Sigma Prime Geosciences, Inc.
Effective Solutions

GEOTECHNICAL STUDY

AARON'S BEACH HOTEL
4150 NORTH CABRILLO HIGHWAY
HALF MOON BAY, CALIFORNIA
APN 047-252-280,290

PREPARED FOR:
PAUL McGREGOR
171 CORONADO AVENUE
HALF MOON BAY, CA 94019

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

APRIL 1, 2025

Sigma Prime Geosciences, Inc.
Effective Solutions

April 1, 2025

Paul McGregor
171 Coronado Avenue
Half Moon Bay, CA 94019

Subject: Geotechnical Report: Aaron's Beach Inn, 4150 North Cabrillo Highway, Half Moon Bay. APN 047-252-280 and 047-252-290
Sigma Prime Job No. 24-137

Dear Mr. McGregor:

As per your request, we have performed a geotechnical study for your proposed hotel at 4150 North Cabrillo Highway, 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.

REGISTERED PROFESSIONAL ENGINEER
CHARLES M. KISSICK
No. 62264
9/30/25
EXPIRES
CIVIL
STATE OF CALIFORNIA

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1. INTRODUCTION

We are pleased to present this geotechnical study report for the proposed hotel at 4150 North Cabrillo Highway 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 construct a new hotel at 4150 North Cabrillo Highway in Half Moon Bay. The hotel will be three stories, with a moderate load, and is expected to be of wood frame 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 3 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 by Sigma Prime on April 24, 2024. The subsurface study consisted of advancing 1 soil boring to a depth of 50 feet. An earlier study was performed for a different project that was not built, by ADCO Engineering in 1997. In that study, two soil borings were drilled to depths of 15 and 40 feet. The approximate locations of the three 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

At the time of our study, the site was undeveloped. The lot is mostly level, with an old sea bluff on the west end, leading down to a paved walkway/bike path, a wide beach and the harbor. The sea bluff is about 16 feet tall and inclined at about 50 percent. The harbor is protected by a breakwater. Therefore, wave action cannot reach the base of the sea bluff.

2.3 REGIONAL AND LOCAL GEOLOGY

Based on Brabb et al (1998), the site vicinity is underlain by Holocene age younger (outer) alluvial fan deposits. This unit is described as unconsolidated fine sand, silt, and clayey silt.

2.4 SITE SUBSURFACE CONDITIONS

Based on the soil borings, the subsurface conditions at the site consist of 33 feet of medium stiff to very stiff clay and sandy clay, with dense clayey sand at 33 to 37 feet, very stiff sandy clay at 37 to 44 feet, and dense sand at 44 to 50 feet. The upper clays have very low to moderate expansive potential, with plasticity indices of 6 and 22.

2.5 GROUNDWATER

Groundwater was encountered in the ADCO boring at a depth of 27 feet, and in our boring at a depth of 25 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 1.5 km to the west. Other faults most likely to produce significant seismic ground motions include the San Andreas, 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 2022 CBC EARTHQUAKE DESIGN PARAMETERS

Based on the 2022 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}
2.42	0.90	2.46	2.36	1.64	1.58

Because the S_1 value is greater than 0.75, Seismic Design Category E is recommended, per CBC Section 1613.2.5. 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 37.5022 and –122.4750, respectively, and were accurately obtained from Google Earth™.

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 nil, 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. In our opinion, due to the thick deposit of medium stiff to very stiff clay, the likelihood of significant damage to the structure from differential compaction is low.

- Slope Stability – The building site is level, but as little as 13.5 feet from a sea bluff that is about 16 feet tall and inclined at about 50 percent. There are no signs of slope instability on the sea bluff at the site or anywhere in the area. The sea bluff only tends to degrade gradually via typical coastal erosion processes instead of landslide-type slope failures. Given the stiff clays and the relatively moderate slope, the slope stability is high.

- Coastal Erosion – Because of the breakwater, the sea bluff is never subject to wave attack. There is a wide beach and an area above the beach with a walking/biking path. Coastal erosion is not a factor. There is no evidence of conventional erosion from rain either, due to the vegetation.

