Climatic Materials Characterization of Fine-Grained Soils

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Characterization of the performance-related characteristics of fine-grained subgrade soils depends largely on the moisture and temperature regime in which they are found. This paper presents the results of a repeated-load testing program designed to produce some important materials properties for a variety of fine-grained subgrade soils. The materials properties characterized are compatible with a newly developed Federal Highway Administration computerized stress and distress analysis system, VESYS II, which represents flexible pavements as linearly viscoelastic layered media. Such characterization has not been done in previous studies. The materials characteristics presented in this study are resilient modulus, residual strain, and permanent deformation characteristics, all of which, as expected, vary with the number of load repetitions. The results, taken from repetitive load tests on three different soils having a range of clay contents of 20 to 70 percent, include the relation of the materials characteristics to mean stress, deviator stress, soil suction, clay content, and temperature. Because the equilibrium suction value of a subgrade soil beneath a pavement is related to the climatically controlled Thornthwaite Moisture Index, it is possible to infer under what conditions and in which parts of the United States special design considerations will be required for pavement structures that rest on the tested soils.

The performance of highway pavements is controlled by the traffic loading, the climatic conditions, and the mechanical properties of the materials in the pavement layers. The properties of subgrade soils that are most important in predicting the performance of pavements under load are its creep compliance and permanent deformation characteristics. These two properties are used as input to the VESYS II pavement-analysis system of the Federal Highway Administration (FHWA) (1, 2, 3). This paper presents the results of a repeated-load testing program at Texas A&M University that was designed to produce some of these important material properties for a variety of fine-grained subgrade soils. The experimental design of the testing program included various levels of temperature, soil suction, stress intensity, clay content, and load repetitions. The properties that were measured included the resilient modulus and the permanent strain properties as currently used in the VESYS II computer program.

Climate influences material properties by changing the temperature and the availability of moisture. This availability is measured by a climatic moisture index that indicates the relative balance between water entering the soil as rainfall and water leaving the soil as either evaporation or transpiration through plants. Russam and Coleman (4) found a reliable correlation between the Thornthwaite Moisture Index and the soil suction, which was measured at depths remote from the seasonal influences of moisture and temperature variations. That relation (Figure 1) shows that in any given climate the suction that is expected to develop beneath a pavement depends on the amount of fines present in the soil. Suction and temperature are thus measurable quantities that are directly related to the local climate and are important variables in the climatic design of pavements.

DEFINITIONS

Several of the terms used in this paper are defined below.

1. Resilient deformation or recoverable deformation is that portion of the total deformation that is recovered after the load is removed.
2. Residual deformation or plastic deformation is that portion of the total deformation that is not recovered before the next load application.
3. Resilient strain or elastic strain (ϵ_r) is the ratio of the resilient deformation to the sample length.
4. Residual strain or plastic strain (ϵ_p) is the ratio of the residual deformation to the sample length.
5. Resilient modulus (M_R) is the ratio of the deviator stress to resilient strain. The resilient modulus is analogous to the elastic modulus in static testing.
6. Soil suction is the energy with which water is attracted to soil and is measured by the work required to move this water from its existing state to a pressure-free, distilled state.

The total suction can be determined by measuring the vapor pressure in equilibrium with the soil water. The total suction can be quantitatively defined by the Kelvin equation, which expresses the total suction (h) in gram·centimeter/gram of water vapor (centimeters of water):

$$h = (RT/gm) \log_e (P/P_o) \tag{1}$$

where

R = gas constant [8.314 J/°C mole (83 million ergs/°C mole)],
T = absolute temperature (°C),
g = gravitational force [981 cm/s^2 (0.39 in/s^2)],
m = molecular weight of water [18.02 g/mole (0.63 oz/mole)],
P = vapor pressure of soil water,
P_o = vapor pressure of free water, and
P/P_o = relative humidity (or relative vapor pressure).

Thus, the total suction is directly related to the relative humidity of the soil. Because the relative humidity is always 1.0 or less, its logarithm is always zero or negative and thus h is always negative. Consequently, the higher the relative humidity is, the more moisture the sample contains and the smaller the absolute value of the suction will be.

