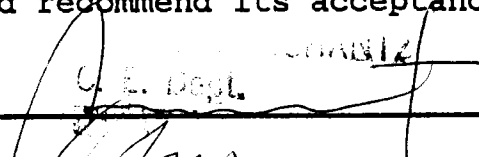
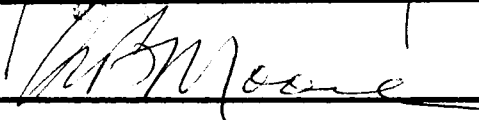


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THE RESILIENT MODULI OF SUBGRADE SOILS IN
THE EAST TENNESSEE AREA

A Thesis
Presented for the
Master of Science
Degree
The University of Tennessee, Knoxville

Teresa Dawn Johnson

December 1986

to my parents

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ABSTRACT

The resilient deformation of a flexible pavement system due to the application of traffic wheel loads on the road surface is a good indicator of pavement performance. In many cases, more than two thirds of the measured deflection has been attributed to the subgrade soil underlying the pavement. The resilient modulus identifies a soil by its behavior under repeated loading conditions.

A program was implemented to determine the resilient moduli of soils in East Tennessee. Laboratory equipment was fabricated and a testing procedure was adopted. A computerized data acquisition system was set-up and computer programs were written to reduce the data.

Fine-grained soil samples were collected from three sites in the East Tennessee area. The soil index properties of each sample were determined from standard laboratory tests. Repeated load tests were conducted on the compacted samples over a range of deviator stresses. Variations in the testing stress level and material properties such as water content and dry density were found to significantly affect the resilient modulus.

A second order polynomial equation, in terms of deviator stress and the degree of saturation, was utilized to predict resilient modulus values.

PREFACE

The Tennessee Department of Transportation contracted with the University of Tennessee Civil Engineering Department and the Transportation Research Center in January 1984 to develop a pavement design procedure for highways and general aviation airfields. Deflection data obtained from non-destructive field testing using a Falling Weight Deflectometer (FWD) is to be incorporated in the design procedure. Field determined deflection is an indicator of pavement condition and remaining useful service life. By reducing or limiting deflections of the pavement structure, fatigue cracking of the asphaltic concrete layer can be prevented and the service life of the pavement will be increased. The pavement design procedure which is based on multilayered linear elastic theory is to be used in the design of asphalt overlays for existing pavements and in the design of new pavement sections (Hight and Moore, 1984).

In order to develop the design procedure, the behavior of each pavement material; asphaltic concrete, aggregate base, and subgrade soil is being investigated. A characterization of the response of three East Tennessee subgrade soils under repeated load conditions is described in this thesis. The testing procedure established for these three subgrade soils will be utilized throughout the remainder of the project to test soils across the state.

TABLE OF CONTENTS

CHAPTER	PAGE
I. INTRODUCTION	1
II. REVIEW OF RESILIENT MODULUS RESEARCH	11
III. COLLECTION AND PREPARATION OF SAMPLES	22
1. Field Sampling	
2. Laboratory Sample Preparation	
IV. REPEATED LOAD TESTING	35
1. Apparatus for Testing	
2. Data Acquisition and Reduction	
3. Testing Procedure	
4. Justification of Testing Procedure	
V. LABORATORY TEST RESULTS	47
1. Standard Tests	
2. Repeated Load Tests	
VI. CHARACTERIZATION OF RESULTS	57
1. Methods of Evaluation	
2. Predictions of the Resilient Modulus	
VII. CONCLUSIONS	76
1. Summary	
2. Recommendations	
REFERENCES	80
APPENDICES	87
APPENDIX A. COMPUTER PROGRAMS AND SAMPLE OUTPUT ...	88
APPENDIX B. CALIBRATION OF THE TESTING EQUIPMENT ..	97
APPENDIX C. LAB DATA MANAGER	99

CHAPTER	PAGE
APPENDIX D. ADDITIONAL LABORATORY TEST DATA	100
APPENDIX E. STANDARD ERROR CALCULATION FOR THE CURVFIT COMPUTER PROGRAM	116
APPENDIX F. PLATES	117
VITA	124

LIST OF TABLES

TABLE		PAGE
I.	Field Data Collected from Three Sites	24
II.	Laboratory Tests and Specifications	27
III.	Standard Laboratory Test Results	48
IV.	Soil Classification	49
V.	Curvefit Data for the Rutledge Pike Samples ...	60
VI.	Curvefit Data for the Jefferson Co. Samples ...	60
VII.	Curvefit Data for the Cookeville Samples	61
VIII.	Predictions of a Rutledge Pike Sample	
	Repeated Load Test	67
IX.	Predictions of a Rutledge Pike Sample	
	Repeated Load Test	68
X.	Predictions of a Rutledge Pike Sample	
	Repeated Load Test	68
XI.	Predictions of a Jefferson County Sample	
	Repeated Load Test	69
XII.	Predictions of a Jefferson County Sample	
	Repeated Load Test	69
XIII.	Predictions of a Jefferson County Sample	
	Repeated Load Test	70
XIV.	Predictions of a Cookeville Sample	
	Repeated Load Test	70

TABLE	PAGE
XV. Predictions of a Cookeville Sample	
Repeated Load Test	71
XVI. Predictions of a Cookeville Sample	
Repeated Load Test	71

LIST OF FIGURES

FIGURE		PAGE
1.	Pavement Layer System	1
2.	Typical Stress-Strain Curve for Repeated Load	
	Triaxial Testing	4
3.	General Soil Region Map of Tennessee	8
4.	East Tennessee Permanent Test Sites	10
5.	Schematic Diagram of the Kneading	
	Compaction Foot	30
6.	Schematic Diagram of the Pneumatic Actuator	36
7.	Cyclic Load versus Time Plot	38
8.	Schematic Diagram Depicting the Computer	
	Determined Resilient Modulus and the	
	Actual Resilient Modulus	40
9.	Plot of Resilient Modulus (E_r) versus Deviator	
	Stress for Several Rutledge Pike Samples	51
10.	Plot of Resilient Modulus (E_r) versus Deviator	
	Stress for Several Jefferson County Samples ..	52
11.	Plot of Resilient Modulus (E_r) versus Deviator	
	Stress for Several Cookeville Samples	53
12.	Plot of Resilient Modulus (E_r) versus Deviator	
	Stress for Samples at Optimum Water Content ..	56
13.	Diagram of the Bi-linear Relationship and Some	
	Typical Tennessee Data	58

FIGURE	PAGE
14. Scatter Plot of "A" versus Degree of Saturation	62
15. Scatter Plot of "B" versus Degree of Saturation	62
16. Plot of "Eo" versus Degree of Saturation	63
17. Plot of Predicted Modulus Values for Three Rutledge Pike Samples	73
18. Plot of Predicted Modulus Values for Three Jefferson County Samples	74
19. Plot of Predicted Modulus Values for Three Cookeville Samples	75
20. Plot of Resilient Modulus (Er) versus Deviator Stress for All of the Rutledge Pike Samples	113
21. Plot of Resilient Modulus (Er) versus Deviator Stress for All of the Jefferson County Samples	114
22. Plot of Resilient Modulus (Er) versus Deviator Stress for All of the Cookeville Samples	115

LIST OF PLATES

PLATE	PAGE
1. Permanent Test Site in Knox County on State Route 1 (Rutledge Pike)	118
2. Core Driller Set-Up on Wheel Path Number One ...	118
3. Wheel-Mounted Core Driller with Six-Inch Diameter Bit (Shown in Operation)	119
4. Kneading Compactor Used to Prepare Soil Samples	120
5. Repeated Load Testing Apparatus	121
6. Computer Monitoring System	122
7. Complete Lab Set-Up with Repeated Load Testing Apparatus and Computer Monitoring System	123

CHAPTER I

INTRODUCTION

When traffic wheel loads are applied to the road surface, the pavement system deforms. The passing of traffic over a road surface results in repeated loading of that pavement system. A flexible pavement system is composed of three basic layers; asphaltic concrete, granular base material, and subgrade soil which are illustrated in Figure 1.

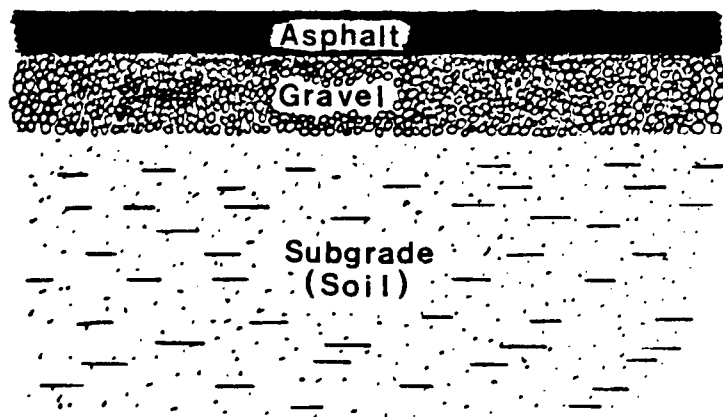


Figure 1. Pavement Layer System.