Half Moon Bay's Local Coastal Land Use Plan includes a list of issues in Policy 7-12 to address for ocean-front properties. The issues, items a through j, are listed below with our discussions on each issue in bold italic:

a. Historic and projected rates of erosion over the anticipated life span of the proposed development, including potential erosion considering future sea level rise, and possible changes in shore configuration and sand transport. Sources to be investigated include recorded land surveys and tax assessment records, historic maps and photographs where available, and best available science on sea level rise and erosion projections such as that developed by USGS, the National Academy of Engineering, the National Academy of Science, the California Geological Survey, and the California Coastal Commission; ***Most estimates of the extent of sea level rise by the year 2100 range from 1 to 8 feet. NASA gives an estimate of 2 feet. The elevation at the base of the bluff is about 13 feet. A typical scenario of sea level rise is unlikely to result in erosion of the bluff, since the breakwater limits waves heights to lees than 3 feet. Seawater from wave runup, assuming it is capable of reaching the base of the bluff would be infrequent or non-existent and have very low energy.***

b. Cliff geometry and site topography, extending the surveying work beyond the site as needed to depict geomorphic conditions that might affect the site and the proposed development; ***The entire bluff and much of the land beyond the base of the bluff have been accurately surveyed. The gradient of the bluff ranges from 45 to 64 percent. The bluff is heavily vegetated and exhibits no signs of erosion or instability.***

c. Geologic conditions, including soil, sediment and rock types and characteristics in addition to structural features such as bedding, joints, and faults; ***Based on Boring B-3, the bluff is underlain by homogeneous, medium stiff to very stiff sandy clay.***

d. Evidence of past or potential landslide conditions, the implications of such conditions for the proposed development, and the potential effects of the development on landslide activity; ***There is no evidence of past or potential landslide conditions.***

e. Wave and tidal action, including effects of marine erosion on bluffs; ***Waves are not able to reach the base of the bluff, as discussed above.***

f. Ground and surface water conditions and variations, including 100-year riverine flooding and its impact/interaction with bluff erosion and ocean forces at creek mouths and low-lying areas, changes to groundwater resulting from rising sea levels, and hydrologic changes caused by the development (e.g., introduction of irrigation water to the ground- water system; alterations in surface drainage); ***Boring B-3 encountered groundwater at a depth of 25 feet, or about 10 feet below the base of the bluff. Surface runoff will be controlled as per the drainage plan for the project.***

g. Potential effects of seismic forces resulting from a maximum credible earthquake; ***Effects on the bluff during a design earthquake are expected to be negligible, given the relatively gentle topography, good soil conditions, and lack of shallow groundwater.***

h. Effects of the proposed development including siting and design of structures, landscaping, drainage, grading, and impacts of construction activity on the stability of the site and adjacent area; ***The proposed project will have no harmful impact on the bluff. In fact, the project will divert roof runoff away from the bluff and send it to the front of the property. Therefore, there will be less surface runoff going over the bluff, post-project.***

i. Any other factors that may affect slope stability; ***None.***

j. Any other factors that may affect slope stability; and Potential erodibility of site and mitigating measures to be used to ensure minimized erosion problems during and after construction (i.e., landscaping and drainage design). ***Adequate landscaping should be maintained to minimize surface erosion. Bare ground is not recommended on the bluff.***

- Settlement – Total and differential settlements due to building loads are expected to be less than ½-inch and ¼-inch, respectively, due to the pier and grade beam foundation.

- 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. Due to the thick cays and deep groundwater, the likelihood structure damage due to liquefaction is very low.

3.3 EARTHWORK

3.3.1 Clearing & Subgrade Preparation

All deleterious materials, including topsoil, roots, vegetation, designated utility lines, etc., should be cleared from building and parking areas. The actual stripping depth required will depend on site usage prior to construction, and should be established by the Contractor during construction. Conventional earthmoving equipment can be used for all earthwork.

3.3.2 Fills

There are no new fills planned for the site, except for utility trench fills. Compaction is discussed below.

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 in loose lifts not exceeding 6 inches. All trench fills should be placed in loose lifts not exceeding 6 to 8 inches in height, and compacted to at least 92% of the maximum dry density, as determined by ASTM D1157-78.

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

Because of the large size of the building, a pier-and-grade-beam type of foundation is recommended. 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.

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 2 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.

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.

3.4.1 Lateral Loads

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 350 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.

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 4 inches of non-expansive fill such as class 2 baserock. 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 55 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 45 pcf.

