TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA/TX-89/473-2		

4. Title and Subtitle	5. Report Date
Effects of Temperature and Moisture on Low-Volume Roads	December 1988
	6. Performing Organization Code
7. Author(s)	8. Performing Organization Report No.
Djan Chandra, Robert L. Lytton, and Weishih Yang	Research Report No. 473-2
9. Performing Organization Name and Address	10. Work Unit No.
Texas Transportation Institute Texas A&M University System College Station, Texas 77843 - 3135	11. Contract or Grant No. Study No. 2-18-87-473
	13. Type of Report and Period Covered
12. Sponsoring Agency Name and Address Texas State Department of Highways and Public Transportation; Transportation Planning Division P. O. Box 5051 Austin, Texas 78763	Interim - September 1986 December 1988
	14. Sponsoring Agency Code

15. Supplementary Notes

Research performance in cooperation with DOT, FHWA. Research Title: Investigation of the Effects of Raising Legal Load Limits to 80,000 lbs. on Farm-to-Market Roads

16. Abstract This report presents a thorough study on the effects of temperature and moisture on the load response of low-volume roads. The procedures developed herein can be used to formulate temperature and seasonal adjustment factors for low-volume roads and to estimate when and where seasonal load restrictions should be applied.

Field studies were performed on two light pavement structures at the Texas Transportation Institute Research Annex. The tests consisted of taking pavement layer temperatures and Falling Weight Deflectometer deflection readings at hourly intervals throughout the day. In addition, six Farm-to-Market roads in different regions of the state of Texas were monitored monthly to evaluate temperature and moisture effects on pavement moduli. Falling Weight Deflectometer deflection readings, rainfall data, pavement temperature, and suction readings were collected over a twelve month period. The effects of rainfall on the moisture condition beneath a pavement were also examined. The deflection readings were then used to backcalculate the layer moduli of the pavements. It was found that the stiffness of low-volume roads increases with the increase of temperature and decrease of moisture. Another aspect of temperature effects on granular materials was examined from the point of view of stress relaxation. Laboratory testings in the form of relaxation tests at different temperatures were performed on a granular material in order to determine the time and temperature dependent properties of the material.

The analytical models were developed based on a micromechanical approach. The granular base course materials were treated as elastic spheres in contact and subjected to temperature and moisture changes. Comparisons between the backcalculated and the predicted pavement layer moduli were made to verify the models.

17. Key Words	18. Distribution Statement
Temperature, Moisture, Micromechanics, Low-Volume Roads	No restriction. This document is available to the public through National Technical Information Service, 5785 Port Royal Road, Springfield, Virginia 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	102	

Form DOT F 1700.7 (8-69)

EFFECTS OF TEMPERATURE AND MOISTURE ON LOW-VOLUME ROADS

by

DJAN CHANDRA, ROBERT L. LYTTON, and WEISHIH YANG

Research Report 473-2

Investigation of the Effects of Raising Legal
Load Limits to 80,000 lbs on Farm-to-Market Roads

Research Study 2-18-87-473

Conducted for

Texas State Department of Highways and Public Transportation

in cooperation with the
U.S. Department of Transportation
Federal Highway Administration

by the

Texas Transportation Institute
The Texas A&M University System
College Station, Texas

December 1988

METRIC (SI*) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol

LENGTH

in	inches	2.54	millimetres	mm
ft	feet	0.3048	metres	m
yd	yards	0.914	metres	m
mi	miles	1.61	kilometres	km

AREA

in^2	square inches	645.2	millimetres squared	mm^2
ft^2	square feet	0.0929	metres squared	m^2
yd^2	square yards	0.836	metres squared	m^2
mi^2	square miles	2.59	kilometres squared	km^2
ac	acres	0.395	hectares	ha

MASS (weight)

oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg

VOLUME

fl oz	fluid ounces	29.57	millilitres	mL
gal	gallons	3.785	litres	L
ft^3	cubic feet	0.0328	metres cubed	m^3
yd^3	cubic yards	0.0765	metres cubed	m^3

NOTE: Volumes greater than 1000 L shall be shown in m^3.

TEMPERATURE (exact)

°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol

LENGTH

mm	millimetres	0.039	inches	in
m	metres	3.28	feet	ft
m	metres	1.09	yards	yd
km	kilometres	0.621	miles	mi

AREA

mm^2	millimetres squared	0.0016	square inches	in^2
m^2	metres squared	10.764	square feet	ft^2
km^2	kilometres squared	0.39	square miles	mi^2
ha	hectores (10 000 m^2)	2.53	acres	ac

MASS (weight)

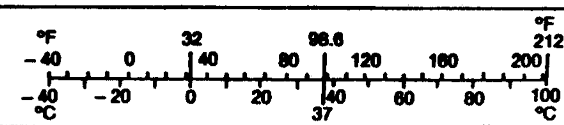
g	grams	0.0353	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams (1 000 kg)	1.103	short tons	T

VOLUME

mL	millilitres	0.034	fluid ounces	fl oz
L	litres	0.264	gallons	gal
m^3	metres cubed	35.315	cubic feet	ft^3
m^3	metres cubed	1.308	cubic yards	yd^3

TEMPERATURE (exact)

°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



These factors conform to the requirement of FHWA Order 5190.1A.

* SI is the symbol for the International System of Measurements

ABSTRACT

This report presents a thorough study on the effects of temperature and moisture on the load response of low-volume roads. The procedures developed herein can be used to formulate temperature and seasonal adjustment factors for low-volume roads and to estimate when and where seasonal load restrictions should be applied.

Field studies were performed on two light pavement structures at the Texas Transportation Institute Research Annex. The tests consisted of taking pavement layer temperatures and Falling Weight Deflectometer deflection readings at hourly intervals throughout the day. In addition, six Farm-to-Market roads in different regions of the state of Texas were monitored monthly to evaluate temperature and moisture effects on pavement moduli. Falling Weight Deflectometer deflection readings, rainfall data, and pavement temperature and suction readings were collected over a twelve month period. The effects of rainfall on the moisture condition beneath a pavement were also examined. The deflection readings were then used to backcalculate the layer moduli of the pavements. It was found that the stiffness of low-volume roads increases with the increase of temperature and decrease of moisture. Another aspect of temperature effects on granular materials was examined from the point of view of stress relaxation. Laboratory testings in the form of relaxation tests at different temperatures were performed on a granular material in order to determine the time and temperature dependent properties of the material.

The analytical models were developed based on a micromechanical approach. The granular base course materials were treated as elastic spheres in contact and subjected to temperature and moisture changes. Comparisons between the backcalculated and the predicted pavement layer moduli were made to verify the models.

SUMMARY

The increasing heavy traffic loads on low-volume roads make it imperative to have a fast and economical method of evaluating the structural integrity of this type of roads. Nondestructive testing methods which measure the surface deflections of pavements have been found to provide the aforementioned capabilities. However, the surface deflections in response to loads are affected to a significant degree by climate, mainly temperature and moisture. As such, the backcalculated properties of the pavement layers vary with the temperature and season at the time when testing is conducted.

This study presents an analytical approach to adjust the stiffness of granular base course layer for temperature and moisture effects. Two separate models were developed, one for temperature and the other for moisture. The models can also be used to predict pavement conditions at a certain month of the year so that seasonal load zoning can be determined. The temperature model assumed that the granular particles in the base course layer are confined in all directions. Due to the confinement, the volumetric expansion caused by temperature increase will increase the confining pressure and, thus, the stiffness of the layer. The moisture model considered the base course materials as a two phase system. One phase represented the soil particles, and the other phase represented an air-water mixture surrounding the soil particles. The temperature model requires the properties of the soil particles, such as the elastic modulus and linear thermal coefficient, as the input, while suction values are required in the moisture model.

In order to verify the models, pavement layer temperatures and Falling Weight Deflectometer deflection readings were taken at hourly intervals throughout day on two pavement test sections at the Texas Transportation Institute Research Annex. In addition, six Farm-to-Market roads in different regions of the state of Texas were monitored monthly. Falling Weight Deflectometer deflection readings, rainfall data, and pavement temperature and suction readings were collected over a twelve month period. The effects of rainfall on the moisture condition beneath a pavement were also examined.

Another aspect of temperature effects on granular materials were examined from the point of view of stress relaxation. Stress relaxation is the time-dependent property of materials which is of great importance in a variety of engineering problems in which long term behavior is of concern. Relaxation tests, at which soil samples were subjected to a constant strain while the stress was continuously monitored for a

period of time, were performed on Ottawa sand at four different temperatures. The relaxation rate and the effect of temperature on the relaxation rate were determined from the tests.

This study provides a better understanding of the mechanisms of temperature and moisture effects on granular materials. It will provide the Texas State Department of Highways and Public Transportation with a better means of interpreting and utilizing the surface deflection data of low-volume roads.

IMPLEMENTATION STATEMENT

This report describes the development of an analytical approach to account for temperature and moisture effects on the stiffness of granular base course materials. The models work very well as verified by the comparison with the field test results. The models can be used for temperature and moisture adjustment of the stiffness of low-volume roads and prediction of pavement conditions for load zoning purposes. These capabilities will be incorporated in a software package which will be completed towards the end of the Study 2-18-8-473.

DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented within. The contents do not necessarily reflect the official views or the policies of the Federal Highway Administration. This report is not a standard, a specification nor a regulation.

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CHAPTER I

INTRODUCTION

There is probably no other engineering structure subjected as much to climatic variations as the pavement structure. Two climatic factors that are generally considered as the most influential to pavement strength and performance are temperature and moisture. Understanding the effects of temperature and moisture on granular materials, which have been commonly used for the pavement base course layer, is, therefore, of great importance in the evaluation of pavement performance. It is more so for low-volume roads, which are light pavement structures with one inch or less surface treatment course placed over the granular base course. The surface treatment is for waterproofing which also serves as a wearing course. The main load bearing layer is the granular base course layer. This understanding will provide a more realistic evaluation of low volume roads and a criterion to estimate when and where seasonal load restrictions should be applied.

One of the methods that has been extensively used to evaluate structural integrity of pavement is measuring pavement surface deflections with nondestructive testing devices. In this type of testing, the surface deflections at various radial distances (i.e., deflection basin) resulting from a static or dynamic load are recorded and then analytical methods are used to predict or match this basin. The material properties which give this matching basin are assumed to be those of the materials in the field. However, the deflection basins are known to vary with the temperature and season at the time when testing is conducted. Owing to this variation, it will not be possible to know the structural integrity of the pavement at another temperature and season.

Seasonal variations in surface deflections are primarily due to movement of moisture beneath the pavement. Higher moisture contents are generally associated with lower resilient moduli of pavement subgrades. Unfortunately, qualitative measures of the moisture beneath a pavement are not readily available in most situations. Studies have been conducted in an attempt to correlate subgrade moisture with rainfall, but different results have been reported. When comparative studies are required, a common method used to account for subgrade moisture variations is to classify a certain area into zones with similar subgrade soil type and climatic factor, and assign different adjustment factors to each zone for different times of the year (Asphalt Institute, 1977; Bandyopadhyay, 1982; Bhajandas et al., 1977; Hines, 1983). Other empirical adjustment factors have also been developed to account for temperature variations. The shortcoming of these approaches is that while they can provide reasonable predictions for the location in which the observations were made, they do not explain the phenomena behind these variations.

Low-volume roads also exhibit a unique behavior when subjected to temperature changes. For thick asphaltic concrete type flexible pavements, higher temperatures are generally associated with lower stiffness, and the temperature dependency of flexible pavements is found to increase with increasing thickness of the asphaltic layer. As noted, for low-volume roads, the asphaltic layer is more of a surface seal than a load carrying component. Consequently, the response of the granular aggregates in the base course layer play a more important role. Scala and Dickinson (1967), while studying low-volume crushed rock type pavements in Australia, found that the surface deflections decreased with increasing temperature for 8 and 10 in. pavements, which suggests that granular materials respond differently from asphalt concrete.

It is believed that the micromechanical approach which examines the behavior of granular soils at the grain level will provide a better understanding of the mechanisms of temperature and moisture effects on granular soils. This study attempts to address the various issues raised.

Another aspect of temperature effects on granular materials that should be examined is stress relaxation. Relaxation and creep of granular soils have been commonly ignored because it has been erroneously believed that granular soils do not creep or relax. Schmertmann (1970) and Lacerda and Houston (1973) observed that these types of behavior do exist in granular soils. The time and temperature dependent properties of granular soils are examined in this study.

OBJECTIVES

The objective of this study is to obtain a better understanding of the response of granular soils subjected to temperature and moisture changes, such that a more realistic evaluation of low-volume roads can be achieved. Models to account for temperature and moisture effects on the load response of granular materials are developed. Two factors will be considered in this study and they are: (1) the stress relaxation of granular materials at different temperatures, and (2) the development of temperature and seasonal adjustment factors for the pavement stiffness of low-volume roads.

METHOD OF APPROACH

The studies undertaken to achieve the stated objectives consist of three main parts, and they are: laboratory stress relaxation testing on granular soil, field testing and theoretical modeling. An outline of the study approach is described in the following chapters.

Chapter II contains a brief description of the nondestructive testing device and the backcalculation program used in this study. The mechanisms of climatic influence on pavement behavior, rainfall factors, and published literature on methods of modeling temperature and moisture effects on pavements are also reviewed.

Chapter III describes the field tests and test results.

Chapter IV contains the formulation of the models for temperature and moisture effects on the stiffness of granular soils.

Chapter V contains the comparison of measured and predicted results, and the application of the models.

Chapter VI contains conclusions and recommendations.

Appendix A contains the description of the stress relaxation test on granular soil under different temperatures, discussion of the test results, and recommendation of the testing procedures. The laboratory and field test data and the results are given in Appendices B and C, while Appendix D contains the listing of a user-friendly computer program which was written based on the models.

CHAPTER II

BACKGROUND

The nondestructive testing device and the backcalculation program used in this study are briefly discussed. The mechanisms of climatic influence on pavement behavior, moisture conditions beneath a pavement and published literature on rainfall effects and methods of modeling temperature and seasonal effects on pavements are also reviewed.

NONDESTRUCTIVE METHODS OF PAVEMENT EVALUATION

There are numerous nondestructive testing devices capable of applying dynamic load and recording deflections at various distances from the loading plate. The Falling Weight Deflectometer (FWD) is one of the more popular of these devices which are reported to be able to simulate pavement response under moving load (Bibbens et al., 1984; Hoffman and Thompson, 1982; Tholen et al., 1985). The FWD imposes an impulse load of between 2500 lbs and 24000 lbs, which is transmitted to the pavement through a 300 mm diameter circular loading plate, and at the same time, surface deflections are recorded by geophones at seven different locations. The loading period roughly corresponds to a wheel speed of 40 to 50 miles per hour. The FWD used in this study was the Dynatest 8000 FWD.

A number of computer programs have been developed to backcalculate layer elastic moduli from deflection basins obtained by nondestructive testing. Most of these programs were developed based upon layered elastic theory or the finite element method. A detailed comparison of these programs for backcalculating layer moduli of low-volume roads has been presented by Chua (1988). In this study, the LOADRATE program (Chua and Lytton, 1984) was used. LOADRATE considered only surface-treated types of pavement. It used regression equations based on results generated from a finite element program, ILLIPAVE. The equations were developed to relate the nonlinear elastic parameters of the bulk stress model (for the base material) and the deviator stress model (for the subgrade material) with the deflections at the load point and at some distance away from the load. Layer moduli were then calculated from these parameters. This program was developed to analyze vast amounts of deflection bowls very quickly and was written for evaluating Farm-to-Market roads.

MECHANISMS OF CLIMATIC EFFECTS

Two Climatic factors that are generally considered as the most influential to pavement performance are temperature and moisture. These factors can affect the pavement strength and performance in the following ways: (1) changing the engineering properties of the component materials, (2) causing disintegration of materials, and (3) inducing volume changes in component materials.

The engineering properties of bituminous mixtures, granular base course materials, and subgrade soils are susceptible to temperature and moisture variations. An increase in temperature will cause a decrease in viscosity accompanied by a reduction in strength of bituminous mixtures. Haynes and Yoder (1963) and Thompson (1969) found that at a high level of moisture saturation the strength of granular materials decreases under repeated loading. Sherif and Burrous (1969) presented a summary of previous works on the effect of temperature changes on the shearing strength of soils. Although most of their data indicated a decrease in the strength of soils with temperature increase, some data has shown contrary results.

In some instances climate-induced deterioration progresses to extent that the materials are almost completely disintegrated. Thompson (1973) stated that it is possible to have a pavement failure caused primarily by climatic factors and not by wheel loading. For rigid pavement systems, corrosion of reinforcing steel and dowel bars is a source of disintegration of materials.

Temperature- and moisture-induced volume change is another mechanism of climatic influence. Volume changes due to low temperature contraction and freeze-thaw cycles will cause a cracking type of failure, and volume changes due to moisture infiltration in expansive soils and frost heaving will produce rough pavement surfaces.

MONITORING MOISTURE PRESENCE IN SOILS

Soils just beneath the base course layer are usually unsaturated. Kersten (1944) monitored moisture conditions of the upper 6 in. of the subgrade beneath flexible pavements in six states and reported that the degree of saturation of the subgrades averaged 73 percent. In the same study, Kersten also found that only 15 percent of the tests showed a saturation value of 90 percent or greater. Unsaturated soil is different from saturated soil in the fact that it is a three-phase system comprised of solid, water and air.

Soil suction has been used to characterize the effect of moisture on the volume and strength properties of unsaturated soils. Soil suction is defined as the free energy present in soil water with respect to a pool of pure water located outside of the soil at the same elevation (Aitchison, 1965). It is made up of two components, the osmotic, due to dissolved salts, and the matrix suction which is a negative pressure that exists in the soil water as a result of the capillary tension in the water. The soil suction can be measured by several methods including a psychrometer which measures the total suction and a thermal moisture sensor which measures the matrix suction. The use of the psychrometer is limited to soils with suctions lower (more negative) than -1 bar (-14.51 psi), while the moisture sensor is used for suctions higher than -1 bar.

The principal ways in which moisture changes can occur in a pavement system are as follows (Thompson, 1973):

1. Seepage of water into the pavement from higher adjacent ground,
2. Rise or fall of the water table level,
3. Percolation of water through the pavement surface,
4. Transfer of moisture, either to or from the soil in the shoulder, as a result of differences in moisture content,
5. Transfer of moisture (liquid phase) to or from lower soil layers,
6. Transfer of water vapor through the soil.

One and/or combinations of the above mechanisms may occur simultaneously.

The effects of rainfall on the moisture condition beneath a pavement have also been extensively studied. Yang (1988) monitored surface deflections using FWD for six test sites at Cornell University for three and a half years and concluded that precipitation is correlated to changes of deflection. Bandyopadhyay and Frantzen (1983) monitored weekly pavement surface deflections with a Dynaflect in Northeast Kansas and concluded that there exists a significant correlation between the amount of rainfall and the subgrade modulus. The time lag for the subgrade to reach its weakest state after a rainfall was found to be dependent on local factors such as runoff characteristics, weather patterns, and soil type. The time lag was found to be as long as three weeks. Stevens et al. (1949) also observed that spring pavement break up in Virginia could be related to the amount of percipitation.

However, not all of the published literature agrees that moisture variation under pavement is related to precipitation. Kubler (1963) analyzed quantities of data on subgrade moisture content and precipitation in West Germany but could not establish a relationship between precipitation and the change in subgrade moisture content. Cumberledge et al. (1974) also found that a comparison of monthly precipitation with moisture variation indicates erratic peaks and no definite increases in moisture due to periods of heavy rainfall. According to Marks and Haliburton (1969), moisture variations are affected by rainfall depending on the surface condition of the pavements. Pavements with cracks and greater surface perviousness were more likely to be affected by rainfall. For pavements with good surface conditions, the moisture variations were primarily attributed to temperature effects. Moulton and Dubbe (1968) showed that the amounts of precipitation occuring at various period prior to moisture content sampling were not statistically significant in explaining the observed variations of moisture content in either the base and subbase materials or in the subgrade soils. They felt that the moisture content in granular base and subbase materials would be more dependent upon the drainage characteristics of the materials and the site than upon precipitation.

METHODS OF MODELING TEMPERATURE AND SEASONAL VARIATIONS

Pavement surface deflections have been found to vary with temperature, especially for the flexible pavements. For the thick asphaltic concrete type of flexible pavement, higher temperatures are generally associated with larger deflections. Various temperature models have been formulated to simulate temperature in a pavement system. Most of the models have been developed to estimate temperature distribution with depth. One widely used empirical method to predict temperature at depth in an asphaltic concrete pavement has been developed by Southgate and Deen (1969). This method estimates the temperature at any depth in a flexible pavement up to 12 in. thick provided that the surface temperature, the 5-day mean air temperature, and the time of the day are known. The analytical type of solution uses the Fourier diffusion equation for determining conductive heat transfer in a pavement system (Schenck, 1963; Straub et al., 1968).

There are two approaches to account for the effects of temperature on a pavement system. The first approach involves assigning incremental deflections for each degree of temperature difference between the pavement temperature and the reference temperature (Cox, 1976; Kingham and Reseigh, 1967; Sebastyan 1961). The second method involves the use of a dimensionless multiplicative factor that is applied to a measured deflection at some known mean temperature of the pavement. This type of adjustment factors has been developed for Benkelman beam (Southgate and Deen, 1969; Asphalt Institute 1977), Road Rater (Cumberledge et al., 1974), and Dynaflect (Bandyopadhyay, 1982).

Thornthwaite moisture index (Thornthwaite, 1948) has been a popular means of empirically estimating moisture conditions in pavement subgrade soils. It is the indication of the availability of moisture in soil in a given year. A negative Thornthwaite moisture index indicates low moisture contents and a positive indicates high moisture content. A very good correlation between subgrade moisture potential and the Thornthwaite moisture index has been observed (Coleman and Russam, 1964; Aitchison and Richards, 1965). However, the accuracy of the method is somewhat questionable since variations in drainage, permeability of surfacing, and type of surrounding vegetation will normally create wide local variations. Theoretical methods have also been developed to model moisture movement and moisture equilibria. Coleman and Russam (1964) generated a method to predict moisture movement based on thermodynamics theory of equilibrium distribution of water in a porous body. Lytton and Kher (1970) formulated a rational method to predict moisture movement in expansive clays. Provided that the input data was of high quality, excellent agreements were obtained between field moisture content and predicted moisture content for various depth below the ground surface.

Other empirical formulas have been developed to estimate seasonal variations of pavement strength. From laboratory test results, Thompson and Robnett (1979) developed a correlation between resilient modulus and degree of saturation for different soil types. Bibbens et al. (1984) developed laboratory-determined resilient modulus versus moisture content curves. Cumberledge et al. (1974) monitored five field test sites in Pennsylvania to collect temperature, engineering properties of subgrade soils and Road Rater deflections. Multilinear regression analysis was performed to relate variations in surface deflections to changes in moisture content, percent of material passing no. 200 sieve, thickness of pavement, liquid limit, and dry unit weight. Among the variables, changes in moisture content were found to be most influential on pavement surface deflections.

CHAPTER III

FIELD STUDIES

TEST SITES AND INSTRUMENTATION

Two field studies were made. The first part involved taking FWD deflection readings on two surface-treated pavement sections at the TTI Research Annex, namely test section 10 and 11, at various temperatures. Wooden rods with thermocouples attached at different depths were inserted into the pavement sections. The tests were conducted at hourly intervals throughout the day for several times in a year. The tests were conducted at different temperatures in the same day in an attempt to eliminate seasonal effects.

The second field study involved collecting data from six Farm-to-Market road sections located in different regions of the State of Texas. Monthly FWD readings as well as monthly subgrade soil suctions and temperatures at different depths were collected for ten months. The basic criteria considered for site selection were the following:

1. The test sites were located in three different climatic zones in the State of Texas.

2. Two different subgrade soil types, one sandy and one clayey, were selected from each climatic zone.

3. The selected roads were typical Farm-to-Market roads with no stabilized base or subbase.

4. Weather data from a nearby weather station had to be available.

The locations of the districts in which the test sections were chosen are shown in Figure 1. District 8 is in the area with mild, dry winters and hot, humid summers. The average annual precipitation from different weather stations in this district ranges from 16 to 27 inches. The climate of District 11 is designated as humid subtropical with very hot summers. The rainfall is quite high, averaging 45 inches annually. The climate of District 21 is classified as semiarid subtropical with warm or hot summers. Precipitation varies from 19 to 27 inches.