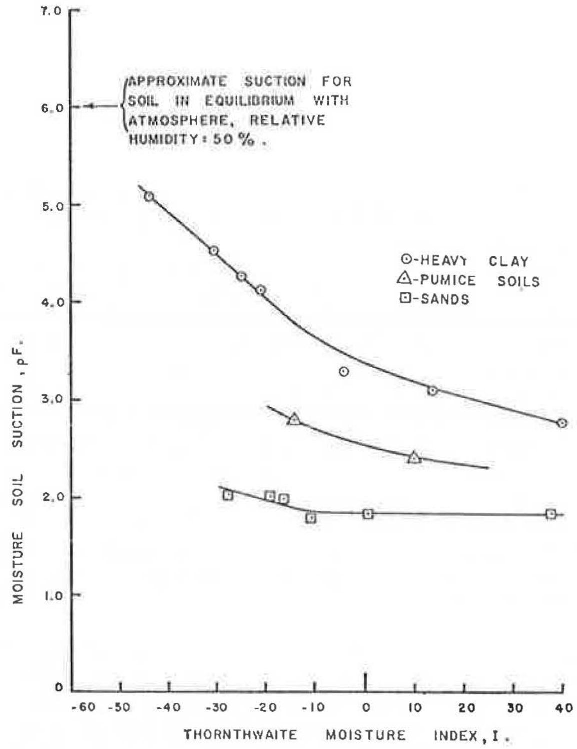
Although soil suction is defined as a negative quantity, its absolute value or positive magnitude is normally used for ease of discussion. Thus, a soil suction of -980 kPa (-142 lbf/in^2) is referred to as a suction of 980 kPa (142 lbf/in^2).

MATERIALS AND TEST EQUIPMENT

Materials

The three soils used in this test program are classified as CH, CL, and ML. For ease in identifying the different soils, each soil is named for the town near which it was obtained. The CH soil was obtained from Moscow, Texas, and consists of dark gray, plastic clay that has a high shrink-swell potential. The CL soil was obtained from Floydada, Texas, and consists of fine, textured clay with alkaline sediments from the high plains. The ML soil, which was obtained from Allenfarm, Texas, con-

Figure 1. Subgrade soil suction versus Thornthwaite Moisture Index.



sists of reddish, calcareous soils that make up the flood plains of the Brazos River; in this soil, therefore, a small percentage of clay is mixed with a large amount of silt. The physical properties of the three soils are given in the table below (1 kPa = 0.145 lbf/in^2).

Property	Soil Moscow	Floydada	Allenfarm
Liquid limit, %	83	30	27
Plastic index, %	55	13	0
Shrinkage limit, %	14	14	23
Optimum moisture content			
(Harvard miniature, 138 kPa), %	31.5	18	16
Soil classification			
AASHO	A-7-6(57)	A-6(7)	A-4(0)
Unified	CH	CL	ML
Specific gravity	2.69	2.70	2.72
Thornthwaite index	+21	-17	0
Percent passing No. 200 sieve	91	71	72
Clay (2 μ), %	70	39	20

The distribution of the soil less than 0.2 mm (0.008 in) was determined by using a hydrometer analysis in accordance with ASTM D422-61T (5). The Moscow, Floydada, and Allenfarm soils have clay percentages of 70, 39, and 20 percent respectively.

Although soil density and soil structure are important factors in determining the dynamic properties of soils, they were not controlled in this study. Johnson and Sallberg (6) report that the kneading compaction method best represents the soil structure obtained in the field. For this reason, Harvard miniature samples were made by using the kneading compaction method with a compressive stress of 138 kPa (20 lbf/in^2), which approximates a compaction effort that produces 97 percent of AASHO T 180 density.

