FAA-RD-78-59

LEVEL VI

75th ANNIVERSARY OF POWERED FLIGHT · 1903-1978

CHARACTERIZATION OF EXPANSIVE SOILS
FOR AIRPORT PAVEMENT DESIGN

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DEPARTMENT OF TRANSPORTATION · UNITED STATES OF AMERICA

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AUGUST 1978

INTERIM REPORT
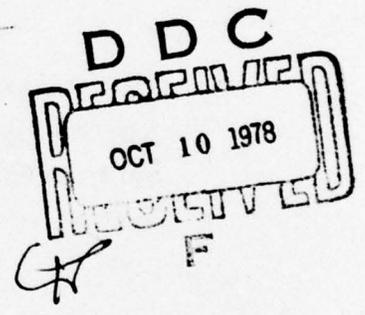
Prepared for

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL AVIATION ADMINISTRATION
Systems Research & Development Service
Washington, D.C. 20590

78 10 06 029

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Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FAA-RD-78-59		

4. Title and Subtitle		5. Report Date
CHARACTERIZING EXPANSIVE SOILS FOR AIRPORT PAVEMENT DESIGN		August 1978
		6. Performing Organization Code

7. Author(s)	8. Performing Organization Report No.
R. Gordon McKeen and John P. Nielsen	CERF-AP-28

9. Performing Organization Name and Address	10. Work Unit No. (TRAIS)
Eric H. Wang Civil Engineering Research Facility, University of New Mexico, Box 25, University Station, Albuquerque, NM 87131	
	11. Contract or Grant No.
	DOT-FA75WAI-531

12. Sponsoring Agency Name and Address	13. Type of Report and Period Covered
U.S. Department of Transportation Federal Aviation Administration Systems Research and Development Service Washington, D.C. 20590	Interim Report February 1977 to April 1978
	14. Sponsoring Agency Code

15. Supplementary Notes

12 I 26 p.

9 Interim rept. Feb 77 – Apr 78

16. Abstract

Characteristics of soil expansion combined with environmental conditions are responsible for differential heaving of airport pavement subgrades. Despite a large technical effort centered on the study of expansive soils, a rapid means of evaluating potential damage is not available. In this study, a reliable rapid method of categorizing expansive soils was sought. Three procedures are recommended: (1) measurement of bulk density change in natural soil clods, (2) determination of clay content or (3) determination of the moisture-suction relationship with particular attention to aggregation. Each of these procedures was developed through correlations with soil compressibility with respect to suction changes, γ_h. This is a fundamental characteristic of the soil and the best indicator of potential expansion. Actual activity depends on imposed loads, initial suction, and final suction. The major obstacle to satisfactory development of this system remains the relation between differential heave and airport pavement roughness. While this problem is to be addressed in future research, there is a present need for criteria. The most acceptable criteria found were categories developed for application to residential concrete slabs on expansive soils. The limitations of this system are recognized but accepted as the best presently available.

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6 Characterization of Expansive Soils for Airport Pavement Design.

17. Key Words		18. Distribution Statement
Expansive Soils Bulk Density Clay Soil Testing Classification Swell Potential Soil Suction		Document is available to the U.S. public through the National Technical Information Service, Springfield, Virginia 22161.

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	119	

Form DOT F 1700.7 (8-72)   Reproduction of completed page authorized

METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in²	square inches	6.5	square centimeters	cm²
ft²	square feet	0.09	square meters	m²
yd²	square yards	0.8	square meters	m²
mi²	square miles	2.6	square kilometers	km²
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft³	cubic feet	0.03	cubic meters	m³
yd³	cubic yards	0.76	cubic meters	m³
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.

Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm²	square centimeters	0.16	square inches	in²
m²	square meters	1.2	square yards	yd²
km²	square kilometers	0.4	square miles	mi²
ha	hectares (10,000 m²)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m³	cubic meters	35	cubic feet	ft³
m³	cubic meters	1.3	cubic yards	yd³
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F

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SECTION 1
INTRODUCTION

BACKGROUND

The Federal Aviation Administration initiated a review of the technical literature on expansive soils in 1975 in order to develop an airport pavement design manual for expansive soil areas. The review was conducted by the Civil Engineering Research Facility of the University of New Mexico (CERF/UNM). The conclusions from that study indicated that some developmental and technology transfer work were required to apply the recent developments from expansive soils research to the problem of airport pavement design. It was also concluded that no further research should be initiated to study the design of lime and cement stabilized layers because an Air Force Study, completed in 1976, had culminated five years of research in this area (Ref. 1).

The developmental work recommended in 1976 was begun in February 1977 at CERF/UNM. It consists of a three-phase research program. Phase 1 is the review of research literature. Phase 2 is intended to provide improved methods of characterizing expansive soils for airport pavement design. Phase 3 consists of the development of a manual for studying moisture retention force (suction) profiles of in situ subgrades. Following the development of a manual, further studies of strain behavior, including evaluation of stabilizers, will be conducted. Phase 4 consists of a study of the pavement roughness and its relationship to the heave characteristics of the subgrade. This phase is intended to result in refined methods of establishing stabilization objectives. The present report presents the results of work on Phase 2 of this project.

OBJECTIVES

1. Evaluate those indicator tests that were found to have merit in FAA-RD-76-66 in order to improve their use as indicators of potential swell. Reliability, speed, and required skill and equipment are considerations that should be evaluated.

2. Coordinate these evaluations with other agencies involved in expansive soils research to avoid duplication of effort.

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SCOPE

This report presents results of laboratory evaluation of several techniques to determine their usefulness in evaluating swell characteristics of airport pavement subgrades. It is not intended to be a comprehensive study of the subject of soil characterization. Only those ideas of potential use in airport pavement design are considered.

SECTION 2
INDICATOR TEST CONSIDERATIONS

WHAT IS AN EXPANSIVE SOIL?

Expansive soils are those that cause early pavement distress due to swelling or shrinking (volume change) of the subgrade soil. While this definition seems simple, it cannot be applied easily to practical problems. One of the clearest findings in studying the technical literature is that it is difficult to quantify the definition of expansive soils. To gain a first approximation of the state of research, a review was made of a number of completed or ongoing "expansive soils" research projects reported in the technical literature. A review of the results of these studies concluded that Atterberg Limits are still the best general indicators of potential expansion (Ref. 2). Figure 1[*] shows plasticity index histograms of several *expansive soil* research projects and sites where expansive soil problems have occurred (i.e., building damage). In comparing these soil property distributions, two conclusions are obvious: (1) the mean plasticity index may vary between 22 and 65 for problem soils, i.e., *expansive* soils; and (2) the range of variation within a given project site varies tremendously. The range for Yazoo Clay is 65, for example; while for the Lance Creek Study, it is 25. It should be kept in mind that each of these sets of data represents a field site that was under study because it was *expansive*.

Clearly the study of expansive soils must include aspects other than the soil properties measured in the laboratory. Soil characterization must include pertinent aspects of the in situ soil. These aspects include the moisture condition and range as well as soil response to the expected changes of load and moisture.

In most work to date, the swell of an expansive soil is evaluated by a unidimensional test. The soil is laterally confined in a ring, inundated, and permitted to swell against a restraining force of varying magnitude depending on test objectives. The swell is reported as void ratio or height changes and plotted as illustrated in Figure 2. It is assumed here that in situ soil behaves as if it were laterally confined and that the soil is saturated when swell stops. Swell is usually said to have stopped when $\Delta L < 0.001$ to 0.0001 inches in a 24-hour period. Expansive soils are thus defined in terms of the swell in this sort of test.

---

[*]Figure 1, p. 4, contains References 3, 4, 5, 6, 7, 8, and 9.

Mancos Shale
Foundation Invest.
n = 25 $\bar{x}$ = 32
REF 3

Pierre Shale
Lance Creek Study
n = 59 $\bar{x}$ = 24
REF 4

Taylor Marl
Waco Ponding Project
n = 68 $\bar{x}$ = 51
REF 6

Beaumont Clay
Foundation Investigation
n = 124 $\bar{x}$ = 34
REF 7

Eagleford Shale
HUD Slab Project
n = 37 $\bar{x}$ = 34
REF 5

Taylor Marl
HUD Slab Project
n = 50 $\bar{x}$ = 42
REF 5

Yazoo Clay
Miss Ponding Project
n = 79 $\bar{x}$ = 65
REF 8

Chinle Shale
Electroosmosis Project
n = 18 $\bar{x}$ = 22
REF 9

FREQUENCY

PLASTICITY INDEX

FIGURE 1. PLASTICITY INDEX HISTOGRAMS FOR "EXPANSIVE SOIL" PROJECTS
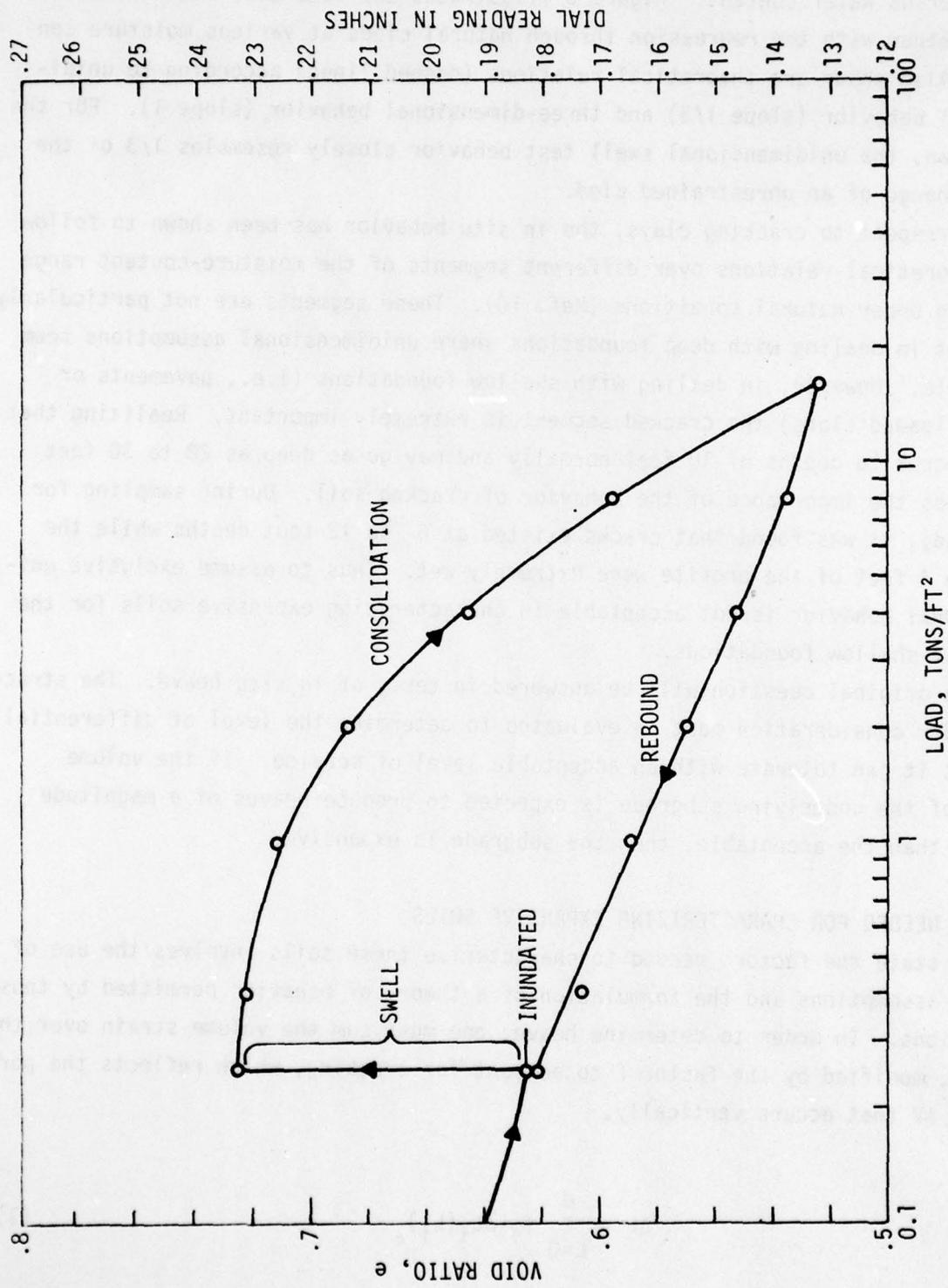
An alternative way of studying the soil behavior is to plot specific volume $(1/\gamma_d)$ versus water content. Figure 3 illustrates the same data from the swell test together with the regression through natural clods at various moisture contents. Also shown are theoretical relations (dashed lines) according to unidimensional behavior (slope 1/3) and three-dimensional behavior (slope 1). For the soil shown, the unidimensional swell test behavior closely resembles 1/3 of the volume change of an unrestrained clod.

In respect to cracking clays, the in situ behavior has been shown to follow both theoretical relations over different segments of the moisture-content range occurring under natural conditions (Ref. 10). These segments are not particularly important in dealing with deep foundations where unidimensional assumptions seem reasonable. However, in dealing with shallow foundations (i.e., pavements or lightly loaded slabs) the cracked segment is extremely important. Realizing that cracks occur to depths of 10 feet normally and may go as deep as 20 to 30 feet emphasizes the importance of the behavior of cracked soil. During sampling for this study, it was found that cracks existed at 6- to 12-foot depths while the top 3 to 4 feet of the profile were extremely wet. Thus to assume exclusive unidimensional behavior is not acceptable in characterizing expansive soils for the design of shallow foundations.

The original question will be answered in terms of in situ heave. The structure under consideration must be evaluated to determine the level of differential movement it can tolerate with an acceptable level of service. If the volume change of the underlying subgrade is expected to produce heaves of a magnitude greater than the acceptable, then the subgrade is expansive.

FACTORS NEEDED FOR CHARACTERIZING EXPANSIVE SOILS

To state the factors needed to characterize these soils involves the use of certain assumptions and the formulation of a theory of behavior permitted by those assumptions. In order to determine heave, one must sum the volume strain over the profile, modified by the factor f to account for cracking, which reflects the portion of ΔV that occurs vertically,

$$\Delta H = \sum_{L=0}^{d} f_i \cdot \Delta L_i (L_i)_0 \tag{1}$$
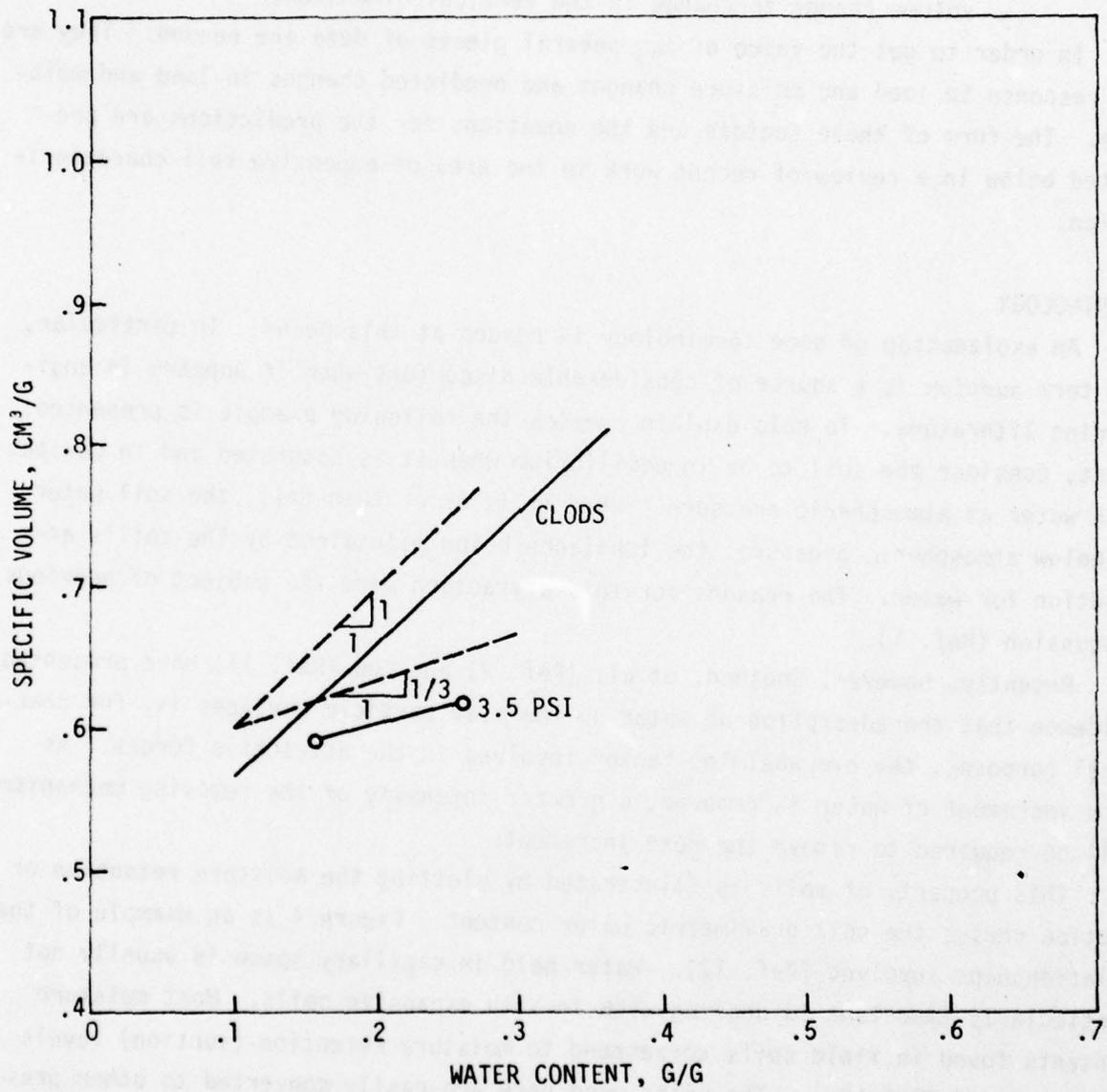
FIGURE 3. ALTERNATIVE DATA PRESENTATION

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where

ΔH = total heave

d = depth below the surface

ΔL_i = vertical dimension change for the ith layer, % as a decimal

$(L_i)_0$ = original thickness of the ith layer

f_i = factor between 1 and 1/3, depending on cracking, which relates volume change to change in the vertical dimension.

In order to get the value of ΔL_i several pieces of data are needed. They are the response to load and moisture changes and predicted changes in load and moisture. The form of these factors and the equations for the predictions are presented below in a review of recent work in the area of expansive soil characterization.

TERMINOLOGY

An explanation of some terminology is needed at this point. In particular, the term *suction* is a source of considerable discomfort when it appears in engineering literature. To help explain *suction* the following example is presented. First, consider the soil to be in equilibrium when it is saturated and in contact with water at atmospheric pressure. When it is drier than this, the soil water is below atmospheric pressure, the imbalance being maintained by the soil's attraction for water. The reasons for this attraction were the subject of previous discussion (Ref. 1).

Recently, however, Snethen, et al. (Ref. 2) and Low (Ref. 11) have presented evidence that the adsorption of water on the clay particle surfaces is, for practical purposes, the overwhelming factor involved in the attractive forces. As each increment of water is removed, a greater intensity of the removing mechanism will be required to remove the next increment.

This property of soils is illustrated by plotting the moisture retention or suction versus the soil gravimetric water content. Figure 4 is an example of the relationships involved (Ref. 12). Water held in capillary space is usually not particularly important in dealing with in situ expansive soils. Most moisture contents found in field soils correspond to moisture retention (suction) levels of 3-5 pF (98-9800 kPa). The units used here are easily converted to other pressure units as shown in Figure 5.

In this report the terms *moisture retention* and *suction* are most often used. Units shown are pF, and kPa primarily. The pF (Ref. 13) is the logarithm to the

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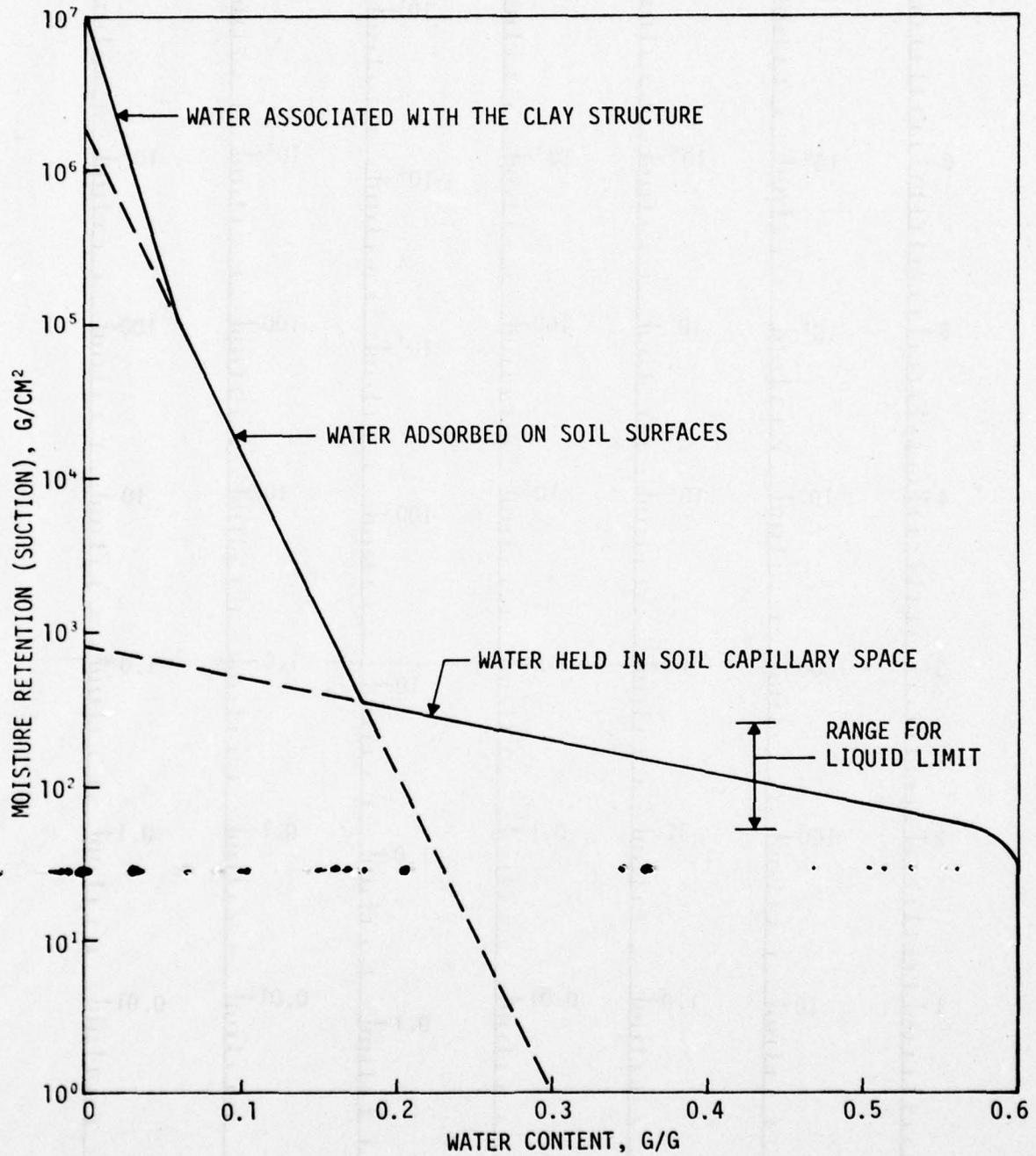
FIGURE 4. MODEL OF MOISTURE-SUCTION BEHAVIOR
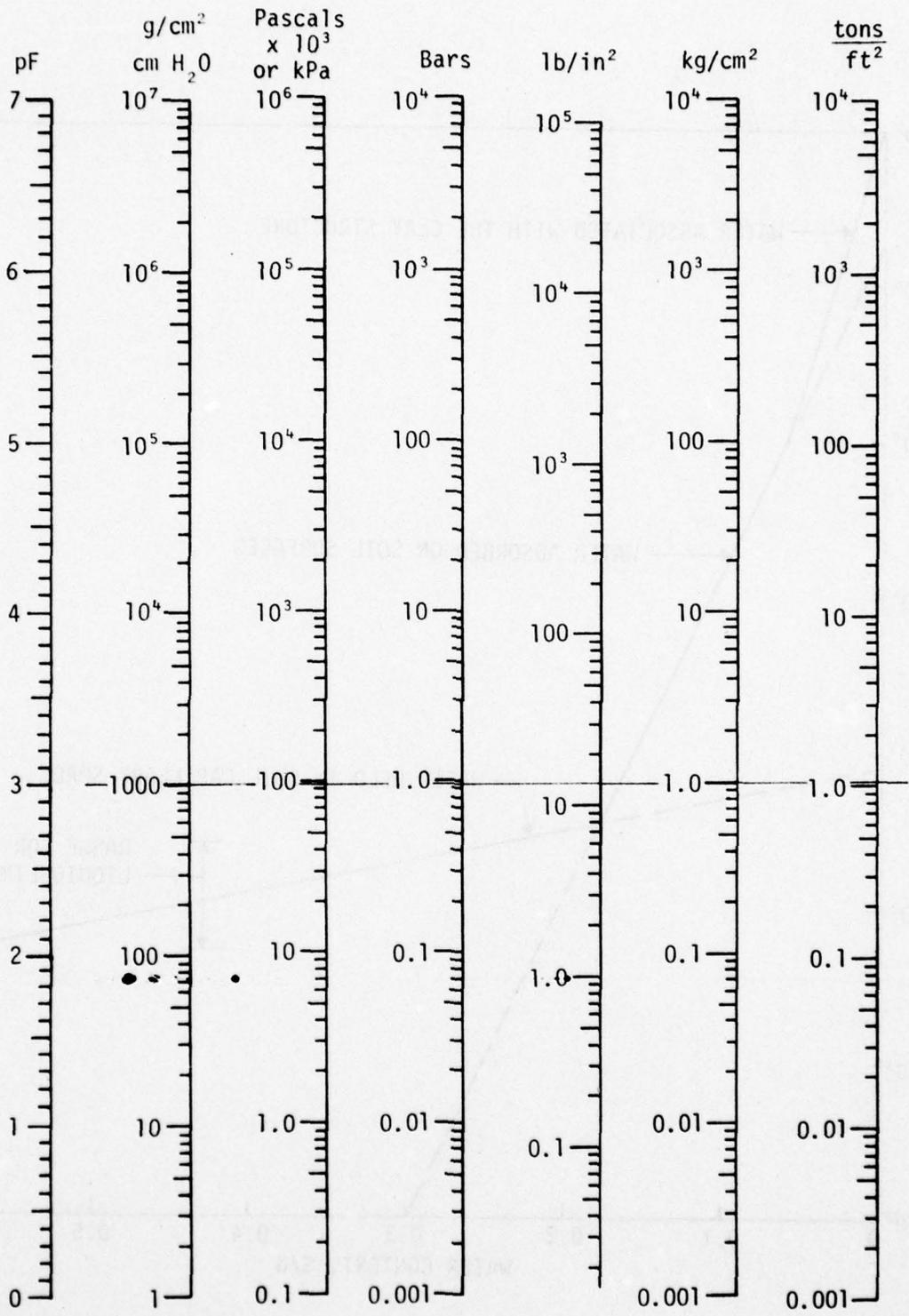
FIGURE 5. VARIOUS SCALES FOR REPORTING SUCTION VALUES

base 10 of the suction measured in centimeters of water or grams per centimeter squared.

RECENT WORK

As reported previously (Ref. 1), the use of strain-suction to characterize expansive soils has provided excellent results for slab foundations. Findings indicated the slope of this relationship does not vary greatly with soil type or overburden loading within the range of pavement loading. A search of the literature provided data to support this assertion.

Figure 6[*] shows data from several sources found in the literature. These data represent a variety of loads (0.7 to 3.5 psi) and were conducted at three different laboratories in three different countries. Data not included were unloaded cases and some samples compacted at dry conditions. All data represent remolded samples.

The conclusions reached from this plot are: (1) the slope varies from about 2 to 1.4 for the conditions represented; (2) as loads increase the slopes are reduced; (3) the slope may not vary greatly from soil to soil thus providing considerable simplicity in swell prediction. Since the equipment and procedures are complex, the measurement of the slope for design purposes is not practical.

The U.S. Army Waterways Experiment Station (WES) has just completed an extensive study of expansive soils for the Federal Highway Administration (Refs. 2, 19, 20). Results published so far by WES have led to a classification system based on correlation of liquid limit, plasticity index, and natural suction with swell as measured in a one-dimensional swell test (Ref. 20). While the end result of this system will provide a categorization of expansive soils, the design portion of the study is still not available. The concept of the WES system is presented in Reference 21. It involves characterizing soil response to moisture changes in terms of the suction-water content relationship and in terms of the specific volume-water content relationship, when unrestrained clods are used. Linear strain is predicted by using the following relation,

$$\frac{\Delta H}{H_0} = \frac{C\tau}{1 + e_0} [(A - Bw_0) - \log(\tau_{mf} + \alpha\sigma_f)] \qquad (2)$$

_____

[*]Figure 6, p. 12, contains References 14, 15, 16, 17, and 18.

11
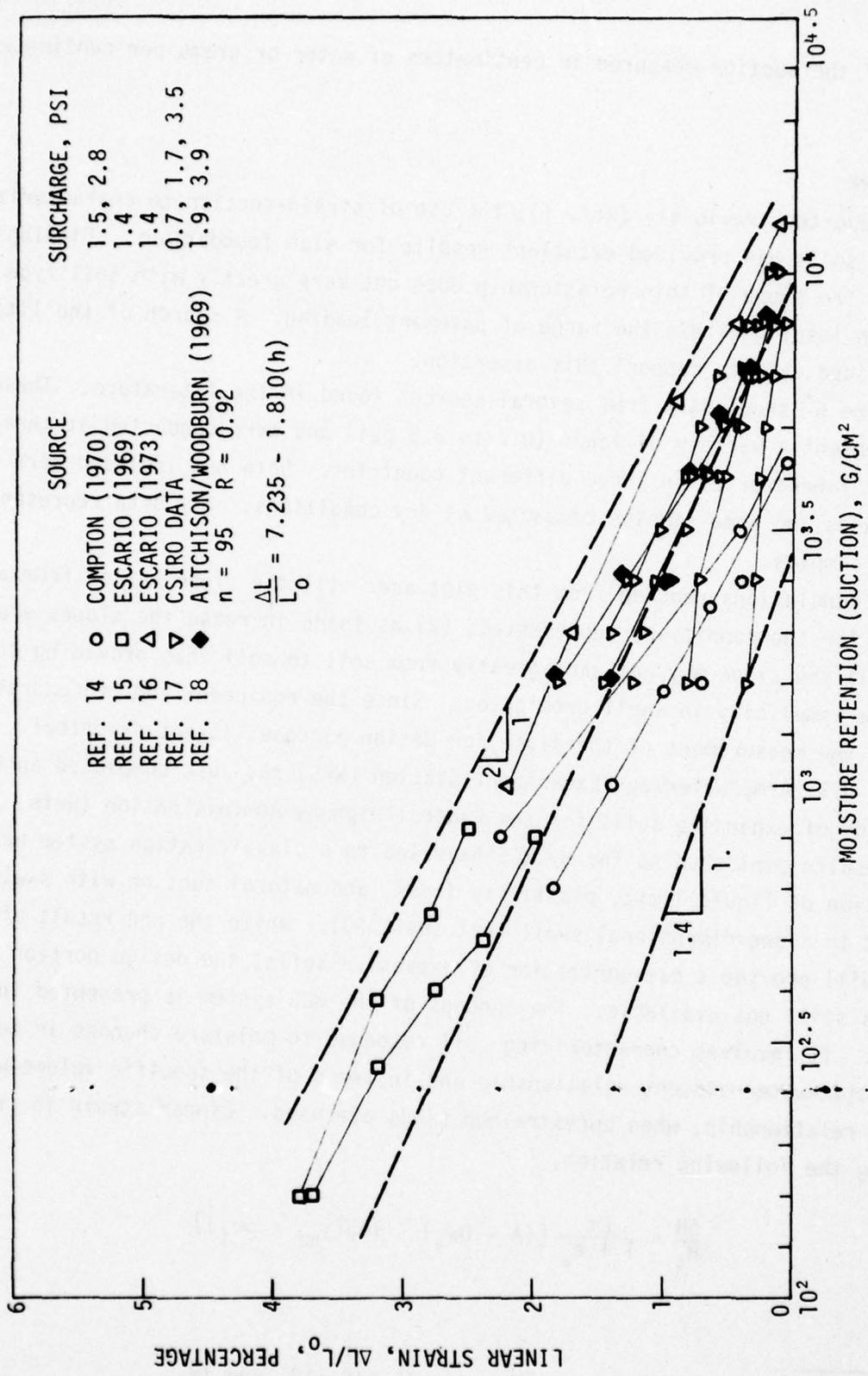
FIGURE 6. EXISTING DATA RELATING STRAIN AND SUCTION

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where

$\frac{\Delta H}{H_0}$ = volume change → one-dimensional behavior

$C\tau$ = suction index = $\sigma\, G_s/100\, B$

e_0 = initial void ratio

w_0 = initial water content, in percent

τ_{mf} = final matrix suction, in tons/ft^2

α = compressibility factor (slope of specific volume versus water content relationship)

σ_f = final applied load, in tons/ft^2

A, B = intercept and slope of the suction (tsf) versus water content plot

G_s = specific gravity of soil solids

The *compressibility factor* (α) is the slope of the specific volume versus moisture-content relationship. It represents the fraction of the applied pressure effective in changing the pore water pressure. It is assumed in this case that the compressibility with respect to load changes equals the compressibility with respect to moisture changes. The development of this method requires a procedure for determining the suction-moisture content relation, the compressibility (slope of specific volume versus water content), and techniques to predict final matrix suction (τ_{mf}).

The Coefficient of Linear Extensibility (COLE) test used by the Soil Conservation Service, National Soil Survey Laboratory (NSSL) is another means used to characterize volumetrically active soils (Ref. 22). The COLE measures the linear extensibility and compressibility calculated from bulk density change that occurs between a moisture retention (suction) of one-third atmosphere and oven dry. This assumes

$$\frac{\Delta L}{L_0} = \left[\frac{\gamma_D}{\gamma_{2.5}}\right]^{1/3} - 1 = COLE \tag{3}$$

where

COLE = $\Delta L/L_0$ = coefficient of linear extensibility

γ_D = bulk density oven dry

$\gamma_{2.5}$ = bulk density of one-third bar moisture tension (2.5 pF equals 1/3 bar)

It is clear that the COLE is directly related to the slope of the relationship shown in Figure 6, because the linear extensibility is always measured over the same change of suction. Another way of stating this correlation is that the

slope of the strain-suction relation is

$$\text{slope} = \frac{\Delta^{\varepsilon} L}{\Delta h} = \frac{\text{COLE}}{4.47 \text{ pF}} \qquad (4)$$

where

slope = change in linear strain ($\varepsilon_L = \Delta L/L_0$) with respect to change
in suction (here suction is in pF)

COLE = same as above

Δh = change in suction used in the COLE test, oven dry is 7.0 pF,
1/3 bar is 2.53 pF

It was demonstrated that the slope of the strain-suction relationship could
be used for design predictions (Refs. 18, 23, 24, 25). This relationship can also
be computed if the COLE is known. The Soil Conservation Service has data that
also relate COLE and clay content as determined by the pipette method, Figure 7
(Ref. 26). From these data the slope for the strain-suction relation is indicated
between 0.5 and 4.0. Keeping in mind that this figure includes a full range of
clay contents (30 to 80 percent) and no overburden loading, it appears to be in
substantial agreement with the previous data.

Another computational method for heave prediction has been derived from mix-
ture theory by Lytton (Ref. 27). The following treatment is taken directly from
his presentation.

