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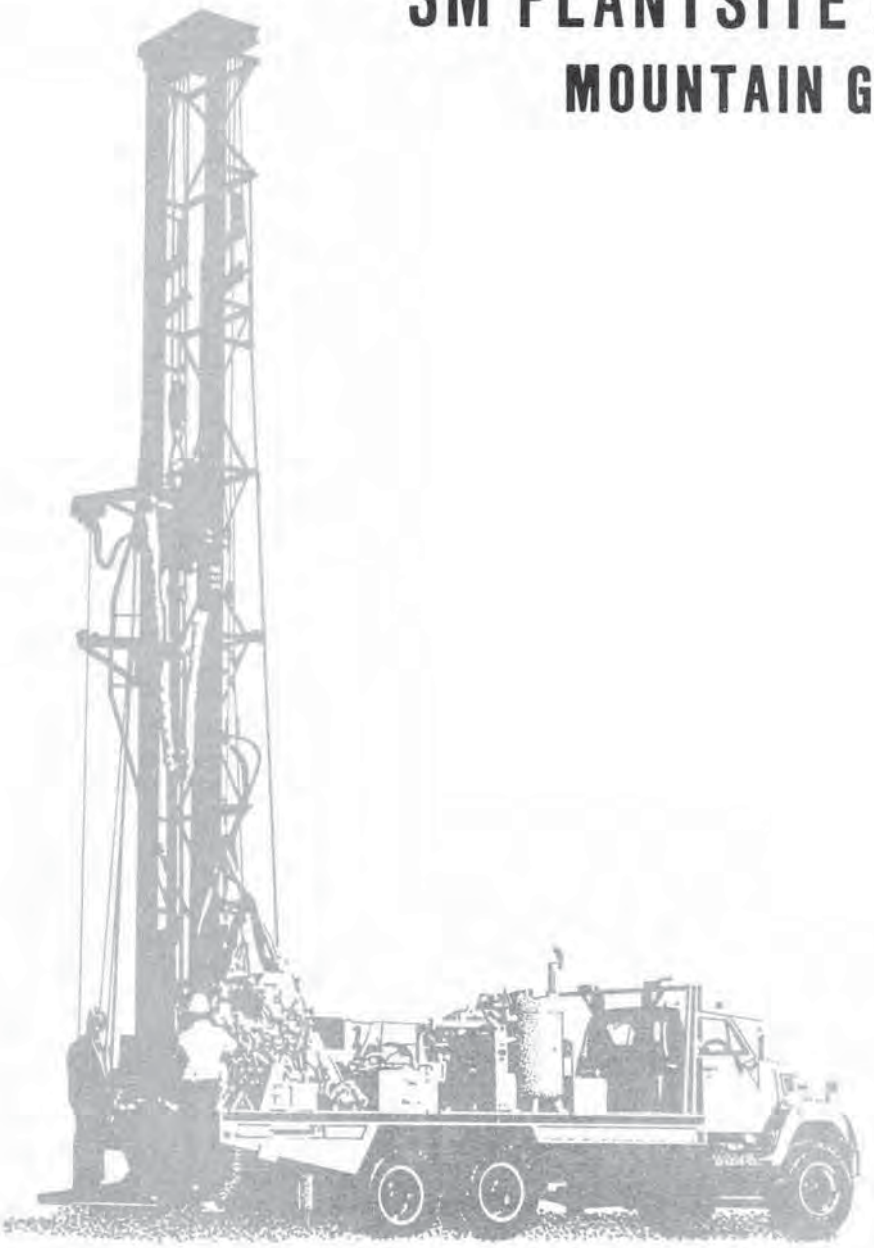
3M PLANTSITE DEVELOPMENT

MOUNTAIN GATE, CALIFORNIA



CH2M HILL

March 1981

CH2M HILL

1525 Court Street P.O. Box 2088, Redding, California 96001, Telephone: 916/243-5831

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TO The 3M Corporation
P.O. Box 3331
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DATE March 25, 1981

RE: 3M PLANTSITE DEVELOPMENT, MTN. GATE, CA

ATTN: Mr. James W. Stoker, Manager
Real Estate and Geology

OUR PROJECT NO. R14202.A5

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The 3M Corporation
P.O. Box 3331
St. Paul, Minnesota 55133

Attention: Mr. James W. Stoker, Manager
Real Estate and Geology

Gentlemen:

Subject: Preliminary Geotechnical Exploration
3M Plantsite Development
Mountain Gate, California

We are pleased to present the results of our preliminary geotechnical exploration for the proposed 3M plantsite development near Mountain Gate, California. This report summarizes our field and laboratory work and presents our preliminary geotechnical design and construction recommendations.

We appreciate the opportunity to work on this interesting project and look forward to the opportunity for future involvement during design and construction. If there are any questions concerning this report, please call.

Sincerely,

Douglas W. Lovell

Douglas W. Lovell
Geotechnical Engineer

Richard L. Quine

Richard L. Quine
Staff Engineering Geologist

vjb
Enclosure (1)

Redding Office
1525 Court Street, P.O. Box 2088, Redding, California 96099 916/243-5831

PRELIMINARY GEOTECHNICAL EXPLORATION

3M PLANTSITE DEVELOPMENT

MOUNTAIN GATE, CALIFORNIA

Prepared for

3M
ST. PAUL, MINNESOTA

Prepared By

CH2M HILL
1525 Court Street
P.O. Box 2088
Redding, California 96099

March 1981

R14202.A5

PREFACE

The preliminary geotechnical exploration was initiated under Contract Redding-04-S, Office Rider 0-1, as authorized by Mr. James W. Stoker in his letter of August 19, 1980. The agreement was subsequently revised by a lump sum activating letter under Contract Redding-04-S, as authorized by Mr. Stoker in his letter of December 11, 1980. The content and requirements of the exploration were detailed in Mr. Stoker's request for proposal dated June 20, 1980, CH2M HILL's proposal dated July 2, 1980, and CH2M HILL's request for additional fees dated November 18, 1980.

A draft of this report was sent to Mr. James W. Stoker on January 1, 1981, for review and comment. Comments were subsequently received in Mr. Stoker's letter of January 21, 1981. The final report either addresses or incorporates all of these comments.

We acknowledge and appreciate the 3M staff involved with the exploration, in particular, Mr. James W. Stoker, whose guidance and comments were extremely useful. Without such interaction between 3M and CH2M HILL, the exploration would not have been tailored to the needs and concerns of the Mountain Gate Project.

SUMMARY

SUMMARY

No subsurface conditions were encountered which preclude the proposed plantsite development; however, some potential problem soils were encountered. With continued geotechnical input during design and construction, economical solutions can be found for these soils.

The subsurface conditions encountered during the preliminary geotechnical exploration typically consist of:

- Soils of mixed coarse- and fine-grained texture, medium to stiff or medium to dense in consistency
- Very dense or hard material at variable depths
- Ground water at depths of 20 feet or greater

Potential problem soils encountered during the preliminary geotechnical exploration include:

- High plasticity, primarily fine-grained soils; these soils are potentially expansive, soft, and compressible, and are present in localized areas at various depths.
- Low resistivity, primarily sandy soils; these soils are potentially corrosive to buried steel (not driven steel piles, however), and are present in localized areas at various depths.

CONTENTS

Chapter		Page
	Preface	ii
	Summary	iii
1	Introduction	
	Purpose and Scope	1-1
	Project Site and Description	1-2
2	Geology and Seismicity	
	Geology	2-1
	Seismicity	2-2
3	Exploration, Testing, and Subsurface Conditions	
	Field Exploration	3-1
	Laboratory Testing	3-1
	Summary of Subsurface Conditions	3-5
4	Conclusions	
	Plantsite	4-1
	Railroad Spur	4-3
	Access Road	4-4
	Overpass	4-6
	Expansive Soils	4-7
	Corrosion	4-8
	Seismicity	4-9
	Topsoil for Landscaping Purposes	4-9
	Construction Considerations	4-9
	Bibliography	

Appendix

A	Figures
B	Exploration and Testing Techniques
C	Boring and Test Pit Coordinates and Elevations

TABLES AND FIGURES

Table		Page
1	Structure Classifications	1-3
2	Significant Regional Faults with Quaternary Displacement	2-4
3	Classification Test Results	3-2
4	Engineering Parameter Test Results	3-5

Figure (See Appendix A)

1 Site Plan
2 Test Pit and Boring Legend
3 Plantsite Test Pit Logs
4 Plantsite Boring Logs
5 Westside Access Area Test Pit Logs
6 Overpass Boring Logs
7 Eastside Access Road Test Pit Logs
8 Eastside Access Road Boring Logs
9 Plasticity Chart
10 Grain Size Analysis
11 Grain Size Analysis
12 Grain Size Analysis
13 Grain Size Analysis
14 Grain Size Analysis
15 Grain Size Analysis
16 Grain Size Analysis
17 Grain Size Analysis
18 Grain Size Analysis
19 Compaction Test
20 Compaction Test
21 Compaction Test
22 Consolidation Test
23 Depth to Very Dense or Hard Material
24 Cut-Fill Transition
25 Expansion Potential

Chapter 1
INTRODUCTION

Chapter 1
INTRODUCTION

PURPOSE AND SCOPE

The purpose of our preliminary geotechnical exploration was to identify and evaluate subsurface conditions for the proposed plantsite development. Special attention was devoted to locating "problem" subsurface conditions (i.e., soft, expansive, corrosive, and organic soils). We understand our report will be used primarily in assisting site layout and preliminary design.

Our scope of work included:

- Review of literature pertaining to geology, seismicity, and previous foundation explorations in the project vicinity
- Site reconnaissance
- Subsurface exploration using borings and backhoe-excavated test pits
- Laboratory testing of materials recovered from the borings and test pits
- Analysis of earthwork, foundation, and pavement alternatives
- Preliminary geotechnical design and construction recommendations

The preliminary geotechnical exploration was performed in accordance with accepted standards of practice to aid in the design and construction of this project. We are not responsible for any other use of this report. The analyses and recommendations are based partly on information obtained from borings, test pits, and laboratory tests which necessarily reflect subsurface conditions only at specific locations. Variations in subsurface conditions may not be reflected. In addition, the passage of time may alter the conditions observed in this exploration, especially with respect to ground water.

The preliminary geotechnical exploration was performed with the understanding that additional work will be performed prior to final design. If variations in subsurface conditions from those described in this report are encountered during subsequent geotechnical exploration or during construction, CH2M HILL should be notified so that the recommendations in this report may be reevaluated.

PROJECT AND SITE DESCRIPTION

3M proposes a project to quarry and process greenstone for the manufacture of roofing granules. This exploration does not address the quarrying or manufacturing aspects of the overall project, but is concerned only with plantsite development. The plantsite development consists of four components--the plantsite itself, a railroad spur, an access road, and an overpass. These components are described below.

Plantsite

The proposed plantsite location is in a meadow surrounded on the south, west, and north by hills (see Figure 1). The meadow is flat to gently rolling, sloping southeast at approximately 20:1 (horizontal to vertical). The meadow is bordered on the east and west by natural drainage courses (branches of Moody Creek) which converge immediately south of the plantsite. The meadow is primarily covered with grasses, manzanita, and scrub oak. There is an orchard of fig trees near the southwest corner of the meadow, indicating that the meadow was used for agriculture in the past.

Specific structures have not yet been identified for the proposed processing plant. In order to provide preliminary foundation recommendations, we have assumed the plant will consist of the classes of structures shown in Table 1. The plant will also contain conveyance, parking, and other appurtenant facilities. We understand maximum cuts of 10 feet and maximum fills of 10 feet are anticipated.

Railroad Spur

We understand the majority of the finished product will be shipped via railroad. A railroad spur is, therefore, proposed from the plantsite to the approximate beginning of the tangent section of the existing Southern Pacific tracks (see Figure 1). The railroad spur will rise slightly in elevation from the existing tracks to the plantsite, at a fairly uniform grade. A maximum cut of approximately 10 feet and a maximum fill of approximately 20 feet are required to provide this grade. The railroad spur transverses four drainage courses in hilly terrain. Existing vegetation consists of scrub oak and manzanita.

Access Road

We understand the access road is to be used primarily for employee and construction traffic. Primary access to the site will be from Interstate 5, east of the plantsite. The access road will follow an existing pioneer road (see Figure 1) from a frontage road paralleling Interstate 5 to a

Table 1
STRUCTURE CLASSIFICATIONS[a]
3M PLANTSITE DEVELOPMENT
MOUNTAIN GATE, CALIFORNIA

Class	Average Structure Pressure (ksf)[b]	Maximum Concentrated Load (kips)	Example	Structural Frame	Comments
Heavily-loaded	>2.5±	>150±[c]	Crushers, processing equipment	Steel or reinforced concrete	Possible dynamic loads
Moderately-loaded	1±	100±	Maintenance and storage	Steel or reinforced concrete	
Lightly-loaded	<0.5±	<50±	Office	Timber, masonry, or reinforced concrete	

[a]Table developed by CH2M HILL.

[b]Average structure pressure $= \frac{\text{Total structure load}}{\text{Total plan area}}$

[c]According to 3M, the maximum concentrated load is approximately 320 kips.

proposed overpass of the existing Southern Pacific tracks. From this point, the road will follow a route approximately parallel to the railroad spur to the plantsite.

This alignment crosses a flat to gently rolling area in the first 2,000 feet adjacent to Interstate 5. This area contains one drainage course, is vegetated by grasses and shrubs, and slopes southeast at approximately 20:1 or flatter. Little cut or fill is anticipated for this area.

The remaining alignment is in hilly terrain, and crosses several natural drainage courses. The vegetation is primarily scrub oak, pine, and shrubs, and maximum existing slopes approach 1:1. Maximum cut and fill heights of 20 feet are anticipated.

Overpass

We understand 3M prefers that the access road cross the existing Southern Pacific tracks via a proposed overpass (versus an underpass or at-grade crossing). Based on a visual examination of possible overpass locations, 3M indicated the location shown on Figure 1 is acceptable. This location provides the maximum relief (30 to 35 feet) within the existing track limits shown. We understand the overpass will be a 1- or 3-span structure, probably constructed of reinforced concrete.

The existing railroad cut slopes at this location are 1:1 on the south and 1-1/2:1 overall on the north side of the tracks. The north slope originally had a bench approximately halfway up, with the slopes above and below cut steeper than 1:1.

Local slump failures have occurred on the north side near the western end of the existing slope. These local failures are probably a result of oversteepening near the bench.

Chapter 2

GEOLOGY AND SEISMICITY

Chapter 2
GEOLOGY AND SEISMICITY

GEOLOGY

Regional Geology

The proposed plantsite development is located in the eastern Paleozoic belt of the Klamath Mountains geologic province. It is also located very near the extreme northern end of the Great Valley province.

The eastern Paleozoic belt contains a diverse assemblage of sedimentary, metamorphic, and plutonic rocks ranging in age from Ordovician to Jurassic. In general, these rocks are an eastward-dipping homoclinal sequence. Formations that underlie various portions of the project include the Copley Greenstone, Kennett(?)[1], Bragdon(?), and Baird(?) Formations.

Bordering the project to the southeast is the Great Valley geologic province. The Great Valley is a northwest-trending geosyncline. It has a complex geologic history involving four episodes of tectonism, and marine and nonmarine sedimentation from the late Jurassic to Recent time. Marine siltstone and sandstone of the Cretaceous Chico Formation are the oldest rocks in the northern Great Valley sequence. Overlying the Chico in the north valley are the Plio-Pleistocene Tehama and Pleistocene Red Bluff Formations.

Local Geology

Copley Greenstone

The Copley Greenstone, of probable middle Devonian age, is the oldest unit underlying the project site. This formation consists of metamorphosed interlayered volcanic flows, pillow lavas, and pyroclastic flows and tuffs, all generally of intermediate composition. Most of the Copley is keratophyre with a characteristic greenish color and aphanitic, foliated, or schistose texture.

---

[1] Marine shale and mudstone encountered during our exploration belong to the Kennett, Bragdon, or Baird Formations. Without geologic field mapping or a petrographic examination of the samples, we cannot state with certainty which formation(s) is/are present.

Devonian to Mississippian Sedimentary Rocks

According to geologic mapping, the Copley Greenstone is overlain by the Kennett, Bragdon, and Baird Formations, characterized as follows:

Kennett Formation:	Dark, thin-bedded siliceous mudstone and shale, tuffaceous shale, and limestone
Bragdon Formation:	Dark greenish-gray to black shale (weathers to buff or brown color)
Baird Formation:	Pyroclastic rocks, mudstone, and keratophyre flows

Alluvial and Colluvial Deposits

Pleistocene to Recent deposits of alluvial and colluvial origin are present over much of the site. Colluvium is present on the sides and near the base of the nearby hills. Alluvial and colluvial deposits generally consist of mixed coarse- and fine-grained soils and are characterized by the presence of angular and subangular gravel.

Fluvial Deposits

Recent deposits of fluvial origin are present within the major drainage courses crossing the site. These deposits are generally well-graded unconsolidated mixtures of sand and gravel with various amounts of cobbles.

SEISMICITY

Our evaluation of seismicity for the project site included the following:

- Review of fault maps and geologic references to determine location and activity of faults within 160 kilometers (100 miles) of the project site
- Search of pertinent references for historic earthquakes in the surrounding region
- Search of NOAA's Earthquake Data File for instrumentally recorded earthquakes within a 100-kilometer (60-mile) radius of the site having magnitudes greater than 2.5 (records date back to 1901)
- Analysis of the above information to assess the overall site seismicity

- Preliminary analysis of soil-liquefaction potential

Capable Faults

Our review of geologic and fault mapping for this area did not reveal any faults crossing the site. Numerous faults have been mapped in the Klamath Mountains, Cascade Volcanics province, and Coast Ranges in the region surrounding the project site. It is significant, however, that the Coast Range and Klamath Mountains faults within about 120 kilometers (75 miles) of the site originated under a set of tectonic forces very different from the ones present today. Although many of these faults are relatively close to the site, they have no known Quaternary movement and are considered inactive.

The Cascade Volcanics region, on the other hand, contains a great number of faults (especially near Mt. Lassen) that do have mapped Quaternary displacements. Because they are associated with volcanic activity (the recurrence of which is not predictable), it is difficult to assess the potential activity and seismic hazard associated with these faults. Since most are located at greater than 140 kilometers (90 miles) from the site, a magnitude 6.5 or larger event would have to occur to generate a bedrock acceleration at the site of 5 percent of gravity (g) or greater. Existing evidence suggests these faults are not capable of generating an earthquake of such magnitude.

Table 2 presents a partial list of faults in the region surrounding the project that have known Quaternary displacements. The far right-hand column shows the site bedrock accelerations that would be associated with the maximum credible earthquake on these faults. Because of the fault locations and attenuation factors, none of these potentially active faults are considered capable of generating site accelerations greater than 5 percent g.

Historic Seismicity

The largest recorded earthquake was a magnitude 5.5 event in 1950 near Mt. Lassen. None of the recorded events were large enough or close enough to the site to have produced a significant (greater than 5 percent g) site acceleration.

Liquefaction

Our literature review discovered no records of liquefaction at or near the site.

