Sigma Prime Geosciences, Inc.
Effective Solutions

August 11, 2020

Greg and Sue Joswiak
736 Arroyo Leon Drive
Half Moon Bay, CA 94019

Re: Geotechnical Report: 2450 Purisima Creek Road, Half Moon Bay.
APN 066-230-050; Sigma Prime Job No. 19-181

Dear Mr. and Mrs. Joswiak:

As per your request, we have performed a geotechnical study for your proposed residence and outbuildings at 2450 Purisima Creek Road in Half Moon Bay, California. The accompanying report summarizes the results of our field study, laboratory testing, and engineering analyses, and presents geotechnical recommendations for the planned structure.

Thank you for the opportunity to work with you on this project. If you have any questions concerning our study, please call.

Yours,

Sigma Prime Geosciences, Inc.

Charles M. Kissick, P.E.

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GEOTECHNICAL STUDY
2450 PURISIMA CREEK ROAD
HALF MOON BAY, CALIFORNIA
APN 066-230-050

PREPARED FOR:
GREG AND SUE JOSWIAK
736 ARROYO LEON DRIVE
HALF MOON BAY, CA 94019

PREPARED BY:
SIGMA PRIME GEOSCIENCES, INC.
332 PRINCETON AVENUE
HALF MOON BAY, CALIFORNIA 94019

AUGUST 11, 2020

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1. INTRODUCTION

We are pleased to present this geotechnical study report for the proposed residence at 2450 Purisima Creek Road in Half Moon Bay, California, at the location shown in Figure 1. The purpose of this investigation was to evaluate the subsurface conditions at the site, and to provide geotechnical design recommendations for the proposed construction.

1.1 PROJECT DESCRIPTION

We understand that you plan to demolish an existing house and construct a new home at 2450 Purisima Creek Road in Half Moon Bay. A large barn, a small horse barn, and a small agricultural housing unit (AHU) and also planned. The existing driveway will be re-routed. Figure 2 shows the approximate location of the proposed house and other buildings. The buildings are expected to be of wood frame construction. Structural loads are expected to be relatively light as is typical for this type of construction.

1.2 SCOPE OF WORK

In order to complete this project we have performed the following tasks:

- Reviewed published information on the geologic and seismic conditions in the site vicinity;
- Geologic site reconnaissance;
- Subsurface study, including 6 soil borings at the site;
- Engineering analysis and evaluation of the subsurface data to develop geotechnical design criteria; and
- Preparation of this report presenting our recommendations for the proposed structure.

2. FINDINGS

2.1 GENERAL

The site reconnaissance and subsurface study were performed on May 8, 2020. The subsurface study consisted of advancing 6 soil borings with hollow stem or flight augers. Borings B-1 through B-6 were advanced to depths of 15 to 26.5 feet. The approximate locations of the borings are shown in Figure 2, Site Plan. The boring logs and the results of laboratory tests are attached in Appendix A.

2.2 SITE CONDITIONS

The property is located on Purisima Creek Road, 2.7 miles inland from Highway 1 in a broad valley. Purisima Creek crosses the property, about 140 feet south of the proposed house site. The creek is incised about 15 feet. The house site is on a gently sloping alluvial terrace, with a gradient of about 6 percent. There is an existing house at the proposed house site on a level building pad. The lower floor of the proposed house will be about 35 feet higher in elevation than the creek bed.

2.3 REGIONAL AND LOCAL GEOLOGY

Based on Brabb et al (1998), The site is underlain by Holocene age colluvium, which is slope wash debris that is derived from the hillside to the north. It is described as firm sand, silt, clay, gravel, and rock debris.

2.4 SITE SUBSURFACE CONDITIONS

Based on the soil borings, the subsurface conditions at the site consist of medium stiff to very stiff clays with small amounts of clayey sand, clayey gravels, and gravelly clays. Sandstone or siltstone bedrock was encountered at depths of 14.5 to 24 feet. The upper clays mostly have high to very high plasticity, with a plasticity index as high as 49.

2.5 GROUNDWATER

Free groundwater was encountered at depths ranging from 11 to 19 feet. Groundwater is not expected to impact the proposed construction.