Thermal moisture sensors were installed in Districts 11 and 21, and measure matrix suction. As for District 8, psychrometers which measure the total suction

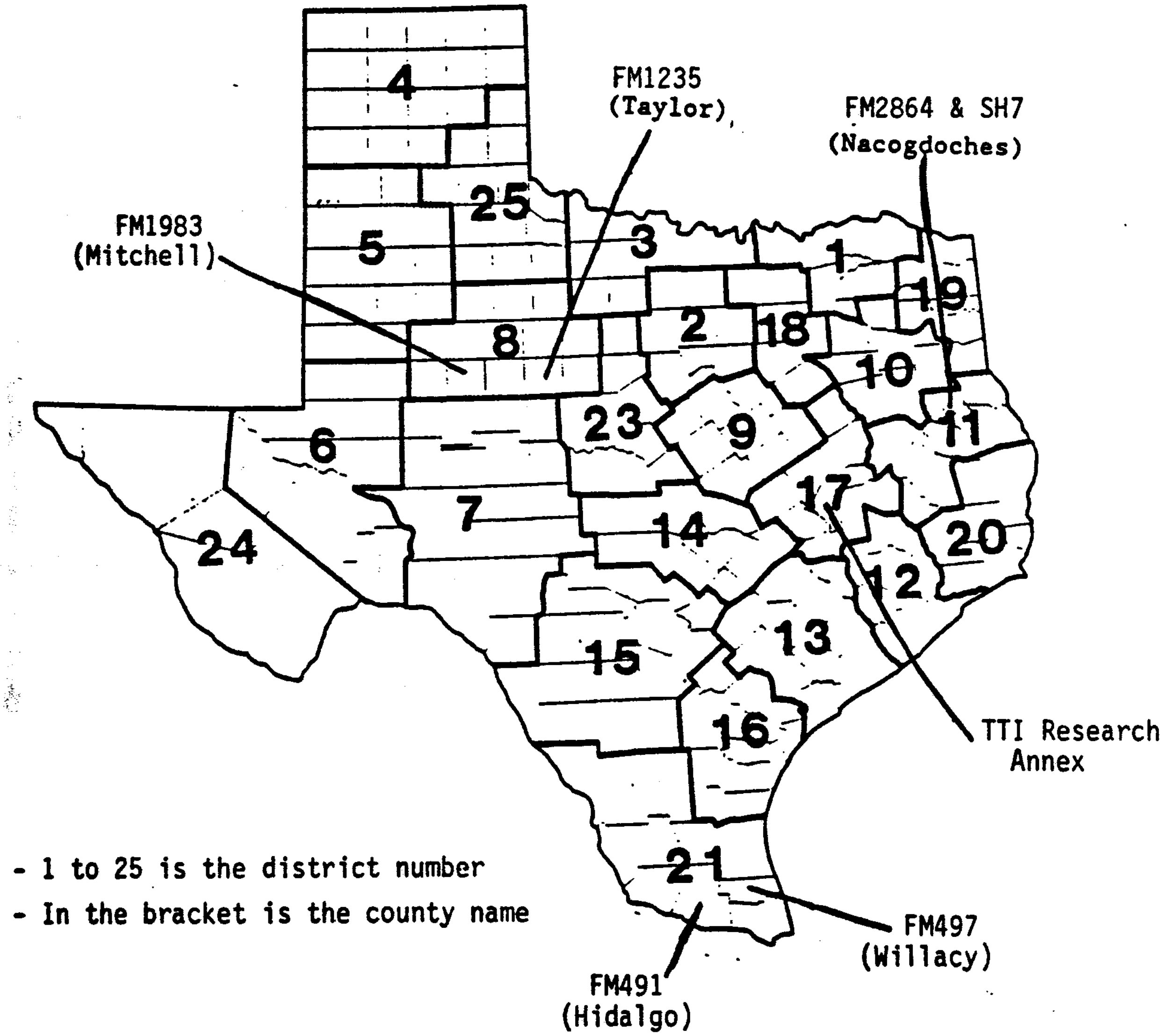


Figure 1. Location of the Test Sites

were used because this area is dry. Base and subgrade materials were also retrieved from all of the test sections for laboratory testing. The characteristics of the test sites are summarized in Table 1.

TEST RESULTS

Observed Temperature Variations and Temperature Effects

The results of the tests obtained from the TTI Research Annex are discussed in this section. Figure 2 shows a plot of air temperature, base course temperature, and subgrade temperature variations within a day as obtained in September for Test Section 11. The base course temperature followed the same pattern as the air temperature, only that there exist, as expected, a time lag between the two. In the afternoon, the base temperature reached its highest point approximately two hours after the air temperature did. At night, the heat trapped in the pavement dissipated slowly causing the temperature of the base course to be higher than the air temperature. This is further illustrated in Figure 3. Again from Figure 2, it appears that the subgrade temperature did not fluctuate much within a day. The same result was also observed for subgrade temperatures at different months (Figure 4). As for the base course, the temperature could vary by as much as 20°F in a day.

The LOADRATE program was used to backcalculate the pavement moduli from the FWD deflection data. The results from September data are presented in Figure 5. The moduli of the base were reported as the composite moduli of the surface treatment layer and the base course. Thus, the base moduli seem to be on the high side. The resilient moduli of the subgrade practically remained constant throughout the day. Even when comparing subgrade resilient moduli determined for different months, little variation was observed (Figure 6). The main reason was probably that the test sections considered here had good surface conditions without any cracks, and that the ground water table was far below the pavement surface. Furthermore, it seemed that temperature did not have pronounced effects on the subgrade moduli, since in one case, the subgrade temperature changed by almost 50 Fahrenheit degrees between April and August (Figure 4), and yet the modulus values changed little.

Two deflection basins obtained at the highest and lowest base course temperature of the day are plotted in Figure 7. Even though the base course modulus was higher at 104°F, it did not necessarily imply that the deflection at every sensor location was

Table 1. Characteristics of the Test Sites

Site	Closest Weather Station	Annual Rainfall (in.)	Base Course				Subgrade			
			Thickness (in.)	%Passing #200	Classification AASHTO	Unified	LL/PL	%Passing #200	Classification AASHTO	Unified
Annex 10	Easterwood	39.1	16	-	-	-	-	-	-	-
Annex 11	Easterwood	39.1	16	-	-	-	-	-	-	-
D8/FM1235*	Abilene	23.26	8	3	A-1-a	GP	43/31	58	A-7-6	CH
D8/FM1983	Roscoe	23.35	8	5	A-1-b	SW	25/16	27	A-2-6	SC
D11/FM2864	Nacogdoches	39.7	8.5	11	A-1-b	SW-SM	43/33	58	A-7-6	CH
D11/SH7	Nacogdoches	39.7	9.5	7	A-1-b	SP-SM	18/-	5	A-2-4	SP-SM
D21/FM491	Raymondville	27.48	8	4	A-1-a	GW	26/17	43	A-6	SC
D21/FM497	Raymondville	27.48	8.5	7	A-1-a	SP-SM	31/20	32	A-2-6	SC

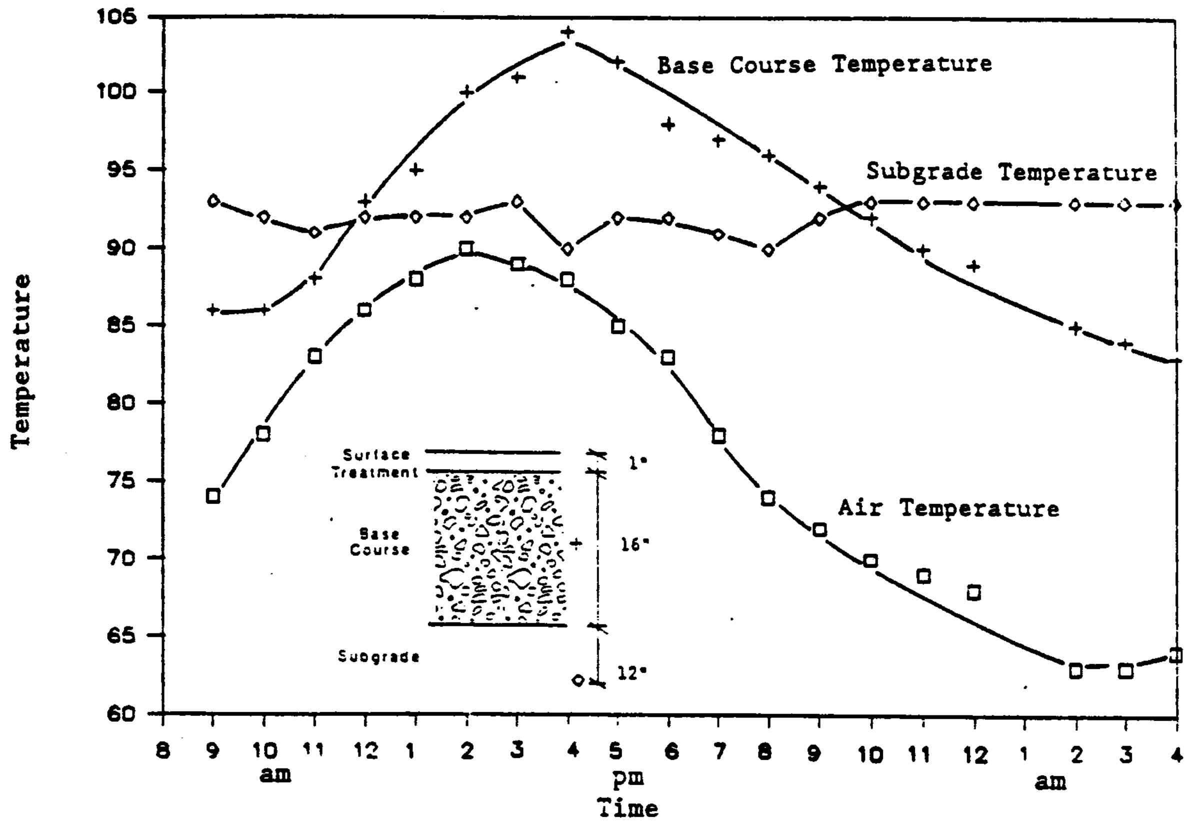


Figure 2. Typical Variation of Temperature within a Day (Section 11 TTI Annex)

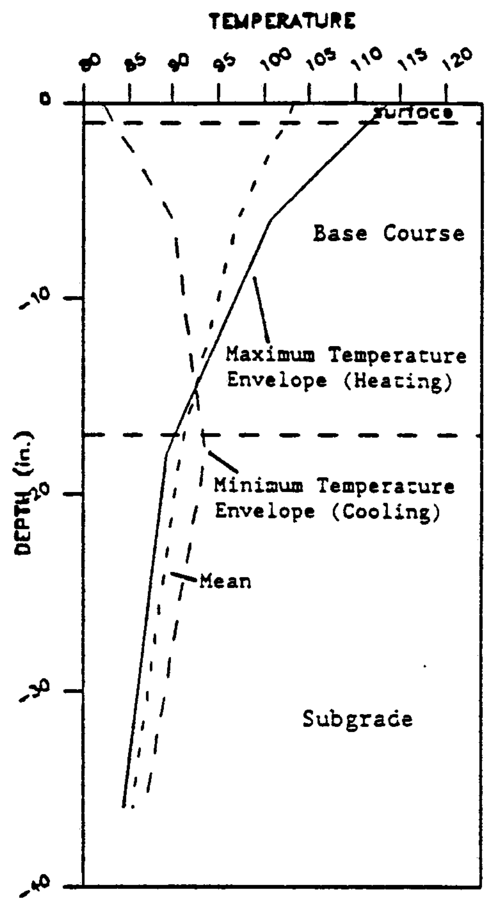


Figure 3. Temperature Variation with Depth

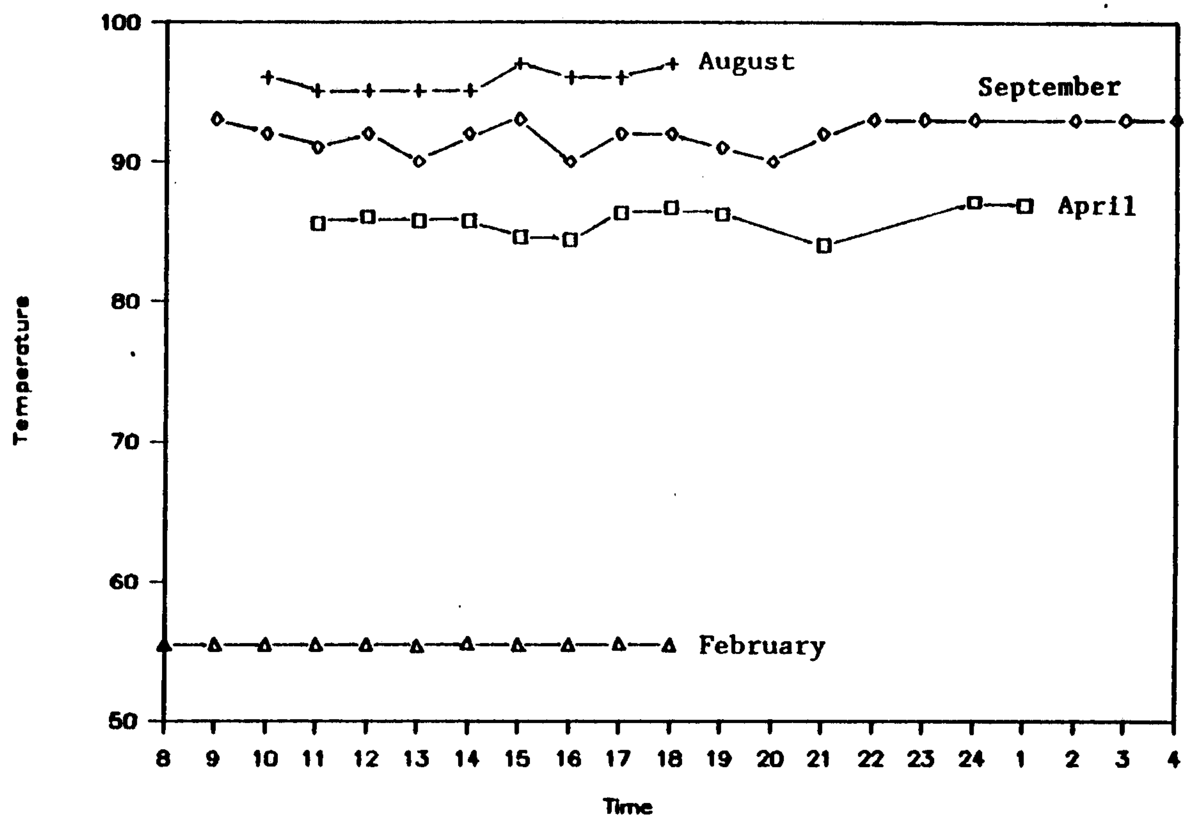


Figure 4. Subgrade Temperature at Different Months

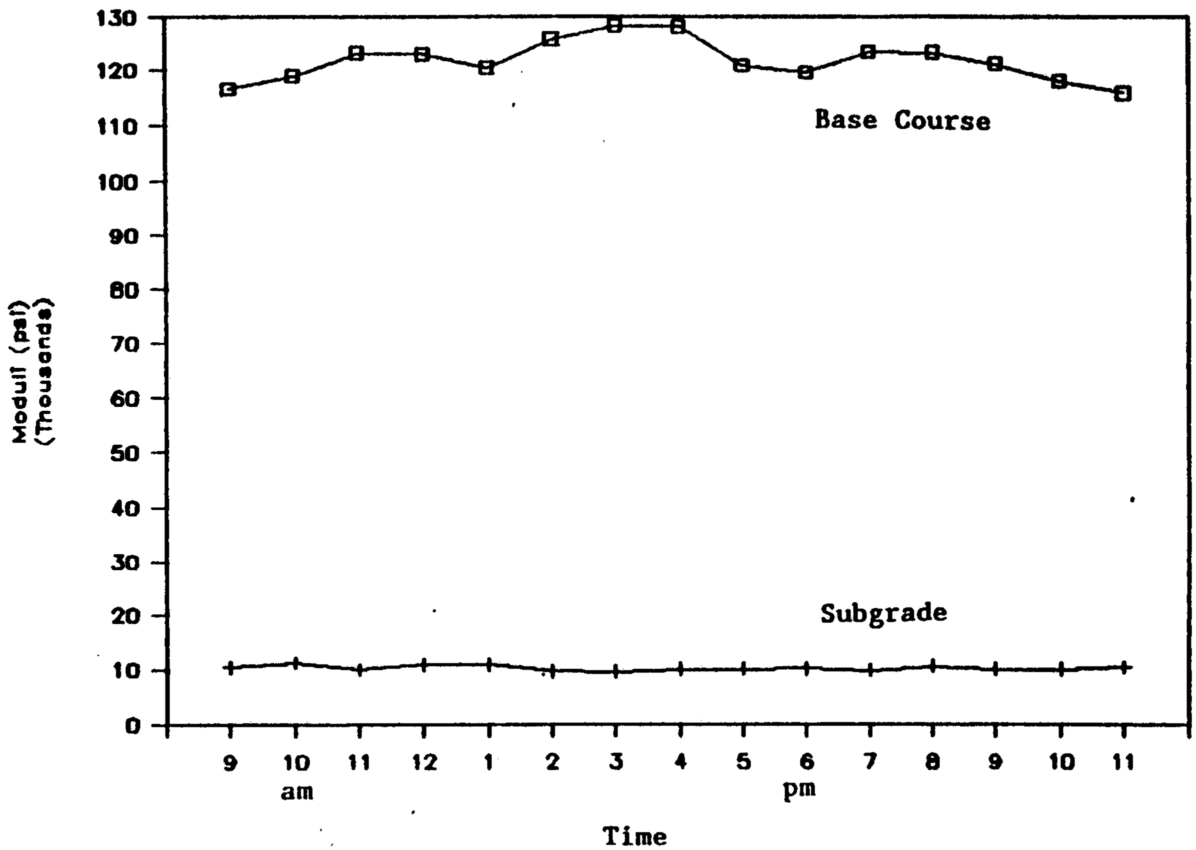


Figure 5. Typical Variation of Pavement Moduli Within a Day (Section 11 TTI Annex)

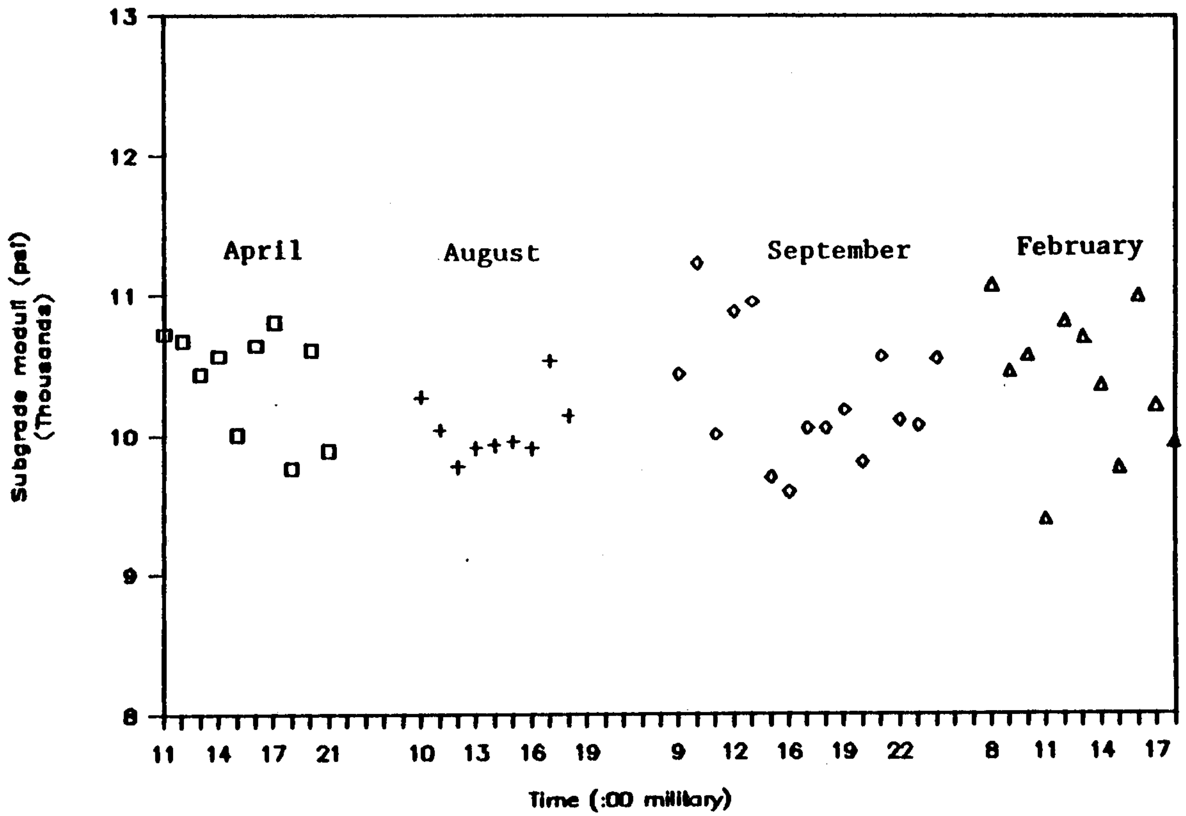


Figure 6. Variation of Subgrade Resilient Moduli at Different Months

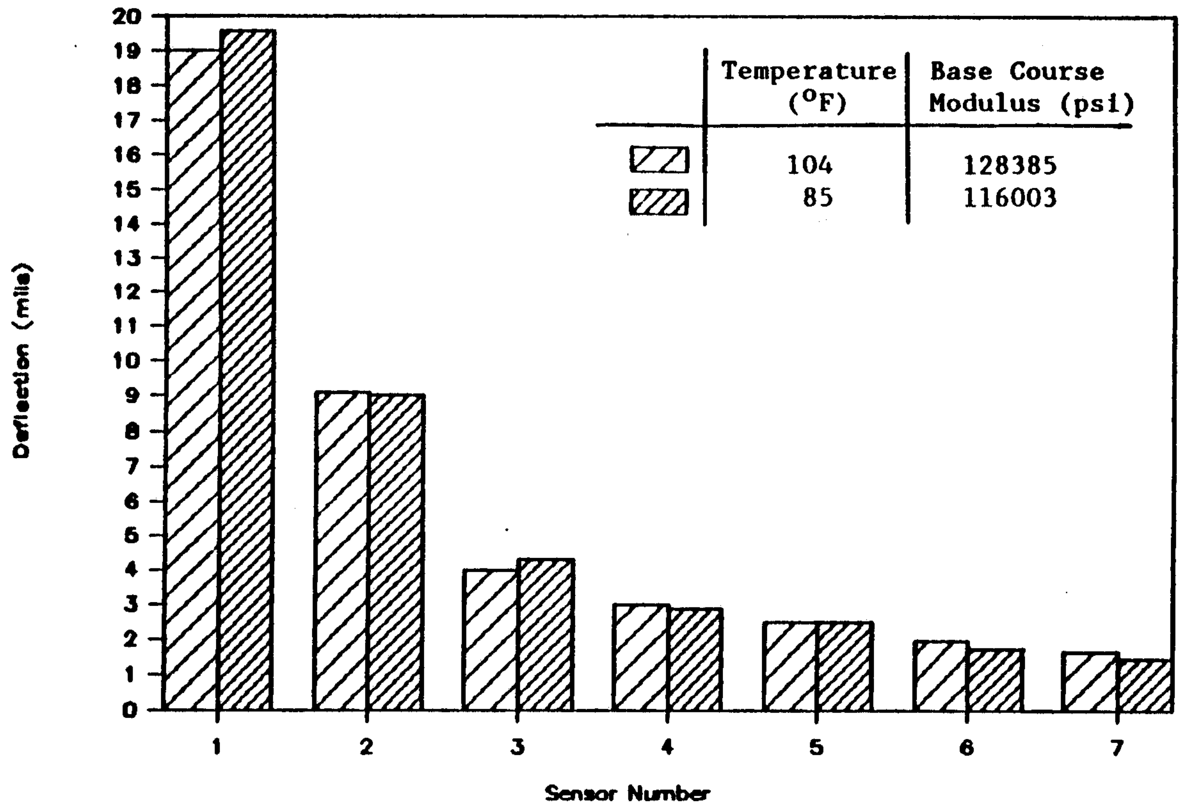


Figure 7. Comparison of Deflection Basins at Two Different Temperatures

lower at 104°F than at 84°F. As such, the deflection change at any individual sensor could not be used as an indication of the variations of the pavement moduli. It was for this reason that the changes of the pavement moduli, rather than the deflection at any one sensor, was used to determine the temperature effects.

