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Expansive Soil Pavement Design Using Case Studies

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SYNOPSIS A study of the field behavior of airport pavements on expansive soils was made for the purpose of developing design procedures for expansive soil areas. Through theoretical developments, computer simulation and empirical calibration a pavement thickness design procedure was developed. The selection of pavement thickness using the method insures a stiff enough pavement to reduce differential movements to acceptable levels based on calculated aircraft response. Differential movements are calculated using a soil model developed from recent concepts of expansive soil behavior. A soil pavement interaction model was derived for calculating the restraint provided by pavement stiffness.

INTRODUCTION

In 1975 the Federal Aviation Administration (FAA) initiated a study intended to provide improved methods for the design of airport pavements on expansive soils. The study was conducted as a joint effort of the FAA and the U.S. Air Force, Engineering and Services Laboratory (AFESC), at the University of New Mexico, Engineering Research Institute (NMERI).

Two interim reports (McKeen (1976), McKeen (1978) and a final report McKeen (1981)) have been published by the FAA. A subsequent paper, McKeen (1982), simplified some aspects of the results and presented a procedure suitable for routine use. The present paper offers further refinements, particularly in the soil model used to calculate differential surface heave for pavement design. The approach taken in the research study was to determine material properties which could be used together with a model to predict field behavior of airport pavements placed on expansive soils. In the following paragraphs these will be discussed.

MATERIAL BEHAVIOR

Expansive soils are clayey soils which exhibit significant volume changes as a result of soil moisture variations. Load changes also induce volume change; however, in shallow structures (i.e., pavements), the moisture-induced variation is most important. The best method of characterizing this behavior is through use of suction-based coefficients. Soil moisture suction is defined as a macroscopic property of the soil which indicates the intensity with which a soil will attract water. The coefficient most suited to expansive soil study is the suction compression index, γ_h , Lytton (1977). It

represents the volumetric response of an expansive clay to a change in moisture suction as follows:

$$\gamma_h = \frac{\Delta V/V_i \cdot f}{\log_{10} \frac{h_f}{h_i}} \quad (1)$$

γ_h = suction compression index

$\Delta V/V_i$ = change of volume with respect to an initial value

h_f, h_i = final and initial suction, in arithmetic units

f = lateral restraint factor
(0.33 to 1.0)

The next important aspect of material behavior is the idea of a soil active zone. This is the portion of the soil which interacts with the atmosphere by exchanging moisture (wetting and drying). The active zone depth (z_o) is best determined by making periodic measurements of soil suction with depth during dry and wet periods of the year. It must also be remembered that year-to-year variations may also occur.

Another concept important in dealing with expansive soils is that of an equilibrium suction condition. When a pavement is built on the soil it will tend to wet or dry toward a condition determined by the soil below the active zone or a water table (if near the surface, within 20 ft.). Movement will be a direct function of the amount of suction change from the soil condition at time of pavement placing, to the equilibrium value. At the equilibrium condition the suction will be uniform except near boundaries exposed to the environment.

In order to obtain a differential heave at the surface of an expansive soil, one of the parameters must vary. As shown later in this paper, (see Appendix A) the term $CV(\gamma_h)$ is introduced, representing the coefficient of variation of the suction compression index. This is an important property of an insitu expansive clay soil and should be included in behavior modeling.

FIELD EXPERIMENT

A field study was planned and conducted to gather data needed to develop design guidelines for airport pavements on expansive soils. Three sites were selected to provide a range of climate and existing airport pavement performance. Those selected were Gallup Airport, New Mexico (GAL); Dallas/Fort Worth Regional Airport, Texas (DFW); and Jackson Airport, Mississippi (JSN). In addition, several existing pavements were surveyed on a one-time basis to develop a data base for performance.

Observations made consisted of the following:

1. suction measurements in the field
2. suction measurements in the laboratory
3. suction compression index determinations
4. active zone depth determinations from suction profiles
5. elevation profiles at 2-ft. intervals over the surfaces of uncovered soil, shoulders, airport pavements and roads.