2.6 FAULTS AND SEISMICITY

The site is in an area of high seismicity, with active faults associated with the San Andreas fault system. The closest active fault to the site is the San Gregorio fault, located about 6 km to the west. Other faults most likely to produce significant seismic ground motions include the San Andreas (8 km to the east), Hayward, Rodgers Creek, and Calaveras faults. Selected historical earthquakes in the area with an estimated magnitude greater than 6-1/4, are presented in Table 1 below.

TABLE 1
HISTORICAL EARTHQUAKES

Date	Magnitude	Fault	Locale
June 10, 1836	6.5[1]	San Andreas	San Juan Bautista
June 1838	7.0[2]	San Andreas	Peninsula
October 8, 1865	6.3[2]	San Andreas	Santa Cruz Mountains
October 21, 1868	7.0[2]	Hayward	Berkeley Hills, San Leandro
April 18, 1906	7.9[3]	San Andreas	Golden Gate
July 1, 1911	6.6[4]	Calaveras	Diablo Range, East of San Jose
October 17, 1989	7.1[5]	San Andreas	Loma Prieta, Santa Cruz Mountains

(1) Borchardt & Toppozada (1996)
(2) Toppozada et al (1981)
(3) Petersen (1996)
(4) Toppozada (1984)
(5) USGS (1989)

2.7 2016 CBC EARTHQUAKE DESIGN PARAMETERS

Based on the 2016 California Building Code (CBC) and our site evaluation, we recommend using Site Class Definition D (stiff soil) for the site. The other pertinent CBC seismic parameters are given in Table 2 below.

Table 2
CBC SEISMIC DESIGN PARAMETERS

S_S	S_1	S_{MS}	S_{M1}	S_{DS}	S_{D1}
2.025	0.702	2.025	null	1.350	null

Because the S_1 value is greater than 0.75, Seismic Design Category E is recommended, per CBC Section 1613.5.6. The values in the table above were obtained from a software program by the Structural Engineers Association of California which provides the values based on the latitude and longitude of the site and the Site Class Definition. The latitude and longitude were measured at 37.4290 and –122.3863, respectively, and were accurately obtained from Google Earth™. These coordinates coincide with location of the main house. Coordinates at the AHU, which is closer to the San Gregorio fault, yielded slightly lower numbers.

3. CONCLUSIONS AND RECOMMENDATIONS

3.1 GENERAL

It is our opinion that, from a geotechnical standpoint, the site is suitable for the proposed construction, provided the recommendations presented in this report are followed during design and construction. Detailed recommendations are presented in the following sections of this report.

Because subsurface conditions may vary from those encountered at the location of our borings, and to observe that our recommendations are properly implemented, we recommend that we be retained to 1) review the project plans for conformance with our report recommendations and 2) observe and test the earthwork and foundation installation phases of construction.

3.2 GEOLOGIC HAZARDS

We reviewed the potential for geologic hazards to impact the site, considering the geologic setting, and the soils encountered during our investigation. The results of our review are presented below:

- Fault Rupture - The site is not located in an Alquist-Priolo special studies area or zone where fault rupture is considered likely (California Division of Mines and Geology, 1974). Active faults are not believed to exist beneath the site, and the potential for fault rupture to occur at the site is low, in our opinion.

- Ground Shaking - The site is located in an active seismic area. Moderate to large earthquakes are probable along several active faults in the greater Bay Area over a 30 to 50 year design life. Strong ground shaking should therefore be expected several times during the design life of the structure, as is typical for sites throughout the Bay Area. The improvements should be designed and constructed in accordance with current earthquake resistance standards.

- Differential Compaction - Differential compaction occurs during moderate and large earthquakes when soft or loose, natural or fill soils are densified and settle, often unevenly across a site. The soils consist of medium stiff to stiff clays minor amounts of clayey sands and gravels to bedrock at a depth of 14.5 to 24 feet. Only Boring B-1 had loose clayey sands, 4.8 feet thick, that will be marginally prone to differential compaction. Our foundation recommendations will mitigate this potential. Therefore, the likelihood of significant damage to the structure from differential compaction is low.