Observed Moisture Variations and Moisture Effects

Monthly data were collected from the Farm-to-Market roads and include temperature and suction for both base course and subgrade, rainfall from the closest weather stations, and FWD deflection readings. There were problems in obtaining the suction readings at FM1983 and FM2864. The psychrometers in FM1983 only gave base course suction readings for several months, while the moisture sensors in FM2864 did not produce any readings at all. However, for the rest of the test sections, both the psychrometers and moisture sensors gave reasonable readings. For suction readings, a larger negative value corresponds to a drier soil and usually indicates a drier month. The effects of rainfall on suction will be discussed later.

The typical fluctuation of subgrade resilient moduli is plotted against suction in Figure 8. The resilient moduli of FM2864 varied by ± 18% but suction readings were not available for this test section. As for the other test sections, the variations were much less, in the order of ± 7%. The subgrade resilient moduli of SH7, FM491 and FM497 where thermal moisture sensors were used showed an increasing trend as the soils became drier. A somewhat similar trend was observed on FM1235, where psychrometers were used, in which, before fluctuating during the spring months, the moduli also became higher as the soil became drier.

Rainfall

In an attempt to correlate rainfall and soil suction, accumulative rainfall data three weeks prior to test dates were collected for each test section and plotted in the same graphs with the suction readings. A typical plot is presented in Figure 9. The suction values in this figure were measured by thermal moisture sensors. It can be seen that the suctions of the base course and subgrade were related to each other as well as to the amount of rainfall. The soil became drier as the rainfall decreased.

The trend of the psychrometer readings of the base course from FM1235 followed the rainfall changes by about two months (Figure 10). Although no reading was

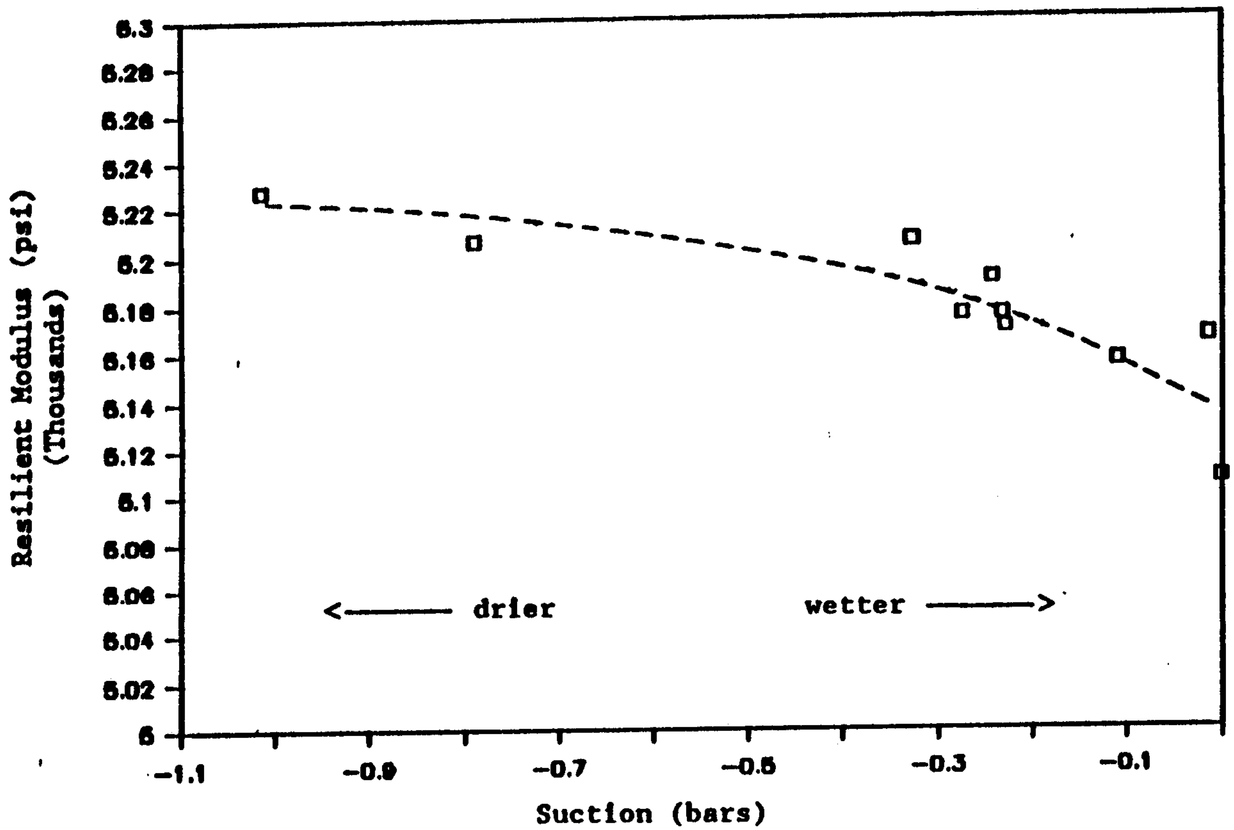


Figure 8. Variation of Subgrade Resilient Moduli of FM491

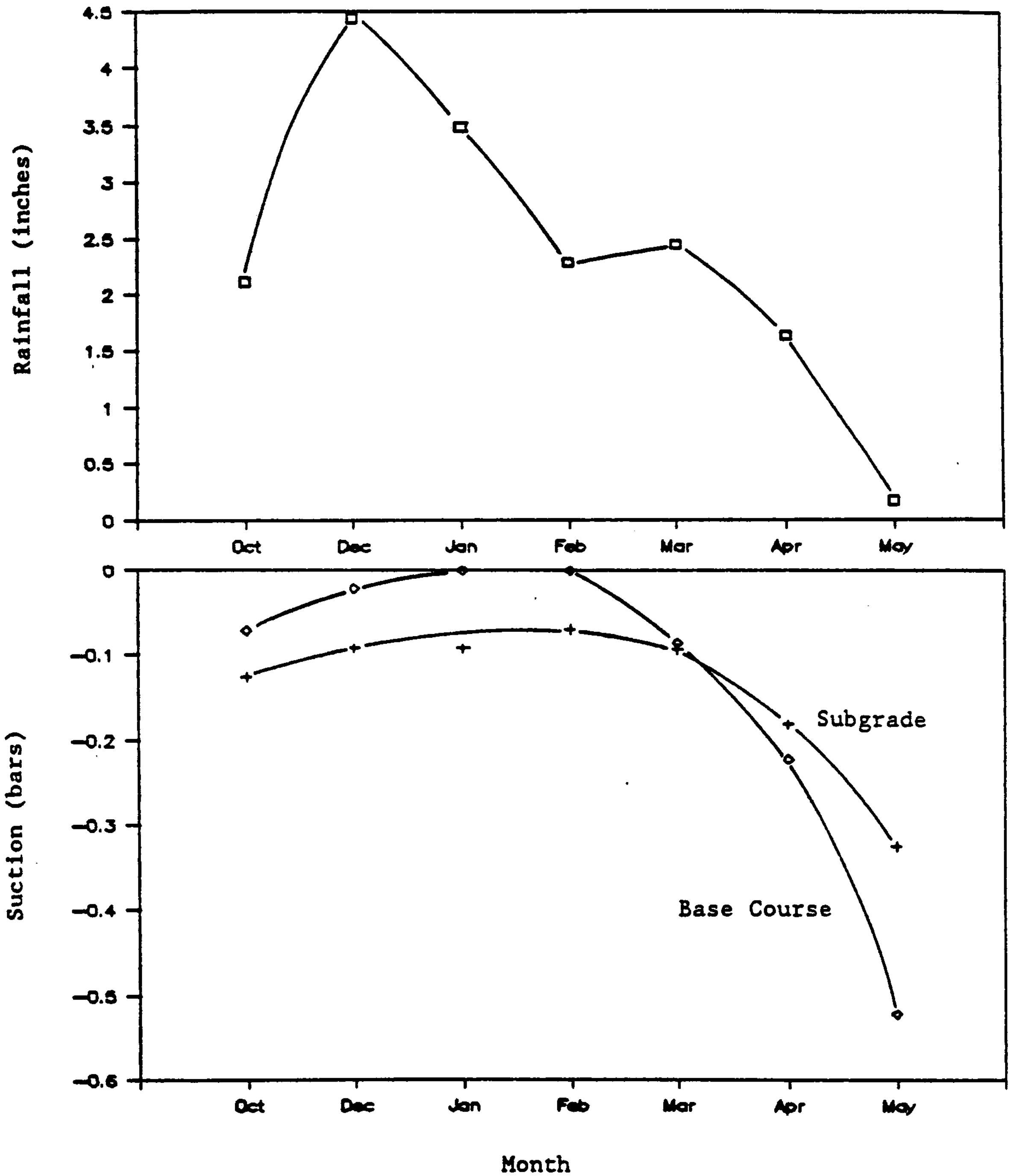


Figure 9. Variation of Rainfall and Suction of SH7

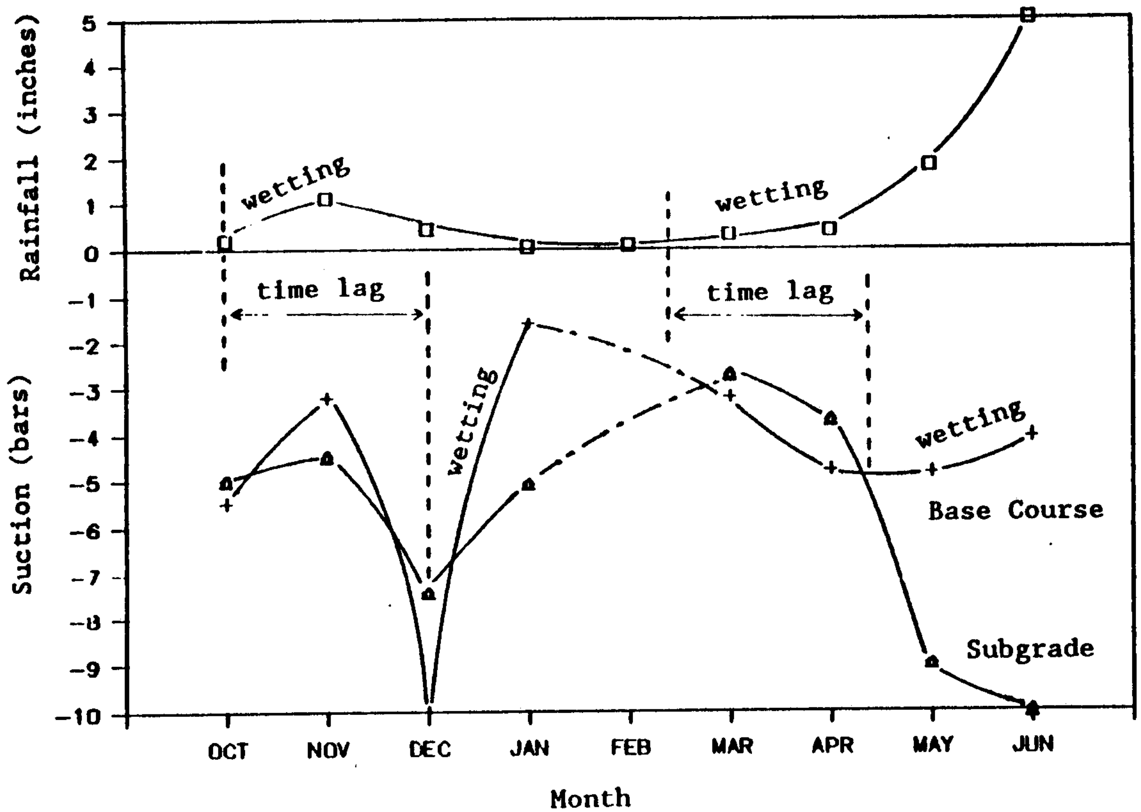


Figure 10. Variation of Rainfall and Suction of FM1235

obtained in February, the suctions continued to follow the two- month lag period behind the rainfall. A greater time lag was observed to occur between the subgrade suctions and rainfall. Even though similar trends were observed, no quantitative correlations between rainfall and soil suction were obtained.

CHAPTER IV

THEORETICAL MODELING

There are two main approaches for modeling soil behavior and they are: the phenomenological approach and the micromechanical approach. The phenomenological approach treats the soil as a continuum which may include thousands to millions of soil grains and pores, and analyzes the mechanism of the continuum of a whole. The micromechanical approach observes the behavior of soil at the grain level, and considers the forces and deformations at contact points between individual particles.

Most micromechanical models seeking to analytically describe the mechanical behavior of granular soils are based on the Hertzian contact theory which deals with a pair of homogeneous, isotropic, elastic spheres in contact, compressed statically by a normal force (Timoshenko and Goodier, 1951). Other models include an extension of the theory by Mindlin (1949) and Mindlin and Deresiewicz (1953) which considers tangential force at contacts. The micromechanical approach has been successful, at least, in the qualitative predictions of the behavior of granular aggregates (Armstrong and Dunlap, 1966; Ko and Scott, 1967). The micromechanical approach is used in this study for two reasons. Firstly, the micromechanical model provides an insight of the physical deformation mechanisms acting within the granular soil mass. Secondly, the temperature effects on pavement materials are hard to measure in the laboratory because the states of stress and boundary conditions in the field can hardly be reproduced. As such, the phenomenological approach which is concerned with the behavior on the size scale of experiment cannot be used.

MODELING APPROACH

The models developed here view the granular soil as an assemblage of soil particles in contact, subjected to temperature and moisture changes. The changes in stress relaxation due to temperature was investigated (Appendix A) but not found to be a significant factor. It was not included in the model. Granular base course soil particles in the field are subjected to overburden pressure and residual stresses. In the temperature model, the soil particles are assumed to be confined in all directions. As such, owing to the inability of the particles to expand due to the confinement, a rise in temperature will cause an increase in the contact forces between particles. The contact pressures which are the confining pressure, $\Delta\theta$, will then affect the stiffness of the soil.

The moisture model treats the soil particles as equal spheres in contact, surrounded by an air-water mixture, each considered as a different phase. Both phases are modeled as homogeneous, isotropic, linear elastic materials. The moisture model is an extension of the micromechanical model formulated by Lamborn (1986) which represents the load-deformation behavior of a partly saturated soil utilizing the thermodynamic laws. When the two-phase system is subjected to moisture or suction changes, the solid phase remains unchanged, but the variation of the air-water phase may result in changes of the principal stresses, $\Delta\theta$. The principal stresses are again related to the stiffness of the soil.

The resilient modulus is assumed to be related to the confining pressure in the following manner:

$$E = K_1 \, \theta^{K_2} \tag{1}$$

where

E = resilient modulus,

θ = bulk stress (sum of the three principal stresses), and

K_1, K_2 = constants.

It should be noted that other nonlinear models for resilient modulus where changes of confining pressure can be implemented, can be used instead of Equation 1. The change of modulus with respect to the change of bulk stress is obtained by taking the derivative of Equation 1, which yields

$$\Delta E = K_1 \, K_2 \, \theta^{K_2 - 1} \, \Delta\theta \tag{2}$$

The above equation can then be rewritten to include the change of the bulk stress caused by the temperature and the suction change,

$$\Delta E = K_1 K_2 \theta^{K_2 - 1} \, (\Delta\theta_T + \Delta\theta_s) \tag{3}$$

where the subscripts T and s denote temperature and suction respectively.

MODEL GEOMETRY

The soil particles are represented as a collection of spheres of equal radii, in contact. The position of the spheres relative to one another is restricted so that they are arranged in ideal packing configurations. For the temperature model, two different packing configurations which represent the densest and loosest arrangement of equal spheres are considered. The packings and the corresponding unit elements are shown in Figure 11.

A unit element has to contain a sufficient portion of the medium to represent the whole arrangement. When unit elements are put together they will form a regular array without addition or substraction of spheres. The unit element of the simple cubic packing (sc) has a porosity of 47.64%, while that of the face- centered cubic (fcc) has a porosity of 25.95%. For typical granular soils, Ottawa sand for example, the simple cubic array and the face-centered cubic array will have dry unit weights of 87.23 pcf and 123.4 pcf, respectively. Since the model is developed for two packing configurations, the states in between the loosest and densest condition are obtained by statistical estimates.

THE SIMPLIFIED TEMPERATURE MODEL

According to Hertzian contact theory, the centers of two spheres in contact under a normal force N will approach one another by an amount Z given by

$$Z = 2\left[\frac{\omega N}{R^{\frac{1}{2}}}\right]^{\frac{2}{3}} \tag{4}$$

in which

R = radius of the spheres,

$\omega = \frac{3}{4}\frac{(1-\nu^2)}{E}$ is a property of the material, and

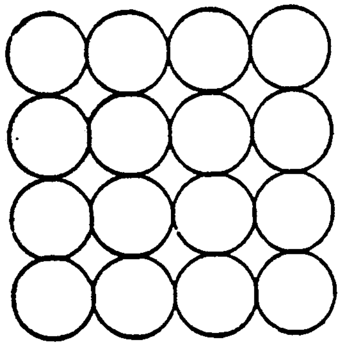
ν = Poisson's ratio.

The volumetric strain of the sphere is given by

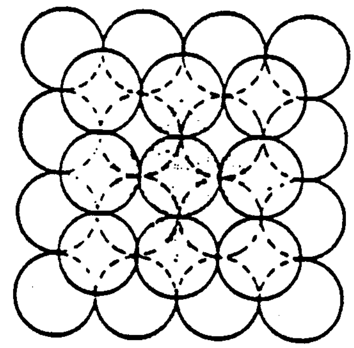
$$\frac{\Delta V}{V} = \frac{3\Delta L}{L} = 3\,\frac{Z}{2R} = \frac{3}{R}\left[\frac{\omega N}{R^{\frac{1}{2}}}\right]^{\frac{2}{3}} \tag{5}$$

Referring to Figure 11(c), when a uniform pressure p acts on the unit element of the simple cubic array, the normal force N is related to p by

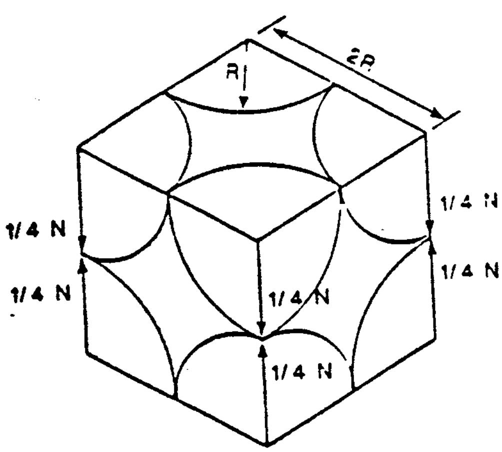
$$N = 4R^2 p \tag{6}$$



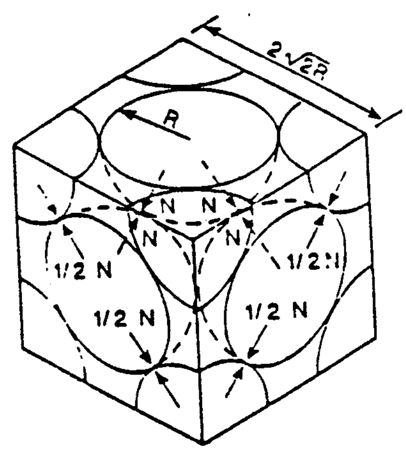
(a) Plan View of Simple Cubic Packing.



(b) Plan View of Face-Centered Cubic Packing.



(C) Unit Element of Simple Cubic.



(d) Unit Element of Face-Centered Cubic.

Figure 11. Modes of Packing of Equal Spheres

and substituting N into Equation 5 gives

$$\frac{\Delta V}{V} = 3\,(4\omega p)^{\frac{2}{3}} \tag{7}$$

Similarly, for face-centered cubic array,

$$N = \sqrt{2}R^2 p \tag{8}$$

and substituting N into Equation 5 gives

$$\frac{\Delta V}{V} = 3(\sqrt{2}\omega p)^{\frac{2}{3}} \tag{9}$$

When the unit elements are subjected to temperature increase, ΔT, there will be an increase in volume, ΔV, which is related by

$$\frac{\Delta V}{V} = \alpha_v\,\Delta T \tag{10}$$

where

α_v = cubical thermal coefficient, which is appoximately three times the linear thermal coefficient, α.

According to Smith et al. (1929), an assembly of randomly packed like spheres may be regarded as an arrangement of separate clusters of simple cubic (sc) array and face-centered cubic (fcc) array, each present in a proportion to yield the observed porosity, n_{obs}, of the assembly. Thus, if x represents the fraction of close-packed spheres, then

$$n_{obs.} = x\,n_{fcc} + (1-x)\,n_{sc} \tag{11}$$

Similarly, the confining pressure, p, acting on a granular medium with a porosity n can be approximated by

$$p = x\,p_{fcc} + (1-x)\,p_{sc} \tag{12}$$

The pressures for the two different cubic arrays can be obtained from Equations 7 and 9, and Equation 12 becomes

$$p = \left(\frac{x}{\sqrt{2}\omega} + \frac{(1-x)}{4\omega}\right)(\frac{1}{3}\alpha_v\Delta T)^{\frac{3}{2}} \tag{13}$$

The pressure in Equation 13 is caused by a change of temperature. If the initial bulk stress is θ, the pressure from the above equation is the change of the bulk stress, $\Delta\theta$, due to temperature variations. The new modulus can then be calculated by adding the change of modulus from Equation 2 to the original modulus.

MATERIAL PROPERTIES

The elastic modulus and thermal coefficent that appear in the equations shown above are for the individual soil particles and not for the soil mass. For their micromechanical model, Ko and Scott (1967) assumed that the sand grains are the same material as silicon glass, and used the elastic modulus of 10×10^6 psi and Poisson's ratio of 0.17. Yong and Wong (1972) reported that the elastic modulus and the Poisson's ratio of Ottawa sand grains is 12.5×10^6 psi and 0.17, respectively. Willis and De Reus (1939) designed and constructed an apparatus which mainly consisted of a temperature control box with optical lever to measure the linear thermal coefficient of different types of rocks. These results are shown in Table 2 which lists the properties, namely elastic modulus and linear thermal coefficient of different types of materials.

MODEL FOR MOISTURE EFFECTS

The mean principal stress acting on a two-phase system, which consists of solid and air-water phase, are related to the Helmholtz free energies per unit initial volume of the two phases, and the strain tensor. The relation can be expressed as:

$$\overline{\theta} = C_s \frac{\partial \overline{F}_s}{\partial \overline{\epsilon}_{kk}} + C_w \frac{\partial \overline{F}_w}{\partial \overline{\epsilon}_{kk}} \tag{14}$$

where

$\overline{\theta}$ = mean principal stress,

C_s, C_w = initial volume fractions for solid and water, respectively,

$\overline{F}$ = Helmholtz free energy, and

$\overline{\epsilon}_{kk}$ = $\Delta V/V$ = volumetric strain.

The overbar denotes the average values of the quantities, and the subscripts s and w represent the solid and water phase, respectively. A change in suction will alter the Helmholtz free energy of the water phase, but not the solid phase. Thus, the first term in the above equation is equal to zero. The change in the mean principal stress, $\Delta\theta$, is obtained by taking the derivative of Equation 14 with respect to the volumetric strain, and yields

$$\Delta\theta = C_w \, (\Delta P_w) \tag{15}$$

Table 2. Material Properties of Rocks (Willis and De Reus, 1939)

Material	Elastic Modulus ($\times 10^6$, psi)	Linear Thermal Coefficient ($\times 10^{-6}$)
Chert	3.1-18.0	6.0-7.2
Quartzite	3.8-10.2	6.2-6.9
Sandstone	2.9- 4.0	6.3-6.6
Basalt	11.4-13.9	3.9-5.9
Granite	7.6- 9.8	2.8-5.3
Limestone	5.1-12.6	1.8-5.4
Dolomite	2.5-10.0	4.0-5.0

where

ΔP_w = change in mean principal stress of the water phase, which is equivalent to the change in suction.

Thus,

$$\Delta\theta = -\Delta(suction)\,\frac{V_w}{V_T} \tag{16}$$

where

V_w = volume of water, and

V_T = total volume.

CHAPTER V

RESULTS

In this chapter, the field test results are used to verify the results predicted from the models. The application of the models are discussed and two examples are presented.

COMPARING PREDICTED AND MEASURED RESULTS

The base course moduli backcalculated by the computer program LOADRATE are compared to the predicted results from the models. Again, the base moduli reported are the composite moduli of surface treatment layer and base course. Thus, it is possible to have a high base moduli. The moduli from LOADRATE are referred to as "measured" moduli since they are calculated from the measured deflection basins.

