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Validation of Procedures for Pavement Design on Expansive Soils

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16. Abstract

This report documents site investigations and design recommendations for airport pavements on expansive soils at three sites. The methods used were developed previously in an FAA-sponsored research project. The design procedure involves soil characterization, determination of surface characteristics and evaluation of soil-pavement interaction. Investigations at three airports are reported; they are Murdo, South Dakota, Mesquite, Texas, and Love Field, Dallas, Texas. Soils data are used along with climatic information to obtain estimates of the equivalent thickness of pavement required for each site. The work at Love Field was somewhat different than the other sites since NMERI was not directly conducting the investigation. Results are reported in the form of pavement thickness required to adequately perform, in terms of surface roughness, on the soil surface characteristics predicted for each site. Recommendations are included for consideration in rehabilitation of each site since all were experiencing surface roughness at the time of the investigations. These results confirm that methods developed for evaluating airport pavements on expansive soils provide useful information for pavement design. This method specifically addresses the expansive soil problem and does not address the effects of traffic or load-induced pavement distress. These aspects are believed to be adequately covered in standard FAA design procedures. Keywords:

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I. INTRODUCTION

The construction of airports provides facilities that directly influence vital transportation activities within surrounding areas. The substantial cost of such facilities and the economic impact of their construction and maintenance have dictated a role for the U.S. Government through the Department of Transportation, Federal Aviation Administration (FAA). One aspect of this role involves the identification and solution of technical problems associated with the design and performance of airport facilities. A significant problem is damage to airport pavements (runways, taxiways, aprons) caused by volume change (shrinking or swelling) of expansive soil subgrades.

In January, 1981 the FAA published results of a research study, performed by the New Mexico Engineering Research Institute (NMERI), addressing this problem and proposing an engineering approach to airport pavement design for expansive soil areas (Ref. 1). The present study applies that approach to airport pavement design problems at three sites to validate and present it in an applied engineering context.

The proposed methods offer an approach to evaluating the behavior of expansive soils as airport pavement subgrades. Portions of the work are empirical and, therefore, must be cautiously extended to new areas. However, in the context of local experience and engineering judgement the tools proposed here are expected to be of significant value in assessing problems associated with pavement structures founded on expansive soils.

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1. McKeen, R. G., **Design of Airport Pavements on Expansive Soils**, FAA-RD-81-25, Federal Aviation Administration, Washington, D.C., January 1981.

II. BACKGROUND

PREVIOUS REPORTS

Several reports and papers have been contributed by NMERI on the design of airport pavements on expansive soils related to the FAA investigation. A literature review was conducted in an attempt to develop a design method from the technical literature (Ref. 2). The results were an engineering approach requiring a research study to develop, and an available procedure for stabilization using lime and cement, but did not address shrink-swell behavior adequately. A laboratory research phase was performed to investigate methods of characterizing expansive soils in terms of meaningful material properties (Ref. 3). A field study of expansive soil behavior and interaction with actual airport pavements resulted in a proposed engineering approach (Ref. 1). Since the completion of that work two further modifications were made and reported (Refs. 4 and 5).

The latest discussion used several characteristics of in situ expansive soils to make predictions of field behavior. A synopsis of the method is included in Section IV. As shown in the synposis, five quantities are required to predict the expected differential heave. They are suction compression index, its variation, lateral restraint factor, active zone depth, and suction change. These properties and the concepts involved are discussed in more detail in the following sections.

---

2. McKeen, R. G., **Design and Construction of Airport Pavements on Expansive Soils**, FAA-RD-76-66, Federal Aviation Administration, Washington, D.C., 1976.

3. McKeen, R. G., and Nielsen, J. P., **Characterizing Expansive Soils for Airport Pavement Design**, FAA-RD-78-59, Federal Aviation Administration, Washington, D.C., 1978.

4. McKeen, R. G., and Lenke, L. R., "Thickness Design for Airport Pavements on Expansive Soils," **Proceedings of the Nineteenth Paving and Transportation Conference**, University of New Mexico, 1982.

5. McKeen, R. G., and Lytton, R. L., "Airport Pavement Design Using Case Studies," **Proceedings of the International Conference on Case Histories in Geotechnical Engineering**, St. Louis, May 1984.

SCOPE OF THIS STUDY

The present study was initiated to apply the methods previously proposed in the context of applied engineering and in locations having different geology/climate from previous study sites. The activities were intended to approach the study in the following sequence: (1) site investigation, (2) laboratory testing, (3) analysis, and (4) recommendation. Some simplification of the concepts previously studied was a desirable result to provide tools that could reasonably be used by practicing engineers. Therefore, the review and evaluation of previous work was an integral part of the present study.

III. EXPANSIVE SOIL CHARACTERISTICS

PROBLEM DESCRIPTION

Figures 3-1 and 3-2 illustrate the results of expansive soil movements beneath a rigid pavement and a flexible pavement. Elevation profile data are presented in Figures 3-3 and 3-4 for comparison. Note that in the case of the rigid pavement illustrated, slabs have apparently behaved as rigid bodies, a typical response. Distress in the flexible pavement manifests itself differently as shown in Figure 3-4. It should be clear that both soil and pavement structure characteristics determine the nature of pavement distress that occurs.

Figure 3-5 shows the centerline profile of a runway measured at various times following construction. This illustrates the progressive nature of profile deterioration with time. Although most expansive soil problems occur within 3 to 5 years, the actual rate may vary widely depending on many factors. Another feature of all distressed pavements shown is that their load carrying capacity is not threatened. The problem created by expansive soils is profile roughness that is not acceptable to the user. Roughness of airport pavements is a complex area of study and is difficult to quantify precisely. In previous work (Ref. 1) the acceptance was established by aircraft simulations using the TAXI computer code (Ref. 6). No further study of these criteria was attempted in this work.

The problem may be summed up as unacceptable roughness in airfield pavements that occurs well before the design life is attained. The distress is progressive in nature, being initiated by soil movements followed by accelerating deterioration as the traffic loads interact with the rough surface.

SOIL SUCTION

Soil suction is a measure of the energy associated with water held in the soil. Expansive soils are invariably fine-grained, made up of significant

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6. Gerardi, A. G., and Lohwasser, A. K., **Computer Program for the Prediction of Aircraft Response to Runway Roughness, Volume I, Program Development**, AFWL-TR-73-109, Volume I, U. S. Air Force Flight Dynamics Laboratory, 1973.

FIGURE 3-2. RUNWAY, MURDO AIRPORT

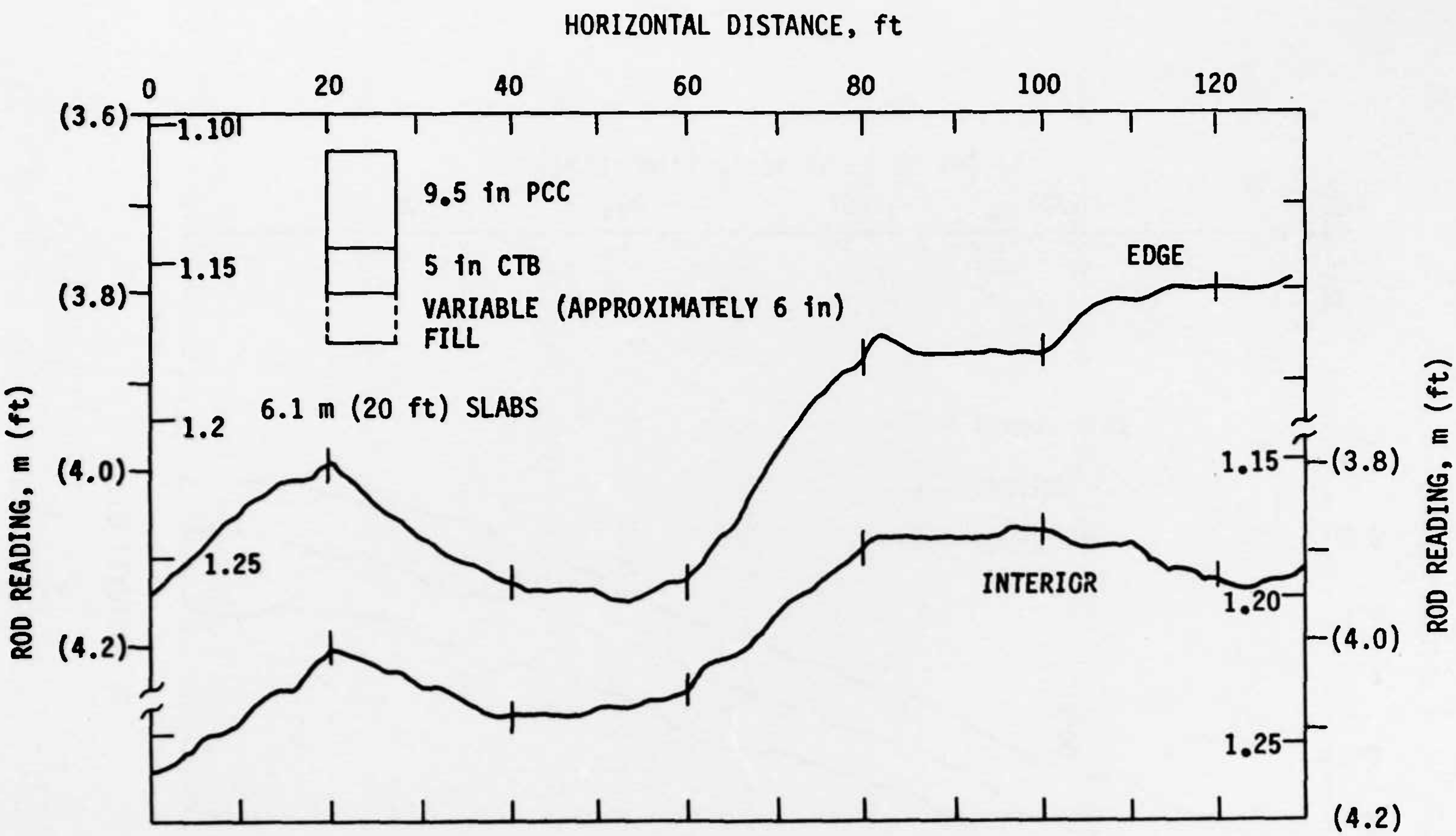

FIGURE 3-3. ACTUAL PROFILES, WEST TAXIWAY, JACKSON AIRPORT

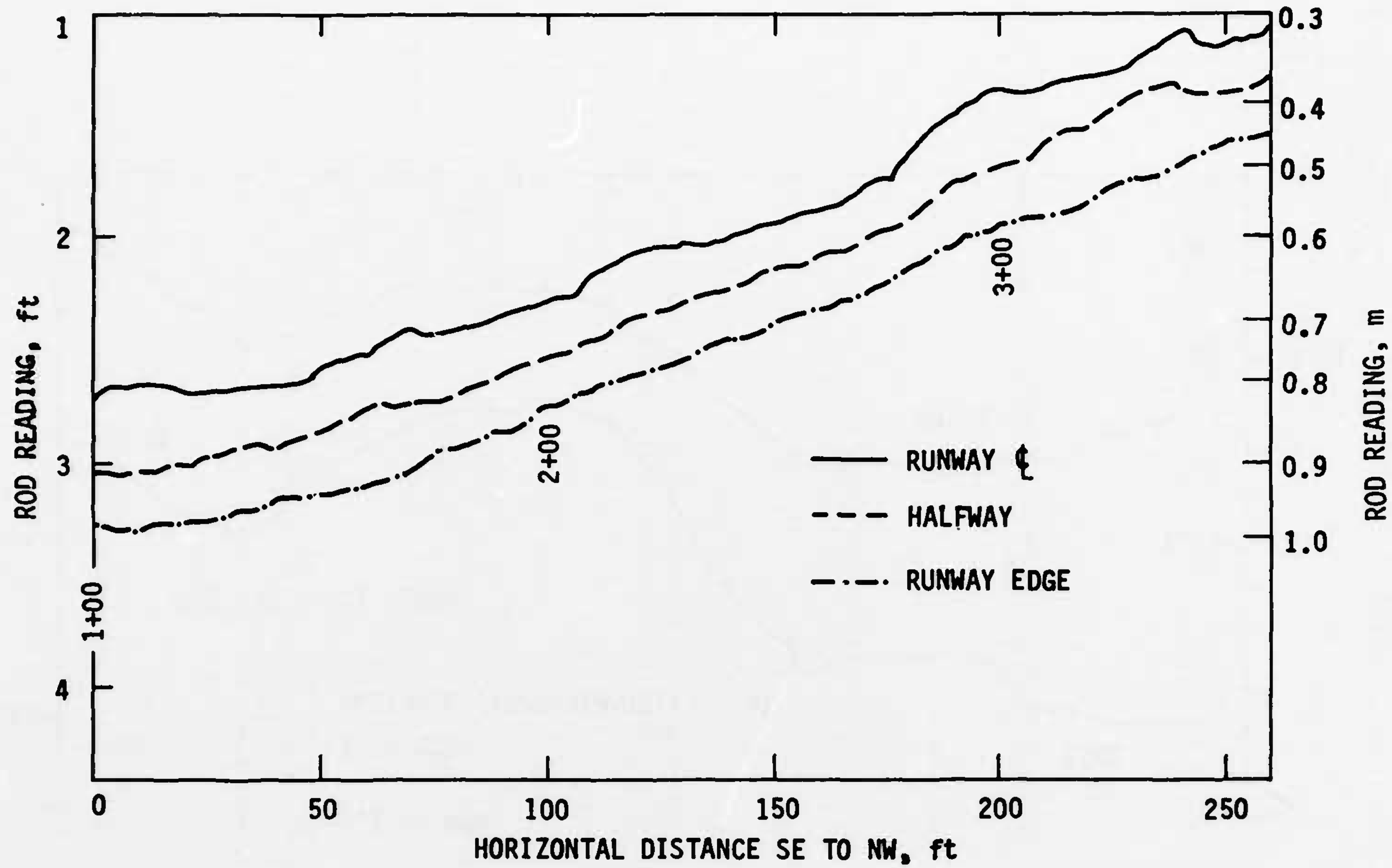

FIGURE 3-4. MURDO AIRPORT RUNWAY PROFILES

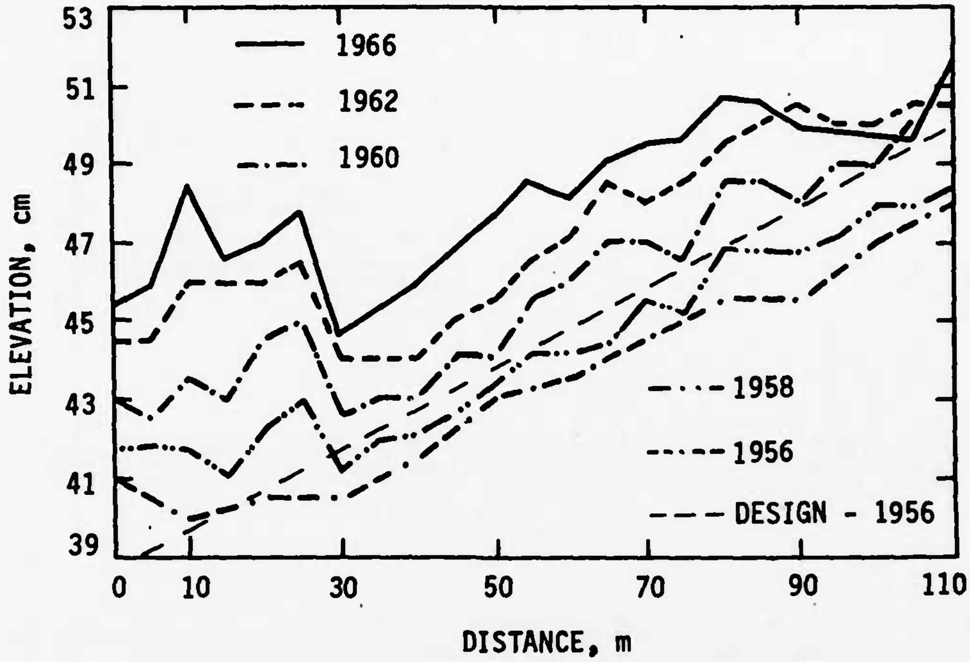

FIGURE 3-5. RUNWAY CENTERLINE PROFILE (REF. 7)

amounts of clay sized particles (<2 μm) containing clay minerals. These materials can attract and hold large quantities of water. Researchers have recognized two components of soil suction. They are matric, because of adsorption and capillary phenomena, and osmotic, associated with variation in salt concentrations in the soil water. The sum of these components is called total suction. In many engineering problems it is acceptable to equate changes in total suction with changes in matric suction. Appendix A presents a discussion of soil suction.

The relationship between soil suction and water content is called the moisture characteristic. Figure 3-6 illustrates data for use in defining this relationship. The slope of the moisture suction relation is an important soil characteristic. A procedure for its determination is presented in Appendix B.

7. Uzan, J, Frydman, S., and Wiseman, G., "Roughness of Airfield Pavement on Expansive Clay," Proceedings of the Fifth International Conference on Expansive Clay Soils, Institution of Engineers, Adelaide, South Australia, 21-23 May 1984, pp. 286-291.

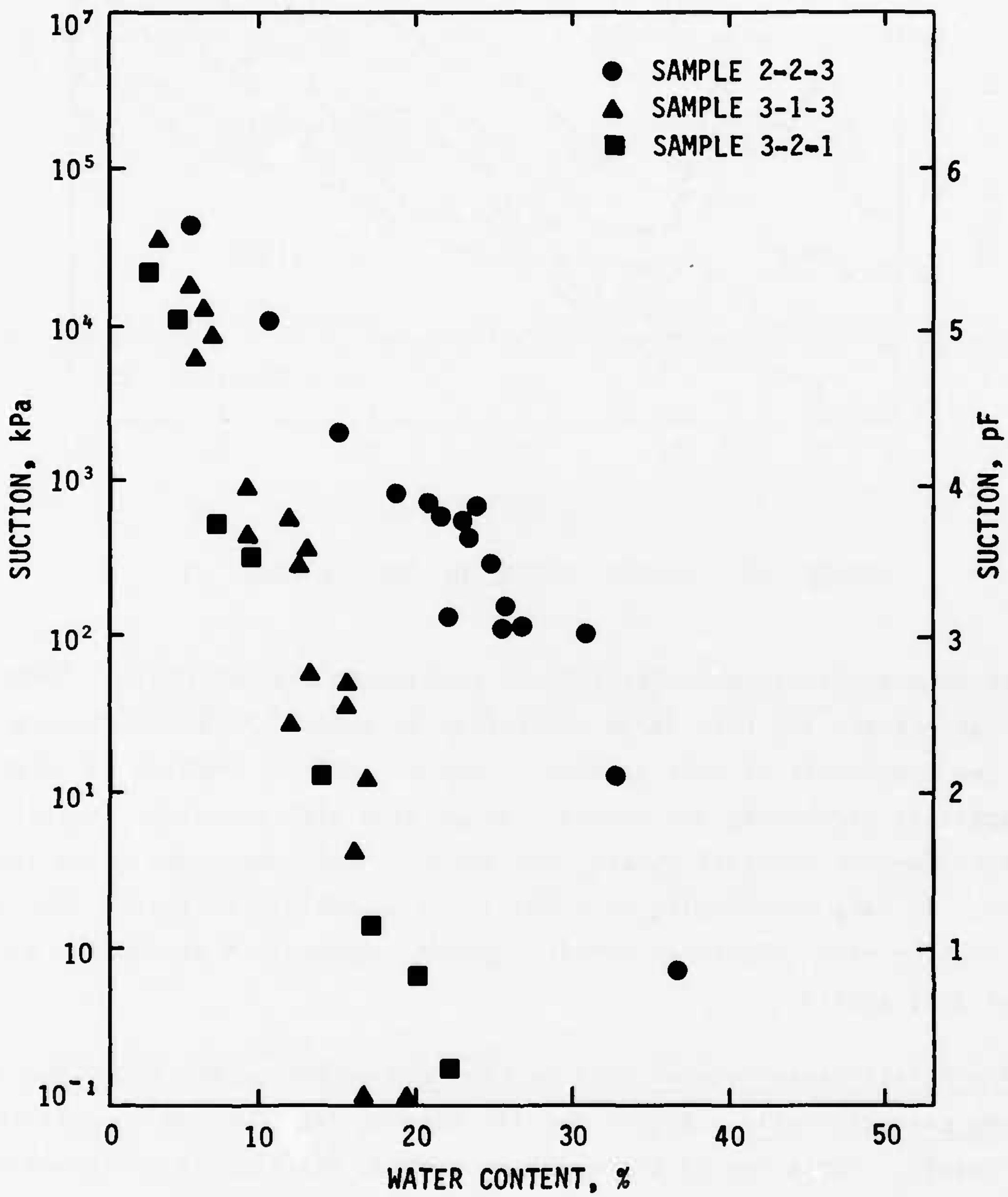

FIGURE 3-6. DALLAS/FORT WORTH AIRPORT SAMPLES

An integral part of site investigation involves the evaluation of suction and water content profiles. It is difficult to use water content to predict profiles because they can show wide variation based on soil properties. Soil suction tends to be uniform or at least show continuous trends, and it is easier to predict profiles as they tend toward equilibrium with a particular environmental effect. It can therefore be easier to predict expansive soil behavior on the basis of suction rather than water content (Ref. 8).

It is vitally important to recognize that data collected at a specific time are like a snapshot of the moisture condition. As illustrated in Figure 3-7, the profiles vary as the active zone of the soil interacts with environmental influences (Ref. 9). Another example of typical behavior is shown in Figure 3-8, as reported by Aitchison and Holmes (Ref. 10). This illustrates the vertical heave at various depths below the surface in an expansive clay through 3 years. A discussion of suction profiles is included in Appendix C.

SUCTION COMPRESSION INDEX

The best approach for modelling stress-strain behavior in unsaturated soils is to describe the stress state using two independent effective stress

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8. Mitchell, P. W., and Avalle, D. L., "A Technique to Predict Expansive Soil Movements," **Proceedings of the Fifth International Conference on Expansive Soils**, Institution of Engineers, Adelaide, South Australia, 21-23 May 1984, pp. 124-130.

9. McKeen, R. G., "Field Studies of Airport Pavements on Expansive Soils," **Proceedings of the Fourth International Conference on Expansive Soils**, Vol. 1, Denver, Colorado, June 16-18, 1980, pp. 242-261.

10. Aitchison, G. D., and Holmes, J. M., "Aspects of Swelling in the Soil Profile," **Australian Journal of Applied Science**, No. 4, 1953, pp. 244.

