, California

Dear Mr. McCune:

This presents the results of our geotechnical investigation for the proposed renovations at 49 Sunnyside Avenue in San Anselmo, California. The scope of our investigation was to review selected geologic references, observe exposed site conditions, drill two test borings in the project area, conduct engineering analyses, and develop geotechnical recommendations for the design and construction of the project. Our scope of work was outlined in our professional services agreement dated June 10, 2022.

PROJECT DESCRIPTION

We understand that the project will consist of renovating the residence and excavating living space beneath the house. Retained cuts are anticipated to range to about 10 feet high. The project is shown on the plans by Andrew McCune Architect dated March 16, 2022.

WORK PERFORMED

We reviewed selected geologic references prior to performing our investigation. We explored the subsurface conditions in the project area on June 30, 2022 to the extent of two test borings approximately 9-1/4 and 11 feet deep and extending into bedrock. Due to limited access, the test borings were drilled with portable drilling equipment. The approximate locations of our test borings are shown on the attached *Site Plan*, Plate 1.

Our personnel observed the drilling, logged the subsurface conditions encountered, and collected soil samples for visual examination and laboratory testing. Samples were retrieved using Sprague and Henwood and Standard Penetration Test samplers driven with a 70-pound hammer. Penetration resistance blow counts were obtained by dropping the hammer through a 30-inch free fall. The number of blows was recorded for each 6 inches of sampler penetration. These

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blow counts were then correlated to equivalent standard penetration blow counts. The blows per foot recorded on the boring logs represent the accumulated number of correlated standard penetration blows that were required to drive the sampler the last 12 inches or fraction thereof.

Logs of the test borings are presented on Plates 2 and 3. The soils encountered are described in accordance with the criteria presented on Plate 4. Bedrock is described in accordance with the *Engineering Geology Rock Terms* presented on Plate 5. The logs depict our interpretation of subsurface conditions on the date and at the depths indicated. The stratification lines on the logs represent the approximate boundaries between soil types; the actual transitions may be gradational.

Selected samples were laboratory tested to determine their moisture content and dry density. Laboratory test results are posted on the boring logs in the manner described on the *Key to Test Data*, Plate 4.

FINDINGS

Site Conditions

The site is located on the western side of Sunnyside Avenue, opposite the intersection with Ross Avenue in San Anselmo, California. The site is situated at the base of a hillside which extends variably up towards the west. The residence is a single-story, wood-framed structure above a slab-on-grade garage and storage level. Cuts for the garage and storage level are supported by a few foot high foundation wall that steps up to a gently sloping crawl space. The house appears to be supported on spread footing foundations. The perimeter foundations of the house have experienced differential movement and localized severe cracking. Roof downspouts for the house discharge onto the ground surface adjacent to the structure.

The upslope (west) side of the house is bounded by planters and a concrete patio. A few foot high brick wall along the upslope side of the patio steps up to a grass covered yard that slopes gently up towards the west. The north and south sides of the house are bounded by planter areas and walkways. The east side of the house is bounded by a planter area and a severely cracked asphalt paved driveway which slopes gently down to Sunnyside Avenue. A few foot high masonry block retaining wall steps down to the sidewalk adjacent to the driveway. This wall has yielded and cracked.

Subsurface Conditions

The site is within the Coast Range Geomorphic Province which includes San Francisco Bay and the northwest-trending mountains that parallel the coast of California. These features were formed by tectonic forces, resulting in extensive folding and faulting of the area. Previous geologic mapping by Rice (1976) indicates the site vicinity to be underlain by sandstone and shale of the Franciscan Assemblage. The mapping indicates the bedrock in the site vicinity to be blanketed by Quaternary aged colluvial soils which have been deposited by slopewash processes.

Our test borings encountered fill and colluvial soils overlying bedrock. The fill encountered generally consists of loose silty sand. The colluvial soils encountered consist of medium dense clayey sand and medium stiff to stiff sandy clay which washed down from upslope areas. The fills encountered in our borings are weak and compressible, and the colluvium is generally moderately compressible. The soils encountered are of low expansion potential. Bedrock encountered in the borings generally consists of firm to moderately hard sandstone and shale.

The approximate test boring locations are shown on the *Site Plan* (Plate 1). The test borings encountered the following profiles:

	Depth (feet)		
Boring	**Fill**	**Residual Soil**	**Bedrock**
B-1	0-1.5	1.5-10.5	10.5-11.0+
B-2	0-2.0	2.0-8.5	8.5-9.3+

Descriptions of the subsurface conditions encountered are presented on the boring logs.

Groundwater

Free groundwater did not develop in the borings prior to backfilling. Groundwater levels at the site are expected to fluctuate over time due to variations in rainfall, surface drainage conditions and other factors. Rainwater percolates through the relatively porous surface soils. On hillsides, the water typically migrates downslope in the form of seepage within the porous soils, at the interface of the soil/bedrock contact, and within the upper portions of the weathered and fractured bedrock.

