Sigma Prime Geosciences, Inc.
Effective Solutions

GEOTECHNICAL STUDY

STEVAUX PROPERTY
PESCADERO CREEK ROAD
PESCADERO, CALIFORNIA
APN 088-090-030

PREPARED FOR:
OLIVIER AND KATIE NOLAN-STEVAUX
340 HAZEL AVENUE
MILLBRAE, CA 94030

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

JANUARY 5, 2024

Sigma Prime Geosciences, Inc.
Effective Solutions

January 5, 2024

Olivier and Katie Nolan Stevaux
340 Hazel Avenue
Millbrae, CA 94030

Re: Geotechnical Report for Proposed Residence: Pescadero Creek Road, Pescadero. APN 088-090-030
Sigma Prime Geosciences Job No. 23-238

Dear Mr. and Mrs. Stevaux:

We have performed a geotechnical study for your proposed residence located at Pescadero Creek Road in Pescadero, 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 residence located on Pescadero Creek Road in Pescadero, California, at the location shown in Figure 1. The purpose of this study 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 single family home on Pescadero Creek Road in Pescadero. 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;
- Subsurface study, including 2 soil borings at the site;
- Laboratory testing of selected soil samples, to establish their engineering properties, and for soil classification purposes;
- 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 December 19, 2023. The subsurface study consisted of drilling 2 soil borings. The borings were advanced to depths 14 and 18 feet. The approximate locations of the borings, numbered B-1 and B-2, are shown in Figure 2. The boring logs and the results of the laboratory tests on soil samples are attached in Appendices A and B, respectively.

2.2 SITE CONDITIONS

At the time of our study, the site was an undeveloped lot. The project site is vegetated with wild grasses and weeds, and trees at the back and front of the property. The building site is relatively level, with a gently sloping driveway down to the house.

2.3 REGIONAL AND LOCAL GEOLOGY

Based on Brabb and Pampeyan (1983), the site vicinity is primarily underlain by Holocene age younger (outer) alluvial fan deposits. The unit is described as unconsolidated fine sand, silt, and clayey silt. The site is also in the vicinity of Holocene age younger (inner) alluvial fan deposits, described as unconsolidated fine to coarse grained sand, silt, and gravel, coarser grained at heads of fans and in narrow canyons

2.4 SITE SUBSURFACE CONDITIONS

Based on the soil borings, the subsurface conditions at the site consist of 8 feet of loose sand and silty sand over stiff clays and medium dense sandy soils. Very dense inner alluvial fan deposits comprised of very dense sandy gravel were encountered at depths of 12.5 to 17 feet.

2.5 GROUNDWATER

Groundwater was not encountered in either boring. Groundwater is not expected to impact the 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 4.5 km to the southwest. 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 C (very dense 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}
1.985	0.725	2.381	1.015	1.588	0.676

Because the S_1 value is less than 0.75, Seismic Design Category D 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 37.2654 and –122.3227, respectively, and were accurately obtained from Google Earth™.

3. CONCLUSIONS AND RECOMMENDATIONS

3.1 GENERAL

It is our opinion that, from a geotechnical viewpoint, 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 and structural calculations 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 Earthquake Fault Zone where fault rupture is considered likely (California Division of Mines and Geology, 1976). Therefore, active faults are not believed to exist beneath the site, and the potential for fault rupture to occur at the site is considered 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 loose sands, the likelihood of some amount of differential compaction is high. We performed a settlement analysis in dry sands, based on Tokimatsu and Seed (1987). After correcting the blow counts to N160 equivalent blow counts, we arrived at total settlement estimates in the two soil borings of 0.6 inches and 1 inch. We conservatively included clayey sands. Therefore, the maximum extent of differential settlement is expected to be less than 1 inch across the length of the house.

- 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 silty sands below a water table were not encountered at the site. Therefore, in our opinion, the likelihood of liquefaction occurring at the site is low.

- Static Settlement –With the proposed foundation type, we expect very little static settlement. Total static settlement should be less than ½-inch, and differential settlement should be less that ¼-inch.