- Liquefaction - Liquefaction occurs when loose, saturated sandy soils lose strength and flow like a liquid during earthquake shaking. Ground settlement often accompanies liquefaction. Soils most susceptible to liquefaction are saturated, loose, silty sands, and uniformly graded sands. Loose, saturated sands were not encountered at the site and are not anticipated, as the borings revealed stiff clays and shallow bedrock below the groundwater surface. Therefore, in our opinion, the likelihood of liquefaction occurring at the site is low.

- Flooding - The site is currently located in an area that is mapped by FEMA as being within the 100-year flood zone. However, the flood hazard map does not include base flood elevations (BFEs) and is clearly not accurate. The flood zone, as mapped, does not follow the creek channel and traverses elevation contours in a way that is not feasible. We have performed a hydrologic study of the watershed to determine the BFE in the area. The study was sent to FEMA for a Letter of Map Amendment (LOMA), and the LOMA was granted. This is a standard procedure that is expected to result in a re-defining of the BFE. Our analysis has resulted in a BFE of 319.5 feet. The proposed floor elevation of the house is 334.75 feet, over 15 feet above the BFE. A LOMA was obtained for a property about 1000 feet upstream of the subject property a few years ago and a similar result was obtained, with the BFE being well below the area depicted in the FEMA map.

3.3 EARTHWORK

3.3.1 Clearing & Subgrade Preparation

All deleterious materials, including asphalt, concrete, topsoil, roots, vegetation, designated utility lines, etc., should be cleared from building and driveway areas. The actual stripping depth required will depend on site usage prior to construction, and should be established by the Contractor during construction. Topsoil may be stockpiled separately for later use in landscaping areas.

3.3.2 Fills

Fills up to 4.5 feet deep are proposed for the main house area. The deeper fill will be in the area of Boring B-2, where shallow medium stiff to stiff clay was encountered. Settlement of the native clay due to the fill load will be negligible. The fills should be placed and compacted as discussed in Section 3.3.3 below. The ground in proposed fill areas should be cleared and scarified, with level benches. The 4.5-foot fill at the north-east corner of the house should include a keyway at the base that is 3 feet wide and 1 foot deep. It should extend from the retaining wall to the walkway.

3.3.3 Compaction

Scarified surface soils should be moisture conditioned to 3-5 percent above the optimum moisture content and compacted to at least 95 percent of the maximum dry density, as determined by ASTM D1157-78. All patio fills, such as at the northeast corner of the main house, should be placed in loose lifts not exceeding 8 inches in height, and compacted to at least 95% of the maximum dry density, as determined by ASTM D1157-78. Other fills in landscaping areas and at the small horse barn may be compacted to at least 92% of the maximum dry density.

3.3.4 Surface Drainage

The finish grades should be designed to drain surface water away from foundations and slab areas to suitable discharge points. For permeable surfaces, slopes of at least 5 percent within 10 feet of the structures are recommended. For impermeable surfaces, slopes of at least 2 percent within 10 feet of the structures are recommended. Ponding of water should not be allowed adjacent to the structure.

3.4 FOUNDATIONS

Due to the nature of the highly expansive soils found on this site, pier-and-grade-beam foundations are recommended for the main house, the large barn, and the AHU. The small horse barn can be founded on spread footings.

Pier and Grade Beam

Piers should be drilled and cast-in-place, and be a minimum of 16 inches in diameter, with the minimum depth determined by the structural engineer.

Per CBC 2016 Section 1705.8, a representative of Sigma Prime shall conform to the following special inspection requirements:

1. Inspect drilling operations and maintain complete and accurate records for each element.
2. Verify placement locations and plumbness, confirm element diameters, bell diameters (if applicable), lengths, embedment into bedrock (if applicable) and adequate end-bearing strata capacity. Record concrete or grout volumes.

The piers may gain support in skin friction acting along the sides of the piers within the lower soils. A skin friction of 500 pounds per square foot (psf) between the piers and the soil should be used in design to calculate the allowable downward capacity. The uplift capacity of the piers may be based on a skin friction value of 350 psf acting below a depth of 4 feet. The skin friction value may be increased by 1/3 for seismic loads and wind loads. Because of the difficulty in cleaning the bottoms of the pier holes, end bearing should be neglected. However, the pier holes should be kept as clean as possible.