GEOLOGIC AND SEISMIC HAZARDS

Landsliding

Regional mapping by Rice (1976) and Wentworth and Frizzell (1975) does not indicate the presence of landsliding within the project area, and maps of slope failures resulting from the severe 1982 storms (Davenport, 1984) and of slope failures resulting from the heavy 1997/1998 storms (USGS, 1999) do not indicate that sliding was reported at the site at either of those times.

The Rice mapping indicates that the site lies immediately east of a boundary separating Slope Stability Zone 3 to the west from Zone 1 to the east. Zone 3 includes areas where the steepness of slopes approach the stability limits of the underlying geologic materials. Zone 1 includes areas underlain by relatively shallow bedrock and areas that occupy stable positions. The zones range from 1 to 4, with Zone 4 being least stable.

We did not observe evidence of landsliding at the site during our investigation, and did not encounter slide debris in our test borings. As such, we judge that the risk of landsliding at the site is low.

Fault Rupture

The property is not within a current Alquist-Priolo Earthquake Fault Zone (EFZ), and we did not observe geomorphic features that would suggest the presence of active faulting at the site. As such, we judge that the risk of ground rupture along a fault trace is low at this site.

Ground Shaking

The San Francisco Bay Region has experienced several historic earthquakes from the San Andreas and associated active faults. Mapped active faults (those experiencing surface rupture within the past 11,000 years) nearest the site are summarized in the following table.

Fault	Distance		Moment Magnitude[1]	Acceleration (g)[2]	
	Miles	Kilometers		M[3]	M+1 [3]
San Andreas (Northern)	7.2	11.6	8.0	0.35	0.63
Seal Cove/San Gregorio	7.7	12.4	7.4	0.29	0.52
Hayward	10.6	17.0	7.3	0.23	0.40
Healdsburg/Rodgers Creek	15.2	24.5	7.3	0.17	0.31

(1) Estimated maximum magnitudes from Caltrans Fault Database (Version 2A).
(2) Peak ground acceleration averaged from New Generation Attenuation (NGA-West 2) relationships by Abrahamson, Silva & Kamai (2104), Boore, Stewart, Seyhan & Atkinson (2014), Campbell &Bozorgnia and (2014), Chiou & Youngs (2014). Estimated shear wave velocity (V_{S30}) = 525 m/s.
(3) M = mean value; M+1 = mean+1 standard deviation value.

Deterministic information generated for the site considering the proximity of active faults and estimated ground accelerations are presented in the table above. The estimated ground accelerations were derived from the above-referenced mean attenuation relationships, and are based on the published estimated maximum earthquake moment magnitudes for each fault, the shortest distance between the site and the respective fault, the type of faulting, and the estimated shear wave velocities of the on-site geologic materials. The deterministic evaluation of the potential for ground shaking assumes that the anticipated maximum magnitude earthquake produces fault rupture at the closest proximity to the site, and does not take recurrence intervals or other probabilistic effects into consideration. This evaluation also does not consider directivity effects, topographic amplification, or other phenomena which may act to amplify ground motions.

Data presented by the U.S. Geological Survey (2016) estimates the chance of one or more large earthquakes (Magnitude 6.7 or greater) in the San Francisco Bay region before the year 2043 to be 72 percent. Consequently, we judge that the site will likely be subject to strong earthquake shaking during the life of the improvements.

Liquefaction/Densification

During ground shaking from earthquakes, liquefaction can occur in saturated, loose, cohesionless sands. The occurrence of this phenomenon is dependent on many factors, including the intensity and duration of ground shaking, soil density, particle size distribution, and position of the ground water table (Idriss and Boulanger, 2008). The soils encountered in our test boring contained a high percentage of fine grained materials (silt and clay). Thus, we judge that the likelihood of liquefaction during ground shaking is low.

Densification can occur in low density, uniformly-graded sandy soils above the groundwater table. We judge that significant densification is unlikely to occur in the areas explored because of the high silt and clay content of the soils encountered in the test boring.

CONCLUSIONS

Our investigation indicates that the project site is blanketed by relatively weak and compressible fills and native soils which are subject to differential settlement due to foundation loading. Mitigating the risk of differential settlement will necessitate extending foundation support into underlying bedrock with either drilled, cast-in-place, reinforced concrete piers or helical piers. We estimate that differential settlements of drilled or helical pier foundations designed in accordance with the recommendations contained in this report will be on the order of half an inch. To avoid damaging differential settlement, interior slabs should be structural slabs designed to span between pier-supported foundations. Non-underpinned foundations and slabs would be subject to settlement relative to pier supported foundations. It will therefore be necessary to extend pier support as necessary to extend pier support to encompass all foundations and slabs in order to avoid differential movement.

Alternatively, if the risk of on the order of a few inches of differential settlement is considered acceptable to the owner and structural engineer, foundation support may be derived from a stiffened mat foundation. It will be necessary to recompact the upper soils and to overexcavate and recompact existing fills beneath the mat to provide more uniform support. If unacceptable future settlement occurs, the mat foundation may be re-leveled by mud-jacking. It would be prudent to design the mat to be capable of resisting corresponding uplift stresses in the event that mud-jacking is required. It will be necessary to extend the stiffened mat system beneath the entire structure to avoid abrupt differential movement.

