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DESIGN AND CONSTRUCTION OF AIRPORT PAVEMENTS ON EXPANSIVE SOILS

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Final Report

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16. Abstract

A literature review was conducted to provide the best available techniques for designing airport pavements on expansive soils. Specific areas studied included identification and classification systems, prediction of heave, setting of acceptable levels of heave, and the design of stabilized soil layers. Methods of identification and classification were found useful for qualitative purposes but unreliable for quantitative prediction of field rates of heave. Prediction of heave is currently based on swell tests in consolidation type equipment and these methods require extreme caution. The technical literature failed to provide sufficient data from which acceptable limits of subgrade heave beneath airport pavements could be established. Stabilization of expansive soils may be accomplished with cement or lime. A procedure is provided for the design of stabilized layers. Present design systems do not provide methods for designing volume changes; therefore, the influence of stabilizers on volume change behavior is not properly accounted for in this procedure. An outline of the research needed to develop a satisfactory design system is presented. The technology required is presently available but a considerable effort is required to produce implementable methods. The approach recommended consists of establishing the load and moisture changes expected to occur at the site and evaluation of the soil response to those changes.

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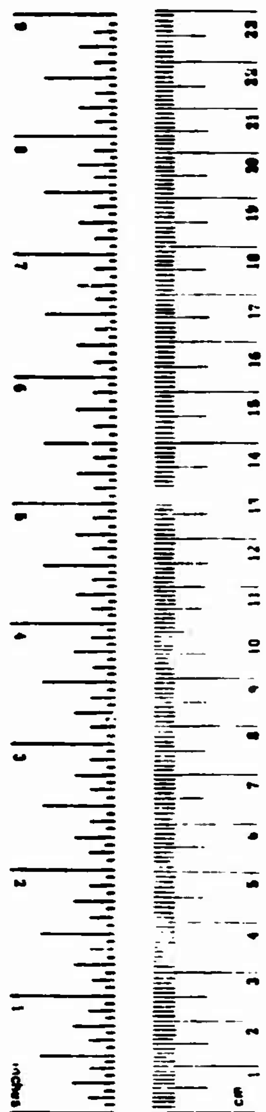
METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

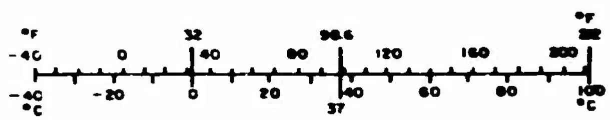
Symbol	When You Know	Multiply by	To Find	Symbol
	LENGTH			
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
	AREA			
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	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
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
	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			
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	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
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
	TEMPERATURE (exact)			
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



CONTENTS

ILLUSTRATIONS

ILLUSTRATIONS (Concl'd.)

TABLES

SECTION 1
INTRODUCTION

BACKGROUND

The pavements of airports (i.e., runways, taxiways, ramps, parking aprons, etc.) constitute a vital part of the overall facility and therefore pavement construction and maintenance costs are important in the planning and operation of airports. Premature failure of these pavements (manifested as surface roughness) effects operational limitations, accelerates aircraft fatigue, and reduces safety; on the other hand, initial construction and material costs prohibit deliberate overdesign of these pavements.

A major cause of premature pavement failure is underlying expansive soils which by shrinking and swelling cause surface roughness. Although current Federal Aviation Administration (FAA) design procedures (ref. 1) do not adequately treat the design of pavements over expansive soils, recognition of expansive soils as a significant engineering problem took place many years ago. A concentrated effort by the world engineering community to solve this problem was begun in 1965 with the First International Conference and has continued with the following national and international conferences:

(1) First International Research and Engineering Conference on Expansive Clay Soils, August 30 - September 3, 1965, Texas A&M University, College Station, Texas.
(2) Second International Research and Engineering Conference on Expansive Clay Soils, 1969, Texas A&M University, College Station, Texas.
(3) Third International Research and Engineering Conference on Expansive Clay Soils, July 30 - August 1, 1973, Haifa, Israel.
(4) Workshop on Expansive Clays and Shales in Highway Design and Construction, sponsored by the Federal Highway Administration, December 13 - 15, 1972, Denver, Colorado.
(5) University-Industry Workshop on Behavior of Expansive Earth Materials, sponsored by the National Science Foundation, October 1974, Denver, Colorado.

The proceedings of these conferences, specialty sessions in the meetings of the International Conference on Soil Mechanics and Foundation Engineering (ICSMFE), and several significant literature reviews form the basis of this report.

OBJECTIVES

This investigation was initiated to review the current engineering literature and synthesize from it a design procedure for stabilizing expansive soils beneath airport pavements. To do this, the study was broken down into six specific areas:

(1) Methods of identifying and classifying the types of soil that are considered expansive and cause early pavement distress

(2) Laboratory and/or field test methods to determine the level of expansion and shrinkage

(3) Selection of the type and amount of stabilizing agent (lime, cement, asphalt, only)

(4) Test methods to determine the physical properties of stabilized soil

(5) Test methods to determine the durability of stabilized soil

(6) Field construction criteria and procedures

SCOPE

This report addresses the above objectives and provides a summary of the current literature pertaining to the subject. Conclusions and recommendations were made based on the current literature, without laboratory verification. Soil volume changes caused by other factors (e.g., frost heave, salt heave) were not studied.

SECTION 2
EXPANSIVE SOILS

ORIGIN AND DISTRIBUTION

Expansive soils are made up of clay particles that result from the alteration of parent materials. Alteration takes place by several processes: weathering, diagenesis, hydrothermal action, neoformation, and post depositional alteration (ref. 2). Most clay minerals are transported by air or water to areas of accumulation. Once deposited, the materials are subjected to the local conditions of accumulation (overburden) and erosion which make up the geologic stress history of the materials (ref. 3). Thus, the existing clay soil at a site is the product of parent material, mode of alteration, and geologic history. Interaction between the soil and the local environment produces continual change in the soil and determines future behavior.

Expansive soils are distributed all over the world. Usually the areas with the most severe problems are those with local climates that produce desiccation. A recent report (ref. 4) provides the results of a study of the distribution of expansive soils in the Continental United States. Distribution is generally a result of geologic history, sedimentation, and local climatic conditions. A more detailed and localized source of distribution information is available through soil surveys published by the U.S. Department of Agriculture Soil Conservation Service. These surveys provide distribution maps and considerable information useful in engineering applications (table 1). In the initial planning of airport facilities, publications reflecting the distribution of soil types in the area should be carefully considered, and the location with the best soil conditions should be selected. The three clay types recognized in engineering studies exhibit distinctly different structures (table 2). Kaolinite is made up of alternate layers of silica tetrahedra and gibbsite bound together by relatively strong hydrogen bonds (ref. 6). The relatively large particles and stable structure are not expansive. Illite is made up of a 2:1 structure consisting of gibbsite sheets surrounded by silica tetrahedra. About 20 percent of the silicons are replaced by aluminum, and the resulting negative charge is balanced by potassium ions between the 2:1 sheets.

Table 1. Estimated Soil Properties Significant to Engineering [after Folks (ref. 5)]

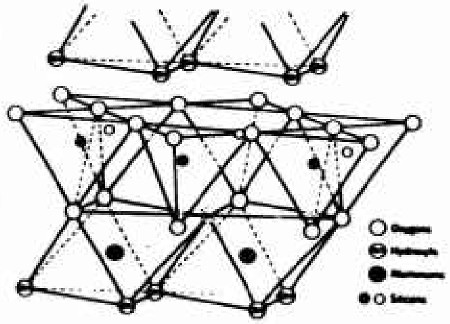
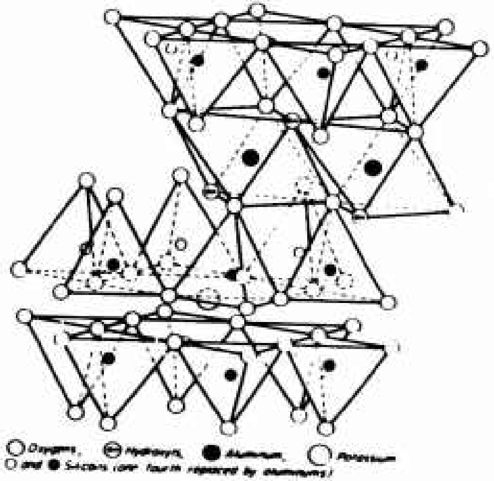
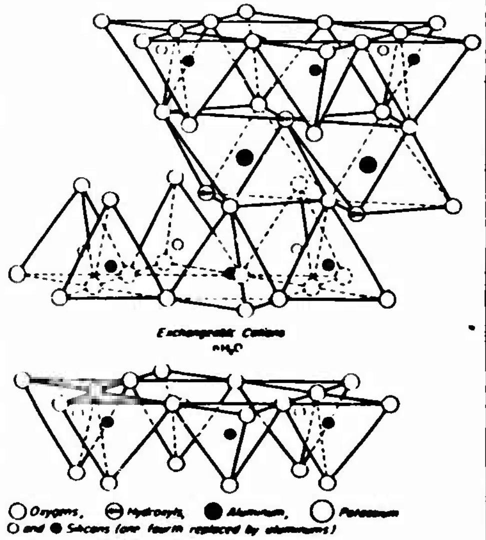
Soil series and map symbols	Depth to bedrock	Depth from surface in representative profile	Classification		
			USDA texture	Unified	AASHO
	Feet	*Inches*			
*Rednun: RD, RE, RG	>5	0-35	Clay loam	CL	A-6 or A-7
For Pena part of RE, see Pena series; for Travessilla part of RG, see Travessilla series.		35-60	Very fine sandy clay loam	CL or ML	A-6 or A-4
Riverwash: RH. Too variable for valid interpretation.					
*Rock outcrop: RK, RL. Too variable for valid interpretation. For Chimayo part of RL, see Chimayo series					
Rock slides: RC. Too variable for valid interpretation.					
Rough broken land: RU. Too variable for valid interpretation.					
*Santa Fe: SF, Sk, SM	½-1½	0-13	Very gravelly clay loam	GC	A-2
For La Fonda part of SF, see La Fonda series. Rock outcrop parts of Sk and SM are too variable for valid interpretation.		13	Bedrock		
*Silver: SP, SR	>5	0-14	Clay (loam surface layer)	CL	A-6 or A-7
For Pojoaque part of SP, see Pojoaque series.		14-45	Silty clay loam	CL	A-6
		45-60	Very fine sandy loam	ML	A-4
Stony rock land: ST. Too variable for valid interpretation.					
*Supervisor: SU, SV	1½-2½	0-23	Gravelly sandy loam and very gravelly light sandy loam.	SM	A-1
Rock outcrop part of SV is too variable for valid interpretation.		23	Bedrock.		
*Tapia: TA	>5	0-21	Clay loam (loam surface layer)	CL	A-6
For Dean part, see Dean series.		21-60	Gravelly loam	SM or SC	A-4
*Travessilla: TB, TR	½-1½	0-10	Loam	ML	A-4
For Bernal part of TB, see Bernal series. Rock outcrop part of TR is too variable for valid interpretation.					
Tuff rock land: TU. Too variable for valid interpretation.					
Wilcoxson, variant: WC	2½-3	0-26	Sandy clay, clay, and gravelly clay.	CH	A-7
		26-31	Coarse sandy loam	SM	A-2
		31	Soft bedrock.		
Willard: WL	>5	0-10	Loam	ML or CL	A-4 or A-6
		10-60	Clay loam	CL	A-6
Witt: WN	>5	0-36	Clay loam and sandy clay loam	CL or ML	A-6
		36-60	Loam	ML or CL	A-4
Zuni, variant: ZU	1½-3½	0-16	Loam and clay loam	ML or CL	A-4 or A-6
		16-20	Clay	CH	A-7
		20	Weathered bedrock.		

[1] In mapping unit Ao corrosivity to uncoated steel is high.
[2] In mapping unit Bf corrosivity to uncoated steel is high throughout.

Coarse fraction greater than 3 inches	Percentage less than 3 inches passing sieve—				Permea-bility	Available water holding capacity	Reaction (1:5 dilution)	Shrink-swell potential	Corrosivity to uncoated steel
	No. 4 (4.7 mm.)	No. 10 (2.0 mm.)	No. 40 (0.42 mm.)	No. 200 (0.074 mm.)					
Percent					*Inches per hour*	*Inches per inch of soil*	*pH*		
....	....	100	90-100	80-90	0.06-0.2	0.19-0.21	7.9-9.0	High	Moderate.
....	....	100	90-100	70-80	0.63-2.0	0.14-0.16	7.9-8.4	Moderate	Moderate.
....	35-55	30-50	25-45	20-35	0.63-2.0	0.08-0.10	6.6-7.3	Low	Moderate.
....	95-100	90-100	90-100	85-95	0.06-0.2	0.14-0.16	7.9-8.4	High	High.
....	95-100	90-100	90-100	85-95	0.2-0.63	0.19-0.21	7.9-8.4	Moderate	Moderate.
....	95-100	90-100	85-95	50-65	0.63-2.0	0.16-0.18	7.9-8.4	Low	Low.
5-15	80-90	55-65	30-40	15-25	2.0-6.3	0.06-0.08	6.1-6.5	Low	Low.
....	95-100	90-100	85-95	75-85	0.63-2.0	0.19-0.21	7.9-9.0	Moderate	Moderate.
25-35	80-90	75-85	60-75	35-50	0.63-2.0	....	8.5-9.0	Low	Low.
0-25	90-100	85-95	65-75	50-60	0.63-2.0	0.14-0.18	7.4-7.8	Low to moderate	Low.
....	90-95	85-95	75-85	65-75	0.06-0.2	0.14-0.16	6.1-7.3	High	High.
....	100	95-100	55-65	25-35	2.0-6.3	0.10-0.12	6.6-7.3	Low	Low.
....	....	100	85-95	60-75	0.63-2.0	0.16-0.18	7.9-8.4	Low to moderate	Moderate.
....	....	100	90-100	70-85	0.2-0.63	0.05-0.07	8.5-9.0	Moderate	High.
....	....	100	80-90	65-75	0.63-2.0	0.16-0.18	7.9-8.4	Moderate	Moderate.
....	....	100	85-95	60-75	0.63-2.0	....	8.5-9.0	Low	Low.
....	....	100	85-95	60-75	0.2-6.3	0.17-0.19	6.1-7.3	Moderate	Moderate.
....	....	100	90-100	75-95	0.06-0.2	0.14-0.16	6.6-7.3	High	High.

[1] In mapping unit Fs corrosivity to uncoated steel is high throughout.

Table 2. Schematic Diagrams and Properties of Clay Minerals (after reference 6)

	Kaolinite	Illite	Montmorillonite
Schematic Structure of Clay Minerals			
Particle Thickness	0.5 to 2 µm	0.003 to 0.1 µm	≥ 9.5 Å
Specific Surface, m^2/g	10 - 20	65 - 180	50 - 840
Cation Exchange Capacity, $\frac{\text{Millequivalents}}{\text{100 grams}}$	3 - 15	10 - 40	80 - 150

The potassium bonds are strong and prevent water from entering between the layers. In montmorillonite a 2:1 structure like that of illite is present, but there is characteristically extensive isomorphous substitution, which determines the behavior of the mineral. As used here, *isomorphous substitution* means the substitution of one metallic ion for another within the tetrahedral or octahedral unit. The important effect of the lattice substitutions is a net negative charge that attracts bipolar water molecules between the layers; this results in an expanded layer structure (fig. 1).

MECHANISMS OF SWELL

Soil volume changes result from an imbalance in the internal energy of the system (soil/water/plants/air). Energy imbalances important in engineering result from moisture movement caused by loads, desiccation, and temperature changes (refs. 7, 8). Response to a specific set of conditions is determined by the composition, structure, and geologic history of the soil. The largest component of volume change is that of the clay micelle which surrounds the individual clay particles in the soil (refs. 6, 9). Water is forced out of the micelle by loads, desiccation, or temperature along energy gradients and a reduction in volume results. When these influences are removed or reduced, the energy gradients are reversed; the available water is forced into the clay micelle and swell is produced (ref. 10). Since several detailed studies (refs. 4, 6, 9, 11) are presented in the literature, discussion here is limited to that required for an understanding of expansive soil behavior.

Water Fixation by Polar Adsorption (Hydration)

Bipolar water molecules are attracted to the clay particle surface by the electric charge imbalance caused by isomorphous substitution, usually negative (refs. 2, 9, 12, 13). A layer of solid-like water forms a new surface of oriented particles, which attracts succeeding layers of oriented water molecules, up to a thickness of 10 to 16 molecular layers or 25 to 40 Å (1Å = 10^{-8}cm). The water beyond this bound layer is mobile and moves freely under any stress gradient (refs. 2, 13, 14). The bound water layers permit adjacent particles to slip past one another without elastic rebound, rupture, or appreciable

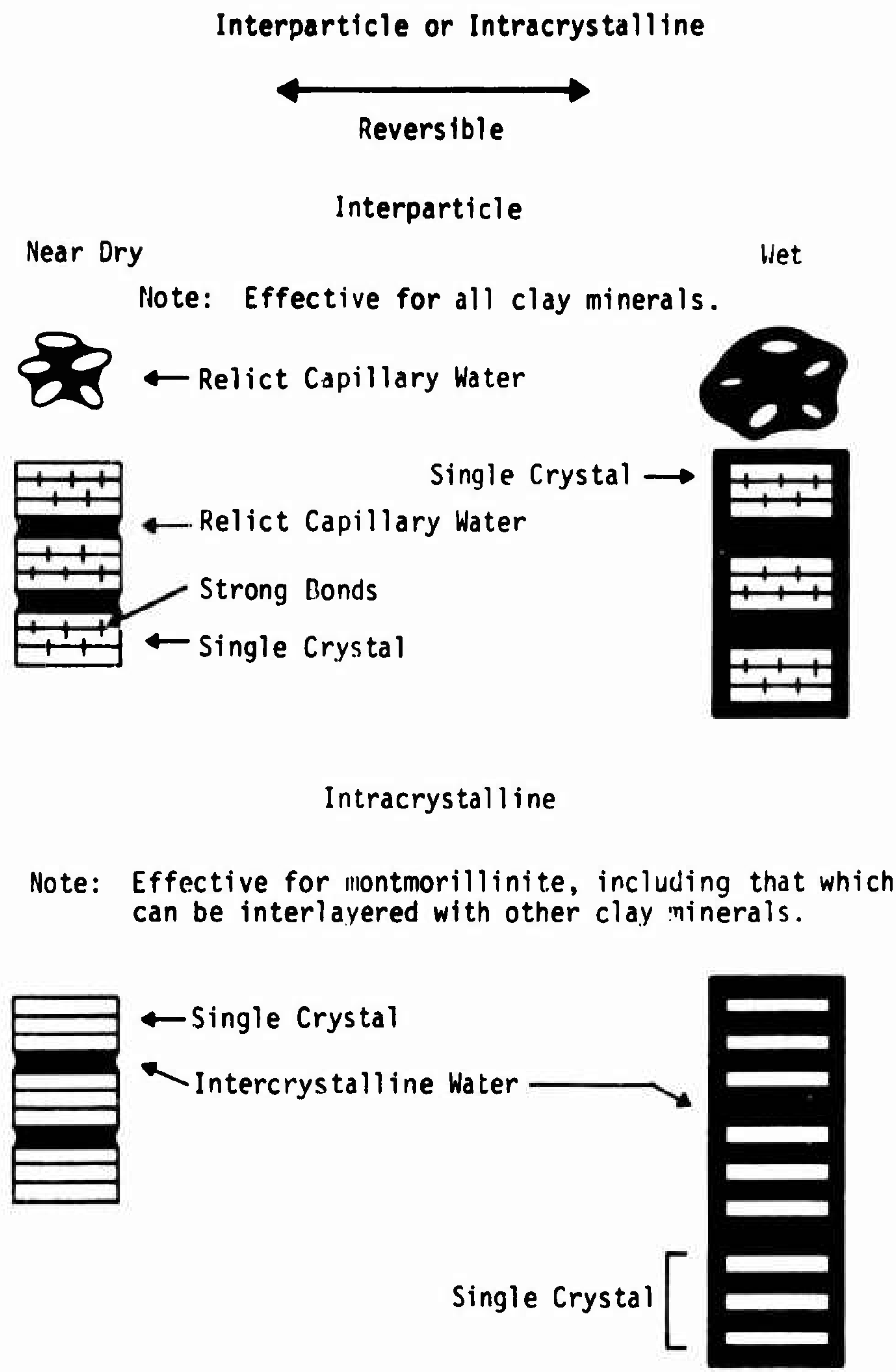


Figure 1. Nature of Hydration Volume Changes (after reference 3)

volume change; this is the well-known plastic behavior of clays. The thin, highly viscous, solid-like water layer is called the *adsorbed layer* and the less viscous layer, which is between the bound and the free water, is called the *electric double layer*.

Osmotic Imbibition

Osmosis can be defined as the passage of a solvent through a semipermeable membrane from a solution of lesser concentration to one of higher concentration to equalize the concentrations (refs. 6, 9). The osmotic pressure is the pressure required to prevent the flow of the solvent.

Water in the soil voids is attracted to the clay surfaces because of isomorphous substitution and the resulting concentration of cations at the clay-particle surface. The electric double layer of viscous water serves as a semipermeable membrane which allows water (solvent) to pass through and dilute the cations (solute) by separating them (volume increase). This process continues until equilibrium conditions are reached. Decreasing the difference in solute concentration decreases the osmotic pressure and therefore the swell (difference between soaking a soil with distilled water and salt water). This phenomenon is used in the drilling industry in drilling through expansive shales. Ordinary water produces swell and binds the drill train; therefore, a brine solution is used to lower the energy level of the water to below that of the soil water.

Several authors (refs. 7, 9, 15, 16) suggest that osmotic pressure is generated by the pressure of cations (solute) against a fluid boundary (double layer) that is free to expand and enlarge the space for the solute ions. The osmotic suction pressure, P_s (force/unit area), can be calculated with the Van't Hoff equation

$$P_s = RT(C_c - 2C_o)$$

where

P_s = swelling pressure, bars (14.50 bars/psi)

R = gas constant, 0.08099 liter-bar-$^0K^{-1}$-mole^{-1}

T = absolute temperature, ^0K

C_c = concentration at mid-distance between clay platelets, moles of ions/liter

C_o = concentration in the bulk solution, moles of ions/liter

C_c can be derived from diffuse double layer theory (ref. 7):

$$C_c = \frac{\pi^2}{v^2 B(d + X_o)^2}$$

where

v = valence of ion

B = temperature-dependent constant (usually taken as 10^{15}cm - millimole^{-1})

d = half the distance between clay platelets, cm

X_o = $4/vBG$, where G = surface charge density, coulomb-cm^{-2}

Approximate values of X_o are as follows: illite, $X_o \cong 1/v\mathring{A}$; kaolinite, $X_o \cong 2/v\mathring{A}$; montmorillonite, $X_o \cong 4/v\mathring{A}$. Ruiz (refs. 9, 17) modified the equation for real soils as follows:

$$P = P_s f$$

where

P = real soil swelling pressure

f = function of moisture content, $f < 1$

Osmosis is possible only in polar fluids, such as water, that are able to disperse exchangeable cations. Swelling varies with the type of cation and generally decreases in the order Na, Li, K, Ca, Mg, and 2H for Wyoming bentonite (refs. 9, 18, 19).

Surface Tension

The spaces between clay particles in soils form capillary tubes. As water is removed from the soil, an air/water interface forms. Attraction of water molecules to the walls of the capillary tube (soil particles) produces menisci (refs. 6, 9). Tension in the water, u(g-cm^{-2}), may be expressed as

$$u = \frac{2T_s}{r} \qquad \text{(ref. 13)}$$

where

T_s = surface tension of water (0.076 g-cm^{-1})

r = radius of capillary tube, cm

As the water content decreases, the menisci recede into the capillaries, drawing particles closer together until no further volume change is possible because of particle contact. The tension in the water is balanced by compression in the soil particles. When additional water becomes available, the water tension is released and the soil particles rebound as the associated compressive stress is relieved.

Thermoosmosis

The movement of soil moisture caused by the energy gradient produced by temperature differences, which cause changes in water vapor pressure, is called *thermoosmosis* (ref. 9). This aspect of moisture movement, although negligible in saturated soils (refs. 20, 21), is significant in unsaturated soils. The swell associated with such moisture movement is small (ref. 9).

Elastic Bending

Elastic deformation and rebound of soil particles under applied loads may contribute to shrinkage and swelling behavior, particularly in soils with flat platy particles (ref. 22). Using mica and dune sand, Gilboy (ref. 23) illustrates this effect. The results of his tests show that the consolidation and rebound of compacted mixtures are proportional to the mica content, and the contribution of elastic bending depends on particle structure and properties as shown below:

Mica, %	Volume Decrease Under 10 kg/cm^2 (142 psi), %	Increase in Void Ratio Upon Removal of Load, %
10	36	26
20	47	31
40	51	42

Entrapped Air

When an initially desiccated clay is allowed to take up water, air may be trapped within the soil mass. This air displaces water in the double layer and induces tensile stresses in the particles surrounding the air pocket. This influence is greater in soils with higher air contents (i.e., drier soils).

SECTION 3
EXPANSIVE SOILS TEST PROCEDURES

The procedures described in this section have been used in engineering studies of expansive soils and in some cases the literature provides considerable data derived from their use. Table 3 was prepared to show the results normally obtained for general soil types. The different procedures for evaluating swell potential are reflected in the variation in swell and swell pressure values reported in the literature (table 4). Other procedures reported in the literature are too expensive, complex, or time consuming for routine engineering design purposes. However, for the interested reader, these techniques can be found in the following references:

Technique	Reference
X-Ray Diffraction	2,24,25,26,27
Electron Microscopy	2,25,26
Differential Thermal Analysis	2,24
Infrared Radiation	27
Dye Adsorption	6,27
Specific Surface Area	9,28,29
Cation Exchange Capacity	2,30
Dielectric Dispersion	31

SWELL

A remolded or undisturbed soil sample is placed in a consolidometer under specified conditions and allowed access to water. The vertical rise of the specimen is then measured. A sample of this procedure is presented in appendix A; numerous versions involving variations in sample preparation, wetting, soaking, specimen size, surcharge loading, etc., are reported in the literature. Because of these various procedures, it is difficult to compare one set of results to another. Even though no single procedure is widely accepted, this is the most popular and reliable technique for evaluating swell potential. This test may be referred to as a *loaded swell test* or a *free swell test*, depending on the

Table 3. Typical Soil Properties (after reference 32)

Soil Property	Heavy Clays	Typical Clays	Silty Soils	Sandy Soils	Test Procedures	
					ASTM	AASHO
Gradation (% of grain size shown in the soil)	80-100	30-80	40-100	50	D422	T88
Grain Size (mm)	≤0.005	≤0.005	0.05-0.005	2.0-0.05	D422	T88
Consistency						
Liquid Limit (%)	80-100	40-60	25-50	Nonplastic	D423	T89
Plastic Limit (%)	-	5-30	5-30	Nonplastic	D424	T90
Plasticity Index (%)	70-80	20-40	10-20	Nonplastic	D424	T91
Shrinkage Limit (%)	-	6-14	15-30	No Volume Change	D427	T92
Maximum Density (lb/ft^3)	-	90-105	100-115	110-135	D698	T99
Optimum Moisture Content(%)	-	20-30	15-25	8-15	D698	T99

Table 4. Typical Results of Swell Tests

Reference	Range of Swell, %	Range of Swell Pressure, psi	Soils Used	Remarks
33	0-13.6	0-83	Texas & Israel	1.4-psi surcharge in swell test.
34	0-13.6	0-83	Texas Gulf Coast	1.4-psi surcharge in swell test.
35	0-15.8	0-284	Israel	USBR Procedures: 1-psi surcharge.
36	0-50.1	0-147	Western U.S.	USBR
37	1.3-39.8	-	Western U.S.	USBR
37	0.1-54.0	-	Pure Clay & Mixtures	USBR
38	-	0-69	Continental U.S.	FHA, PVC Swell Index

type of loading applied to the sample. Results may be expressed in percent swell under the specific load used.

SWELL PRESSURE

A test similar to that described above, except that the sample is loaded in increments so that the volume remains constant, may be performed to determine swell pressure--the pressure required for zero volume change. This test in combination with the free swell test is often performed on the same sample in some test procedures (appendix A). It is also referred to as a *no-volume-change* test.

POTENTIAL VOLUME CHANGE

Potential volume change is determined by a no-volume-change test in a specified apparatus developed for the Federal Housing Administration and used for soil classification (ref. 39). Test duration is two hours. The pressure required for zero volume change is called the *swell index* (given in pounds per square foot) and it is used in classifying the soil. Figure 2 illustrates the use of the swell index to classify soils based on the method of sample preparation (i.e., wet, dry, moist).

EXPANSION INDEX

The expansion index, EI, is an index property of a soil determined in a specified consolidometer ring apparatus developed for evaluation of soil expansion (ref. 40). The EI is calculated by

$$EI = (1000)\Delta hF$$

where

Δh = vertical expansion measured

F = fraction of the sample < #4 sieve (4.76 mm); only the minus #4 material is used in the test.

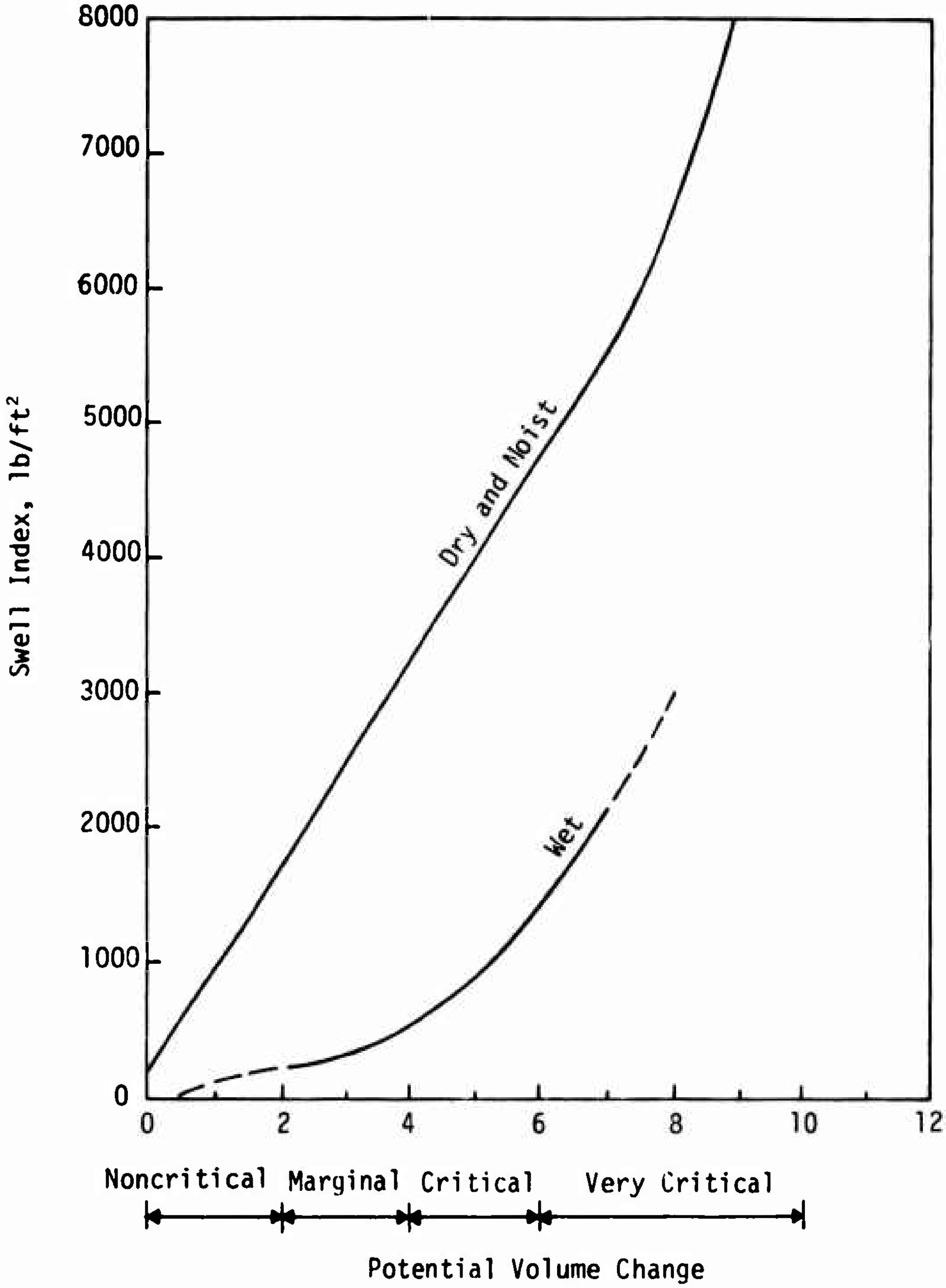


Figure 2. Swell Index Versus Potential Volume Change
[after Lambe (ref. 39)]

Weighting factors are assigned by depth to compute a weighted EI as follows: 0 to 1 ft, 0.4; 1 to 2 ft, 0.3; 2 to 3 ft, 0.2; and 3 to 4 ft, 0.1. The EI for each soil layer is multiplied by the appropriate weighting factor and summed to determine the weighted EI. The soil at a site is then classified as low, moderate, high, or very high potential expansion (table 5). This procedure was developed for residential slab construction in Southern California and experience with it is limited to that application (ref. 41). Table 5 shows a comparison of EI to other tests.

ATTERBERG LIMITS AND INDEXES

The meaning of the Atterberg limits and indexes used in all engineering soil classification systems is illustrated in figure 3. These tests, which have been used in soils engineering for many years, provide a widely acceptable means of rating soils. In the three general soil-classification systems used in the United States, fine-grained soils are classified on the basis of liquid limit, LL, and plasticity index, PI.

CLAY CONTENT

A test is used to determine the quantity of material in a soil sample that is smaller than a selected size, expressed as a percentage by weight of the total sample. Sizes used are 2 μm (0.002 mm) and 1 μm (0.001 mm); the upper limit of the clay range is generally considered to be 2 to 5 μm. The test usually requires a hydrometer analysis.

ACTIVITY

Activity, A, the ratio of the plasticity index divided by the percent clay (% < 2 μm), was first defined and used by Skempton (ref. 42). This property has been used by various investigators to classify soils.

Table 5. Approximate Relationship of Expansion Index to Other Tests (after reference 41)

Soil Test	Approximate Range			
Plasticity Index, %	5-15	10-25	20-45+	35+
Clay Content (0.002 mm), %	5-15	10-25	20-30	30-45
Swell (60 lb In-Situ), %	0-4	3-9	8-12	12+
Swell (144 lb In-Situ), %	0-2	2-6	6-10	10+
Swell (650 lb In-Situ), %	0-1	1-3	3-5	5+
Expansion Classification	Low	Moderate	High	Very High
Weighted Expansion Index	0-20	20-60	60-100	100+

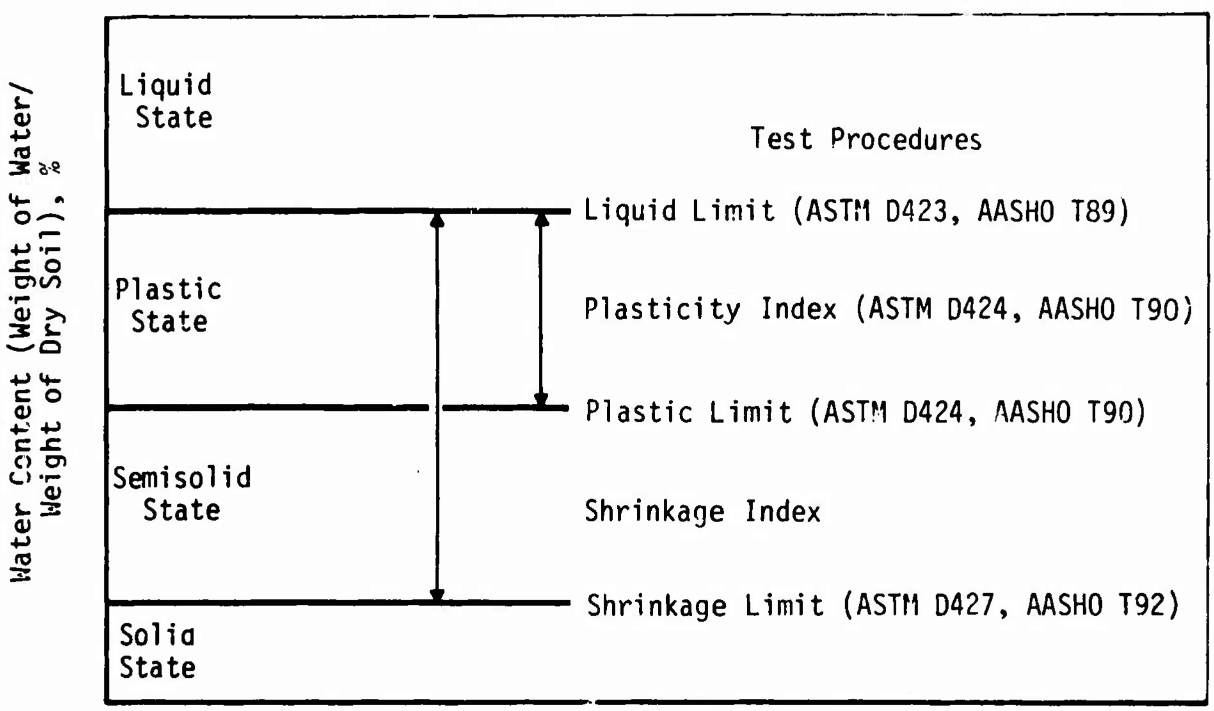


Figure 3. Consistency Limits and Indexes

DENSITY

Density is the weight per unit volume of dry soil calculated in accordance with standard procedures (ASTMD698, AASHO T99) and usually expressed in pounds per cubic feet (g/cm^3 in CGS System; kg/m^3 in SI System).

MOISTURE CONTENT

Moisture content, w, is the weight of water in a given mass of soil determined in accordance with standard procedures (ASTMD698, AASHO T99) and expressed as a percentage of the dry weight of the soil.

LINEAR SHRINKAGE

Linear shrinkage, LS, is the change in length of a soil sample as it dries to the shrinkage limit, SL, expressed as a percentage of the original length. A test procedure is given in appendix B.

FREE SWELL

The free swell test consists of placing a known volume of dry soil in water and noting the swelled volume after the material settles, without any surcharge, to the bottom of a graduated cylinder. Mixed success is reported for this test (refs. 43, 44).

SURFACE AREA

The surface area is the sum of the internal and external surfaces of soil particles. There are several methods used in agriculture, but no standard engineering procedure exists. The method should be specified. Values are reported in square meters per gram unless otherwise specified.

EQUILIBRIUM MOISTURE CONTENT

The equilibrium moisture content is the moisture content a soil will reach when exposed to an environment with a constant humidity; for example, a subscript 85 (w_{85}) represents the equilibrium moisture content at a relative humidity of 85 percent.

PENETRATION RESISTANCE

Penetration resistance is the resistance of a soil to penetration by a sampling tube expressed as blows per foot and determined in accordance with ASTM D1586 and AASHO T206, unless otherwise specified.

COEFFICIENT OF LINEAR EXTENSIBILITY

The coefficient of linear extensibility, COLE, is used in U.S. Department of Agriculture soil surveys to classify clay soils. It is determined from the change in bulk density of a soil clod, 5 to 8 cm in diameter (ref. 45).

$$COLE = \left(\frac{D_{bd}}{D_{bm}}\right)^{1/3} - 1$$

where

D_{bd} = bulk density (dry)

D_{bm} = bulk density (moist)

PERCENT SILT AND CLAY

The amount of silt and clay is expressed as weight of material passing the No. 200 sieve (0.074 mm) as a percentage of the sample dry weight.

SWELL INDEX

The swell index is the ratio of the natural water content to the liquid limit of the soil (ref. 33). Both quantities are in percent so the swell index is dimensionless. This common term has several definitions and thus it should be clearly defined when used.

SECTION 4
IDENTIFICATION AND CLASSIFICATION SYSTEMS

INTRODUCTION

Currently identification and classification of expansive soils are either based on a direct measurement of swell potential or on correlation of simpler test results with swell potential measurements. These are referred to as direct and indirect techniques, respectively. Before using any soil classification system, engineers should understand the data base from which it was derived and establish its limitations; otherwise, poor reliability and lack of confidence in the system may result. Two types of identification and classification systems are discussed here: first, general classification systems which have evolved over many years and are based largely on correlation with actual performance; and second, those devised specifically for identification and classification of expansive soils. These systems are based on indirect and direct predictions of swell potential, as well as combinations, to arrive at a rating. Generally, these methods are based on the performance of certain types of structures in specific geographic areas. For example, the U.S. Bureau of Reclamation system was developed in the western United States on construction jobs involving hydraulic structures. Unfortunately, none of the expansive soil systems in the literature are based on experience with airport pavements.

Several important considerations in reviewing identification and classification techniques are the reliability, cost (equipment and time), and method of establishing the rating scheme. The techniques reported in the literature are empirical and are derived from experience with specific types of structures. Since the rating schemes are related to functional failure of specific structures (e.g., canals), use in evaluations for other types must be done cautiously. For example, the amount of expansion detrimental to a residential concrete slab in Southern California may not adversely affect an asphalt airport pavement in Ohio. In this effort, it was desirable to select a system of identifying and classifying expansive soils as to their influence on airport pavements throughout the United States. Although a high degree of reliability is desirable, the time and cost of testing are also important considerations in any type of construction. Therefore, it is desirable to know, quantitatively, the reliability of the system. With these objectives in mind, the methods of identification and classification were reviewed.