The actual deformation of the layers due to the wheel loads is composed of two parts -- irrecoverable permanent or plastic deformation and recoverable resilient or elastic deformation. Under normal traffic wheel loads, plastic deformations compose only a small portion of the total deformation. Field studies by several agencies have

correlated resilient deformation and fatigue failures in the asphalt pavement. The magnitude of the resilient deformation was determined to be a good indicator of pavement performance (Robnett and Thompson, 1973).

Multilayer linear elastic theory can be used to analyze pavement layer systems and to predict pavement performance. Elastic theory employs the following assumptions (Yoder and Witczak, 1975):

1. Material properties of each layer are homogeneous.
2. Each layer has a finite thickness, except the subgrade which is semi-infinite.
3. Each layer is isotropic.
4. Full friction is developed between layers at each interface.
5. There are no shearing forces at the surface.
6. The relationship between stress and strain is completely described by two material properties for each layer (by definition for isotropic and homogeneous materials):

E = Elastic or Resilient Modulus

μ = Poisson's Ratio.

The modulus value used in elastic theory analysis is generally determined from laboratory repeated load testing. Modulus values are determined for the asphaltic concrete, granular base material, and subgrade soil. Behavior of the subgrade soil is of primary importance because of the role the subgrade plays in deflection. As much as 67% of the total deflection from traffic wheel loads has been attributed to the subgrade (Robnett and Thompson, 1973).

The property that relates a soil's elastic deformation to an applied stress is the resilient modulus. It can be defined by the following equation:

$$E_r = \sigma_d / \epsilon_r$$

where: E_r = the resilient modulus

σ_d = the deviator stress ($\sigma_1 - \sigma_3$)

ϵ_r = the recoverable (elastic) strain.

The resilient modulus is typically determined in a triaxial test where the axial stress (σ_1) is applied to the top of the sample and (σ_3) is the corresponding confining stress or chamber pressure. Deviator stress is the difference between the axial and confining stress. A plot of deviator stress versus axial strain for repeated loading is shown in Figure 2. For a given deviator stress,

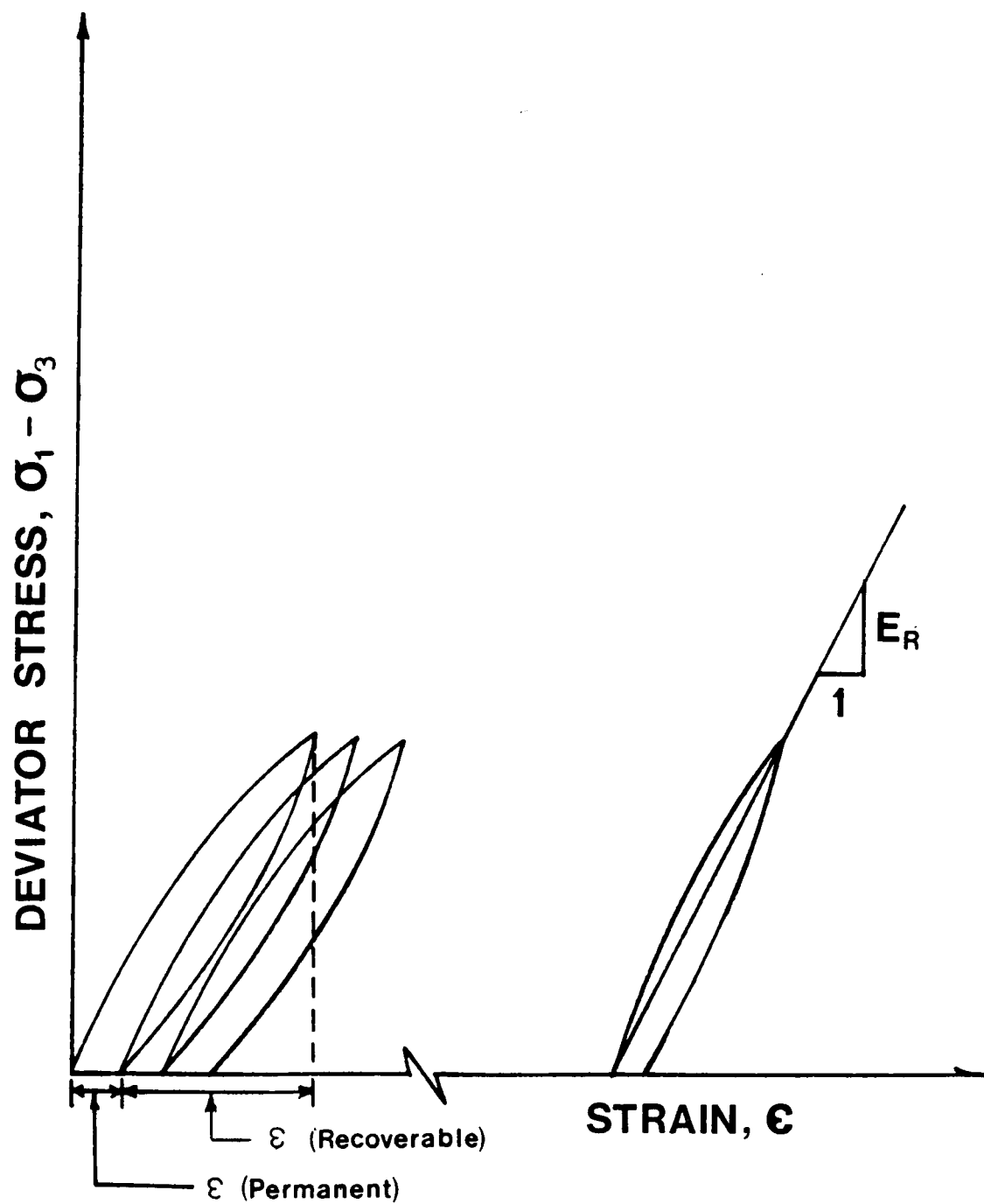


Figure 2. Typical Stress-Strain Curve for Repeated Load Triaxial Testing.

the ratio of recoverable strain to total strain increases as the number of load cycles increases. For most soils, the magnitude of recoverable strain per load cycle becomes constant after about 100 load cycles. Consequently, the resilient modulus at a given stress level also becomes constant. This constant value of E_r is reported as the resilient modulus.

Young's Modulus or the modulus of elasticity is defined as the ratio of applied axial stress to the corresponding axial strain. A distinction is made between the modulus of elasticity and the resilient modulus because the modulus of elasticity is determined from "monotonic loading", while the resilient modulus is determined from repeated load applications. Monotonic loading implies a single application of increasing load.

The resilient modulus of any soil is a function of the state of stress. As the applied stress is varied, the strain in the soil may vary disproportionately. Coarse grained materials generally exhibit a decrease in strain at higher stress levels, and fine grained soils generally exhibit an increase in strain at higher stress levels. This behavior explains why granular materials are sometimes referred to as "stress hardening" materials while fine grained soils are referred to as "stress softening" materials. An increase in strain value causes the

resilient modulus to decrease when greater stresses are applied. The decrease in the resilient modulus for fine grained soils is clearly visible on graphs of the resilient modulus versus the applied deviator stress. This may be seen in typical test data presented in Chapter V.

The behavior of soil due to repeated loading is dependent on factors other than applied stress. Some other factors that have been found to affect the resilient modulus include moisture content, degree of saturation, density, method of compaction, grain size and mineralogy (Robnett and Thompson, 1973).

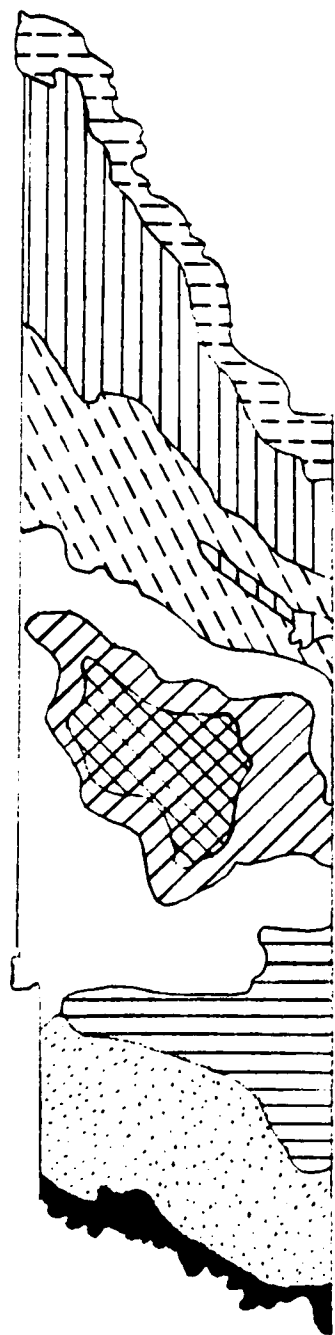
Characterization of Tennessee subgrade soil behavior due to repeated loading has been a primary goal of a research project at the University of Tennessee. A preliminary study was conducted in 1984 to determine the types of soil found across the state and to decide which types of soil should be tested (Hightner and Moore, 1984).

A source of detailed information concerning soil types is the Department of Agriculture Soil Conservation Service (SCS). The SCS in cooperation with the Tennessee Agricultural Experiment Station mapped the soil types in each county of Tennessee. Although the primary goal of SCS is to provide information to farmers, some engineering properties of soils were determined while soil samples were being evaluated (Birdwell, 1984).