Excavations will expose weak soils that are subject to caving. It will therefore be necessary to maintain vertical support for the structure and to shore excavations in order to maintain lateral support for adjacent areas and to provide safe working conditions. Shoring should be designed to resist lateral earth pressures as well as surcharge loads using the design criteria presented in this report. Underpinning, shoring and the stability of excavations and existing structures should be contractually established as solely the responsibility of the Contractor and is excluded from our scope of work.

It is important that surface and subsurface water be controlled to reduce moisture variations in the weak on-site soils. Perimeter subdrains should be provided to reduce water infiltration beneath the structure, and retaining walls should be provided with adequate backdrainage to prevent hydrostatic buildup. All drains and downspouts should be collected in new closed conduits and discharged at an approved storm drain or at approved erosion resistant outlets well away from improvements.

RECOMMENDATIONS

Seismic Design

Based on the results of our investigation, the following seismic design criteria were developed in accordance with the *2019 California Building Code* and *ASCE 7-16*:

Site Class	C
Site Coefficient F_a	1.2
Site Coefficient F_v	1.4
0.2 sec Spectral Acceleration S_S	1.50
1.0 sec Spectral Acceleration S_1	0.60
0.2 sec Max Spectral Response S_{MS}	1.80
1.0 sec Max Spectral Response S_{M1}	0.84
0.2 sec Design Spectral Response S_{DS}	1.20
1.0 sec Design Spectral Response S_{D1}	0.56
Design Category	D

Underpinning and Shoring

Unless non-yielding (i.e. tiedback or rigidly-braced) shoring is provided, underpinning should be installed where excavations will extend below a 1-1/2:1 line projected down from the ground surface adjacent to existing foundations. Underpinning should consist of drilled piers, helical piers or deepened pit footings which are designed in accordance with the recommendations presented in the *Foundations* section of this report. Excavations for underpinning must be properly shored.

The Contractor should slope excavations in accordance with OSHA standards or install shoring as the excavation proceeds in order to maintain lateral support. All underpinning, temporary slopes and shoring should be contractually established as solely the responsibility of the

Contractor. Shoring should be designed to resist lateral earth pressures and surcharge loading from structures and retaining walls as outlined in the *Retaining Walls* section of this report.

Foundation Support

Drilled Piers

Drilled, cast-in-place, reinforced concrete piers should be at least 18 inches in diameter, and should extend at least 6 feet into approved competent bedrock. Design pier depths and diameters should be calculated by the Project Structural Engineer using the criteria presented below. The actual depths to competent bedrock should be determined by our representative in the field during pier drilling.

The sidewalls of pier holes allowed to remain open may be subject to desiccation and deterioration which adversely impacts skin friction capacity. If concrete is not placed in pier holes within 72 hours of drilling, we should be notified to reevaluate the holes to determine if they need to be reamed or re-drilled.

Piers should be interconnected with grade beams. The portion of piers and grade beams extending at least 12 inches below finished grade can impose a passive equivalent fluid pressure of 150 pounds per cubic foot (pcf). For piers this pressure should be assumed to act over 2 pier diameters. The portion of piers extending into approved competent bedrock can impose a passive equivalent fluid pressure of 450 pcf acting over 2 pier diameters and vertical dead plus real live loads of 1000 pounds per square foot (psf) in skin friction. The portion of piers and grade beams designed to impose passive pressures should have at least 7 feet of horizontal confinement from the face of the nearest retaining wall. Where allowed by code, these values may be increased by 1/3 for seismic and wind loads, but should be decreased by 1/3 for determining uplift resistance. Skin friction should be neglected in the material located above the bedrock, and end bearing should be neglected due to the uncertainty of mobilizing end bearing and skin friction simultaneously.

If groundwater is encountered, it may be necessary to dewater the holes and/or to place concrete by the tremie method. If caving soils are encountered, it will be necessary to case the holes. Casing should be carefully maintained ahead of the drill to avoid causing settlement of adjacent improvements. Casing should be removed from the holes simultaneous with concrete placement. Hard drilling or coring will be required to achieve the required bedrock penetrations.

Helical Piers

Helical piers should consist of end bearing Chance Anchors (A.B. Chance Company), or equivalent, which are installed using a rotary type torque motor. The helical piers should be installed and corrosion protected in accordance with the manufacturer's specifications. Helical piers extending into approved competent bedrock should be designed using an allowable bearing capacity of 12,000 pounds per square foot (psf) for dead plus code live loads. The actual bearing capacity of the piers should be evaluated based on measured torque values obtained in the competent bedrock during installation. If the piers are not contracted on a guaranteed design-build basis, load testing should be performed on at least one pier to verify capacity.

Helical piers should be interconnected with grade beams to support structural loads and to resist lateral loads. The portion of grade beams extending at least 12 inches below finished grade can impose a passive equivalent fluid pressure of 150 pounds per cubic foot (pcf). The portion of grade beams designed to impose passive pressures should have at least 7 feet of horizontal confinement from the face of the nearest retaining wall. No lateral resistance should be derived from the helical shafts.