A total of about six observations were made at approximately two month intervals, these data are fully documented in the FAA final report and are not repeated here. A summary of the results is presented in Table 1. Some new quantities are shown; each is explained here.

Table 1. Summary of Field Study Results

Site:	GAL	DFW	JSN
Property			
$\bar{\gamma}_h$	0.18	0.15	0.14
$CV(\gamma_h)$	0.19	0.33	0.18
z_o (ft)	4	7	12
$\Delta\bar{A}$ (ft/pF)	0.05	0.068	0.075
λ_o	18	26	34
f	1.00	0.55	0.67

The average suction compression index in the soil active zone ($\bar{\gamma}_h$) was determined by numerous tests on samples throughout the soil

active zone. The $CV(\gamma_h)$ is the coefficient of variation of γ_h measured on samples taken within the soil active zone. Because the procedures are simple, a large number of tests is not expensive. A minimum of two samples per foot of depth is recommended. The active zone depth (z_o) is identical to that described previously. The quantities \bar{A} and λ_o are amplitude and wavelength properties of the surfaces studied. The \bar{A} was determined using a Fast Fourier Transform (FFT) to compute amplitude components at discrete frequencies from the space domain elevation data. From these amplitudes a weighted average was calculated which is called \bar{A} . The \bar{A} varies significantly with suction changes in the soil active zone. Based on the data in the field study, the quantity $\Delta\bar{A}$ was defined as the slope of the \bar{A} vs. h_a relationship. Here h_a is the average active zone suction in pF units. The pF is the \log_{10} of the pressure in cm. of water.

Another technique taken from digital signal analysis is the autocorrelation function. The value of the autocorrelation function indicates the degree to which a function repeats itself. The autocorrelation function is initially equal to 1.0 (indicating perfect agreement) at zero lag, and decreases as the lag is increased. The distance at which the autocorrelation function equals zero is called the decorrelation distance. This quantity was selected for use as λ_o . The lateral restraint factor (f) was back calculated from field data using equation (5), see Appendix A. These are the quantities shown in Table 1.

The values for amplitude and wavelength were derived through calculational methods that are not exact. The quantities actually obtained are not truly amplitudes and wavelengths in the sense that a sine wave has an amplitude and a wavelength. They are related to such values, however. It was determined that precise analytical solutions could be developed for these values; however, it seemed more appropriate to seek correlations with field data as a first step.

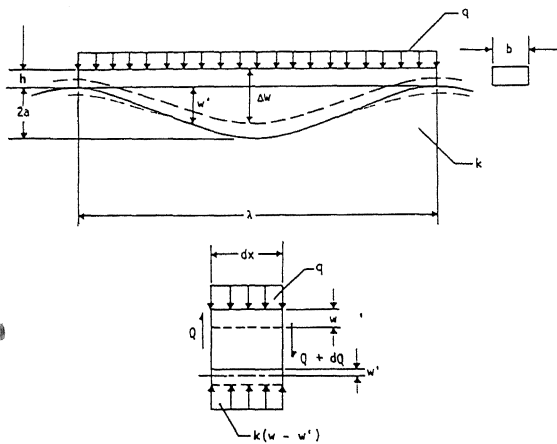
SOIL-PAVEMENT INTERACTION MODEL

Figure 1 illustrates the deformed beam-on-elastic foundation model used to calculate the soil pavement interaction. Figure 2 is the solution to this model. It is completely derived in McKeen (1981) and (1982). The procedure in McKeen (1982) is slightly different from that given earlier. Several parameters are introduced; their explanation follows. The nondimensional quantities $\Delta w/2a$ and $\beta\lambda$ appear in the solution (Figure 2). Referring to Figure 1, note that Δw is the vertical difference in the pavement surface over the distance $\lambda/2$. The quantity $2a$ is the vertical difference of the foundation over the same distance. The quantity measured in the field experiment as \bar{A} is the same as $2a$ in the model, for elevation measurements

on uncovered soil. Similarly the \bar{A} measured on pavement surfaces is equivalent to Δw . The $\beta\lambda$ quantity consists of the beam stiffness (β) and the model wavelength (λ). The stiffness can be expressed as:

$$\beta = \left[\frac{kb}{4EI} \right]^{\frac{1}{4}} = \left[\frac{3k}{Eh^3} \right]^{\frac{1}{4}} = \left[\frac{K}{h^3} \right]^{\frac{1}{4}} \quad (2)$$

All quantities shown are defined in Figure 1 except the K. This is simply included to represent $3k/E$.