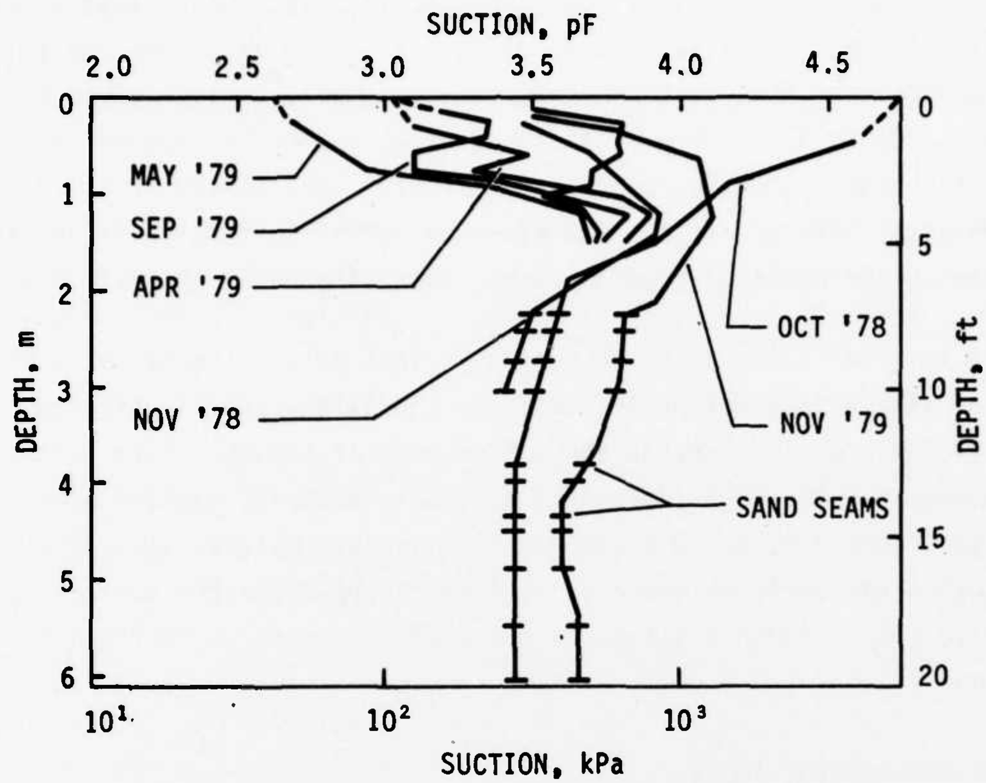

FIGURE 3-7. DALLAS/FORT WORTH SUCTION PROFILES (REF. 9)

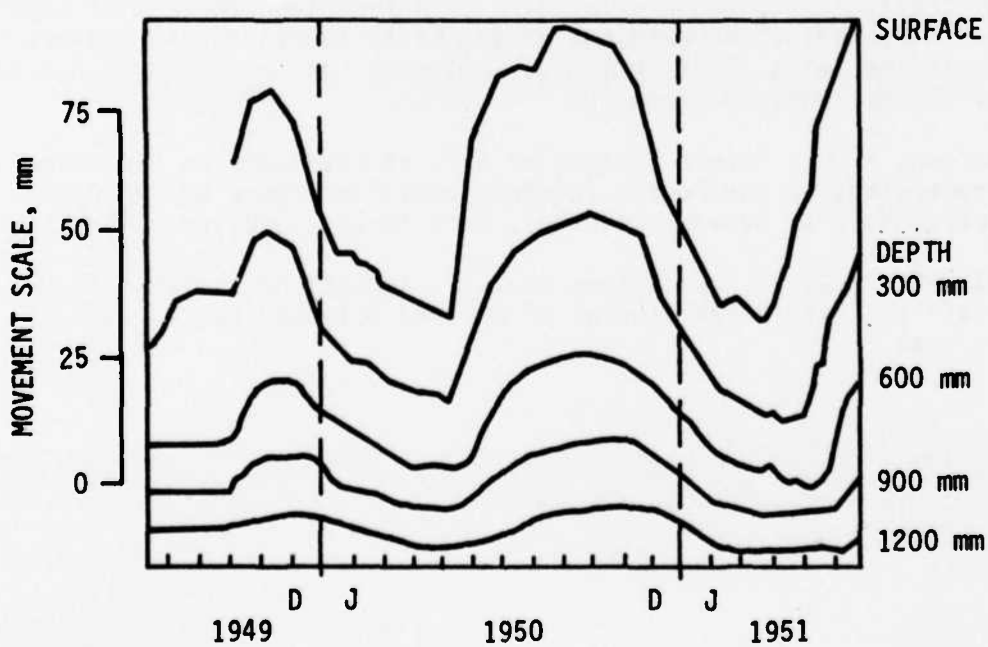

FIGURE 3-8. OBSERVED SEASONAL SOIL MOVEMENTS (REF. 10)

variables (Ref. 11). In this representation the stress associated with overburden and mechanical loads is represented by $(\sigma - u_a)$. Stress associated with the pore pressures is represented by the matrix suction variable $(u_a - u_w)$.

Figure 3-9 illustrates the idealized volumetric behavior following this approach. In this case e is the void ratio of the soil, $(u_a - u_w)$ represents matric suction, C_m is the rate of change of void ratio (e) with change in the logarithm to the base 10 of matric suction, $(\sigma - u_a)$ is the applied stress, and C_c is the rate of change of e with change in the logarithm to the base 10 of applied stress. This concept exhibits a smooth transition into the saturated (positive pore pressure) region. The present study further assumes the load variation involved is rather small, and that moisture stresses are the overwhelming factor controlling soil volume changes of importance in pavement design.

In previous studies the suction compression index, γ_h, has been used to represent the slope of the volume change-suction relationship. The present report uses another variable C_h to represent the suction compression index, following the idea proposed by Hamberg (Ref. 12). This corresponds to C_m in Figure 3-9. The differences between γ_h and C_h are that γ_h expresses volume change in reference to oven-dry volume, while C_h expresses volume change using the initial volume as the reference. These correspond to C_m in Figure 3-9, which, like γ_h, is developed from using small increments of change, making the reference volume unimportant.

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11. Fredlund, D. G., and Morgenstern, N. R., "Stress State Variables for Unsaturated Soils," **Journal of the Geotechnical Engineering Division, ASCE**, Vol. 103, No. GT5, 1977, pp. 447-466.

12. Hamberg, D. J., **A Practical Method for Predicting Heave in Expansive Soils**, Thesis, Colorado State University, Ft. Collins, Colorado, May 1955.

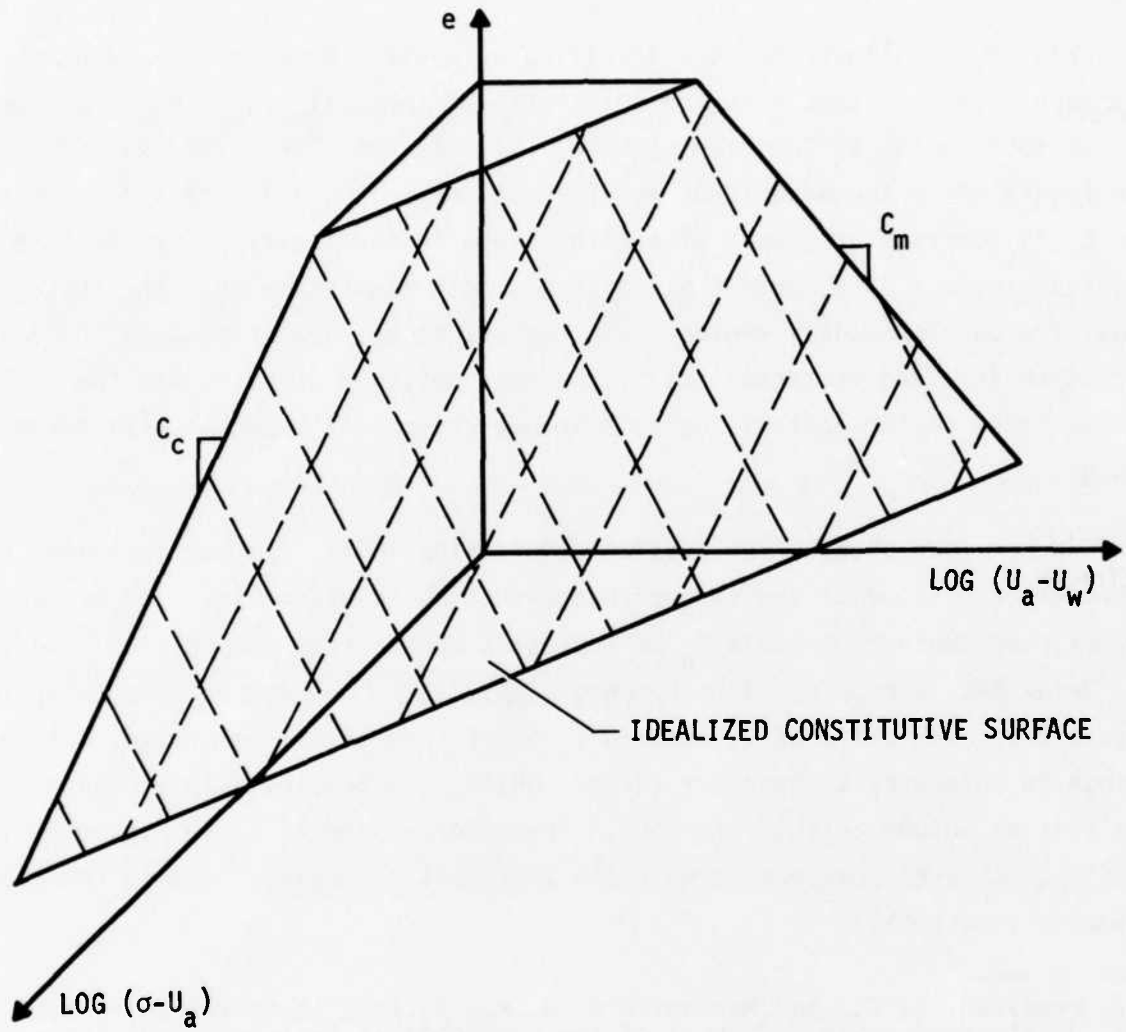

FIGURE 3-9. CONSTITUTIVE MODEL OF SOIL BEHAVIOR (REF. 11)

The original suction compression index, (γ_h), was presented by Lytton (Ref. 13) and took the form

$$\gamma_h = \frac{\Delta V/V}{\Delta h} = \frac{\Delta e/[1 + e_f]}{\Delta h}$$

where

γ_h = suction compression index

$\Delta V/V$ = volumetric strain

Δh = suction change = $(h_f - h_i)$

h_i, h_f = initial and final suction

Δe = change in soil void ratio = $e_f - e_i$

e_i, e_f = initial and final void ratio

This index employed an incremental approach involving small volume changes. Therefore, the question of which volume as represented by V on the above equation was not important. However, the previous NMERI work carried this idea a step further and used the principles of the Soil Conservation Service's Coefficient of Linear Extensiblity (COLE) test to measure γ_h . First, the COLE is determined as follows:

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

where

COLE = coefficient of linear extensibility

Db_d = dry bulk density of the oven-dried sample

Db_m = dry bulk density of the sample equilibrated at 1/3 bar

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13. Lytton, R. L., "The Characterization of Expansive Soils in Engineering," Presentation of the Symposium on Water Movement and Equilibrium in Swelling Soils, American Geophysical Union, San Francisco, California, December 1977.

In terms more familiar to civil engineers this may be written

$$\text{COLE} = \left[\frac{\Delta V}{V_d} + 1\right]^{1/3} - 1 = \left[\frac{\Delta e}{1 + e_d} + 1\right]^{1/3} - 1$$

where

ΔV = change in soil volume

V_d = volume of over-dry soil sample

Δe = change in soil void ratio = $e_f - e_i$

e_d = void ratio of the oven-dry soil sample (in this case $e_f = e_d$)

Observations of actual soils yield values of $\Delta V/V_d$ in the range of 0.05 to 0.30. Calculation of COLE corresponding to these yields

$$(1.05)^{1/3} - 1 = 0.0164$$
$$(1.30)^{1/3} - 1 = 0.0914$$

The procedure for calculating COLE converts the volumetric strain ($\Delta V/V_d$) to an estimated linear strain ($\Delta L/L_d$), where ΔL is a length change and L_d is a length on the oven-dry sample. If the ratio of linear strain to volumetric strain is determined in the COLE procedure, it is found to be in the range 0.3 (at $\Delta V/V_d$ = 0.3) to 0.33 (at $\Delta V/V_d$ = 0.05). This ratio is called the lateral restraint factor (f). A popular assumption for the value of this ratio is 1/3 (Refs. 14 and 15). It is also important to note that COLE values are based on a volume strain over a very wide range (from near saturation to oven dry) and the reference volume is the oven dry volume.

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14. Richards, B. G., "Moisture Flow and Equilibrium in Unsaturated Soils for Shallow Foundations," **Permeability and Capillarity of Soils**, ASTM, Special Technical Publication No. 417, 1967.

15. McDowel, Chester, "Interrelationship of Load, Volume Change and Layer Thicknesses of Soils to the Behavior in Engineering Structures," **Proceedings of the Highway Research Board**, Vol. 35, 1956, pp. 754-772.

The clod test proposed by NMERI (Refs. 1 and 3) was developed from the COLE procedure and the oven dry volume was the reference volume, but no assumption was made about the ratio of linear strain to volume strain. Instead the lateral restraint factor was used to represent this ratio, as shown below:

$$\gamma_h = \frac{[\Delta V/V_d] \cdot f}{\log_{10}\left(\frac{h_f}{h_i}\right)}$$

where

γ_h = suction compression index

$[\Delta V/V_d]$ = volume strain with respect to dry volume

$$= \frac{(\gamma_d)\ dry}{(\gamma_d)_{nat}} - 1$$

f = lateral restraint factor

$(\gamma_d)_{dry}$ = dry bulk density of the oven dry soil = Db_d

$(\gamma_d)_{nat}$ = dry bulk density of the soil at natural moisture = Db_m

h_i, h_f = initial and final suction (arithmetic units)

It was proposed by Hamberg (Ref. 12) to redefine the suction compression index as C_h using the volume at natural moisture condition as the reference volume. This is clearly a better approximation of the volume strain occurring in a field soil. The value is calculated as follows:

$$C_h = \frac{[\Delta V/V_{nat}] \cdot f}{\log_{10}\left(\frac{h_f}{h_i}\right)}$$

where

C_h = suction compression index

$\Delta V/V_{nat}$ = volume strain with respect to natural volume

$$= \left[\frac{(\gamma_d)\ dry}{(\gamma_d)_{nat}} - 1\right]\frac{(\gamma_d)_{nat}}{(\gamma_d)_{dry}}$$

(other variables same as above)

Throughout this report the variable C_h will be used to represent the compression index for the soils studied. The procedure for its determination is detailed in Appendix D. Table 3-1 provides example data comparing these indexes for several samples.

TABLE 3-1. COMPARISON OF VARIOUS VOLUME CHANGE MODELS

Sample Number	h_i pF	$(\gamma_d)_{wet}$ g/cc	$(\gamma_d)_{dry}$ g/cc	COLE	γ_h	C_h
17-1	4.592	1.650	2.033	0.0721	0.256	0.208
17-6	3.137	1.400	2.038	0.1333	0.193	0.133
18-9	2.501	1.673	1.939	0.0504	0.050	0.046
TH-1-4-IV	4.200	1.578	1.944	0.0720	0.178	0.145
TH-2-4-I	2.530	1.415	1.843	0.0920	0.104	0.080

ACTIVE ZONE DEPTH (Z_a)

The surfaces of all soil deposits in nature are exposed to environmental forces. Typically these materials cycle through wet and dry periods in response to the precipitation and transpiration produced by the climate. In most problems involving expansive soils it is the active zone that is subjected to environmental change causing movement and the resulting distress in engineering structures. It is therefore necessary to establish the active zone depth for design of airport pavements on expansive soils.

The best method for determination of active zone depth is observation of the moisture profile (suction and water content) through a full cycle of wetting and drying. A comparison of the weather observed with historical records then leads to criteria for selection of the active zone depth at or below the depth to which seasonal changes occur. An example of this procedure is illustrated using data in Figure 3-7. Data shown are profiles of suction at a site on the Dallas/Ft. Worth Airport during 1978 and 1979. The Thornthwaite Moisture Index calculated for 1979 was -8 while the normal is -12 (Ref. 1). This indicates a near normal year, slightly wetter than the average. The active zone depth (Z_a) is selected as the deepest penetration of moisture changes from the surface (1.8 m [6 ft]) plus an additional increment for more severe conditions (0.3 m [1 ft]), making the active zone depth for design 2.1 m [7 ft].

where

σ^2 = variance

σ = standard deviation

$\bar{x}$ = mean

CV = coefficient of variation

x_n = individual observation

N = number of observations

MODULUS OF LACK OF SWELL CURVE (k)

A value of the modulus of the lack of swell curve is used in computations for the evaluation of soil-pavement interaction. It is determined as the pressure increment preventing swell divided by the amount of swell prevented. A popular way to measure k is with conventional consolidation equipment. The soil is loaded at the overburden pressure (p_1), then inundated and allowed to swell (ΔL), subsequent loads then return the sample to its orginal height at a pressure (p_2) greater than p_1. Following this method yields an estimate of k for the soil,

$$k = \frac{p_2 - p_1}{\Delta L}$$

In the previous report (Ref. 1) values for the study sites were determined in this manner yielding the following values.

Site	k, kPa/m (pci)
GAL	1.65×10^5 (608)
DFW	3.56×10^5 (1310)
JSN	2.26×10^5 (587)

IV. EVALUATION/DESIGN PROCEDURES

INTRODUCTION

Previous research has produced recommendations for the design of airport pavements on expansive soils (Refs. 1, 4, and 5). A three-step approach is involved in the work, consisting of soil characterization, prediction of surface characteristics and evaluation of soil-pavement interaction. The result obtained from this procedure is a pavement thickness needed to provide sufficient bending stiffness to remain acceptable for commercial aircraft considering the given soil and environmental conditions. The basis for the method is the field experiment previously reported. Thickness obtained is independent of traffic levels. The design is used to evaluate the soil and environment and provides a pavement thickness to meet predicted soil behavior. As a result, general aviation facilities such as the Murdo and Mesquite airports may require significant pavement structures to perform satisfactorily on expansive soils, since traffic is not the key factor. The effects of expansive soils may be reduced by recognizing the factors involved and incorporating this into the design/construction approach. Permitting moisture equilibrium beneath pavements prior to final paving is a significantly beneficial technique. Construction of horizontal or vertical moisture barriers has been proven of benefit. Stabilization methods including mechanical compaction or chemical additivies can be evaluated using the design method. In the following paragraphs the procedures are described, then they are applied to the runways at Murdo, Mesquite and to a limited extent Love Field.

SOIL CHARACTERIZATION

As shown in Figure 4-1 four characteristics of the soil profile are important. They are discussed in the previous section in detail. The methods recommended are valuable for several reasons. First, they are related to the theoretical models that best describe the behavior of expansive soils. Second, the test procedures are cheap and quick, thus a large number of tests can be run. A greater testing frequency is particularly significant for soils in which the property variation ($CV[C_h]$) is important. Third, the use of suction profiles in evaluating the moisture condition of an expansive soil is

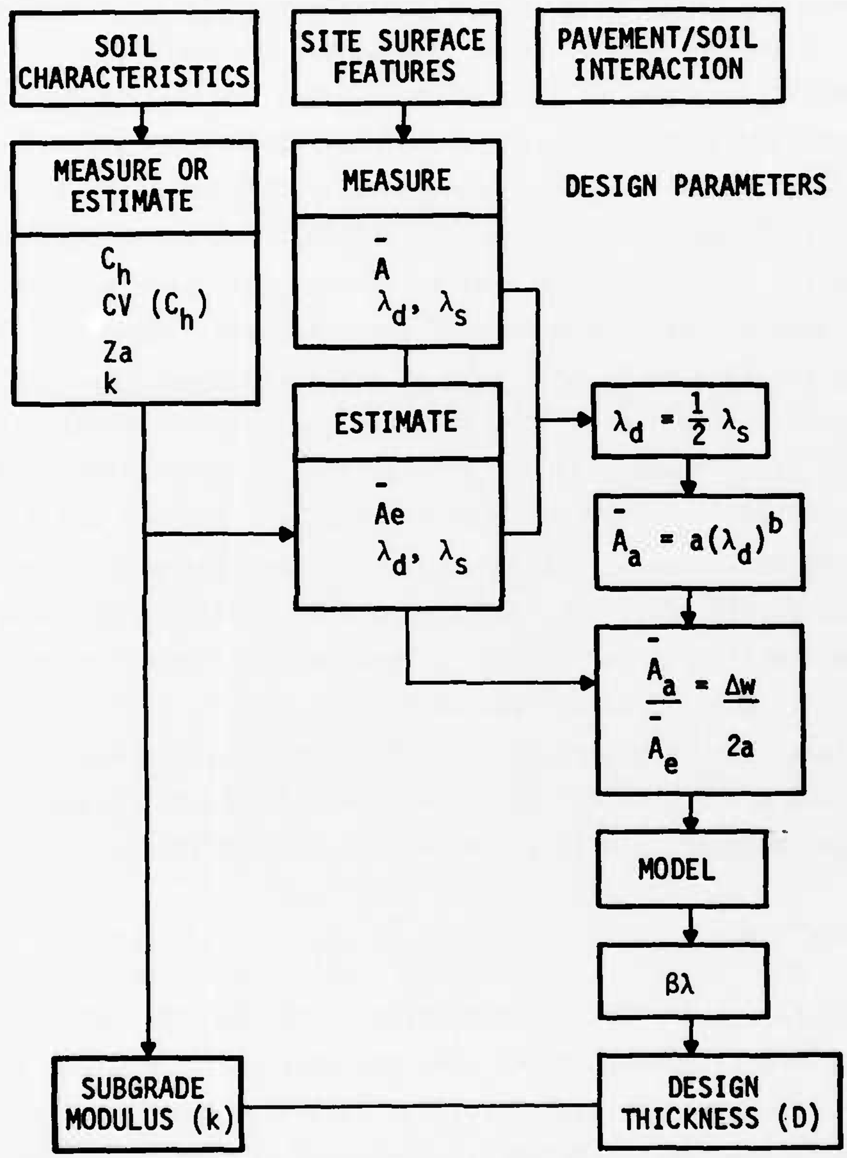


FIGURE 4-1. EVALUATION/DESIGN PROCEDURE

much better than water content. The use of suction data yields trends that are meaningful in terms of gradients, moisture equilibrium and prediction of future changes.