Pit Footings

Hand-excavated pit footings for underpinning should be at least 24 inches square, and should be bottomed in approved firm soils at least 18 inches below a 2:1 line projected up from the base of planned excavations. Footing excavations should be shored as necessary to prevent ground loss. The footings can be designed to impose dead plus code live load bearing pressures and total design load bearing pressures of 2,000 and 2,600 psf, respectively. The portion of footings extending at least 12 inches below a 2:1 line projected up from the base of planned excavations can impose a passive equivalent fluid pressure of 250 pounds per cubic foot (pcf) and a friction factor of 0.25 times net vertical dead load. The portion of pit footings designed to impose passive pressures should have at least 7 feet of horizontal confinement from the face of the nearest slope or wall.

Stiffened Mat

If piers will not be used and a few inches of differential movement is considered acceptable, a stiffened mat may be used. Excavations beneath planned mats should be deepened as necessary to remove existing fills. Soils beneath planned mats and exposed by overexcavation should be scarified to a minimum depth of 8 inches, brought to near optimum moisture content, and recompacted to at least 90 percent relative compaction. Relative compaction refers to the in-place dry density of a soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557 test procedure. Optimum moisture content is the water content of the soil (percentage by dry weight) corresponding to the maximum dry density.

In areas of overexcavation, approved fill material should be placed in lifts not exceeding 8 inches in uncompacted thickness, moisture conditioned to within 3 percent of optimum moisture content, and compacted to at least 90 percent relative compaction to establish subgrade. All fill material should be free of organic matter. The fill material should not contain rocks or lumps larger than 4 inches in greatest dimension, and no more than 15 percent should be larger than 2 inches. Considerable moisture conditioning of on-site material may be required prior to reuse as fill. Imported fill material should have a plasticity index of 15 percent or less, and a maximum liquid limit of 40 percent. Herzog Geotechnical should approve all imported fill prior to it being brought to the site.

Mat foundations should be at least 12 inches thick, and can be designed to impose dead plus code live load bearing pressures and total design load bearing pressures of 1,000 and 1,300 pounds per square foot (psf), respectively. A modulus of subgrade reaction of 20 pounds per cubic inch (pci) should be used for design. Mats should be designed to span 6 foot square zones of non-support under full dead load, and to cantilever at least 3 feet at building edges and corners under full dead load. Resistance to lateral forces can be obtained using a passive equivalent earth pressure of 150 pcf and a soil friction factor of 0.25 times net vertical dead load. Passive pressure should be neglected in the top 6 inches where the ground surface will not be covered by exterior slabs. The portion of mats designed to impose passive pressures should have at least 7 feet of horizontal confinement from the face of the nearest slope or wall.

Mat subgrade should be sloped to drain into a 12 inch deep trench excavated beneath the middle of the mat. The trenches should be lined completely with a filter fabric such as Mirafi 140N, or equivalent. A 4-inch diameter rigid-perforated PVC or ABS (Schedule 40, SDR 35 or equivalent) pipe should be placed on a 1-inch layer of drain rock at the bottom of the trench with perforations down. The trench should be backfilled with drain rock up to mat subgrade elevation. The filter fabric should be wrapped over the top of the drain rock. The pipe should be sloped to drain by gravity to a non-perforated pipe which discharges at an approved outlet. The trench for the non-perforated pipe should be backfilled with properly compacted soil.

The mat should be underlain by a capillary moisture break consisting of at least 4 inches of free-draining crushed rock or gravel at least 1/4 inch, and no larger than 3/4 inch, in size. Moisture vapor detrimental to floor coverings or stored items will condense on the underside of the mat. A moisture vapor barrier should therefore be installed over the capillary break. The barrier should be specified by the mat designer. It should be noted that conventional concrete mat construction is not waterproof. The local standard of crushed rock and vapor barrier will not prevent moisture transmission through mats. Where moisture sensitive floor coverings are to be installed, a waterproofing expert and/or the flooring manufacturer should be consulted for their recommended moisture and vapor protection measures, including moisture barriers, concrete admixtures and/or sealants.

Retaining Walls

Retaining walls should be supported on mat or drilled pier foundations which are designed in accordance with the recommendations presented in this report.

Free-standing retaining walls should be designed to resist active lateral earth pressures equivalent to those exerted by a fluid weighing 45 pounds per cubic foot (pcf) where the backslope is level, and 60 pcf for backfill at a 2:1 slope. Retaining walls restrained from movement at the top should be designed to resist an "at-rest" equivalent fluid pressure of 60 pcf for level backfill and 75 pcf for backfill at a 2:1 slope. For intermediate slopes, interpolate between these values. A minimum factor of safety against instability of 1.5 should be used to evaluate static stability of retaining walls.

Seismic wall stability may be evaluated based on a uniform lateral earth pressure of 12xH psf (where H is the height of the wall in feet). This pressure is in addition to the active equivalent fluid pressures presented in this report. For restrained walls, seismic pressures may be assumed to act in combination with active rather than at-rest earth pressures. The factor of safety against instability under seismic loading should be at least 1.1.