A map of the principal soil regions of Tennessee,

shown in Figure 3, illustrates the variation and complexity of soils across the state of Tennessee. The nine soil regions depicted on the map are based on physiographic regions of the state. Soils of the Major Stream Bottoms (A) and Soils of the Coastal Plain Region (C) have alluvial (water transported) soil origins, while Soils of the Loess Region (B) have an aeolian (air transported) soil origin. The remaining six soil regions have residual soil origins i.e. weathered in place from bedrock. These nine regions were subdivided by the SCS into different soil groups resulting in 376 different soils across the state of Tennessee. The differences are due to variations in temperature, rainfall, vegetation, and underlying rock. (Springer and Elder, 1980).

The decision concerning which of the 376 soils to test was based on location, prevalent soil types and the amount of traffic. Only those soils underlying interstates and state routes were considered. Measurements of the length in miles of each soil type were made for I-24, I-40, I-65, I-75, I-81 and nineteen different state routes. Average daily traffic (ADT) values were also recorded for each soil type on each roadway. The ADT values in conjunction with the soil mile values were used to determine priority soil types that should be tested (Hightler and Moore, 1984).



-  A -- Soils of the Major Stream Bottoms
-  B -- Soils of the Loess Region
-  C -- Soils of the Coastal Plain Region
-  D -- Soils of the Highland Rim
-  E -- Soils of the Outer Part of the Nashville Basin
-  F -- Soils of the Inner Part of the Nashville Basin
-  H -- Soils of the Cumberland Plateau
-  J -- Soils of the Ridges and Valleys
-  K -- Soils of the Unaka Mountains

Figure 3. General Soil Region Map of Tennessee.

Three permanent test sites were chosen in the eastern part of the state. Permanent test sites in Middle and West Tennessee are to be chosen after the investigation of the three eastern sites has been completed. The East Tennessee sites were chosen to give a representative sample of the area's subgrades with special consideration of areas with high average daily traffic values. The locations of the three East Tennessee permanent test sites are shown in Figure 4.

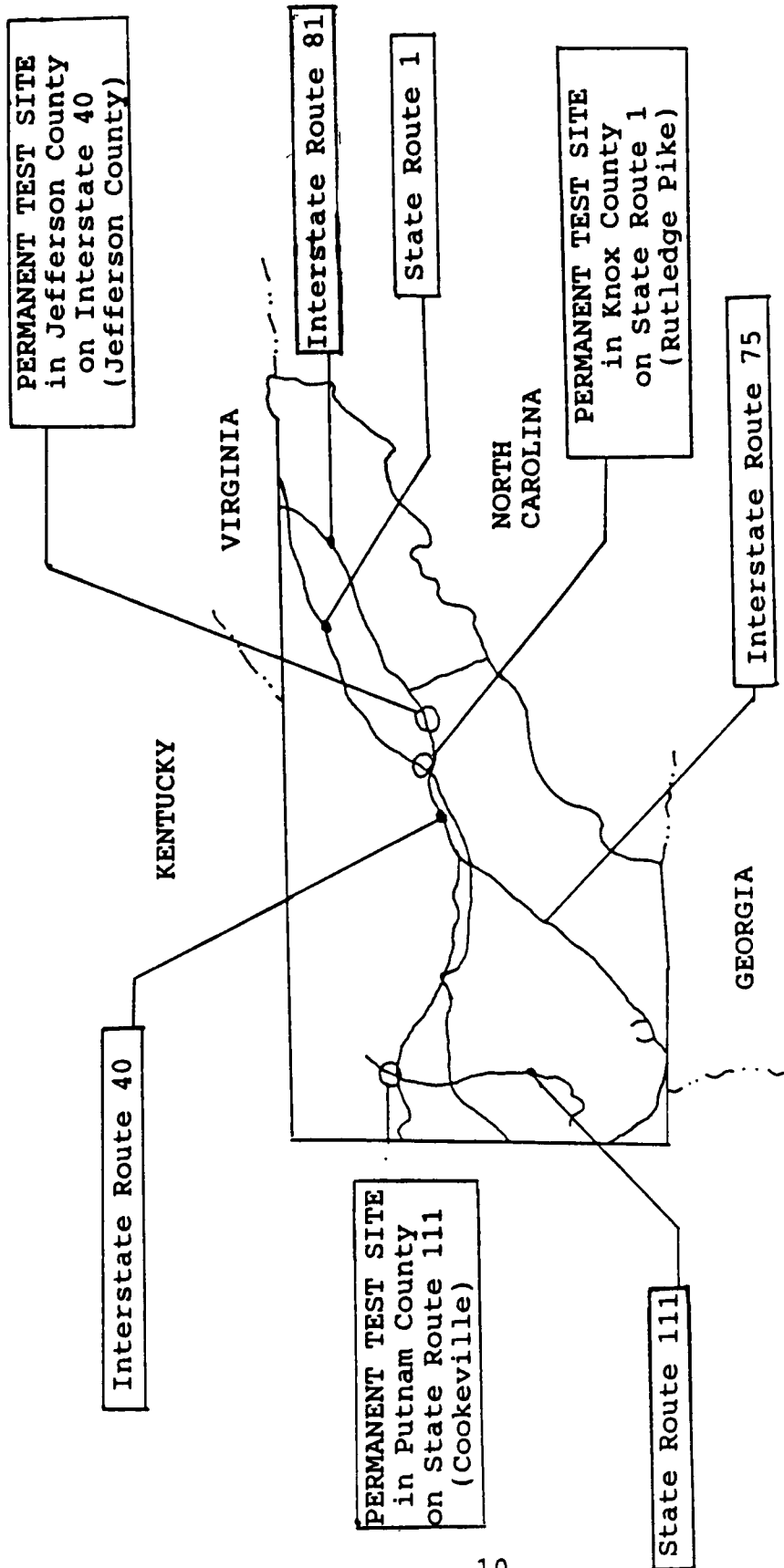


Figure 4. East Tennessee Permanent Test Sites.

CHAPTER II

REVIEW OF RESILIENT MODULUS RESEARCH

Repeated load testing of subgrade soils was first proposed when the data from the Washington Association of State Highway Officials (WASHO) and the American Association of State Highway Officials (AASHO) Road Tests showed significantly greater deflections than the values calculated from static load applications. Research was begun in the late 1950's to characterize repeated load behavior of soils and to determine the factors that affect the resilient modulus (Seed and Monismith, 1962).

Comparisons between statically determined modulus values (E_s) and those obtained from repeated load tests (E_r) revealed significant differences between moduli. At any given stress level the modulus determined from a conventional test was found to be 1.05 to 2 times greater than the modulus from a repeated load test. A general trend of increased moduli (E_s and E_r) was found to occur when water content was kept constant while dry density was increased. For a given dry density, modulus values were found to decrease as the water content was increased (Ahmed and Larew, 1962).

Seed, Chan and Lee experimented with the number of stress applications, time interval between compaction and testing, stress intensity, compaction method, and

saturation effects in conjunction with repeated load tests. Each of the factors investigated has a significant effect on the resilient modulus of a soil sample and subsequent research has been based on their findings. The highest recorded deformations were found to occur between one application and five thousand applications and then deformations decreased substantially for over 10,000 stress applications. The time interval between compaction and testing was determined to affect the resilient modulus due to thixotropic strength gains. Samples compacted to a high degree of saturation by a method that induces shearing strains were found to exhibit strength increases when they were placed in a moist cure room for several days before testing. Stress intensity was also found to have a major effect on the resilient modulus. As the deviator stress is increased (from 3 psi to 15 psi), the resilient modulus decreases rapidly then levels off at stress values greater than 15 psi. A flocculated or random soil structure was found to occur when compaction at low saturation levels was conducted by any method. Seed, Chan and Lee concluded that kneading compaction simulates the conditions of compaction in the field by inducing the necessary shearing strains and producing a dispersed or oriented soil structure. "Worst field conditions" were checked by compacting samples to a high degree of

saturation with a kneading compactor (Seed, Chan, and Lee, 1962).

Several studies correlating field and laboratory resilient moduli were published in the late sixties. In one study, modulus values obtained from plate load tests in the field were determined to be from two to four times greater than modulus values determined from laboratory triaxial tests. The resilient modulus was found to increase with undrained shear strength and decrease with increasing shear stress, but no useful trends were proposed due to inconsistencies in the data (Soderman, Kim, and Milligan, 1968). In another study, displacement in laboratory repeated load tests was compared to deflections in pavement subgrades using existing AASHTO Road Test data. The "threshold stress" concept which describes the onset of creep deformation was used to explain when and how plastic strains occur during repeated loading (Glynn and Kirwan, 1969).

Stress and strain controlled laboratory tests along with plate load field tests were implemented in the study of a glacial lake clay by Hanna and Adams. The field determined modulus values were higher than those determined from laboratory tests. Values from controlled stress tests were consistently higher than for controlled strain tests (Hanna and Adams, 1968).

An evaluation of a southeast Asian airfield pavement

by repeated plate load bearing tests was compared to repeated load behavior of saturated remolded kaolin samples in the laboratory by Ting. A "transformation" equation incorporating stress, strain, number of repetitions and a time factor was determined from laboratory tests. A form of the "transformation" equation was applied to the evaluation of repeated plate load bearing test data. The equation provided a simple and accurate method of evaluating the field data (Ting, 1970).