The proposed retaining walls may be supported on shallow spread footings, also called T-footings. Footing widths and depths should be determined by the structural engineer, based on our data and recommendations. The geotechnical engineer should check the footing excavations to evaluate whether the depth is adequate.

Footings should be designed for allowable bearing pressures of 2,500 pounds per square foot for dead plus live loads, with a one-third increase allowed for total loads including wind or seismic forces. The weight of foundation concrete extending below grade may be disregarded for downward loads.

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.

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 hotel located at 4150 North Cabrillo Highway in Half Moon Bay. 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

ADCO Engineering, 1997, Soil and Foundation Investigation, 4150 Cabrillo Highway, Half Moon Bay, unpublished report, December 2.

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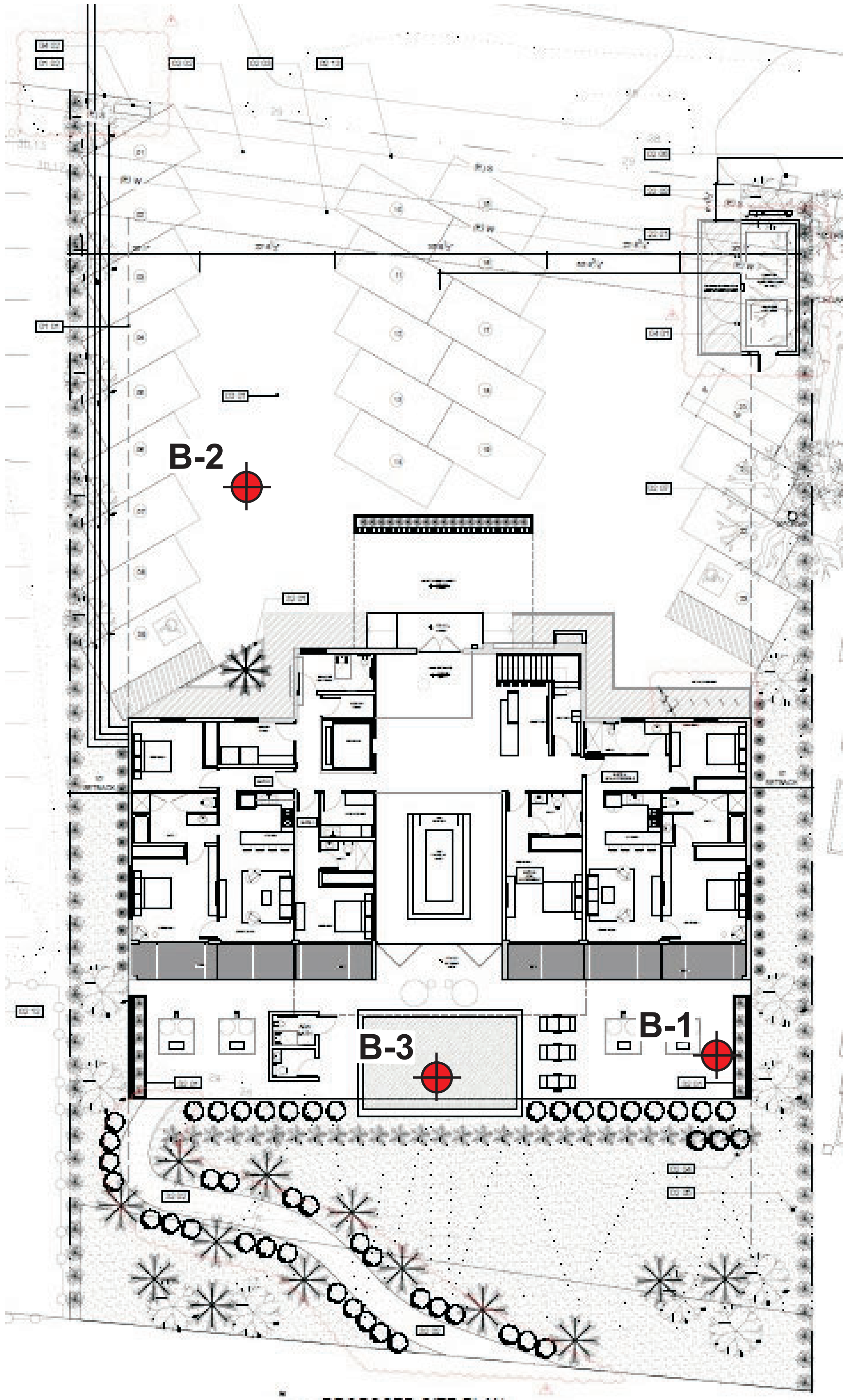
N