Volume Strain

Since the size of the volume strain depends on the size of the suction and
of its change as well as on the size of total pressure and of its change, the
following incremental volume strain equation is proposed.

$$\frac{\Delta V}{V} = -\gamma_h \overbrace{\frac{\Delta h}{h}}^{\text{for constant pressure}} = -\gamma_\sigma \underbrace{\frac{\Delta \sigma}{\sigma}}_{\text{for constant suction}} \qquad (5)$$

where γ_h and γ_σ are positive valued compression constants, h is suction and σ
is stress. In integral form, Eq. (5) becomes:

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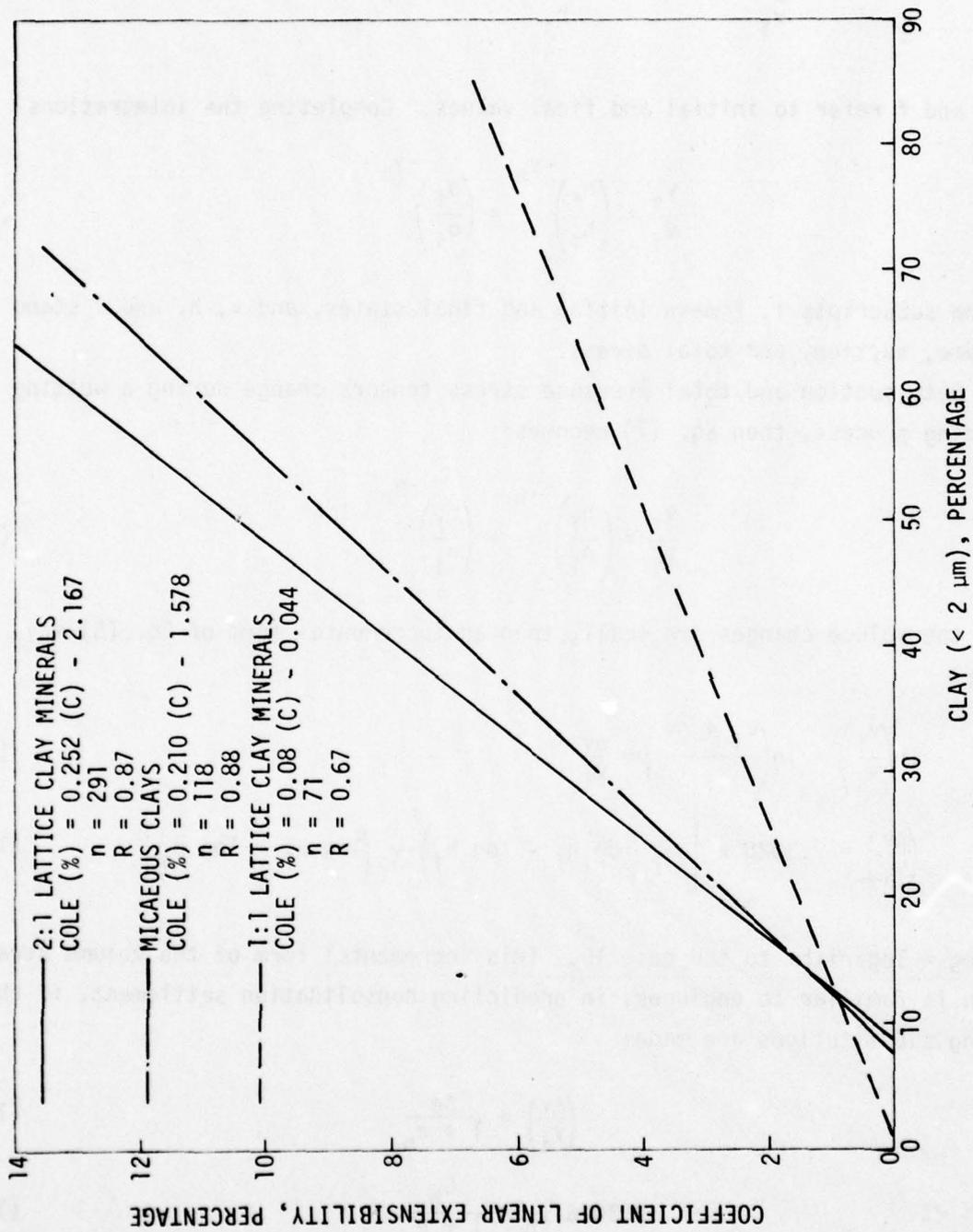
FIGURE 7. CORRELATION OF COLE, CLAY AND MINERALOGY

$$\int_{v_i}^{v_f} \frac{dv}{v} = -\gamma_h \int_{h_i}^{h_f} \frac{dh}{h} = -\gamma_\sigma \int_{\sigma_i}^{\sigma_f} \frac{d\sigma}{\sigma} \tag{6}$$

where i and f refer to initial and final values. Completing the integrations gives:

$$\frac{v_f}{v_i} = \left(\frac{h_f}{h_i}\right)^{-\gamma_h} = \left(\frac{\sigma_f}{\sigma_i}\right)^{-\gamma_\sigma} \tag{7}$$

where the subscripts i, f mean initial and final states, and v, h, and σ stand for volume, suction, and total stress.

If both suction and total pressure stress tensors change during a wetting and loading process, then Eq. (7) becomes:

$$\frac{v_f}{v_i} = \left(\frac{h_f}{h_i}\right)^{-\gamma_h} + \left(\frac{\sigma_f}{\sigma_i}\right)^{-\gamma_\sigma} \tag{8}$$

If the volume changes are small, then an incremental form of Eq. (8) may be used:

$$\ln\left(\frac{v_f}{v_i}\right) = \ln\left(\frac{v_i + \Delta v}{v_i}\right) \approx \frac{\Delta v}{v_i} \tag{9}$$

$$\left(\frac{\Delta v}{v_i}\right) \cong 2.3026 \times \left[-\gamma_h\left(\log h_f - \log h_i\right) - \gamma_\sigma\left(\log \sigma_f - \log \sigma_i\right)\right] \tag{10}$$

where log = logarithm to the base 10. This incremental form of the volume strain equation is familiar to engineers in predicting consolidation settlement, if the following substitutions are made:

$$\left(\frac{\Delta v}{v_i}\right) = \frac{\Delta e}{1 + e_o} \tag{11}$$

$$2.3026 \, \gamma_h = \frac{C_h}{1 + e_o} \tag{12}$$

$$2.3026 \, \gamma_\sigma = \frac{C_\sigma}{1 + e_o} \tag{13}$$

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where

e_o = initial void ratio

C_h, C_σ = suction and total stress compression indexes respectively.

Another form of Eq. (10) provides still more insight into the mechanism of swelling.

$$\left(\frac{\Delta v}{v_i}\right) = 2.3026 \, \gamma_h \left[\log h_i - \log h_f \left(\frac{\sigma_f}{\sigma_i}\right)^{\gamma_\sigma/\gamma_h}\right] \quad (14)$$

This equation has also been proposed by Snethen and Johnson (Ref. 21), except that in their equation,

$$h_f \left(\frac{\sigma_f}{\sigma_i}\right)^{\gamma_\sigma/\gamma_h} = h_f + \alpha\sigma_f \quad (15)$$

where α is the equivalent factor relating pressure change to suction change.

Algebraic manipulation of Eq. (15) gives an expression for α in terms of measureable stresses and the compression constants:

$$\alpha = \left(\frac{h_f}{\sigma_f}\right)\left[\left(\frac{\sigma_f}{\sigma_i}\right)^{\gamma_h/\gamma_\sigma} - 1\right] \quad (16)$$

The following observations can be made from this equation. If a soil is insensitive to volume change with a change of suction, γ_h is zero and α will be zero. The value of α will depend upon the ratio of the final values of suction and total stress in any swelling or shrinking process.

The value of σ_i to be used in Eq. (14) is the value of the nominal pressure that is applied to a laboratory soil sample in performing a "swell test," i.e., 0.1 tons/ft² or 9.6 kPa. The value of σ_f is the isotropic pressure applied to the soil by the overburden pressure and lateral restraint. When the soil is swelling, σ_f will be equal to the overburden pressures; but when the soil is shrinking, σ_f will be less than the overburden pressure because the lateral restraint is reduced by shrinkage cracking. Heave will occur at all depths above which the following conditions are met:

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$$(1) \quad \frac{h_i}{h_f} \geq 1$$

$$(2) \quad \frac{h_i}{h_f} \left(\frac{\sigma_i}{\sigma_f}\right)^{\gamma_h/\gamma_\sigma} \geq 1 \left.\rule{0pt}{40pt}\right\} \text{(Heave)} \tag{17}$$

Vertical contraction will occur at all depths above which both of the above criteria are less than 1.0.

Initial Suction

The initial value of suction has been found to be represented conveniently by the following equation (Ref. 2):

$$\log h_i = c - dw_i \tag{18}$$

where c and d are constants for a given soil and a given wetting or drying process, and w_i is the initial gravimetric water content in decimal form.

Typical values of c and d for a variety of soils in a drying process are given in Table 1. The constants will not be greatly different for a swelling process as long as there is not much hysteresis in the suction-water content curve between the adsorption and desorption phase. For more detailed information on the soils listed in Table 1, see Reference 2.

Compressibility Coefficient for Constant Pressure, γ_h

A test to determine the suction change compressibility of expansive soils in their natural condition was first proposed by the SCS (Refs. 28, 29, 30) and the resulting measurement was called the COLE test. The change of suction imposed on the soil is between 32.7 kPa (4.7 psi) and oven dry (approximately 9.805×10^5 kPa). COLE data obtained by McKeen (Ref. 31) from the SCS showed that the value of γ_h depends upon the clay mineral type and the percent clay present in the soil. For 2:1 expanding lattice soils of which montmorillonite is typical:

$$\gamma_h' = 2.3026 \, \gamma_h = 0.00056(\% \text{ clay}) - 0.00433 \tag{19}$$

For clay-mica soils, illite being "typical" mineral:

$$\gamma_h' = 2.3026 \, \gamma_h = 0.00047(\% \text{ clay}) - 0.00351 \tag{20}$$

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TABLE 1.   CONSTANTS RELATING SUCTION (kPa) TO GRAVIMETRIC WATER
CONTENT (DECIMAL) [AFTER SNETHEN AND JOHNSON (REF. 2)]

Site No.	Geologic Formulation	State	c	d
1	Yazoo	Mississippi	7.195	10.68
2	Hattiesburg	Mississippi	5.721	13.45
3	Alluvium	Louisiana	5.642	8.80
4	Prairie Terrace	Louisiana	4.899	11.52
5	Taylor	Texas	4.658	10.39
6	Vale	Texas	11.896	77.07
7	Washita	Oklahoma	8.202	39.36
8	Hennessey	Oklahoma	10.493	52.74
9	Chinle	Arizona	5.173	18.80
10	Chinle	Arizona	7.812	24.54
11	Mancos	Utah	4.461	12.05
12	Blue Hill	Kansas	6.575	16.01
13	Graneros	Kansas	8.381	33.86
14	Pierre	Colorado	4.953	8.11
15	Laramie	Colorado	8.434	16.20
16	Denver	Colorado	7.800	31.40
17	Mowry	Wyoming	6.403	15.07
18	Pierre	Wyoming	8.573	33.21
19	Bearpaw	Montana	8.184	33.86
20	Pierre	S. Dakota	8.177	21.07

For 1:1 expanding lattice clay minerals such as kaolinite,

$$\gamma_h' = 2.3026 \, \gamma_h = 0.00018(\% \text{ clay}) - 0.000098 \tag{21}$$

where γ_h' refers to the compressibility coefficient on a logarithmic base of 10.
The constant γ_h is the same coefficient relative to a logarithmic base, e.   The
γ_h coefficients will range between zero and slightly over 0.05.

Compressibility Coefficient for Constant Suction, γ_σ

Typical values of γ_σ' (= 2.3026 γ_σ) were calculated from the results of swell
tests and swell pressure tests reported by Vijayvergiya and Sullivan (Ref. 19),

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according to Eq. (22).

$$\gamma_\sigma' = \frac{(\Delta v/v)_o}{\log\left(\dfrac{\sigma_p}{\sigma_i}\right)} \tag{22}$$

where

σ_p = swell pressure

σ_i = nominal pressure of 0.1 tons/ft² (9.6 kPa) applied during the swell test

$(\Delta v/v)_o$ = volume strain in swell test, decimal units

The range of these calculated values of γ_σ' fell between 0.02 and 0.09 indicating that the ratio γ_h/γ_σ is typically at or slightly below 1.0. Table 2 shows selected values of γ_σ' as calculated from the published data (Ref. 7). The corresponding values of C_σ were calculated using Eq. (13), and their values range between 0.030 and 0.145. These values of C_σ may be compared directly with values of the compression index C_o, which is used in calculating settlement.

This presentation illustrates the relation between many aspects of the subject. It is significant that this body of theoretical knowledge is beginning to correlate with much of the laboratory work being conducted. In order to use the swell-suction relation for design, one needs a means of predicting suction changes. One technique may be to measure suction-water content relations. If, as the WES data suggest, a linear relation exists in the range of in situ moisture conditions some other techniques are possible: namely, a linear relation between swell and water content in the range of in-situ moisture contents. While this subject is to be studied in later work, some suction measurements were made in the current work as the result of the author's finding a promising technique in the literature. This procedure involves equilibrating calibrated filter papers with soil samples (Ref. 32). By measuring the moisture content of the paper, one can measure the moisture retention or suction. This technique is inexpensive, simple, and has been used extensively by the United States Geological Survey, Water Resources Division (Refs. 33, 34, 35). A significant advantage is the wide range of this test method.

EXPERIMENTAL FRAMEWORK

Experiments were conducted to evaluate the variability of the swell-suction relationship from soil to soil and with variations in other more easily determined properties. These four properties were Atterberg Limits, clay content,

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TABLE 2.   TYPICAL VALUES OF COMPRESSIBILITY COEFFICIENT,
γ'_σ AND COMPRESSION INDEX, C_σ
[AFTER VIJAYVERGIYA, ET AL. (REF. 7)]

Test No.	Swell Pressure, kPa	Decimal Volume Strain	γ'_σ	e_o	C_σ
1	46.0	0.0225	0.033	0.38	0.046
2	38.3	0.0120	0.020	0.48	0.030
3	37.4	0.0145	0.025	0.45	0.036
4	153.2	0.0510	0.042	0.43	0.060
5	35.4	0.0195	0.034	0.46	0.050
6	75.7	0.0390	0.043	0.45	0.062
7	114.9	0.0400	0.037	0.49	0.055
8	153.2	0.0530	0.044	0.45	0.064
9	91.0	0.0390	0.040	0.40	0.056
10	134.0	0.0300	0.026	0.46	0.038
30	326.0	0.0890	0.058	0.33	0.077
42	374.0	0.1360	0.086	0.46	0.126
47	517.0	0.0900	0.052	0.39	0.072
53	345.0	0.1280	0.082	0.40	0.115
93	575.0	0.0970	0.055	0.37	0.075
132	418.0	0.1300	0.080	0.43	0.114
133	479.0	0.0830	0.049	0.59	0.073
170	192.0	0.1200	0.092	0.58	0.145

COLE and bar linear shrinkage. In addition, a modification of the COLE test using clods at varying moisture content was evaluated. The suction-water content relationship, as determined by the use of calibrated filter papers, was also studied.

The emphasis in this laboratory work was on the evaluation of soil behavior as responses to changes in load or moisture condition. The objective was to obtain a rapid means of determining the response for unit change in load or suction. These soil properties, combined with predictions of the total change anticipated in situ, provide the necessary data for soil response (heave) predictions as indicated in the technical literature.

SECTION 3
TEST METHODS

INTRODUCTION

Numerous sampling and testing methods are used in the engineering evaluation of soil behavior for obtaining design data. The presentation and description of these is often inadequate in the technical literature. The methods used in this study will be presented as clearly as possible, although questions of the effects of various aspects may remain unanswered. The following section describes the methods employed.

SAMPLING - UNDISTURBED MATERIALS

Throughout this report samples are referred to as disturbed or undisturbed. It is recognized that no soil sample removed from its in situ environment is truly undisturbed. The samples referred to as undisturbed were obtained by pushing large diameter steel sampling tubes into the soil with a hydraulic ram mounted on a mobile drilling rig. Part of the samples were obtained by the U.S. Army Corps of Engineers, Waterways Experiment Station and furnished to CERF for this program. The remaining samples were obtained by CERF personnel. The sampling tubes which were used are described in Table 3.

The WES samples were sealed with expanding packers to prevent the loss of moisture and stored in a cool, dry warehouse at Vicksburg, Mississippi. They were taken during 1975 and stored until mid 1977 at which time they were shipped to CERF. When received in the laboratory, the samples were removed by cutting the tubes. Soils were generally in good condition, although some moisture loss had occurred.

TABLE 3. CHARACTERISTICS OF SAMPLING TUBE

Dimension, in	WES	CERF
Outside Diameter	5.25	4.5
Inside Diameter	5.0	4.375
Wall Thickness	0.125	0.0625
Length	~36	24
Approximate Sample Length	~24	~18

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The CERF samples were extruded in the field, sealed in double 4-mil plastic bags and packed in metal cans with vermiculite; or they were sealed in the tubes. Samples sealed in the tubes were plugged with paraffin approximately one-half inch thick. Soils in the tubes were extruded in the laboratory within about two weeks of the sampling. Materials were sampled during October through December, 1977. A diagram illustrating testing performed on undisturbed materials is shown in Figure 8. The tests are discussed below.

SAMPLING - DISTURBED MATERIALS

Disturbed samples were collected from auger cuttings or removed with hand tools (pick and shovel) from shallower depths. They were placed in plastic bags and sealed with tape to preserve the sample moisture condition. Some disturbed materials were obtained using steel sampling tubes to extract the soil, followed by extruding and sealing as previously described. Moisture content samples were removed and placed in cans in the field and sealed with electrical tape. The samples were taken at a variety of depths through the profiles. All tests on soils obtained by WES were made on material taken from the sampling tubes. Figure 9 shows tests for disturbed materials.

STANDARD METHODS OF TESTING

Standard methods of testing as published by the American Society for Testing and Materials (ASTM) were used in evaluating soils. Those tests used are listed in Table 4. The American Association of State Highway and Transportation Officials (AASHTO) method was used for evaluation of organic content on those soils having a dark color.

OTHER TESTS

In addition to the standard test methods mentioned above, test procedures standardized and used by other organizations were included. These were the co-efficient of linear extensibility (COLE), bar linear shrinkage (BLS), and the determination of moisture retention (suction) of soils by the filter paper technique. Some description of these procedures is presented to benefit readers not familiar with them.

COLE TEST

The COLE test is performed routinely by the Soil Conservation Service in preparation of a variety of studies by the National Soil Survey Laboratory (NSSL)

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                        ┌──────────────┐
                        │ UNDISTURBED  │
                        │   SAMPLE     │
                        └──────┬───────┘
                               │
                               │          ┌──────────────┐
                               ├─────────▶│   NATURAL    │
                               │          │   MOISTURE   │
                               │          └──────────────┘
                               │
        ┌──────────┬───────────┼──────────────┬──────────────┐
        ▼          ▼           ▼              ▼
┌──────────────┐ ┌──────────────┐ ┌──────────────────┐ ┌──────────────┐
│  COLE TEST   │ │  CLOD TEST   │ │MOISTURE RETENTION│ │ SWELL TESTS  │
│SCS PROCEDURE │ │CERF PROCEDURE│ │  FILTER PAPER    │ │              │
└──────┬───────┘ └──────────────┘ └──────────────────┘ └──────┬───────┘
       │                                                      │
       ▼                                                      ▼
┌──────────────┐                              ┌──────────────────────┐
│ OTHER TESTS  │                              │ MOISTURE RETENTION   │◀─┐
│              │                              │     (BEFORE)         │  │
└──────────────┘                              └──────────────────────┘  │
                                                                         │
                                              ┌──────────────────────┐   │
                                              │    CLOD TEST         │◀──┘
                                              │     (AFTER)          │
                                              └──────────────────────┘
```
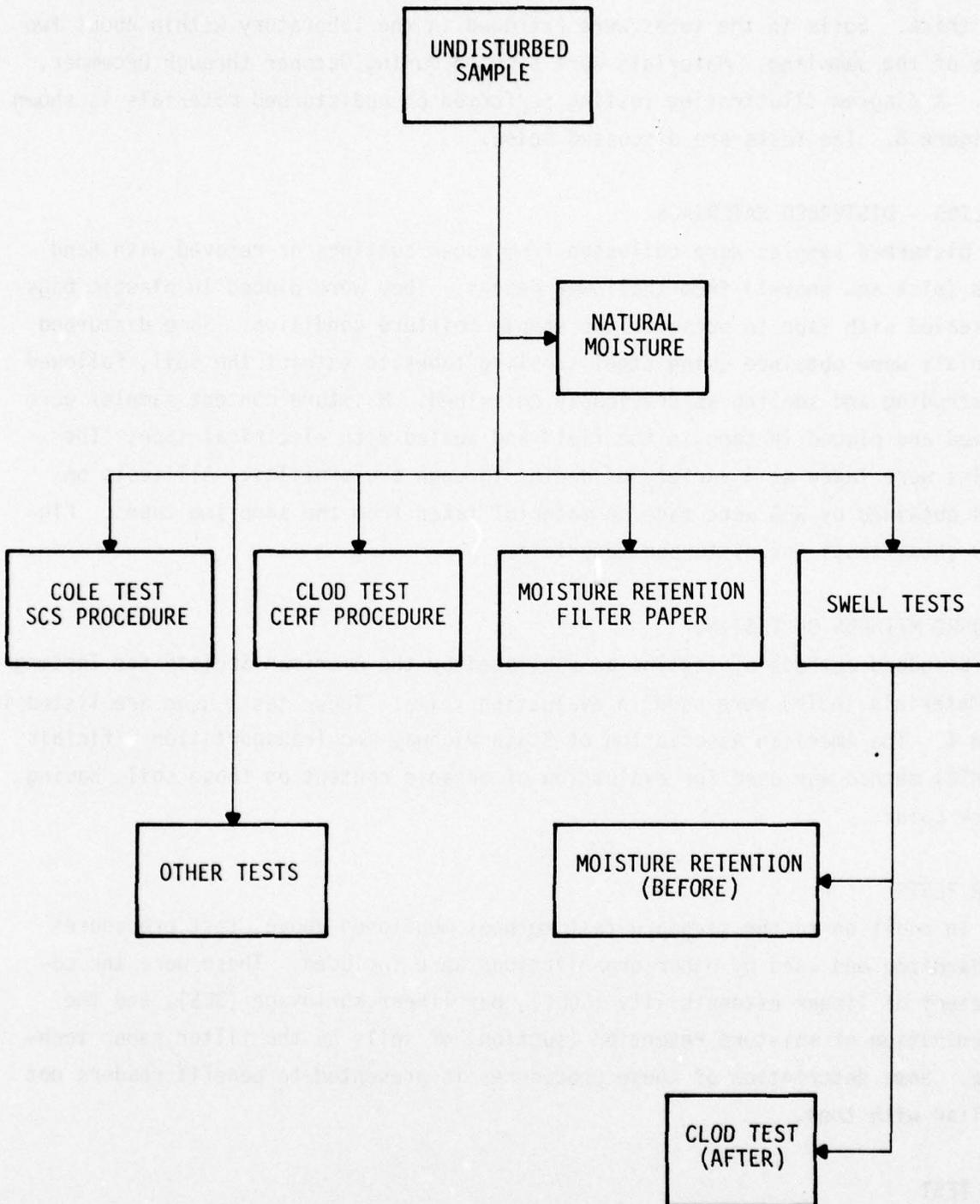
FIGURE 8. UNDISTURBED SOIL TESTING

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                    ┌─────────────────────┐
                    │  DISTURBED SAMPLE   │
                    │    SEALED IN BAG    │
                    └─────────────────────┘
                              │
              ┌───────────────┴───────────────┐
              │                               │
    ┌─────────────────┐           ┌─────────────────────┐
    │    AIR DRY       │           │  NATURAL MOISTURE   │
    │    2000 G        │           │      SAMPLE         │
    └─────────────────┘           └─────────────────────┘
              │
    ┌──────────────────┐  > NO. 10  ┌──────────┐    ┌──────────────────┐
    │  SEPARATE ON     │───────────▶│  OVEN    │───▶│  COARSE SIEVE    │
    │  NO. 10 SIEVE    │            │  DRY     │    │  ANALYSIS        │
    └──────────────────┘            └──────────┘    │  ASTM D422       │
              │ < NO. 10                             └──────────────────┘
```

MOISTURE RETENTION (FILTER PAPER) · SPECIFIC GRAVITY ASTM D854 · HYDROMETER ASTM D422 · FINE SIEVE ASTM D422 · OTHER TESTS

FINE SIEVE ASTM D422

SEPARATE ON NO. 40 SIEVE > NO. 40

< NO. 40

DISCARD

ORGANIC CONTENT AASHTO T194 · ATTERBERG LIMITS ASTM D423, D424, D427 · BAR LINEAR SHRINKAGE TEXAS METHOD

FIGURE 9. DISTURBED SOIL TESTING

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TABLE 4. STANDARD TESTS USED

Title of Method	Designation
Liquid Limit of Soils	ASTM D423-66
Plastic Limit and Plasticity Index of Soils	ASTM D424-59
Shrinkage Factors of Soils	ASTM D427-61
Specific Gravity of Soils	ASTM D854-58
Particle Size Analysis of Soils	ASTM D422-63
Determination of Organic Matter in Soils by Wet Combustion	AASHTO T194-70

in Lincoln, Nebraska. Previous study of the technical literature (Ref. 1) indicated the COLE of a sample might be a good indicator of field behavior of soils. As performed by the NSSL, the test consists of the following steps (Ref. 22); also see Appendix A:

1. Collect natural clods 50 to 200 cm^3.

2. Immerse each clod briefly in a plastic solution using a thread or fine wire.

3. Apply additional coats in the laboratory and permit the clod to absorb water by capillary action (Fig. 10).

4. Equilibrate the clod by the pressure plate technique at one-third atmosphere (Fig. 11); then determine weight and volume by Archimedes principle.

5. Then oven dry the clod and determine its weight and volume in the same manner.

6. Computation is made as follows (Ref. 22):

$$COLE = \left[\frac{Db_d}{Db_m}\right]^{1/3} - 1 \qquad (23)$$

where Db_d and Db_m are dry and moist bulk densities respectively.

A large volume of COLE data has been obtained by the NSSL. Therefore, it was decided the procedure evaluated should duplicate that used by the NSSL. The procedure used and reported as COLE in this study follows that presently used at the Lincoln Laboratory, except that it was performed at CERF. In order to insure duplication of COLE procedures, a CERF employee spent two days at the NSSL for training.

FIGURE 10.   COLE SAMPLES ON TENSION TABLE
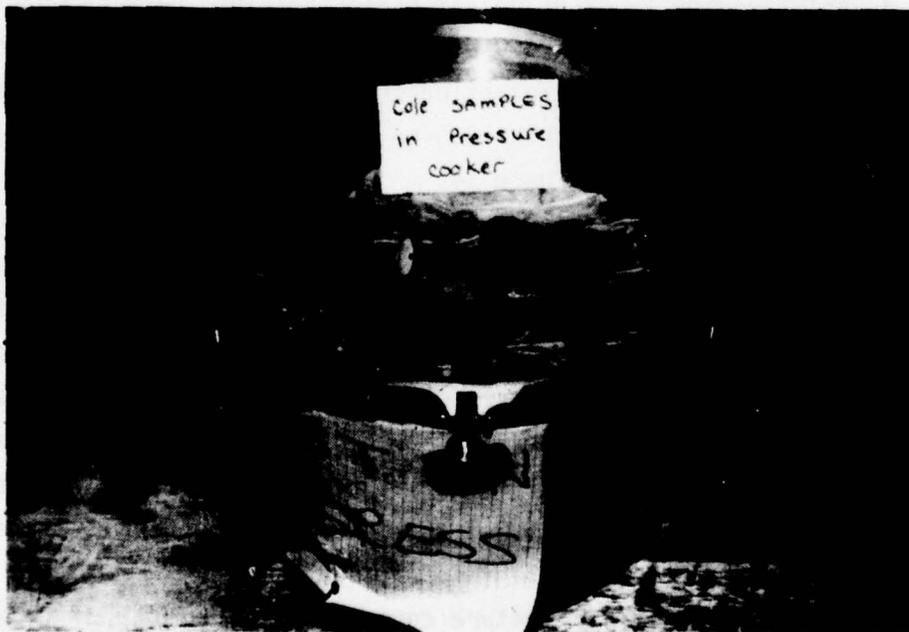
FIGURE 11.   COLE SAMPLES ON PRESSURE PLATE

BAR LINEAR SHRINKAGE TEST

The bar linear shrinkage test (Ref. 36) was performed following the Texas DOT procedure (Tex-107-E) with slight modification. Along with the measurements of shrinkage, the weight change with time was determined. These data provide a shrinkage curve for the pulverized soil as illustrated in Figure 12. In addition to the slope of the shrinkage curve, the range of volumetric activity is important. These data can be used to characterize the volumetric response of expansive soils to moisture changes. It is clear the total shrinkage usually measured in the test can be represented in terms of the slope and range.

MOISTURE RETENTION (SUCTION)

The measurement of the intensity with which a soil retains moisture is a fundamental characteristic of that soil. A tremendous amount of research has been devoted to the study of moisture characteristics or moisture-suction relationships of soils. Although agricultural scientists have routinely measured moisture retention, engineers have not, primarily because of the complexity of such a determination. More recently the use of thermocouple psychrometers (Ref. 2) and calibrated filter papers (Ref. 32) have been demonstrated to have potential for routine measurements in engineering applications.

Figure 13 illustrates the types of relationships obtained between moisture retention and moisture content for a heavy clay soil (Ref. 37). The relationship between Atterberg Limits and moisture retention characteristics have been studied in detail and shown to correlate to a high degree (Refs. 38, 39, 40, 41, 42). Typical data for a soil is shown in Figure 14.

The test method is fully described in Appendix A. A summary is provided here for the purpose illustrating the technique. A calibrated filter paper was placed in a moisture can with the sample (clod or pulverized) as shown in Figure 15. The filter paper was separated from the clod by a small piece of paper towel cut the same size to prevent soil particles from adhering to the filter paper. The can was then sealed with plastic tape and allowed to equilibrate in an insulated chest, Figure 16. After seven days the filter paper was removed and its moisture content determined. By use of the calibration curve, the moisture retention was determined, Figure 17.

The method requires use of a temperature controlled room (20° ± 1° C) as developed (Ref. 32). In order to facilitate the use of the technique by conventional soil testing laboratories, this restriction was disregarded. During the tests at CERF, temperature varied between 20 and 27° C.

28
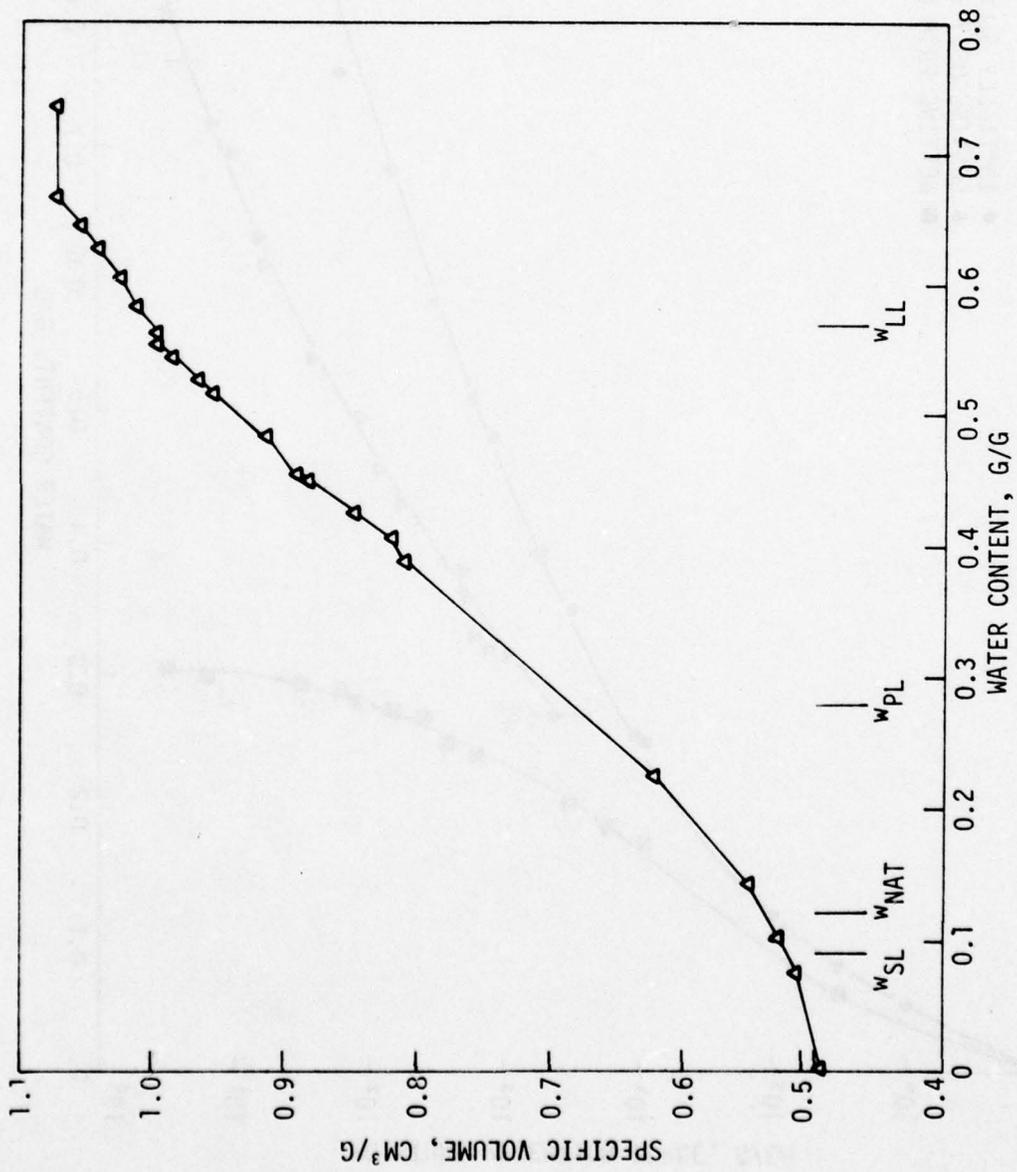
FIGURE 12. SHRINKAGE CURVE FROM BLS TEST

29
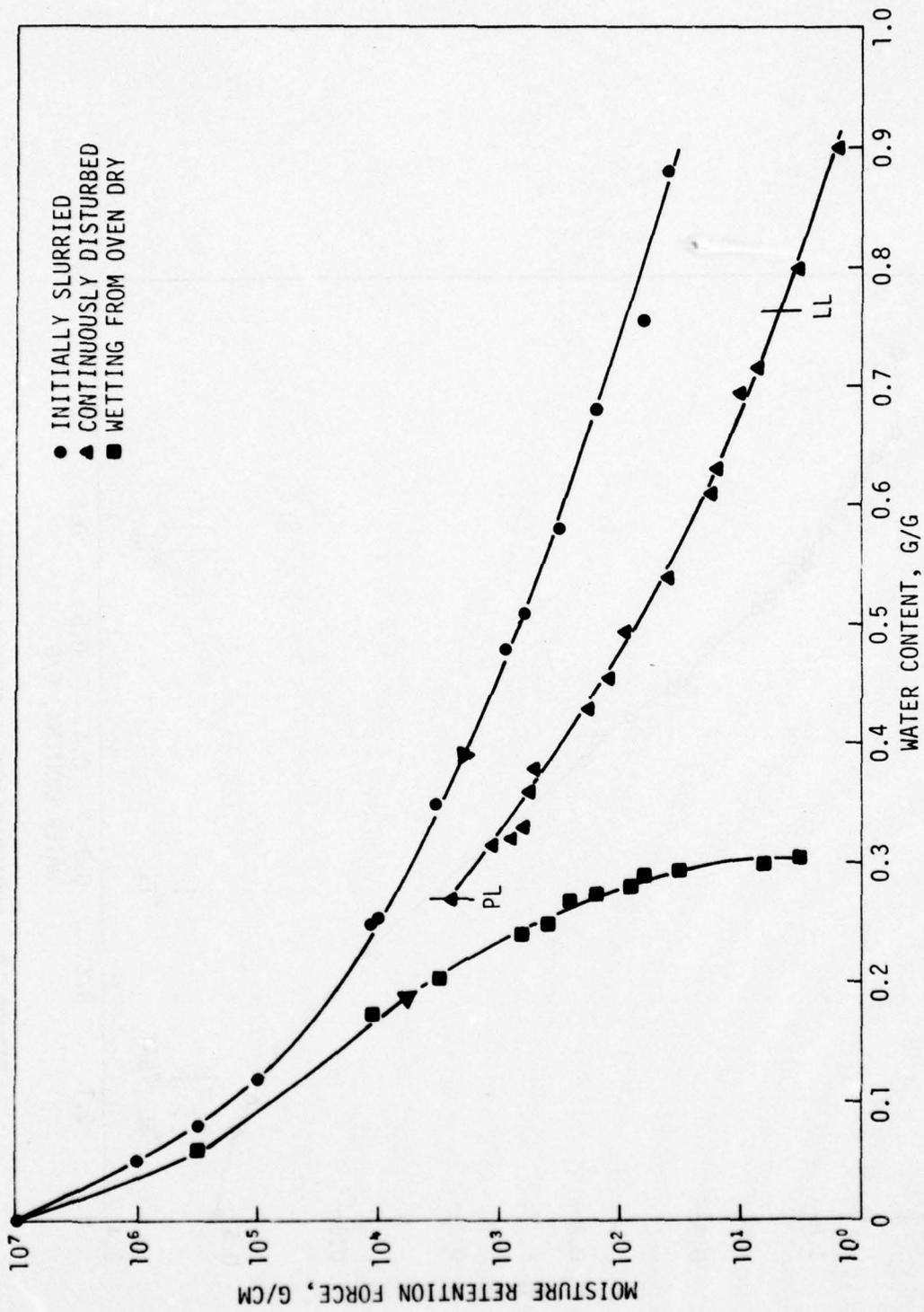
FIGURE 13. MOISTURE RETENTION (SUCTION) RELATIONSHIPS
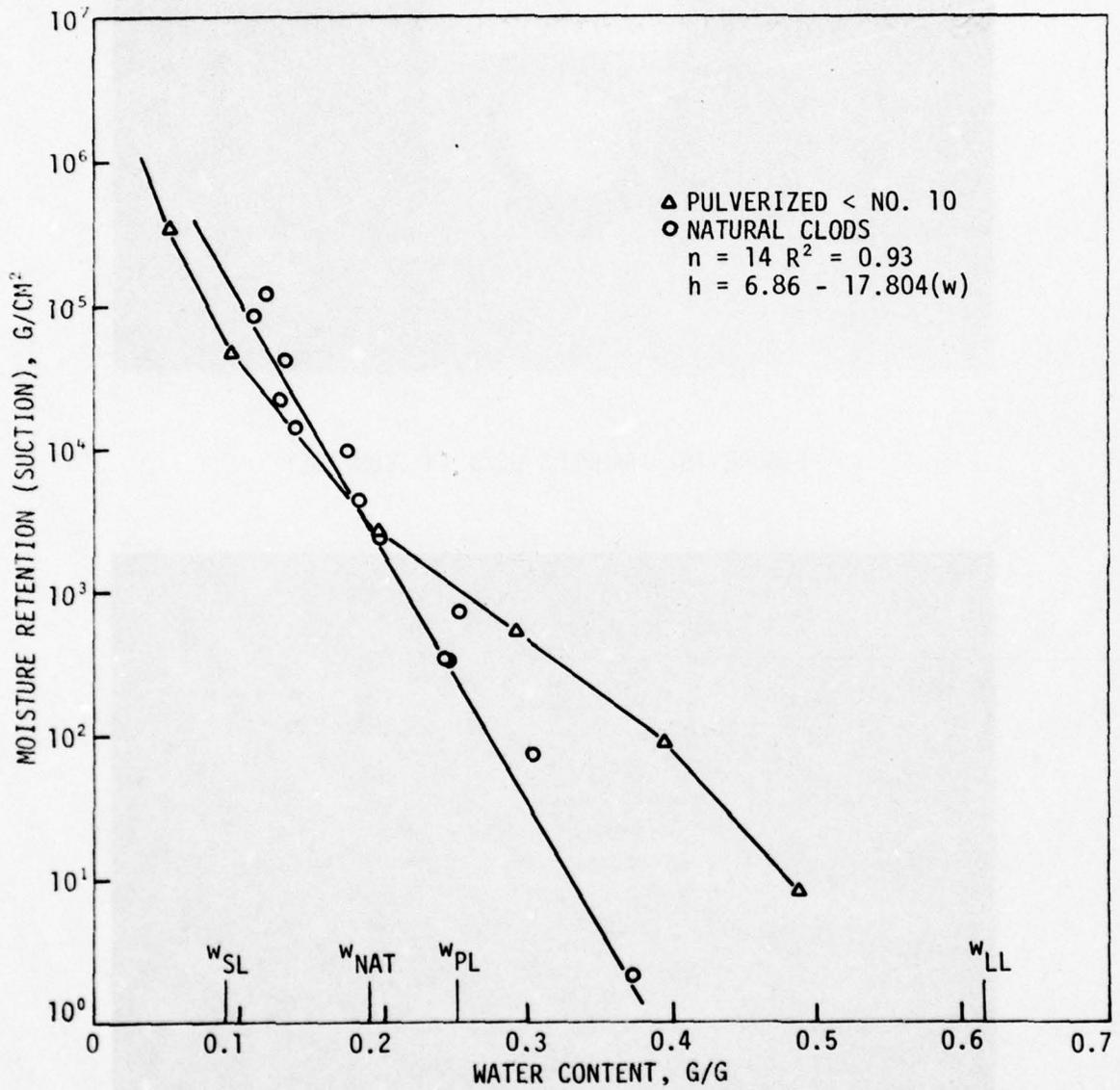
FIGURE 14. TYPICAL MOISTURE-SUCTION RELATION
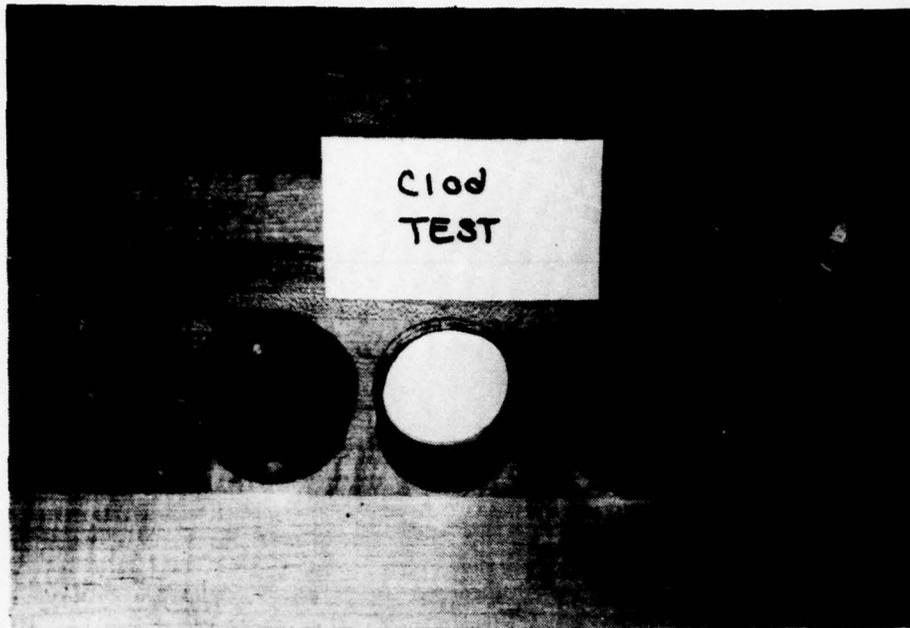
FIGURE 15. SAMPLES USED IN CLOD TEST