Mechanical, pneumatic and hydraulic devices used in repeated load laboratory testing were compared to determine the limitations of each. For long term tests on samples with pore pressure measurement, servo-controlled hydraulic testing equipment was preferred by the researchers (Cullingford, Lashine, and Parr, 1972). Lashine conducted a study of a silty clay under repeated loading using the servo-controlled test equipment. Undrained compressive strength and moisture content were found to be directly affected by the deviator stress which resulted in failure (Lashine, 1973).

A study on the effective stress response of saturated clay samples under repeated loading revealed a linear relationship between pore pressure increases and applied repeated stress when that stress was below a critical value (Sangrey, Henkel, and Esrig, 1969). The "critical value" of stress below which nonfailure equilibrium is

reached but above which nonrecoverable strain is recorded was investigated further by Wilson and Greenwood. Closed stress-strain hysteresis loops and elastic behavior were found to accompany stresses below the "critical value" (Wilson and Greenwood, 1974).

As a first step in the prevention of fatigue failures in asphalt pavements, factors affecting the resilient modulus of a silty clay and a sand were investigated by Tanimoto and Nishi. An equation characterizing the behavior of the sand was obtained from the study but not for the silty clay (Tanimoto and Nishi, 1970). Unsaturated silty loam and sandy clay loam were subjected to repeated load tests by Lou. The resilient moduli obtained from the tests were introduced into pavement design equations (Luo, 1973). In London, repeated load tests on two subgrade materials were made utilizing apparatus and procedures developed by H. B. Seed. Permanent deformation was found to be high even in the early stages of loading (Clarke, 1974). Similar resilient modulus research was also being conducted on "low bank roads" in Japan (Yamanouchi and Yasuhara, 1975).

In a study by Brewer, kaolinite samples were subjected to repeated loading while pore pressure and axial strain measurements were made. Cyclic strain was found to be directly related to the frequency and amplitude of the applied axial stress. Cyclic confining

stress was applied to samples to investigate the cyclic pore pressure response. An "equilibrium pore pressure" was found to correspond to a given cyclic confining stress (Brewer, 1972).

Researchers at the University of Illinois began an extensive study of the resilient properties of subgrade soils in 1973 (Robnett and Thompson, 1973). Characterization of all subgrade soils in the state of Illinois by their resilient moduli was proposed. Incorporation of the subgrade response in a computer program called ILLIPAV was used to predict pavement deflections in the field (Thompson and Robnett, 1976).

The U.S. Army Waterway Experiment Station conducted a study on end restraint in cyclic loading tests and found that clays (unlike sands) were not affected by end restraint (Vernese and Lee, 1977). In another of their investigations, a heavy clay was studied to reveal how stiffness related to rutting potential, elastic strain, resilient modulus, and Poisson's ratio. Rutting potential, as indicated by plastic axial strain, was found to increase with the magnitude and number of stress applications. The resilient modulus was observed to decrease initially with stress repetitions, then showed a gradual increase as additional stress applications were applied. The researchers concluded that proper

conditioning would limit this effect (Townsend and Chisolm, 1976).

Several groups published literature in an attempt to define some of the problems associated with repeated load testing and to establish a "standard testing procedure". Dynamic testing procedures for all highway materials were compiled in one document by the Transportation Research Board's Committee on Strength and Deformation Characterization of Pavement Sections (Barksdale, et al., 1975). A report outlining the affects of sample conditioning and stress level on repeated load test results was published by ASTM (Fredlund, Bergan, and Wong, 1977). In another investigative study, air diffusion through membranes, problems with end restraint, and the existance of undrained creep were found to effect the repeated load test results. Based on the fact that no specific cyclic loading rate was found to have any clear advantages over any other cyclic loading rate, the researchers concluded that the choice of loading rate is dependent on the purpose of the test (Sangrey, Pollard, and Egan, 1978).

Several models have been used to predict a soil's response to repeated loading. Non-linear behavior of soft clay during cyclic loading has been evaluated using a Ramberg-Osgood model (Idriss, Dobry, and Singh, 1978). The elasto-plastic deformation of soil was described by an

anisotropic hardening model (Mroz, Norris, and Zienkiewicz, 1979). Constitutive modeling of cyclic triaxial tests with emphasis on the "critical state" of the soil was also proposed (Carter, Booker, and Wroth, 1980).

Research conducted in Canada on a Leda clay revealed two distinct behavior modes of repeated load specimens. A stress of approximately 54% of the static failure stress of the samples was found to separate the behavior modes. Below the critical stress, permanent deformation continues to increase as the number of stress cycles increases. The resilient modulus stabilizes at a large number of cycles. Above the critical stress, some of the Leda clay samples experienced sudden shear failure (Raymond, Gaskin, and Addo-Abedi, 1979). Similar research into the behavior of sensitive clays during repeated loading was conducted with pore pressure measurement. The effective stress path was found to be a straight line connecting the initial stress and the critical stress (Yudhbir and Rehman, 1977).

The effects of such climatic properties as temperature and moisture on the resilient modulus of fine grained soils were investigated in another study. The resilient modulus of a soil was found to be characterized by soil suction, deviator stress, saturation, and the number of load cycles. An equation utilizing these factors to determine the resilient modulus was proposed. The most

important term in the equation was determined to be the number of load cycles. The equation was found to be more dependent on moisture properties than soil suction as clay content of a sample decreased. The resilient modulus was found to be easier to predict at low clay contents. A fundamental change in the behavior of fine-grained soil samples was found to occur at the "optimum -2%" level. Soil samples wetter than this value exhibit dramatic soil property changes with small increases in water content. The properties of soil samples with water contents below this value are more closely related to soil suction than actual water content (Edris and Lytton, 1977). The influence of soil suction on unsaturated clay samples subjected to repeated loading was being investigated by another researcher. Illitic clay samples were placed under suction at different suction levels using several different extraction methods (Motan, 1981).

The American Society of Testing and Materials (ASTM) presented an instrumented triaxial cell for cyclic loading of clays as a guideline for experimental work (Brown, Austin, and Overy, 1980). The Transportation Research Board also revealed apparatus and results from a preliminary study on repeated load testing (Chaddock, 1982).

Symposia on ground freezing and recent advances in

geotechnical engineering have been concerned with repeated loading. Soils undergoing freeze-thaw cycles were found to be more sensitive to repeated loading (Alkire and Morrison, 1982). Repeated load testing using a new testing apparatus with precision pore pressure measurement permitted the evaluation of effective stress using elasto-plastic models (Akai, Ohnishi, Yamanaka, and Nakagawa, 1981).

The American Association of State Highway and Transportation Officials (AASHTO) established a standard testing method for determining the resilient modulus of subgrade soils in 1982. Recommended apparatus, a variety of compaction procedures, and a detailed testing procedure are given in T274-82 (AASHTO, 1982). The American Society of Testing and Materials (ASTM) has a proposed standard testing method similiar to the AASHTO procedure that has yet to be approved by committee (ASTM, 1985).

With a few noted exceptions, the majority of recent repeated load research has been concerned with earthquake response and wave induced loads on offshore platforms. A simplified method of obtaining material characteristics of drained cohesionless soil has been proposed for use in determining the permanent deformation or rutting that occurs in flexible highway pavements due to repeated loading (Diyaljee and Raymond, 1982). A study evaluating the resilient moduli of Brazilian soils applicable for

pavement design was conducted (Medina and Preussler, 1983). In another study, normally consolidated clay samples were found to act like overconsolidated samples due to pore pressure build-up following repeated load testing. Reductions in shear strength were found to occur in normally consolidated clayey soil specimens that had been subjected to repeated load applications (Togrol and Guler, 1984).

CHAPTER III

COLLECTION AND PREPARATION OF SAMPLES

1. FIELD SAMPLING

Field sampling in the East Tennessee area was conducted at three permanent test sites. The three test sites, shown previously in Figure 4 on page 10, were Rutledge Pike (SR1), Jefferson Co. (I-40), and Cookeville (SR111). These sites were chosen to represent varying thicknesses of the asphaltic concrete layer and different underlying subgrade soils.

The outside traffic lane was blocked so that the subgrade soil underlying the pavement could be sampled. The soil was sampled through a six inch diameter core hole which had been drilled in the outside wheel path of that lane. Photographs of the field set-up are shown in Appendix F as Plates 1 and 2.

A small wheel-mounted coring machine with a six inch diameter bit (Plate 3 in Appendix F) was used to core the asphalt layer. Water was supplied continuously to the inside of the bit for cooling purposes. Since the actual insitu water content of the subgrade soil was desired, the affect of the core water on the water content was assessed. It was observed that the core water accumulated primarily at the surface and in the base material due to

the large permeability of the base material with respect to the asphaltic concrete and subgrade soil. Visual inspection of the subgrade slightly below the surface of that layer indicated little change of water content. The water content of the subgrade was assumed to be unaffected by the core water due to the observations of infiltration and the low permeability of fine grained subgrade soils with respect to the permeability of the base materials.