0 1000
Scale - feet

Sigma Prime Geosciences, Inc.	Figure 1
	Date: 6-24-24
	Job No.: 24-137

Location Map

4150 North Cabrillo Hwy., Half Moon Bay



EXPLANATION

B-1	Soil Boring, Drilled, 11-20-97
B-2	Soil Boring, Drilled, 11-20-97
B-3	Soil Boring, Drilled, 4-24-24

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Figure	2
Date:	6-24-24
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Site Plan

4150 North Cabrillo Hwy., Half Moon Bay

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.

Project Name	Project Number					
McGregor	24-137					
Location: See Figure 2						
Drilling Method	Hole Size	Total Depth	Soil Footage	Rock Footage	Elevation	Datum
Flight Auger	5“	50’	50’	0	30’	NAVD88
Drilling Company: Cenozoic Drilling			Logged By: C. Kissick			
Type of Drill Rig: Simco 2400 SK-1	Type of Sampler(s): Mod Cal, SPT		Hammer Weight and Fall: 140 lb, 30“			

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Boring No.	B-3
Page	1 of 3
Date(s)	4/24/24

Depth (feet)	Description	Graphic Log	Class	Blow Count	Sample No.	Sample Type	Comments
0	0’-4’: Clay: dark brown; medium stiff; moist.		CL	4 7 10	1	MC	Lab, Sample #1: Moisture%=15.1% Dry Density=113.2 pcf LL=23, PL=17, PI=6
4	4’-33’: Sandy Clay: orange-brown; stiff; moist.						
5				4 5 5	2	SPT	
10	Medium stiff.		CL	3 4 4	3	SPT	
15	Very stiff.			6 7 10	4	SPT	
20	Stiff.			4	5	SPT	(Cont. on next page)

Project Name					Project Number		
McGregor					24-137		
Location: See Figure 2							
Drilling Method	Hole Size	Total Depth	Soil Footage	Rock Footage	Elevation	Datum	
Flight Auger	5"	50'	50'	0	30'	NAVD88	

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Drilling Company: Cenozoic Drilling	Logged By: C. Kissick
Type of Drill Rig: Simco 2400 SK-1	Type of Sampler(s): Mod Cal, SPT
Hammer Weight and Fall: 140 lb, 30"	

Boring No.	**B-3**
Page	2 of 3
Date(s)	4/24/24

Depth (feet)	Description	Graphic Log	Class	Blow Count	Sample No.	Sample Type	Comments
20	4'-33': Sandy Clay: (cont.)		CL	6 8	5	SPT	
25				3 5 6	6	SPT	∇ Groundwater @ 25'
30	Very stiff.			5 11 16	7	SPT	
	33'-37': Clayey Sand w/ Gravel: orange-brown; dense; moist.		SC				
35				17 18 16	8	SPT	
	37'-44': Sandy Clay: grayish brown; very stiff; saturated.		CL				
40				6	9	SPT	(Cont. on next page)

Project Name	Project Number
McGregor	24-137

Location: See Figure 2

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Drilling Method	Hole Size	Total Depth	Soil Footage	Rock Footage	Elevation	Datum
Flight Auger	5“	50’	50’	0	30’	NAVD88