In addition to lateral earth pressures, retaining walls must be designed to resist horizontal pressures that may be generated by uphill retaining walls and foundation loads. Where an imaginary 1-1/2:1 (horizontal:vertical) plane projected downward from the base of an upslope retaining wall intersects the downslope wall, that portion of the downslope wall below the intersection should be designed for an additional horizontal uniform pressure equivalent to the maximum calculated lateral earth pressure at the base of the upslope wall. Where an imaginary 1-1/2:1 plane projected downward from the outermost edge of a surcharge load or footing intersects a retaining wall, we should be contacted to provide appropriate lateral surcharge criteria.

Retaining walls should be fully backdrained. The backdrains should consist of 4-inch diameter, rigid perforated pipe surrounded by a drainage blanket. The top of the drain pipe should be at least 8 inches below lowest adjacent downslope grade. The pipe should be PVC Schedule 40 or ABS with an SDR of 35 or better, and the pipe should be sloped to drain at least 1 percent by gravity to an approved outlet. Accessible subdrain cleanouts should be provided, and should be maintained on a routine basis. The drainage blanket should consist of clean, free-draining crushed rock or gravel wrapped in a filter fabric such as Mirafi 140N. Alternatively, the drainage blanket could consist of Caltrans Class 2 "Permeable Material", in which case the filter fabric may be omitted. A prefabricated drainage structure such as Mirafi Miradrain may also be used provided that the backdrain pipe is embedded in at least 1 cubic foot of Class 2 Permeable Material or fabric-wrapped crushed rock per lineal foot of wall. The drainage blanket should be continuous, at least 1 horizontal foot thick, and should extend to within 1 foot of the surface. The uppermost 1 foot should be backfilled with compacted soil to exclude surface water from entering the backdrain.

Where migration of moisture through retaining walls would be detrimental or undesirable, retaining walls should be waterproofed as specified by the Project Architect or Structural Engineer.

Wall backfill should conform with the fill requirements outlined previously. Wall backfill should be spread in level lifts not exceeding 8 inches in thickness, brought to near the optimum moisture content, and compacted to at least 90 percent relative compaction. Relative compaction refers to the in-place dry density of a soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557 test procedure. Optimum moisture content is the water content of the soil (percentage by dry weight) corresponding to the maximum dry density. Retaining walls will yield slightly during backfilling. Therefore, walls should be backfilled prior to building onto or adjacent to the walls. Backfilling adjacent to walls should be performed only with hand operated equipment to avoid over-stressing the walls, and the walls should be properly braced during the backfilling operations.

Even well-compacted backfill will settle about 1 percent of its thickness. Therefore, slabs and other improvements crossing the backfill should be designed to span or to accommodate this settlement.

Slabs

Interior and settlement sensitive slabs should be structurally designed to span between foundation supported elements.

Slab subgrade should be sloped to drain into a 12 inch deep trench excavated beneath the middle of each slab. The trenches should be lined completely with a filter fabric such as Mirafi 140N, or equivalent. A 4-inch diameter rigid-perforated PVC or ABS (Schedule 40, SDR 35 or equivalent) pipe should be placed on a 1-inch layer of drain rock at the bottom of the trench with perforations down. The trench should be backfilled with drain rock up to slab subgrade elevation. The filter fabric should be wrapped over the top of the drain rock. The pipe should be sloped to drain by gravity to a non-perforated pipe which discharges at an approved outlet. The trench for the non-perforated pipe should be backfilled with properly compacted soil.

Slabs should be underlain by a capillary moisture break consisting of at least 4 inches of free-draining, crushed rock or gravel (slab base rock) at least 1/4 inch, and no larger than 3/4 inch, in size. Positive drainage should be provided from the slab base rock. Moisture vapor detrimental to floor coverings or stored items will condense on the undersides of slabs. A moisture vapor barrier should therefore be installed over the capillary break. The barrier should be specified by the slab designer. It should be noted that conventional concrete slab-on-grade construction is not waterproof. The local standard under-slab construction of crushed rock and vapor barrier will not prevent moisture transmission through slab-on-grade. Where moisture sensitive floor coverings are to be installed, a waterproofing expert and/or the flooring manufacturer should be consulted for their recommended moisture and vapor protection measures, including moisture barriers, concrete admixtures and/or sealants.

Geotechnical Drainage

The ground surface within 5 feet of the perimeter of the residence should be sloped to drain at least 2 percent away from the structure. Ponding of surface water should not be allowed. Drop inlets should be installed at low areas. Provisions should be made for fail-safe drainage around the residence to prevent flooding in the event that the drains become clogged. All roofs should be provided with gutters and downspouts. All surface drains and downspouts should be connected to new non-perforated pipes which discharge at approved erosion resistant outlets well away from improvements. Downspout and drop inlet conduits should consist of rigid PVC or ABS pipe which is SDR 35, Schedule 40, or equivalent. Surface drains and downspouts should be maintained entirely separate from foundation drains and slab/mat underdrains. Downspouts, surface drains and subsurface drains should be checked for blockage and cleared and maintained on a regular basis.