- q = unit pressure of beam material
- k = subgrade modulus
- b = width of beam
- a = half-amplitude of deformed soil
- h = thickness of beam
- Δw = deflection of beam
- w' = shape of soil surface
- q = shear
- λ = wavelength
- dx = incremental length

FIGURE 1. BEAM ON DEFORMED ELASTIC FOUNDATION PROBLEM.

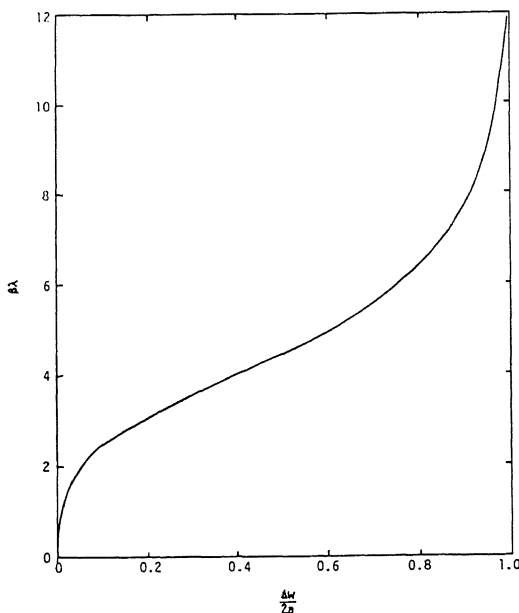


FIGURE 2. SOLUTION TO PAVEMENT MODEL EQUATION.

The model assumes a beam of one material. In order to deal with a single material thickness all pavements studied were converted to an equivalent thickness of a single material with a modulus (E) of 5×10^5 psi. An example of the conversion calculation is illustrated in the Appendix B along with modular values assumed.

APPLICATION TO THE DESIGN PROBLEM

In order to use the model it is necessary to determine what wavelength (λ) to use for design and what amplitude (\bar{A}) is acceptable on the pavement surface. The results of the field study provided the data shown in Figure 3. These data show the equivalent depth of the pavements studied plotted versus ratio of pavement surface wavelength (λ) to uncovered soil wavelength (λ_0). From these data it was concluded that a wavelength of one-half the uncovered soil wavelength would be appropriate as a design wavelength for airport pavements. This is because a given \bar{A} causes greater accelerations for shorter wavelengths, therefore a design wavelength equal to $0.5 \lambda_0$ is a conservative value.

Determination of the acceptable amplitude was accomplished using aircraft simulations. The TAXI, Gerardi (1973) computer code was used to simulate B727-100, B727-200, DC-9-40 and a TriJet composite taxiing at 100 fps on the pavements studied. Using vertical acceleration at the pilot station as a criteria, a line was drawn separating acceptable ($<0.3 g$) from unacceptable performance. Results are illustrated in Figure 4. The acceptable amplitude can then be found from,

$$\bar{A}_a = 0.00153 [\lambda_d]^{1.354} \quad (3)$$

where

\bar{A}_a = acceptable amplitude, ft.

λ_d = wavelength used for design, ft. = $\lambda_0/2$

It was then determined that λ_0 was a function of the active zone depth, z_0 as follows:

$$\lambda_0 = 8.178 [z_0]^{0.579} \quad (4)$$

where

λ_0 = uncovered soil wavelength, ft.

z_0 = active zone depth, ft.

Another result of this study was an expression for the expected soil differential heave or amplitude \bar{A}_e as a function of γ_h , f, z_0 ,

Δh_a and $CV(\gamma_h)$; (see Appendix A):

$$\bar{A}_e = 0.37 \gamma_h f z_o CV(\gamma_h) h a \quad (5)$$

where

\bar{A}_e = expected value of soil amplitude, ft.