Boring No.	B-3
Page	3 of 3
Date(s)	4/24/24

Drilling Company: Cenozoic Drilling

Logged By: C. Kissick

Type of Drill Rig: Simco 2400 SK-1

Type of Sampler(s): Mod Cal, SPT

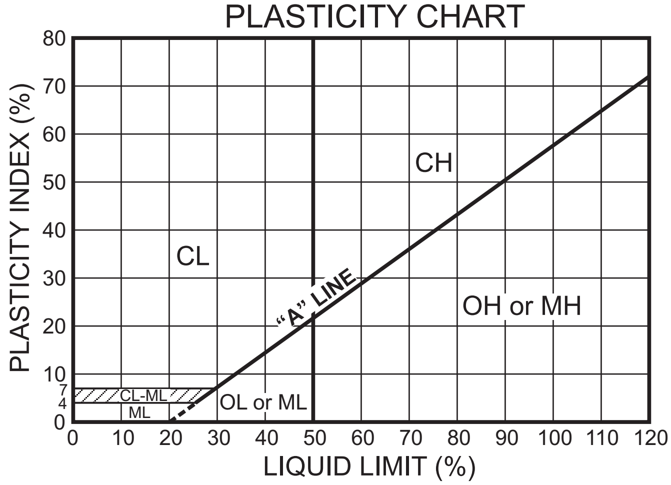
Hammer Weight and Fall: 140 lb, 30“

Depth (feet)	Description	Graphic Log	Class	Blow Count	Sample No.	Sample Type	Comments
40	37’-44’: Sandy Clay: (cont.)		CL	7 11	9	SPT	
	44’-50’: Sand: grayish brown; dense; saturated. Fine, poorly sorted sand.		SP	12 15 18	10	SPT	
45				15 16 17	11	SPT	
50	Bottom of Hole @ 50’ Groundwater encountered @ 25’.						
55							
60							

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-85)

MATERIAL TYPES	CRITERIA FOR ASSIGNING SOIL GROUP NAMES			GROUP SYMBOL	SOIL GROUP NAMES & LEGEND
COARSE-GRAINED SOILS > 50% RETAINED ON NO. 4 SIEVE	**GRAVELS** > 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS < 5% FINES	Cu > 4 AND 1 < Cc < 3	**GW**	WELL-GRADED GRAVEL
			Cu < 4 AND/OR 1 > Cc > 3	**GP**	POORLY-GRADED GRAVEL
		GRAVELS WITH FINES > 12% FINES	FINES CLASSIFY AS ML OR CL	**GM**	SILTY GRAVEL
			FINES CLASSIFY AS CL OR CH	**GC**	CLAYEY GRAVEL
	SANDS > 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN SANDS < 5% FINES	Cu > 6 AND 1 < Cc < 3	**SW**	WELL-GRADED SAND
			Cu < 6 AND/OR 1 > Cc > 3	**SP**	POORLY-GRADED SAND
		SANDS WITH FINES > 12% FINES	FINES CLASSIFY AS ML OR CL	**SM**	SILTY SAND
			FINES CLASSIFY AS CL OR CH	**SC**	CLAYEY SAND
FINE-GRAINED SOILS > 50% PASSING NO. 200 SIEVE	**SILTS AND CLAYS** LIQUID LIMIT < 50	INORGANIC	PI > 7 AND PLOTS > "A" LINE	**CL**	LOW-PLASTICITY CLAY
			PI > 4 AND PLOTS < "A" LINE	**ML**	LOW-PLASTICITY SILT
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	**OL**	ORGANIC CLAY OR SILT
	SILTS AND CLAYS LIQUID LIMIT > 50	INORGANIC	PI PLOTS > "A" LINE	**CH**	HIGH-PLASTICITY CLAY
			PI PLOTS < "A" LINE	**MH**	HIGH-PLASTICITY SILT
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	**OH**	ORGANIC CLAY OR SILT
HIGHLY ORGANIC SOILS		PRIMARILY ORGANIC MATTER, DARK COLOR, ORGANIC ODOR		**PT**	PEAT

PLASTICITY CHART



SAMPLE TYPES

- B — BULK SAMPLE
- ST — PUSHED SHELBY TUBE
- SPT — STANDARD PENETRATION
- MC — MODIFIED CALIFORNIA
- P — PITCHER SAMPLE
- C — ROCK CORE

ADDITIONAL TESTS

CA - CHEMICAL ANALYSIS
CN - CONSOLIDATION
CP - COMPACTION
DS - DIRECT SHEAR
PM - PERMEABILITY
PP - POCKET PENETROMETER
Cor. - CORROSIVITY
SA - GRAIN SIZE ANALYSIS
(20%) - (PERCENT PASSING #200 SIEVE
SW - SWELL TEST
TC - CYCLIC TRIAXIAL
TU - CONSOLIDATED UNDRAINED TRIAXIAL
TV - TORVANE SHEAR
UC - UNCONFINED COMPRESSION
WA - WASH ANALYSIS
▽ - WATER LEVEL AT TIME OF DRILLING AND DATE MEASURED
▼ - LATER WATER LEVEL AND DATE MEASURED

LEGEND TO SOIL DESCRIPTIONS

Sigma Prime Geosciences, Inc.