The key to all expansive-soil classification systems is the method of measuring swell potential, since soils are rated by their measured swell potential. Swell potential may be measured directly in a swell test or indirectly determined by correlation of other test results with swell test data. In almost every case swell potential is evaluated in the laboratory in a consolidation test device. This may yield swell potentials far different from those for in-situ soils. Thus, a reasonably good correlation between swell potential and other test results for purposes of classification is meaningless for prediction of in-situ heave. These procedures, however, do provide good indicators of the swell potential when the soil is subjected to the conditions used in the test.

GENERAL CLASSIFICATION SYSTEMS

The following general soil classification systems are used in the United States:

(1) Unified Soil Classification System

(2) AASHO Soil Classification System

(3) FAA Soil Classification System

In a comprehensive review of these systems, Yoder (ref. 46) stated that their ability to predict swell and, therefore, to classify soils as to their swell potential was derived from the consistency indexes on which the systems are based. The Federal Housing Administration has published a guide that correlates swell potential with Unified Soil Classification (ref. 47):

Category	Symbol	Soil Classification in Unified System
Little or no expansion	1	GW, GP, GM, SW, SP, SM
Moderate expansion	2	GC, SC, ML, MH
High volume change	3	CL, OL, CH, OH
No rating		PT

More problems are encountered with CL, OL, CH, and OH soils than with the others in housing construction (based on FHA experience). Briefly summarized:

(1) All clay and organic soils exhibit high volume change.
(2) All clayey gravels and sands and all silts exhibit moderate volume changes.
(3) All sands and gravels exhibit little or no expansion.

This procedure is not useful in the design process for airport pavement structures; however, it does provide an initial alert that further investigations may be required when fine-grained soils are encountered.

EXPANSIVE-SOIL CLASSIFICATION SYSTEMS

A review of the identification and classification systems for expansive soils that appear in the technical literature follows:

Kantey and Brink, 1952 (ref. 48)

Expansive soils are recognized by the following criteria:

$$\text{Liquid Limit} > 30\ \%$$
$$\text{Plasticity Index} > 12\ \%$$
$$\text{Linear Shrinkage} > 8\ \%$$

These criteria, which are based on the A-line of the plasticity chart developed by Casagrande, are used in the Unified Soil Classification System. The linear shrinkage criteria are included to detect those silt-clay and silty soils that are expansive.

Skempton, 1953 (ref. 42)

The activity of soils as determined by the plasticity index and % < 2 μm is as follows:

$$A = PI : \% < 2\ \mu m$$

Soils are rated low ($A < 0.75$), medium ($0.75 < A < 1.25$), or high ($A > 1.25$) with regard to potential expansion.

DeBruyn, et al., 1956 (ref. 28)

Potential expansion is rated in terms of specific clay surface area and the soil equilibrium moisture content at 85 percent relative humidity. The rating scheme is as follows:

Rating	Potential Expansion	Specific Surface,* m^2/g	Equilibrium Moisture Content at 85 % Relative Humidity, %
Good	Nonexpansive	< 70	< 3
Medium	Moderately Expansive	70-300	3-10
Bad	Expansive	300	> 10

*Reported as total surface (internal and external) determined by glycol retention.

Holtz and Gibbs, 1956 (ref. 43) and Holtz, 1959 (ref. 49)

The U.S. Bureau of Reclamation method was developed in the western United States on reclamation and water resources projects. Criteria were first presented in 1956 and then later modified by experience in 1959.

Data from Index Tests				
Colloid Content, % < 0.001 mm	PI, %	SL, %	Probable Expansion at 1 psi, dry→sat. Volume Change, %	Degree of Expansion
> 28	> 35	< 11	> 30	Very High
20-31	25-41	7-12	20-30	High
13-23	15-28	10-16	10-20	Medium
< 15	< 18	> 15	< 10	Low

These tests are performed on all soils classified CH and CL in the Unified Classification System that have a LL > 40 percent. Soils in the low category are not subject to special construction procedures; all others (medium to very high) are tested for quantitative volume change from the initial to anticipated final operating conditions of the structure to determine what special procedures are required during construction.

Altmeyer, 1956 (ref. 44)

Altmeyer reports successful use of the following system for identifying expansive soils:

SL, %	LS, %	Potential Expansion, %	Volume Change
< 10	> 8	> 1.5	Critical
10-12	5-8	0.5-1.5	Marginal
> 12	< 5	< 0.5	Noncritical

Linear shrinkage measured as moisture content is reduced from field moisture equivalent AASHO Method T93 to a lower limit beyond which no volume change occurs (shrinkage limit ASTM D427, AASHO T92)

Williams, 1958 (ref. 50)

Plasticity index and % < 2 μm are used as criteria and soils are placed into four categories as illustrated in figure 4.

McDowell, 1956 (ref. 51)

A curve relating plasticity index to volume change of the soil (fig. 5) is prepared based on construction experience with Texas highways. McDowell warns of the limitations of the graph and recommends its use as a rough estimate only.

Lambe, Federal Housing Administration, 1960 (refs. 38, 39, 52)

Lambe developed the FHA Soil Potential Volume Change (PVC) Meter for the Federal Housing Administration to provide a quick field identification of expansive soils. The device measures the swell pressure of compacted soil as it swells against a restraining force for 2 hr. The following categories have been established:

Category	PVC Rating
Noncritical	< 2
Marginal	2-4
Critical	4-6
Very Critical	> 6

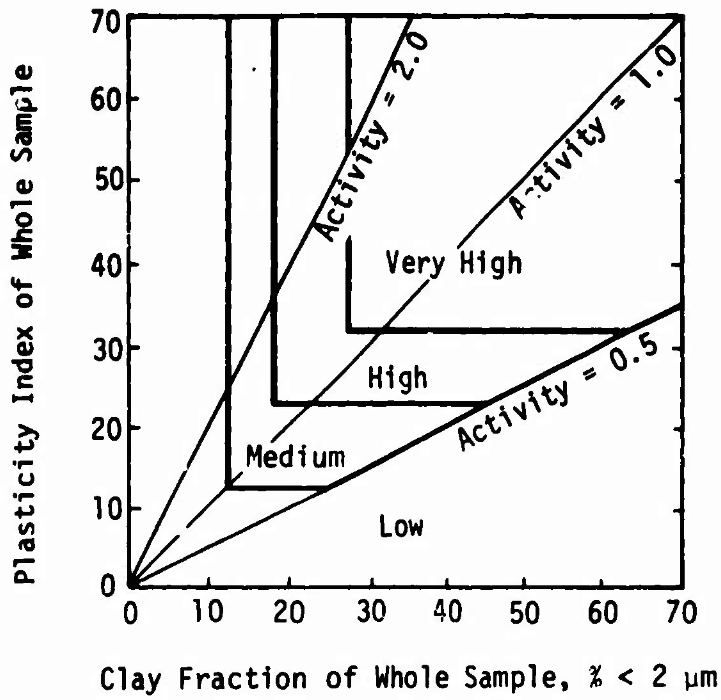


Figure 4. Determination of Potential Expansiveness of Soils [after Williams (ref. 50)]

Note: Specimens subjected to swell under average of 1-psi surcharge.

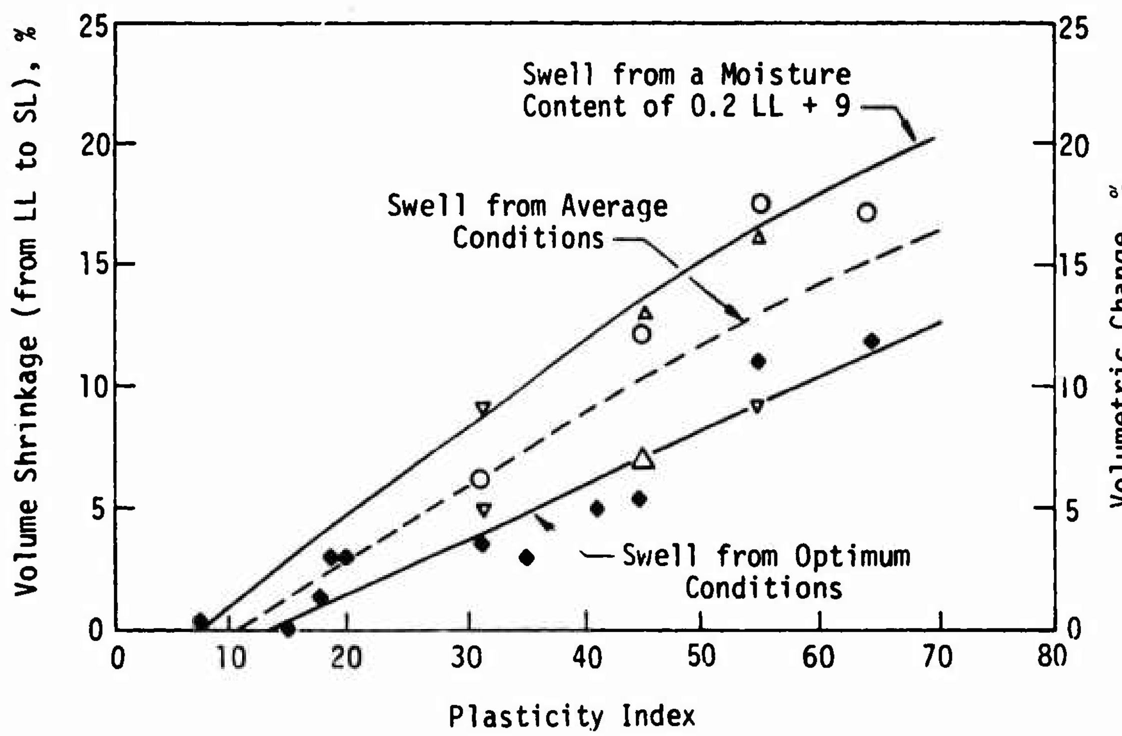


Figure 5. Interrelationship of Plasticity Index and Volume Shrinkage [after McDowell (ref. 51)]

Ladd and Lambe, 1961 (ref. 53)

A modification to the PVC Method consists of a combined PVC rating based on correlations with swell under 200 lb/ft^2, plasticity index, moisture content at 100 percent relative humidity, and the volume change occurring between the field moisture equivalent and the shrinkage limit. The system results in the following ratings:

PVC Rating	Degree of Expansion
< 2	Noncritical
2-4	Marginal
4-6	Critical
> 6	Very Critical

This test procedure involves a relatively large amount of laboratory work which the results do not seem to justify.

Seed, Woodward, and Lundgren, 1962 (ref. 37)

The swell potential of an expansive soil is defined from correlations of percent swell from oedometer tests of laboratory prepared and compacted samples (maximum dry density and optimum moisture content, AASHTO T-99) under 1-psi surcharge with % < 2 μm and soil activity. A statistical relationship is defined for swell potential in terms of clay content and activity and compared with measured volume change. The proposed classification for natural soils is shown in figure 6. With appropriate charts, the swell potential may be categorized as follows:

Swell Potential(S),	Degree of Expansion
0-1.5	Low
1.5-5	Medium
5-25	High
> 25	Very High

Ranganathan and Satyanarayana, 1965 (ref. 54)

This classification system is based on shrinkage index (liquid limit minus shrinkage limit) only. Data published by Seed, et al. (ref. 37) are used.

USBR Classification	SI, %	Probable Expansion at 1-psi (dry→sat.), %
Low	0-20	< 10
Medium	20-30	10-20
High	30-60	20-30
Very High	> 60	> 30

Chen, 1965 (ref. 55)

To simplify the USBR Method, a correlation is made between swell data and % < No. 200 sieve, liquid limit, and standard penetration resistance (ASTM D1586, AASHO T206).

Laboratory and Field Data

< No. 200 Sieve, %	LL, %	Standard Penetration, Blows/ft	Probable Expansion, %	Degree of Expansion
< 30	< 30	< 10	< 1	Low
30-60	30-40	10-20	1-5	Medium
60-95	40-60	20-30	3-10	High
> 95	> 60	> 30	> 10	Very High

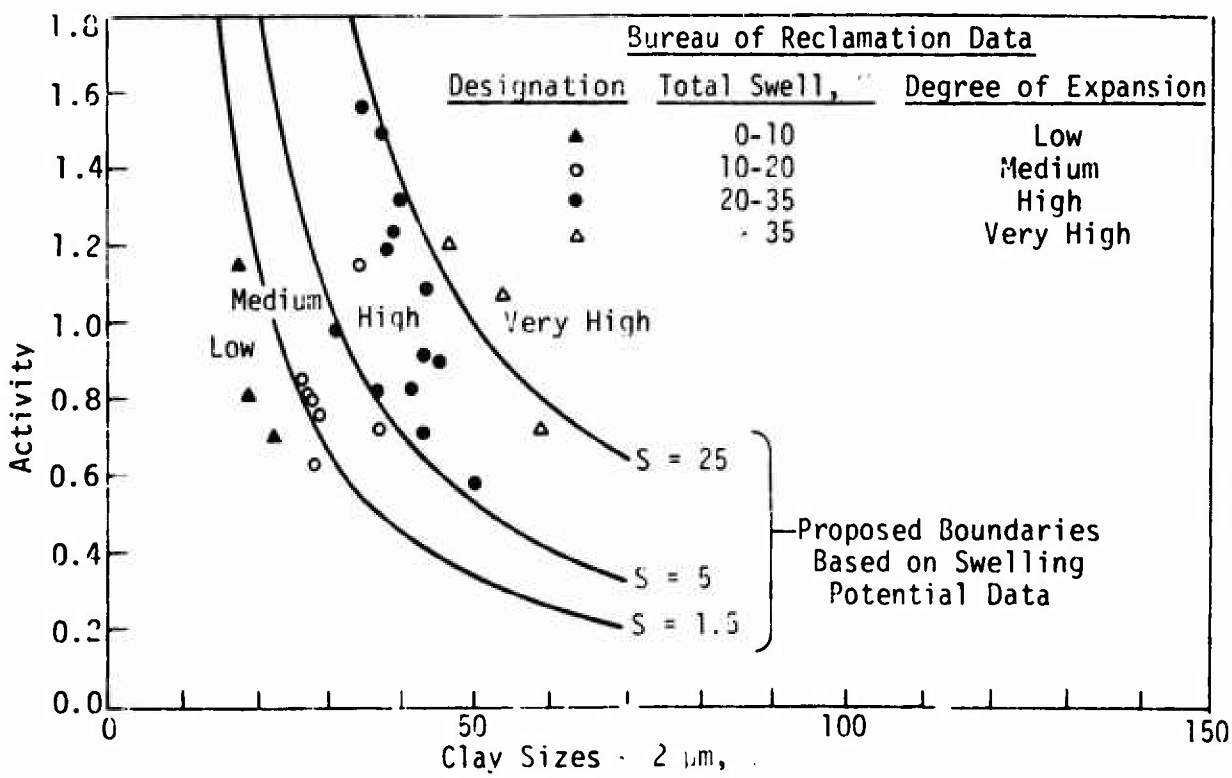


Figure 6. Applicability of Proposed Chart for Classification of Twenty-Seven Natural Soils [after Seed, et al. (ref. 37)]

Komornik and David, 1969 (ref. 35)

Through a statistical approach, an empirical equation is developed for the prediction of swell pressure in terms of liquid limit, natural dry density, and natural moisture content.

$$\log P = \bar{2}.132 + 0.0208(LL) + 0.000665(\gamma d) - 0.0269(w)$$

where

P = swell pressure, kg/cm^2
LL = liquid limit, %
γd = natural dry density, kg/m^3
w = natural moisture content, %

The data utilized involve a wide range of soil properties and thus lend credibility to the results. The system is used to predict swell pressure; it is not a classification system as such.

Packard, 1973 (ref. 56)

The guide shown below is used in the concrete airport pavement design manual of the Portland Cement Association. However, no procedures are given for handling expansive soils when they are present.

PI, % (ASTM D424)	Degree of Expansion	Approximate Swell, % (ASTM D1883)
0-10	Nonexpansive	< 2
10-20	Moderately Expansive	2-4
> 20	Highly Expansive	> 4

Vijayvergiya and Sullivan, 1973 (ref. 34)

With the liquid limit and dry unit weight of soil, the chart in figure 7 is used to predict swell. The ratings shown below are a guide for slab foundations on Beaumont Clay in Southeastern Texas.

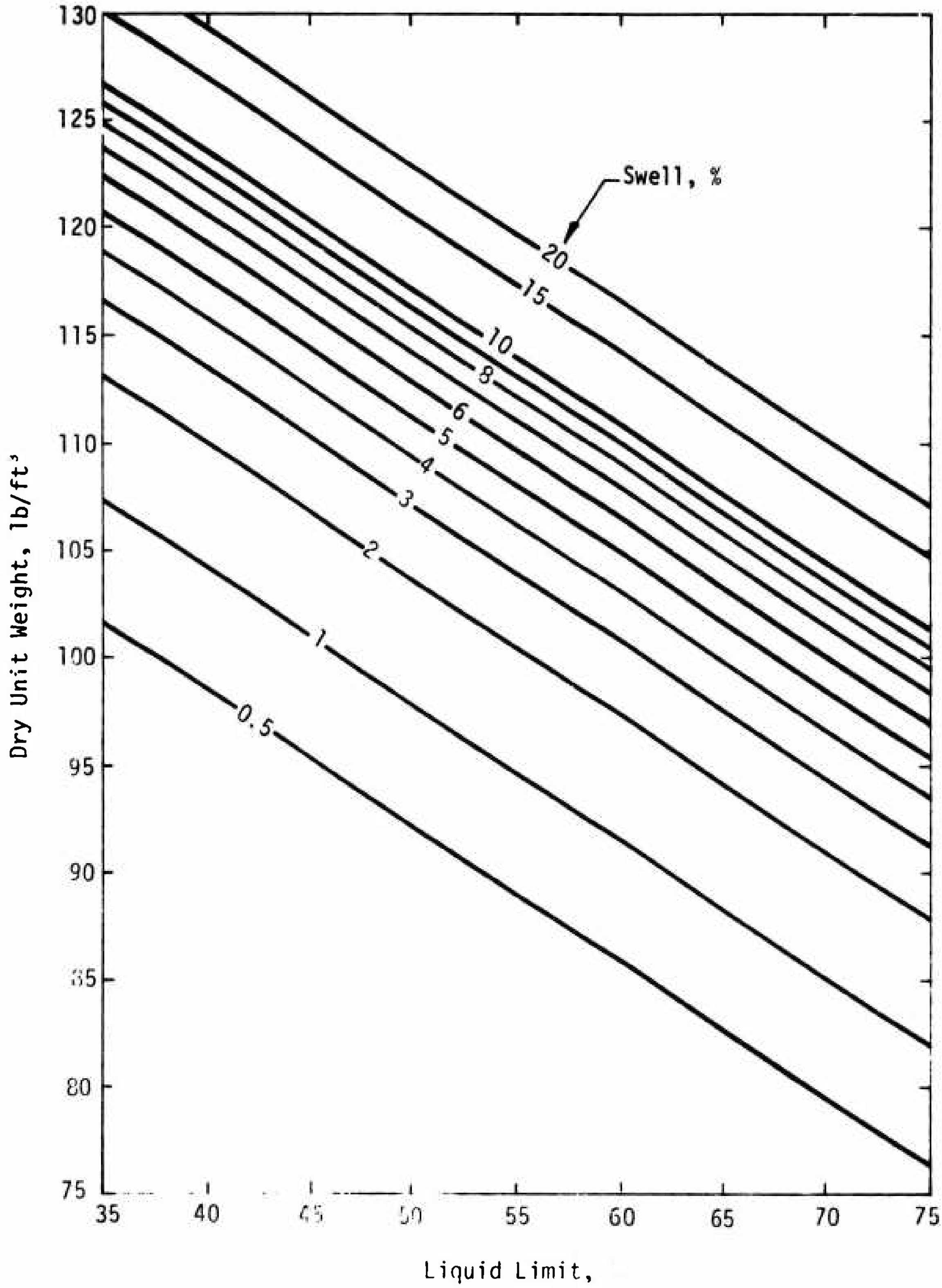


Figure 7. Correlation of Swell, Liquid Limit, and Dry Unit Weight [after Vijayvergiya and Sullivan (ref. 34)]

Swell Under 200 lb/ft², %		Heave Potential
< 1	Low	- No damage.
1-4	Moderate	- No damage with proper attention to design (< 1 in).
> 4	High	- Detailed investigation warranted.

Vijayvergiya and Ghazzaly, 1973 (refs. 4, 33)

This method defines a swell index for an expansive soil as the ratio of the natural water content to liquid limit and correlates it with one-dimensional swell (0.1-ton/ft^2 surcharge) and swell pressure data. Rather than a specific degree of expansion, limits of probable swell and swell pressure are defined as follows:

Swell Index	Probable Swell Pressure, ton/ft^2	Probable Swell, %
> 0.5	< 0.3	< 1
0.37-0.5	0.3-1.25	1-4
0.25-0.37	1.25-3.0	4-10
< 0.25	> 3.0	> 10

The method is based on data collected from a large number of samples. It is very simple to use; i.e., all that is required is the natural water content and the liquid limit. However, experience with regard to application of the method is limited.

Krazynski, 1973 (ref. 40)

A proposed test method for directly measuring expansion under a set of standard conditions is presented. The computed expansion index is then used to classify the soil for use beneath concrete slabs as follows:

Expansion Index	Potential Expansion
1-20	Very Low
21-50	Low
51-90	Medium
91-130	High
> 130	Very High

The expansion index is developed in conjunction with work on residential slabs and is intended only for classification purposes. There is little experience with this method, and no data for comparison with other methods are available.

Fernando, Smith, and Arulanandan, 1975 (ref. 57)

With the method described earlier by Arulanandan (ref. 31), a comparison is made between expansion index (ref. 40) and the magnitude of dielectric dispersion. The correlation is good for the soils tested and the authors establish the following criteria:

Magnitude of Dielectric Dispersion	Expansion Index	Potential Expansion
1-10	1-20	Very Low
11-25	21-50	Low
26-45	51-90	Medium
46-65	91-130	High
> 65	> 130	Very High

EVALUATION

Table 6 presents a summary of the criteria reviewed for most of the systems described.

The identification and classification systems presented in the literature reflect numerous attempts to correlate simpler test results with swell potential. However, it is impossible to select a suitable procedure based on the data presented. As part of this review, an evaluation and comparison were attempted, but because of the lack of continuity in the reported research, all systems

Table 6. Summary of Expansive-Soil Classification Systems

Reference	48			42	28		44			51		54	56	
Category	LL	PI	LS	A	w_{85}	SA	LS	SL	FS	FS	PI	SI	FS	PI
Nonexpansive	< 30	< 12	< 8		< 3	< 70								
Low				0.75			< 5	> 12	< 0.5	< 2	< 10	< 20	< 2	10
Medium				75-1.25	3-10	70-300	5-8	10-12	0.5-1.5	< 5	10-20	20-30	2-4	10-20
Expansive	> 30	> 12	> 8		> 10	> 300								
High				> 1.25			> 8	< 10	> 1.5	> 5	> 20	30-60		
Highly Expansive													> 4	> 20
Very High												> 60		

Reference	50		49				38	37		55			40	33	
Category	PI	% < 2 μm	FS	PI	SL	% < 1 μm	PVC	FS	PI	PR	LL	% < #200	EI	I_s	FS
Very Low													1-20		
Low	< 12	< 12	< 10	< 18	> 15	< 15	< 2	< 1.5	< 15	< 10	< 30	< 30	21-50	> 0.5	< 1
Medium	12-23	12-28	10-20	15-28	10-16	13-23	2-4	1.5-5	15-24	10-20	30-40	30-60	51-90	0.37-0.5	1-4
High	23-32	18-28	20-30	25-41	7-12	20-31	4-6	5-25	24-46	20-30	40-60	60-95	91-130	0.25-0.37	4-10
Very High	> 32	> 28	> 30	> 35	< 11	> 28	> 6	> 25	> 46	> 30	> 60	> 95	> 130	< 0.25	> 10

could not be included. The literature review, however, did produce several references (33, 34, 35, 37, 38) with sufficient data to make some comparisons.

PVC Rating and Plasticity Index

The Federal Housing Administration has published PVC swell index and plasticity index data for 151 soils from around the continental United States (ref. 38). A comparison of these data is shown in figure 8. The regression line computed for the data corresponds to the line relating swell index and PVC rating. It is obvious that based on these data the plasticity index is superior to the PVC rating for soil classification because of cost and routine availability.

Linear Shrinkage and Plasticity Index

The linear shrinkage of clay soils has been shown to be a better indicator of swell potential than the plasticity index (ref. 60). It has also been illustrated with a large amount of data that these two soil characteristics are closely related (ref. 61) and could be used interchangeably. Since the linear shrinkage test is quick and simple, it is a promising technique for evaluating swell potential. The literature indicates that this test may be superior to the plasticity index test because it involves a volume change mechanism. Although sufficient data are not available to compare and evaluate these tests, their combined use seems to be promising for qualitative indication of soil swell potential.

Multiple and Single Parameter Systems

Multiple tests are used in many simple identification and classification systems to classify clay soils. A comparison of four such systems is shown in figure 9. The data used were published by Komornik and David (ref. 35). Again, the plasticity index is superior since the multiple parameter systems involve considerably more laboratory work and offer no advantage in predicting swell potential.

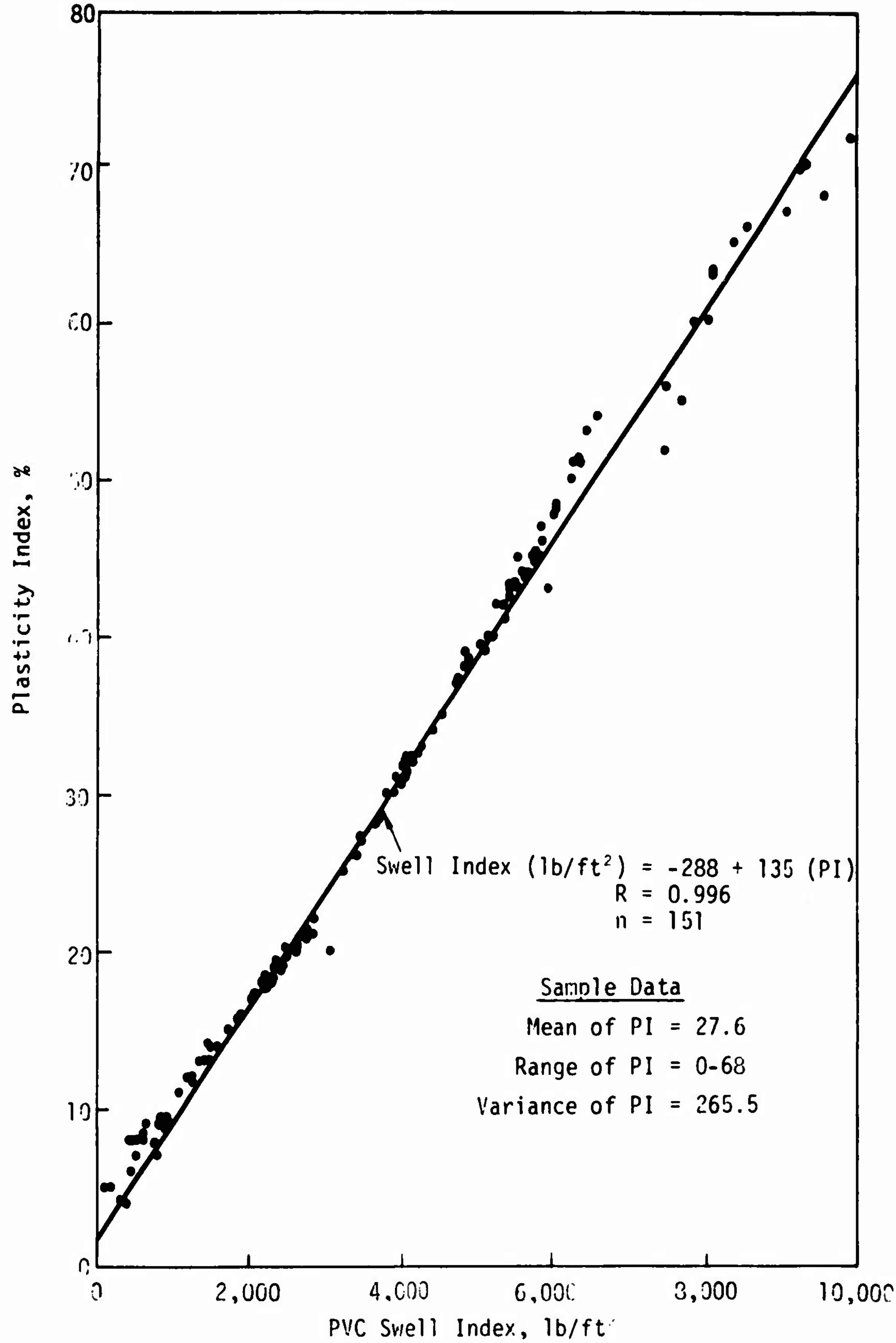


Figure 8. Relationship Between Plasticity Index and PVC Swell Index

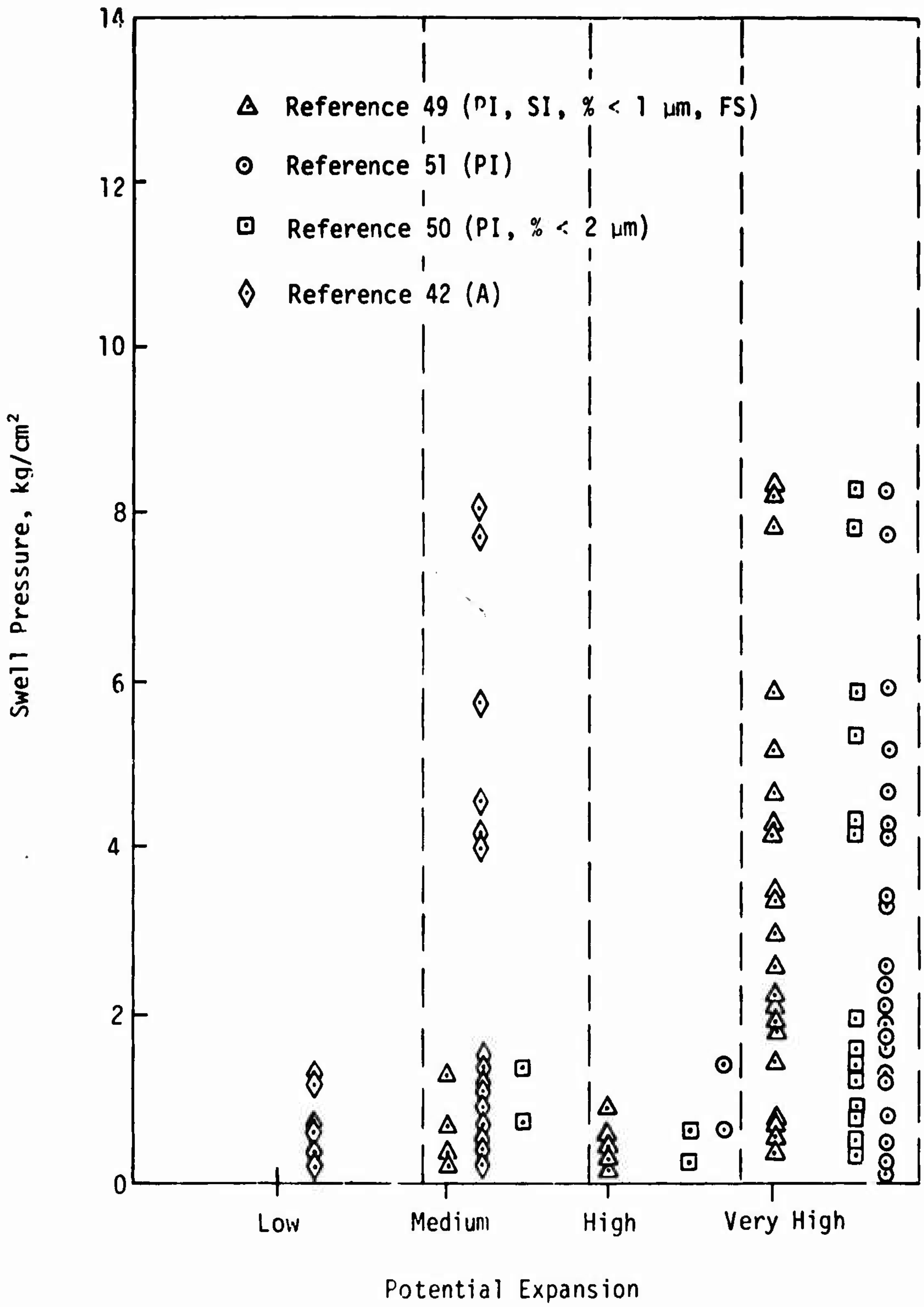


Figure 9. Comparison of Multiple and Single Parameter Classification Systems

Statistical Comparison

A further study of the reliability of predictions was made. Several systems presented in the literature or synthesized from published data were compared (table 7). To provide a measure of the reliability of each method, predicted values were calculated and compared to measured values of swell potential by linear regression analysis. The degree of correlation is represented by the correlation coefficient, R. To determine the effect of soil variability, the mean, $\overline{x}$, and the variance, σ^2, of the sample plasticity index were determined for all data groups studied. The distribution of the sample plasticity index was also determined after it was discovered that some of the systems in the literature were derived from data for which the plasticity index was quite low. Most engineering problems on expansive soils occur on soils with a high plasticity index (> 30 %). The correlation coefficient for some samples was compared to that for the same data without the data points with plasticity indexes less than 20 percent (table 8). From this statistical analysis, the following facts are evident:

(1) Method 1 is not a reliable predictor of swell potential for soils with a plasticity index greater than 20 percent (i.e., highly expansive soils).

(2) Method 2 provides widely differing correlation coefficients between predicted and measured values of swell potential. These results are inconclusive, and lack of data in the literature prevents further evaluation.

(3) Method 3 results in fairly consistent results, with the correlation coefficient dropping as the sample variance increases.

(4) Method 4 predicts swell potential in much the same manner as method 3, with a slightly wider range of correlation coefficients.

(5) Method 5 gives the best correlation coefficient (0.60), considering the distribution and variance of the data on which it was based.

Thus the most reliable simple technique for predicting swell potential is that provided by Komornik and David (ref. 35); correlation is based on swell

Table 7. Swell Potential Prediction Methods Used for Statistical Comparison

Method	Indicator	Prediction Equation
1 (ref. 37)	Plasticity Index, %	Swell, % = $60k(PI)^{2.44}$ $k = 3.6 \times 10^{-5}$
2 (ref. 54)	Shrinkage Index, %	Equations Based on Regression Swell, % = $0.55(SI)^{0.97}$ Swell Pressure, kg/cm² $= 0.52(SI)^{0.13}$
3 (ref. 33)	Swell Index $\left(I_s = \dfrac{\text{natural water content, \%}}{\text{liquid limit, \%}}\right)$	Equation Based on Regression Swell, % = $0.49(I_s)^{-1.9}$
4 (ref. 34)	Liquid Limit (LL), % Dry Density (γd), lb/ft³	Prediction Based on Table Relating LL, γd, and Swell
5 (ref. 35)	Liquid Limit (LL), % Dry Density (γd), kg/m³ Initial Moisture Content (w), %	$\log P = a + b(LL) + c(\gamma d) + d(w)$ where P = swell pressure, kg/cm² $a = \bar{2}.132$ $b = 0.0208$ $c = 0.000665$ $d = -0.0269$

pressure as a measure of the swell potential. There is no simple indicator presented in the literature, with a correlation coefficient greater than 0.60, that can reliably classify a wide variety of clay soils according to their expansion potential.

Swell Percentage and Swell Pressure

Both swell percentage and swell pressure are used throughout the literature for quantifying the swell potential of the soil. These two measures, although related, are not necessarily interchangeable. For one group of tests (ref. 34), these two measures of swell potential were compared by linear regression. The swell under a 1-psi surcharge was predicted as follows:

$$\text{Swell} = 1.12 + 2.16 \times \text{Swell Pressure}$$

Table 8. Results of Statistical Analysis of Swell Potential Prediction Methods

Method	Sample Data (Plasticity Index)							Correlation Coefficient (R)	Remarks
	n	$\bar{x}$	σ^2	Distribution, %					
				> 60	60-40	40-20	< 20		
1 (PI)	21	23	278	9	5	25	61	0.98	Pure Clays
	38	27	195	0	22	36	42	0.68	Natural Soils
	184	34	95	0	28	68	4	0.18	Beaumont Clay
	24	35	125	0	38	62	0	0.45	Natural Soils (PI $\geq$ 20 Only)
2 (SI)	125	42	254	16	39	35	10	0.06	Natural Soils
	38	27	195	0	22	36	42	0.80	Natural Soils
	24	35	125	0	38	62	0	0.74	Natural Soils (PI $\geq$ 20 Only)
3 (I_s)	265	35	87	0	23	69	3	-0.63	Natural Soils
	125	42	254	16	39	35	10	-0.56	Natural Soils
4 (LL, γ_d)	184	34	95	0	28	68	4	0.68	Beaumont Clay
	125	42	254	16	39	35	10	0.51	Natural Soils
5 (LL, γ_d, w)	125	42	254	16	39	35	10	0.60	Natural Soils

For this equation and the data in reference 34, the correlation coefficient is 0.84 and the standard error is 1.64. It should be recognized that for each set of samples, the reliability will vary. In design of airport pavements, swell is the more meaningful measure of swell potential. A thick airport pavement (e.g., 36 in of concrete and 36 in of stabilized base) places on the subgrade a pressure that is far below the measured swell pressures of expansive soils (refs. 34, 35, 38). Seed (refs. 58, 59) presents data that demonstrate volume changes of a fraction of a percent greatly reduce swell pressures.

SECTION 5
PREDICTION OF IN-SITU HEAVE

Since many prediction methods in use today involve direct measurement with a consolidometer, this device and its limitations were studied. Factors important in evaluating in-situ heave were reviewed and the methods currently available for predicting heave were analyzed within this framework.

CONSOLIDOMETER TESTING

Evaluation of soil volume changes by consolidometer testing is the most widely used method for predicting in-situ heave (appendix A). Test methods influence the nature of the results obtained and this must be considered in evaluating the methods. The consolidometer was originally designed by Terzaghi to simulate field settlement in the laboratory (ref. 59); it has also proven useful in the study of swelling clays (refs. 8, 43, 62, 63).

Consolidometer tests are usually of two types--free swell and constant volume. In a free swell test the soil sample takes on water, either by submersion or capillary action, and swells under a token load (e.g., 1 psi) until no further volume change occurs in a specified amount of time. When equilibrium is reached, the load is increased until the sample is compressed to its original volume; the pressure required to accomplish this is one measure of the swell pressure of the soil. In a constant volume test, the load is increased to prevent expansion as the sample is allowed access to water. When no change in pressure is required in a specified time to retain the same volume, the test is ended; this load is the swell pressure. The sample is usually unloaded to establish the rebound or expansion curve. These two test results are often taken as the boundaries of soil behavior, with actual in-situ behavior assumed to be somewhere between these data as plotted on a void-ratio-versus-log-of-pressure curve. Some investigators have reported reasonably accurate predictions of swell, but the rate of swell cannot be determined by these tests since the moisture gradients produced in the consolidometer are drastically different from those in in-situ conditions (ref. 59).

Several sources of significant error should be considered in performing the tests and interpreting the results (ref. 59). For example:

(1) Friction in the measuring apparatus is significant at low loads (< 0.5 kg/cm^2 or 7.11 psi). At a pressure of 0.01 kg/cm^2 (0.14 psi), the load applied to the sample has been found to be in error by 100 percent for one type of consolidometer.

(2) Compression characteristics of the apparatus are important. Consolidometers should be tested to establish the compressibility of the loading frame and volume change measuring apparatus. Calibration curves should be prepared and no components should be switched without verifying the compression characteristics of the apparatus. The compression characteristics do not vary significantly from cycle to cycle.

(3) Porous discs produce a high degree of compressibility. Smoothly grooved thick stones are most desirable.

(4) Filter paper used between samples and porous stones produces significant compressibility in swell tests on Bearpaw Shale (ref. 64).

(5) Sample seating against the porous discs is difficult to evaluate. As the load increases the significance is reduced.

(6) In measuring the swell pressure, very small volume changes result in large differences in measured swell pressure. (All sources of volume change must be considered in arriving at a measured value.)

(7) Lateral confinement of the sample may not duplicate field conditions; a correction factor may be required.

Since the results of consolidometer tests are influenced by the above factors, these sources of error must be considered; if they are not, low estimates of swell usually result. Caution must be exercised in using swell-test results for design data.

FACTORS INFLUENCING IN-SITU HEAVE (refs. 6, 9, 40, 65)

Clay Thickness

In any study of a construction site, the thickness of all soil layers must be determined. The location of each layer with respect to the completed structure is important in evaluating the effects of changes in overburden load, availability of water, and drainage of surface water. When an expansive soil layer is identified it may be removed and replaced with a better material, if it is thin enough and suitable replacement material is available. Thicker layers may require alternate designs to accommodate or reduce the soil surface movement.

Water Table Depth

When soil is sealed off from the atmosphere by a pavement structure, it reaches an equilibrium moisture condition. The depth of the water table is important in determining this equilibrium condition. The existence of perched or temporary water tables must be taken into consideration in the study of in-situ heave.

Initial Moisture Condition

One of the primary factors in the study of in-situ heave is the actual initial moisture condition of the soil. This determines the point from which the system moves toward an equilibrium moisture condition. Depending on the specific equilibrium conditions, the initial moisture conditions determine whether shrinkage or swelling actually occurs. The effects of removing vegetation must be considered. Changes in the moisture condition of the soil between clearing and grubbing and the placing of structures over the soil must be considered in arriving at the actual initial moisture condition of the soil when it is sealed off by construction.