Table 2
SIGNIFICANT REGIONAL FAULTS
WITH QUATERNARY DISPLACEMENT
3M PLANTSITE DEVELOPMENT
MOUNTAIN GATE, CALIFORNIA

Fault Name	Distance (miles)/ Direction	Maximum Credible Earthquake (Richter Magnitude)	Maximum Credible Bedrock Site Acceleration (%g)
Battle Creek	25/SE	5.0(?)[a]	<5
Hatchet Mountain	36/NE	5.0(?)	<5
Hat Creek Area (Numerous Faults)	50/NE	5.0(?)	<5
Mt. Lassen Volcanic Activity	50/SE	6.0(?)	<5
Likely	90/NE	7.5	<5
Honey Lake	100/SE	7.25	<5
Freshwater	84/SW	7.25	<5
San Andreas	102/SW	8.25	<5

[a]Question mark indicates the maximum credible earthquake is not quoted in the literature, but has been estimated based on fault length to magnitude relationships (Bonilla, 1970).

Chapter 3

EXPLORATION, TESTING, AND SUBSURFACE CONDITIONS

Chapter 3
EXPLORATION, TESTING, AND SUBSURFACE CONDITIONS

FIELD EXPLORATION

Our field exploration consisted of a site reconnaissance, excavation of 46 backhoe test pits, and drilling 16 soil borings. Ten borings were drilled and 24 test pits excavated in the plantsite area. The area of the railroad spur and access road on the west side of the Southern Pacific tracks was explored with 15 test pits. The access road area on the east side of the tracks was explored with seven test pits and four borings. The overpass location was explored with two borings. The boring and test pit logs are shown on Figures 2 through 8, and locations are shown on Figure 1 and summarized in Appendix C. A detailed description of the field exploration is presented in Appendix B.

Some of the soil encountered in our borings and test pits was residual soil (derived from in-place weathering of underlying rock materials, not transported and deposited). A gradual transition from soil to rock with no distinct line of demarcation was observed in this residual profile; however, a distinct line has been shown on the logs for presentation purposes. The test pits generally show rock at shallower depths because visual examination of the test pit sidewalls allows identification of the relic rock structure. This structure is generally obscured in the disturbed samples obtained in the borings.

Surface soil containing organic material (topsoil) was encountered in some locations. This soil was generally less than 1 foot thick and is not shown on the logs because of its minor thickness. Chapter 4 contains a discussion of the quantity and location of topsoil for landscaping purposes.

All natural drainage courses were dry at the time of our exploration.

LABORATORY TESTING

Samples obtained during the field exploration were taken to CH2M HILL's Civil Engineering Laboratory in Redding, California. Representative samples were selected for classification tests consisting of natural moisture content, Atterberg limits, and grain size distribution. Classification test results are summarized in Table 3 and presented in Figures 9 through 18.

Testing to determine engineering parameters for preliminary design was then performed on selected samples. Expansion, corrosion, R-value, strength, compaction, specific gravity,

Table 3
CLASSIFICATION TEST RESULTS
3M PLANTSITE DEVELOPMENT
MOUNTAIN GATE, CALIFORNIA

Location	Sample	Depth (ft)	Natural Moisture Content (%)	Atterberg Limits (%)			Grain Size Distribution (%)			Observed Maximum Size (in)	Visual Classification	Unified Classification
				Liquid Limit	Plastic Limit	Plas-ticity Index	Clay & Silt (Minus #200)	Sand (#200 to #4)	Gravel (#4 to 3 in)			
PT-1	S-1	6.0	22								Sandy gravelly clay	
PT-2	S-1	1.0	13								Sandy gravelly silt	
	S-2	6.0-7.0	22							4	Clayey gravelly sand	
PT-4	S-1	5.0	25	50	32	18	20	76	4		Silty sand	SM
			26	53	34	19	19					
PT-5	S-1	1.0	2				21	50	29	<1.5	Silty gravelly sand	SM
	S-2	5.0	7				8	31	61	<2	Fractured and weathered rock	GW-GM
PT-6	S-1	3.0	21				16	61	23	6	Silty gravelly sand	SM
	S-2	6.0	32	69	33	36					Clayey silt	MH
	S-3	7.0	25	63	31	32					Clayey silt	MH
PT-7	S-1	8.0	22								Weathered rock	
	S-2	11.0	20								Weathered rock	
PT-8	S-1	3.0	12							3	Gravelly sandy silt	
PT-9	S-1	3.0	9								Gravelly sandy silt	
	S-3	11.0	40								Gravelly clay	
PT-10	S-1	3.0-4.0	15								Sandy clay	
	S-2	7.0	19								Sandy clay	
	S-3	10.0	22								Sandy clay	
PT-12	S-1	3.0-4.0					3	19	78	10	Sandy gravel with cobbles	GP
	S-2	7.5		37	15	22					Weathered rock	CL
PT-13	S-1	1.0	19	64	32	32	23				Clayey silty sand	SM-SC
PT-14	S-1	3.0-4.0	8				50				Sand and silt	
	S-2	8.5	20								Clayey sand	

Table 3 (Cont'd)
CLASSIFICATION TEST RESULTS
3M PLANTSITE DEVELOPMENT
MOUNTAIN GATE, CALIFORNIA

Location	Sample	Depth (ft)	Natural Moisture Content (%)	Atterberg Limits (%) Liquid Limit	Atterberg Limits (%) Plastic Limit	Atterberg Limits (%) Plasticity Index	Grain Size Distribution (%) Clay & Silt (Minus #200)	Grain Size Distribution (%) Sand (#200 to #4)	Grain Size Distribution (%) Gravel (#4 to 3 in)	Observed Maximum Size (in)	Visual Classification	Unified Classification
PT-15	S-1	3.0	13								Clayey sand	
PT-17	S-1	2.0	22								Sandy clay	
	S-2	8.0	29								Silty clay	
	S-3	10.0-12.0		70	40	30	38				Gravelly silty sand	SM
PT-20	S-1	7.0	35	74	41	33	48				Sand and silt	SM-MH
PT-24	S-1	2.0-3.0	22	64	31	33	15				Clayey sand	SC
PB-3	ST-1	9.0-10.3	26	48	34	14	30	60	10	<3/8	Silty sand	SM
PB-4	SS-2	5.0-6.5								1	Gravelly sandy silt	
	SS-3	8.0-9.5	26	47	30	17					Clayey silt	ML
			26									
WA-1	S-1	1.0	11	39	24	15	50	45	5	<3/8	Sandy clay	CL-SC
			11				50					
WA-3	S-1	2.5	9							6	Gravelly sandy silt	
WA-4	S-2	3.0	5								Rock	
WA-10	S-1	2.0-3.0	16				22	78	0	<1/8	Weathered rock	SM
OB-1	SS-1	5.0-6.5	16								Gravelly sandy clay	
	SS-2	10.0-11.5	24								Gravelly sandy clay	
	SS-3	15.0-16.5	34								Gravelly sandy clay	
	SS-4	20.0-21.5	34	51	38	13					Sandy silt	MH
	SS-5	25.0-26.5	35									
			21				37				Silty sand	
	SS-6	30.0-31.5	17									
			24								Weathered claystone	

Table 3 (Cont'd)
CLASSIFICATION TEST RESULTS
3M PLANTSITE DEVELOPMENT
MOUNTAIN GATE, CALIFORNIA

Location	Sample	Depth (ft)	Natural Moisture Content (%)	Atterberg Limits (%)			Grain Size Distribution (%)			Observed Maximum Size (in)	Visual Classification	Unified Classification
				Liquid Limit	Plastic Limit	Plasticity Index	Clay & Silt (Minus #200)	Sand (#200 to #4)	Gravel (#4 to 3 in)			
OB-2[a]	SS-1	5.0-6.5	16	51	35	16	32				Silty sand	SM
	SS-2	10.0-11.5	16	51	35	16	32				Silty sand	SM
EA-1	S-1	2.0	8								Sandy silty clay	
			10									
EA-2	S-1	1.0-2.0	11								Clayey silt	
EA-3	S-1	3.0	15								Gravelly clayey sand	
	S-2	5.0	22								Weathered shale	
EA-4	S-1	2.0	16	42	20	22	20		7	3	Clayey sand	SC
			15									
	S-2	8.0	21								Sandy clay	
	S-3	9.0	20								Sandy clay	
EA-5	S-1	2.5-3.0	17	45	22	23	42		23	1.5	Gravelly clayey sand	SC
			17									
EA-7	S-1	1.5-2.0	17	39	12	27	26	65	9	1	Clayey sand	SC
			16									
EB-4	S-1	0.5-1.0	14								Sandy clay	

[a]Tests performed on combined sample from SS-1 and SS-2

Table 4
ENGINEERING PARAMETER TEST RESULTS
3M PLANTSITE DEVELOPMENT
MOUNTAIN GATE, CALIFORNIA

			UBC Expansion Test		Corrosion Testing			R-Value Test		Natural	Unconfined Compression Test				Modified Proctor Compaction	
Location	Sample	Depth (ft)	Expansion Index	Expansion Potential	pH	Resistivity (ohms/cm)	Sulfate Concentration (ppm)	R-Value at Exudation Pressure = 300 psi	R-Value by[a] Expansion	Dry Density (pcf)	Compressive Strength (tsf)	Failure Strain (%)	Moisture Content (%)	Specific Gravity	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
PT-4	S-1	5.0	12	Very low				37	53					2.79[b]	103.5	21
PT-6	S-2	6.0	131	Very high	6.80	60,000	<100									
PT-15	S-2	8.0			6.26	50,000										
PT-17	S-3	10.0-12.0	122	High												
PT-20	S-1	7.0			6.65	6,100	<100									
PB-2	SS-1	2.0-3.5			5.71	130,000										
PB-3	ST-1	9.0-10.3			7.08	1,500	200			91.2				2.80[b]		
PB-4	SS-2	5.0-6.5			5.43	130,000										
	SS-3	8.0-9.5								97.1	2.4	4	26			
WA-1	S-1	1.0						19	27					2.75[b]	112	15
WA-10	S-1	2.0-3.0						9	22							
OB-1	SS-5	25.0-26.5								92.8	2.2	7	24			
	SS-6	30.0-31.5								82.2	0.6[c]	5[c]	35			
EA-4	S-1	2.0						20	100							
EA-5	S-1	2.5-3.0	81	Medium				11	11							
EA-7	S-1	1.5-2.0						19	>27							
EB-4	S-1	0.5-1.0								88.5				2.75[b]	113	15

[a] Based on traffic index = 6.0, gravel equivalent factor = 1.0, and unit weight of cover = 130 pcf
[b] Test run on total sample
[c] Crumbling-type failure, intact rock structure evident

natural dry density, and compressibility tests were performed. The results are presented in Table 4 and on Figures 19 through 22.

A detailed description of the laboratory tests appears in Appendix B. Unless special arrangements are made with CH2M HILL, all samples will be discarded after 30 days.

SUMMARY OF SUBSURFACE CONDITIONS

Our summary of subsurface conditions divides the site into four areas for purposes of discussion (see Figure 1).

Most of the surface and near-surface soil encountered during our exploration is of alluvial or colluvial origin. Alluvial soils are present in the plantsite and eastside access road areas. Residual soils, primarily derived from marine shale and mudstone, are present at locations throughout the site.

Plantsite

Soils encountered at the plantsite varied from fine-grained (clay and silt) to coarse-grained (sand, gravel, and cobbles) in texture. The depth to very dense or hard material varied from approximately 7 feet (backhoe refusal in PT-3 and PT-5) to over 45.5 feet (PB-3) and was generally between 10 and 20 feet.

The predominant soil encountered was a mixture of sand, gravel, and silt which typically extended from the ground surface to depths in excess of 10 feet. This soil was typically loose to medium in consistency near the ground surface and medium to dense below 5 feet.

Potential problem soils include high-plasticity mixtures of silt and clay which are probably highly weathered residual soils. These soils were encountered and tested in Test Pits PT-6, PT-17, PT-20, and PT-24. These soils are likely to be compressible, may be soft at high moisture content, and exhibit a high shrink-swell potential. In addition, some low resistivity mixtures of silt, clay, and sand were encountered in Boring PB-3 and Test Pit PT-20. These soils are likely to be corrosive to buried steel.

Ground water was encountered in two borings and none of the test pits. The depth to ground water was approximately 35 feet, but this measurement was made at a time of the year when ground water elevations are generally at their lowest.

Railroad Spur and Westside Access Road

Both coarse and fine-grained soils were encountered in this area, with the predominant soil type being a mixture of sand, gravel, and silt. These soils were typically loose to

medium in consistency near the ground surface. Hard or very dense material was encountered at depths of 2 to 5 feet in six of the test pits (indicated by backhoe refusal in WA-3, WA-4, and WA-12 through WA-15). Other test pits extended to 10 feet without reaching hard or very dense material.

Some mixtures of silt and clay were encountered, typically overlying rock. A sample tested from WA-1 indicated the soil is of low plasticity and would present few, if any, problems.

No ground water was encountered within the depths explored, but ground water may rise during wetter periods of the year.

Overpass

The soils encountered in the two overpass borings consisted of 28 to 40 feet of a sand, silt, and clay mixture containing various amounts of gravel and some cobbles. The mixture was typically medium to very dense or stiff to hard near the ground surface, changing to medium consistency below 15 to 20 feet. The underlying material was a weathered marine sediment, very dense or hard in consistency. No ground water or high plasticity soils were encountered.

Eastside Access Road

The soils in this area are predominantly clayey sand and sandy clay with various amounts of silt and gravel. They are medium to very dense or stiff to hard in consistency. A very dense or hard weathered marine shale was encountered at a depth of 6 and 15 feet in two locations. The remaining borings and test pits typically extended to 8 to 15 feet without encountering the very dense or hard stratum.

Ground water was not encountered. However, based on topography and experience in adjacent areas, we expect ground water near the ground surface during periods of high precipitation.

Although no high plasticity fine-grained soils were encountered, medium plasticity soils were. These medium plastic soils are most likely of moderate compressibility, may be soft at high moisture content, and exhibit medium shrink-swell potential.

Chapter 4
CONCLUSIONS

Chapter 4
CONCLUSIONS

PLANTSITE

Foundations

Foundation types appropriate for the anticipated structures include:

- Reinforced concrete shallow foundations - spread and continuous footings
- Reinforced concrete mat foundations
- End-bearing deep foundations

For preliminary design, shallow foundations may be sized for a maximum allowable bearing pressure of 1,500 to 3,000 psf. Footings should be embedded at least 2.5 feet and have a minimum width of 1.5 feet.

Insufficient information is presently available for sizing mat foundations.

Our analysis of deep foundation types indicates the following should be considered during final design:

- Augered or bored cast-in-place concrete piles
- Driven steel H or pipe piles
- Driven treated timber piles

Allowable capacities of 20 to 50 tons per pile may be used for preliminary design. We anticipate deep foundations will penetrate 5 to 15 feet of very dense to hard material, depending upon the required capacity and bearing material characteristics. Considerations for the selection of pile type include the variable depth to very dense or hard material (timber pile lengths and to a lesser extent steel pile lengths are not easily varied in the field) and the presence of groundwater (cast-in-place concrete piles generally require casing below the groundwater table). Additional considerations include the ability to determine when the bearing stratum is reached (relatively certain for driven piles and less certain for augered or bored piles) and, of course, cost.

We anticipate shallow foundations are appropriate for lightly loaded structures.

Moderately loaded structures may be founded on shallow, mat, or deep foundations. The foundation type will primarily depend on the allowable total and differential settlement for the structure. We therefore recommend that the preliminary selection of foundation type be based on allowable total settlement. Structures with total allowable settlements of 2 to 3 inches may be supported on mat or shallow foundations. Structures sensitive to settlement greater than 1 inch should be founded on deep foundations.

Concentrated loads for heavily loaded structures will require deep foundations. Other parts of heavily loaded structures may be supported on mat or shallow foundations. Preliminary selection of foundation type for these other areas should be based on the settlement criteria stated for moderately loaded structures. Because the predominant soil type encountered at the plantsite (silt, sand, and gravel) is potentially susceptible to vibration-induced settlement, vibrating loads may require deep foundations.

The depth to very dense or hard material varies in the plantsite area. Figure 23 shows a summary of the information obtained during our exploration. This figure may be useful for positioning moderately and heavily loaded structures, and for minimizing the depth of deep foundations. Our interpretation of this figure indicates the depth to very dense or hard material is greatest near the center of the meadow and decreases near the edges. We caution that this figure is based on preliminary information, and variations between locations should be expected.

Consideration should be given to removal and recompaction of native soils to improve foundation support. This approach may allow shallow foundations to be positioned nearer the ground surface, or may allow the use of a mat foundation instead of a deep foundations.

Potentially expansive soils were encountered in localized areas at the plantsite and are discussed later on in this chapter. The presence of these soils may necessitate modification of our foundation conclusions or design of mitigating measures.

Earthwork

We believe earthwork operations can be performed by conventional earthmoving equipment. Very dense to hard material may require ripping, but blasting is unlikely. For preliminary design and earthwork balance, cut and fill slopes of 1.5:1 and an overall shrinkage factor of 15 percent should be used. The shrinkage factor will, however, vary according to soil type. In addition, permanent slopes that will be vegetated should be 2:1 or flatter.

With the exception of potentially expansive soils, all soil encountered in the plantsite area is suitable for fill. The compaction requirements will depend upon the soil type and imposed structure loads. Present information on proposed structures is insufficient to formulate compaction requirements.

Future Exploration

Upon identification of structure types, loads, and locations, additional field and laboratory work should be performed for final design. This work should include subsurface exploration under each major structure and additional tests, primarily to determine engineering parameters. If alternative locations for structures are identified, they should be explored. We have found that this extra effort can ultimately save money and avoid delays.

RAILROAD SPUR

Alignment and Grade

Except for expansive soils, our field exploration encountered no soil conditions that would restrict the railroad spur alignment. Given the strict tolerances for railroad alignment and grade, we anticipate isolated areas of expansive soils cannot be avoided and mitigating measures may be necessary.

Cut Sections

We believe cuts can be made with conventional earthmoving equipment and that ripping but not blasting will be necessary. For preliminary design and earthwork balance, cut slopes in soil should be 1.5:1 and cut slopes in rock should be 1:1. To provide a transition between cut and fill sections without abrupt differential settlement, we recommend a transition slope of 2:1 or flatter be used as shown on Figure 24.

Subgrade preparation should ensure a relative compaction[1] of at least 95 percent to a depth of 0.5-foot or greater. Moisture content during subgrade preparation should be that required to obtain the necessary relative compaction and to minimize long term volume changes.

Fill Sections

With the exception of potentially expansive soils, all soil we encountered is suitable for fill under the railroad spur.

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[1]Relative compaction is defined as the ratio of the as-compacted field dry density to the maximum dry density determined in accordance with ASTM D 1557.

For preliminary design and earthwork balance, fill slopes should be 1.5:1. Based on our experience in the area, an overall shrinkage factor of 10 percent should be used. We do, however, expect this factor will vary widely according to soil or rock type.

Fill should be placed and compacted to at least 90 percent relative compaction, except for the top 2.5 feet below subgrade elevation which should be compacted to at least 95 percent relative compaction. Compaction moisture content should be adequate to achieve the necessary relative compaction and minimize long term volume changes.

Structural Section

In cut sections, 6 to 12 inches of aggregate base should be used and in fill sections 8 to 12 inches should be used. Aggregate base should consist of Class 2 Aggregate Base, 3/4-inch maximum as specified in the State of California Standard Specifications (Caltrans, 1981).

Ballast, placed over the aggregate base, should consist of at least 6 inches of 1.5-inch crushed rock.

We encountered no suitable sources of onsite aggregate base or ballast. It may be possible to crush and process Copley Greenstone for aggregate base and ballast, otherwise these materials must be imported.

Future Exploration

Upon location of the line and grade of the railroad spur, additional field and laboratory work should be performed.