Typical results from the September tests are presented. For that test, the base course temperature varied from 85°F to 104°F in the same day. In order for the temperature model to predict the base course moduli at different temperatures, a reference modulus at a known temperature was required as one of the inputs. The base course mean temperature for the day, which was 94°F, was selected as the reference temperature. Since the system can be expected to come to equilibrium at the mean daily temperature, the reference temperature was chosen to correspond to this temperature. The reference modulus was obtained by averaging the projected modulus values at 94°F from each modulus at different temperatures. The material properties used in the calculation were those of limestone, with an elastic modulus of $10\text{x}10^6$ psi, Poisson's ratio of 0.17, and linear thermal coefficient of $5\text{x}10^-6$ /°F. Other factors needed were the values of K_1 and K_2 in Equation 1. The constant K_1 was obtained from LOADRATE and K_2 was 0.33, which was the value used when the program was developed.

The results of the prediction are plotted in Figure 12. The predicted moduli and the measured results showed a similar trend of increase as the temperature increased. The prediction results of other tests were also plotted against the measured results (Figure 13). The points cluster along the 45- degree line, which indicates that the model predictions and the actual values are in agreement. The standard deviation of 5,200 psi was made up of random errors due to measurement, systematic errors

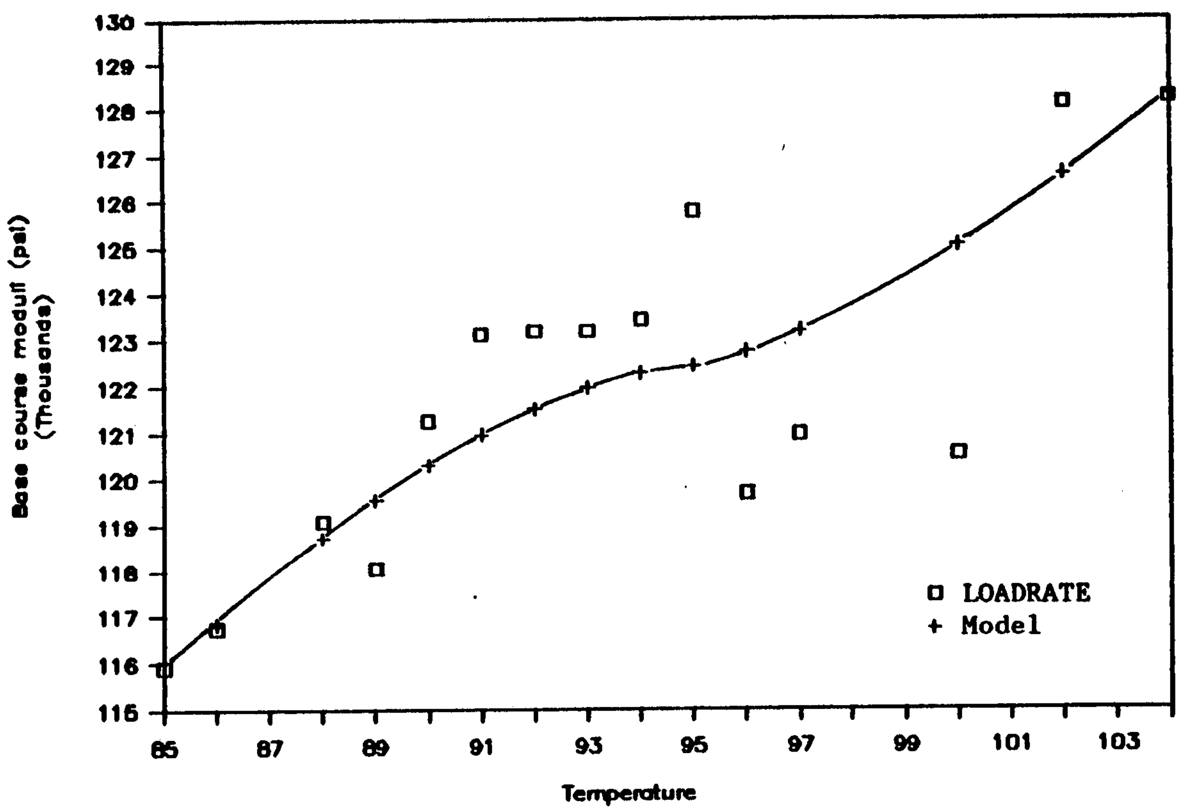


Figure 12. Comparison of the Base Course Moduli from LOADRATE and the Model (Section 11 TTI Annex)

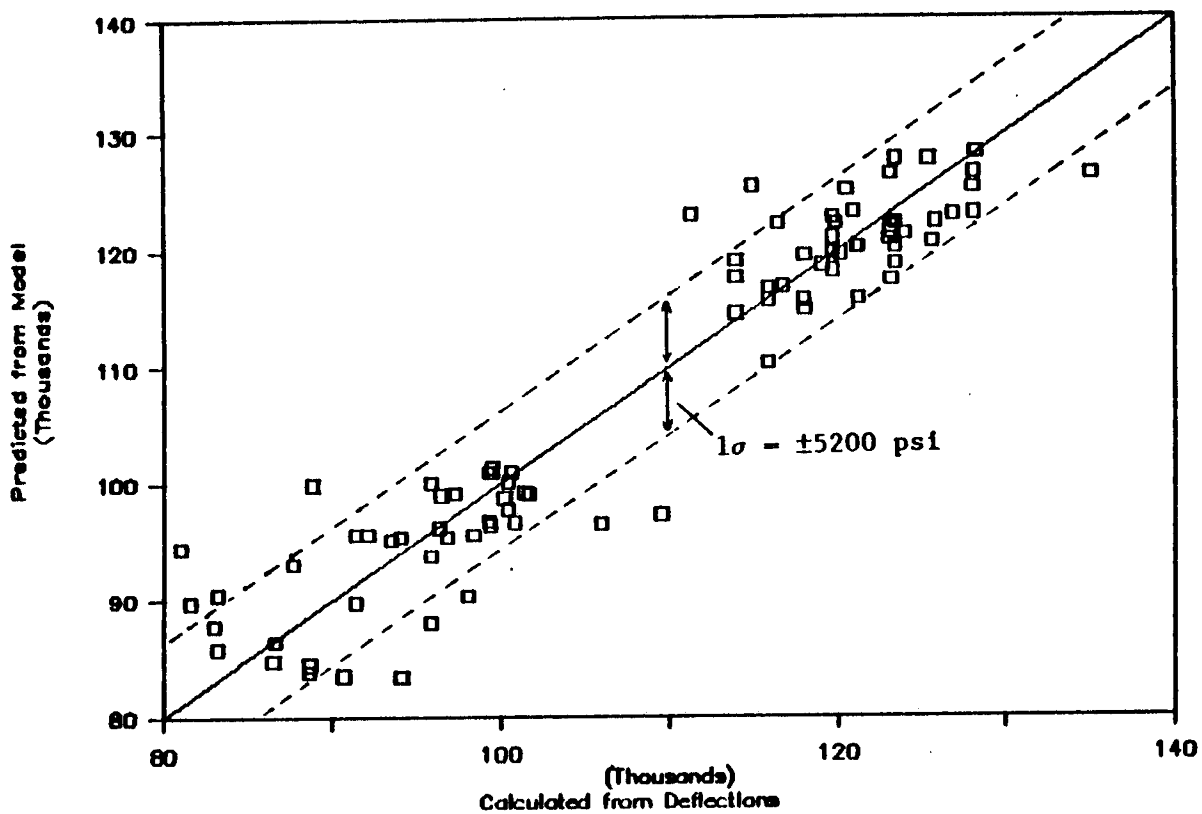


Figure 13. Predicted versus Backcalculated (TTI Annex)

due to backcalculation procedure used, and errors due to the effects of suction change which were not accounted for in the predictions.

In order to isolate the moisture effects on the base course moduli, deflection data with identical base temperatures were analyzed. The result is presented in Table 3. Since the range of moisture content variations of granular materials was small, the initial volume fraction (C_w) was assumed to be 0.13 for all of the calculations. It can be seen that for all the test sections, the base course moduli in different months but with the same base course temperature varied by less than 7%. Thus, the effects on the modulus of the base course due to changes of suction were too small to be measured reliably by the backcalculation method used.

However, when the fluctuation of the suctions was large, as in the case of FM1235, the effects of suctions on the modulus values were apparent. In Figure 14, the month of October was used as the reference, and all the other moduli were predicted from the October modulus. The deflection readings were collected from different months in which there was a wide spread of base course temperatures. The base course modulus for each month was the mean value of the moduli backcalculated from ten deflection basins taken at the same spot. The solid line in the figure denotes the predicted moduli without considering the suction effects. The dotted line, calculated by considering both temperature and suction variations, yields a much better prediction.

The same method was used to fit the base course moduli of SH7, where the suction readings were obtained by thermal moisture sensors. The result is plotted in Figure 15, which shows a good agreement between the predicted and measured results.

APPLICATIONS OF THE MODELS

The nondestructive testing devices which measure pavement surface deflections provide a fast and rational method of evaluating pavement conditions. The results of the evaluation have been used to determine the allocation of highway maintenance and rehabilitation fund. However, the surface deflections are found to vary with temperature and season. As such, direct comparison among the pavements cannot be made when the surface deflections are taken at different temperature and season. To account for these variation, empirical adjustment factors have been developed. The problem with the empirical method is that it cannot be expected to provide reasonable accuracy when the site and climatic conditions deviate greatly from those used to

Table 3. Moisture Effects on Base Course Elastic Moduli

Site	E (psi)	K_1	θ (psi)	ΔSuction (psi)	Δθ (psi)	Measured Change of Modulus	Predicted Change of Modulus
FM1235	110,000	33,700	34.8	-59	+7.7	+7,000	+8,032
FM1983	74,000	21,800	39.1	-15	+2.0	-2,900	+1,261
SH7	110,000	36,000	28.5	-2	+0.25	+5,000	+321
FM491	46,000	15,300	27.8	-10	+1.3	-3,000	+717
FM497	49,000	17,700	21.1	-1	+0.13	-3,400	+101

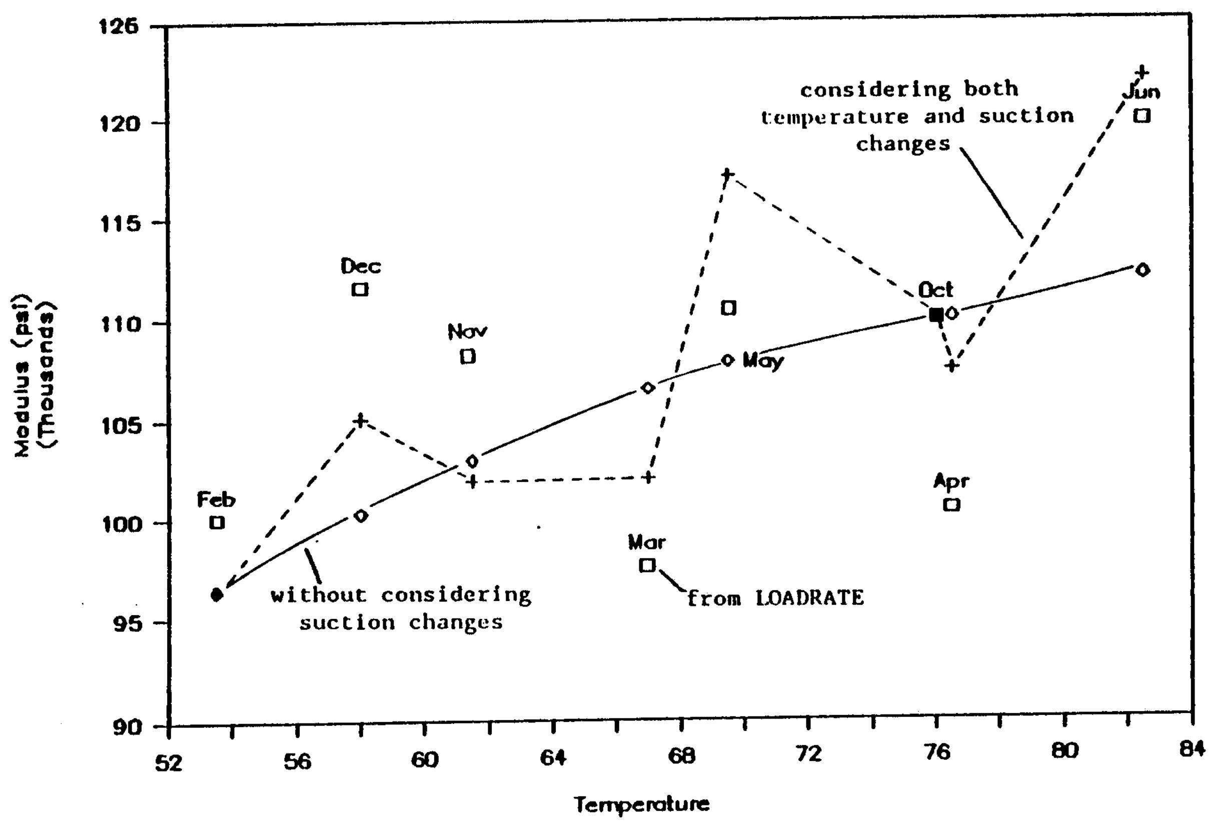


Figure 14. Comparison of the Base Course Moduli from LOADRATE and the Model (FM1235)

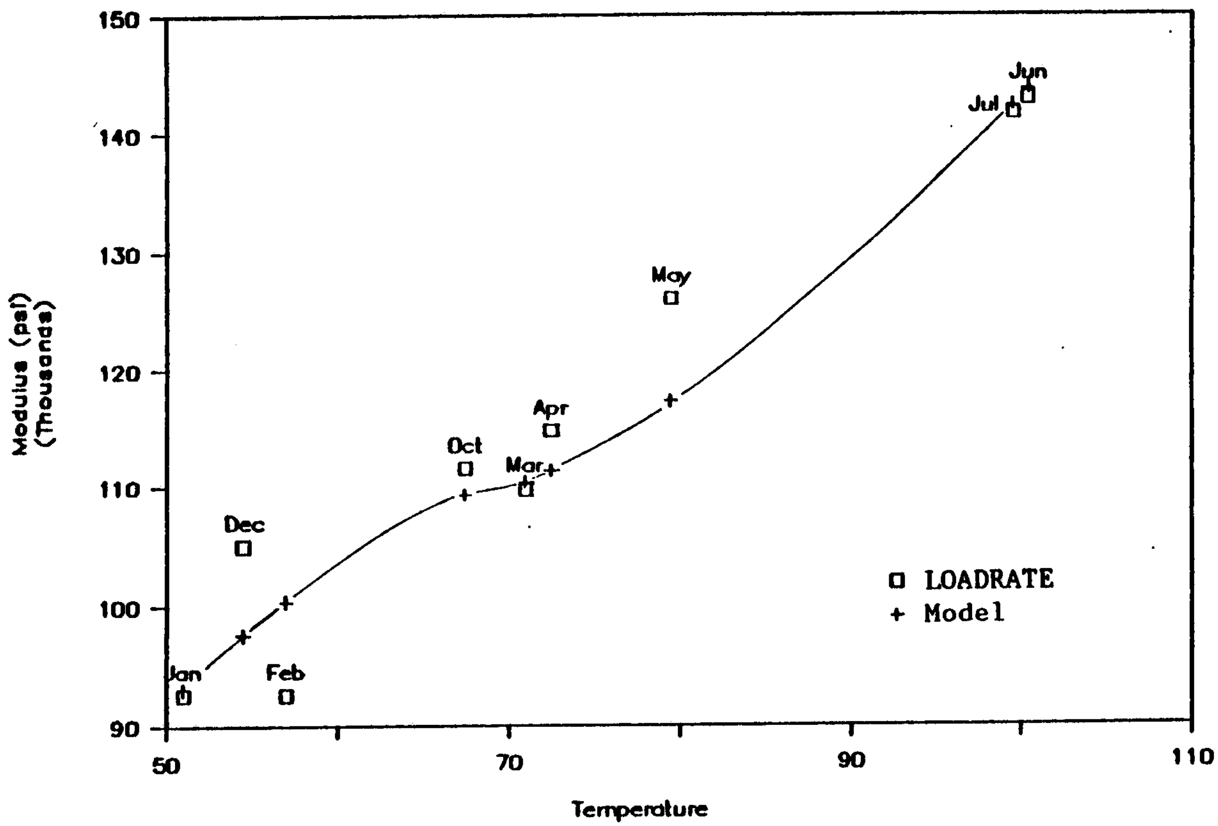


Figure 15. Comparison of the Base Course Moduli from LOADRATE and the Model (SH7)

develop the adjustment factors. The temperature and moisture models presented in the previous chapter can provide a rational method of comparing pavement structural integrity from deflection basins taken at different climatic conditions. Example 1 illustrates the application of the models for this type of comparative study. The models are used to adjust the moduli at different temperature and season to the moduli at the reference temperature and suction.

Excessivé moisture beneath a pavement, either from spring thawing or heavy rainfall, may result in a decrease in the stiffness and the associated strength of a pavement structure. To reduce the pavement deterioration that can occur, load restrictions for truck traffic are often applied. The criteria for the load restrictions primarily depend on the pavement condition during that period. In order to know the pavement condition, deflection measurements have to be taken at that period, which is not practical at all. The significance of the temperature and moisture models can be seen from the fact that they can be used to project the pavement structural integrity at that critical condition by using deflection data taken at other time. Whether or not seasonal load restrictions are required can then be determined.

Example 1: Consider two low-volume roads, A and B. The base course modulus of road A backcalculated from the deflection basin was 60,000 psi. The temperature and suction of the base course when the deflection data were taken was 50°F and -10 psi, respectively. The base course modulus of road B was found to be 70,000 psi at 110°F and the suction was -100 psi. The properties of the two base course materials are summarized in Table 4. The problem was to find out which pavement was stronger by comparing the base course modulus.

The temperature of 70°F and suction of -10 psi were selected as the reference condition. Assuming that the dry unit weight of the base course for both roads is 120 pcf, the porosity, n_{obs}, could be calculated. The fraction of the total volume representing face-centered cubic arrays, obtained from Equation 11, was used in Equation 13 to obtain the change of bulk stress caused by temperature variation. The change of bulk stress due to suction was obtained from Equation 16. Equation 3 was used to calculate the change of modulus. The equivalent moduli at the reference condition were then calculated for both roads. The results are also given in Table 4. At the first glance, road B seemed to be stronger than road A because the base course modulus of road B was higher. However, since the base course temperatures and suctions at the time the moduli were obtained were not the same, a comparative study could not

Table 4. Comparison of Base Course Moduli

Road	Base Course temperature (^oF)	Base Course modulus (psi)	Base Course suction (psi)	Base course material	Material properties	Calculated modulus at 70^oF and 10 psi suction
A	50	60,000	-10	Limestone	K_1 = 10,000 psi K2 = .45 μ = .17 α = $8x10^{-6}$ $/^oF$ E = $8x10^{-6}$ psi	61,400
B	110	70,000	-100	Crushed stone	K_1 = 15,000 psi K2 = .40 μ = .21 α = $6x10^{-6}$ $/^oF$ E = $10x10^{-6}$ psi	59,400

Note: The value of K_1 and K_2 are from Rada and Witczak (1981) be performed unless the equivalent moduli were calculated. It turned out that the modulus of road A was slightly higher than that of road B at the same temperature and moisture condition.

Example 2: The data from FM1235 were used to illustrate another application of the models. Referring to Figure 10 on page 22, the base course of FM1235 was found to be in the wettest condition in January, where the suction and temperature were recorded as -2 bars (-29 psi) and 67°F. Since the decision of load restrictions on a pavement has to be made as early as possible, the models can be used to predict the base course modulus at its wettest condition in January from the modulus in December. However, the base course temperature and suction in January have to be known. In this example, the measured temperature and suction were used. As for December, the suction and the temperature of the base course layer was -10 bars (-145 psi) and 58°F, respectively.

The modulus in December was first calculated from the deflection basins using LOADRATE, and turned out to be 112,000 psi. The constant K_1 was 32,200 psi. Considering the softening of the base course caused by moisture increase which was counteracted by the hardening due to temperature increase, the modulus in January was found to be 99,000 psi, which agreed well with the measured modulus of 94,000 psi. Knowing that the strength of the pavement reduces by 12% in January, the highway agency has to decide whether load restrictions are required. Other factors that have to be considered for applying load restrictions include the expected number of passes of overweight vehicles and the tolerable degree of damage of the pavement.

CHAPTER VI

CONCLUSIONS AND RECOMMENDATIONS

Rigorous field studies and laboratory testings were performed to study the effects of temperature and moisture on low-volume roads. An analytical approach to model those effects on granular base course materials were developed. The models provide a rational method of adjusting the stiffness of granular base course materials for temperature and moisture effects. They can also be used to estimate pavement conditions when excessive moisture is present, which will provide a means to estimate whether or not seasonal load restrictions should be applied. The temperature model requires the material properties of the soil particles, base course temperatures and base course moduli as the input, while soil suctions are needed for the moisture model. The variations of base course moduli due to temperature and suction changes predicted by the models agree well with the backcalculated moduli.

A computer program was written based on the models. Three groups of granular base course materials with their respective properties, namely the linear thermal coefficient, elastic modulus and Poisson's ratio, are given in the program. The users can either select one of the material groups or input their own material properties. The program can be used by itself provided that the stiffness of the base course layer is known beforehand. It can also be implemented as a subroutine of another program which calculates the stiffness of the pavement from surface deflections. In this study, the models will be incorporated in a software package which calculates the structural characteristics of low-volume roads from surface deflections. In this case, the temperature and suction of the base course layer should be measured during FWD testings. Alternatives to this measurement of temperature and suction are discussed in the recommendations for future studies.

The following conclusions can be drawn from this study:

1. The granular base course moduli of low-volume roads show a trend of increase as the temperature rises and suction become more negative, which can be explained as the result of an increase of the contact pressure between particles due to thermal expansion and suction changes.

2. It was shown that the proposed models are capable of predicting the changes of the stiffness of granular base course layer due to temperature and moisture changes.

3. The temperature and resilient moduli of subgrade soils do not fluctuate much within a day. However, the resilient moduli can vary by ± 18% due to seasonal changes.

4. The amount of rainfall is closely related to the variations of suction beneath a pavement.

5. Granular materials do relax under constant strain, but the relaxation rate is much lower than fine-grained materials. For dry Ottawa sand, the relaxation rate in the power law formulation was found to be approximately between 0.0122 and 0.0476.

The following recommendations were made for future studies:

1. Efforts to measure temperature and suction of pavement layers are laborious and timely. A thorough look into the methods, both theoretical and empirical, to estimate moisture and temperature beneath a pavement and verification of the methods with field test results would be worthy of consideration.

2. Another alternative to estimate soil suctions is to establish soil suction profiles for different types of pavement at different region of Texas. The profiles should cover different seasons throughout the year. This will involve statewide installment of psychrometers and moisture sensors for different types of pavement and long time monitoring of the suction changes.

3. Laboratory testings should be performed on different types of granular materials which are commonly used in low-volume roads to obtain the material properties of the soil particles. Those properties should include the linear thermal coefficient, the elastic modulus, and the stress relaxation rate.

4. The relaxation modulus from the laboratory testing sould be implemented into the models by means of the Correspondence Principle (Schapery, 1984). This may lead to a more realistic approach since it considers the time-dependent properties of the materials.

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Appendix A - Stress Relaxation in Granular Soils

STRESS RELAXATION IN GRANULAR SOILS

Stress relaxation is generally defined as the time-dependent decrease of stress in solid under constant constraint at constant temperature. Figure A1 illustrates stress relaxation behavior where the stress gradually decreases under constant strain. Creep, which can be related to stress relaxation, is the time-dependent increase of strain under constant stress. Stress relaxation and creep tests have been commonly performed on plastics, fibers and metals, and temperature is found to be the most influential factor on the test results. For geological materials, relaxation test was performed only on fine-grained soils and intact rocks. It has been commonly believed that granular soils do not creep or relax. However, it has been observed that foundations in sand exhibit a continuing settlement with time in the manner similar to the creep type phenomenon (Nonveiler, 1963). Schmertmann (1970) even suggested a correction factor for creep to be used in settlement calculation of foundations in sand. Up to now, the only published literature on relaxation test of granular soils was the test done by Lacerda and Houston (1973) on Monterey sand. In this chapter, the aspect of temperature effects on granular materials is examined from the point of view of stress relaxation. Since there is no available specification or published literature on the procedures of relaxation tests on soils, numerous problems were encountered. The test variables, testing procedures, encountered problems and test results of relaxation tests on a granular material under different temperatures are discussed in the following sections.