Soil Structure

Soil structure is a property of the soil that might be confusing if it is discussed without being placed into a frame of reference. Four such reference frames are thus established:

(1) Clay Particle Structure - the types of clay mineral present are identified by unique crystal structures (i.e., montmorillonite, kaolinite, or illite) and ionic substitutions in these structures. The behavior of each configuration is affected by the particle surface charge, ionic substitutions, surface area, and bonding which are characteristic of that particular clay mineral structure.

(2) Clay Particle Arrangement - The arrangement of clay particles in the soil mass has an influence on soil behavior. This has been recognized for sometime and is illustrated in figure 10 (ref. 66). This arrangement is determined by origin, geologic history, and local conditions.

(3) Clod Structure - As the clay mass interacts with the local environment, a higher-order structure is formed. The properties of this structure depend on the factors which formed the soil. When a clay layer is exposed to cycles of loading and unloading, or wetting and drying, a pattern of fissures (cracks) forms throughout the soil. As these seams are opened and closed, the clod structure between them takes on a distinct character, unlike either that of the bulk soil or the grain structure produced by the clay particle arrangement, which influences the behavior of the soil. The only study in the literature on this aspect deals with the field of agriculture (ref. 45). Little information is available on the influence of clod properties on the engineering behavior of natural soils.

(4) Bulk Structure - The overall macrostructure of the soil (fissures, voids, etc.) contributes to the bulk structure of a clay soil layer. Another aspect of bulk structure in clay soils is the formation of gilgai (fig. 11). This characteristic rolling surface structure, a natural result of soil/environment interaction, extends to depths greater than those disturbed in normal pavement construction. The reflection of this natural structure through the pavement produces roughness and thus pavement repair or replacement is required.

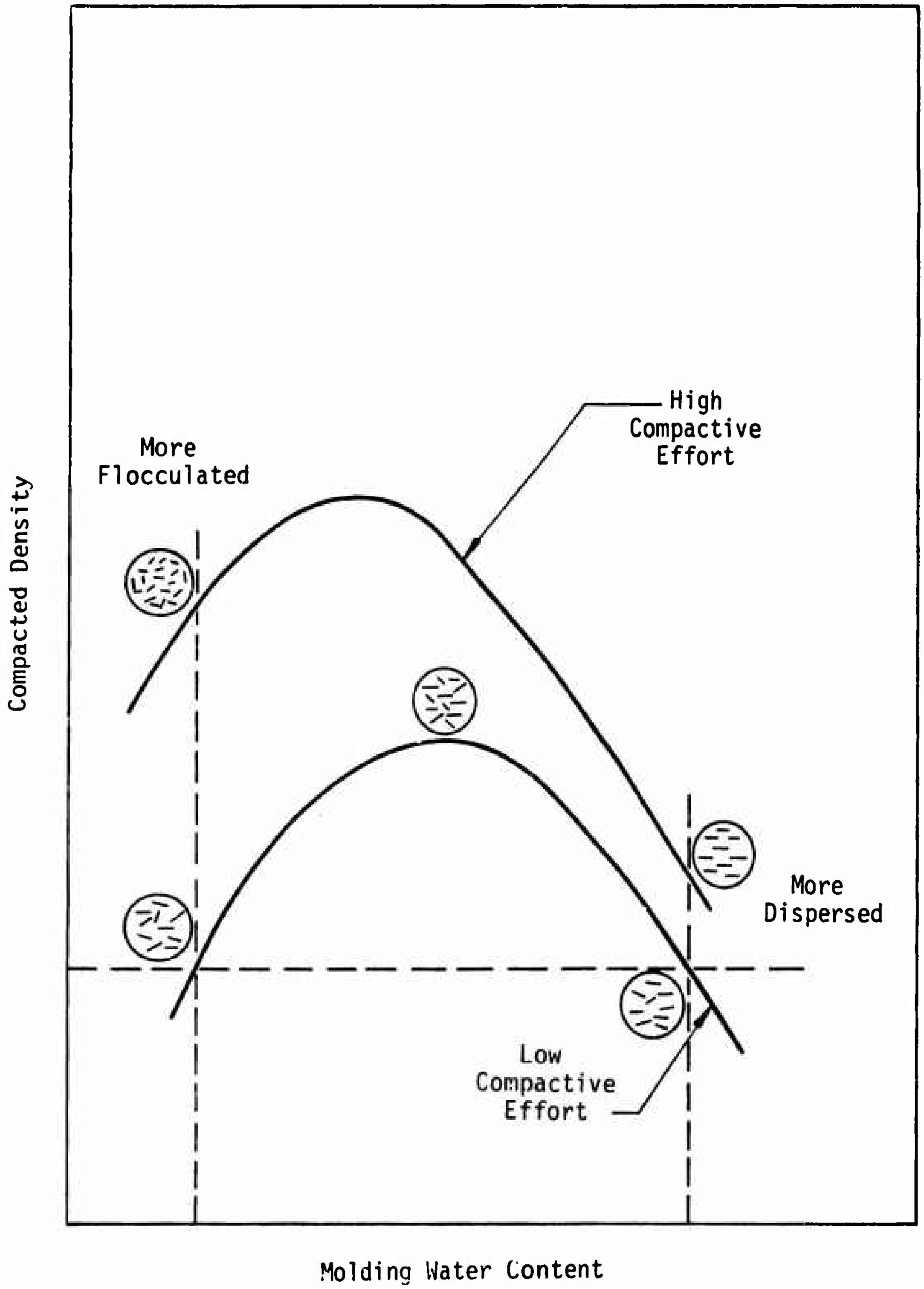


Figure 10. Effects of Placement Conditions on Structure [after Lambe (ref. 66)]

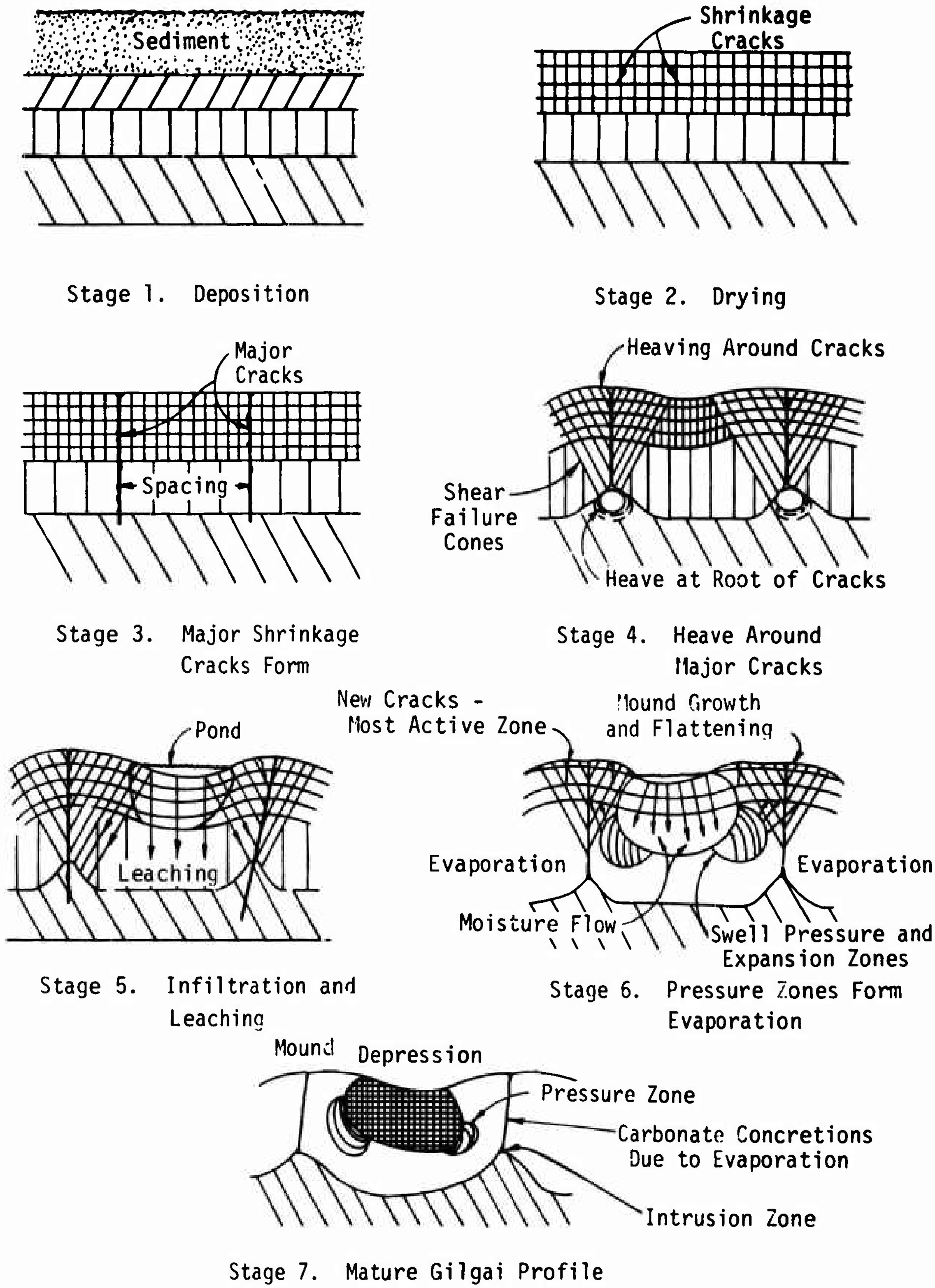


Figure 11. Stages of Genesis of Normal Gilgai [after Lytton et al. (ref. 67)]

Gilgai is a relatively new subject and little engineering information is available.

Any procedure for the estimation of soil behavior should be directed toward determining soil behavior within one of the above reference frames. Most tests in use today measure the effects of clay-particle arrangements that are produced in the laboratory by remolding soils. The designer must recognize the practical significance and limitations of the data from which his design must be derived. Most behavior in the field is controlled by the clod and bulk properties of the soil. Certainly today there is a gap in understanding the bulk behavior of soils and in adequately measuring the engineering bulk properties. This is one area that requires careful study in the process of soils evaluation for pavement design.

Initial Stress Condition

The initial stress condition is the loading applied to a soil prior to construction. Usually this is made up of the overburden loads, both vertical and horizontal components, and the environmentally induced stresses; some contribution may be made by vegetation such as trees. It is important to distinguish between loading conditions at the time construction begins (clearing and grubbing) and the conditions at the time the structure is placed (earthwork and paving). The importance of any difference in these conditions must be evaluated in the design process in order to establish the conditions for evaluating the behavior of the soil.

Final Moisture Conditions

The prediction of final moisture conditions under a structure has been studied for over two decades. Many designers assume a saturated condition, believing that to be the *worst case*. In many situations this is overdesigning since the moisture content never reaches saturation; in other circumstances, the soil could actually shrink in attaining equilibrium with its new environment. Several reasonable methods for predicting the final equilibrium moisture content are proposed in the literature. Appendix C outlines a recommended procedure for such predictions. (Also see appendix D.)

Final Stress Condition

The final stress condition is evaluated in the same way as the initial stress condition. Principal considerations are the structure loads, the loads associated with cut or fill sections of the pavement, and any environmentally induced stresses.

Load/Water Content/Volume Relationship

Determination of the soil response to changes in load and water content is the real key to the prediction of in-situ heave. In most techniques for the prediction of in-situ heave, the soil response to changes in load and moisture content is measured. The main differences are in the assumptions made in determining the percentage of swell and the procedures for establishing the initial and final moisture conditions. The standard consolidometer is the most reliable instrument available for field use in evaluation of soil response to environmental changes (refs. 24, 63, 68); it is used in many evaluation procedures. Several techniques in which the loads and soil suction* are independently controlled and made to duplicate measured or predicted in-situ values have been published (refs. 69, 70, 71, 72). Clearly, the closer the sample and test conditions duplicate the in-situ soil, the better the estimation of heave will be. Careful study of the test conditions is necessary in every case.

Rate of Volume Change

The rate at which volume changes occur may be an important factor in determining the soil/structure interaction. Primarily, the rate of swell depends on the soil permeability. An initially dry, fissured soil swells rapidly at first as water moves through the existing shrinkage cracks. As these passages are closed by swelling, the permeability is drastically reduced and a much slower rate of swell results. The moisture gradients in the field are very different from those in any known laboratory test (ref. 59). There is presently no means of accurately studying field rates of heave in the laboratory.

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*Appendix D elaborates on *soil suction* and related terminology.

Seasonal Variations

Since soil near the surface interacts with the local environment, the moisture conditions are constantly changing. This seasonal influence is apparent to a definite depth (usually 5 to 10 ft). When a pavement is constructed over the soil, the response of the soil to changes in the local environment is reduced or eliminated. Cracks in a pavement or a highly permeable pavement will certainly allow water from the environment to penetrate to the underlying soil. Experience reported in the literature (refs. 9, 68, 73, 74, 75) indicates that pavement subgrades tend to approach an equilibrium moisture condition. The soil below a pavement and within the zone of seasonal variation may shrink, swell, or remain unchanged depending on the initial moisture condition and the equilibrium moisture condition. The initial condition of the soil in the zone of seasonal variation must be known for rational design over expansive soils.

PREDICTION METHODS

Although great effort has been devoted to the prediction of in-situ heave in expansive soils, little progress has been made in recent years so far as implementable procedures are concerned (table 9). The one-dimensional swell test with a consolidometer type apparatus is the most widely used and reliable procedure. However, no standard method exists and there are almost as many methods reported in the literature as there are researchers. Only recently have serious attempts at standardization appeared in the literature (refs. 40, 76). The most promising method in the current literature (Australian Method) is by no means implementable. However, the merits of this technique do justify a comprehensive development effort. The Holtz Method (ref. 76) is the best implementable technique available at this time. (See appendix A.)

Consolidometer Methods

Numerous methods of measuring swell potential directly in a consolidometer are reported in the literature. Figure 12 illustrates the type of data obtained from these tests. For example, a specimen may be loaded to its in-situ

Table 9. Summary of In-Situ Heave Prediction Methods

Factors Involved in Heave	Direct Model (77)	Double Oedometer (65, 78)	Sullivan and McClelland (79)	Sampson et al. (80)	Mississippi Method (81-84)	Salas and Serratosa (20)	Noble (7)	Navy (85)	USBR (63)	Volumeter (87)	Holtz (76)	Richards (88)	McDowell (51)	Van Der Merwe (89)	Australian (69)	Simple Oedometer (86)
	Method (reference)															
Clay Thickness	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
Water Table Depth	✓	✓	✓	x	x	✓	x	x	x	x	x	✓	✓	x	✓	✓
Initial Moisture	✓	x	✓	✓	✓	x	✓	✓	✓	x	✓	✓	✓	x	✓	✓
Soil Structure																
Clay Particles	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	x	✓	✓	✓	✓
Particle Arrangement	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	x	✓	x	✓	✓
Clod Structure	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	x	✓	x	✓	✓
Bulk Structure	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Initial Stress	✓	✓	✓	✓	✓	✓	x	✓	✓	✓	✓	x	✓	x	✓	✓
Final Moisture	x	✓	x	x	x	✓	x	x	x	x	x	✓	x	x	✓	✓
Final Stresses	✓	✓	✓	✓	✓	✓	✓	✓	✓	x	✓	x	✓	x	✓	✓
Load/Moisture/Volume Relationship	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	x	✓	x	✓	✓
Rate of Volume Change	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Seasonal Variations	x	x	x	x	x	x	x	x	x	x	x	x	x	x	✓	✓

Note: ✓ = considered in the prediction; x = not considered in the prediction.

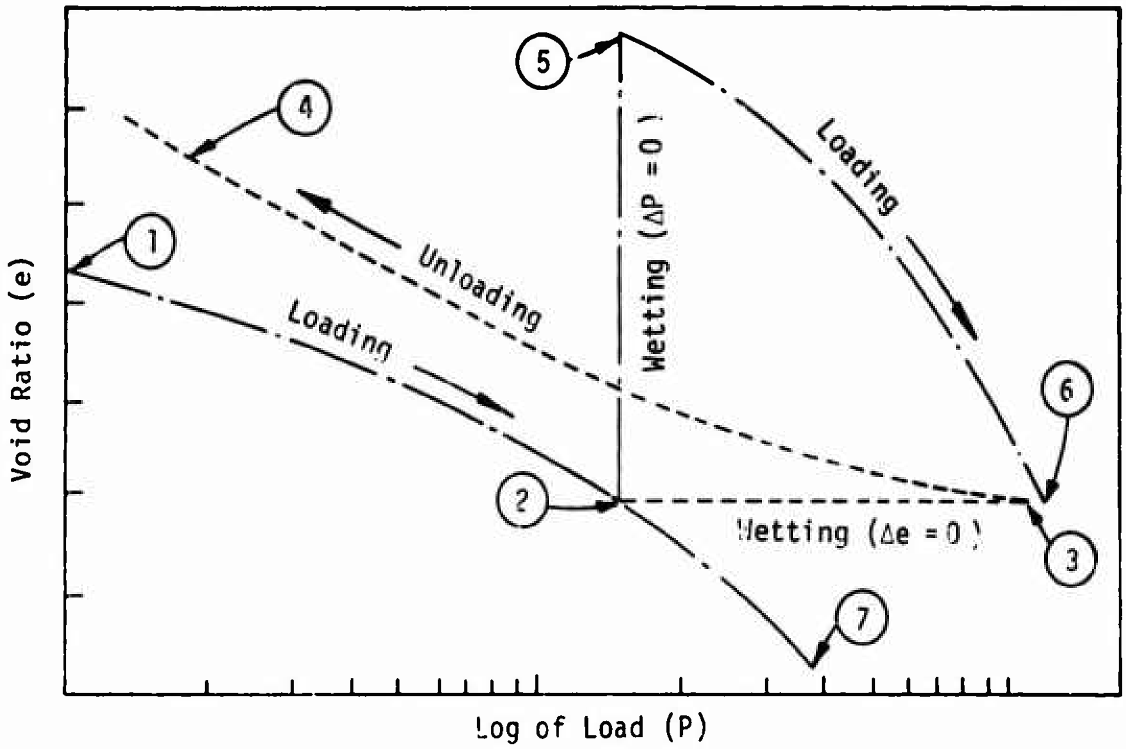

Figure 12. Types of Swell Test Data

overburden pressure, (1 to 2). Then it is subjected to a change in moisture condition and maintained at constant volume until equilibrium is reached (3). This pressure is called the *swell pressure* (no-volume-change test). The pressure is then released to a small arbitrarily selected load or to a specific design load, (3 to 4). Another test procedure loads the soil to the overburden pressure (2), allows it to swell under constant load to (5), and loads the sample to the original void ratio (6). With this kind of test procedure, swell may be calculated as follows:

$$S = \frac{\Delta e}{1 - e_1} (\Delta H)$$

where

Δe = change in void ratio (final to initial)

e_1 = original void ratio

ΔH = thickness of soil layer

The curve (1 to 7) illustrates a test in which a soil is loaded to the initial overburden pressure, (7), unloaded to a final overburden pressure, (2), and permitted access to water; then the swell is determined (analysis of cut sections). In each situation, events follow a specific sequence. The closer these duplicate in-situ conditions, the better the prediction of soil behavior. Those methods reported in the literature in which some form of the consolidometer test is used are as follows:

(1) Direct Model Method, Texas Highway Department (ref. 77)
(2) Jennings and Knight's Double Oedometer Test (refs. 65, 78)
(3) Sullivan and McClelland's Method (ref. 79)
(4) Sampson, Schuster, and Budge's Method (ref. 80)
(5) Mississippi Method (refs. 81, 82, 83, 84)
(6) Salas and Serratosa's Method (ref. 20)
(7) Noble's Method (ref. 7'
(8) Navy Method (ref. 85)
(9) Simple Oedometer Method (ref. 86)
(10) USBR Method (ref. 63)
(11) Volumeter (ref. 87)
(12) Holtz's Method (ref. 76)

Each of these methods has some similarity with the others as well as some differences. Some involve multiple samples (e.g., 2 and 10); others do not. No one method is clearly better than another for airport pavement construction. Any procedure that is used must be adapted to a particular situation and an effort must be made to simulate these actual in-situ conditions. At best these methods provide estimates of questionable accuracy unless they are used with considerable experience with the specific soil and climatic conditions under study (refs. 40, 63).

Predictions of in-situ heave are made by testing each soil layer in the system to determine its response to changes in load and moisture. The individual layers may represent different types of soil, the same soil in different moisture conditions, or the same soil at different densities. Once each layer is identified and a swell percentage is assigned by testing in the consolidometer, the calculation of surface heave is straightforward as shown below.

Depth, ft	Thickness of Soil Layer, ft	Overburden Pressure,* lb/ft^2	Swell, %	Vertical Rise Due to Layer, in	Vertical Rise at Layer Surface, in
0-2	2	125	8	1.92	7.68
2-4	2	375	4	0.96	5.76
4-10	6	875	3	2.16	4.80
10-12	2	1375	3	0.72	2.64
12-20	8	2000	2	1.92	1.92
20-24	4	2750	0	0	0
Bedrock	-	-	-	-	-

In this illustration, the predicted surface heave is 7.68 in. The designer should carefully evaluate the procedures used in establishing the initial moisture conditions and load as well as the final moisture conditions and load used in the tests. These parameters and their relationship to in-situ conditions will determine to a large degree the accuracy of the prediction. With some methods a lateral restraint factor may be used to reduce swell values for certain soils (e.g., particularly highly fissured clays). The amount of testing required for this type of analysis can be great in terms of time and money. The variability of the soil system must be studied in order to arrive at the amount of testing required to adequately evaluate the swell potential. Once these data are available, the effect of soil removal, stabilization, compaction, etc. may be evaluated quantitatively, provided swell data are also gathered for the stabilized and/or compacted materials.

Richards' Method (ref. 88)

Using curves of moisture content versus matrix suction plotted from measured values, Richards predicts moisture content changes as soils reach their

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*Average at center of layer, based on density of overlying material and structural load. A density of 125 lb/ft^3 was assumed for all soils in this illustration.

equilibrium moisture conditions under pavements. Assuming the volume change of the soil is equal to the volume of water taken up, he gives

$$\frac{\Delta V}{V} = \frac{\Delta w G_s}{100 + w_o G_s}$$

and

$$\frac{\Delta L}{L} = \frac{1 \Delta V}{3V} = \frac{\Delta H}{H}$$

where

w_o = initial water content, %
Δw = change in water content (w_o - w_f), %
G_s = specific gravity of solids (approximately 2.70)
V,L,H = volume, length, height, respectively

With empirical relationships developed for Australian conditions, the final equilibrium moisture content under a pavement is predicted. With this relationship, w_f may be predicted from the moisture/suction curves previously determined for each soil layer. A sample calculation is shown in table 10.

McDowell's PVR Method (ref. 51)

An undisturbed sample of each soil layer in the system is introduced into a triaxial cell and the sample is allowed to absorb water under a small (2 psi) lateral pressure. After the sample has absorbed water for 15 days (or a number of days equal to the plasticity index, if it is greater) the volume change, ΔV, is converted to a linear vertical rise, ΔL, from an empirical chart. With another empirical chart, ΔL is reduced according to the stress imposed by the overburden load. The remaining percentage of vertical rise is then summed for each increment. An example of this procedure is shown in table 11. Columns show: (1) the increments of overburden loads into which the system is divided, (2) the average overburden load, (3) the volume change measured under the existing overburden pressure, (4) the linear swell corresponding to the volume change, (5) the thickness of each increment in the system, and (6) the conversion of column (4) to a percent and the total vertical rise at the surface.

Table 10. Sample Calculation of Soil Movement [after Richards (ref. 88)]

From Driest Condition to Equilibrium Profile

L, cm	Initial Suction (h_o), cm H_2O	Initial Effective Stress ($\bar{\sigma}_o$), cm H_2O	Final Suction (h_f), cm H_2O	Final Effective Stress ($\bar{\sigma}_f$), cm H_2O	w_o, %	Δw, %	ΔL, cm
0-10	90,000	90,000	1400	1400	11.5	15.7	1.08
10-20	45,000	45,000	1400	1400	14.6	12.6	0.82
20-30	10,000	10,000	1300	1300	21.0	6.4	0.37
30-40	5,000	5,000	1300	1300	23.3	4.1	0.23
40-60	3,200	3,200	1300	1300	24.6	2.8	0.30
60-80	2,500	2,500	1300	1300	25.1	2.3	0.25
80-100	1,800	1,800	1300	1300	26.4	1.0	0.11
100-120	1,500	1,500	1300	1300	27.1	0.3	0.03
120-140	1,400	1,400	1300	1300	27.2	0.2	0.02
				Surface Movement 3.21 cm			

From Driest to Wettest Condition (i.e., Seasonal Movement)

L, cm	Initial Suction (h_o), cm H_2O	Initial Effective Stress ($\bar{\sigma}_o$), cm H_2O	Final Suction (h_f), cm H_2O	Final Effective Stress ($\bar{\sigma}_f$), cm H_2O	w_o, %	Δw, %	ΔL, cm
0-10	90,000	90,000	800	800	11.5	16.7	1.15
10-20	45,000	45,000	700	700	14.6	13.8	0.90
20-30	10,000	10,000	600	600	21.0	7.8	0.45
30-40	5,000	5,000	520	520	23.3	5.8	0.33
40-60	3,200	3,200	580	580	24.6	4.3	0.46
60-80	2,500	2,500	870	870	25.1	2.9	0.32
80-100	1,800	1,800	1120	1120	26.4	1.1	0.13
100-120	1,500	1,500	1350	1350	27.1	-	-
120-140	1,400	1,400	1300	1300	27.2	-	-
				Surface Movement 3.73 cm			

Van Der Merwe's Method (ref. 89)

Another empirical approach involves classifying the soil by the Williams' Method (ref. 50) into the categories shown below.

Williams' Criteria PI, %	Williams' Criteria Clay, %	Potential Expansiveness*	Unit Heave, in/ft
< 12	< 12	Low	0
12-24	12-18	Medium	0.25
24-32	18-28	High	0.50
> 32	> 28	Very High	1.00

Each category is assigned a unit heave value in inches of heave per foot of soil layer thickness. An empirical relationship for the change in potential

Table 11. Conversion of Volume Change to Potential Vertical Rise [after McDowell (ref. 51)]

Load, psi (1)	Average Load, psi (2)	Swell, % Volume (Average) (3)	Swell, % Linear (4)	Depth of Layer, ft (5)	Vertical Movement, in (6)
0		15.0			
1.5-2.5	2.00	9.1	2.90	1.0 x 1.15 = 1.15	2.9% x 1.15 x 12 = 0.40
2.5-5.0	3.75	7.5	2.40	2.5 x 1.15 = 2.87	2.4% x 2.87 x 12 = 0.82
5.0-7.5	6.25	5.5	1.80	2.5 x 1.15 = 2.87	1.8% x 34.40 x 12 = 0.62
7.5-10.0	8.75	4.5	1.50	2.5 x 1.15 = 2.87	1.5% x 34.40 x 12 = 0.52
10.0-12.5	11.25	3.5	1.10	2.5 x 1.15 = 2.87	1.1% x 34.40 x 12 = 0.38
12.5-15.0	13.75	2.6	0.80	2.5 x 1.15 = 2.87	0.8% x 34.40 x 12 = 0.28
15.0-17.5	16.25	2.0	0.60	2.5 x 1.15 = 2.87	0.6% x 34.40 x 12 = 0.21
17.5-20.0	18.75	1.5	0.50	2.5 x 1.15 = 2.87	0.5% x 34.40 x 12 = 0.17
20.0-22.5	21.25	1.0	0.30	2.5 x 1.15 = 2.87	0.3% x 34.40 x 12 = 0.10
22.5-25.0	23.75	0.8	0.25	2.5 x 1.15 = 2.87	0.25% x 34.40 x 12 = 0.09
25.0-27.5	26.25	0.5	0.20	2.5 x 1.15 = 2.87	0.2% x 34.40 x 12 = 0.07
27.5-31.0	29.25	0.2	0.10	3.5 x 1.15 = 4.03	0.1% x 4.03 x 12 = 0.05
				Total Depth = 33.88	Total PVR Due to Swell = 3.71

*All soils with $A = (PI \div \% < 2\ \mu m) \leq 0.5$ are in the low category.

heave with depth because of overburden load was developed for South African soils.

$$D = K\log_{10} F$$

where

D = depth, ft (negative sign for depths below the surface)

K = constant (20 for the soils studied)

F = a factor indicating the relative decrease in heave at depth D compared to the surface (table 12a)

The total heave is then determined by summing the potential volume change over the soil profile, each layer being reduced by the appropriate value of F, as shown in table 12b.

Australian Method (refs. 69, 90)

The Australian Method involves the use of initial load and soil suction values and predicted values of final load and soil suction. The key to this method is the evaluation of soil response to load changes with simultaneous control of the load and soil suction in a specially modified consolidometer. The data from such tests are plotted to provide $\Delta H/H$ (linear strain) versus soil suction for various loading levels. With this relationship (in terms of the slope of the curve) known, measurement of initial suction and prediction of final suction provide sufficient data for an accurate estimate of the in-situ heave of the soil (table 13). The units pF equal the $\log_{10}$ of the suction in centimeters of water. (See appendix D.)

Although excellent results have been reported (refs. 90, 91), field implementation requires some development work. Measurements of soil suction can be made economically in the field with commercially available thermocouple psychrometers (refs. 92, 93). (See appendix D.) Since the slope of the swell/suction curve at various loads is required for design purposes, easier methods to obtain this are required. Correlation of the slope with soil index properties is a promising approach (ref. 94). This method should be very accurate for practical field use since it is based on an in-situ measure of the soil moisture status. With proper attention to predicting the final moisture condition and drainage design, good swell predictions should be available for design purposes.

Table 12. Van Der Merwe's Heave Prediction
[after Van Der Merwe (ref. 89)]

(a) Value of F with D from Relation $D = 20 \log_{10} F$

Depth, ft	Mean Value of F		Depth, ft	Mean Value of F	
0-1	F_1	0.943	15-16	F_{16}	0.168
1-2	F_2	0.842	16-17	F_{17}	0.150
2-3	F_3	0.750	17-18	F_{18}	0.133
3-4	F_4	0.668	18-19	F_{19}	0.119
4-5	F_5	0.596	19-20	F_{20}	0.106
5-6	F_6	0.531	20-21	F_{21}	0.094
6-7	F_7	0.473	21-22	F_{22}	0.084
7-8	F_8	0.422	22-23	F_{23}	0.075
8-9	F_9	0.376	23-24	F_{24}	0.067
9-10	F_{10}	0.335	24-25	F_{25}	0.060
10-11	F_{11}	0.298	25-26	F_{26}	0.053
11-12	F_{12}	0.266	26-27	F_{27}	0.047
12-13	F_{13}	0.237	27-28	F_{28}	0.042
13-14	F_{14}	0.211	28-29	F_{29}	0.038
14-15	F_{15}	0.188	29-30	F_{30}	0.034

(b) Sample Calculation

Depth, ft	Description	Potential Expansion, in	Predicted Heave $[F_D(PE)_D]$, in
0-1	Grey Sand	Low = 0	0.94 x 0 = 0.00
1-4	Yellow Lateritic Sandy Clay	Low = 0	2.26 x 0 = 0.00
4-10	Grey Slickensided Sandy Clay with Iron Concretions	High = 1/2	2.73 x 1/2 = 1.37
10-12	Nodular Lime in Sandy Clay	High = 1/2	0.56 x 1/2 = 0.28
12-20	Grey and Yellow Slickensided Sandy Clay with Iron Concretions	High = 1/2	1.31 x 1/2 = 0.66
20-21	Pebble Marker	Low = 0	0.09 x 0 = 0.00
21-30	Yellow Micaceous	Medium = 1/4	0.50 x 1/4 = 0.13
			Total Heave = 2.44
			Say 2.4

Table 13. Calculation of Total Swell [after Lytton (ref. 90)]

Depth, ft	Vertical Increment (H), in	Suction, pF Initial	Suction, pF Final	Overburden and Surcharge Pressure, lb/ft^2	Slope $(\Delta H/H)$, 0.1 pF	$\frac{\Delta H}{H}$	Lateral Restraint Factor (f)	$f\left(\frac{\Delta H}{H}\right)$	Average $f\left(\frac{\Delta H}{H}\right)$ for the Increment	Incremental Vertical Movement, in	Total Vertical Movement, in
0		4.0	3.2	0	0.0034	0.0270	1.00	0.0270			1.00
1	12	3.9	3.2	112	0.0031	0.0215	1.00	0.0215	0.0242	0.29	0.71
2	12	3.8	3.2	224	0.0028	0.0170	1.00	0.0170	0.0190	0.23	0.48
3	12	3.7	3.2	336	0.0026	0.0130	1.00	0.0130	0.0150	0.18	0.30
6	36	3.3	3.2	672	0.0017	0.0017	1.00	0.0017	0.0075	0.27	0.03
9	36	3.2	3.2	1008	0.0009	0	1.00	0	0.0008	0.03	0
12	36	3.2	3.2	1344	0.0000	0	1.00	0	0	0	0

Computer Methods

Three attempts to use computer codes to predict heave are reported in the literature. Richards' Method (ref. 95), which has been adapted to computer use, was the first example in the literature of two-dimensional suction distribution in which discontinuous, suction-dependent diffusion constants measured in the laboratory were used (ref. 14). Lytton and Watt (ref. 96) presented a method in which a computer-predicted change of moisture content is used to calculate the consequent change of soil volume. Although this method is simple and practical in approach, the soil data required for input are not readily available from routine soils investigations. The results presented are quite limited but appear promising. Recently Johnson and Desai (ref. 97) presented a finite-difference method for predicting heave with time for heterogeneous expansive soils beneath structures, idealized as two-dimensional. In this method a relationship between water content, suction, and plasticity index, developed empirically for English soils, is assumed; a relationship between volumetric swell, potential volume change, and surcharge pressure derived from data on Texas soils is also assumed. Despite these assumptions, reasonable results are obtained quickly and cheaply. Further development of these assumed relationships may be worthwhile. None of the numerical methods reported have demonstrated sufficient reliability to justify their implementation for routine use.

SECTION 6
STABILIZATION OF EXPANSIVE SOILS

INTRODUCTION

Once it is determined that an airport pavement is to be built on a soil which exhibits excessive volume changes, the following specific design approaches are available to the designer:

(1) make the pavement accommodate the movement of the subgrade,

(2) remove the undesirable material and replace it with a better soil,

(3) through construction techniques, prevent moisture changes in the subgrade after construction (prewetting, membranes, etc.), and

(4) alter the properties of the existing soil and make it suitable for a pavement subgrade.

In practical applications, (1) is not economical for airport pavements; (2) is often impractical or uneconomical because of the depth of the expansive soil or the availability of suitable fill material; and (3), although it may warrant careful study when the time for proper testing and design are available (refs. 82, 98), is not within the scope of this study. More experience with these techniques is required to develop implementable construction criteria. Choice (4) is often used in airport pavement construction and has been the subject of extensive research by the U.S. Air Force since 1969. Soil properties may be altered mechanically by compaction or by blending the soil with better soils or chemically by adding chemicals to the soil. This study was concerned with the stabilization of expansive soils with lime, cement, and bituminous materials.

Current pavement design procedures are largely empirical methods developed through extensive experience or testing. Subgrade soils are characterized by a single parameter such as the California Bearing Ratio (CBR) or the modulus of subgrade reaction, k. In the methodologies that have evolved for the design of stabilized soils, criteria that are compatible with pavement design procedures are used. The requirements for stabilized soils are established in terms of strength increases and no data are available for developing volume change reduction as criteria. A design method for expansive soil subgrades must contain volume-change criteria. Thus, the effects of stabilization methods on volume change must be established. There is a need for research in this area if an expansive soils design system is to be developed.

Stabilization objectives must presently be established in terms of strength increases produced in the soil layer. These increases, which may be determined quantitatively, provide a means of reducing the thickness of the overlying pavement layers. To attain a specific strength increase in a soil, a combination of stabilizing agents may be required. At the present time strength and durability testing are the only reliable means of evaluating stabilized materials. It is important to recognize that stabilization objectives should not be determined without considering the cost of achieving these objectives. This requires that the in-situ soil properties and the required soil properties be established. The designer is then required to select from the methods available those that will produce the required changes in the soil within the cost restrictions imposed on the facility. The current state-of-the-art for establishing the in-situ soil properties involves determination of the k value or CBR of the subgrade soil. Based on design charts relating soil supporting value to pavement thickness, an economical design value is selected. No current design procedure covers the durability and volume stability of the subgrade.

CHEMICAL STABILIZATION

Lime

The reactions that occur between lime and soil (e.g., ion exchange, flocculation, carbonation, and pozzolanic reaction) depend on the composition of the soil. When lime and soil are mixed, cation exchange and agglomeration/flocculation reactions occur; immediately this reduces the plasticity and improves the workability of practically all fine-grained soils (ref. 99). The rate at which these reactions occur depends on mixing, particle size, temperature, etc., as do all chemical reactions. Finer materials react better with lime; thus, heavy clay soils are most readily stabilized with lime. Carbonation reactions are produced when carbon dioxide reacts with the lime. This produces a weak calcium-carbonate cementing material and reduces overall strength (ref. 100). Up to this point no significant increase in strength is produced. A modified soil is the result of these reactions; most soils react to some extent. The greatest increase in strength results from pozzolanic reactions. These reactions produce strong cementing agents from lime, water, and aluminous or siliceous substances in the soil. In order for these reactions to occur, alumina or silica compounds must be present in the soil and become soluble in the high pH environment produced by the lime. This is the case with most clay soils. Organic compounds, sulfates, and iron may interfere with these reactions by reacting with the needed ingredients or by coating the clay particles. The process of designing lime-stabilized soils must provide for detection of these substances in order to produce satisfactory results over a wide geographical area. Lime stabilization, rather than modification, is produced when the pozzolanic reactions occur. Thompson proposed a strength increase of 50 psi as indicative of lime reactivity (ref. 101); this establishes a dividing line between modification and stabilization with lime.

Cement

The addition of Portland cement to soil produces changes in the behavior of the soil. These changes result from the hydration of the cement and therefore are highly dependent on the amount of cement. In some fine-grained soils, the free lime produced during cement hydration reacts with the soil to increase the strength with curing time. The hydration of cement produces calcium aluminate and silicate bonding materials which form bonds between and around the soil grains; this results in a matrix that encloses the soil particles. Thus, finer soils require greater amounts of cement, and very heavy clays may not be economically stabilized with cement alone. The major difference between cement- and lime-stabilization is that in cement-stabilization the cement contains the necessary ingredients for the pozzolanic reactions, but in lime-stabilization the soil must furnish part of the reactants. Therefore, cement/soil mixtures harden faster than lime/soil mixtures, although both mixtures continue to gain strength with time.

Lime/Cement

In some soils a combination of lime and cement is the most advantageous stabilizing agent. The stabilization procedure consists of two phases. A modification of the soil occurs with the addition of lime. This increases the workability, reduces the plasticity, and provides a much more suitable material for cement treatment. Then with the addition of cement, a lime/cement/soil mixture with significant strength improvements is formed. All soils react differently with lime and cement, and they do not work in combination for all soils. Testing remains the only reliable means of designing mixtures. The added expense of using two stabilizers is justified only for soils that are not lime reactive and require increased workability for mixing with cement.

Lime/Bituminous Material

Bituminous materials are widely used to stabilize granular materials for base and subbase applications. The mechanism is principally mechanical--waterproofing and cementing the soil grains together. Since bituminous materials cannot be used directly with fine-grained soils, clays must first be modified with lime into a granular material. There is little experience with the combination of lime and bituminous materials for stabilization; however, this method appears promising for soils that do not react with lime, require excessive amounts of cement, or cannot be blended with bituminous materials directly (i.e., heavy clays). Insufficient experience with this method is reported in the literature and therefore no implementable procedures can be formulated at this time.

SELECTION OF STABILIZING AGENTS

The U.S. Air Force initiated an extensive research effort in 1969 to develop implementable procedures for the selection of stabilizing agents and the evaluation of the stabilized materials. After the initial development (ref. 102), a laboratory validation was conducted to provide a basis for modifications where needed (ref. 103). The final system, the Air Force Soil Stabilization Index System (SSIS), has been selected by the U.S. Army Corps of Engineers for the design of stabilized soil layers (ref. 104). A follow-on research project just completed has further validated this system through laboratory tests and field surveys (ref. 105). A review of the technical literature indicates that these reports comprise the most satisfactory methodology for the selection of admixtures and the evaluation of stabilized materials for airport pavement construction.

The SSIS is a guide to the selection of chemical admixtures (lime, cement, and bituminous materials) and admixture quantities. Research included an extensive literature review, consultation with over 40 acknowledged experts in soil stabilization, and laboratory validation. The overall system provides procedures for the selection of the stabilizer or combination of stabilizers (fig. 13) based only on soil classification test data. Once the stabilizer is selected, procedures for selecting the quantity are provided based on the use (base or subgrade) and the military situation (expedient or nonexpedient). The adaption of this system to the stabilization of expansive soils involves the use of the procedures for stabilizing nonexpedient subgrade soils. Where the expedient techniques are promising, they are discussed since they save considerable time.

STABILIZED SOIL DESIGN AND EVALUATION

Subsystem for Lime Stabilization

The SSIS procedure for lime stabilization is shown in figure 14. Four distinct steps are involved in selecting the optimum lime content:

(1) Estimate the lime content using a pH test.
(2) Determine the strength (lime reactivity) of the mixture.
(3) Determine the durability of the mixture.
(4) Select the optimum lime content. (Steps 2 and 3 are performed at several lime contents.)

