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REPORT
GEOTECHNICAL INVESTIGATION
RESIDENTIAL CONSTRUCTION
129 PERALTA AVENUE
MILL VALLEY, CALIFORNIA

Job No.: 2038.20

Prepared for:

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By

Nersi Hemati, P.E.
Geotechnical Engineer - 390

September 14, 2020

INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed residential construction at 129 Peralta Avenue, Mill Valley, California. We understand that the existing house will be removed and a new house, swimming pool and ADU is proposed. The proposed project is shown on the preliminary plans Sheets A1.0-A1.3 by Polsky Perlstein Architects, dated 08/10/20.

The purpose of the geotechnical investigation is to evaluate the soil conditions at the site and provide geotechnical conclusions and recommendations to aid the design and construction of the project.

The scope of our work included exploring and evaluating the subsurface conditions with test borings and laboratory tests, analyzing the results of the field and laboratory work, and presenting our findings in this report. Our report provides the following geotechnical information:

1. A description of the soil and geologic conditions observed.

2. An opinion of project feasibility from a geotechnical standpoint.

3. Design recommendations for new foundations, retaining walls and pool walls.

4. Site grading and soil engineering drainage recommendations.

The scope of our work does not include an evaluation of soil or groundwater hazardous waste contamination, toxicity, or corrosion potential at the site. The scope of our work also does not include an evaluation of the existing foundations or the foundation support of the existing structures.

WORK PERFORMED

We performed a reconnaissance of the site and reviewed the following geological maps:

> Rice, S.J., and Smith, T.C., 1976; Geology of the Lower Ross Valley, Corte Madera, Homestead Valley, Tamalpais Valley, Tennessee Valley and Adjacent Areas, Marin County, California: Scale 1:12,000.

On September 2, 2020 we explored the subsurface conditions at the site to the extent of 5 test borings. The test borings were drilled with portable power augers to depths ranging from 1’ to 8’, terminating in bedrock. The approximate locations of the borings are indicated on the attached Boring Location Sketch, Plate 1. We observed the drilling, logged the conditions encountered, and obtained samples for visual examination, classification and laboratory testing.

Detailed descriptions of the materials encountered in our borings are presented on the logs of boring. The attached boring logs and related information depict subsurface conditions only at the approximate location shown on Plate 1 and on the date designated on the log; subsurface conditions at other locations and times could differ from the conditions occurring at our boring location.

Details of the field and laboratory work are presented in the appendix at the end of the report.

SITE AND SOIL CONDITIONS

The property is located on a relatively level to down-sloping lot on the upslope north side of Peralta Avenue in Mill Valley. The property is relatively level in the existing house area but generally slopes down to the northwest beyond the house and backyard at average gradients of approximately 4 or 3:1 (horizontal to vertical). It appears that some fill material is present in the northerly portions of the site. There is an existing residence, driveway and garage existing on the property. Many of these are to be replaced or demolished.

Geology and Soils

The area has been mapped as containing Franciscan melange sandstone bedrock at shallow depths (Rice and Smith, 1976). We observed bedrock outcrops to the northeast of the existing house. Our test borings mainly encountered sandstone and shale bedrock at a shallow depth (about 1 to 2 feet) except for borings 2 and 5 which encountered bedrock at about 6 and 3.5 feet depths, respectively. Bedrock was overlain by brown sandy clay with rock fragments and roots (partially fill). Detailed descriptions of the materials encountered are presented in logs of Borings, plates 2-6.

Groundwater

Free groundwater was not encountered at the time of drilling. However, fluctuations in the groundwater level could occur due to variation in rainfall and/or other factors and groundwater may be encountered during construction.

CONCLUSIONS AND DISCUSSION

Based on our field, laboratory and office studies we judge that the project is feasible from a geotechnical engineering standpoint provided that the recommendations presented in this report are incorporated in the design and construction. The main geotechnical consideration is the need for potentially hard excavation in bedrock.