SOIL SURFACE FEATURES

After the soil characteristics C_h, $CV(C_h)$, Z_a and k are available for design the soil surface features must be obtained. The information required is $\overline{A}$ and λ_s, the soil surface weighted amplitude and characteristic wavelength respectively. In the original field study (Ref. 1), these were obtained by measuring soil surface elevation at 0.6-m (2-ft) intervals over a 40-m (130-ft) distance. These data were transformed to the frequency domain using a standard Fast Fourier Transform (FFT) found in signal analysis and statistical packages for most computer systems. The individual amplitudes of each frequency component calculated, a_n, were then used to compute $\overline{A}$ as follows:

$$\overline{A} = \sum_{n=1}^{N/2} a_n \cos \frac{2\pi n}{N}$$

where

a_n = maximum amplitude of the nth frequency component
N = number of components calculated
n = 1, 2, 3 ... N

This procedure yields a single number to rate the degree or magnitude of the differential surface elevation in the measurements.

The soil surface characteristic wavelength, λ_s, is obtained by calculating the autocorrelation function of the elevation profiles and plotting it versus the lag distance. The lag distance where the autocorrelation function goes to zero is called the decorrelation distance and is the characteristic wavelength used. Autocorrelation computations are a standard feature of statistical analysis packages.

In the previous field study the measured values of $\overline{A}$ and λ_s were correlated with other characteristics which may be predicted from them. The expected amplitude, $\overline{A}_e$, can be predicted as follows:

$$\overline{A}_e = C \cdot C_h \cdot f \cdot Z_a \cdot CV(C_h) \cdot \Delta h_a$$

where

$\bar{A}_e$ = expected amplitude of the soil surface

C = an empirical constant, from the field study

C_h = suction compression index

f = lateral restraint factor

Z_a = active zone depth

$CV(C_h)$ = coefficient of variation of C_h

Δh_a = change in the average suction in the active zone, in logarithmic units

The previous work resulted in an estimate of C equal to 0.37. This value was developed assuming the active zone suction change averaged about two orders of magnitude. The f at GAL was taken as 1.0 yielding the value for C. Subsequently, f values for DFW and JSN were determined. In the current study, the values of C_h are different from γ_h previously used (see Section III) and a different value of Δh_a is appropriate (see Appendix C). As a result of these changes, the prediction of $\bar{A}_e$ is made using the following equation:

$$\bar{A}_e = 0.48 \cdot C_h \cdot f \cdot Z_a \cdot CV(C_h) \cdot \Delta h_a$$

and taking the approximations of $f = 1.0$, $\Delta h_a = 1.25$ log suction yields

$$\bar{A}_e = 0.6 \cdot C_h \cdot Z_a \cdot CV(C_h) \quad (\bar{A}_e \text{ has same units as } Z_a)$$

Once the appropriate data are available [C_h, Z_a, $CV(C_h)$], $\bar{A}_e$, an estimate of $\bar{A}$, may be calculated.

Another important feature of the surface is the acceptable amplitude on the pavement surface, denoted by $\bar{A}_a$. In the field study it was obtained in relation to surface wavelength by aircraft simulation. Those results yield a relation for $\bar{A}_a$ in terms of wavelength:

$$\bar{A}_a = 0.0023\ (\lambda_d)^{1.354} \qquad (\lambda_d, \bar{A}_a \text{ in m})$$

$$\bar{A}_a = 0.00153\ (\lambda_d)^{1.354} \qquad (\lambda_d, \bar{A}_a \text{ in ft})$$

These $\overline{A}_a$ values are considered to be the maximum acceptable, based on aircraft simulations reported in earlier work (Ref. 1). The criterion used to establish this value was pilot station vertical acceleration calculated using the TAXI computer code.

Estimates of λ_s can be computed from a correlation between λ_s and Z_a previously reported (Refs. 1 and 4). It is in the form of a regression equation between the two variables:

$$\lambda_s = 4.958\ (Z_a)^{0.579} \qquad (Z_a,\ \lambda_s \text{ in m})$$

$$\lambda_s = 8.178\ (Z_a)^{0.579} \qquad (Z_a,\ \lambda_s \text{ in ft})$$

In the field study it was found the pavement structure changed the soil wavelength. Since the shortest wavelengths produce the worst roughness, they are of interest in design. The shortest values found were equal to half the soil wavelength. The design wavelength is thus 0.5 λ_s.

SOIL-PAVEMENT INTERACTION EVALUATION

A mathematical model was derived for an elastic beam on a deformed foundation. It permits the calculation of reduced swelling of the soil because of the pavement and its stiffness. The solution to the model is shown in Figure 4-2 as a plot of $\beta\lambda$ versus $\Delta w/2a$. The β is a characteristic distance representative of the pavement-soil system. It is expressed as

$$\beta = \left(\frac{kb}{4EI}\right)^{1/4}$$

where

k = a modulus for swell reduction due to load

b = width of the pavement, taken as unity

E = modulus of the pavement material

I = moment of inertia of pavement cross section

The wavelength, λ, is the design wavelength representing a mound spacing defined in the problem initial condition. It is assumed equal to λ_d. The

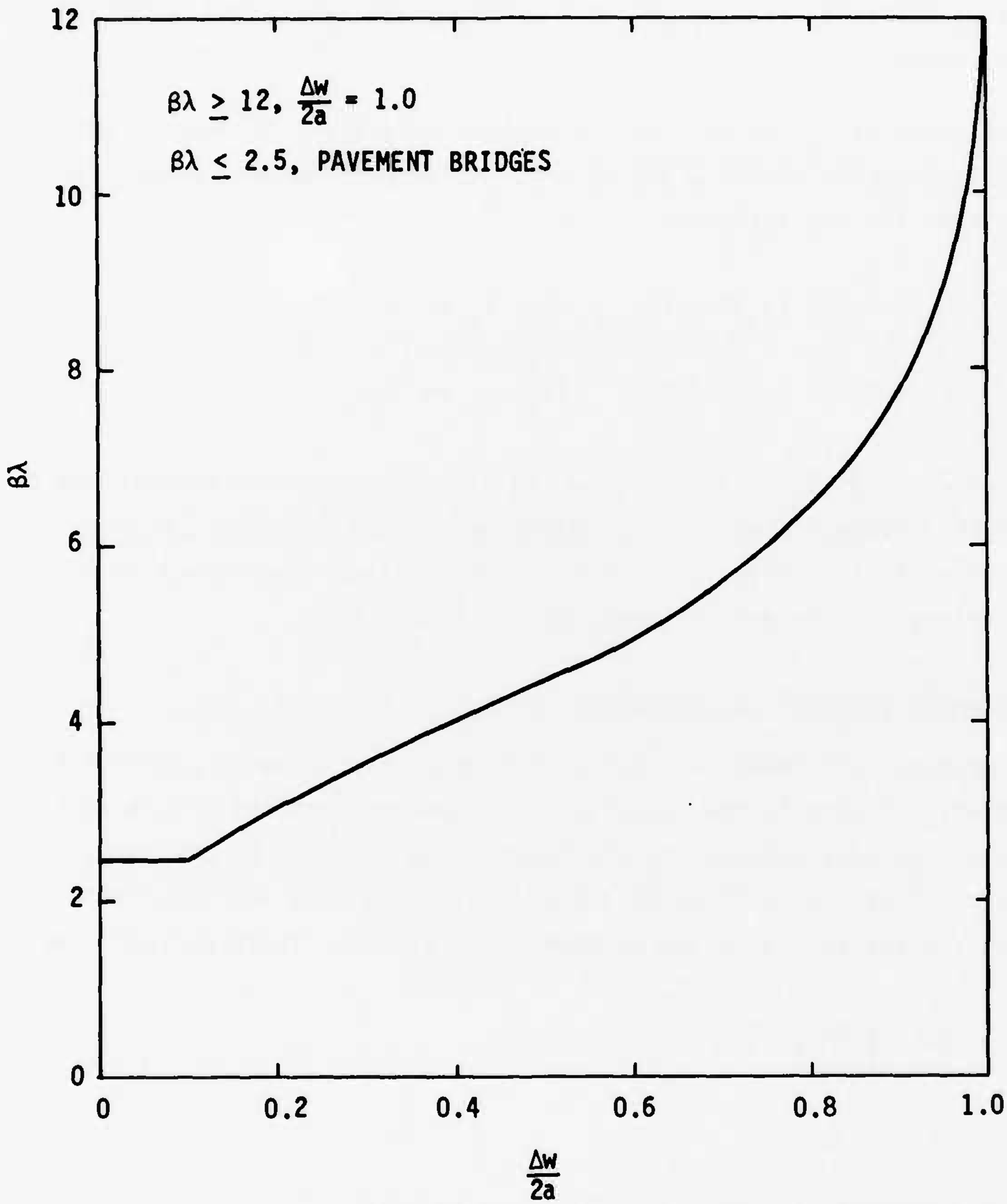


FIGURE 4-2. SOLUTION TO PAVEMENT MODEL EQUATION

differential elevation in the pavement surface is Δw, and 2a represents the soil surface differential elevation. Using the values previously determined, this may be computed as

$$\frac{\Delta w}{2_a} = \frac{\bar{A}_a}{\bar{A}_e}$$

The value of βλ is then picked from Figure 4-2. Using λ_d for λ in the model yields a value of β. For purposes of comparison an E of 3.4×10^6 kPa (5×10^5 lb/in^2) is used and all materials in the pavement are converted to an equivalent thickness. This procedure is illustrated by an example below.

EXAMPLE CALCULATIONS

In this example, data from the field experiment at the Gallup Airport are used for illustration. Results of soil characterization tests are as follows:

1. Soil characteristics

$$C_h = 0.137$$
$$CV(C_h) = 0.19$$
$$Z_a = 1.2 \text{ m (4 ft)}$$
$$k = 1.9 \times 10^5 \text{ kPa/m [700 (lb/in}^2\text{/in)]}$$

2. Surface characteristics

$\bar{A}$ (measured) = 0.0146 m (0.048 ft)

λ_s (measured) = 5.43 m (17.8 ft)

3. Calculated results (Z_a in meters)

$$\bar{A}_e = 0.6\ C_h \cdot Z_a \cdot CV(C_h) = 0.0187 \text{ m}$$
$$\lambda_s = 4.958\ (Z_a)^{0.579} = 5.5 \text{ m}$$
$$\lambda_d = 0.5\ \lambda_s = 2.755 \text{ m}$$
$$\bar{A}_a = 0.0023(\lambda_d)^{1.354} = 0.0091 \text{ m}$$

4. Calculated results (Z_a in feet)

$$\bar{A}_e = 0.6\ C_h \cdot Z_a \cdot CV(C_h) = 0.062 \text{ ft}$$
$$\lambda_s = 8.178\ (Z_a)^{0.579} = 18.2 \text{ ft}$$
$$\bar{\lambda}_d = 0.5\ \lambda_s = 9.1 \text{ ft}$$
$$\bar{A}_a = 0.00153\ (\lambda_d)^{1.354} = 0.0304 \text{ ft}$$

5. Soil-pavement interaction

$$\frac{\Delta w}{2a} = \frac{\bar{A}a}{\bar{A}_e} = 0.49 \text{ (using est. } \bar{A}_e\text{)} = 0.63 \text{ (using meas. } \bar{A}\text{)}$$

$\beta\lambda$ (from Figure 4-2) = 4.42 (est.) or 5.25 (meas.)

$$\beta = \frac{\beta\lambda}{\lambda_d} = \begin{array}{l} 0.040 \text{ in}^{-1}\text{(est.) or } 0.049 \text{ in}^{-1} \text{ (meas.)} \\ 0.016 \text{ cm}^{-1}\text{(est.) or } 0.0193 \text{ cm}^{-1} \text{ (meas.)} \end{array}$$

Pavement thickness may then be obtained as follows:

$$\beta = \left(\frac{kb}{4EI}\right)^{1/4} = \left(\frac{3k}{Eh^3}\right)^{1/4} \text{ using } E = 3.4 \times 10^6 \text{ kPa } (5 \times 10^5 \text{ psi})$$

$$h = \left(\frac{3k}{E\beta^4}\right)^{1/3}$$

h = 30.0 cm (11.8 in) (based on estimated $\bar{A}$)

h = 23.0 cm (9.0 in) (based on measured $\bar{A}$)

The pavement at the site consists of an equivalent depth of 39.9 cm (15.7 in) of asphalt concrete. Therefore, based on this procedure the existing pavement has sufficient bending resistance to perform satisfactorily on the soil.

The above procedure illustrates a prediction based on estimated soil amplitude [using C_h, Z_a, $CV(C_h)$] and calculated soil-pavement interaction. Also illustrated is an alternate method in which surface elevations are measured; a Fast Fourier Transform (FFT) is used to compute amplitudes of frequency components, and the measured soil amplitude, $\bar{A}$, is used in the soil-pavement interaction calculation.

V. SITE INVESTIGATIONS

INTRODUCTION

Site investigations are required to provide a means of obtaining data for use in design calculations. The required characteristics are suction compression index, its variation, the active zone depth, suction change, and the lateral restraint factor and modulus. The site investigation and subsequent analysis must either measure these or provide a basis for selecting a value by some other means. In order to proceed with the analysis of expansive soil profiles several measurements are made. They are suction and water content profiles, moisture characteristic of each significant soil type, suction compression index throughout the profile, and climatic data for computation of the Thornthwaite Moisture Index. In the following paragraphs these data are presented and discussed for each site involved in the present study. Detailed test procedures are in the appendix for reference.

For the validation of procedures, the criteria set up for site selection included: (1) pavements existing more than five years since construction, (2) evidence of expansive soil distress, and (3) not in the same geological formation as previous studies. A review of sites known to have expansive soil related distress was made and two sites were selected. Both are general aviation airports, one located in Mesquite, Texas the other in Murdo, South Dakota. A third site was included to some extent as a result of an ongoing engineering investigation at Love Field in Dallas, Texas. While the results are not as complete in regard to the proposed methods, a limited evaluation was made and believed to be valuable.

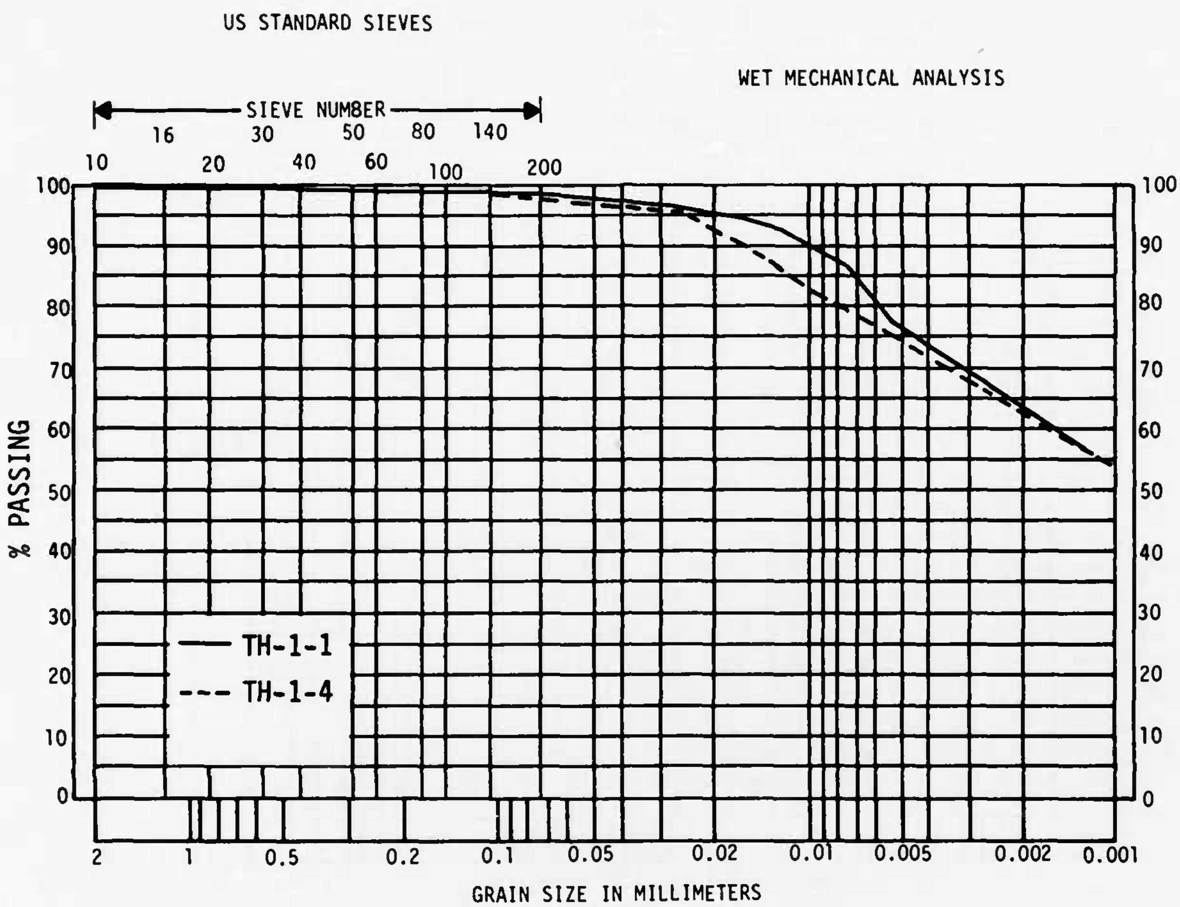
MURDO AIRPORT, SOUTH DAKOTA (MDO)

The Murdo Airport is located in south central South Dakota. The pavement and airport layout are discussed later. The underlying soil is residual Pierre Shale, typically weathered to a clay near the surface, with highly jointed shale below that and unweathered shale at greater depth. The site investigation involved three borings. Boring Number 1 was at about station 13+40 of the runway and 6.1 m (20 ft) west of the pavement edge in a fill area. Boring Number 2 was near station 1+88, and 6.1 m (20 ft) west of the pavement edge in a cut area. Boring Number 3 was at station -0+72, 6.1 m (20 ft) west of the pavement edge and also in a cut.

The Murdo Airport consists of a 15.2 x 1036 m (50 x 3400 ft) runway, a connecting taxiway and parking apron. It is used primarily for general aviation supporting crop dusters and other small aircraft. The runway was constructed in 1974 consisting of 0.22 m (8.5 in) of asphalt concrete pavement (FAA Specification P201), 0.15 m (6 in) of lime stabilized clay and 0.15 m (6 in) of compacted subgrade. Construction records indicate the weather prior to construction were extremely dry and problems were involved with proper construction of the lime-stabilized soil layer. The pavement became severely rough and was closed to traffic in 1979. A rehabilitation project was undertaken in 1980. Work consisted of milling the rough surface, especially the outer one-third of the runway. This material was recycled and used as a leveling course followed by a 0.04 m (1.5 in) overlay with P201 material. In addition four areas of particularly severe distress were excavated to 1.8 m (6 ft) and recompacted. As of September 1984, during the site investigation for this study, the runway was rough. There are large transverse cracks, believed to be due to thermal cycling, as well as numerous areas distorted and heaved due to clay movements.

Results of classification testing are presented in Figures 5-1 and 5-2 for the fill and cut areas respectively. The fill was 2.3 m (7.5 ft) thick at Boring Number 1. The fill appears to be material similar to the natural shale below it. However, the structure has been radically altered by the process of compaction. The shale was easily identified by inspection of samples during sampling. The shale contains numerous joints, cracks and seams with a variety of iron stains, gypsum, and other materials infilling the seams. The natural shale appeared very dry at this location, much drier than the fill material. Data in Figures 5-3 and 5-4 support this observation. Suction and water content profiles are shown in Figure 5-3 for Boring Number 1. The data are field samples as well as a few laboratory samples. The field samples were used to draw the curve. The top of the fill has dried to a depth of 1.2 m (4 ft) due to interaction with the climate. The moisture and suction at that level are believed to be approximately the same as constructed conditions. Below 1.2 m (4 ft) the fill material has dried somewhat. The slope of the suction versus depth curve represents an energy gradient that drives water from the lower

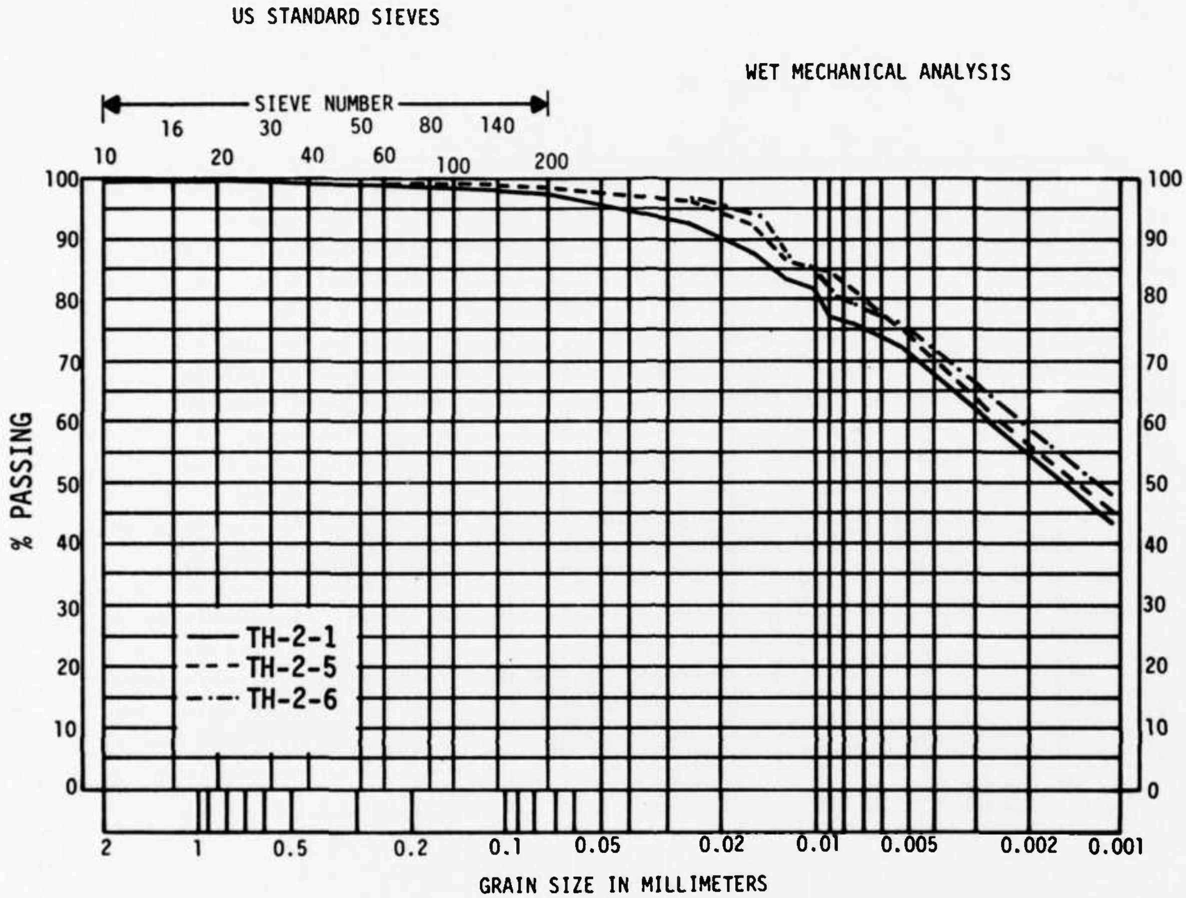
SITE: MURDO AIRPORT (FILL)



