AHTRC

Arkansas Highway and
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Dept. of Civil Engineering
College of Engineering
University of Arkansas
Fayetteville, Arkansas

RESILIENT PROPERTIES OF ARKANSAS SUBGRADES

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TRC-94
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16. Abstract

The resilient behavior of 15 Arkansas soils was studied. Moisture content, freeze-thaw, and deviator stress were found to significantly affect the soils' resilient moduli. Moisture content is the most critical variable for Arkansas subgrades. Recommendations are made for simplifying routine resilient modulus testing and for selecting the design resilient modulus. The testing simplifications consist of: 1) shortening the sample conditioning to 200 cycles of a single deviator stress, 2) testing at a single confining pressure and two deviator stresses, and 3) reducing the number of deviator stress repetitions to 50.

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FINAL REPORT

TRC-94

RESILIENT PROPERTIES OF ARKANSAS SUBGRADES

PRINCIPAL INVESTIGATOR - Robert P. Elliott

CO-PRINCIPAL INVESTIGATOR - Sam I. Thornton

GRADUATE ASSISTANTS - Kee Yong Foo
Kin Wah Siew
Ronota Woodbridge

Conducted by

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Engineering Experiment Station
University of Arkansas

in cooperation with

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and

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The contents of this report reflect the view of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Arkansas State Highway and Transportation Department or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

SI CONVERSION FACTORS

1 inch = 25.4 mm

1 foot = 0.305 m

1 pcf = 16 kg/m^2

1 psi = 6.9 kN/m^2

1 ksi = 6.9 MN/m^2

1 lb = 4.45 N

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CHAPTER 1

INTRODUCTION

STATEMENT OF PROBLEM

The Arkansas State Highway and Transportation Department (AHTD) designs its pavements using the AASHTO Guide for Design of Pavement Structures (1). Prior to 1986, the AASHTO procedure incorporated subgrade evaluation in terms of a Soil Support scale that was undefined in terms of method of test or relationship between test results and Soil Support value. As a result, users of the procedure had to select their own method of test and test to Soil Support relationship. AHTD used the R-value test.

In 1986, AASHTO adopted a revised procedure that incorporated the resilient modulus test as the method of subgrade support evaluation. Because AHTD had not previously used the resilient modulus test and because the resilient modulus had been determined for only a limited number of Arkansas soils, research project TRC-94, Resilient Behavior of Arkansas Subgrades, was undertaken.

PROJECT OBJECTIVES

The general objectives of the study were to develop a knowledge of the resilient behavior of Arkansas soils and to establish specific methods to be used in the selection of the resilient modulus to for in pavement design. The specific objectives to be accomplished and addressed under the project were:

1. Determine the effects of the following variables on the resilient modulus of selected, representative Arkansas soils:

   a. Moisture content

   b. Density

   c. Deviator stress

   d. Confining pressure

   e. Freeze-thaw cycles.

2. Establish recommendations for the testing procedures to be used routinely by AHTD for determining subgrade resilient modulus.

3. Develop a method for estimating resilient modulus based on falling weight deflectometer (FWD) data.

4. Examine the feasibility of developing relationships between resilient modulus and other soil properties as a means for reducing the time and cost of testing.

SELECTION OF SOILS FOR STUDY

The soils tested in the study were selected to provide a general representation of subgrades typically encountered in Arkansas. Figure 1-1 shows the locations from which the soils were obtained. AHTD selected three soils for the initial phase of the study. The remaining twelve soils were selected by analysis of Arkansas soil maps.

The Arkansas General Soil Map (2) identifies eight geologic areas in the State and lists the major soil associations occurring in each area. Soils were selected to represent the predominate soil series in each major soil association and each geologic area. The soil associations represented by the 12 soils selected in this manner cover nearly 70 percent of the State (Table 1-1).

Table 1-1. Soils tested during the major phases of the study.

SOIL SERIES	SOIL ASSOCIATION AREAL COVERAGE, %	GEOLOGIC AREA
Enders	10.8	Boston Mountains and Arkansas Valley & Ridges
Carnasaw	9.4	Ouachita Mountains
Guyton	9.2	Coastal Plain
Sharkey	8.3	Bottom Lands & Terraces
Calloway	6.2	Loessial Plains & Hills
Sacul	6.1	Coastal Plain
Smithdale	5.6	Coastal Plain
Clarksville	4.6	Ozark Highlands
Foley	3.0	Bottom Lands & Terraces
Leadvale	2.9	Arkansas Valley & Ridges
Perry	2.6	Bottom Lands & Terraces
Houston	1.0	Blackland Prairie
TOTAL	69.7	

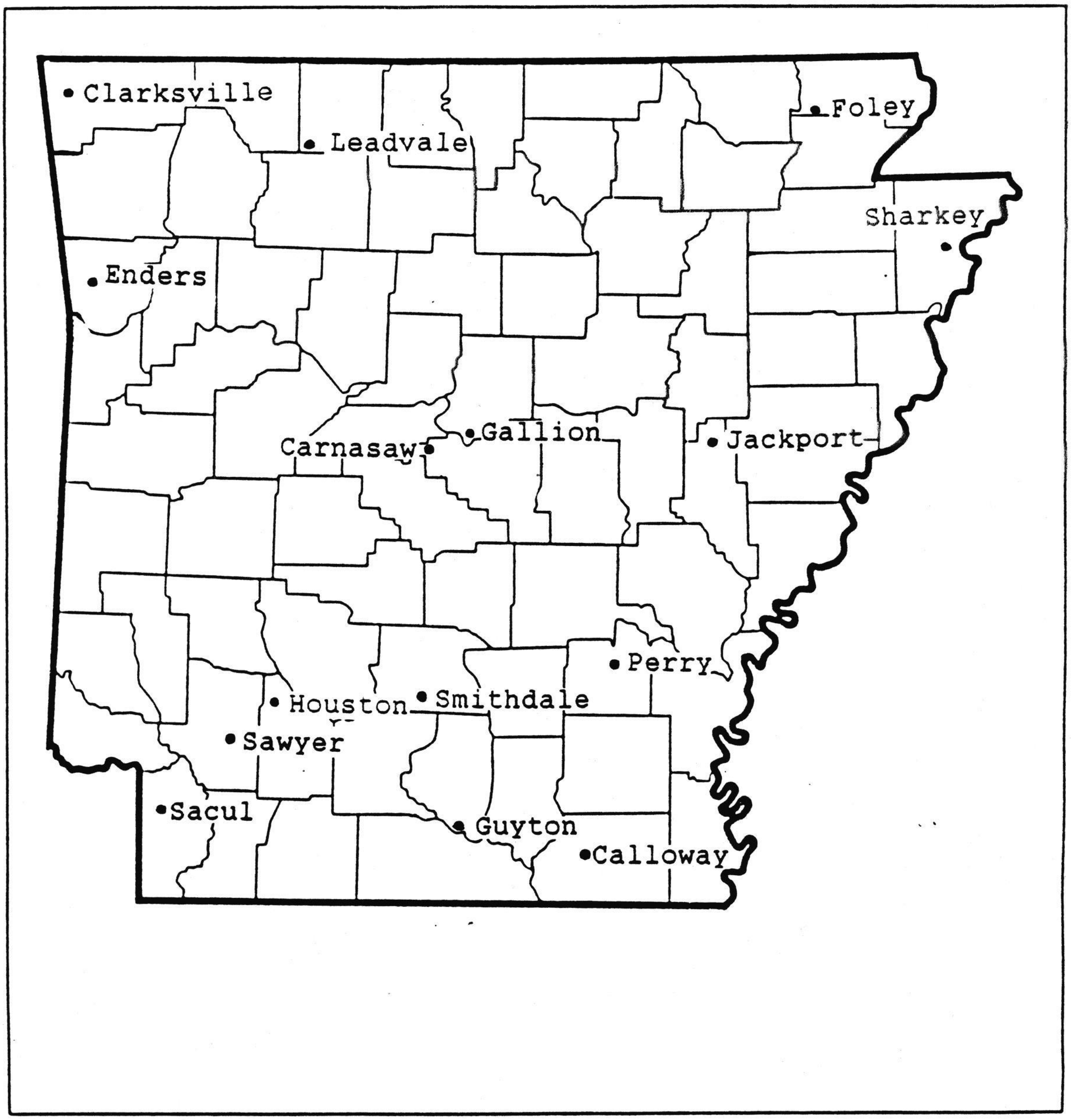


Figure 1-1. Location Map of Soils Used in Study.

CHAPTER 2

FACTORS AFFECTING SOIL RESILIENT MODULUS

The objective of resilient modulus testing is to simulate the in-service behavior of the soil as a support medium for the pavement. As such, the testing should simulate field conditions as closely as possible and must take into account those factors that affect resilient modulus in the field. The standard method of test for resilient modulus (AASHTO T274) has been designed to accomplish this. However, the AASHTO test method is quite complex and time consuming. Results from TRC-94 demonstrate that the test can be simplified.

STANDARD TEST REQUIREMENTS

The standard AASHTO test requirements were developed recognizing the stress dependent nature of soil. Testing is to be done in a triaxial chamber so that a lateral confining pressure can be applied.

For cohesive soils, three confining pressures are required (0 psi, 3 psi, and 6 psi). The traffic loading effect is simulated by applying a vertical load (referred to as a deviator stress) for a duration of 0.1 second at a repeated interval of 1 to 3 seconds. Five levels of deviator stress are required (1, 2, 4, 8, and 10 psi). Each deviator stress is repeated for 200 cycles at each of the three confining pressures.

The resilient modulus is determined for each combination of deviator stress and confining pressure by the equation:

$$M_r = s_d/e_r$$

where

M_r = the resilient modulus

s_d = the deviator stress

e_r = the resilient or recoverable strain.

With three levels of confining pressure and five levels of deviator stress, 15 M_r values are determined for each test specimen. Using 2 seconds between stress cycles, the testing time for one specimen by AASHTO T274 is 100 minutes exclusive of sample preparation and conditioning. By reducing the number of stress cycles, confining pressures, and deviator stresses, the testing time can be reduced significantly.

STRESS CYCLES

Deformation readings were taken at 50, 100, and 200 load cycles for each test during the testing of the first three soils. These data were analyzed to determine the necessity of having 200 cycles before recording the resilient deformation.

In most cases, the resilient deformation at 50, 100, and 200 cycles were found to be identical. The number of variations that did occur were more frequent at 50 cycles and were found to increase as the deviator stress increased. The 50 cycle readings varied from the 200 cycle reading only 17 of 324 times (5% of the time) at 8 psi and 53 of 324 times (16% of the time) at 10 psi; and, the maximum variation amounted to less than 6 percent of the

200 cycle deformation. Therefore, the number of loading cycles may be reduced from 200 to 50 without changing the test results.

CONFINING PRESSURE

Initially all tests conducted under the study were performed at three confining pressures as prescribed by AASHTO T274. In those tests the resilient modulus tended to be higher with higher confining pressures (Figure 2-1). The increase from 0 to 3 psi was greater than the increase from 3 to 6 psi. The increase was also found to be less as the moisture content increased. At 120 percent of optimum (AASHTO T99) , the difference between 0 and 3 psi ranged from none to about 15 percent.

For routine purposes, there is no reason to test at more than one confining pressure. Consideration might be given to testing in the unconfined state (0 psi). Unconfined testing would simplify the test and would not require use of a triaxial cell. However, unconfined testing cannot be used for non-plastic soils. Ideally, the confining pressure should be representative of the pressure expected in the completed subgrade. Subgrade confining pressure is typically about 3 psi. Consequently, a test confining pressure of 3 psi is recommended.

DEVIATOR STRESS

The resilient behavior of fine grained soils is known to be stress dependent. The resilient modulus generally decreases as the deviator stress is increased and the rate of decrease becomes

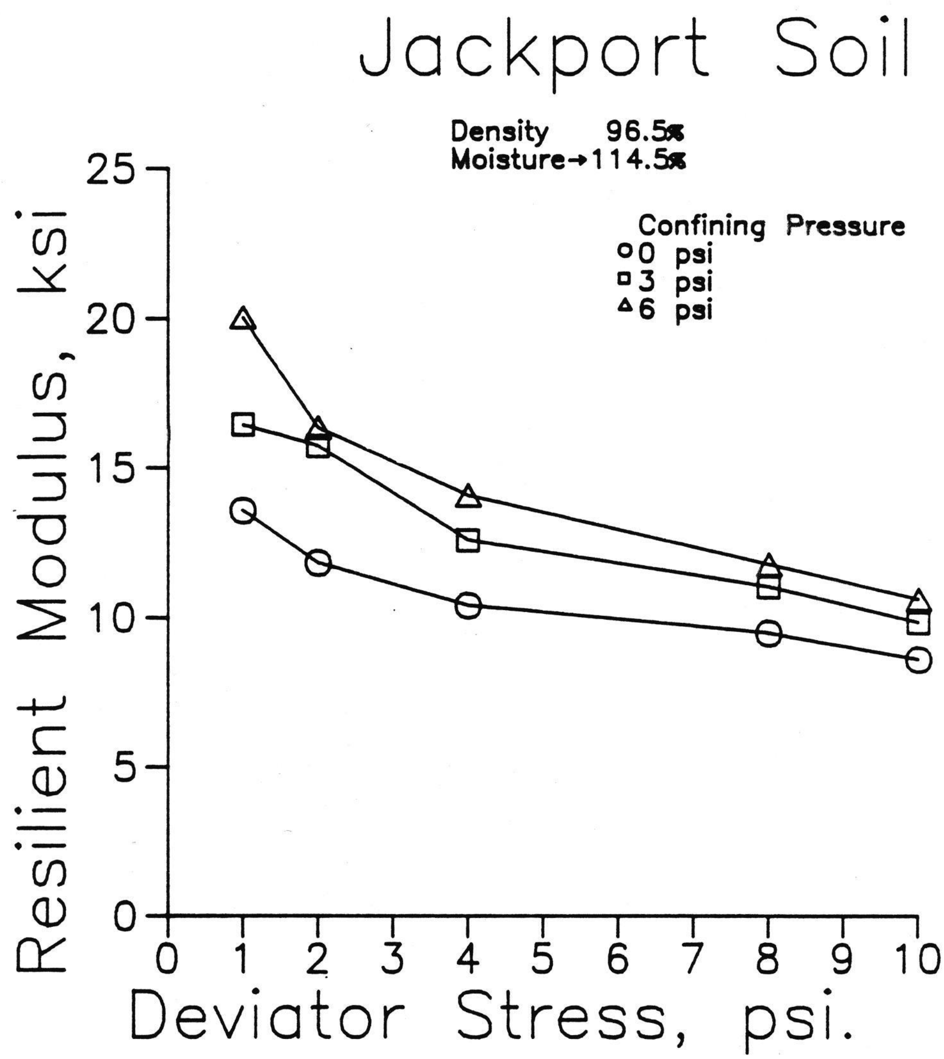


Figure 2-1. Effect of Confining Pressure on Jackport Soil.

less as the deviator stress is increased.

In a previous study of subgrade resilient modulus, Thompson and Robnett (3) characterized the stress dependent behavior as two intersecting straight lines. The point of intersection was called the "breakpoint" resilient modulus and "breakpoint" deviator stress. The "breakpoint" deviator stress was found to be between 4.14 and 9.00 psi. The slopes at stresses below the "breakpoint" ranged from 0.27 to 3.21 and from 0.01 to 0.42 above the "breakpoint".

The Arkansas subgrade soils were found to fit this general pattern. Although the tests were not conducted so that specific points of intersection could be determined, the slopes from 2 to 4 psi and from 8 to 16 psi were consistent with Thompson and Robnett's results; and the 4 to 8 psi data suggests that the points of intersection would be within the 4 to 8 psi range. For soils tested wet of optimum, the slopes of resilient modulus versus deviator stress were found to be:

	DEVIATOR STRESS RANGE		
	2 to 4	4 to 8	8 to 16
MEAN SLOPE, ksi/psi	0.77	0.35	0.13
RANGE OF SLOPES	0.21 - 1.64	0.00 - 0.82	0.04 - 0.27

These results suggest that deviator stress should receive some consideration in selecting the resilient modulus for design. Because the slopes of resilient modulus versus deviator stress vary from soil to soil, deviator stress can be considered only if testing is conducted at more than one deviator stress.

To determine the appropriate deviator stress for testing and for selection of a design modulus, stress analyses were performed on typical pavement cross sections using the elastic layered theory. From these analyses, a general relationship was observed between the deviator stress and the pavement structural number (Figure 2-2). The deviator stress is generally 4 psi or less for structural numbers greater than 4.5 and is 8 psi or more for structural number less than 2.5. As a result, deviator stresses of 4 and 8 psi appear to be appropriate for routine testing.

COMPACTION METHOD

AASHTO T274 also specifies the method of sample compaction. The method to be used depends upon the expected field degree of saturation during construction and later in service. Table 2-1 summarizes the sample compaction specifications.