Test Equipment

In this program, the following were to be measured: (a) soil suction before, during, and after testing; (b) vertical deformation, both permanent and recoverable, at any time during the test; and (c) magnitude of the applied vertical load. The repetitive loading apparatus used in the study is a pneumatically operated testing machine that applies an axial load to a standard triaxial cell. The axial pressure pulse consists of 0.2 s with the load applied and 1.8 s with the load off. This frequency corresponds to a highway speed of 72.4 km/h (45 mph). A psychrometer was chosen to measure the suction of the soil samples because this instrument has a large range and could be incorporated into an end cap, thus measuring the soil suction during the test. By using the dew-point method of measurement, temperature corrections could be made easily; the results are accurate to within ±5 percent. A pair of induction coils were used to measure the axial deflections by measuring the change in the magnetic field caused by a change in the spacing of the coils. The accuracy of the deflection measurements is ±0.0127 mm (±0.0005 in).

Test Matrix

The following table gives the variables considered in the testing program [1 kPa = 0.145 lbf/in^2 and 1°C = (1°F - 32)/1.8]:

Variable	High	Medium	Low
Soil suction, kPa			
Moscow (CH)	1725	759	414
Floydada (CL)	1035	345	138
Allenfarm (ML)	966	193	69
Temperature, °C	0	22.2	38
Stress condition			
Axial pressure (σ_1), kPa	118.7	241.5	172.5
Confining pressure (σ_3), kPa	24.1	138	103.5
Mean stress (σ_m), kPa	55.9	172.5	126.3
Stress ratio [$(\sigma_1 - \sigma_3)/\sigma_m$]	1.69	0.60	0.55
Clay fraction (-2 μ)	0.70	0.39	0.20

Different suction levels were used on each soil, and the higher suctions were used on the soils having higher clay content. The midlevel of suction was set at about optimum moisture content.

Method of Analysis

The equations presented in this paper were developed by using the two-step regression method of the SELECT computer program (7, 8). The first step is a linear regression on the logarithms of the dependent and independent variables. When the antilog is taken, the coefficients obtained in this first step become powers of the independent variables. The second step is a linear regression of the independent variables raised to the powers determined in the first step. With this method, there is no preset power law or polynomial form of the equation, and experience has shown that this method consistently produces higher coefficients of determination (R^2).

RESILIENT MODULUS

Although the resilient modulus is not used in the VESYS II computer programs, it is used in other pavement systems analysis programs such as FPS-BISTRO (9) and PDMAP (10); it is included here to show how subgrade stiffness depends on the climatically controlled variables of temperature and suction. The equation developed in this study for the resilient modulus (M_R) at a constant

Table 1. Constants for resilient-modulus equation.

Constant	Soil Moscow (CH)	Floydada (CL)	Allenfarm (ML)
b	0.084	0.145	0.081
c	3.6	3.3	1.4
d	-0.60	-0.60	-0.16
e	3.6	2.0	-0.26
f	-0.27	-0.23	0.063
g	-3.3	-2.25	-0.30
a_0	-4791.99	7980.89	-1827.72
a_1	-27 272.4	2981.64	171 705.0
a_2	-45.0169	64.397	0.6566
a_3	-3.733	-4.2008	-4.4849
a_4	1.706×10^{-7}	-2.002×10^{-3}	64.6522
a_5	-5.0763	-3.7228	-1.6108
a_6	-0.1288	-0.1639	-0.001 155
a_7	0.059 99	-0.1974	-14.8816
a_8	-5.8416	-4.2766	-1.5899
R^2	0.534	0.453	0.766
Standard error, kPa	45 000	27 973	10 771

Note: 1 kPa = 0.145 lbf/in².

temperature of 22.2°C (72°F) is as follows:

$$M_R = a_0 + a_1 [(h_f/h_i)^{0.20} N^b] \{1 + a_2 (1 - n)^c [1 + a_3 (\sigma_1 - \sigma_3)^d] + a_4 (S)^e [1 + a_5 (\sigma_1 - \sigma_3)^d + a_6 \sigma_m^f] + a_7 (nS)^g [1 + a_8 (\sigma_1 - \sigma_3)^d]\} \quad (2)$$

where

h_f = final suction (kPa × 0.145);
h_i = initial suction (kPa × 0.145);
N = number of load cycles;
(1 - n) = volumetric soil content in decimal form;
$(\sigma_1 - \sigma_3)$ = deviator stress (kPa × 0.145);
S = saturation (percent);
σ_m = mean stress (kPa × 0.145); and
nS = volumetric moisture content in decimal form.