FIGURE 16. INSULATED CHEST USED FOR EQUILIBRATION
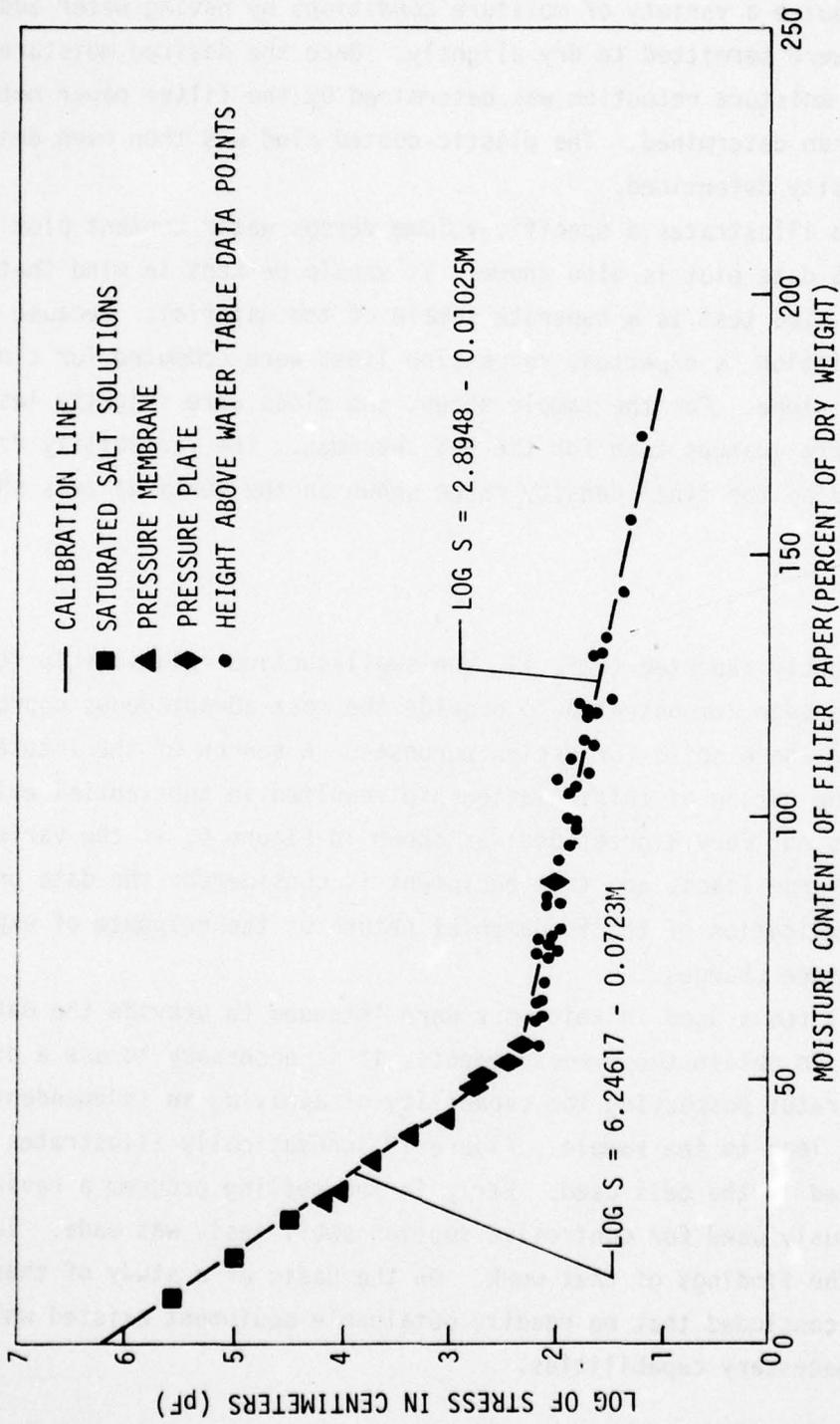
FIGURE 17. SUMMARY OF CALIBRATION DATA

CLOD TESTS

Measurements of the change of bulk density of natural clods were made by use of the procedures involved in the COLE tests as previously described. Clods were allowed to acquire a variety of moisture conditions by having water added to some while others were permitted to dry slightly. Once the desired moisture condition was obtained, moisture retention was determined by the filter paper method. Bulk density was then determined. The plastic-coated clod was then oven dried and its final dry density determined.

Figure 18 illustrates a specific-volume versus water-content plot for a clod test. The BLS data plot is also shown. It should be kept in mind that each data point for the clod test is a separate sample of the material. Because the sample-to-sample variation is expected, regression lines were computed for clod data to determine the slope. For the sample shown, the clods were slightly less responsive to moisture changes than for the BLS specimen. The variability from clod to clod reflected by the final density range shown on the vertical axis should be noted.

SWELL TESTS

As previously reported (Ref. 1), the swell-suction relationship for expansive soils has been demonstrated to provide the most advantageous approach to characterizing these soils for design purposes. A search of the literature for evidence of the nature of this relationship resulted in substantial evidence that the slope does not vary a great deal as shown in Figure 6. If the variety of materials, surcharge loads, and test equipment is considered, the data provide a significant indication of the fundamental nature of the response of expansive soils to moisture changes.

The swell tests used in this work were intended to provide the data illustrated here. To obtain these measurements, it is necessary to use a pressure membrane apparatus possessing the capability of applying an independent surcharge or mechanical load to the sample. Figure 19 schematically illustrates the principles involved in the cell used. Early in the testing program a review of techniques previously used for controlled suction swell tests was made. Table 5[*] illustrates the findings of that work. On the basis of a study of these techniques, it was concluded that no readily obtainable equipment existed which could provide the necessary capabilities.

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[*]Table 5, p. 37, contains References 43, 44, and 45.

34
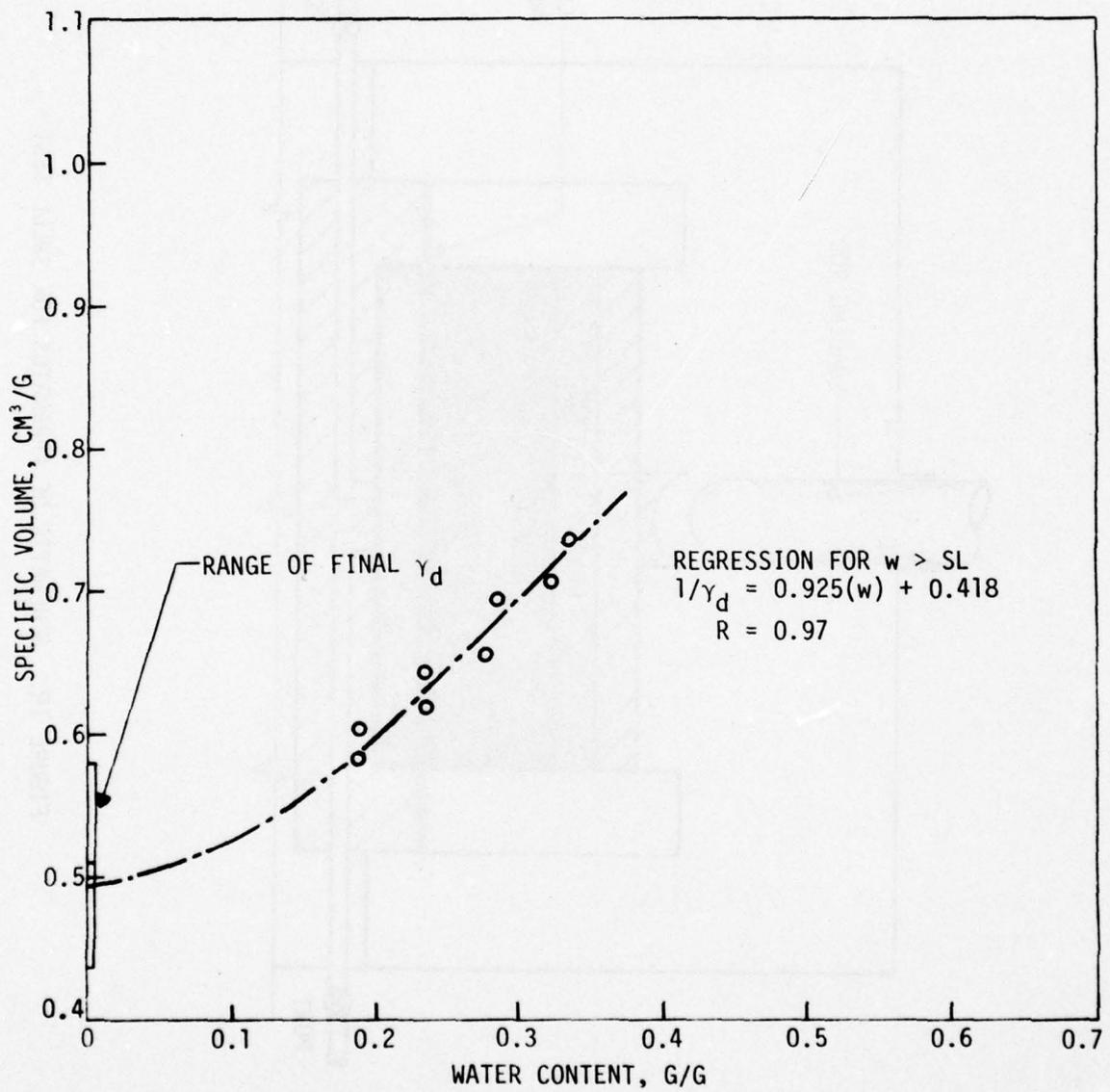
FIGURE 18.   SPECIFIC VOLUME-MOISTURE CONTENT PLOT FROM CLOD DATA

35
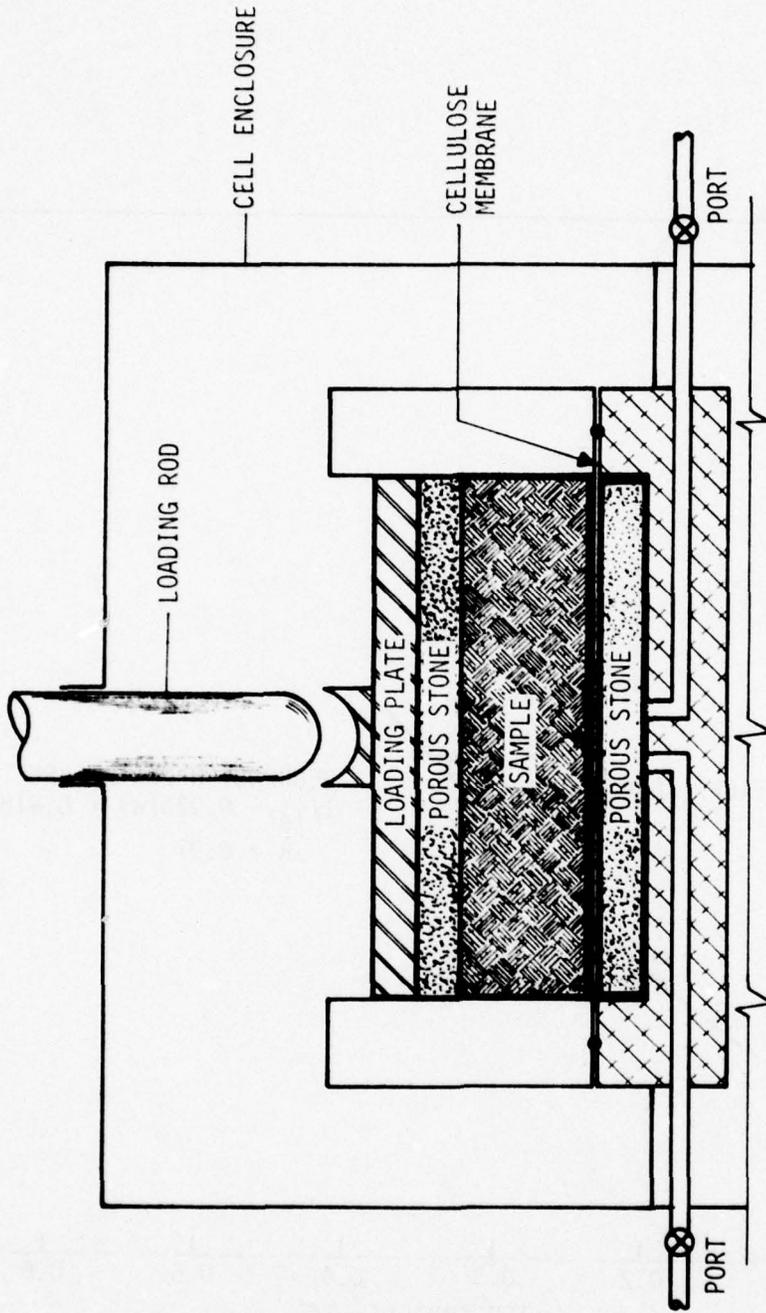
FIGURE 19. SCHEMATIC OF APPARATUS FOR SWELL TEST

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TABLE 5.   PREVIOUS WORK INVOLVING CONTROLLED-
SUCTION SWELL TESTS

Source	Equipment and Membrane	Sample	Suction Pressure Range
CSIRO (Ref. 43)	Modified Wykeham Farrance Consolidation Frame - Nalo Cellulose	Remolded 3 in (7.6 cm) Dia. 1 in (2.5 cm) High	9.8 - 980 kPa (1.4 - 142 psi)
Escario (Ref. 15)	Mercury Suction Plate - 1 Bar Porous Plate	Remolded	< 9.8 kPa (< 1.4 psi)
Escario (Ref. 16)	Pressure Membrane Cell - Cellulose	Remolded 2.75 in (7 cm) Dia. 0.7 in (1.8 cm) High	0 - 1765 kPa (0 - 256 psi)
Compton (Ref. 14)	Modified Bishop Oedometer - 5 Bar Porous Ceramic	Remolded 3 in (7.6 cm) Dia. 0.75 in (1.9 cm) High	0 - 490 kPa (0 - 71 psi)
Fredlund (Ref. 44)	Modified Anteus Oedometer - 4 and 15 Bar Porous Ceramic	Remolded	
Kassiff (Ref. 45)	Modified Consolidation Apparatus - Cellulose	Remolded 1.8 in (4.5 cm) Dia. 0.6 in (1.5 cm) High	0 - 1570 kPa (0 - 228 psi)

Equipment on hand at the CERF was modified for use in these tests. Large
triaxial testing cells were modified to provide loading capability on a sample
mounted on a cellulose membrane inside the cell. Figure 20 illustrates the dis-
assembled cell components. Samples were cut from undisturbed materials to be
3.0 in (7.6 cm) in diameter by 0.75 in (1.9 cm) in height. They were placed in
the cell on a cellulose membrane supported by a 1/8-inch-thick porous stone in
the base. Another porous stone and loading plate were positioned on top of the
sample. Externally applied loads were placed on the loading plate through a
loading rod. Samples were initially equilibrated ($\Delta L \leq 0.0001$ in 24 hours) at a
cell pressure of 120 psi (~10^4 kPa). The cell pressure was then reduced in incre-
ments to allow reduction of the suction pressures, thus permitting the sample to
take on water through the semipermeable membrane.

It was necessary to utilize compensating pistons (Figs. 20 and 21) to counter-
act the air pressure on the loading rod in the cell. As a result of the configura-
tion of these cells, an imbalance was present in the system. Imbalances were
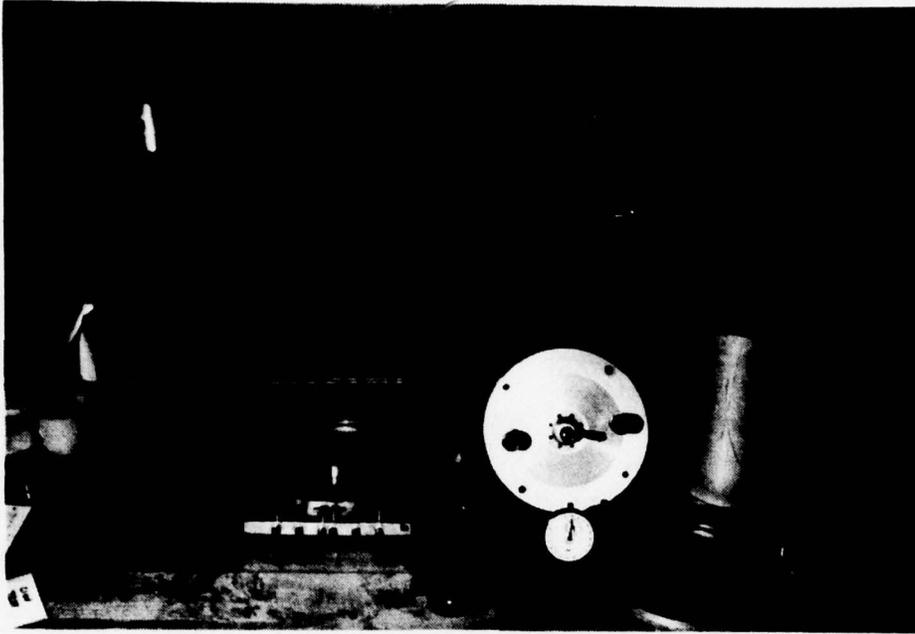
FIGURE 20.   DISASSEMBLED CELL COMPONENTS
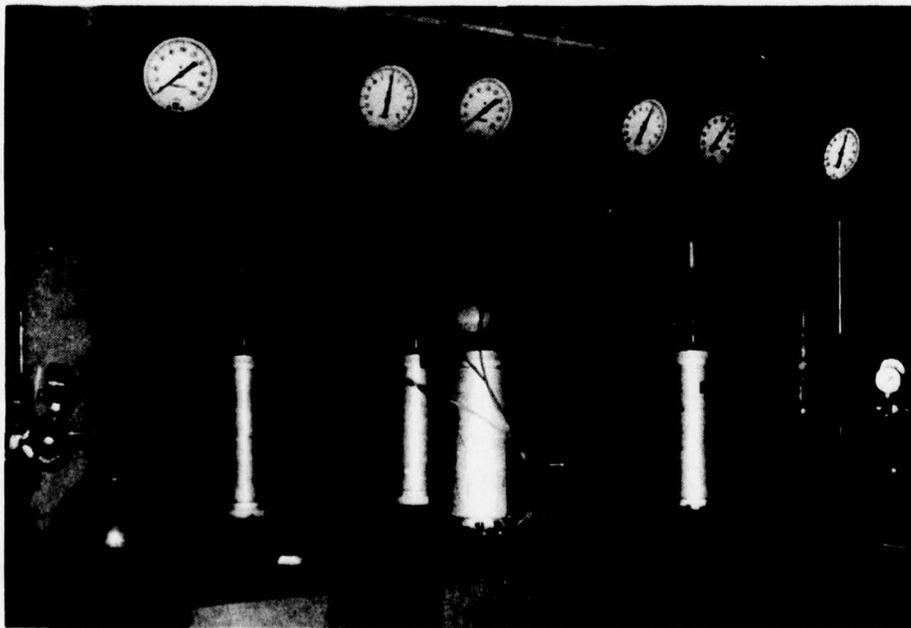
FIGURE 21.   ASSEMBLED CELLS

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accounted for by calibrating each component of the cell as well as the assembled device over the full range of operation of the cell. A variety of mechanical pressures were applied by the use of the loading rod in order to evaluate the response of the samples under various loadings.

SECTION 4
DATA PRESENTATION AND ANALYSIS

INTRODUCTION

In this section the results of the tests involved in this study are presented and discussed. The factors affecting the results are enumerated and evaluated as fully as possible. In some respects this presentation is detailed. It is recognized that this aspect of the report is significant to subsequent researchers but is not particularly important in applications to practical problems.

Results of standard tests are presented in Appendix B. The format consists of grain-size distribution curves and tables containing results of measurements of various samples at each site. Except for one MH soil, all soils tested were classified CH or CL on the basis of the Unified Soil Classification System (USCS). The range of soil types was made intentionally wide in order to cover the full range of soil behavior.

MOISTURE RETENTION (SUCTION)

Suction measurements were made on samples of natural soil clods by use of the filter paper or wide-range method (Ref. 32). These measurements were included for three specific reasons. First, the filter paper technique has great potential for routine use in soil engineering but has not been evaluated in this context. Second, the WES study (Ref. 2) indicated that the suction/water-content relationship is of significant value in characterizing expansive soil behavior. Thirdly, Lytton (Ref. 27) has used the suction-water content relation for obtaining initial suction values used in his mixture theory model of expansive soil behavior.

Before extensive testing began, three aspects of the technique were evaluated: calibration, equilibrium time and hysteresis effects. A calibration curve has been published by McQueen and Miller (Ref. 32). Several points were checked by using solutions prepared with distilled water and reagent grade potassium chloride. Additional data were obtained from the pressure plate used for the COLE tests. The results of a regression analysis coincided very nearly with the previously published curve. These results made it possible to use the previously published calibration for all computations.

Multiple specimens of one sample were used to evaluate the equilibrium time required for the filter paper. Measurements were made at 2, 5, 7, 12, and 14 days. All measurements after five days were consistent. For all tests in this study, a

seven-day equilibration time was used. For most data found in the literature, the suction-water content relation exhibits hysteresis between the wetting and drying portions. However, thermocouple psychrometer data (Ref. 2) did not indicate the existence of hysteresis. Two samples were used to evaluate hysteresis effects by means of the filter paper technique. Figure 22 shows the results. The data indicate no significant hysteresis in the suction-water content relationship for these samples.

Results of moisture-suction measurements for soils in this study are shown in Appendix B. Thermocouple psychrometer measurements are also shown for samples provided to CERF by WES. These were all made on separate samples. Regression lines were computed for these data but did not prove meaningful. The data were then enclosed in an envelope. This approach seemed to enclose the data better and to provide a measure of the variability of the material. It is this variation within a clay formation that results in differential heave.

A summary of the results is presented in Table 6 where the slope and intercept of a line through the center of the data band are shown. In addition, the apparent maximum water content is shown. For several samples, the delineation of this maximum water content is clear. Other samples did not have such a boundary. The concept of this phenomena was presented as an aggregation model (Ref. 46). The implications are that each soil develops bonds as it goes through cycles of wetting and drying and, secondly, that when the soil takes on water, the swell is restricted by these bonds. The importance of aggregation will be apparent as the results of this testing are presented and discussed. The slope shown in Table 6 for Hennessy No. 7 is 12.05, indicating a volumetrically active soil for moisture changes. However, due to aggregation, this soil will not take on water in excess of 13 percent by weight. In contrast, the Mcquino sample is less responsive to unit change (slope = 15.32) but responds over a much wider range (w_{max} = 0.4). These characteristics are detectable by the wide range (filter paper) method of measuring soil suction.

COLE TESTS

All COLE values reported here were obtained by duplicating the NSSL procedure. Previous workers have correlated COLE with clay content ($\% < 2 \mu$m) (Refs. 26, 47). These are shown in Figure 23 together with data from this study. All efforts to correlate COLE-clay relations with mineralogy were unsuccessful. However, a threefold scheme for evaluation appears to be appropriate. Equations (1),

41
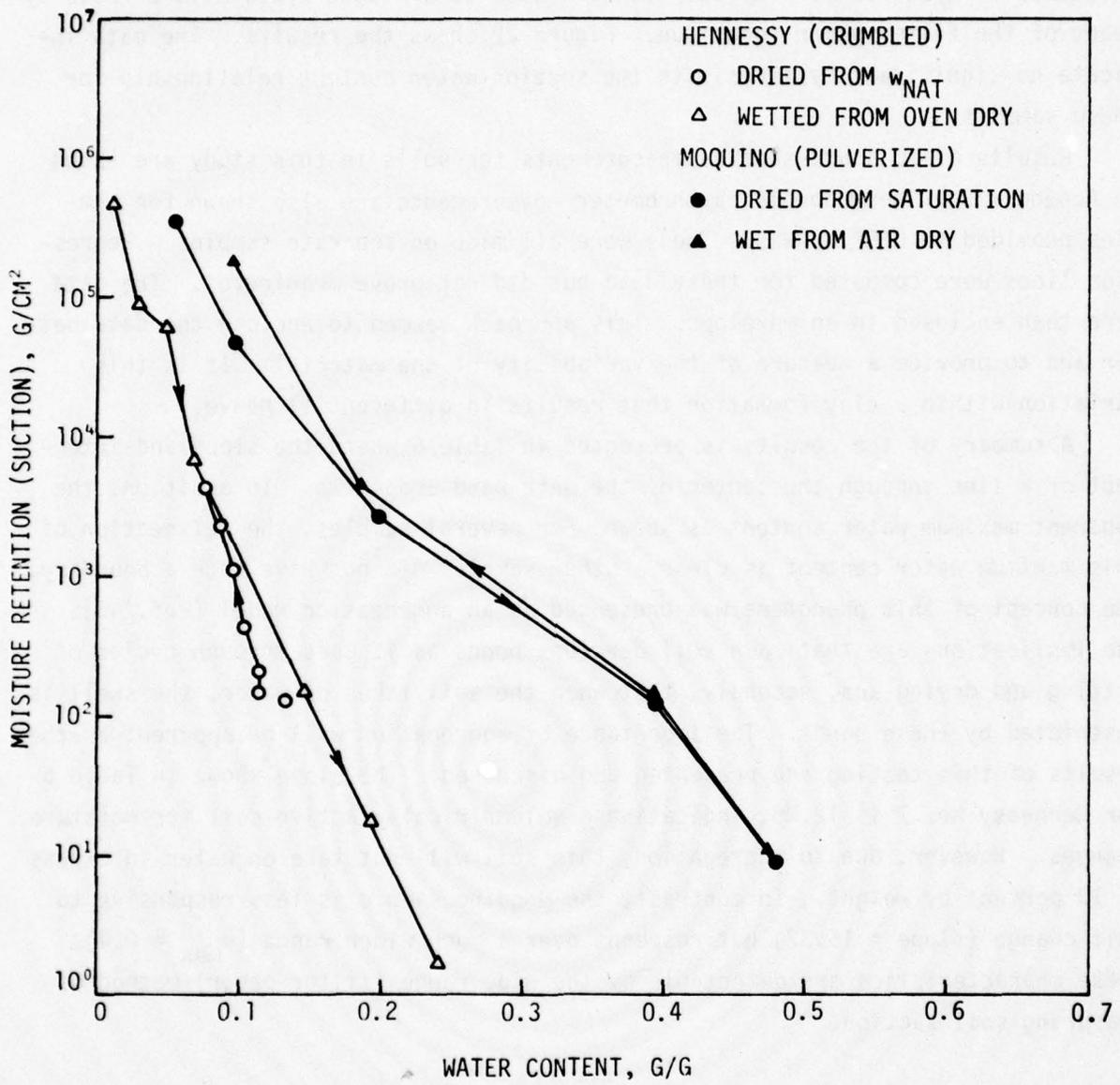
FIGURE 22. RESULTS OF HYSTERESIS EXPERIMENT

TABLE 6. SUCTION-MOISTURE RELATIONSHIPS

Site	Sample	Slope, $\Delta \log h/\Delta w$	Intercept, $\log h$	w_{max}, g/g
Ellsworth	2,5,10	11.17	6.08	0.25
Hennessy	4	20.30	5.70	0.11
Hennessy	7	12.05	5.82	0.13
Holbrook	2,5,8	13.66	6.80	0.33 - 0.49*
San Antonio	4,6,8	8.02	6.46	0.41
DFW	2-2	9.16	6.28	0.34*
DFW	2-3	5.90	6.25	0.65*
DFW	3-1	27.83	6.34	0.20
Moquino	---	15.32	6.54	0.40
Tucumcari	1	10.98	6.12	0.25
Tucumcari	2	14.32	5.78	0.22*
Kelly	---	13.29	6.27	0.33*

*Limited data.

(2), and (4) were obtained by Brasher (Ref. 26) and are based primarily on soils data from the western United States. The clay contents were determined by the standard NSSL pipette method (Ref. 22). This method routinely includes removal of organics and cementing materials by chemical treatment. The data for Eq. (5) were apparently similarly obtained for some Ohio soils. Data obtained by CERF [data points and Eq. (3)] involved a variety of soils. Clay contents were determined by the hydrometer method (ASTM D422) with a dispersing agent as the only treatment.

The CERF data indicate that mineralogical influence is largely masked in this measure of soil response. This masking is probably due to the marked influence of the level of aggregation discussed in the previous section.

The results of the COLE tests may be categorized in three groups. First, a highly expansive group for which the relationship between COLE, compressibility, and clay is:

$$\gamma_h = \sim \frac{1}{3} COLE = 0.00179(\% \ C) - 0.041 \tag{24}$$

Clays with this behavior are not difficult to identify because natural deposits are highly fissured as the result of movements within the material. The shearing

TABLE 8. WETTING MOISTURE RELATIONSHIPS

EQUATION	n	R	SLOPE	INTERCEPT
1	291	0.87	0.00252	.0217 (NSSL)
2	118	0.83	0.00210	.0210 (NSSL)
3	29	0.84	0.00172	.0171 (CERF)
4	71	0.67	0.00080	.0004 (NSSL)
5	25	0.96	0.00092	.0175 (McCORMICK)

CLAY (< .002 mm), PERCENTAGE
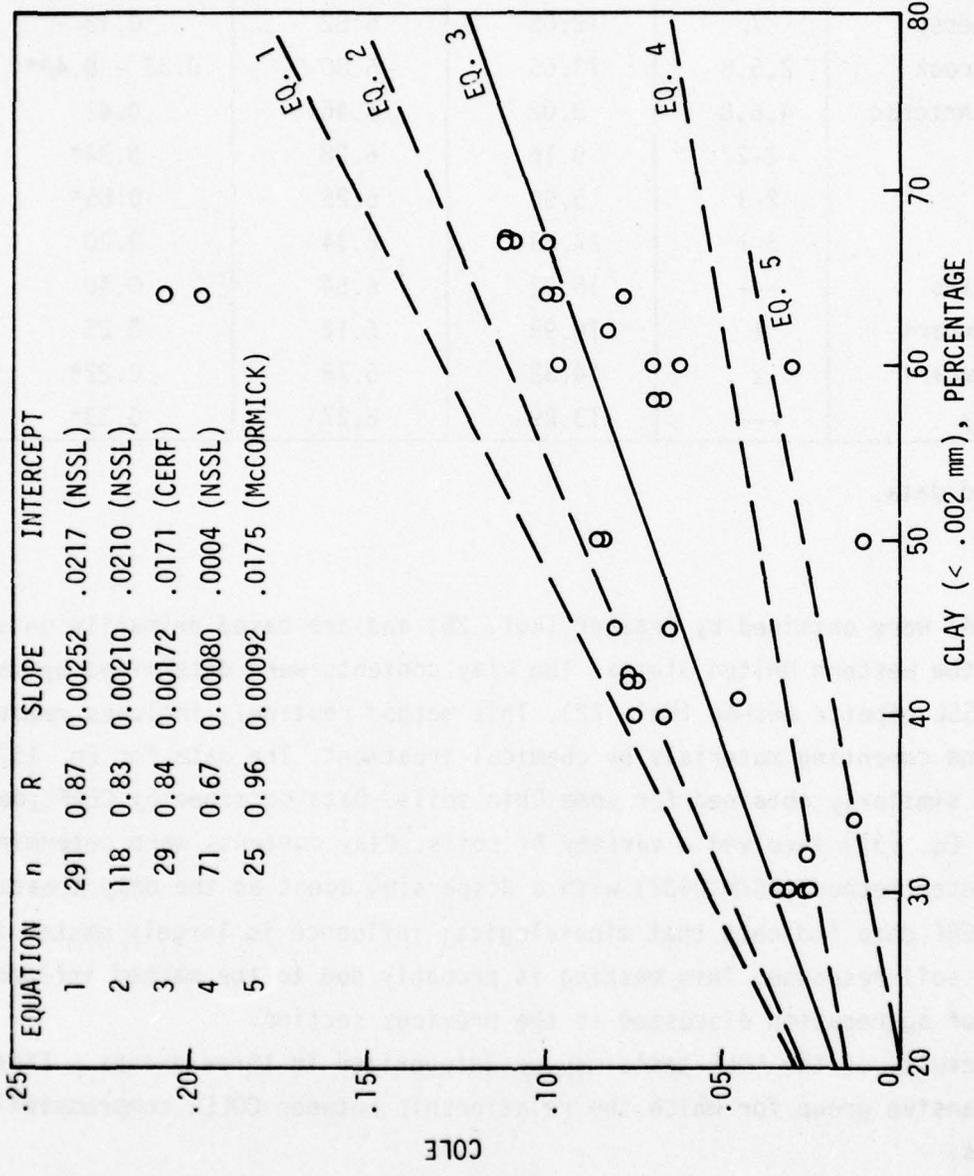
COLE

FIGURE 23. CORRELATIONS BETWEEN COLE AND CLAY

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planes or "slickensides" are easily identified. Clay contents in these materials are characteristically high (60 to 80 percent) as evidenced by clays of the Eagleford Formation in Texas and the Yazoo in Mississippi.

For most clays the following equation is more appropriate:

$$\gamma_h = \sim \frac{1}{3} \text{ COLE} = 0.00057(\% \text{ C}) - 0.0057 \tag{25}$$

or

$$\gamma_h = \sim \frac{1}{3} \text{ COLE} = 0.00057(\% \text{ C}) + 0.0139 \tag{26}$$

The first of these equations [Eq. (25)] is the regression equation from CERF tests. The second [Eq. (26)] is the same equation to which 1.96 times the standard deviation has been added in order to overpredict for 95 percent of the materials. A third equation could be used for the low data points. These materials, like the high ones, are easily identified by their rigid, stable structure and their inactiveness.