Δh_a = change in average suction in the active zone, pF

f = lateral restraint factor

z_o = active zone depth, ft.

γ_h = average suction compression index

$CV(\gamma_h)$ = coefficient of variation of γ_h

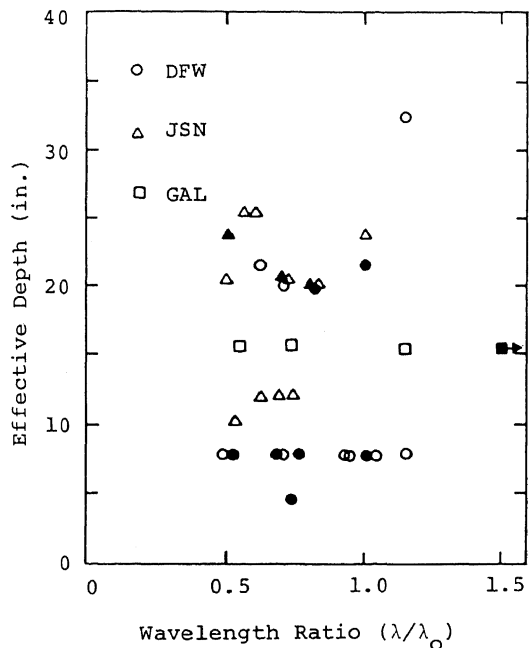


Figure 3. Pavement Depth-Wavelength Relationship

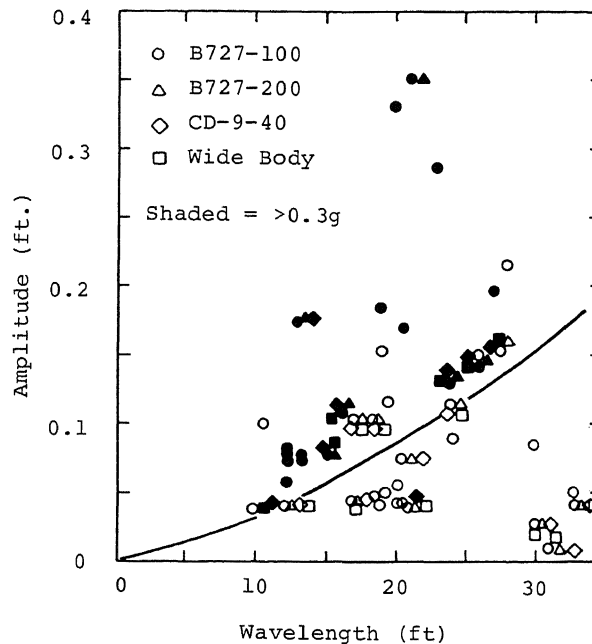


Figure 4. Acceptability Criteria

Table 2. Sample Data from Field Experiment

	calculated values ($h = 2pF$)		
Site	GAL	DFW	JSN
$\bar{\gamma}_h$	0.18	0.15	0.14
z_o	4	7	12
λ_d (ft)	9.1	12.6	17.2
\bar{A}_a (ft)	.0304	.0467	.0720
\bar{A}_e (ft)	0.101	0.135	0.150
$\frac{\Delta w}{2a}$	0.30	0.35	0.48
$\beta\lambda$	3.55	3.81	4.37

In Table 2 data from the field experiment (Table 1) are used to obtain design values using the above equations. The value of $\Delta w/2a$ is computed as \bar{A}_a/\bar{A}_e . Then the value of $\beta\lambda$ is taken from Figure 2. The data required for a thickness selection are now available. A value of $\Delta h = 2pF$ was used for the above computations.

THICKNESS SELECTION

The data in Table 2 show a value of $\beta\lambda$ for each site. If this is divided by λ_d (also in

Table 2) a value of β in $(\text{inches})^{-1}$ is obtained. Recall the expression for β previously presented,

$$\beta = \left[\frac{K}{h^3} \right] \frac{1}{4} \quad (6)$$

The procedure then is to evaluate K by using the pavements studied at various sites. These are shown in Table 3.