After the asphalt core and underlying base material were removed, small Shelby samples (1.3 inch diameter by 3 inches high) were hand-driven for field density determinations and for unconfined compression testing. Water content samples were also collected. These samples provided the data on the actual compacted subgrade under the pavement. The thicknesses of the asphaltic concrete and base layers were measured directly from the cored hole. The water content of the subgrade soil was checked periodically throughout the year to determine seasonal variations of the subgrade water content. The effects of varying water content were investigated in the laboratory and are discussed in the section on sample preparation. Table I contains a compilation of field data from each of the three sites. The reported water content values are the average of three samples.

Table I. Field Data Collected from Three Sites

Site	Subgrade Data						
	Thickness (in.) AC	Thickness (in.) Base	Dry Density (lb/cu.ft)	Seasonal Water W	Sp	S	Optimum Water Content (%) (from Standard Proctor test)
Rutledge Pike	6.0	4.0	110.8	23.6	32.4	21.8	37.5
Jefferson County	17.5	5.5	118.5	20.0	22.8	23.3	20.5
Cookeville	10.5	5.0	108.7	18.2	20.5	21.3	33.6

In addition to the samples taken from the core hole, a bag sample was also taken from the shoulder of the road adjacent to the core hole to provide sufficient material for the preparation of remolded laboratory samples. The bag sample was taken at a depth of 12 inches and greater to avoid the humus layers of the soil. Shelby tube samples were also taken at the same depth below the surface to determine the density of the bank material. Water content samples were taken to determine the moisture content of the bank material.

Although the bank sample material was the same as the subgrade soil under the pavement surface at each of the sites, the water contents and densities were found to vary. The dry density of each subgrade sample was greater than that of the corresponding bank sample. This is expected since the design specifications for construction require a minimum dry density for the subgrade underlying a roadbed to be 100 % of the maximum dry density determined from a Standard Proctor test conducted in the laboratory (TNDOT, 1981). The embankment material that was sampled was undisturbed residual soil from a cut section which naturally has a lower density than the same material that has been compacted for use as a subgrade.

The dry density of each subgrade was also found to be greater than the maximum density determined from Standard Proctor tests conducted in the laboratory. This

is due to the fact that 100% of the maximum density determined from a Standard Proctor test is a minimum requirement for Department of Transportation specifications (TNDOT, 1981). There are also two notable factors that contribute to higher field densities. First, the subgrade soil samples were slightly densified due to friction along the walls of the Shelby tubes during sampling and extrusion. Although a lubricant was applied to the surface of the Shelby tube in order to minimize these effects, densification was observed both in the field during the sampling process and in the laboratory where samples were extruded. Second, the Shelby samples from the Rutledge Pike and Cookeville sites contained cherty fragments typical of residual soils. The cherty fragments have a much greater density than the fine-grained soil. The Standard Proctor test was conducted on "processed soil" which had the larger cherty fragments removed prior to compaction by passing the soil through a No.4 sieve. Therefore, the calculated density of "processed soil" would be less than that of the soil with cherty fragments.

After the subgrade material was processed, standard laboratory tests were run to determine the index properties of the soil samples. (Results from these standard laboratory tests are given later in Tables III and IV which appear in Chapter V.) A listing of the

laboratory tests that were performed and the corresponding ASTM and AASHTO standard specifications that served as guidelines for these tests is given in Table II.

Table II. Laboratory Tests and Specifications.

Laboratory Test	Specification	
	ASTM	AASHTO
Atterberg Limits:		
Liquid Limit	D423	T89
Plastic Limit	D424	T90
Grain Size Analyses:		
Sieve Analysis	D1140	T88
Hydrometer Analysis	D422	T88
Specific Gravity of Solids	C854	T100
Proctor Test:		
Maximum Dry Density	D698	T99
Optimum Moisture Content	D698	T99
Unconfined Compression Test	D2166	T208

The purpose of running the standard laboratory tests was to obtain index properties and standard data on the soils for correlation with resilient modulus values. A good correlation between index properties and resilient moduli had been found for coarse-grained materials in a previous study of Illinois soils. However, the same study

did not reveal a correlation between index properties of glacially deposited fine-grained soils and resilient moduli (Robnett and Thompson, 1973). Although the samples in the Tennessee study were fine-grained, a correlation between index properties and moduli values was attempted because the residual Tennessee soils are characteristically different from Illinois soils. As described later in Chapter VI, no simple correlations were found between the index properties and the resilient modulus data.

2. LABORATORY SAMPLE PREPARATION

Laboratory samples were prepared for use in repeated load tests. The decisions concerning choice of water content, sample size, compaction method, testing equipment and procedure are discussed in the following paragraphs.

Tennessee Department of Transportation (TNDOT) Standard Specification 207.04 requires the subgrade to be placed at optimum water content as determined from a compaction test performed in accordance with ASTM D698 (TNDOT, 1981). A previous study of placement moisture content along two sections of interstate in Illinois revealed that significant volumes of the subgrade had been placed wet of optimum. Characterization of the subgrade moisture content utilizing a normal distribution indicated

that approximately 43% of the subgrade was placed wet of optimum (Robnett and Thompson, 1973).

In this investigation, laboratory samples were prepared at two water contents to reflect actual field compaction conditions and to evaluate the effects of water content on the resilient modulus. "Optimum water content", the water content that corresponds to the maximum dry density determined from a Standard Proctor compaction test was a logical choice based on TNDOT specifications. "Optimum + 2%" was chosen to determine the effect of compaction at wetter than optimum conditions on the resilient modulus.

Samples 2 inches in diameter and between 3.25 and 4.25 inches in height were used. Sample size was based on the size limitation of the compaction and testing equipment, ease of handling and by the small amount of material required. The sample size was sufficiently greater than the maximum particle size of the fine-grained soils to be tested. In addition, results from the study on Tennessee soil behavior were easy to correlate with a previous study which had utilized the same sample size (Robnett and Thompson, 1973).

All laboratory samples were compacted in membranes by a kneading compaction method. The decision to compact samples in membranes was made so that existing equipment could be utilized. The compaction mold was actually a two

part membrane stretcher which had been fitted with a collar and base plate to hold the mold in place during compaction. Compaction within the membrane also reduced moisture loss and preparation time.

Field compaction, by sheep's foot roller, was simulated in the laboratory by a kneading type compaction which induces shearing strains and produces a dispersed soil structure (Seed, Chan, and Lee, 1962). A kneading compactor (Plate 4 in Appendix F) which was designed for the preparation of four inch diameter and six inch diameter soil and asphalt specimens, was modified for two inch diameter sample preparation. The compactor was fitted with a sixty degree wedge shaped compaction foot as shown in Figure 5.

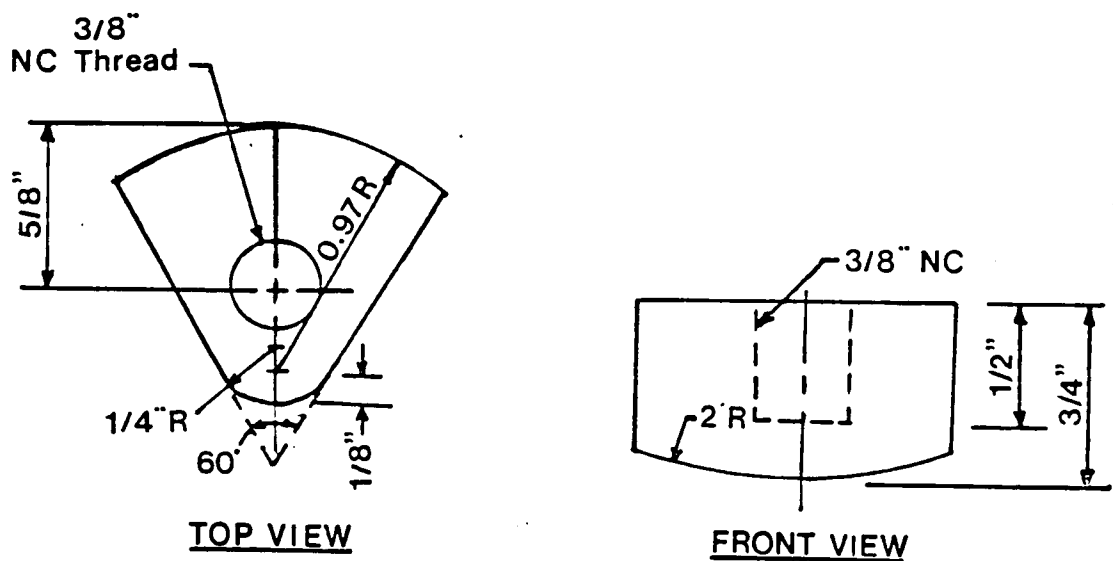


Figure 5. Schematic Diagram of the Kneading Compaction Foot.

The compaction foot was based on a design from the University of Illinois (Robnett and Thompson, 1973). The kneading compactor uses air pressure to apply the load to the surface. The compaction foot provides a pressure between 70 and 90 psi to the soil surface for one second during each stroke. The pressure was adjusted by trial and error until the desired sample density was achieved. A turntable base was adjusted to rotate the sample forty-five degrees after each stroke of the compaction foot. The fifteen degree overlap along with the orientation of the compaction foot in the mold provided the geometry required for kneading action to take place.