SAMPLE NO.	DEPTH	NATURAL		G_s	ATTERBERG LIM			CLAY <.002	BAR L.S.	C_h	UNIFIED CLASSF.
		ω	γ_d		LL	PI	SL				
—	FT	%	lb/ft³	—	%	%	%	%	%	—	—
TH-1-1	0.5-1.5	27.5	87.0	2.76	76	46	---	64		0.100	CH
TH-1-4	7-8	24.7	94.1	2.73	74	42	---	63		0.075	CH

FIGURE 5-1. SOIL CLASSIFICATION DATA

SITE : MURDO AIRPORT (CUT)



SAMPLE NO.	DEPTH	NATURAL		G_s	ATTERBERG LIM			CLAY <.002	BAR L.S.	C_h	UNIFIED CLASSF.
		ω	γ_d		LL	PI	SL				
—	FT	%	lb/ft³	—	%	%	%	%	%	—	—
TH-2-1	0-1	23.8	89.9	2.77	80	51	---	55		0.128	CH
TH-2-5	10.5-11	28.1	92.9	2.72	77	46	---	57		0.085	CH
TH-2-6	14.5-15.0	27.5	93.2	2.78	79	47	---	59		0.092	CH

FIGURE 5-2. SOIL CLASSIFICATION DATA

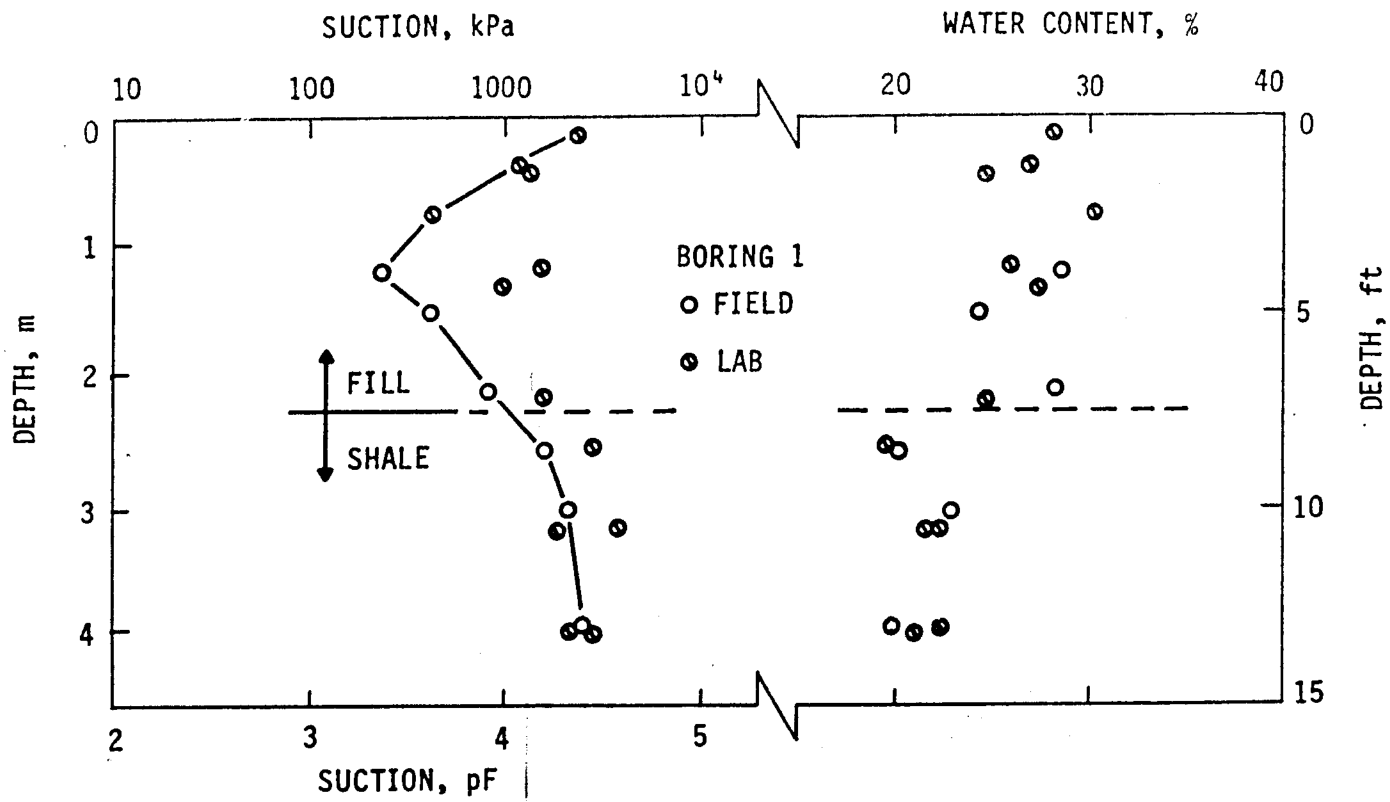


FIGURE 5-3. MURDO BORING 1 (FILL)

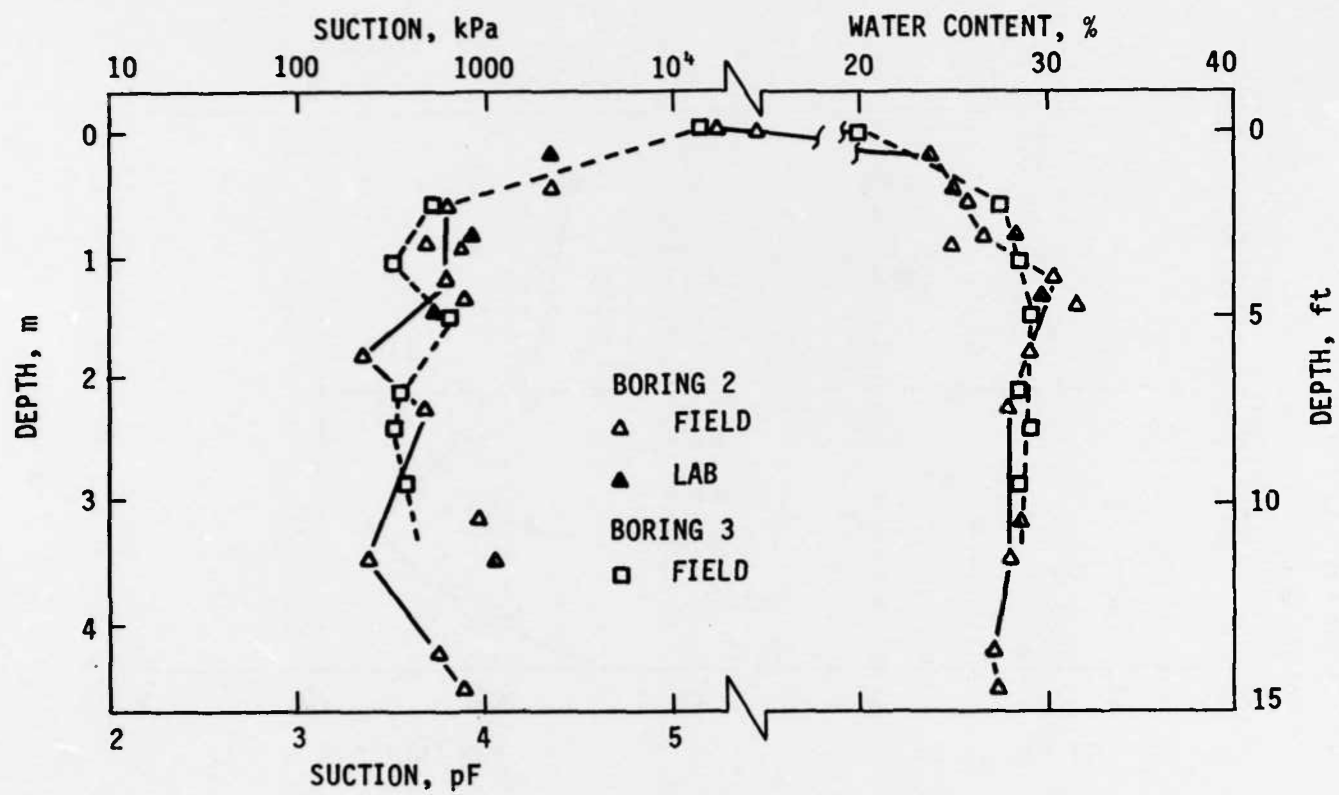


FIGURE 5-4. MURDO BORINGS 2 AND 3 (CUT)

suction (fill) to the higher suction (natural shale). Examination of the shale [below 2.3 m (7.5 ft)] shows a dry profile that has wetted slightly at the fill-shale interface. The profile continues to get drier with depth. Ordinarily, in the absence of water tables, the suction at depth reaches an equilibrium value. When the suction is still varying it must be concluded the profile is not in moisture equilibrium. This is an important factor and will be discussed further.

Suction and water content profiles for Boring Numbers 2 and 3 are presented in Figure 5-4. These are both in cut areas and therefore are in natural shale from the surface. By examining suction profile data, drying effects are evident to about 0.6 m (2 ft), indicating that moisture movement into and out of the shale and fill are apparently quite different, as one might anticipate. Study of the data with depth reveals an increasing suction and decreasing water content. As before, this is a nonequilibrium profile. The evaluation of equilibrium conditions may be taken a step further by examining the climate using the Thornthwaite Moisture Index (TI) (Ref. 1). Figure 5-5 is a plot of rainfall and potential evapotranspiration for a typical year at the Murdo site. The average TI is -0.46 m (-18.0 in) for the 10 years of records available (1975-1984). Figure 5-6 shows relationships, developed by British researchers (Ref. 16) and validated in the United States (Ref. 1), between equilibrium suction and the long term TI of the climate. These data indicate the suction at depth should be about 980 kPa (4 pF). A further step in this evaluation is replotting the suction-water content data as shown in Figure 5-7, to determine the number of soil types in the profile. These data indicate that a single moisture characteristic can be used to represent the soils (see Appendix B). The relationship can be represented by the following equations:

$$\log h \text{ (kPa)} = 5.24 - 0.092(w)$$

$$h \text{ (pF)} = 6.25 - 0.092(w)$$

$$w = \text{water content, percent}$$

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16. Russam, K., and Coleman, J. D., "The Effect of Climatic Factors on Subgrade Moisture Condition," **Geotechnique**, Vol. III, No. 1, 1961, pp. 22-28.

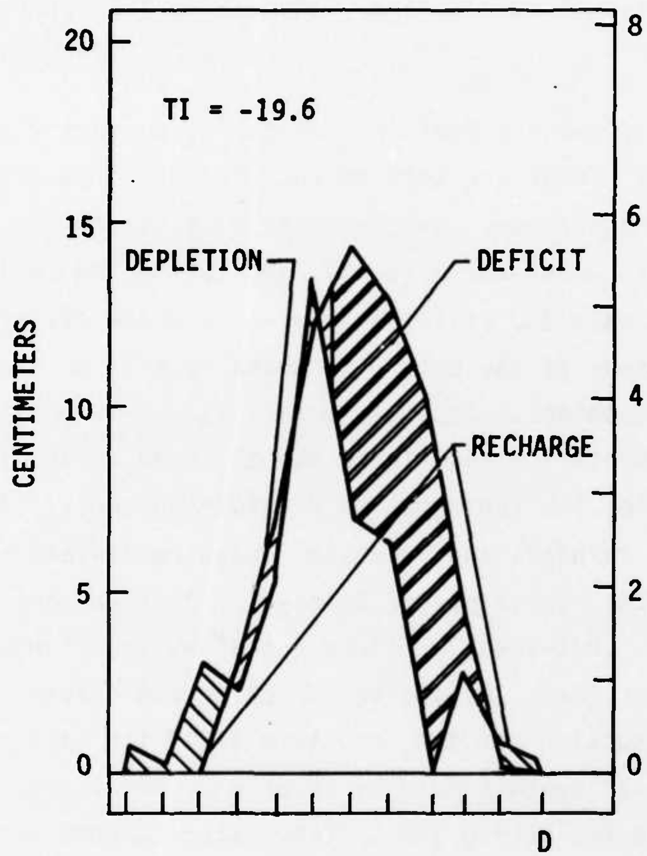

FIGURE 5-5. THORNTHWAITE DATA FOR MURDO (1979)

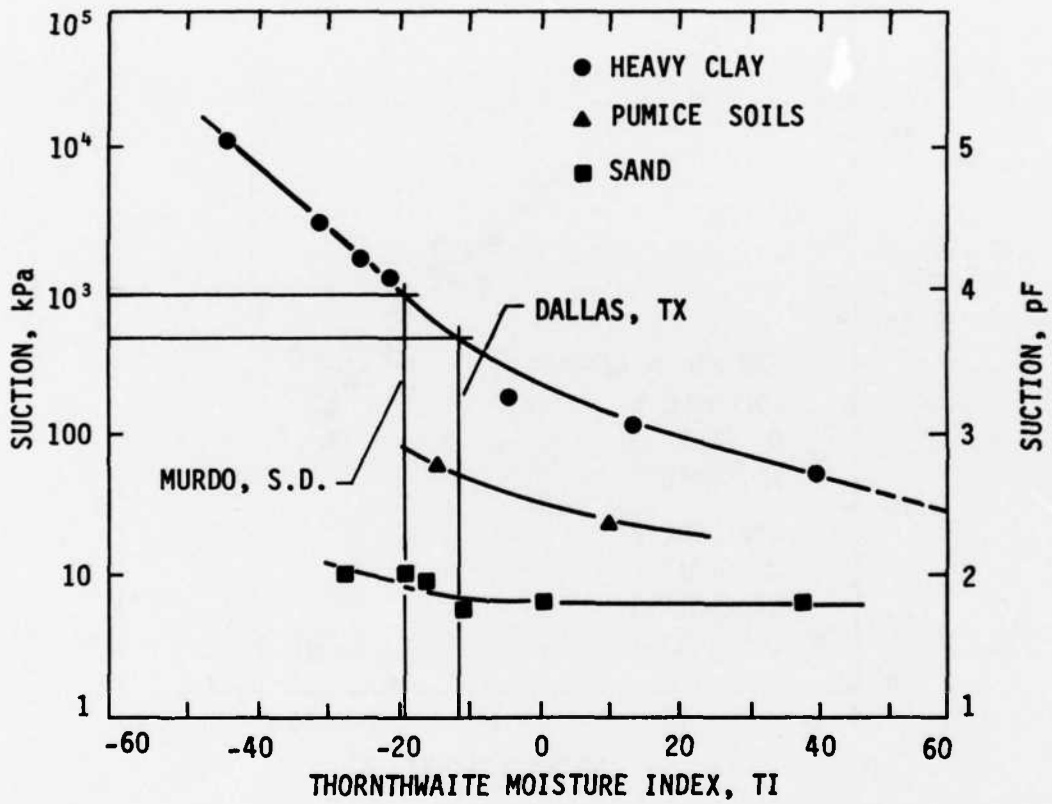

FIGURE 5-6. SUBGRADE SUCTION AS A FUNCTION OF MOISTURE INDEX (REF. 16)

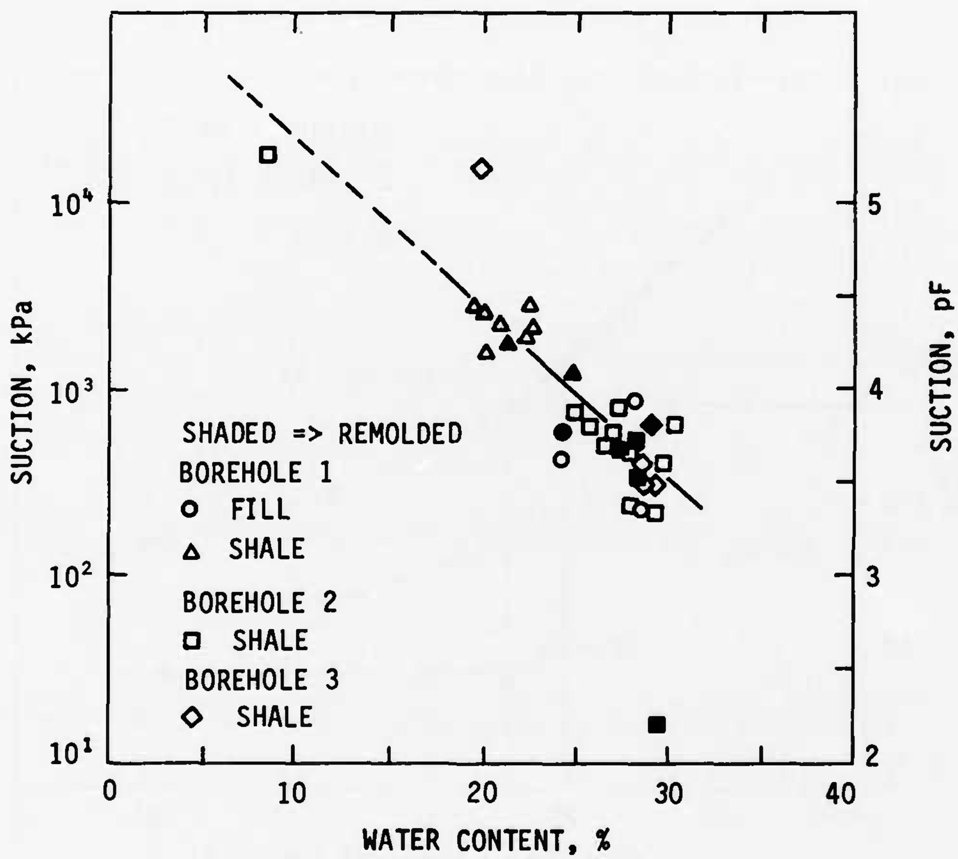

FIGURE 5-7. MOISTURE CHARACTERISTIC, MURDO AIRPORT

The suction of 980 kPa (4 pF) is equivalent to a water content of 27.4 percent.

Comparison of these values (4 pF, 27.4 percent) with data in Figure 5-3 clearly shows the soil at depth is drier. In the case of Figure 5-4 the soil at depth is wetter than these values. Normally borings would be advanced to a depth at which the soil moisture condition is at the equilibrium condition, in order to arrive at an estimate of the active zone depth. In this investigation equipment limitations prevented advancing borings to such a depth. Therefore an alternate procedure was used to obtain an estimate of the active zone depth. This consisted of extrapolating a line through the water content profile to the depth at which it reached 27.4 percent. Similarly, the suction profile was extrapolated to a depth at which it equaled 980 kPa (4.0 pF). Both methods yield a depth of 7 m (23 ft). The presence of the fill at boring 1 (Figure 5-3) makes such a procedure very difficult. It is clear the material was extremely dry at the time the fill was placed and remained that way. Due to the deep wetting at borings 2 and 3 (Figure 5-4) and the dry condition at boring 1, under the fill (Figure 5-3) it is concluded that vertical infiltration of surface water is the principal source of water in the soil profile at this site.

The final soil characteristics of interest are the suction compression index and its variation. Using the clod test (Appendix D) these were measured and are plotted in Figure 5-8. The data exhibit some variation from point to point as is typical in most profiles. The mean values of C_h are somewhat different for the soils involved (fill, shale below fill, shale in cut). The mean value of C_h is 0.107 with a coefficient of variation of 29.2 percent.

Surface characteristics of the sites were determined on the uncovered soil, pavement edge, pavement centerline and halfway between the edge and centerline. The lines were surveyed using conventional rod and level surveying techniques. Two sections were measured yielding a larger data base than previously used. The FFT and autocorrelation computations that were made yielded the results in Table 5-1., representing measured values of $\overline{A}$, the soil surface weighted amplitude.

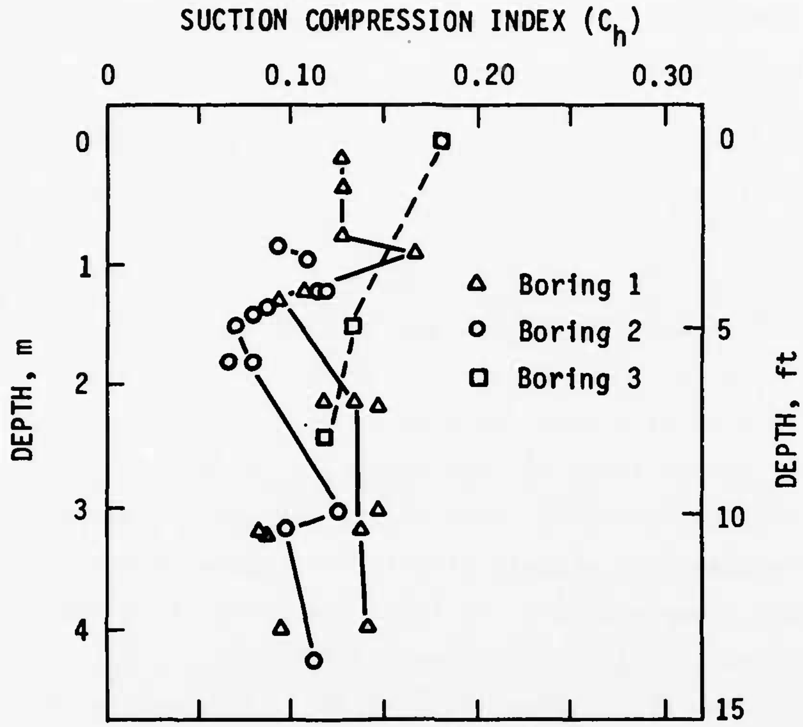

FIGURE 5-8. MURDO SUCTION COMPRESSION INDEX

TABLE 5-1. MEASURED SURFACE CHARACTERISTICS--MURDO

Site	Centerline, m (ft)		Halfway, m (ft)		Edge, m (ft)		Soil m (ft)	
	λ_o	$\bar{A}$	λ_o	$\bar{A}$	λ_o	$\bar{A}$	λ_s	$\bar{A}$
MDO(1)	6.7 (22)	0.006 (0.019)	4.6 (15)	0.003 (0.009)	4.9 (16)	0.004 (0.014)	4.1 (13.5)	0.020 (0.067)
MDO(2)	6.1 (20)	0.025 (0.083)	5.5 (18)	0.16 (0.054)	6.6 (21.5)	0.009 (0.030)	7.6 (25)	0.026 (0.086)
MDO(3)	7.0 (23)	0.021 (0.070)	6.1 (20)	0.020 (0.065)	4.6 (15)	0.012 (0.042)	4.9 (16)	0.028 (0.093)

λ_o = pavement surface wavelength calculated from measured elevation data

λ_s = soil surface wavelength calculated from measured elevation data

$\bar{A}$ = weighted amplitudes calculated from measured elevation data

HUDSON AIRPORT, MESQUITE, TEXAS (MSQ)

The Hudson Airport is located in Mesquite, Texas just east of Dallas in the north central part of the state. The facility was originally established by private owners, and then taken over by the city for operation and maintenance. The soil of the site consists of heavy clays derived by weathering from the underlying Ozan Formation, also known as Lower Taylor Marl. It is described as "clay, marly, calcareous content decreases upward, montmorillontic, some glauconite, phosphate pellets and hematite and pyrite nodules. It weathers to a light gray to grayish orange and white. Thickness is about 68.6 m (225 ft)" (Ref. 17).