3.3 EARTHWORK

3.3.1 Clearing & Subgrade Preparation

All deleterious materials, including topsoil, roots, vegetation, 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.

After the site has been properly cleared, stripped, and excavated to the required grades, the exposed surface soil in areas to receive a slab-on-grade should be scarified to the depth recommended in Section 3.4.2, moisture conditioned to at least 3-5 percent over optimum moisture content, and compacted to the specifications listed below under the section captioned "compaction."

3.3.2 Compaction

The scarified surface soils should be moisture conditioned to 3-5 percent above the optimum moisture content and compacted to at least 90 percent of the maximum dry density, as determined by ASTM D1557-78. All trench backfill should also be moisture conditioned to 3-5 percent above the optimum moisture content and compacted to at least 90 percent of the maximum dry density. The upper 3 feet of trench backfill below foundations or paved areas should be compacted to 95 percent of the maximum dry density in 6-inch loose lifts.

3.3.3 Surface Drainage

The finish grades should be designed to drain surface water away from foundations and slab areas, to suitable discharge points. On pervious surfaces, such as soil, slopes of at least 5 percent within 10 feet of the structures is required by the building code. The slope can be reduced to 2 percent for impervious surfaces. Ponding of water should not be allowed adjacent to the structure.

3.4 FOUNDATIONS

There is a high potential for seismically-induced settlement, however the extent of differential settlement is expected to be low, as discussed in Section 3.2. However, there is always some uncertainty in settlement analyses based on soil borings. Therefore, we recommend a pier and grade beam foundation with the piers extending down to the very dense gravels.

Piers should be drilled and cast-in-place, and be a minimum of 16 inches in diameter, and should be a minimum of 20 feet deep, as measured from the bottom of the adjacent grade beam. This depth is meant to penetrate the sands that may experience settlement during an earthquake.

The piers may gain support in skin friction acting along the sides of the piers within the lower soils. A skin friction of 350 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 250 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 250 pcf be used to calculate the passive resistance against the piers.

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 base rock.

Where floor wetness would be detrimental, a vapor barrier, such as Stego wrap or equivalent may be used.

3.5 CONSTRUCTION OBSERVATIONS 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 at Pescadero Creek Road in Pescadero, California. 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 study; 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 a study 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

Borchardt, G. and Toppozada, T.R., 1996, Relocation of the "1836 Hayward Fault Earthquake" to the San Andreas Fault, Abstracts, American Geophysical Union Fall Meeting, December, San Francisco.

Brabb, Earl E. and Pampeyan, Earl H., 1983, Geologic Map of San Mateo County, California, USGS Miscellaneous Investigations Series Map I-1257-A, Scale 1:62,500.