The lime content is estimated by the Eades and Grim (ref. 106) pH Test (appendix E). Strength is evaluated by measuring the unconfined compressive strength, q_u, after 28 days of curing; the durability is evaluated as residual strength after 24 hours of immersion in water. By using the lime content initially estimated by the pH Test and $\pm$ 2 percent of it, an optimum value can be selected for design. A modification of this procedure was studied at the

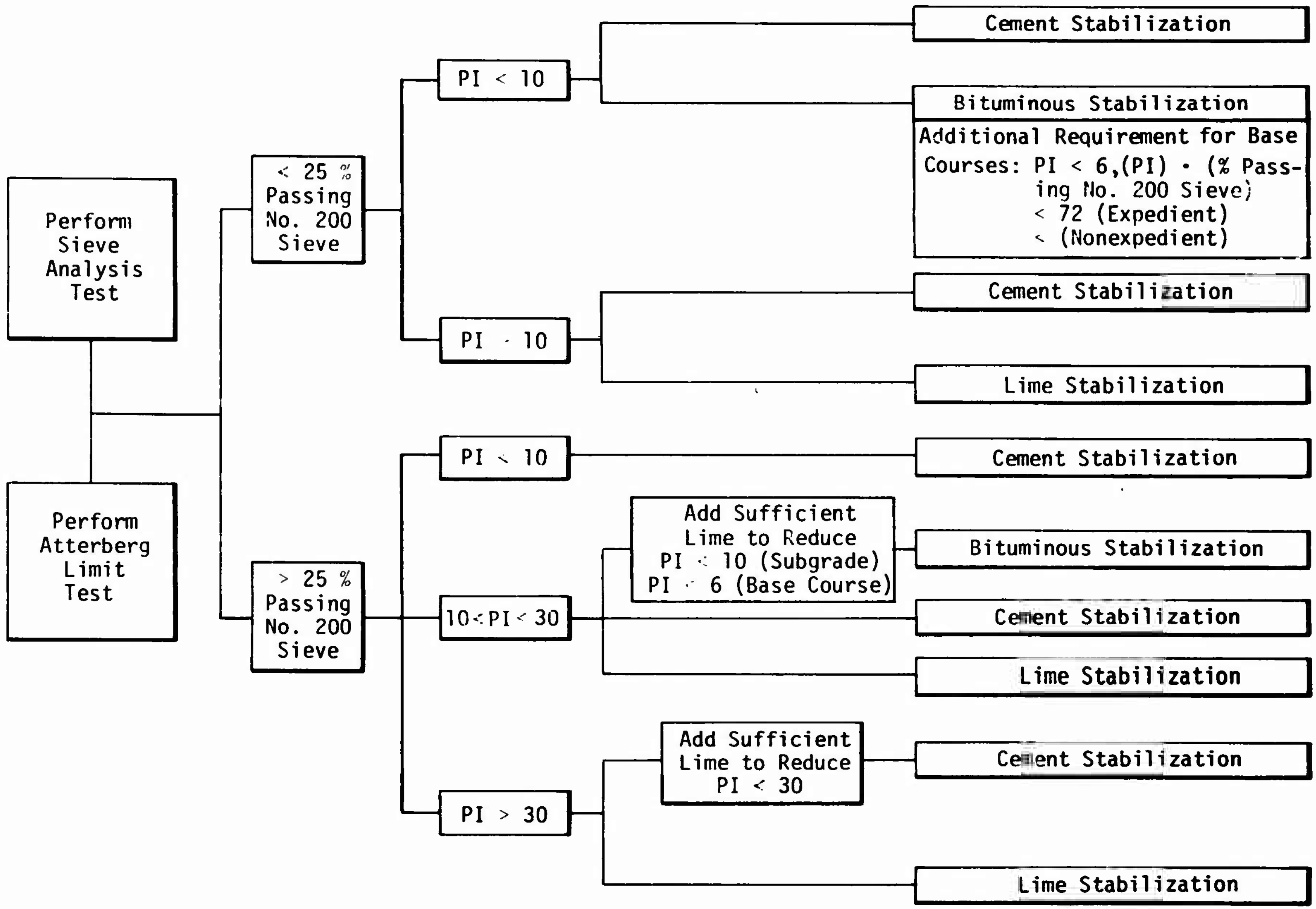


Figure 13. Selection of Stabilizer [after Dunlap et al. (ref. 103)]

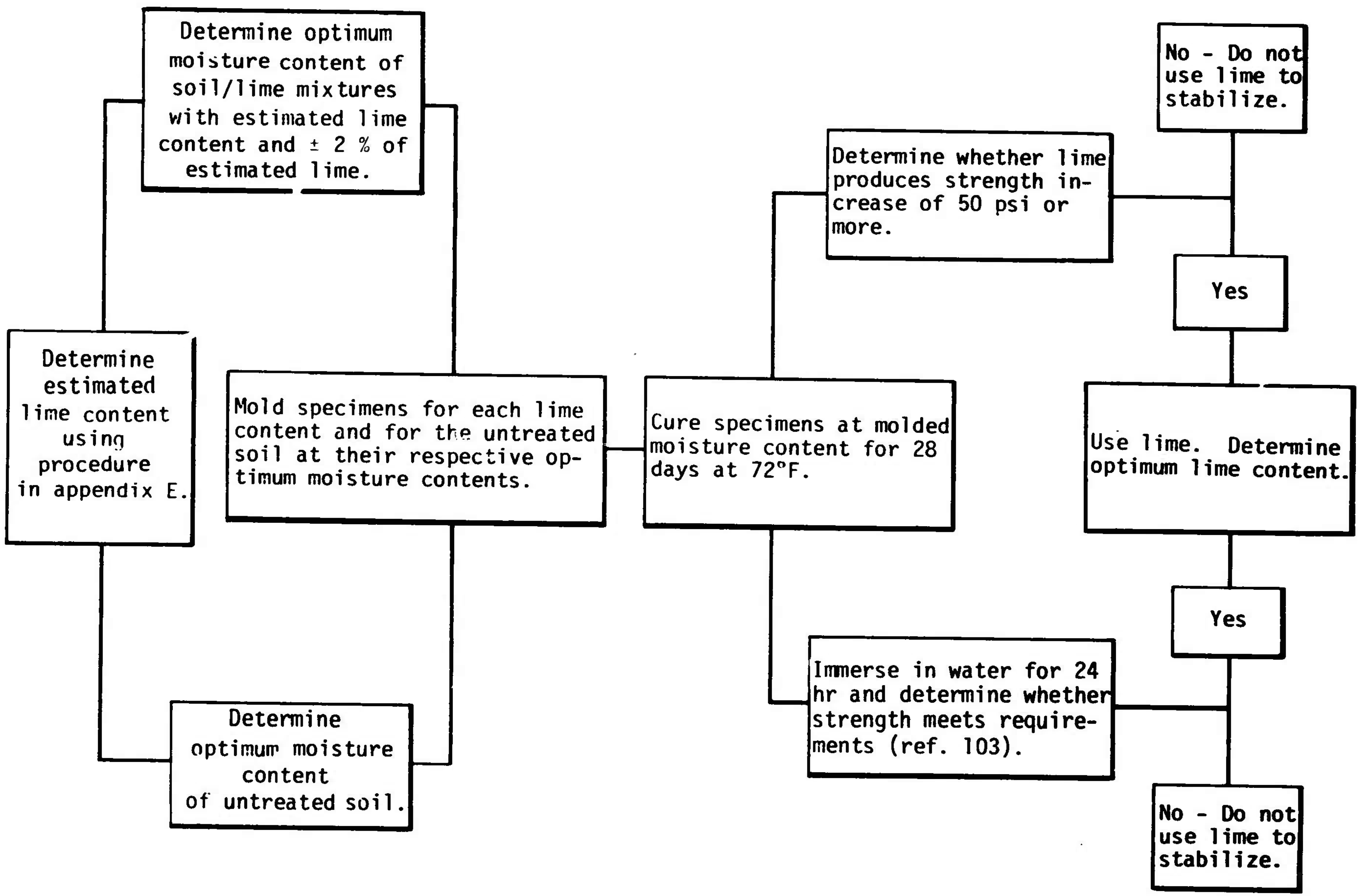


Figure 14. SSIS Subsystem for Nonexpedient Subgrade Stabilization with Lime [after Dunlap et al. (ref. 103)]

U.S. Air Force Academy (ref. 105). A wide range of soils was used in the study (appendix F). The objectives of this validation study and a summary of the results are as follows:

(1) Verify the validity of lime stabilization of soils with a plasticity index greater than 10. Results verified the validity of PI > 10 as an indicator of pozzolanic potential; however, soils with PI < 10 may also be lime reactive. These conclusions are based on a ≥ 50-psi increase in 28-day unconfined compressive strength.

(2) Verify the validity of the Eades and Grim pH Test in estimating the optimum lime content. Results indicated a 0.96 correlation coefficient between the optimum lime content based on strength tests and that based on the pH Test (fig. 15). Caution must be exercised when the test is used on soils with high organic content.

(3) Verify the validity of accelerated curing and establish curing times and temperatures. For the wide variety of soils tested (all those considered for lime stabilization), rapid cure may be used in place of normal cure (fig. 16). Rapid curing consists of 30 hr at 120°F (49°C) instead of 28 days at 73°F (23°C). Rapid cure values are generally slightly conservative.

(4) Evaluate three-cycle freeze/thaw strength as an indicator of field durability in varied environments. Results indicated that no significant strength loss occurred after seven freeze/thaw cycles. The strength loss after seven freeze/thaw cycles can be predicted from the strength loss after three freeze/thaw cycles with the family of curves shown in figure 17. Thus, with accelerated curing and three-cycle freeze/thaw techniques, testing may be completed in about one week.

(5) Evaluate strength after vacuum soak as an indicator of field durability in varied environments. An acceptable correlation between three-cycle freeze/thaw data and vacuum-immersion data was found. On this basis a design chart was developed (fig. 18).

(6) Investigate the effects of sulfates and organics on lime stabilization of soils. Above 1 percent, sulfates reduce the

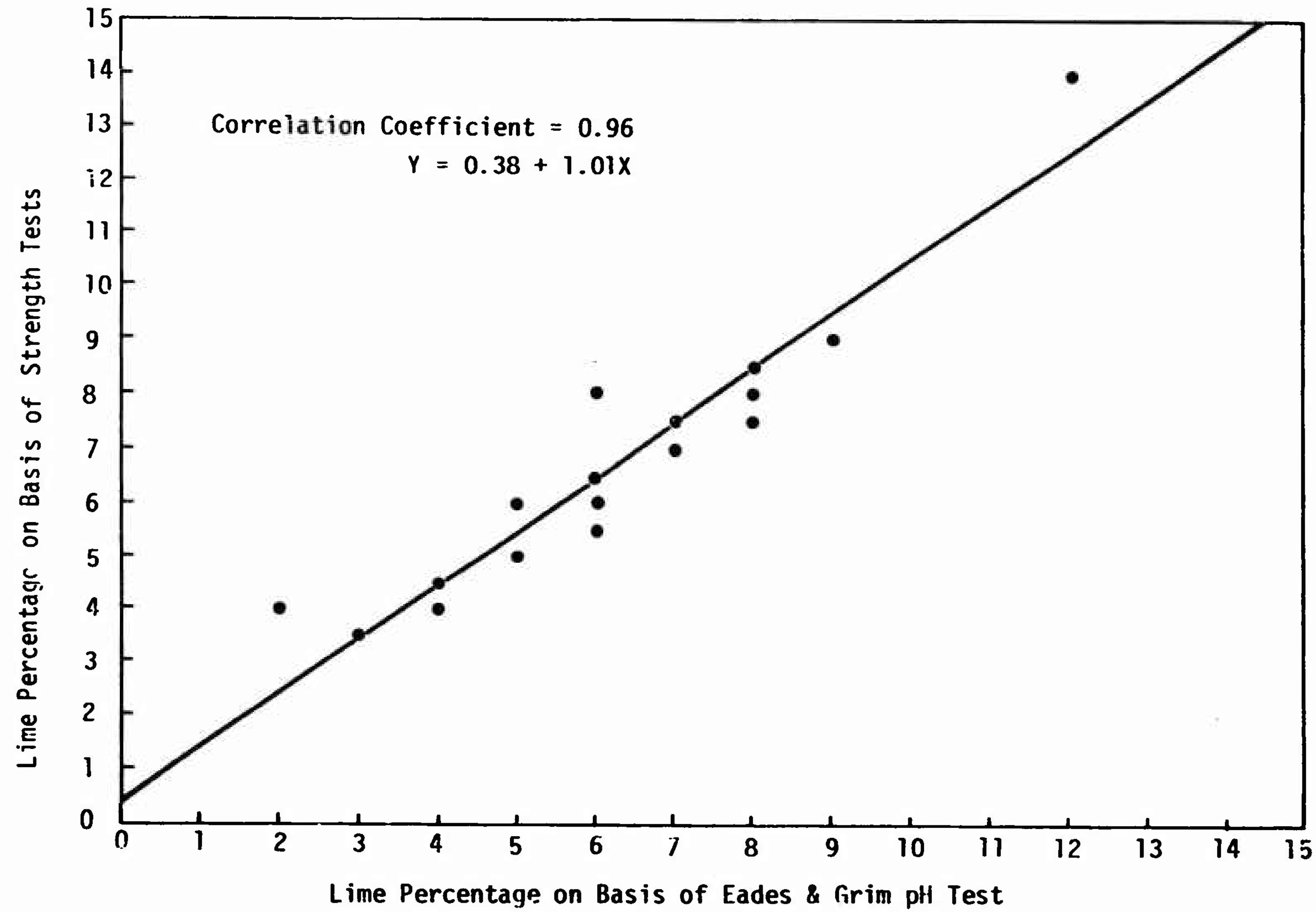


Figure 15. pH Test Versus Strength Test as Predictor of Optimum Lime Content [after Currin et al. (ref. 105)]

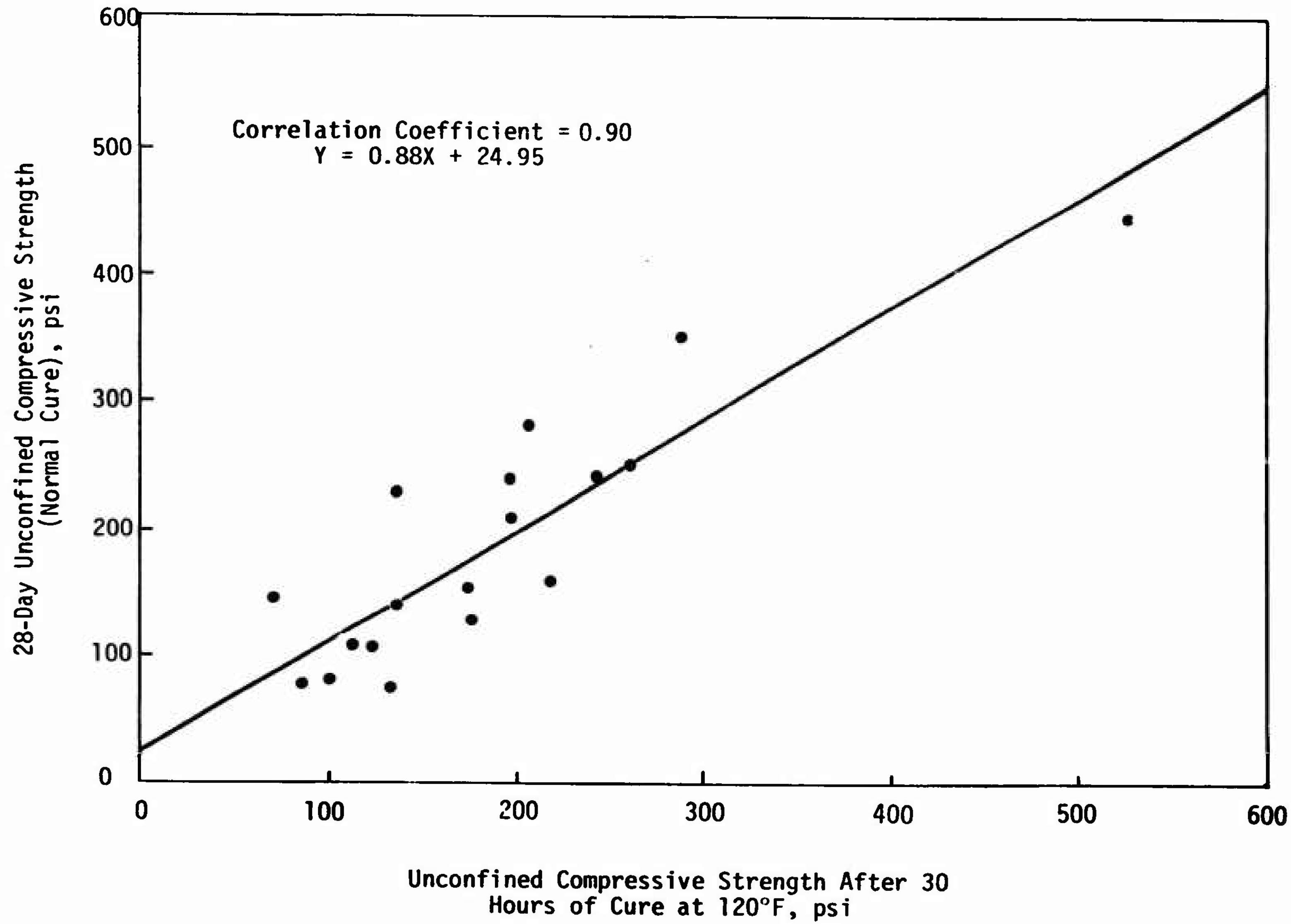


Figure 16. Twenty-Eight-Day Strength Predicted by Accelerated Cure [after Currin et al. (ref. 105)]

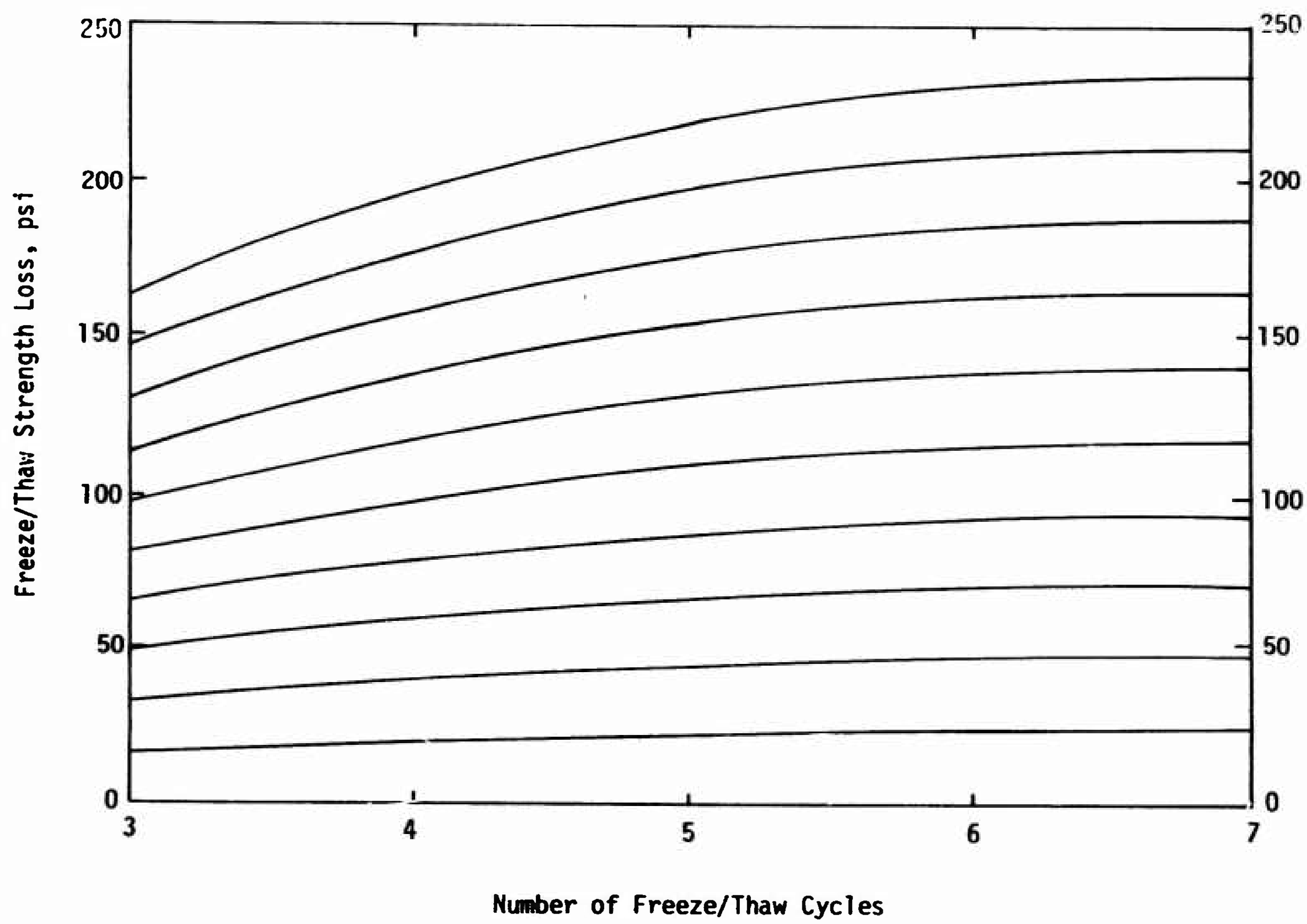


Figure 17. Design Chart for Freeze/Thaw Loss [after Currin et al. (ref. 105)]

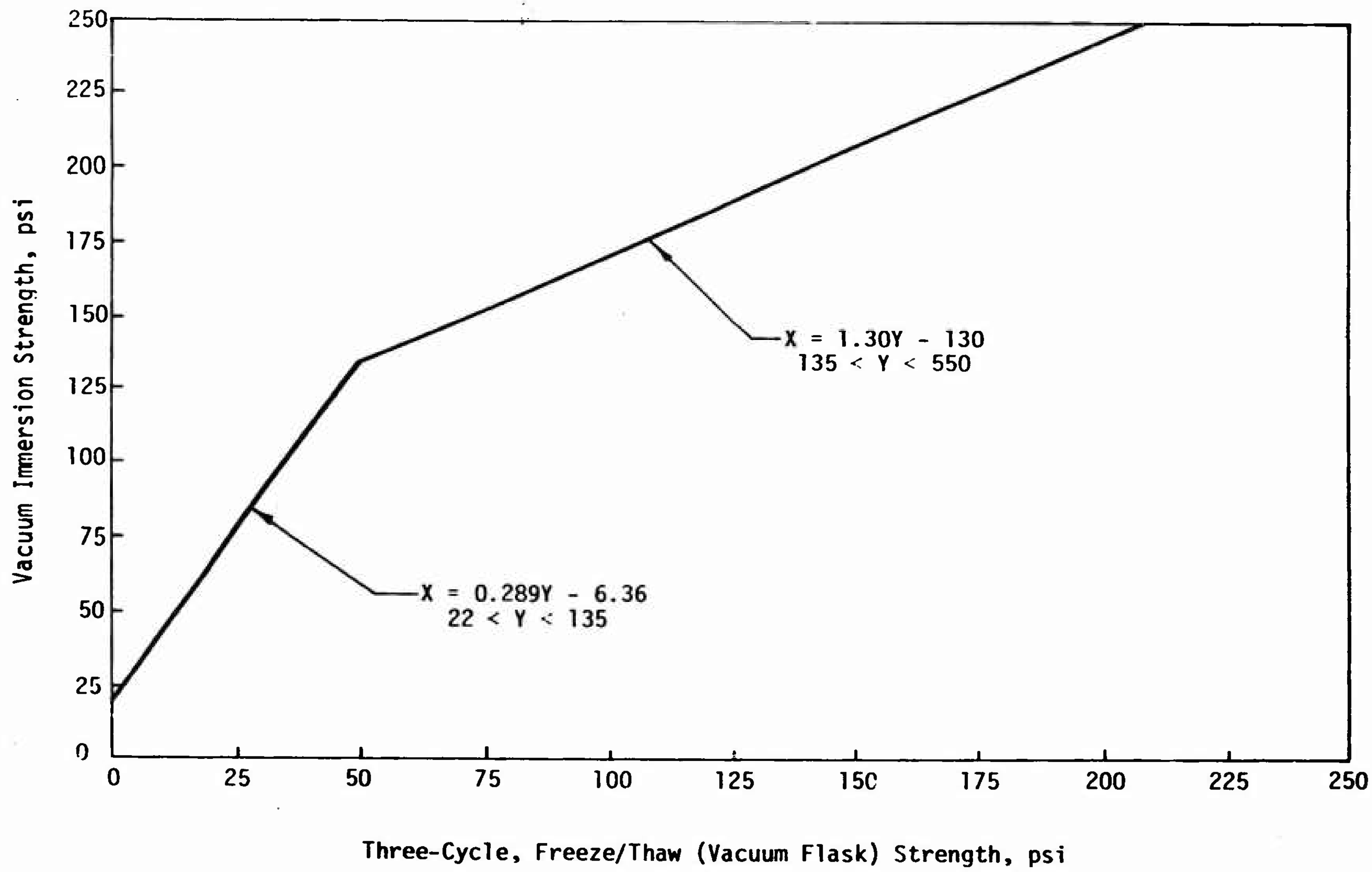


Figure 18. Design Chart for Three-Cycle, Freeze/Thaw Strength From Vacuum Immersion Strength [after Currin et al. (ref. 105)]

durability of lime/soil mixtures. The presence of organic matter seems to increase the amount of lime required but does not appear to influence the strength or durability. The data were quite limited in this area and no implementable conclusions were made.

Based on the Air Force Academy validation study, the design subsystem for nonexpedient subgrade stabilization with lime shown in figure 19 is recommended.

Subsystem for Cement Stabilization

The SSIS procedure for nonexpedient subgrade stabilization with cement is shown in figure 20. It consists of the PCA procedures for sandy soils (< 50 percent silt and < 20 percent clay) and the PCA base course procedure (ref. 107), preceded by nonstandard tests for harmful amounts of organics and sulfates. The test used to detect harmful amounts of organics was first introduced by MacLean and Sherwood (ref. 108) and is shown in appendix G. If sufficient organic material is present to prevent the development of a pH of 12.1, the cement hydration reaction will be impaired. Thus many near-surface soils cannot be stabilized with cement. The test for sulfates is made by two procedures (appendix G), but because of the complications of the test it should not be used unless high sulfate content is suspected.

In the validation of the SSIS performed at the U.S. Air Force Academy (ref. 105), the soils shown in appendix F were tested. Based on the technical literature, hypothesized design subsystems were constructed and validation objectives were established. These objectives and the conclusions reached are as follows:

(1) Verify the prediction of 28-day unconfined compressive strength by 7-day unconfined compressive strength. Results are shown in figure 21. The authors recommend the use of 7-day unconfined compressive strength to predict 28-day unconfined compressive strength.

(2) Determine whether an accelerated freeze/thaw cycle can be substituted for the standard freeze/thaw cycle (ASTM D560).

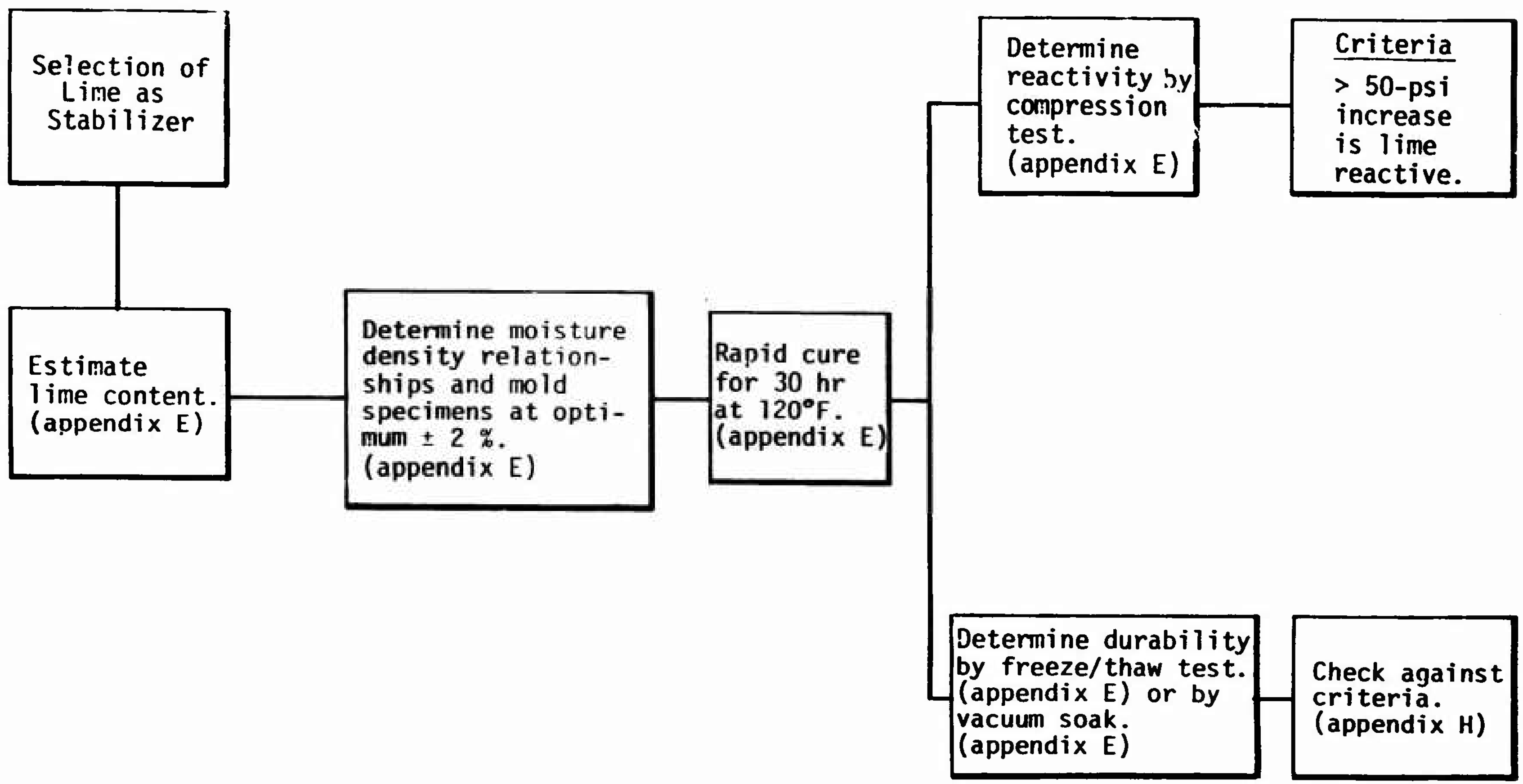


Figure 19. Design Subsystem for Nonexpedient Subgrade Stabilization with Lime [after Currin et al. (ref. 105)]

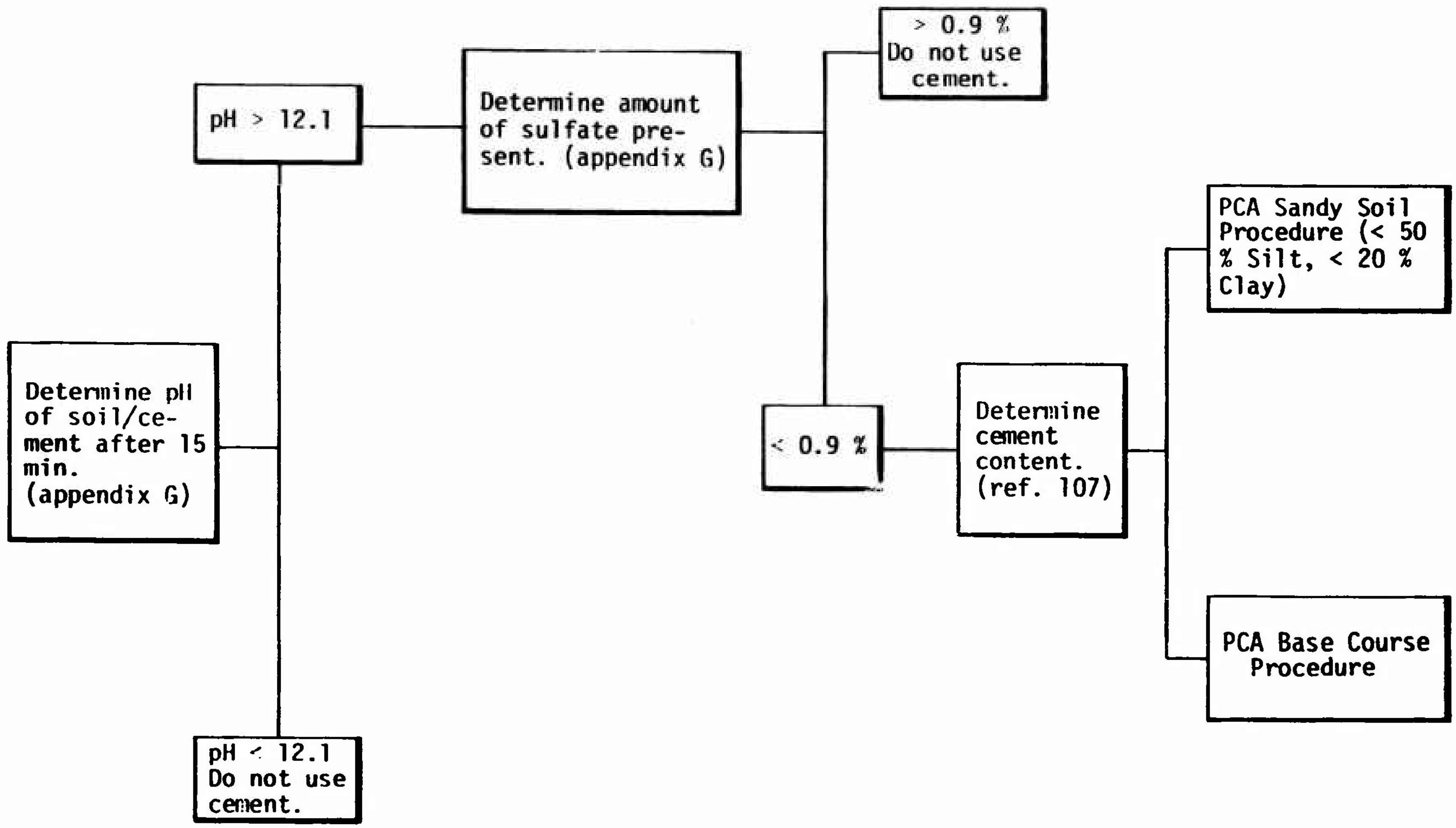


Figure 20. SSIS Subsystem for Nonexpedient Subgrade Stabilization with Cement [after Dunlap et al. (ref. 103)]

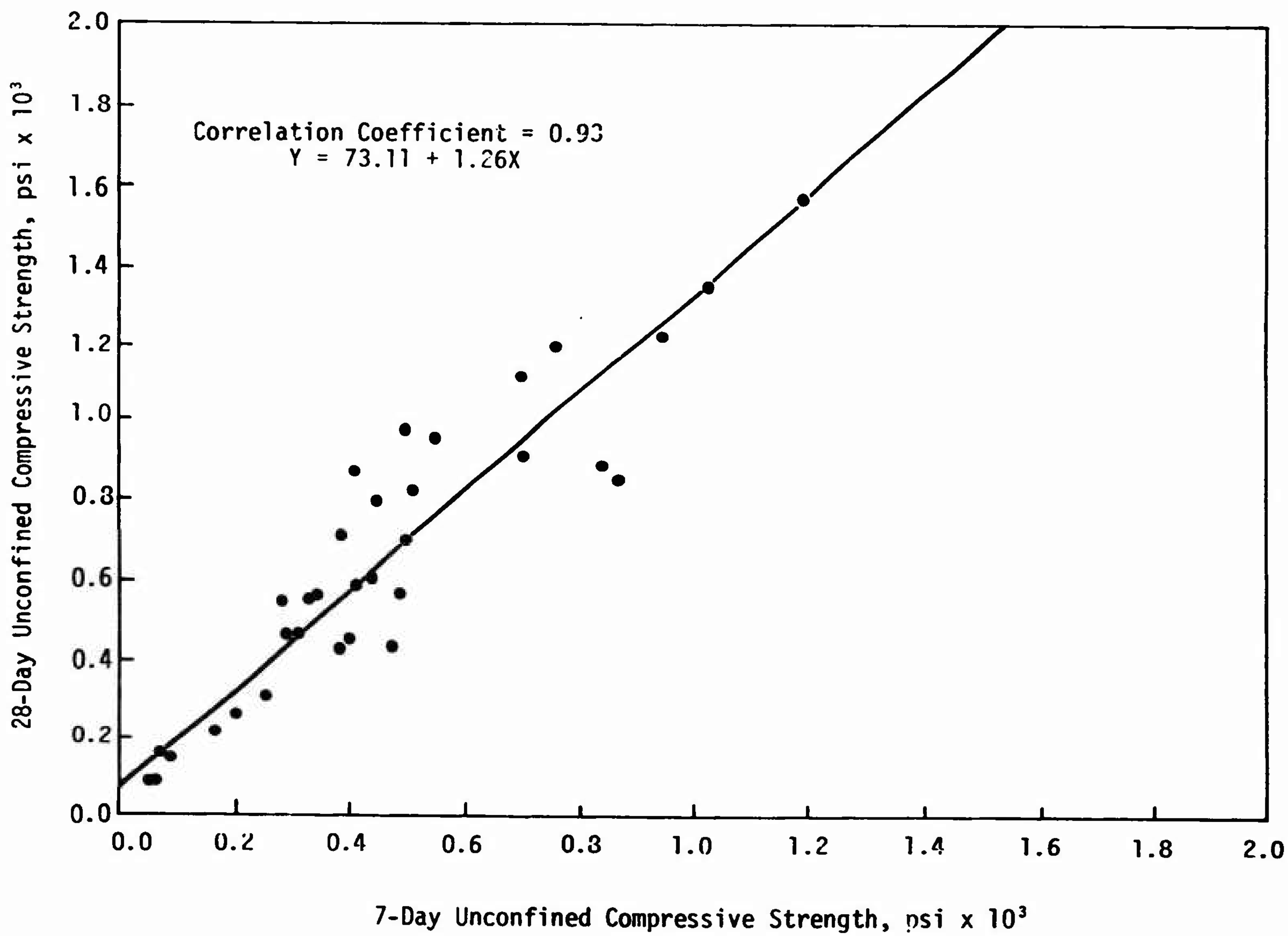


Figure 21. Correlation Between Seven-Day and Twenty-Eight-Day Unconfined Compressive Strength [after Currin et al. (ref. 105)]

(3) Determine whether the standard 12-cycle freeze/thaw weight loss can be predicted in a fewer number of freeze/thaw cycles. It was found that a shortened freeze/thaw cycle of 8 and 4 hr, respectively (each cycle is 24 hr in the standard test), could be substituted for the standard procedure. Correlations for 6-cycle accelerated freeze/thaw loss with standard 12-cycle freeze/thaw loss are shown in figure 22. The data reflect excellent correlation but are based on only seven soil types. Before this system can be put into general use more soils must be tested. With the correlations shown, the PCA weight loss criteria would be altered as shown in table 14. Since the PCA criteria are established by the AASHO soil group and the seven soils tested represented seven different groups, these results are certainly significant. Every effort should be made to further test the validity of these correlations.

(4) Determine whether immersion strength tests (long-term immersion or vacuum immersion) can predict durability. These techniques yielded a low degree of correlation with standard freeze/thaw durability (ASTM D560). Because of the promising results of the accelerated freeze/thaw test, these procedures are not recommended.

(5) Determine whether the wet/dry test is a valid durability predictor. Because of the additional cement hydration permitted in this test, it is much less severe than the well-established freeze/thaw test. Because of the promising results with the accelerated freeze/thaw tests, the wet/dry test is not recommended as a durability indicator.

(6) Verify the PCA procedure for selecting the optimum cement content. For all soils tested the PCA procedure gave satisfactory results.

(7) Investigate the pH Test used in the SSIS (ref. 103) for evaluation of the organic content. Based on SSIS, the procedure is recommended for use; no additional information regarding the test was obtained in this study.