Table 3. Comparison to Performance

Pavement	D (in)	$\beta(\text{in}^{-1})$	$K(\text{in}^{-1})$	U or S
GAL 2, 3 (original)	12.2	.0325	0.0020	U
GAL 2, 3 (w/overlay)	15.7	.0325	0.0043	S
DFW 3	32.3	.0252	0.0136	S
DFW 6	7.9	.0252	0.0002	U
NAS 1, 2	21.0	.0252	0.0037	U
FTW	22.0	.0326	0.120	S
JSN 5	20.4	.0212	0.0017	U
JSN 3	23.6	.0212	0.0027	U
JSN 4	12.0	.0212	0.0003	U

Recall that D is pavement equivalent depth, performance is rated as satisfactory (S) or unsatisfactory (U).

The data gathered indicate that $K = 0.004$ is the dividing line between satisfactory and unsatisfactory performance of airport pavements. All data obtained in this study fit this value. With the above procedures the designer can determine a required stiffness, β . It is known that K must be at least 0.004 in.^{-1} , so a minimum h (thickness) can then be obtained. The above equations also permit the use of reduced z_o through stabilization to reduce the pavement thickness required.

CLOSURE

A concept of expansive soil field behavior is presented and equations are developed using both theoretical derivations and empirical observations. As a result, tools are provided with which a designer can check the stiffness of an airport pavement at the sites studied. This will indicate whether the

pavement will develop unacceptable roughness as the soil equilibrates under the pavement structure. This is the thickness required for expansive soil interaction and does not consider load requirements.

A need to evaluate the constants developed from field observation exists. This can be done by determining the data in equation 5, and calculating thickness of pavement required. These results should then be compared to pavements with known performance with respect to expansive clays. Tests and calculations required are detailed in the FAA final report, McKeen (1981).

Additional study is needed to better define the values of K and provide understanding of the compressibility of expansive soils under large covered areas.

ACKNOWLEDGEMENT

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APPENDIX A

Equation for Differential Heave Prediction

Lytton (1977) proposed the following soil behavior model for expansive clays:

$$\frac{\Delta V}{V_i} = \int_0^{z_o} f \left[-\gamma_h \ln \left(\frac{h_f}{h_i} \right) \right] dz + \int_{z_i}^{z_o} f \left[-\gamma_\sigma \ln \left(\frac{\sigma_f}{\sigma_i} \right) \right] dz$$

- where:
- $\Delta V/V_i$ = volume change
 - f = lateral restraint factor
 - ln = logarithm to the base e
 - h_f, h_i = final and initial suction
 - γ_h, γ_σ = compression indexes for volume response to suction and load respectively
 - z, z_i, z_o = various depths below the surface

This may be rewritten,

$$\frac{\Delta V}{V_i} = \int_0^{z_o} f \left[-\gamma_h 2.3026 r(z_o - z) \right] dz +$$

$$\int_{z_i}^{z_o} f \left[-\gamma_{\sigma} 2.3026 \left(\frac{z}{z_i} - 1 \right) \right] dz$$

where: $r = \Delta h / z_o$

$\Delta h =$ suction change at the surface in pF units

The variance of volume change may be written,

$$\begin{aligned} \text{Var}(\Delta) &= \left(\frac{\partial \Delta}{\partial \gamma_h} \right)^2 \text{Var}(\gamma_h) + \left(\frac{\partial \Delta}{\partial \Delta h} \right)^2 \text{Var}(\Delta h) \\ &+ \left(\frac{\partial \Delta}{\partial z_o} \right)^2 \text{Var}(z_o) + \left(\frac{\partial \Delta}{\partial \gamma_{\sigma}} \right)^2 \text{Var}(\gamma_{\sigma}) \end{aligned}$$