In our judgment, the proposed structures may be supported on shallow spread footing foundations bottomed in bedrock. Drilled, cast-in-place, reinforced concrete piers may be used where bedrock is not encountered at foundation level. The piers should extend through the weak surface soils and into the underlying bedrock. The swimming pool may need to be supported on drilled piers due to the amount of fill material that may be present in that location.

There is the potential for encountering difficult excavations in bedrock to achieve the proposed grades. Heavy ripping and/or jack hammering may become necessary.

Temporary excavations during construction should be properly sloped backed or shored in accordance with OSHA requirements. Construction safety and stability of temporary excavations during construction is solely the responsibility of the contractor.

Surface and subsurface drainage facilities should be constructed as discussed below in the "Recommendations" section of the report.

Like the entire San Francisco Bay Area, the site is subject to strong ground shaking during earthquakes. It will be necessary to design and construct the project in strict conformance with current standards for earthquake resistant construction. The U.S. Geological Survey (USGS, 2015) predicts a 72% chance of a large earthquake (Richter Magnitude 6.7 or greater) occurring in the Bay Area in the next 30 years.

All conclusions and recommendations presented in this report are contingent upon Nersi Hemati being retained to: 1) Review the geotechnical engineering aspects of the final grading and foundation plans prior to construction; and 2) Observe construction of the project as outlined below in the "Supplemental Services" section of this report.

RECOMMENDATIONS

Faulting and Seismicity

The site is not within a current Alquist-Priolo Special Studies Zone, and the geologic maps reviewed indicate that active faults are not considered to exist within the site. The nearest known active faults are the San Andreas Fault, located about 8 kilometers to the southwest, and the Hayward Fault located about 17 kilometers to the east. Maximum credible earthquake magnitudes of 7.9 and 7.1 (Richter scale) have been postulated for these faults, respectively. The site soils may be categorized as Class “C” in the seismic design of the project in accordance with the 2019 California Building Code (CBC) and ASCE 7-16 Standard. The following seismic parameters should be used:

Mapped Spectral Acceleration Values:	Ss = 1.5 g S1 = 0.602 g
Site Coefficients:	Fa = 1.2 Fv = 1.4
Spectral Response Accelerations:	Sms = 1.8 g Sm1 = 0.842 g
Design Spectral Response Accelerations:	Sds = 1.2 g Sd1 = 0.562 g

Site Grading

Areas to be developed should be cleared of vegetation, debris, the existing structures, slabs, and foundations. The site should be stripped of the upper few inches of soil containing organic matter. The strippings should be removed, or if suitable, stockpiled for re-use as topsoil in landscaping.

Following initial site preparation, excavation should be performed as necessary. We anticipate that, with the exception of organic matter and rocks larger than 6 inches in diameter, the excavated material will be suitable for re-use as compacted fill.

In sloping ground areas steeper than about 5:1 a keyway should be excavated at the toe of the fill extending at least 2 feet into bedrock. A subdrain consisting of minimum 4-inch diameter perforated pipe (SDR 35 or better) encased in Class 2 permeable materials (Caltrans Specification) should be installed in the keyway. The exposed subgrade to receive fill should be prepared by scarifying to a depth of 6 inches, moisture-conditioning as necessary, and compacting to at least 90% of the maximum dry density of the materials as determined by the ASTM D-1557 laboratory compaction test procedure. Fill material should then be spread in 8-inch thick loose lifts, moisture-conditioned as necessary, and compacted to at least 90% relative compaction. As successive layers of fill are placed they should be continually keyed into rock or strong soil. The final lift below slabs and pavements should be compacted to at least 95% Relative Compaction.

Imported fill should be non-expansive; that is, it should have a plasticity index of 15 or less. The imported fill material should be free of organic matter and of rocks or lumps larger than 6 inches in diameter. Not more than 15% of the rocks or lumps should exceed 2.5 inches.

Generally, grading is most economically performed during the summer months when on-site soils are usually dry of optimum moisture content. Delays should be anticipated in site grading performed during the rainy season due to excessive moisture in on-site soils. Special and relatively expensive construction procedures should be anticipated if grading must be completed during the winter.