The constants for this equation are given in Table 1.

The final suction and the number of load cycles are directly related to the resilient modulus whereas the deviator stress is inversely related. The power of the suction ratio is constant in all the equations at about 0.20; the power of the number of load cycles varies between 0.081 and 0.145. The power of the deviator stress is negative and becomes smaller when the clay content decreases below 40 percent. The suction and the number of load cycles have a larger influence on the resilient modulus than does the deviator stress.

Several general observations can be made from the three equations. As the clay content decreases, the power of the volumetric soil content, the volumetric moisture content, and the saturation decreases. However, the decrease is not linear with decreasing clay content. The single most important term in the equations is the number of load cycles. As the clay content decreases, the equations are less dependent on the soil suction and more dependent on the volumetric moisture properties. From a comparison of the coefficients of determination it appears that the resilient modulus becomes more predictable with lower clay contents.

A temperature correction must be made to this equation for temperatures other than 22.2° C (72° F). The resilient modulus of Equation 2 is multiplied by the following correction factor:

$$f_{M_R} = a_0 - a_1 (D/D_0)^b + a_2 (h/h_0)^c + a_3 (T/T_0)^d \times \{1 - a_4 (h/h_0)^c (D/D_0)^b + a_5 (N/N_0)^e [1 - a_6 (h/h_0)^c + a_8 (h/h_0)^c (D/D_0)^b - a_7 (D/D_0)^b]\} \quad (3)$$

where

(D/D_0) = deviator-stress ratio [D_0 = 94.6 kPa (13.7 lbf/in²)];
(h/h_0) = soil-suction ratio [h_0 = 759, 345, and 193 kPa (110, 50, and 28 lbf/in²) for CH, CL, and ML soils respectively];
(T/T_0) = temperature ratio [T_0 = 22.2° C (72° F)];
(N/N_0) = ratio of number of load cycles (N_0 = 10 000);
b = -1.7013 + 6.2014(PL);
c = 0.0271 - 0.2873 log (clay);
d = 0.0697 - 0.9846 (clay);
e = 0.0582 - 0.002 26 (clay);
a_0 = -125.574(SL) - 2764.13(PL) + 21 234.1 (SL × PL);
a_1 = -465.052(SL) - 2890.01(PL) + 23 642.5 (SL × PL);
a_2 = -37.6644 + 279.813 (SL + PL)²;
a_3 = -15.0184 + 13 786.434 (SL × PL)²;
a_4 = 0.8088 + 0.3006 (clay);
a_5 = 30.8763 - 306.7167(LL)²;
a_6 = 7.5058(SL) - 6.0135(PL) + 41.1548 (SL × PL);
a_7 = 3.6476(PL) + 2.0336(LL) - 7.3402 (PL × LL);
a_8 = 4.370(SL) - 6.1516(PL) + 53.4137 (SL × PL);
LL = liquid limit;
PL = plastic limit;
SL = shrinkage limit; and
clay = clay content in decimal form.

The coefficients of determination for the power relations are all above 0.90. As with the deviator-stress ratio, the soil suction and the number of load-cycle-ratio changes are directly related to the changes in resilient modulus. Whereas the powers were generally related to the clay content, the coefficients are generally related to the Atterberg limits. The coefficients of determination for the coefficient relations are all above 0.90.

STRAIN RELATIONS

A fundamental change occurs in the behavior of fine-grained soils at a water content about 2 percent dry of optimum, as determined by the Harvard miniature compaction procedure. When the soil is wetter than this amount its properties change dramatically with a small change in water content. When the soil is drier than this amount its properties are much less dependent on water content and much more dependent on the level of suction in the soil. The soil suction at this threshold is related to the clay fraction in the soil by the following equation:

$$h = 21.481 + 181.14(c) \quad (4)$$

where

h = soil suction in kPa (1 kPa = 0.145 lbf/in²) and
c = clay fraction in the soil in decimal form.