By taking appropriate derivatives, dividing by $\frac{1}{4} r^2 z_o^2$, then taking the square root, we obtain the following

$$\left[\frac{\text{Var}(\Delta)}{r^2 z_o^2} \right]^{\frac{1}{2}} = \frac{\text{root mean squared amplitude change}}{\text{mean suction change (pF units)}} = \Delta \bar{A}$$

which is:

$$\begin{aligned} \Delta \bar{A} = \frac{\bar{A}}{\Delta h} &= (-2.3026 \gamma_h f z_o) \text{CV}(\gamma_h) + \\ &(-2.3026 \gamma_h f z_o - \frac{\gamma_{\sigma} f z_o}{r} \left[\frac{1}{z_i} - \frac{z}{z_o} + \frac{z_i}{z_o^2} \right]) \text{CV}(f) \\ &+ (-2.3026 \gamma_h f z_o) \text{CV}(r) \\ &+ (-2.3026 \gamma_h f z_o - \frac{2\gamma_{\sigma} f z_o}{r} \left[\frac{1}{z_i} - \frac{1}{z_o} \right]) \text{CV}(z_o) \\ &+ \left(-\frac{\gamma_{\sigma} f z_o}{r} \left[\frac{1}{z_o} - \frac{z}{z_o} + \frac{z_i}{z_o^2} \right] \right) \text{CV}(z_o) \end{aligned}$$

The data obtained in the field study provided some observations which may be used to evaluate this expression. The γ_{σ} terms were dropped because there was little influence on the field experiment results from load changes.

If the above expression is written,

$$\frac{\bar{A} / \Delta h}{\gamma_h z_o \text{CV}(\gamma_h)} = C_f$$

where C = a constant.

From the data in Table 1 values of C_f are calculated and shown below. Assuming the value of $f = 1.00$ at GAL, the value of C may be

determined followed by the value of f for DFW and JSN.

Site	C_f	f	C
GAL	0.37	1.00	0.37
DFW	0.20	0.53	0.37
JSN	0.25	0.67	0.37

The final equation for differential heave is,

$$\bar{A} = 0.37 \gamma_h f z_o \Delta h \text{CV}(\gamma_h)$$

APPENDIX B

Pavement Equivalent Depth Calculations

Layer	Thick-ness	Material	Modulus	Modular Ratio
1	10 in.	PCC	4×10^6 psi	8.
2	6 in.	Cement-Treated Base	8×10^5 psi	1.6
3	6 in.	Lime-Stabilized Soil	3×10^3 psi	0.006

Reference modulus $E_o = 5 \times 10^5$ psi

$$\bar{y} = \frac{\left(\frac{E_1}{E_o} \right) h_1 y_1 + \left(\frac{E_2}{E_o} \right) h_2 y_2 + \left(\frac{E_3}{E_o} \right) h_3 y_3}{\left(\frac{E_1}{E_o} \right) h_1 + \left(\frac{E_2}{E_o} \right) h_2 + \left(\frac{E_3}{E_o} \right) h_3}$$

$$I_{x_1}' = I_{x_1} + A_1 b_1^2 \quad A_i = \left(\frac{E_i}{E_o} \right) h_i$$

$$I_{x_2}' = I_{x_2} + A_2 b_2^2 \quad b_i = (y_i - \bar{y})$$

$$I_{x_3}' = I_{x_3} + A_3 b_3^2 \quad I_{x_i} = \left(\frac{E_i}{E_o} \right) \frac{(h_i)^3}{12}$$

$$I_x = \sum_{i=1}^{i=3} I_{x_i}$$

$$D = \left[\frac{12 I_x}{1. \text{ in}} \right]^{\frac{1}{3}}$$

where:

$\bar{y} =$ distance from bottom of section to neutral axis

$E_o, E_1, E_2, E_3 =$ modulus values for the materials

y_i = distance from bottom of section to centroid of the i^{th} layer

h_i = thickness of the i^{th} layer

I_x = moment of inertia of the section about its centroid axis

D = equivalent depth of the pavement in thickness of the reference material, E_o .

Modular values assumed:

<u>Material</u>	<u>Modulus, E (psi)</u>
Portland Cement Concrete	4×10^6
Asphalt Concrete	5×10^5
Cement-Treated Base	2×10^5
Lime-Stabilized Soil	3×10^3
Sand-Clay Fill	1×10^3
Asphalt-Stabilized Base	2×10^5

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