A compaction procedure developed at the University of Illinois was adopted for sample preparation. Samples were compacted in four equal layers with sixteen strokes of the compaction foot per layer except for the top layer which received thirty-two strokes to obtain a more uniform density of all four layers. The orientation of the compaction foot for the first stroke of layers two, three, and four was offset ninety degrees from the previous layer. This helped provide uniform compaction. The surface of each compacted layer was scarified before the next layer of soil was added to insure bonding between layers (Robnett and Thompson, 1973).

Each sample was then trimmed, weighed, and length and diameter measurements were made. A water content check was

then made from the soil trimmings to determine the actual water content of each prepared sample. Wet and dry densities were determined based on sample mass and volume.

TNDOT Standard Specification 207.04 requires the compacted subgrade to be not less than 100% of the maximum density as determined from a standard proctor test (TNDOT, 1981). Resilient modulus samples that were not within 1% of this specified density were discarded. Samples that met the density and height requirements were put in plastic bags and then placed in a moist curing room. The curing room maintains 100% moisture conditions and a constant temperature of 77 degrees. Samples were left in the moist curing room for at least a week before testing.

Samples compacted by methods that induce shearing strains such as kneading type compaction may show strength gains from the time of compaction. The term "thixotropy" is used to describe a strength increase with time at a constant soil composition. In a pioneering research study by Seed, Chan, and Lee, samples tested immediately after compaction had lower resilient moduli than samples that were allowed to rest for a few days. Samples that were allowed to cure for several months before testing had resilient moduli approximately equal to samples that were cured only one week. A one week curing time before testing of repeated load test specimens minimized the effect that

thixotropy had on the determination of the resilient modulus (Seed, Chan, and Lee, 1962).

Field data from the three East Tennessee sites revealed seasonal changes in the water content of the subgrade as indicated in Table I on page 24. Spring thaw and heavy spring rains generally increase the water content and degree of saturation of the subgrade. Due to lack of rainfall in the East Tennessee area, the subgrade soils did not reach water contents significantly above optimum during the course of this study. Instead, water contents below optimum were commonly observed.

To simulate the "drier than optimum" conditions that were encountered at the three East Tennessee test sites, samples that had been prepared at optimum density and moisture content were "dried out" until the desired water content was reached. Samples were prepared at optimum water content and then subsequently dried to insure that the soil structure was the same as the actual subgrade, because repeated load stress-strain behavior of a soil sample can be dependent upon its structure. Compaction at water contents significantly below optimum creates a random or flocculated soil structure rather than an oriented or dispersed soil structure which results from compaction at or around optimum water content.

The drying procedure used in this study was determined from a trial and error process. After the usual

one week curing period, the membranes were removed from the samples. The samples were allowed to air dry in the laboratory for several hours. Water contents of some samples were checked to determine the amount of air drying time that was needed to achieve the desired water content. Actual drying times varied from three hours to twenty-four hours. After drying, the samples were wrapped in a moist paper towel, covered with a plastic bag, and then placed back in the moist cure room. The moisture loss of the sample was determined to be greatest along the surface. The wet paper towel was used to restore moisture to the surface and to achieve a more uniform moisture content of the sample. The samples were stored in the moist curing room for another week before testing to allow the water content to equalize throughout the sample.

CHAPTER IV

REPEATED LOAD TESTING

Repeated load testing simulates the loading and unloading that occurs in a subgrade when traffic wheel loads are applied to the road surface. Special laboratory equipment is needed to apply the load and monitor the results. The loading apparatus and testing procedure used in this study are outlined in the following sections. The procedures for data acquisition and reduction are also discussed. Appendix F contains several plates of the repeated load testing equipment.

1. APPARATUS FOR TESTING

A Research Engineering Model RE-SS-2400 load frame was utilized for cyclic triaxial testing. A pneumatic actuator mounted on the top of the loading frame supplied the cyclic load to the sample. A schematic diagram of the actuator is shown in Figure 6. Steady state and cyclic air pressure inputs were provided by an electropneumatic cyclic loader. A modified Exact Electronics waveform generator electronically controlled the shape, size, and frequency of the applied load pulse. The "Amplitude" and "DC Offset" dials allowed for variations in pulse size and static load left on the sample. A load versus time plot

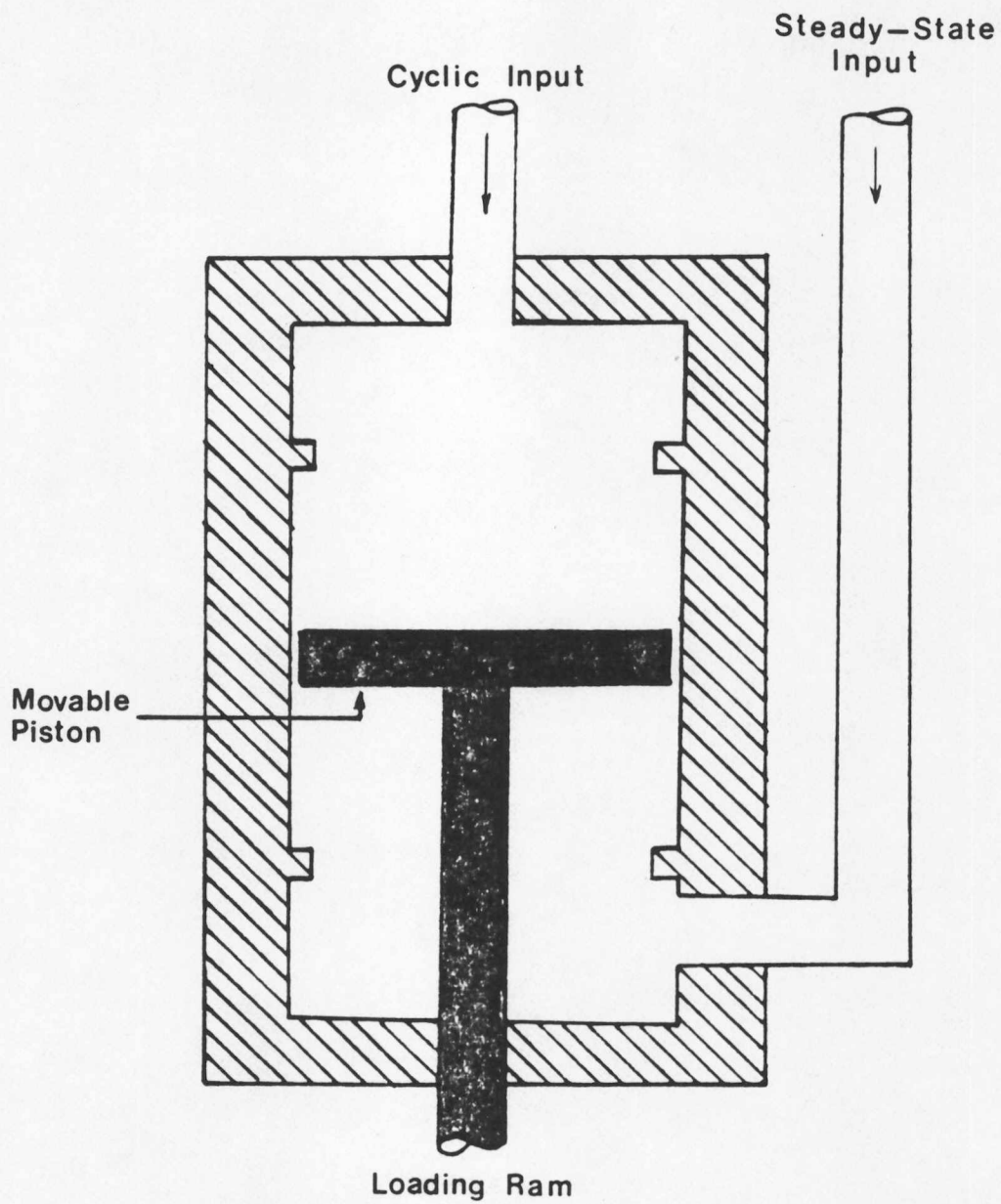


Figure 6. Schematic Diagram of the Pneumatic Actuator.

using actual data for two different loads is shown in Figure 7.

An Interface load cell (rated at + 1000 lb.) was mounted in line with the loading ram. A spring-loaded Linear Variable Differential Transducer (LVDT) with a range of ± 1 inch was clamped to the loading shaft to record axial deformation. Due to the geometry of the triaxial cell, the LVDT was placed two inches to the right of the loading shaft. The effect of this offset was not thought to be significant due to the rigidity of the mounting bracket and the slow loading rate.

2. DATA ACQUISITION AND REDUCTION

Voltages from the load cell and LVDT corresponding to the applied load and vertical displacement of the sample were displayed on Schaevitz digital transducer readouts. The signal from the load cell was boosted using a D.C. Bridge amplifier before it reached the digital display (Clevite Corporation, 1969). An explanation of the calibration procedure of the load cell/amplifier and LVDT is given in Appendix B.