All three of these are empirical equations based on limited data. Clay contents were from 30 to 67 percent and COLE values were from 0.01 to 0.22. These equations must be used with caution outside the range of data from which they were developed. Further, it should be noted that the COLE tests were performed on undisturbed clods of the soils. The results thus reflect the aggregation of the natural soil. When this natural structure or fabric is destroyed by remolding, the behavior may be altered significantly.

CLOD TESTS

Natural soil clods were allowed to acquire a variety of moisture contents and then equilibrated with filter papers to measure suction. These same materials were then coated according to COLE procedures and their bulk density determined. After the materials were oven dried, their final bulk density was measured. From these measurements the volume strain ($\Delta v/v_i$) was calculated and converted to linear strain ($\Delta L/L_i$) by the COLE procedure. These data, which appear in the Appendix, were plotted as linear strain versus the initial suction of the clod. As with suction/water-content relations, a band of two enclosing lines was constructed from the data. The summary of these lines is shown in Figure 24.

Once the suction level reached about 2.2 pF (15.5 kPa), volumetric activity apparently ceased. At the other extreme, as the soil dried, volume loss ceased between 5 and 6 pF. All volumetric activity in the soil occurred within the same

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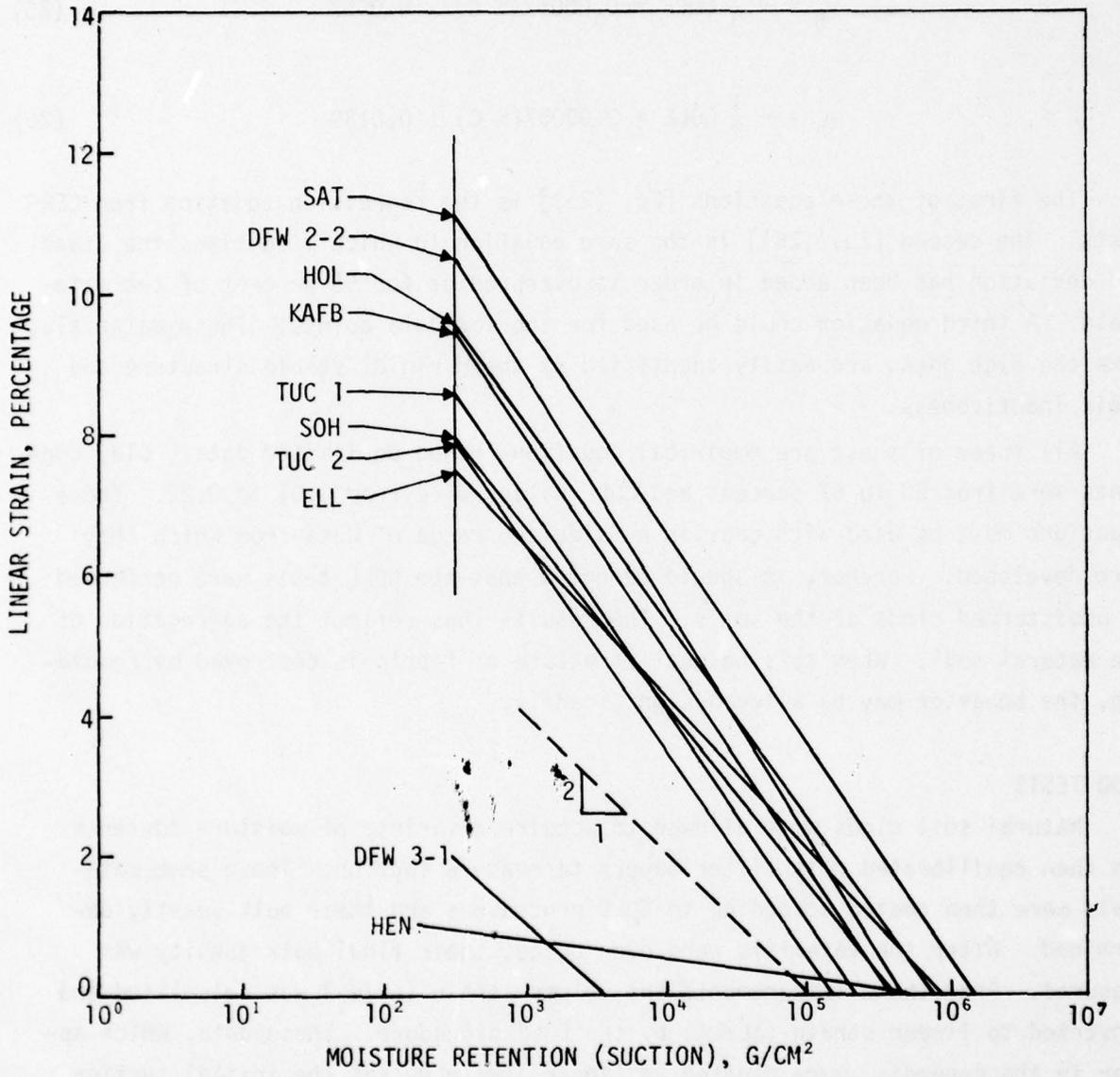
FIGURE 24.   COMPARISON OF LINEAR STRAIN OF CLODS

range of suction values. The slopes were not greatly different. Establishing these boundaries for soil activity is an important step toward simplifying soil characterization.

The bulk density changes measured were used to prepare shrinkage curves (specific volume versus water content) as shown in Section 3. The slope of these relations has been used for heave predictions (Ref. 21). Table 7 shows that the slopes (α values) are flatter for clods than for pulverized material (bar linear shrinkage). Also shown in the table are the initial and final water contents which reflect the range over which the slope applies. The effects of aggregation and unsaturation are evident in the flatter slopes of the natural clods. If it is assumed that the values are the same for load compressibility, a procedure for heave prediction becomes available.

BAR LINEAR SHRINKAGE

The bar linear shrinkage test involves measuring the shrinkage of a pulverized soil sample from above the liquid limit to oven dry. In this work, weight change and final density were determined for each test specimen. On the basis of these data, a shrinkage curve was then plotted (specific volume versus water content). The value of the slope was interpreted as a measure of the soil compressibility and is shown in Table 7. By comparing these to values determined using natural clod samples, one can see clearly that pulverization significantly alters soil behavior. The total linear shrinkage of pulverized materials is clearly dependent on the moisture content range over which the soil is active. The restricting effect of soil structure or fabric reduces the activity of the same soil in a natural condition. Thus, it is pointless to predict behavior of natural materials by using results derived from pulverized soil.

SWELL TEST

Swell tests were performed in a unidimensional mode under conditions in which the suction could be controlled by the introduction of air pressure in the sample chamber. The sample was in contact with water at atmospheric pressure through a semipermeable membrane. Samples were placed in the cell at their existing moisture and density condition. After each sample was equilibrated at 120 psi air pressure, the air pressure was lowered in increments. The pressures were maintained until the vertical dimension change was less than 10^{-4} inches over a 24-hour period. These criteria are similar to those used in many conventional swell tests.

TABLE 7. COMPRESSIBILITIES FROM BULK DENSITY DATA

Site	Sample	Clods			BLS			
		α_C	Range		α_B	Range		$\Delta L/L_i$, %
		$\dfrac{cm^3/g}{g/g}$	w_i	w_f	$\dfrac{cm^3/g}{g/g}$	w_i	w_f	
Ellsworth	2,5,10	0.77	0.11	0.28	0.978	0.15	0.53	11.3
Hennessy	4,7	1.05	0.08	0.17	0.955	0.15	0.35	8.9
Holbrook	2,5,8	1.10	0.11	0.35	1.123	0.09	0.61	15.3
San Antonio	4,6,9	0.87	0.12	0.43	1.175	0.13	0.77	22.0
DFW	2-2	0.86	0.10	0.37	1.084	0.09	0.65	20.0
DFW	3-1	0.80	0.11	0.26	1.061	0.09	0.29	13.4
Moquino	---	0.90	0.10	0.52	1.007	0.10	0.50	18.5
Tucumcari	1	1.04	0.11	0.37	1.096	0.13	0.44	13.0
Tucumcari	2	0.97	0.12	0.43	0.920	0.11	0.64	16.0
Kelly	---	0.89	0.09	0.47	1.165	0.13	0.63	21.0

Early in the testing it was evident that several factors determine the slope of the volume strain-suction relation (compressibility coefficient, γ_h). The suction is a function of the soil particle characteristics (microscale) and their arrangement (macroscale). Properties such as clay type and amount are microscale characteristics, while density is a macroscale factor. The soils used in these tests were originally under in-situ conditions of overburden load (σ_0) and moisture suction (h_o). Upon removal during sampling, σ_0 was released, allowing the soil to rebound. This load would be transformed into work to alter the particle arrangement, which would be resisted by adhesion of the water and clay particles. An increase in moisture suction results. At the beginning of the test, each sample was loaded and at the same time pressurized and put in contact with water at atmospheric pressure. Therefore, both load and suction were altered simultaneously. The response to suction was then measured as the chamber pressure was reduced. The start point in each case varied as the properties of the undisturbed soil varied.

Several operational problems altered the analysis of the swell test data. The cells used were calibrated to provide data for use in arriving at accurate measurements of volume change and load. It was found that slight variations

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occurred between the load generated in the compensating piston and in the cell. The result was a positive applied stress to the specimen. Although these loads were small, they were considered in all computations. Criteria for the end of swell at each stage of the test were set up like those for conventional swell tests. Height changes less than 0.0001 inches in a day (~ 24 hours) were taken as end of swell.

As the results were reviewed, it became apparent that the samples were not reaching the equilibrium conditions determined in this way. Suction measurements made after the tests revealed much higher levels of suction than the axis trans- lation procedure indicated. It was necessary then to compute the compressibility coefficient for each sample in these tests by using the before-and-after suction measurements to arrive at values for the compressibility coefficient under vari- ous loads. Table 8 shows the results of these tests.

Several facts about the nature of expansive soils became evident. The sample-to-sample variation was high. Also both the initial density and moisture condition varied considerably as demonstrated in the table. The magnitude of these variations was greater than originally anticipated. In addition, a number of these materials exhibited variations in the natural fabric or structure of the soil. In the Holbrook, San Antonio, Tucumcari, and DFW 2 soils, the vari- ations were significant due to the presence of a "packeted" structure in the materials. Each of the packets was apparently different from its neighbors. The clearest conclusions to be drawn from these tests were that the material varies greatly over short distances and that some measure of this variation is needed.

TABLE 8. SWELL TEST DATA

Test	Sample	Final Suction, kPa	Initial Suction, kPa	Applied Stress, kPa	Compressibility Coefficient, γ_h	Initial Density, g/cm³
1	SAT 4	252.0	1,133.7	23.2	0.0435	1.362
2	SAT 4	252.6	1,518.8	30.1	0.0101	1.437
3	SAT 9	229.4	1,100.3	1.0	0.0110*	1.291
4	SAT 6	148.1	802.6	3.7	0.0383	1.333
5	SAT 6	212.6	1,642.5	31.7	0.0108	1.218
6	HOL 2	390.4	20,678.0	2.1	0.0137*	1.805
7	HOL 8	77.9	34,396.1	---	0.0007	1.729
8	SOH 1-2	69.7	2,016.1	6.9	0.0055*	1.737
9	SAT 9	18.6	1,554.2	0.8	0.0219*	1.293
10	HEN 4	24.7	2,585.3	4.1	0.0018	2.016
11	SAT 4	18.5	2,463.3	71.0	0.0065	1.402
12	SAT 6	52.8	1,796.8	2.1	0.0602	1.453
13	SOH 1-2	75.8	1,275.0	31.7	0.0306	1.701
14	HOL 8	75.4	11,053.7	3.3	0.0600	1.786
15	HOL 2	120.4	22,830.2	67.7	0.0218	1.883
16	HEN 7	201.6	6,018.8	3.7	0.0007	2.233
17	DFW 3-1-3	49.2	222.6	2.1	0.0171*	1.849
18	SAT 4	18.5	1,821.8	3.7	0.0157*	1.497
19	TUC 2-4	191.7	2,336.2	0.8	0.0183*	1.936
20	DFW 3-1-3	239.2	301.7	71.0	0.0133	1.789
21	TUC 2-4	206.3	1,433.9	72.3	0.0168	1.793
22	DFW 2-2-3	82.9	140.0	2.1	0.1500	1.528
23	SOH 1-2	79.5	1,275.0	73.0	0.0060	1.587
24	DFW 2-2-3	95.4	445.2	31.3	0.0269	1.554
26	KAFB	69.4	246.3	72.3	0.0086	1.507
29	TUC 2-4	390.4	998.9	3.0	0.0705	1.878

*Poor contact load and sample.

SECTION 5
CONCLUSIONS

INTRODUCTION

The objective of this study was to investigate various methods of assessing the swell potential of soils. These methods can be used in preliminary investigations to determine whether soils present sufficient potential for damage to require special precautions. Soil responds to load and suction changes by volumetric expansion or contraction. The nature of the response depends on the composition of the soil (clay type, amount, density) and the nature of the initial and final moisture and load conditions. These aspects of the problem must be quantified for purposes of evaluation. Two composition characteristics are necessary: the compressibilities due to suction and to load. These together with the environmental characteristics of load and suction change are required.

COMPRESSIBILITY DUE TO SUCTION, γ_h

The composition characteristics identified above determine the response of soils to unit changes of load and suction. These response characteristics have been referred to as γ_σ and γ_h respectively. Several measures of these compressibilities have been proposed for use in engineering evaluations of expansive soils. Conclusions regarding their use follow.

The compressibility coefficient, γ_h, may be determined directly from the COLE test. The beginning condition is always one-third atmosphere or 2.53 pF. A point at which volume change ceases must be selected. On the basis of clod tests in this study, that point was found to lie between 5 and 6 pF. Thus if COLE results are available, γ_h may be calculated,

$$\gamma_h = - \frac{COLE}{\log \frac{h_f}{h_i}} = 0.337 \ COLE \text{(for } h_i = 5.5 \text{ pF)} \tag{27}$$

The current classification system used by the Soil Conservation Service in Soil Survey Reports is shown in Table 9.

These categories were developed through correlation with the classification data used in the Building Research Advisory Board (BRAB) criteria for residential slab on ground construction. It seems appropriate to caution users in extrapolating such a categorization from residential slabs to airport pavements. Further

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TABLE 9. SOIL CLASSIFICATIONS FROM COLE TESTS

SCS Ratings	COLE	Compressibility γ_h (h_i = 5.5 pF)	Probability
Very Low	< 0.01	< 0.0034	0.059
Low	0.01 - 0.03	0.0034 - 0.0101	0.176
Moderate	0.03 - 0.06	0.0101 - 0.0202	0.176
High	0.06 - 0.10	0.0202 - 0.0336	0.529
Very High	> 0.10	> 0.0336	0.059

consideration of this aspect is presented later. The last column provides the probability density function for soils used in this study. This indicates that a full range of soils are represented, but most fall in the high category.

The time required to perform the COLE test makes it an unattractive tool for routine engineering use. A modified version of this test was also evaluated. Clods were removed at natural moisture and permitted to attain a variety of water contents. Suction was measured first by the filter paper technique. Then bulk density was measured by the COLE procedures. These data were plotted and are shown in Appendix B. Compressibilities were determined from the summary curves shown in Figure 24. These are plotted versus compressibilities determined from COLE in Figure 25.

The most important fact suggested by this comparison is the apparent reduction in compressibility in the COLE test. This reduction is the result of the sequence used in the test. Clods are coated at the natural moisture condition and then allowed to take on water. In the clod test, samples were allowed to change moisture without restriction, i.e., no coating. The restricting effect of plastic coatings on clods has previously been noted by Tunny (Ref. 48). The two samples above the line of equality were of very low natural suctions (DFW 2-2 and 3-1). Those farthest below the line had the highest natural suctions.

In an attempt to quantify the reduction of swell by the coating, probability distributions were calculated for γ_h derived from COLE tests, clod tests and loaded swell tests. (In Figure 26, it is clear that the coating in the COLE test reduces the compressibility coefficient by about half the reduction due to load in the swell tests.) The mean load for the swell tests included was 3.0 psi (20.7 kPa). The probability distributions were used in this calculation because the sample-to-sample variation distorted the relationships.

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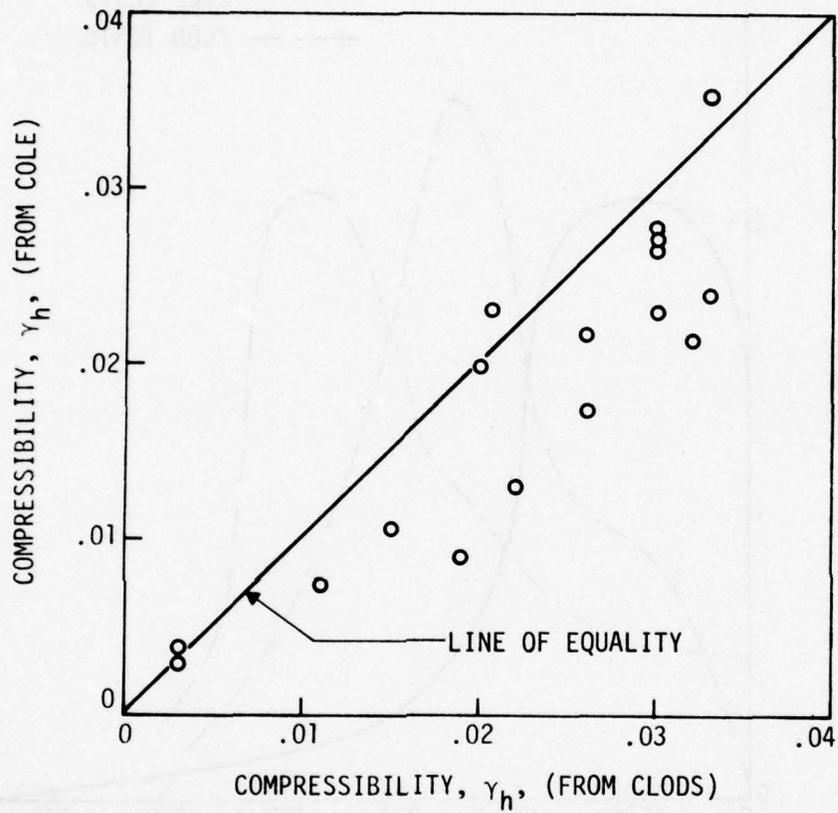
FIGURE 25. COMPARISON OF COMPRESSIBILITY COEFFICIENTS

53
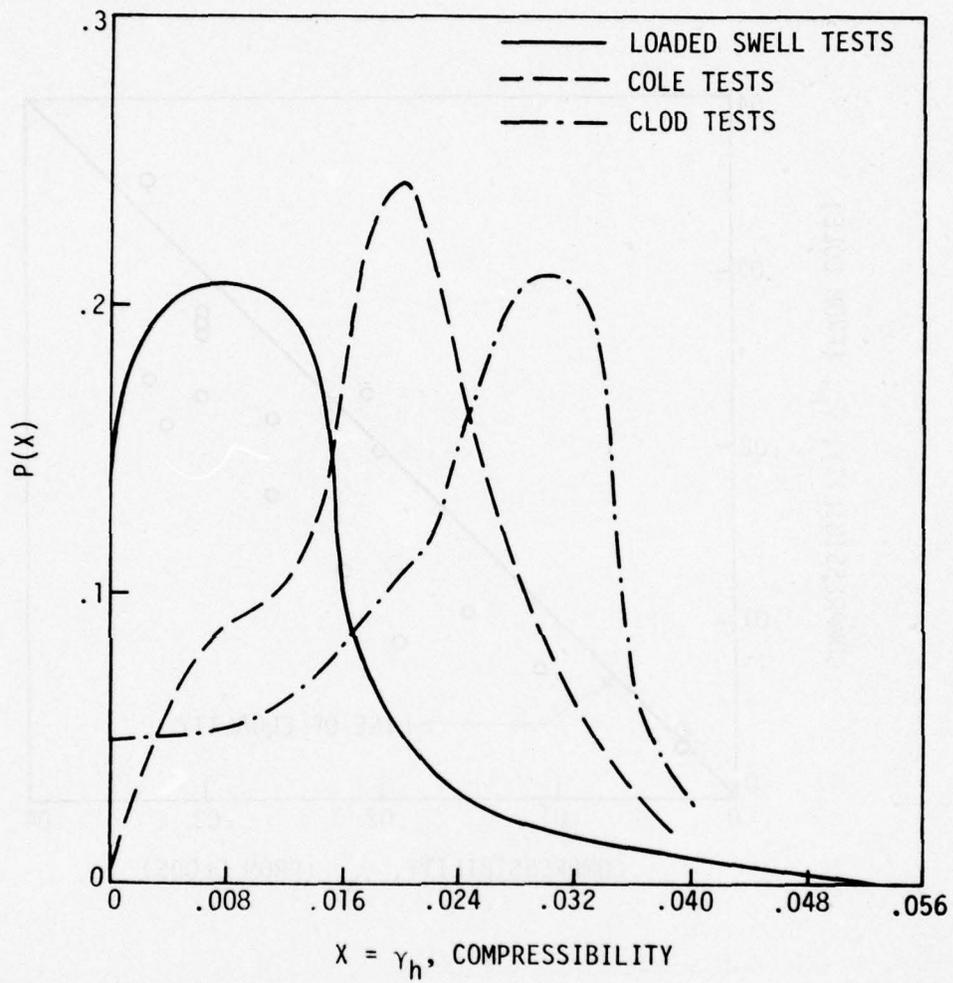
FIGURE 26. PROBABILITY DISTRIBUTION OF γ_h FOR VARIOUS TESTS

From these results it is clear that the plastic coating restricts the volume change of the samples during the test. The amount seems to be equivalent to about 1.5 psi (10.3 kPa). Since this study is of soils beneath pavement structures, it is reasonable to assume that the minimum load of the overlying pavement will always be present. Therefore the COLE test better duplicates the *in situ* boundary conditions, although the free swell of the clod is a better representation of *general soil response* characteristics.

Two other simpler evaluation techniques were also studied. These were (1) the clay content, through correlation with COLE values, and (2) the moisture suction relation. Equations for computing γ_h from clay content were presented earlier. By use of these three equations, values of γ_h were computed. These values are compared with those derived from COLE tests in Figure 27. Shaded points show the restricting influence of aggregation on some samples.

When a modified prediction equation

$$\gamma_h = 0.00057(C) + 0.0139 \tag{28}$$

where

  γ_h = compressibility coefficient
  C = clay content, $\% < 2 \ \mu m$

is used, 95 percent of all data are equal to or less than the predicted value. It seems appropriate to use this equation to make predictions for those samples that do not show aggregation. The normal equation

$$\gamma_h = 0.00057(C) - 0.0057 \tag{29}$$

seems appropriate for aggregated soils. The detection of aggregation requires the development of moisture suction data by use of the wide range method.

The relationship between moisture and suction in soils has been the subject of continuing interest for many years. Correlations between this relationship and other properties have been demonstrated in the technical literature (Refs. 38, 39, 40, 41, 42). The difficulty of making suction measurements has prevented the use of such relations in practical work. In this study the filter paper technique was used to obtain these relations. Due to its low cost and simplicity, this technique has great potential for use in routine engineering work. Therefore, correlation of moisture-suction characteristics with compressibility was studied. Figure 28 illustrates the slope of the moisture-suction relation versus compressibility data. The samples exhibiting strong indication of aggregation are dark.

EQUATION FOR NORMAL SOILS
EQUATION FOR HIGH SWELL SOILS
NOT AGGREGATED
AGGREGATED

LINE OF EQUALITY

COMPRESSIBILITY, γ_h, (FROM COLE)

COMPRESSIBILITY, γ_h, (FROM CLAY)

.08   .07   .06   .05   .04   .03   .02   .01
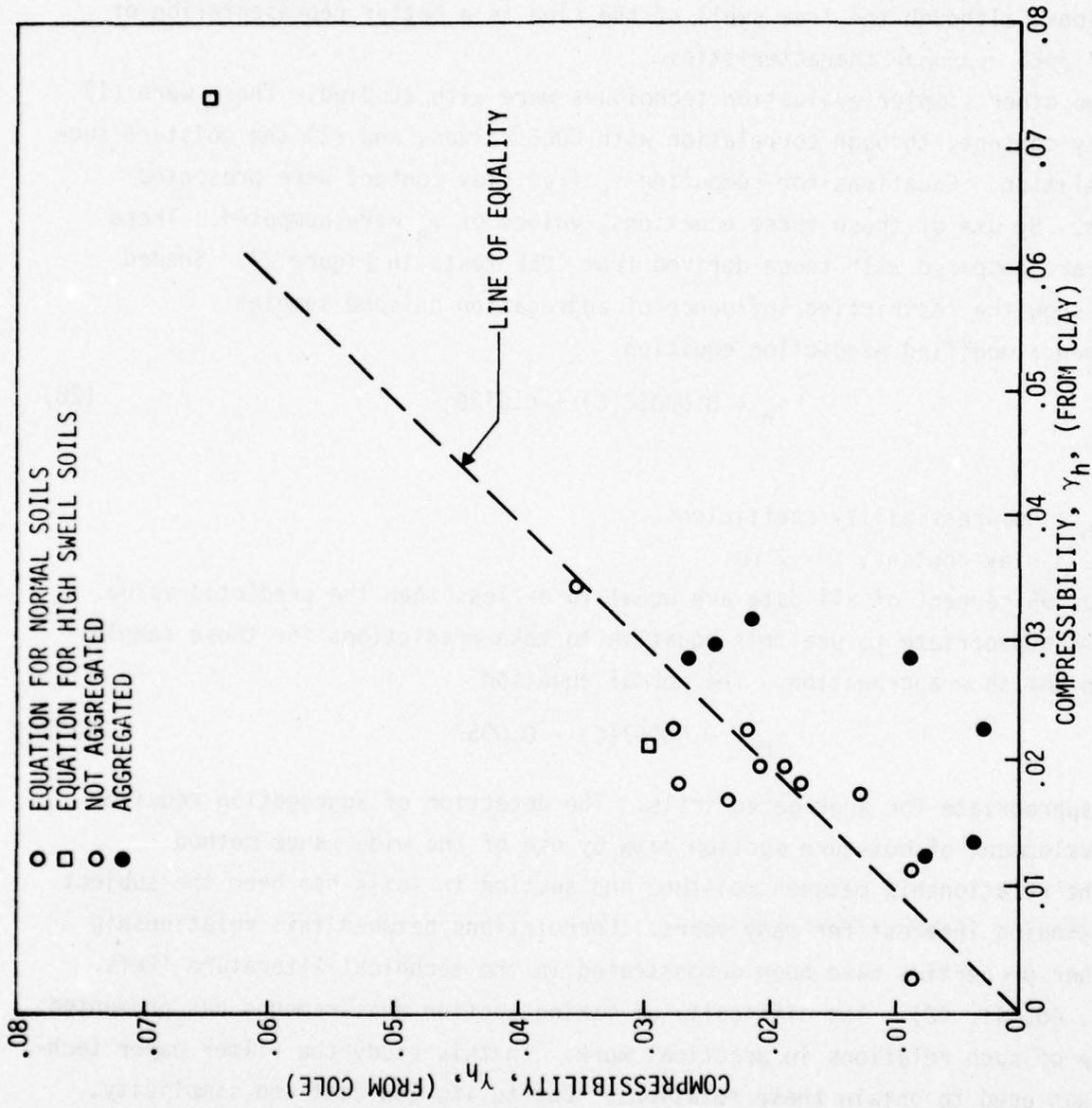
FIGURE 27. COMPARISON OF γ_h FROM COLE AND CLAY

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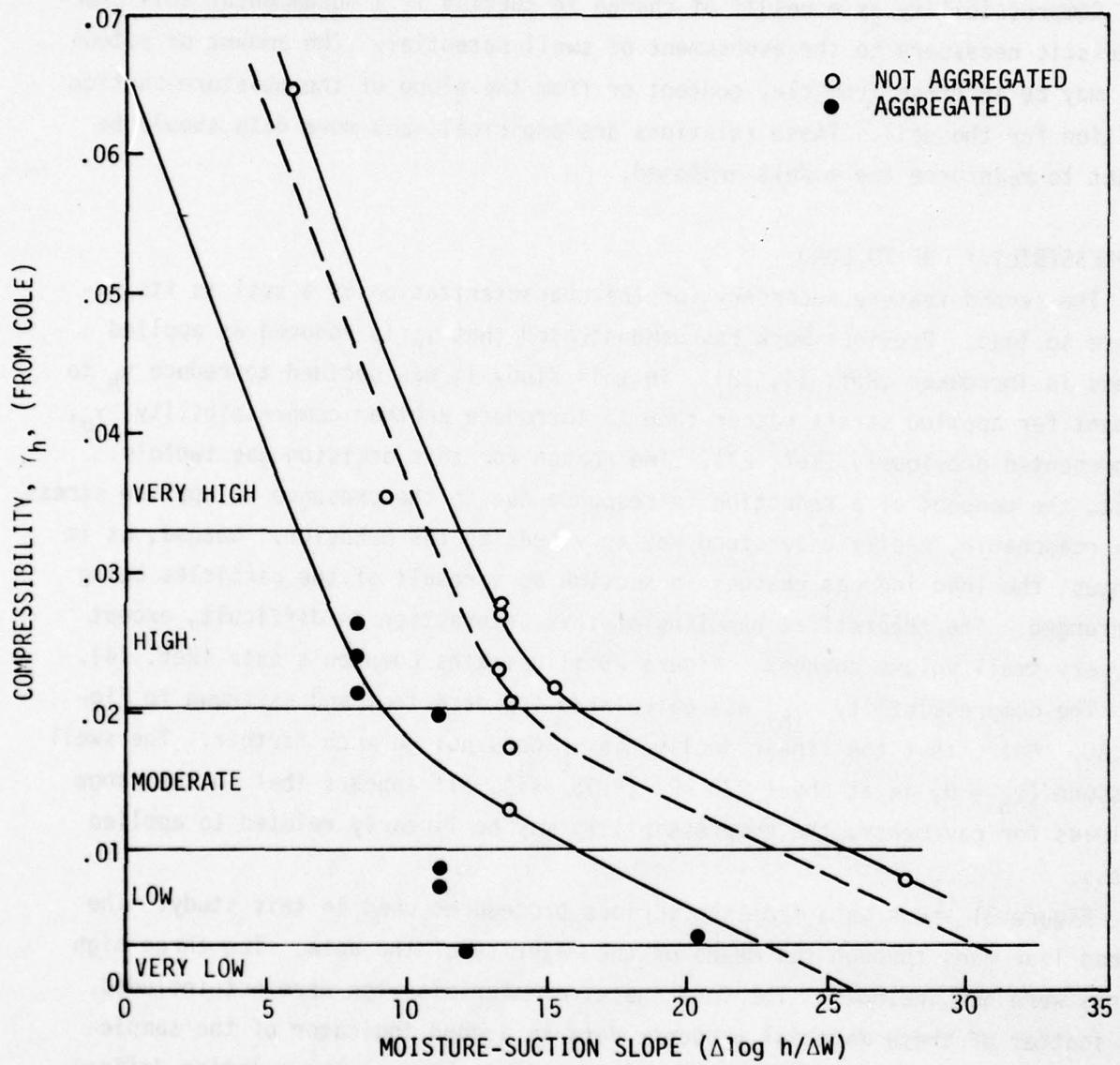
FIGURE 28. CORRELATION OF γ_h (COLE) WITH MOISTURE-SUCTION SLOPE

The effect of aggregation was questionable or non-existent in the other samples. The detection of aggregation is possible with the filter paper technique. On the basis of this data, Table 10 was prepared to illustrate the categorization of swell potential by means of the slope of the moisture-suction relationship.

Compressibility as a result of change in suction is a fundamental soil characteristic necessary to the assessment of swell potential. The amount of potential may be inferred from clay content or from the slope of the moisture-suction relation for the soil. These relations are empirical, and more data should be sought to reinforce the models proposed.

COMPRESSIBILITY DUE TO LOAD

The second feature necessary for the characterization of a soil is its response to load. Previous work has demonstrated that γ_h is reduced as applied stress is increased (Ref. 14, 18). In this study it was decided to reduce γ_h to account for applied stress rather than to introduce another compressibility, γ_σ, as presented previously (Ref. 27). The reason for this decision was twofold. First, the concept of a reduction in response due to the presence of applied stress is a reasonable, easily understood way to visualize the behavior. Second, as it changes, the load induces changes in suction as a result of the particles being rearranged. The theoretical handling of this interaction is difficult, except for very small volume changes. Figure 29 illustrates Compton's data (Ref. 14).

The compressibility, γ_h, was calculated for each test and is shown in Figure 30. Note, that the linear decline in γ_h does not go much farther. The swell pressure ($\gamma_h = 0$) is at about 230 kPa (~ 35 psi). It appears that in the range of loads for pavements, the compressibility may be linearly related to applied stress.

Figure 31 shows data from the various procedures used in this study. The dashed line goes through the means of the majority of the data. The three high points were not included. The very inactive materials also were not included. The scatter of these material property data is a good indicator of the sample-to-sample variation for the materials used in this study. The relation defined previously is also shown. The indication here is that load is more effective in reducing the activity of remolded soils as compared to undisturbed material.

A search was made to determine whether this behavior had been modeled before in the technical literature. Data were found that involved swell under variable loads, controlled strain and the data previously illustrated. If the data are

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TABLE 10. CATEGORIES OF SWELL POTENTIAL FROM MOISTURE-SUCTION DATA

Category	Compressibility Coefficient, γ_h	Slope of h-w $-\left(\dfrac{\log h_1 - \log h_2}{w_1 - w_2}\right)$	P(x)
Very Low	< 0.0034	> 30	
Low	0.0034 - 0.0101	30-23	0.188
Moderate	0.0101 - 0.0202	23-14	0.063
High	0.0202 - 0.0336	14-10.5	0.438
Very High	> 0.0336	< 10.5	0.313
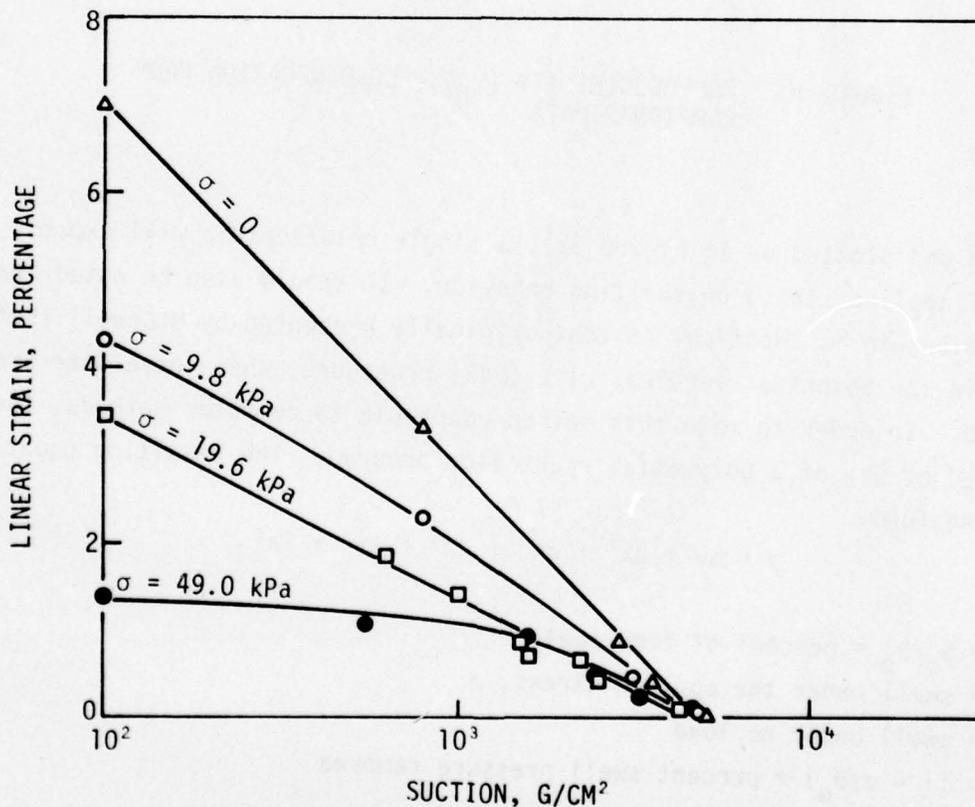
FIGURE 29. COMPTON'S SWELL DATA

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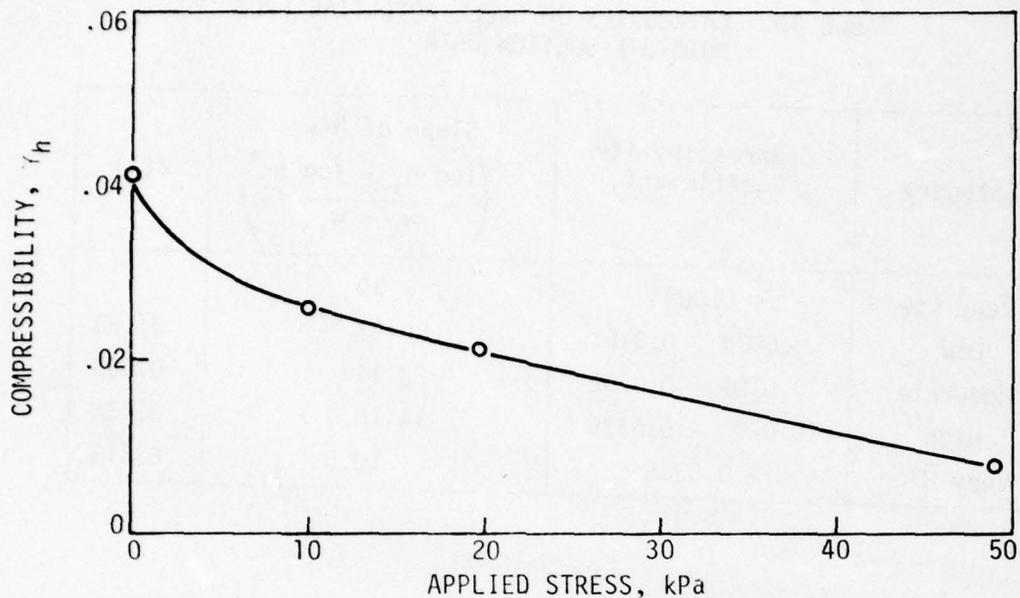
FIGURE 30.   COMPRESSIBILITY (γ_h) - LOAD RELATION FROM
COMPTON'S DATA

normalized and plotted as in Figure 32[*], a single relationship will express the
effects of applied stress on swelling behavior. It should also be noted that
this relationship is identical to that originally presented by McDowell (Ref. 50)
and used in the potential vertical rise (PVR) procedure, when these data are
normalized. In order to make this method adaptable to computer methods, the data
were fitted by use of a polynomial regression program. The resulting equations
were of the form:

$$y = ax + bx^2 + cx^3 + dx^4 + ex^5 + fx^6$$

where

$y = S_\sigma/S_0$ = percent of free swell

S_σ = swell under the applied stress, σ

S_0 = swell under no load

$x = (1 - \sigma/\sigma_0)$ = percent swell pressure removed

σ = applied stress

σ_0 = applied stress for zero swell

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[*]Figure 32, p. 62, contains Reference 49.