Field data from several Arkansas projects were analyzed to determine the magnitude and variability of density and moisture content typical for Arkansas. Estimates of the degree of saturation following compaction indicated that 75 to 80 percent of the soils were compacted at moisture contents that resulted in greater than 80 percent saturation after compaction. AASHTO T274 requires kneading compaction (Table 2-1) if field compaction moisture contents typically result in greater than 80 percent

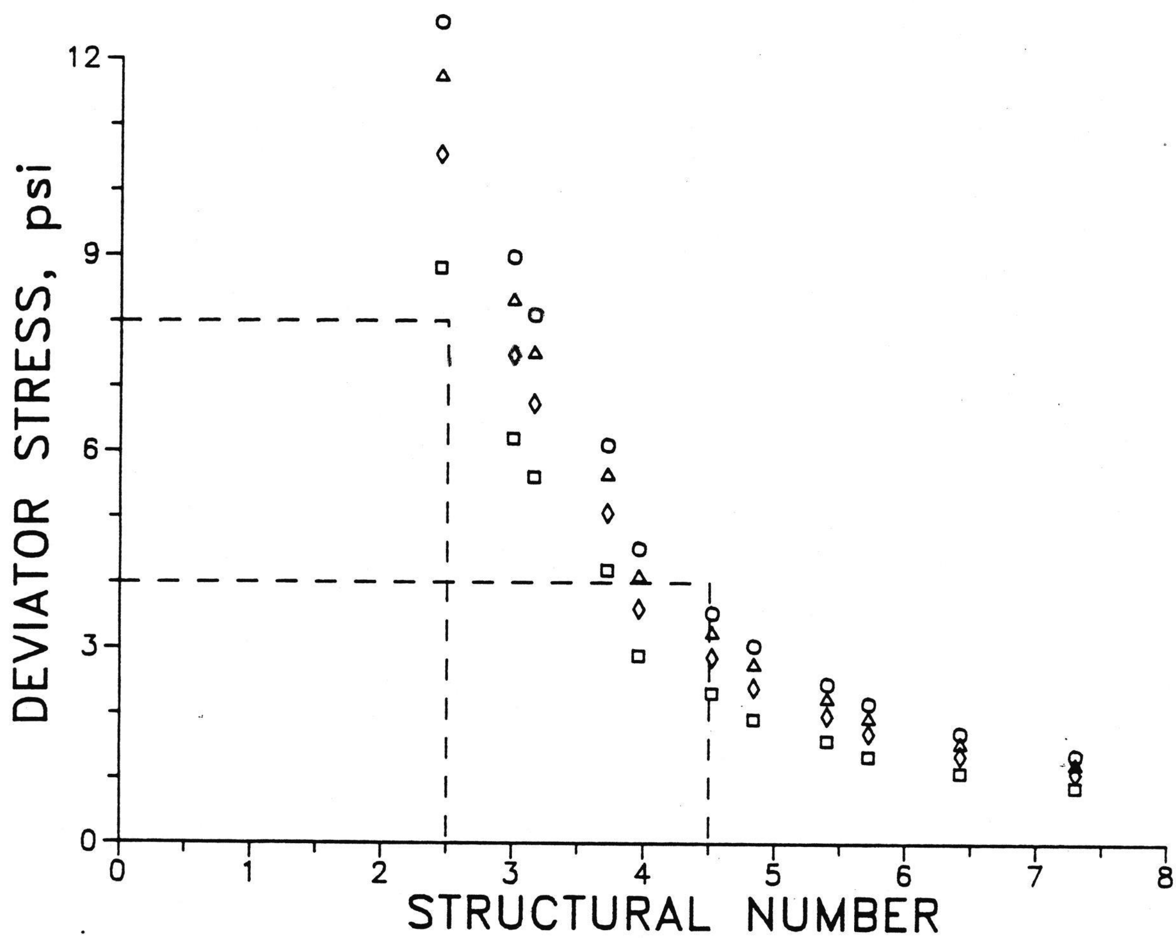


Figure 2-2. Relationship between Structural Number and Subgrade Deviator Stress.

Table 2-1. Compaction Method Requirements by AASHTO T274.

FIELD DEGREE OF SATURATION		LABORATORY COMPACTION METHOD
As Compacted	In-service	(at in-service moisture content)
< 80%	< 80%	gyratory, kneading, or static
< 80%	> 80%	static
> 80%	> 80%	kneading

saturation.

Static compaction, however, is simpler and would be preferred if it produced the same or essentially the same M_r values. To determine whether static compaction could be used, specimens of five soils were prepared using static compaction as well as kneading compaction. Of the five soils tested using both compaction methods, two were found to be significantly affected by compaction method (Figures 2-3 and 2-4). The statically compacted specimens exhibited a higher resilient modulus than did the kneading compacted specimens. The other three soils showed little to no influence due to method of compaction (Figure 2-5).

No indication was found that particular types of soil were influenced by method of compaction more than others. Therefore, the use of static compaction cannot be justified and kneading compaction is recommended.

DENSITY

Analysis of field density data (4) showed 95 percent of maximum density to be a realistic target density for sample preparation. To examine the sensitivity of test results to the actual density achieved, test specimens of three soils were prepared at approximately 95 and 100% of maximum. Within the range of 95 to 100%, resilient modulus was not significantly affected by density (Figures 2-6 and 2-7).

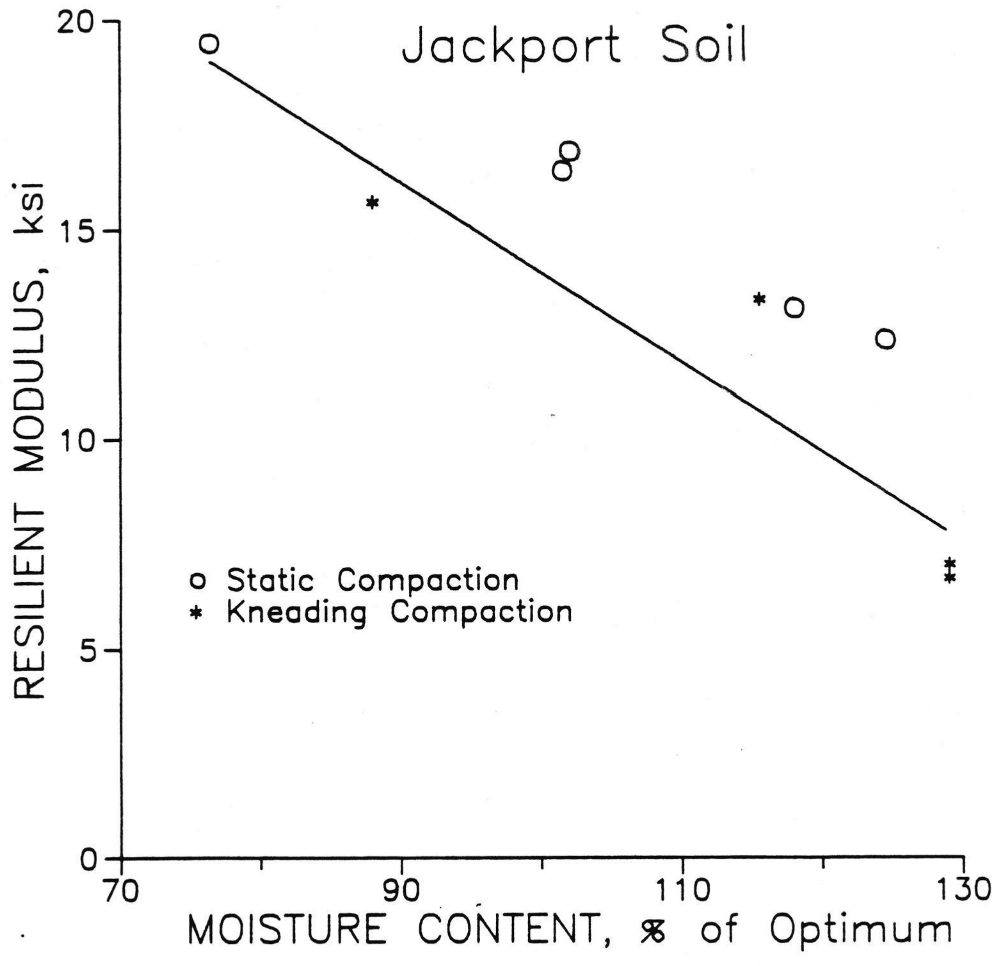


Figure 2-3. Effect of Compaction Method on Jackport Soil.

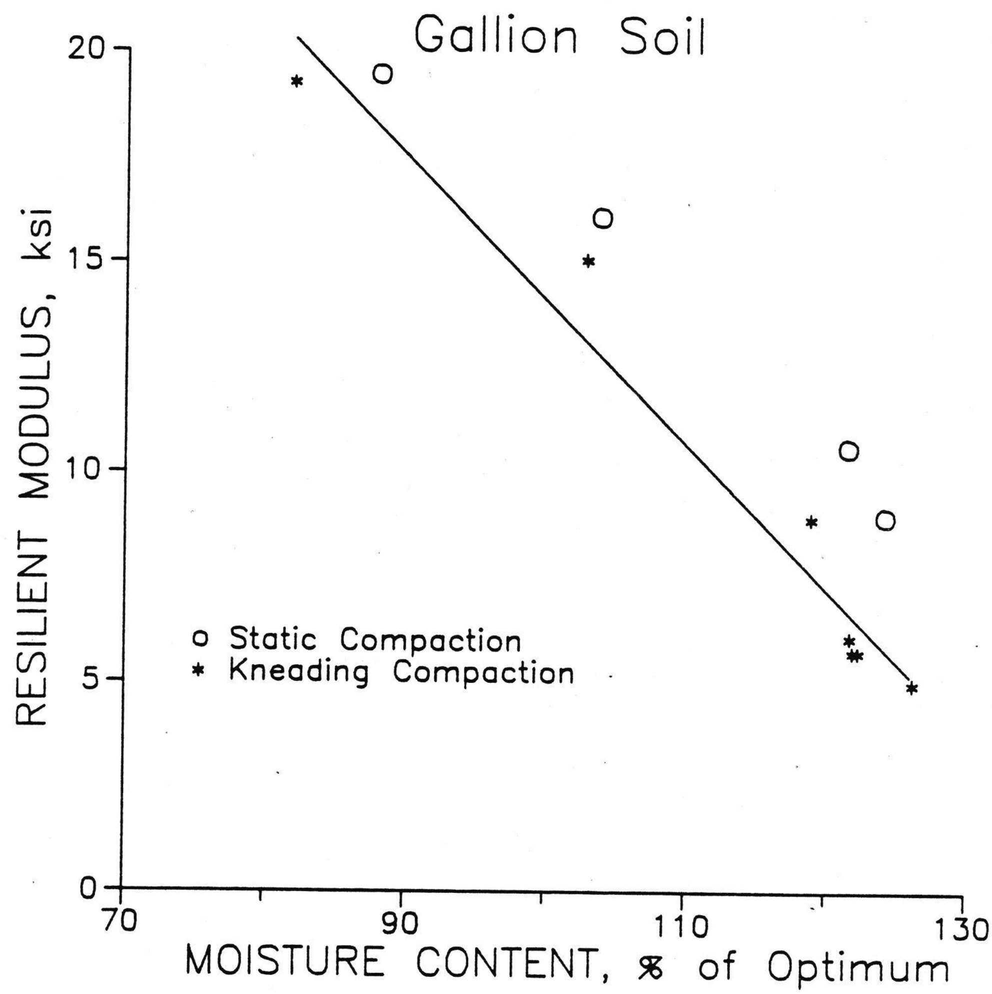


Figure 2-4. Effect of Compaction Method on Gallion Soil.

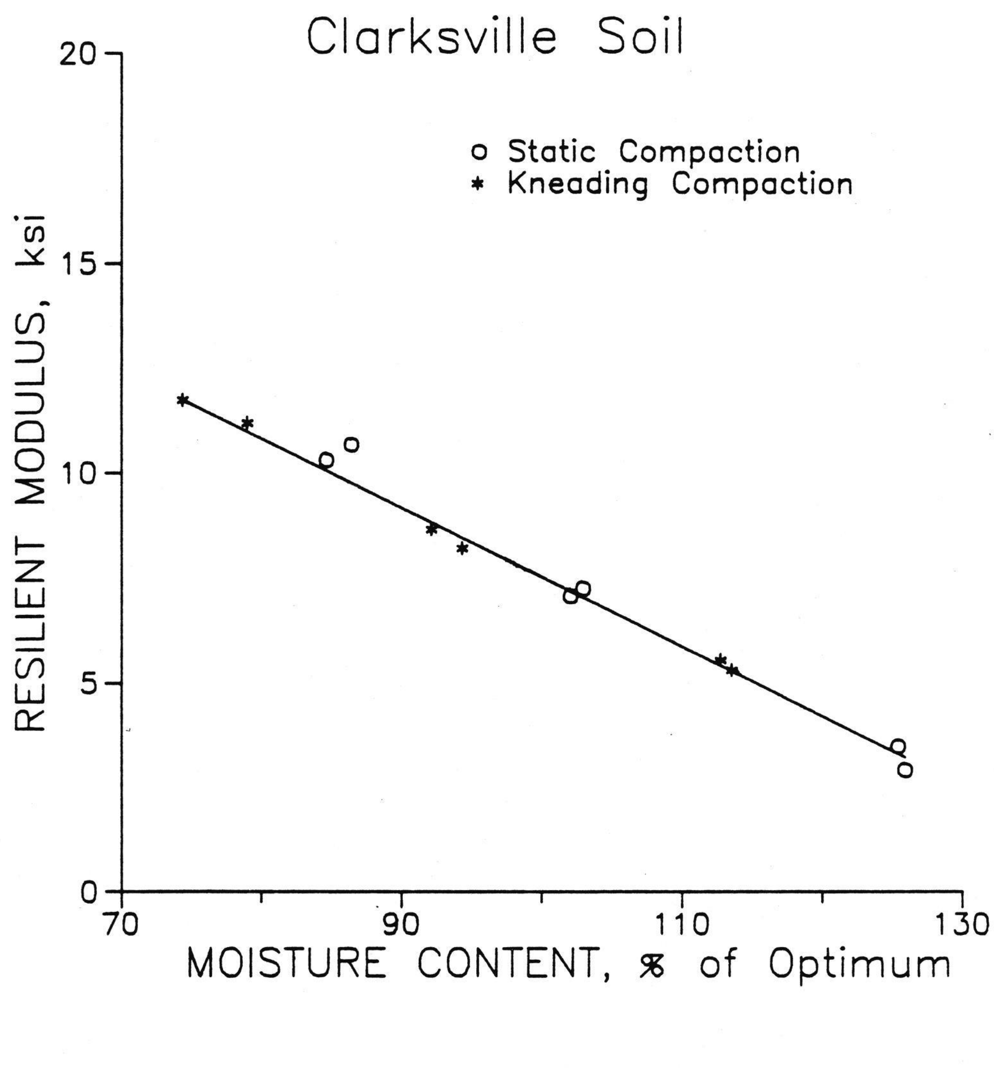


Figure 2-5. Effect of Compaction Method on Clarksville Soil.

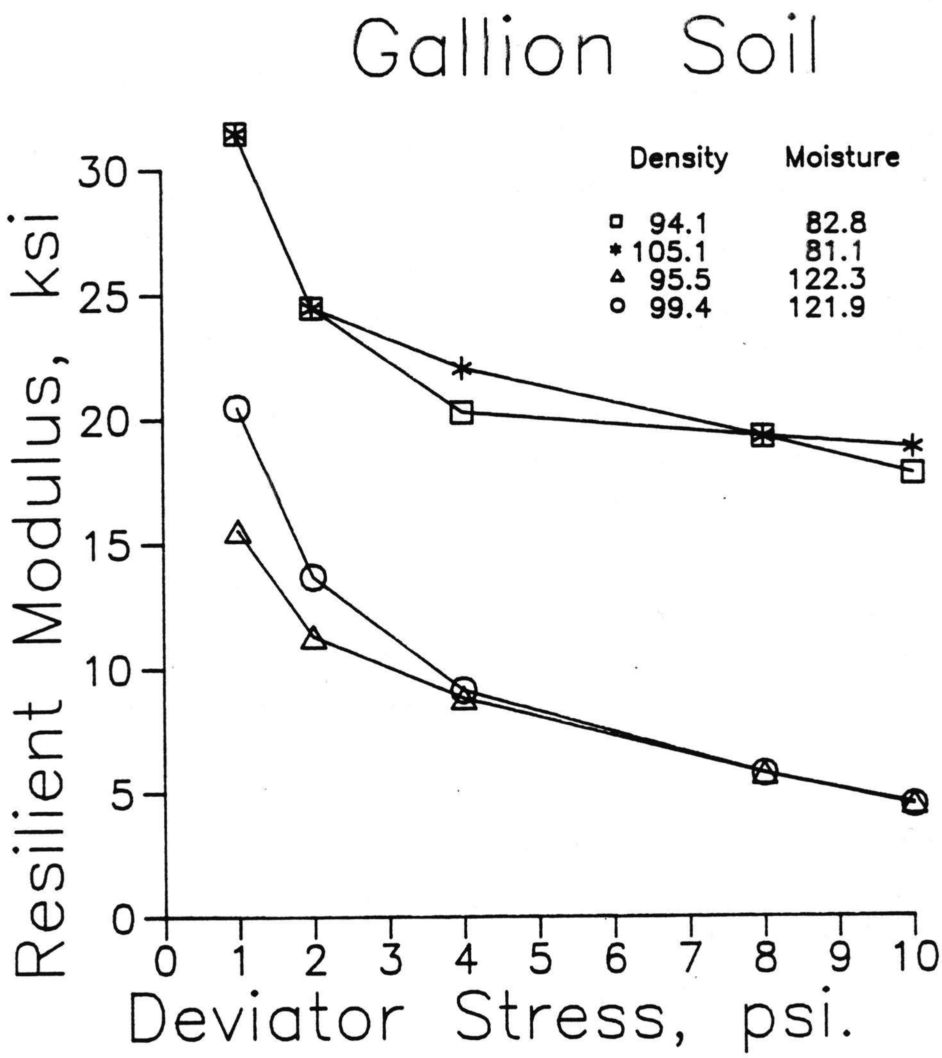


Figure 2-6. Effect of Density on Gallion Soil.