Potential sources of aggregate base and ballast (including crushing and processing of onsite Copley Greenstone) should be explored. This will allow for an economical design of a structural section. The durability of excavated rock should be assessed to determine whether it can be placed as a rock fill or must be broken down prior to placement.

ACCESS ROAD

Alignment and Grade

Except for potentially expansive soils, our field exploration encountered no soil conditions that would restrict the alignment of the access road. Expansive soils generally should be avoided, but they are present only in limited and localized areas and should be a relatively minor concern for alignment.

Due to the expected high ground-water table, consideration should be given to a raised roadway in the eastside access road area. We anticipate approximately 2 feet of fill will be sufficient. However, if special consideration is given to drainage in this area, a raised roadway may not be necessary.

Cut and Fill Sections

For preliminary design, the same conclusions as stated for the railroad spur apply to the roadway.

Structural Section

Soils of variable expansion potential and compressibility plus variable cut and fill depths will be encountered along the access road alignment. We therefore consider a flexible pavement (asphalt concrete surfacing) to be appropriate for the subsurface conditions.

We have performed a preliminary analysis for the structural section of an asphalt concrete roadway. Our preliminary analysis is based on the R-value test, which generally accounts for expansion in soils with low to medium expansion potential. Soils with high to very high expansion potential require testing and analysis specifically directed toward volume change characteristics. The parameters assumed in our preliminary analysis are:

- Class 2 aggregate base with an R-value of 78
- Subbase material with an R-value of 44
- Minimum R-value of basement soil is 9 (minimum measured)
- Traffic index of 6, corresponding to between 23,500 and 47,300 equivalent 18 kip axle loads, calculated for a design period of 20 years
- Two-lane construction

The design method is that used by the California Department of Transportation (Caltrans, 1975). Using the above parameters, we arrived at a preliminary section as follows:

Material	Thickness
Asphalt Concrete	0.25 foot
Aggregate Base	1.0 foot
Subbase	1.6 feet

Allowing an increase in the assumed basement R-value to 19 results in a 0.2-foot reduction in the subbase thickness. This may be justified since several basement soils along the potential route tested in that range. The above parameters are based on using materials that should be readily available.

Future Exploration

Upon location of the line and grade of the access road, additional field and laboratory work should be performed. This work should concentrate on the following:

1. Defining areas of potentially expansive soils.

2. Analysis of the structural section with respect to expansive soils.

3. Defining suitable native subbase material and assessing the design R-value.

4. Establishing groundwater conditions, especially in the eastside access road area.

Potential sources of aggregate base should be explored as for the railroad spur.

In addition, staged construction should be analyzed. Staged construction may involve placement of subbase, base, and a macadam surfacing followed by asphalt concrete placement after plantsite construction is essentially complete. This approach may prove advantageous, especially in light of the heavy traffic involved during construction.

OVERPASS

Foundations

The subsurface conditions encountered in our two borings appear suitable for a deep foundation. A deep foundation will also minimize the consequence of potential slope instability--presently evidenced by small slumps on the north side of the tracks.

The conclusions regarding deep foundations at the plantsite are also appropriate for the overpass. For the overpass, however, additional embedment in very dense or hard material will be necessary to resist lateral loads. This constraint may make augered or bored cast-in-place concrete piles appropriate.

Future Exploration

Additional exploration should be performed, primarily to recover samples for tests to determine engineering parameters at the exact overpass location.

EXPANSIVE SOILS

Soils of high to very high shrink-swell potential were encountered and tested in the plantsite area. These soils may also be present in other areas. Identification of these additional areas will be necessary prior to final design. Figure 25 shows a plot of plasticity index versus Uniform Building Code (UBC) expansion index. From this figure, we conclude that the plasticity index is a good measure of potential volume change and can be used effectively during future explorations to identify expansive soils.

There are, however, other factors which influence the volume change behavior of soils which are not addressed in the above tests. These factors include:

- Percentage of fine-grained, primarily clay-size, particles
- Natural moisture content and expected moisture variation
- Stress state of the material; i.e., depth below the ground surface and imposed structure loads

Soils having medium to high shrink-swell potential can also present problems, especially if they occur near the ground surface or immediately under a structure. Soils with medium to high shrink-swell potential were encountered and tested in the eastside access road area and may be present in other areas.

Options we consider appropriate for mitigating shrink-swell problems include:

- Avoidance--relocate the structure or roadway
- Complete or partial removal of expansive soils and replacement with nonexpansive material
- Admixtures such as lime treatment
- Control of volume changes through the proper sizing of imposed loads--increase the structural section of roadways or size footings for bearing pressures greater than or equal to swell pressures
- Separation--use grade beams and suspended slabs to separate slabs from the ground and allow swell
- Control of moisture changes--condition the soil to a moisture content equal to its expected long-term moisture content

A more detailed investigation of expansive soil problems and site-specific recommendations for remedial measures are necessary. These needs can be best addressed during final design.

Additional field and laboratory work is necessary to adequately characterize and locate potentially expansive soils. Laboratory tests such as confined swell tests should be performed and the results correlated with existing classification data. The need for remedial measures can then be determined and appropriate measures selected.

CORROSION

Some of the soils at the plantsite are corrosive to buried steel. Two out of six samples tested were found to have low electrical resistivity, a common characteristic of corrosive soils. Other localized areas of corrosive soil may be present throughout the site. The rate of corrosion of buried steel cannot be predicted, because it will vary according to the way the steel is installed. Steel components such as pipe which are installed by excavation and backfill will be most susceptible to corrosion because this type of construction aerates the soil, making it more corrosive. Placing concrete slabs over buried steel also increases the corrosivity of the soil by increasing the moisture content. Driven steel piling has a low-corrosion rate because the soil around it is not disturbed except at grade.

Corrosion protection should be provided for buried steel. Zinc galvanizing of steel surfaces is the minimum level of acceptable protection. An organic (paint) coating and cathodic protection are preferable, particularly for piping. Driven steel piles should be coated from a point 1 foot above grade to 2 feet below grade.

Stainless steel should be avoided for buried service because it has unpredictable corrosion characteristics in soil. If stainless steel is necessary, however, cathodic protection will provide adequate corrosion control.

Construction materials which will not require any special corrosion control measures include ductile iron, copper, and concrete.

During the design phase, field resistivity tests should be conducted at locations where buried steel is proposed. These tests will be used to determine if the soil at those locations is corrosive and to assess the need for cathodic protection.

SEISMICITY

Based on our review of available literature and the earthquake data file, there appears to be no significant seismic hazard associated with the proposed project site. Although our exploration did not identify any historic earthquake or capable faults which have resulted in site accelerations greater than 5 percent g, we believe it prudent to design for a higher ground acceleration. We therefore recommend design accelerations be determined in accordance with applicable building codes.

An analysis of soil-liquefaction potential takes into consideration the soil particle size distribution, the location of the water table, the soil type and relative density, and the maximum potential ground accelerations from seismic events. Detailed analyses require structure- and site-specific information and are beyond the scope of this exploration.

Structure-specific liquefaction potential should be assessed during final design. We anticipate simplified analysis procedures, which are generally conservative, will confirm that no special design considerations are associated with potential liquefaction.

TOPSOIL FOR LANDSCAPING PURPOSES

Potential sources of topsoil for landscape purposes include the plantsite and eastside access road areas. Essentially, no topsoil is available in the westside access road area.

Both the plantsite and eastside access road areas support substantial growths of grass. The root zone in these areas is typically within 1 foot of the ground surface and contains grass, leaves, and roots in various stages of decomposition. Of the typical 1-foot (or less) thickness, we estimate approximately 0.5 feet may be salvaged for landscaping purposes. Addition of fertilizer and soil amendments should be expected. If additional topsoil is required, reconditioning of the soil below the root zone should be considered. Additional sampling and testing is necessary to establish the reconditioning requirements.

CONSTRUCTION CONSIDERATIONS

The weather in the Redding area is generally acceptable for earthwork operations from April through September. The months of July through September are usually hot and dry, presenting moisture content problems for fill placement. If possible, site grading work should be planned for late spring or early summer to take advantage of weather and soil moisture.

To a large degree, the ultimate performance of the plantsite development will depend on construction quality control. A program involving the project geotechnical engineers (during construction) and utilizing quality control testing by trained technicians under the direction of those engineers can help ensure that the geotechnical recommendations are followed. This type of program will also serve to identify soil conditions exposed during construction that differ from those conditions upon which the geotechnical recommendations are based.

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BIBLIOGRAPHY

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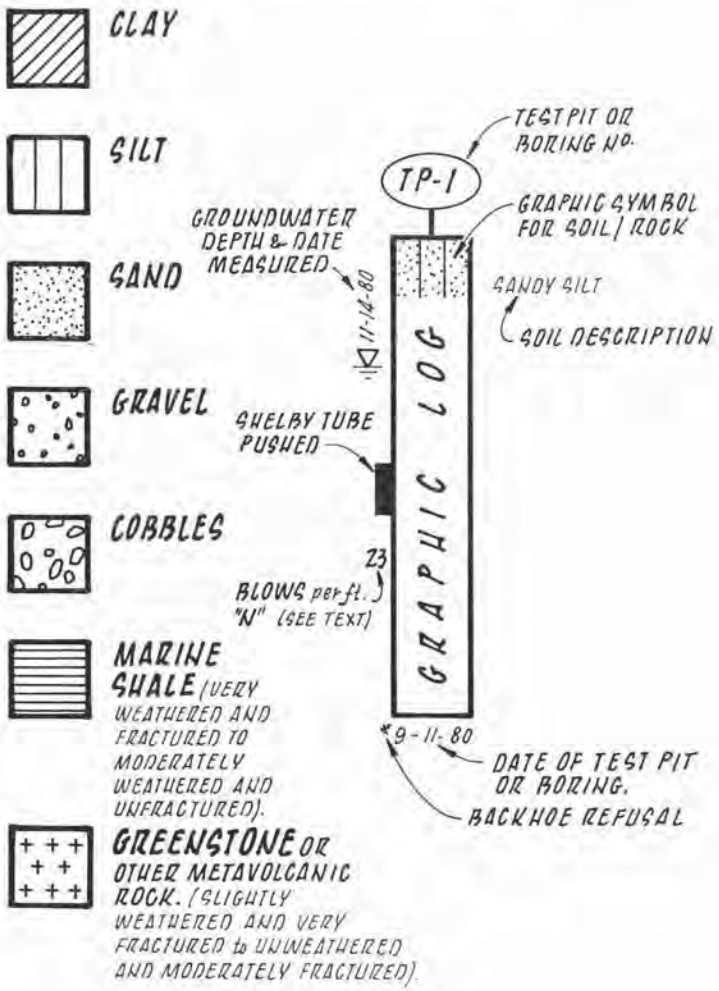
Appendix A
FIGURES

SYMBOLS and SAMPLE LOG



NOTES

NO GROUNDWATER WAS ENCOUNTERED IN TEST PITS

SOLID OR DENSE SYMBOL IS THE MAJOR SOIL CONSTITUENT. DASHED OR SPARSE SYMBOL IS THE MINOR SOIL CONSTITUENT.

BACKHOE REFUSAL IS THE INABILITY OF THE BACKHOE AND OPERATOR TO ADVANCE THE DEPTH OF THE TEST PIT MORE THAN 1 TO 2 INCHES PER SCOOP.

FIGURE-2
TEST PIT and BORING LEGEND
3M PLANTSITE DEVELOPMENT
Mountain Gate, California