Cut and fill slopes should be no steeper than 2:1. Where steeper banks are required, retaining walls should be used. Slopes should be planted with fast growing, deep-rooted groundcover to reduce sloughing or erosion.

Spread Footings

Spread footings should be used in level areas excavated into bedrock. Spread footings should be at least 16 inches wide, and should extend at least 12 inches into bedrock.

The footings should be stepped as necessary to produce level tops and bottoms. Footings should be deepened as necessary to provide at least 7 feet of horizontal confinement between the footing bottoms and the face of the nearest slope.

Footings installed in accordance with these recommendations may be designed using allowable bearing pressures of 2500, 3000, and 4000 pounds per square foot (psf), for dead loads, dead plus code live loads, and total loads (including wind and seismic), respectively. A passive equivalent fluid pressure and friction factor of 400 pcf and 0.40 may be used respectively, to resist sliding.

Drilled Piers

Drilled piers should be at least 18 inches in diameter and should extend at least 6 feet into rock but as designed by the project structural engineer. The final depth of the piers should be determined by the Geotechnical Engineer during the pier drilling operations. In sloping ground areas, the piers should be designed and reinforced to resist lateral creep forces equivalent to an average 4-foot thick zone exerting an equivalent fluid pressure of 60 pounds per cubic foot (pcf) acting on 2 pier diameters. The thickness of the creep zone may be reduced where proposed excavations remove existing soil (such as the swimming pool area) and should generally equal depth to bedrock. The piers should be designed by the project structural engineer. However, we recommend that all piers be reinforced with at least four No. 5 bars. The piers should be connected with grade beams and tie beams. The grade beams should span between the piers in accordance with structural requirements. The steel from the piers should extend sufficient distance into the grade beams to develop its full bond strength.

The portion of the piers extending into rock may be designed using an allowable skin friction of 800 pounds per square foot (psf). End bearing should be neglected because of the difficulty of cleaning out small diameter pier holes, and the uncertainty of mobilizing end bearing and skin friction simultaneously. Lateral loads on piers will be resisted by passive pressure on the rock. An equivalent fluid pressure of 400 pcf acting on 2 pier diameters should be used and the stability of the system should be calculated using a minimum factor of safety of 1.5.

If ground water is encountered, it may be necessary to dewater the holes and/or place the concrete by the tremie method. If caving soils are encountered, it may be necessary to case the holes. Hard drilling may be required to achieve the required penetration.

Retaining Walls

New retaining walls should be designed to resist lateral earth pressures plus additional lateral pressures that may be caused by surcharge loads at the ground surface behind the walls such as for surcharge from nearby foundations and walls. Retaining walls supporting a relatively level backfill should be designed to resist an active equivalent fluid pressure of 45-pcf acting in a triangular pressure distribution. Where the backfill slopes up at a 2:1 gradient, the walls should be designed for an equivalent fluid pressure of 60 pcf. Values can be interpolated for flatter gradients. Retaining walls restrained from movement at the top should be designed for “at-rest” pressures of 60 and 75 pcf for level and sloping backfills respectively. Lower lateral pressures may apply where retaining walls support bedrock cuts. Non-restrained retaining walls may deflect about 1% of the height of the wall at the top of the wall. A minimum factor of safety of 1.5 should be used to evaluate static stability of retaining walls.

We recommend a uniform pressure (in psf) equal to 10 times the height of the retaining walls (measured in feet) be used as seismic surcharge. For restrained walls, seismic pressures may be assumed to act in combination with active rather than at-rest pressures. The factor of safety against instability under seismic loading should be at least 1.1.

Where an imaginary 1-1/2:1 line projected down from foundations intersects retaining walls, the portions of the retaining walls below the intersection should be designed for an additional horizontal surcharge load. Where an 1-1/2:1 line projected down from the toe of a retaining wall intersects a lower retaining wall, the portion of the lower wall below the intersection should be designed for an additional surcharge load. The upper wall surcharge load should be assumed to be a uniform lateral pressure equal to the height of the upper retaining wall times the equivalent fluid pressure acting on that wall. Where retaining wall backfill is subject to vehicular traffic, the walls should be designed to resist an additional surcharge pressure equivalent to 2 feet of additional backfill.