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FIGURE 31. RELATION BETWEEN COMPRESSIBILITY AND APPLIED STRESS
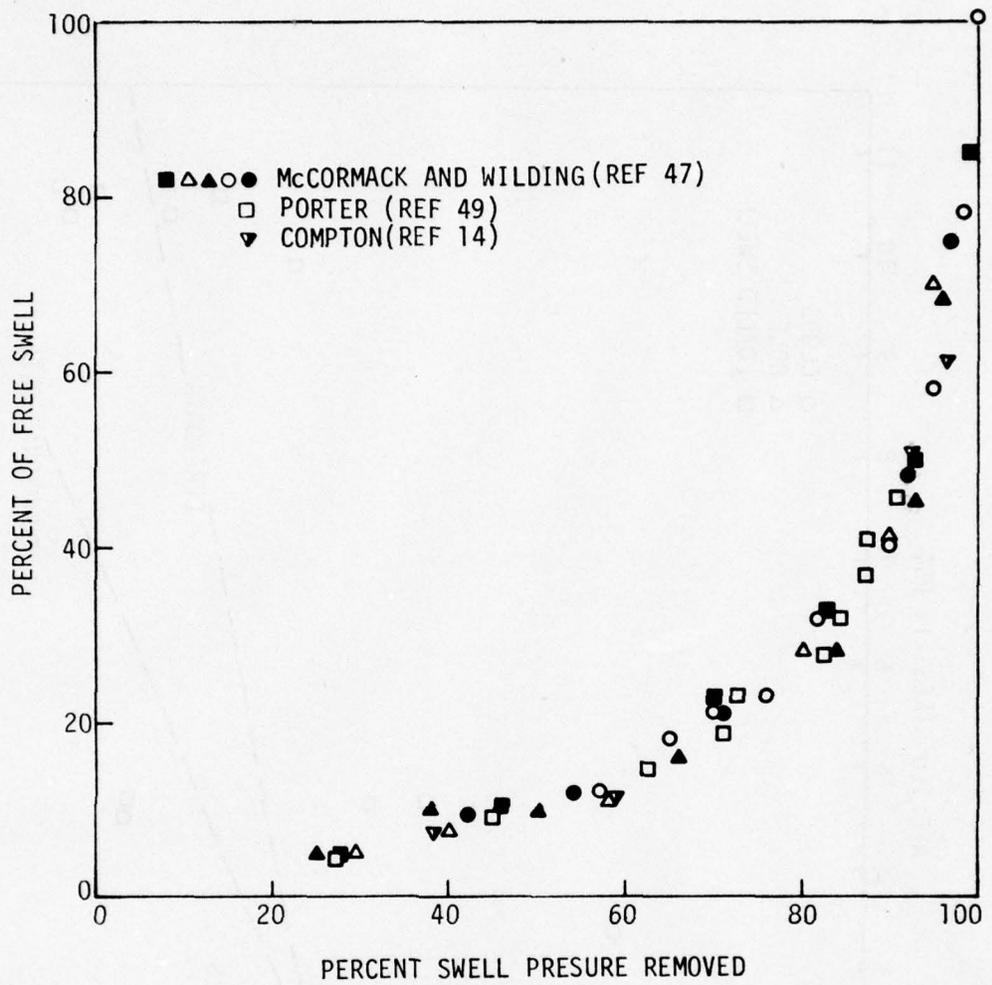
FIGURE 32. NORMALIZED DATA FOR EFFECT OF APPLIED STRESS

The coefficients found for a 4th and 6th degree polynomial are shown below, as well as the correlation coefficient for the data.

Coefficient	4th Degree	6th Degree
a	-0.0812	0.07148
b	2.4794	2.7937
c	-6.3843	-18.304
d	4.9861	49.137
e	---	-57.664
f	---	24.96582
R^2	0.970	0.981

In order to use this model, one must estimate the swell pressure of the natural soil. Probably the most readily available method is the PVC meter developed by the Federal Housing Administration (Ref. 51). With this estimate, a reasonable method of prediction could then be provided for use in evaluating soils.

ENVIRONMENTAL CHARACTERISTICS

The environmental characteristics needed for expansive soil study are the initial and final suction and applied soil load. Once these data are available, the evaluation of differential heave is possible. Initial suctions may be obtained through the filter paper technique (Ref. 32) or the thermocouple psychrometer method (Ref. 2). The distribution of initial suction of soils is dependent on the environment in which the soil exists. This includes rainfall, drainage, ground water table, and potential evapotranspiration. In this study, initial suctions varied from 2.65 pF (43.8 kPa) to 4.6 pF (3904.0 kPa). The WES study reported initial values between 2.50 pF (31 kPa) and 4.48 pF (2961.48 kPa). These data indicate that initial suction values cover the full range expected for in situ soils in the active zone. Here the active zone is the soil that interacts with the environment. The initial suction should be measured rather than estimated. This is especially the case since the filter paper technique is available.

Final suction values are needed in order to compute the change expected for the soil. Several comprehensive studies of final moisture conditions are in the technical literature. This question will be studied in detail in the next phase of work. Since this portion is concerned with categorization of

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soils to identify potentially expansive soils, the final suction will be as-
sumed to be 2.5 pF (31 kPa). The clods in this study appeared to cease volume
change at this point. This figure also corresponds to the field capacity con-
cept used in agriculture, i.e., the point at which moisture will begin to run
off rather than soak into the soil.

The load changes associated with construction of airport pavements should
be determined or estimated (for initial evaluation). The testing involved in
this study was intended to consider the range of loads associated with the
pavement itself and not the effects of large fills or cuts.

SECTION 6
RECOMMENDATIONS

COMPRESSIBILITY COEFFICIENT, γ_h

The fundamental element in characterizing expansive soils is the response to changes of suction, γ_h. It may be estimated by several methods, three of which are outlined below.

(1) <u>Method 1</u> - Natural clods (50 to 200 g) can be obtained from the field site and placed in moisture cans with filter paper to determine the initial suction. After equilibration, the clod is removed and coated with plastic, to determine bulk density. It is then oven dried and the dry bulk density determined. The compressibility coefficient, γ_h, is determined:

$$\gamma_h = - \frac{\frac{1}{3}\left[\frac{\gamma_d}{\gamma_{nat}} - 1\right]}{\log\left(\frac{h_{nat}}{31,010.5 \text{ kPa}}\right)}$$

where

γ_h = compressibility coefficient

γ_d = bulk density oven dry

γ_{nat} = bulk density at natural conditions

h_{nat} = natural moisture suction, kPa

This assumed volume change ceases at 5.5 pF (31,010.5 kPa).

Example:

h_{nat} = 435.02 kPa

γ_{nat} = 1.605 g/cc

γ_d = 1.817 g/cc

$$\gamma_h = - \frac{\frac{1}{3}\left[\left(\frac{1.817}{1.605}\right) - 1\right]}{\log\left(\frac{h_{nat}}{31,010.5 \text{ kPa}}\right)} = 0.024$$

This value can then be converted to an actual estimate by reducing it according to the model in Figure 32. If a swell pressure test is performed, the polynominal model may be used for the load correction. Total testing time for this type of analysis is estimated to be about two weeks.

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The sampling should include a minimum of 5 samples to a maximum of 10 samples. Data scatter is expected. The natural variations are the important considerations. Samples should be taken throughout the site in groups, and each group should cover a linear distance of about 25 feet at intervals of 4 to 5 feet. An estimate of the differential swell can be obtained by computing the estimated swell of each group of samples and finding the differences. Estimated swell is found as follows:

$$\Delta H = \sum_{i=o}^{i=d} \left(\gamma_h\right)_i \log \frac{h_f}{h_i}$$

where

ΔH = surface heave

d = depth of active zone

$\left(\gamma_h\right)_i$ = compressibility coefficient for the ith layer

h_i = initial suction, kPa, of the ith layer

h_f = final value of suction, here assumed to be 31 kPa

There is at this time no field data to evaluate this procedure. Phase 3 should provide some data for use in evaluating this technique. The best evidence for use presently is the BRAB criteria presented before. The ratings are as follows:

Damage Potential	γ_h
Very Low	< 0.0034
Low	0.0034 - 0.0101
Moderate	0.0101 - 0.0202
High	0.0202 - 0.0336
Very High	> 0.0336

These categories represent experience with residential slab foundations. Damage in these structures is the result of either edge heave or shrinkage as the result of the effects of environmental interaction. While such damage occurs where pavement edges are not protected, it is not the design problem of principle concern for airport pavements. The heterogeneous nature of expansive soil formations (the gilgai pattern) will cause differential heave in the material even when all of it is isolated from atmospheric influences. This differential heave must be estimated and provided for in the pavement design. Thus, the above criteria are somewhat conservative, since they were set with use in

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airport pavement structures in mind. However, any subgrade soil reflecting a high or very high rating should be analyzed to evaluate the heave magnitudes expected. The effect of these movements on the structure would then have to be evaluated.

Caution should be exercised in applying this procedure to soils that are very dry (i.e., h_{nat} > 4.0 pF, 980 kPa). In such cases the natural suction is close to the end point of volumetric activity. It was assumed for this procedure that volume change stopped at 5.5 pF (31,010.5 kPa). The values actually measured in this testing program ranged from 5 to 6 pF for the clay soils. As the natural suction increases, the effect of an error in this assumption becomes more significant. This error could be corrected by performing the test on clods to which water had been added.

(2) Method 2 - The second method is based on the correlation of COLE and clay content. Two equations were developed:

$$\gamma_h = 0.00179(C) - 0.041$$

and

$$\gamma_h = 0.00057(C) - 0.00057$$

where

γ_h = compressibility coefficient
C = percent < 2 μm (ASTM D422)

The first equation was developed from data where 40 < C < 70 percent and evidence of high activity in the form of fissures, slickensides, etc. appears. The second equation is valid for other clay soils, 25 < C < 70 not exhibiting this evidence of high activity. Refer to method 1 for the other details.

(3) Method 3 - The suction-moisture relation has been shown to be a fundamental characteristic of soils. The slope of this relationship can be determined by using natural soil clods. The samples are either dried slightly or wetted slightly to achieve a variety of moisture contents, and the suction is measured using the filter paper technique. A line should be constructed from two lines enclosing the data points as shown in the Appendix. The following categories may then be used for classification (based on Figure 28).

| | Slope of h-w |
Category	$\left(\dfrac{\log h_1 - \log h_2}{w_1 - w_2} \right)$
Very Low	≥ 32
Low	22-32
Moderate	14-22
High	10-14
Very High	≤ 10

Another alternative is to select the γ_h directly from Figure 28 by using the slope of the suction-moisture relation. Once this relation is obtained, the procedures under method 1 may be followed.

INITIAL SUCTION, h_i

The wide range or filter paper method of measuring suction appears to be a sorely needed breakthrough if suction is to be used in evaluating expansive soil. The equipment required is inexpensive and easy to use. The data agree with data on similar samples obtained with the thermocouple psychrometer. This instrument has been extensively used by the U.S. Army Waterways Experiment Station and is currently being implemented by the Federal Highway Administration. Therefore, it is assumed that correlation with the psychrometer is a good indication of reliability in the procedure.

Initial suction values should be determined by use of the filter paper technique. The profile should be developed as shown in Figure 33. Ideally a study involving several measurements should be made in order to identify the depth of seasonal activity. The dashed line illustrates the assumed final suction discussed above (i.e., 2.5 pF or 31 kPa). Clearly in the case of the other sample, this assumption does not seem justified; as the material is very dry at depths of 12 feet. Specific procedures for handling this assumption must be made for each site. This procedure will depend on the conditions at the site and the experience in testing the soil involved. The final suction will also be controlled by the drainage of surface water from the site. Proper evaluation and design of these appurtenances cannot be overemphasized.

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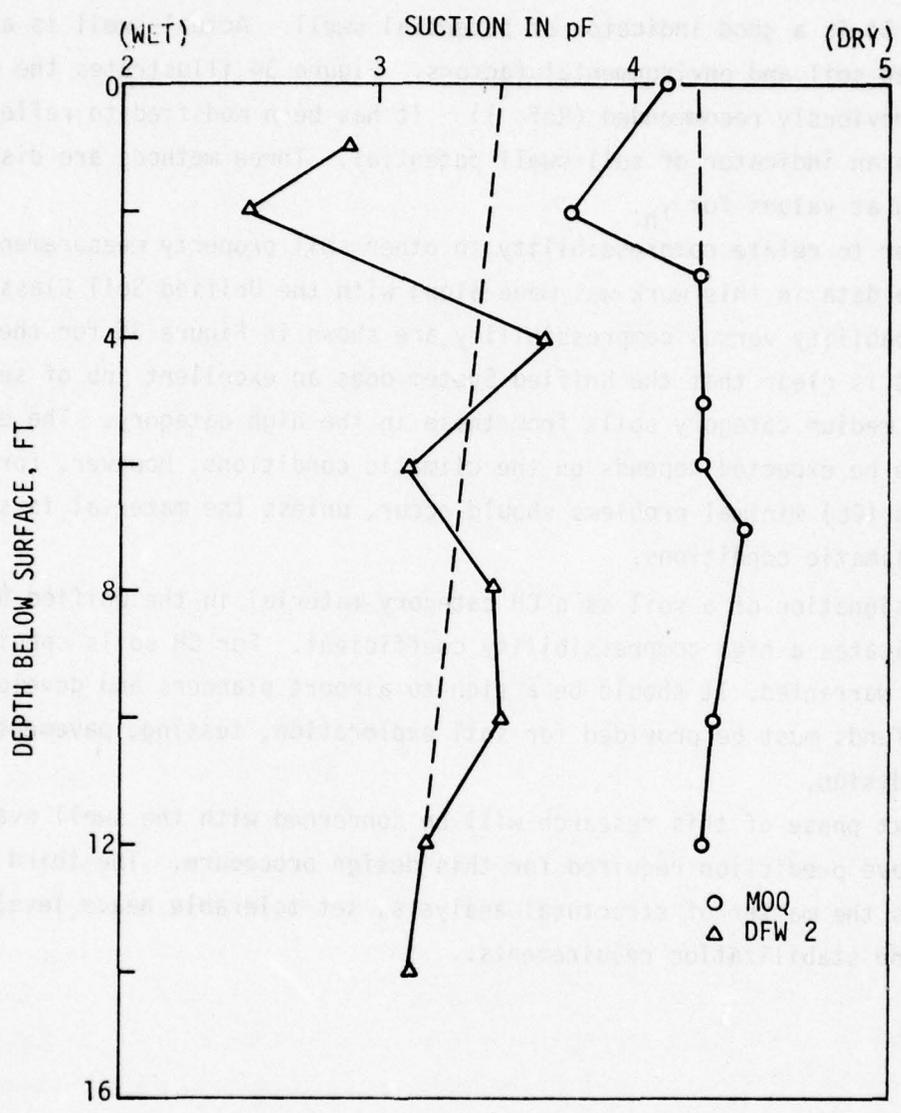
FIGURE 33.   SUCTION PROFILES FOR TWO SITES

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SUMMARY

The compressibility coefficient, γ_h, for changes in suction is a fundamental property of the material. It can be shown to relate to the volumetric activity of a soil. It is a good indicator of potential swell. Actual swell is a function of other soil and environmental factors. Figure 34 illustrates the design procedure previously recommended (Ref. 1). It has been modified to reflect the use of γ_h as an indicator of soil swell potential. Three methods are discussed for arriving at values for γ_h.

In order to relate compressibility to other soil property measurements, a study of the data in this work was made along with the Unified Soil Classifications. Probability versus compressibility are shown in Figure 35 for the soils studied. It is clear that the Unified System does an excellent job of separating the low and medium category soils from those in the high category. The extent of movements to be expected depends on the climatic conditions. However, for low and medium soils (CL) minimal problems should occur, unless the material is subjected to severe climatic conditions.

The designation of a soil as a CH category material in the Unified System clearly indicates a high compressibility coefficient. For CH soils special design studies are warranted. It should be a sign to airport planners and developers that additional funds must be provided for soil exploration, testing, pavement and foundation design.

The next phase of this research will be concerned with the swell evaluation and heave prediction required for this design procedure. The third phase will address the matter of structural analysis, set tolerable heave levels, and determine stabilization requirements.
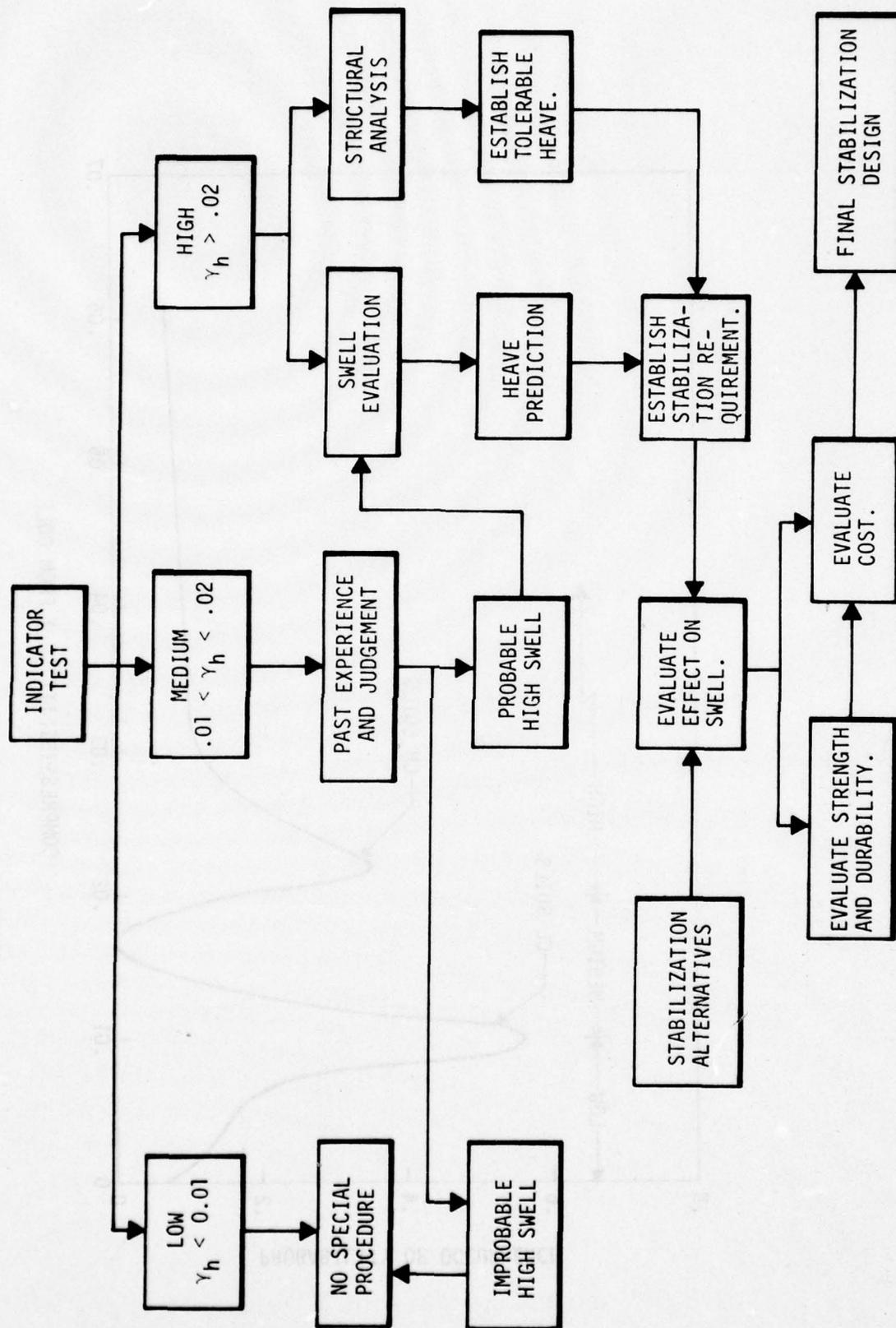
FIGURE 34. RECOMMENDED DESIGN PROCEDURE
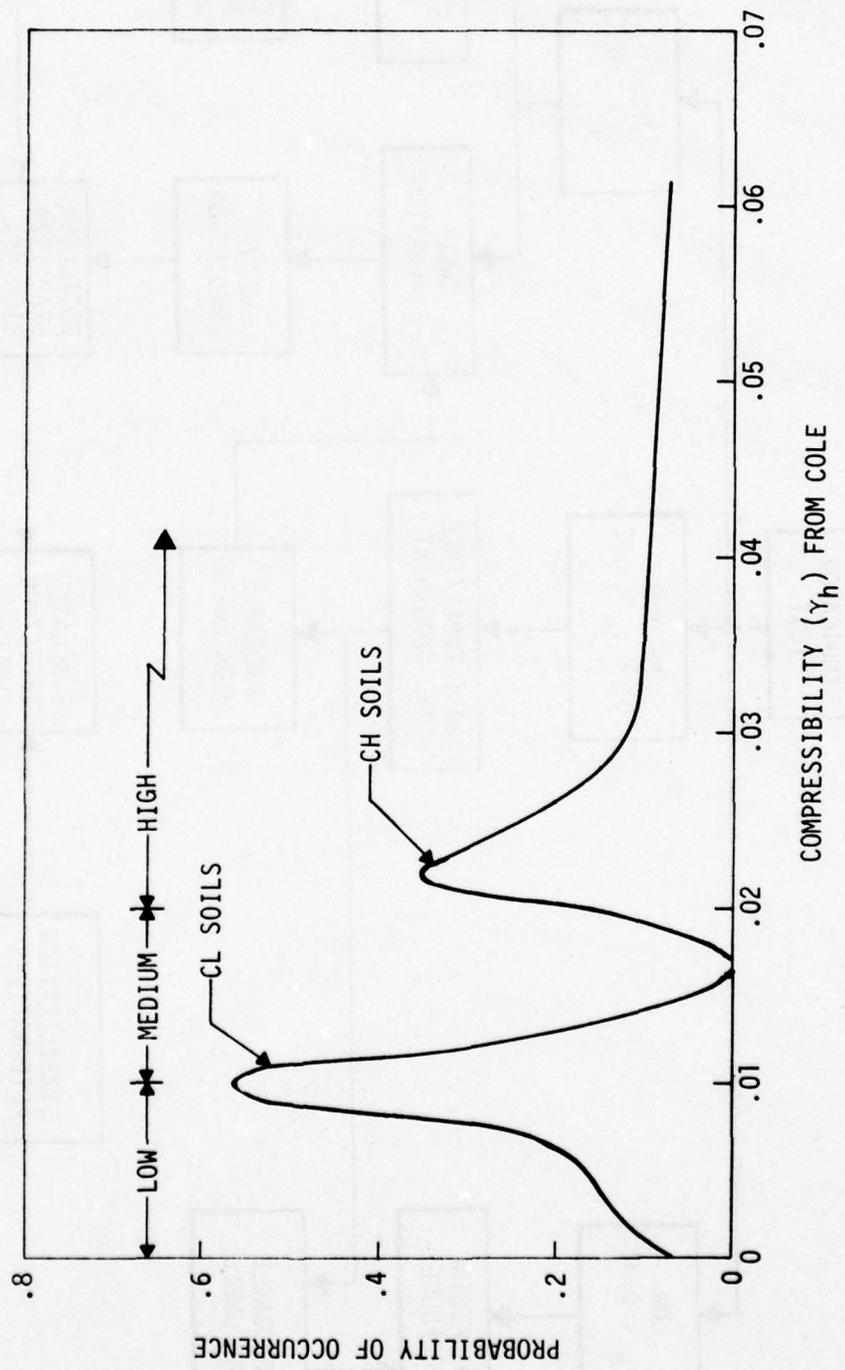
FIGURE 35. DISTRIBUTION OF γ_h FOR SOILS STUDIED

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APPENDIX A
TEST PROCEDURES

WIDE RANGE GRAVIMETRIC METHOD FOR MEASURING MOISTURE STRESS

The following description was obtained from the original published procedure prepared by McQueen and Miller. This procedure was followed in all respects except the temperature control requirements. This exception was made in order to make application to routine use by engineering soils laboratories. The following material is presented by permission of the copyright holder. References in this material are McQueen's and Miller's and are listed in their text, p. 231. These references do not appear in the main Reference List of this report.

CALIBRATION AND EVALUATION OF A WIDE-RANGE
GRAVIMETRIC METHOD FOR MEASURING
MOISTURE STRESS

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INTRODUCTION

The soil-moisture research staff of the Soil and Moisture Conservation Program, Water Resources Division, U. S. Geological Survey recognized a need for a wide-range method for measuring moisture stress in soils while conducting studies on the moisture requirements of arid land plants in the western United States. The standard or popular methods for measuring moisture stress either did not cover the range of stress values expected on arid lands or they were not adaptable to use on field samples.

Various methods, described in the literature, were examined to determine if they could be used for arid lands research. A method proposed by Robert Gardner about 1936, using filter papers as indirect moisture stress sensors was investigated, modified, and eventually adopted.

HISTORY

The use of paper as a moisture stress sensor has gradually evolved in Europe and the United States. Hansen (5) working at the University of Copenhagen used blotting paper as a carrier for sugar solutions. Blotting paper strips, saturated with four different concentrations of sugar solutions, were exposed to soil samples in closed chambers. The sugar solution that did not lose or gain weight was assumed to represent the stress level in the sample. Stoker (13) used a similar procedure with a larger number of sugar solution concentrations for better accuracy. Gradman (4) improved the method by using a single strip of blotting paper soaked in a salt solution and then calibrated for weight versus stress. These sensors were enclosed with soil samples until complete equilibrium was reached. This method with some minor refinements has been used in France by Eckhardt (1).

The first use of paper as a moisture stress sensor without a hygroscopic salt and probably the only previous use in the United States was reported by Gardner (2).

METHOD

The basic concept of using filter paper as a passive gravimetric moisture stress sensor as proposed by Gardner has been followed by the authors but details of the method have been changed to eliminate some hazards and difficulties, and to adapt it to use with routine gravimetric soil moisture sampling programs.

Apparatus and Supplies. In addition to the equipment required for routine gravimetric soil moisture tests, the following are needed: (a) an analytical balance accurate to 0.0002 gm.; (b) small lightweight weighing boxes such as Soiltext Catalog No. LT-15; (c) constant temperature chamber (20°C); (d) filter paper—Schleicher and Schuell No. 589 White Ribbon, 5½ cm dia. circles was used in this study. (Other grades of paper may require calibration); (e) pentachlorophenol "Dowcide-7" reagent grade or equivalent if obtainable; (f) ethanol or methanol reagent grade solvent; (g) plastic electrical tape to seal soil moisture cans.

Procedure. (a) Filter paper discs, (d) above, are pretreated to inhibit biological decomposition by dipping them into a solution of pentachlorophenol in ethanol and allowing to air dry. Extensive tests have shown that a 2 per cent solution will leave sufficient protection on the discs for two or three weeks. The pentachlorophenol is insoluble in water so reagent grade solvent must be used. Methanol may be satisfactory but it is more dangerous if accidentally ingested. Penta is nonhygroscopic and we have been unable to detect any differences in moisture retention characteristics due to concentration of the solution used so we are using a 3 per cent solution to insure protection of samples that may be processed later than scheduled.

(b) One treated filter paper disc is placed in

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the top of each gravimetric soil-moisture sample when it is obtained in the field and the container is sealed with plastic insulating tape (item g above). It is suggested that the soil-moisture sample should fill at least a 4-ounce sample can, especially if the sample consists of coarse sand with a small surface area. At field capacity the filter paper will absorb about 0.1 gm. of water and if there are only about 5 or 6 gms. of water in the sample this could change the measured stress.

(c) The samples are transported to a laboratory and allowed to equilibrate in a constant temperature chamber at 20° ± 1°C for at least one week. Moist samples should be transported with care. A sample with a low stress may change its stress value without a change in moisture content by simply rearranging its pore sizes and shapes. A few miles on a rough road is all it takes.

(d) After equilibration the filter paper is removed from the soil sample can, placed in an aluminum weighing box (item b above) and its moisture content is accurately determined. The soil sample should be treated as a routine soil-moisture content sample after the filter paper is removed.

In order to determine the moisture content of the filter papers accurately, some departures from normal laboratory procedures have been used. Transference of the filter papers from the soil-sample containers to the small aluminum weighing boxes must be done as rapidly as possible and with as little contact of the paper with hands and tools as possible. If there are sand grains clinging to the paper they should be quickly flipped off. The wet weight of the paper should be obtained immediately because evaporation is rapid in dry laboratory air. The weighing boxes are placed in the oven with their lids partly open to permit rapid drying. Drying time is normally overnight, but it can be shortened to two hours if necessary. The lids of the weighing boxes are closed while the samples are still in the oven. Upon removal from the oven the boxes are placed on a heavy aluminum plate for 30 seconds to cool and then they are weighed immediately. Previous experience in our laboratory has proved that desiccators are a source of error in oven dry weighings, so we don't use them. The light weight weighing boxes will cool in less than 30 seconds and the weight

can be obtained before the papers can adsorb a weighable quantity of moisture.

The moisture stress in the soil sample may be computed from the moisture content of the filter paper or it may be obtained from a plot of the calibration relationship.

CALIBRATION

The filter papers were calibrated under conditions and procedures that were as much as possible like those that exist during normal use of the method. However, additional conditions and procedures were investigated to help define limits of accuracy and time requirements for equilibrium.

For high stress

For stress levels above 15 bars, filter papers were exposed to saturated salt solutions in closed containers in a constant temperature chamber. Periodic weighings were made of both initially wet and initially dry papers to define the time required for equilibrium and to determine equilibrium moisture contents. Technical data for this phase of the calibration are given in table 1.

Results agree with the corresponding portion of Gardner's calibration curve for which he exposed the papers to sulfuric acid solutions in evacuated chambers. Available data for relative humidity above saturated salt solutions do not agree but the range of disagreement shown in table 1 is small.

For medium stress

For stress levels from one bar to fifteen bars, samples of several soils were brought to given stress levels on a pressure membrane extractor and then sealed in cans with wet and/or dry filter papers. For points below one bar stress, soil samples were brought to given stress on a pressure plate assembly and then sealed in cans with wet and/or dry filter papers. All calibration samples were held in a constant temperature chamber at 20°C for equilibration.

A summary of calibration data for stress between 0.1 and 15 bars is given in table 2.

The procedure for routine use of filter papers to determine the stress in moisture samples involves use of initially dry papers so only the data for initially dry papers was used for the calibration curve. Also, some data obtained

TABLE 1

Calibration data for stress levels controlled by saturated salt solutions in a constant temperature chamber

Salt	$Na_2S_2O_3$		Na_2SO_4		$CaSO_4$	
Relative vapor pressure*	0.78		0.93		0.98	
Log of stress in cm.	5.55		5.00		4.44	
Stress in bars	335		97.9		27.25	
Moisture Content	A†	B	A	B	A	B
Time: 0 days	22.79	3.50	21.95	3.50	22.04	3.61
7 days	10.64	8.01	17.84	16.36	23.70	22.71
14 days	10.54	7.90	17.90	16.35	24.05	23.10
28 days	10.42	7.96	17.93	16.39	24.12	23.93
76 days	10.30	8.04	17.79	16.58	25.21	26.10

* Published values do not agree. Data given are from report by O'Brien (7). Corresponding values computed from International Critical Tables (Washburn [14]) are: 0.798, 0.94, and 0.98. The range of disagreement is small.

† "A" papers were premoistened by exposing over distilled water for two days prior to start of test. "B" papers were started in air-dry condition.

under varying conditions to define limits of accuracy are not included in the calibration data.

For low stress

Calibration for stress levels below 0.2 bars was determined from field samples obtained at known heights above a water table. A sampling program conducted in conjunction with a study of water use by phreatophytes on the Gila River in Arizona provided data for calibration. Eighteen profiles were sampled to the water table. Moisture stress in centimeters of water was determined from a tentative calibration curve, and the stress plotted against depth of sample below land surface in centimeters. Portions of several of these profiles could be represented by straight lines with similar slopes. The tentative calibration curve was adjusted to make the slopes of these lines approach a 1:1 relationship. The depths of the zero stress intercepts of these lines were compared with known depths to water and they were found to be in agreement. The adjusted calibration curve agreed with the data obtained with the pressure plate assembly at 0.1 and 0.2 bars stress.

Calibration curves

All the calibration data were plotted on semilogarithmic paper, and the best fit was found to be two straight line segments that intersect at 0.21 bars. Formulas for the lines are

$$\log_{10} S_B = 3.2380 - 0.0723M \quad (M < 54\%)$$

TABLE 2

Summary of calibration data from samples prepared on pressure membrane extractor (M) and pressure plate assembly (P)

Stress (Bars)	Stress Control* Apparatus	Samples (No.)	Moisture in Paper in Per Cent	
			Mean	Computed
15	M	20	27.89	28.52†
10	M	9	29.76	30.95
5	M	12	33.90	35.12
2	M	12	41.45	40.62
1	M & P	4§	43.29	44.79
0.5	P	8	49.12	48.95
0.3	P	3	52.21	52.02
0.2	P	8	58.17	58.1‡‡
0.1	P	4	88.33	87.48‡‡

* See Richards (10) page 109.

† $M = (3.238 - \log_{10} S_B) \div 0.0723$.

‡‡ $M = [(9.8966 - 10) - \log_{10} S_B] \div 0.01205$.

§ Several samples rejected because of leakage in pressure plate.

and

$$\log_{10} S_B = (9.8966 - 10)$$

$$- 0.01025M \quad (M > 54\%)$$

where S_B is stress in bars and M is the moisture content of the filter paper in per cent of dry weight.

In order to avoid use of awkward negative characteristics of logarithms the stress can be expressed in centimeters of water, or pF as suggested by Schofield (1935).

$pF = log_{10}S = 6.24617 - 0.0723M \quad (M < 54\%)$

$pF = log_{10}S = 2.8948 - 0.01025M \quad (M > 54\%)$

DISCUSSION

A detailed explanation of why there are two calibration curves is not within the scope of this paper but it will be discussed in a subsequent paper. It may be noted however that the intersection at 0.21 bars is near the accepted field capacity stress values of 0.1 to 0.3 bars. Others have reported a break in conductivity and moisture retention curves at this stress level. W. R. Gardner (3) in discussing capillary conductivity said that conductivity of soil becomes limiting at 0.15 to 0.2 bars.

This abrupt change in slope represents a change in energy level when gravity drainage is replaced by other modes of moisture movement such as film flow and vapor diffusion. It may be the change from capillary to pellicular moisture described by Rode (11).

Figure 1 shows a comparison between the calibration curve published by Gardner and the curve obtained during this investigation. Gardner said of his curve: "The upper portion (low stress end) of the curve is no doubt somewhat above the true value, as only drying papers were used." This no doubt contributed to the differences. However, Gardner used a centrifuge for this portion of his curve and recent investigations on the effects of temperature on centrifuge moisture tests (Prill and Johnson, [9]) indicate that without temperature and humidity control the centrifuge is not an accurate moisture stress instrument. For the high stress end of his curve Gardner exposed papers to sulfuric acid solutions in evacuated chambers. The pressure plate and pressure membrane extractors that are currently used for moisture stress measurements were not available to Gardner. These instruments, used in this investigation have permitted a more accurate calibration of this method.
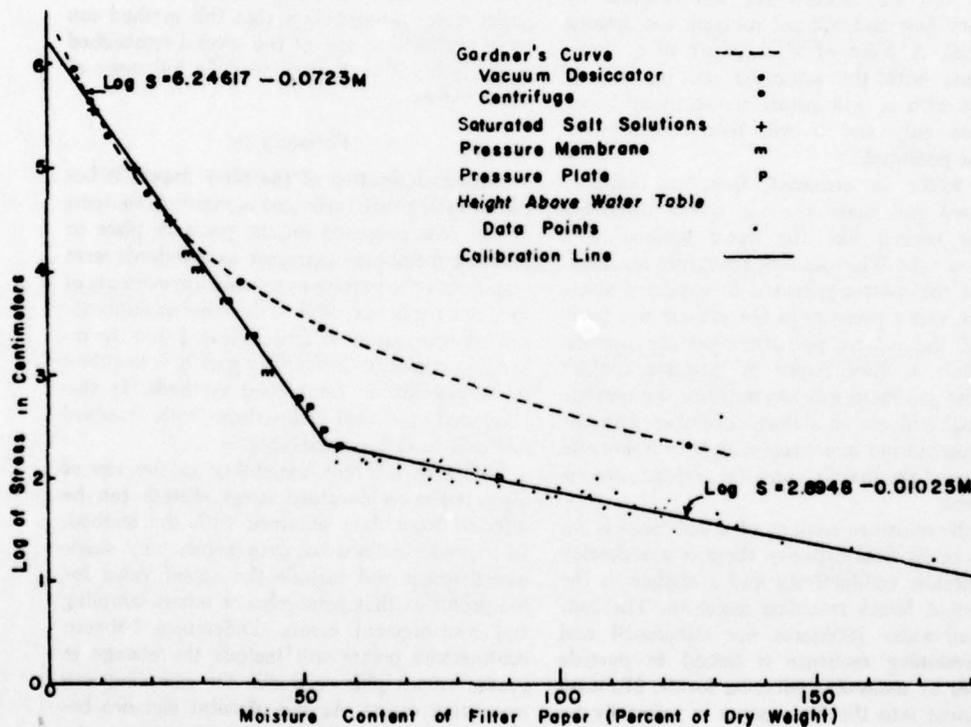
FIG. 1. Summary of calibration data showing agreement between four methods used in this study and one of the two methods used by Gardner.