CH2M HILL

R 14202.A5 – March 1981

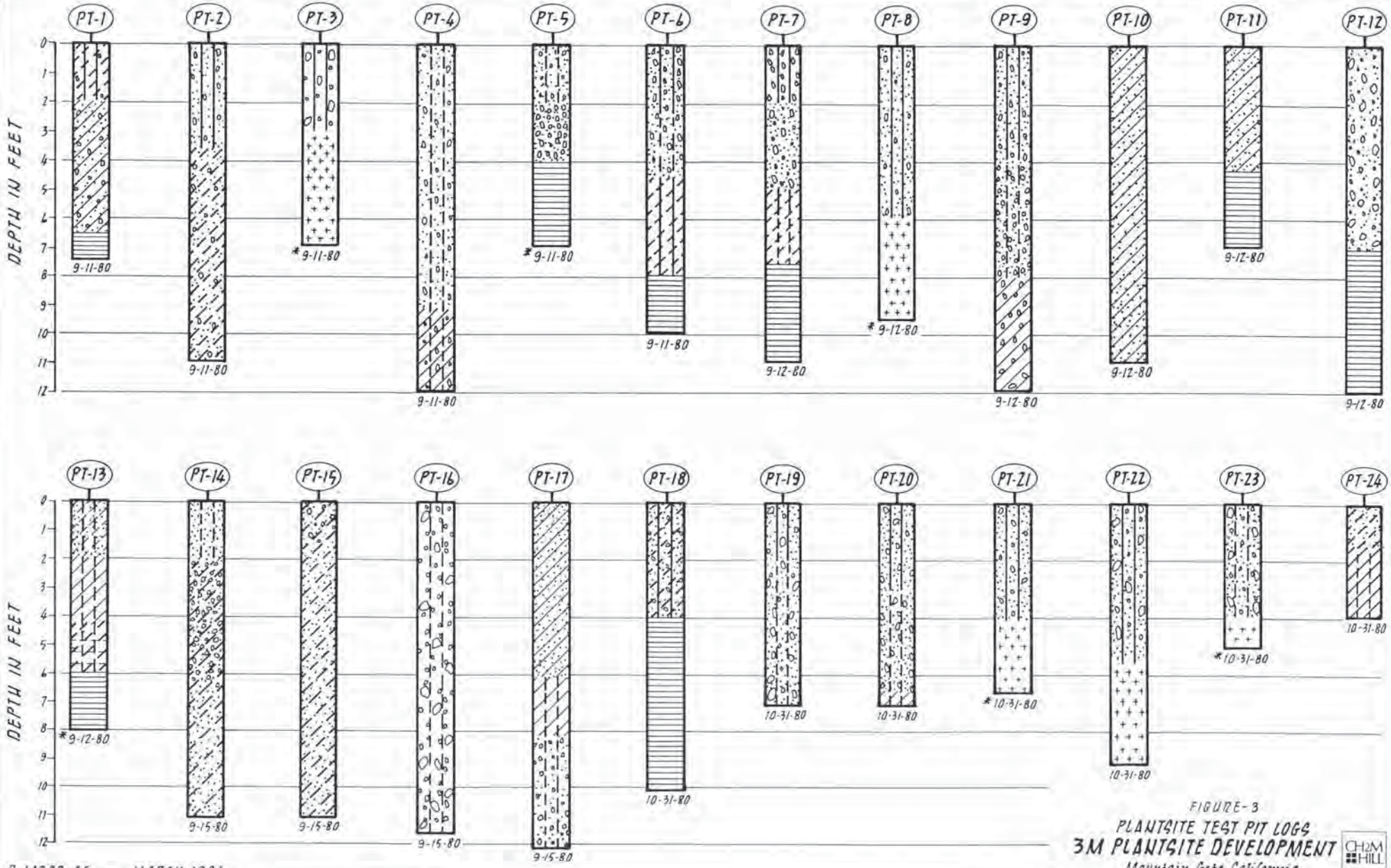


FIGURE-3

PLANTSITE TEST PIT LOGS

3M PLANTSITE DEVELOPMENT

Mountain Gate, California

CH2M HILL

R 14202 A5 — MARCH 1981

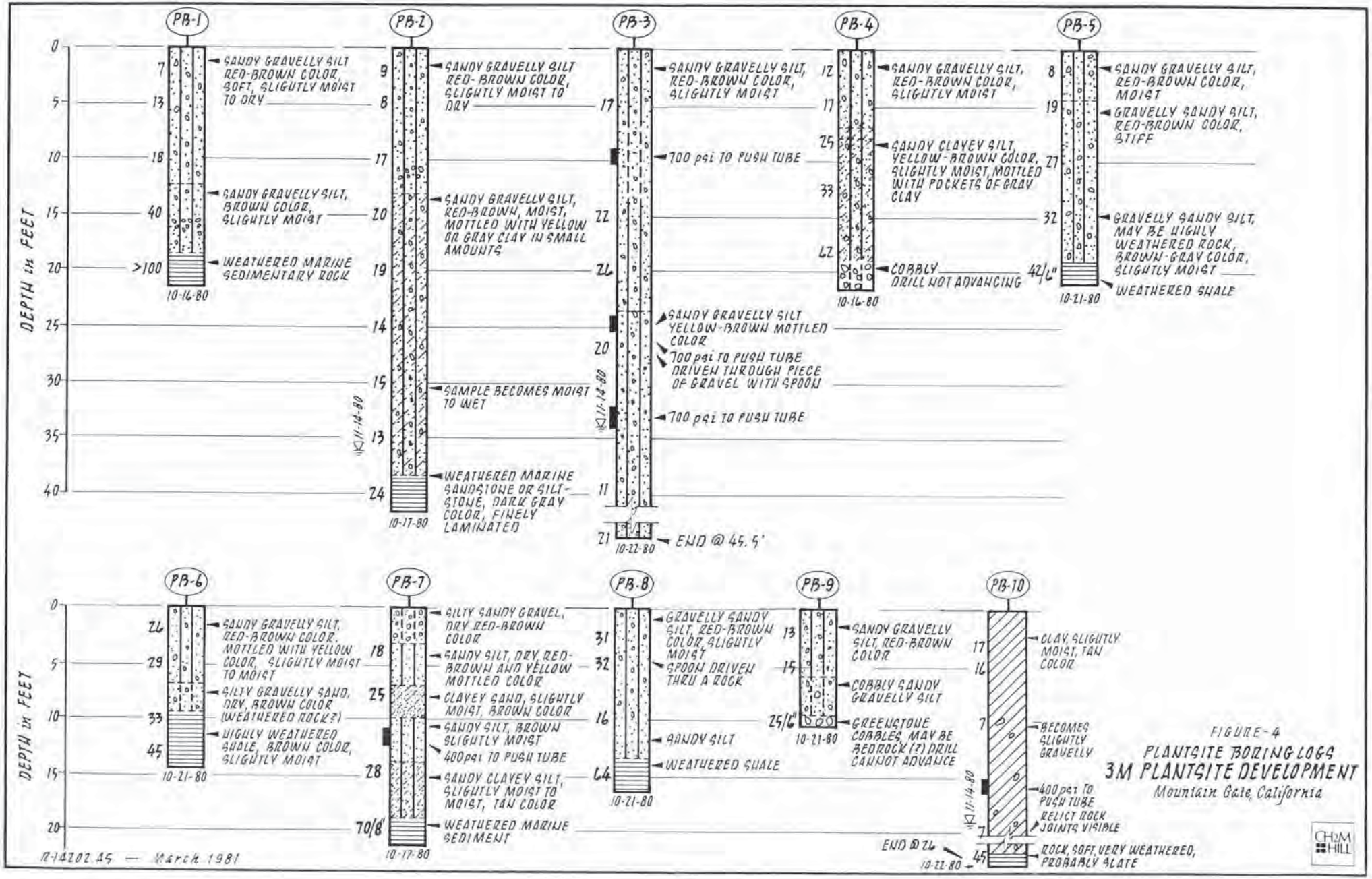


FIGURE-4

PLANTSITE BORING LOGS

3M PLANTSITE DEVELOPMENT

Mountain Gate, California

CH2M HILL

R-14202.A5 — March 1981

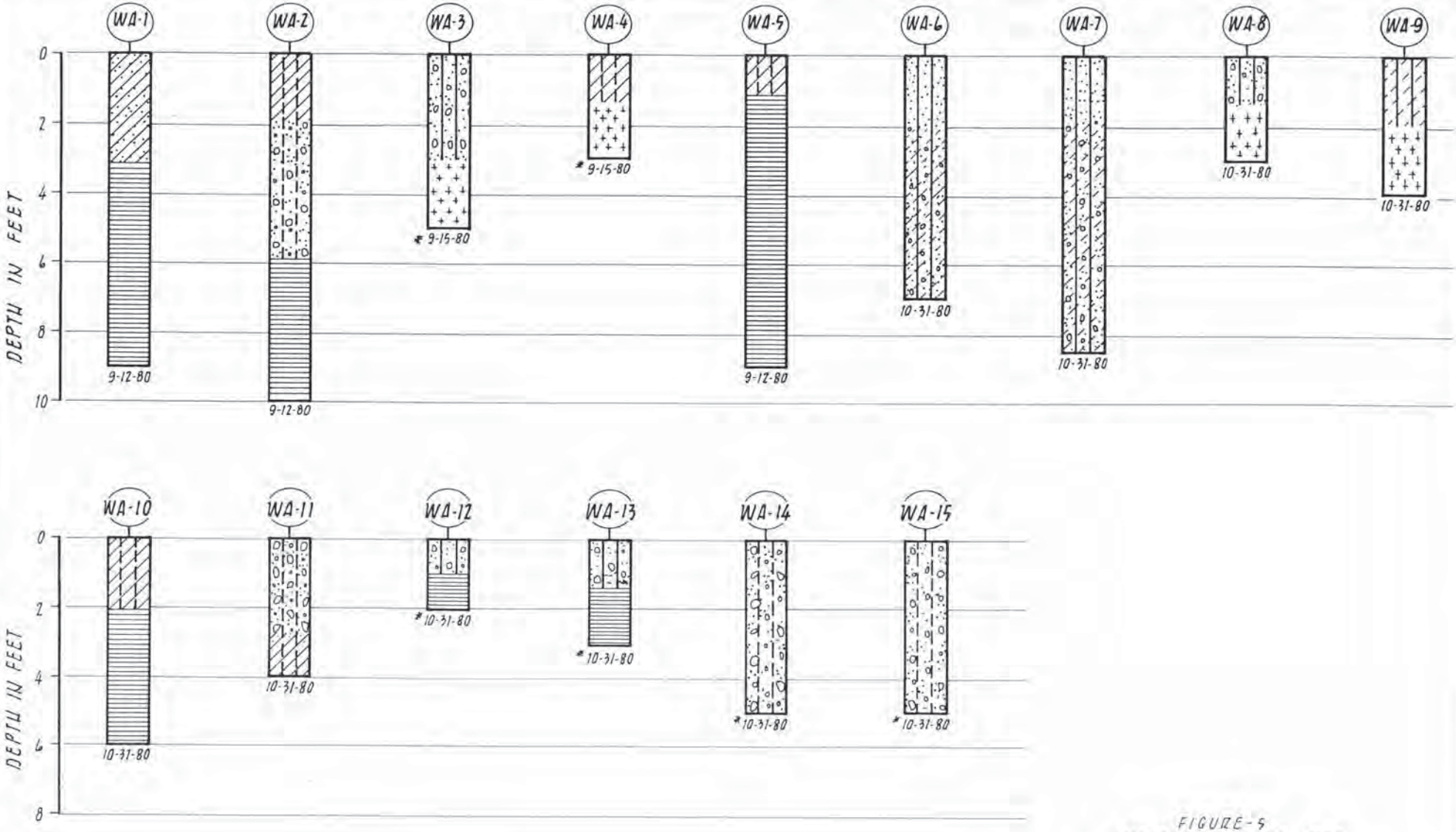
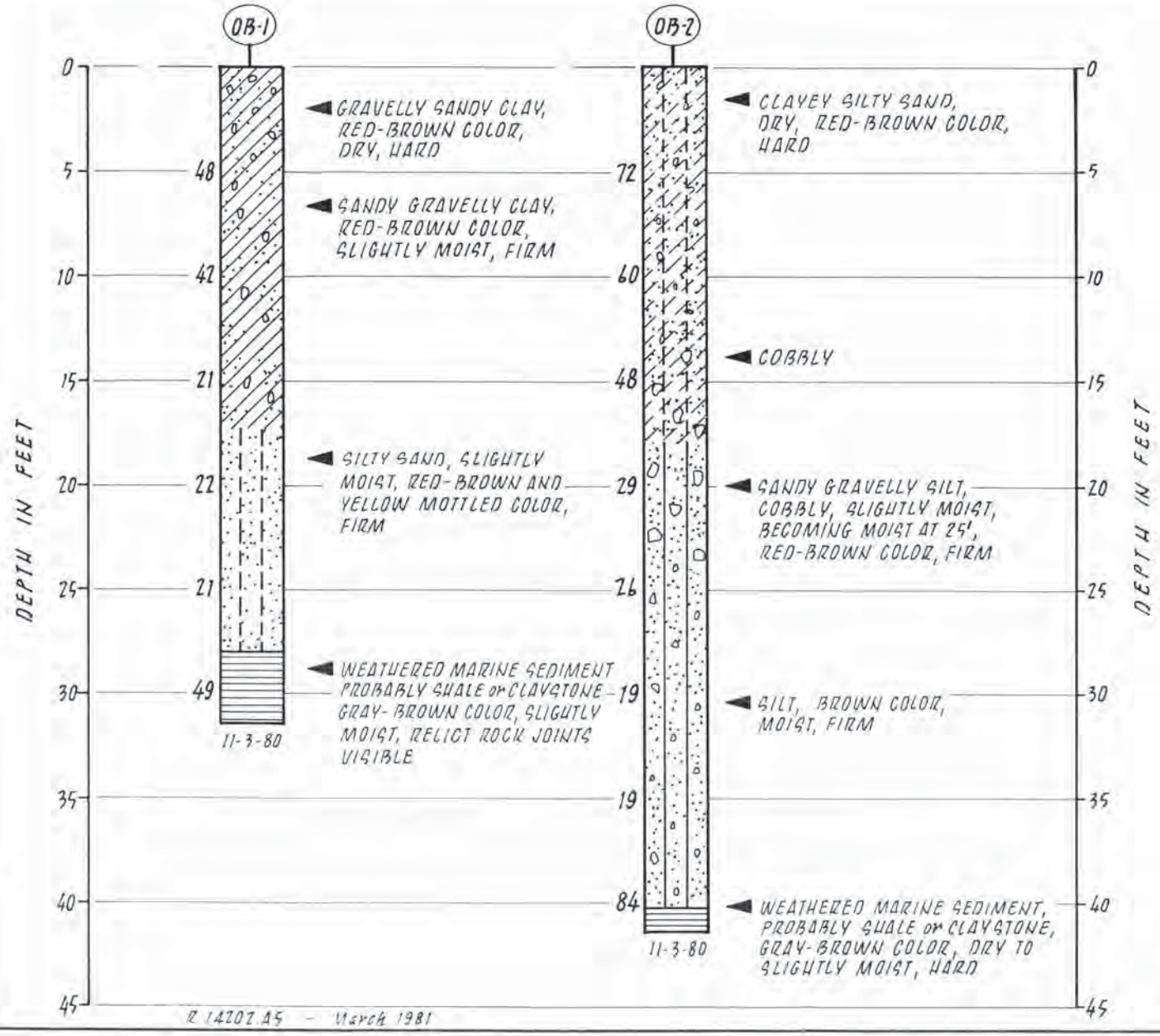


FIGURE-5
WESTSIDE ACCESS AREA
TEST PIT LOGS
3M PLANTSITE DEVELOPMENT
Mountain Gate, California

CH2M HILL

R 14202.A5 – March 1981



R 14207.A5 - March 1981

FIGURE-6

OVERPASS BORING LOGS

3M PLANTSITE DEVELOPMENT

Mountain Gate, California

CH2M HILL

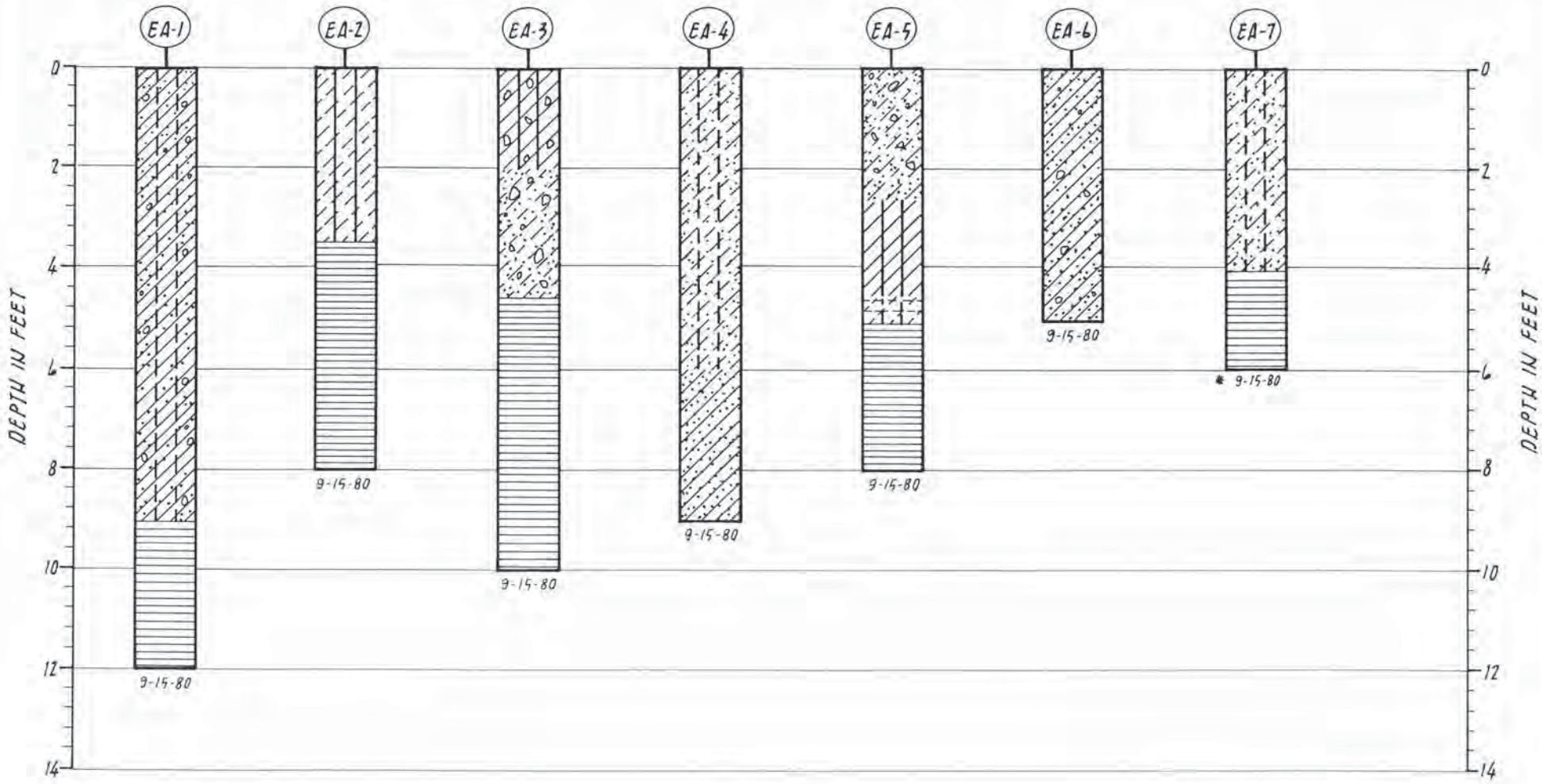


FIGURE-7
EASTSIDE ACCESS ROAD
TEST PIT LOGS
3M PLANTSITE DEVELOPMENT
Mountain Gate, California

CH2M HILL

R14202 A5 MARCH 1981

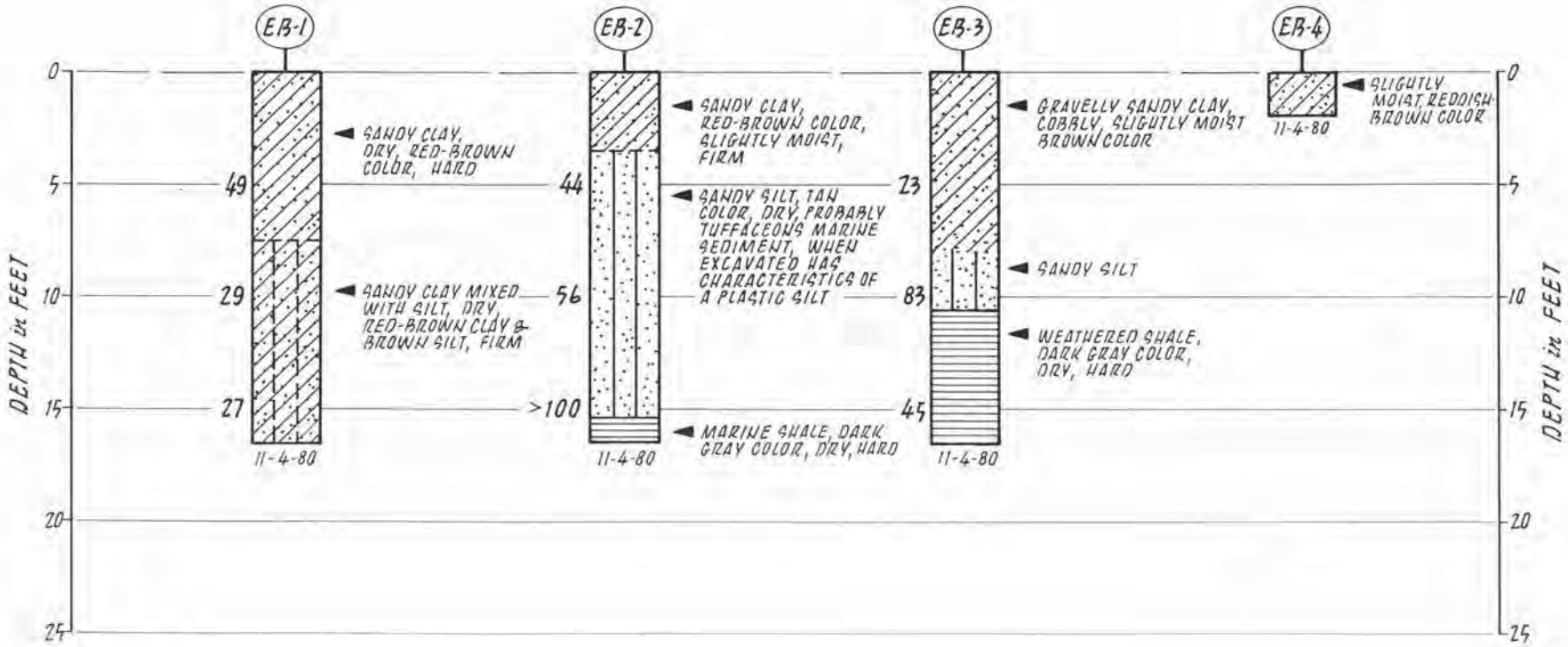


FIGURE-8
EASTSIDE ACCESS ROAD
BORING LOGS
3M PLANTSITE DEVELOPMENT
Mountain Gate, California