RELAXATION BEHAVIOR OF SOILS

Although relatively few studies have been done on stress relaxation of soils, particularly granular soils, most researchers suggested that after a certain period of time the decay of stress is essentially linear with the logarithm of time. The slope of the stress decay, which is the stress relaxation rate, varies with different soil types.

Murayama and Shibata (1961) performed tests on Osaka City Clay, and found that the decay of the deviator stress with the logarithm of time was linear up to approximately 500 minutes, and then showed a tendency to remain constant afterwards. The existence of this final relaxed level of stress was not found by other investigators.

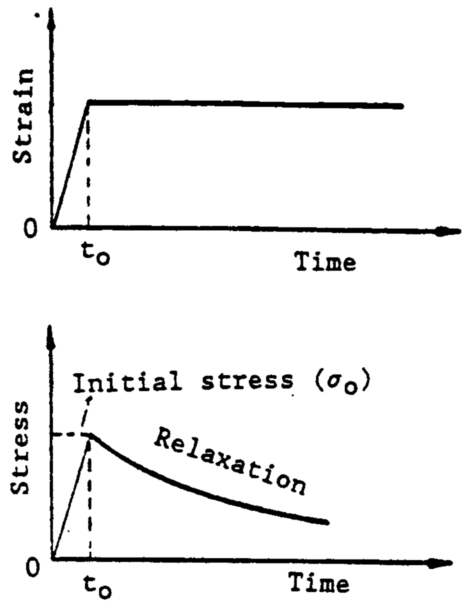

Figure A1. Stress Relaxation at Constant Strain

Vialov and Skibitsky (1961) presented the relaxation curves of dense clay which were approximately linear with logarithm of time. Saada (1962) showed that the deviator stress decreased linearly with logarithm of time until 50 days before it fell abruptly to zero. Results from other investigators also show that for a significant period of time there exists a linear portion of the deviator stress with logarithm of time (Akai et al., 1975; Folque, 1961; Lacerda and Houston, 1973).

Relaxation modulus, E(t), is the ratio of stress to the constant strain at some point in time t. The power law has been used to model the relaxation modulus for various type of materials, such as aluminum (Rohde and Swearengen, 1979), plastics (Williams, 1967), rocks (Obert and Duvall,1967), and soils (Schapery and Riggins, 1982; Stevenson, 1973). In the power law formulation, the relaxation modulus is given by

$$E(t) \;=\; E_1\, t^{-n} \tag{1}$$

where t represents time, and E_1, n are material constants. The constant n is the slope of the relaxation curve which is also the relaxation rate. The larger the value of n the more pronounced the relaxation behavior of the material. Chua and Lytton (1986) backcalculated the relaxation modulus of sand surrounding a pipe from the pipe deflections which had been measured for a period of time and obtained n values of approximately 0.01. Again using power law, Chua and Lytton curve-fitted the empirical correction factors for creep to be used for settlement of footings over sand as suggested by Schmertmann (1970) and obtained an n value of 0.02. Lacerda and Houston (1973) showed that the stress relaxation rate of Monterey sand was approximately four times less than that of fine-grained soils. Lade et al. (1987), while examining the stability of granular materials, found that creep rates of sand decreased to very small values after 60 minutes. Riggins (1981) presented n values that ranged from 0.082 to 0.104 for high plasticity clays. Obert and Duvall (1967) reported n values that were as high as 3.3 for various types of rock. Table A1 summarizes the relaxation rate, n, for different types of soil.

TEST VARIABLES

Confining Pressure

Different amount of confining pressure has been used for relaxation tests on soils. Murayama et al. (1974) performed relaxation tests on clay with 28.4 psi confining

Table A1. Stress Relaxation Rate (n) for Different Types of Soils

Reference	Type of Soil	Properties*				n
		LL(%)	PL(%)	CC(%)	Gs	
Lacerda and Houston (1973)	San Francisco Mud	88	36	59	2.75	0.0938
	Ygnacio Valley Clay	40	17	43	2.64	0.0421
	Kaolinite	57	30	100	2.64	0.0546
	Monterey Sand	-	-	-	2.65	0.0123
Murayama (1974)	Fujinomori Clay	44	26	32	2.68	0.0465
Akai (1975)	Fukakusa Clay	54	27	6	2.68	0.0447
Vialov (1961)	Dense Clay	-	-	-	-	0.0374
Campanella (1965)	Dry Illite	-	-	-	-	0.0464
Murayama (1969)	Osaka Clay:					
	50°F	-	-	-	-	0.0349
	68°F	-	-	-	-	0.0350
	86°F	-	-	-	-	0.0369
	104°F	-	-	-	-	0.0434
Riggins (1981)	Marine Sediments	101	37	80	2.73	.08-.1

* LL - Liquid Limit
PL - Plastic Limit
CC - Clay Content
Gs - Specific Gravity

pressure, while Vialov and Skibitsky (1961) used a confining pressure of 42.6 psi on dense clay. Lade et al. (1987) applied 4.26 psi confining pressure for creep tests on fine silica sand. Murayama and Shibata (1964) who did a series of relaxation tests with confining pressure ranging from 2.84 psi to 14.2 psi reported that the initial deviator stress and the rate of stress relaxation due to the application of the initial strain is independent of the confining pressure. The same conclusion was reported by Lacerda and Houston (1973).

AASHTO (1986) recommended confining pressures of 20, 15, 10, 5, and 1 psi for resilience testing of granular soils. Hicks and Monismith (1971) indicated that the range of confining pressure encountered in field pavements is from 0 to 10 psi. For this study, a confining pressure of 10 psi was used for all the tests.

Preconditioning

It was found that the test specimens should be preconditioned at a given stress level by applying cyclic deviator stress prior to testing. Some of the samples were initially tested without preconditioning. The test results were rather random and unpredictable which were probably due to the different arrangement of soil grains for different samples and the seating problem between the load cap, porous stone, and the sample. In order to eliminate these problems and, at the same time, simulate the field condition where base course materials are subjected to repeated loading, two levels of cyclic load, namely 5 psi and 17 psi, were selected. The 5 psi cyclic load was applied for 200 cycles as recommended by AASHTO (1986) for resilient modulus test, and the 17 psi was applied up to a point where no permanent deformation occured between successive cycles. It took approximately 25 cycles to reach the stage. For the latter case, a strain corresponding to a stress level of 17 psi was applied and held constant. The reason for the application of the 17 psi preconditioning and 17 psi initial stress level will be discussed in the section under 'Test Results'.

Strain

Murayama and Shibata (1961) and Vialov and Skibitsky (1961) presented relaxation test data in which the slope of the relaxation curves became slightly steeper as the strain increased. On the other hand, at relatively large strains, the rate of stress relaxation was found to be approximately independent of the magnitude of the imposed strain (Akai et al., 1975; Lacerda and Houston, 1973; Murayama et al., 1974).

The criteria for selection of strain in this study was that the strain had to be large enough to avoid imposing influence on the relaxation rates. On the other hand, the ratio of deviator stress to confining pressure had to be within a range such that failure did not occur. Using the Mohr-Coulomb failure criteria and an angle of internal friction of 35° for Ottawa sand, the principal stress ratio had to be less than 2.5 in order to avoid failure.

Strain Rate

The modulus obtained from the relaxation test where the sample is subjected to a constant strain is not exactly equal to the relaxation modulus because the time when the relaxation starts is not zero, but equal to t_o as shown in Figure A1. However, the value will approach the relaxation modulus at times which are long compared to the loading time, t_o.

Assuming that the relaxation modulus follows the power law, the ratio of measured to theoretical relaxation modulus at time t is given by (Schapery, 1987):

$$R_E(t) = \frac{E_R(t)}{E(t)} = \frac{t_R}{1-n}\left[1-(1-\frac{1}{t_R})^{1-n}\right] \tag{2}$$

where $t_R = t/t_o$. The relation of R_E to t_R for n=0.02 is illustrated in Figure A2.

From Figure A2, it is obvious that the longer the test time t and the faster the loading time t_o, the closer the measured modulus is to the theoretical modulus. For example, if the sample is loaded in 100 seconds, after 10 minutes the value of R_E is 1.0018. In other words, the error between measured and theoretical value is 0.18% after 10 minutes.

Lacerda and Houston (1973), who performed relaxation tests for at least 100 minutes, found that stress relaxation rates were independent of strain rates. They also found that the strain rate had a significant influence on the time at which stress relaxation started. The greater the initial rate of strain used to bring a soil to a given deformation, the more quickly stress relaxation began. Both phenomena agreed with the aforementioned theoretical explanation. For this study, the test time was at least 4 hours. As such, the test results were not affected by the strain rate.

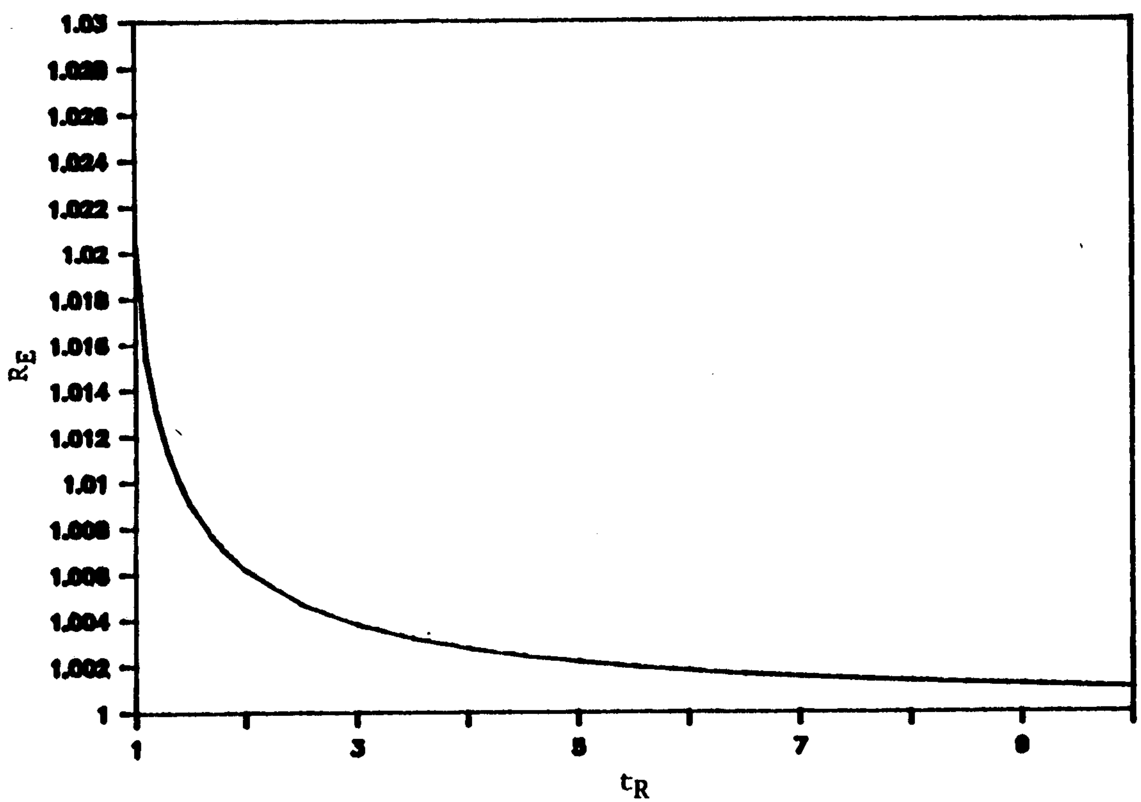

Figure A2. The Ratio of Measured to Theoretical Relaxation Moduli for a Power Law Modulus

Temperature

According to Murayama's (1969) test results on Osaka clay, the rate of stress relaxation increased at higher temperature. For the current research, temperatures of 40°, 70°, 100°, and 140°F were selected to cover a wide range of temperatures encountered in the field.

TEST MATERIAL

The material used in this study was a clean uniform Ottawa sand, which was assumed to be representative of the fundamental behavior of granular soils. As shown by the grain size distribution in Figure A3 it was a medium sand with a coefficient of uniformity of 1.22. The sand grains were angular and had a specific gravity of 2.67.

The sand was used for the test, with the reason that: (1) it was uniform and could be modeled easily, and (2) since to some extent this is a study on the fundamental behavior of granular materials, the additional results will provide a more comprehensive character of the sand which had been extensively studied.

TEST EQUIPMENT

Figure A4 shows the equipment used to prepare the test samples. Samples were formed by the split mold which had an inside diameter of 2.81 inches and produced a sample of 6 inches in height. A vibratory table was used to densify the samples. A surcharge, as shown in Figure A4, was placed on top of the material to help the densification process. A conventional triaxial cell was used for the test.

The loading equipment consisted of the MTS 810 machine with a microprofiler which was capable of applying both static and cyclic load. For the cyclic load, the microprofiler was capable of producing both haversine and sinusoidal load. The axial loads and displacement were measured by the built-in transducers. A MINC computer was connected to the loading machine for data acquisition.

To control the temperature of the sample, the triaxial cell with sample in it was placed in the temperature chamber which was seated on the MTS machine. The sample was loaded by the extension bar from the load cell through a hole in the roof of the chamber. Liquid carbon dioxide which was connected to the chamber was used to lower the temperature of the samples down to 40°F. The temperature chamber had an accuracy of $\pm$ 2°F.

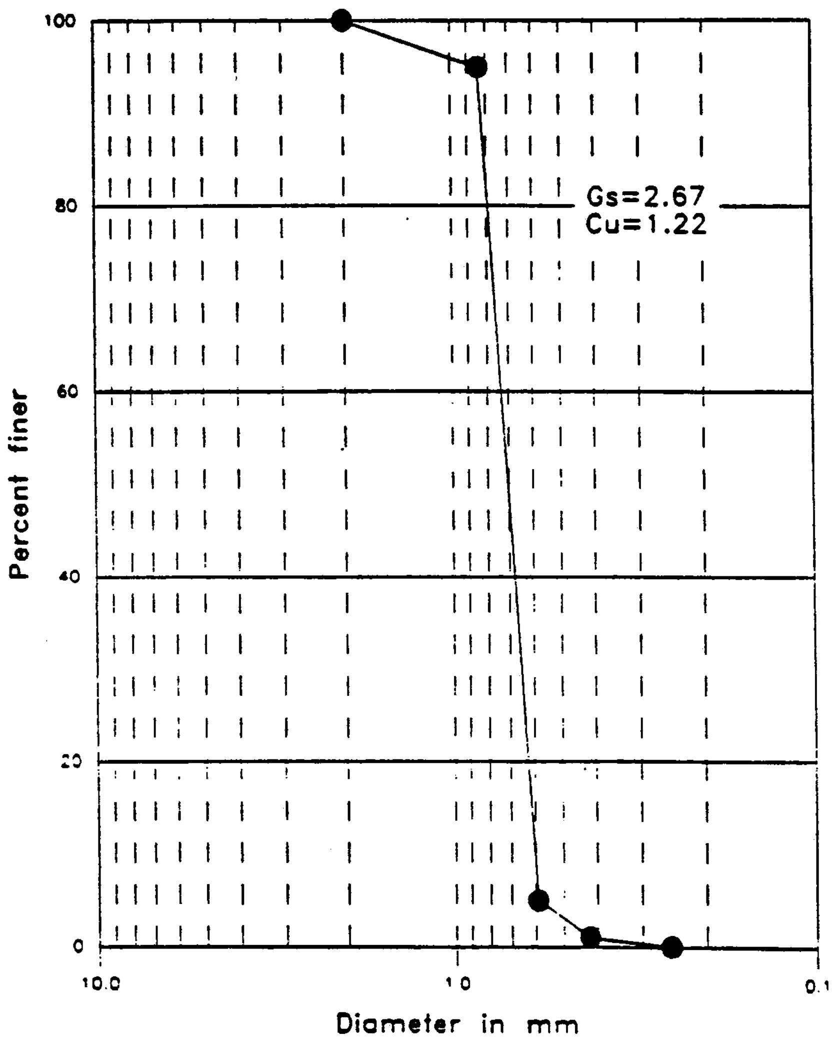

Figure A3.Grain Size Distribution

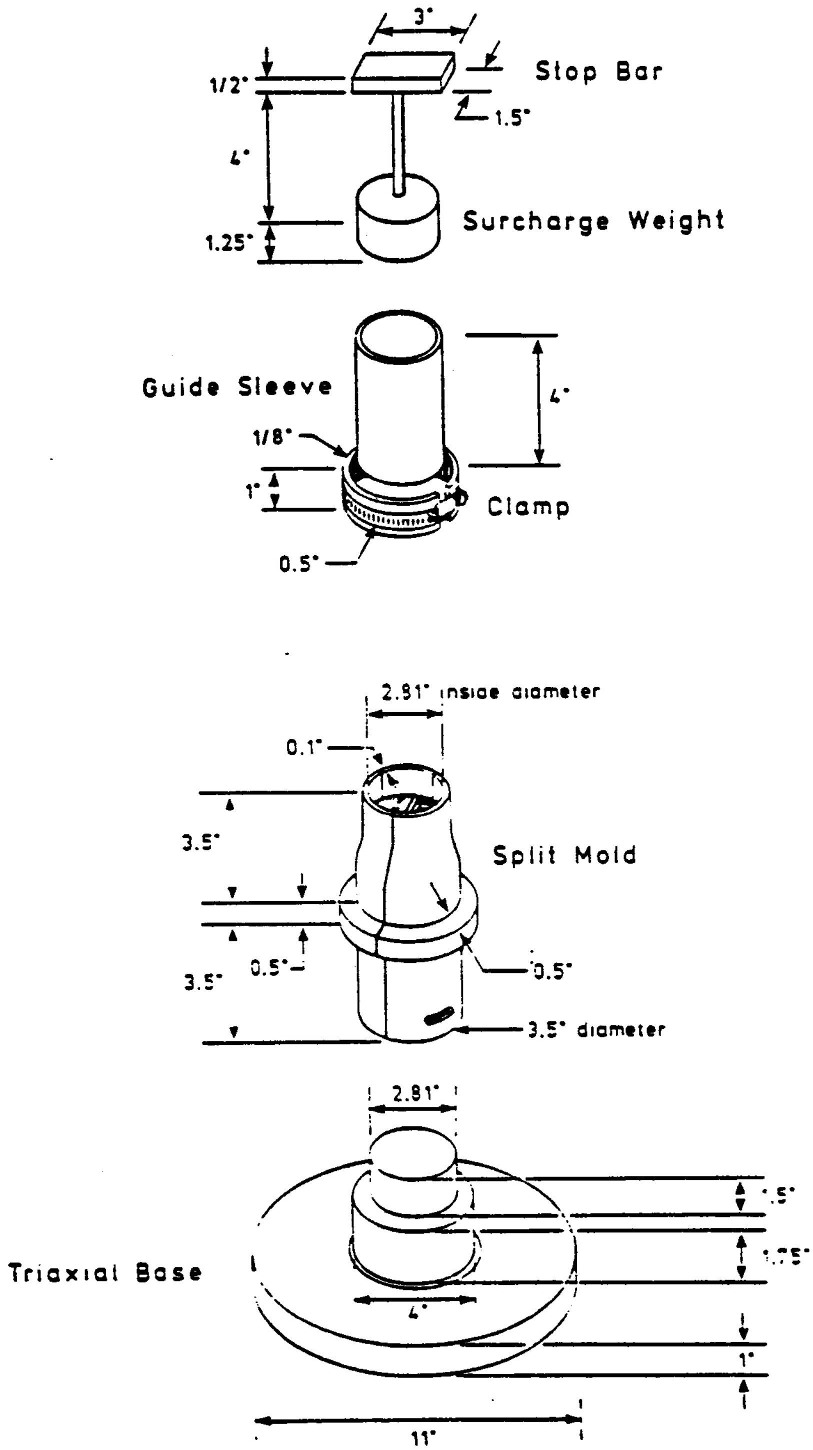

Figure A4. Test Apparatus for Sample Preparation

SAMPLE PREPARATION

Samples were prepared directly on the base of the triaxial cell. This method was found to be necessary, especially for granular materials, because it was not possible to move the sample after preparation from a separate mold to the triaxial base without causing at least some amount of damage. To constitute a test sample, a rubber membrane was mounted on the base of the triaxial cell with two o-rings clamped around it. Two o-rings were used not only to insure that there was no leakage when vacuum was later applied to the sample but also to make the split mold tightly fit into the base, such that when the whole set-up was vibrated the mold and the base would stay together. A steel mold split in three equal segments was clamped around the base of the cell. The membrane was then stretched and folded around the top edge of the mold. Prior to that, the inner wall of the mold was wetted with water to help hold the membrane against the sides of the mold. The guide sleeve was fastened to the top of the mold and the assembly was then placed on the vibratory table.

The correct amount of air-dry sand for each layer was weighed out and poured continuously into the the mold. The assembly was vibrated with the surcharge placed on top of the material until a desired density was reached. Higher density was achieved by increasing the number of layer, degree of vibration, and weight of the surcharge. For this study, the density of all specimens before the application of stresses was approximately 115 pcf. The load cap was then placed on the sample and the membrane was rolled up. Two o-rings were placed around the load cap and the membrane. Before removing the mold, a 3-psi vacuum was applied so that the sample could support its own weight and the weight of the cap. The sample was then ready for testing.

TESTING PROCEDURE

Before proceeding with the stress relaxation tests, duplicate samples were tested to obtain the stress-strain curves. The purpose of the above was to obtain the amount of strain to be applied to the samples for the relaxation tests. As mentioned before, the strain has to be relatively large but not too large to fail the sample. Figure A5 shows three different stress-strain curves and the selected stress levels for samples tested without preconditioning, preconditioned at 5 psi, and preconditioned at 17 psi. The stress levels, corresponding to relatively large strains but less than 2.5 times the confining pressure, met the criteria for selecting the applied strains. After several

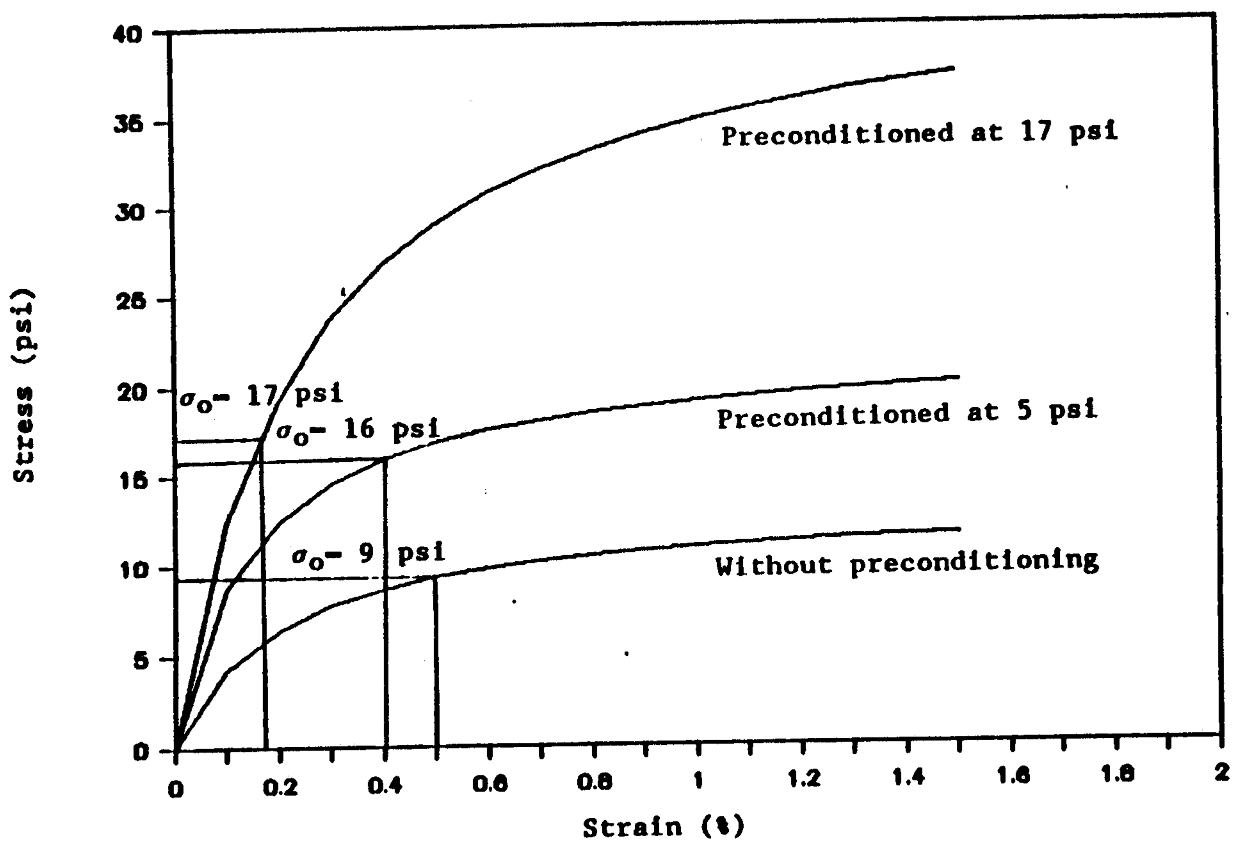

Figure A5. Stress-Strain Curves at Different Preconditioning Level and the Selected Initial Stress tests, it was found that a better control of the initial load on the sample was more easily achieved by loading the sample to the desired initial stress and then holding the strain corresponding to the initial stress constant, rather than loading the sample up to the desired strain. For the first several tests with no preconditioning, the latter method was used and the applied strains were varied from 0.5%, 1% to 2%.