This value of suction is henceforth referred to as the threshold suction.

Analysis of Strain Data

The residual strain was analyzed as an exponential function of the number of load repetitions (N):

$$\epsilon_p = I\, N^S \quad (5)$$

The constants (I) and (S) in this equation were then related to ratios of the other independent variables in the test series. The values of these independent variables are given in the following table [1 kPa = 0.145 lbf/in² and 1°C = (1°F - 32)/1.8]. (The strain equation was developed by using customary unit values.)

Variable	Value	Variable	Value
Soil suction (h_0), kPa		Temperature (T_0), °C	22.2
Moscow (CH)	759	Mean stress (m_0), kPa	55.9
Floydada (CL)	345	Stress ratio (s_{r0})	1.69
Allenfarm (ML)	193	Clay fraction (c_0)	0.4

Residual Strain

The large range of residual strain that occurred during the first few load cycles is attributed to seating error because the samples were not preloaded. To compensate for this the residual strain was set at zero at the one-hundredth load repetition. The differences in the residual strain are thus due to differences in the samples and not to differences caused by the seating error.

The constants (I) and (S) in the residual strain equation are expressed as follows:

$$I = -6.017 \times 10^{-4} + 3.28 \times 10^{-4}(h/h_0)^{-1.15} + 3.22 \times 10^{-5}(m/m_0)^{1.22} + 9.88 \times 10^{-6} \times (T/T_0)^{-4.30} + 1.13 \times 10^{-4}(c/c_0)^{-2.0} \quad (R^2 = 0.45) \tag{6}$$

$$S = -1.553 + 0.451(h/h_0)^{0.15} + 0.597(m/m_0)^{-0.275} + 0.737(T/T_0)^{0.87} + 0.442(c/c_0)^{0.33} \quad (R^2 = 0.39) \tag{7}$$

where

(h/h_0) = soil-suction ratio,
(m/m_0) = mean-stress ratio,
(T/T_0) = temperature ratio, and
(c/c_0) = clay-content ratio.

The intercept or I equation produces numbers around 10^{-4} to 10^{-7}. The intercept varies inversely with the soil-suction ratio, the temperature ratio, and the clay-content ratio but directly with the mean-stress ratio.

The residual strain increases as the number of load cycles increases; the slope or S equation will therefore be positive. The range of the slope is 0.1 to 1.4. In this equation the slope (S) varies directly with the suction ratio, the temperature ratio, and the clay-content ratio and inversely with the mean-stress ratio.

PERMANENT DEFORMATION RELATION

In predicting the rutting behavior of pavements the VESYS II computer program calculates the fractional increase of permanent strain that develops with each load that passes. The clearest explanation of this calculation appears to be that by Rauhut, O'Quin, and Hudson (11); the following has been extracted from their discussion.

The residual strain equation is $\epsilon_p = I\ N^S$ (Equation 5). The change of residual strain with each load is

$$d(\epsilon_p)/dN = IS\ N^{S-1} = \Delta\epsilon_p \tag{8}$$

The total strain at a given load cycle is the sum of the resilient strain and the incremental increase of permanent strain given above. Consequently, the fraction of the total strain that becomes permanent strain with each load cycle is

$$F(N) = \Delta\epsilon_p/(\epsilon_r + \Delta\epsilon_p) \approx ISN^{S-1}/\epsilon_r \tag{9}$$

The function [F(N)] must be specified in the input to VESYS II as

$$F(N) = \mu\ N^{-\alpha} \tag{10}$$

from which it is found that

$$\alpha = 1 - S \tag{11}$$

and

$$\mu \cong IS/\epsilon_r \tag{12}$$

both of which are related to the constants (I) and (S) in Equations 6 and 7.