CH2M HILL

R 14202 A5 - March 1981

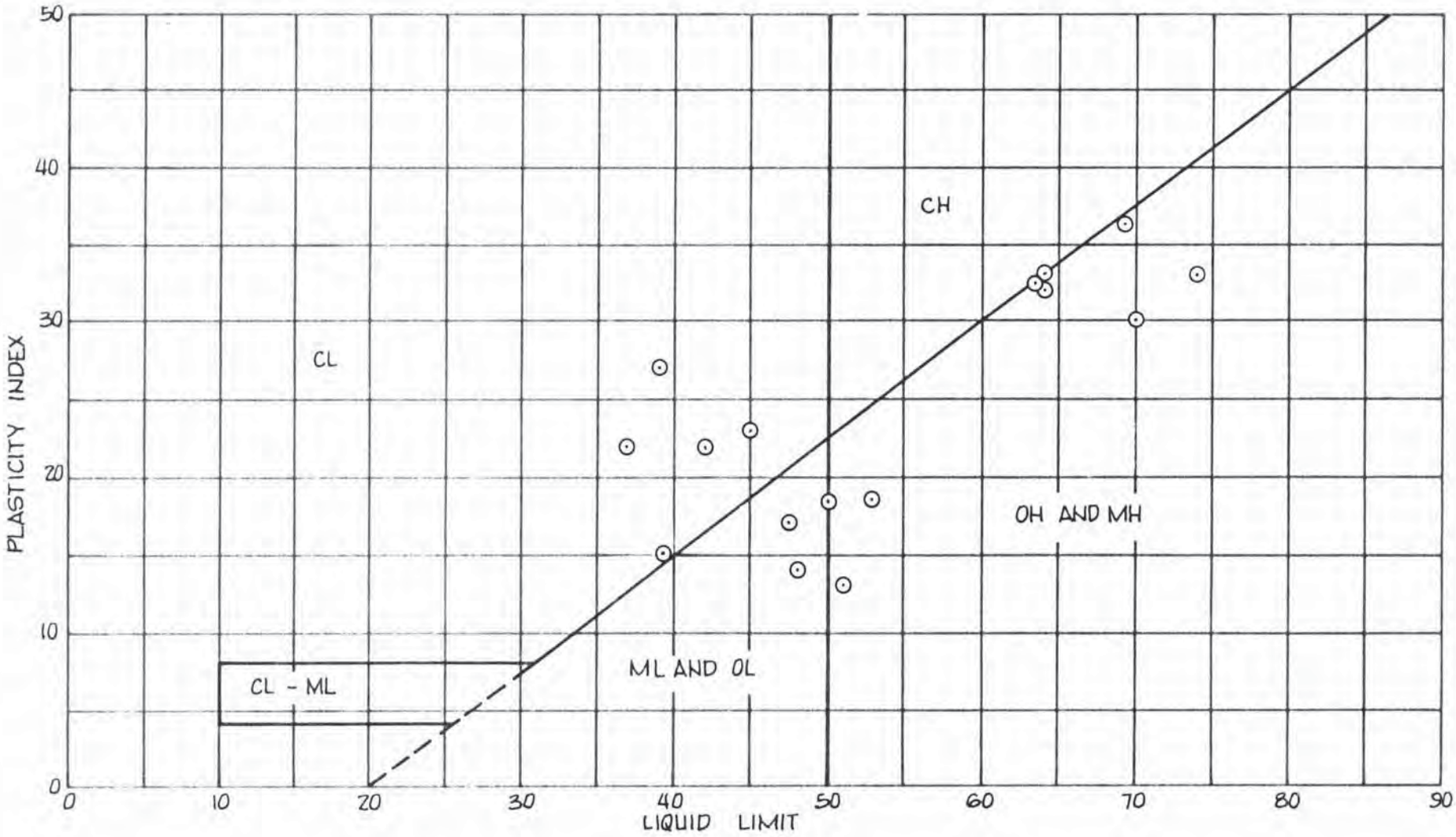


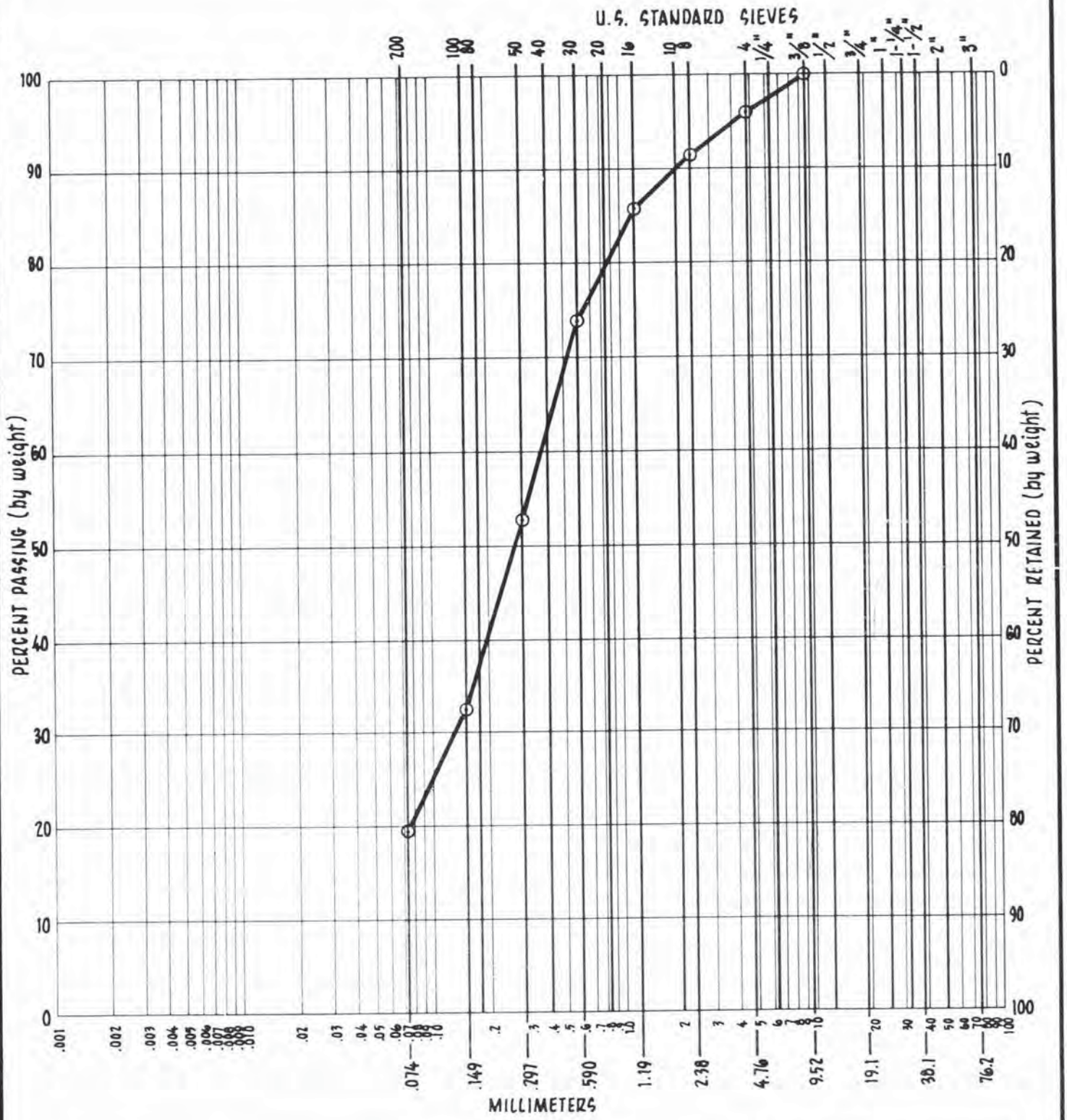
Figure 9

PLASTICITY CHART

3M PLANTSITE DEVELOPMENT
MOUNTAIN GATE, CALIF.

R-14202.A5
MARCH 1981

CH2M HILL



CLAY TO SILT (FINES)	SAND			GRAVEL		COBBLES
	FINE	MEDIUM	COARSE	FINE	COARSE	

LOCATION: PT-4
SAMPLE: S-1
DEPTH: 5.0'

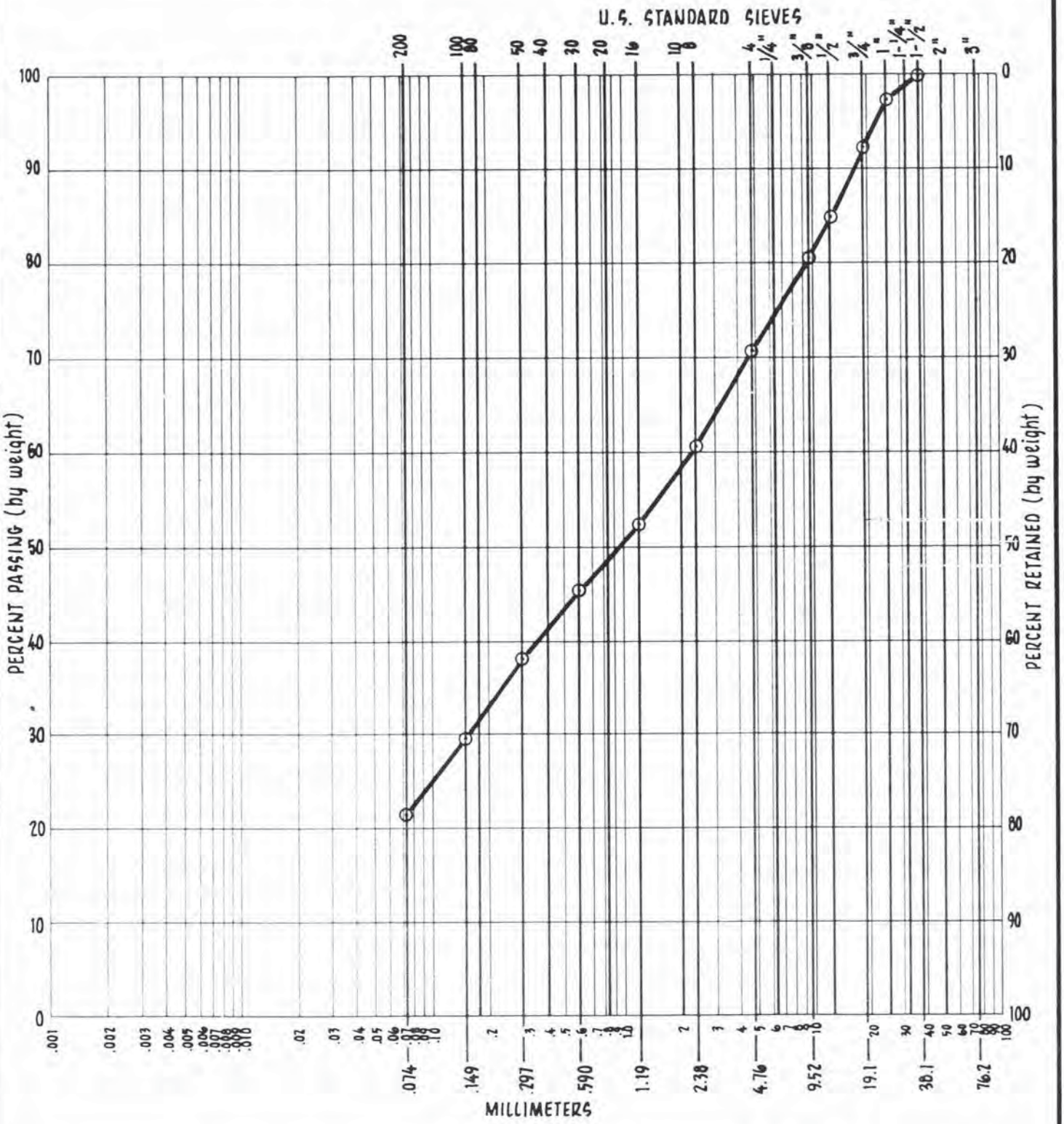
Figure 10

GRAIN SIZE ANALYSIS

3M PLANTSITE DEVELOPMENT
MOUNTAIN GATE, CALIF.

CH2M HILL

R-14202.A5 MARCH 1981



CLAY TO SILT (FINES)	SAND			GRAVEL		COBBLES
	FINE	MEDIUM	COARSE	FINE	COARSE	

LOCATION: PT-5
SAMPLE: G-1
DEPTH: 1.0'

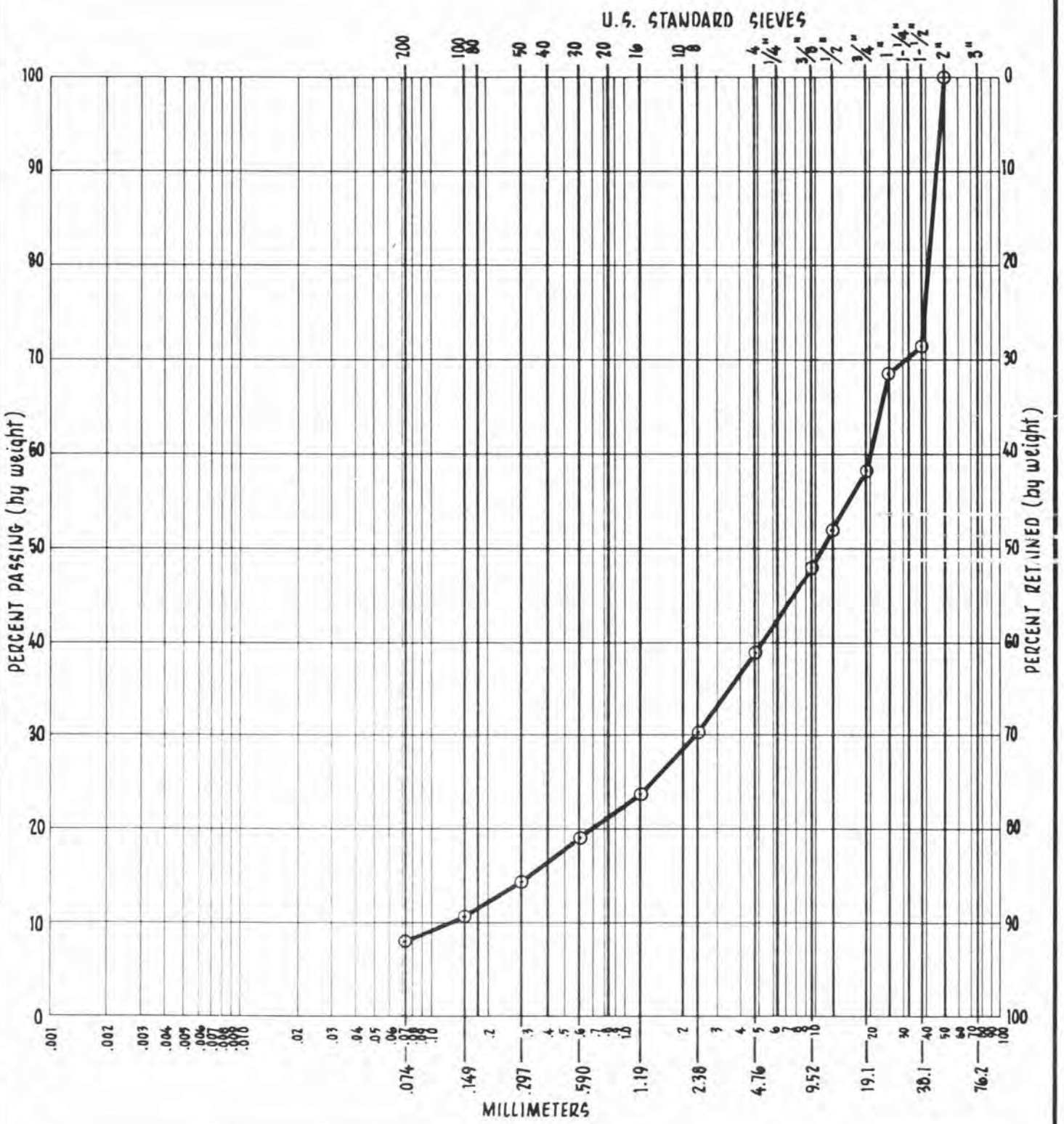
Figure 11

GRAIN SIZE ANALYSIS

3M PLANTSITE DEVELOPMENT
MOUNTAIN GATE, CALIF.

CH2M HILL

R-14202.A5 MARCH 1981



CLAY TO SILT (FINES)	SAND			GRAVEL		COBBLES
	FINE	MEDIUM	COARSE	FINE	COARSE	

LOCATION: PT-5
SAMPLE: S-2
DEPTH: 5.0'

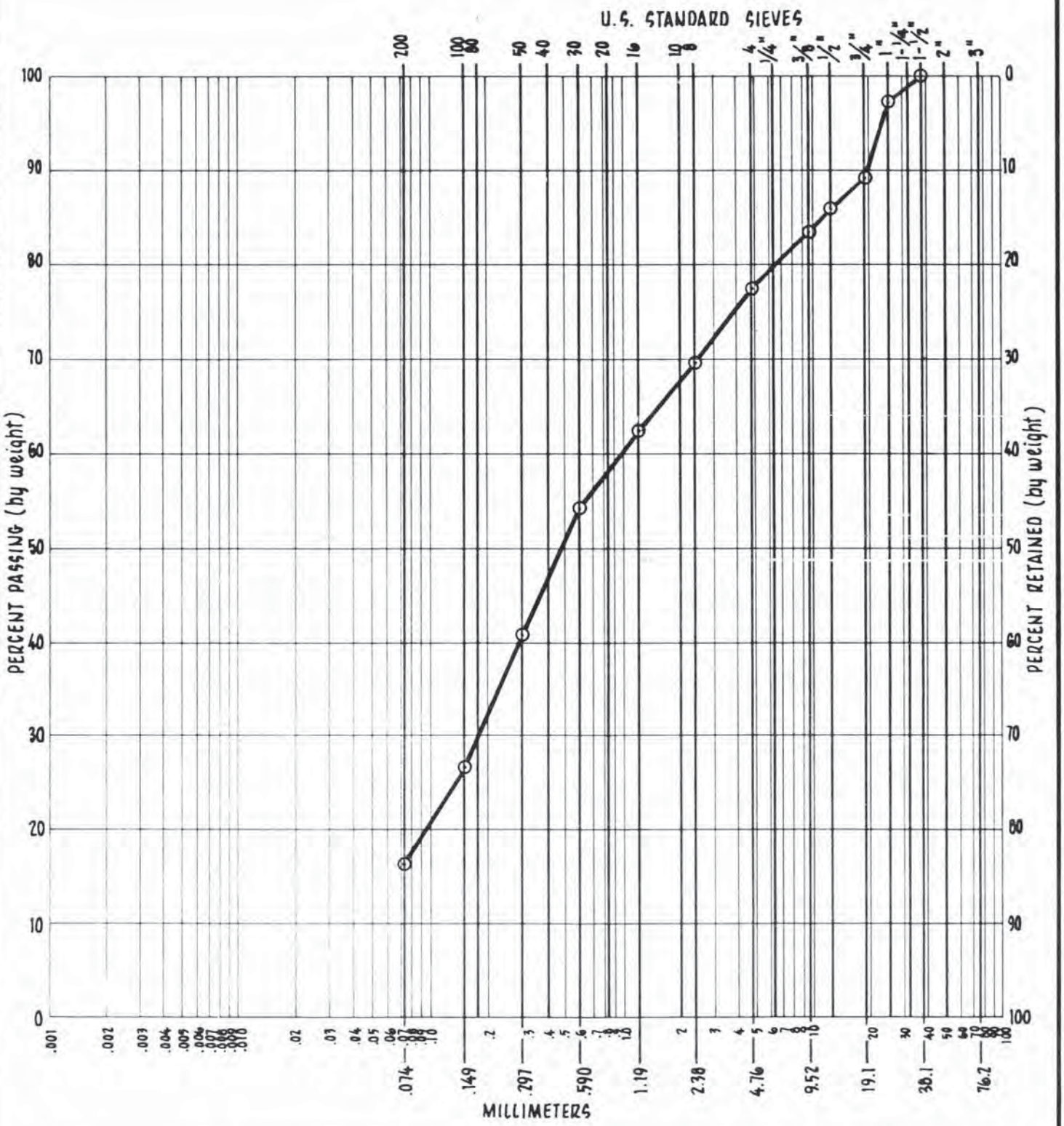
Figure 12

GRAIN SIZE ANALYSIS

3 M PLANTSITE DEVELOPMENT
MOUNTAIN GATE, CALIF.

CH2M HILL

R-14202.A5 MARCH 1981



CLAY TO SILT (FINES)	SAND			GRAVEL		COBBLES
	FINE	MEDIUM	COARSE	FINE	COARSE	

LOCATION: PT-6
SAMPLE: S-1
DEPTH: 3.0'

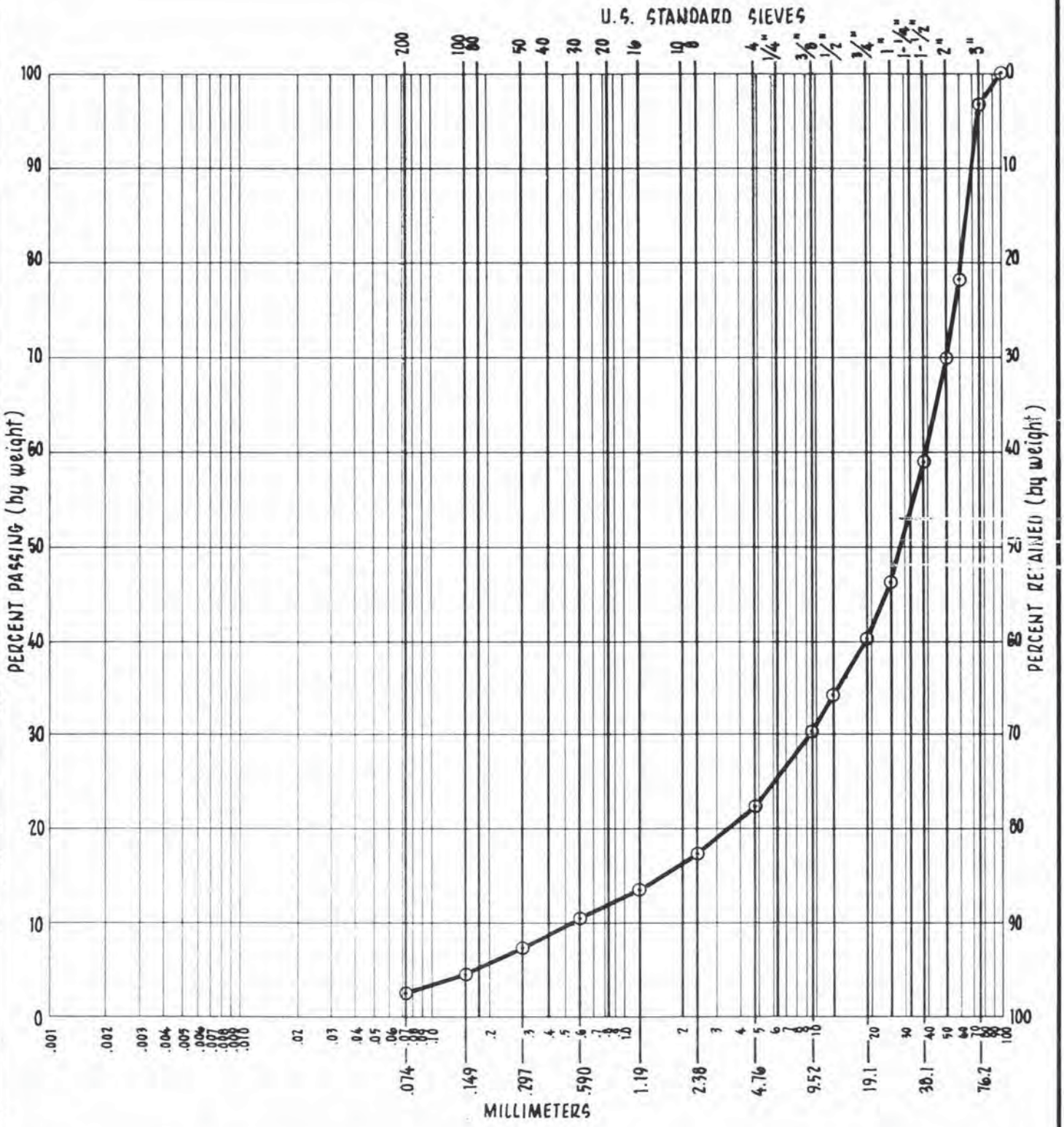
Figure 13

GRAIN SIZE ANALYSIS

3 M PLANTSITE DEVELOPMENT
MOUNTAIN GATE, CALIF.

CH2M HILL

R-14202.A5 MARCH 1981



CLAY TO SILT (FINES)	SAND			GRAVEL		COBBLES
	FINE	MEDIUM	COARSE	FINE	COARSE	

LOCATION: PT-12
SAMPLE: S-1
DEPTH: 3.0'-4.0'