The Mesquite Airport consists of a 15.2- x 1150-m (50- x 3760-ft) runway with a parallel taxiway, three connecting taxiways, and several parking aprons. The area is flat and no cut or fill is evident on the site. The existing pavement section along the runway consists of a bituminous material surface 0.089 to 0.102 m (3.5 to 4.0 in) in thickness. However, in one area at the south end it was only 0.038 m (1.5 in) thick. Apparently this was the original thickness with overlays placed at a later time. Taxiways are surfaced with 0.038 m (1.5 in) of bituminous material. All pavements are underlain by a crushed limestone base material ranging in thickness from 0.15 m (6 in) on the runway to 0.102 (4 in) on the taxiway. The natural dark clay underlies this base course. An investigation was conducted at the site in June 1984 for design of a rehabilitation/upgrade of the facility. A total of 15 borings was made in that investigation. Samples were not available to NMERI.

The NMERI site investigation consisted of three borings, Numbered 16, 17, and 18 to continue the numbers used by Southwestern laboratories in their investigation in June 1984. Boring Number 16 was 6.1 m (20 ft) west of the taxiway pavement edge and 418 m (1370 ft) south of the connecting taxiway. Boring Number 17 was 6.1 m (20 ft) west of the runway pavement edge and 287 m (940 ft) south of the connecting taxiway. Boring Number 18 was 6.1 m (20 ft) west of the runway pavement edge and 93 m (300 ft) north of the connecting taxiway. Results of classification tests are shown in Figures 5-9 through

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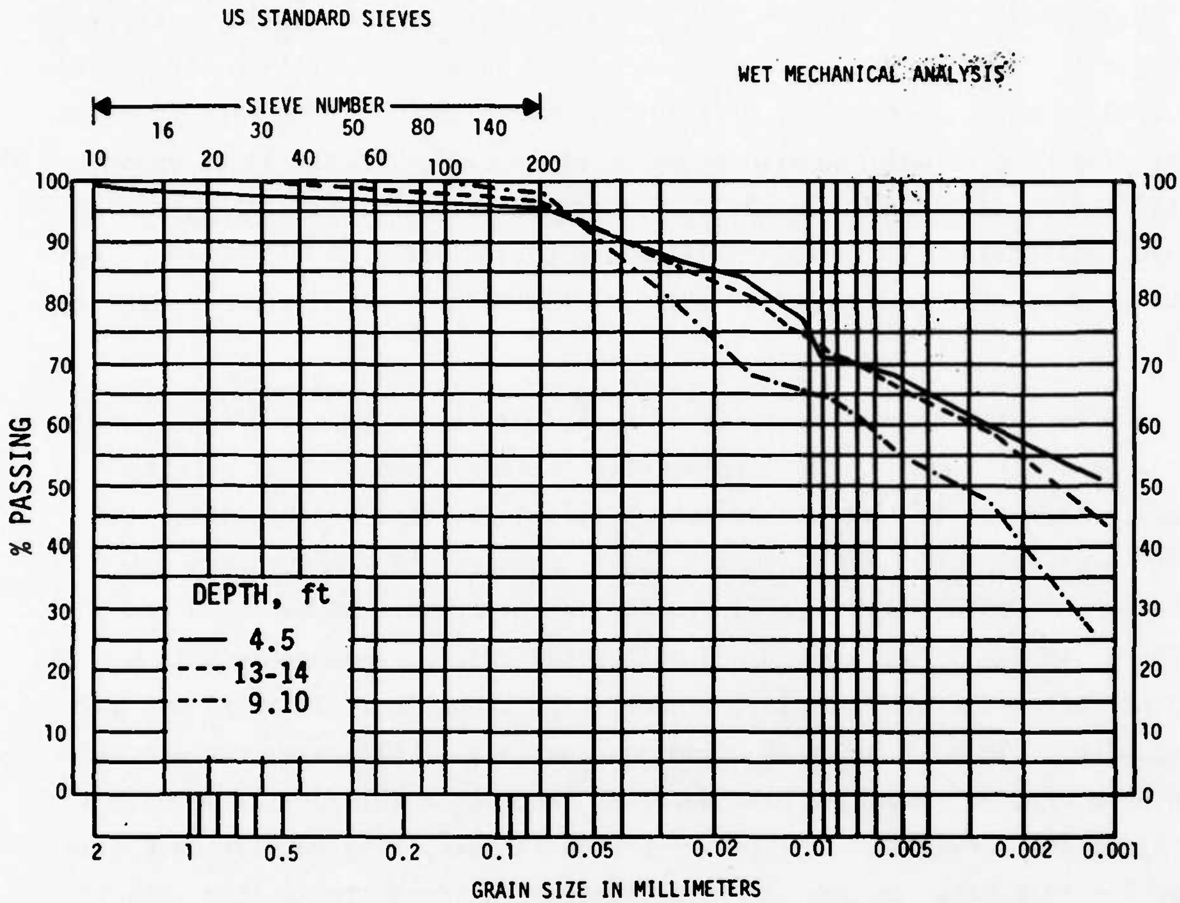
17. Barnes, V. E., **Geologic Atlas of Texas, Waco Sheet**, University of Texas at Austin, Bureau of Economic Geology, 1970.

SITE: MESQUITE, TEXAS



SAMPLE NO.	DEPTH	NATURAL		G_s	ATTERBERG LIM			CLAY <002	BAR L.S.	C_h	UNIFIED CLASSF.
		ω	γ_d		LL	PI	SL				
—	FT	%	lb/ft³	—	%	%	%	%	%	—	—
16	4.5	29.7	92.1	2.70	72.7	45.4	---	57	[a] 18	0.142	CH
16	9-10	25.5	98.3	2.80	64.2	37.1	---	40	---	0.115	CH
16	13-14	20.7	107.0	2.73	64.2	33.9	---	59	---	0.070	CH

[a] From SWL

FIGURE 5-9. SOIL CLASSIFICATION DATA

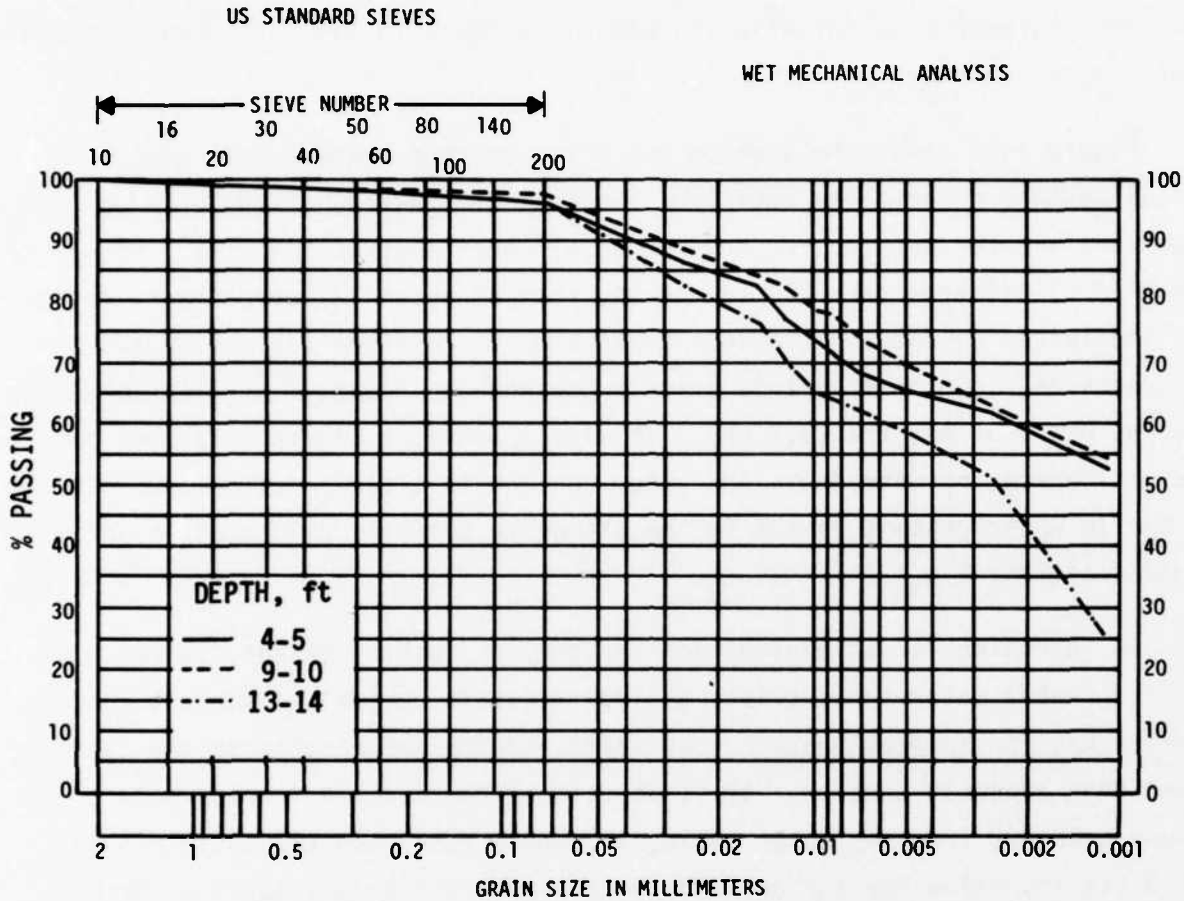
5-12. These data indicate typical values for clays in this area, ranging from CH to CL in the Unified Classification System. It is also apparent that the surface contains a higher clay percentage which decreases with depth. This reduction in expansion potential is clearly evident in all data commonly associated with expansion potential (LL, PI, C, C_h).

Figure 5-13 shows the suction and water content measurements plotted versus depth for the three borings. The initial reaction to these results is that they are too wet for the environmental conditions at the site. Figure 5-14 illustrates data plotted in the form of potential evapotranspiration and precipitation showing the Thornthwaite Moisture Index Categories and the TI, which is -11.8. Using this value together with Figure 5-6 yields a suction at depth of 495 kPa (3.7 pF). It also appears in Figure 5-13 that suction is decreasing with depth and water content is clearly decreasing. The change in water content is due to the change in soils as evidenced in the classification data.

In searching for an explanation for the low suction values, it was discovered that a water table exists at 19.8 m (65 ft) below the surface and has been stable for several years. It is believed the water table is the cause of lower than expected suction. If it is assumed the suction increases on a one-to-one gradient from the water table, it should reach 148 kPa (3.18 pF) at 4.6 m (15 ft) below the surface and 194 kPa (3.3 pF) at the surface (dashed line in Figure 5-13). In view of the normal variations in climate and the uncertainity of the water table depth, it appears reasonable to conclude that the water table is controlling the moisture condition in the soil. It is expected that the result would be a reduced active zone depth. The estimate based on these data is that the active zone depth is 2.0 m (6.5 ft), which is about equal to the 2.1 m (7 ft) observed at DFW Airport, just west of Dallas, with a similar climate.

Figure 5-15 shows the moisture characteristic data for the site. Suction is plotted versus water content yielding three different curves based on the soil description obtained during drilling and sampling activities. This is the reason small variations in suction with depth are accompanied by rather large variations in water content. All moisture characteristic curves are forced through on intercept of 1.74×10^6 kPa (6.25 pF) yielding slopes tabulated as follows:

SITE : MESQUITE, TEXAS

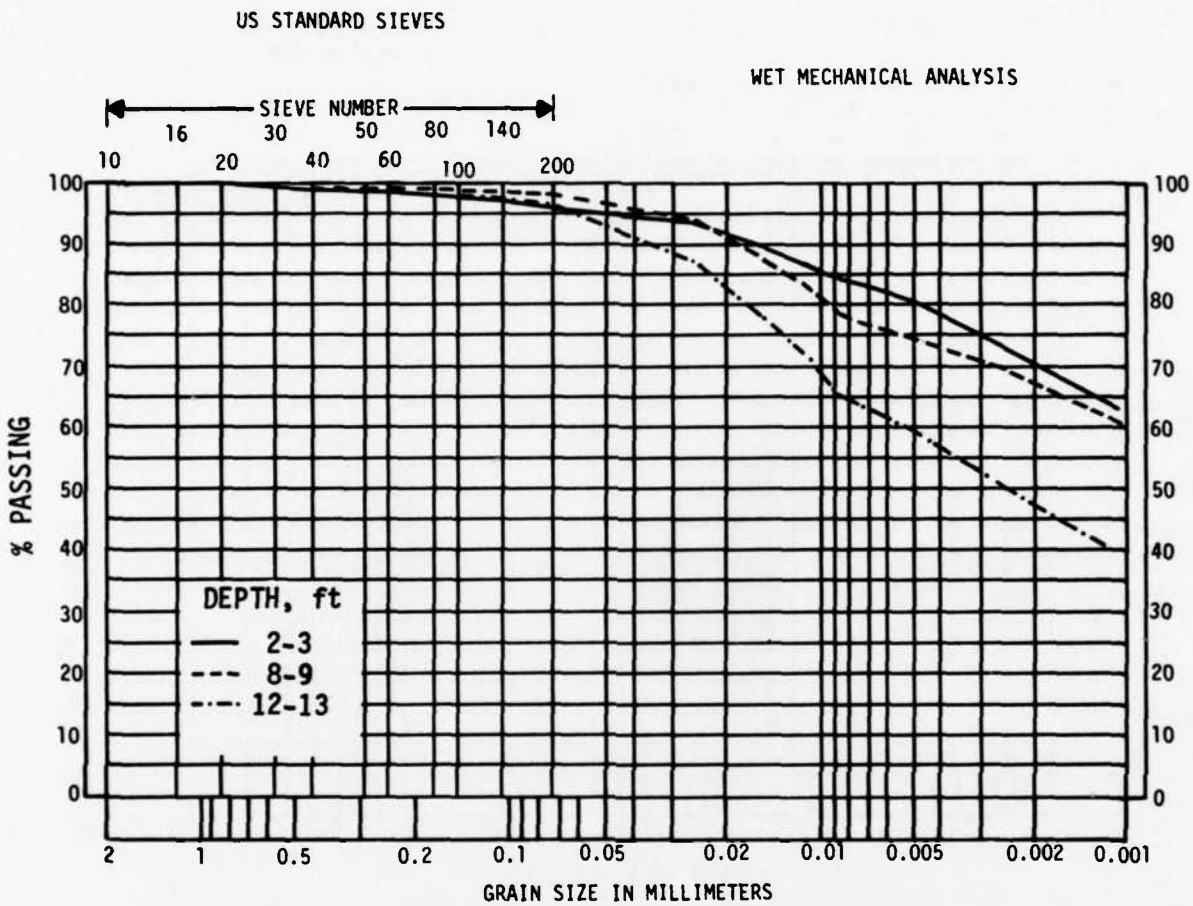


SAMPLE NO.	DEPTH	NATURAL ω	NATURAL γ_d	G_s	ATTERBERG LIM LL	ATTERBERG LIM PI	ATTERBERG LIM SL	CLAY <.002	BAR L.S.	C_h	UNIFIED CLASSF.
—	FT	%	lb/ft³	—	%	%	%	%	%	—	—
17	4-5	29.9	90.8	2.74	82.0	53.1	---	58	---	0.150	CH
17	9-10	[a]30.0	89.2	2.68	76.7	49.0	---	61	---	0.130	CH
17	13-14	[a]22.6	[a]103.7	2.72	59.7	42.7	---	43	---	0.090	CH

[a]Average

FIGURE 5-10. SOIL CLASSIFICATION DATA

SITE : MESQUITE, TEXAS

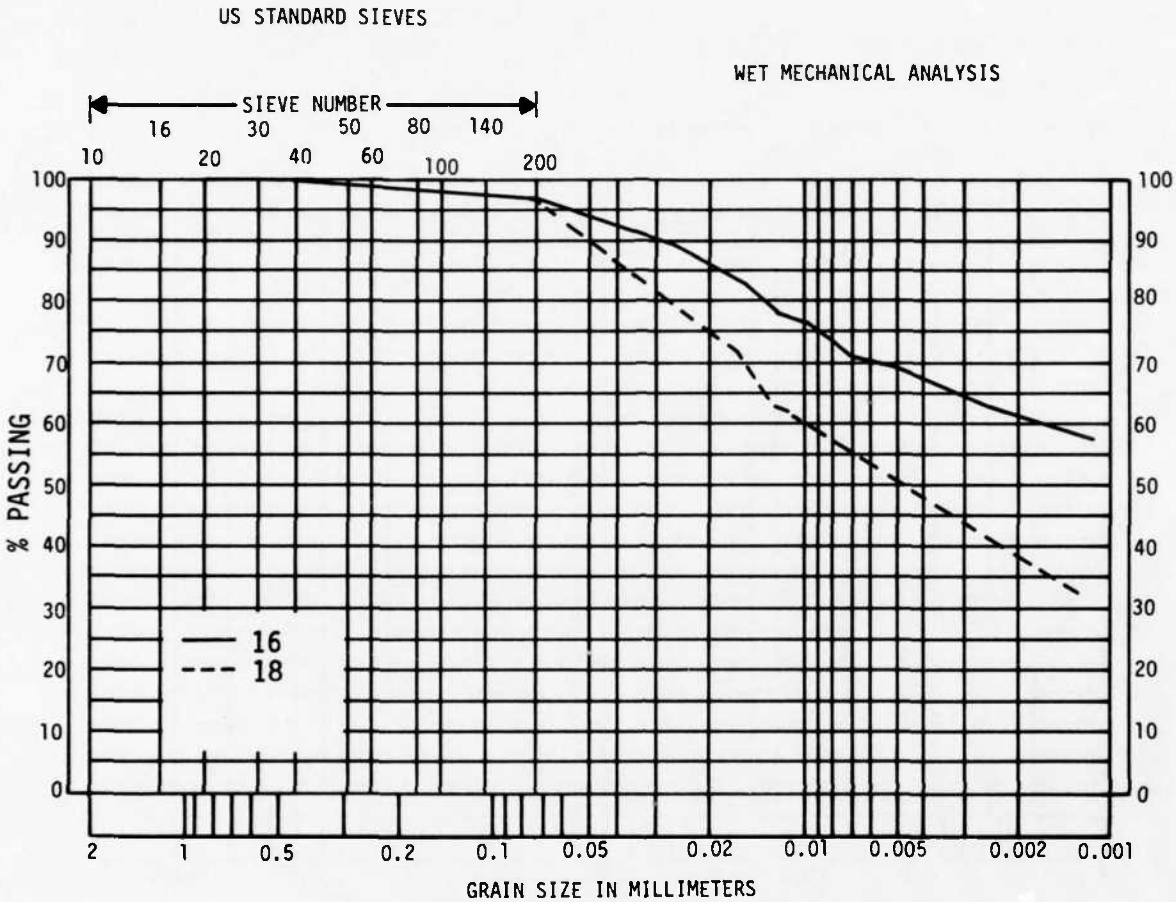


SAMPLE NO.	DEPTH	NATURAL		G_s	ATTERBERG LIM			CLAY <.002	BAR L.S.	C_h	UNIFIED CLASSF.
		ω	γ_d		LL	PI	SL				
—	FT	%	lb/ft³	—	%	%	%	%	%	—	—
18	2-3	33.4	80.2	2.64	79.3	45.1	---	70	[a]19	0.155	CH
18	8-9	31.8	87.9	2.64	83.4	53.2	---	68	[a]19	0.130	CH
18	12-13	20.0	104.4	2.65	46.8	27.7	---	47	---	0.050	CL

[a]From SWL

FIGURE 5-11. SOIL CLASSIFICATION DATA

SITE : MESQUITE, TEXAS



SAMPLE NO.	DEPTH	NATURAL		G_s	ATTERBERG LIM			CLAY <.002	BAR L.S.	C_h	UNIFIED CLASSF.
		ω	γ_d		LL	PI	SL				
—	FT	%	lb/ft³	—	%	%	%	%	%	—	—
16	2-3	20.0	86.0	2.72	66	40	---	62			CH
18	14-15	20.5	104.0	2.73	45	30	---	38		0.082	CL