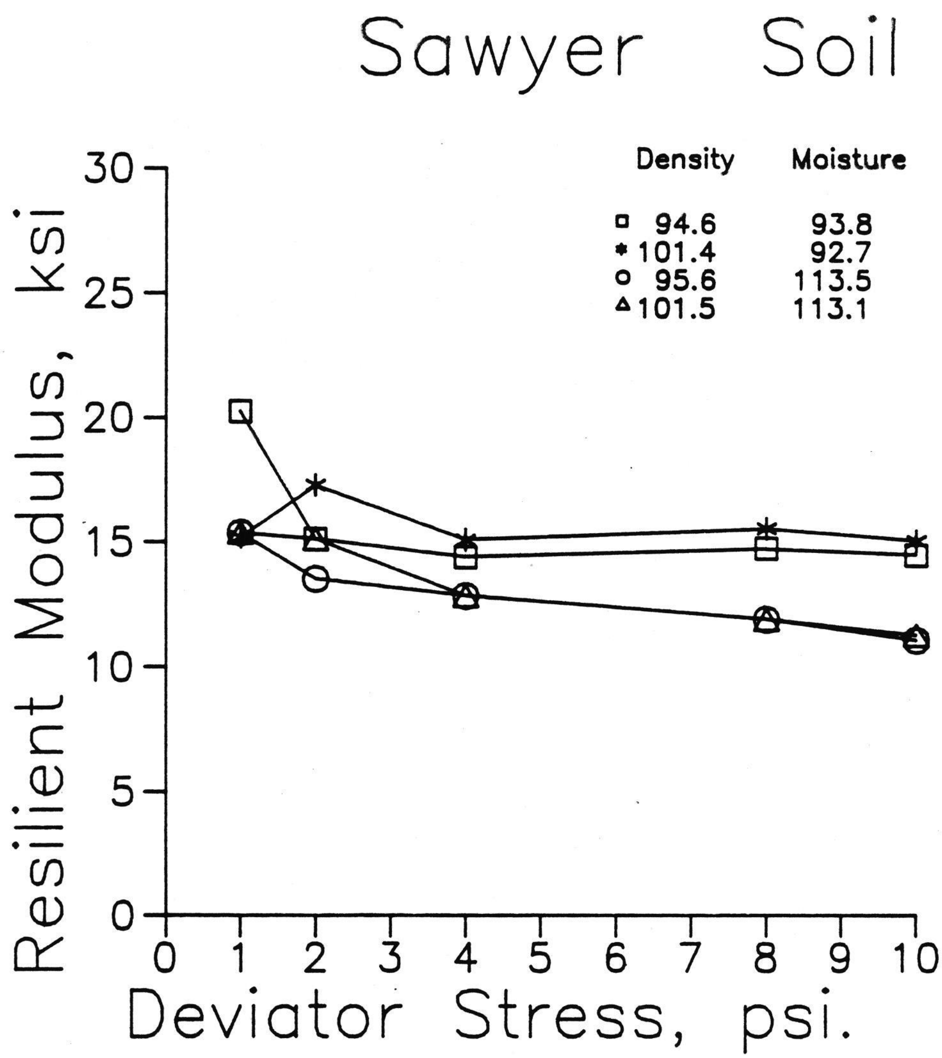


Figure 2-7. Effect of Density on Sawyer Soil.

MOISTURE CONTENT

Resilient modulus moisture sensitivity is defined as the unit change in M_r per unit change in soil moisture content. Moisture sensitivity of Arkansas soils was investigated by testing each soil at moisture contents from below optimum to approximately 120% of optimum.

Table 2-2 lists the moisture sensitivity of each soil tested. The moisture sensitivity is reported for deviator stresses of 4 and 8 psi and relative to both optimum moisture content and absolute moisture content. The units of measure are change in resilient modulus (ksi) per change in either: 1) percentage of optimum moisture content or 2) absolute moisture content percentage. Table 2-2 also lists each soil's resilient modulus at 120 percent of optimum.

The moisture sensitivity data shows that Arkansas soils are quite sensitive to moisture content. Also, the degree of sensitivity varies significantly from soil to soil. On the average, a one percent change in moisture content will result in a 1.4 ksi change in the resilient modulus. However, for the individual soils tested, the change ranged from about 0.2 ksi to nearly 4.3 ksi.

As a consequence, <u>the moisture content of the soil at the time of test is critical</u>. The test moisture content needs to be representative of the moisture content that will exist in the subgrade after the pavement is in-service. A reasonable prediction of this moisture content is essential to determining the

Table 2-2. Moisture Sensitivity of Soils Tested.

SOIL	OPTIMUM MOISTURE %	DEVIATOR STRESS psi	RESILIENT MODULUS ksi @ 120% of optimum	MOISTURE SENSITIVITY ksi per % relative to % of Optimum	% Moisture
Calloway	17.4	4	4.1	.12	.70
		8	3.5	.13	.72
Carnasaw	15.0	4	5.9	.09	.60
		8	4.2	.13	.84
Clarksville	14.8	4	4.6	.18	1.21
		8	4.3	.17	1.12
Enders	17.0	4	2.4*	.10	.61
		8	3.1*	.06	.34
Foley	20.0	4	6.2	.05	.26
		8	6.0	.05	.25
Gallion	25.0	4	10.6	.28	1.12
		8	7.7	.35	1.39
Guyton	16.2	4	6.2	.44	2.70
		8	4.1	.38	2.35
Houston	16.0	4	11.3	.05	.31
		8	9.3	.05	.33
Jackport	20.0	4	12.1	.45	2.27
		8	11.1	.48	2.40
Leadvale	21.5	4	7.0	.24	1.12
		8	5.0	.25	1.16
Perry	37.4	4	2.1	.08	.21
		8	1.6	.16	.44
Sacul	19.5	4	1.8*	.77	3.93
		8	1.7*	.51	2.59
Sawyer	22.5	4	11.0	.46	2.04
		8	8.8	.51	2.27
Sharkey	28.5	4	6.5	.13	.47
		8	5.9	.14	.47
Smithdale	11.5	4	5.7*	.49	4.28
		8	5.5*	.46	4.03

* These soils failed prematurely when tested at 120% of optimum. The M_r value listed was extrapolated from the highest moisture content that could be tested.

appropriate resilient modulus to use for pavement design.

FREEZE-THAW CYCLES

The effect of freeze-thaw was investigated by testing three Arkansas soils. Four specimens of each were prepared with two of each being subjected to one freeze-thaw cycle prior to testing. The freeze-thaw specimens were wrapped in plastic prior to freezing to retain the as-molded moisture content. A single freeze-thaw cycle reduces the soil's resilient modulus by about 50 percent (Figure 2-8). Previous work by Robnett and Thompson (5) had similar results for Illinois soils. Robnett and Thompson also found that additional freeze-thaw cycles resulted in only a small additional decrease in resilient modulus.

SUMMARY OF FACTORS AFFECTING RESILIENT MODULUS

Subgrade resilient modulus is sensitive to deviator stress, moisture content, and freeze-thaw; but, relatively insensitive to the number of stress cycles, confining pressure, and density. Some soils are sensitive to the method of compaction.

The degree of sensitivity to moisture content, deviator stress, and freeze-thaw needs to be reflected in the routine test procedure and in the selection of the design resilient modulus. Moisture content appears to be of prime importance. Yet, the ability to predict field moisture content is poor. A practical method to realistically estimate in-service moisture contents is needed.

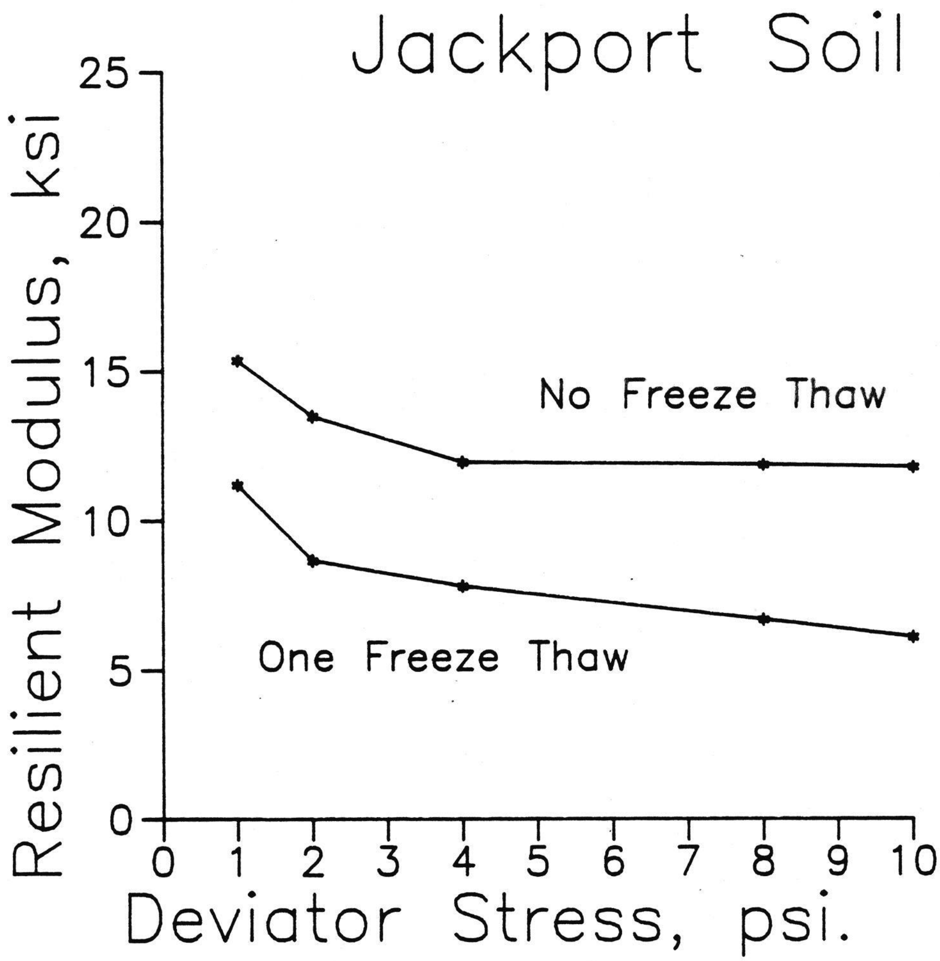


Figure 2-8. Freeze-Thaw Effect on Jackport Soil.

CHAPTER 3

RESILIENT MODULUS FROM FWD DATA

Five Arkansas sites were selected for FWD (falling weight deflectometer) deflection testing to provide data that could be used to develop a method for estimating resilient modulus from FWD deflection data. The sites and the associated soils are listed in Table 3-1.

The procedure for estimating resilient modulus from deflection data involves the calculation of modulus values that will produce a similar deflection pattern when used in a structural model. This procedure is generally referred to as backcalculation. Various researchers (6, 7, 8) have studied the process of backcalculation and have developed several different methods of backcalculation.

Seven methods of estimating resilient modulus from deflection data were available for this study. Four of the methods were true backcalculation methods using relatively complex computer programs based on elastic layer theory. These four methods were:

1) ELMOD, which was provided by the manufacturer of the AHTD FWD;
2) BISDEF, which is based on the Bisar elastic layer program;
3) ELSDEF, which is similar to BISDEF but uses the ELSYM5 elastic layer program; and
4) FPEDD1, which also uses ELSYM5 and was developed at the University of Texas for the Texas Highway Department.

Table 3-1. FWD Test Locations.

SOIL	SITE DESCRIPTION	PAVEMENT
Calloway	Highway 82, near Hamburg, Ashley County	7" ACHM, 8" Gran. Base over an old roadbed
Carnasaw	Highway 113, west of Little Rock	1.3" ACHM, 5" Gran. Base, 2′ select fill
Enders	Highway 162, east of Cedarville, Crawford Co.	1.5" ACHM, 12" Gran. Base
Sacul	Highway 71, south of Texarkana, Miller County	3.5" ACHM, 10" Gran. Base, 6" Select Matl.
Sharkey	Highway 140, near Osceola, Mississippi County	3.5" ACHM, 6" Stab. Base, 4" Gran. Subbse.

The other 3 methods were relatively simple backcalculation algorithms (equations) developed from the finite element program ILLI-PAVE (9).

FWD deflection measurements were made at each of the five Arkansas sites several times during the course of the study. Cores were also taken at least once during the study to determine material thicknesses and to get undisturbed shelby tube samples for resilient modulus testing in the laboratory. Comparison of measured and backcalculated resilient modulus values was expected to identify the "best" method of backcalculation.

None of the methods for estimating resilient modulus from deflections proved to be satisfactory. Except for one ILLI-PAVE algorithm, the methods frequently produced unrealistically high resilient modulus values. In many cases, the modulus value predicted for the subgrade was greater than that predicted for the base. Table 3-2 displays the laboratory and backcalculated resilient modulus values for those test sites and times for which shelby tube samples were obtained.

A major problem with backcalculation appears to be the need for a good measure of the depth to bedrock. To examine the significance of this parameter, BISDEF was used to backcalculate the subgrade resilient modulus for one set of data using different depths to bedrock. The results, displayed on Figure 3-1, shows the backcalculated resilient modulus value is very sensitive to depth of bedrock for depths less than about 30 feet.

Table 3-2. Comparison of Backcalculated Resilient Modulus with Laboratory Tests on Shelby Tube Samples.

SOIL	LAB RESULTS σ_d	M_r, ksi	BACKCALCULATED RESILIENT MODULUS, ksi (a)	(b)	(c)	(d)	(e)	(f)
Calloway	4	5.9	19.9	15.0	15.4	3.1	11.2	8.2
	8	5.5						
Carnasaw	4	9.7	39.5	18.6	48.8	0.7	16.4	16.6
	8	5.6						
	4	6.0						
	8	3.3						
Enders	4	8.0	47.8	35.0	65.7	9.0	57.1	18.8
	8	6.7						
Sacul	4	4.7	-	-	44.5	45.0	54.8	17.5
	8	4.4						
	4	7.8						
	8	7.0						
	4	7.5	-	-	40.9	47.9	27.8	15.7
	8	6.6						
	4	10.8						
	8	10.1						

(a) BISDEF, (b) ELMOD, (c) FPEDD1, (d) ILL-1, (e) ILL-2, (f) ILL-3

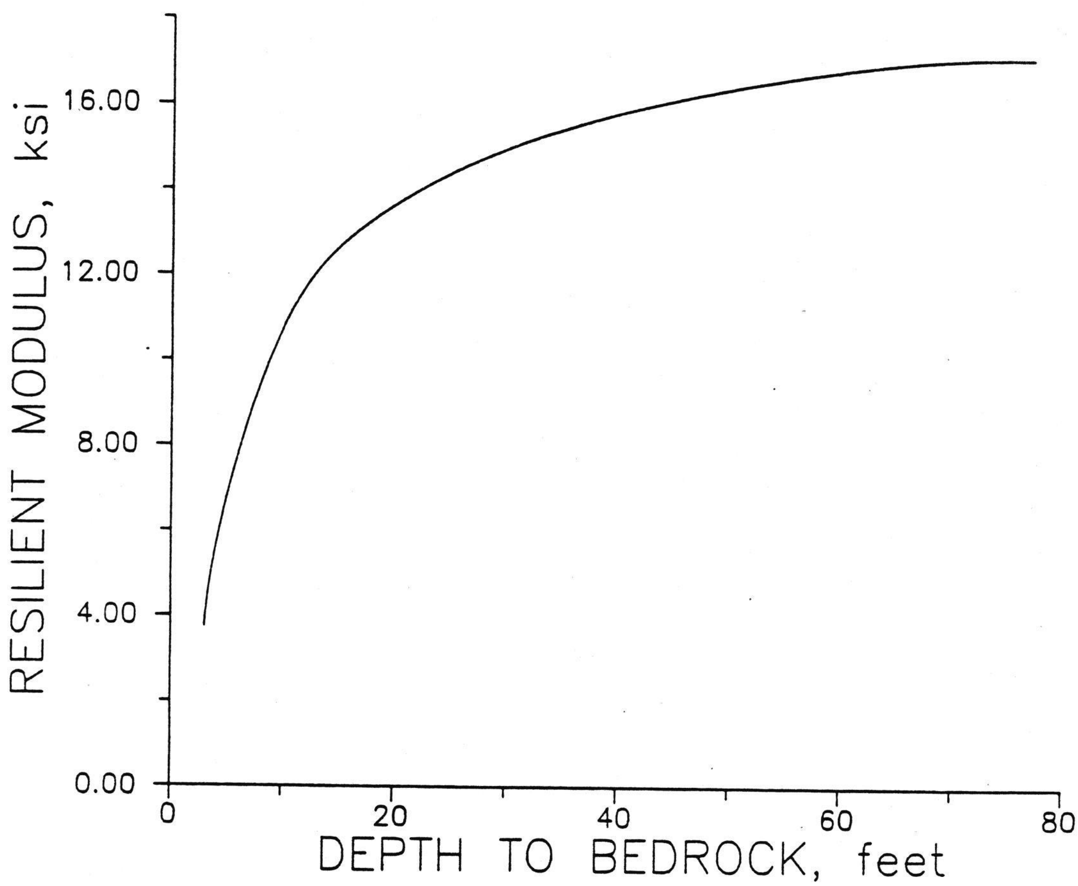


Figure 3-1. Effect of Depth of Bedrock on Backcalculated Subgrade Modulus.

The FPEDD1 program incorporates a method that estimates the depth of bedrock from the deflection data if the depth is unknown. Despite this feature, FPEDD1 also provided unrealistic estimates of the subgrade resilient modulus.

Only the ILLI-PAVE algorithm referred to as ILL-3 in Table 3-2 consistently produced reasonable values. However, the production of reasonable values does not demonstrate that ILL-3 is correct or in anyway superior to the other methods. The ILL-3 equation is such that only reasonable values can be obtained. The equation is:

$$M_r = 24.06 - 5.08\ D_3 + .28\ D_3^2$$

where

M_r = resilient modulus in ksi

D_3 = the deflection at 3′ from the center of loading in mils.

Also, the ILL-3 values do not correspond well to the laboratory test results.

Nevertheless, the expectation of a strong correlation between tests on shelby tube samples and backcalculation results may not be realistic. The shelby tube provides a measure of the resilient modulus on a very small, finite sample. Backcalculation, on the other hand, provides a gross, overall estimate for the entire depth of the subgrade. Since resilient modulus varies significantly with soil parameters (e.g. moisture) that can vary with depth, a strong correlation between the finite shelby tube measure and the overall backcalculation measure may not be possible.

The problem of correlation is demonstrated by the dual shelby tube samples tested from the Sacul and Carnasaw sites. At each site, two samples were tested from the same location but at different depths. The test results from the different depths differed significantly (Table 3-2).

In summary, the TRC-94 efforts to establish a method for estimating resilient modulus from FWD deflection data were unsuccessful. Seven methods of estimation were examined with only one consistently producing reasonable results; and, there is no evidence that the method is correct. Nevertheless, the data, test sites, and analyses have been continued in project TRC-8705, NDT Overlay Design.