What the filter paper measures

The term "soil-moisture stress" or "moisture tension" sometimes means the total pressure difference that a plant must overcome to extract moisture or it may mean vapor pressure deficit in the soil air, the capillary stress or some combination of stress components. Often the characteristics of the measuring system determine what is meant by "moisture stress." An understanding of the basic principles of a measuring system is essential for proper use and interpretation of data obtained.

Moisture stress may be defined as the difference in free energy between the water in the soil and a body of distilled water at the same elevation and temperature. In a saturated soil mass, water movement occurs as liquid flow through interstices in response to gravity, capillary stress or other force gradients. The only stress component at saturation is the osmotic potential due to dissolved salts. A piece of filter paper placed in contact with a saturated soil will absorb the soil solution by capillary flow and will not measure the osmotic potential. A piece of filter paper in a closed container with the saturated soil but not in contact with it, will adsorb moisture by vapor diffusion only and it will then measure the osmotic potential.

As water is removed from an initially saturated soil mass the air water interfaces become curved like the water surface in a capillary tube. The radius of curvature is a function of the matrix potential or capillary stress but the vapor pressure in the soil air is a function of the matrix potential and the osmotic potential. A filter paper in intimate contact with the soil mass will not measure the osmotic potential and one in a closed chamber with the soil mass but not in contact with it, will measure the sum of the matrix potential and the osmotic potential.

As the moisture content of a soil mass is reduced below field capacity there is a reduction in hydraulic conductivity and a change in the pattern of forces retaining moisture. The concave air-water interfaces are eliminated and the remaining moisture is bound to particle surfaces by molecular adhesion forces. Moisture movement into the filter paper is primarily by vapor diffusion and the filter paper measures the total stress.

In normal use, matrix stress is measured at high moisture contents while total soil moisture stress is measured at low moisture contents, due to a lack of direct contact between the soil moisture and the paper.

EVALUATION

Evaluation of new quantitative methods for measuring a soil parameter usually involves direct comparisons with established or "standard" methods. In evaluating the use of filter paper as moisture stress sensors this was not considered expedient because the standard methods cover limited ranges of stress values, cannot be used in the same manner as the filter papers and are not directly comparable. All established methods available to our laboratory were used for calibration and, therefore, could not be considered as independent evaluation standards. Data obtained with a new method, when corroborated by independent data, provides an alternate means for evaluation. Some examples chosen from data obtained with filter paper stress sensors show that this method can be as accurate as any of the several established methods and it is effective over the full range of stress values.

Variability

During calibration of the filter papers it became evident that variations in moisture contents of soil pats prepared on the pressure plate or pressure membrane extractor as standards were larger than the variations in moisture contents of the filter papers exposed to the same or duplicate soil moisture samples. This indicated that the inherent variability of the filter papers is less than the variability in established methods. It also precluded statistical comparisons with standard methods to define variability.

Limits of inherent variability in the use of filter paper as moisture stress sensors can be inferred from data obtained with the method. In a profile defined by data points, any single measurement will include the actual value for the profile at that point plus or minus sampling and measurement errors. Differences between contiguous points will include the change in profile values plus or minus the sampling and measuring errors. As the physical distance between points is decreased by increasing the number of points, the change in profile values

between points is decreased and the differences between contiguous points approach the variability due to errors in sampling and measurement. In a frequency distribution analysis of 18 moisture stress profiles, 50 per cent of the differences in moisture contents of filter papers from contiguous samples were between −1 and +1 per cent. From this we concluded that in the moisture stress range above one bar the variability in results due to the method is probably less than 2 per cent of the moisture content value.

At moisture contents below field capacity (stresses greater than 0.2 bars) moisture is held on the surfaces of particles by electrostatic and molecular adhesion forces and disturbance of the sample has little or no effect on the stress measurement. At moisture contents above field capacity (stress less than 0.2 bars) the additional moisture is held within the soil pores and disturbance during sampling or transport alters the shape of the pores and their moisture holding capacity. The result is a change in the stress level in the sample and an increase in the total variability of measurements.

Variability at low stresses can be limited by obtaining relatively undisturbed samples and limiting the handling shocks they receive while being transported to the laboratory.

Accuracy

The accuracy of a measuring system should be defined in terms of primary standards. Unfortunately, there are no primary standards available for measurements of moisture stress in soils. Marshall (6) said: "The tensiometer is the only tool available for measuring suction directly in the field." But the tensiometer could not be used as an absolute primary standard because its range is limited and it is influenced by temperature gradients within the instrument.

It is possible to increase the accuracy of a measuring system over that of a calibrating standard by selecting and averaging a mass of data. Also, confidence in the accuracy of a calibration is increased if it is obtained with several independent calibrating methods. Both of these concepts were used to increase the accuracy of calibration of the filter paper method. Figure 1 shows data obtained using six different calibration methods. Five of these

methods are corroborative. The only deviation is the centrifuge method used by Gardner and he recognized that it could be in error.

Use of method

This filter paper method has been used in several projects with different objectives. A brief description of a few of them will illustrate its versatility and accuracy.

In a study of hydrologic effects of water spreading in Box Creek Basin, Wyoming by Hadley and McQueen (1961)[1] it was used to estimate infiltration of water during flood flows. These estimates compared favorably with inflow-outflow data on two floods and on a third flood the estimate was used because inflow-outflow records were unobtainable.

In a study of plant communities and soil moisture relationships near Denver, Colorado (Branson, Miller, and McQueen, 1965)[2] an extremely stony soil introduced variability in moisture data that prevented rational interpretation until it was compared with moisture stress data obtained with filter papers.

The ultimate soil moisture stress that a given plant community can induce is being determined for several rangeland plant species in eastern Montana. Preliminary results indicate that this ultimate stress or what the authors prefer to call "moisture stress competence" for western wheatgrass (*Agropyron smithii*) is 32 bars. This compares favorably with published values for maximum stress levels for grasses (Perrier *et al.*, 1961).

The Moisture Stress Competence for big sagebrush (*Artemisia tridentata*) is about 40 bars; for greasewood (*Sarcobatus vermiculatus*) is about 55 bars; and saltbush (*Atriplex nuttallii*) is about 60 to 65 bars.

Moisture stress gradients are being measured to help define hydrologic processes in several projects. The effects of radiation or shading on movement of soil moisture is being studied in Arizona. The movement of water in the un-

[1] Hadley, R. F., and McQueen, I. S. 1961. Hydrologic effects of water spreading in Box Creek basin, Wyoming. U.S. Geol. Survey, Water-Supply Paper 1532-A.

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saturated zone above a shallow watertable is being studied in Arizona and in Colorado.

Filter papers have been used to define the effectiveness of land treatment practices in conserving and utilizing soil moisture. They are now being used to calibrate experimental devices for recording moisture stress in soils.

CONCLUSIONS

Filter paper used as an indirect gravimetric moisture stress sensor makes it possible to obtain a moisture stress value for each gravimetric soil moisture sample.

The method is versatile, accurate, convenient, and economical. It is effective over the entire stress range from 0.001 bars to 1,500 bars: from 1 to 1,500,000 centimeters of water (pF 0 to 6.2). It can and should be used by anyone conducting a gravimetric soil moisture content sampling program either as control for neutron moisture measurements or in lieu of them.

Although calibration of the filter paper required six different methods it is considered accurate enough to be used as a calibration standard for systems being developed for in situ recording of soil moisture stress.

Moisture stress may be determined by this filter paper method with an accuracy that is comparable to or better than the accuracy of other methods with limited ranges.

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COLE TEST

The following are excerpts from the procedures used for COLE tests (Ref. 22).

BULK DENSITY

Density is defined as mass per unit volume. Soil density differs from most density in that the mass of the liquid phase is excluded. Also, the volume over which the weight is determined includes interparticle space. Because of these irregularities, soil density has been called bulk density, Db, to distinguish it from the more usual density that is based on intraparticle volume only. Furthermore, since the volume of a given mass of soil depnds on its water content, subscripts are added to designate the moisture condition when the measurement was made. Thus Db_m is the bulk density of a moist sample; $Db_{1/3}$ is the bulk density of a clod sample equilibrated at 1/3-bar tension; and Db_d is the bulk density of a dry sample.

Saran-Coated Clods

Reagents

Methyl ethyl ketone.

Dow Saran F310[*].—The Saran resin dissolves readily in acetone or methyl ethyl ketone. In this method methyl ethyl ketone is used as a solvent because it is less soluble in water than is acetone and there is less penetration of the Saran-solvent solution into a moist clod. Saran-solvent ratios of 1:4 to 1:8 are used, depending on the porosity of the soil to be coated.

To mix the plastic solution, fill a weighed container with solvent to about three-fourths its volume. From the weight of the solvent, calculate the resin required to obtain a predetermined resin-solvent ratio and add to the solvent. Since the solvent is flammable and its vapors form explosive mixtures with air, mix the plastic with an air-powered or non-sparking electric stirrer under an exhaust hood.[1] If a high-speed stirrer is used, the resin dissolves in about 1 hour. Metal cans (1 gal) are satisfactory containers for mixing and storing the plastic. Keep the containers tightly closed to prevent evaporation of the solvent.

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[*] Registered Trademark Dow Chemical Co.

[1] Information on the safe handling and use of methyl ethyl ketone is available in Chemical Safety Data Sheet SD-83, Manufacturing Chemists' Association, Inc., 1825 Connecticut Avenue NW., Washington, D.C.

Procedure

Collect natural clods of about 50 to 200 cc. Chip a piece of soil larger than the clod from the face of a sampling pit with a spade. From this piece remove a clod by gently cutting or breaking off protruding peaks and material sheared by the spade. If roots are present, they can be cut conveniently with scissors. In some soils, clods can be removed directly from the face of a pit with a knife or spatula. No procedure for taking clod samples fits all soils; the procedure must be adjusted to meet the conditions in the field at the time of sampling.

Hold the separated clod by a thread or fine wire and immerse it briefly in the plastic solution. For convenience, either of two concentrations of plastic solution is usually used—a 1:7 solution for the majority of soil samples or a 1:4 solution for clods that have large pores. Then suspend the immersed clod from a line to allow the coating to dry, usually 15 to 30 minutes.[2] If bulk density at field-moisture content is desired, store the clods in waterproof plastic bags as soon as the coating dries since the coating is permeable to water vapor. Although the coating keeps the clods intact, they may be crushed in transport unless they are packed in rigid containers.

In the laboratory apply additional coatings of plastic to make the clod waterproof and to prevent its disruption during wetting. Then weigh the clod, either in its natural moisture condition or in an adjusted moisture condition (e.g., 1/3-bar tension) in air and in water to obtain its volume by Archimedes' principle. Subsequent changes in moisture condition and volume of the soil sample can be followed by reweighing the coated clod in air and in water. Finally, weigh the ovendry clod in air and in water.

Be careful not to lose any soil material because the weight of material lost is calculated as soil moisture, and calculated bulk densities depend on the final ovendry weight of the clod.

Bulk-density values determined by this method are reported on the basis of the fine-earth fabric. Determinations are made on clod samples that may contain particles larger than 2 mm; but after the measurement is made, the weight and volume of the coarse fraction are subtracted. The remainder consists of the

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[2] Clods coated in this way can be transported to the laboratory and examined macroscopically in an undisturbed state.

weight of the <2-mm material and the volume of these fine-earth particles and the pore space associated with them.

Sometimes it is necessary to correct for weight and volume of the plastic coating. The coating has a density of about 1.3 g per cubic centimeter and it loses 10 to 20 percent of its airdry weight on ovendrying at 105° C. Thus, the amount of correction becomes smaller as bulk density of the soil approaches density of the coating and as moisture content of the soil approaches the weight loss of the coating.

Calculations

The example given is for a clod equilibrated at 1/3-bar tension.

$$Db_{1/3} = \frac{\text{wt clod}_{od} -- \text{wt} > 2 \text{ mm} -- \text{wt coat}_{od}}{\text{vol clod}_{1/3} -- \text{vol} > 2 \text{ mm} -- \text{vol coat}}$$

$$Db_{od} = \frac{\text{wt clod}_{od} -- \text{wt} > 2 \text{ mm} -- \text{wt coat}_{od}}{\text{vol clod}_{od} -- \text{vol} > 2 \text{ mm} -- \text{vol coat}}$$

$$W_{1/3} = \frac{\text{wt clod}_{1/3} -- \text{wt clod}_{od} -- (\text{wt coat}_{ad} -- \text{wt coat}_{od})}{\text{wt clod}_{od} -- \text{wt} > 2 \text{ mm} -- \text{wt coat}_{od}}$$

where

$Db_{1/3}$ is bulk density of <2-mm fabric at 1/3-bar tension in grams per cubic centimeter

Db_{od} is bulk density of <2-mm fabric at ovendryness in grams per cubic centimeter

$W_{1/3}$ is water content of fine earth at 1/3-bar tension as weight percentage

wt clod$_{od}$ is weight of ovendry coated clod

wt clod$_{1/3}$ is weight of coated clod equilibrated at 1/3-bar tension

vol clod$_{od}$ is volume of ovendry coated clod

vol clod$_{1/3}$ is volume of coated clod equilibrated at 1/3-bar tension

vol > 2 mm is volume of material > 2 mm separated from clod after ovendrying

wt > 2 mm is weight of material > 2 mm separated from clod after ovendrying

wt coat$_{ad}$ is weight of Saran coating before oven drying

wt coat$_{od}$ is weight of Saran coating after oven drying

vol coat is volume of Saran coating (estimated).

It is not always necessary to correct for the weight and volume of the Saran coating.

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Linear Extensibility (LE)

Linear extensibility is a measure of the change in clod dimension on going from a dry to a moist state. It has also been expressed as COLE (coefficient of linear extensibility). COLE = LE ÷ 100.

$$LE \text{ (pct.)} = 100 \left[\frac{L_m - L_d}{L_d} \right]$$

where

L_d = length of clod, dry

L_m = length of same clod, moist

Airdry or Ovendry to 30 cm, 1/3-Bar or 1/10-Bar Tension

Linear extensibility can be estimated from laboratory bulk-density data and the coarse-fragment conversion factor (Cm).

$$LE \text{ (pct.)} = 100 \left[\left(\frac{1}{Cm \frac{Db_m}{Db_d} + (1 - Cm)} \right)^{1/3} - 1 \right]$$

where

$$Cm = \frac{Vol \text{ moist} < 2\text{-mm fabric}}{Vol \text{ whole soil}}$$

Db_m = bulk density of the fine-earth fabric at 30 cm, 1/3 bar, or 1/10 bar

Db_d = bulk density of the fine-earth fabric at oven- or air-dryness

If there is no coarse material, Cm = 1 and the equation reduces to

$$LE \text{ (pct.)} = 100 \left[\left(\frac{Db_d}{Db_m} \right)^{1/3} - 1 \right]$$

LE calculated for the fine-earth fabric alone can be referred to as LE_f (or $COLE_f$).

CLOD TESTS

These tests consisted of obtaining clods of the natural soil (about 10). Several were dried in the laboratory for varying periods. Small quantities of water were added to several of the clods. A filter paper was placed in the moisture can which was then sealed and placed in an insulated chest for seven days. At the end of seven days, the filter paper was removed and weighed to determine suction. The clods were coated with plastic and weighed in air and water to

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determine bulk density. They were then oven dried, and the bulk density again determined. Data obtained included water content, suction, density and change of density on oven drying.

APPENDIX B
SOILS DATA

Soil samples were obtained from eight different sites in the United States. A variety of materials were intentionally included in the testing program. The sites are as follows:

Site Location	Symbol	Remarks
Ellsworth, Ks	ELL	Highway Site
Hennessy, Ok	HEN	Highway Site
Holbrook, Az	HOL	Highway Site
Irving, Tx	DFW	Airport Site
Kelly AFB, Tx	KAFB	Airport Site
Moquino, NM	SOH	Building Site
San Antonio, Tx	SAT	Highway Site
Tucumcari, NM	TUC	Highway Site

As shown in Figure B-1, most materials classified as CH or CL soils in the Unified System. On the basis of the USDA textual classification, a wider variety is evident in Figure B-2.

The following pages provide classification data, natural conditions, COLE, moisture-suction data and strain-suction data.
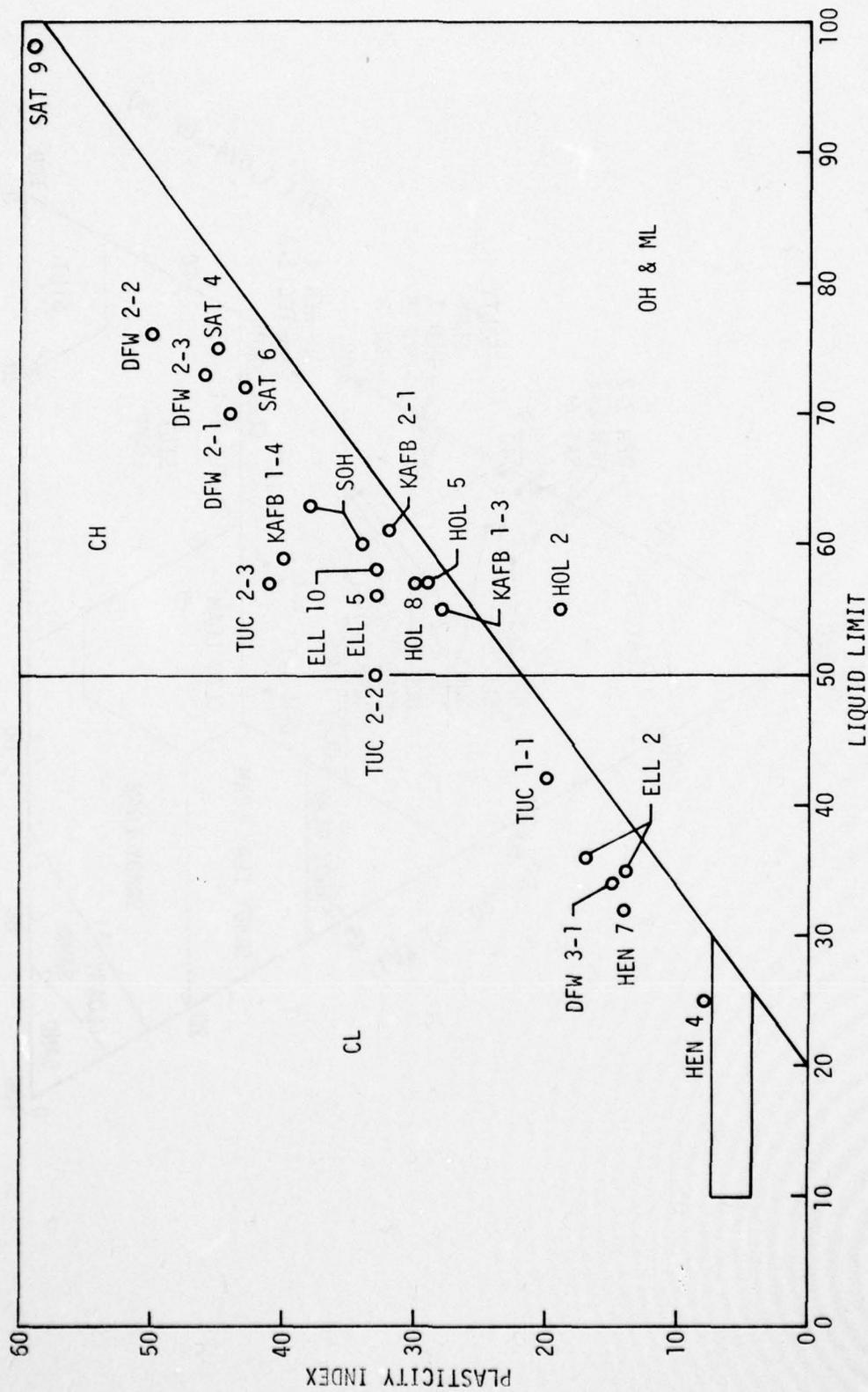
FIGURE B-1. SOILS IN CERF STUDY

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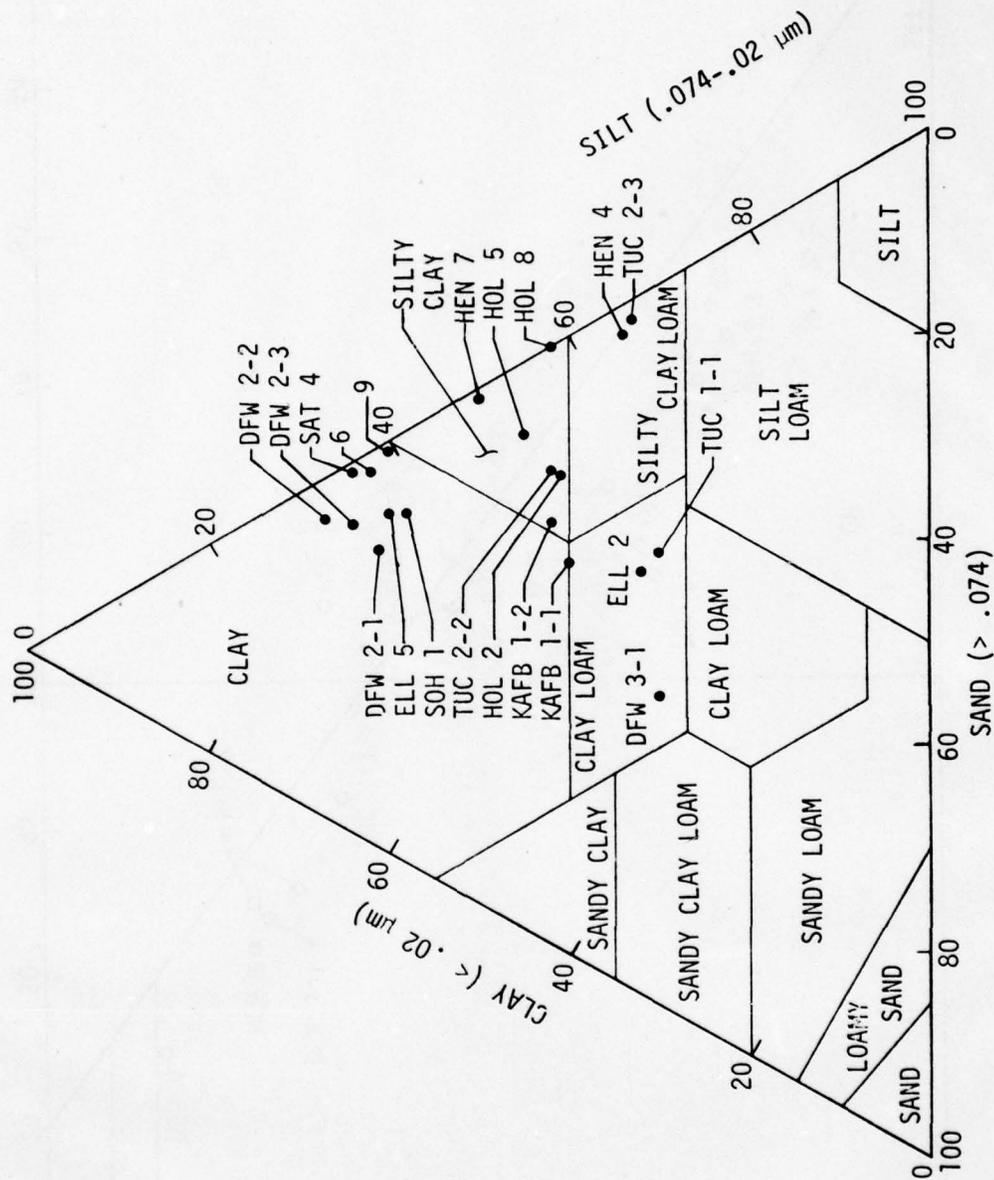
FIGURE B-2. TEXTUAL CLASSIFICATION OF SOILS

SITE Ellsworth, Kansas

US STANDARD SIEVES

WET MECHANICAL ANALYSIS

GRAIN SIZE IN MILLIMETERS

| SAMPLE NO. | DEPTH | NATURAL | | G_s | ATTERBERG LIM | | | CLAY <.002 | BAR L.S. | COLE | UNIFIED CLASSF. |
		ω	γ_d		LL	PI	SL				
—	FT	%	lb/ft³	—	%	%	%	%	%	—	—
2	3.6-5.9	15.3	104	2.65	36	16	18	32	7.8	.026	CL
5	8.1-10.3	19.9	104	2.61	56	33	11	60	13.6	.029	CH
10	15.1-17.3	14.8	105	2.61	58	33	15	45	12.5	.065	CH

FIGURE B-3. SOIL CLASSIFICATION DATA

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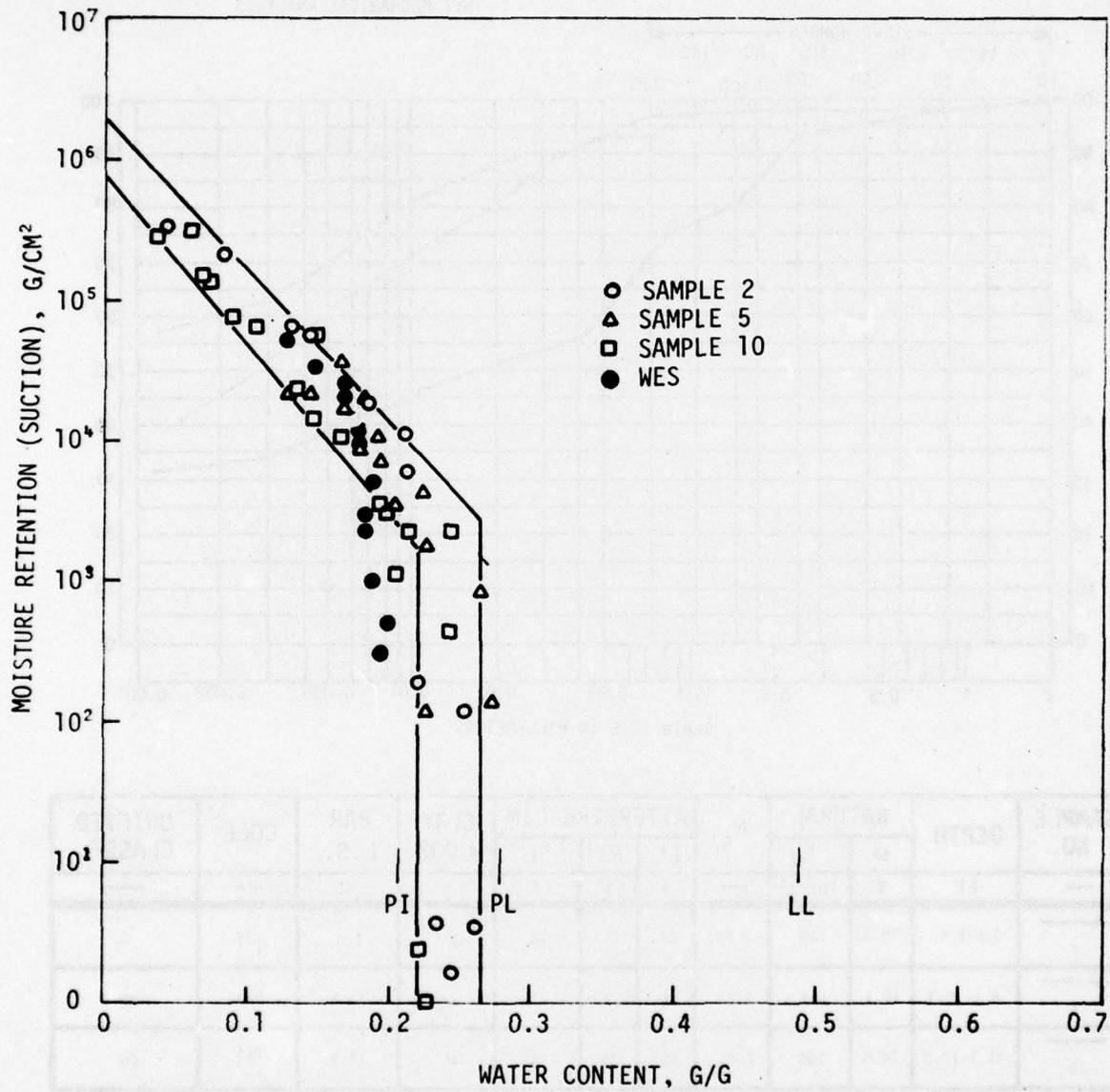
FIGURE B-4. ELLSWORTH, KANSAS MOISTURE DATA
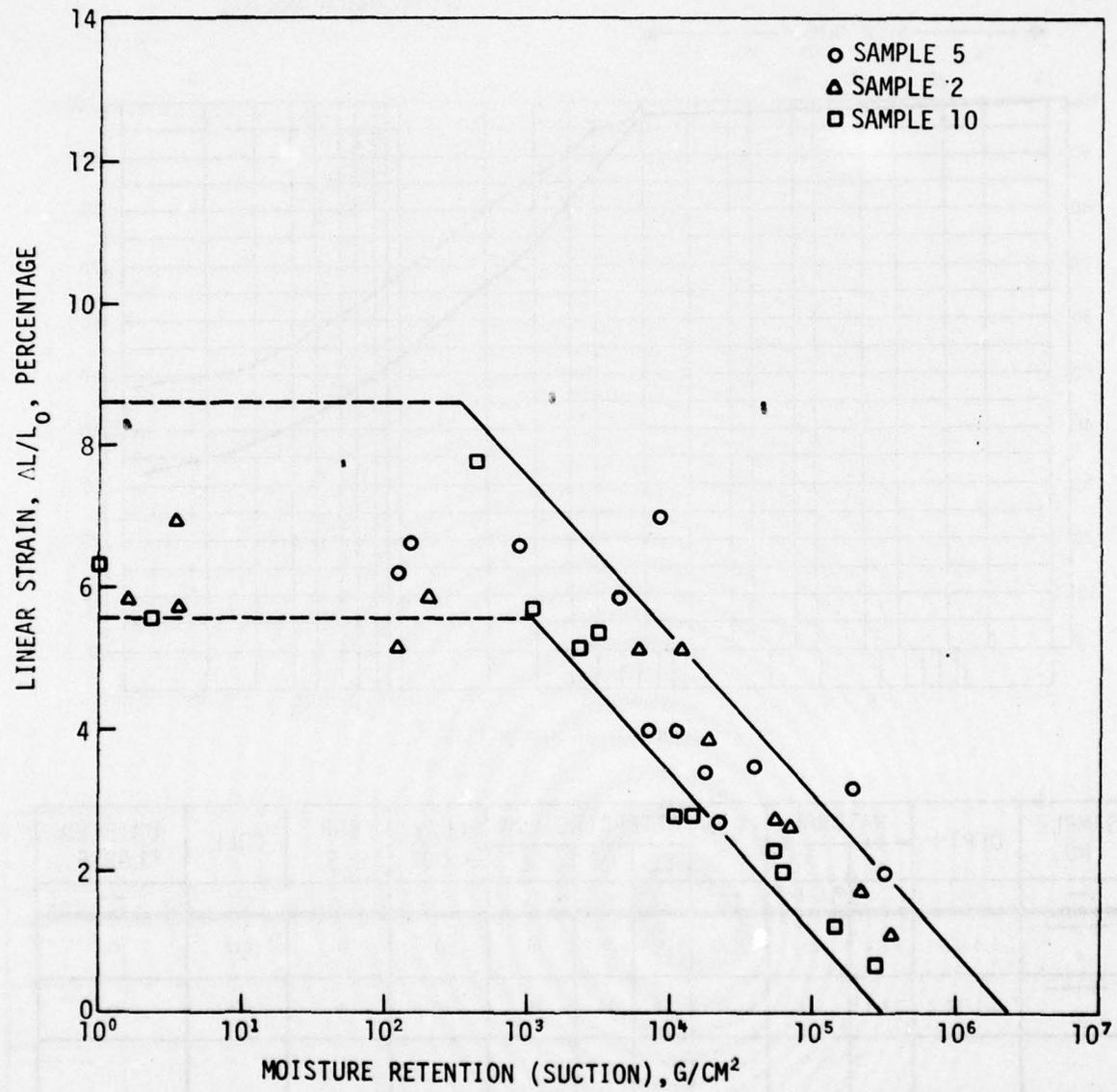
FIGURE B-5. ELLSWORTH, KANSAS STRAIN-SUCTION DATA

SITE Hennessy, Okla.
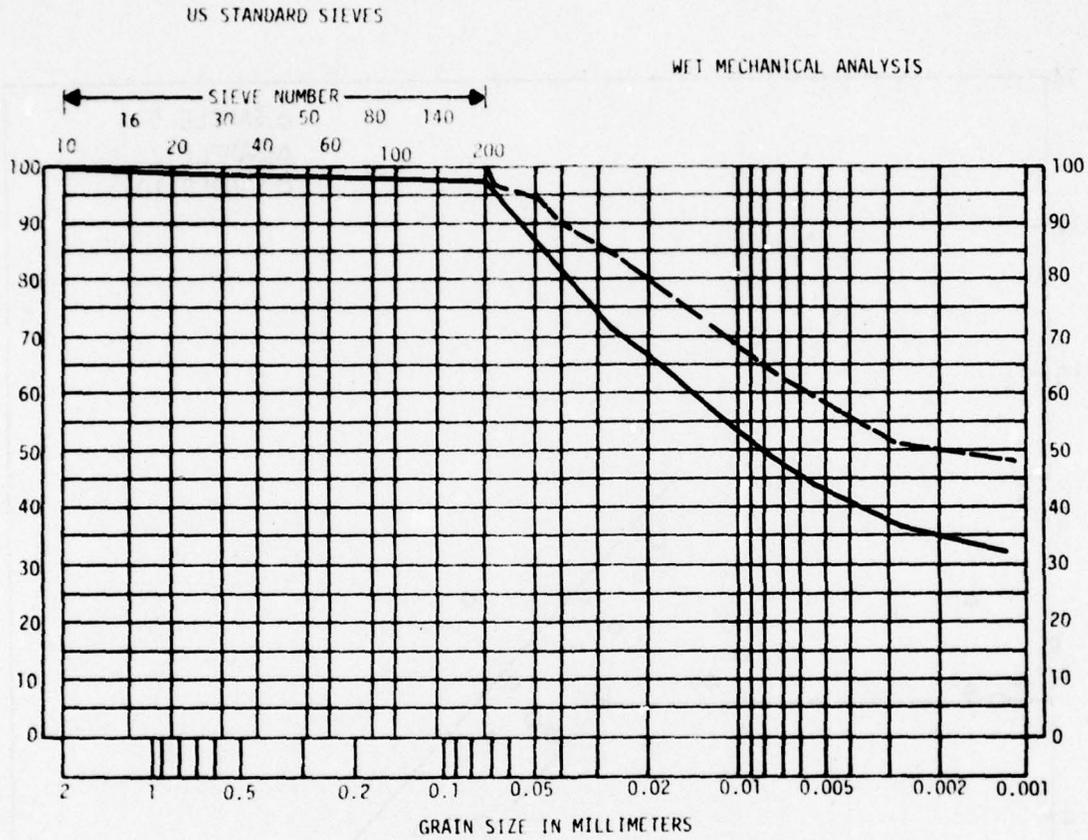
US STANDARD SIEVES

WET MECHANICAL ANALYSIS

GRAIN SIZE IN MILLIMETERS

SAMPLE NO.	DEPTH	NATURAL		G_s	ATTERBERG LIM			CLAY <.002	BAR L.S.	COLE	UNIFIED CLASSF.
		ω	γ_d		LL	PI	SL				
—	FT	%	lb/ft³	—						—	—
4	5.5-7.9	13.4	123	2.77	25	8	14	34	8	.012	CL
7	10.1-12.2	13.9	124	2.80	32	14	15	50	9	.009	CL