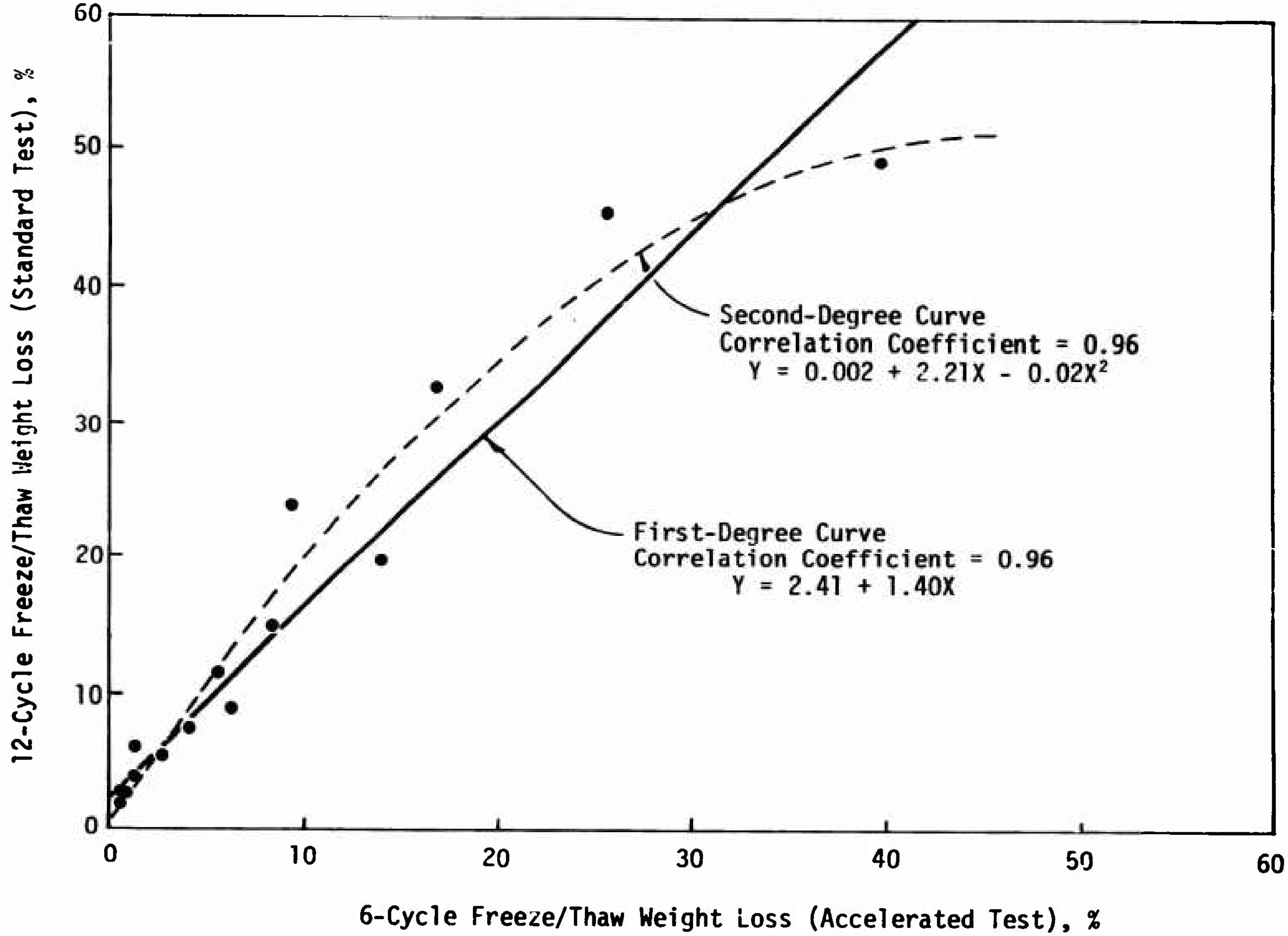


Figure 22. Correlations Between Six-Cycle Accelerated and Twelve-Cycle Standard Freeze/Thaw Weight Loss [after Currin et al. (ref. 105)]

Table 14. PCA Soil/Cement, Freeze/Thaw Weight Loss Criteria

PCA Criteria		Accelerated 6-Cycle Freeze/Thaw Test Weight Loss, %	
AASHO Soil Group	Maximum Weight Loss, %	1st-Degree Curve	2nd-Degree Curve
A-1,A-2-4,A-2-5,A-3	< 14	8.3	6.5
A-2-6,A-2-7,A-4,A-5	< 10	5.4	4.7
A-6,A-7	< 7	3.3	3.3

(8) Investigate the effect of sulfate on cement-stabilized soils and establish the maximum allowable percentage. The upper limit of 0.9 percent set by the SSIS procedure should be used. Because of the nature of the test, it should be conducted only when a high sulfate content is suspected.

Based on the Air Force Academy validation study, the design subsystem for cement stabilization of nonexpedient subgrades illustrated in figure 23 is recommended.

SOIL STABILIZATION SYSTEM FOR EXPANSIVE SOILS

Introduction

The procedure recommended for selecting type and amount of admixture for stabilizing expansive soils is based on the SSIS developed for the U.S. Air Force by Texas A&M University (ref. 103) and the validation study just completed at the Air Force Academy (ref. 105). These systems were developed through four years of laboratory research and consultation with most of the recognized experts in the field of chemical soil-stabilization with lime, cement, and bituminous materials (refs. 102, 103, 105).

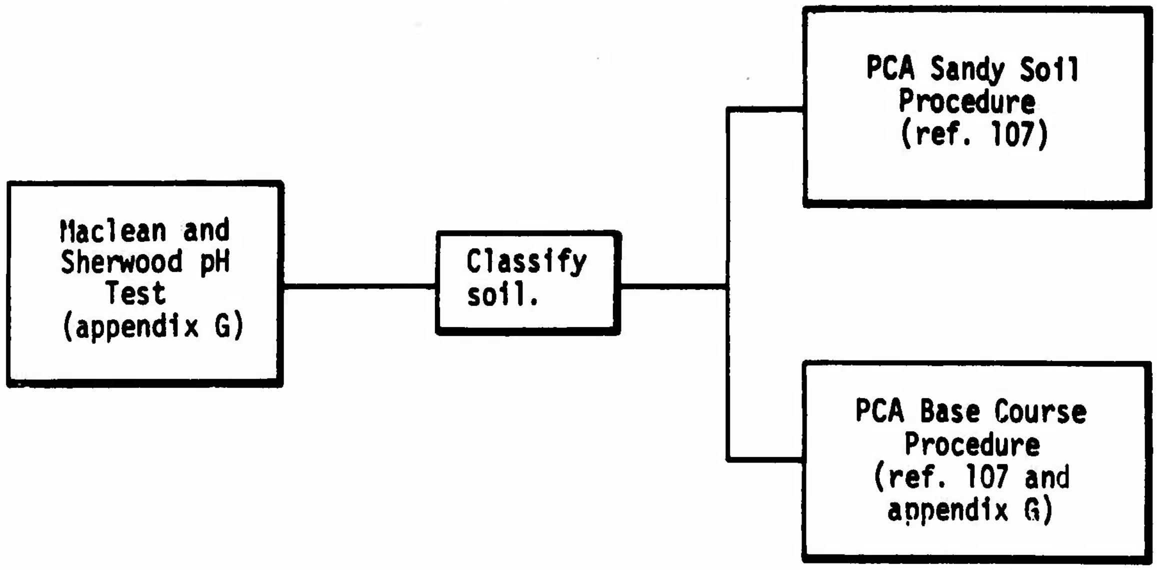


Figure 23. Design Subsystem for Nonexpedient Subgrade Stabilization with Cement [after Currin et al. (ref. 105)]

The procedures presented here are for use of lime, cement, and their combination; bituminous materials are not applicable to the stabilization of expansive soils. The literature provides very little data to support procedures for lime modification of soils followed by bituminous stabilization. It is felt that the validity and usefulness of such methods are highly questionable and, therefore, no such procedures were considered.

Some of the tests involved in the current system are nonstandard. The development of the procedures was based on these tests and, therefore, they must be used until correlations with standard tests are established by laboratory testing or until these methods are standardized. In this report the best available system is presented in what is believed to be an implementable procedure. Figure 24 illustrates the selection of the type of admixture for expansive soil stabilization.

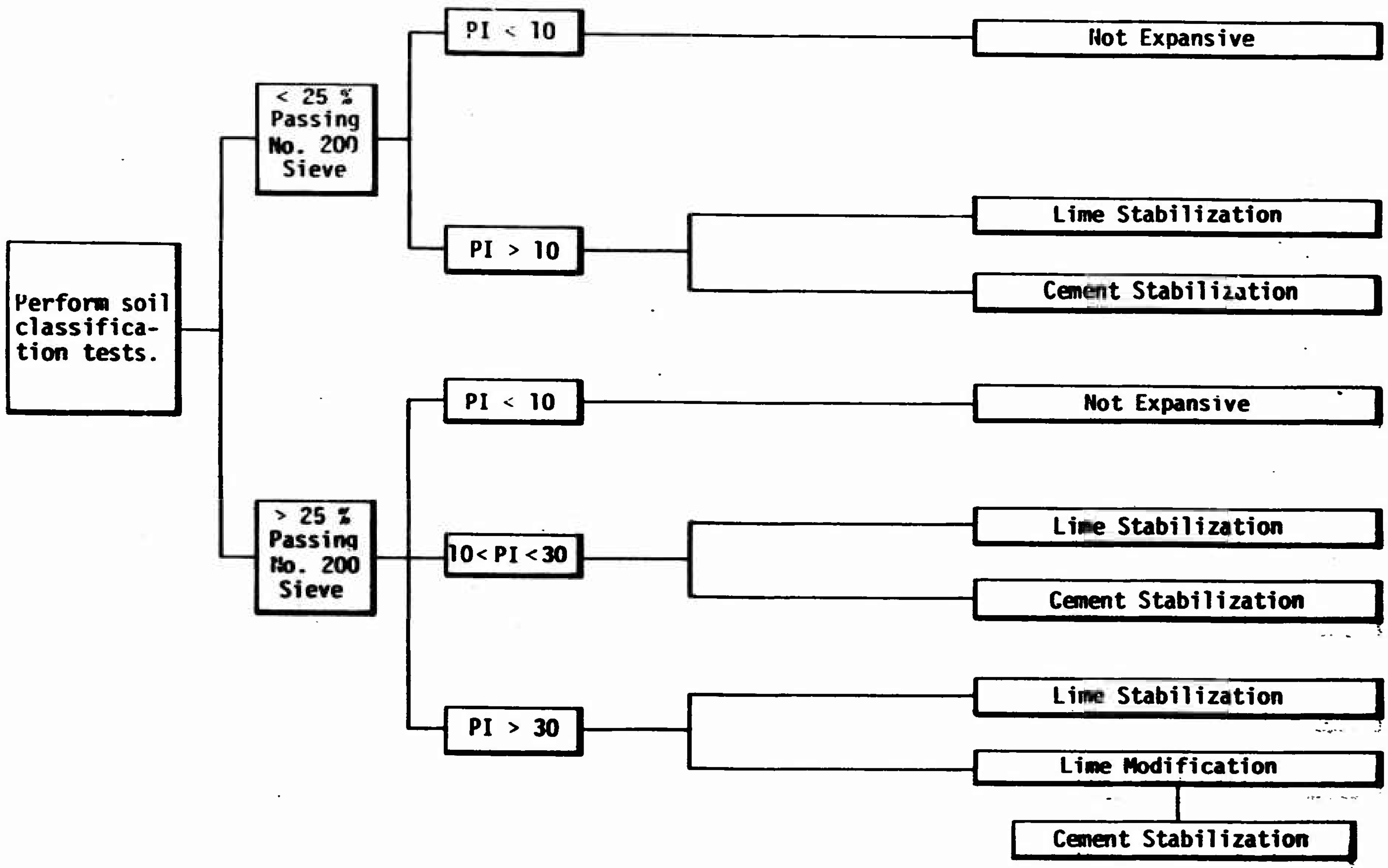


Figure 24. Selection of Type of Admixture for Expansive Soil Stabilization

Subsystem for Lime Stabilization

The subsystem synthesized from the Air Force SSIS for stabilization of expansive soils with lime is shown in figure 25. Test procedures are shown in the appendixes indicated. An initial lime content is selected on the basis of the pH test. Specimens of soil and lime/soil are prepared for freeze/thaw and lime-reactivity determinations and cured using the accelerated method recently developed (ref. 105). Tests for lime reactivity and durability are conducted and results are evaluated. As part of the SSIS validation, residual strength requirements for airport pavements were determined (ref. 105). These are described in appendix H and constitute better criteria than those previously used. Volume change reduction criteria are indicated but at present do not exist.

Subsystem for Cement Stabilization

The subsystem for cement stabilization of expansive soils is illustrated in figure 26. An initial pH test is used to detect harmful amounts of organic material. The sulfate test is recommended only when there is reason to believe that > 1 percent sulfates may be present in the soil. The remainder of the procedure is the PCA method (ref. 107), except for the freeze/thaw weight loss criteria which are in appendix H. Again, the volume change reduction is indicated in the procedures, but no attempt to develop such procedures has been made.

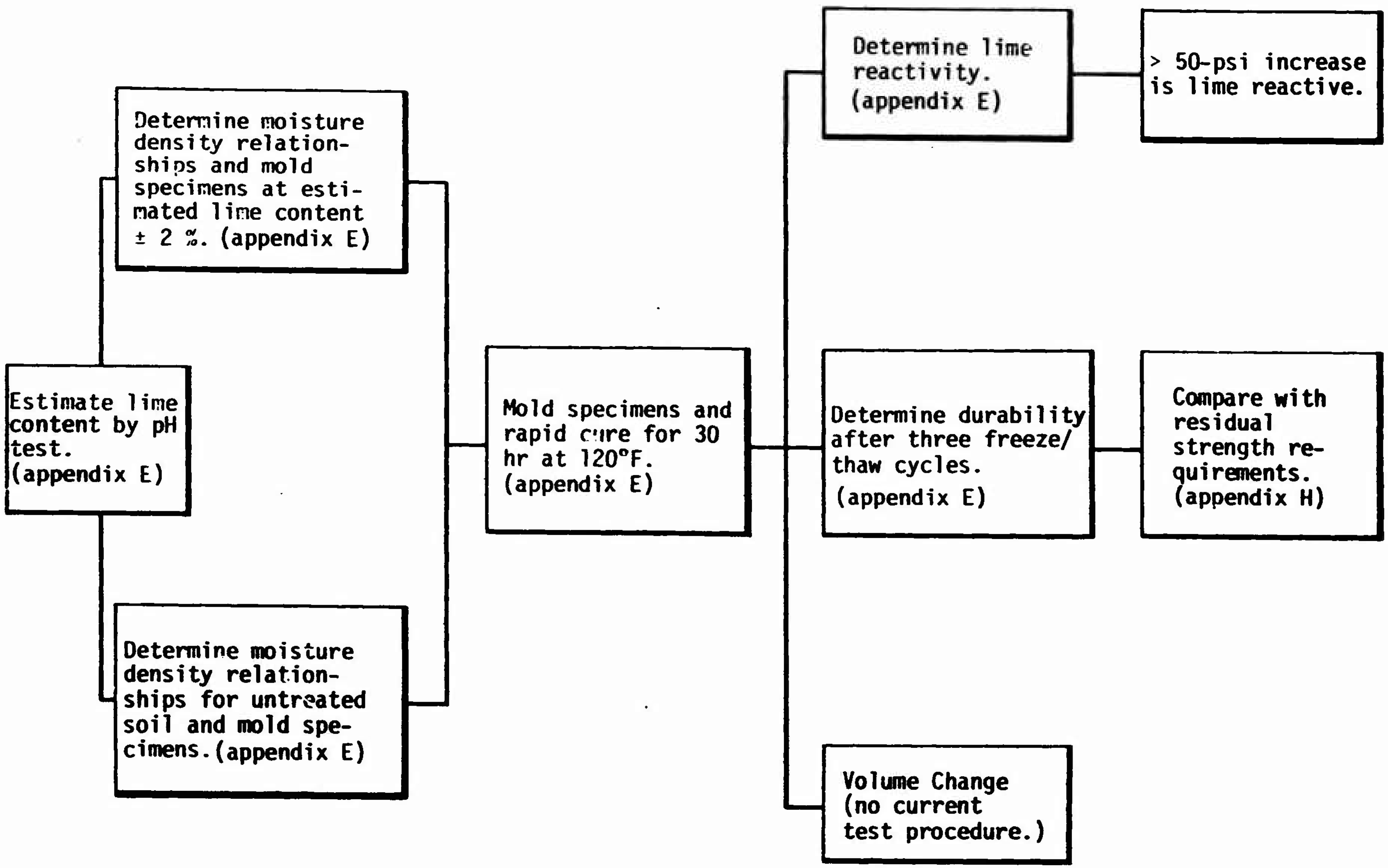


Figure 25. Subsystem for Expansive Soils Stabilization with Lime

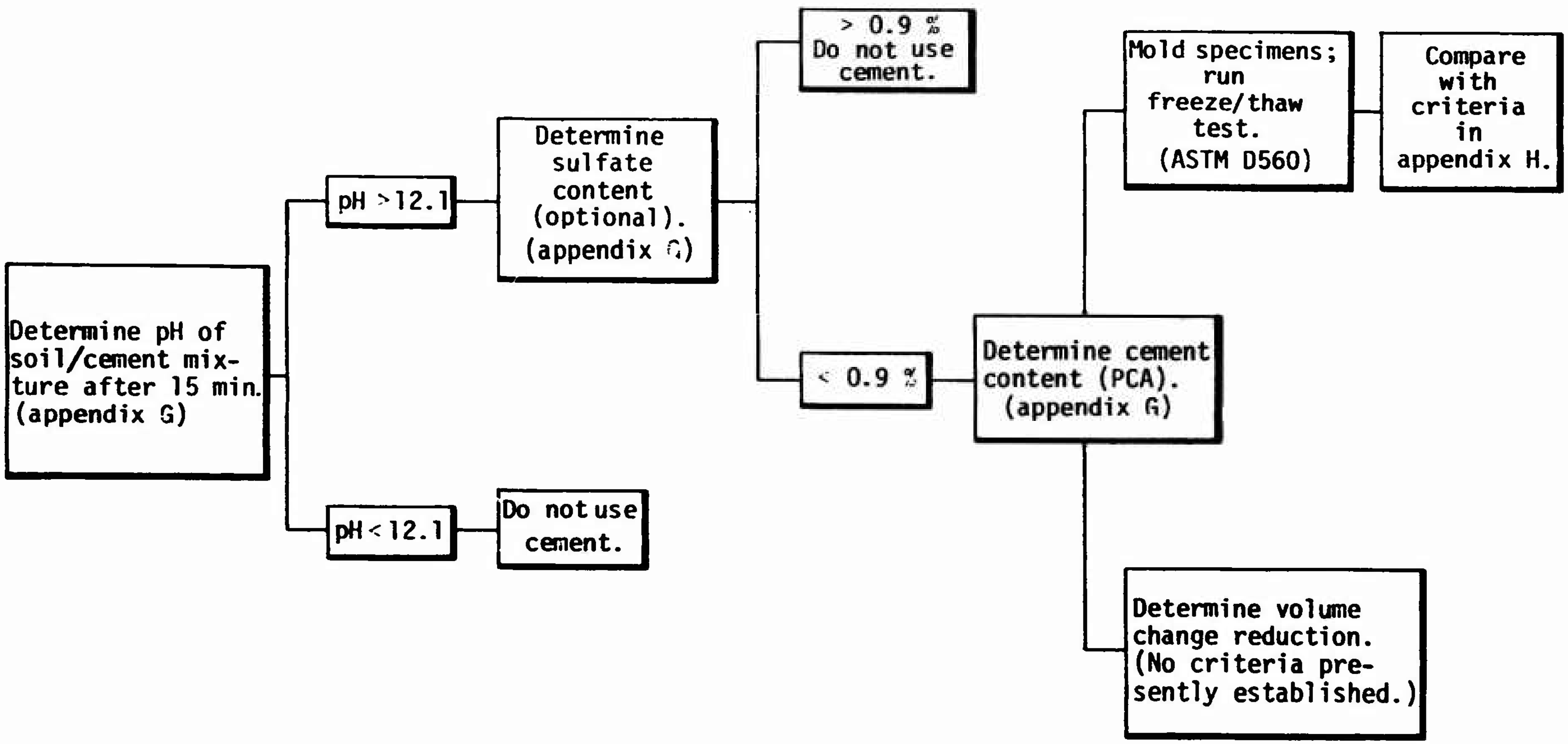


Figure 26. Subsystem for Expansive Soils Stabilization with Cement

SECTION 7
CONSTRUCTION OF CHEMICALLY STABILIZED SOILS

The most important aspect of chemical soil-stabilization is the field-mixing procedure used. The vast difference in uniformity of mixing, subdivision, and pulverization between laboratory and field procedures must be considered in the design and construction of chemically stabilized soils. Field strengths seldom match laboratory strengths (ref. 109). A mixing efficiency of 60 percent is typical (ref. 110); it may be as low as 10 percent for highly plastic soils (i.e., expansive soils) (ref. 111). Mixing efficiency is defined as the ratio of field-mixed strength to laboratory-mixed strength. Until mixing equipment for clay materials becomes more efficient, the significant difference between field and laboratory strength will continue. The efficiency of the mixing equipment must be determined before any realistic benefit of the stabilization may be determined in the design process. The important consideration is to determine the mixing efficiency and take it into account in the design.

LIME STABILIZATION PROCEDURES

Soil Scarification and Pulverization

After the soil is at the required grade, it should be scarified with a grader/scarifier, a disc harrow, a rotary mixer, or a similar device to the required depth to reduce the material to a clod size of less than 2 in. Although this may require a higher number of passes for heavy clay soils, modern rotary mixers can achieve this degree of pulverization. All debris such as stumps, roots, and aggregations larger than 3 in should be removed.

Lime Spreading

Lime may be spread dry or in slurry form (typically, 1 ton of lime to 500 gallons of water); lime may be distributed directly on the soil with a variety of spreading devices from self-unloading bulk tanker trucks. The engineer should insure that uniform distribution is achieved and that dry lime is not lost through wind erosion or other disturbances. Lime slurry can be prepared in a central mix plant by one of several techniques (ref. 112). The slurry can be transported to the subgrade with some provision for agitation to prevent lime solids from settling out of the solution. Provisions must be made to prevent the runoff of slurry on slopes once it is deposited on the soil. Standard water or asphalt distributors are the most common equipment used to distribute lime slurry; those with a pressure distributor are preferred since they distribute the slurry more uniformly (ref. 112).

Lime and soil may be mixed in a central facility and transported to the construction site. In this case, the material must be excavated, transported, treated, and distributed at the site. It should be spread by a spreader to avoid piles of material which cause density variations. The compaction and curing methods are the same as those for mixed-in-place lime/soil mixtures.

Mixing and Curing

Lime stabilization of heavy clays requires two mixing stages (ref. 102). The initial mixing distributes the lime throughout the soil layer, reduces plasticity, and improves the workability (mellowing). The time required varies from 2 days for silts to 7 days for very heavy clays. Ideally, the soil should be mixed with a rotary mixer so it is reduced to the point that 100 percent passes the 1-in sieve and 60 percent passes the No. 4 sieve (refs. 102, 112). The lime/soil mixture is then lightly compacted to prevent carbonation and to protect it from rain.

After the curing period, the lime/soil is mixed with high-speed rotary mixers to achieve the requirement of 100 percent passing the 1-in sieve and 60 percent passing the No. 4 sieve. The high-speed rotary mixer is the only device available today that can meet the requirements for gradation and uniformity in the mix.

Compaction

Compaction is normally done in one lift. A sheepsfoot roller is first used, then a 10-ton pneumatic tire roller, and finally a steel flat wheel or light pneumatic rollers for finishing. The requirement for compaction is 95 percent of the maximum AASHO T99 (ASTM D698) test (ref. 112). The test must be based on field samples of a lime/soil mixture.

Final Curing

After compaction the lime/soil should be cured 3 to 7 days by sprinkling the surface to maintain a moist condition, or by sealing the surface with a membrane material. No criteria are available to specify the curing time; however, the longer the curing time, the stronger the material will be when the final pavement layers are constructed. Construction cutoff dates must be established to allow completion of curing before freezing temperatures occur. Provisions must be made to prevent erosion due to heavy rainfall during curing, if it is anticipated.

CEMENT STABILIZATION PROCEDURES

The best available source of information on the subject of soil/cement stabilization is a series of bulletins published by the PCA (refs. 107, 113, 114). These procedures are the result of 30 years of experience; practically all other procedures used today are derived from them.

Soil Scarification and Pulverization

Soils amendable to cement stabilization are usually not heavy clays (PI < 30). Thus, mixing is easily accomplished and pulverization is less critical. When the cement is to be applied to a windrow, the soil must be scarified and placed in a windrow. For application directly over the subgrade in a uniform layer, the soil may not need to be scarified. Rotary mixers generally used with this procedure are capable of breaking up all but the very hardest soils.

Cement Spreading

Cement is usually spread from a bulk transport truck or closed dump truck by a mechanical spreader attached to the rear. Spreading should be done at a moisture content below optimum, even if drying the soil is required; cement will not mix adequately with wet soil.

Mixing

The cement and soil are mixed with the appropriate amount of water for the final mixture. The precise sequence of events varies with the type of equipment and mixer used. The final mixture should meet the criteria of 100 percent passing the 1-in sieve and 60 percent passing the No. 4 sieve (ref. 113).

A central plant may be used to mix the soil, cement, and water. A more uniform mixture is usually obtained, but the cost of excavating and transporting may be significant. The soil/cement mixture should be distributed with spreader boxes rather than by dumping and spreading to reduce density variations (ref. 102). Compaction requirements are the same as those for mixed-in-place soil/cement mixtures.

Compaction

A variety of compaction equipment has been used to satisfactorily compact soil/cement mixtures (ref. 102). Sheepsfoot rollers should be used for all but highly granular soils. Contact pressures should be 75 to 125 psi for friable silty and clayey sandy soils, 100 to 200 psi for clayey sands, lean clays, and silts, and 150 to 200 psi for medium to heavy clays (ref. 115). An 8-in thickness is the maximum depth for standard length feet on the rollers (ref. 113). Minimum density required is 95 percent of the maximum density of the field soil/cement as determined by AASHO T134 or ASTM D558.

LIME/CEMENT STABILIZATION PROCEDURES

The procedures for the construction of lime/cement stabilized soil are identical to those for the individual stabilizers. The expense and time involved in using both materials are justified only if a heavy clay soil (PI > 30) is not lime reactive and will not mi. with cement. In this situation, the lime improves the workability and reduces the plasticity index but does not significantly increase the strength. Lime is added to produce a more friable material for cement stabilization. Some degree of interaction occurs but this mechanism is not understood and little research has been performed (ref. 116). Combination stabilization is normally used for soils that cannot be economically stabilized otherwise.

SECTION 8
CONCLUSIONS AND RECOMMENDATIONS

EXPANSIVE SOIL DESIGN

Engineering problems associated with expansive soils are significant and warrant the implementation of special design procedures to supplement those normally used. Expansive soils may be detected by observing the performance of engineering structures. When such observations are impossible or inconclusive, other means are needed. An economical and fast test is desirable to provide an early indication that special testing and design are needed. The most meaningful indicators are the plasticity index and the linear shrinkage. In the initial investigation, the soil should be rated as follows:

	Soil Expansion Potential		
Soil Property	Low	Medium	High
Plasticity Index, %	≤ 10	$> 10, < 20$	> 20
Linear Shrinkage, %	≤ 8	$> 8, < 12$	> 12

Soils in the low category require no special provisions to account for swell in the design of airport pavements; soils in the high category require evaluation of potential heave. The medium category serves as an alert to the designer. In this case further study should be made to evaluate previous experience with similar structures of at least 5 years of age in the area. The designer must determine whether further tests to place the soil in the high or low category are justified. Soils in the high category are studied by the swell test described in appendix A. The test samples should represent each significant soil layer being evaluated. Representative samples of compacted or undisturbed layers to a depth beyond which no significant change in the soil condition is anticipated are required. In thick clay soil deposits this depth may be 50 ft or more. The percentage of swell estimated by the test and the layer thickness, of which the sample is representative, are used to compute the heave. The subgrade surface heave is the sum of the heaves of all underlying soil layers.

The test procedure recommended is the best implementable test found in the literature for evaluating swell and predicting subgrade heave; however, there are limitations: moisture conditions of the test are significantly different from those in-situ, and no conclusions regarding the rate of swell can be made from test results. Caution must be exercised in preserving the in-situ moisture condition and structure of the undisturbed samples. In this test, it is assumed that soil under the pavement will gain moisture after construction. Thus, before using the test data for design, initial and final estimated moisture conditions should be evaluated to determine the validity of the final moisture content in the swell test. Deformation and inconsistancies of the measuring apparatus and loading frame must be evaluated and corrections must be made in the data analysis. Time required for the test is excessive (up to 6 weeks per test).

The variation in heave from one area to another is called *differential heave*. The differential heave of subgrade soils is the cause of pavement failure. The normal procedure for designing on expansive soils is to assume that the heave measured in the swell test is the differential heave. It is important to note that this is an assumption, and the validity should be considered in each case. A study of heave values, soil variation, and drainage is needed to select a design value for differential heave.

The design differential heave must be compared to an allowable differential heave, but no procedure is available for computing this allowable differential heave. A structural analysis of the pavement section to be built must be made. The analysis consists of placing a mound of soil equal to the height of the predicted swell and computing the effect on the pavement section (fig. 27). Stress and strain in the pavement section and surface roughness caused by the mound must be considered. Surface roughness is one of the first indications of differential movement in the subgrade soil. Adequate procedures for dealing with expansive soil induced roughness are not reported in the technical literature. At this time, the allowable differential heave must be determined by structural analysis techniques.

A stabilization objective is established as the difference between design heave and allowable heave. When the design differential heave exceeds the allowable, some action must be taken to reduce it. This reduction may be accomplished in many ways, but every effort to reduce the design differential heave must be

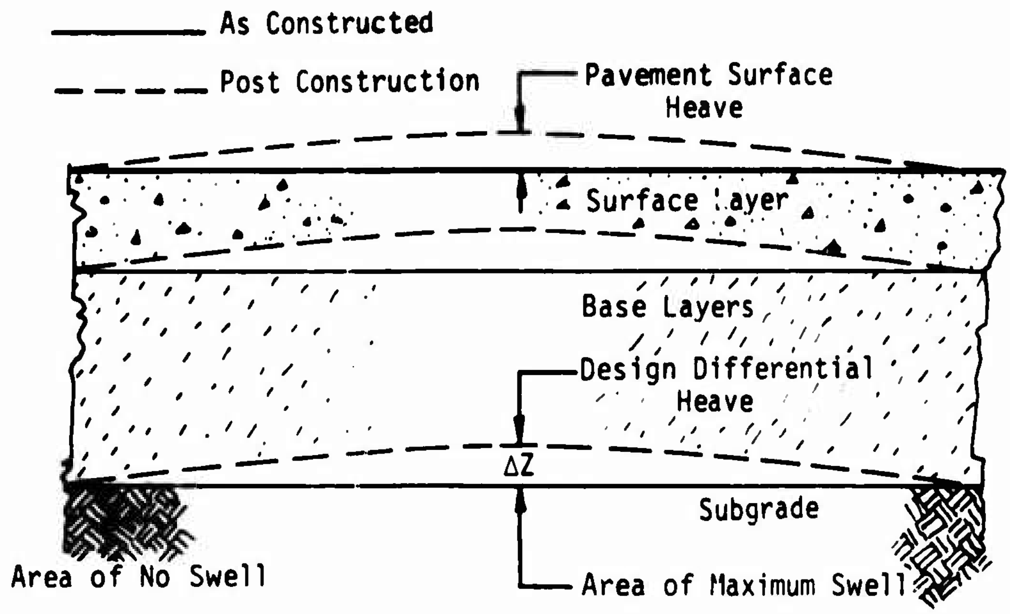


Figure 27. Diagram for Computing Allowable Heave

quantitatively evaluated. Additional swell tests are required to evaluate the swell of stabilized soils (i.e., lime treated). The addition of lime or cement admixtures to a soil reduces but does not eliminate swell. All layers affected by treatment must be tested after treatment to determine the final heave for comparison with the allowable heave. A stabilization system must not only adequately reduce the heave to below the allowable, but it must also meet the conventional strength and durability requirements. The procedures contained in this report are satisfactory for designing lime- and cement-stabilized layers. These procedures have been developed from thorough laboratory studies and they are sound, implementable procedures. The overall design procedure is shown in figure 28.

RESEARCH NEEDS

The expansive soil design method described above has specific limitations. However, the technology required to overcome these limitations exists in every

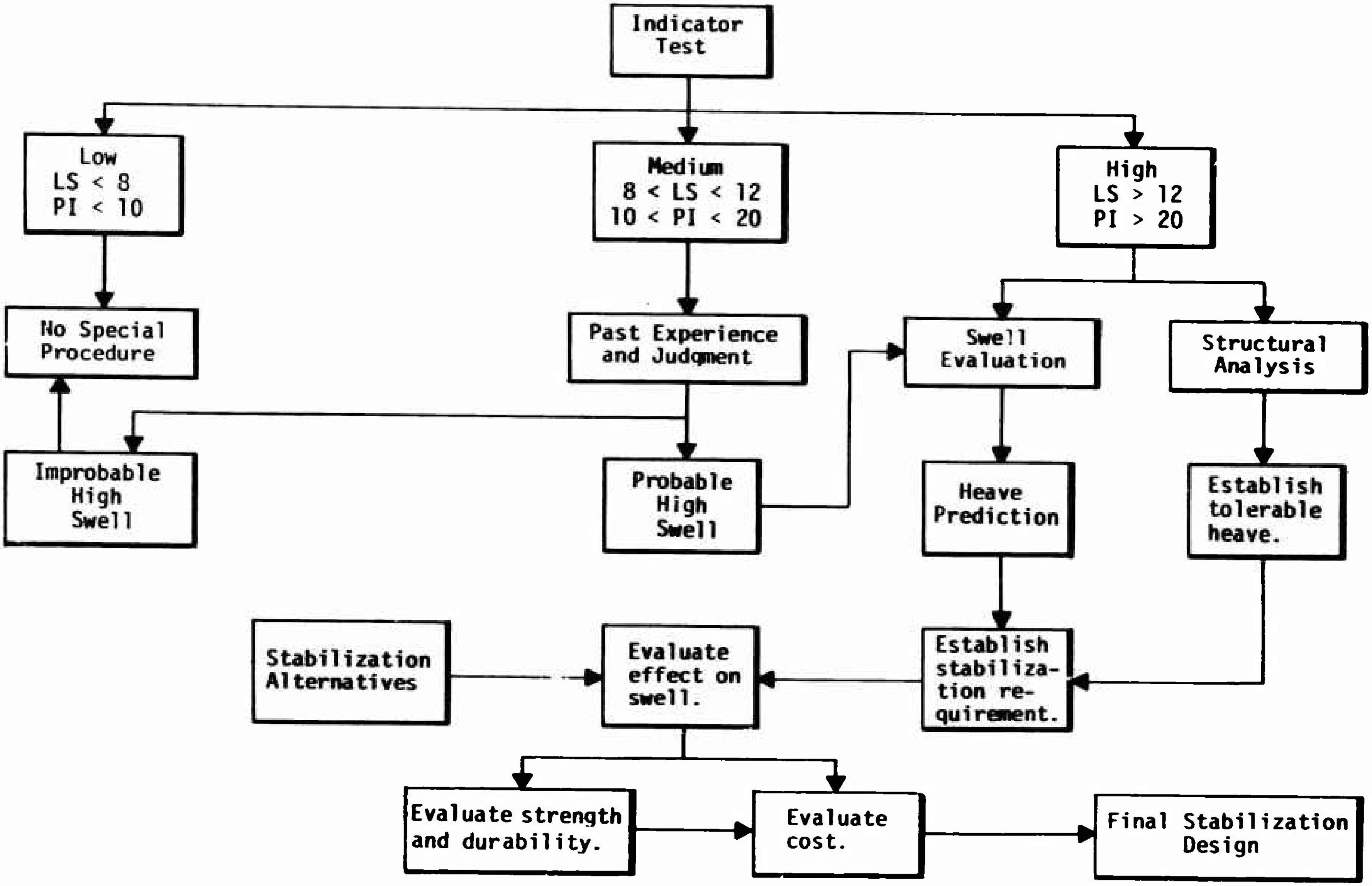


Figure 28. Recommended Design Procedure

case. Research is required to transform what are now research tools into implementable design tools. Simple expansive-soil tests have evolved over many years on a piecemeal basis, and no comprehensive, large-scale research effort has been directed toward improving these simple procedures for evaluating swell potential. In two programs funded by the Federal Highway Administration, various simple test procedures are being compared. Hopefully, a procedure that will yield a better correlation than 0.6 (the best available today) between predicted and measured swell potential will be developed. A reliable simple test is required to determine the need for detailed heave prediction.

No simple procedure to accurately predict in-situ heave will be developed. It has been established in the literature (refs. 117, 118) that initial load and soil suction and final load and soil suction are the critical factors which determine soil behavior. The data needed to predict in-situ soil behavior are the initial and final conditions and the response of the soil to the change. Structural design of pavements requires consideration of loads before and after construction; therefore, the load data are routinely available. Soil suction, however, is relatively new to engineering. Soil suction is a measure of the energy balance of the soil water (appendix D). Until recently, measurement of high in-situ soil water suction (i.e., low water content) was not practical. Progress during the last ten years in the field of soil science has produced thermocouple psychrometers for measuring suction in the field. These instruments, together with conventional hydraulic tensiometers, can be used to cover the full range of soil suction encountered in natural soils (fig. 29). The Thornthwaite Index, an indicator of the moisture balance between the atmosphere and the soil, has been correlated with equilibrium soil suction (refs. 74, 88, 119). Drier soils could be tested in the laboratory under controlled conditions using undisturbed samples. Testing is needed to determine the moisture content/disturbance relationship for various soils and to provide values for a and b (fig. 29). A test program in which currently available equipment could be used is needed to develop a procedures manual for the use of thermocouple psychrometers and hydraulic tensiometers in establishing initial soil suction in expansive soils.

The final equilibrium soil suction under pavements has been studied extensively. Recent developments with mathematical models for predicting moisture movement

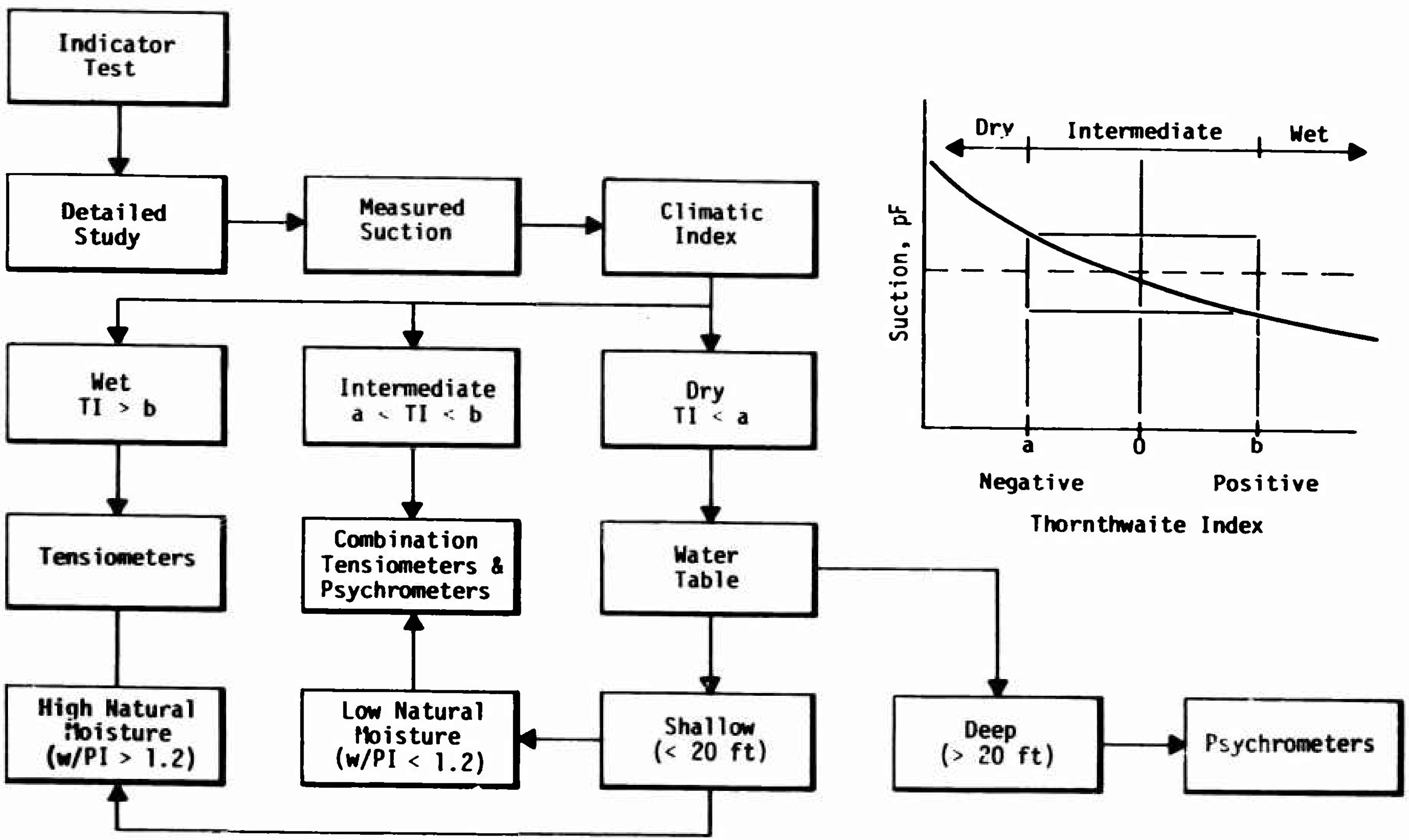


Figure 29. Suction Measurement Procedures

are promising but as yet far from implementable. An estimate can be made by measuring the suction at a depth below seasonal influence and using the procedure described in appendix C to provide values for the rest of the soil. It is important to recognize that once initial and final design conditions are selected, they must be maintained through proper attention to surface drainage, water table fluctuations, and other sources of change.

To complete the data required for expansive soil design, the soil's response to the change from initial to final conditions must be characterized. Data used for this purpose are obtained with an oedometer that provides for independent load and soil-suction control. With the slope of the linear strain/suction curve (fig. 30), computations are made to predict swell. Because of its complexity, however, the oedometer is not an instrument that can be used for routine design testing. Correlation of the slope of the linear strain/suction curve with simple soil properties promises to provide an implementable technique (ref. 94). Testing must be performed on a wide range of soils to establish the necessary correlations. The design of stabilized soil layers would be facilitated by developing similar data for stabilized materials (dashed lines in figure 30). The present methods, based on strength and durability, provide nothing for the designer to use in estimating the heave reduction attained through stabilization.

The remaining weakness of the current state-of-the-art is in establishing the allowable differential subgrade heave below a pavement; structural analysis is cumbersome for routine design work. It seems reasonable that categories for pavement sections may be established in terms of allowable heave. Recent studies of pavement surface roughness have demonstrated the capability of present technology in describing the allowable pavement roughness (ref. 120). It remains to establish the relationships between pavement characteristics, subgrade properties, and roughness. This would permit the setting of acceptable levels of roughness in terms of subgrade differential heave. Figure 31 illustrates the use of such data in establishing the most economical combination of pavement stiffness and stabilizer for an allowable level of roughness.