CHAPTER 4

PREDICTION OF RESILIENT MODULUS FROM SOIL PROPERTIES

Each soil included in the study was also tested to determine its Atterberg limits (AASHTO T89 and T90), grain size distribution (AASHTO T88), organic content (AASHTO T194), R-value (AASHTO T190), maximum density, and optimum moisture content (AASHTO T99). The soil property tests were conducted to examine the feasibility of predicting resilient modulus from other soil properties. The results of these tests are listed with descriptions of each soil in Appendix A. The resilient modulus test results are in Appendix B.

Correlation analyses were performed between resilient modulus and the other soil properties. The moduli used in the analyses were at 4 and 8 psi deviator stress and a moisture content of 120% of optimum. Only clay content and colloid content were found to have a significant correlation with resilient modulus (Table 4-1). The significant correlations with clay and colloid contents do not infer "cause and effect" relationships; and the lack of significant correlation with the other properties (Atterberg limits, organic content, etc.) does not mean that they do not influence resilient modulus.

Regression analyses also were performed to identify equations for estimating resilient modulus from routine soil tests. Several regression methods were used that involved different approaches for selecting the independent variables (soil properties) to be

Table 4-1. Results of Correlation Analyses.

SOIL PROPERTY	CORRELATION COEFFICIENT WITH RESILIENT MODULUS	
	s_d = 4 psi	s_d = 8 psi
Silt Content, %	0.006	-0.011
Clay content, %	0.575*	0.522*
Colloids, %	0.592*	0.542*
Organic Content, %	0.170	0.169
Liquid Limit, %	0.267	0.208
PI, %	0.326	0.282
Group Index	0.395	0.363
R-value	-0.333	-0.277
Maximum Density, pcf	-0.251	-0.232
Optimum Moisture Content, %	-0.062	-0.111

* Significant at the 5% level

included in the equation. The backward elimination technique produced the results that appears to be the most useful.

The backward elimination technique begins by performing a regression analysis that includes all of the independent variables. The variable contributing least to the prediction of the dependent variable (resilient modulus) is then removed and a regression analysis is performed with the remaining variables. The variable removal step is repeated until the only variables that remain are those that have a significant contribution to the prediction.

The results of the backward elimination regression analyses are contained in Table 4-2. The analyses produced similar equations for both the 4 psi and 8 psi deviator stress data. The final prediction equations include clay content, PI, and optimum moisture content. The equations are:

For deviator stress of 4 psi

$$M_r = 11.21 + .17*CL + .20*PI - .73*w_{opt} \qquad \text{(Eq 4-1)}$$

$R^2 = .80$, Standard Error of Estimate = 1.78

For deviator stress of 8 psi

$$M_r = 9.81 + .13*CL + .16*PI - .60*w_{opt} \qquad \text{(Eq 4-2)}$$

$R^2 = .77$, Standard Error of Estimate = 1.53

where

CL = clay content, percent

PI = plasticity index, percent

w_{opt} = optimum moisture content, percent

Resilient moduli predicted by these equations are compared

Table 4-2. Results of Regression Analyses.

Analysis Relative to Resilient Modulus at 4 psi Deviator Stress

INTERCEPT a ksi	Clay %	PI %	Opt Moist %	Organic %	Colloids %	LL %	Max Den pcf	R^2	RMSE
	REGRESSION COEFFICIENTS, b								
19.78	0.68	0.53	-0.67	-2.20	-0.50	-0.24	-0.02	.91	1.49
17.05	0.68	0.53	-0.66	-2.16	-0.49	-0.23		.91	1.40
15.00	0.56	0.30	-0.80	-1.96	-0.40			.87	1.56
13.66	0.17	0.27	-0.73	-1.46				.84	1.66
11.21	0.17	0.20	-0.73					.80	1.78

Analysis Relative to Resilient Modulus at 8 psi Deviator Stress

INTERCEPT a ksi	Clay %	PI %	Opt Moist %	LL %	Organic %	Colloids %	Max Den pcf	R^2	RMSE
	REGRESSION COEFFICIENTS, b								
27.98	0.54	0.47	-0.53	-0.29	-1.48	-0.39	-0.11	.89	1.32
14.17	0.51	0.46	-0.49	-0.26	-1.28	-0.35		.88	1.31
12.67	0.16	0.40	-0.45	-0.22	-0.82			.84	1.42
11.24	0.16	0.35	-0.46	-0.21				.82	1.43
9.81	0.13	0.16	-0.60					.77	1.53

Form of the regression equation:

$$M_R = a + b*CL + b.......$$

with the measured moduli in Figures 4-1 and 4-2.

Equations 4-1 and 4-2 may be used to estimate a design resilient modulus when time constraints or other factors make actual testing impractical. However, the equations should not be used for cohesionless soils even though one of the soils tested was non-plastic.

The equations also should not be used routinely to replace testing. The fact that the equations give reasonable predictions of the resilient moduli measured in this study does not verify their reliability. As AHTD begins resilient modulus testing on a regular basis, data should be accumulated and used to verify and/or modify the prediction equations.

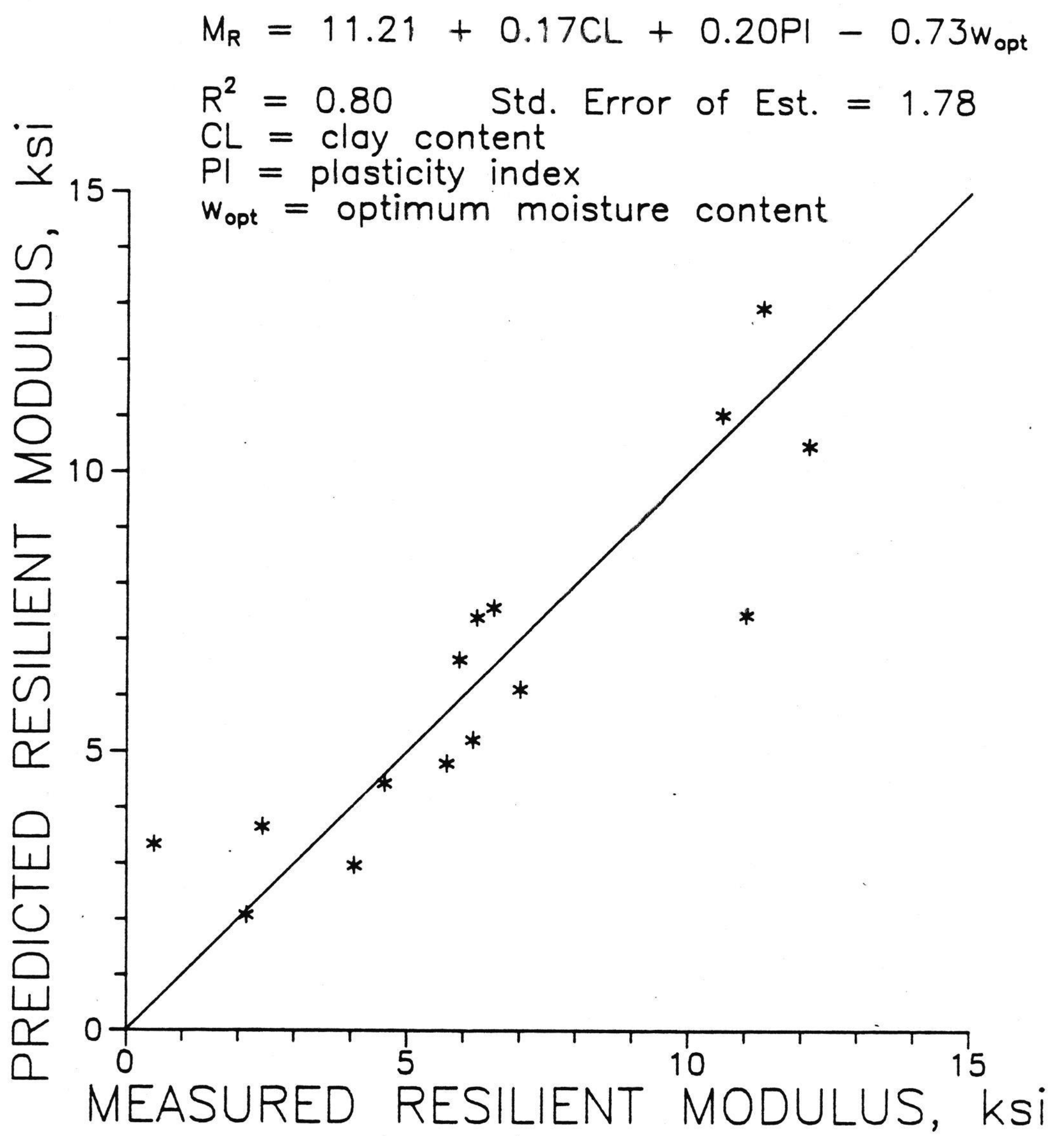


Figure 4-1. Predicted versus Measured Resilient Modulus at 4 psi Deviator Stress.

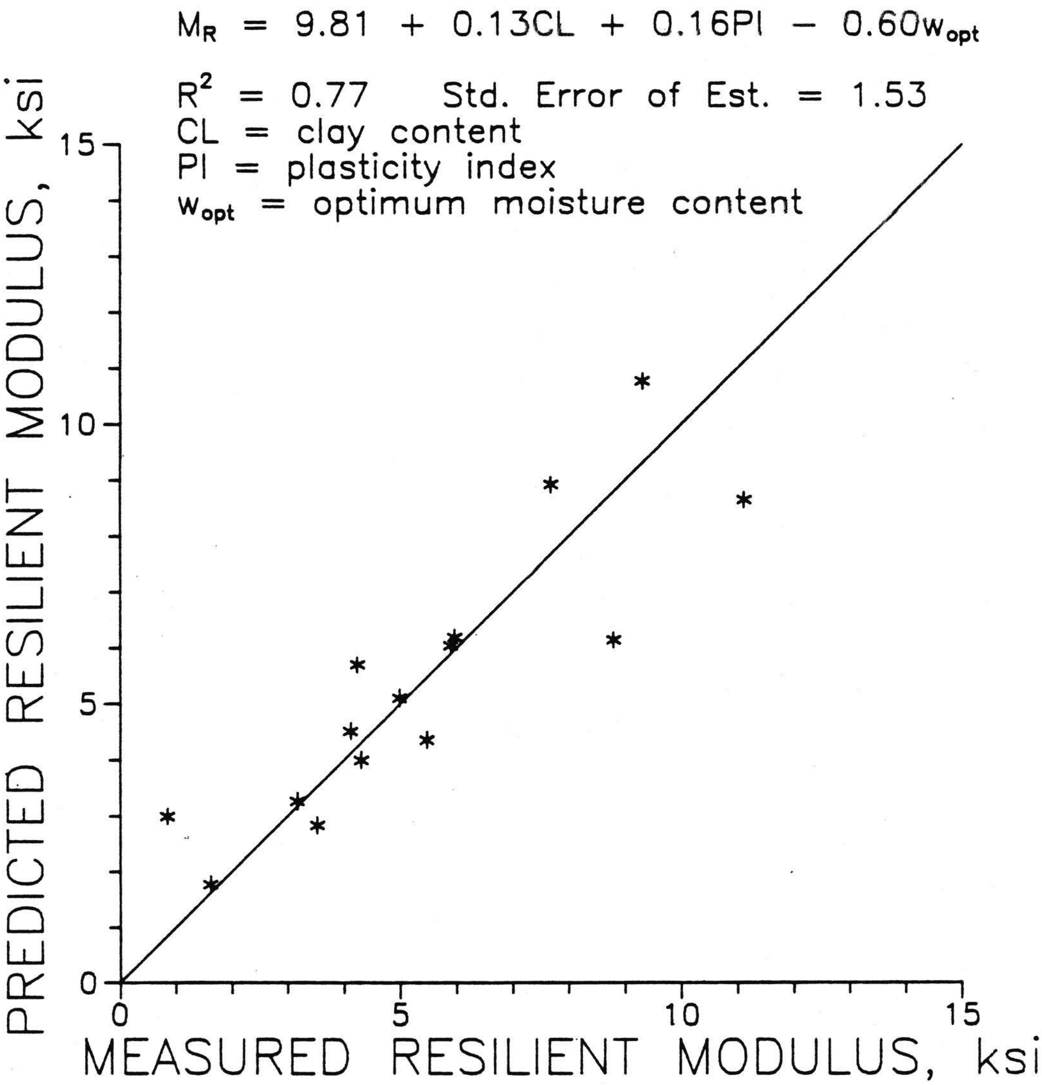


Figure 4-2. Predicted versus Measured Resilient Modulus at 8 psi Deviator Stress.

CHAPTER 5

ROUTINE TESTING RECOMMENDATIONS

SAMPLING AND PRELIMINARY TESTING

Resilient modulus testing provides soil behavior data for pavement design. Soil samples must provide a reasonable representation of the subgrade under the completed pavement. To get representative samples, a soil survey and sampling plan must be established for each design project.

County agricultural soil maps are suggested for planning the soil survey. Each soil type that will be encountered along the highway alignment as well as each soil type that will be taken from borrow pits should be sampled and tested. A minimum of two tests should be reported for each soil type. Additional tests should be made for predominate soil types.

As a part of the sampling plan, the approximate percentage of each soil type should be determined. This can be done by plotting the project centerline on the county maps and scaling the distances of each soil type.

Atterberg limits (AASHTO T90) and moisture-density relationships (AASHTO T99) should be determined for each soil prior to resilient modulus testing. As discussed below, the optimum moisture content and maximum density will be used as the basis for the target density and the moisture contents to be used in the resilient modulus tests.

SAMPLE PREPARATION

Two specimens shall be prepared for each soil to be tested. The moisture contents of the specimens should bracket (one above and one below) the design moisture content. Until a better estimate becomes available, the design moisture content should be 120% of the optimum moisture content for cohesive soils and 100% of optimum for well drained, non-plastic soils.

The amount of soil required for each specimen plus enough for a moisture determination shall be weighed out. Water will be added to bring the soil to the target moisture content. After the water is mixed with the soil, the soil will be stored in a sealed plastic bag at least 24 hours prior to molding the specimens.

The specimens shall be prepared using a kneading compactor. Test specimens used in the research were prepared using R-value molds. The specimens were then trimmed to the appropriate size using a portion of a shelby tube. However, for routine testing it is suggested that molds be made for specimens having the appropriate height to diameter ratio (2:1) without requiring trimming. Molds of the proper size (e.g. 3 inch diameter by 6 inch height) are especially important if the sample contains particles larger than 1/4 inch.

The target density for the test specimens will be 95% of the maximum density. The actual densities achieved and moisture contents shall be determined by weighing and measuring the specimens and weighing and drying a sample of the soil remaining after compaction. Densities between 92% and 98% of maximum will be considered to be acceptable. Moisture contents shall bracket the design moisture content and the low moisture content shall not be less than 105% of optimum.

After compaction and before being removed from the mold, the specimen may be subjected to a static compressive load to assure that the end surfaces are plane. The specimen shall then be removed from the mold. A thin leak-proof membrane shall be placed over the specimen and solid end platens shall be placed on the ends. The ends shall be sealed with O-rings. The specimen shall be stored in sealed plastic bags at least 24 hours prior to testing.

RESILIENT MODULUS TESTING

The resilient modulus test will be conducted on equipment conforming to the requirements of AASHTO T274. Several manufacturers make equipment that meets these requirements.

Prior to testing, the solid end platens shall be replaced with porous stones and the specimen shall be mounted on the platens in a triaxial chamber. The membrane shall cover the porous stones and be sealed to the triaxial end platens.

The test will be conducted at a confining pressure of 3 psi with the bottom drainage line open to the atmosphere. Two levels of deviator stress will be applied, 4 psi and 8 psi. The deviator stress will be applied for a duration of 0.1 seconds and be repeated on a 2 second cycle.

The 8 psi deviator stress will be applied first for 200 repetitions. (The 200 repetitions will serve for both the conditioning phase and the testing.) The resilient (or recovered) deformation during the final repetition shall be recorded.

Occasionally, a soil may be encountered that cannot be tested at 8 psi at the recommended moisture contents. When this occurs, the soil should not be used in the subgrade unless it is improved by use of a modifying agent (e.g. hydrated lime).

Following the 8 psi testing, the deviator stress will be reduced to 4 psi. Fifty repetitions will be applied at 4 psi with the resilient deformation during the final repetition being recorded.

The resilient modulus shall be calculated for each deviator stress using the following equation:

$$M_r = s_d/e_r$$

where

M_r = the resilient modulus, psi

s_d = applied deviator stress, 4 or 8 psi

e_r = resilient strain, resilient deformation divided by specimen length.

REPORTING

The resilient modulus test report shall include sufficient information to allow the designer to select a modulus value consistent with the areal distribution of each soil type, design moisture content, and the subgrade deviator stress. As a minimum, the following information should be reported to the designer:

For each soil

Maximum density (AASHTO T99)

Optimum moisture content (AASHTO T99)

Liquid Limit (AASHTO T90)

Plasticity Index (AASHTO T90)

Approximate percentage of project soil represents

For each test specimen

Density, % of maximum

Moisture content, % of dry density

Resilient modulus at 4 psi

Resilient modulus at 8 psi

CHAPTER 6

SELECTION OF THE DESIGN RESILIENT MODULUS

AASHTO SELECTION METHOD

Pavement design by the AASHTO Guide requires the selection of a single resilient modulus value to represent the subgrade. However, as described in Chapter 2, many factors affect the resilient modulus of a soil. As a result, the modulus value is not a single constant but is a value that changes with moisture, stress states, freeze-thaw cycling, etc.