Foundation drains should be installed adjacent to perimeter foundations. Perimeter retaining wall backdrains may be substituted for foundation drains. The drains should consist of trenches which extend 18 inches deep, or 12 inches below lowest adjacent interior or crawl space grade, whichever is deeper, and which are sloped to drain at least 1 percent by gravity. The trenches should be lined completely with a filter fabric such as Mirafi 140N, or equivalent. A 4-inch diameter rigid perforated PVC or ABS pipe (Schedule 40, SDR 35 or equivalent) should be placed on a 1-inch thick layer of drain rock at the bottom of the trenches with perforations down. Frequent cleanout risers should be provided for the drain, and sweeps or sanitary wyes should be used to allow for future inspection and maintenance of the drain. The pipes should be sloped to drain at least 1 percent by gravity to a non-perforated pipe (Schedule 40, SDR 35 or equivalent) which discharges at an approved erosion resistant outlet. The trench for the perforated pipe should be backfilled to within 6 inches of the ground surface with drain rock. The filter fabric should be wrapped over the top of the drain rock. The upper 6 inches of the trenches should be backfilled with compacted clayey soil to exclude surface water. The trench for the non-perforated outlet pipe should be completely backfilled with compacted soil.

Supplemental Services

Our conclusions and recommendations are contingent upon Herzog Geotechnical being retained to review the project plans and specifications to evaluate if they are consistent with our recommendations, and being retained to provide intermittent observation and appropriate field and laboratory testing during overexcavation, scarification and recompaction, backfill placement and compaction. We should also observe pier drilling, helical pier installation and load testing, mat subgrade compaction, slab and mat base rock installation, backdrain installation, and wall backfilling. We should also be notified to observe the completed project. Steel, concrete, slab moisture barriers, corrosion protection and/or waterproofing should be inspected by the designer. Inspection of temporary slopes, shoring and underpinning should be performed by the respective designers, and are specifically excluded from our scope of services.

If during construction subsurface conditions different from those described in this report are observed, or appear to be present beneath excavations, we should be advised at once so that these conditions may be reviewed and our recommendations reconsidered. The recommendations made in this report are contingent upon our being notified to review changed conditions.

If more than 18 months have elapsed between the submission of this report and the start of work at the site, or if conditions have changed because of natural causes or construction operations at or adjacent to the site, the recommendations of this report may no longer be valid or appropriate. In such case, we recommend that we review this report to determine the applicability of the conclusions and recommendations considering the time elapsed or changed conditions. The recommendations made in this report are contingent upon such a review.

We should be notified at least 48 hours before the beginning of each phase of work requiring our observation, and upon resumption after interruptions. These services are performed on an as-requested basis and are in addition to this geotechnical reconnaissance. We cannot provide comment on conditions, situations or stages of construction that we are not notified to observe.

LIMITATIONS

This report has been prepared for the exclusive use of Andrew McCune and his consultants for the proposed project described in this report. Our services consist of professional opinions and conclusions developed in accordance with generally-accepted geotechnical engineering principles and practices. We provide no other warranty, either expressed or implied. Our conclusions and recommendations are based on the information provided us regarding the proposed construction, the results of our field exploration and laboratory testing programs, and professional judgment. Verification of our conclusions and recommendations is subject to our review of the project plans and specifications, and our observation of construction.

The test boring log represents subsurface conditions at the location and on the date indicated. It is not warranted that is representative of such conditions elsewhere or at other times. Site conditions and cultural features described in the text of this report are those existing at the time of our field exploration, and may not necessarily be the same or comparable at other times. The location of the test boring was established in the field by reference to existing features, and should be considered approximate only.

Our work only addressed the proposed renovations, and did not include an evaluation of existing site walls, driveway pavements, or other items/areas. Our scope of services did not include an environmental assessment or an investigation of the presence or absence of hazardous, toxic or corrosive materials in the soil, surface water, ground water or air, on or below, or around the site, nor did it include an evaluation or investigation of the presence or absence of wetlands. Our work also did not include an evaluation of any potential mold hazard at the site.

We appreciate the opportunity to be of service to you. If you have any questions, please call.