FIGURE B-6. SOIL CLASSIFICATION DATA

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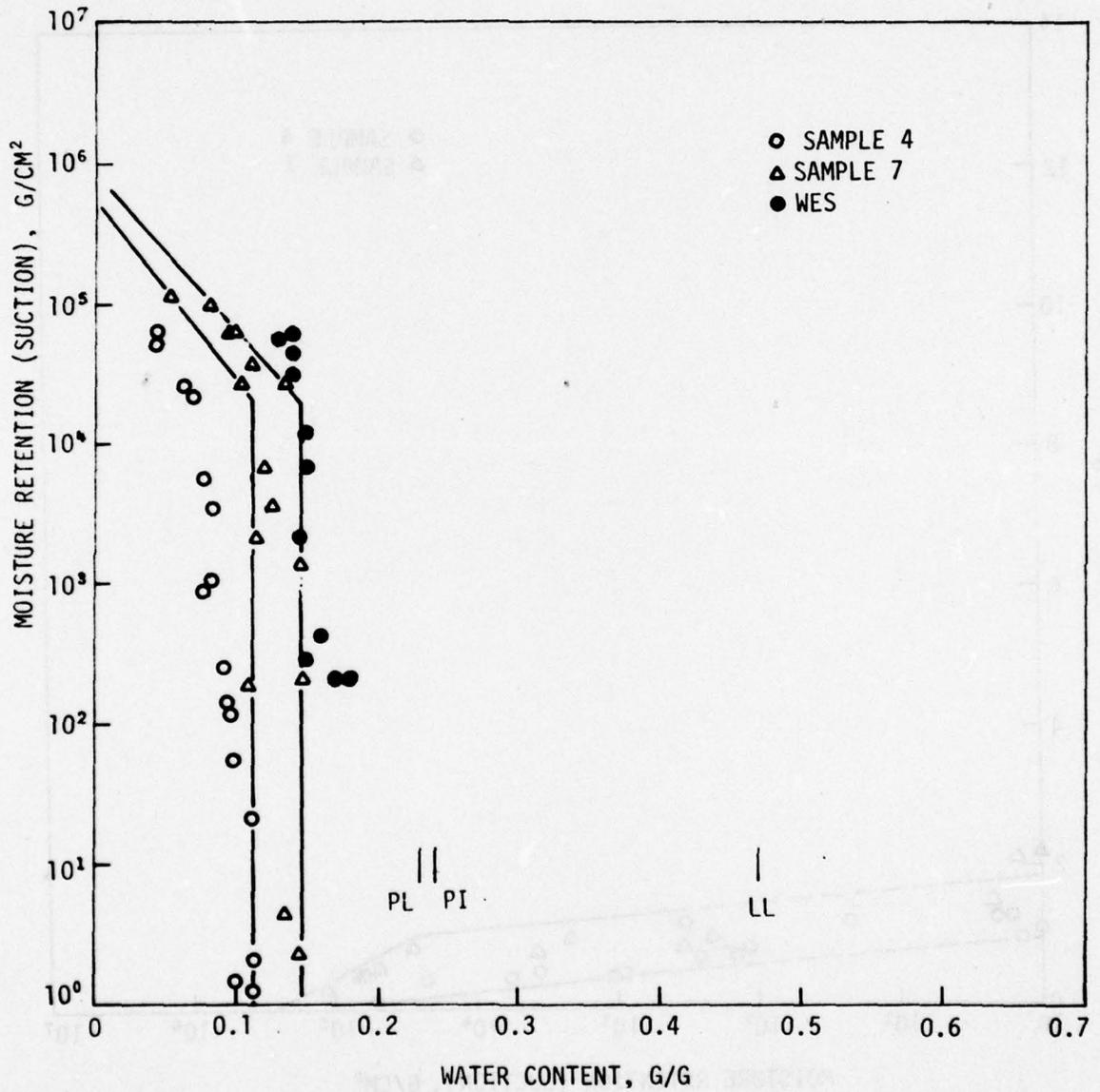
FIGURE B-7.   HENNESSY, OKLAHOMA MOISTURE DATA
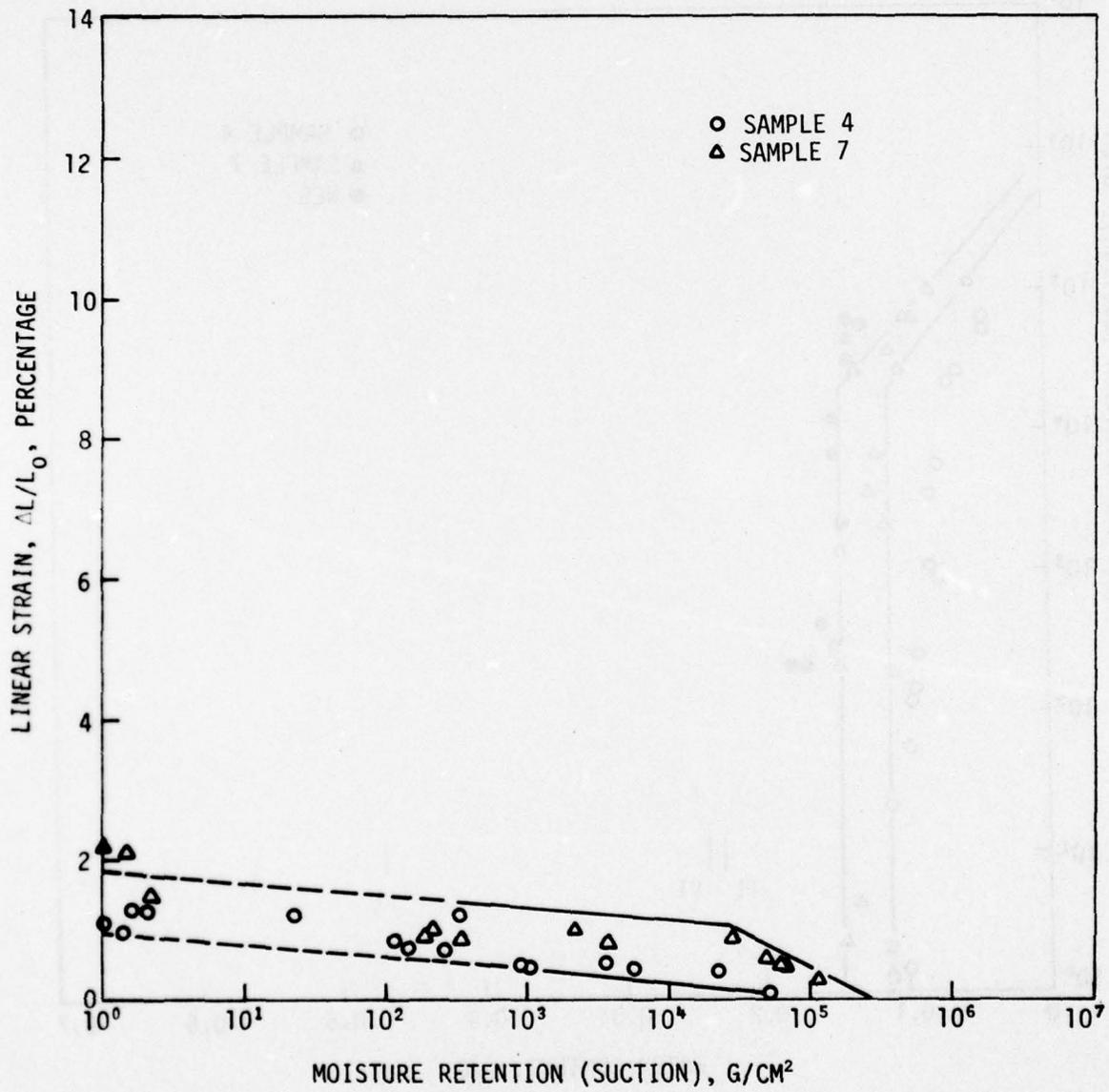
FIGURE B-8.   HENNESSY, OKLAHOMA STRAIN-SUCTION DATA

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SITE Holbrook, Ariz.
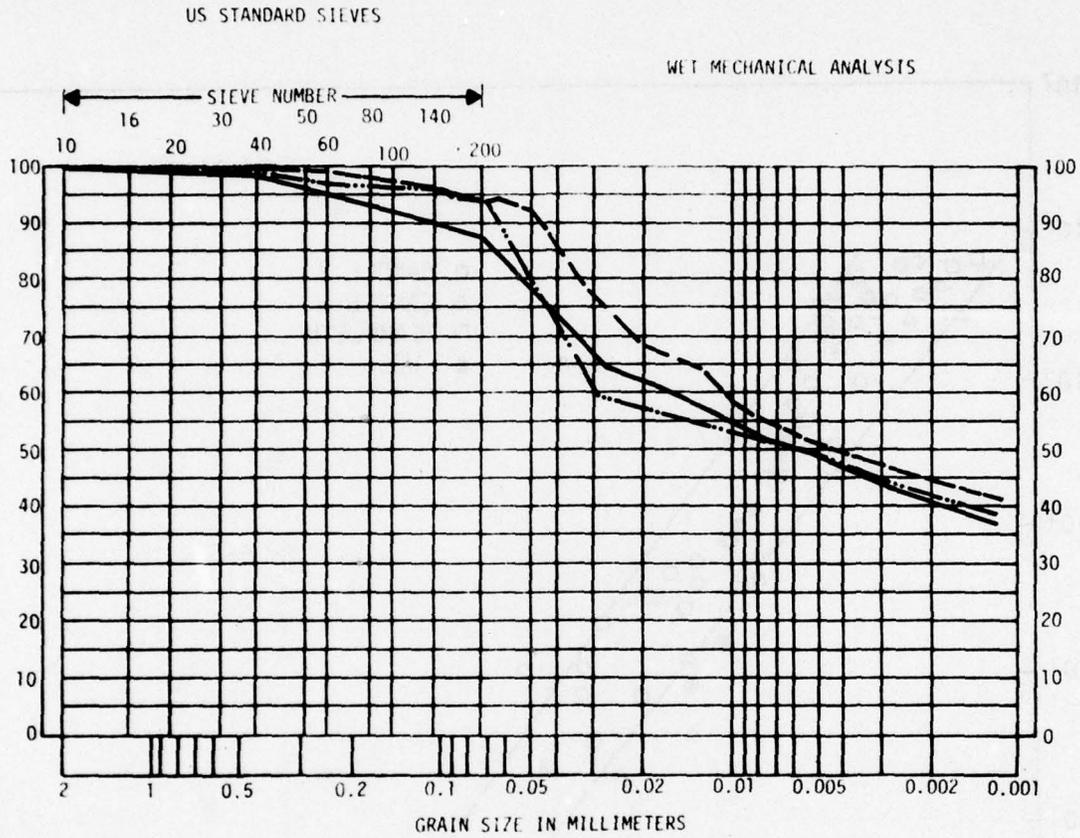
US STANDARD SIEVES

WET MECHANICAL ANALYSIS

GRAIN SIZE IN MILLIMETERS

SAMPLE NO.	DEPTH	NATURAL		G_s	ATTERBERG LIM			CLAY < .002	BAR L.S.	COLE	UNIFIED CLASSF.
		ω	γ_d		LL	PI	SL				
—	FT		lb/ft³	—						—	—
2	4.5-6.2	16.3	109	2.80	55	19	9	41*	20	.045	MH
5	8.4-10.6	12.3	118	2.81	57	29	9	45	20	.071	CH
8	14.9-16.8	17.2	104	2.79	57	30	12	42*	19	.076	CH

FIGURE B-9. SOIL CLASSIFICATION DATA

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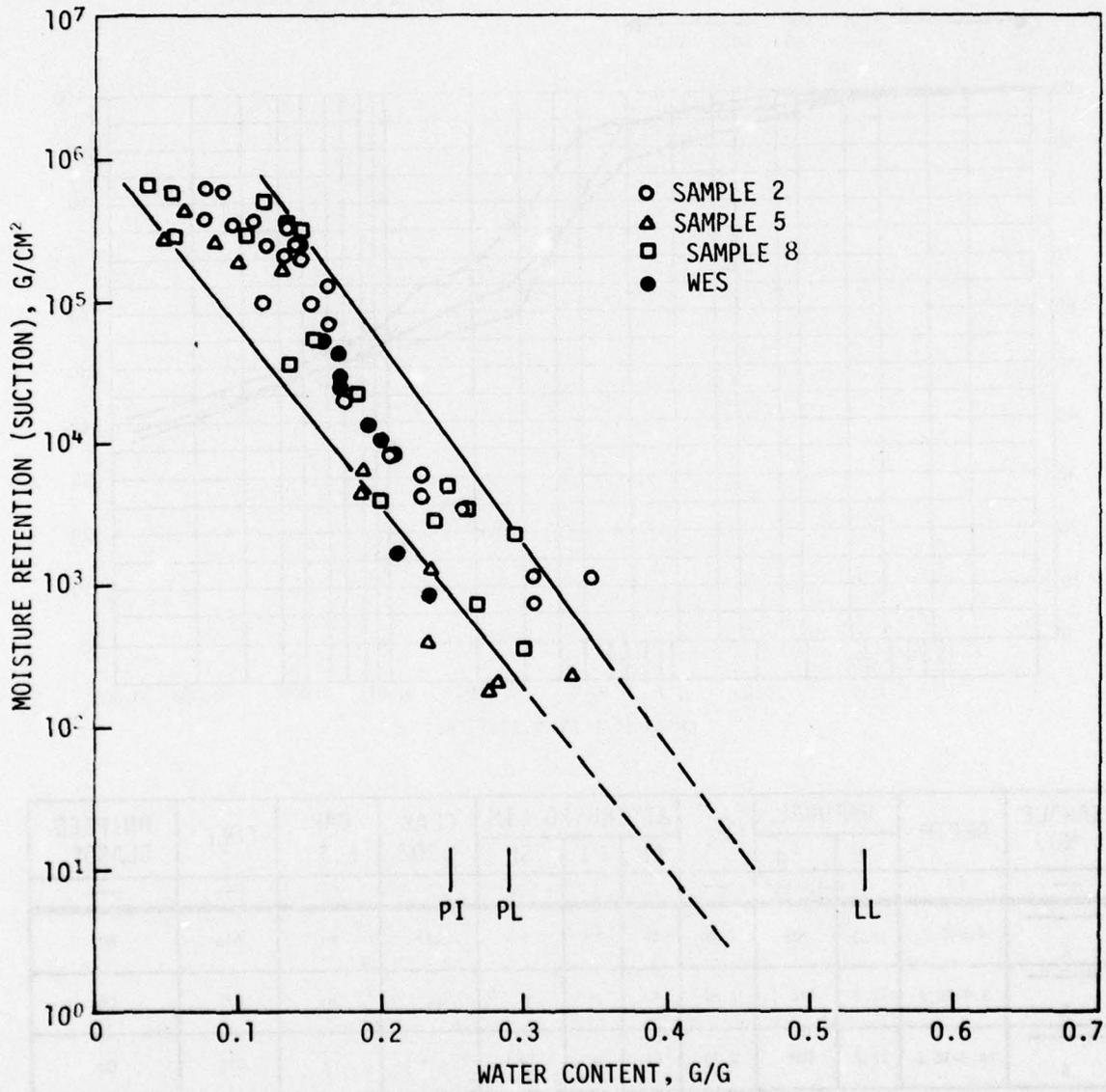
FIGURE B-10. HOLBROOK, ARIZONA MOISTURE DATA
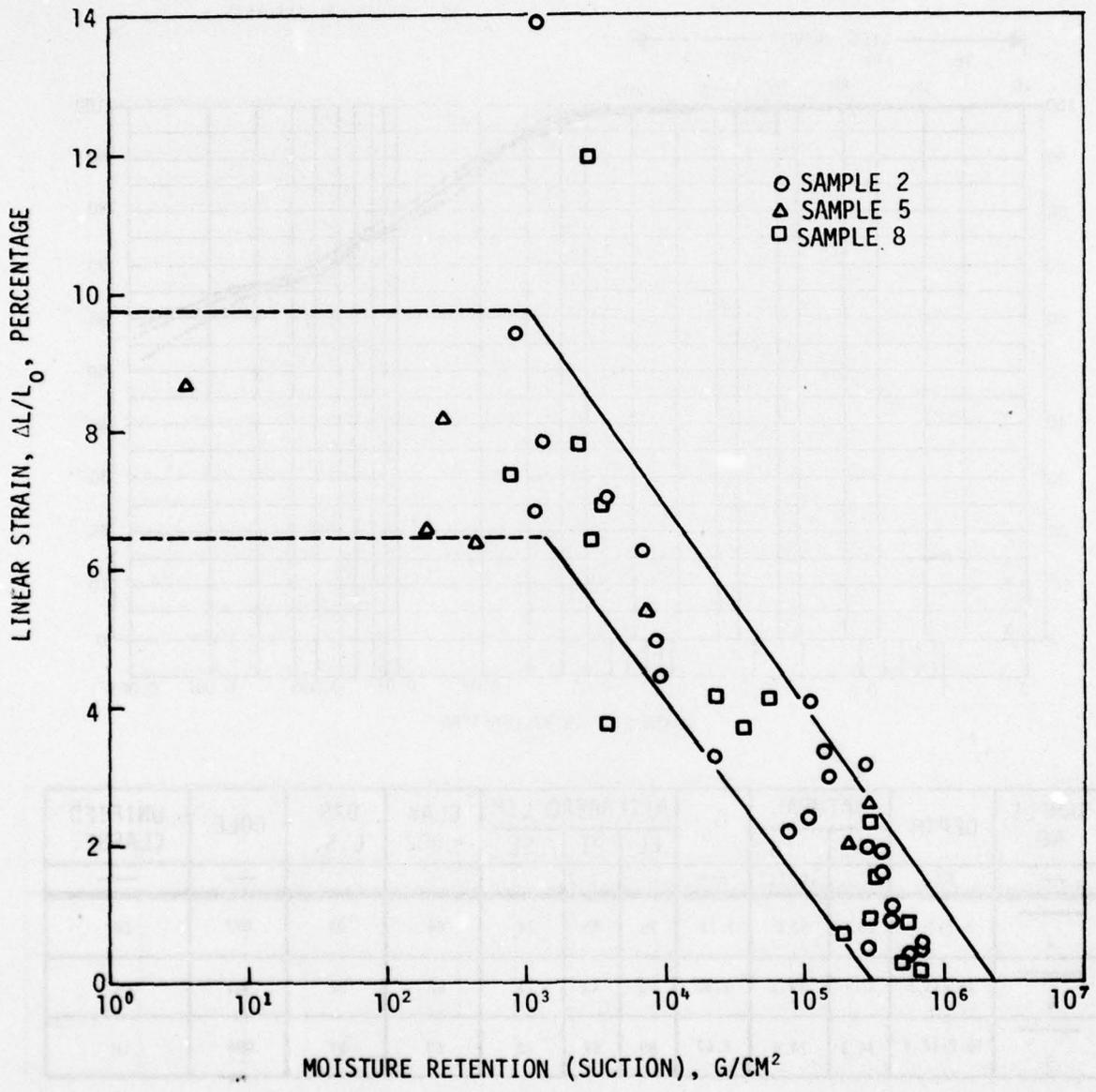
FIGURE B-11. HOLBROOK, ARIZONA STRAIN-SUCTION DATA

SITE San Antonio, Texas
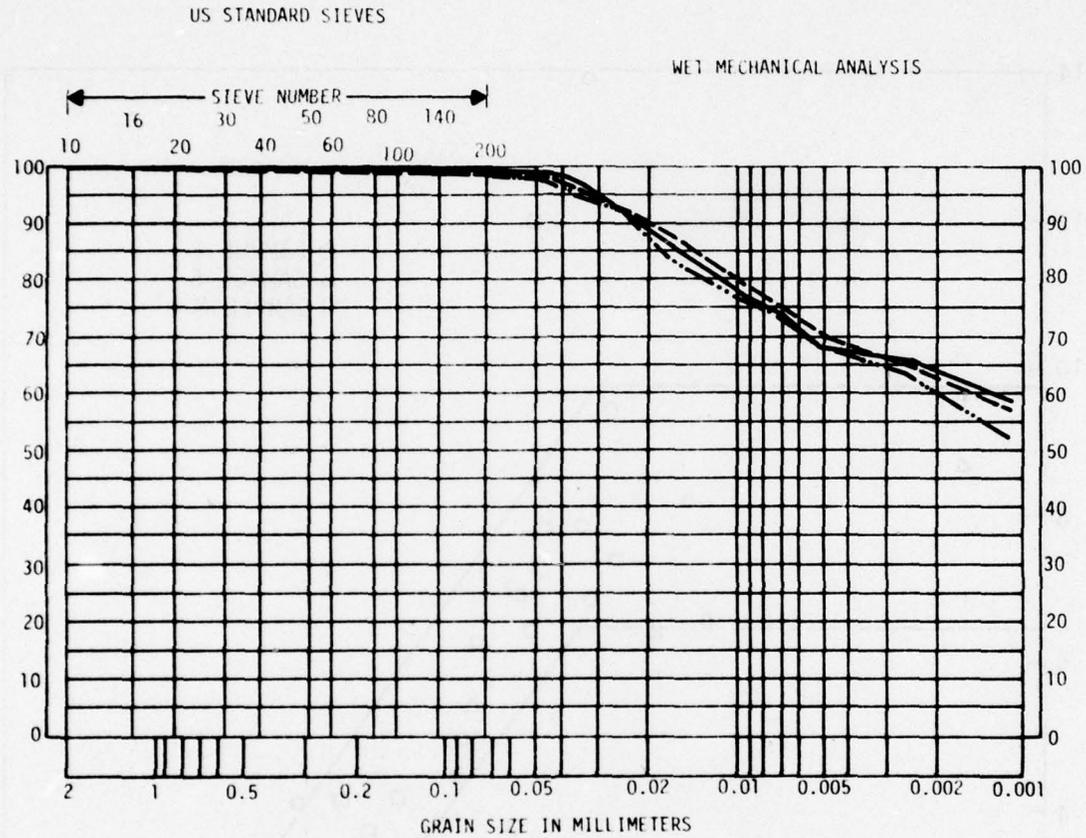
US STANDARD SIEVES

WET MECHANICAL ANALYSIS

GRAIN SIZE IN MILLIMETERS

SAMPLE NO.	DEPTH	NATURAL		G_s	ATTERBERG LIM			CLAY <.002	BAR L.S.	COLE	UNIFIED CLASSF.
		ω	γ_d		LL	PI	SL				
—	FT		lb/ft³	—						—	—
4	5.2-7.4	28.3	82.6	2.71	75	45	14	64	23	.077	CH
6	10.-11.6	32.7	79.2	2.50	72	43	13	62	22	.081	CH
9	16.2-17.3	34.3	74.6	2.67	99	59	12	60	21	.096	CH

FIGURE B-12.   SOIL CLASSIFICATION DATA
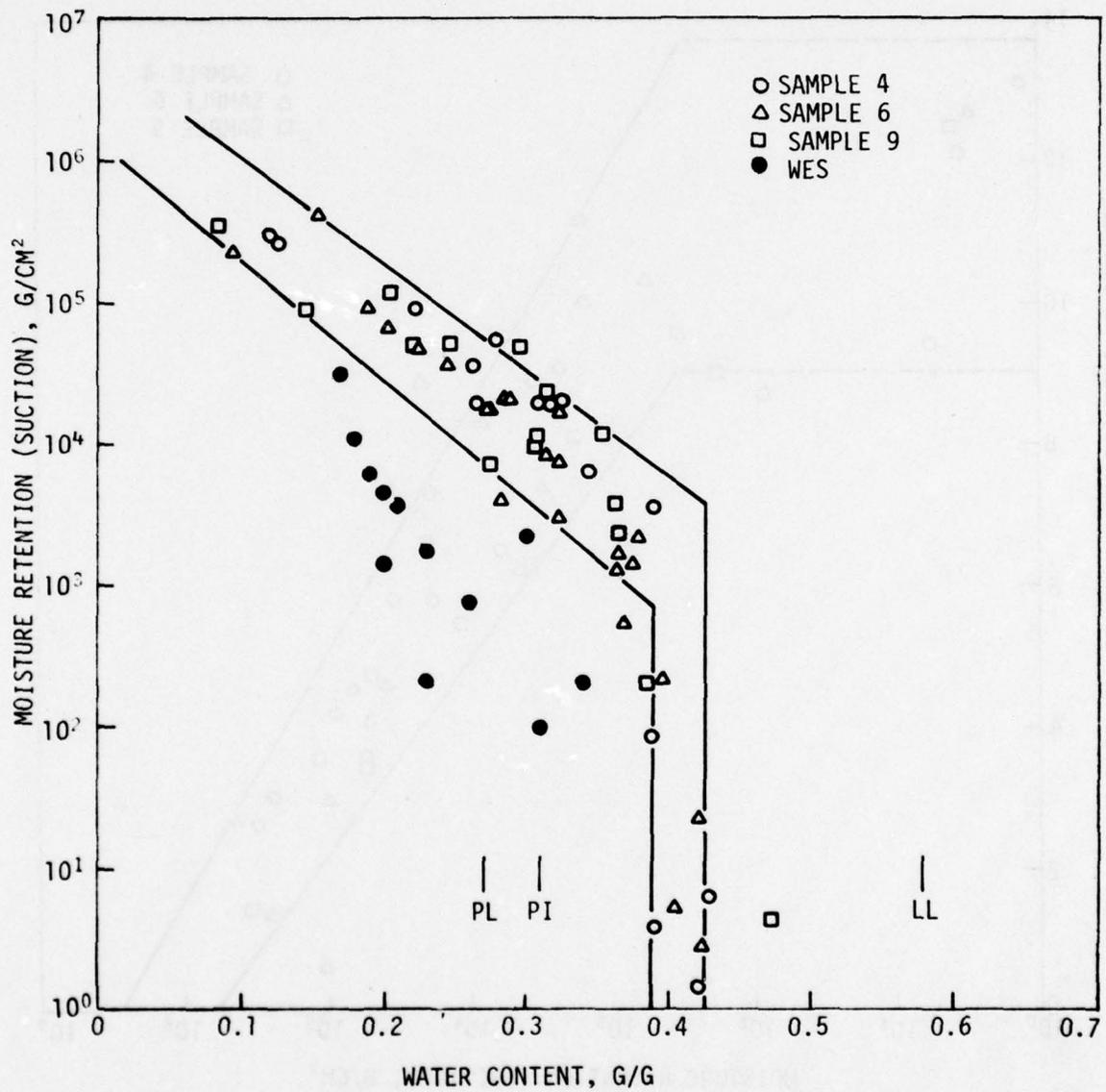
FIGURE B-13. SAN ANTONIO, TEXAS MOISTURE DATA
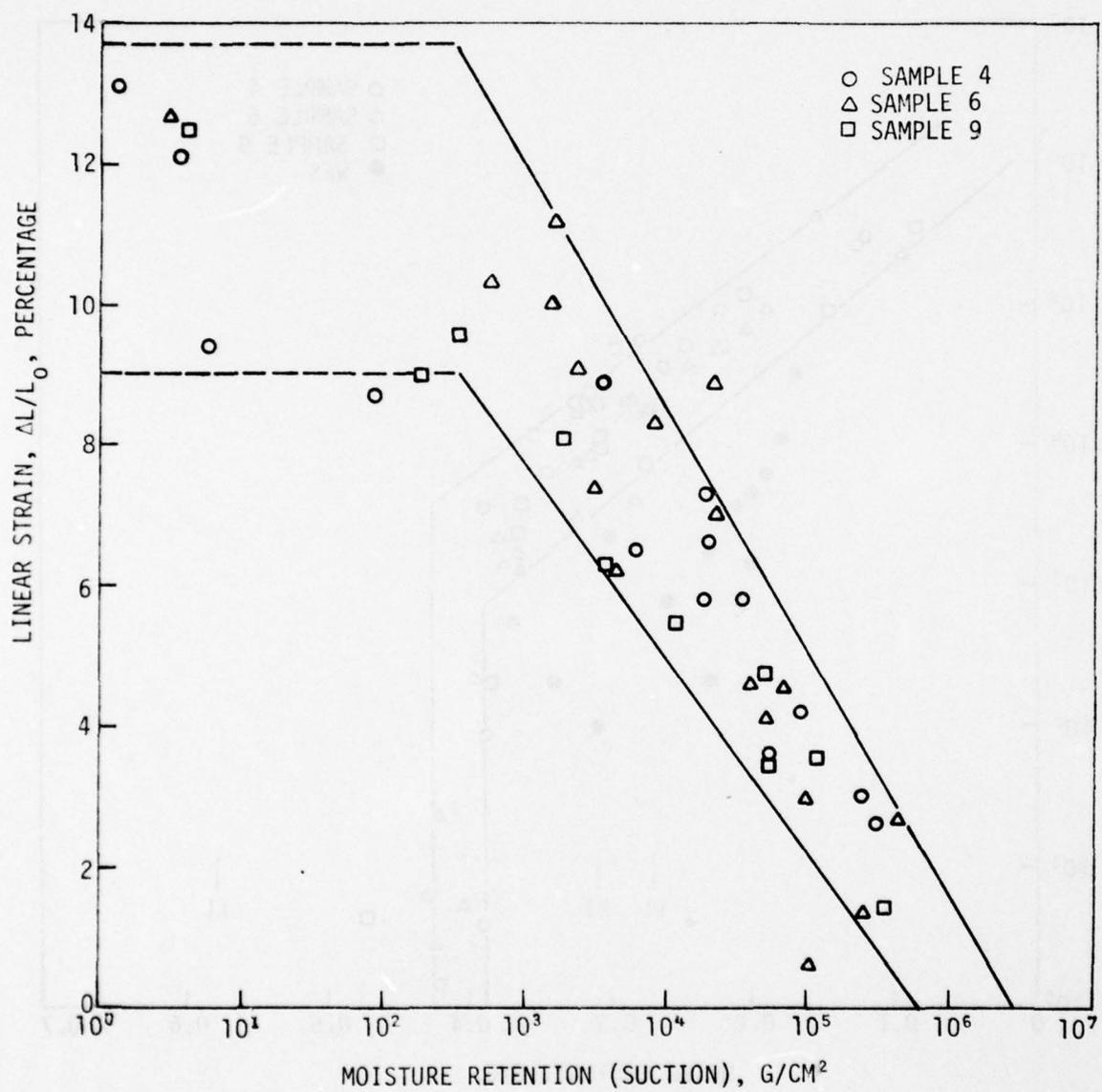
FIGURE B-14. SAN ANTONIO, TEXAS STRAIN-SUCTION DATA

US STANDARD SIEVES

WET MECHANICAL ANALYSIS

GRAIN SIZE IN MILLIMETERS

SAMPLE NO.	DEPTH	NATURAL		G_s	ATTERBERG LIM			CLAY <.002	BAR L.S.	COLE	UNIFIED CLASSF.
		ω	Y_d		LL	PI	SL				
—	FT		lb/ft³	—						—	—
2-1	.5-2.5	31	88	2.72	70	44	7	61	21	.109	CH
2-2	3.-4.	26	92	2.70	76	50	10	67	20	-	CH
2-3	6-10	31	90	2.78	73	46	14	64	19	.202	CH

FIGURE B-15. SOIL CLASSIFICATION DATA

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SITE Dallas/Ft Worth Airport, Texas   Site 3
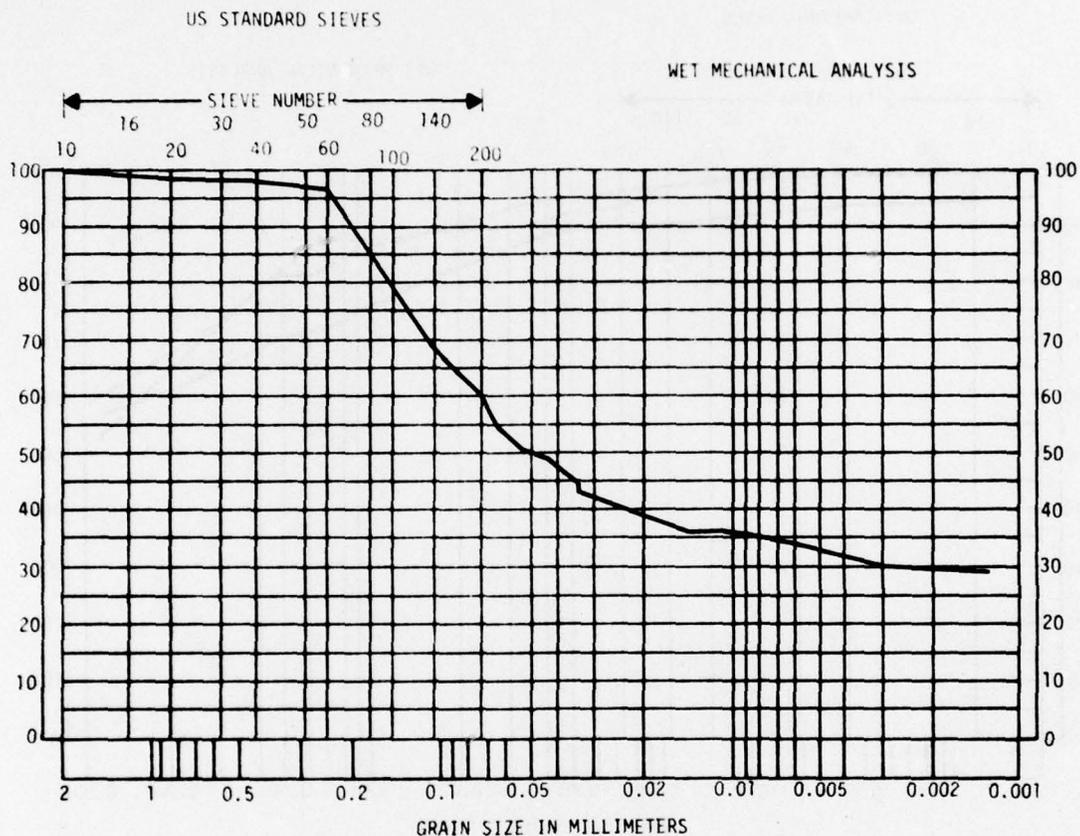
US STANDARD SIEVES

WET MECHANICAL ANALYSIS

GRAIN SIZE IN MILLIMETERS

SAMPLE NO.	DEPTH	NATURAL		G_s	ATTERBERG LIM			CLAY <.002	BAR L.S.	COLE	UNIFIED CLASSF.
		ω	γ_d		LL	PI	SL				
—	FT	%	lb/ft³	—	%	%	%	%	%	—	—
3-1	2-3	15	105	2.71	34	15	11	30	11	.025	CL