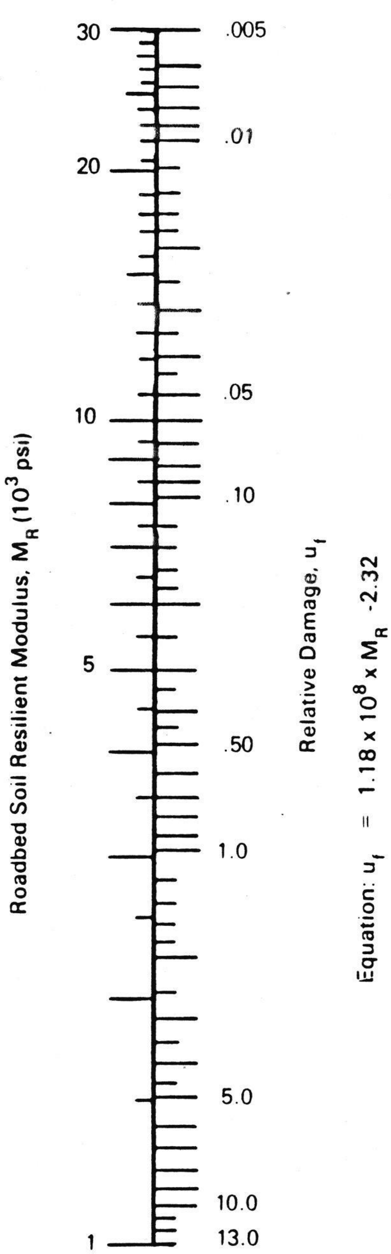
The AASHTO Guide for the Design of Pavement Structures provides a method for selecting the subgrade resilient modulus to be used in design. The AASHTO selection method recognizes the seasonal variation that can result due to moisture variation and freeze-thaw. The method consists of:

1) Estimating monthly or bi-monthly resilient modulus values.
2) Assigning relative damage factors for each month or bi-monthly period.
3) Calculating the 12 month average damage factor.
4) Selecting the design resilient modulus to correspond to the average damage factor.

The AASHTO selection method is illustrated in Figure 6-1 and is described in more detail in the interim report for TRC-94 (4).

The AASHTO method is logical and straight forward. However,

Month	Roadbed Soil Modulus, M_R (psi)	Relative Damage, u_f
Jan.	2200	2.1
Feb.	2200	2.1
Mar.	2200	2.1
Apr.	2200	2.1
May	2700	1.3
June	3900	0.6
July	6000	0.2
Aug.	6800	0.2
Sept.	6800	0.2
Oct.	6000	0.2
Nov.	5400	0.3
Dec.	4500	0.4
	Summation: Σu_f =	11.8



Average: $\bar{u}_f = \frac{\Sigma u_f}{n} = 1.0$

Effective Roadbed Soil Resilient Modulus, M_R (psi) = 3000 (corresponds to $\bar{u}_f$)

Figure 6-1. AASHTO Form for Selecting Design Resilient Modulus.

no guidance is provided for estimating the seasonal moisture variation, freeze-thaw cycles, or representative stress states. Until such guidance is available, the AASHTO selection process is too difficult for routine design. A practical selection process needs to reflect the sensitivity of the design to the resilient modulus and the capability to predict the various parameters that influence resilient behavior.

DESIGN SENSITIVITY AND ACCURACY

The resilient modulus used for design should represent the effect of the subgrade's resilient behavior on the life of the pavement. As such, the ideal selection process should consider the environmental and other factors discussed in Chapter 2. However, as a practical matter, the testing and selection process does not need to be more sophisticated than is warranted by:

1) the capability to predict in-service variables that affect the modulus (e.g. moisture content and freeze-thaw),
2) the sensitivity of the design to the resilient modulus, and
3) the accuracy of the prediction model used for design (i.e. the AASHTO design equation).

Design sensitivity was examined as an early part of TRC-94 and is discussed in detail in the interim report. In practical terms, a 30% change in resilient modulus was found to result in an thickness change of about 1 to 1.5 inches in a Full Depth asphalt pavement (Figure 6-2). A 1 to 1.5 inch thickness change is about the same as the accuracy of the AASHTO design equation.

The AASHTO design equation is based on the AASHO Road Test (10). The basic equation from the Road Test had a standard error of estimate of 0.31 on the logarithm of the number of axle applications (log W). This standard error is approximately equivalent to 30% error in the resilient modulus when applied to the modified equation used in the AASHTO Guide.

The influence of prediction accuracy is considered in the AASHTO Guide design process by the standard deviation (S_o) term which is used with the design reliability. S_o has two components: 1) pavement performance prediction error and 2) traffic prediction error. Any error in testing or selection of the appropriate resilient modulus is reflected in S_o as an increase in the pavement performance prediction error.

The effect of resilient modulus error on S_o was examined for two levels of traffic prediction accuracy. With no error in either traffic prediction or resilient modulus, S_o is 0.31 (the standard error from the AASHO Road Test). With a traffic prediction error of 75% (standard deviation of log W = 0.24) but no error in the resilient modulus, S_o becomes 0.39. When an error in resilient modulus is added, S_o increases (Figure 6-3). With a 15% error in resilient modulus, the change in S_o represents less than 0.25" of asphalt surfacing. Therefore, an error in selecting the appropriate resilient modulus has little effect as long as the error remains below about 15 percent.

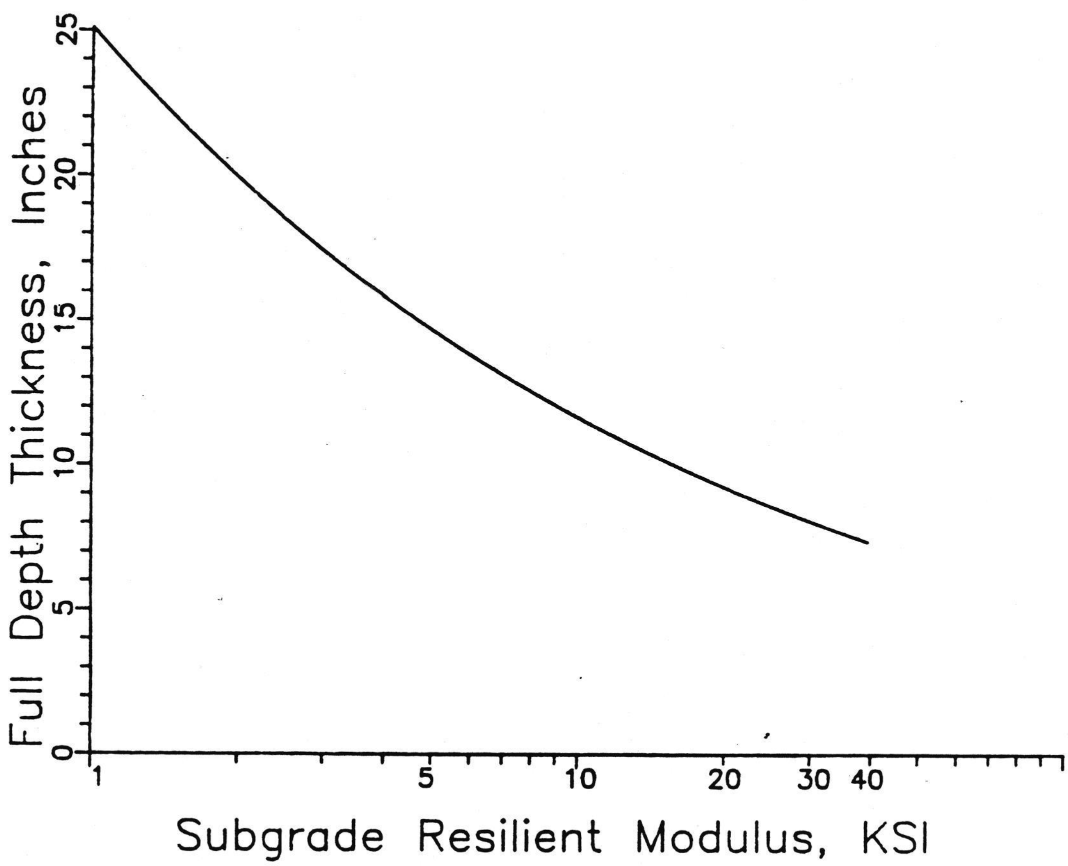


Figure 6-2. Effect of Subgrade Modulus on Design Thickness.

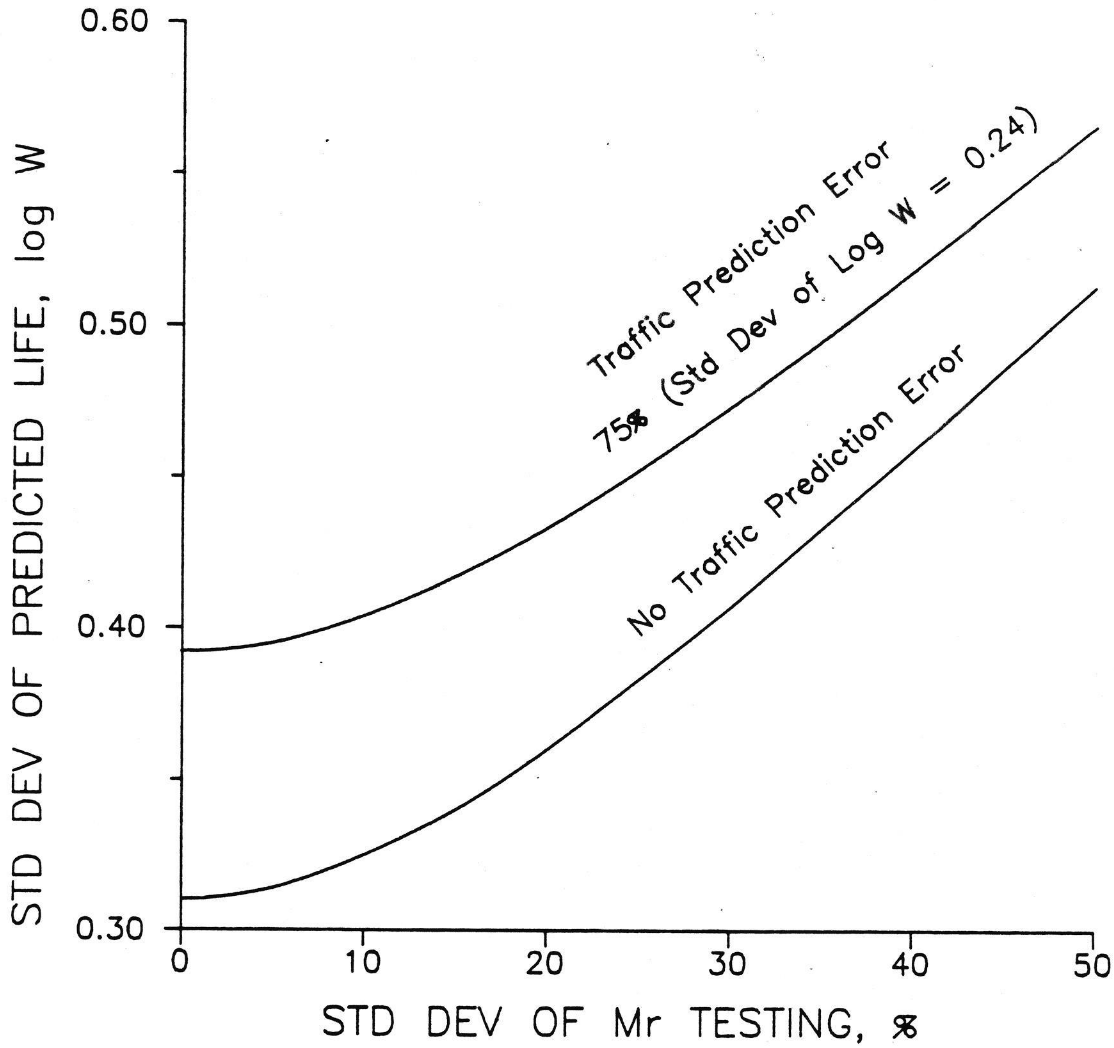


Figure 6-3. Effect of Testing Accuracy on the Standard Deviation of Predicted Pavement Life.

SUBGRADE MOISTURE CONTENT

The moisture content of the subgrade is a critical parameter in the determination of the appropriate resilient modulus for design (see Chapter 2). Because of the moisture sensitivity, significant efforts were devoted to identifying a practical method for predicting the in-service moisture contents of Arkansas subgrades.

One approach to predicting subgrade moisture content is to consider the pavement in terms of external and internal factors. External factors are climatic conditions which influence the supply of moisture to the pavement. Precipitation and temperature are the principal external factors. Internal factors are those properties of the pavement geometry, soil, and materials that interact with moisture. Significant internal factors are: 1) drainability, 2) permeability, 3) soil type, 4) geometry of roadway, 5) surrounding topography, 6) water table depth.

External Factors

The most obvious external factor is precipitation. Studies by the Corps of Engineers (11) confirm that the amount of precipitation has considerable influence on the moisture conditions in pavement subgrade.

However, other factors complicate the prediction of moisture content. For example, in West Germany Kubler (12) found that he could not establish a relationship between precipitation and the change in moisture. He concluded that precipitation alone is not sufficient to predict the ground moisture.

Similarly, Marks and Haliburton (13) indicated that moisture variations beneath pavements in Oklahoma were predominantly temperature dependent. High moisture contents occurred during cold seasons and decreased during summer months, but variations could not be correlated to measured precipitation. Moisture variations resulting from temperature changes were usually between 1 and 5 percent. Moisture content variations were considerably affected by precipitation only where pavements were in poor condition (cracked and pervious). Most moisture variations due to temperature occur on an annual cycle with maximum moisture content occurring during winter months. Moisture variations resulting from infiltration such as those found in shoulders and beneath most overlays were found to lag rainfall by four to six weeks.

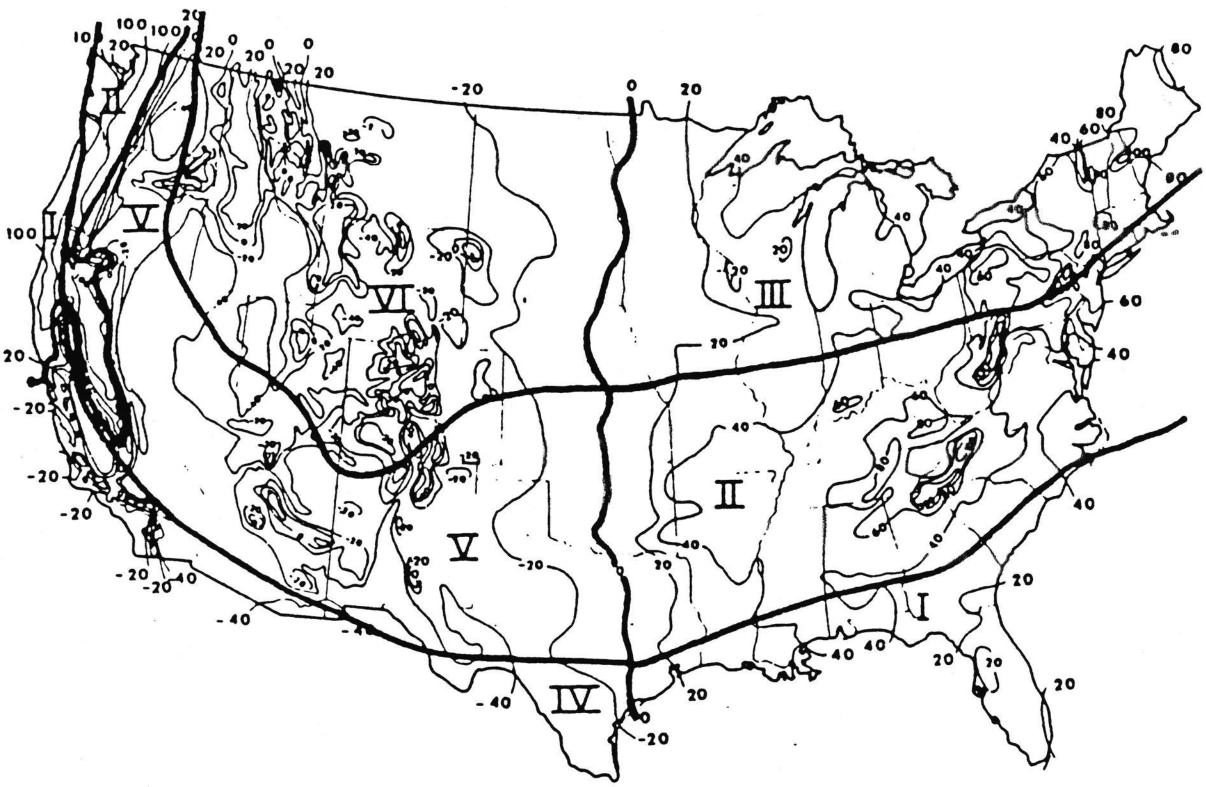
The 1986 AASHTO Guide for Design of Pavement Structures (1) divides the United States into climatic regions (Figure 6-4). These regions represent areas of similar expected pavement performance based on moisture availability in the subgrade and the influence of temperature. Arkansas is in region II indicating that pavements in Arkansas are subjected to high moisture contents throughout the year and experience some freeze-thaw cycling.

Internal Factors

Moisture in pavement systems may come from several sources (Figure 6-5):

1) Seepage of water into the subgrade from higher ground.

2) High water table (this can be expected in the winter



REGION	CHARACTERISTICS
I	Wet, no freeze
II	Wet, freeze - thaw cycling
III	Wet, hard-freeze, spring thaw
IV	Dry, no freeze
V	Dry, freeze - thaw cycling
VI	Dry, hard freeze, spring thaw

Figure 6-4. AASHTO Climatic Regions.