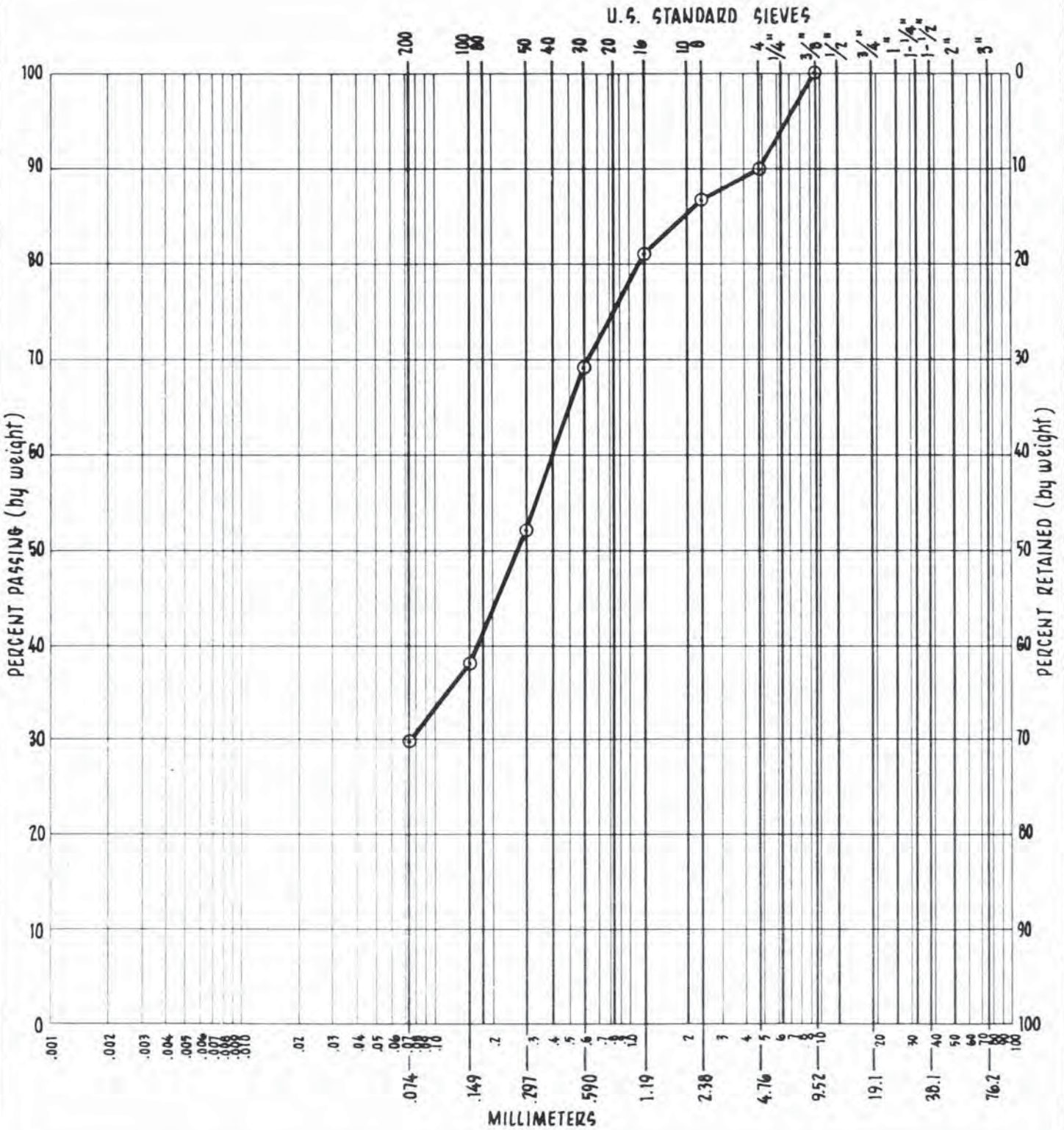
Figure 14

GRAIN SIZE ANALYSIS

3M PLANTSITE DEVELOPMENT
MOUNTAIN GATE, CALIF.

CH2M HILL

R-14202.A5 MARCH 1981



CLAY TO SILT (FINES)	SAND			GRAVEL		COBBLES
	FINE	MEDIUM	COARSE	FINE	COARSE	

LOCATION: PB-3
SAMPLE: ST-1
DEPTH: 9.0'-10.3'

Figure 15

GRAIN SIZE ANALYSIS

3M PLANTSITE DEVELOPMENT
MOUNTAIN GATE, CALIF.

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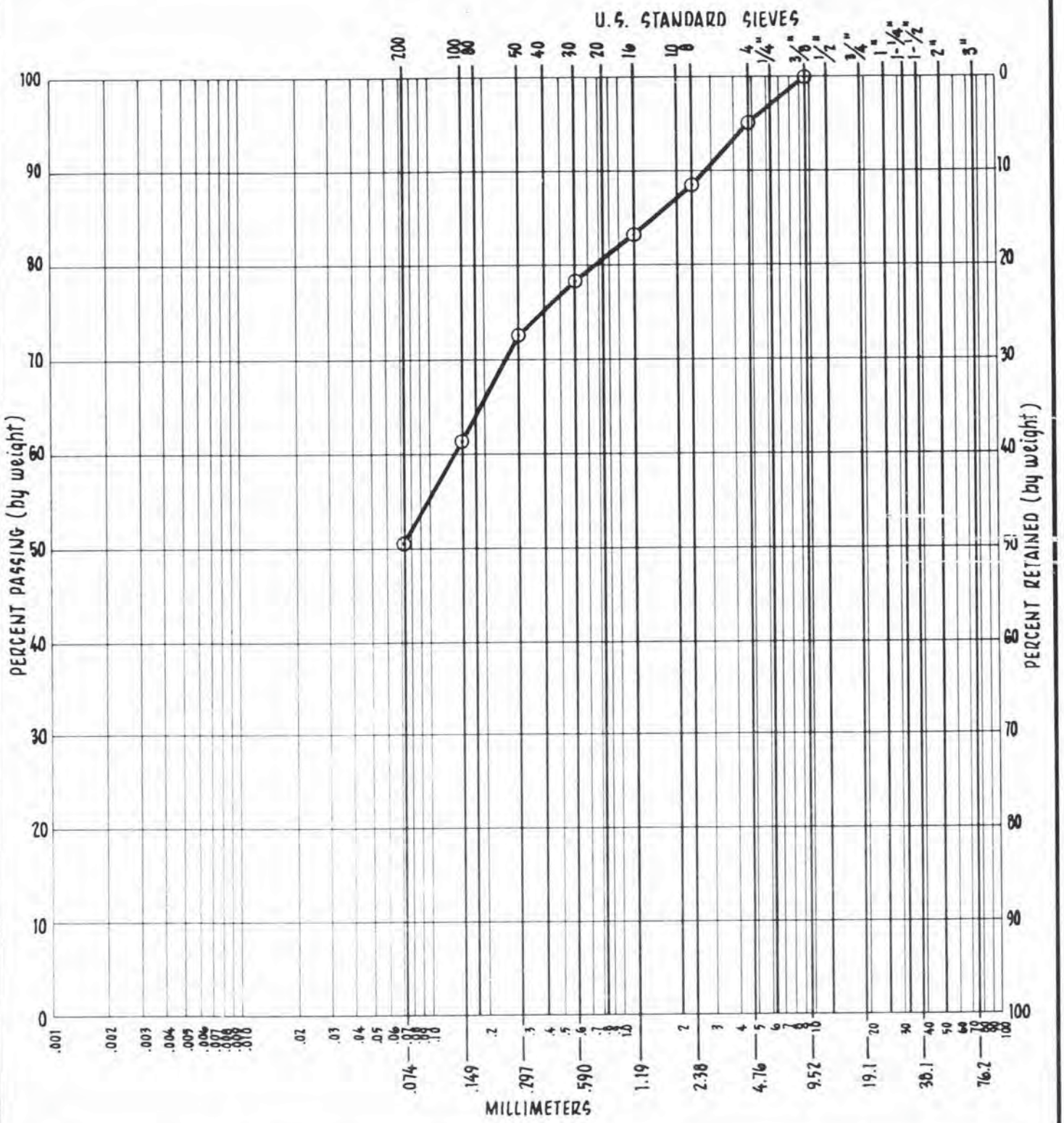

CLAY TO SILT (FINES)	SAND			GRAVEL		COBBLES
	FINE	MEDIUM	COARSE	FINE	COARSE	

LOCATION: WA-1
SAMPLE: S-1
DEPTH: 1.0'

Figure 16

GRAIN SIZE ANALYSIS

3M PLANTSITE DEVELOPMENT
MOUNTAIN GATE, CALIF.

CH2M HILL

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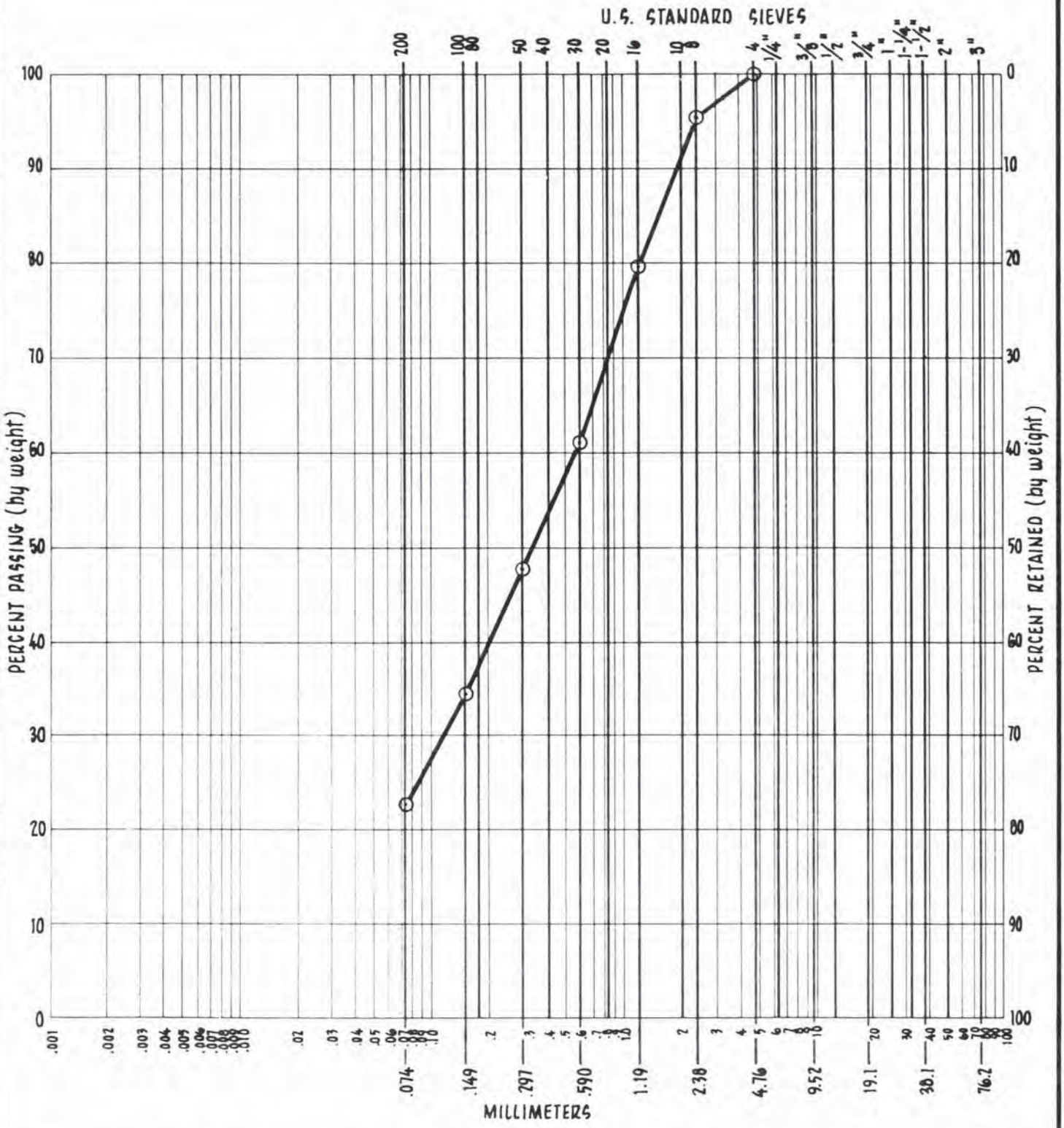


CLAY TO SILT (FINES)	SAND			GRAVEL		COBBLES
	FINE	MEDIUM	COARSE	FINE	COARSE	

LOCATION: WA-10
SAMPLE: S-1
DEPTH: 2.0'-3.0'

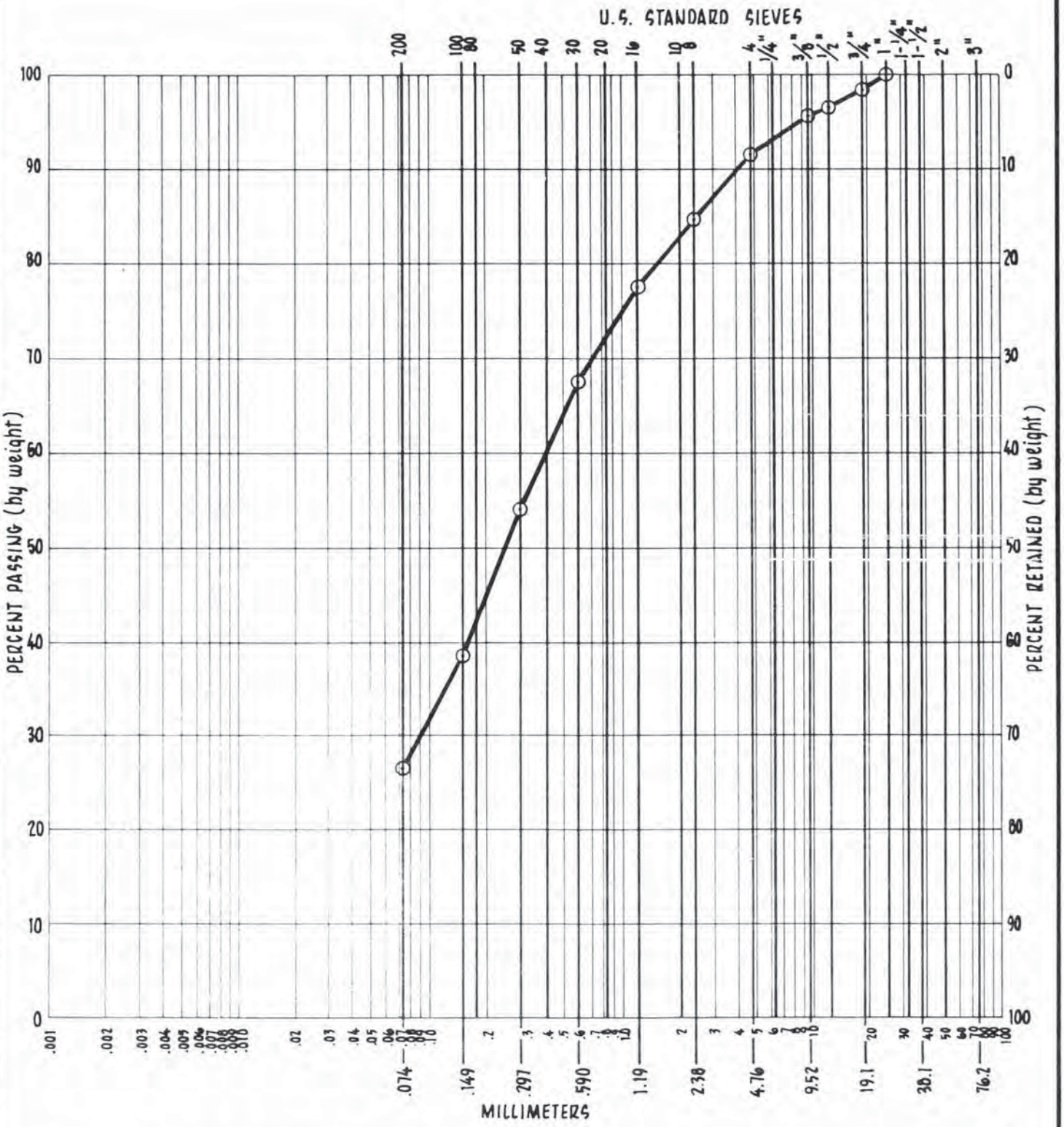
Figure 17

GRAIN SIZE ANALYSIS

3M PLANTSITE DEVELOPMENT
MOUNTAIN GATE, CALIF.