FIGURE B-16.   SOIL CLASSIFICATION DATA

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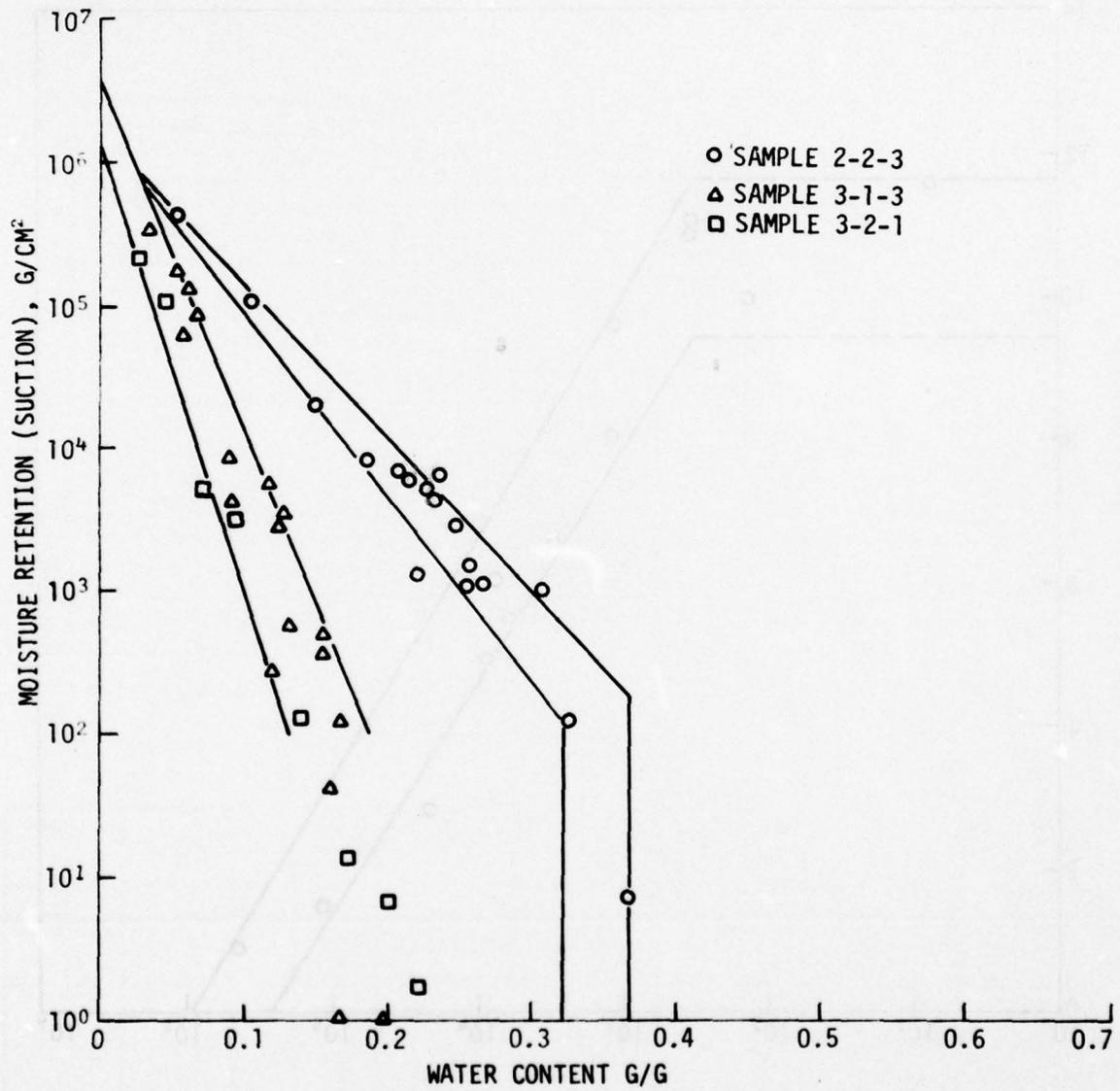
FIGURE B-17. DALLAS/FORT WORTH AIRPORT MOISTURE DATA
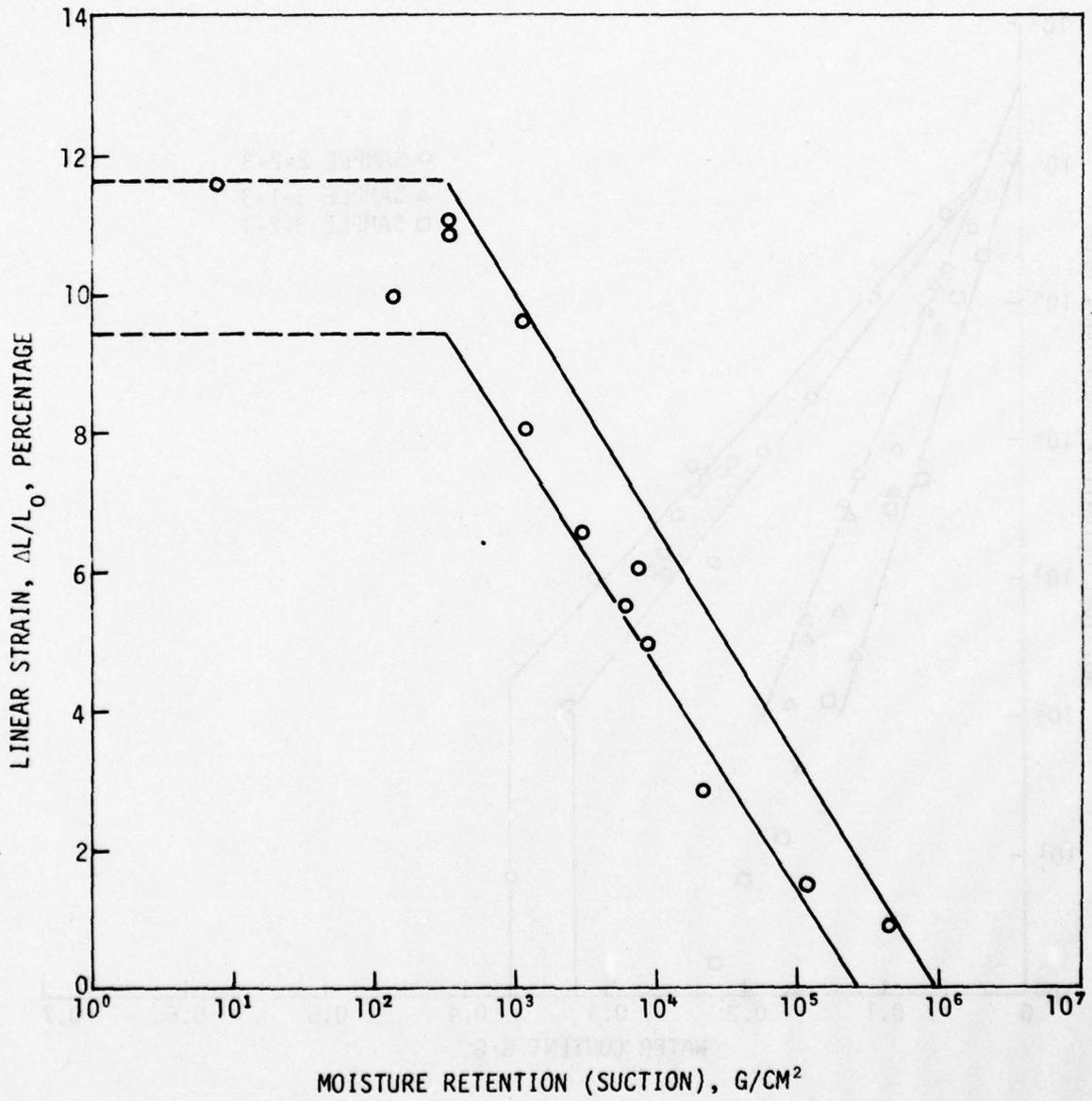
FIGURE B-18.   DALLAS/FORT WORTH AIRPORT STRAIN-SUCTION DATA, SAMPLE 2-2
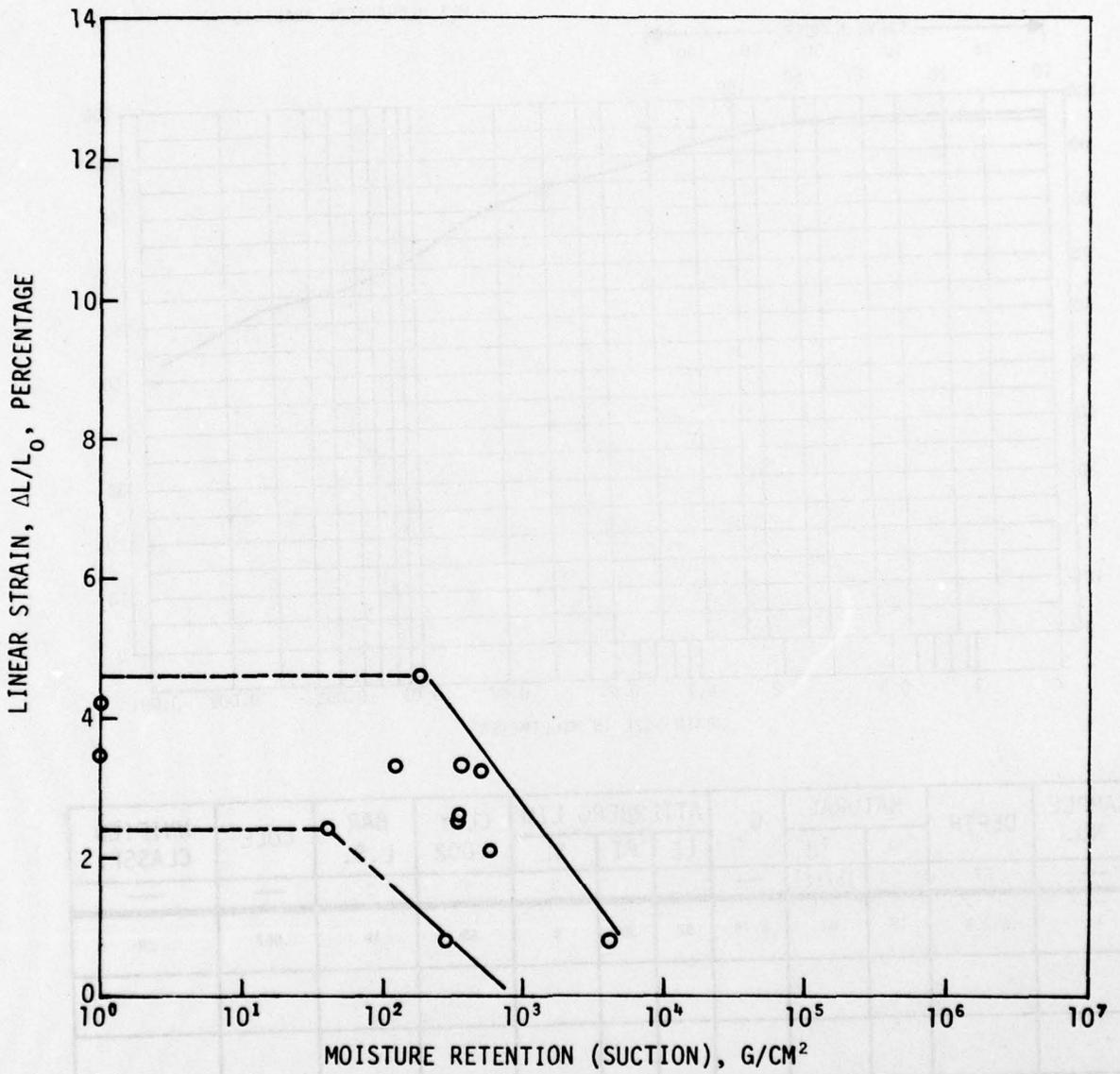
FIGURE B-19. DALLAS/FORT WORTH AIRPORT STRAIN-SUCTION DATA, SAMPLE 3-1

SITE Moquino, New Mexico
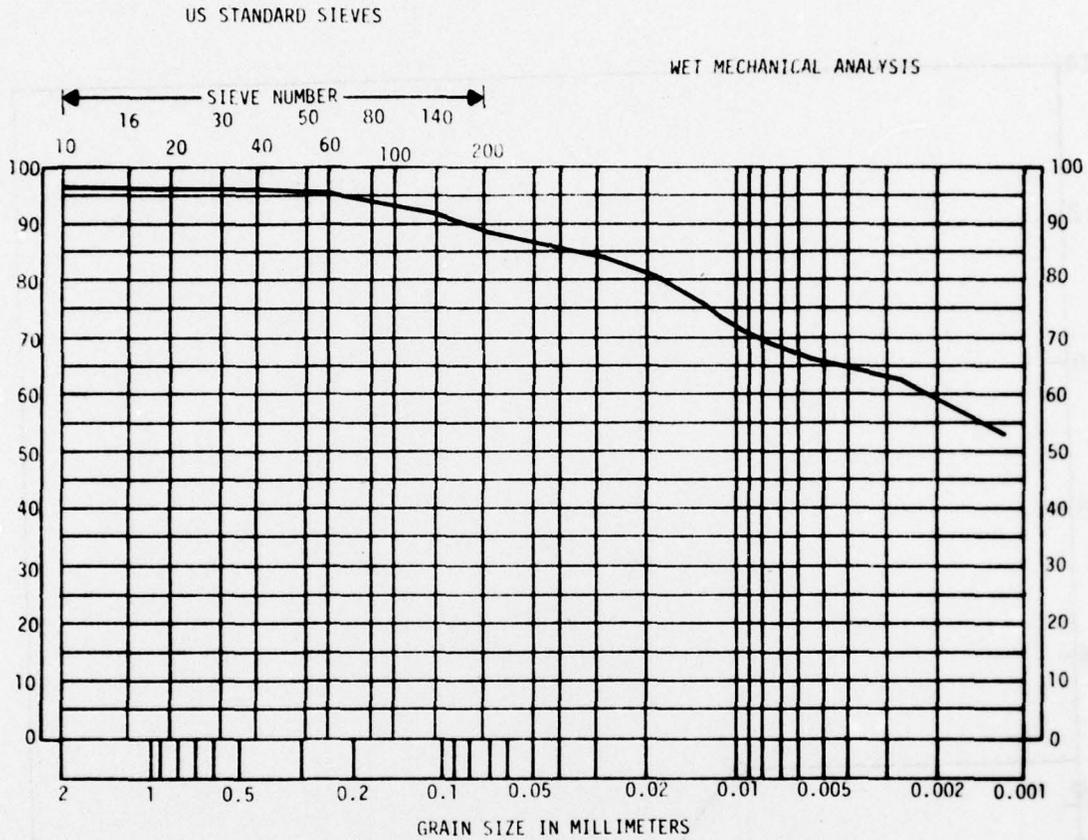
US STANDARD SIEVES

WET MECHANICAL ANALYSIS

GRAIN SIZE IN MILLIMETERS

SAMPLE NO.	DEPTH	NATURAL		G_s	ATTERBERG LIM			CLAY <.002	BAR L.S.	COLE	UNIFIED CLASSF.
		ω	Y_d		LL	PI	SL				
—	FT		lb/ft³	—						—	—
1	.5-2.5	19	87	2.74	62	36	9	58	16	.067	CH

FIGURE B-20. SOIL CLASSIFICATION DATA
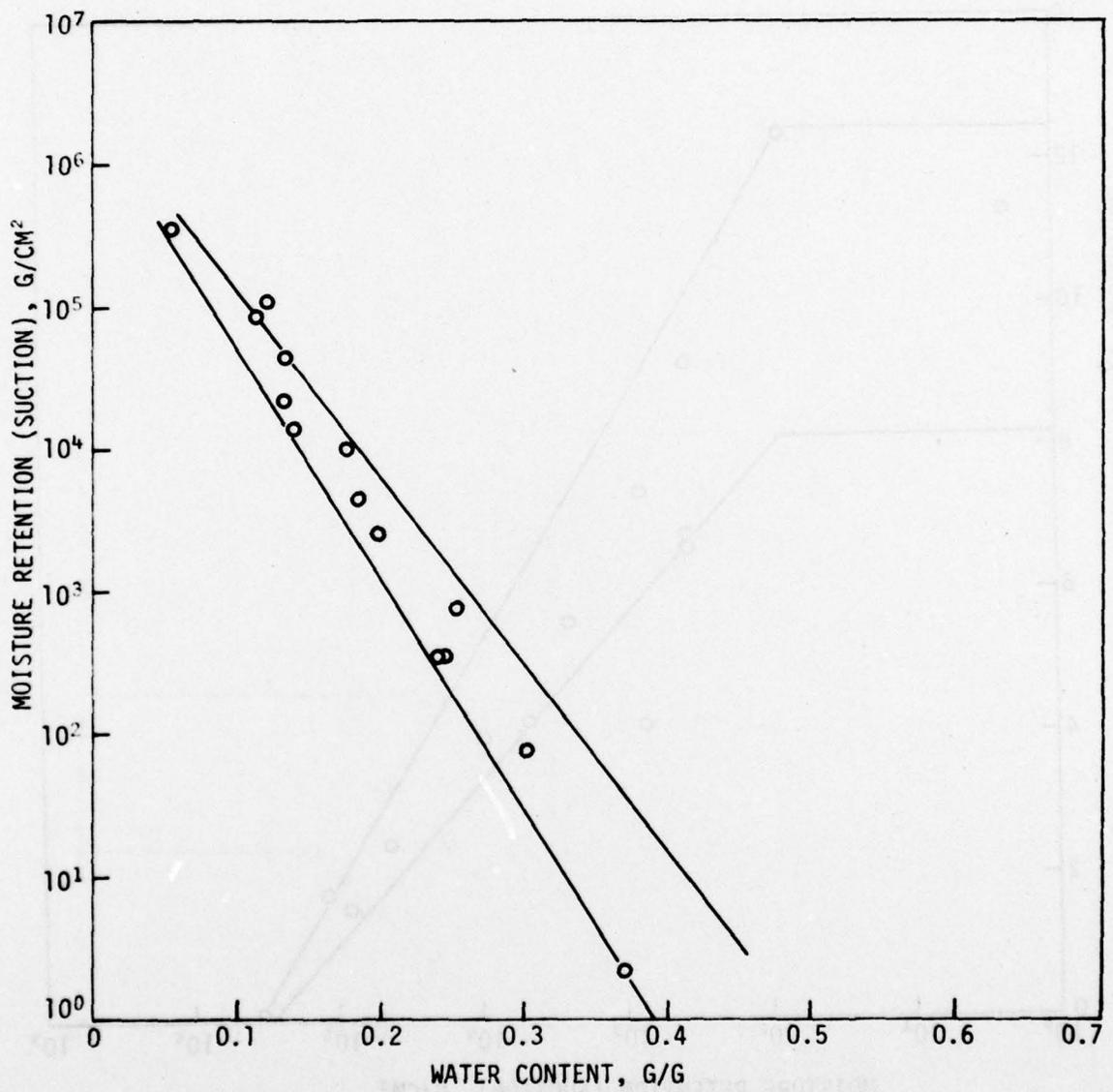
FIGURE B-21. MOQUINO, NEW MEXICO MOISTURE DATA
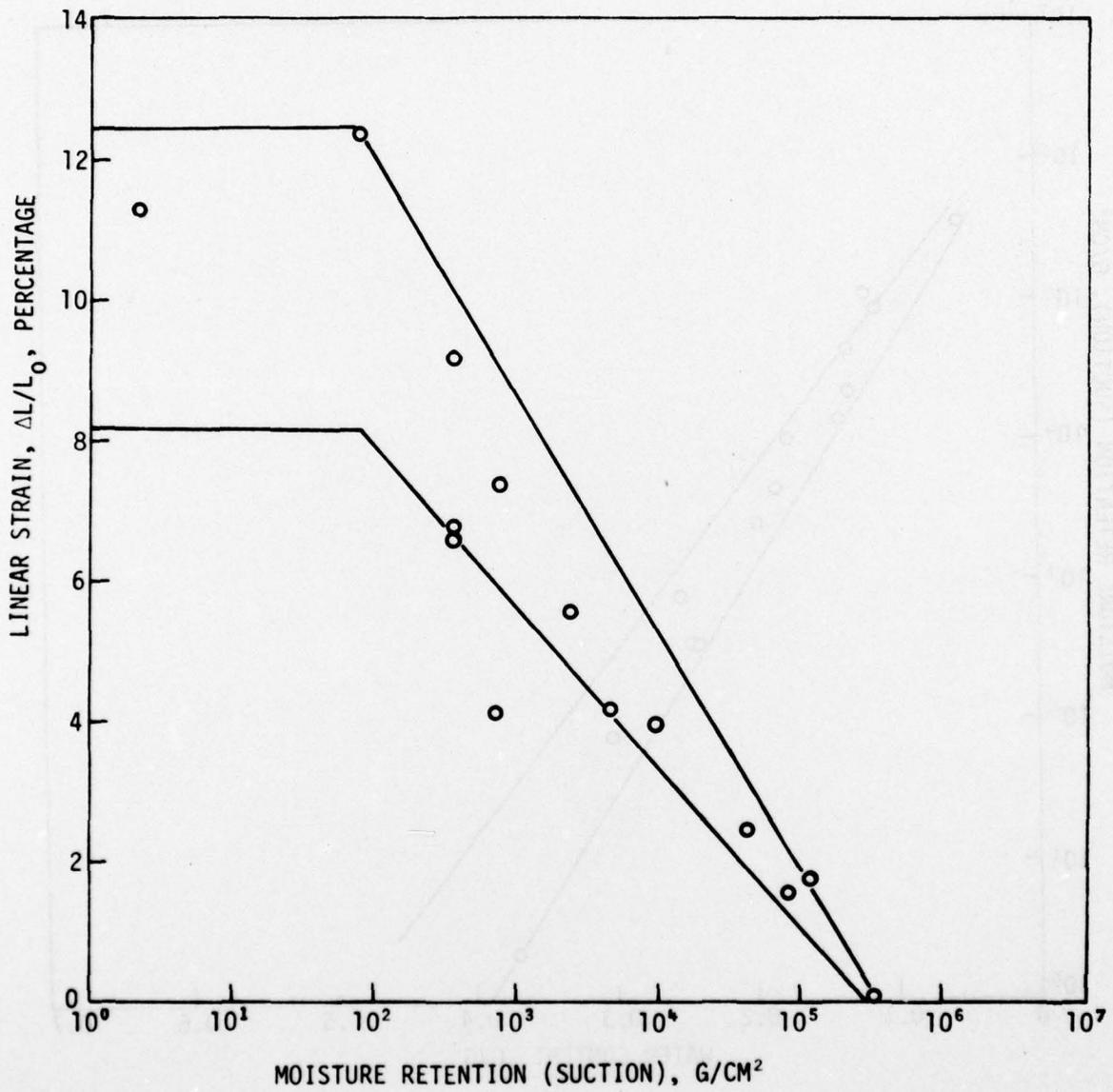
FIGURE B-22. MOQUINO, NEW MEXICO STRAIN-SUCTION DATA

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SITE Tucumcari, New Mexico

SAMPLE NO.	DEPTH	NATURAL		G_s	ATTERBERG LIM			CLAY <.002	BAR L.S.	COLE	UNIFIED CLASSF.
		ω	γ_d		LL	PI	SL				
—	FT		lb/ft³	—						—	—
1	1-3	18	89	2.72	42	20	11	30	13	-	CL
2-2	1.5-2.5	12	104	2.76	50	33	13	42	5	-	CH
2-3	4-5	12	104	2.80	57	41	12	33	16	.075	CH

FIGURE B-23. SOIL CLASSIFICATION DATA

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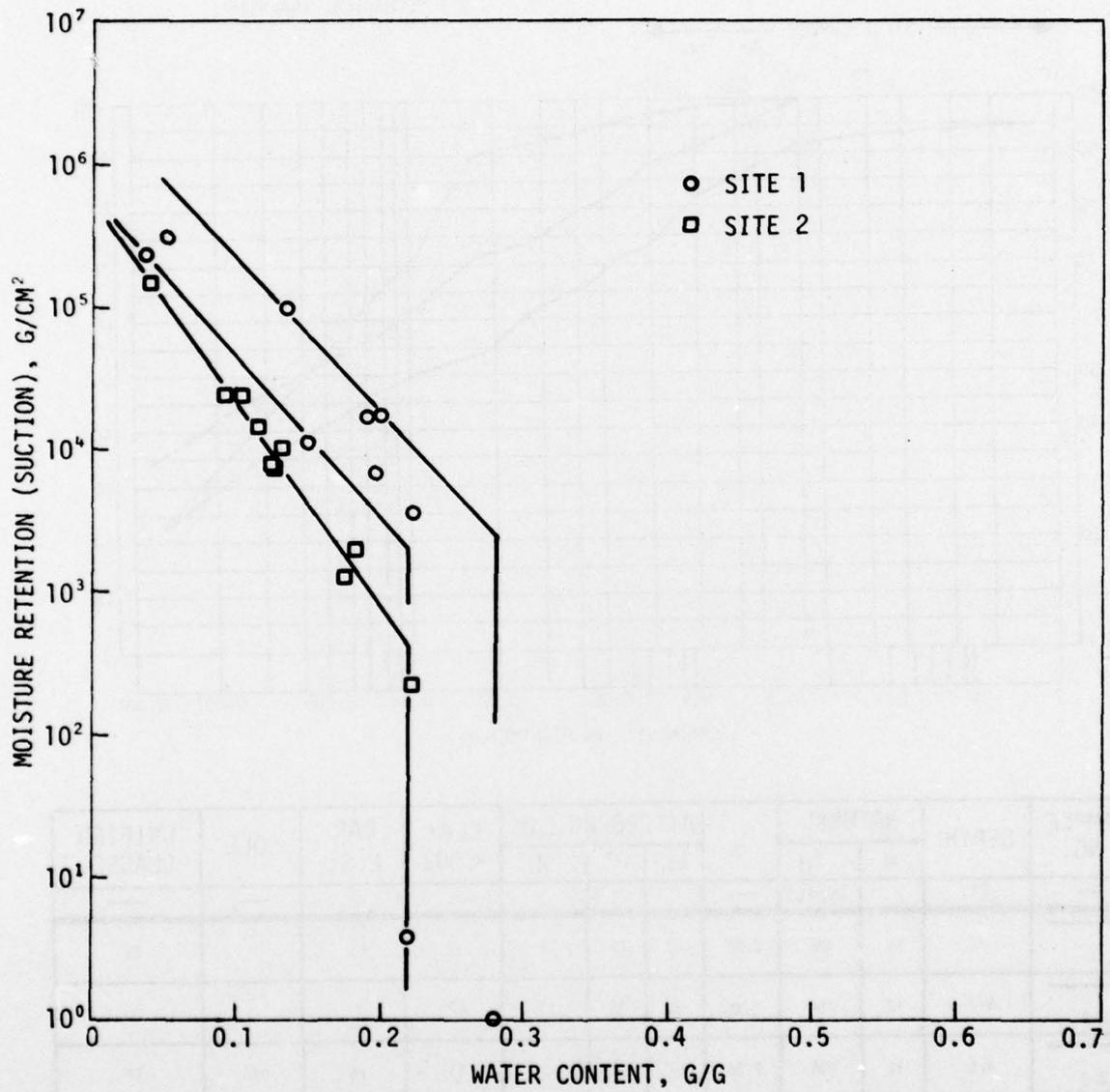
FIGURE B-24.   TUCUMCARI, NEW MEXICO MOISTURE DATA

110
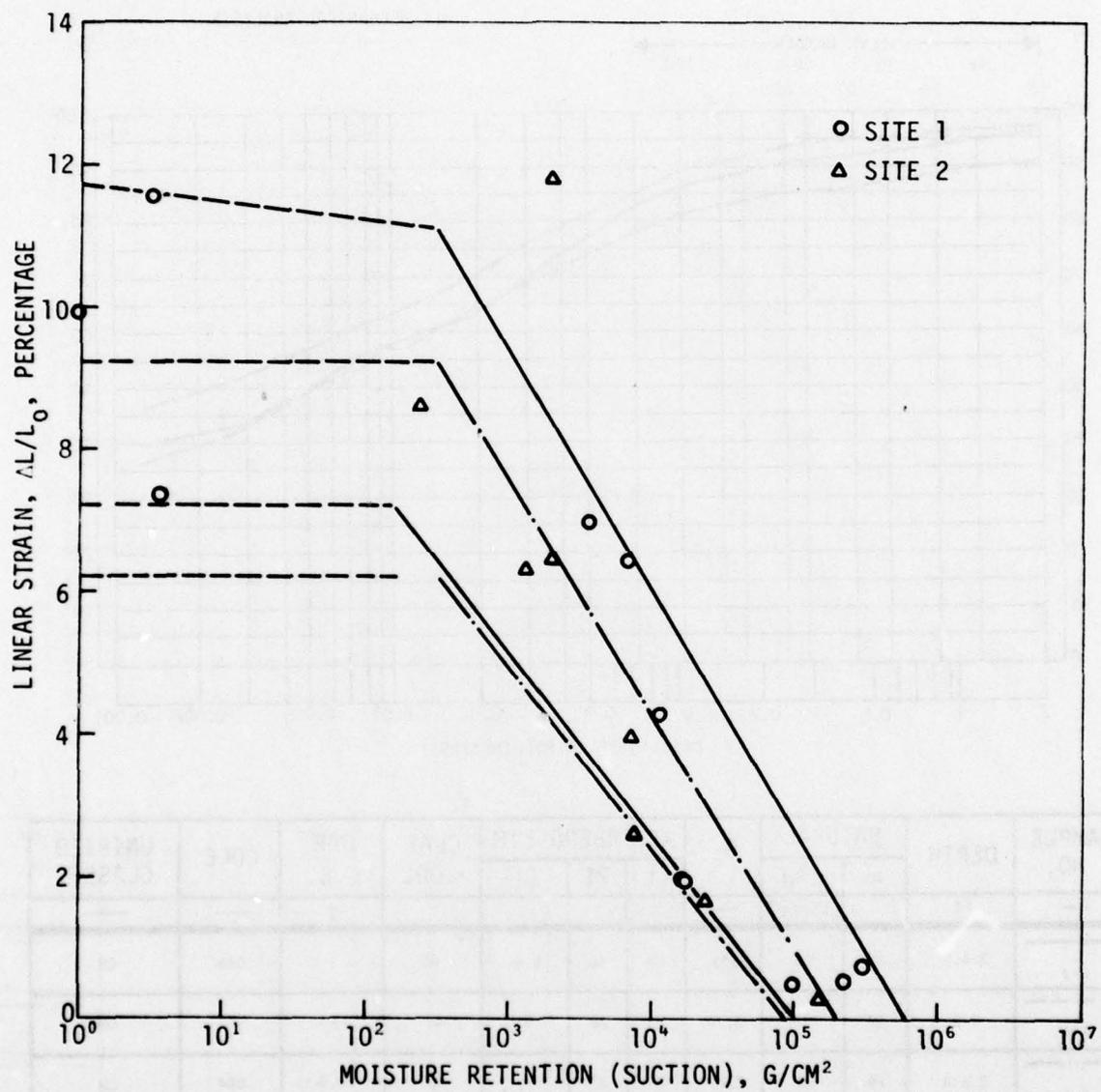
FIGURE B-25. TUCUMCARI, NEW MEXICO STRAIN-SUCTION DATA

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SITE Kelly AFB, Texas

US STANDARD SIEVES
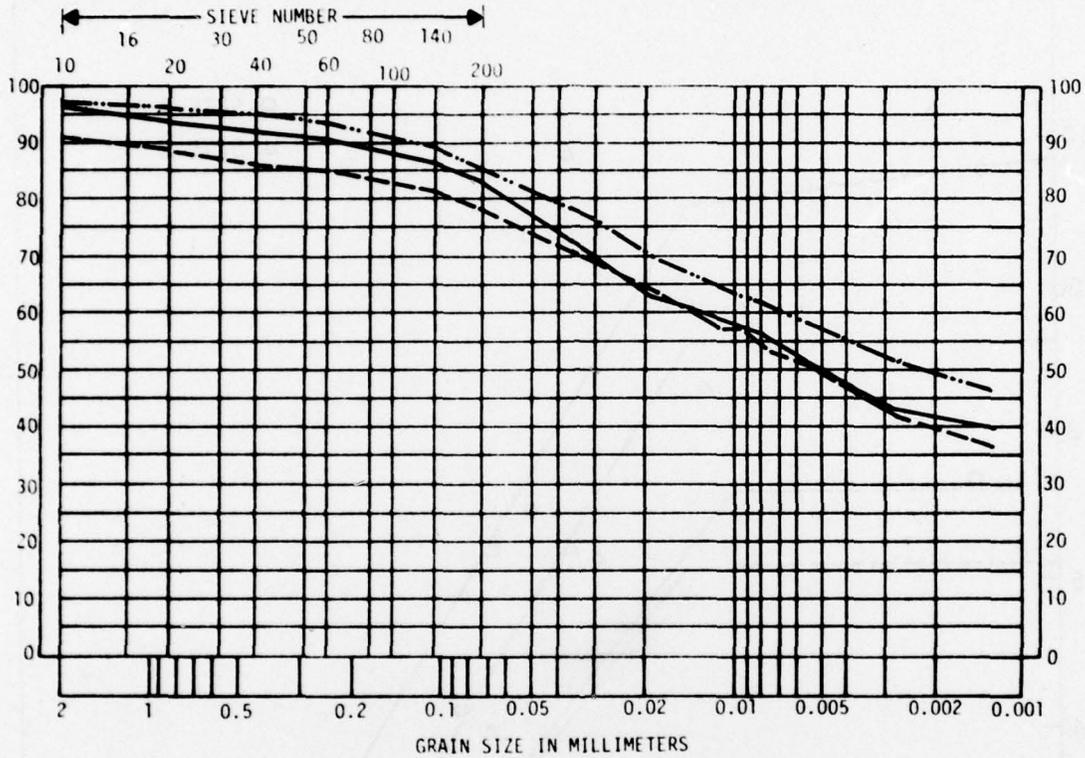
WET MECHANICAL ANALYSIS

GRAIN SIZE IN MILLIMETERS

SAMPLE NO.	DEPTH	NATURAL		G_s	ATTERBERG LIM			CLAY <.002	BAR L.S.	COLE	UNIFIED CLASSF.
		ω	γ_d		LL	PI	SL				
—	FT		lb/ft³	—						—	—
1-3	3-4.5	24	89	2.73	59	40	5.6	40	-	.070	CH
1-4	4-5	24	97	2.70	55	28	9.	42	-	-	CH
2-1	2.5-4	26	90	2.71	61	32	7.	50	20.5	.084	CH

FIGURE B-26.   SOIL CLASSIFICATION DATA

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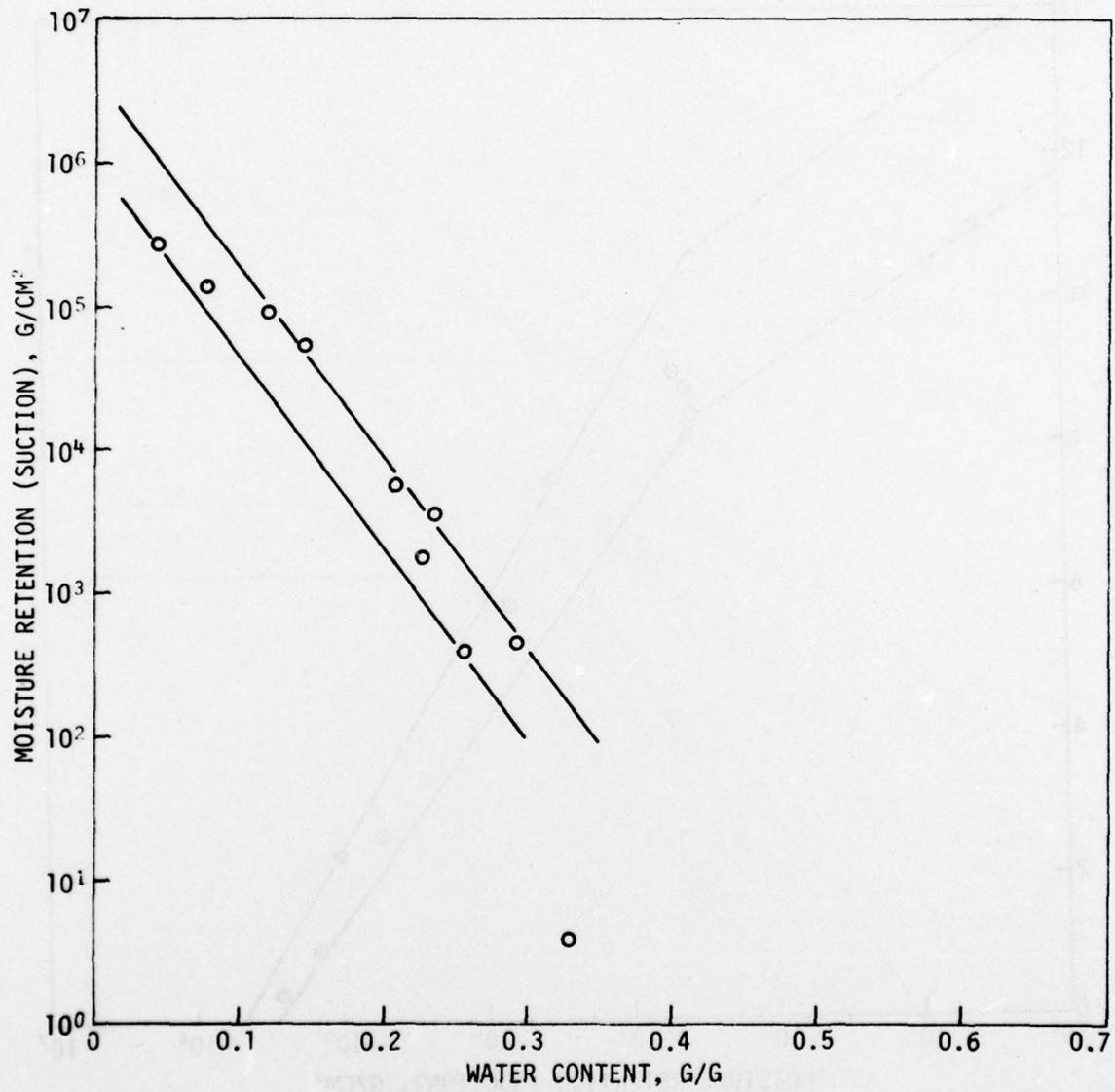
FIGURE B-27. KELLY AIR FORCE BASE, TEXAS MOISTURE DATA
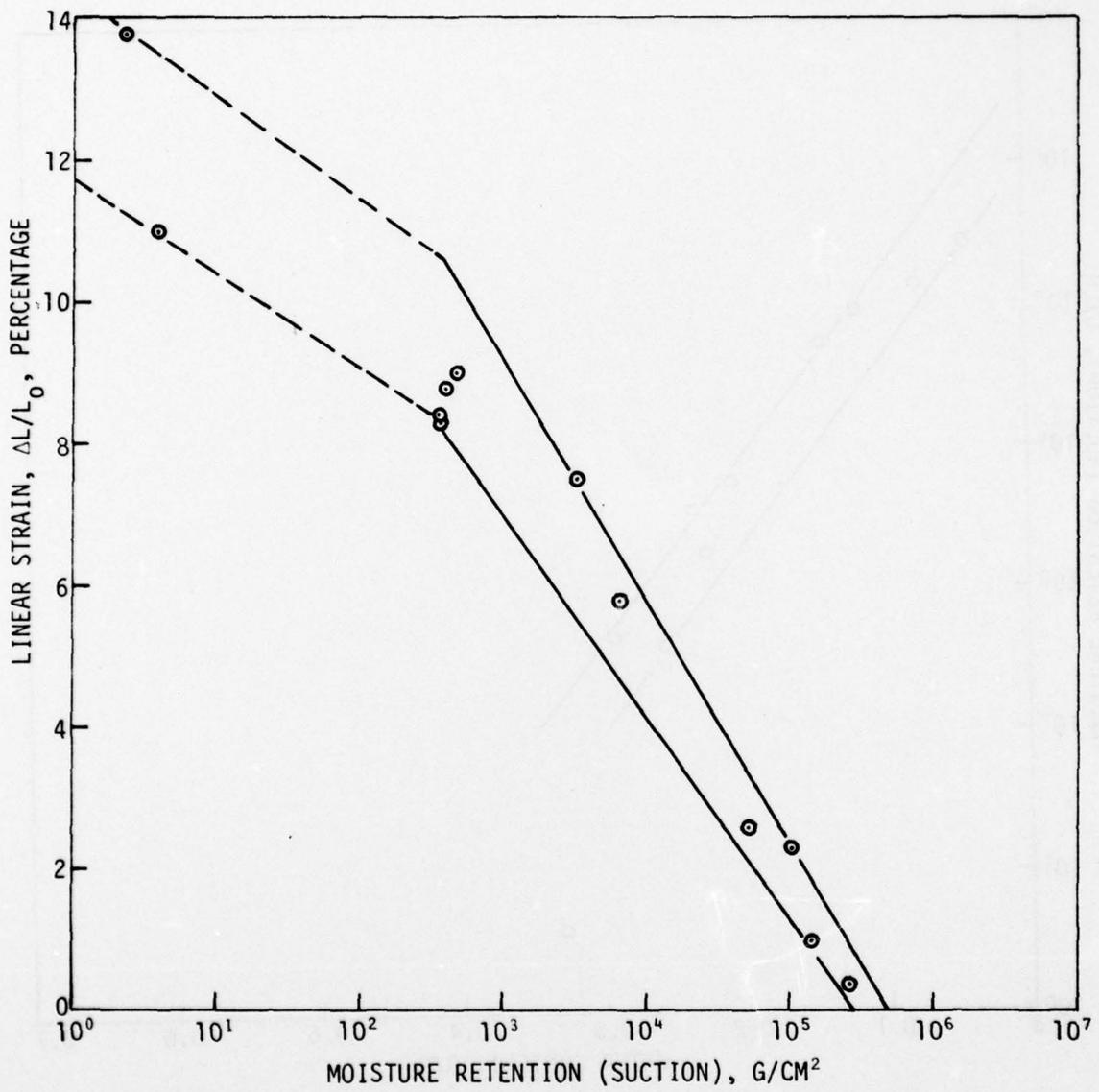
FIGURE B-28.   KELLY AIR FORCE BASE, TEXAS STRAIN-SUCTION DATA

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