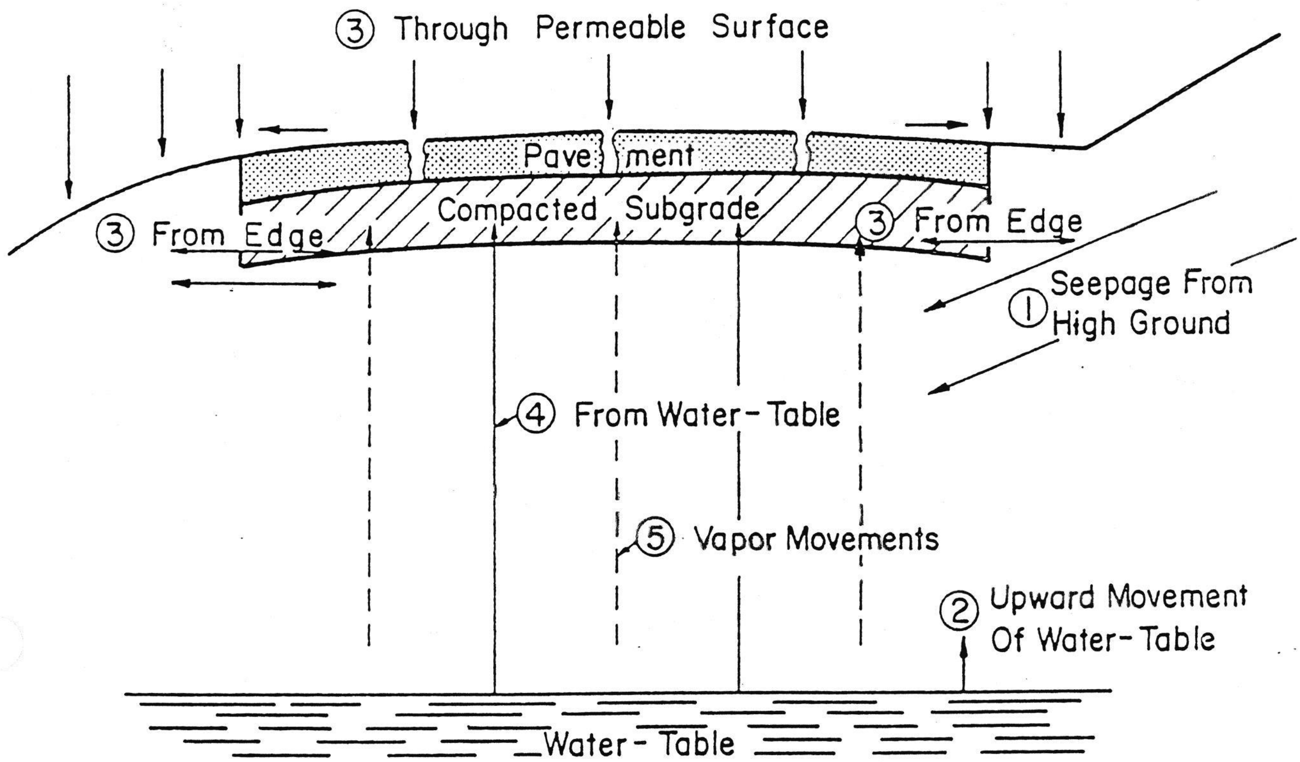


Figure 6-5. Potential Sources of Moisture in Pavements. (Ref. 37)

and spring seasons).

3) Percolation of water through joints and cracks in the pavement surface, penetration at the edges of pavement, and migration of water from shoulder slopes or verges.

4) Water moving vertically in capillaries or interconnected water films.

5) Transfer of water vapor, depending upon adequate temperature gradients and air void space.

Each of these sources influence subgrade moisture content and moisture variation. For example, Kersten (16) found a slight increase in moisture content with increase in depth (the average difference in moisture between the subgrade surface and a depth of 30 inches was 1.0 and 1.5 percent). However, in a study of subgrade moisture contents in Missouri by Guinnee and Thomas (14), the moisture variations were greater in the top levels than at deeper levels.

Some researchers have also concluded that moisture contents at the pavement edges are generally higher than those at the interior location. Guinee and Thomas (14) noted that water gets into a pavement more easily and in greater volumes at the edge of the pavement. Benkelman (15), in an analysis of WASHO Road Test deflection data, concluded that adverse moisture conditions existed at the pavement edges. Kersten (16) also found that moisture contents are ordinarily higher under the edge than under the central portion of the surfacing.

The depth to the groundwater table also plays a major influence on subgrade moisture content. In New Jersey, Turner and Jumikis (17) found that precipitation could modify the position of the water table and subgrade moisture content. In South Carolina, Chu and Humphries (18) found that moisture contents in pavement systems varied with season, and location in the pavement system, and are influenced by the depth of the ground-water table. Marks and Haliburton (13) noted that stable subgrade moisture conditions exist only at sites where the groundwater table was consistently high.

Drainability is a soil parameter that greatly influences subgrade moisture content. Drainability is a function of permeability, physical geometry, and composition of the drainage material. The drainability of a material is related to the ability of a material to exhibit an attraction for water (soil suction).

Elzeftway and Dempsey (19) found that a high attraction for moisture meant poor subgrade drainage. Figure 6-6 shows how different soils exhibit different moisture attraction capacities at the same moisture content. Consequently, soil type can also influence subgrade moisture content.

Kersten (16) found that fine textured soils exhibited a greater tendency to attain moisture contents than coarse textured soils. Clays often attained moisture contents in excess of their plastic limit; but coarse textured soils such as sandy loam rarely had moisture contents as great as their plastic limits. This effect is compounded because clays have higher plastic limits than do sandy loams. Kersten's observations were based in

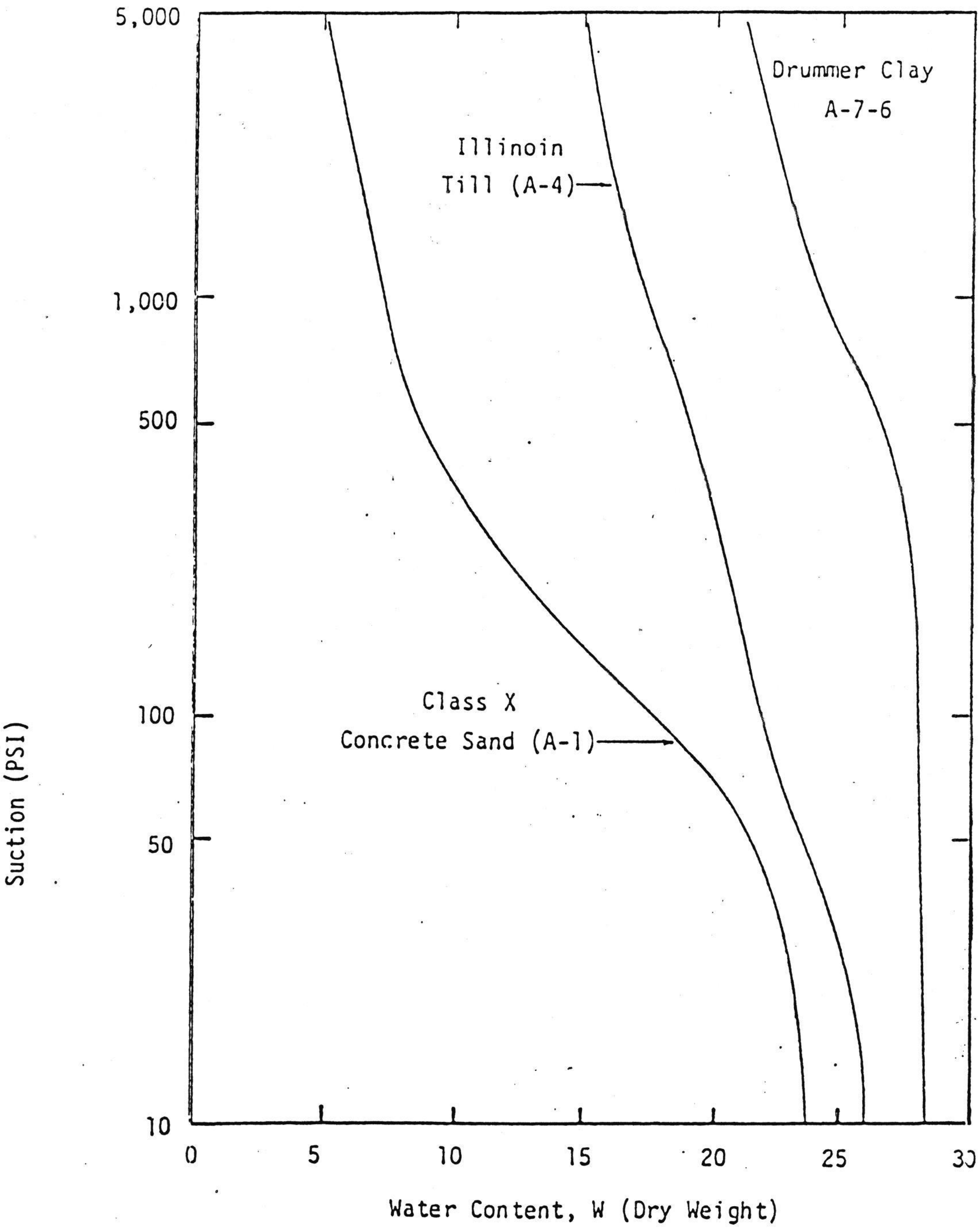


Figure 6-6. Soil Suction versus Moisture Content for Various Soils. (Ref. 37)

part on a survey of subgrade moisture in Arkansas.

Arkansas Subgrade Moisture Study

Kersten (16) analyzed moisture data from 130 Arkansas subgrades. The survey was on pavements with bituminous surfaces, stabilized gravel bases, and clay loam and clay subgrades. Kersten found that the average percent of saturation was 86%, with over half of the tests having saturation of 90% or greater. There was an increase in saturation and a decrease in air void content as the texture varied from light sandy loam soils through clay.

The average moisture content of subgrades in the Kersten study (Table 6-1) was 103% of optimum moisture content. Fifty four percent of the 125 tests exceeded the optimum moisture. Kersten also noted that the fine textured soils such as clays exhibited a marked tendency to attain moisture contents in excess of their plastic limit. Sandy loams rarely had moisture contents as great as their plastic limit.

Methods of Predicting Subgrade Moisture

Prediction of moisture conditions in the subgrade is complex and difficult. Accurate prediction involves the understanding of thermodynamics governing moisture movement in the pavement and influencing climatic factors. Due to the complexity involved, many researchers have developed empirical relationships which mainly relate moisture contents to soil properties and climatic indices.

The Thornthwaite moisture index (20), which relates subgrade

Table 6-1. Arkansas Subgrade Moisture Contents from Kersten's Study (Ref. 16).

SOIL TYPE	AVERAGE MOISTURE CONTENT		PERCENT EXCEEDING	
	Percent of Plastic Limit	Percent of Optimum	Plastic Limit	Optimum
Sandy Loam	72	73	0	17
Loam	69	102	0	47
Clay Loam	82	100	13	41
Clay	105	109	56	70

moisture conditions to climatic indices, has been a popular empirical method of predicting moisture conditions in pavement subgrades. Other researchers have also developed empirical equations based on their observations and accumulated experiences for estimating subgrade moisture under pavements.

Several researchers have reported predictive relationships based on the soil's plastic limit (PL). Swansberg and Hansen (21) found that the water contents of highway subgrades in Minnesota could be estimated in terms of the Plastic limit (PL) using the equation:

$$W = 1.16PL - 7.4.$$

Wooltorton (22) developed a similar equation relating subgrade moisture to the plastic limit. His equation is:

$$W = 1.17PL - 4$$

A Navy study (23) concluded that subgrade moisture contents generally exceed the plastic limit by about 2 percent. Kersten (16) found that the water content for sand and clay soils in damp regions is between 80 percent and 120 percent of the plastic limit.

Nevertheless, the Organization for Economic Cooperation and Development (24) has indicated that many of the empirical formulas for estimating subgrade moisture are not reliable and cannot be described as methods of prediction. Estimates of water content had results scattered around regression lines. The standard deviations of the regression lines usually represent as much as 4 percent moisture content.

Theoretical methods have also been developed. The various theoretical studies indicate the complexity of subgrade moisture movement and consequently the difficuluty of predicting subgrade moisture contents.

A theoretical model for predicting moisture movement and moisture equilibrium in pavement systems usually involves climatic factors and includes model validation and calibration by laboratory and field data. Soil properties and soil suction are important aspects of laboratory studies. The relationship expressed in a soil suction-moisture content characteristic curve is of fundamental importance in the analysis of moisture movement and moisture equilibrium in subgrade soils.

Janssen and Dempsey (25) studied soil suction relationships for a significant number of soils. They found that the soil suction versus moisture content curve (Figure 6-6) can be used to predict the equilibrium moisture content at various levels above the water table.

Researchers from the British Road Research Laboratory (26, 27, 28, 29) developed a rational method based on the thermodynamic theory of equilibrium distribution of water in a porous body. Mathematical formulas based on thermodynamic principles for predicting moisture movement caused by nonisothermal and isothermal conditions have been proposed by other researchers (31, 32, 33, 34, 35).

Figure 6-7 illustrates the results of a theoretical model by Lytton and Kher (36). The figure depicts the accuracy that can be achieved using mathematical models with accurate input data.

Unfortunately, the data needed are not readily available for Arkansas soils and pavement configurations.

Recommended Moisture Content for Design Modulus

The current ability to predict moisture content is extremely limited. Therefore a selection process that incorporates seasonal moisture variation is not warranted until the ability to predict those changes becomes available.

Nevertheless, a moisture content must be selected for the testing. The interim report for TRC-94 (4) recommended testing at either the plastic limit or 120% of optimum moisture content. Except for well drained, non-plastic soils, this recommmendation continues to appear appropriate (although conservative). For well drained, non-plastic soils, the recommended moisture content is 100% of optimum. A study devoted to the determination of appropriate moisture contents for testing is needed.

FROST PENETRATION IN ARKANSAS

A single freeze-thaw cycle reduces the resilient modulus of Arkansas soils by about 50 percent (Chapter 2). Additional cycles apparently result in only minor additional reductions (5). However, additional cycles would prolong the time over which a reduced modulus would be in effect. The significance of the reduced modulus depends upon the frequency of frost penetration into the subgrade.

Northern Arkansas counties can expect frost penetration into the subgrade on the average of 1 year in 10. Analysis of Arkansas

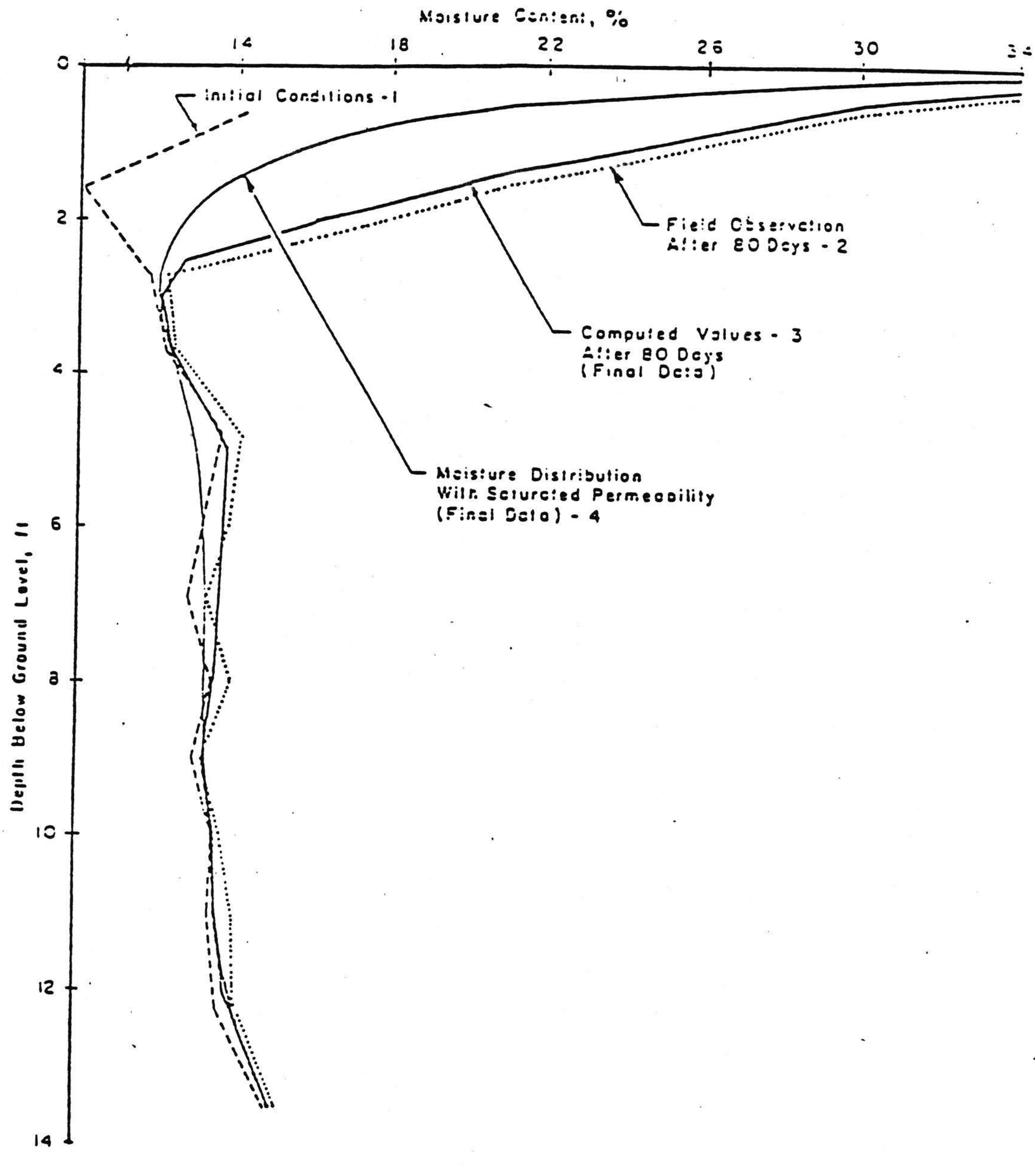


Figure 6-7. Comparison of Predicted and Measured Moisture Profiles. (Ref. 36)

weather data was contained in the project interim report (4). The analysis results were consistent with the Corps of Engineer's map (Figure 6-8).

Therefore, freeze-thaw need not be considered for the lower two thirds of Arkansas because the subgrade seldom freezes. However, some allowance for freezing is warranted for thinner pavements to be constructed in the northern two or three rows of counties.