The voltages from the load cell and LVDT were monitored by a data acquisition system (Lab Data Manager), developed by Interactive Microware, in conjunction with an Apple II+ 48K computer (Warne, 1982). The data acquisition

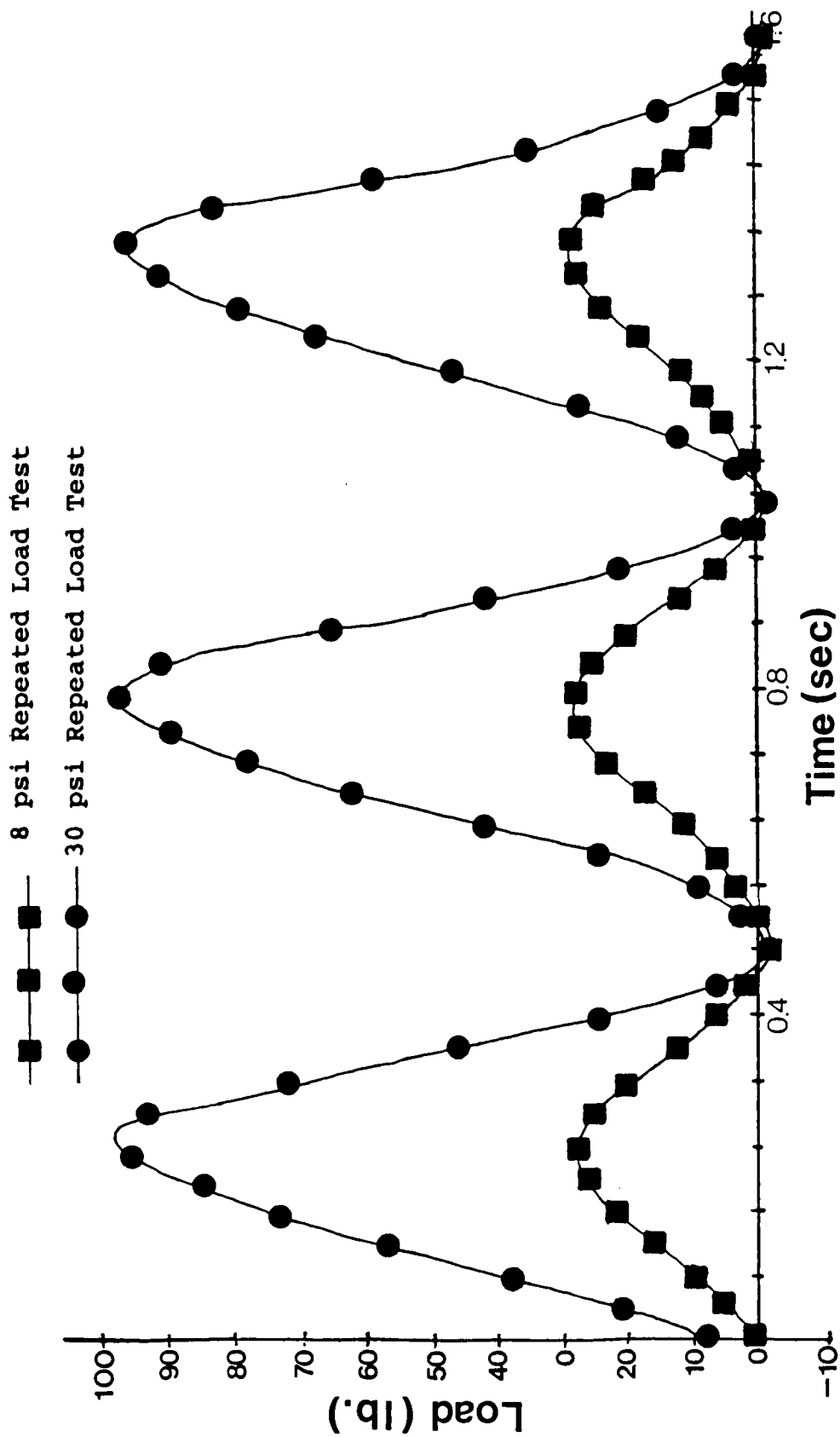


Figure 7. Cyclic Load versus Time Plot.

system collected twenty-five samples per second throughout each ten second testing period. A brief explanation of Lab Data Manager and the hardware used in collecting the load and displacement data is given in Appendix C.

The collected load and displacement data were stored as separate temporary files on a disk. A computer program called "FILE" took these two separate load and displacement files and combined them into one permanent file. The file was identified by sample type, sample number, deviator stress level and the testing data. Another computer program called "CONVERT" was created to reduce the collected data into stress and strain data. After "CONVERT" reduced the voltage data, a search of the data was made to determine the maximum recorded stress and strain values. The value of the maximum stress divided by the maximum strain is the reported resilient modulus. The use of these maximum values for the determination of the resilient modulus assumes that the intermittent computer sampling actually took place at the instant when the stress and strain were at their maximum level. This assumption is justified due to the relatively high sampling rate of twenty-five samples per second, the loading frequency of two Hertz and the ten second duration of the test. The deviation between the actual resilient modulus and computer determined resilient modulus is depicted schematically in Figure 8. With approximately

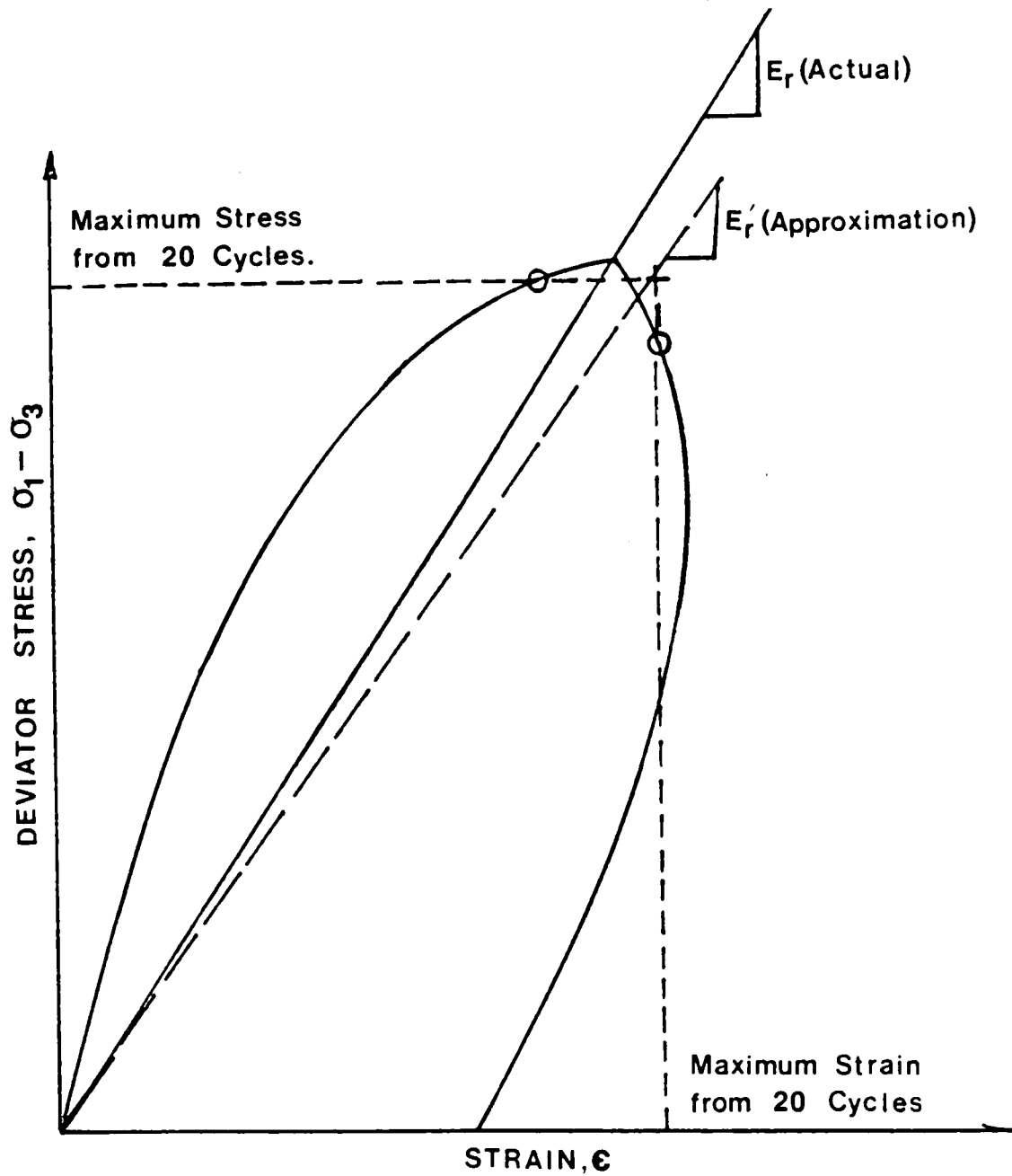


Figure 8. Schematic Diagram Depicting the Computer Determined Resilient Modulus and the Actual Resilient Modulus.

twenty values of maximum stress and strain, this deviation is expected to be small and the procedure should yield a good approximation of the resilient modulus. Appendix A contains a listing of the programs "Set", "File", and "Convert". Sample output from the "Convert" program is also contained in Appendix A.

3. TESTING PROCEDURE

A sample, encased in a membrane, was taken from the moist cure room and placed in the loading frame. Porous stones were placed on the ends of the sample. The alignment of the sample with respect to the loading base and cap was checked. The membrane was then slipped over the stones and onto the loading base and cap.