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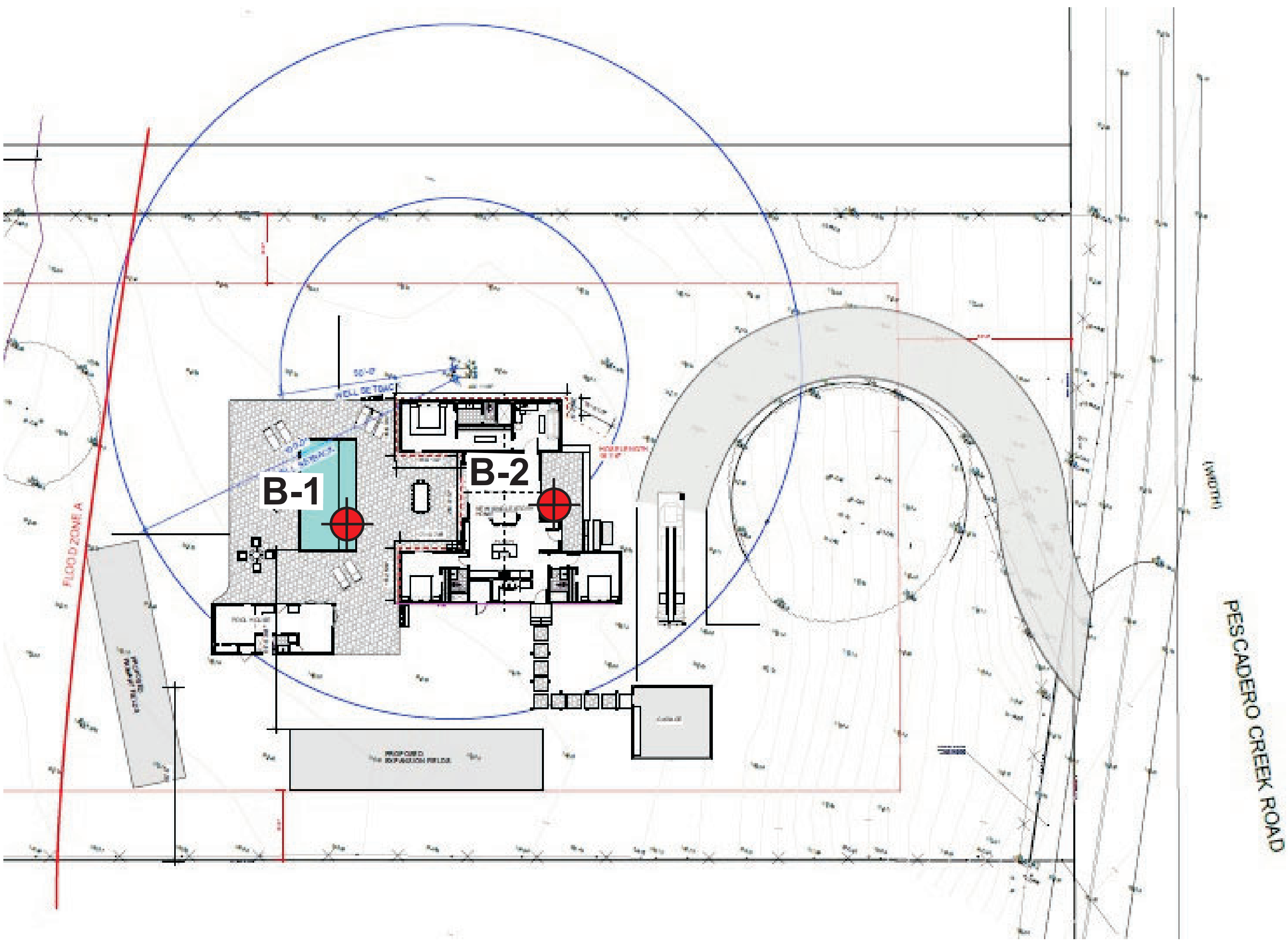
Figure	1
Date:	1/2/24
Job No.:	23-238

Location Map

Stevaux, Pescadero Creek Rd., Pescadero



Note: This is a preliminary design. House, pool, and driveway layout may change.

EXPLANATION

B-1 Soil Boring, Drilled, 12-19-23

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Figure	2
Date:	1/2/24
Job No.:	23-238

Site Plan

Stevaux, Pescadero Creek Rd., Pescadero

APPENDIX A

SUBSURFACE STUDY

The soils encountered during drilling were logged by our representative, and samples were obtained at depths appropriate to the study. The samples were taken to the 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 a standard split spoon sampler the last 12 inches. The blow counts are recorded on the boring logs at the appropriate depth. Use of the standard split spoon sampler defines a Standard Penetration Test (SPT), and yields an SPT-equivalent blow count. A modified California (Mod-Cal) sampler was also used, which results in blow counts that are higher than an SPT-equivalent blow count, due to the Mod-Cal sampler's larger diameter. For analyses, it is normal practice to reduce the Mod-Cal blow counts to correspond to an SPT-equivalent blow count. The blow counts from the Mod-Cal sampler are uncorrected on the logs. The results of these field tests are also 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 groundwater 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.