FIGURE 5-12. SOIL CLASSIFICATION DATA

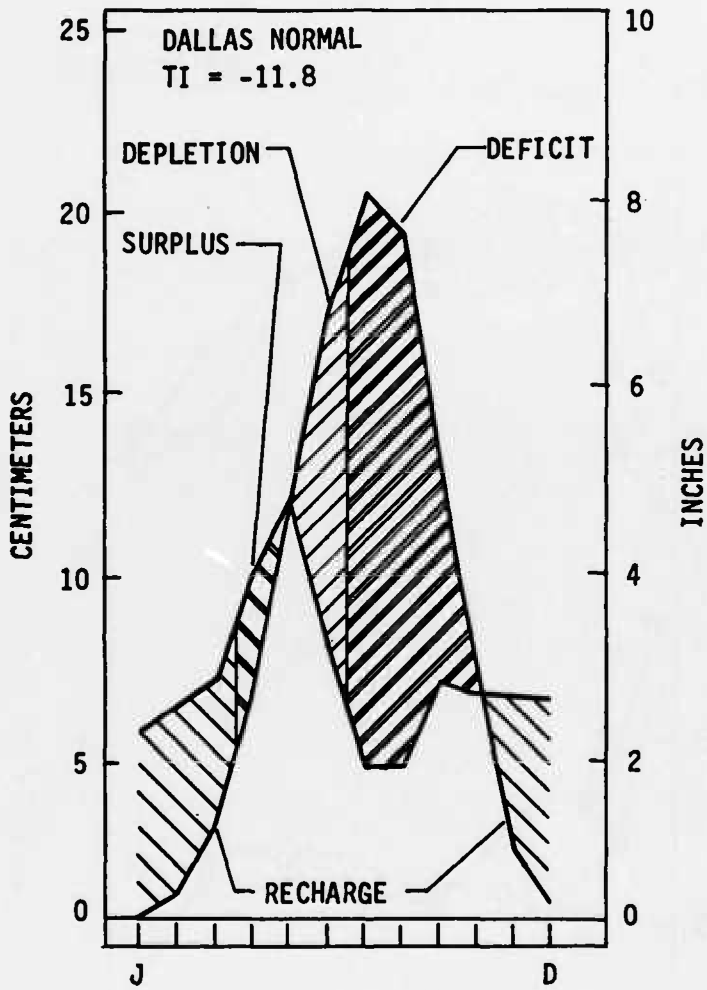


FIGURE 5-14. THORNTHWAITE DATA FOR DALLAS

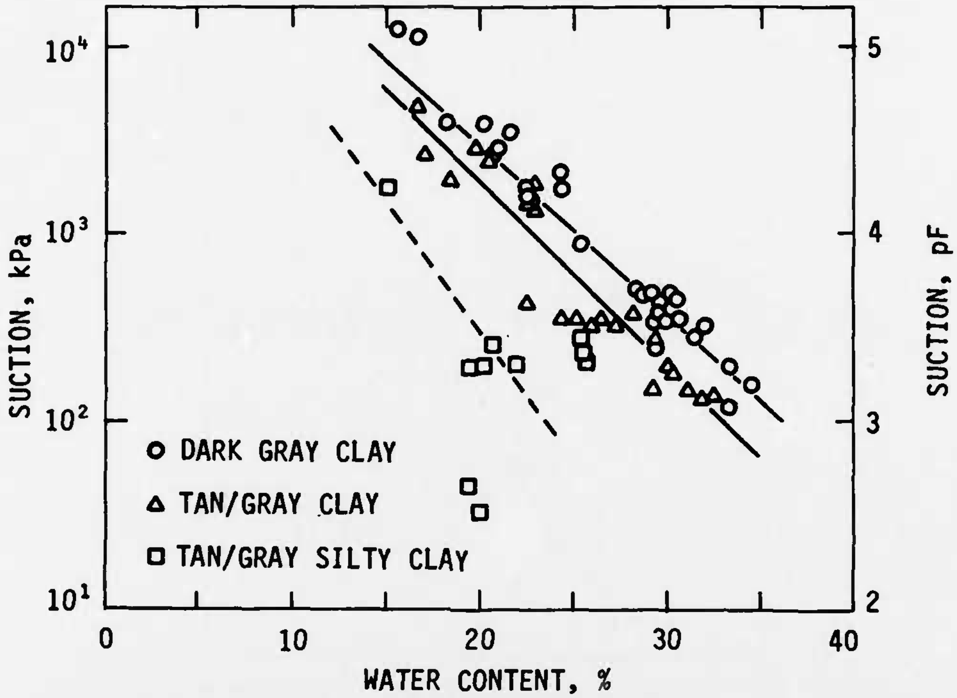


FIGURE 5-15. MOISTURE CHARACTERISTIC, MESQUITE AIRPORT

Soil description	Slope	B. H. 16, m (ft)	Depth interval B. H. 17, m (ft)	B. H. 18, m (ft)
Dark gray clay	-0.089	0-1.2 (0-4)	0-1.5 (0-5)	0-1.5 (0-5)
Tan/gray clay	-0.099	1.2-1.8 (4-6)	1.5-3.4 (5-11)	1.5-3.4 (5-11)
Tan/gray silty clay	-0.138	1.8+ (6+)	3.4+ (11+)	3.4 + (11+)

Also note the variation in depth intervals for the various soils found in the site exploration program. The suction compression index was also measured for the sampled soils. The data are plotted in Figure 5-16 versus depth. The data are strikingly uniform in comparison to other sites studied. The data were examined by soil type as shown below.

Soil description	No. tests	C_h	CV (C_h)
Dark gray clay	11	0.153	9.7
Tan/gray clay	8	0.131	5.3
Tan/gray silty clay	8	0.085	34.4

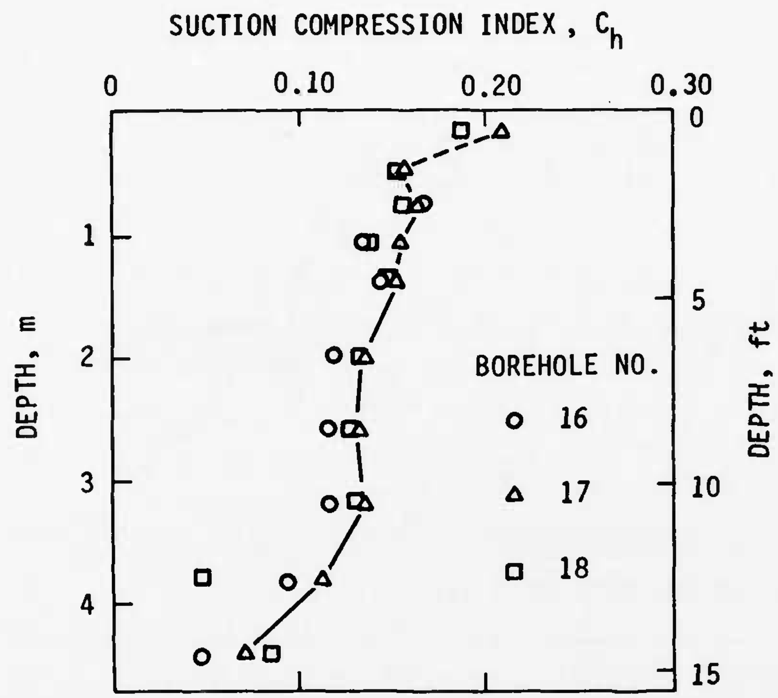


FIGURE 5-16. . SUCTION COMPRESSION INDEX, MESQUITE AIRPORT

The data shown are the number of tests (n), suction compression index (C_h) and its variation CV (C_h). The silty clay variation is quite high in relation to other values. The high coefficient of variation is more typical of expansive clays. As stated earlier, the uniformity of the upper soil layers is unusual.

Surface characteristics were determined at the Mesquite Airport using methods described previously. A total of ten lines was surveyed. Three (soil, edge, centerline) near boring 16 on the taxiway, four (soil, edge, centerline, halfway) near boring 17 on the runway and three (soil, edge, centerline) near boring 18 on the runway. Results are in Table 5-2 below.

TABLE 5-2. MEASURED SURFACE CHARACTERISTICS--MESQUITE

Site	Centerline, m (ft)		Halfway, m (ft)		Edge, m (ft)		Soil m (ft)	
	λ_o	$\bar{A}$	λ_o	$\bar{A}$	λ_o	$\bar{A}$	λ_s	$\bar{A}$
MSQ(16)	6.7 (22.0)	0.020 (0.066)	--- ---	--- ---	4.7 (15.5)	0.019 (0.062)	4.8 (15.6)	0.21 (0.068)
MSQ(17)	3.4 (17.6)	0.010 (0.032)	5.3 (17.3)	0.006 (0.019)	4.0 (13.0)	0.011 (0.036)	4.5 (14.9)	0.029 (0.096)
MSQ(18)	6.1 (19.9)	0.012 (0.038)	--- ---	--- ---	3.4 (11.2)	0.010 (0.034)	5.0 (16.3)	0.039 (0.127)

LOVE FIELD, DALLAS, TEXAS (DAL)

Love Field is located in the northwest part of Dallas, Texas. An airfield has existed on the site since the 1920's. The area involved in this study was the southeast end of Runway 31L constructed in 1961. The area of concern was from Station 72 + 81 to 121 + 06. A partial evaluation of the site was made possible through an agreement between the engineering firm evaluating the runway for the city of Dallas in the fall of 1984. A total of twenty borings was made and materials were characterized through tests conducted at a laboratory in the Dallas area using NMERI procedures (Ref. 1). The NMERI effort consisted of evaluating the moisture condition of the subgrade and determining its stability and potential for future movements.

Classification data are summarized in Table 5-3 from the tests conducted on soil samples from the borings. The soils beneath the pavement consisted of a maximum of 1.2 m (4 ft) of fill consisting of materials ranging from clay to fine sand. The next layer was a dark gray clay, separated at about the 2.1 m (7 ft) level on the basis of tests by the Dallas laboratory. A fourth category of soil was tan sandy clay below the dark gray clay. Figure 5-17 is the boring log for Boring B-14, a representative profile. The fill material was not characterized for the NMERI work.

TABLE 5-3. SOIL PROPERTIES AT LOVE FIELD

Boring	Soil description	Depth, m (ft)	w, %	γ_d mg/m^3 (pcf)	LL	PI
1	Cement stab. base	0.9 (3)	15	1.95 (122)	---	---
1	Brown, gray clay (fill)	1.5 (5)	26	1.57 (98)	39	20
1	Brown, gray clay	2.7 (9)	24	1.73 (108)	59	40
2	Light brown clay (fill)	1.2 (4)	16	1.99 (124)	25	13
2	Dark brown silty clay	2.4 (8)	22	1.71 (107)	59	40

As a result of the procedure followed, it was not possible to develop profiles of suction and water content for this site. The first step taken was to plot suction test results versus water content for the three materials

Project No	Boring No	Project
4-0665-01	B-14	Pavement Subgrade Evaluation, Runway 31L Dallas Love Field Airport

Location	Completion Depth	Completion Date	Water Observations
See Plan of Boring	25'	11-3-84	Seepage @ 18' during drilling.

Surface Elevation:

Type: Undisturbed

Depth, Ft.	Stratum Description	Blows/Ft. Pen Reading, T.S.F.	Passing No. 200 Sieve, %	Liquid Limit, %	Plastic Limit, %	Plasticity Index	Linear Shrinkage, %	Moisture Content, %	Unit Dry Wt. Lbs/Cu. Ft.	Unconfined Compression Lbs/Sq. Ft.
	Concrete (13.1")									5075
									149	
	Cement stabilized base (6.1")	4.5+						19	113	763
5	Light brown & reddish-tan sandy clay, hard, lime treated, Fill	4.0						23		
		2.5		61	17	44	23	23	103	
	Light gray to gray silty clay, very stiff, w/calcareous nodules and iron stains	2.7						23		
10		2.2						24		
15		3.0						20		
20	Light gray and reddish-tan silty, sandy, clay, soft to firm, w/ some iron stains	1.0						24		
25		10								
30										
35										
40										
45										

LOG OF BORING NO. B-14

Rone Engineers

FIGURE 5-17. BORING LOG AT LOVE FIELD

identified. The data are shown in Figure 5-18 for pressure plate tests and clod tests. The clod test results on the tan sandy clay were obviously not like the relationship developed from the other data points. This result indicates a clay similar to that in the dark gray clay (from the suction-water content relationship). However, it also contains sandy material. Typically in the clod test, these type materials fall apart, as they aparently did in this case. In the pressure membrane test the sample is restrained by a sample holder without falling apart, thus enabling a complete test. It is concluded from the data in Figure 5-18 that a single relationship may be used for these materials. It is represented by

$$\log h \text{ (kPa)} = 5.24 - 0.116 \text{ (w)}$$
$$h \text{ (pF)} = 6.25 - 0.116 \text{ (w)}$$

The suction compression index, C_h was determined from the clod test results shown in Table 5-4.

The difference between mean values are too small to regard them as significant in view of the variation in the data. It was therefore decided to use the overall mean value for the suction compression index.

The climatic data in Figure 5-14 is from the Love Field weather station. It is apparent that suction at depth (equilibrium) is 491 kPa (3.7 pF). Referring to Figure 5-18, yields a value of 22.0 percent for water content in these materials at the equilibrium suction. Data from the boring logs provided water content data beneath the pavement structure. A total of 68 measurements yielded a mean of 22.6 percent with a standard deivation of 3.7 percent. Clearly the water content data strongly indicate moisture equilibirum exists for these soils. Assuming a deviation of two standard deviations indicates a lack of equilibrium, four locations are found that are apparently not in equilibrium. Two of these were found to be soils that deviated from those characterized in the laboratory testing. Two borinys remained that had water contents two standard deviations from the equilibrium water content, one wet and one dry. It was concluded that satisfactory explanations could not be reached based on the available data. It was also not possible to establish the active zone depth from the available data.

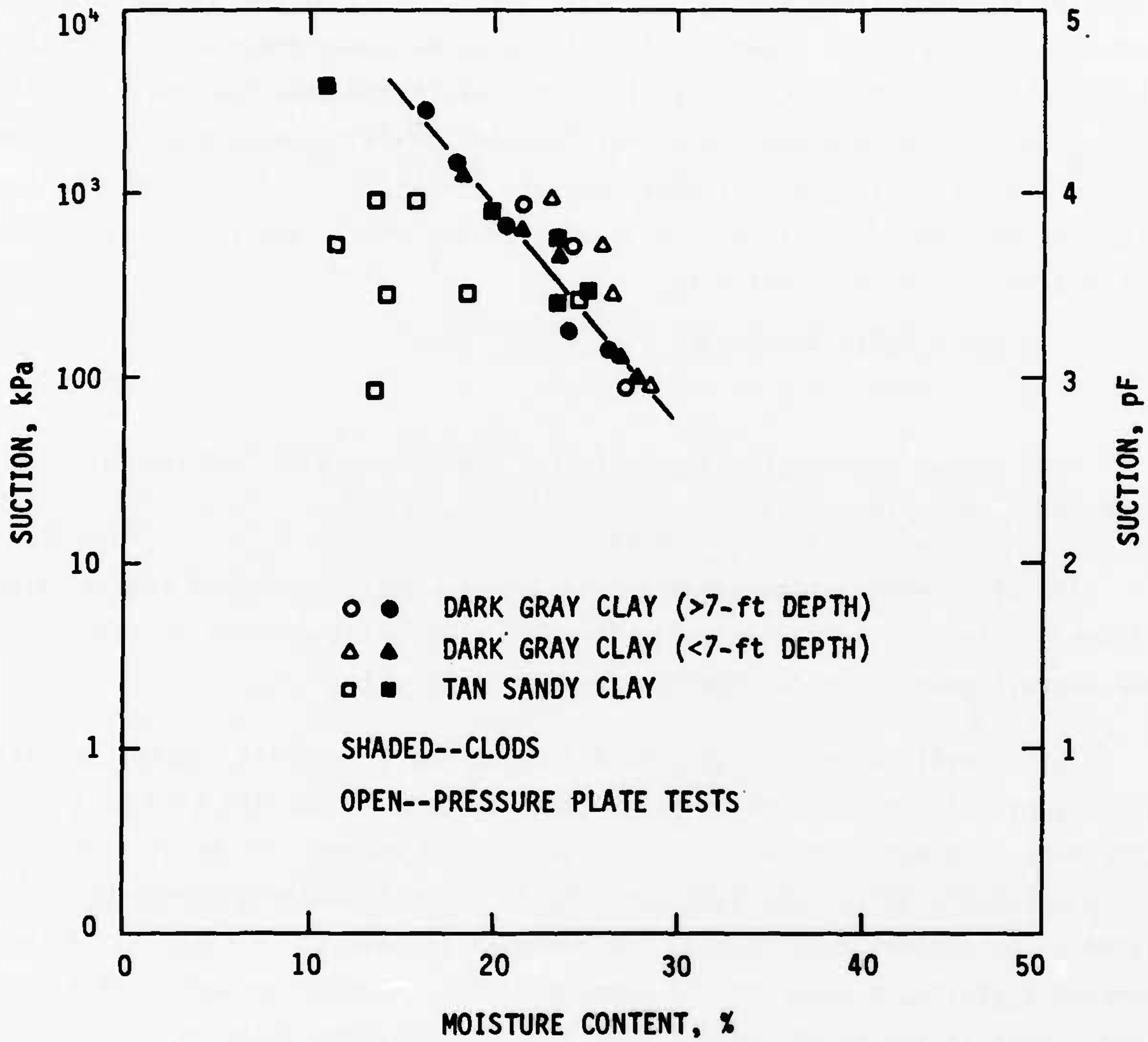


FIGURE 5-18. MOISTURE-SUCTION DATA (DAL)

TABLE 5-4. SUCTION COMPRESSION INDEX RESULTS

Soil description	n	C_h	CV, %
Dark gray clay (shallow)	5	0.078	20.3
Dark gray clay (deep)	5	0.086	33.8
Tan sandy clay	4	0.107	26.5
All Data	14	0.089	29.0

VI. SUMMARY AND RECOMMENDATIONS

RESULTS OF SITE INVESTIGATIONS

Soil characteristics obtained from the tests performed at the sites studied are show in Table 6-1. Using the procedure previously discussed, required pavement thickness was determined as shown in Table 6-2. Data previously obtained for GAL, JSN, and DFW are also included in order to represent the whole data base. Note the values of C_h are lower than γ_h values previously reported for these sites. All data were recalculated following the definition of C_h . Table 6-3 lists the equivalent thickness of pavements at each site studied. The ratings shown indicating acceptability were based on simulated aircraft runs and user comments (Ref. 1). The ratings for current sites are based on user comments and NMERI inspection. The required equivalent thicknesses are shown again for comparison. All thicknesses are computed based on modulus values assumed and converted to an equivalent thickness of material with a modulus of 3.4×10^6 kPa (5×10^5 lb/in^2). A pavement rating is also shown for comparison. The following paragraphs will provide more detailed discussion.

TABLE 6-1. SOIL CHARACTERISTICS

Site	C_h	CV(C_h)	Z_a m	(ft)	k (pci), kPa/m	(pci)
DFW	0.108	0.33	2.1	(7)	2.28×10^5	(840)
JSN	0.102	0.18	3.7	(12)	2.44×10^5	(900)
GAL	0.137	0.19	1.2	(4)	1.90×10^5	(700)
MDO(1)	0.127	0.201	1.2	(4)	2.69×10^5	(992)
MDO(2)	0.098	0.157	1.5	(5)	3.49×10^5	(1286)
MDO(3)	0.155	0.228	2.0	(6.5)	2.21×10^5	(813)
MSQ(16)	0.139	0.142	2.0	(6.5)	2.46×10^5	(906)
MSQ(17)	0.160	0.156	2.0	(6.5)	2.14×10^5	(788)
MSQ(18)	0.151	0.125	2.0	(6.5)	2.26×10^5	(834)
DAL	0.078	0.203	2.1	(7.0)	4.38×10^5	(1615)

TABLE 6-2. COMPUTATIONS OF REQUIRED PAVEMENT THICKNESS

Site	λ_s, m (ft)	$\bar{A}_a$, m (ft)	$\bar{A}_e$, m (ft)	$\frac{\Delta W}{2a}$	$\beta\lambda$	h_{pred}, m (in)
DFW	7.7 (25.2)	0.014 (0.047)	0.046 (0.150)	0.316	3.65	0.62 (24.6)
JSN	10.5 (34.5)	0.022 (0.072)	0.040 (0.132)	0.547	4.70	0.69 (27.3)
GAL	5.5 (18.2)	0.009 (0.030)	0.019 (0.063)	0.486	4.42	0.29 (11.6)
MDO(1)	5.5 (18.2)	0.009 (0.030)	0.015 (0.061)	0.496	4.50	0.32 (12.7)
MDO(2)	6.3 (20.8)	0.011 (0.037)	0.018 (0.058)	0.632	5.12	0.36 (14.0)
MDO(3)	7.4 (24.2)	0.014 (0.045)	0.042 (0.138)	0.324	3.68	0.58 (22.8)
MSQ(16)	7.4 (24.2)	0.014 (0.045)	0.023 (0.077)	0.581	4.85	0.41 (16.3)
MSQ(17)	7.4 (24.2)	0.014 (0.045)	0.030 (0.097)	0.459	4.28	0.47 (18.4)
MSQ(18)	7.4 (24.2)	0.014 (0.045)	0.022 (0.074)	0.607	5.00	0.39 (15.3)
DAL	7.7 (25.2)	0.014 (0.047)	0.020 (0.067)	0.711	5.68	0.43 (16.9)

TABLE 6-3. PAVEMENT EQUIVALENT THICKNESSES AT SITES STUDIED

Site	Pavement feature	Line No.	h (exist), m (in)	h (rqd), m (in)	Rating
DFW	Taxiway	3	0.82 (32.2)	0.62 (24.6)	Sat.
	Isolation apron	2	0.11 (4.5)	0.62 (24.6)	Sat.
	Access road	5-14	0.20 (7.9)	0.62 (24.6)	Unsat.
JSN	Taxiway	2,3	0.60 (23.6)	0.69 (27.3)	Unsat.
	Taxiway	5-9	0.52 (20.4)	0.69 (27.3)	Unsat.
	Apron	11,12	0.64 (25.1)	0.69 (27.3)	Unsat.
	Roadway	10	0.26 (10.1)	0.69 (27.3)	Unsat.
	Apron	4,15,16	0.31 (12.0)	0.69 (27.3)	Unsat.
GAL	RW/TW	2-6	0.40 (15.7)	0.29 (11.6)	Unsat/sat.
MDO	RW (original)	1-3	0.22 (8.8)	0.32 (12.7)	Unsat.
	RW (w/overlay)	1-3	0.31 (12.2)	0.36 (14.0)	Unsat.
				0.58 (22.8)	Unsat.
MSQ	Runway	17,18	0.17 (6.2)	0.41 (16.3)	Unsat.
	Taxiway	16	0.06 (2.5)	0.47 (18.4)	Unsat.
				0.39 (15.3)	Unsat.
DAL	Runway	---	0.70 (27.7)	0.43 (16.9)	Sat.

MURDO AIRPORT, SOUTH DAKOTA

Results of this design analysis indicate substantial pavement structures are required to provide adequate stiffness for the clay soil characteristics measured. A comparison of the fill area to the cut area clearly shows a significant advantage of the remove and replace technique developed for Interstate Highway construction in this area. A difference in required equivalent depth of pavement of up to 0.25 m (10 in) between site 1 and 3, is indicated. Site 3 is in a portion of the pavement where some of the material was excavated and recompacted during the rehabilitation work in 1980. It appears from these results the soil characteristics obtained may be sensitive to such features. Indicators are the $\bar{A}$ and λ_s values obtained.