FIGURE A-1

Logged By: J.B.	Exploratory Boring Log	Hole No. B-1

Job No. 97-1168-S1

Dry Density p.c.f.	Moisture Content %	Penet. Resist. Blows/ft.	Unconf. Comp. Strenght, k.s.f.	Direct Shear Test "C" k.s.f.	Direct Shear Test "0" Degree	Sample Number	Depth in Feet	Boring Log	DESCRIPTION
									Dark Brown Silty Clay, Wet, Stiff
									Light Brown sandy silty clay moist, stiff
108.4	18.5	22		0.9	18	1-1	5		
							10		
110.2	17.3	25				1-2	15		More Sand
							20		
107.8	16.9	27				1-3	25		
									Free Water Standing ▽
							30		

Remarks:

Figure 4 – Logs of Test Borings

ADCO ENGR.

Logged By: J.B.	Exploratory Boring Log	Hole No. B-1

Job No. 97-1168-S1

Dry Density p.c.f.	Moisture Content %	Penet. Resist. Blows/ft.	Unfonf. Comp. Strenght, k.s.f.	Direct Shear Test "C" k.s.f.	Direct Shear Test "0" Degree	Sample Number	Depth in Feet	Boring Log	DESCRIPTION
									Light Brown sandy silty clay moist, stiff
							35		
							40		
									Boring terminated @ 40'

Remarks:

Figure 4 — Logs of Test Borings

ADCO ENGR.

Logged By: J.B.	Exploratory Boring Log	Hole No. B-2

Job No. 97-1168-S1

Dry Density p.c.f.	Moisture Content %	Penet. Resist. Blows/ft.	Unconf. Comp. Strenght, k.s.f.	Direct Shear Test "C" k.s.f.	Direct Shear Test "Ø" Degree	Sample Number	Depth in Feet	Boring Log	DESCRIPTION
									Dark Brown Silty Clay, Wet Stiff
109.5	18.9	25				2-1	5		Light Brown Silty Clay, moist stiff
							10		
							15		More Sand text
									Boring terminated @ 15'

Remarks:

Figure 5 – Logs of Test Borings

ADCO ENGR.

PRIMARY DIVISION			GROUP SYMBOL	SECONDARY DIVISION
COARSE GRAINED SOILS — MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS — MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (LESS THAN 5% FINES)	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GRAVEL WITH FINES	GM	Silty gravels, gravel-sand-clay mixtures, non-plastic fines.
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines.
	SANDS — MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (LESS THAN 5% FINES)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES	SM	Silty sands, sand-silt mixtures, non-plastic fines.
			SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS — MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS — LIQUID LIMIT IS 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 silts and organic silty clays of low plasticity.
	SILTS AND CLAYS — LIQUID LIMIT IS GREATER THAN 50%		MH	Inorganic silts, micaceous or diatomaceous 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.

DEFINITION OF TERMS

SANDS AND GRAVELS	BLOWS / FOOT [I]
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

RELATIVE DENSITY

SILTS AND CLAYS	STRENGTH [II]	BLOWS / FOOT [I]
VERY SOFT	0 - 1/4	0 - 2
SOFT	1/4 - 1/2	2 - 4
FIRM	1/2 - 1	4 - 8
STIFF	1 - 2	8 - 16
VERY STIFF	2 - 4	16 - 32
HARD	OVER 4	OVER 32

CONSISTENCY

U.S. STANDARD SERIES SIEVE: 200, 40, 10, 4

CLEAR SQUARE SIEVE OPENINGS: 3/4", 3", 12"

SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

GRAIN SIZES

[I] Number of blows of 140 pound hammer falling 30 inches to drive a 2 inch O.D. (1-3/8 inch I.D.) split spoon (ASTM D-1586).

[II] Unconfined compressive strength in tons/sq. ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D-1586), pocket penetrometer, torvane, or visual observation.

Figure 3 Unified Soil Classification — ADCO ENGR.

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 one soil sample in accordance with ASTM D 422. These results are presented on boring log B-3, at the appropriate sample depth.