To determine a reasonable allowance for freezing, analyses were performed (Appendix C) using the AASHTO relative damage approach for selecting the design resilient modulus (1). For the analyses it was assumed that the resilient modulus would be reduced 50% for one month following a thaw. The analyses showed that the design resilient modulus should be reduced 12% to account for an annual freeze-thaw, 6% to account for a freeze-thaw every other year, and 3% to account for a freeze-thaw every five years. Based on these analyses, it is recommended that the design resilient modulus be reduced by 5% for flexible pavements in the northern two or three rows of counties if the total thickness is less than about 15 inches.

DEVIATOR STRESS

Cohesive soils exhibit a resilient modulus stress dependency that is a function of the applied deviator stress (Chapter 2). The design resilient modulus should be selected to reflect the magnitude of load induced deviator stress. The deviator stresses

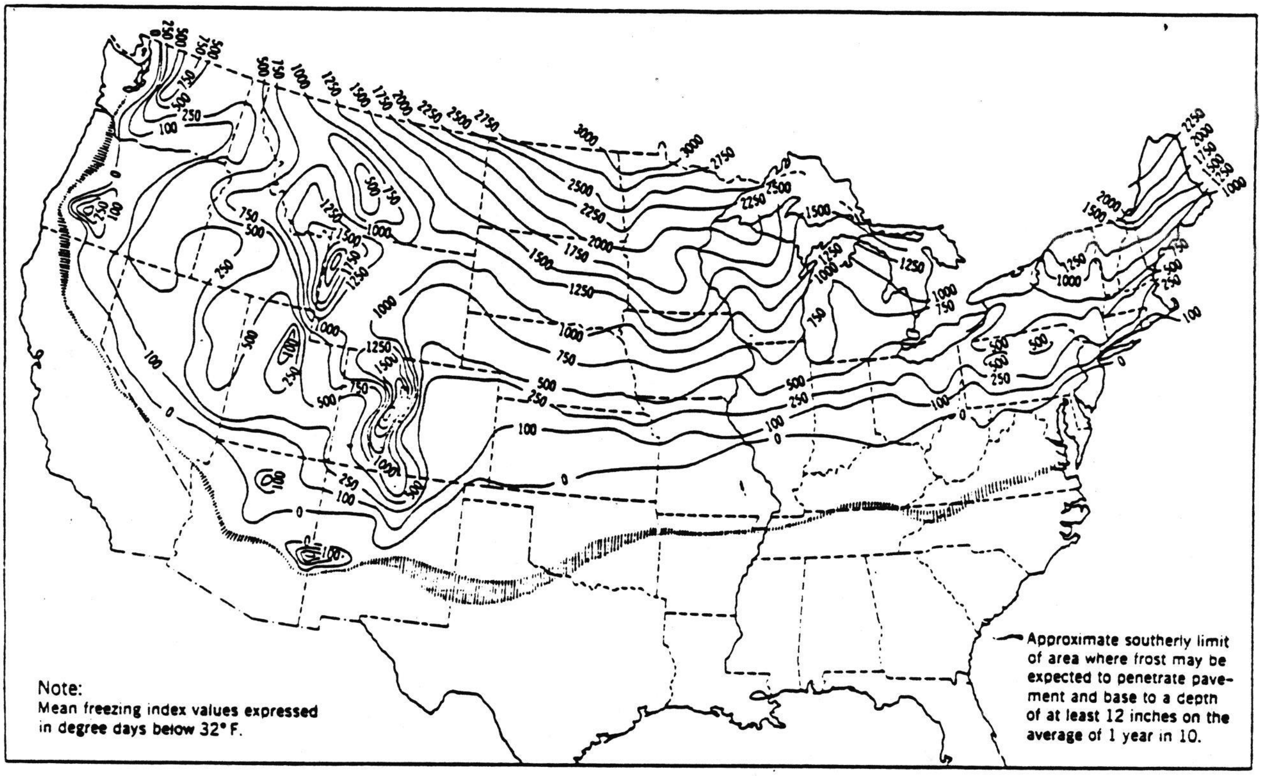


Figure 6-8. Map Showing Limits of Subgrade Freezing. (Ref. 9)

caused by a standard 9000 pound wheel load were determined for typical pavement sections using the elastic layered computer program ELSYM5. Figure 2-2 is a plot of the deviator stresses versus the structural numbers of the pavements.

As would be expected, deviator stress decreases as the structural number increases. For structural numbers greater than about 4.5, the deviator stress is 4 psi or less. For structural numbers less than about 2.5, the deviator stress is 8 psi or greater.

RECOMMENDATION FOR DESIGN MODULUS SELECTION

Selection of the Design Modulus for One Soil

The design modulus selection process needs to consider each of the factors discussed previously that significantly affect the resilient modulus of the soil. However, these factors cannot be considered realistically until they can be predicted and anticipated realistically. Consequently, the influence of seasonal moisture variation cannot be considered at the present time.

As described in Chapter 5, resilient modulus will be determined and reported to the designer at two levels of deviator stress (4 and 8 psi) and at two moisture contents that will bracket the expected field subgrade moisture content. (Until a better method of prediction becomes available, the expected moisture content will be assumed to be 120% of optimum moisture content for the soil.) The measured modulus values will be used to estimate the modulus at the expected field moisture content by the following linear interpolation:

$$M_{rd} = M_{rl} - \left(\frac{d-l}{h-l}\right)(M_{rl}-M_{rh})$$

where

M_{rd} = resilient modulus at the design moisture content

M_{rl}, M_{rh} = resilient modulus at the lower and higher moisture contents respectively

l, h, d = the lower, higher, and design moisture content respectively.

The design resilient modulus for the soil will be selected to be consistent with the deviator stress versus structural number relationship (Figure 2-2). For pavements that will require a structural number of 4 or greater, the design resilient modulus will be M_{rd} at 4 psi. If the required structural number will be less than 2.5, the design resilient modulus will be M_{rd} at 8 psi. For structural numbers between 2.5 and 4.0, the mean of the two M_{rd} values will be used (representing a deviator stress of 6 psi). An example selection of the design resilient modulus for a single soil is contained in Figure 6-9.

If, in the future, it becomes possible to make a reliable prediction of the seasonal subgrade moisture contents, the AASHTO Guide selection process should be added to the above process. The procedure described above would continue to be used in selecting the seasonal modulus values. The seasonal values would then be used in the manner illustrated in Figure 6-1 to select the design resilient modulus for a single soil.

Selection of Project Design Resilient Modulus

The design resilient modulus for the project is to be based on samples representative of all the soils along the project. A soil survey is required to identify the different soil types along the alignment and establish a sampling and testing plan. The soil survey must also identify the percent of the project covered by each soil.

The design modulus will be the average modulus for the subgrade. Past practice (prior to the 1986 AASHTO Guide) was to design pavements based on a lower percentile, weaker soil. However, the 1986 AASHTO Guide bases design on average values. The effect of strength variablity is to be considered with the reliability concept. If appreciably different resilient modulus values are encountered, the design project may be divided into subprojects of similar soils and separate design analyses may be made.

The design modulus is the average modulus for the subgrade, but not necessarily the average of the resilient modulus test values. The average modulus for the subgrade must reflect the areal coverage of each soil. Consequently, the design modulus should be calculated as a weighted average. For example, assume a given project has three soils and that soil A covers 50% of the project, soil B covers 35%, and soil C covers 15%. If the resilient modulus values for the three soils were 6,000 psi for soil A, 9,000 for soil B, and 8,000 for soil C, the design resilient modulus would be:

SELECTION OF DESIGN RESILIENT MODULUS

REPORTED TEST RESULTS

	% Moist.		M_r @ 4 psi	M_r @ 8 psi
Low moisture test (l)	19.0	M_{rl} =	5.28	4.71
High moisture test (h)	22.3	M_{rh} =	4.12	3.47
Optimum water content	17.5			
Design water content (d)	21.0	(120% of optimum)		

CALCULATION EQUATION:

$$M_{rd} = M_{rl} - \left(\frac{d-l}{h-l}\right)(M_{rl} - M_{rh})$$

CALCULATION OF M_{rd} @ 4psi

$$M_{rd} = 5.28 - \frac{(21.0 - 19.0)}{(22.3 - 19.0)}(5.28 - 4.12)$$

$$= \underline{\qquad}$$

CALCULATION OF M_{rd} @ 8psi

$$M_{rd} = 4.71 - \frac{(21.0 - 19.0)}{(22.3 - 19.0)}(4.71 - 3.47)$$

$$= 3.96$$

DESIGN MODULUS SELECTION
(check)
(one)

____ Design Structural Number expected to be less than 2.5 (use M_{rd} @ 8 psi)

____ Design Structural Number expected to be greater than 4.0 (use M_{rd} @ 4 psi)

✓ Design Structural Number expected to be between 2.5 & 4.0 (use average of M_{rd} @ 4 and 8 psi)

DESIGN RESILIENT MODULUS 4.27

Figure 6-9. Example Selection of Design Resilient Modulus.

$$M_r = .50 * 6000 + .35 * 9000 + .15 * 8000$$
$$= 7350 \text{ psi}$$

<u>Summary of the Selection Process</u>

The recommended process for selecting the design resilient modulus is a two step process. In the first step, resilient modulus values are determined for each soil along the project. The process for this step is illustrated in Figure 6-9. If more than one set of tests is conducted on a given soil, a modulus for each set would be determined and the average modulus would be used for the soil.

The second step is to calculate the project design resilient modulus from the values determined for each soil. The design resilient modulus is calculated as a weighted average that takes into account the percent coverage of each soil tested.

CHAPTER 7

CONCLUSIONS, IMPLEMENTATION, AND RECOMMENDATIONS

CONCLUSIONS

TRC-94 has provided a knowledge of the resilient behavior of subgrade soils typically encountered in Arkansas. The study has also developed recommendations for the testing of soils and selecting the subgrade resilient modulus for pavement design. Specific conclusions of the study are:

1. The factors having a significant effect on the resilient modulus of a soil are moisture content, freeze-thaw, and deviator stress.
2. The most significant variable affecting resilient modulus is moisture content. Selection of the moisture content for resilient modulus testing is critical.
3. Density, within the range of 95 to 100 percent of maximum (AASHTO T99), does not have a significant effect on resilient modulus.
4. Although freeze-thaw can reduce resilient modulus by 50%, freeze-thaw need not be considered in Arkansas except for relatively thin pavements in the northern counties. A 5% reduction in the design resilient modulus is appropriate in that area.
5. The standard test procedures of AASHTO T274 may

be simplified without adversely affecting the reliability of the test results. Simplifications include reducing the number of stress cycles to 50, the number of deviator stresses to two, the number of conditioning cycles to 200, and testing at a single confining pressure.

IMPLEMENTATION

Implementable items developed under TRC-94 include:

1. ROUTINE TESTING

Specific recommendations were developed for the routine testing of soils for pavement design. The recommendations incorporate the testing simplifications mentioned above. Details of the recommendations are contained in Chapter 5. These recommendations may be implemented directly into AHTD material testing practice.

2. SELECTION PROCESS FOR DESIGN RESILIENT MODULUS

A specific procedure for selecting the Design Resilient Modulus from the routine test results was also developed. This procedure is presented in Chapter 6. The resilient modulus selection procedure can be incorporated into the AHTD pavement design process.

3. PREDICTION OF RESILIENT MODULUS

Two equations (Eq 4-1 and 4-2) were developed that provide a reasonably good prediction of the resilient modulus of the soils tested in this study. These equations predict resilient modulus from the routine soil properties clay content, PI, and optimum moisture content. When time constraints or other factors preclude resilient modulus testing, the equations may be used to provide a reasonable estimate of the modulus. However, they should be used with caution and should not be used for cohesionless soils.

RESEARCH RECOMMENDATIONS

TRC-94 has demonstrated that the moisture content of the soil at the time of test is critical. A change of 1 to 2% in moisture content can cause a change of 4 ksi or more in the measured resilient modulus of some soils (Table 2-2). A testing error of this magnitude can result in a pavement thickness design error of several inches. A reasonable prediction of subgrade moisture content is essential to resilient modulus testing and good pavement design practice.

Nevertheless, the current ability to predict moisture content is limited and inadequate. If the results of TRC-94 are to be of any real practical value, research must be initiated to develop a reliable, practical method for predicting subgrade moisture.

A second, but less pressing, research need is to validate and/or improve the resilient modulus prediction equations (EQ 4-1 and 4-2). This need not require a formal research effort, but may be accomplished by AHTD using the results from routine testing.

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APPENDIX A

DESCRIPTION OF SOILS TESTED

CALLOWAY

SAMPLE DESCRIPTION

Sample Site: Ashley County, Arkansas; Sec. 26, T17S, R7W; on Highway 82 section 8, two miles south of Hamburg.

Sample Information: Dark yellowish brown silt loam; poorly drained and found in level and nearly level terrain.

SOIL PROPERTIES

Liquid Limit: 34.8

Plasticity Index: 12.5

AASHTO Class: A-6(3)

Grain Size:
- % Silt (0.002-0.074 mm): 36
- % Clay (<0.002 mm): 11
- % Colloids (<0.001 mm): 10

R-Value:
- @ 240 psi: NT
- @ 300 psi: 25

Moisture Density (T-99):
- max. dens.: 107.1 pcf
- w_{opt}: 17.4 %

Organic Content: 1.75 %

Specific Gravity: NT

CARNASAW

SAMPLE DESCRIPTION

Sample Site: Pulaski County, Arkansas; Sec. 25, T3N, R16W; on Highway 113 section 2, 1/2 mile north of Highway 10.

Sample Information: Yellowish brown gravelly silt loam; well drained and found in gently sloping to steep terrain.

SOIL PROPERTIES

Liquid Limit: 32.8

Plasticity Index: 10.0

AASHTO Class: A-4(5)

Grain Size:
- % Silt (0.002-0.074 mm): 43
- % Clay (<0.002 mm): 25
- % Colloids (<0.001 mm): 20

R-Value:
- @ 240 psi: 11
- @ 300 psi: 16

Moisture Density (T-99):
- max. dens.: 112.9 pcf
- w_{opt}: 15.0 %

Organic Content: 2.65 %

Specific Gravity: 2.6736

CLARKSVILLE

SAMPLE DESCRIPTION

Sample Site: Benton County, Arkansas; on Highway 102 approximately 0.8 miles north of Decatur from an exposed slope on the north side of the road.

Sample Information: Cherty silt loam pale brown in color with significant rock fragments; excessively to moderately well drained and found in gently sloping to steep terrain.

SOIL PROPERTIES

Liquid Limit: 23.6

Plasticity Index: 6.0

AASHTO Class: A-4(3)

Grain Size:
- % Silt (0.002-0.074 mm): 66
- % Clay (<0.002 mm): 16
- % Colloids (<0.001 mm): 13

R-Value:
- @ 240 psi: 65
- @ 300 psi: 69

Moisture Density (T-99):
- max. dens.: 109.0 pcf
- w_{opt}: 14.8 %

Organic Content: 1.40 %

Specific Gravity: 2.6303

ENDERS

SAMPLE DESCRIPTION

Sample Site: Crawford County, Arkansas; Sec. 1, T10N, R32W; on Highway 162 section 0, 1/2 mile east of Cedarville.

Sample Information: Strong brown stony fine sandy loam; well drained and found in gently sloping to very steep terrain.

SOIL PROPERTIES

Liquid Limit: 22.3

Plasticity Index: 4.0

AASHTO Class: A-4(1)

Grain Size:
- % Silt (0.002-0.074 mm): 45
- % Clay (<0.002 mm): 23
- % Colloids (<0.001 mm): 22

R-Value:
- @ 240 psi: 20
- @ 300 psi: 25

Moisture Density (T-99):
- max. dens.: 107.9 pcf
- w_{opt}: 17.0 %

Organic Content: 2.75 %

Specific Gravity: 2.6134

FOLEY

SAMPLE DESCRIPTION

Sample Site: Clay County, Arkansas; south of Peach Orchard on Highway 90, 0.2 miles north of the Green County line on the east side of the road at the edge of a plowed field.

Sample Information: Grayish brown silty loam changing to a silty clay loam at approximately 16 inches; poorly drained and found in level terrain.

SOIL PROPERTIES

Liquid Limit: 48.9

Plasticity Index: 24.3

AASHTO Class: A-7-6(25)

Grain Size:
- % Silt (0.002-0.074 mm): 58
- % Clay (<0.002 mm): 34
- % Colloids (<0.001 mm): 32

R-Value:
- @ 240 psi: <5
- @ 300 psi: <5

Moisture Density (T-99):
- max. dens.: 96.7 pcf
- w_{opt}: 20.0 %

Organic Content: 2.70 %

Specific Gravity: 2.6330

GALLION

SAMPLE DESCRIPTION

Sample Site: Faulkner County, Arkansas; Sec. 12, T13N, R14W; on Highway 365, southwest corner of County south of Mayflower.

Sample Information: Reddish brown silty clay loam; well drained and found in level and nearly level terrain.

SOIL PROPERTIES

Liquid Limit: 67.9

Plasticity Index: 42.7

AASHTO Class: A-7-6(40)

Grain Size:
- % Silt (0.002-0.074 mm): 30
- % Clay (<0.002 mm): 55
- % Colloids (<0.001 mm): 54

R-Value:
- @ 240 psi: 3
- @ 300 psi: 4

Moisture Density (T-99):
- max. dens.: 94.3 pcf
- w_{opt}: 25.0 %

Organic Content: 3.00 %

Specific Gravity: 2.6199

GUYTON

SAMPLE DESCRIPTION

Sample Site: Calhoun County, Arkansas; on Highway 167 approximately 0.8 miles north of the Quachita river bridge and approximately 100 yards west of the road at a clearing for a large power line.

Sample Information: Brown silty loam to clay loam; poorly drained and normally associated with low lying frequently flooded bottom lands.