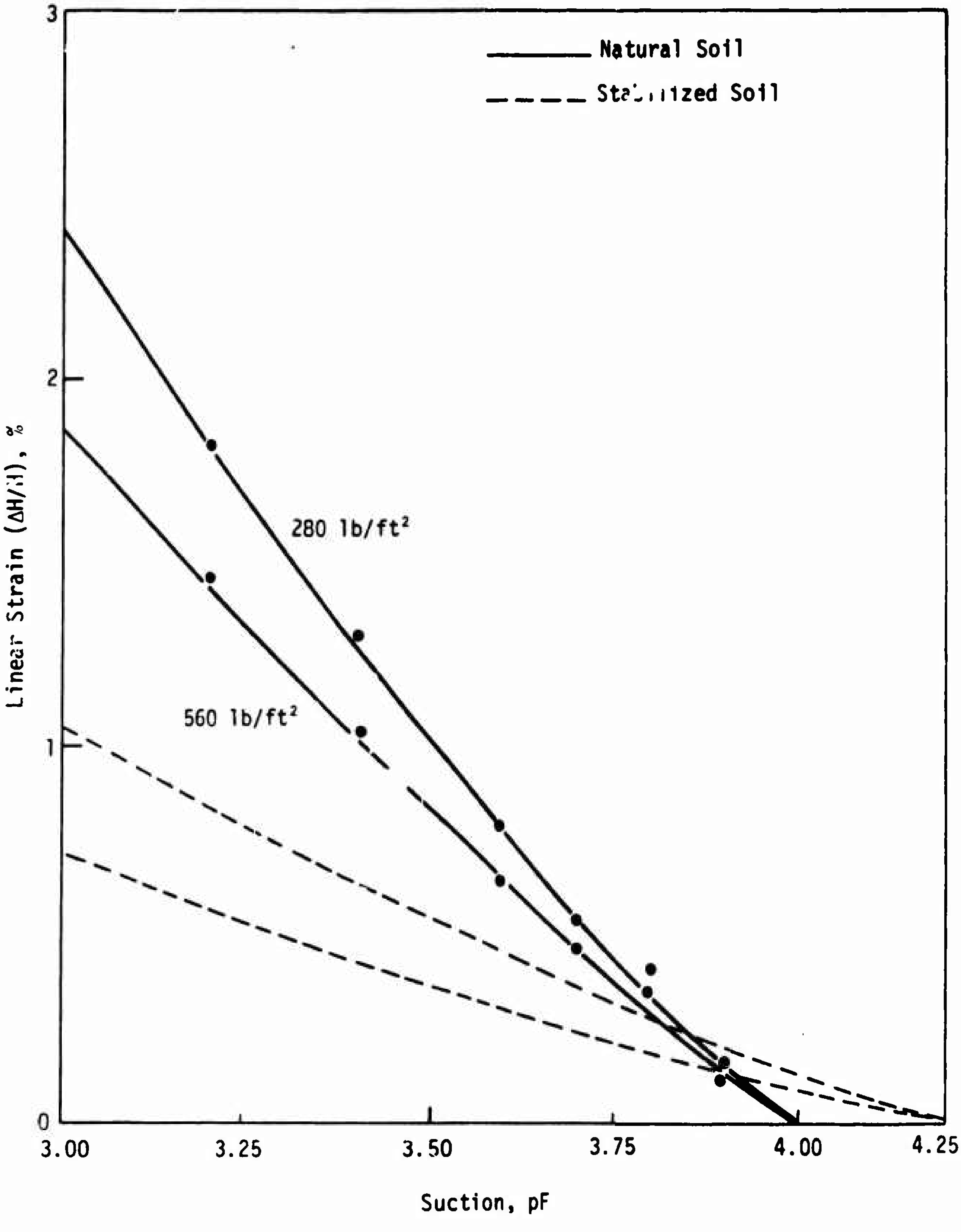
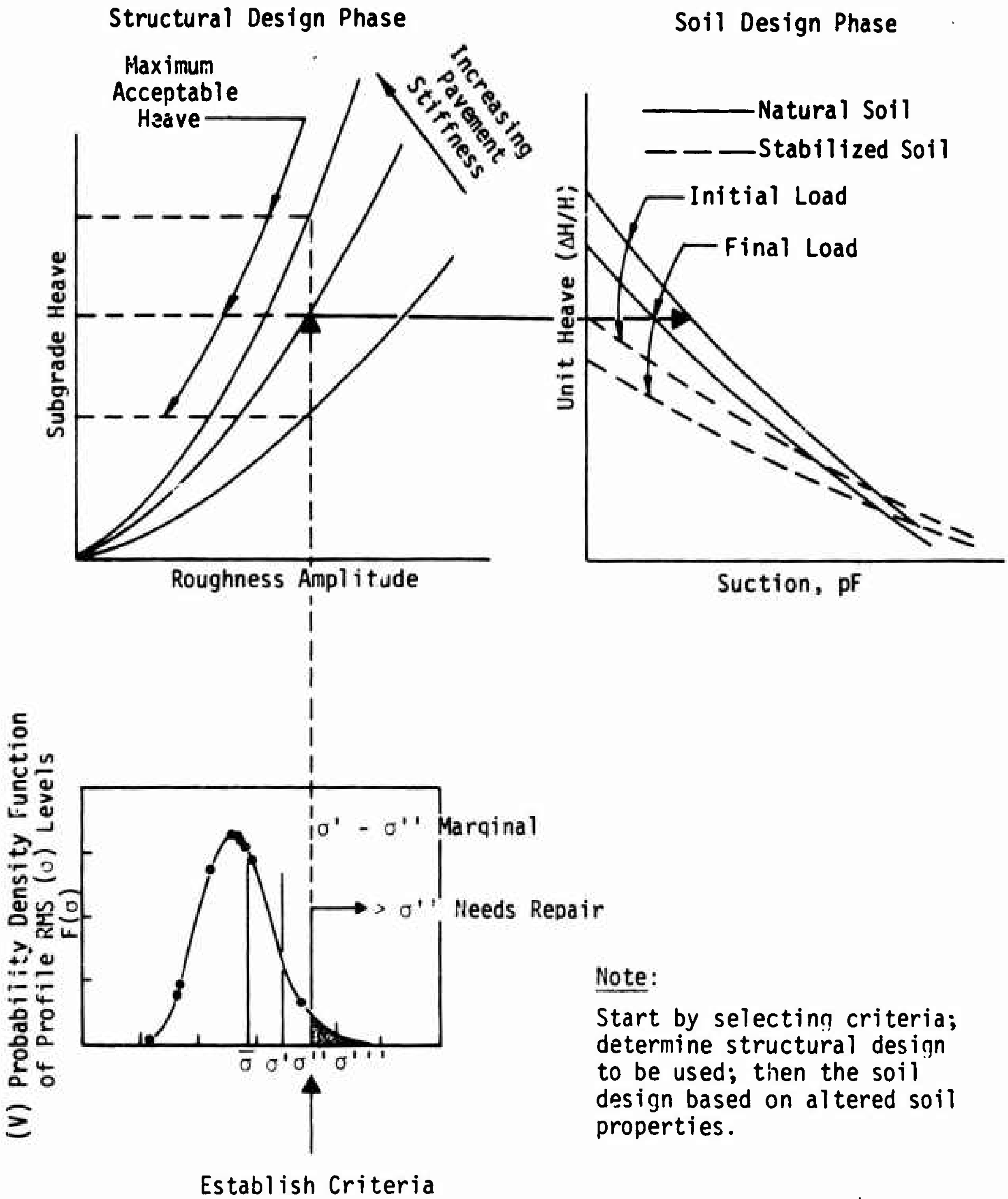


Figure 30. Evaluation of Natural and Stabilized Soils



Note:
Start by selecting criteria; determine structural design to be used; then the soil design based on altered soil properties.

Figure 31. Heave/Roughness Amplitude Relationship for Design

IMPLEMENTATION

The procedure presented offers no marked improvement over currently used techniques. The swell test has been utilized to predict heave for many years. The improvement of this procedure is dependent on further research. The method presented here for expansive soil stabilization is derived directly from the U.S. Air Force system. Current criteria are based on strength and durability and no provision is made to determine the swell reduction associated with the stabilizers.

All tests recommended in this report are implementable. Their use, however, will not provide a marked improvement over the procedures currently employed by any conscientious soil engineer. The procedures are not new and have changed little in the last ten years. The sequence of tests for stabilization design is new. It was recently developed for the U.S. Air Force and provides a well-based system which utilizes the strength and durability of the stabilized materials as the design criteria. Implementation of all recommendations in this report offers little progress in dealing with expansive soils. The procedure for soil stabilization, however, is a significant improvement and should be implemented.

APPENDIX A
SWELL TEST PROCEDURE

SUGGESTED METHOD OF TEST FOR ONE-DIMENSIONAL EXPANSION AND UPLIFT PRESSURE OF CLAY SOILS[1]

SUBMITTED BY W. G. HOLTZ[2]

1. Scope

1.1 This method explains how to make expansion tests on undisturbed or compacted clay soil samples that have no particle sizes greater than $\frac{3}{16}$ in. (passing the No. 4 standard ASTM sieve[3]). The test is made to determine (*1*) magnitude of volume change under load[4] or no-load conditions, (*2*) rate of volume change, (*3*) influence of wetting on volume change, and (*4*) axial permeability of laterally confined soil under axial load or no load during expansion. Saturation (no drainage) takes place axially. Permeant water is applied axially for determining the effect of saturation and permeability. The specimens prepared for this test may also be used to determine the vertical or volume shrinkage as the water content decreases. Total volume change for expansive soils is determined from expansion plus shrinkage values for different ranges of water content.

1.2 Expansion test data may be used to estimate the extent and rate of uplift in subgrades beneath structures or in structures formed from soils, and shrinkage tests may be used to estimate the volume changes which will occur in soils upon drying, provided that natural conditions and operating conditions are duplicated.

2. Significance

2.1 The expansion characteristics of a soil mass are influenced by a number of factors. Some of these are size and shape of the soil particles, water content, density, applied loadings, load history and mineralogical and chemical properties. Because of the difficulty in evaluating these individual factors, the volume-change properties cannot be predicted to any degree of accuracy unless laboratory tests are performed. When uplift problems are critical, it is important to test samples from the sites being considered.

2.2 The laboratory tests described herein are primarily intended for the study of soils having no particles larger than the No. 4 standard sieve size ($\frac{3}{16}$ in.). If the test is made on the minus No. 4 fraction of soils containing gravel material (plus No. 4), some adjustment is required in any analysis. Gravel reduces volume change because it replaces the more active soil fraction.

3. Apparatus

3.1 *Consolidometer*—Conventional laboratory consolidometers are used for the expansion test. Consolidometers most used in the United States are of the fixed-ring and floating-ring types. Figure *1* illustrates the fixed-ring type. Either

[1] This suggested method has no official status in the Society but is published as information only. The method is based on the experience of the submitter. Comments are solicited.

[2] Consulting Civil Engineer, Denver, Colo.

[3] See ASTM Specification E 11, for Wire-Cloth Sieves for Testing Purposes, *Annual Book of ASTM Standards*, Part 30.

of these is suitable. Both types are available commercially. In the fixed-ring container, all specimen movement relative to the container is upward during expansion. In the floating-ring container, movement of the soil sample is from the top and bottom away from the center during expansion. The specimen containers for the fixed-ring consolidometer and the floating-ring consolidometer consist of brass or plastic rings, and other sion tests the larger diameter consolidation rings are preferred as they restrain the soil action to a lesser degree. In a test using the floating-ring apparatus, the friction between the soil specimen and container is smaller than with the fixed-ring apparatus. On the other hand, the fixed-ring apparatus is more suitable for saturation purposes and when permeability data are required. Porous stones are required at the top and bottom of the

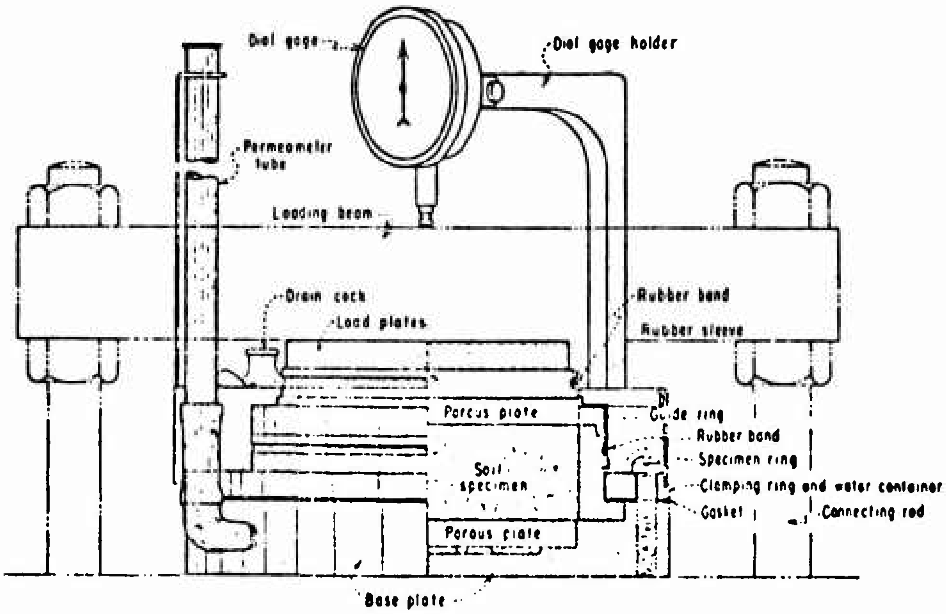


FIG. 1 - Fixed-Ring Consolidometer.

component parts. Sizes of container rings most commonly used vary between 4¼-in. diameter by 1¼ in. deep and 2½-in. diameter by ¾ in. deep, although other sizes are used. However, the diameter should be not less than 2 in. and the depth not greater than three tenths of the diameter, except that the depth must not be less than ¾ in. for specimens of small diamter. Lesser depths introduce errors caused by the magnitude of surface disturbance, while large depths cause excessive side friction. For expan- specimen to allow application of water. The apparatus must allow vertical movement of the top porous stone for fixed-ring consolidometers, or vertical movement for top and bottom porous stones for floating-ring consolidometers, as expansion takes place. A ring gage machined to the height of the ring container to an accuracy of 0.001 in. is required; thus, the ring gage for 1¼-in.-high specimens will have a height of 1.250 in. Measure the diameter of the specimen container ring to 0.001 in.

3.2 *Loading Device*—A suitable device for applying vertical load to the specimen is required. The loading device may be platform scales of 1000 to 3000-lb capacity mounted on a stand and equipped with a screw jack attached underneath the frame. The jack operates a yoke which extends up through the scale platform and over the specimen container resting on the platform. The yoke is forced up or down by operating the jack, thus applying or releasing load to the soil specimen. The desired applied pressure, which is measured on the scale beam, becomes fully effective when the beam is balanced.

3.2.1 Another satisfactory loading device utilizes weights and a system of levers for handling several tests simultaneously. Hydraulic-piston or bellows-type loading apparatus are also very satisfactory if they have adequate capacity, accuracy, and sensitivity for the work being performed. Apparatus such as described in ASTM Method D 2435, Test for One-Dimensional Consolidation Properties of Soils,[4] is satisfactory and may be used.

3.3 *Device for Cutting Undisturbed Specimens*--This apparatus consists of a cutting bit of the same diameter as the ring container of the consolidometer, a cutting stand with bit guide, and knives for trimming the soil. Wire saws or trimming lathes may be used if a uniform tight fit of the specimen to the container is obtained.

3.4 *Device for Preparation of Remolded Specimens*--Compacted soil specimens are prepared in the consolidometer ring container. In addition to the container, the apparatus consists of an extension collar about 4 in. in depth and of the same diameter as the container. A compaction hammer of the same type required in Method A of ASTM Method D 698, Test for Moisture-Density Relations of Soils, Using 5.5-lb Rammer and 12-in. Drop.[4]

[4] *Annual Book of ASTM Standards*, Part 11.

4. Procedure-Expansion Test

4.1 *Preparation of Undisturbed Specimens*—Perform the tests on hand-cut cube samples or core samples of a size that will allow the cutting of approximately ½ in. of material from the sides of the consolidometer specimen. (Alternatively, obtain a core of a diameter exactly the same as the diameter of the consolidometer specimen container, and extrude the core directly into the container. This procedure is satisfactory provided that the sampling has been done without any sidewall disturbance and provided that the core specimen exactly fits the container. Place the undisturbed soil block or core on the cutting platform, fasten the cutting bit to the ring container, and place the assembly on the sample in alignment with the guide arms. With the cutting stand guiding the bit, trim the excess material with a knife close to the cutting edge of the bit, leaving very little material for the bit to shave off as it is pressed gently downward. (Other suitable procedures to accommodate guides for wire saws, trimming lathes, or extrusion devices may be used in conformance with the use of alternative apparatus and samples.) In trimming the sample, be careful to minimize disturbance of the soil specimen and to assure an exact fit of the specimen to the consolidometer container. When sufficient specimen has been prepared so that it protrudes through the container ring, trim it flush with the surface of the container ring with a straightedge cutting tool. Place a glass plate on the smooth, flat cut surface of the specimen, and turn the container over. Remove the cutting bit, trim the specimen flush with the surface of the container ring, and cover it with a second glass plate to control evaporation until it is placed in the loading device.

4.2 *Preparation of Remolded Specimens*—Use about 2 lb of representative soil that has been properly moistened to the degree desired and processed free from lumps and from which particles or aggregations of particles retained by a 3/16-in. (No. 4) sieve have been excluded. Compact the specimen to the required wet bulk density after adding the required amount of water as follows: Place the extension collar on top of the container ring and fasten the bottom of the container ring to a baseplate. Weigh the exact quantity of the processed sample to give the desired wet density when compacted to a thickness 1/4 in. greater than the thickness of the container ring. Compact the specimen to the desired thickness by the compaction hammer. Remove the extension collar and trim the excess material flush with the container ring surface with a straightedge cutting tool. Remove the ring and specimen from the baseplate and cover the specimen surfaces with glass plates until the specimen is placed in the loading device. If, after weighing and measuring the specimen and computing the wet density, as described below, the wet density is not within 1.0 lb/ft^3 of that required, repeat the preparation of the remolded specimen until the required accuracy is obtained.

4.3 *Calibration of Dial Gage for Height Measurements*—Prior to filling the container ring with the soil specimen, place a ring gage in the specimen container with the same arrangement of porous plates and load plates to be used when testing the soil specimen. Place the assembly in the loading machine in the same position it will occupy during the test. After the apparatus has been assembled with the ring gage in place, apply a load equivalent to a pressure of 0.35 psi (or 0.025 kgf/cm^2) on the soil specimen. The dial reading at this time will be that for the exact height of the ring gage. Mark the parts of the apparatus so that they can be matched in the same position for the test.

4.4 *Initial Height and Weight of Soil Specimen*—Clean and weigh the specimen container ring and glass plates and weigh them to ±0.01 g before the ring is filled. After filling and trimming is completed, weigh the soil specimen, ring, and glass plates to ±0.01 g. Determine the weight of the soil specimen. Assemble the specimen container and place it in the loading device. If the specimen is not to be saturated at the beginning of the test, place a rubber sleeve around the protruding porous plates and load plates to prevent evaporation. Apply the small seating load of 0.35 psi (or 0.025 kgf/cm^2) to the specimen. By comparing the dial reading at this time with the dial reading obtained with the ring gage in place, determine the exact height of the specimen. Use this information to compute the initial volume of the specimen, the initial density, void ratio, water content, and degree of saturation. The true water content of the specimen will be determined when the total dry weight of the specimen is obtained at the end of the test.

4.5 *Saturation and Permeability Data*—To saturate the specimen attach the percolation tube standpipe, fill it with water, and wet the specimen. Take care to remove any air that may be entrapped in the system by slowly wetting the lower porous stone and draining the stone through the lower drain cock. After the specimen is wetted, fill the pan in which the consolidometer stands with water. After saturation has been completed, permeability readings can be taken at any time during the test by filling the percolation tube standpipe to an initial reading and allow the water to percolate through the specimen. Measure the amount of water flowing through the sample in a given time by the drop in head.

4.6 *Expansion Test:*

4.6.1 *General Comments*—The expansion characteristics of an expansive-type soil vary with the loading history, so that it is necessary to perform a separate test or several specimens for each condition of loading at which *exact* expansion data are required. However, one procedure is to test only two specimens: (*1*) loaded-and-expanded, and (*2*) expanded-and-loaded. From these data, an estimate of expansion can be made for any load condition as shown by Curve C, Fig. 2, in which Specimen No. 1 was loaded and expanded by saturation with water, (Curve B) and Specimen No. 2 was expanded by saturation with water and then loaded (Curve A).

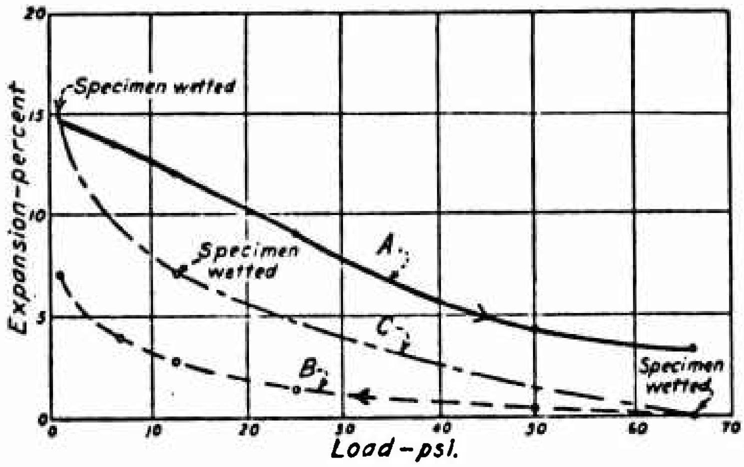


FIG. 2—Load-Expansion Curves.

4.6.2 *Loaded and Expanded Test*—To measure expansion characteristics where the soil specimen is saturated under full load and then allowed to expand, apply the seating load of 0.35 psi (or 0.025 kgf/cm^2) to Specimen No. 1, and secure initial dial readings. Then saturate the soil specimen as described in 4.5. (The permeameter tube head should be sufficiently low so that the specimen is not lifted.) As the specimen begins to expand, increase the load as required to hold the specimen at its original height. Then reduce the load to $\frac{1}{2}$, $\frac{1}{4}$, and $\frac{1}{8}$ of the maximum load and finally to the seating load of 0.35 psi (or 0.025 kgf/cm^2) and measure the height with each load. Use a greater number of loadings if greater detail in the test curve is required. Maintain all loads for 24 h, or longer if needed, to obtain constant values of height. Remove the specimen from the ring container and weigh it immediately and again after drying to 105 C. From the water content, dry bulk density, and specific gravity of the specimen, calculate the volume of air and, assuming it to be the same as the volume of air following the determination of permeability, calculate the water content and degree of saturation.

4.6.3 *Expanded and Loaded Test*—To measure expansion characteristics where the soil is allowed to expand before loading, apply the seating load of 0.35 psi (or 0.025 kgf/cm^2) to Specimen No. 2, and secure initial dial gage readings.

Then saturate the specimen as described in 4.5. Allow the specimen to expand under the seating load for 48 h or until expansion is complete. Load the specimen successively to $\frac{1}{8}$, $\frac{1}{4}$, $\frac{1}{2}$ and 1 times the maximum load found in 4.6.2, to determine the reconsolidation characteristics of the soil. Use a greater number of loadings, if greater detail in the test curve is required. Follow the procedures specified in 4.6.2 for making loadings and all measurements and determinations.

4.6.4 *Individual Load Expansion Test*—When it is desired to perform separate expansion tests for other conditions of loading apply the seating load of 0.35 psi (or 0.025 kgf/cm²) to the specimen and measure the initial height. Then load the specimen to the desired loading, saturate the specimen as described in 4.5, and allow the specimen to expand under the applied load for 48 h, or until expansion is complete. Measure the height of the expanded specimen. Reduce the load to that of the seating load. Allow the height to become constant and measure; then remove the specimen from the ring and make the determination specified in 4.6.2.

5. Procedure—Shrinkage Test

5.1 *Specimen Preparation*—When measurements of shrinkage on drying are needed, prepare an additional specimen as described in 4.1 or 4.2. Cut this specimen from the same undisturbed soil sample as the expansion specimens, or remolded to the same bulk density and water content conditions as the expansion specimens. Place the specimen in the container ring, and measure the initial volume and height as described in 4.4. Determine the water content of the soil specimen by weighing unused portions of the original sample of which the specimen is a part, drying the material in an oven to 105 C, and reweighing it.

5.2 *Volume and Height Shrinkage Determinations*—To measure volume shrinkage, allow the specimen in the ring to dry in air completely or at least to the water content corresponding to the shrinkage limit (ASTM Method D 427, Test for Shrinkage Factors of Soils).[4] After the specimen has been air-dried, remove it from the ring container, and obtain its volume by the mercury-displacement method.

5.2.1 To perform the mercury displacement measurement, place a glass cup with a smoothly ground top in an evaporating dish. Fill the cup to overflowing with mercury, and then remove the excess mercury by sliding a special glass plate with three prongs for holding the specimen in the mercury over the rim. Pour the excess mercury into the original container and replace the glass cup in the evaporating dish. Then immerse the air-dried soil specimen in the glass cup filled with mercury using the special glass plate over the glass cup to duplicate the initial mercury volume determination condition. (See Method D 427 for general scheme of test and equipment.) Transfer the displaced mercury into a graduated cylinder, and measure the volume. If the shrinkage specimen is cracked into separate parts, measure the volume of each part, and add the individual volumes to obtain the total. (A paper strip wrapped around the specimen side and held by a rubber band is effective in holding the specimen intact during handling.)

5.2.2 If the height of the air-dried specimen is desired, place the specimen and ring container in the loading machine. Apply the seating load of 0.35 psi (or 0.025 kgf/cm²), and then read the dial gage.

6. Calculations

6.1 *Expansion Test Data*—Calculate the void ratio as follows:

$$e = \frac{\text{volume of voids}}{\text{volume of solids}} = \frac{h - h_s}{h_s}$$

where:

e = void ratio,

h = height of the specimen, and

h_s = height of the solid material at zero void content

Calculate the expansion, as a percentage of the original height, as follows:

$$\Delta, \text{percent} = \frac{h_2 - h_1}{h_1} \times 100$$

where:

Δ = expansion in percentage of initial volume,

h_1 = initial height of the specimen, and

h_2 = height of the specimen under a specific load condition.

6.2 *Permeability Test Data*—Calculate the permeability rate by means of the following basic formula for the variable head permeameter:

$$k = \frac{A_p \times L_s}{A_s \times 12} \times \frac{1}{t} \ln \frac{H_i}{H_f}$$

where:

k = permeability rate, ft/year,

A_p = area of standpipe furnishing the percolation head, in.²,

A_s = area of the specimen, in.²,

L_s = length of the specimen, in.,

H_i = initial head, difference in head between headwater and tailwater, in.,

H_f = final head, difference in head between headwater and tailwater, in., and

t = elapsed time, years.

6.3 *Shrinkage Test Data*—Calculate the volume shrinkage as a percentage of the initial volume as follows:

$$\Delta_s = \frac{v_i - v_d}{v_i} \times 100$$

where:

Δ_s = volume shrinkage in percentage of initial volume,

v_i = initial volume of specimen (height of specimen times area of ring container), and

v_d = volume of air-dried specimen from mercury displacement method.

Calculate the shrinkage in height as follows:

$$\Delta_{h_s} = \frac{h_i - h_d}{h_i} \times 100$$

where:

Δ_{h_s} = height of shrinkage in percentage of initial height,

h_i = initial height of specimen, and

h_d = height of air-dried specimen.

6.3.1 To calculate the total percentage change in volume from "air-dry to saturated conditions," add the percentage shrinkage in volume on air drying Δ_s to the percentage expansion in volume on saturation Δ_e, as described in 6.1. This value is used as an indicator of total expansion but is based on initial conditions of density and water content. Since expansion volume data are determined for several conditions of loading, the total volume change can also be determined for several conditions of loading.

6.3.2 To calculate the total percentage change in height from saturated to air-dry conditions, add the percentage shrinkage in height Δ_{h_s} to the percentage expansion Δ when the specimen is saturated under specific load conditions.

7. Plotting Test Data

7.1 *Expansion Test*—The test data may be plotted as shown on Fig. 2.

8. Reports

8.1 *Expansion Test*—Include the following information on the soil specimens tested in the report:

8.1.1 Identification of the sample (hole number, depth, location).

8.1.2 Description of the soil tested and size fraction of the total sample tested.

8.1.3 Type of sample tested (remolded or undisturbed; if undisturbed, describe the size and type, as extruded core, hand-cut, or other).

8.1.4 Initial moisture and density conditions and degree of saturation (if remolded, give the comparison to maximum density and optimum water content (see Methods D 698)).

8.1.5 Type of consolidometer (fixed or floating ring, specimen size), and type of loading equipment.

8.1.6 A plot load versus volume change curves as in Fig. 1. A plot of void ratio versus log of pressure curve may be plotted if desired.

8.1.7 A plot log of time versus deformation if desired.

8.1.8 Load and time versus volume-change data in other forms if specifically requested.

8.1.9 Final water content, bulk dry density, and saturation degree data.

8.1.10 Permeability data and any other data specifically requested.

8.2 *Shrinkage Test*—For the report on shrinkage, include data on the decrease in volume from the initial to air-dried condition and, if desired, other information such as the total change in volume and total change in height. Report the load conditions under which the volume change measurements were obtained. Include also Items 8.1.1 through 8.1.5 and 8.1.9.

APPENDIX B
LINEAR SHRINKAGE TEST

The apparatus used in the linear shrinkage test is shown in figure 1. Soil-preparation methods for the test and the actual test procedure for volumetric shrinkage are adequately described under Method D-427 in *Procedures for Testing Soils* by the American Society for Testing Materials. The following excerpts are from reference 61:

> "The linear shrinkage of a soil binder is the percentage of shrinkage, on the basis of the original wet length, acquired by a soil bar in drying from its liquid limit down to its shrinkage limit, i.e., the moisture content at which all shrinkage ceases. The preliminary shrinkage of soils containing water in excess of the liquid limit is vertically downward so that the linear shrinkage of the soil bar as measured in this test will not be materially changed if the specimen is molded with an amount of water slightly in excess of the liquid limit. In fact, more consistent results will be obtained with samples that are mixed too wet than with those mixed too dry, provided that the mixture is not wet enough to allow segregation of the larger particles to the bottom."
>
> "Apparatus for test: (a) One 4-in spatula (stainless steel), (b) one 4-in evaporating dish, (c) one metal mold, preferably stainless steel (fig. 1), (d) one drying oven that can be used to dry out samples between 200 and 225°F, (e) thermometer graduated to 400°F by 2-deg intervals, (f) grooving tool, (g) Vaseline for greasing shrinkage molds, and (h) one stainless steel shrinkage gage."
>
> "The importance of a thorough and uniform mixing of the sample with distilled water in this and all other tests cannot be overemphasized."
>
> "A test to determine when the proper consistency for molding is reached is performed by shaping the sample into a smooth layer about 1/2 in thick on the bottom or side of the container. A liquid limit grooving tool is then placed against the bottom of the dish and drawn through the layer of soil. If the material just flows into and closes the groove at the bottom on its own accord the sample is at the proper consistency for molding."
>
> "The inside walls of the mold should be thinly greased with Vaseline before the specimen is formed; this will prevent the sample from sticking to the walls of the mold. The material should be worked evenly into the mold and made to fill it completely with a gentle jarring of the mold to assist in the removal of any entrapped air bubbles."

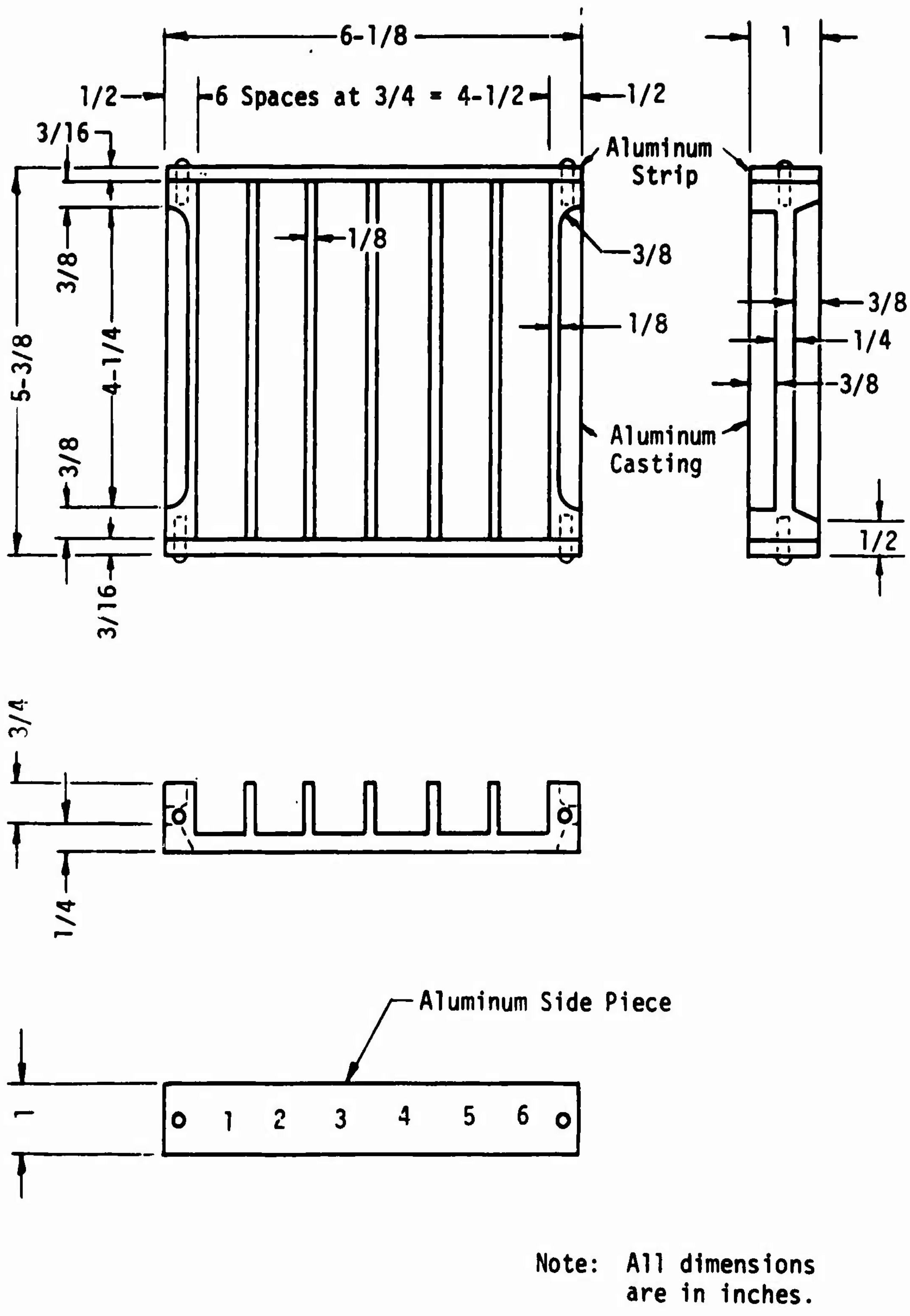


Figure 1. Test Apparatus [after Heidema (ref. 61)]

"The stainless steel shrinkage gage may be used to measure the shrinkage directly in percentage."

"An investigation by the Highway Laboratory showed that accuracy closer than ± 2 percent cannot be assured by the bar method." In other words, if the measured linear shrinkage of the bar is 20 percent, the actual shrinkage may be as low as 19.6 or as high as 20.4 percent.

To these remarks should be added the statement that the molded specimens are to be air-dried overnight in an air-drier prior to being placed in the drying oven. If this is not done, some of the more cohesive specimens would tend to curl up and this would make it impossible to measure the shrinkage accurately.

APPENDIX C

ESTIMATION OF FINAL EQUILIBRIUM MOISTURE CONTENT UNDER PAVEMENTS

A number of researchers have studied the problem of predicting final equilibrium moisture content beneath covered areas such as pavements. It is appreciated that saturation, as used in most swell tests, is a poor approximation. A review of the methods reported (refs. 51, 73, 74, 75, 88, 90, 121) resulted in a rather simple procedure (fig. 1). In areas where a water table is shallow (< 20 ft), the increase in suction from the water table to the surface is roughly 1 cm H_2O per 1 cm vertical rise (refs. 71, 90). In areas with deep water tables, the equilibrium moisture content below the level of seasonal fluctuation may be assumed to equal the equilibrium moisture content under a pavement after construction.

Significant progress is reported on the development of theoretical models for prediction of moisture changes in soils beneath pavements (refs. 122, 123). These approaches are certainly more realistic than the procedure just described, but considerable research is required to develop implementable procedures for theoretical models and the related computer codes.

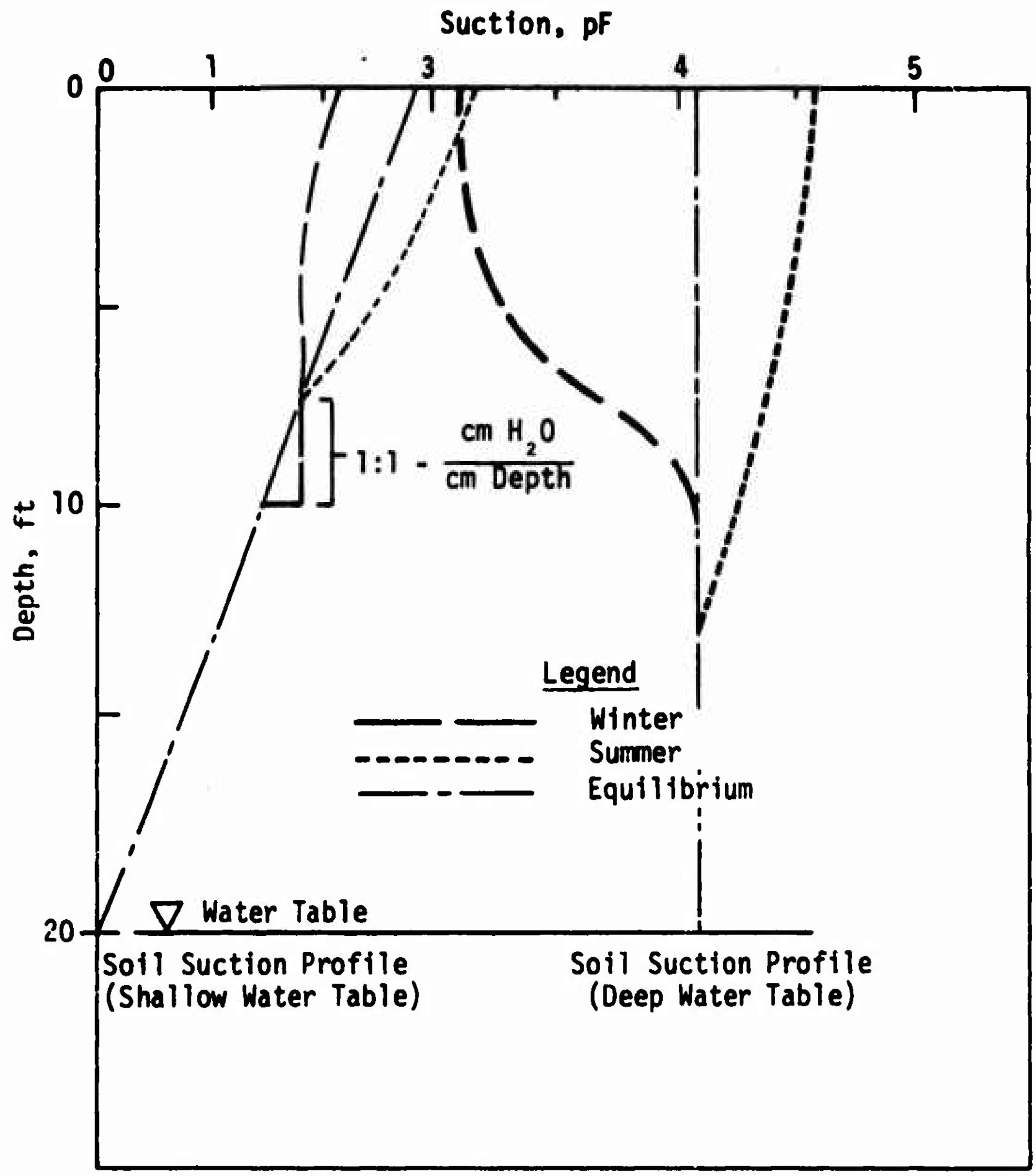


Figure 1. Soil Suction Profile Data

APPENDIX D
SOIL SUCTION

Soil suction is a macroscopic property of soil which indicates the intensity with which a soil sample will attract water. Suction is normally defined as a negative gage pressure and is not to be confused with pore water pressure, which is a component of suction. Pore water pressure is normally associated with the density of liquid, distance from a free surface, and surface tension forces (ref. 12).

Suction results from the interplay of attraction and repulsion forces of charged clay particles and polar water molecules, together with surface tension forces in water, solution potentials due to dissolved ions, and gravity potential. The representation of suction, the sum of all these forces, as an equivalent height of water has been called the *capillary model*. This model was a controversial subject until 1960 when at the London Conference on Pore Pressures and Suction in Soils (ref. 117) substantial agreement was finally reached. At this conference, Aitchison carefully defined the range of validity of the model and concluded that it is a useful concept over a very wide range of suction pressures.

Terminology is very important in this discussion. There is a difference between tension in pore water and suction in the water. Tension applies to the actual pressure state of the pore water; suction is a total head term which includes pore water pressure, osmotic pressure, and adsorptive pressure as components.