Retaining walls should be fully backdrained. The backdrains should consist of 4-inch diameter, rigid perforated pipe embedded in drain rock. The pipe should be PVC Schedule 40, SDR 35, or equivalent, and the pipe should be sloped to drain to outlets by gravity. Drain rock should consist of clean, free-draining crushed rock or gravel. The rock should be wrapped in filter fabric such as Mirafi 140N or equivalent. Alternatively, Class 2 permeable rock (Caltrans Specification) should be used in lieu of drain rock and filter fabric. The top of the pipe should be at least 8 inches below the lowest adjacent grade. The crushed rock or gravel should extend to within 1 foot of the surface. The upper one-foot should be backfilled with compacted soil to exclude surface water. The ground surface behind retaining walls should be sloped to drain.

Where migration of moisture through retaining walls would be detrimental, retaining walls should be waterproofed. Retaining walls will yield slightly during backfilling. Therefore, walls should be backfilled prior to building on or adjacent to the walls.

Swimming Pool

It will be necessary to extend the pool excavation into undisturbed rock. Where the pool excavation does not extend into rock, the excavation may be deepened with the subgrade soils over-excavated and replaced with compacted crushed rock or drilled pier support may be employed. The pool should be constructed on at least 6 inches of compacted crushed rock with 4-inch diameter SDR 35 bottom-perforated pipes sloped to drain by gravity. The pool walls should be designed for the anticipated earth and water pressures and the downslope walls should also be designed to free-stand and withstand hydrostatic pressure from within, for the entire pool depth. Apron slabs, grade supported decks and concrete flat work should either be structurally supported or supported on compacted subgrade. However, we judge compacted fill would be subject to some settlement on the order of 1% of the fill thickness. Positive surface drainage should be provided away from the pool to outlets.

Soil Engineering Drainage

Surface water should be diverted away from slopes and foundations. Roofs should be provided with gutters, and the downspouts should be connected to closed conduits discharging well away from foundations and slopes. Roof downspouts and surface drains must be maintained entirely separate from foundation drains and retaining wall backdrains. The outlets should discharge into erosion-resistant areas, and should be provided with rock rip-rap or other energy dissipaters, if they discharge onto the ground.

Foundation subdrains should be installed to control subsurface water. The foundation drains should consist of trenches that extend to at least 12 inches below the level of any crawl space and installed around the entire perimeter of the structures except the downhill side and where retaining wall back drains are present.

The subdrains should consist of 4-inch diameter perforated pipe embedded in drain rock. The pipe should be PVC Schedule 40 or SDR 35 pipe, and the pipe should be sloped to drain to outlets by gravity. Drain rock should consist of clean, free-draining crushed rock or gravel. The rock should be wrapped in filter fabric such as Mirafi 140N or equivalent. Alternatively, class 2 permeable rock (Caltrans specification) may be used in lieu of drain rock and filter fabric. The top of the pipe should be at least 8 inches below the lowest adjacent crawl space grade. The crushed rock or gravel should extend to within 6 inches of the surface. The upper 6 inches should be backfilled with compacted soil to exclude surface water.

The ground surface should be sloped to drain away from foundations. Piped outlets should be provided to allow drainage through foundations.

Even with the above provisions, some water may be encountered in the crawl space areas due to the topography and geology of the area.

<u>Slab-On-Grade</u>

Unless bedrock is encountered at subgrade level, living area slabs should consist of structural slabs spanning between adjacent foundations. Slabs-on-grade subgrade should be rolled to produce a dense, uniform surface. Loose fill and areas of soft soil should be over-excavated and re-compacted as engineered fill.

The slabs should be underlain with a capillary moisture break consisting of at least 4 inches of clean, free draining crushed rock or gravel at least 1/4 inch and no larger than 3/4 inch in size. Where migration of moisture vapor through slabs would be detrimental, an impermeable membrane moisture vapor barrier, 15 mils or thicker, should be provided between the drain rock and the slabs. The membrane should be covered with 2 inches of sand to protect it during construction. However, we defer to flooring and waterproofing specialists who should be consulted for these items especially the use of sand over the membrane. Outlets should be provided from the slab drain rock.

