Sigma Prime Geosciences, Inc.
Effective Solutions

GEOTECHNICAL STUDY

AARON'S INN
1390 MAIN STREET
MONTARA, CALIFORNIA

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

JANUARY 6, 2022

Sigma Prime Geosciences, Inc.
Effective Solutions

January 6, 2022

Paul McGregor
171 Coronado Avenue
Half Moon Bay, CA 94019

Subject: Geotechnical Report: Aaron's Inn, 1390 Main Street, Montara (APN:036-052-150)   Sigma Prime Job No. 21-187

Dear Mr. McGregor:

As per your request, we have performed a geotechnical study for your proposed hotel at 1390 Main Street, Montara. 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
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1. INTRODUCTION

We are pleased to present this geotechnical study report for the proposed hotel at 1390 Main Street in Montara, 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 22-room hotel at 1390 Main Street in Montara. The building will be three stories, including a parking garage on the lower level. The floor elevation of the parking garage will be at existing grade in the southwest corner, and 15 feet below grade at the northeast corner. The building is 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 2 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 July 27, 2021. The subsurface study consisted of advancing 2 soil borings with continuous drive sampling. Borings B-1 and B-2 were advanced to a depths of 12 and 12.9 feet, respectively. 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

At the time of our study, the site was partially developed with a small house in the southeast corner of the much larger property. The house will be demolished. The rest of the lot is moderately sloped to the west and mostly covered with wild grasses and weeds.

2.3 REGIONAL AND LOCAL GEOLOGY

Based on Brabb et al (1998), the site vicinity is underlain by Pleistocene marine terrace deposits. This unit is described as poorly consolidated and poorly indurated, well to poorly sorted, sand and gravel.

2.4 SITE SUBSURFACE CONDITIONS

Based on the soil borings, the subsurface conditions at the site consist of about 3 feet of loose sandy fill in the northeast corner, underlain by silty sand. AT Boring B-1, the silty sand is loose in the upper 6 feet, then grades to dense at about 10 feet. In Boring B-2, there is no fill. The soil in Boring 2 consists of 2 feet of clayey topsoil over medium dense silty sand that grades to very dense at a depth of 9 feet. There is a hard sandy clay lenes from 7 to 9 feet. The upper sandy clay and the fines in the silty sand have very low plasticity, with plasticity indices of 2 to 6.

2.5 GROUNDWATER

Groundwater was not encountered in either boring. 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 1.3 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 2019 CBC EARTHQUAKE DESIGN PARAMETERS

Based on the 2019 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}
2.182	0.893	2.182	null	1.455	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 37.5423 and –122.5154, 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). Figure 1 indicates that the site is between the special studies zones for the San Andreas fault and the Hermit fault. 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. In our opinion, due to the medium dense to dense nature of the underlying sandy soils, the likelihood of significant damage to the structure from differential compaction is low, provided the foundation recommendations in this report are followed. For most of the building, the uppermost, less dense soil, as well as all the fill material, will be excavated for the parking garage.

- Slope Stability – The site and surrounding areas have gentle topography. The soil is medium dense to dense silty sand. The site and the surrounding area are not in a State-mapped seismically-induced landslide hazard zone. Given the gentle slopes and dense sandy soils, the slope stability is very high.

- 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. Loose, saturated silty sands were not encountered at the site and are not expected at depth. The site and the surrounding area are not in a State-mapped seismically-induced liquefaction hazard zone. The marine terrace deposits are not prone to liquefaction due to age and relatively high density. Therefore, in our opinion, the likelihood structure damage due to liquefaction is low.

3.3 EARTHWORK

3.3.1 Clearing & Subgrade Preparation

All deleterious materials, including foundations, 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. 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 large extent of the building and the large difference in excavation depth for the parking garage, there is a potential for differential settlement if shallow foundations are used. Therefore, we recommend a pier and grade beam foundation. 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 2019 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 400 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 300 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 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.

3.4.2 Slabs-on-Grade

The lower level of the building will be the parking garage and some small rooms, with a slab foundation throughout. Slabs-on-grade should be tied into the grade beams. We recommend that the slab-on-grade be underlain by at least 12 inches of drain rock with a network of perforated pipes to facilitate positive drainage out from beneath the slab. 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. The walls should be founded on drilled piers with the same requirements as those discussed above. 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 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 1390 Main Street in Montara, California (APN 036-052-150). 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.