Samples for the relaxation test were placed in the temperature chamber. The triaxial cell was separately heated in an oven for high temperature tests or cooled in the refrigerator for low temperature tests. In this way, the time to reach the temperature equilibrium state after the sample was placed in the triaxial cell was considerably reduced. A confining pressure of 10 psi was then applied to the sample and the vacuum was removed simultaneously.

A preconditioning stress of 5 psi or 17 psi was applied to the sample. This step was skipped for the test without preconditioning. The various levels of preconditioning and applied initial stresses are summarized in Table A2. Axial strain was then applied to the sample. At the desired initial stress the strain was held constant, thus beginning the stress relaxation test. The axial load was continuously monitored by the load cell and recorded by the MINC computer.

TEST RESULTS

The mechanisms of deformation that take place in granular materials comprise (1) interparticle sliding, which occurs when the friction between particles is exceeded, (2) elastic compression of the soil grains, and (3) crushing of grains at interparticle contact points, as a result of localized stresses at contact points exceed the yield strength of the material.

Typical results of relaxation modulus versus time are plotted in logarithm scale in Figure A6. It can be seen that the granular material underwent three distinct phases under the constant strain. The first phase happened in the first few minutes after the beginning of the test and involved mainly with interparticle sliding and particles repositioning to distribute applied load within the soil mass. The second phase involved densification with all mechanisms of deformation took place simultaneously. The third phase involved creep or relaxation.

If the material was not preconditioned or preconditioned at lower stress level, a more random and unpredictable results were observed. The stress paths of the two different loading conditions are illustrated in Figures A7 and A8. At the end of the

Table A2. Stress Levels for the Relaxation Test

Type	Preconditioning level (psi)	Initial stress (psi)
I	none	9
II	5	16
III	17	17

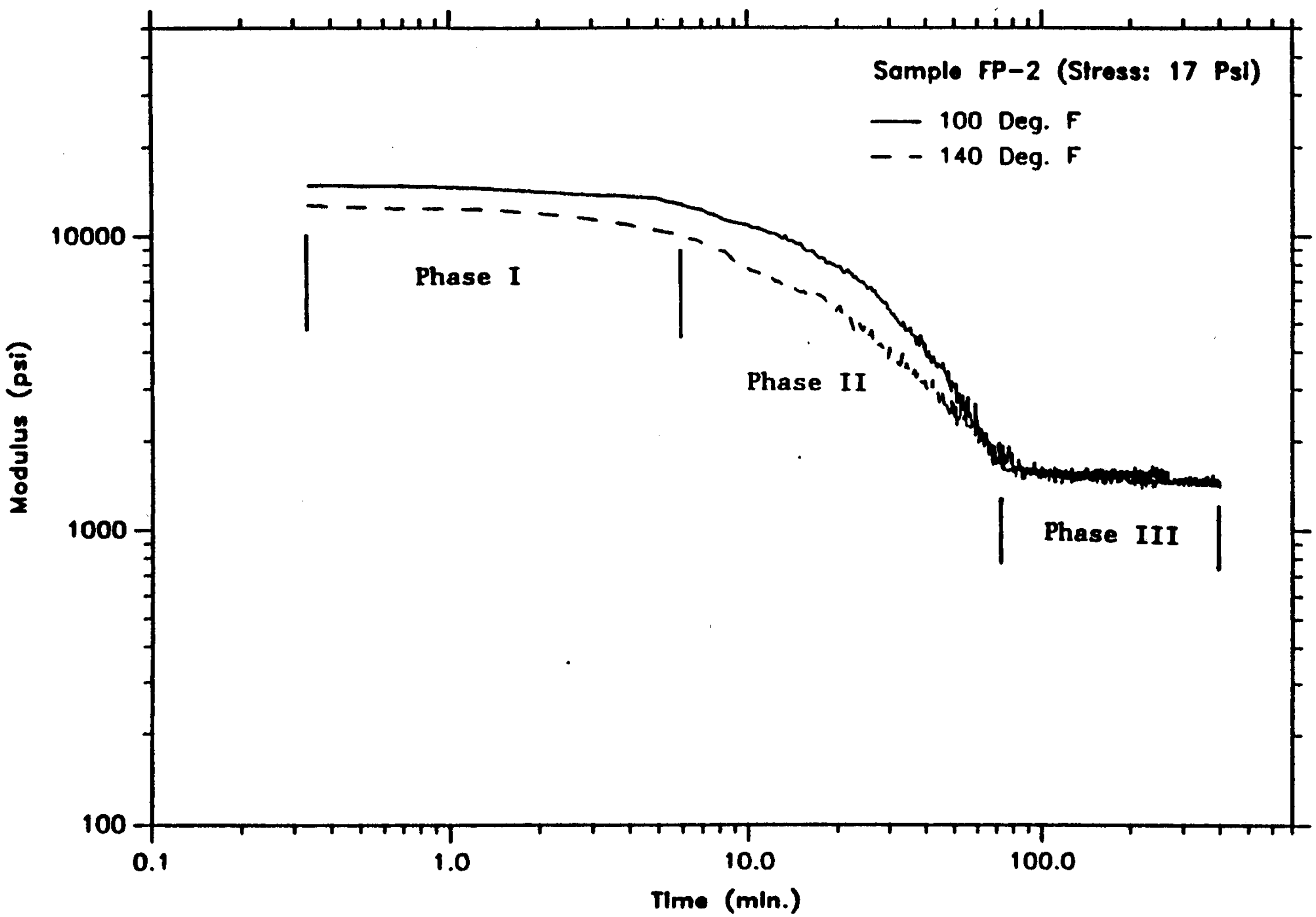

Figure A6. Relaxation Curves at Different Temperatures
(Preconditioned at 17 psi)

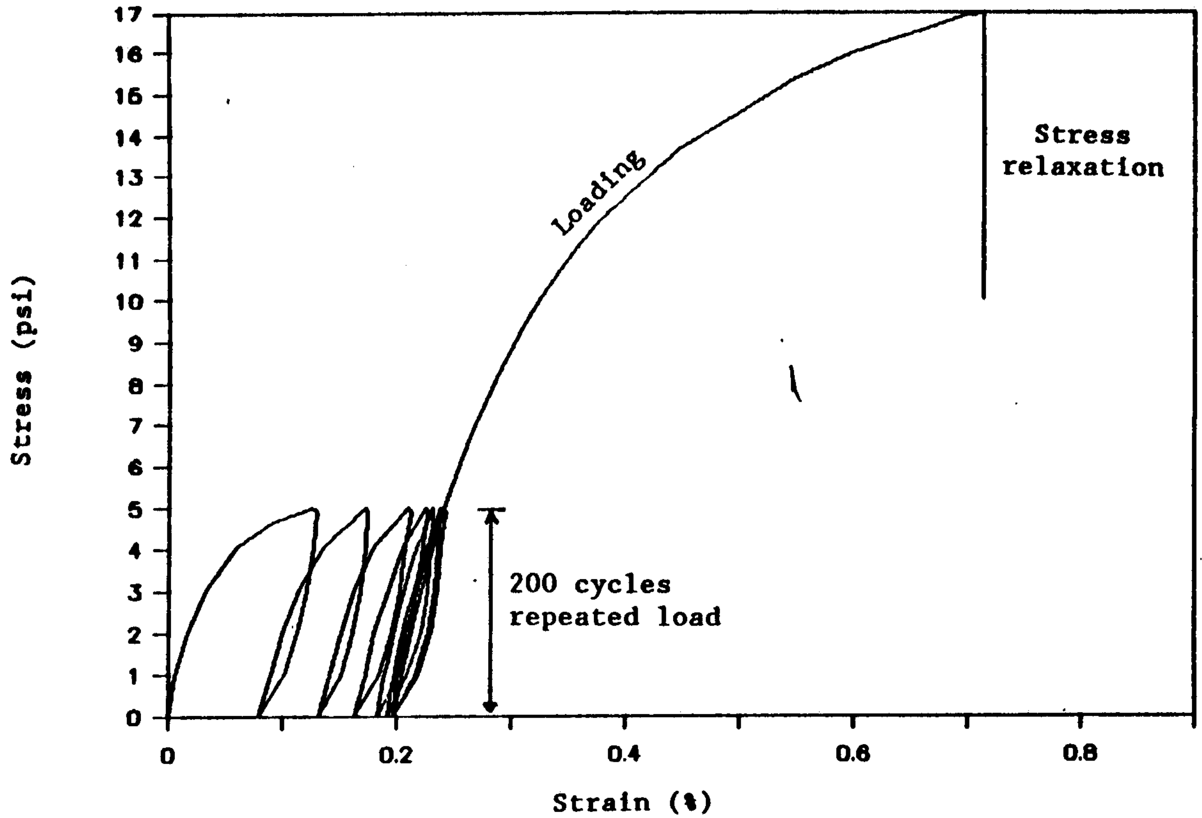

Figure A7. Loading Path of the Samples Preconditioned at a Low Stress Level

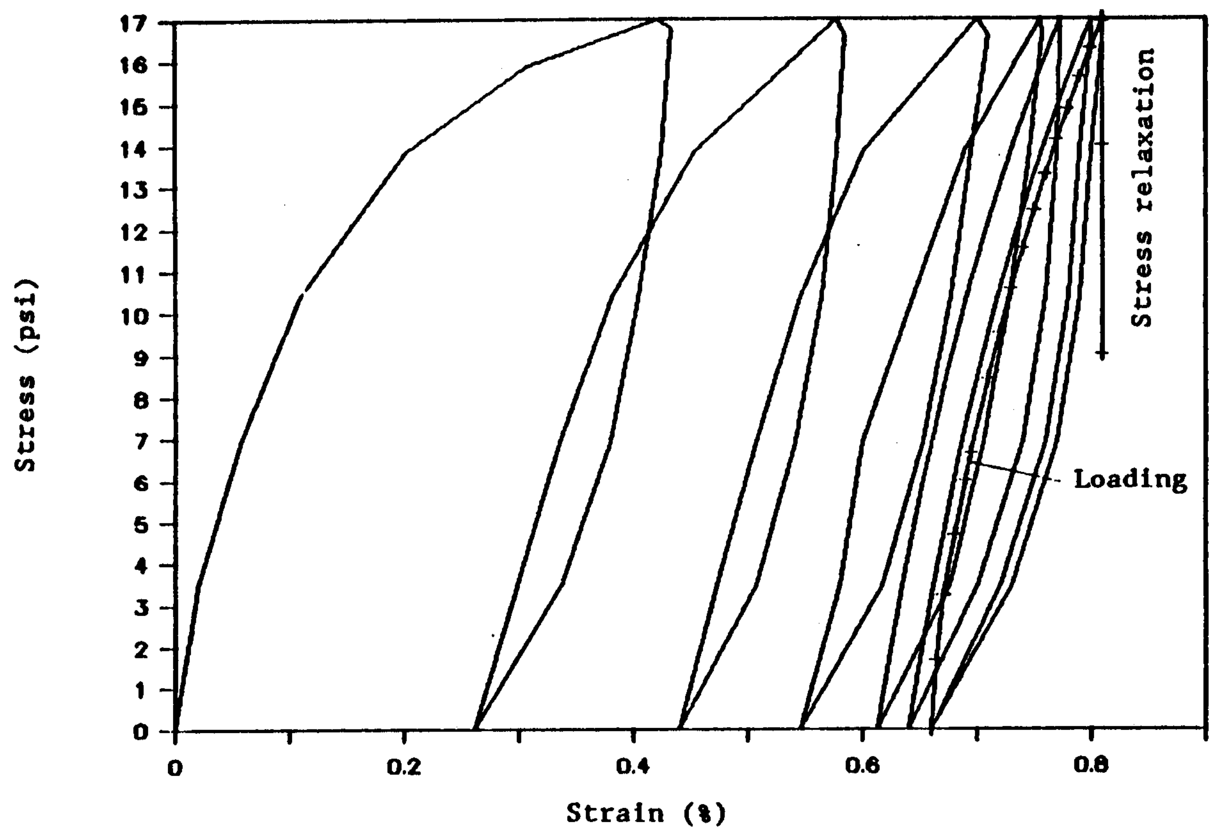

Figure A8. Loading Path of the Samples Preconditioned at a High Stress Level preconditioning process, the samples for the two cases have rearranged into a more stable structure. However, when the sample was loaded to a higher load as shown in Figure A7, the stable structure was destroyed and deformation in the form of particles rearrangement once again took place. The mechanisms of deformation involved were not identical for different samples, thus yielded somewhat random results. The same behavior was observed for the test without preconditioning. In the case where the amount of initial stress at the beginning of the test was the same as the applied preconditioning stress (Figure A8), the stability of the soil structure was not disturbed by the application of the initial stress and, thus, yielded consistent results.

For relaxation tests under different temperatures, the variability of the test results was caused not only by the temperature changes but the variability of the samples as well. To eliminate this error, the same specimen was tested at different temperatures. Since the same initial stress was used and the specimen had reached its stable state, the reloading path will be parallel to the first loading path. As such any changes in the test results were due to temperature changes. However, enough time had to be given between loading cycles to allow the temperature of the sample to reach equilibrium and to let the deformation recover. As shown in the loading path in Figure A9, there exist deformation right after the load was removed. This deformation was partially recoverable because the elastic energy stored during loading would cause the soil skeleton to expand. At the same time, partial reversal of the sliding which took place during the load application would occur.

The method of using the same sample for different temperature tests was further justified by the finding of Lacerda and Houston (1973) who performed repeated stress relaxation test on a single specimen with different strain rates. Identical specimens were tested in the reversed order of strain rates and it was found that the order of application of the strain rate did not affect the result of the tests. Repeated stress relaxation tests could be performed on a single specimen, and each test exhibited the same behavior.

Stress relaxation rates were obtained by performing regression analysis on the third phase of the curves. The results are given in Appendix B. The n values range from 0.0122 to 0.0476. However, temperature changes did not seem to have prominent effects on the relaxation behavior.

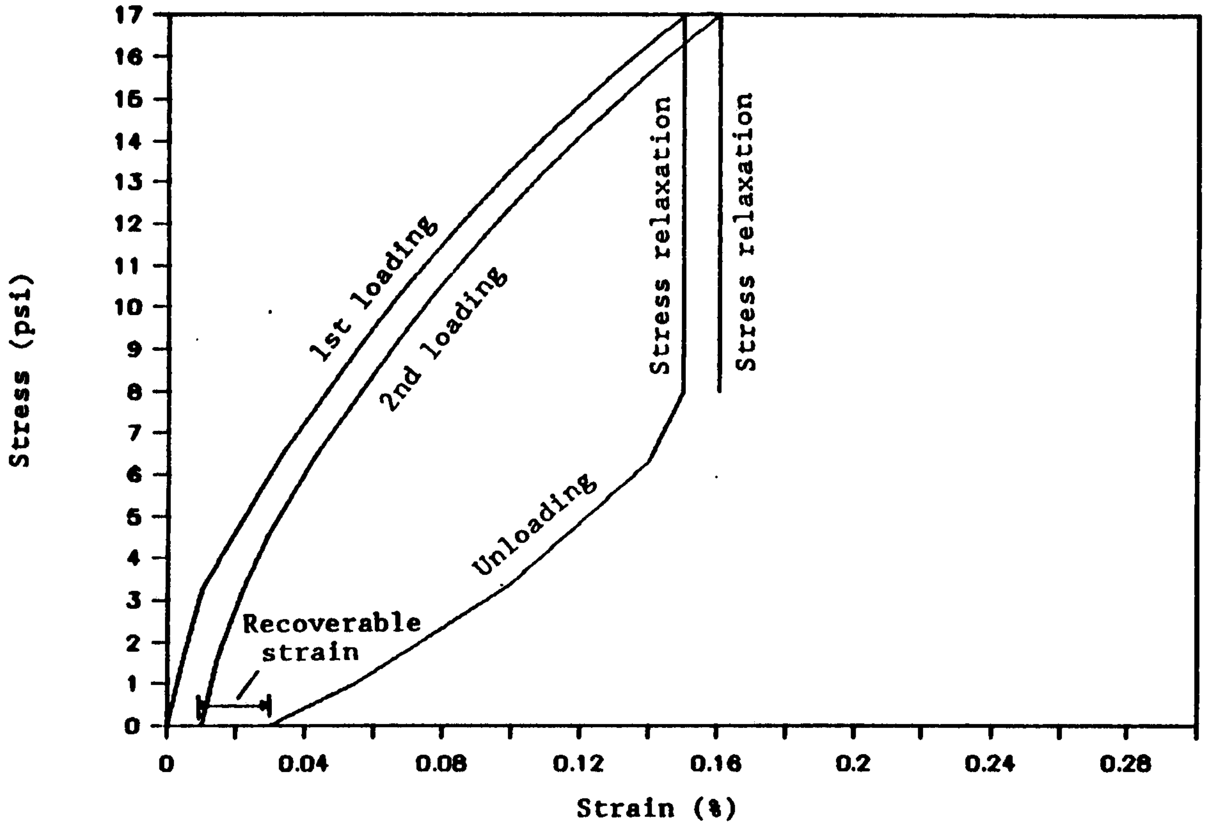

Figure A9. Loading Path for Stress Relaxation Testing at Different Temperature

CONCLUSION

The stress-strain curve of a material should be obtained before a relaxation test is performed, so that the amount of the applied initial stress could be derived. The sample should be preconditioned up to the desired initial stress level for appoximately 25 cycles or until no permanent deformation is observed between cycles. The purpose of this procedure is to eliminate the inconsistency of the test results and to simulate the field condition where the base course materials are subjected to repeated loading. The same sample can be used for tests at different temperatures, as long as the applied initial stress is not changed. Using the power law formulation, the relaxation rate, n, of air-dry Ottawa sand was found to be between 0.0122 and 0.0476. A lack of temperature dependency of the stress relaxation behavior of the Ottawa sand was observed.

In the development of the theoretical model, the relaxation behavior due to temperature changes was not considered since it was relatively insignificant compared to the thermal expansion of the material.

Appendix B - Relaxation Curves of Ottawa Sand

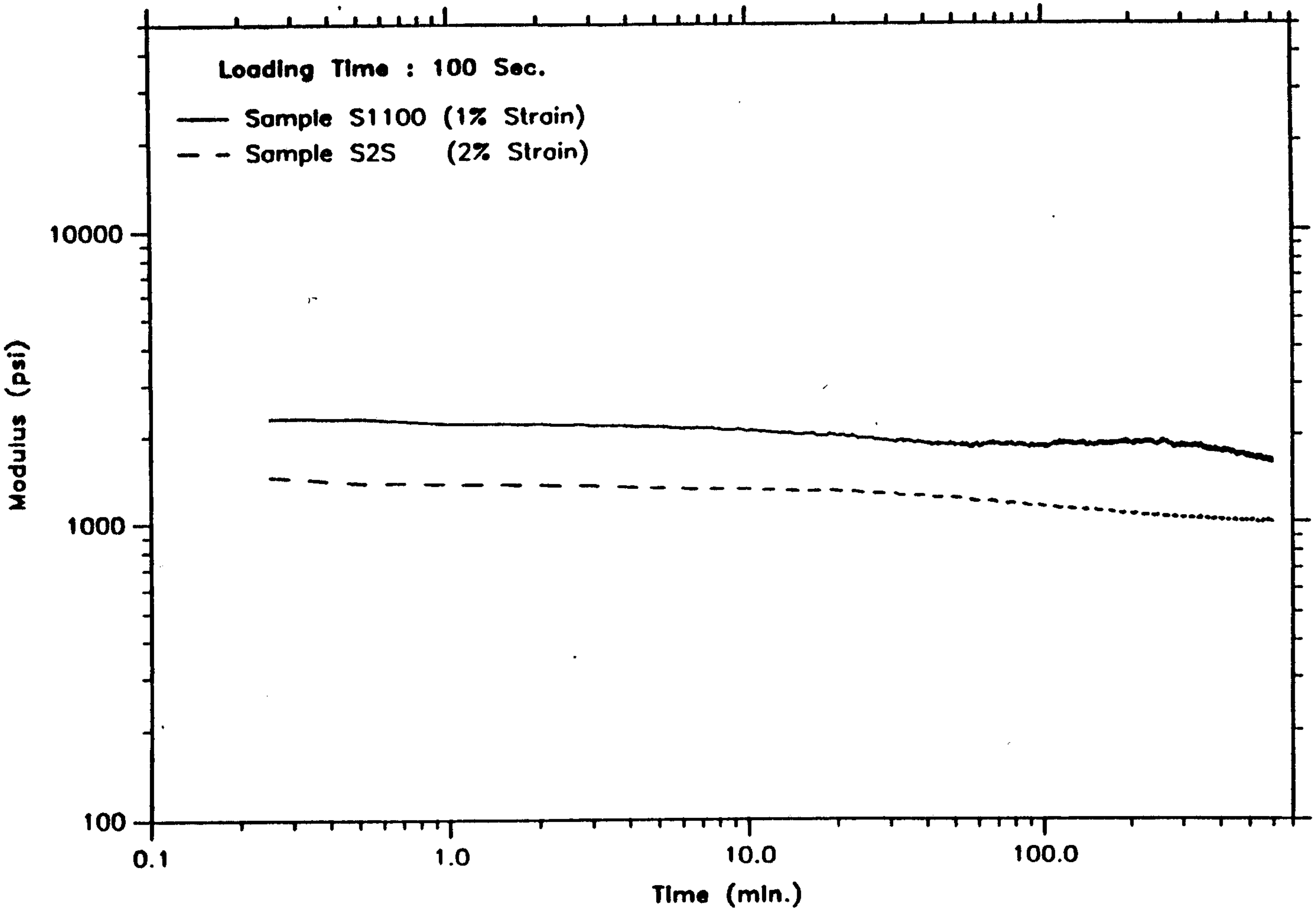

Fig. B1. Relaxation Curve at Room Temperature (Without Preconditioning)

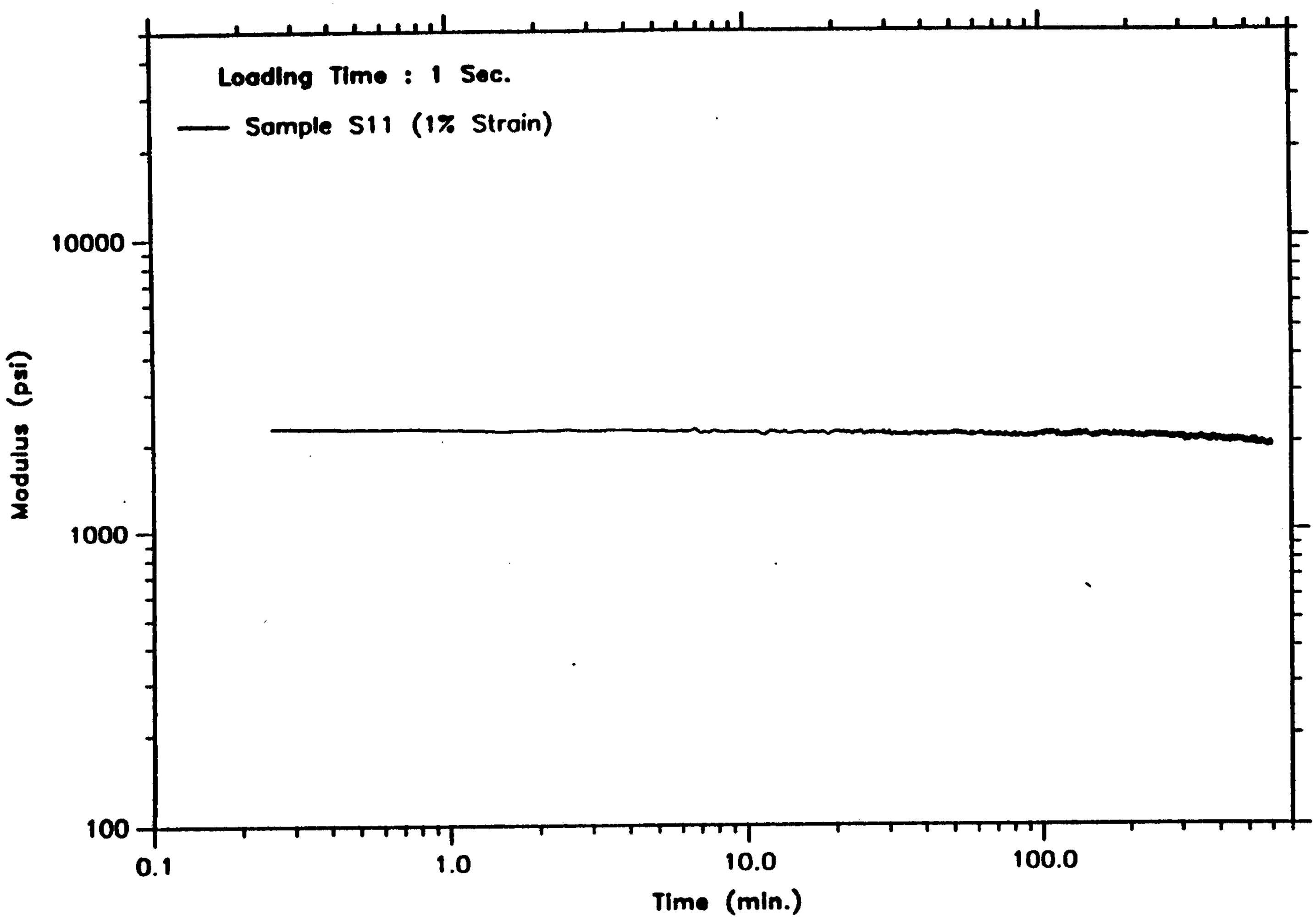

Fig. B2. Relaxation Curve at Room Temperature (Without Preconditioning)