The International Society of Soil Science has given definitions of soil suction, its components, and the different potentials which make up the total potential of soil water (table 1). Basically, soil suction is considered to be composed of matrix suction and osmotic or solute suction. Matrix suction is a negative gage pressure which will hold soil water in equilibrium through a porous membrane with the same soil water within a sample of soil. This is also known as *capillary suction*. Osmotic or solute suction is a negative gage pressure which will hold pure water in equilibrium with soil water through a membrane which allows only water molecules to pass.

Table 1. Definitions of Suction and Potential (after reference 118)

Term	Definition	Common Units
Total Suction (τ)	The *negative gage pressure*, relative to the external gas pressure on the soil water, to which a pool of pure water must be subjected in order to be in equilibrium through a semipermeable (permeable to water molecules only) membrane with the soil water	cm of H_2O pF = $\log_{10}$ (cm H_2O) bars, atmospheres
Osmotic (Solute) Suction (τ_s)	The *negative gage pressure* to which a pool of pure water must be subjected in order to be in equilibrium through a semipermeable membrane with a pool containing a solution identical in composition with the soil water	
Matrix (Soil Water) Suction (τ_m)	The *negative gage pressure*, relative to the external gas pressure on the soil water, to which a solution identical in composition with the soil water must be subjected in order to be in equilibrium through a porous permeable wall with the soil water	
Total Potential (ψ)	*Amount of work required per unit quantity* of pure water to transport reversibly and isothermally an infinitesimal quantity of water from a pool of pure water at a specified elevation at atmospheric pressure to the soil water	bars, atmospheres pF, cm of H_2O
Osmotic (Solute) Potential (ψ_s)	*Amount of work required per unit quantity* of pure water to transport reversibly and isothermally an infinitesimal quantity of water from a pool of pure water at a specified elevation at atmospheric pressure to a pool containing a solution identical in composition with the soil water but in all other respects identical with the reference pool	
Gravitational Potential (ψ_g)	*Amount of work required per unit quantity* of pure water to transport reversibly and isothermally an infinitesimal quantity of water from a pool containing a solution identical in composition with the soil water at a specified elevation at atmospheric pressure to a similar pool at the elevation of the point under consideration	
Matrix (Capillary) Potential (ψ_m)	*Amount of work required per unit quantity* of pure water to transport reversibly and isothermally an infinitesimal quantity of water from a pool containing a solution identical in composition with the soil water at the elevation and the external gas pressure of the point under consideration to the soil water	
External Gas Pressure Potential (ψ_p)	This component is considered only when the external gas pressure differs from atmospheric pressure, i.e., in a pressure membrane apparatus	

There is a close relationship between these suction components and their corresponding potentials in the soil water. The total potential of soil water at a certain position is the amount of isothermal work per unit volume that must be done on a small quantity of water to move it from a pool of pure water at atmospheric pressure and a specified elevation to the soil water at the point under consideration. At least five components of this total potential can be identified in most problems:

(1) osmotic or solute potential,
(2) gravitational potential,
(3) matrix or so-called *capillary* potential,
(4) gas pressure potential, and
(5) structural or overburden pressure potential.

In many engineering problems, some of these potentials may be neglected. For example, soils containing small quantities of soluble salts which are rather uniformly dispersed will not be greatly affected by solute potentials. The gas pressure potential should be considered only when the gas pressure is greatly different from the atmospheric pressure. Structural or overburden pressure may need to be considered in most problems.

From thermodynamic theory, total suction may be inferred from the relative humidity within the soil macrostructure with the Kelvin equation (refs. 7, 92, 93, 124).

$$\tau = -\frac{RT}{V_w} \ln \frac{P}{P_o} = -\psi$$

where

τ = total suction, bars (a positive quantity)
ψ = soil water potential, bars (a negative quantity)
R = universal gas constant (80.99 cm^3 bar $^0K^{-1}$ $mole^{-1}$)
T = absolute temperature ($^0K = {}^0C + 273^0$)
V_w = volume of a mole of liquid water (18.02 cm^3 $mole^{-1}$)
P = water vapor pressure in equilibrium with soil water vapor, bars
P_o = pressure of saturated water vapor, bars

Assumptions (ref. 124):

(1) Water behaves as an ideal gas

(2) Water vapor in the air space where the relative humidity is determined as in equilibrium with the soil water vapor
(3) Isothermal conditions ($\Delta T \leq \pm 3°C$)
(4) Absence of soluble salts
(5) Absence of external force fields

As shown in the Kelvin equation, $\tau = -\psi$, where τ is a negative gage pressure (a positive quantity) and ψ is the amount of work required to bring water at reference conditions to equilibrium with the soil water (a negative quantity).

$$\frac{\text{work}}{\text{unit volume}} = \frac{\text{force x distance}}{(\text{distance})^3} = \frac{\text{force}}{(\text{distance})^2} = \text{pressure}$$

The reference selected here is pure water at atmospheric pressure. This is a higher energy level than soil water in unsaturated soils.

Figure 1 is a plot of τ versus P/P_0 x 100 percent in accordance with the Kelvin equation, at T = 20°C. The data points on the curve indicate the range of variation associated with a ΔT from 0° to 40°C; this seems to justify the assumption of isothermal conditions for $\Delta T = \pm 3°C$. Also illustrated is the usable range of several types of suction-measuring devices for field use, as well as a qualitative description of soil conditions. It is apparent that very accurate measurements of relative humidity are required in the range of practical application to real soils. The development of the thermocouple psychrometer in the past decade has provided the required instrument for practical use of soil-suction measurements (refs. 92, 93, 124, 125, 126).

Thermocouple psychrometers are of two general types--the Spanner or cooling current (ref. 125) and the Richards and Ogata or dew point (ref. 126). The Spanner psychrometer involves a thermocouple instrument which evaporates water into the chamber air after Peltier cooling has condensed water onto the thermocouple. By measuring the temperature difference, the relative humidity may be inferred quite accurately. The other type involves evaporation of a drop of water placed in a ring. Although both are appropriate for laboratory work, the Spanner psychrometer is best-suited to field study and has been commercially produced.

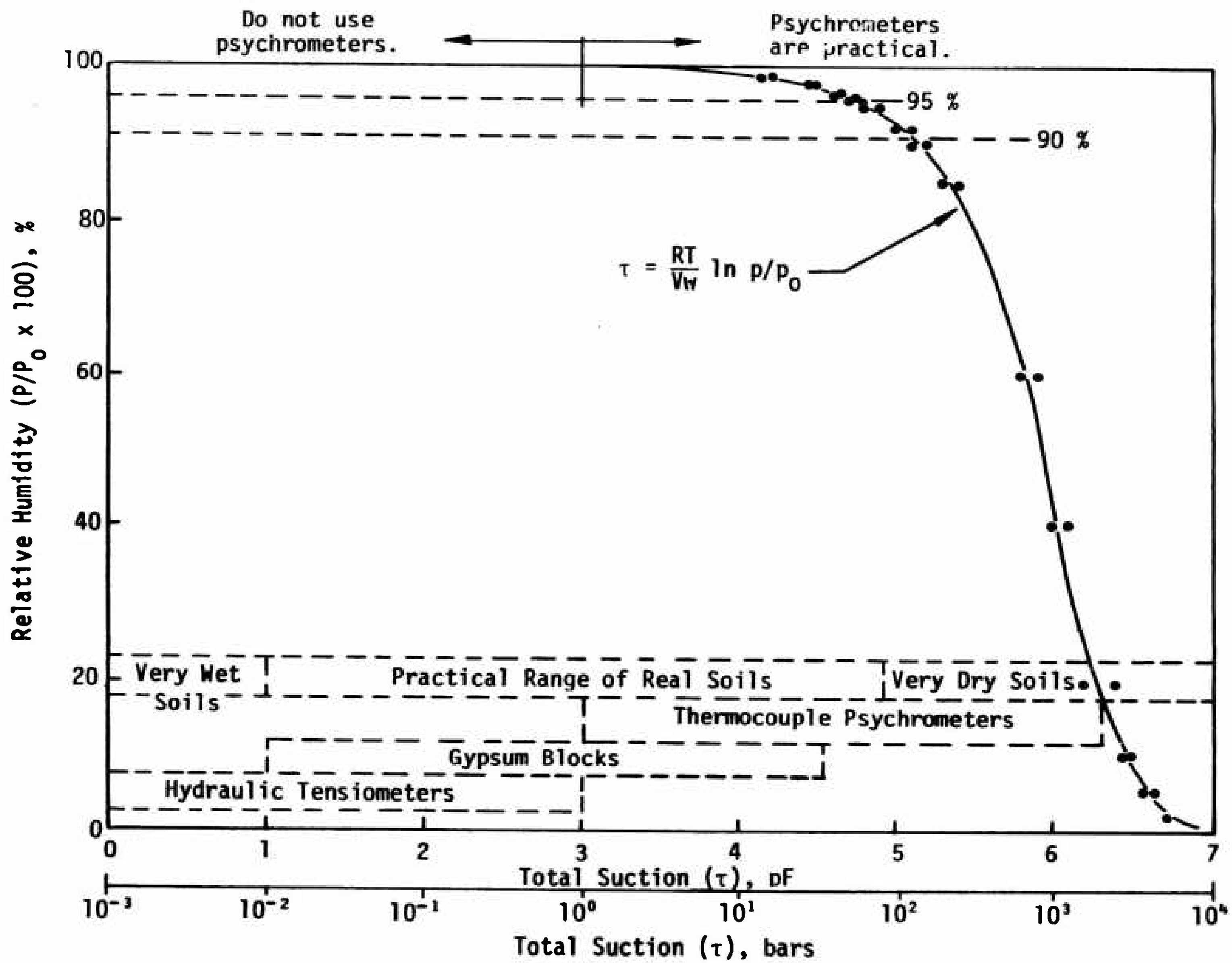


Figure 1. Relationship Between Relative Humidity and Suction

APPENDIX E
LIME STABILIZATION PROCEDURES

This appendix provides the procedures used in the laboratory testing of soil stabilization with lime. Although these are not standard tests in each case, these procedures were used in the development of the data on which the system in this report is based. The material presented is taken directly from the cited references. Only those changes needed for conformance to this format have been made.

TEST FOR pH TO DETERMINE LIME REQUIREMENT (REF. 103)

Materials

Lime to be used for soil stabilization

Apparatus

1. pH meter (the pH meter must be equipped with an electrode having a pH range of 14)
2. 150-ml (or larger) plastic bottles with screw-top lids
3. 50-ml plastic beakers
4. CO_2 - free distilled water
5. Balance
6. Oven
7. Moisture cans

Procedure

1. Standardize the pH meter with a buffer solution having a pH of 12.45.
2. Weigh to the nearest 0.01 g representative samples of air-dried soil, passing the No. 40 sieve and equal to 20.0 g of oven-dried soil.
3. Pour the soil samples into 150-ml plastic bottles with screw-top lids.
4. Add varying percentages of lime, weighed to the nearest 0.01 g, to the soils. (Lime percentages of 0, 2, 3, 4, 5, 6, 8, and 10, based on the dry soil weight, may be used.)
5. Thoroughly mix soil and dry lime.

6. Add 100 ml of CO_2 - free distilled water to the soil/lime mixtures.

7. Shake the soil/lime and water for a minimum of 30 sec or until there is no evidence of dry material on the bottom of the bottle.

8. Shake the bottles for 30 sec every 10 min.

9. After 1 hr, transfer part of the slurry to a plastic beaker and measure the pH.

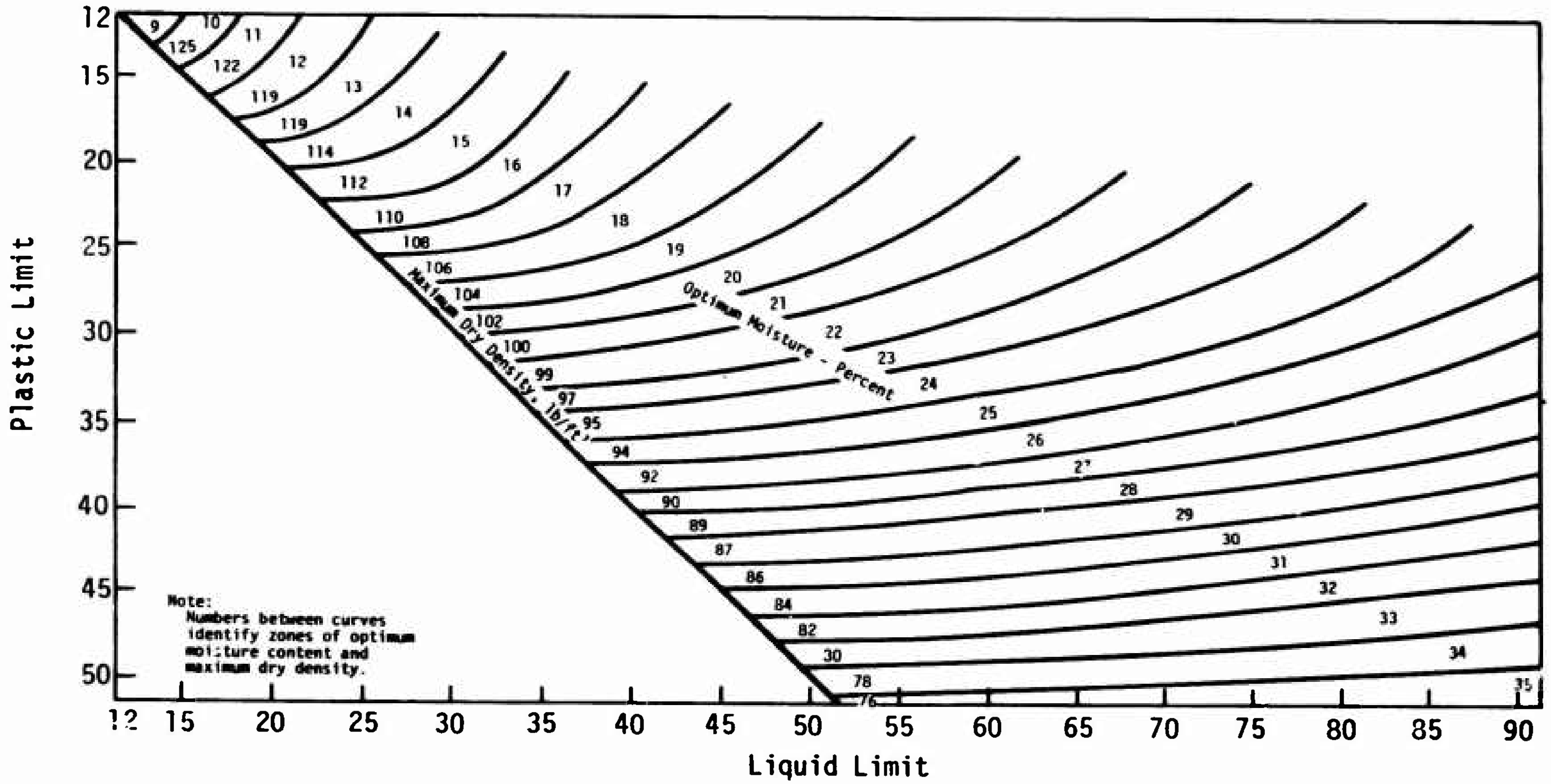
10. Record the pH for each of the soil/lime mixtures. The lowest percent of lime giving a pH of 12.40 is the percent required to stabilize the soil. If the pH does not reach 12.40, the minimum lime content giving the highest pH is that required to stabilize the soil.

MOISTURE/DENSITY RELATIONSHIPS OF LIME/SOIL MIXTURES (REF. 105)

To find the optimum moisture content corresponding to the maximum dry density of a lime/soil mixture, a method similar to that found in ASTM D698-70 is used. Figure 1 gives the approximate optimum moisture and maximum dry density based upon known Atterberg Limits of the untreated soil. The optimum moisture content of lime/soil mixtures is always higher than the soil untreated. The maximum dry density is lower. A Vicksburg Miniature Compaction Apparatus is used to fabricate specimens. The apparatus produces a 2-in-diameter by 4-in-high specimen with similar densities produced with the compaction equipment employed in ASTM D698-70. The procedures used in determining the optimum moisture content for lime/soil mixtures are as follows:

(1) The untreated soil is first passed through a No. 4 sieve. It may be necessary to air dry the soil to permit pulverization to the proper size.

(2) Estimate the approximate moisture content from figure 1. Determine the proportions of soil, lime, and water required for fabrication of approximately five specimens. Approximately 2100 g of mix will be required. See the lime/soil mixture calculations that follow these procedures.



Example: Given: Plastic Limit - 20 Liquid Limit - 35

Find: Average Maximum Dry Density and Optimum Moisture Content

Answer: 110 lb/ft^3 (Density) 16 percent (Moisture Content)

Figure 1. Approximate Moisture/Density Relationship [after Ring, et al. (ref. 127)]

(3) Weigh the Vicksburg Mold to the nearest 0.1 g and record on data sheet.

(4) Measure the inside height of the Vicksburg Mold with the entire assembly in place to the nearest 0.01 in (from the base to the top of the collar). Measure the inside diameter of the mold to the nearest 0.01 in and record the values on the data sheet.

(5) Weigh out the soil, lime, and water required to the nearest g/ml as per calculations.

(6) Thoroughly mix soil and lime together either by hand or with an electric soil mixer until all free lime is blended with the soil.

(7) Add water evenly to the blended soil and lime (care must be taken to prevent excess loss on the sides of the mixing pot). Mix the entire blend thoroughly. After the soil is mixed, cover with a damp paper towel to prevent moisture loss.

(8) Weigh approximately five equal portions of the soil mixture to be compacted. Approximately 75 to 85 g per layer will produce a specimen of the proper size.

(9) Pour soil into compaction mold, level soil, and compact the first layer with five blows of the sliding hammer (take weight to its full height on the sliding rod before dropping).

(10) Measure the height from the top of the collar to the top of the first compacted layer of soil. By subtracting this value from the total height (step 4), you will obtain the thickness of the compacted layer. Multiply this figure by 5 (number of total layers) and this will give you an approximation of the total height of the specimen. Adjust the amount of soil in the following layers so that the final specimen will be 4 ± 0.25 in.

(11) Scarify the top of each compacted layer to a depth of 1/8 in with an ice pick to insure adequate bond with following layers.

(12) After compacting the last layer, measure from the top of the specimen to the top of the collar using a steel rule with 0.01-in accuracy and record on the data sheet.

(13) Remove the collar and mold. Trim the excess soil from the inside of the mold to make the specimen level across the top.

(14) Weigh the mold with the compacted sample to the nearest 0.1 g and record.

(15) Extrude the sample from the mold.

(16) Break the specimen into five equal parts and take an equal amount of soil from the center of each portion. Place all five portions in a preweighed tare and weigh to the nearest 0.01 g. Place tare in oven and obtain a moisture content the following day.

(17) Repeat above steps for varying water contents, adding water as per calculations. Do not recompact samples.

(18) Calculate dry density of specimens and moisture content.

$$\text{Dry Density} = \frac{\text{Wet Density}}{1 + \text{Moisture Content}}$$

$$\text{Moisture Content} = \frac{\text{Weight of Water}}{\text{Weight of Solids}} \times 100\ \%$$

(19) Plot data and select optimum moisture content for the percentage of lime.

Sample Problem

Given:

Percent lime (by weight) required	=	6 %
Desired initial H_2O content	=	15 %

H_2O content of untreated soil = 10 %
(determined earlier)

Calculations:

<u>Total Desired Mixture Formula:</u>

Lime		Water		Soil Solids	
$0.06\ W_s$	+	$0.15\ W_s$	+	$1\ W_s$	= 2100 g (will make approximately five samples)

Solve for W_s

$1.21\ W_s$ = 2100 g

W_s = 1735.54 g

$W_{lime} = 0.06\ W_s$ = 104.13 g

$W_{water} = 0.15\ W_s$ = 260.33 g

Check 2100 g

<u>Actual Water Required (considering H_2O content of natural soil):</u>

<u>Water in Untreated Soil</u>

W_s = 1735.54 g (from above)

H_2O content of untreated soil = 10 %

0.10 (1735.54 g) = 173.55 g of H_2O natural soil

Water to Add for Desired H_2O Content

Weight of water desired	260.33 g
Weight of water in soil	- 173.55 g
Weight of water to add	86.78 g

Total Soil, Lime, and Water Required:

(1) Soil Required

$W_s + W_{water}$ Untreated Soil = Soil Required

1735.54 g + 173.55 g = 1909 g

(2) Lime Required

$W_{lime} = 104$ g

(3) Water Required

$W_{water} = 87$ g or ml

Water Required to Increase Moisture Content:

$$\frac{\text{No. Specimens Left in Batch} \times \text{Original } W_s}{5}$$

x ___% Increase Desired = Water to Add

Example (for a desired 2-percent increase, 3 specimens left in batch)

$\frac{3}{5} \times 1735.54 \text{ g} \times 0.02 = 21$ g or ml of water

Note: Actual moisture contents will be higher than calculated due to loss of soil during fabrication.

SPECIMEN FABRICATION (REF. 105)

The procedure given above is followed by specimen fabrication. No more than three specimens may be compacted from a batch of soil/lime/water mixture to insure proper mixing and good quality control. Approximately 1450 g of soil is required for fabrication of three specimens. A moisture content is taken from the uncompacted mix during compaction of each specimen. Each specimen height, moisture content, and dry density is determined and must meet the following specifications:

Specimen Height .... 4 $\pm$ 0.125 in
Moisture Content .... Optimum $\pm$ 1 %
Dry Density .... Maximum Dry Density $\pm$ 2 lb/ft^3

The specimens are triple wrapped in thin plastic membrane and taped to prevent moisture loss.

RAPID CURE (REF. 105)

Lime/soil specimens are placed in an oven for 30 hr $\pm$ 15 min. The oven must be capable of holding a temperature of 120° $\pm$ 2°F with quick temperature recovery when the door is opened for removal of specimens. After completion of curing, the specimen is allowed to cool for 15 min prior to strength testing and 2 hr prior to water immersion testing. Care must be taken during cure to totally prevent specimen moisture loss.

FREEZE/THAW DURABILITY (REF. 105)

This test is for the determination of the change in unconfined compressive strength for cured 2-in-diameter by 4-in-high lime/soil specimens which have been subjected to repeated cycles of alternate freezing and thawing. The apparatus used consists of: (a) a commercial wide mouth vacuum flask with an internal diameter of about 2.5 in and depth of about 6 in; (b) a specimen holder of low thermal conductivity lucite for holding the cylindrical specimen inside the vacuum flask. The base of the specimen holder was perforated to permit the access of water to the bottom of the lime/soil specimens; (c) demineralized water; (d) a freezer maintained at 22° ± 2°F.

The procedure for conducting the freeze/thaw durability test is as follows:

(1) The specimens are placed in the plastic specimen holders. The specimen holders are then inserted into the vacuum flasks. Enough demineralized water is placed in the vacuum flasks so that the bottom 1/4 in of the lime/soil specimens will be immersed when placed in the flasks. This water level is maintained throughout the entire test.

(2) The vacuum flasks and specimens are placed in the freezer (22° ± 2°F) for 16 hr.

(3) After the 16-hr freezing period, the vacuum flasks are removed from the freezer. The specimens in the plastic holder are removed from the flasks and allowed to thaw for 8 hr at 77° ± 2°F. The bottom 1/4 in of the specimens remain immersed in water during the thawing period. One freeze/thaw cycle in 16 hr of freezing and 8 hr of thawing.

(4) The process is repeated for three cycles of freezing and thawing, after which the specimen is removed and the unconfined strength determined (ASTM D2166-66).

LIME REACTIVITY (REF. 105)

Samples at three lime percentages (pH estimated lime percent, + 2 percent and - 2 percent) are prepared using 2-in-diameter by 4-in-high molds and the Vicksburg compaction apparatus. Specimens are thoroughly wrapped to totally prevent moisture loss and then rapid cured for 30 hr at 120°F. After rapid cure is complete, determine the unconfined compression strength (ASTM D2166-66). The soil is lime reactive if the strength is in excess of 110 psi. Should lower strengths result, lime treatment should not be used.

APPENDIX F

SSIS SOIL SAMPLES

In the development of any design method it is desirable to include a wide variety of materials which are representative of most types likely to be encountered in practice. It was, therefore, desired to utilize a wide variety of soils in the testing. The soils used in the development of the Air Force Soil Stabilization Index System (SSIS) are described in tables 1 through 3. Examination of the data in these tables indicates the wide range of soil materials used.

Table 1. Soils Used in Initial Validation of SSIS (ref. 103)

Soil	Classification		Consistency		Moisture/Density		pH
	AASHO	Unified	LL*,%	PI*,%	γ_d*,lb/ft^3	OMC*,%	
Tuy Hoa	A-1-b	G	---	NP	---	--	5.1
Altus SB	A-2-4	SC	14.5	NP	---	--	7.4
Dyess	A-7-6(12)	CL	40.3	23.2	102.7	19.7	7.4
Altus SG	A-7-6(12)	CL	40.7	19.8	97.7	23.6	7.5
Tyler	A-7-5(15)	OH	52.5	21.1	91.7	22.3	2.3
Houma	A-7-6(20)	CH	63.7	40.8	86.4	23.7	6.95
Perrin B	A-7-6(20)	CH	65.0	41.7	92.4	24.1	7.3
Perrin A	A-7-6(20)	CH	72.0	40.0	97.5	23.7	4.5
Perrin AB	A-7-6(20)	CH	69.4	43.3	95.0	23.9	6.7
Panama A	---	CH	72.5	32.8	83.4	35.1	5.3
Panama B	---	CH	75.5	35.5	82.8	35.1	6.27
North Carolina	---	CH	61.0	26.9	98.6	23.5	5.05
Dallas Regn'l	A-7-6(20)	CH	68.0	50.1	---	--	7.73
WES Clay	A-6(9)	CL	37.5	13.6	107.8	17.8	--
Buckshot	A-7-6(20)	CH	67.1	43.0	---	--	--
Chenault	A-7-6(17)	CH	45.6	29.6	---	--	7.70

* LL = Liquid Limit, PI = Plasticity Index, γ_d = Maximum Dry Density, OMC = Optimum Moisture Content.

Table 2. Lime Stabilized Soils, AFACAD Validation (ref. 105)

Soil	Classification		Consistency Atterberg Limits		Moisture/Density		pH
	AASHO	Unified	LL*,%	PI*,%	γ_d*,lb/ft^3	OMC*,%	
Dyess	A-7-6(12)	CL	40.3	23.2	102.7	19.7	7.40
Altus	A-7-6(12)	CL	40.7	19.8	97.7	23.6	7.50
Tyler	A-7-5(15)	OH	52.5	21.1	91.7	22.3	2.30
Houma	A-7-6(20)	CH	63.7	40.8	86.4	23.7	6.95
Perrin A	A-7-6(20)	CH	72.0	30.0	97.5	23.7	4.50
Perrin B	A-7-6(20)	CH	65.0	41.7	92.4	23.1	7.30
Perrin AB	A-7-6(20_	CH	69.4	43.3	95.0	23.9	6.70
Bergstrom	A-6(7)	CL	32.0	14.1	121.90	14.75	8.70
Carswell	A-7-6(20)	CH	48.6	18.6	101.6	22.6	8.62
Tinker	A-6	CL	30.0	12.0	112.8	16.5	8.18
LeMoore	A-7-6(16)	CH	58.4	33.4			8.25
Malmstrom	A-6	CL	34.1	14.9			7.50
Cannon	A-1-b	SM	25.0	3.5	114.0	14.0	8.80
Estiraodo	A-7-5(8)	CL	28.7	9.7			
Ellington	A-7(20)	CH	60.0	32.5	102.7	25.0	8.70
Barksdale	A-2-4	CL-ML	30.0	8.3	114.0	17.1	8.53
Ellsworth	A-2-7	SW-SC	30.7	24.0			8.83
Moody	A-2-5	SM	26.0	4.8			8.00
Robbins	A-2-4	ML	25.2	3.6			8.95

* LL = Liquid Limit, PI = Plasticity Index, γ_d = Maximum Dry Density, OMC = Optimum Moisture Content

Table 3. Cement Stabilized Soils, AFACAD Validation (ref. 105)

Soil	Classification AASHO	Classification Unified	Consistency LL*,%	Consistency PI*,%	Moisture/Density γ_d*,lb/ft³	Moisture/Density OMC*,%	pH
Tuy Hoa	A-1-b			NP			5.10
Altus Subbase	A-2-4	SC	14.5	NP			7.40
Dyess	A-7-6		40.3	23.2	102.7	19.7	7.40
Altus Subgrade	A-7-6	CL	40.7	19.8	97.7	23.6	7.50
Tyler	A-7-5	OH	52.5	21.1	91.7	22.3	2.30
Houma	A-7-6	CH	63.7	40.8	92.4	24.1	7.30
Perrin B	A-7-6	CH	65.0	41.7	92.4	24.1	7.30
Perrin A	A-7-6	CH	72.0	40.0	97.5	23.7	4.50
Clark	A-1-b	SM-SC		NP	117.2	11.2	
Patrick	A-1-b			NP	112.5	10.6	
Holloman	A-1-b			NP	139.0	5.9	
Moody	A-1-b			NP	121.0	11.3	
Robbins	A-2-7		45.2	22.0	122.6	11.1	
Laughlin	A-6	CL	33.2	13.0	105.0	18.7	
Charleston	A-1-a	GW		NP	125.0	9.8	
Norton	A-1-b	SP		NP	102.5	16.9	
Vance	A-1-b			NP		8.4	
Ellington	A-2-4	SW			126.2	9.0	
George	A-3				118.0	12.5	
Hamilton	A-4		27.4	5.7	112.0	16.5	
Tinker	A-6		37.3	20.4	107.9	18.6	
Kelly	A-7-5		82.0	45.2	89.0	20.0	

* LL = Liquid Limit, PI = Plasticity Index, γ_d = Maximum Dry Density, OMC = Optimum Moisture Content

APPENDIX G
CEMENT STABILIZATION PROCEDURES

This appendix provides the procedures used in the laboratory testing of soil stabilization with cement. These procedures are taken directly from the literature cited. The data and procedures described in this report are based on testing in accordance with these procedures.

TEST FOR pH OF SOIL/CEMENT MIXTURES (REF. 103)

Materials

Portland cement to be used for soil stabilization

Apparatus

1. pH meter (the pH meter must be equipped with an electrode having a pH range of 14)
2. 150-ml plastic bottles with screw-top lids
3. 50-ml plastic beakers
4. Distilled water
5. Balance
6. Oven
7. Moisture cans

Procedure

1. Standardize the pH meter with a buffer solution having a pH of 12.00.
2. Weigh to the nearest 0.01 g, representative samples of air-dried soil, passing the No. 40 sieve and equal to 25.0 g of oven-dried soil.
3. Pour the soil samples into 150-ml plastic bottles with screw-top lids.
4. Add 2.5 g of the Portland cement.
5. Thoroughly mix soil and Portland cement.

6. Add sufficient distilled water to make a thick paste. (Caution: too much water will reduce the pH and produce an incorrect result.)

7. Stir the soil, cement, and water until thorough blending is achieved.

8. After 15 min, transfer part of the paste to a plastic beaker and measure the pH.

9. If the pH is 12.1 or greater, the soil organic matter content should not interfere with the cement stabilizing mechanism. To determine the required percent of cement, refer to design methods outlined in section 6 of this report.

DETERMINATION OF SULFATE IN SOILS - GRAVIMETRIC METHOD (REF. 103)

Scope

This method is applicable to all soil types with the possible exception of soils containing certain organic compounds. This method should permit the detection of as little as 0.05 percent sulfate as SO_4.

Reagents

1. Barium chloride, 10-percent solution of $BaCl_2 \cdot 2H_2O$. (Add 1 ml of 2-percent HCl to each 100 ml of solution to prevent formation of carbonate.)

2. Hydrochloric acid, 2-percent solution (0.55 N)

3. Magnesium chloride, 10-percent solution of $MgCl_2 \cdot 6H_2O$.

4. Demineralized water

5. Silver nitrate, 0.1 N solution

Apparatus

1. Beaker, 1000 ml
2. Burner and ring stand
3. Filtering flask, 500 ml
4. Buchner funnel, 9 cm
5. Filter paper, Whatman No. 40, 9 cm
6. Filter paper, Whatman No. 42, 9 cm
7. Saranwrap
8. Crucible, ignition, or aluminum foil, heavy grade
9. Analytical balance
10. Aspirator or other vacuum source

Procedure

(1) Select a representative sample of air-dried soil weighing approximately 10 g. Weigh to the nearest 0.01 g. (Note: When sulfate content is anticipated to be less than 0.1 percent, a sample weighing 20 g or more may be used.) (The moisture content of the air-dried soil must be known for later determination of dry weight of the soil.)

(2) Boil for 1-1/2 hr in beaker with mixture of 300 ml water and 15 ml HCl.

(3) Filter through Whatman No. 40 paper, wash with hot water, dilute combined filtrate and washings to 50 ml.

(4) Take 100 ml of this solution and add $MgCl_2$ solution until no more precipitate is formed.

(5) Filter through Whatman No. 42 paper, wash with hot water, dilute combined filtrate and washings to 200 ml.

(6) Heat 100 ml of this solution to boiling and add $BaCl_2$ solution very slowly until no more precipitate is formed. Continue boiling for about 5 min and let stand overnight in warm place, covering beaker with Saranwrap.

(7) Filter through Whatman No. 42 paper. Wash with hot water until free from chlorides (filtrate should show no precipitate when a drop of $AgNO_3$ solution is added).

(8) Dry filter paper in crucible or on sheet of aluminum foil. Ignite paper. Weigh residue on analytical balance as $BaSO_4$.

Calculation

$$\text{Percent } SO_4 = \frac{\text{Weight of Residue}}{\text{Oven-Dry Weight of Initial Sample}} \times 411.6$$

where

$$\text{oven-dry weight of initial sample} = \frac{\text{Air-Dry Weight of Initial Sample}}{1 + \dfrac{\text{Air-Dry Moisture Content (percent)}}{100 \text{ percent}}}$$

Note: If precipitated from cold solution, barium sulfate is so finely dispersed that it cannot be retained when filtering by the above method. Precipitation from a warm, dilute solution will increase crystal size. Due to the absorption (occlusion) of soluble salts during the precipitation of $BaSO_4$ a small error is introduced. This error can be minimized by permitting the precipitate to digest in a warm, dilute solution for a number of hours. This allows the more soluble small crystals of $BaSO_4$ to dissolve and recrystallize on the larger crystals.

DETERMINATION OF SULFATE IN SOILS - TURBIDIMETRIC METHOD (REF. 103)

Reagents

1. Barium chloride crystals (Grind analytical reagent grade barium chloride to pass 1-mm sieve.)

2. Ammonium acetate solution (0.5N) (Add dilute hydrochloric acid until the solution has a pH of 4.2.)

3. Distilled water

Apparatus

1. Moisture can

2. Oven

3. 200-ml beaker

4. Burner and ring stand

5. Filtering flask

6. Buchner funnel, 9 cm

7. Filter paper, Whatman No. 40, 9 cm

8. Vacuum source

9. Spectrophotometer and standard tubes (Bausch and Lombe Spectronic 20 or equivalent)

10. pH meter

Procedure

(1) Take a representative sample of air-dried soil weighing approximately 10 g and weigh to the nearest 0.01 g. (The moisture content of the air-dried soil must be known for later determination of dry weight of the soil.)

(2) Add the ammonium acetate solution to the soil. (The ratio of soil to solution should be approximately 1:5 by weight.)

(3) Boil for about 5 min.

(4) Filter through Whatman No. 40 filter paper. If the extracting solution is not clear, filter again.

(5) Take 10 ml of extracting solution (this may vary depending on the concentration of sulfate in the solution) and dilute with distilled water to about 40 ml. Add about 0.2 g of barium chloride crystals and dilute to make the volume exactly 50 ml. Stir for 1 min.

(6) Immediately after the stirring period has ended, pour a portion of the solution into the standard tube and insert the tube into the cell of the spectrophotometer. Measure the turbidity at 30-sec intervals for 4 min. Maximum turbidity is usually obtained within 2 min and the readings remain constant thereafter for 3 to 10 min. Consider the turbidity to be the maximum reading obtained in the 4-min interval.

(7) Compare the turbidity reading with a standard curve and compute the sulfate concentration (as SO_4) in the original extracting solution. (The standard curve is secured by carrying out the procedure with standard potassium sulfate solutions.)

(8) Correction should be made for the apparent turbidity of the samples by running blanks in which no barium chloride is added.

Sample Problem

Given:

Weight of air-dried sample = 10.12 g

Water Content = 9.36 %

Weight of dry soil = 9.27 g

Total volume of extracting solution = 39.1 ml

10 ml of extracting solution was diluted to 50 ml after addition of barium chloride (step 5). The solution gave a transmission reading of 81.

Calculations:

From the standard curve, a transmission reading of 81 corresponds to 16.0 ppm (fig. 1). Therefore, concentration of original extracting solution = 16.0 x 5 = 80.0 ppm.

$$\text{Percent } SO_4^{--} = \frac{80.0 \times 39.1 \times 100}{1000 \times 1000 \times 9.27} = 0.0338\ \%$$

Determination of Standard Curve:

(1) Prepare sulfate solution of 0, 4, 8, 12, 16, 20, 25, 30, 35, 40, 45, 50 ppm in separate test tubes. The sulfate solution is made from potassium sulfate salt dissolved in 0.5 N ammonium acetate (with pH adjusted to 4.2).

(2) Continue steps 5 and 6 of the procedure.

(3) Draw standard curve as shown in figure 1 by plotting transmission readings for known concentrations of sulfate solutions.

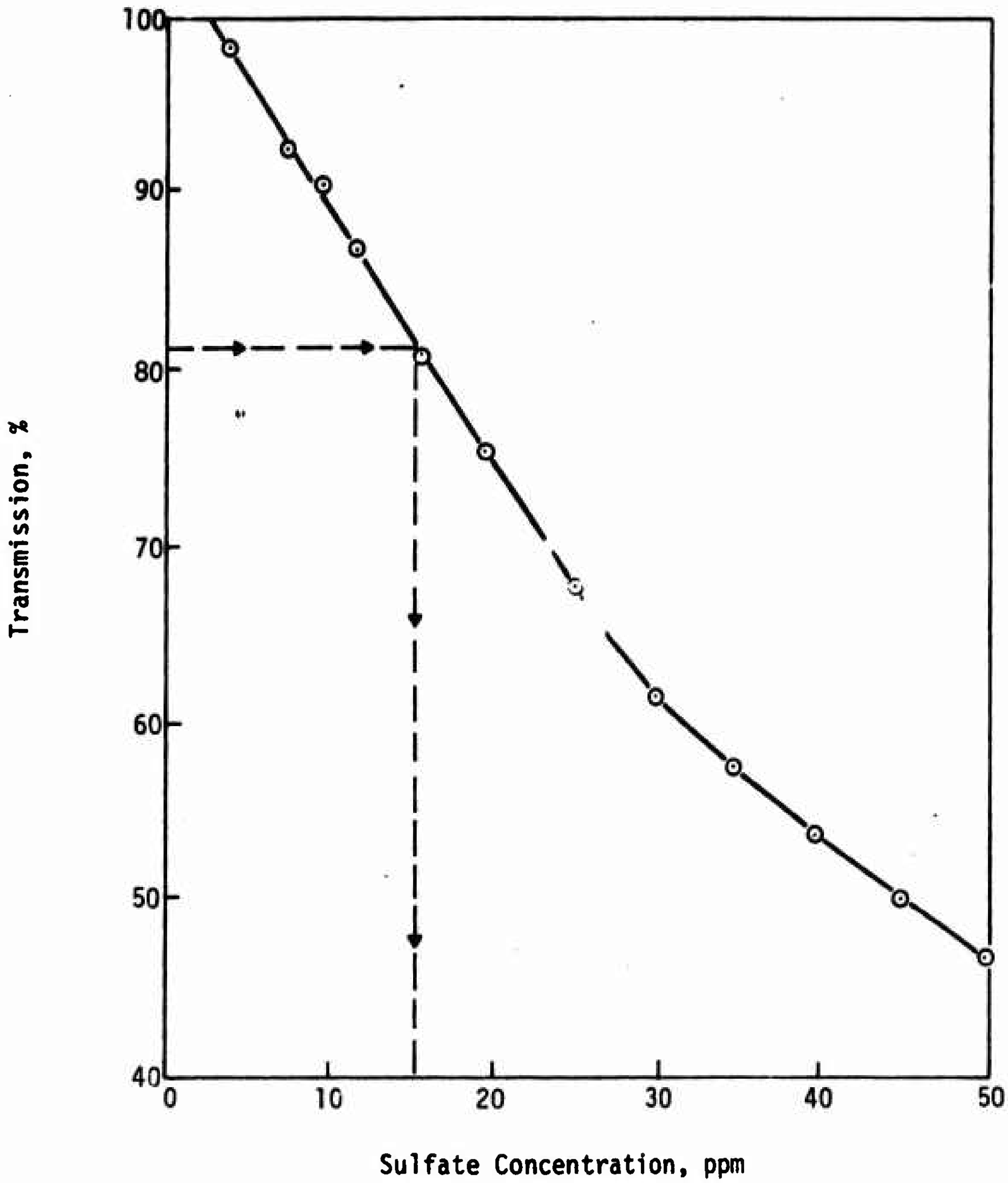


Figure 1. Example Standard Curve for Spectrophotometer [after Dunlap (ref. 103)]

SELECTION OF CEMENT CONTENTS FOR TESTS*

This chapter will be of major interest to the laboratory engineer because it will assist him in determining what cement contents to investigate in the soil/cement tests. The field engineer and administrative engineer will also be interested because the properties of soil/cement mixtures and the relationships existing among these properties and various test values are discussed. Information is presented that will enable engineers to estimate probable cement factors so that job estimates can be made before any tests are made.