Sincerely,
HERZOG GEOTECHNICAL

REGISTERED PROFESSIONAL ENGINEER
CRAIG W. HERZOG
No. 002383
GEOTECHNICAL
STATE OF CALIFORNIA

Craig Herzog, G.E.
Principal Engineer

Attachments: References
Plate 1 - 5

REFERENCES

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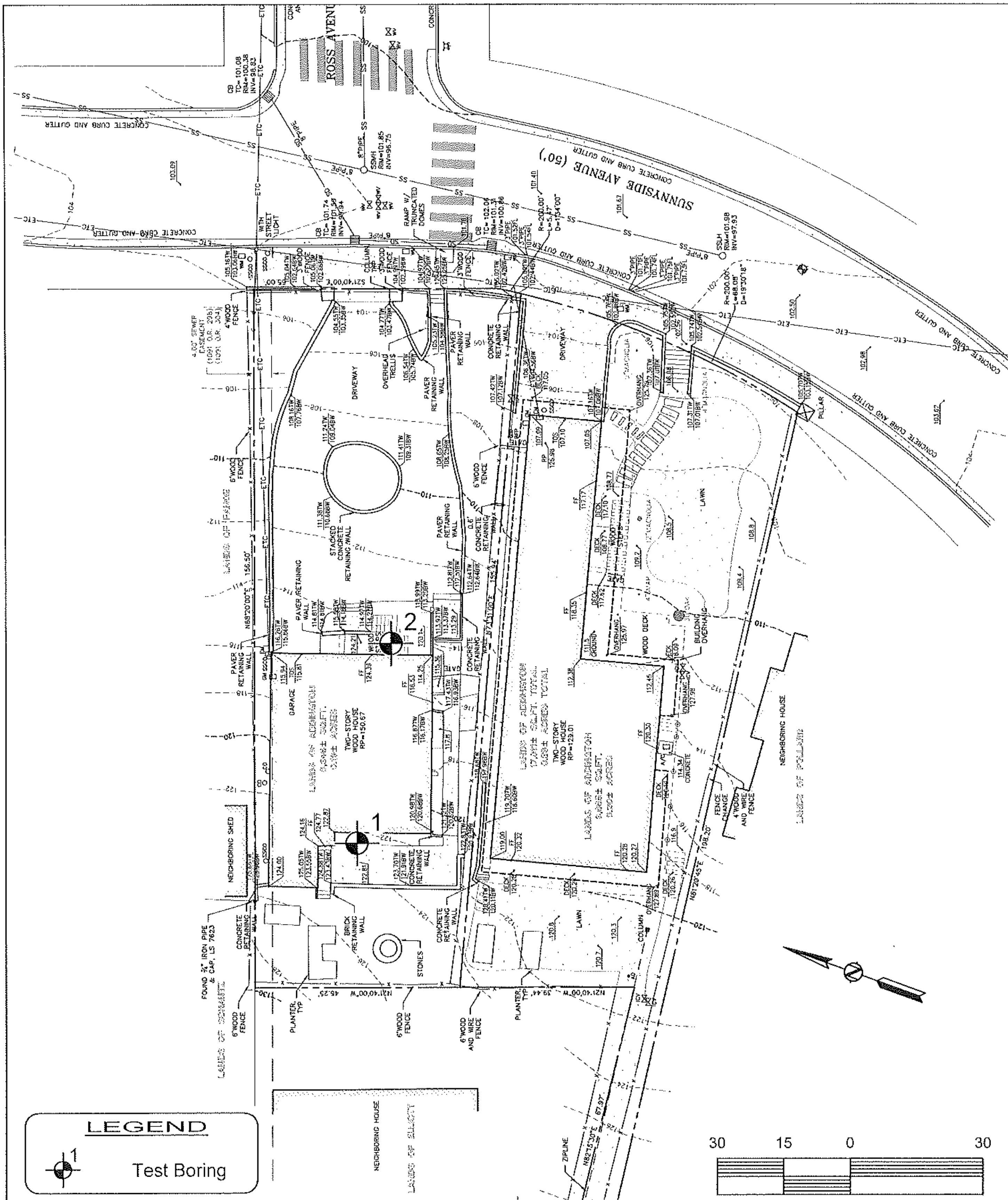

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HERZOG GEOTECHNICAL CONSULTING ENGINEERS	Job. No: 4167-01-22 Appr: CH Drwn: LPDD Date: JUL 2022	SITE PLAN 49 Sunnyside Avenue San Anselmo, California	PLATE 1

EQUIPMENT: 4" Flight Auger

LOGGED BY: C.H.

ELEVATION: **

START DATE: 6-30-22

FINISH DATE: 6-30-22

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	DEPTH (FEET)	Description
						0	GRAY-BROWN SILTY SAND (SM), loose, moist (Fill)
		12.4	104		21	1	
						1.5	MOTTLED YELLOW-ORANGE-GRAY CLAYEY SAND (SC), medium dense, dry, with decomposed sub-angular rock fragments
						4	becomes moist at 4'
		17.9	111		24	4.5	
						5.5	ORANGE-BROWN GRAVELLY CLAY WITH SAND (CL), medium stiff to stiff, moist
		16.9	111		32	8	MOTTLED ORANGE-GRAY-BROWN CLAYEY SAND (SC),
					34	9	
					55	10	
						10.5	ORANGE-BROWN SANDSTONE, firm, friable, highly weathered
						11	BOTTOM OF BORING 1 @ 11 FEET No Free Water Encountered

* Converted to equivalent standard penetration blow counts.