Due to the potential for expansion of the upper expansive soils, we recommend that the piers also be designed to resist an uplift force calculated using a skin friction of 1,000 psf acting over the upper 4 feet of the piers. Similarly, grade beams should not rest on soil. To minimize uplift on grade beams, a 4-inch-thick void should be left beneath the bottom of the grade beams. The gap can be filled with compressible material such as cardboard forms or a suitable equivalent. The perimeter grade beams should extend at least 8-inches below the crawl space grade or the building pad soils below the gravel placed for the garage slab.

When concrete is poured into the pier holes, care must be taken to preserve vertical sides to the piers. In other words, the concrete should not be allowed to flow away from the tops of the piers, creating an upside-down bell shape, or mushroom at the top. A bell-shaped pier cap will allow expansive soil to lift the piers upward. Sonotubes can be used to keep a smooth, vertical side to each pier.

Drilled piers should have a center-to-center spacing of not less than three pier diameters. Our representative should be present during pier drilling operations to assure that piers holes are sufficiently deep and that pier holes are kept free of loose soil. Pier excavations should be poured as soon as practical after drilling. If there is water in the pier holes, it should be pumped out prior to pouring concrete, or the concrete should be tremied into the hole, thereby displacing the water. The concrete should not be allowed to free-fall more than 5 feet.

<u>Spread Footings for Horse Barn</u>

The proposed horse barn may be supported on shallow spread footings. Footings should be at least 12 inches wide and should extend at least 24 inches below the lowest adjacent grade. To eliminate uplift forces on the sides of the footings, caused by expansive soil “grabbing” footings and lifting them up as they expand, we recommend that the upper 2 feet of the in-situ soils within 1 foot of the footings be replaced with non-expansive imported fill.

The geotechnical engineer should check the footing excavations to evaluate whether the depth is adequate. The bottoms of footings should be kept wet until concrete is placed, to keep the clays from shrinking due to moisture loss from evaporation. A hose with a low-pressure spray nozzle may be used to keep the soils moist.

Footings should be designed for maximum allowable soil pressures of 2,500 psf. The weight of foundation concrete extending below grade may be disregarded for downward loads.

3.4.1 Lateral Loads

Piers

Resistance to lateral loads may be provided by passive pressure acting against the piers, neglecting the upper 2 feet of the pier, and acting across two pier diameters. We recommend that an equivalent fluid weight of 300 pcf be used to calculate the passive resistance against the upper 8 feet of the piers. No passive resistance should be considered in design below a depth of 8 feet.

Spread Footings

A passive pressure equivalent to that provided by a fluid weighing 300 pcf and a friction factor of 0.3 may be used to resist lateral forces and sliding against spread footing foundations. These values include a safety factor of 1.5 and may be used in combination without reduction. Passive pressures should be disregarded for the uppermost 12 inches of foundation depth, measured below the lowest adjacent finished grade, unless confined by concrete slabs or pavements. However, the pressure distribution may be computed from the ground surface.

3.4.2 Slabs-on-Grade

Slabs-on-grade should be constructed as free-standing slabs, structurally isolated from surrounding grade beams. We recommend that the slab-on-grade be underlain by at least 24 inches of non-expansive fill. The upper 4 inches of this fill should consist of ½- to ¾-inch clean crushed rock. Where floor wetness would be detrimental, a vapor barrier, such as Stego wrap or equivalent may be used.

3.5 RETAINING WALLS

Retaining walls should be designed to resist lateral earth pressure from the adjoining natural soils and/or backfill. We recommend that walls that are restrained from lateral movement be designed to resist an at-rest equivalent fluid pressure of 75 pounds per cubic foot (pcf). Retaining walls that are not restrained from lateral movement should be designed to resist an active equivalent fluid pressure of 60 pcf.