In order to obtain the maximum amount of information from the wet/dry and freeze/thaw tests, it is important that the laboratory engineer design the soil/cement specimens properly. For instance, if specimens are designed with very high cement contents, they will pass the wet/dry and freeze/thaw tests, and a minimum cement factor will not have been determined. On the other hand, if the specimens are designed with inadequate cement contents, they will all fail in the tests.

The principal requirement of a hardened soil/cement mixture is that it withstand exposure to the elements. Strength might also be considered a principal requirement; however, since most soil/cement mixtures that possess adequate resistance to the elements also possess adequate strength, this requirement is secondary.

Therefore, in a study to determine when a certain soil/cement mixture has been adequately hardened, the requirement of adequate resistance to exposure is the first considered. That is, will the hardened soil/cement mixture withstand the wetting and drying and the freezing and thawing cycles of nature and still maintain at least the stability inherent in the mass at the time the roadway was opened to traffic?

For instance, consider a hypothetical road subgrade made from a clay loam soil without cement, packed to maximum density at a moisture content slightly less

* This material was taken directly from reference 107 by permission of the Portland Cement Association.

than its optimum moisture content. This mass can withstand relatively heavy loads without failure, although it cannot offer much resistance to abrasive forces.

The same soil mixed with cement and compacted to maximum density at optimum moisture content will have stability before the cement hydrates at least equal to that of the raw soil.

But consider the two cases at a later date under a condition of slow drainage when moisture, by capillary action or in some other manner, has permeated the masses. The voids in the raw soil become filled with water and the soil loses the original inherent physical stability that was built into it by compaction to maximum density. This is not so however, with the adequately hardened soil/cement mixture, which has continually increased in stability since its construction because of cement hydration and resultant cementation. Its air voids may become filled with water too, but its stability will still be much greater than that built into it originally.

The next important requirement to consider is economy. Available data indicate that about 85 percent of all soils likely to be used for soil/cement can be adequately hardened by the addition of 14 percent cement or less. To determine whether or not a soil falls into this category would not require much testing. However, more than 50 percent of all soils so far tested for soil/cement require only 10 percent cement or less for adequate hardening. To identify these soils requires more testing. Since soil/cement is in the low-cost paving field, the testing engineer on large jobs should determine by test the minimum quantity of cement that can be safely used with each soil. By this procedure, the lowest-cost soil/cement construction possible will be obtained.

Estimating Cement Requirements

The following information will aid the engineer in estimating cement requirements of the soils proposed for use and in determining what cement factors to investigate in the laboratory tests.

As a general rule, it will be found that the cement requirement of soils increases as the silt and clay content increases, gravelly and sandy soils requiring less cement for adequate hardness than silt and clay soils.

The one exception to this rule is that poorly graded, one-size sand materials that are devoid of silt and clay require more cement than do sandy soils containing some silt and clay.

In general, a well-graded mixture of stone fragments or gravel, coarse sand, and fine sand either with or without small amounts of slightly plastic silt and clay material will require 5 percent or less cement by weight. Poorly graded one-size sand materials with a very small amount of nonplastic silt, typical of beach sand or desert blow sand, will require about 9 percent cement by weight. The remaining sandy soils will generally require about 7 percent. The nonplastic or moderately plastic silty soils generally require about 10 percent cement by weight, and plastic clay soils require about 13 percent or more.

Table 1 gives the usual range in cement requirements for subsurface soils of the various AASHO soil groups. A horizon soils may contain organic or other material detrimental to cement reaction and may require higher cement factors. For most A horizon soils, the cement content in table 1 should be increased four percentage points if the soil is dark grey to grey and six percentage points if the soil is black. It is usually not necessary to increase the cement factor for a brown or red A horizon soil. These cement contents can be used as preliminary estimates, which are then verified or modified as additional test data become available.

STEP-BY-STEP PROCEDURE

The following procedure will prove helpful to the testing engineer in setting up cement contents to be investigated:

(1) Determine from table 1 the preliminary estimated cement content by weight based on the AASHO soil group.

Table 1. Cement Requirements of AASHO Soil Groups

AASHO Soil Group	Usual Range in Cement Requirement		Estimated Cement Content(that used in moisture/density test), percent by weight	Cement Contents for Wet/Dry and Freeze/Thaw Tests, percent by weight
	percent by volume	percent by weight		
A-1-a	5- 7	3- 5	5	3- 4- 5- 7
A-1-b	7- 9	5- 8	6	4- 6- 8
A-2	7-10	5- 9	7	5- 7- 9
A-3	8-12	7-11	9	7- 9-11
A-4	8-12	7-12	10	8-10-12
A-5	8-12	8-13	10	8-10-12
A-6	10-14	9-15	12	10-12-14
A-7	10-14	10-16	13	11-13-15

(2) Use the preliminary estimated cement content obtained in step 1 to perform the moisture/density test.

(3) Verify the preliminary estimated cement content by referring to table 2 if the soil is sandy or to table 3 if it is silty or clayey. These tables take into consideration the maximum density and other properties of the soil, which permits a more accurate estimate. In the case of A horizon soils, the indicated cement factor should be increased as discussed above for table 1.

Sandy Soils:

(a) Using the percentage of material smaller than 0.05 mm, the percentage of material retained on the No. 4 sieve, and the maximum density obtained by test in step 2, determine from table 2 the estimated cement content.

(b) Mold wet/dry and freeze/thaw test specimens at the estimated cement

Table 2. Average Cement Requirements of B and C Horizon Sandy Soils

Material Retained on No. 4 Sieve, percent	Material Smaller Than 0.05 mm, percent	Cement Content, percent by weight					
		Maximum Density, lb/ft^3					
		105-109	110-114	115-119	120-124	125-129	130 or More
0-14	0-19	10	9	8	7	6	5
	20-39	9	8	7	7	5	5
	40-50	11	10	9	8	6	5
15-29	0-19	10	9	8	6	5	5
	20-39	9	8	7	6	6	5
	40-50	12	10	9	8	7	6
30-45	0-19	10	8	7	6	5	5
	20-39	11	9	8	7	6	5
	40-50	12	11	10	9	8	6

Table 3. Average Cement Requirements of B and C Horizon Silty and Clayey Soils

Group Index*	Material Between 0.05 and 0.005 mm, percent	Cement Content, percent by weight						
		Maximum Density, lb/ft³						
		90-94	95-99	100-104	105-109	110-114	115-119	120 or More
0- 3	0-19	12	11	10	8	8	7	7
	20-39	12	11	10	9	8	8	7
	40-59	13	12	11	9	9	8	8
	60 or more	--	--	--	--	--	--	--
4- 7	0-19	13	12	11	9	8	7	7
	20-39	13	12	11	10	9	8	8
	40-59	14	13	12	10	10	9	8
	60 or more	15	14	12	11	10	9	9
8-11	0-19	14	13	11	10	9	8	8
	20-39	15	14	11	10	9	9	9
	40-59	16	14	12	11	10	10	9
	60 or more	17	15	13	11	10	10	10
12-15	0-19	15	14	13	12	11	9	9
	20-39	16	15	13	12	11	10	10
	40-59	17	16	14	12	12	11	10
	60 or more	18	16	14	13	12	11	11
16-20	0-19	17	16	14	13	12	11	10
	20-39	18	17	15	14	13	11	11
	40-59	19	18	15	14	14	12	12
	60 or more	20	19	16	15	14	13	12

*Group index values determined by charts used in AASHO M 145-49 (fig. 2). The newer group index chart used with Interim Recommended Practice for the Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes (AASHO M 145-661) cannot be used to determine group index values for table 3 since this table is based on AASHO M 145-49.

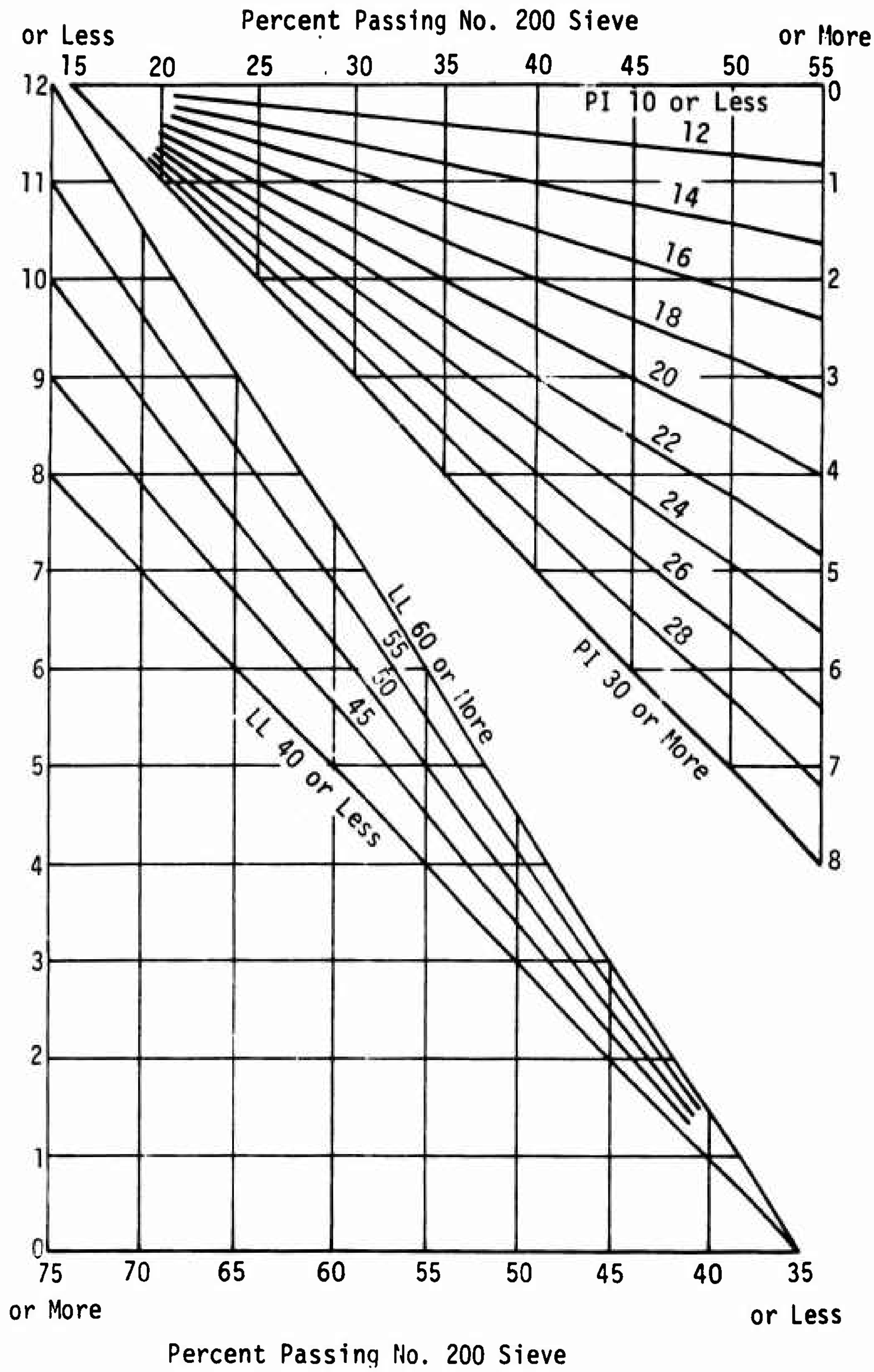


Figure 2. Charts for Calculating Group Index Values

content by weight obtained in (a) and at cement contents two percentage points above and below that cement factor.*

Silty and Clayey Soils:

(a) Using the percentage of material between 0.05 and 0.005 mm, the AASHO group index, and the maximum density obtained by test in step 2, determine from table 3 the estimated cement content.

(b) Mold wet/dry and freeze/thaw test specimens at the estimated cement content obtained in (a) and at cement contents two percentage points above and below that cement factor.

To help in determining how well the soil reacts, it is advantageous to save half of the last moisture/density test specimen and to place it in an atmosphere of high humidity for inspection daily. This half specimen, called the *tail-end* specimen, is obtained during the usual procedure of cutting the last specimen of the moisture/density test in half vertically so that a representative moisture sample can be taken. Generally, tail-end specimens are satisfactorily hardened in two to four days and it is not uncommon for them to be satisfactory a day after molding.

A study of compressive-strength data is also helpful in checking the estimated cement factor.

Miscellaneous Soils

A number of miscellaneous materials or special types of soils such as caliche, chert, cinders, scoria, shale, etc., have been used successfully in soil/cement construction. In some cases, these materials have been found in the roadway or street that was to be paved with soil/cement; in other cases, in order to reduce the cost of the project, they have been used as borrow materials to replace soils that required high cement contents for adequate hardening.

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*If the estimated cement content is 5 percent or less, it is good practice to use 1-percentage-point increments below 5 percent.

The procedure for testing miscellaneous materials is the same as that used for regular soils. Average cement requirements of a number of miscellaneous materials and cement contents to be investigated in the laboratory tests are given in table 4. As test data are accumulated and experience is gained with local miscellaneous materials, it may be found that future testing can be reduced or eliminated for similar materials.

Table 4. Average Cement Requirements of Miscellaneous Materials

Type of Miscellaneous Material	Estimated Cement Content (that used in moisture/density test)		Cement Contents for Wet/Dry and Freeze/Thaw Tests, percent by weight
	percent by volume	percent by weight	
Shell Soils	8	7	5- 7- 9
Limestone Screenings	7	5	3- 4- 5- 7
Red Dog	9	8	6- 8-10
Shale or Disintegrated Shale	11	10	8-10-12
Caliche	8	7	5- 7- 9
Cinders	8	8	6- 8-10
Chert	9	8	6- 8-10
Chat	8	7	5- 7- 9
Marl	11	11	9-11-13
Scoria Containing Material Retained on No. 4 Sieve	12	11	9-11-13
Scoria Not Containing Material Retained on No. 4 Sieve	8	7	5- 7- 9
Air-Cooled Slag	9	7	5- 7- 9
Water-Cooled Slag	10	12	10-12-14

APPENDIX H
RESIDUAL STRENGTH REQUIREMENTS
FOR
STABILIZED SOIL MIXTURES

The material in this appendix is taken directly from reference 105. It has been rewritten to fit into the format of this report; however, the content is not changed. This approach to establishing the residual strength requirements for stabilized soils is superior to any other technique currently in use for airport pavement design.

INTRODUCTION

Residual strength requirements generally used in soil stabilization were derived for highway pavements and associated loadings by means of linear analytical techniques. Because airport pavement thicknesses and wheel loadings are significantly different from highway pavements, it was necessary to determine the strength requirements for airport pavement sections. Furthermore, as high quality stabilized layers exhibit greater stiffness than underlying natural materials, they act in a flexural mode. The limiting value of strength for these layers to be investigated is flexural strength.

The AFPRE/AFPAV nonlinear computer codes (ref. 128) were used to account for the nonlinear stress/strain relationships of paving materials, particularly natural subgrades and unbound granular layers.

The objective of this phase of the investigation was to determine required flexural strengths of stabilized pavement layers. These values were correlated to unconfined compressive strengths, q_u, that would be required in the pavement after the first freeze/thaw season. Procedures described elsewhere in this report are then used to determine the required q_u prior to freeze/thaw on the basis of the anticipated number of freeze/thaw cycles.

ANALYSIS PROCEDURE - LIME-STABILIZED LAYERS

Typical flexible and rigid pavements were analyzed for aircraft in the three design categories described in Air Force Manual 88-6, Chapter 1 (ref. 129). The F-4E aircraft was selected for light load analysis since it has tne highest gear load in that category. The C-141 was used for the medium load category, and the B-52 for heavy load design. Flexible pavements were not included in the heavy load category analysis, nor were rigid pavements analyzed for the light load category. A wide range of subgrade types, pavement thicknesses and stabilized layer properties was investigated. Tables 1 through 4 summarize the physical properties of the pavement sections containing lime-stabilized layers. The aircraft landing gear configurations and wheel loads cover the range with which the military engineer will be involved. In addition, the majority of civilian jetliners exhibit similar characteristics.

Table 1. Light Load Design Pavement Parameters - Flexible Pavements [after Currin (ref. 105)]

Pavement Layer	Thickness Range, in	Maximum Shear Modulus (G_{max}), psi	Poisson's Ratio
Asphalt Concrete	3	60,000	0.40
Crushed Stone Base	0-35	20,000	0.35
Stabilized Layer (Lime)	10, 18, 36	25,000-90,000	0.15
Subgrade	300	1,000-12,000 (CBR Range: 1.5-23)	0.45

Table 2. Medium Load Design - Flexible Pavements [after Currin (ref. 105)]

Pavement Layer	Thickness Range, in	Maximum Shear Modulus (G_{max}), psi	Poisson's Ratio
Asphalt Concrete	3	60,000	0.40
Crushed Stone Base	0-14.5	20,000	0.35
Stabilized Layer (Lime)	6, 10, 18	25,000-90,000	0.15
Subgrade	250	1,100-12,000 (CBR Range: 2-23)	0.45

Table 3. Medium Load Design - Rigid Pavements
[after Currin (ref. 105)]

Pavement Layer	Thickness Range, in	Maximum Shear Modulus (G_{max}), psi	Poisson's Ratio
Portland Cement Concrete	15-20	1,250,000	0.20
Stabilized Base (Lime)	6	25,000-90,000	0.15
Subgrade	300	(K Range: 25-250)	0.45

Table 4. Heavy Load Design - Rigid Pavements
[after Currin (ref. 105)]

Pavement Layer	Thickness Range, in	Maximum Shear Modulus (G_{max}), psi	Poisson's Ratio
Portland Cement Concrete	20-30	1,250,000	0.20
Stabilized Base (Lime)	6	25,000-90,000	0.15
Subgrade	300	(K Range: 25-250)	0.45

The pavement thicknesses were determined from the CBR design procedure as outlined in Air Force Manual 88-6, Chapter 2 (ref. 130), for flexible pavements. The entire pavement thickness was reduced by the lime equivalency factor of 1.1 as recommended by reference 131. The rigid pavement thicknesses were determined from the design curve included in AFM 88-6, Chapter 8 (ref. 132), upon the basis of the modulus of subgrade reaction, K.

COMPUTER CODES

The AFPRE/AFPAV nonlinear finite elements computer codes are described in reference 128. Asphalt concrete, Portland cement concrete and stabilized layers were considered to be linear elastic materials. Subgrade and unbound granular layer shear stress/strain relationships were input to the programs on the basis of typical values of G_{max}, reference shear strain, and the shape of the curve as represented by the "a" value (refs. 133, 134, 135).

Values of G_{max} for the stabilized layers and surface courses were derived from the relationship

$$G_{max} = \frac{E}{2(1 + \nu)}$$

where

E = flexural modulus

ν = Poisson's ratio

The values of G_{max} listed for the lime/soil layers cover a range of flexural modulus values from 50,000 to 200,000 psi. This range is typical of the values reported by Thompson (ref. 136).

Table 5 shows pertinent parameters used in this phase. Three loading steps were used in each analysis.

Table 5. AFPRE/AFPAV Parameters [after Currin (ref. 105)]

Aircraft	Gear	Half-Period Fourier Load Function, in	Individual Tire Footprint, in	Tire Pressure, psi
F-4E	Single, Tri-cycle	150	8.9 x 11.5	265
C-141A	Twin Tandem, Tricycle	150	12.6 x 16.5	180
B-52H	Twin-Twin, Bicycle	200	12.2 x 17.6	285

For all aircraft used in this analysis, the tire groups were located to provide a symmetrical loading pattern. Therefore, an even Fourier Series was obtained, requiring only cosine terms in the load function. Fifteen cosine terms were used for the F-4E, seventeen for the B-52H, and seventeen for the C-141A.

SOIL/CEMENT SECTIONS

A number of flexible pavement sections containing soil/cement layers were analyzed. Stabilized layer and asphalt concrete layer thicknesses were the same as for the lime sections. Material properties were similar to the lime sections, with the exception that G_{max} of the soil/cement layer was 700,000 psi. This value corresponds to a flexural modulus of 1,600,000 psi, an intermediate value of the wide range over which this parameter may vary, depending upon the type of soil stabilized.

RESIDUAL STRENGTH

It is assumed that the most critical period in the life of a pavement containing stabilized layers occurs immediately after the first freeze/thaw period. It is at this point that natural subgrades may be least stable due to high moisture contents and stabilized layers have suffered the deteriorating influence of the winter freeze/thaw cycles. Thompson and Dempsey (ref. 137) have shown that stabilized materials will continue to gain strength with increased curing time after the first winter and that freeze/thaw damage occurring in subsequent winters is not cumulative. Therefore, the flexural strength required after the first freeze/thaw season may be regarded as the minimum necessary for satisfactory pavement performance.

Values of maximum flexural stress in the stabilized layers were determined for each pavement section. Because it was assumed that the stabilized material would begin to gain strength immediately after the freeze/thaw season, it was decided to allow the flexural stresses to represent 80 percent of the flexural strength during this period. With the ensuing strength gain, this percentage would drop before sufficient load cycles to cause fatigue failure could be applied. The required flexural strength for each section was determined by dividing the calculated flexural stress by 0.80. Flexural strength was converted to unconfined compressive strength, q_u, using the relationship

$$\text{Flexural Strength} = 0.25q_u$$

as reported by Thompson (ref. 136). The values of q_u thus obtained represent minimum required values of unconfined compressive strength (residual strength) the stabilized materials must exhibit in the field immediately after the first freeze/thaw season.

RESULTS

Flexible Pavements

Figures 1 through 4 show residual strength requirements for stabilized layers (lime and cement) for airport flexible pavements. The design procedure utilizing these figures should include the following steps:

(1) Use standard CBR design procedures to determine required pavement thickness.

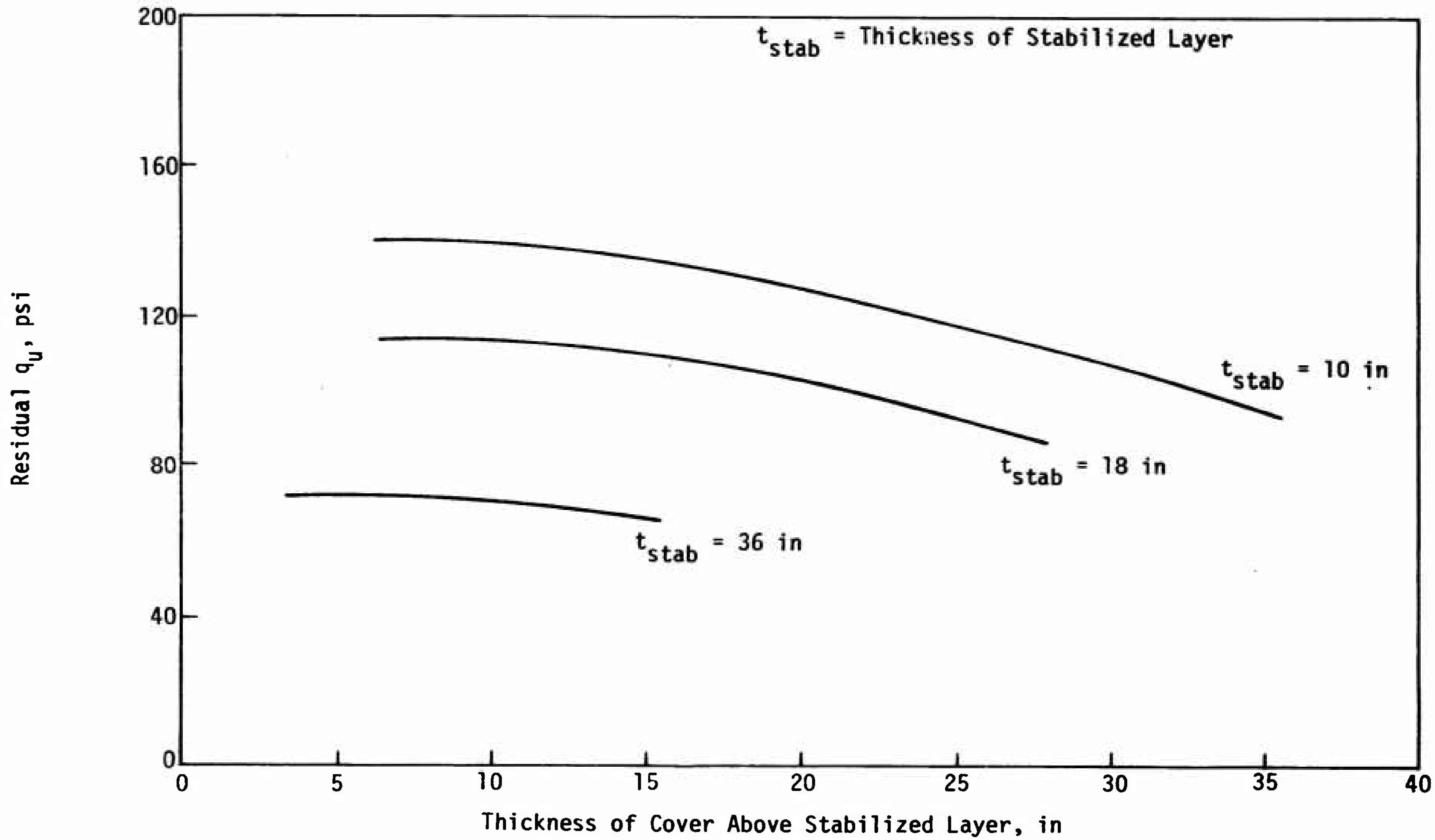

Figure 1. Residual Strength Requirements for Lime-Stabilized Layers in Airport Flexible Pavements - Medium Load Design [after Currin (ref. 105)]

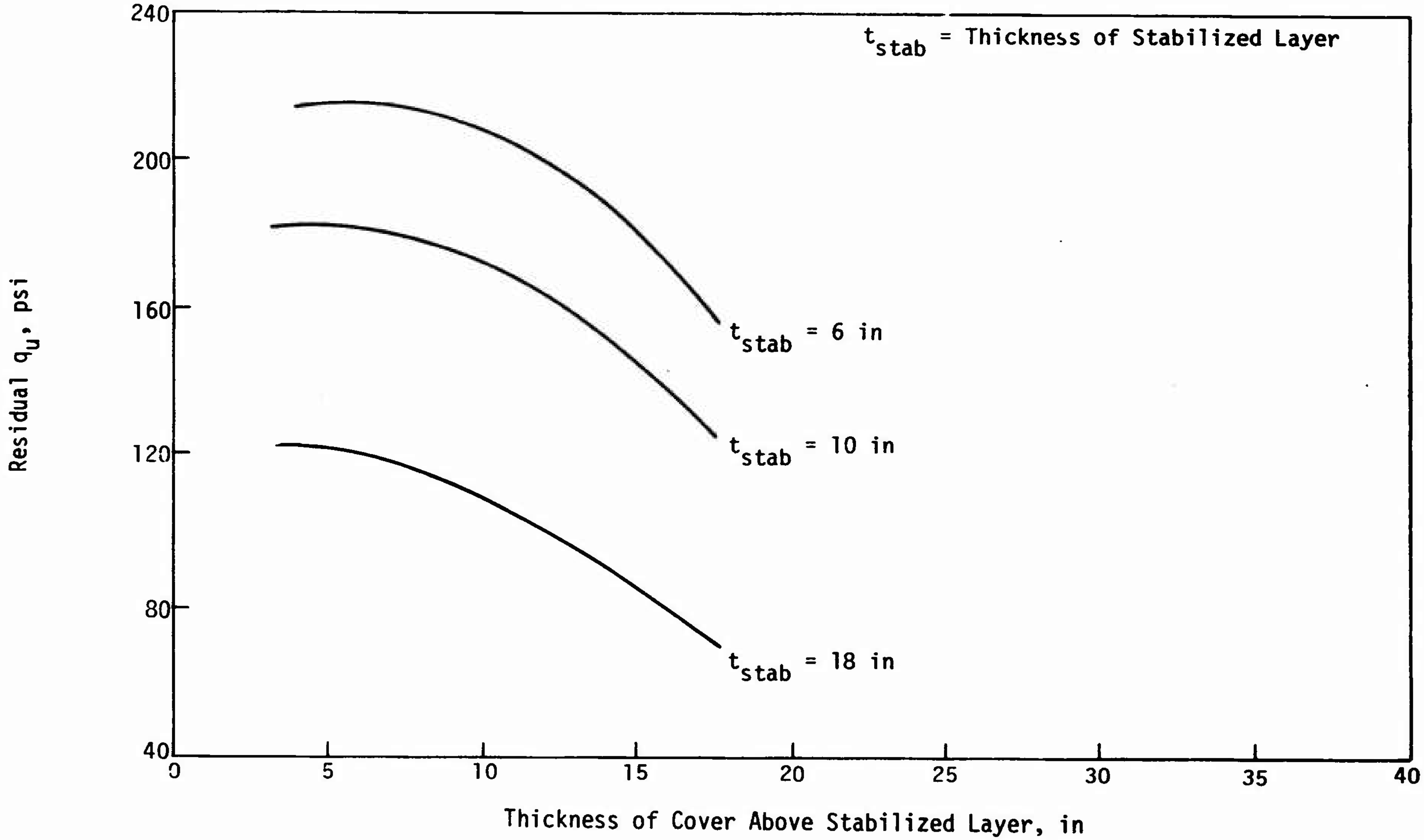

Figure 2. Residual Strength Requirements for Lime-Stabilized Layers in Airport Flexible Pavements - Light Load Design [after Currin (ref. 105)]

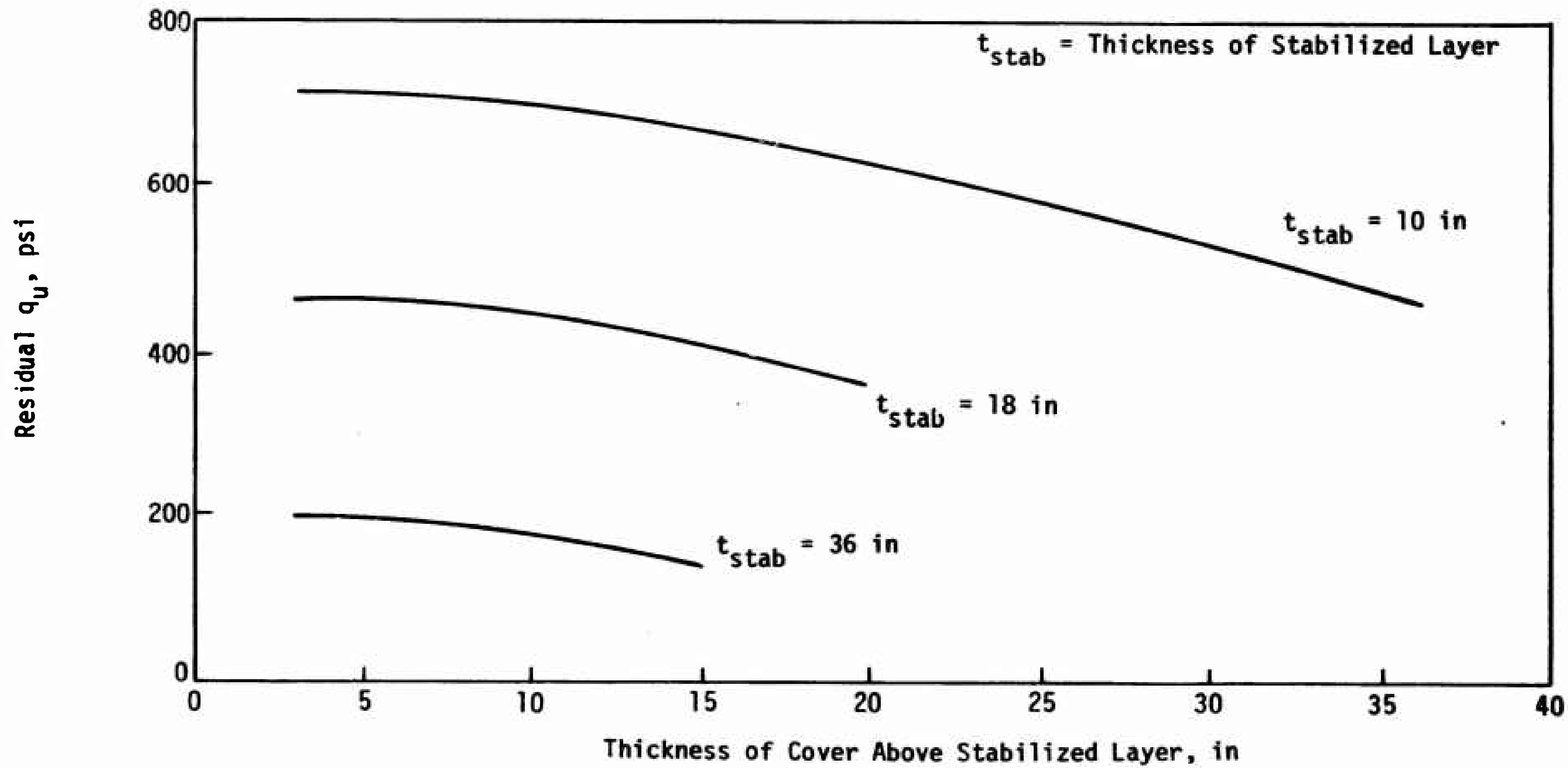

Figure 3. Residual Strength Requirements for Cement-Stabilized Layers in Airport Flexible Pavements - Medium Load Design [after Currin (ref. 105)]

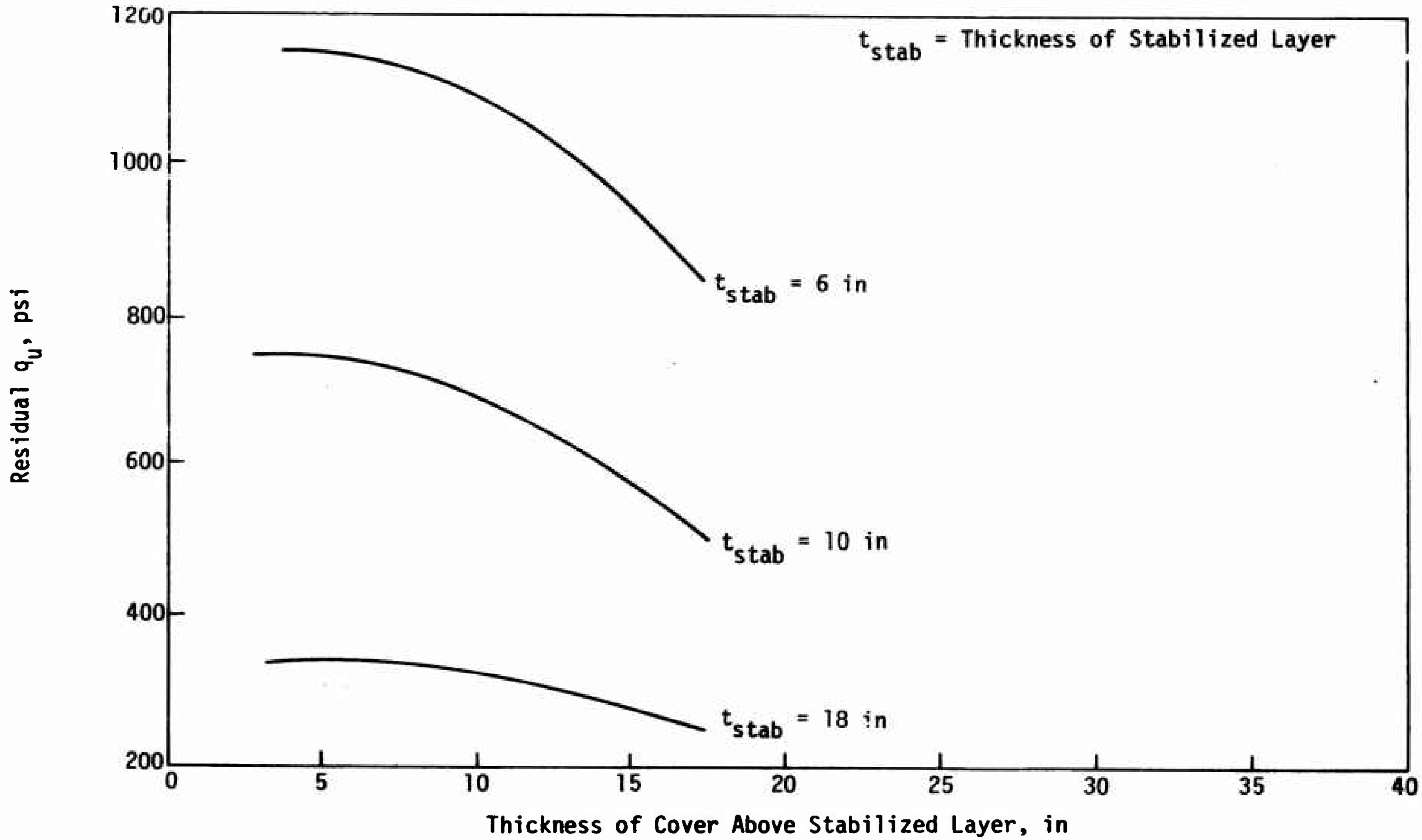

Figure 4. Residual Strength Requirements for Cement-Stabilized Layers in Airport Flexible Pavements - Light Load Design [after Currin (ref. 105)]

(2) Select individual layer thicknesses.

(3) Enter the appropriate figure (design category, type of stabilizer) with the thickness of cover (thickness of material above the top of the stabilized layer). Then read the required residual q_u from the appropriate curve for stabilized layer thickness.

(4) Using procedures outlined in reference 105, determine the strength loss for the number of freeze/thaw cycles anticipated for the first season. (See section 6 of this report, or reference 105.)

(5) Add the anticipated strength loss to the residual strength. This value represents the q_u required in the field after construction and initial curing and prior to the first freeze/thaw season.

(6) To determine the laboratory strength necessary to ensure sufficient field strength, allowances must be made for field variability and mixing efficiency. Thompson and Dempsey (ref. 137) have suggested that efficiency $\left(\frac{\text{field mix strength}}{\text{lab mix strength}}\right)$ values of 0.75 (granular) and 0.65 (fine-grained) for mixed-in-place procedures and 0.85 for plant mix operations. They also recommend that field coefficients of variation for strength be taken as 15 percent for plant mixes and 25 percent for mixed-in-place operations.

Rigid Pavements

Because of the thickness and high modulus of concrete surface courses, calculated flexural strengths varied over a small range (< 10 psi). Therefore, it is recommended that residual strength values of 60 to 80 psi be required for stabilized bases for rigid airport pavements. These values are higher than would be indicated by the relation flexural strength = $0.25q_u$. However, lower strength mixes exhibit lower values of flexural modulus which would allow larger strains and possibly more severe cracking in the stabilized layer.

DISCUSSION

The most critical step in the suggested design procedure is determining the number of freeze/thaw cycles that the stabilized layers will undergo. Several points should be discussed:

(1) Pavement system characteristics are as important as geographic and climatic considerations. As shown in reference 137, increasing the asphalt-concrete surface thickness from 2 to 4 in can decrease the number of freeze/thaw cycles occurring at a point 2 in below the top of the base course from 8 to less than 4. Similarly, numbers of freeze/thaw cycles under Portland cement concrete surfaces are drastically reduced as compared to those occurring at the same depth in the natural deposit. In addition, the type of stabilized material has an effect on freeze/thaw cycles in flexible pavements. As shown in reference 137, the ratio of freeze/thaw cycles for stabilized fine grained to freeze/thaw cycles for stabilized granular was 0.7.

(2) Although maximum strength loss in durability testing of soils in this study occurred after seven freeze/thaw cycles, it is unlikely that the majority of airport stabilized layers would undergo this number of cycles because of the insulating effects of the asphalt concrete or Portland cement concrete surface course. On the basis of published highway pavement data for the state of Illinois alone, the worst case for a pavement consisting of 3 in of asphalt concrete over a stabilized fine-grained base would be 5 to 6 cycles and 2 to 3 cycles for 8 in of Portland cement concrete. In the absence of definitive climatological data, it is recommended that these figures be considered to be the governing values for airport pavements.

ILLUSTRATIVE EXAMPLE

Given:

Medium load design

Lime-stabilized fine-grained subbase

Thickness of asphalt concrete surface = 3 in
Thickness of granular base = 12 in
Thickness of stabilized layer = 18 in
Number of freeze/thaw cycles = 5
Strength loss after three freeze/thaw cycles in laboratory = 50 psi
Mixed-in-place operation

Solution:

(1) From figure 1, with thickness of cover = 15 in and t_{stab} = 18 in, residual strength = 110 psi.

(2) From figure 17 (section 6), the five-cycle strength loss = 70 psi.

(3) Required field strength prior to freeze/thaw = 110 psi + 70 psi = 180 psi.

(4) Adjust for mixing efficiency (mixed-in-place efficiency = 0.65)

$$\frac{180}{0.65} = 277 \text{ psi}$$

(5) Adjust for field variability (coefficient of variation = 0.25)

$$\frac{277}{0.75} = 369 \text{ psi for 84\% of the material to have } q_u > 277 \text{ psi}$$

The laboratory 28-day q_u must be 369 psi to develop the required field strength of 110 psi after five freeze/thaw cycles. (Note: For a plant mix operation, the laboratory strength requirement would be reduced to 249 psi because of increased mixing efficiency and reduced field variability.)

The pavement designer should be cognizant that the preceding analysis has been concerned with pavement response to repeated dynamic wheel loadings; that is, flexural fatigue was the main consideration. It is recognized that stabilized pavement sections develop ultimate strength far in excess of the stresses that lead to initial cracking (ref. 138). For a situation where low traffic volumes are anticipated, the required strengths of stabilized layers may be significantly overestimated by figures 1 through 4. Examination of the data presented by Suddath and Thompson (ref. 138) indicates that ultimate strengths may be at least two to three times as large as those predicted by Meyerhof's ultimate load theory. Clearly, for these situations, the designer is justified in accepting lower strengths than those indicated by figures 1 through 4.

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