Two factors are important in attempting to evaluate the Murdo site. The suction profiles reach only a depth of 4.6 m (15 ft) (due to equipment limitations) but they indicate nonequlibrium conditions exist to greater depth. Beneath the fill, the natural shale is extremely dry showing a wetting trend only near the fill. While this indicates the material may remain dry, it certainly has a great potential for deep seated swell. Normal equilibrium suction is expected to be about 980 kPa (4) pF. Therefore, any activity at the site that may alter surface or subsurface drainage in the vicinity of the fill should be studied carefully. Also drilling operations should require proper sealing of drill holes that penetrate this dry material in order to reduce the possibility of surface water or perched water tables directing water to the shale. It would be helpful to extend borings to about 12 m (40 ft) to determine if the moisture conditions are at equilibrium as discussed in Section III of this report. The fill material at a depth of four feet exhibits a moisture content equal to the constructed conditions.

The second factor of interest involves the suction profiles measured in the cut areas (borings 2 and 3). Both indicate soil that is wetter than equilibrium conditions dictated by the climate, to depths below that of sampling. Analysis of pavement and soil surface features indicate active zone depth of 1.5 m (5 ft) and 2.08 m (6.5 ft) as shown previously. It is believed that deep wetting represents long term response to the climate and that it occurs slowly. Perhaps the trend has resulted from three or four years of wetter than normal conditions. Since it occurs slowly, is deep seated, and appears rather uniform over a large area (borings 2 and 3 are 79 m [260 ft] apart) it has not been a significant factor in pavement performance. Certainly this reasoning mitigates some of the concern over deep seated swelling beneath the fill discussed above. In fact, the Thornthwaite Index calculated for the last several years confirms this (Table 6-4).

Since the pavement at the time of the site investigation exhibited distress, it is logical to make a recommendation for rehabilitation based on this analysis. Some limitation of the actual site investigation must be stated then recommendations will be addressed. No borings were made through the pavement structure. Therefore, the condition beneath the pavement is not known. This is a very important piece of information particularly in relation to the large transverse cracks present in the pavement. Actual thickness of pavement layers were not verified and their physical properties (modulus) were assumed, rather than measured. It is recommended that tests be conducted for these data in a site investigation if rehabilitation is undertaken.

Apparently the soil beneath the pavement is still undergoing moisture variation producing the distress in the pavement structure. This may be due to horizontal water movement through discontinuities in the shale or vertical movement through the pavement. If it were vertical the distress would necessarily coincide with discontinuities in the pavement structure. Inspection of the site in September 1984 indicated no correlation whatsoever. In fact several areas of major distress were not associated with the cracking patterns in the pavement structure. Based on these observations it is concluded that water is infiltrating by horizontal movement.

The problem now becomes one of determining how best to stabilize the moisture condition of the soil in the active zone beneath the pavement. One method is remove and replace, as was done for the fill at Murdo. Results of this study indicate a reduction in the required thickness of pavement, but it is still a substantial structure. Therefore, if 2.1 m (7 ft) of material were removed and replaced, a required pavements structure, equivalent to 0.32 m

TABLE 6-4. THORNTHWAITE MOISTURE INDEX, MURDO

Year	Total rainfall, m	(in)	Average temperature, °C	(°F)	Thornthwaite Index m	(in)
1975	0.53	(20.9)	8.2	(46.7)	-0.37	(-14.7)
1876	0.18	(7.3)	9.8	(49.6)	-1.08	(-42.5)
1977	0.70	(27.4)	8.5	(47.3)	-0.32	(-12.6)
1978	0.48	(19.0)	6.8	(44.3)	-0.24	(-9.6)
1979	0.41	(16.3)	7.4	(45.4)	-0.50	(-19.6)
1980	0.35	(13.7)	5.1	(41.2)	-0.92	(-36.4)
1981	0.51	(20.2)	10.7	(51.3)	-0.34	(-13.4)
1982	0.68	(26.7)	8.6	(46.5)	-0.05	(-1.8)
1983	0.49	(19.5)	9.1	(48.4)	-0.24	(-9.4)
1984	0.47	(18.6)	9.6	(49.3)	-0.34	(-13.2)

(12.7 in) of asphalt concrete is calculated (Table 6-2). In view of the existing structure, this alternative seems unrealistic as a rehabilitation alternative.

Another way to prevent moisture movement in in situ soils is to fill the seams, cracks, and fissures with an impermeable material. This is accomplished by chemical injection under pressure. The technique has been successful in areas with an extensive network of passages to allow the chemicals to move into the soil mass. Prior to serious consideration of the method, a testing program sould be carried out to ensure the injection procedures will distribute the chemicals adequately in this material. Tests of the effect of chemicals used on the soils should also be conducted as part of the evaluation. The depth of treatment should be set based on further study of the suction profiles at the site.

The Texas Department of Highways and Public Transportation has developed the use of vertical moisture barriers for preventing moisture movement beneath pavements. The technique involves placing a synthetic, water proof material to a depth of 2.5 m (8 ft). Field experiments have demonstrated that moisture

A second factor for consideration also relates to the wetter than normal subgrade. Wetter climates have a deeper active zone than drier climates. The introduction of a water table causes a wetter soil moisture condition. This in turn has a greater potential for change under a prolonged drought, due to the wetter initial condition. In view of this possibility, the provision of horizontal membranes at the pavement edge is considered an excellent feature for this site. The maintenance of water tightness of this apron should be a high priority item.

In summary, the soils at the Mesquite Airport are highly expansive and should be given special consideration. The wetting influence of the water table should stabilize the pavement under normal climatic conditions. Under a severe drying period the wetter condition is a less desirable starting point. The proposed pavement sections (Ref. 18) are much better suited to the subgrade onsite than the existing pavement. The asphalt concrete sections are a little low when compared to the calculated requirement. The performance will depend on the actual moisture variation experienced. If it undergoes the variation assumed in design, it may require a smoothing overlay in a few years. The portland cement concrete sections are adequate. The maintenance of joints and prevention of water entering these pavements is a high priority requirement.

LOVE FIELD, DALLAS, TEXAS

The predicted equivalent depth required is less than the existing pavement for this site. Current roughness of the runway has been attributed to differential settlement in the fill, rather than expansive soil movements. Evaluation of the moisture condition indicates a water content very near that expected based on the Thornthwaite Index. In this case the existing pavement should not undergo further movements due to moisture variation in the clay subgrade. It should be made clear the NMERI investigation was limited and did not include an evaluation of the fill material. Since tests by others provided evidence that the fill was compressing, its moisture condition is of great interest. Referring to Figure 5-3, it can be seen that the fill and

18. "Subsurface Investigation for Airport Improvements Hudson Municipal Airport, Mesquite, Texas," Southwestern Laboratories, SWL Report No. 84-200, June 1984.

natural clay were clearly exchanging moisture at Murdo Airport. It is reasonable to expect that at Love Field a similar moisture exchange may have occurred or is occurring, which could affect surface profile. The conclusion reached is that the pavement structure is stable with respect to volume change. This finding dictates the use of some type of overlay to rehabilitate the pavement rather than reconstruction.

RECOMMENDATIONS

This report has documented the application of a method developed for determining the required thickness of airport pavements for expansive soil subgrades. It is outlined in Section IV with sufficient detailed methods in the Appendixes for application in practice. The method departs from normal approaches to pavement design in that interaction of the environment, soil and pavement is the behavior considered; while most methods focus on aircraft loading. The problem of expansive soil damage usually does not involve load-induced pavement distress.

Methods for characterizing the suction and volume change behavior of expansive soils are provided. These methods have been in use at NMERI since the mid 1970s and have proven reliable in furnishing meaningful soil characteristics. The data and procedures are unlike many standard soil mechanics tests. Technician training specifically for these tests is believed to be an important part of their implementation. Similarly, suction data reveal information about the moisture in a soil profile, its present condition, previous trends of movement and potential future movement. In general geotechnical and pavement engineers are not trained in the measurement of moisture suction or interpretation of suction data. The potential value gained by measurement of suction is viewed as great. Here again, training is an essential step toward implementation of the method.

It became clear in the present study that subgrade moisture condition evaluation could be accomplished easily using the methods developed for the expansive soils design method. When an overlay is considered, the moisture condition of the subgrade should be evaluated to assess its potential for future changes. This information should be considered in the decision making process. The selection of an alternative for rehabilitation is affected by the subgrade moisture condition. For example, as this report indicates, a method to stabilize the active zone soil beneath the pavement (such as chemical injection) should be considered for the Murdo Airport while at Love Field simply overlaying the runway appears the best alternative.

In the present study the testing was much the same as previously used. The expansion of the experience base was an important aspect of the work. The sites studied included the presence of fill materials, a water table influencing the subgrade and a highly jointed shale. The experience gained was a valuable addition to the previous work. The application to these new variations also demonstrates the applicability of the aproach to a wide range of clay soil problems. Sites located in potential expansive areas should be used to further refine the methods studied and extend the experience base.

In Figures 5-3 and 5-4, results of field samples as well as laboratory samples are shown. The laboratory samples are consistently at higher suction than corresponding field samples. This is an important factor in planning and conducting site investigations. The increase in suction may be a direct result of overburden stress relief during sampling. Its absence in near-surface soils seems to confirm this. If this reasoning is valid, a method of back calculating the coefficient of earth pressure at rest (K_o) may be derived from such testing. The proper timing of tests is certainly shown to be important.

This study has been directed to the design of new pavements. Rehabilitation of existing pavements is another problem requiring a somewhat different approach. The main focus of an investigation for pavement rehabilitation is deriving useful data from the existing pavement. The results of suction tests clearly reveal trends in the soil moisture behavior that are important factors in evaluating present condition as well as possible future changes. The combination of the construction records and existing data yield the information required to back calculate soil characteristics such as the diffusion coefficient (α) which is directly applicable to estimates of swell rate as well as the amount.

The FAA Expansive Soil Pavement Design Procedure was applied to three airport pavements. The calculated thicknesses appear to be reasonable based on experience at each site. A much greater variety of conditions were involved in these sites than previous study sites. This factor indicates a versatile and widely useful set of analytical tools for investigating behavior of engineering structures and selecting among design alternatives.

APPENDIX A
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. 19).

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. The model was a controversial subject until 1960 when at the London Conference on Pore Pressures and Suction in Soils (Ref. 20) 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 betweeen 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 A-1). Basically, soil suction is considered to

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19. Low, Phillip F., "Fundamental Mechanisms Involved in Expansion of Clays as Particularly Related to CLay Mineralogy," **Proceedings of a Workshop on Expansive Clays and Shales in Highway Design and Construction**, Vol. 1, May 1973, pp. 70-91.

20. **Pore Pressure and Suction in Soils**, Conference organized by the British National Society of the International Society of Soil Mechanics and Foundation Engineering, Butterworths, 1961, p. 151.

TABLE A-1. DEFINITIONS OF SUCTION AND POTENTIAL (Ref. 21)

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
Osomotic (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	

21. Statement of the Review Panel: "Engineering Concepts of Moisture Equilibria and Moisture Changes in Soils," Symposium in Print, Butterworths, 1965, pp. 7-21.

TABLE A-1. CONCLUDED

Term	Definition	Common units
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	

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.

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
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

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

where

τ = total suction, bars (a positive quantity)

ψ = soil water potential, bars (a negative quantity)

R = universal gas constant (80.88 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. 22)

1. Water behaves as an ideal gas
2. Water vapor in the air space where the relative humidity is determined is in equilibrium with the soil water vapor
3. Isothermal conditions ($\Delta T < \pm 3°C$)
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).

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

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22. Johnson, L. D., **An Evaluation of the Thermocouple Psychrometer Technique for the Measurement of Suction in Clay Soils**, Technical Report S-74-1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, January 1974, p. 67.

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

Figure A-1 is a plot of τ versus $P/P_0 \times 100$ 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.

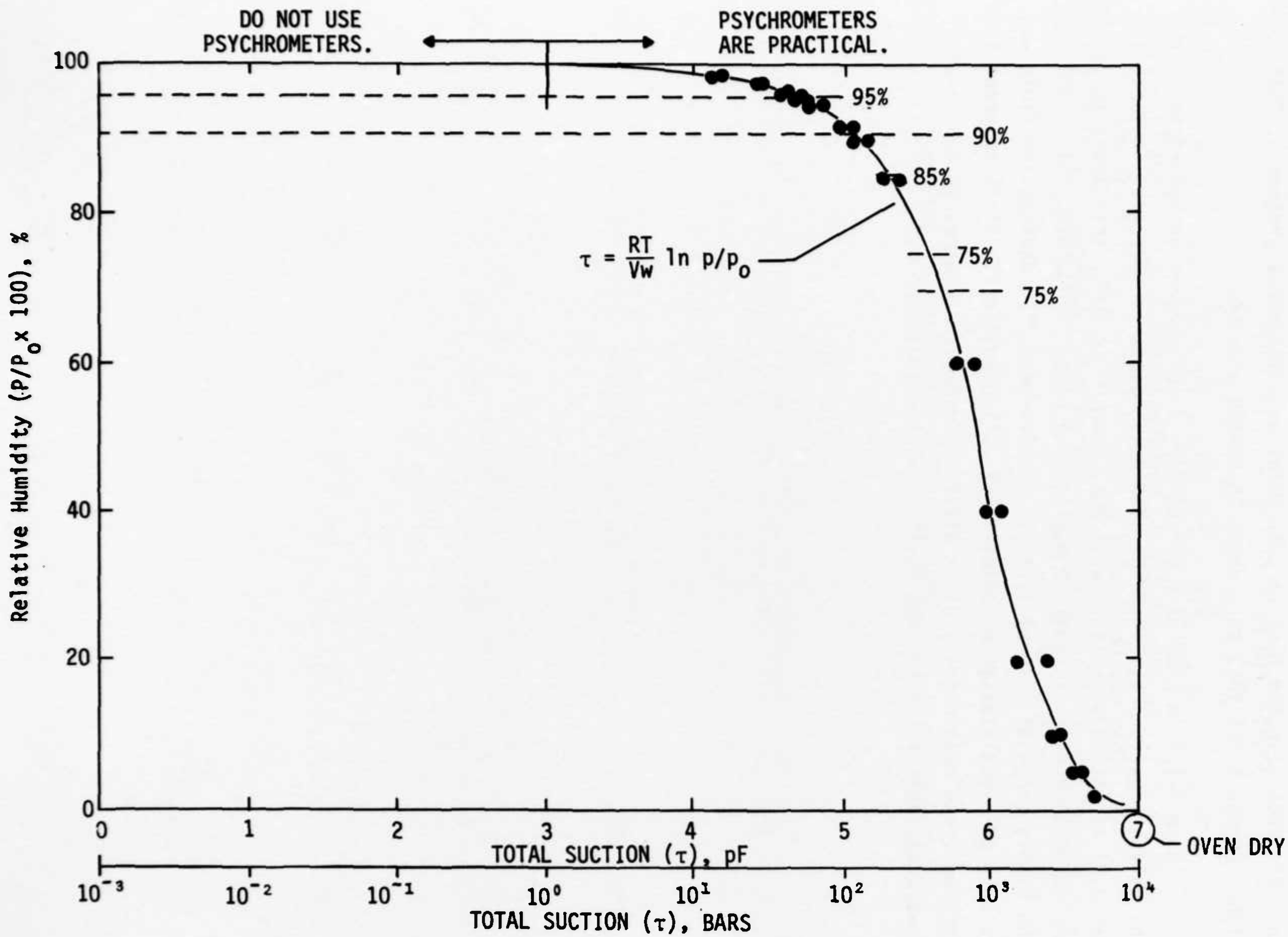

FIGURE A-1. RELATIONSHIP BETWEEN RELATIVE HUMIDITY AND SUCTION

APPENDIX B

PROCEDURE FOR DETERMINING MOISTURE CHARACTERISTICS

The moisture characteristic is the relationship between suction and water content for a soil. It can usually be developed from data obtained in site investigations. This may be accomplished by measuring suction and water content on samples of the soil. The following procedure is then used to determine the moisture characteristic.

Figure B-1 is a plot of suction versus water content for samples taken from a clay soil profile. Ten values of suction and water content are plotted. The mean for them is obtained and plotted. The moisture characteristic is established by constructing a line through the point 3.1×10^5 kPa (6.25 pF) and the average suction and water-content. The slope of this line, the moisture characteristic curve is then equal to

$$m = \frac{\Delta h}{\Delta w} = \frac{(h)_{avg} - 6.25 \text{ pF}}{(w)_{avg}} \text{ (in pF units)}$$

or

$$m = \frac{\Delta \log h}{\Delta w} = \frac{\log(h)avg - \log(3.1 \times 10^5 \text{ kPa})}{(w)_{avg}} \text{ (in kPa units)}$$

The slope m will be the same in either set of units and should be thought of as the change in log of suction per unit change in water content.

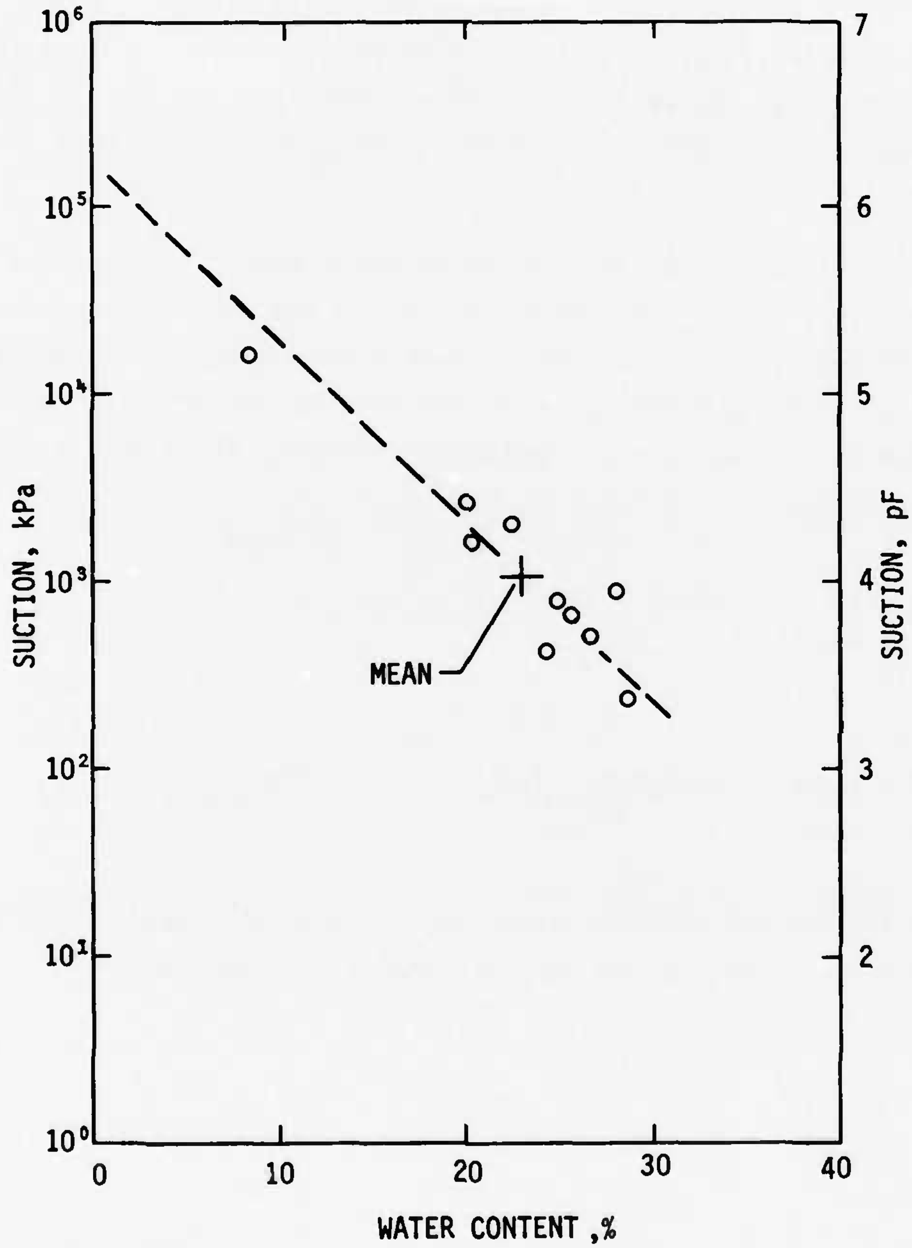

FIGURE B-1. EXAMPLE OF MOISTURE CHARACTERISTIC DETERMINATION

APPENDIX C
EQUILIBRIUM SUCTION PROFILES

The procedures included in this report provide a rational approach to estimating expansive soil behavior. One element of this approach is estimating suction change. Suction change is usually determined by estimating the initial and final profiles of suction for design. The suction profile is simply a plot of soil suction versus depth. An initial profile may be estimated by sampling the materials and measuring suction. Some allowance should be made for changes prior to construction. It is then necessary to estimate the final equilibrium suction profile. In this appendix some measured final profiles will be shown, followed by procedures for selecting the final profile for design.

MEASURED SUCTION PROFILES

Figure C-1 is a plot of suction profiles measured on soil samples removed from a site in the Dallas, Texas area. The profiles shown were measured in October 1978 (□), after a very dry period; May 1979 (o), following spring rains; and July 1979 (Δ), during normal summer drying. The jogs or bends in the profiles are believed to accurately reflect minor deviations of weather that interrupt the major trends. In the profiles for May and July such interruptions are apparent. The profile for October does not show this feature due to the long time period involved and the lack of any significant interruption of the trend.

Figure C-2 illustrates data measured in an area affected by the water use of a tree and a nearby area overlain by a pavement. These are typical suction profiles where water tables are absent. In the presence of water tables, it is usually assumed that suction increases at a one-to-one rate with distance above the water table.