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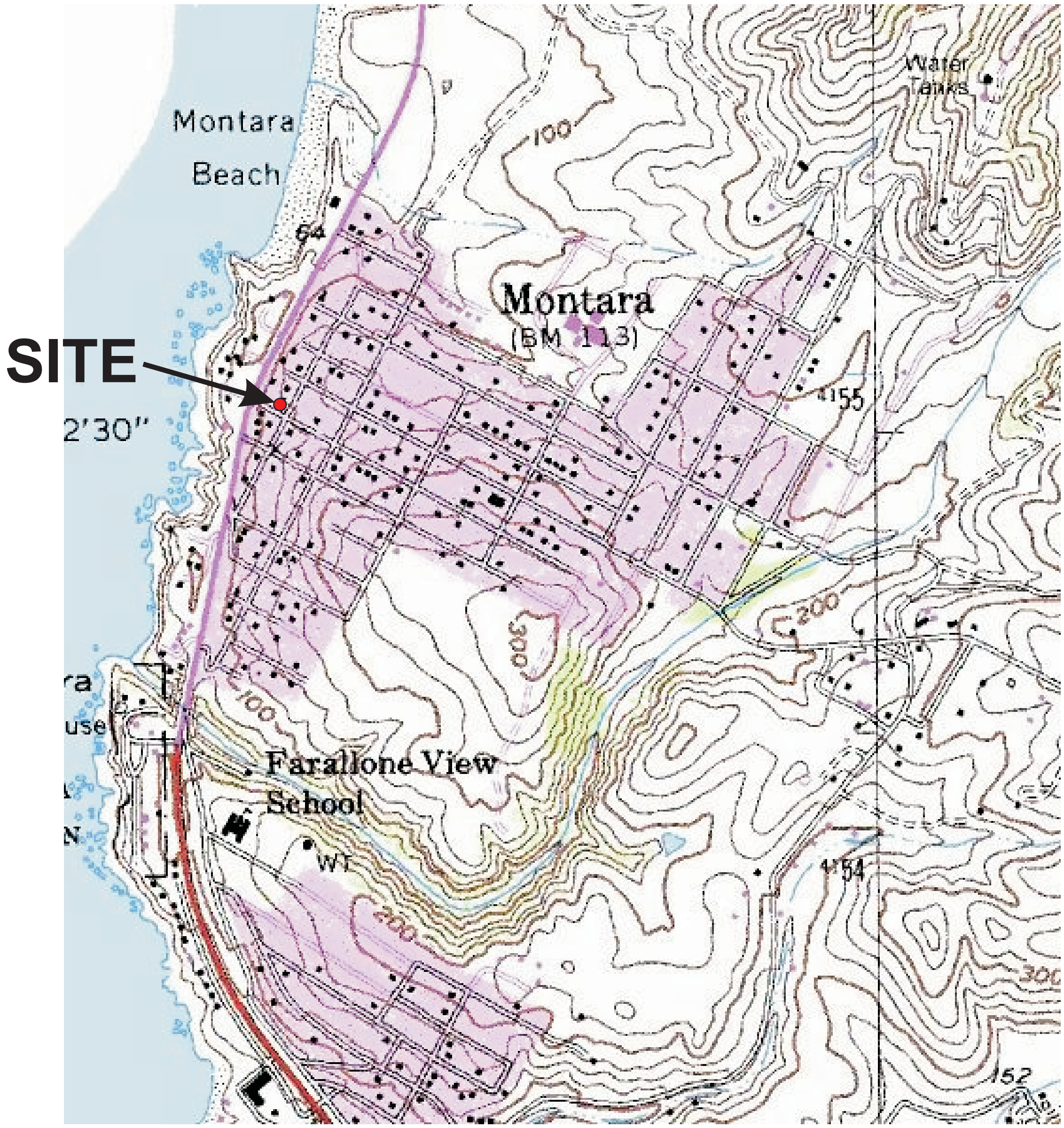

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Figure	1
Date:	1/6/22
Job No.:	21-187

Location Map

Aaron's Inn, 1390 Main Street, Montara

EXPLANATION

B-1 Soil Boring, Drilled, 7/27/21

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Figure	2
Date:	1/6/22
Job No.:	21-187

Site Plan

Aaron's Inn, 1390 Main Street, Montara

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: Aaron's Inn						Project Number: 21-187	Sigma Prime Geosciences, Inc.	
Location: NE Corner of Lot								
Drilling Method	Hole Size	Total Depth	Soil Footage	Rock Footage	Elevation	Datum		
Continuous	4"	12'	12'	0'	130'	NAVD88	**Boring No.**	**B-1**
Drilling Company: Access Soil Drilling				Logged By: CMK			**Page**	1 of 1
Type of Drill Rig:	Type of Sampler(s): Mod Cal, 2½, SPT			Hammer Weight and Fall: 140 lb, 30"			**Date(s)**	7-27-21