In habitable areas, slab subgrade should be sloped to drain into subdrain trenches which should be approximately 12 inches deep (as grades permit) with 4-inch diameter, bottom perforated SDR 35 pipes encased in drain rock, sloped for gravity drainage to proper discharge locations. The drain rock should consist of ¾" clean crushed rock wrapped in Mirafi 140N filter fabric or equivalent.

The future expansion potential of the sub-grade soils should be reduced by thoroughly presoaking the slab sub-grade prior to concrete placement. Slabs should be at least 5 inches thick, and should be reinforced with at least #4 bars on 12-inch centers each way. These are minimum requirements as the slabs should be designed by the project structural engineer. Slabs should be grooved at regular intervals to induce and control cracking.

Flatwork / Exterior Slabs

We recommend that the subgrade be scarified at least 6 inches and compacted to 95% Relative Compaction. At least 6 inches of Class 2 aggregate base rock should be placed and compacted to 95% Relative Compaction below the concrete surface.

The slabs should be at least 5 inches thick, and should be reinforced at least with No. 4 bars at 18-inch centers to reduce cracking. These are only minimum requirements and the actual thickness and reinforcement should be based on specific design by the project structural engineer. Slabs should be grooved at regular intervals to induce and control cracking.

LIMITATIONS

Our services consist of professional opinions, conclusions and recommendations that are made in accordance with generally accepted geotechnical engineering principles and practices. This warranty is in lieu of all other warranties, either expressed or implied.

Geotechnical engineering is characterized by uncertainty. Therefore, we are unable to eliminate all risks or provide guarantees.

We judge that construction in accordance with these recommendations will be stable, and that the risk of future instability is within the range generally associated with construction on hillsides in Mill Valley. Subsurface conditions are complex, and may differ from those indicated by surface features and those encountered at the test hole locations.

Due to limitations inherent in geotechnical investigations, it is not uncommon to encounter variations in the soil conditions during construction nor is it practical to determine all such variations during an acceptable program of subsurface exploration for a project of this scope.

Soil conditions and standard of practice change. Therefore, we should be consulted to update this report if construction is not performed within 18 months.

SUPPLEMENTAL SERVICES

We should review the final plans for conformance with the intent of our recommendations. During construction, we should observe the conditions encountered in construction excavations and modify our recommendations, if warranted. We should observe and test fill placement and compaction.

We should observe footing excavations or pier drilling operations to determine the actual depths required. Our services during foundation construction are limited to observation of soil and bedrock conditions, depth of excavation or drilling, and the condition of excavations or pier holes prior to concrete placement. Our services do not include observation or approval of steel, concrete, or asphalt nor do they include establishing or verifying construction lines and grades. This should be performed by the appropriate party. Upon completion of the project, we should perform a final observation. We should summarize the results of this work in a final report.

These supplemental services are performed on an as-requested basis, and we cannot accept responsibility for items that we are not notified to observe. These supplemental services are in addition to this soil investigation, and are charged for on an hourly basis in accordance with our Schedule of Charges.

MAINTENANCE

Periodic land maintenance will be required. Surface and subsurface drainage facilities should be checked frequently, and cleaned and maintained as necessary. A dense growth of deep-rooted ground cover must be maintained on all slopes to reduce sloughing and erosion. Sloughing and erosion that occurs must be repaired promptly before it can enlarge into sliding.

APPENDIX - FIELD EXPLORATION AND LABORATORY TESTING

Field Exploration

Our field investigation consisted of a site reconnaissance and subsurface exploration. Due to site inaccessibility, we drilled 4-inch diameter exploratory borings with portable power auger equipment at the approximate locations shown on the Boring Location Sketch, Plate 1.

The materials encountered in the test borings were continuously logged in the field. Logs of our borings are included as Plates 2-6. The soils encountered in our exploratory borings are classified in accordance with the Unified Soils Classification System presented on Plate 7.