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Project Name	Project Number		
Stevaux	23-238		
Location: See Figure 2			

Drilling Method	Hole Size	Total Depth	Soil Footage	Rock Footage	Elevation	Datum
Continuous	4“	14’	14’	0’	104’	NAVD88

Boring No.	B-1
Page	1 of 1
Date(s)	12-19-23
Drilling Company	Access Soil Drilling
Logged By	CMK
Type of Drill Rig	
Type of Sampler(s)	Mod Cal, 2½, SPT
Hammer Weight and Fall	140 lb, 30“

Depth (feet)	Description	Graphic Log	Class	Blow Count	Sample No.	Sample Type	Comments
0	0’ - 1’: <u>Sandy Clay</u>: moderate brown; soft; moist.		CL	2 2 4 5	1	MC	Lab, Sample #1: Moisture%=20.2% Dry Density=90.3 pcf
	1’ - 5’: <u>Silty Sand</u>: yellowish brown; loose; moist.		SM	5 5 7 9	2	MC	
				4 6 5 6	3	2½“	
5	5’ - 8’: <u>Sand</u>: tan; loose; moist. Fine sand, poorly sorted.		SP	5 4 4 5	4	2½“	
	8’ - 10’: <u>Sandy Clay</u>: orange-brown; soft; moist. Very thin laminations.		CL	3 4 4 6	5	SPT	
10	10’ - 12.5’: <u>Sand</u>: tan; loose; moist. Fine sand, poorly sorted.		SP	3 4 5 8	6	SPT	
	12.5’ - 14’: <u>Sandy Gravel</u>: gray; very dense; moist.		GP	7 26 26 26	7	SPT	
	Bottom of Hole 14’ below ground surface. No groundwater encountered.						
15							
20							

Project Name	Stevaux	Project Number	23-238
Location	See Figure 2		

Sigma Prime Geosciences, Inc.

Drilling Method	Hole Size	Total Depth	Soil Footage	Rock Footage	Elevation	Datum
Continuous	4“	18’	18’	0’	106’	NAVD88

Boring No.	B-2
Page	1 of 1
Date(s)	12-19-23

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	Graphic Log	Class	Blow Count	Sample No.	Sample Type	Comments
0	0’ - 1’: Sandy Clay: moderate brown; soft; moist.		CL	5 8 8 8	1	MC	
	1’ - 8’: Silty Sand: yellowish brown; loose; moist.		SM	6 6 7 7	2	MC	
5				4 4 3 4	3	2½“	
				4 4 4 4	4	2½“	
	8’ - 11’: Sandy Clay: moderate brown; stiff; moist.		CL	3 4 5 6	5	2½“	
10				3 6 12 13	6	SPT	
	11’ - 12.5’: Sand: moderate brown; medium dense; moist.		SP				
	12.5’ - 17’: Clayey Sand: moderate brown; medium dense; moist.		SC	11 7 7 7	7	SPT	
15				3 5 7 8	7	SPT	
	17’ - 18’: Sandy Gravel: gray; very dense; moist.		GP	5 12 14 27	7	SPT	
	Bottom of Hole 18’ below ground surface. No groundwater encountered.						
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: $Cu = D_{60}/D_{10}$

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

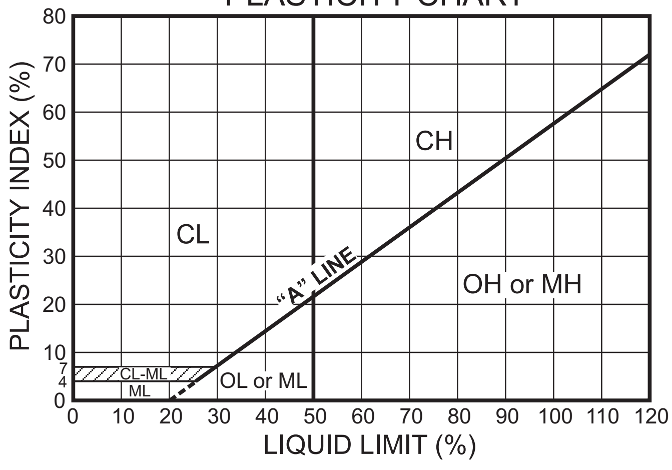
SAMPLE TYPES

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

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



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.