CH2M HILL

R-14202.A5 MARCH 1981



CLAY TO SILT (FINES)	SAND			GRAVEL		COBBLES
	FINE	MEDIUM	COARSE	FINE	COARSE	

LOCATION: EA-7
SAMPLE: S-1
DEPTH: 1.5'-2.0'

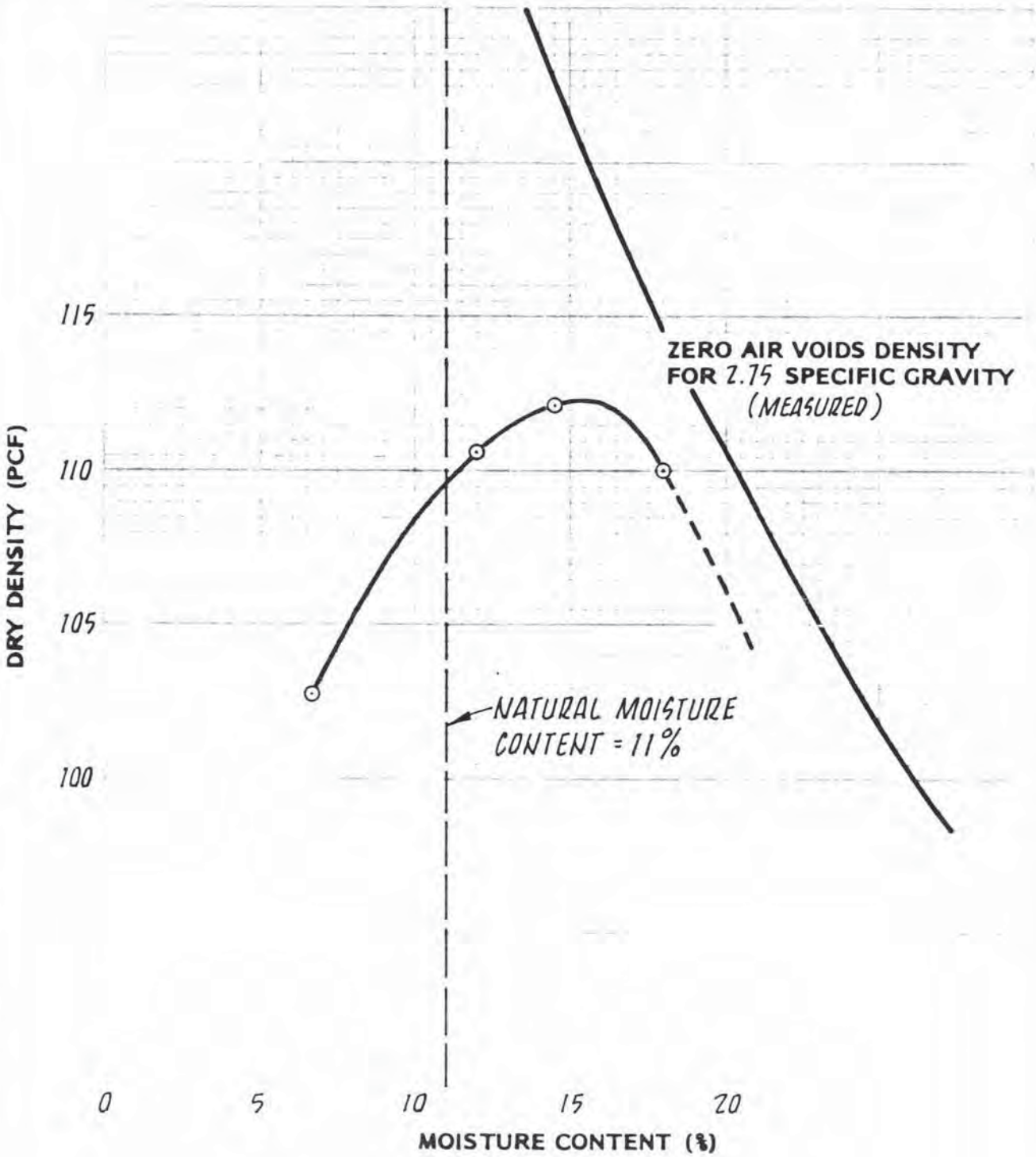
Figure 18

GRAIN SIZE ANALYSIS

3M PLANTSITE DEVELOPMENT
MOUNTAIN GATE, CALIF.

CH2M HILL

R-14202.A5 MARCH 1981



COMPACTION TEST METHOD	MAXIMUM DRY DENSITY (PCF)	OPTIMUM MOISTURE CONTENT (%)
ASTM D1557-A	112	15

LOCATION: WA-1
SAMPLE: S-1
DEPTH: 1.0'-3.0'

Figure 19
COMPACTION TEST
3M PLANTSITE DEVELOPMENT
MOUNTAIN GATE, CALIF.

CH2M HILL

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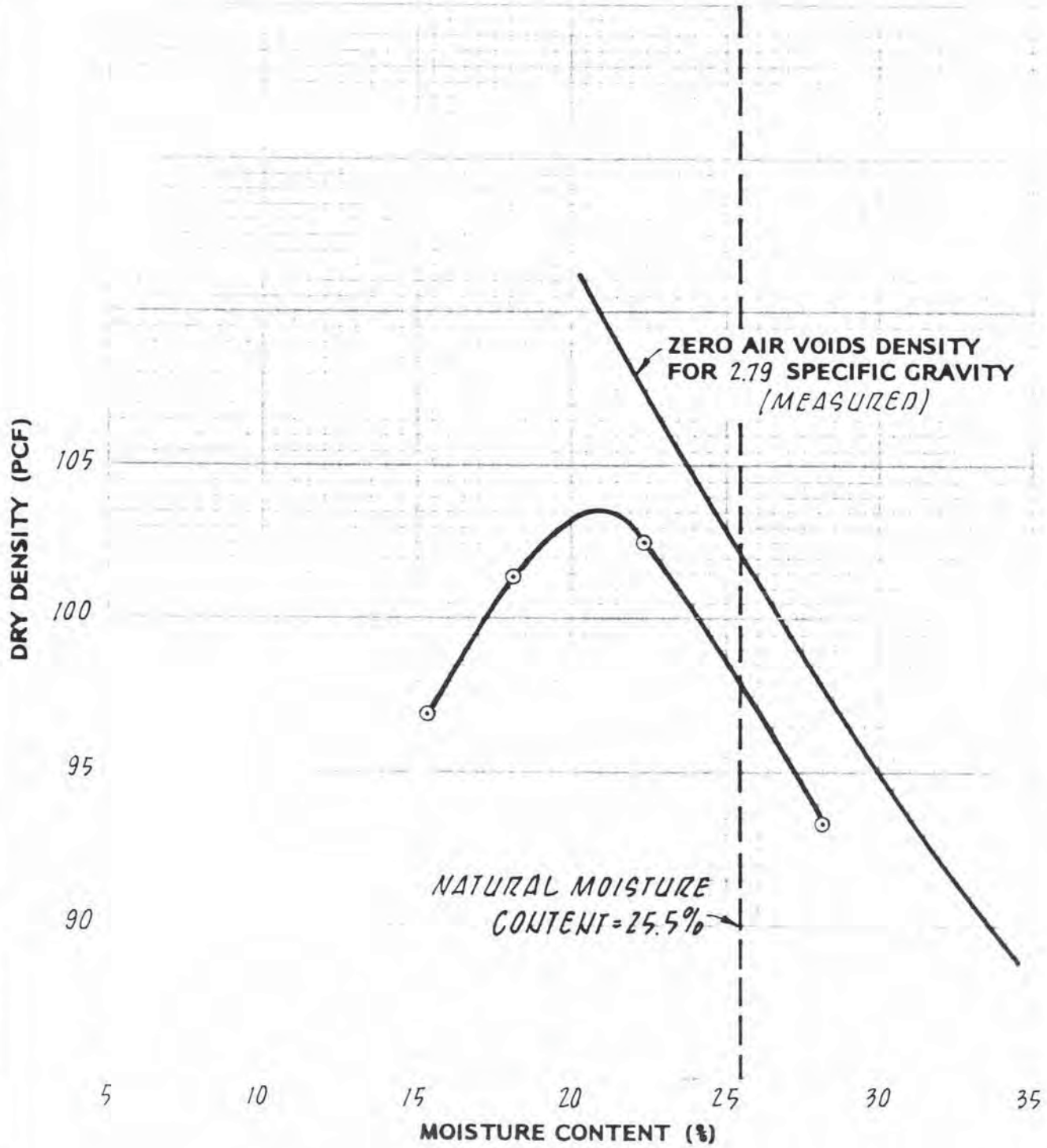

COMPACTION TEST METHOD	MAXIMUM DRY DENSITY (PCF)	OPTIMUM MOISTURE CONTENT (%)
ASTM D1557-C	103.5	21

LOCATION: PT-4
SAMPLE: S-1
DEPTH: 5'

Figure 20

COMPACTION TEST

3M PLANTSITE DEVELOPMENT
MOUNTAIN GATE, CALIF.

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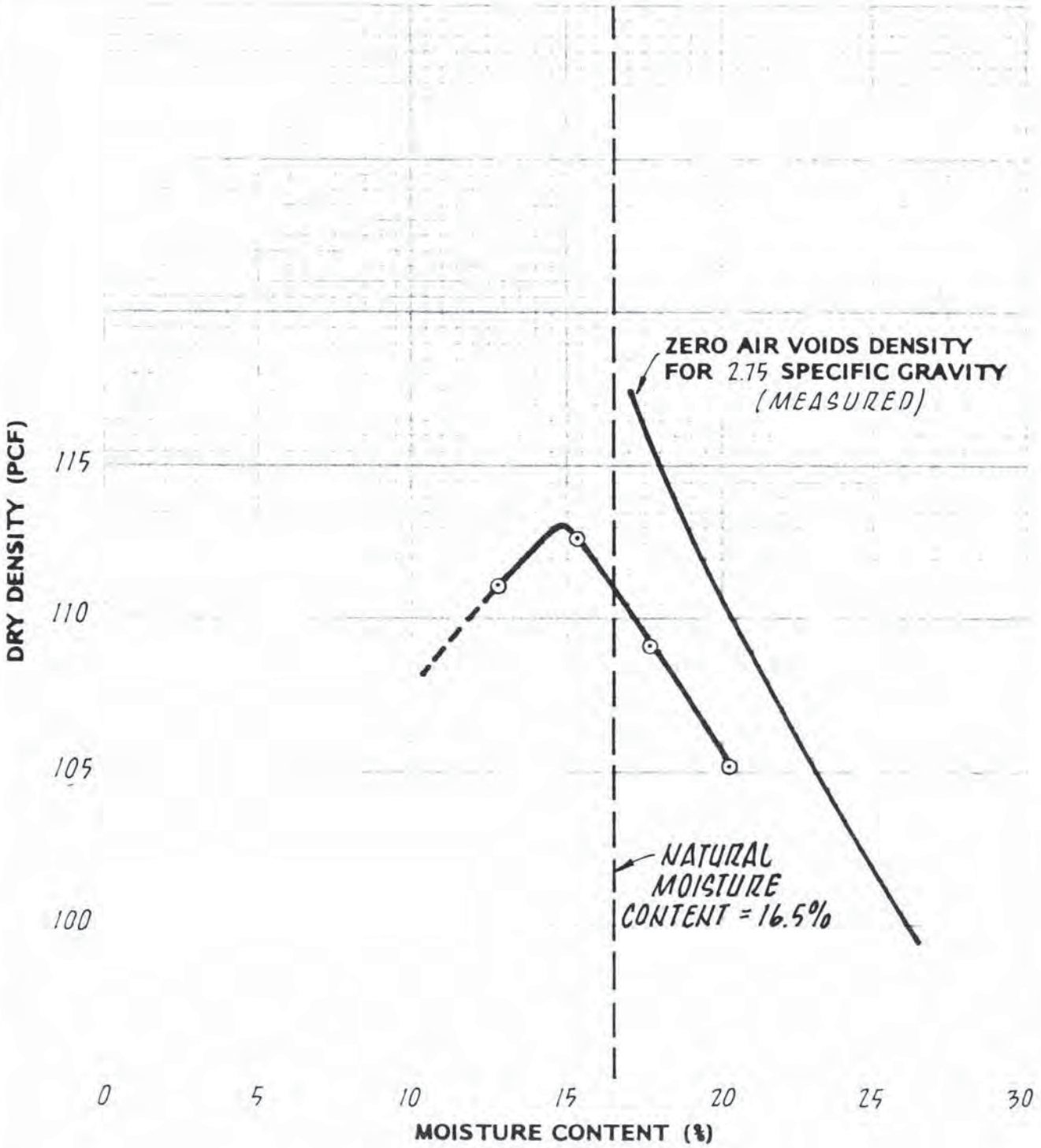

COMPACTION TEST METHOD	MAXIMUM DRY DENSITY (PCF)	OPTIMUM MOISTURE CONTENT (%)
ASTM D1557-C	113	15

LOCATION: EA-7
SAMPLE: S-1
DEPTH: 1.5'-2.0'

Figure 21
COMPACTION TEST
3M PLANTSITE DEVELOPMENT
MOUNTAIN GATE, CALIF.

CH2M HILL

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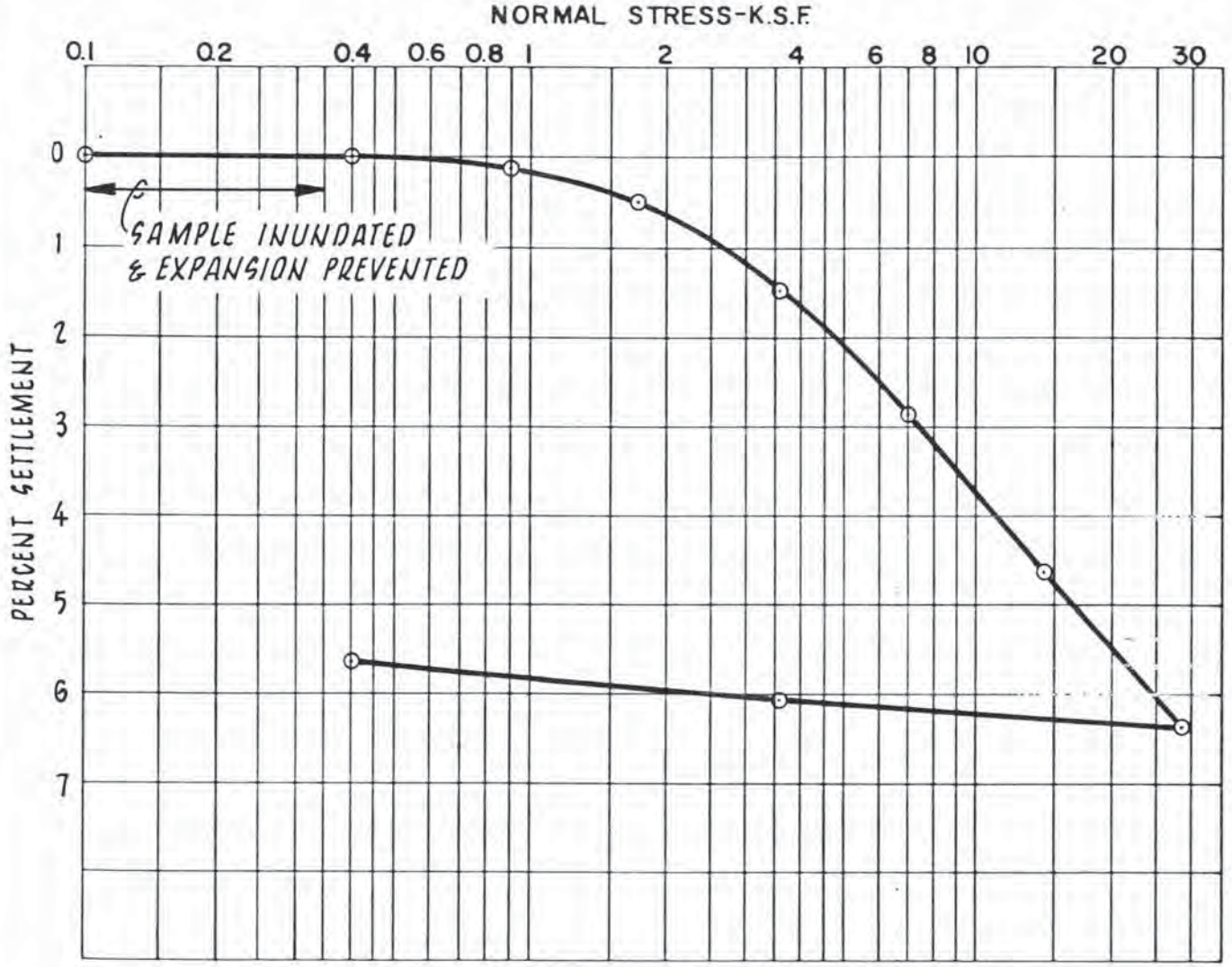

INITIAL CONDITIONS:

MOISTURE CONTENT = 26%
DRY DENSITY = 93.2 pcf
DEGREE OF SATURATION = 83%
VOID RATIO = 0.88

RESULTS:

COMPRESSION INDEX (C_c) = 0.11
RECOMPRESSION INDEX (C_R) = 0.007
PRECONSOLIDATION PRESSURE = 3.5± ksf
OVERCONSOLIDATION RATIO (OCR) = 3±

LOCATION: PB-3
SAMPLE: ST-1
DEPTH: 9.0'-10.3'

Figure 22

CONSOLIDATION TEST

3M PLANTSITE DEVELOPMENT
MOUNTAIN GATE, CALIF.

CH2M HILL

R-14202.A5 MARCH 1981

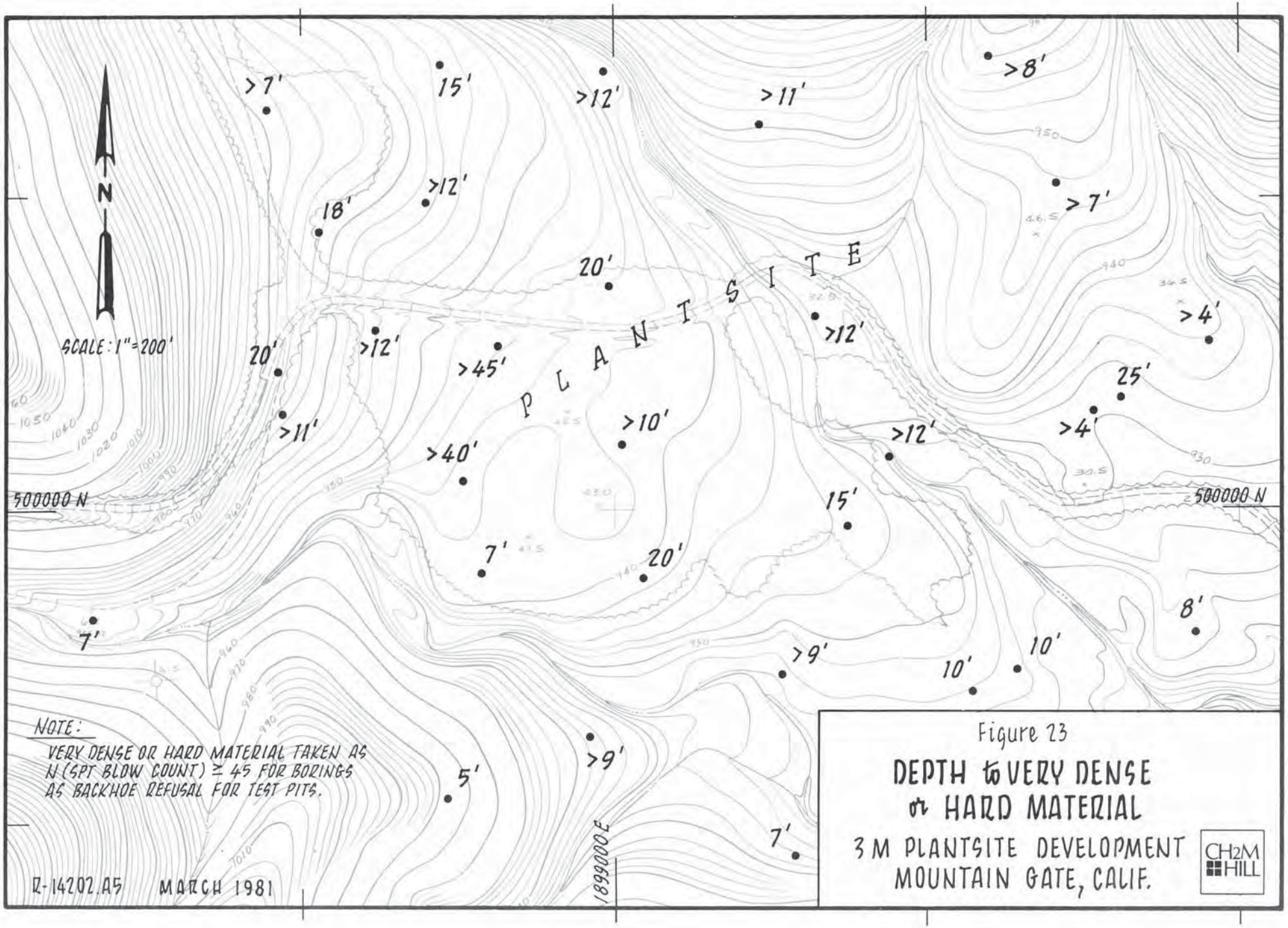

Figure 23

DEPTH to VERY DENSE or HARD MATERIAL

3M PLANTSITE DEVELOPMENT
MOUNTAIN GATE, CALIF.