Relatively undisturbed soil samples were obtained from the exploratory borings at selected depths appropriate to the subsurface investigation. The samples were obtained with the 2.4" inside diameter Modified California Sampler as well as the Standard Penetration Test (SPT) sampler.

The blow counts were obtained by dropping a 70- pound hammer through a 30-inch free fall. The sampler was driven 18 inches, or a shorter distance where hard resistance was encountered, and the number of blows were recorded for each 6 inches of penetration. The blow per foot recorded on the boring logs represent the accumulated number of blows that were required to drive the sampler the last 12 inches or the number of inches indicated where the sampler did not penetrate the full 18 inches.

The blows per foot recorded on the boring log have been adjusted to represent the standard penetration test. The approximate location of the exploratory borings was established in the field by pacing and tape methods. Boring locations were not established by surveying methods and the approximate locations indicated on the Boring Location Sketch should be assumed accurate only to the degree implied by the method used.

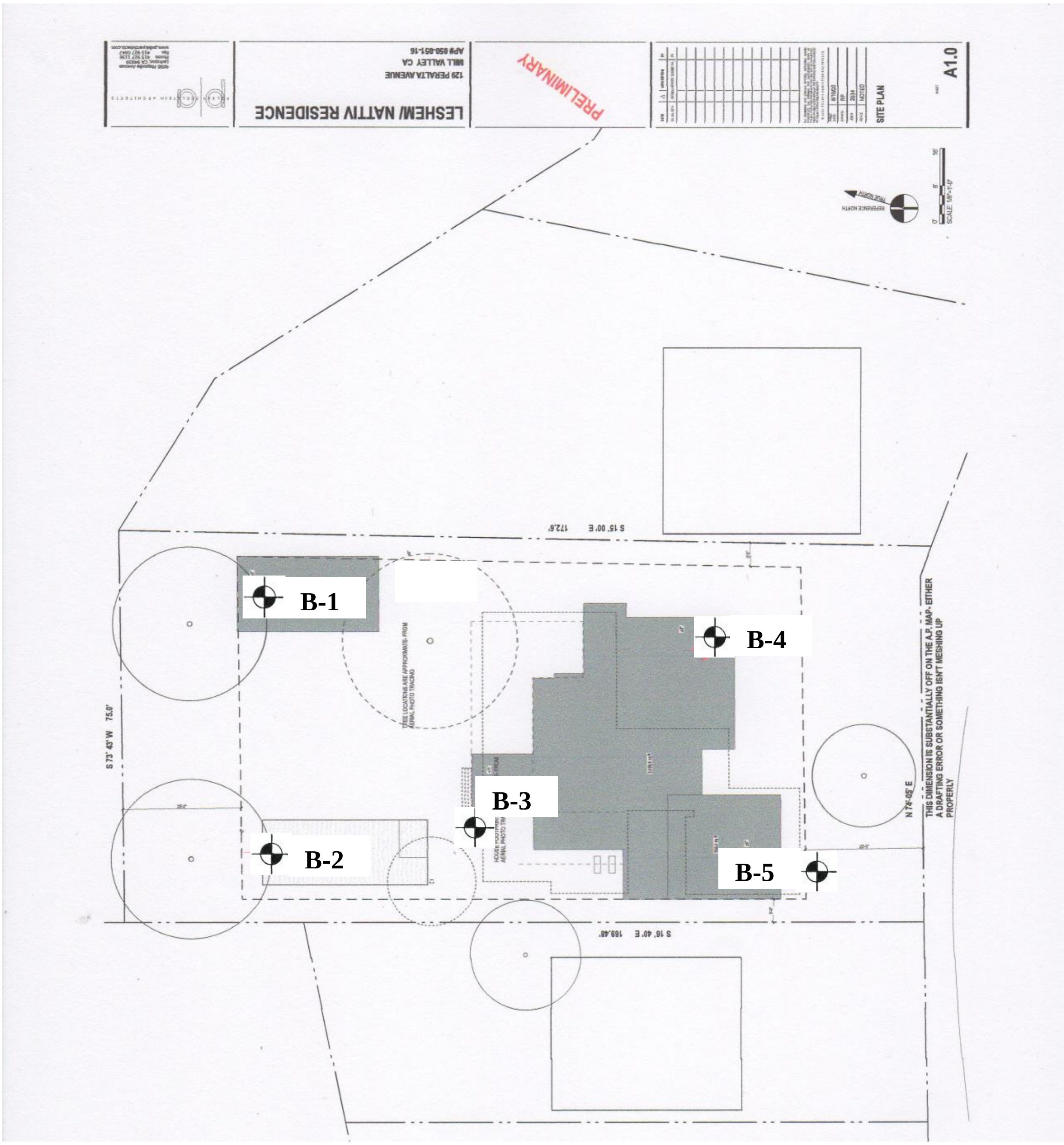
The boring logs show our interpretation of the subsurface conditions on the dates and at the locations indicated and it is not warranted that they are representative of the subsurface conditions at other locations and times. The stratification lines on the borings represent the approximate boundaries between the material types; actual transitions may be gradual.