Depth (feet)	Description	Graphic Log	Class	Blow Count	Sample No.	Sample Type	Comments
0	0' - 3': Clayey Sand (FILL): moderate brown; loose; moist.		SC	13 11 8 5	1	MC	
	3' - 12': Silty Sand (NATIVE): orange-brown; loose; moist. Very fine to fine grained.		SM	5 6 8 8	2	MC	Lab, Sample #2: Moisture%=10.4% Dry Density=125.0 pcf LL=14, PL=12, PI=2
5				4 5 4 4	3	2½"	
	Medium dense.			5 8 15 19	4	2½"	
	Dense.			10 14 16 19	5	SPT	
10				16 17 19 20	6	SPT	
	Bottom of Hole 12' below ground surface. No groundwater encountered.						
15							
20							

Project Name	Project Number	
Aaron's Inn	21-187	Sigma Prime Geosciences, Inc.

Location: SW Corner of Lot

Drilling Method	Hole Size	Total Depth	Soil Footage	Rock Footage	Elevation	Datum
Continuous	4"	12'-10"	12'-10"	0'	119'	NAVD88

Boring No.	**B-2**
Page	1 of 1
Date(s)	7-27-21

Drilling Company	Logged By
Access Soil Drilling	CMK

Type of Drill Rig	Type of Sampler(s)	Hammer Weight and Fall
	Mod Cal, 2½, SPT	140 lb, 30"

Depth (feet)	Description	Class	Blow Count	Sample No.	Sample Type	Comments
0	0' - 2': Sandy Clay: moderate brown; medium stiff to stiff; moist.	CL	5 7 10 14	1	MC	Lab, Sample #1: Moisture%=7.3% Dry Density=102.4 pcf LL=21, PL=15, PI=6
	2' - 7: Silty Sand: orange-brown; medium dense; moist. Very fine to fine grained.	SM	13 12 15 19	2	MC	
5			13 16 20 21	3	2½"	
			15 16 24 27	4	2½"	
	7 - 9': Sandy Clay: orange-brown; hard; moist.	CL	18 18 23 34	5	SPT	Lab, Sample #5: Sieve: 21% fines 79% sand. (Lower half of sample)
10	7' - 12: Silty Sand: orange-brown; very dense; moist. Very fine to fine grained.	SM	30 30 34 33	6	SPT	
			40 50/4	7	SPT	Refusal.
	Bottom of Hole 12'-10" below ground surface. No groundwater encountered.					
15						
20						

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

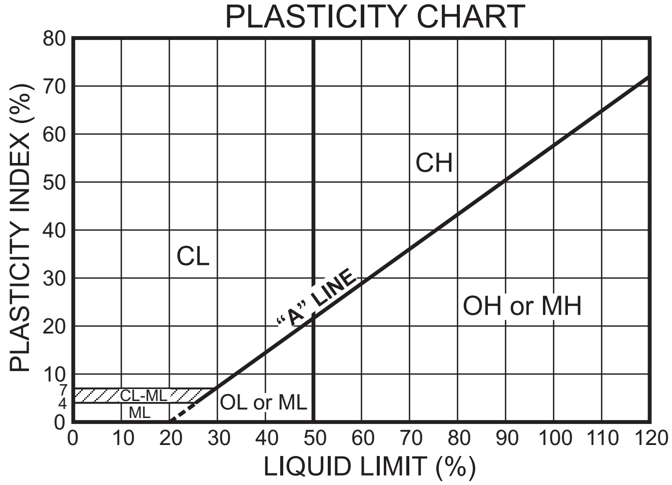
NOTE: $Cu = D_{60}/D_{10}$

$Cc = (D_{30})^2/(D_{10}+D_{60})$

BLOW COUNT

THE NUMBER OF BLOWS OF THE HAMMER REQUIRED TO DRIVE THE SAMPLER THE LAST 12 INCHES OF AN 18-INCH DRIVE. THE NOTATION 50/4 INDICATES 4 INCHES OF PENETRATION ACHIEVED IN 50 BLOWS.

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

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FIGURE A-1

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 two soil samples in accordance with ASTM D 422. These results are presented on the boring logs, at the appropriate sample depths.