The building code calls for a geotechnical investigation that shall include “a determination of lateral pressures on basement and retaining walls due to earthquake motions.” Some methods still being used, such as the Mononobe-Okabe or the Seed and Whitman methods, include either an inverted triangular distribution or a rectangular distribution for the seismic surcharge pressure. However, recent research indicates that there is no need to include a seismic surcharge pressure if (a) the walls are designed for the at-rest condition, and (b) the conventional factors of safety are applied to the wall design. Furthermore, extensive observations by international teams of seismic experts following recent large earthquakes have not resulted in any documented failures of retaining walls that could be attributed to seismic surcharge pressures.

Based on our current understanding of the state-of-the-practice regarding seismic surcharge pressures, we recommend that (a) no seismic surcharge pressure be used if the walls are designed for the higher at-rest earth pressures, and (b) a uniform (rectangular) seismic surcharge pressure of 10 H psf (where H is the “free” wall height in feet above the finished grade in front of the wall) be used if the walls are designed for the lower active earth pressures.

3.6 CONSTRUCTION OBSERVATION AND TESTING

The earthwork and foundation phases of construction should be observed and tested by us to 1) Establish that subsurface conditions are compatible with those used in the analysis and design; 2) Observe compliance with the design concepts, specifications and recommendations; and 3) Allow design changes in the event that subsurface conditions differ from those anticipated. The recommendations in this report are based on a limited number of borings. The nature and extent of variation across the site may not become evident until construction. If variations are then exposed, it will be necessary to reevaluate our recommendations.

4. LIMITATIONS

This report has been prepared for the exclusive use of the owner for specific application in developing geotechnical design criteria, for the currently planned residence and other buildings at 2450 Purisima Creek Road in Half Moon Bay, California (APN 066-230-050). We make no warranty, expressed or implied, except that our services were performed in accordance with geotechnical engineering principles generally accepted at this time and location. The report was prepared to provide engineering opinions and recommendations only. In the event that there are any changes in the nature, design or location of the project, or if any future improvements are planned, the conclusions and recommendations contained in this report should not be considered valid unless 1) The project changes are reviewed by us, and 2) The conclusions and recommendations presented in this report are modified or verified in writing.

The analyses, conclusions and recommendations contained in this report are based on site conditions as they existed at the time of our investigation; the currently planned improvements; review of previous reports relevant to the site conditions; and laboratory results. In addition, it should be recognized that certain limitations are inherent in the evaluation of subsurface conditions, and that certain conditions may not be detected during an investigation of this type. Changes in the information or data gained from any of these sources could result in changes in our conclusions or recommendations. If such changes do occur, we should be advised so that we can review our report in light of those changes.

5. REFERENCES

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APPENDIX A

FIELD INVESTIGATION

The soils encountered during drilling were logged by our representative, and samples were obtained at depths appropriate to the investigation. The samples were taken to our laboratory where they were carefully observed and classified in accordance with the Unified Soil Classification System. The logs of our borings, as well as a summary of the soil classification system, are attached.

Several tests were performed in the field during drilling. The standard penetration resistance was determined by dropping a 140-pound hammer through a 30-inch free fall, and recording the blows required to drive the 2-inch (outside diameter) sampler 24 inches. The standard penetration resistance is the number of blows required to drive the sampler the last 12 inches of an 18-inch drive. Because the sampler was driven 24 inches instead of 18 inches, the blow counts are a modification of a standard penetration test. Accordingly, we use engineering judgment when evaluating the soils. The results of these field tests are presented on the boring logs.

The boring logs and related information depict our interpretation of subsurface conditions only at the specific location and time indicated. Subsurface conditions and ground water levels at other locations may differ from conditions at the locations where sampling was conducted. The passage of time may also result in changes in the subsurface conditions.

APPENDIX B

LABORATORY TESTS

Samples from the subsurface study were selected for tests to establish some of the physical and engineering properties of the soils. The tests performed are briefly described below.

The natural moisture content and dry density were determined in accordance with ASTM D 2216 on selected samples recovered from the borings. This test determines the moisture content and density, representative of field conditions, at the time the samples were collected. The results are presented on the boring logs, at the appropriate sample depth.

The plasticity of selected clayey soil samples was determined on five soil samples in accordance with ASTM D 422. These results are presented on the boring logs, at the appropriate sample depths.