Figures 2 and 3 show the dependence of μ and α respectively on soil suction and stress ratio at a constant temperature and number of load repetitions for the Allenfarm soil (ML). These figures show the striking change of behavior that occurs at the threshold suction value. The Allenfarm soil, a low-plastic silt, is similar to the subgrade soil of the AASHO Road Test. The values of μ and α to the wet side of the threshold suction are in the same range as those calculated from AASHO Road Test data by Rauhut, O'Quin, and Hudson (11) in which α varied from 0.63 to 1.0 and μ was around 0.1. For the other two soils tested, the peaks were not so sharp, the levels of μ were generally lower (0.001 to 0.01), and the levels of α were similarly lower (0.3 to 0.7).

The μ and α relations show several general characteristics. As the number of load repetitions increases, μ increases slightly and α remains nearly constant. Temperature is another important influence: As temperature increases both μ and α decrease and vice versa. Both of these soil properties are strongly dependent on temperature; α actually becomes negative for soils with higher clay contents, as shown in Figure 4. A negative value of α indicates a progressively increasing permanent strain with each additional load cycle.

IMPLICATIONS FOR CLIMATIC DESIGN OF PAVEMENTS

In addition to the more detailed effects to be derived from pavement analysis, at least two broad-scale climatic effects are implied by the results of this study. These effects concern (a) the threshold suction and (b) the dependence of α on temperature.

Threshold Suction

Because the Thornthwaite Moisture Index is related to the suction level in subgrade soils, it is possible to draw a climatic map of the United States (Figure 5) that delineates those regions where ML, CL, and CH soils must either be treated or protected by extra pavement thickness or stiffness from the likelihood of excessive rutting. The shaded areas in Figure 5 represent climatic areas where subgrade suction can be expected to be less than the threshold suction value. As the figure shows, silty soils will require special pavement-design considerations throughout most of the United States.

Negative α

When α becomes negative in a soil, that soil will suffer progressively increasing permanent strain ($\Delta\epsilon_p$) with each passing load. No part of the United States has temperatures higher than 35°C (95°F) during substantial portions of the year. The southern states, however, can have subgrades of high clay content whose temperatures range above this value for brief periods of time each day, usually toward evening. If this higher temperature period coincides with a period of heavy traffic, accelerated pavement rutting may be expected. In these instances, a thick pavement overlying the subgrade will (a) reduce the traffic stresses in the subgrade and (b) provide thermal insulation to keep the subgrade from reaching a temperature at which α will become negative.

CONCLUSIONS

The following conclusions can be drawn from the results of this study.

1. The dynamic properties of fine-grained subgrade soil depend strongly on factors of traffic, climate, and

Figure 2. μ versus soil suction and stress ratio at 10 000 load repetitions and 22.2°C (Allenfarm soil).

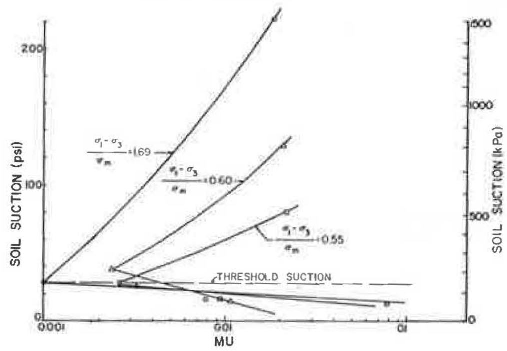


Figure 3. α versus soil suction and stress ratio at 10 000 load repetitions and 22.2°C (Allenfarm soil).

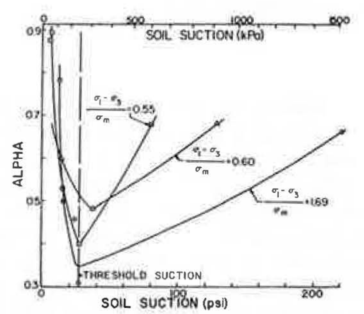


Figure 4. α and μ versus temperature and stress ratio for threshold suction at 10 000 load repetitions (Moscow soil).