** Existing ground surface at time of investigation.

HERZOG GEOTECHNICAL CONSULTING ENGINEERS

Job No: 4167-01-22

Appr: CH

Drwn: LPDD

Date: JUL 2022

LOG OF BORING 1

49 Sunnyside Avenue

San Anselmo, California

PLATE 2

EQUIPMENT: **4" Flight Auger**

ELEVATION: **

LOGGED BY: **C.H.**

START DATE: **6-30-22**

FINISH DATE: **6-30-22**

Other Laboratory Tests	Pocket Penetrometer (ksf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200 sieve	Blows/Foot *	Sample	DEPTH (FEET)
							0
							1
		20.9	103		7	■	
							2
							3
		17.1	112		26	■	4
							5
							6
							7
							8
		33/3"				■	9

Depth	Description
0	GRAY-BROWN SILTY SAND (SM), loose, moist (Fill)
2	MOTTLED ORANGE-GRAY-BROWN CLAYEY SAND (SC), loose, moist
3	becomes medium dense at 3'
8.5	GRAY SHEARED SHALE, firm to moderately hard, weak to moderately strong, highly weathered, vertically oriented shear fabric

BOTTOM OF BORING 2 @ 9.3 FEET
No Free Water Encountered

* Converted to equivalent standard penetration blow counts.

** Existing ground surface at time of investigation.

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LOG OF BORING 2

49 Sunnyside Avenue

San Anselmo, California

PLATE 3

MAJOR DIVISIONS				TYPICAL NAMES
COARSE GRAINED SOILS More than Half > #200 sieve	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL GRADED GRAVELS, GRAVEL-SAND
			GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, POORLY GRADED GRAVEL-SAND-SILT MIXTURES
			GC	CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS
			SP	POORLY GRADED SANDS, GRAVELLY SANDS
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS, POOORLY GRADED SAND-SILT MIXTURES
			SC	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES
FINE GRAINED SOILS More than Half < #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 WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACIOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS

UNIFIED SOIL CLASSIFICATION SYSTEM

Shear Strength, psf

Confining Pressure, psf

Consol	Consolidation	Tx	2630 (240)	Unconsolidated Undrained Triaxial
LL	Liquid Limit (in %)	Tx sat	2100 (575)	Unconsolidated Undrained Triaxial, saturated prior to test
PL	Plastic Limit (in %)	DS	3740 (960)	Unconsolidated Undrained Direct Shear
PI	Plasticity Index	TV	1320	Torvane Shear
Gs	Specific Gravity	UC	4200	Unconfined Compression
SA	Sieve Analysis	LVS	500	Laboratory Vane Shear
■	Undisturbed Sample (2.5-inch ID)	FS	Free Swell	
◪	2-inch-ID Sample	EI	Expansion Index	
◩	Standard Penetration Test	Perm	Permeability	
⊠	Bulk Sample	SE	Sand Equivalent	

KEY TO TEST DATA

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SOIL CLASSIFICATION CHART AND KEY TO TEST DATA

49 Sunnyside Avenue

San Anselmo, California

PLATE

4

ROCK SYMBOLS

SHALE OR CLAYSTONE	CHERT	SERPENTINITE
SILTSTONE	PYROCLASTIC	METAMORPHIC ROCKS
SANDSTONE	VOLCANIC	DIATOMITE
CONGLOMERATE	PLUTONIC	SHEARED ROCKS

LAYERING

MASSIVE	Greater than 6 feet
THICKLY BEDDED	2 to 6 feet
MEDIUM BEDDED	8 to 24 inches
THINNLY BEDDED	2-1/2 to 8 inches
VERY THINNLY BEDDED	3/4 to 2-1/2 inches
CLOSELY LAMINATED	1/4 to 3/4 inches
VERY CLOSELY LAMINATED	Less than 1/4 inch

JOINT, FRACTURE, OR SHEAR SPACING

VERY WIDELY SPACED	Greater than 6 feet
WIDELY SPACED	2 to 6 feet
MODERATELY SPACED	8 to 24 inches
CLOSELY SPACED	2-1/2 to 8 inches
VERY CLOSELY SPACED	3/4 to 2-1/2 inches
EXTREMELY CLOSELY SPACED	Less than 3/4 inch

HARDNESS

SOFT - Pliable; can be dug by hand

FIRM - Can be gouged deeply or carved with a pocket knife

MODERATELY HARD - Can be readily scrached by a knife blade; scratch leaves heavy trace of dust and is readily visable after the powder has been blown away

HARD - Can be scratched with difficulty; scratch produces little powder and is often faintly visable

VERY HARD - Cannot be scratched with pocket knife; leaves a metallic streak

STRENGTH

PLASTIC - Capable of being molded by hand

FRIABLE - Crumbles by rubbing with fingers

WEAK - An unfractured specimen of such material will crumble under light hammer blows

MODERATELY STRONG - Specimen will withstand a few heavy hammer blows before breaking

STRONG - Specimem will withstand a few heavy ringing hammer blows and usually yields large fragments

VERY STRONG - Rock will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments

DEGREE OF WEATHERING

HIGHLY WEATHERED - Abundant fractures coated with oxides, carbonates, sulphates, mud, etc., thourough discoloration, rock disintegration, mineral decomposition

MODERATELY WEATHERED - Some fracture coating, moderate or localized discoloration, little to no effect on cementation, slight mineral decomposition

SLIGHTLY WEATHERED - A few stained fractures, slight discoloration, little or no effect on cementation, no mineral decomposition

FRESH - Unaffected by weathering agents, no appreciable change with depth