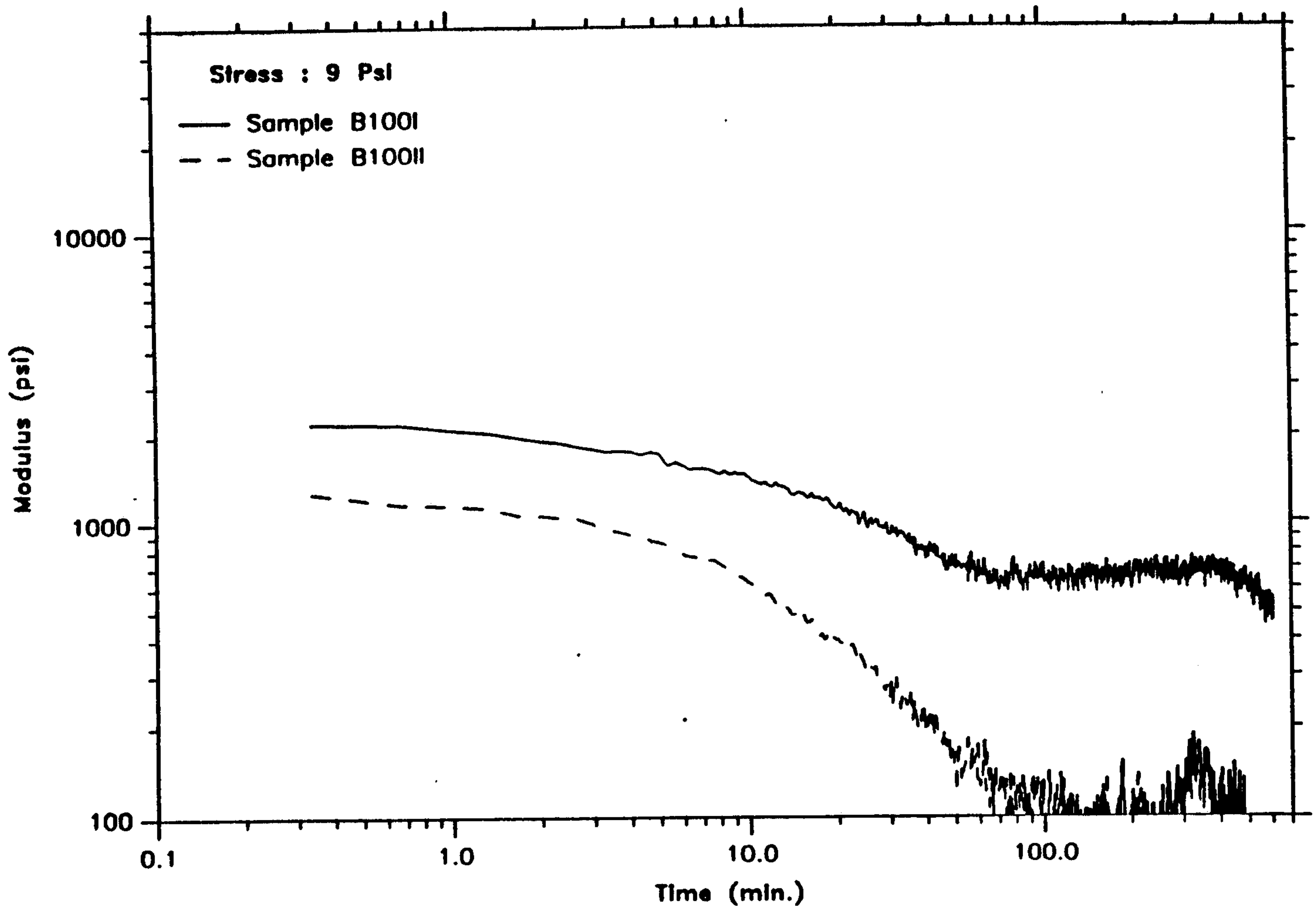

Fig. B3. Relaxation Curve at 100 Degree Fahrenheit (Without Preconditioning)

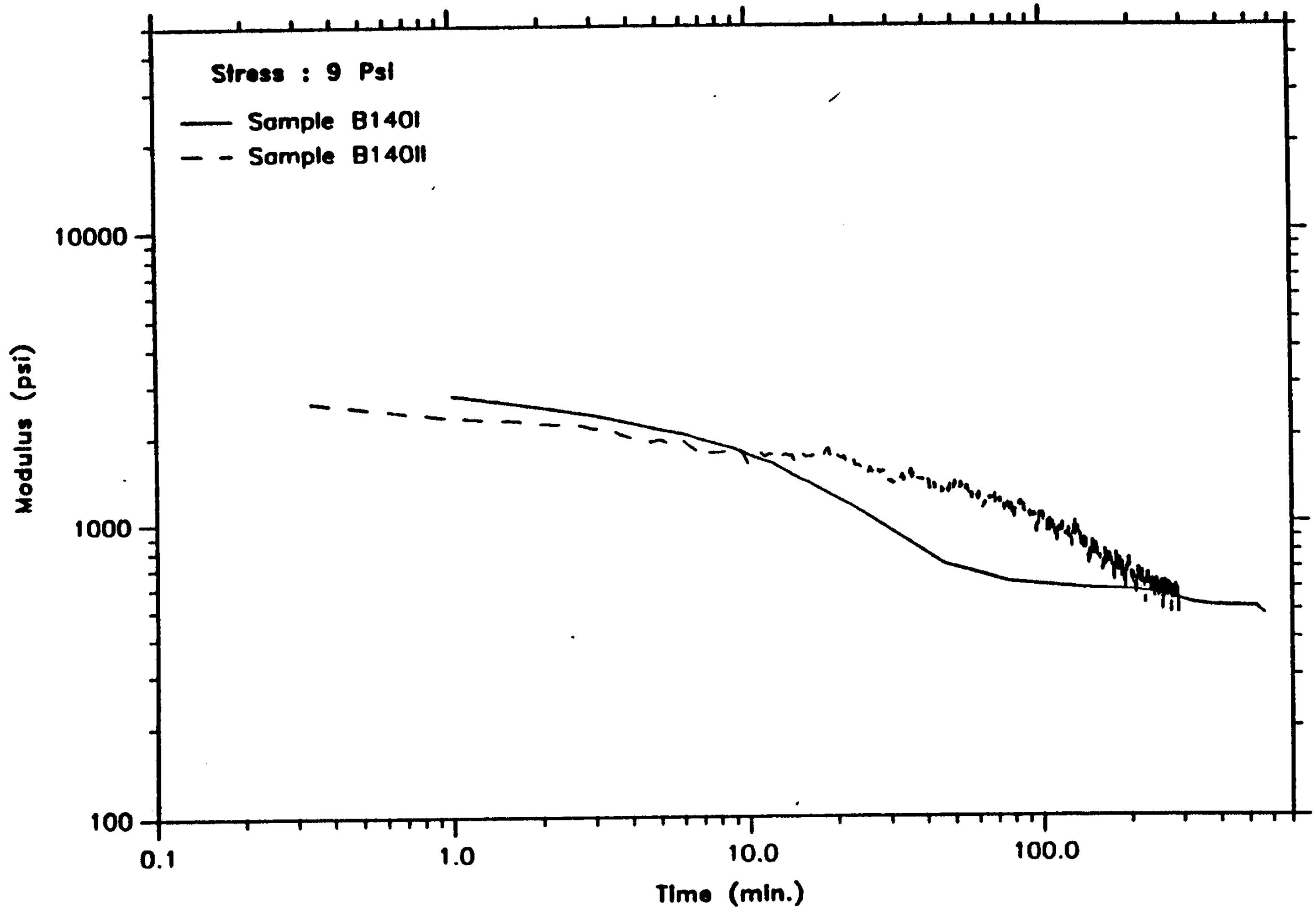

Fig. B4. Relaxation Curve at 140 Degree Fahrenheit (Without Preconditioning)

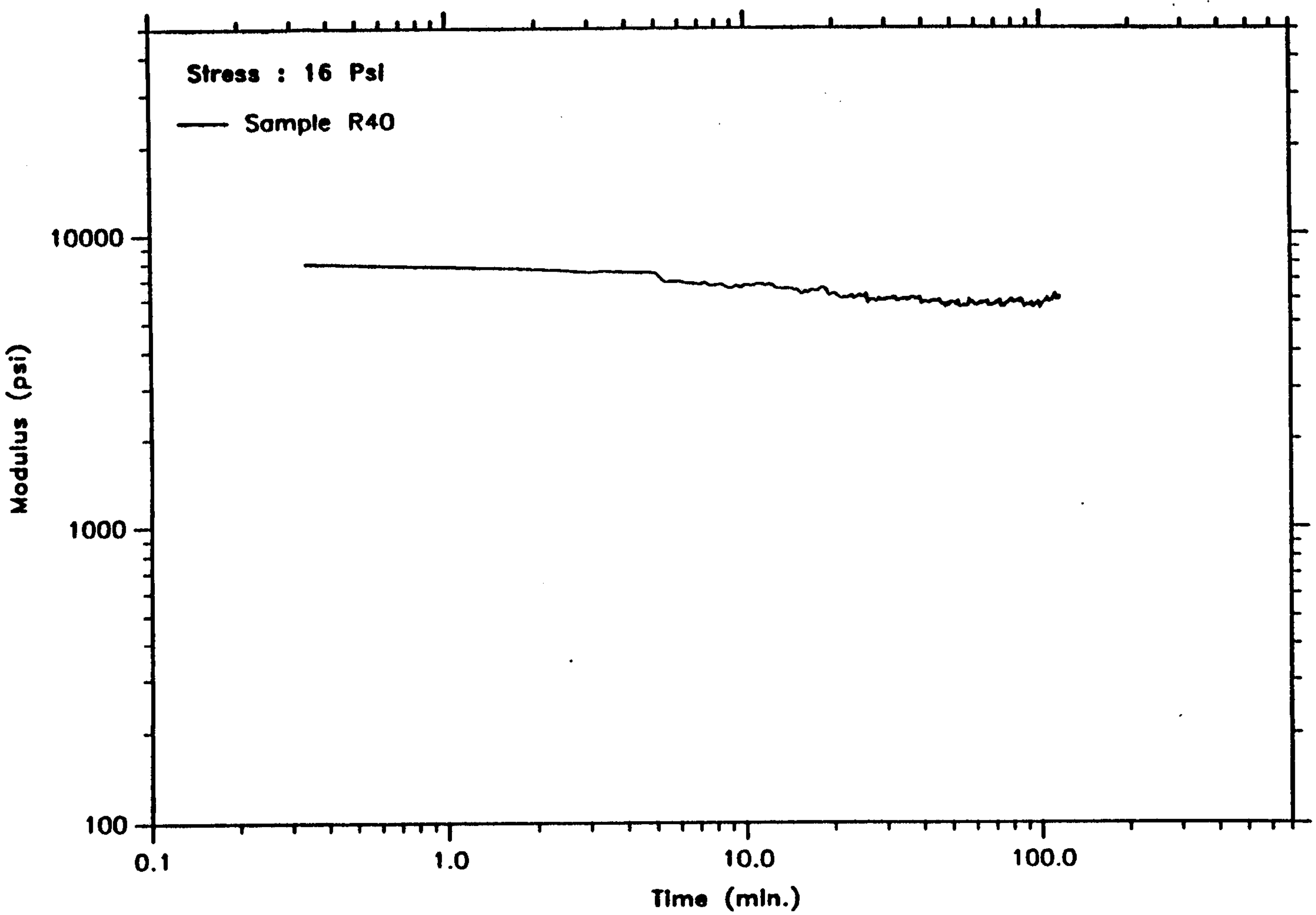

Fig B5. Relaxation Curve at 40 Degree Fahrenheit (Preconditioned at 5 Psi)

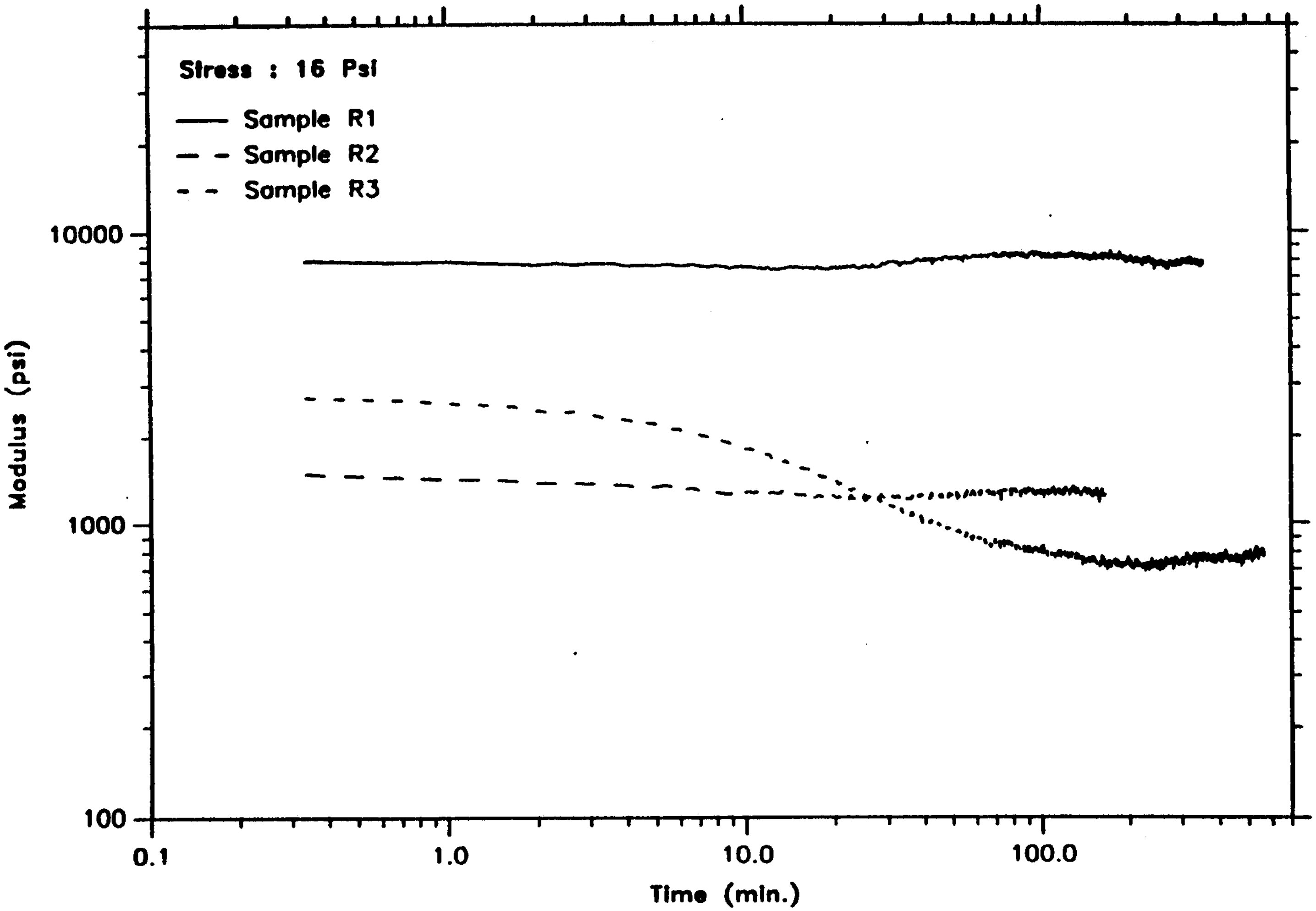

Fig. B6. Relaxation Curve at Room Temperature (Preconditioned at 5 Psi)

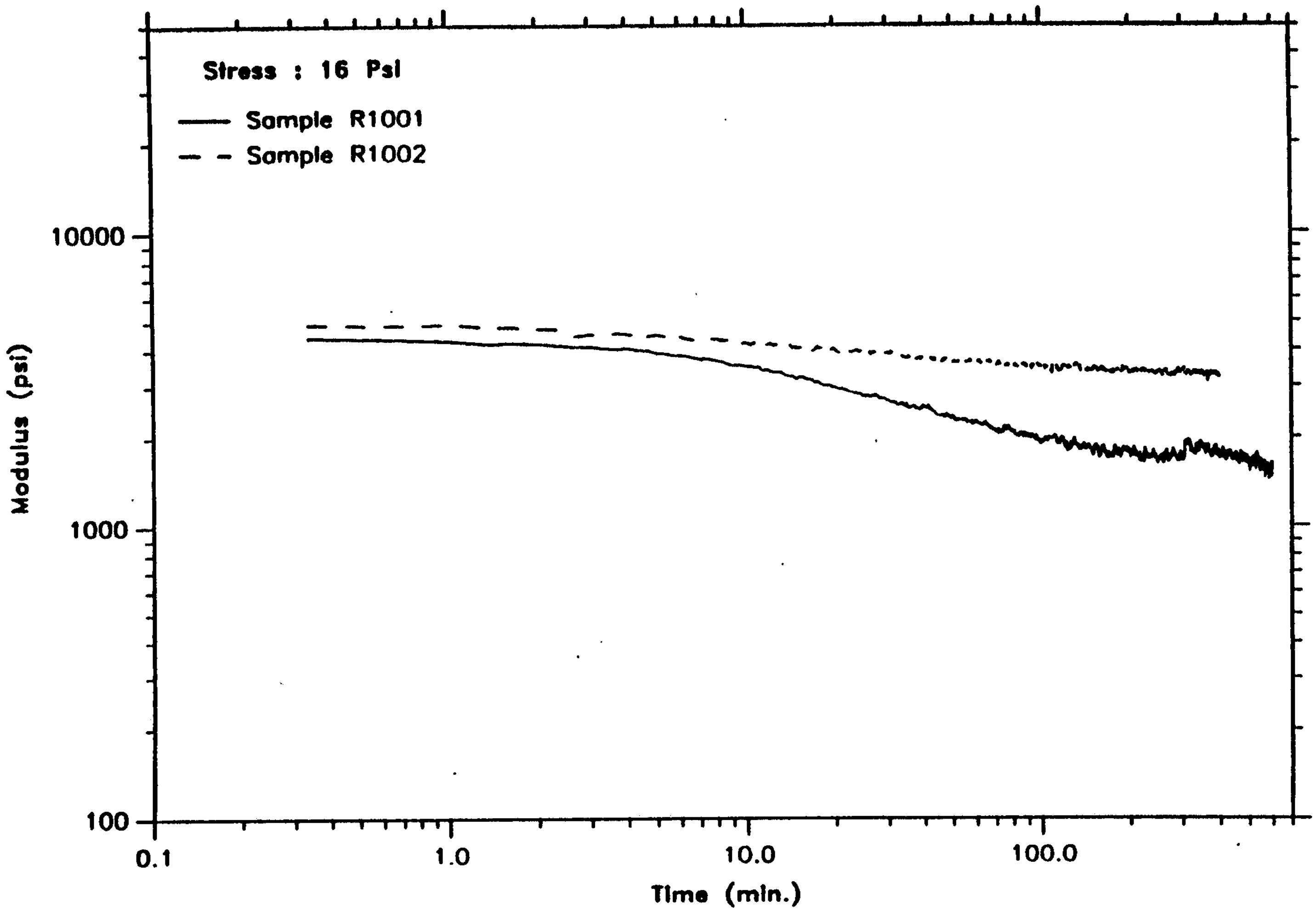

Fig. B7. Relaxation Curve at 100 Degree Fahrenheit (Preconditioned at 5 Psi)

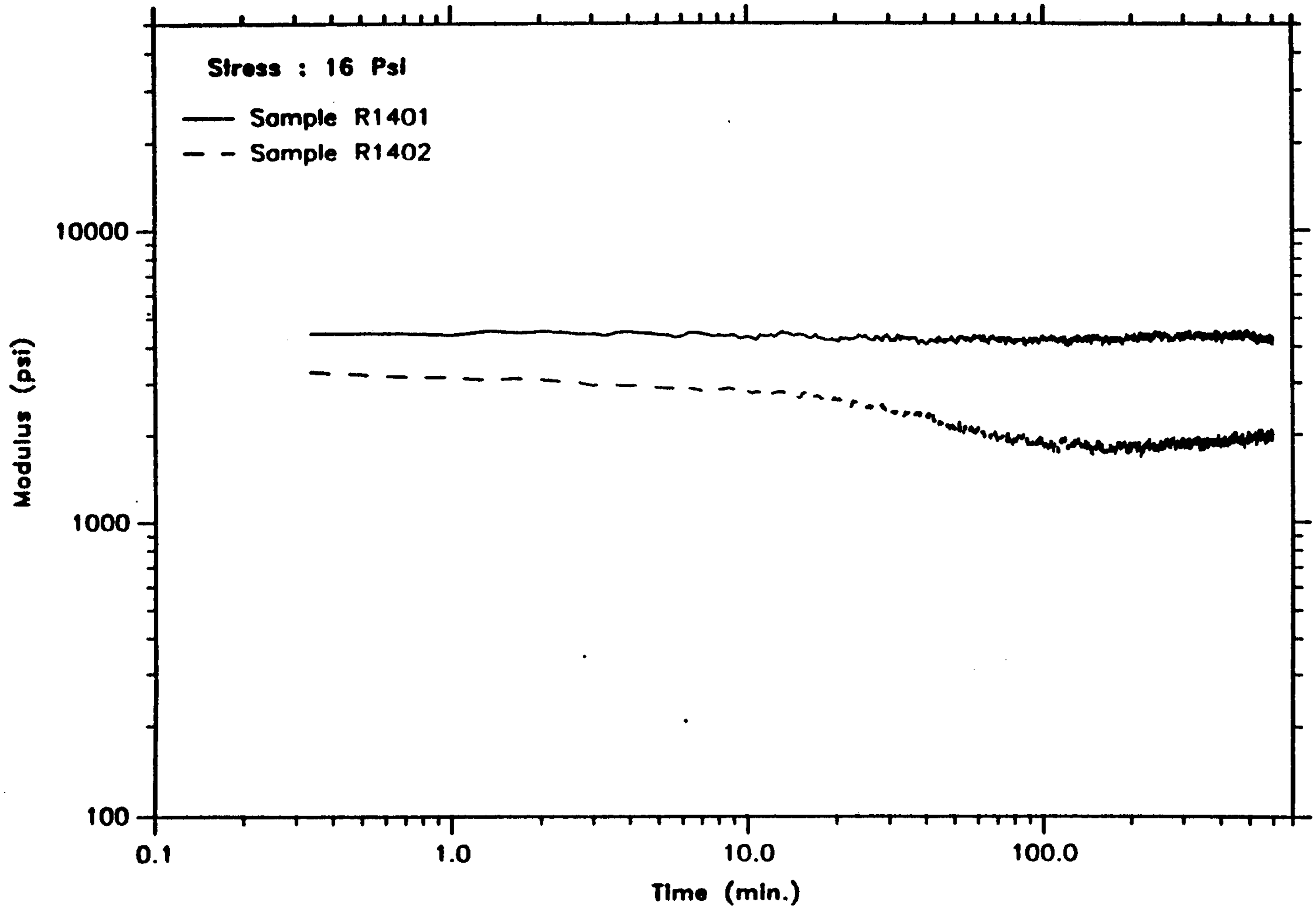

Fig. B8. Relaxation Curve at 140 Degree Fahrenheit (Preconditioned at 5 Psi)