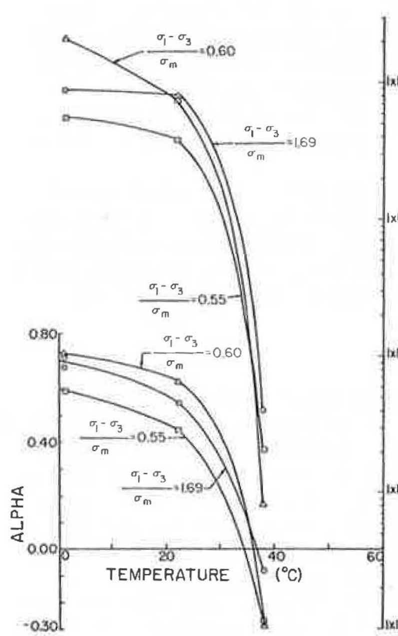
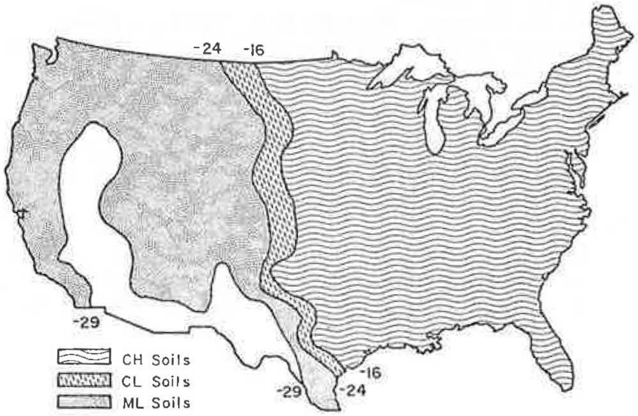


Figure 5. Climatic map corresponding to Thornthwaite Moisture Index.



soil composition. The important climatically related factors are soil suction and temperature.

2. Resilient moduli, resilient strain, and residual strain may be predicted reliably from these factors.

3. Equations have been developed that relate stress, suction, temperature, and clay content to the permanent deformation factors (μ) and (α) currently used in the VESYS II pavement-analysis computer program of FHWA.

4. Climatic design of pavements will recognize the importance of the threshold suction value. Subgrades that have expected values of suction lower than the threshold value will require special consideration in terms of added thickness or stiffness or stabilization.

5. A negative α can develop under certain thermal conditions. At a temperature above 35°C (95°F), a subgrade soil that has a high clay content can suffer a progressively increasing permanent strain with each load cycle. Thus, thick pavements in the warmer climates have two functions: reduction of traffic stress and thermal insulation.

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REFERENCES

1. F. Moavenzadeh, J. E. Soussou, H. K. Findakly, and B. Brademeyer. Synthesis for Rational Design of Flexible Pavements—Operating Instructions and Program Documentation. Department of Civil Engineering, MIT, Cambridge, Res. Rept. R74-26, Part 3, Feb. 1974.
2. VESYS Users Manual. Office of Research, Federal Highway Administration, U.S. Department of Transportation, March 1976.
3. W. L. Hufferd and J. S. Lai. VESYS II Computer Program and Documentation. Univ. of Utah, in preparation.
4. K. Russam and J. D. Coleman. The Effect of Climatic Factors on Subgrade Moisture Conditions. Geotechnique, Vol. 11, 1961, pp. 1-22.
5. Grain-Size Analysis of Soils. ASTM Standards, Part 4, 1961, pp. 1272-1283.
6. A. W. Johnson and J. R. Sallberg. Factors That

Influence Compaction of Soils. HRB, Bulletin 272, 1960, pp. 29-48.
7. D. A. Debuse. Variable Selection Procedure, Implementing the Hocking-LaMottee-Leslie Method. Institute of Statistics, Texas A&M Univ., Nov. 1970.
8. D. Y. Lu, R. L. Lytton, and W. M. Moore. Forecasting Serviceability Loss of Flexible Pavements. Texas Transportation Institute, Texas A&M Univ., Res. Rept. 57-1F, Nov. 1974.
9. D. Y. Lu, C. S. Shih, and F. H. Scrivner. The Optimization of a Flexible Pavement System Using Linear Elasticity. Texas Transportation Institute, Texas A&M Univ., Res. Rept. 123-17, March 1973.
10. Woodward-Clyde Consultants. Users Manual for Computer Program PDMAP. NCHRP, Project 1-10B, Aug. 1975.
11. J. B. Rauhut, J. C. O'Quin, and W. R. Hudson. Sensitivity Analysis of FHWA Model VESYS II. Austin Research Engineers, Austin, Tex., and Federal Highway Administration, U.S. Department of Transportation, Repts. FHWA-RD-76-23 and FHWA-RD-76-24, March 1976.