SOIL PROPERTIES

Liquid Limit: 30.2

Plasticity Index: 6.3

AASHTO Class: A-4(6)

Grain Size:
- % Silt (0.002-0.074 mm): 66
- % Clay (<0.002 mm): 26
- % Colloids (<0.001 mm): 22

R-Value:
- @ 240 psi: 20
- @ 300 psi: 26

Moisture Density (T-99):
- max. dens.: 108.5 pcf
- w_{opt}: 16.2 %

Organic Content: 1.75 %

Specific Gravity: 2.6077

JACKPORT

SAMPLE DESCRIPTION

Sample Site: Monroe County, Arkansas; Sec. 31, T13N, R2W; southwest of Brinkley in the maintenance yard.

Sample Information: Dark grayish brown silty clay loam; poorly drained and found in level terrain.

SOIL PROPERTIES

Liquid Limit: 54.9

Plasticity Index: 33.8

AASHTO Class: A-7-6(32)

Grain Size:
- % Silt (0.002-0.074 mm): 48
- % Clay (<0.002 mm): 41
- % Colloids (<0.001 mm): 38

R-Value:
- @ 240 psi: <5
- @ 300 psi: <5

Moisture Density (T-99):
- max. dens.: 94.0 pcf
- w_{opt}: 20.0 %

Organic Content: 3.00 %

Specific Gravity: 2.4405

HOUSTON

SAMPLE DESCRIPTION

Sample Site: Nevada County, Arkansas; at the Highway 19 overpass of I-30, adjacent to the north- bound off ramp from the backslope of the outside ditchline.

Sample Information: Very dark gray clay; moderately well drained and found in nearly level terrain.

SOIL PROPERTIES

Liquid Limit: 59.3

Plasticity Index: 37.7

AASHTO Class: A-7-6(35)

Grain Size:
- % Silt (0.002-0.074 mm): 52
- % Clay (<0.002 mm): 34
- % Colloids (<0.001 mm): 32

R-Value:
- @ 240 psi: 6
- @ 300 psi: 7

Moisture Density (T-99):
- max. dens.: 94.0 pcf
- w_{opt}: 16.0 %

Organic Content: 4.00 %

Specific Gravity: 2.6453

LEADVALE

SAMPLE DESCRIPTION

Sample Site: Boone County, Arkansas; on Highway 43 approximately 3.2 miles north of the Newton County line from an exposed slope on the northwest side of the road.

Sample Information: Silty clay loam brown with red and gray mottles; moderately well drained and found in level to gently sloping terrain.

SOIL PROPERTIES

Liquid Limit: 49.8

Plasticity Index: 20.7

AASHTO Class: A-7-6(19)

Grain Size:
- % Silt (0.002-0.074 mm): 45
- % Clay (<0.002 mm): 37
- % Colloids (<0.001 mm): 36

R-Value:
- @ 240 psi: 16
- @ 300 psi: 17

Moisture Density (T-99):
- max. dens.: 99.2 pcf
- w_{opt}: 21.5 %

Organic Content: 1.85 %

Specific Gravity: 2.6665

PERRY

SAMPLE DESCRIPTION

Sample Site: Lincoln County, Arkansas; on Highway 11 section 3 approximately three miles north of Fresno directly east of the right-of-way at the edge of a plowed rice field.

Sample Information: Gray brown clay; poorly drained and found in level terrain.

SOIL PROPERTIES

Liquid Limit: 94.2

Plasticity Index: 64.0

AASHTO Class: A-7-5(55)

Grain Size:
- % Silt (0.002-0.074 mm): 48
- % Clay (<0.002 mm): 31
- % Colloids (<0.001 mm): 25

R-Value:
- @ 240 psi: NT
- @ 300 psi: NT

Moisture Density (T-99):
- max. dens.: 81.2 pcf
- w_{opt}: 37.4 %

Organic Content: 4.90 %

Specific Gravity: 2.6135

SACUL

SAMPLE DESCRIPTION

Sample Site: Miller County, Arkansas; Sec. 31 & 36, T16S, R27 & 28 W; on Highway 71 section 2, one mile south of Ferguson.

Sample Information: Red silty clay; moderately well drained and found in nearly level to steep terrain.

SOIL PROPERTIES

Liquid Limit: 33.6

Plasticity Index: 11.6

AASHTO Class: A-6(5)

Grain Size:
- % Silt (0.002-0.074 mm): 36
- % Clay (<0.002 mm): 23
- % Colloids (<0.001 mm): 22

R-Value:
- @ 240 psi: 14
- @ 300 psi: 18

Moisture Density (T-99):
- max. dens.: 102.5 pcf
- w_{opt}: 19.5 %

Organic Content: 2.40 %

Specific Gravity: 2.6314

SAWYER

SAMPLE DESCRIPTION

Sample Site: Hempstead County, Arkansas; Sec. 21, T12S, R24W; at the junction of I-30 and Highway 29 near Hope.

Sample Information: Yellowish brown silty clay loam; moderately well drained and found in nearly level to gently sloping terrain.

SOIL PROPERTIES

Liquid Limit: 48.3

Plasticity Index: 27.6

AASHTO Class: A-7-6(23)

Grain Size:
- % Silt (0.002-0.074 mm): 40
- % Clay (<0.002 mm): 41
- % Colloids (<0.001 mm): 36

R-Value:
- @ 240 psi: 3
- @ 300 psi: 4

Moisture Density (T-99):
- max. dens.: 96.0 pcf
- w_{opt}: 22.5 %

Organic Content: 3.00 %

Specific Gravity: 2.6546

SHARKEY

SAMPLE DESCRIPTION

Sample Site: Mississippi County, Arkansas; Sec. 34, T13N, R10E; on Highway 140 section 8, 1/4 mile east of Highway 119Y.

Sample Information: Dark grayish brown silty clay; poorly drained and normally associated with broad flats.

SOIL PROPERTIES

Liquid Limit: 71.4

Plasticity Index: 36.3

AASHTO Class: A-7-5(43)

Grain Size:
- % Silt (0.002-0.074 mm): 39
- % Clay (<0.002 mm): 57
- % Colloids (<0.001 mm): 50

R-Value:
- @ 240 psi: <5
- @ 300 psi: <5

Moisture Density (T-99):
- max. dens.: 87.7 pcf
- w_{opt}: 28.5 %

Organic Content: 3.60 %

Specific Gravity: 2.6691

SMITHDALE

SAMPLE DESCRIPTION

Sample Site: Dallas County, Arkansas; on Highway 9 between the Y intersection of Highway 273 in the east ditch.

Sample Information: Orange red sandy loam; well drained and found in gently sloping to moderately steep terrain.

SOIL PROPERTIES

Liquid Limit: 13.8

Plasticity Index: NP

AASHTO Class: A-2-4(0)

Grain Size:
- % Silt (0.002-0.074 mm): 14
- % Clay (<0.002 mm): 11
- % Colloids (<0.001 mm): 10

R-Value:
- @ 240 psi: 75
- @ 300 psi: 76

Moisture Density (T-99):
- max. dens.: 122.2 pcf
- w_{opt}: 11.5 %

Organic Content: 1.40 %

Specific Gravity: 2.6069

APPENDIX B

RESILIENT MODULUS TEST RESULTS

APPENDIX B

RESILIENT MODULUS TEST RESULTS
(specimens prepared by kneading compaction except as noted)

MOISTURE % of Optimum	RESILIENT MODULUS, ksi Test Deviator Stress, psi 2	4	8	10	16	COMMENT
		-- CALLOWAY --				
69.5	12.2	10.6	10.4	--	10.4	
86.6	9.2	7.5	6.5	--	5.8	
109.1	6.1	5.3	4.7	--	*	
115.1	6.1	4.9	4.1	--	*	
118.7	6.3	4.3	3.7	--	*	
87.4	10.2	7.8	6.6	--	*	static compact.
102.2	9.1	7.1	6.6	--	*	static compact.
107.5	6.4	5.6	5.5	--	5.0	static compact.
		-- CARNASAW --				
91.3	11.0	8.3	8.4	--	8.2	
96.0	11.8	9.1	8.0	--	7.6	
98.1	8.7	8.1	7.7	--	7.0	
112.0	11.3	7.5	4.7	--	3.2	
114.0	11.1	6.1	3.6	--	2.7	
114.9	7.3	6.1	4.8	--	3.7	
116.0	11.8	7.9	4.6	--	3.1	
128.0	6.3	5.0	4.3	--	3.9	
137.0	6.3	4.0	3.0	--	2.6	
		-- CLARKSVILLE --				
74.4	14.9	12.7	11.7	--	11.5	
79.0	13.3	12.1	11.2	--	11.2	
92.1	10.9	9.5	8.6	--	8.2	
94.3	10.0	9.2	8.2	--	8.0	
112.7	6.3	5.7	5.5	--	5.1	
113.5	6.3	5.5	5.3	--	5.1	
84.6	16.5	12.2	10.3	--	10.1	static compact.
86.4	16.6	12.7	10.7	--	10.0	static compact.
102.0	10.3	8.0	7.1	--	7.2	static compact.
102.9	11.0	8.2	7.2	--	7.1	static compact.
125.4	4.6	3.9	3.5	--	3.0	static compact.
125.9	3.8	3.5	2.9	--	3.1	static compact.

* Specimen failed during testing.

MOISTURE % of Optimum	RESILIENT MODULUS, ksi Test Deviator Stress, psi 2	4	8	10	16	COMMENT
	--	ENDERS	--			
70.2	19.8	21.8	20.7	--	17.3	
76.4	11.0	8.3	5.9	--	5.2	
77.6	12.1	10.0	8.4	--	*	
80.6	5.5	4.4	4.1	--	*	
90.0	6.5	4.7	4.0	--	*	
90.6	7.5	5.5	4.7	--	*	
103.5	5.4	4.5	4.1	--	3.6	
113.8	3.2	2.5	2.9	--	*	
115.9	2.6	2.5	3.0	--	*	
Wetter specimens could not be tested.						
	--	FOLEY	--			
83.6	10.0	8.5	8.0	--	7.3	
83.7	9.8	8.4	7.7	--	7.2	
110.2	8.6	7.8	7.3	--	7.1	
114.1	8.5	7.4	7.2	--	7.1	
121.0	6.7	6.0	5.7	--	5.0	
127.0	6.7	6.2	5.4	--	4.7	
	--	GALLION	--			
82.8	24.4	20.2	19.2	17.7	--	
102.8	16.9	16.0	15.0	14.6	--	
118.8	13.3	11.4	8.9	--	3.6	
121.6	12.3	9.5	6.1	4.8	--	
122.0	13.6	9.1	5.7	4.5	--	
125.6	13.5	9.7	5.5	--	*	
126.4	12.2	8.1	5.0	3.6	--	
84.8	20.3	19.4	19.3	18.6	--	static compact.
91.2	23.9	20.9	19.6	18.9	--	static compact.
100.4	19.7	18.7	17.6	15.6	--	static compact.
107.2	17.7	16.0	14.5	13.8	--	static compact.
121.0	15.1	13.3	10.8	9.9	--	static compact.
122.0	13.9	12.3	10.4	9.5	--	static compact.
123.6	14.9	12.7	10.9	6.9	--	static compact.
124.4	12.5	10.8	8.7	7.6	--	static compact.
124.4	13.4	11.9	9.2	7.8	--	static compact.

* Specimen failed during testing.

MOISTURE % of Optimum	RESILIENT MODULUS, ksi Test Deviator Stress, psi 2	4	8	10	16	COMMENT
	-- GUYTON --					
83.0	17.1	15.8	14.9	--	13.6	
90.4	17.5	15.2	13.9	--	12.5	
94.4	17.4	15.2	14.3	--	13.5	
96.6	17.7	16.4	13.6	--	11.6	
100.2	17.1	14.9	10.6	--	8.7	
115.9	10.6	7.8	5.1	--	4.5	
121.7	8.2	5.5	3.7	--	3.6	
	-- HOUSTON --					
82.3	12.1	12.0	10.6	--	10.4	
109.4	14.7	11.6	9.9	--	7.9	
150.0	12.1	10.3	7.6	--	6.1	
160.6	13.3	9.2	6.5	--	3.4	
184.0	6.7	6.5	5.4	--	3.6	

This soil has a very flat moisture density curve and its optimum moisture was unusually low relative to its liquid limit.

MOISTURE % of Optimum	2	4	8	10	16	COMMENT
	-- JACKPORT --					
88.0	16.9	15.6	15.6	15.9	--	
115.5	14.9	14.2	13.3	13.1	--	
129.0	9.1	8.1	7.0	6.5	--	
129.0	9.3	8.0	6.7	6.1	--	
75.5	20.5	19.5	19.4	18.8	--	static compact.
77.5	20.5	19.5	19.4	19.2	--	static compact.
101.5	18.2	17.3	16.4	16.1	--	static compact.
102.0	18.3	17.3	16.9	17.0	--	static compact.
118.0	16.4	14.9	13.2	12.2	--	static compact.
124.5	14.8	14.3	12.4	11.7	--	static compact.
	-- LEADVALE --					
74.7	20.2	18.5	18.0	--	17.3	
76.7	20.0	18.3	16.5	--	15.0	
84.0	17.3	18.3	16.0	--	14.6	
105.6	12.9	11.6	8.8	--	7.7	
115.3	10.1	7.6	5.5	--	4.6	
133.5	7.8	5.2	3.4	--	1.7	
81.4	32.2	27.1	24.9	--	22.8	static compact.
84.7	27.3	25.3	21.8	--	19.8	static compact.
100.3	16.2	14.7	14.0	--	10.6	static compact.
107.5	16.4	13.6	11.4	--	8.0	static compact.
134.4	8.2	5.3	3.4	--	1.3	static compact.

MOISTURE % of Optimum	RESILIENT MODULUS, ksi Test Deviator Stress, psi					COMMENT
	2	4	8	10	16	
		-- PERRY --				
82.1	7.3	6.5	8.4	--	8.7	
93.7	7.3	7.0	5.8	--	4.7	
112.8	4.9	4.3	3.2	--	2.1	
116.0	2.8	2.6	2.2	--	1.4	
117.0	2.4	2.2	2.1	--	1.4	
120.0	2.5	2.1	1.6	--	*	
		-- SACUL --				
74.3	21.6	19.8	19.1	--	18.9	
75.6	16.3	15.1	14.6	--	14.6	
84.8	11.9	10.9	9.8	--	9.2	
90.9	12.6	11.3	8.7	--	6.9	
106.6	13.1	11.0	7.7	--	5.4	
107.0	12.4	10.1	7.3	--	5.2	
118.2	2.1	1.8	1.7	--	*	
Wetter specimens could not be tested.						
		-- SAWYER --				
67.6	17.3	17.5	16.4	15.7	--	
89.3	17.5	17.5	16.8	16.9	--	
93.8	15.1	14.4	14.7	14.4	--	
96.0	14.9	14.2	14.1	14.0	--	
113.3	13.5	12.8	11.9	11.0	--	
115.6	13.3	13.3	11.1	10.1	--	
128.6	7.8	6.6	4.3	--	2.6	
129.4	7.7	5.9	3.8	--	2.5	
75.6	16.6	15.0	14.7	14.7	--	static compact.
91.2	23.9	20.9	19.6	18.9	--	static compact.
100.4	19.7	18.7	17.6	15.6	--	static compact.
107.2	17.7	16.0	14.5	13.8	--	static compact.
121.0	15.1	13.3	10.8	9.9	--	static compact.
122.0	13.9	12.3	10.4	9.5	--	static compact.
123.6	14.9	12.7	10.9	6.9	--	static compact.
124.4	12.5	10.8	8.7	7.6	--	static compact.
124.4	13.4	11.9	9.2	7.8	--	static compact.

* Specimen failed during testing.

MOISTURE % of Optimum	RESILIENT MODULUS, ksi Test Deviator Stress, psi 2	4	8	10	16	COMMENT
	-- SHARKEY --					
82.0	12.9	12.4	12.1	--	11.9	
93.0	10.9	9.9	9.0	--	8.1	
106.3	8.6	8.0	7.5	--	6.8	
106.5	7.4	8.4	8.1	--	7.6	
110.8	7.4	7.0	6.3	--	5.9	
127.4	6.5	6.1	5.6	--	4.8	
138.0	5.1	4.4	3.8	--	2.8	
94.7	8.2	7.8	7.8	--	7.0	static compact.
120.7	6.6	5.9	5.3	--	3.5	static compact.
	-- SMITHDALE --					
64.7	32.7	29.9	27.5	--	23.6	
84.4	27.3	25.3	22.6	--	19.0	
103.6	13.6	11.9	9.9	--	*	
104.1	11.8	10.3	9.5	--	*	
104.3	13.9	12.1	10.3	--	*	
110.1	11.9	9.7	8.5	--	*	

Wetter specimens could not be tested.
This is a non-plastic soils and normally would not be tested at moisture contents much above optimum.

* Specimen failed during testing.

APPENDIX C

ANALYSIS OF THAW SOFTENED SUBGRADE

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ANALYSIS OF THAW SOFTENED SUBGRADE

ASSUMPTIONS:

1. AASHTO relative damage factor equation.
2. Design M_r with no freeze-thaw is representative of all periods without thaw softening.
3. Thaw softening reduces M_r by 50% for one month following the thaw.
4. Period of frozen subgrade is negligible.

AASHTO Relative Damage Factor Equation:

$$u_f = 1.18*10^8 * M_r^{-2.32}$$

ANALYSIS:

$$\text{Mean } u_f = (1.18*10^8)(\sum M_{ri}^{-2.32})/n$$

$$\text{Design } M_r = \{(\text{Mean } u_f)/1.18*10^8\}^{(1/-2.32)}$$

$$= \{(\sum M_{ri}^{-2.32})/n\}^{(1/-2.32)}$$

For one thaw per year:

$$\text{Design } M_r = \{(11M_r^{-2.32}+(.5M_r)^{-2.32})/12\}^{(1/-2.32)}$$

$$= 0.88\ M_r$$

For one thaw every 2 years:

$$\text{Design } M_r = \{(23M_r^{-2.32}+(.5M_r)^{-2.32})/24\}^{(1/-2.32)}$$

$$= 0.94\ M_r$$

For one thaw every 5 years:

$$\text{Design } M_r = \{(59M_r^{-2.32}+(.5M_r)^{-2.32})/60\}^{(1/-2.32)}$$

$$= 0.97\ M_r$$