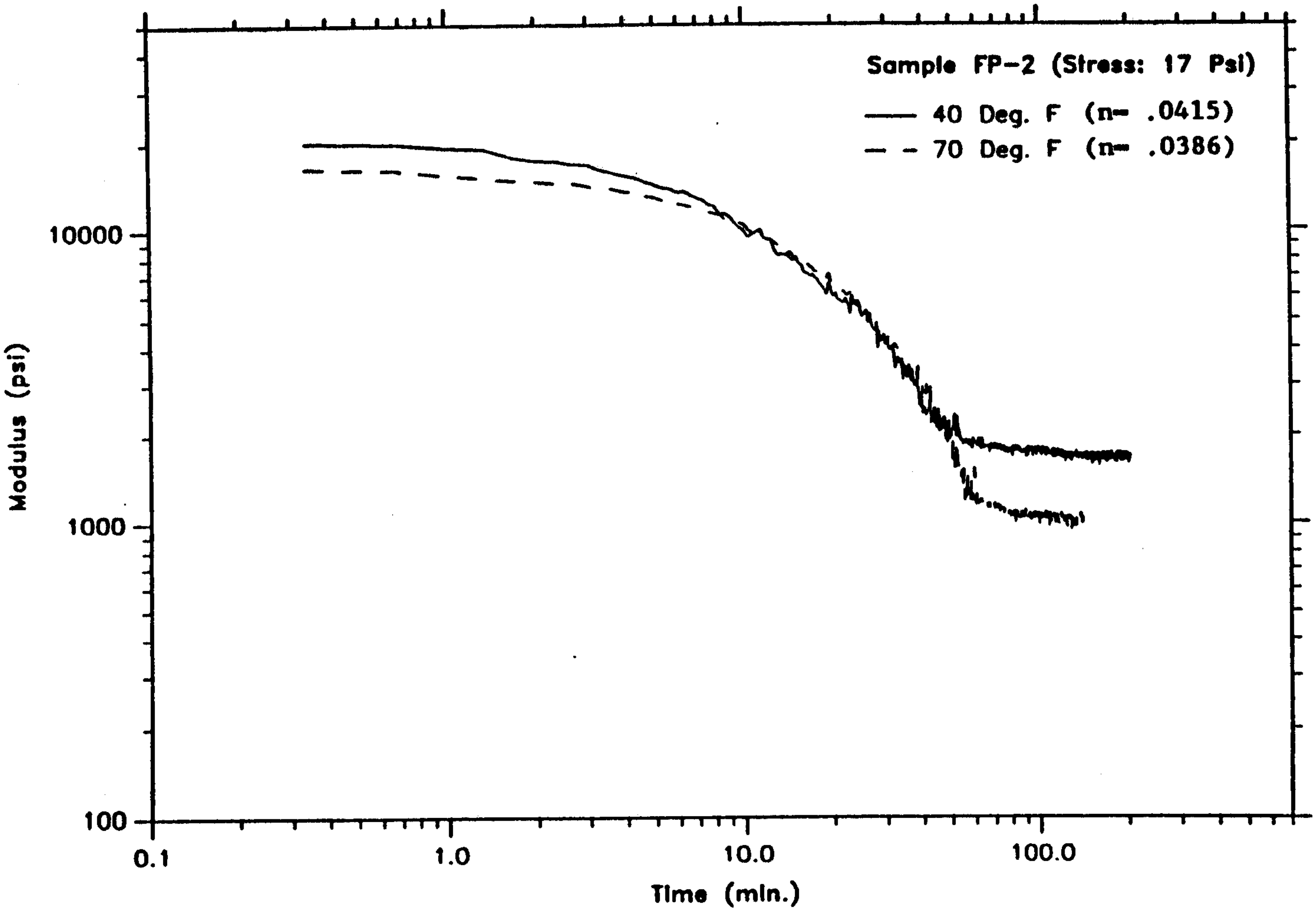

Fig. B9. Relaxation Curve at Different Temperatures (Precondition at 17 Psi)

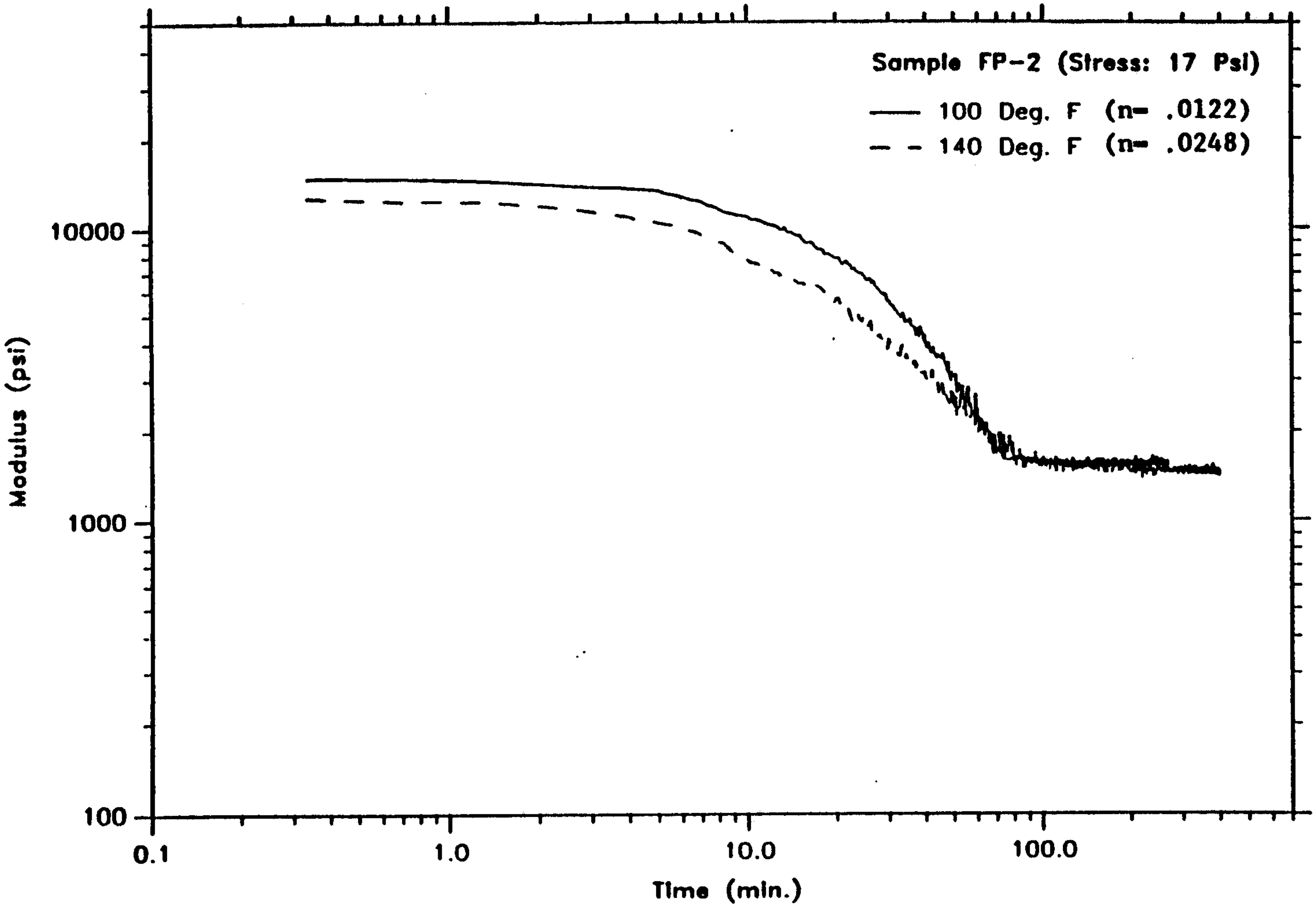

Fig. B10. Relaxation Curves at Different Temperatures (Preconditioned at 17 Psi)

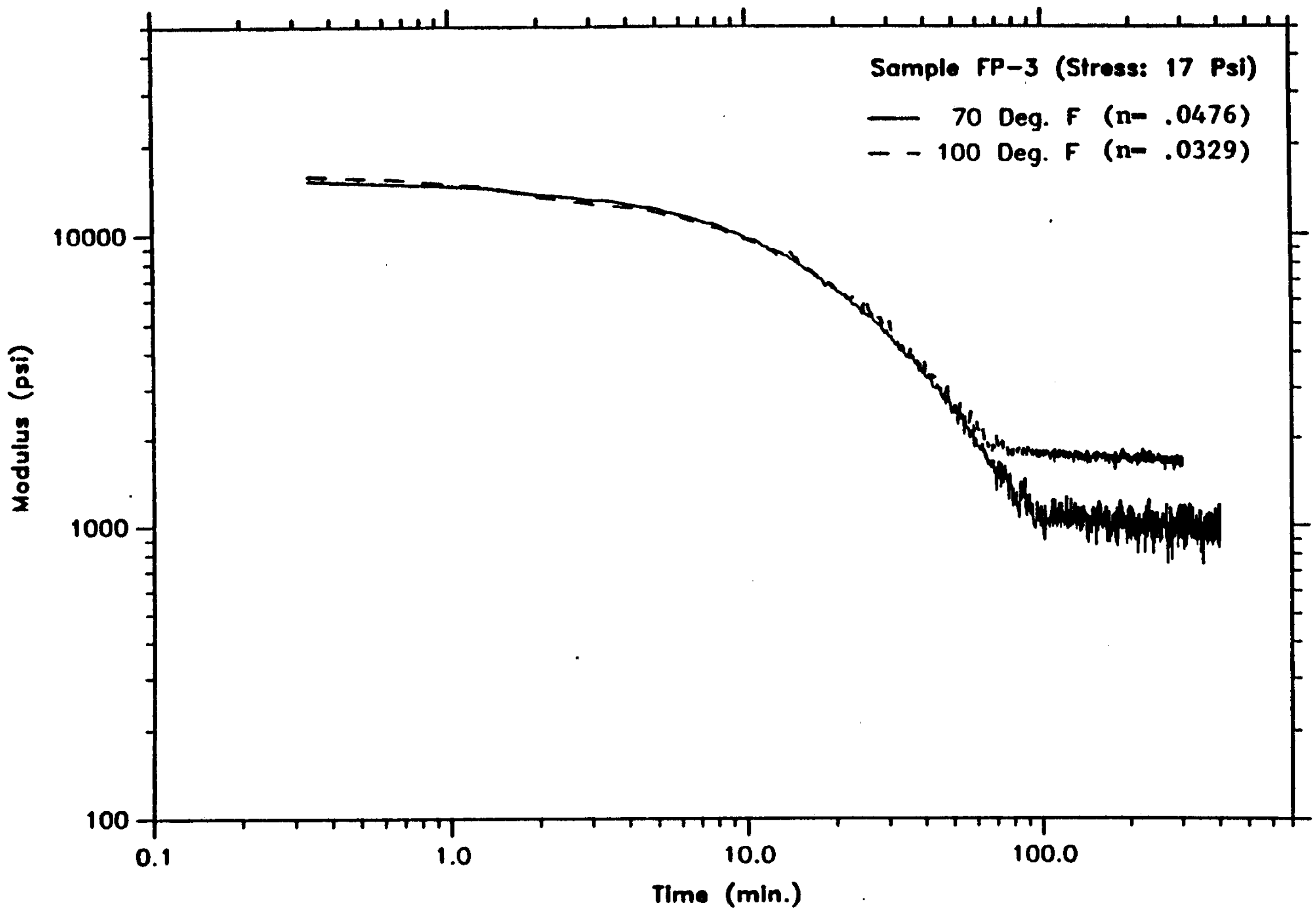

Fig. B11. Relaxation Curves at Different Temperatures (Preconditioned at 17 Psi)

Appendix C - Field Test Results

Table C1. Test Results From FM1235 (District 8)

Month	Base Course			Subgrade		
	Suction (bar)	Temperature (deg. F)	Modulus (psi)	Suction (bar)	Temperature (deg. F)	Modulus (psi)
Oct	-5.01	76	109989.4	-5.48	80	6346.6
Nov	-4.49	62	112481.6	-3.21	70	6842.6
Dec	-7.42	58	111548.3	-11.94	60	7117.4
Jan	-	-	-	-	-	-
Feb	0	54	100000.1	0	55	7326.4
Mar	-2.74	67	93432.8	-3.21	64	6792.4
Apr	-3.71	77	91535.1	-4.79	86	7188.1
May	-8.99	73	90593.5	-4.85	80	6575.2
Jun	-15.23	83	119781.9	-4.06	85	6878.5
Jul	0	101	78011.7	-3.27	92	7147.8
Aug	-17.58	98	55364.8	-5.24	96	5186.7
Sep	0	101	62181.1	0	91	7190.4
Oct	-8.44	70	86044.2	-4.22	76	6930.1

Note: 1 bar = 14.505 psi = 100 kPa

Table C2. Test Results from FM1983 (District 8)

Month	Base Course			Subgrade		
	Suction (bar)	Temperature (deg. F)	Modulus (psi)	Suction (bar)	Temperature (deg. F)	Modulus (psi)
Oct	-0.759	70	73954.64	-	64	7032.3
Nov	-2.254	60	70511.63	-	67	7165.9
Dec	-0.657	48	62451.33	-	59	6876.3
Jan	-	-	-	-	-	-
Feb	-	57	64070.47	-	53	7166.8
Mar	0	68	67894.06	-	64	7214.6
Apr	-	66	63482.41	-	69	7531.3
May	-2.553	71	71107.51	-	77	7542.2
Jun	0	91	72420.56	-	93	7652.1
Jul	-3	104	70387.69	-	93	7610.7
Aug	-0.83	97	32162.51	-	95	6614.4
Sep	-11.61	94	62180.95	-	93	7190.4
Oct	-	84	64321.14	-	84	7136.3

Note: 1 bar = 14.505 psi = 100 kPa

Table C3. Test Results from FM2864 (District 11)

Month	Base Course			Subgrade		
	Suction (bar)	Temperature (deg. F)	Modulus (psi)	Suction (bar)	Temperature (deg. F)	Modulus (psi)
Oct	-	79	34493.25	-	75	10955
Nov	-	-	31541.03	-	-	-
Dec	-	66	31541.03	-	60	9535
Jan	-	61	27847.17	-	52	9481
Feb	-	57	22675.92	-	57	9693
Mar	-	64	30089.81	-	69	10047
Apr	-	70	27932.25	-	76	10008
May	-	69	31531.09	-	80	10361
Jun	-	97	32671.01	-	95	10849
Jul	-	92	40135.92	-	89	11283
Aug	-	106	39366.81	-	108	11977
Sep	-	-	-	-	-	-
Oct	-	83	44340.19	-	87	11098

Note: 1 bar = 14.505 psi = 100 kPa

Table C4. Test Results from SH7 (District 11)

Month	Base Course			Subgrade		
	Suction (bar)	Temperature (deg. F)	Modulus (psi)	Suction (bar)	Temperature (deg. F)	Modulus (psi)
Oct	-0.071	68	111557.3	-0.126	70	6270.7
Nov	-	-	-	-	-	-
Dec	-0.021	55	105000.1	-0.093	55	6520.4
Jan	0	51	92578.3	-0.093	49	6380.1
Feb	0	57	92578.3	-0.071	59	6380.1
Mar	-0.086	71	109793.8	-0.093	71	6304.1
Apr	-0.222	73	114764.3	-0.182	64	6109.8
May	-0.522	80	126099.4	-0.325	69	6139.9
Jun	0	100	143006.8	-1.099	99	6229.7
Jul	-0.036	100	141670.2	-0.078	78	6952.3
Aug	0	103	147788.1	-0.372	95	6745.6
Sep	-		-	-	-	-
Oct	0	76	128851.1	-0.069	60	7185.9

Note: 1 bar = 14.505 psi = 100 kPa

Table B3. Test Results from FM2864 (District 11)

Table C5. Test Results from FM491 (District 21)

Month	Base Course			Subgrade		
	Suction (bar)	Temperature (deg. F)	Modulus (psi)	Suction (bar)	Temperature (deg. F)	Modulus (psi)
Oct	-0.410	111	39942.18	-0.243	88.5	5220.3
Nov	-0.506	99	35880.84	-0.328	90	5206.4
Dec	-0.011	90	44983.57	-0.232	83	5227.4
Jan	0	54	51864.78	-0.275	58	5135.3
Feb	0	72	50616.58	0	62	5067.8
Mar	0	105	46198.5	-0.111	76.5	5155.7
Apr	-0.678	107	38018.82	-1.015	81.5	5139.0
May	-0.096	114	34330.9	-0.791	94.5	5183.4
Jun	-0.025	115	57251.05	-0.229	98	5129.7
Jul	0	126	56038.84	-0.014	102	5153.1
Aug	0	116	51822.54	0	99	5153.2
Sep	-	-	-	-	-	-
Oct	-0.053	106	40080.35	0	90	5111.0

Note: 1 bar = 14.505 psi = 100 kPa

Table C6. Test Results from FM497 (District 21)

Month	Base Course			Subgrade		
	Suction (bar)	Temperature (deg. F)	Modulus (psi)	Suction (bar)	Temperature (deg. F)	Modulus (psi)
Oct	-0.274	98	48916.39	-0.771	91	5325.1
Nov	-0.325	100	45576.77	-0.281	88	5335.7
Dec	-0.253	85	48735.81	0	79	5301.5
Jan	-0.202	54	58501.71	0	68	5189.8
Feb	-0.13	63	52545.94	0	67	5158.2
Mar	0	94	42490.05	-0.116	78	5289.7
Apr	0	91	46702.81	-0.401	83	5278.5
May	0	104	51839.87	-0.189	92	5312.1
Jun	0	107	51356.77	-0.501	98	5223.7
Jul	0	109	44056.05	-0.549	100	5306.3
Aug	0	114	46333.42	-0.378	98	5308.0
Sep	-	-	-		-	-
Oct	0	96	46128.43	0	90	5242.6

Note: 1 bar = 14.505 psi = 100 kPa

Appendix D - Listing of the Computer Program

```
10 CLS
20 PRINT:PRINT:PRINT:PRINT
30 PRINT TAB(20) "============================================"
40 PRINT TAB(20) "|                                          |"
50 PRINT TAB(20) "|     THIS PROGRAM PREDICTS THE MODULUS    |"
60 PRINT TAB(20) "|     OF GRANULAR MATERIALS AT DIFFERENT   |"
70 PRINT TAB(20) "|     TEMPERATURES AND MOISTURES BASED     |"
80 PRINT TAB(20) "|     ON A MICROMECHANICAL APPROACH        |"
90 PRINT TAB(20) "|                                          |"
100 PRINT TAB(20) "============================================"
110 PRINT:PRINT:PRINT:PRINT TAB(25)
120 INPUT "PRESS ANY KEY TO CONTINUE........";PRESS
130 CLS
140 '
150 '=======INPUT========
160 PRINT:PRINT:PRINT
170 PRINT TAB(25) "TYPE OF MATERIAL:" : PRINT
180 PRINT TAB(25) "  1 = LIMESTONE"
190 PRINT TAB(25) "  2 = BASALT, GRANITE, DOLOMITE"
200 PRINT TAB(25) "  3 = CHERT, QUARTZITE, SANDSTONE   "
210 PRINT TAB(25) : INPUT MTYPE
220 IF (MTYPE<=0 OR MTYPE>3) GOTO 130
230 IF (MTYPE=1) THEN ALP=5*10^(-6) : E=6.4*10^6 : U=.17 : K1=14000 :K2=.4
240 IF (MTYPE=2) THEN ALP=6.5*10^(-6) : E=7.8*10^6 : U=.2 :K1=24000 :K2=.37
250 IF (MTYPE=3) THEN ALP=5*10^(-6) : E=8.534001*10^6 : U=.17:K1=7210:K2=.45
260 CLS : PRINT : PRINT
270 PRINT TAB(20) "PROPERTIES OF THE MATERIAL SELECTED:"
280 PRINT
290 PRINT TAB(25) "1. LINEAR THERMAL EXPANSION = ";ALP
300 PRINT TAB(25) "2. ELASTIC MODULUS          = ";E
310 PRINT TAB(25) "3. POISSON'S RATIO          = ";U
320 PRINT TAB(25) "4. K1                       = ";K1
330 PRINT TAB(25) "5. K2                       = ";K2
340 PRINT: PRINT
350 PRINT TAB(20):INPUT "DO YOU WANT TO CHANGE? 1=YES  2=NO ";CHOICE
360 IF (CHOICE <> 1 AND CHOICE <> 2) THEN GOTO 350
370 IF (CHOICE=2) GOTO 460
380 PRINT : PRINT TAB(20) : INPUT "WHICH LINE ";WL
390 PRINT TAB(20) : INPUT "INPUT NEW VALUE "; NV
400 IF (WL=1) THEN ALP=NV
410 IF (WL=2) THEN E=NV
420 IF (WL=3) THEN U=NV
430 IF (WL=4) THEN K1=NV
440 IF (WL=5) THEN K2=NV
450 GOTO 260
460 CLS : PRINT:PRINT:PRINT TAB(25)
470 INPUT "UNIT WEIGHT OF MATERIAL (pcf) :   ", UW
480 PRINT:PRINT TAB(25)
490 INPUT "MODULUS OF MATERIAL           : ",EI
500 PRINT:PRINT
510 PRINT TAB(20) "CONDITIONS AT WHICH THE MODULUS IS OBTAINED: "
520 PRINT
530 PRINT TAB(25) : INPUT "TEMPERATURE (deg. F)   :   ",TTEMP
540 PRINT TAB(25) : INPUT "SUCTION (psi)          :   ", HI
```

```
550 PRINT:PRINT
560 PRINT TAB(20) "INPUT WANTED CONDITIONS: "
570 PRINT
580 PRINT TAB(25) : INPUT "TEMPERATURE (deg. F)    :   ",RTEMP
590 PRINT TAB(25) : INPUT "SUCTION (psi)           :   ", HL
600 '
610 '=======CALCULATION========
620 VOID=1/(UW/2.67/62.4)-1
630 N=VOID/(1+VOID)                'POROSITY
640 X=(.4764-N)/(.4764-.2595)
650 IF (X<0) THEN X=0
660 IF (X>1) THEN X=1
670 PI=3.1415927#
680 PRO1=3/4*(1-U^2)/E
690 '==K2 IS THE POWER IN THE EQUATION E=K1*TETHA^K2==
700 K2=1/3
710 THETA=10^(LOG(EI/K1)/LOG(10)/K2)
720 DTEMP=RTEMP-TTEMP   'RTEMP IS THE REFERENCE TEMP.
730 DT=ABS(DTEMP)
740 IF (DTEMP=0) THEN NE=EI : GOTO 800
750 DELV=DT*ALP*3
760 PSC=X/2^(1/2)/PRO1*(1/3*DELV)^(3/2)
770 PFCC=(1-X)/4/PRO1*(1/3*DELV)^(3/2)
780 PT=(PSC+PFCC)*DTEMP/DT
790 DET=K1*K2*THETA^(K2-1)*PT
800 PS=-(HL-HI)*.13
810 DES=K1*K2*THETA^(K2-1)*PS
820 TOTDE=DET+DES
830 NE=EI+TOTDE
840 '
850 '=======OUTPUT========
860 CLS
870 PRINT:PRINT:PRINT TAB(10)
880 PRINT "INPUT CONDITIONS:                    WANTED CONDITIONS:"
890 PRINT
900 PRINT USING"  TEMPERATURE =    ###.#   deg. F      TEMPERATURE =  ###.#   deg. F
"; TTEMP; RTEMP
910 PRINT USING"  SUCTION     =   ####.##  psi         SUCTION     = ####.##  psi
"; HI; HL
920 PRINT USING"  MODULUS     = ######.##  psi";EI
930 PRINT : PRINT
940 PRINT USING "  CHANGE OF MODULUS DUE TO TEMPERATURE CHANGE (psi) : ######.##";DET
950 PRINT USING "  CHANGE OF MODULUS DUE TO SUCTION CHANGE (psi)     : ######.##";DES
960 PRINT
970 PRINT USING "        MODULUS AT WANTED CONDITIONS (psi)  : ######.##";NE
980 END
```