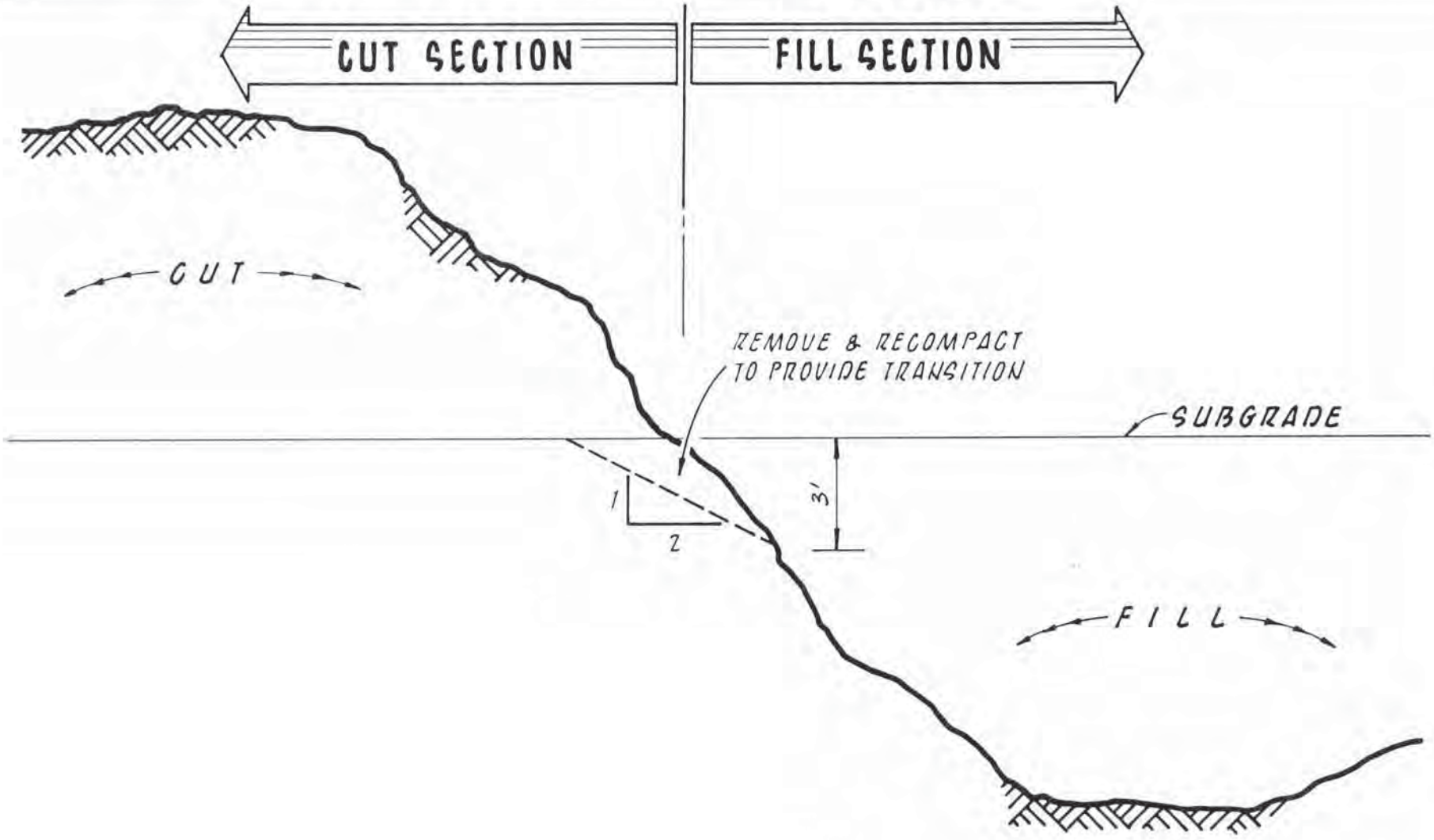

Figure 24

CUT-FILL TRANSITION

3M PLANTSITE DEVELOPMENT

MOUNTAIN GATE, CALIF.

CH2M HILL

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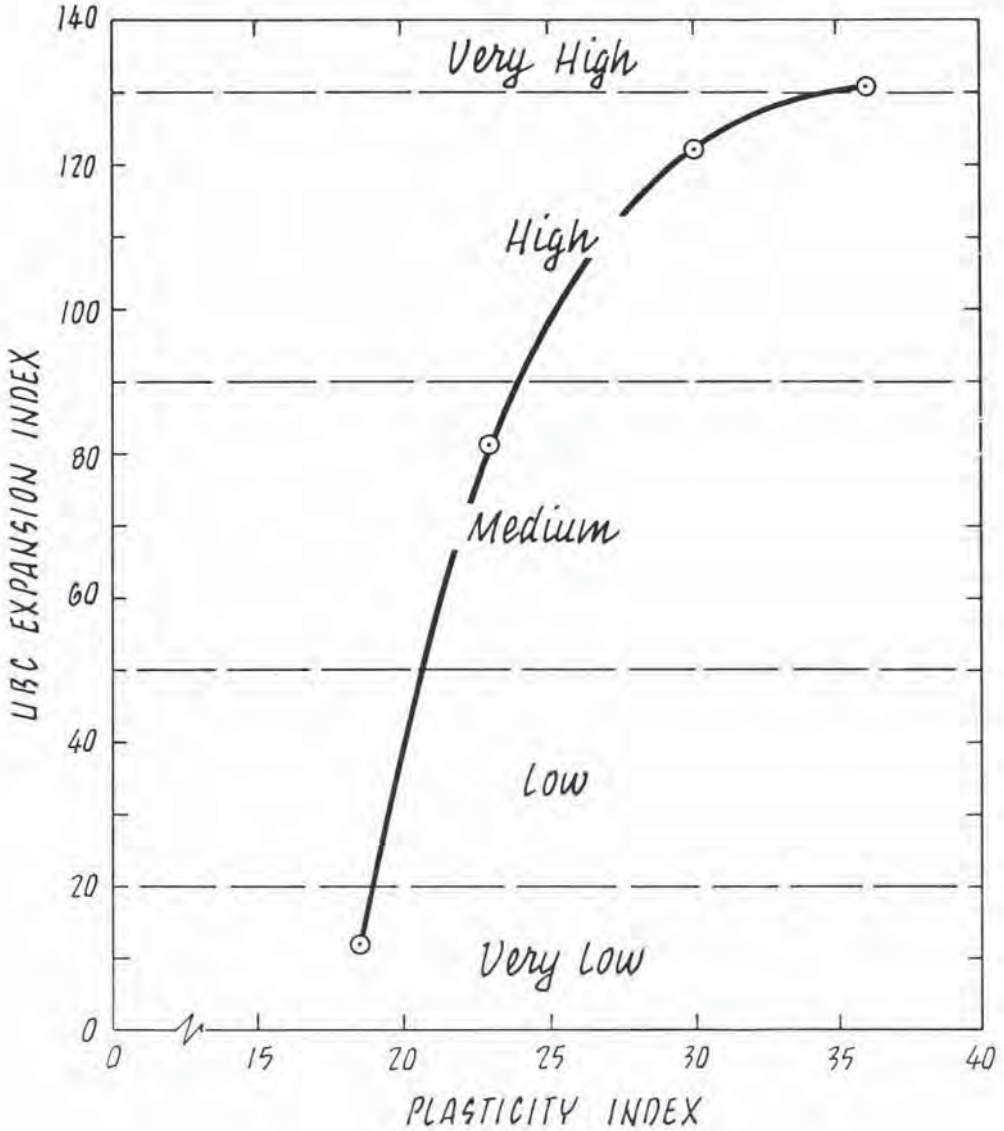

Figure 25

EXPANSION POTENTIAL

3 M PLANTSITE DEVELOPMENT
MOUNTAIN GATE, CALIF.

CH2M HILL

R-14202.A5 MARCH 1981

Appendix B

EXPLORATION AND TESTING TECHNIQUES

Appendix B
EXPLORATION AND TESTING TECHNIQUES

CLASSIFICATION

All test pits and borings were made under the direct supervision of a CH2M HILL staff engineering geologist. Soils were classified in approximate accordance with ASTM D 2488, "Description of Soils (Visual-Manual Procedure)." Classification using the ASTM standard helps establish a uniformity in nomenclature with respect to soil descriptions, but does not ensure it. Although field classifications were verified by laboratory visual classification and laboratory classification testing, not all the soil types were sampled and fewer were tested. Judgment is, therefore, the primary factor in soil classification, and the boring and test pit logs represent the integration of our field classification, laboratory visual classification, and laboratory classification testing. In addition, the natural variability in soils creates a complex picture of the subsurface if precise descriptions are employed for each stratum encountered. Soil descriptions in this report have been simplified for interpretation purposes.

TEST PIT EXCAVATION AND SAMPLING

Test pits were excavated using a Case 580C rubber-tired backhoe with a 2-foot bucket. Disturbed bulk samples were taken of representative soil materials for laboratory testing. Test pits were logged and sampled by a CH2M HILL staff engineering geologist. All test pits were backfilled on completion of logging and sampling.

DRILLING AND SAMPLING

Borings were made by a CME 550 drill rig mounted on an all-terrain vehicle. Fifteen of the sixteen borings were made using 3-3/4-inch I.D. by 7-1/2-inch O.D. hollow augers. One boring was made using a 3-7/8-inch tricone rotary bit and mud rotary methods. The rotary boring used biodegradable drilling mud ("Revert"). All borings were left open for approximately 3 weeks to allow ground-water levels to stabilize, then water level measurements were taken and the holes were backfilled to the surface.

Disturbed soil samples were obtained in the borings by the Standard Penetration Test (SPT), conducted according to American Society for Testing and Materials (ASTM) Specification D 1586, except that the split spoon did not have a ball check-valve or vents. The SPT consists of driving a 2-inch outside diameter split-barrel spoon with a 140-pound hammer dropped 30 inches vertically. The spoon is driven 18 inches, and the number of blows is recorded for each

6 inches. The penetration resistance (N-value) is reported as the number of blows required to drive the last 12 inches or, in the case of extremely hard driving, as the number of blows required to drive the stated distance (example: 58/8"). A free-fall hammer of the "trip monkey" type was used for the sampling.

The sample retained in the spoon is classified and can be used for various index and classification tests. The N-value gives an indication of the sample's consistency and has been very approximately correlated with certain soil characteristics and behavior.

Undisturbed soil samples were obtained in three borings by pushing Shelby tube thin-wall samplers. The tubes used were 3 inches in diameter by 36 inches long. The sample retained in the tube is classified and can be used for index and classification tests or tests which assess the soil's engineering properties such as consolidation or strength.

BORING AND TEST PIT LOCATIONS

Boring and test pit locations were estimated in relation to noticeable topographic features. Locations were determined in the field using a 1" to 200'± enlarged airphoto as a base map. Boring and test pit locations were then transferred to the topographic map (Figure 1) and coordinates and elevations determined. These are reported in Appendix C. Boring and test pit locations, coordinates, and elevations should be considered approximate. Precise locations (e.g. by horizontal and vertical surveying) were not considered appropriate for the preliminary geotechnical exploration in light of the cost involved.

CLASSIFICATION TESTS

Natural moisture contents were determined in accordance with ASTM D 2216 and Atterberg limits determined in accordance with ASTM D 423 and D 424. Grain size analysis was performed in accordance with ASTM C 117 and C 136.

The Atterberg limits are two index tests: the plastic limit and the liquid limit. These are both moisture contents at which a specified laboratory test response is observed. The natural moisture content and Atterberg limits are useful in grouping and classifying soils, and can provide useful correlations for strength, compressibility, and compaction parameters. The grain size distribution of soils is useful in classifying the soil and has been roughly correlated to permeability.

In addition, classification tests generally provide information to identify and correlate similar soil types on which more sophisticated and costly laboratory tests are performed.

COMPACTION TEST(S)

Compaction tests were conducted in accordance with ASTM D 1557. This test consists of compacting soil into a mold with a 10-pound hammer dropped 18 inches vertically. The mold is filled in five approximately equal layers, each receiving 25 blows from the hammer. The density and moisture content of the compacted soil are then determined. The test is repeated at different soil moisture contents and the relationships shown on Figures 19 through 21 are generated. Compaction tests are useful in assessing the moisture sensitivity and moisture conditioning requirements for potential fill materials.

SPECIFIC GRAVITY OF SOLIDS

The specific gravity of solids was determined according to ASTM D 854. The specific gravity was necessary to construct the zero air voids curve for compaction tests and to calculate the degree of saturation and void ratio for the consolidation test.

CONSOLIDATION TEST

A consolidation test was performed on Sample ST-1 from Boring PB-3. This test was conducted in accordance with ASTM D 2435 with the following exceptions and additions.

- The test was performed on an undisturbed Shelby-tube sample.
- Samples were inundated at a seating pressure of 100 psf and confined to prevent swell by the intermittent addition of load.
- No interpretation was made of time rate results except to establish t_{100}.
- The resulting percent settlement versus log of pressure curve represents the total movements recorded for a load duration of 30 minutes.
- t_{100} was generally less than 2 minutes.
- Loads were generally maintained for 24 hours.

A consolidation test was attempted on Sample ST-3 from Boring PB-3. This sample contained a relic rock structure with partially weathered rock fragments. Upon loading, the sample compressed unevenly, rendering the results uninterpretable.

R-VALUE TEST(S)

R-value tests were conducted in accordance with ASTM D 2844 to evaluate pavement design parameters.

UNCONFINED COMPRESSION TESTS

Unconfined compression tests were performed on disturbed split-spoon samples. Tests were performed using the controlled stress method in accordance with ASTM D 2166.

CORROSION TESTING

Test samples for corrosion testing were first air-dried, then pulverized and passed over a No. 50 sieve. The minus No. 50 fraction was used in subsequent testing.

pH was determined by mixing 20 grams of prepared soil with 20 mls of pH7 deionized water. This mixture was stirred three times in a period of 30 minutes, then cured for 1 hour, after which the pH was measured by a Model 701 Orion Specific Ion Meter.

Resistivity and sulphate concentration were determined by mixing 100 grams of prepared soil with 300 mls of deionized water. This mixture was allowed to stand overnight, then was filtered, producing a clear aqueous extract.

Resistivity was then measured on the aqueous extract in approximately accordance with ASTM D 1125, Method A.

Sulphate concentration was determined on the aqueous extract by the turbidimetric method conducted in accordance with ASTM 516, Method B.

EXPANSION TESTS

Soil expansion characteristics were determined in accordance with the Uniform Building Code (UBC) Expansion Test Standard 29-2. The UBC Expansion Test is a confined-swell index test performed on the minus No. 4 fraction of the sample. The sample is compacted at a moisture content producing 50-percent saturation (±1 percent) by compacting with a 5.5-pound hammer dropped 12 inches vertically. Compaction takes place in two lifts, each approximately 1 inch high, with 15 blows of the hammer per lift. A 1-inch-high specimen is then trimmed and set in a floating ring consolidometer. A 144-psf (1-psi) confining pressure is applied and the sample is inundated. Expansion is measured when the sample stabilizes and the expansion index is reported as:

$$E.I. = \frac{\Delta t}{t_o} \times 1{,}000$$

where,

$E.I.$ = Expansion index
Δt = Change in thickness
t_o = Initial thickness

The expansion index has been correlated to expansion potential and the results are generally reported in the form of index values; medium, high, etc.

Tests to predict swell pressure or potential expansion (heave) were not conducted for the preliminary exploration. To produce meaningful results from these tests, the following conditions must be known or estimated:

- Imposed structure load
- Depth (after site grading)
- Initial moisture content
- Expected moisture change

At this time, only the initial moisture content is known. To perform confined swell tests without accurate knowledge of the other variables would not have been appropriate for the preliminary exploration. We anticipate confined swell tests will be necessary for final design and recommend they be performed after the significant variables are estimated.

Appendix C

BORING AND TEST PIT COORDINATES AND ELEVATIONS

Appendix C
BORING AND TEST PIT COORDINATES AND ELEVATIONS

Boring or Test Pit	North-South Coordinate	East-West Coordinate	Ground Surface Elevation (feet)
PB-1	500220N	1898470E	966
PB-2	500040N	1898760E	943
PB-3	500260N	1898820E	946
PB-4	500440N	1898530E	962
PB-5	500350N	1899000E	944
PB-6	500710N	1898730E	955
PB-7	499890N	1899050E	939
PB-8	499980N	1899370E	933
PB-9	499740N	1899640E	924
PB-10	500170N	1899820E	931
OB-1	501050N	1902230E	968
OB-2	501150N	1902340E	964
EB-1	500980N	1902980E	933
EB-2	500850N	1903790E	896
EB-3	500410N	1905120E	876
EB-4	500510N	1905310E	875
PT-1	500640N	1898450E	969
PT-2	500150N	1898470E	963
PT-3	499820N	1898160E	966
PT-4	500280N	1898620E	954
PT-5	499900N	1898790E	941
PT-6	500100N	1899020E	942
PT-7	499730N	1899270E	941
PT-8	499700N	1899570E	926
PT-9	500080N	1899440E	926
PT-10	500610N	1899240E	950
PT-11	501260N	1899090E	1,020
PT-12	500700N	1899000E	938
PT-13	499800N	1899930E	923
PT-14	499740N	1900430E	915
PT-15	499720N	1900190E	916
PT-16	500300N	1899320E	932
PT-17	500490N	1898710E	956
PT-18	499230N	1900000E	902
PT-19	499300N	1899840E	911
PT-20	499430N	1899620E	917
PT-21	499440N	1899280E	933
PT-22	499640N	1898960E	
PT-23	499540N	1898730E	978
PT-24	500150N	1899770E	930

Boring or Test Pit	North-South Coordinate	East-West Coordinate	Ground Surface Elevation (feet)
WA-1	501030N	1901340E	928
WA-2	501940N	1901040E	947
WA-3	500190N	1900540E	935
WA-4	500480N	1900660E	948
WA-5	501190N	1901560E	942
WA-6	500510N	1899720E	948
WA-7	500710N	1899600E	958
WA-8	500520N	1900060E	947
WA-9	500350N	1900180E	938
WA-10	500140N	1900110E	939
WA-11	500260N	1899950E	936
WA-12	500250N	1900870E	923
WA-13	500420N	1901000E	933
WA-14	500570N	1901160E	936
WA-15	500740N	1901020E	937
EA-1	500600N	1905540E	872
EA-2	500840N	1904990E	880
EA-3	500940N	1903560E	892
EA-4	501120N	1902850E	935
EA-5	500900N	1902670E	950
EA-6	500940N	1902960E	939
EA-7	500920N	1904200E	890

Note: All coordinates and elevations are approximate. See Appendix B for a description of boring and test pit locations.