Laboratory Testing

Water Content And Dry Density

The natural water content and dry density were determined on several samples of the materials recovered from the borings respectively; these are recorded on the boring logs at the appropriate sample depths.

<u>LIST OF PLATES</u>



Base Map by Polsky Perlstein Architects

BORING LOCATION SKETCH **129 Peralta Avenue** **Mill Valley, CA**		**Not to Scale**
Nersi Hemati, P.E., G.E. **Consulting Soil Engineer**	**JOB NO: 2038.20**	**PLATE 1**

BORING LOGS

PROJECT	129 Peralta Ave, Mill Valley	BORING NO: 1
DATE OF BORING	9/2/20	
TYPE OF BORING	4" Augers	
HAMMER WEIGHT	#70	

SAMPLES

DESCRIPTION OF MATERIALS	DEPTH IN FEET	*BLOWS PER FOOT	DRY DENSITY (PCF)	WATER CONTENT (%)	OTHER TESTS
BROWN SANDY CLAY WITH ROOTS (CL)					
stiff, with some gravel		30/11"			
LIGHT BROWN SANDSTONE			hard drilling	9.6	
weathered, fractured (BEDROCK)		30/4"		drilling refusal	
	5-				
	10-				
	15-				
Bottom of Boring 2' 9"					
* Blow counts have been converted to SPT					
No Groundwater Encountered					

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PROJECT	129 Peralta Ave, Mill Valley					BORING NO: 2
DATE OF BORING	9/2/20	SAMPLES				
TYPE OF BORING	4" Augers					
HAMMER WEIGHT	#70					
DESCRIPTION OF MATERIALS		DEPTH IN FEET	*BLOWS PER FOOT	DRY DENSITY (PCF)	WATER CONTENT (%)	OTHER TESTS
BROWN SANDY CLAY WITH YELLOW BROWN SANDSTONE FRAGMENTS (CL)						
stiff	(PARTIALLY FILL)		20			
					8.1	
			harder drilling			
			20			
DARK BROWN SANDY CLAY WITH ROCK FRAGMENTS (CL), stiff, with roots		5-		101	9.4	
BROWN SANDSTONE						
friable, dry	(BEDROCK)					
			25/6"		8.9	
			30/6"			
		10-				
		15-				
Bottom of Boring 8'						
* Blow counts have been converted to SPT						
No Groundwater Encountered						
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Consulting Soil Engineer		JOB NO:	2038.20		PLATE 3	

PROJECT	129 Peralta Ave, Mill Valley	**BORING**	**NO: 3**
DATE OF BORING	9/2/20		
TYPE OF BORING	4" Augers		
HAMMER WEIGHT	#70		