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Desiccation of Soils Derived From Volcanic Ash

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Five volcanic-ash-derived soils from the island of Hawaii were studied to determine the relation of changing moisture content to engineering behavior. Two soils weathered on the leeward side of the island under relatively dry conditions; the other three developed on the windward side under high mean annual rainfalls. The dry soils show little change on desiccation whereas the desiccation of the wet soils is accompanied by an irreversible hardening that causes drastic changes in the index properties. Engineering behavior of the dry soils is similar to that of a sandy silt. The wet soils behave as plastic clays but, when they are dried out, their engineering characteristics change to those of a sand. Mercury porosimetry tests reveal that, in wet soils, volume changes between field moisture content and the oven-dry state are about 150 percent. The dry soils exhibit very small volumetric shrinkage. Drying tests under controlled relative humidity provide data on drying rates and critical moisture contents. The mineralogy of the soils was studied by using X-ray diffraction and fluorescence, differential thermal, and thermogravimetric analyses. The predominant minerals are gibbsite, iron oxides, and allophane. Mineralogical studies indicate that irreversible hardening is accompanied by an increase in gibbsite content. Mercury porosimetry results and mineralogical analysis indicate that a major portion of the shrinkage is due to contraction of the intermediate-size pores and that the extent of shrinkage is a function of allophane content.

The behavior of some soils derived from volcanic ash in very wet tropical areas has been of considerable interest to pedologists and soil engineers. Highway-construction experience with these soils in Hawaii has been discussed by Hirashima (7, 8), who pointed out problems associated with compaction at very high moisture contents. In general, ash soils are characterized by low density, high permeability, and high water-holding capacity (4, 7, 11, 15). Birrell (4), who worked with ash-derived soils from New Zealand, has reported that the preconsolidation pressure of the soils as determined in conventional consolidation tests is frequently greater than the overburden pressure but that once this preconsolidation pressure is exceeded the soils are highly compressible. Yamanouchi (16) has pointed out that the ash-derived soils of Japan are difficult to stabilize with conventional additives if the soils are high in organic content.

One of the most interesting properties of these soils is that they harden irreversibly when dried. This hardening is often accompanied by shrinkage. At field moisture contents the ash soils are plastic and claylike; if they are allowed to dry, however, sand- or silt-size aggregates form. Drying reduces liquid limits and decreases clay contents in these soils (3, 6, 7). If the soils are allowed to dry partially at relatively high moisture contents, they retain their plastic nature. No data were found to indicate the moisture content at which an ash-derived soil loses its plastic character. The component of ash soils that is responsible for irreversible hardening and for some other unique properties has been referred to variously as palagonite (7), amorphous clay (15), amorphous colloids (4), and allophane (6). Few quantitative data on the mineralogy of ash-derived soils are to be found in the literature. The confusion in terminology and the lack of quantitative data arise from the vague definition of the poorly crystalline components in these soils and from the resultant difficulties in making measurements.

The data presented in this paper may help to provide a better understanding of the desiccation process in ash soils and thus contribute to evaluating their potential for chemical stabilization, classifying them for engineering purposes, and predicting their behavior in the field.

SOILS STUDIED

Three ash-derived soils known to exhibit irreversible hardening when dried are the Kukaiau, Honokaa, and Hilo series from the windward side of the island of Hawaii. These soils were weathered from a volcanic ash known as the Pahala ash under very high mean annual rainfalls ranging from 178 to 457 cm. As a basis for comparison, two other soils were studied that weathered from the same parent material but do not exhibit irreversible hardening. These two soils, Waimea and Kilohana, developed under mean annual rainfalls ranging from 51 to