An approximation proposed by Hamberg (Ref. 15) is shown in Figure C-3. It amounts to the assumption that suction of the surface goes from the shrinkage limit 31 MPa (5.5 pF) to about the plastic limit 97 kPa (3.0 pF) at the surface. It is necessary to determine the suction at depth and the depth

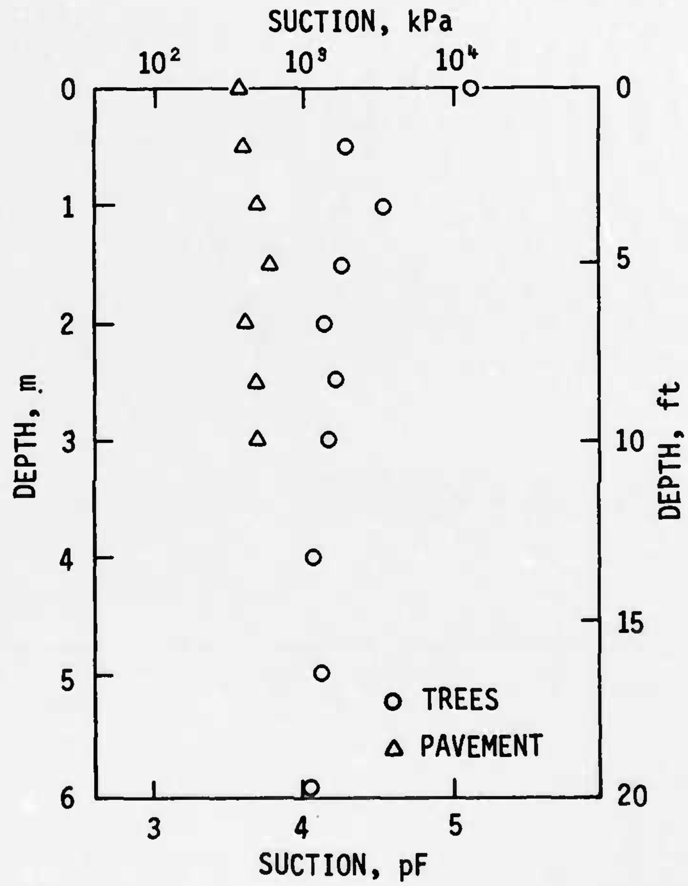

FIGURE C-2. SUCTION BENEATH TREES AND PAVEMENT

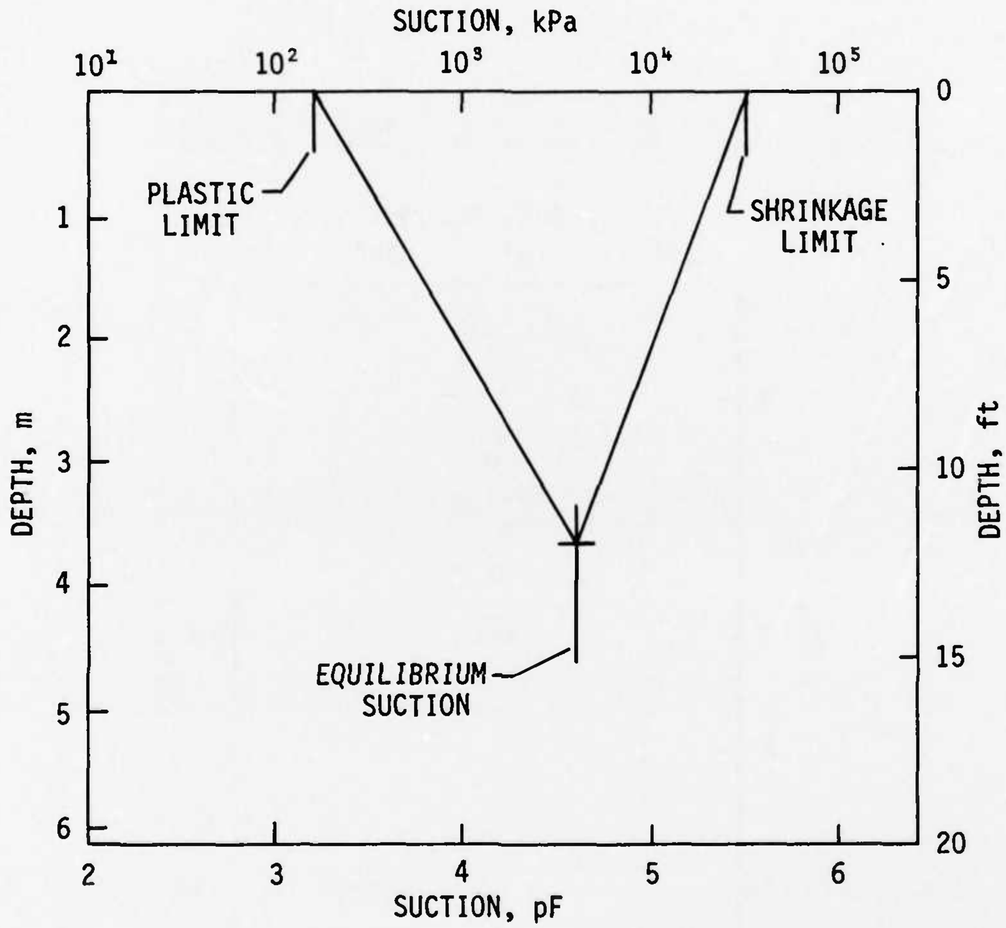

FIGURE C-3. METHOD OF APPROXIMATING SUCTION CHANGES FOR DESIGN (REF. 12) considered active for design. If this method is applied to Figure C-1, and 3 m (10 ft) is selected for the active zone depth, the average suction change for the profile is a change of 1.25 log kPa or 1.25 pF. This is the average change over an active zone depth of 3 m (10 ft).

APPENDIX D

PROCEDURE FOR DETERMINING SUCTION COMPRESSION INDEX (C_h)

INTRODUCTION

The suction compression index, C_h, is calculated using a suction measurement and two determinations of bulk density. The suction measurement may be made by using a variety of measurement methods. In this discussion the filter paper method is used. It is described below in detail. The result obtained is a total suction measurement, but is usually assumed to equal the matric suction value. The method presented here was developed from that presented by McQueen and Miller (Ref. 23). Bulk density measurements are made using a modified COLE (Coefficient of Linear Extensibility) procedure (Ref. 24). The difference is in initial conditions and the manner in which the data are interpreted and used. The test involves use of a Saran™ resin coating that is permeable to water vapor but almost impermeable to liquid water. This permits bulk density measurements by submerging in water, yet allows drying through the coating eliminating the need to strip it off the specimen.

FILTER PAPER SUCTION MEASUREMENTS

This test method uses laboratory filter papers as passive sensors to measure the moisture suction of soil samples. This involves placing the papers in a moisture can with the sample and equilibrating at a constant temperature for a week. It is assumed the water vapor in the air, soil sample and paper are in equilibrium after this period of time. The details of the test procedure are provided below.

Calibration--The paper used must be ash-free quantitative filter paper. In this work Schleicher and Schuell No. 589, White Ribbon has been used exclusively. The paper was calibrated following procedures used by McQueen and Miller (Ref. 25). Their work resulted in a two-part relationship. The upper

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23. McQueen, Irel S., and Miller, R. F., "Calibration and Evaluation of a Wide-Range Gravimetric Method for Measuring Moisture Stress," **Soil Science**, Vol. 106, No. 3: 1968, pp. 225-231.
24. SCS, **Soil Survey laboratory Methods and Procedures for Collecting Soil Samples**, Soil Survey Investigations Report No. 1, Soil Conservation Service, U.S. Department of Agriculture, April 1972.
25. McQueen, I. S., and Miller, R. F., "Approximating Soil Moisture Characteristics from Limited Data: Empirical Evidence and Tentative Model," **Water Resources Research**, Vol. 10, No. 3, 1974, pp. 521-527.

segment represents moisture retained as films adsorbed to particle surfaces while the lower segment represents moisture retained by capillary or surface tension forces. Figure D-1 illustrates the NMERI calibration curves used throughout this study. Using the calibration curve, a filter paper water content can be converted to a suction value.

Procedure--Two sizes of moisture cans are used in the test. A 29.6 cm^3 (8 ounce) can, coated with zinc chromate (to retard rusting) is used for the initial sample equilibration. A soil sample, nominally 120 gm (0.26 lb), is placed in the can. Two filter papers are placed on top of the sample. If the sample is wet, its contact with the filter paper should be broken to prevent wetting by capillary action. This may be accomplished using rubber O rings, screen wire, etc., between the filter paper and the soil. Normally, expansive soils are drier than the plastic limit and this is not a problem. The can lid is put in place and sealed with a plastic tape (Scotch 88 was used by NMERI.) The sealed can is then placed in an insulated chest, and placed in a location with temperature variation of less than 3°C. Temperature variations inside the chest should be less than 1°C. The samples are allowed 7 days to equilibrate.

At the end of the equilibration period, each filter paper will require its own preweighed 7.4 cm^3 (2 oz) aluminum moisture can. These are weighed to the nearest 0.0001 gm, designated T_c (tare-cold), before the soil sample cans are removed from the insulated chest. The cans are numbered by imprinting with a metal stamp. The cans should not be written on with any type of marker. It is suggested the tare weights be taken immediately prior to weighing the papers.

Utilizing a pair of tweezers, transfer the top filter paper from the soil sample can into the smaller preweighed aluminum can. Repeat this procedure for the second filter paper using the second preweighed aluminum can. This entire process must be completed in three to five seconds per paper. It is helpful to place lids loosely on the cans (not ajar) in performing this requirement. Care must be taken to replace lids after each transfer, i.e. take the filter paper from the large moisture can, replace lid, place paper into small can and place its lid on the can. Repeat this for the second

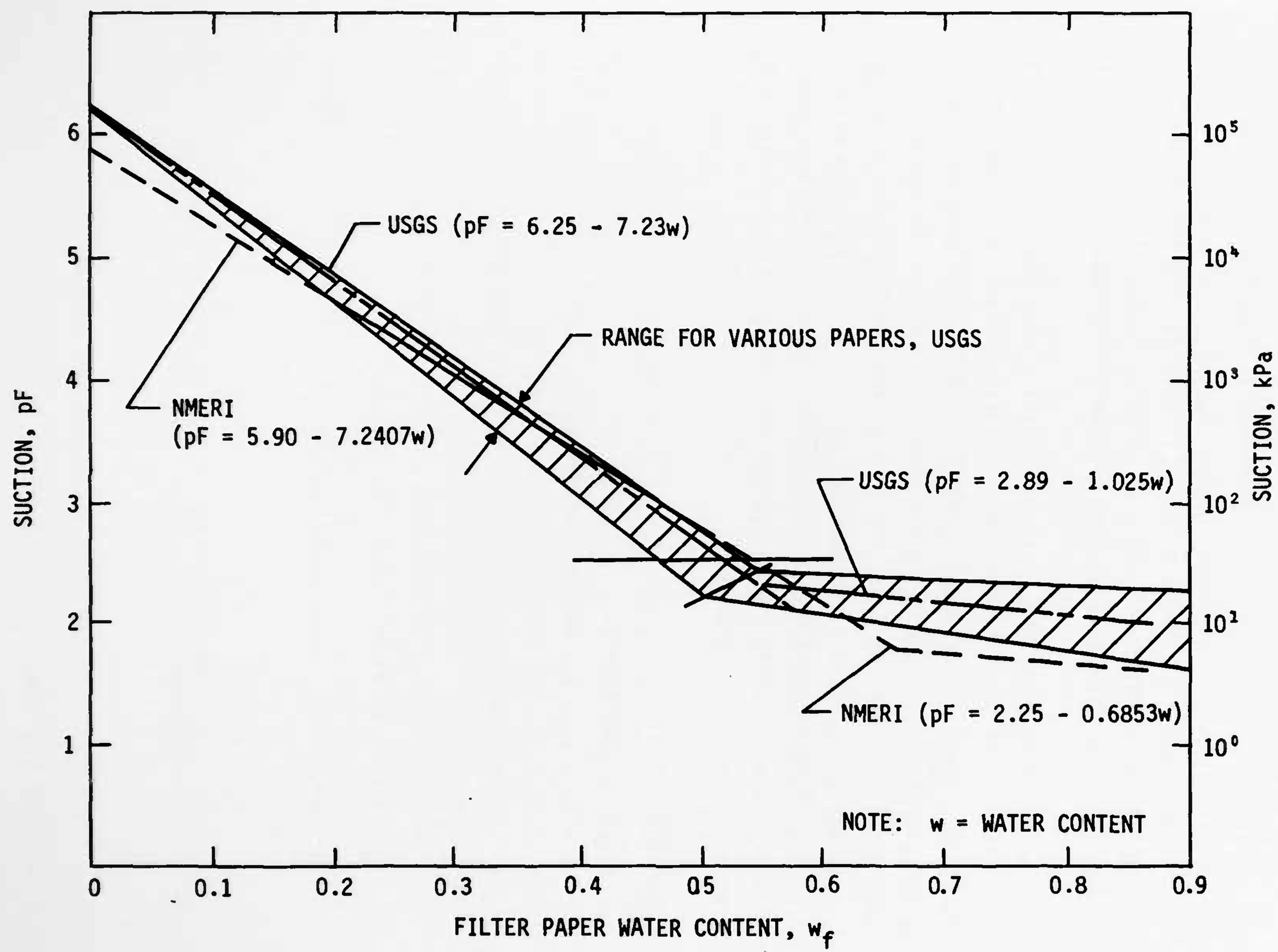

FIGURE D-1. FILTER PAPER CALIBRATION RELATIONSHIPS paper. The lids must be kept in place to ensure that ambient air does not alter the moisture condition of the sample or filter papers.

Both filter paper cans should be individually weighed immediately with the lids tightly closed. This weight is $W_1 = W_f + W_w + T_c$, consisting of the weights of filter paper, water and cold tare. The cans are then placed in an oven at 110 ±5°C (230 ± 9°F) with the lids slightly ajar to permit moisture to escape. The cans remain in the oven for a minimum of 16 hours.

After the minimum time, the lids are tightly closed and left in the oven for another 15 minutes to allow temperature equilibration. The cans are removed one at a time, placed on an aluminum block for 30 s to cool, then weighed to determine $W_2 = W_f + T_h$ to the nearest 0.0001 gm. Immediately remove and discard the filter paper and reweigh the can to 0.0001 gm to determine T_h (the hot tare). The aluminum block acts as a heat sink and will reduce the temperature variation during weighing. Water content or other properties of the soil sample are determined following the appropriate ASTM procedures or the clod test procedure.

Computations--

Measured quantities:

$W_1 = W_f + W_w + T_c$--wet weight plus tare (cold)

$W_2 = W_f + T_h$--dry weight plus tare (hot)

T_c = cold tare

T_h = hot tare

W_f = weight of dry filter paper

W_w = weight of water in the filler paper

Determine filter paper water content:

$$W_f + W_w = W_1 - T_c$$

$$W_f = W_2 - T_h$$

$$w_f = \frac{(W_w + W_f) - W_f}{W_f} = \frac{W_w}{W_f}$$

The capital W is used to denote weights measured while the lower case w is used to represent the water content of the filter paper.

Convert the filter paper water content (w_f) to a suction value:

$$h = m\ (w_f) + b$$

Where m and b are the slope and intercept of the filter paper calibration curve. The units of h depend on the units associated with m and b; they must be consistent.

Example--

$$
\begin{aligned}
W_1 &= 15.7629 \text{ g} \\
T_c &= 15.4993 \text{ g} \\
W_2 &= 15.6882 \text{ g} \\
T_h &= 15.4957 \text{ g} \\
\left.\begin{aligned} m &= 6.2407 \\ b &= 5.90 \end{aligned}\right\} & \quad \text{Fig. D-1} \\
w &= 0.369 \text{ or } 36.9 \text{ percent} \\
h(pF) &= 5.90 - 6.2407\ (0.369) \\
h(pF) &= 3.60 \text{ pF}
\end{aligned}
$$

CLOD TEST PROCEDURE

Measurement Procedure--Soil samples weighing 120 ± 20 g are separated from undisturbed samples and placed in 30 cm^3 (8 ounce) moisture cans, as soon after sampling as practical. This can easily be accomplished in the field if samples are extruded from the samplers. Other tests may be performed after samples are returned to the laboratory in order to vary the moisture condition to develop data for a wide range of moisture, if desired. Samples are normally wetted to three moisture contents wetter than natural and three drier by assuming a value of in situ moisture and adjusting sample moisture based on weight.

Suction measurements are made following procedures given above for filter paper suction measurements. After equilibration the filter paper and soil sample are separated. Filter paper is treated as described above, the soil sample is treated as described by the following.

Samples are weighed (W_1), followed by preparation of the sample for bulk density measurements. A wire, tag, and possibly a hair net are attached to provide a means of handling the sample. Hair nets are used only for those samples that fall apart without support. In tests involving moisture adjustments, these tare items are added before the moisture altered. A second weight is measured (W_2).

The next step involves coating the sample with Saran™ resin. The solutions used are 1:7 or 1:4 (only for coarse soils), Saran™: methyl ethyl ketone. This procedure should be performed in a well-ventilated area, preferably under a fume hood. The sequence for applying the coating is as follows:

1. For 1:4 solution: dip in liquid, dry 5 minutes; dip in again, dry 55 minutes.

2. For 1:7 solution: dip in liquid, dry 5 minutes; dip again, dry eight minutes; dip again, dry 55 minutes.

These procedures for coating are based on the Soil Conservation Service method.

Immediately weigh the sample in air and water at the end of the drying period. These weighings are designated W_3 and W_4 respectively. In the normal NMERI method, W_4 is the buoyant force exerted on the sample, which is measured directly, rather than the submerged weight of the sample.

Samples are then air dried to an approximately constant weight. They are weighed in air and water again, yielding values designated as W_5 and W_6. The samples are then placed in a cool oven which is started and raised to 105°C, and dried for 48 hours. This procedure is used to prevent the coating from separating from the sample due to thermal shock.

Samples are removed from the oven and cooled until they can be easily handled. They are weighed in air and water again (W_7 and W_8). All weights measured with the sample in water are buoyant force on the sample (W_4, W_6, W_8).

<u>Summary of Measurements</u>--

W_1 = Weight of wet sample = $W_s + W_w$
W_2 = Weight of wet sample plus tare = $W_s + W_w + T$
W_3 = Weight of wet sample, tare, coating = $W_s + W_w + T + W_r$
W_4 = Buoyant force on submerged sample
W_5 = Weight of air-dried sample, coating, tare = $W_s + (W_w)_a + T + W_r$

W_6 = Buoyant force on submerged air-dried sample

W_7 = Oven dry weight = $W_s + T + 0.85(W_r)$

W_8 = Buoyant force on sample

T_4, T_6, T_8 = Water temperatures at which the buoyant force measurements (W_4, W_6, W_8) are made

where

W_s = weight of solids

W_w = weight of water

T = tare weight (wire, tag, hair net)

W_r = weight of Saran™ resin coating (when oven dried the resin loses 15 percent of its weight).

γ_r = density of Saran™ resin = 1.2 g/cc

Computations--

1. Weight of solids:

$$(W_s) = W_7 - 0.85(W_3 - W_2) - (W_2 - W_1)$$

2. Water content (gravimetric):

$$(w) = \frac{W_1 - W_s}{W_s}$$

3. Dry bulk density of moist sample:

$$[Dbm] = \left[\frac{W_s}{\dfrac{W_4}{(\gamma_w)_4} - \dfrac{W_3 - W_2}{1.3}}\right]$$

where

$(\gamma_w)_4$ = water density at T_4

4. Void ratio of moist sample:

$$e = \frac{G_s\gamma_w}{Dbm} - 1$$

where

G_s = specific gravity of solid particles

5. Degree of saturation of moist sample:

$$S = \frac{1}{e}\left[\frac{(1 + w)(Dbm)(1 + e)}{(\gamma_w)_4}\right] - G_s$$

6. Water content (gravimetric) after air drying:

$$(w)_a = \left[\frac{W_5 - W_s - (W_2 - W_1) - (W_3 - W_2)}{W_s}\right]$$

7. Dry bulk density after air drying:

$$Dba = \left[\frac{W_s}{\dfrac{W_6}{(\gamma_w)_6} - \dfrac{W_3 - W_2}{1.3}}\right]$$

8. Void ratio after air drying:

$$(e)_a = \frac{G_s\gamma_w}{Dba} - 1$$

9. Degree of saturation of air dried sample:

$$(S)_a = \frac{wGs}{e_a}$$

10. Dry bulk density of the oven dried sample:

$$(Dbd) = \left[\frac{W_s}{\dfrac{W_8}{(\gamma_w)_8} - \dfrac{W_3 - W_2}{1.3}\ 0.85}\right]$$

where

$(\gamma_w)_8$ = density of water at T_8

11. Suction compression index (C_h) =

$$\left[\frac{Dbd}{Dbm} - 1\right]\frac{Dbm}{Dbd}\left[\frac{1}{\log\dfrac{h_f}{h_i}}\right]$$

or

$$C_h = \frac{\Delta V/V_m}{\Delta \log h}$$

where

h_i = measured initial suction

h_f = final assumed to be 5.5 pF or 31.0 MPa or use a plot of $\Delta V/V_m$ versus suction to obtain h_f.

Example (weights in grams)--

h_i = 3.850 pF = 694 kPa

W_1 = 108.11

W_2 = 109.16

W_3 = 113.07

W_4 = 62.63

W_5 = 93.00

W_6 = 52.66

W_7 = 89.33

W_8 = 50.01

$T_4 = T_6 = T_8 = 71°F$

γ_w = 0.9772

G_s = 2.77

W_s = (89.83) - 0.85(113.07 - 109.16) - (109.16 - 108.11) = 85.46 g

$$w = \frac{108.11 - 85.46}{85.46} = 0.265 \text{ or } 26.5 \text{ percent}$$

$$Dbm = \frac{85.46}{\frac{62.63}{0.9772} - \frac{(113.07 - 109.16)}{1.3}} = 1.399 \text{ g/cm}^3$$

$$e = \frac{2.77 \cdot (1.0)}{1.399} - 1 = 0.980$$

$$S = \frac{1}{0.980}\left[\frac{(1.265)(1.399)(1.980)}{1.0}\right] - 2.77 = 0.805 \text{ or } 80.5 \text{ percent}$$

$$w_a = \frac{93.00 - 85.46 - (113.07 - 109.16) - (109.16 - 108.11)}{85.46} = 0.03 \text{ or}$$

3 percent

$$Dba = \frac{85.46}{\dfrac{52.66}{0.9772} - \dfrac{(113.07 - 109.16)}{1.3}} = 1.680 \text{ g/cm}^3$$

$$e_a = \frac{(2.77)1.0}{1.680} - 1 = 0.649$$

$$s_a = \frac{0.03\ (2.77)}{0.649} = 0.128 \text{ or } 12.8 \text{ percent}$$

$$Dbd = \frac{85.46}{\dfrac{50.1}{0.9772} - \dfrac{[(113.01) - 109.16]}{1.3}\,0.85} = 1.758 \text{ g/cm}^3$$

$$C_h = \left[\frac{1.758}{1.399} - 1\right] \frac{1.399}{1.758} \left[\frac{1}{\log \dfrac{31.0}{0.694}}\right] = 0.124$$

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