DESCRIPTION OF MATERIALS	**DEPTH IN FEET**	**SAMPLES: *BLOWS PER FOOT**	**SAMPLES: DRY DENSITY (PCF)**	**SAMPLES: WATER CONTENT (%)**	**SAMPLES: OTHER TESTS**
GRAVEL SURFACE					
BROWN TO LIGHT BROWN SANDY CLAY WITH					
ROOTS		35			
stiff					
LIGHT BROWN TO YELLOW BROWN			107	8.6	
SANDSTONE (BEDROCK)					
friable, dry					
		35			
				11.9	
	5-				
	10-				
	15-				
Bottom of Boring 5.5'					
* Blow counts have been converted to SPT					
No Groundwater Encountered					

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Consulting Soil Engineer	**JOB NO:**	2038.20	**PLATE 4**

PROJECT	129 Peralta Ave, Mill Valley				**BORING NO: 4**	
DATE OF BORING	9/2/20	**SAMPLES**				
TYPE OF BORING	4" Augers					
HAMMER WEIGHT	#70					
DESCRIPTION OF MATERIALS		**DEPTH IN FEET**	***BLOWS PER FOOT**	**DRY DENSITY (PCF)**	**WATER CONTENT (%)**	**OTHER TESTS**
GRAVEL SURFACE				hard drilling		
YELLOW BROWN SANDSTONE BEDROCK			25/6"		9.4	
		5-				
		10-				
		15-				
Bottom of Boring 1'						
* Blow counts have been converted to SPT						
No Groundwater Encountered						
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Consulting Soil Engineer		**JOB NO:**	2038.20		**PLATE 5**	

PROJECT	129 Peralta Ave, Mill Valley					BORING NO: 5
DATE OF BORING	9/2/20	SAMPLES				
TYPE OF BORING	4" Augers					
HAMMER WEIGHT	#70					
DESCRIPTION OF MATERIALS		**DEPTH IN FEET**	***BLOWS PER FOOT**	**DRY DENSITY (PCF)**	**WATER CONTENT (%)**	**OTHER TESTS**
3" ASPHALT						
MOTTLED LIGHT BROWN AND BROWN SANDY						
CLAY WITH ROCK FRAGMENTS (CL)			14			
medium stiff	(PARTIALLY FILL)					
MOTTLED LIGHT GRAY AND ORANGE BROWN				94	26.7	
SANDY SILTY CLAY (CL)						
stiff, very moist				firm drilling		
GRAY SHALE	(BEDROCK)		40			
weak, moist						
		5-		116	16.4	
		10-				
		15-				
Bottom of Boring 5.5'						
* Blow counts have been converted to SPT						
No Groundwater Encountered						
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MAJOR DIVISIONS				TYPICAL NAMES
COARSE GRAINED SOILS (MORE THAN HALF IS LARGER THAN #200 SIEVE)	GRAVELS — MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL GRADED GRAVELS, GRAVEL - SAND MIXTURES
			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 SIZE	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, POORLY GRADED SAND - SILT MIXTURES
			SC	CLAYEY SANDS, POORLY GRADED SAND - CLAY MIXTURES
FINE GRAINED SOILS (MORE THAN HALF 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 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		320	(2600)	Unconsolidated Undrained Triaxial
LL	- Liquid Limit (in %)	Tx CU		320	(2600)	Consolidated Undrained Triaxial
PL	- Plastic Limit (in %)	DS		2750	(2000)	Consolidated Drained Direct Shear
PI	- Plasticity Index	FVS		470		Field Vane Shear
G_s	- Specific Gravity	UC		2000		Unconfined Compression
SA	- Sieve Analysis	LVS		700		Laboratory Vane Shear
■	"Undisturbed" Sample	SS	- Shrink Swell			
⊠	Bulk or Disturbed Sample	EXP	- Expansion			
◩	Standard Penetration Test	P	- Permeability			
□	Sample Attempt with No Recovery					

Note: All strength tests on 2.8" or 2.4" diameter sample unless otherwise indicated.

KEY TO TEST DATA

SOIL CLASSIFICATION AND KEY TO TEST DATA 129 Peralta Avenue Mill Valley, CA		
Nersi Hemati, P.E., G.E. Consulting Soil Engineer	JOB NO: 2038.20	PLATE 7