The sample was conditioned by applying 100 cycles at the specified ten testing stresses (2, 4, 6, 8, 10, 15, 20, 25, 30, and 35 psi). The function generator settings of "Amplitude" and "DC Offset" were recorded at each of these stress settings. The height of the sample, after conditioning, was then recorded for later use in strain calculations. The function generator dials were adjusted for the 2 psi setting and the computer was prompted. The starting signal was given to the computer. A toggle switch on the function generator was turned on to begin cycling. The computer monitored the testing for 10 seconds and recorded the load and displacement data on a disk file.

The cyclic load was then turned off to prevent unnecessary stress cycle application while the computer was storing the data. The function generator dials were then adjusted for the next stress level (4 psi).

This procedure was repeated for each of the stress levels. After the 35 psi test, the sample was removed from the test apparatus and a final height of the specimen was recorded. A water content check was also made from the soil sample.

4. JUSTIFICATION OF TESTING PROCEDURE

A standard method of testing for the resilient modulus of subgrade soils was established in 1982 by the American Association of State Highway and Transportation Officials (AASHTO, 1982). The American Society for Testing and Materials (ASTM) has a tentative method that has not yet been approved, but the tentative procedure is similar to the AASHTO procedure (ASTM, 1986). The Transportation Research Board (TRB) has described test procedures for determining dynamic stress-strain properties of highway materials (Barksdale, et al., 1975).

The AASHTO procedure sets loose guidelines allowing for a variety of compaction methods and sample sizes. The testing procedure utilizes three different confining stresses while testing over a range of deviator stresses. A series of tests are first performed at the 6 psi

confining stress level and then again with a confining stress of 3 psi. Finally, the series of tests are run utilizing no confining pressure. Samples are subjected to deviator stresses of 2, 4, 6, 8, and 10 psi at each of the confining stresses. The AASHTO procedure incorporates the simplified TRB method as part of the standard testing procedure with the simplification being the elimination of testing at the 6 and 3 psi confining stress levels. The AASHTO procedure and TRB method are based in part on the early testing procedures developed at the University of Illinois (Robnett and Thompson, 1973). The procedure used in this study was adapted primarily from the Transportation Research Board's (TRB) suggested simplified method of resilient modulus determination (Barksdale, et al., 1975). Some modifications were made to the standard procedures based on the limitations of the testing equipment used in the Tennessee study. Deviations from the suggested procedures are discussed in the following paragraphs.

Using linear elastic theory or some other method such as finite element analysis, the magnitude of the confining pressure existing in the upper regions of the subgrade has been found to be less than 5 psi (Robnett and Thompson, 1973). In previous studies, no significant differences in resilient moduli were found for tests with low confining

pressure and tests with no confining pressure (Seed, Chan, and Lee, 1962). The testing procedure is significantly simplified if no confining pressure is used, consequently this approach was adopted. With the application of no cell pressure, the deviator stress equals the axial stress.

A sinusoidal load pulse was modified to simulate traffic loading of the subgrade. A sine wave is composed of both positive and negative parts which would cause compressive and tensile loading of the sample. Since the actual subgrade does not experience tensile loading, modifications must be made to the load pulse. This can be accomplished by supplying a zero output throughout the duration of the negative portion of the sine wave. Alternately, only non-negative values can be supplied by offsetting the entire sine wave to a position above zero as was done in this study (refer to Figure 7, page 38). Recommended pulse durations vary between 0.06 seconds and 2.0 seconds. The longer pulse durations correspond to slower movement of traffic on the road surface. In the University of Illinois study, a 0.06 second pulse duration was chosen and the rate of application was twenty pulses per minute (Robnett and Thompson, 1973). Each pulse was generated separately with a significant pause between pulses which allowed the sample to rest. The function generator used in the University of Tennessee study supplied a continuous string of pulses. Since the test

sample would not be allowed to rest between pulses, a relatively slow frequency of 2 Hertz was chosen. A range of frequencies are recommended by the TRB depending on traffic speed. The 0.06 second load pulse duration is at the high end of the acceptable range corresponding to high traffic speeds, while the 2 Hertz loading rate adopted in this study corresponds to slower traffic movement.

In the study conducted at the University of Illinois, conditioning consisted of 1000 applications of a predetermined axial stress. The magnitude of the conditioning axial stress was equal to the vertical subgrade stress that occurs due to a nine kip dual wheel load placed on a typical Class III flexible pavement (Robnett and Thompson, 1973). The conditioning procedure recommended by the TRB consists of 200 repetitions at deviator stresses equal to 1, 3, 5, 7, and 10 psi for a total of 1000 repetitions (Barksdale, et al., 1975). The TRB procedure suggests conditioning samples at deviator stress levels that correlated with actual testing stress levels. The conditioning stresses are limited to a maximum of 10 psi, although during the testing phase samples are subjected to higher deviator stresses (Barksdale, et al., 1975). The AASHTO procedure recommends conditioning samples at each deviator stress level that is used in the laboratory testing procedure (AASHTO, 1982). The conditioning procedure implemented in the University of

Tennessee study utilized conditioning stresses equal to the deviator stress levels used for actual testing. The conditioning process covered the full range of test stresses from 2 psi to 35 psi. The conditioning procedure consisted of 1000 total stress applications with 100 applications at each of the ten deviator stress levels.

The resilient modulus of fine-grained soil decreases as deviator stress is increased. Sufficient testing was needed to characterize this "stress softening" behavior. The TRB procedure suggested testing a single sample at increasing deviator stress levels. Starting with a 3 psi test and then incrementally increasing the deviator stress by 3 to 5 psi for subsequent tests (Barksdale, et al., 1975). The AASHTO procedure suggests testing at deviator stresses of 2, 4, 6, 8, and 10 psi (AASHTO, 1982). In the Tennessee study, each sample was tested at ten deviator stress levels. Testing was conducted at 2, 4, 6, 8, 10, 15, 20, 25, 30, and 35 psi. Testing was stopped during the procedure if significant permanent deformation occurred.

CHAPTER V

LABORATORY TEST RESULTS

The laboratory testing segment of the project included a wide range of standard tests to determine the soil properties of the three subgrade samples and repeated load tests on a variety of laboratory compacted samples. The results of these tests are given in the following sections.

1. STANDARD TESTS

Standard laboratory tests were performed to determine the soil properties of the three subgrade materials and to form a data base for correlation of soil properties with repeated load test data. The results of some of the standard laboratory tests that were performed on the three subgrade materials are listed in Table III. The similarities between the Rutledge Pike subgrade material and the Cookeville subgrade material soil properties are clearly visible. The Jefferson County subgrade soil had lower Atterburg limit values and a lower optimum moisture content. An appreciable amount of the Jefferson County material passed the No. 200 sieve. The maximum dry density and ultimate strength of the Jefferson County subgrade material were also significantly greater than the same

Table III. Standard Laboratory Test Results.

Sample Site Location	% Passing # 200 Sieve	Atterburg Limits		S.G. of Solids	Proctor Test Data		Unconfined Compression	
		LL	PL		Max. Dry Density (lb/cuft)	Opt. M. C. (%)	Max. Stress	Test Data (psi) Ult. Stress
Rutledge Pike	28.7	68.5	39.2	2.71	80.2	37.5	55.0	23.5
Jefferson County	45.9	37.5	18.3	2.69	104.5	20.5	64.5	48.9
Cookeville	32.9	63.6	36.5	2.78	86.1	33.6	50.8	25.2

properties for the Rutledge Pike and Cookeville subgrade materials.

Each subgrade material was classified according to three classification systems : (1) the Soil Conservation Service (SCS), (2) the Unified Soil Classification System (USCS), and (3) the American Association of State Highway and Transportation Officials (AASHTO). The SCS system was designed primarily for agricultural use and is based on physiographic regions of the state (Springer and Elder, 1980). The USCS and AASHTO classification systems, used predominately by engineers, are based on grain size distribution and Atterberg Limits (Holtz and Kovacs, 1981). Table IV contains soil classification data on the subgrade materials from the three field sites.

Table IV. Soil Classification.

Location	Classification Systems		
	SCS	USCS	AASHTO
Rutledge Pike	J-11	SM	A-2-7
Jefferson County	J-21	SC-CL	A-6
Cookeville	D-51	SM	A-2-7

2. REPEATED LOAD TESTS

Repeated load tests were performed on laboratory compacted fine-grained soil samples from each of the field sites. Each sample was tested over a range of deviator stresses. The results from several of these tests are shown in plots of the resilient modulus (E_r) versus deviator stress (σ_d) as indicated in Figures 9, 10, and 11. Information concerning the water content and degree of saturation of each sample is given on the graphs. Appendix D contains " E_r vs. σ_d " graphs for all of the tested samples.

The actual deviator stress levels used to test the samples varied somewhat from the proposed stress levels established in the testing procedure. The variance resulted from manual adjustments made on the signal conditioner dials to achieve the desired stress level as seen on the digital output. The response digital meter is not sufficient to monitor the entire stress variation being applied to the sample. Since the dials are set by visual observation, the actual stress being applied is somewhat different than the deviator stress outlined in the testing procedure. The differences between the proposed stress levels and the actual testing stress levels are of little consequence since the variations of the resilient modulus resulting from deviator stress

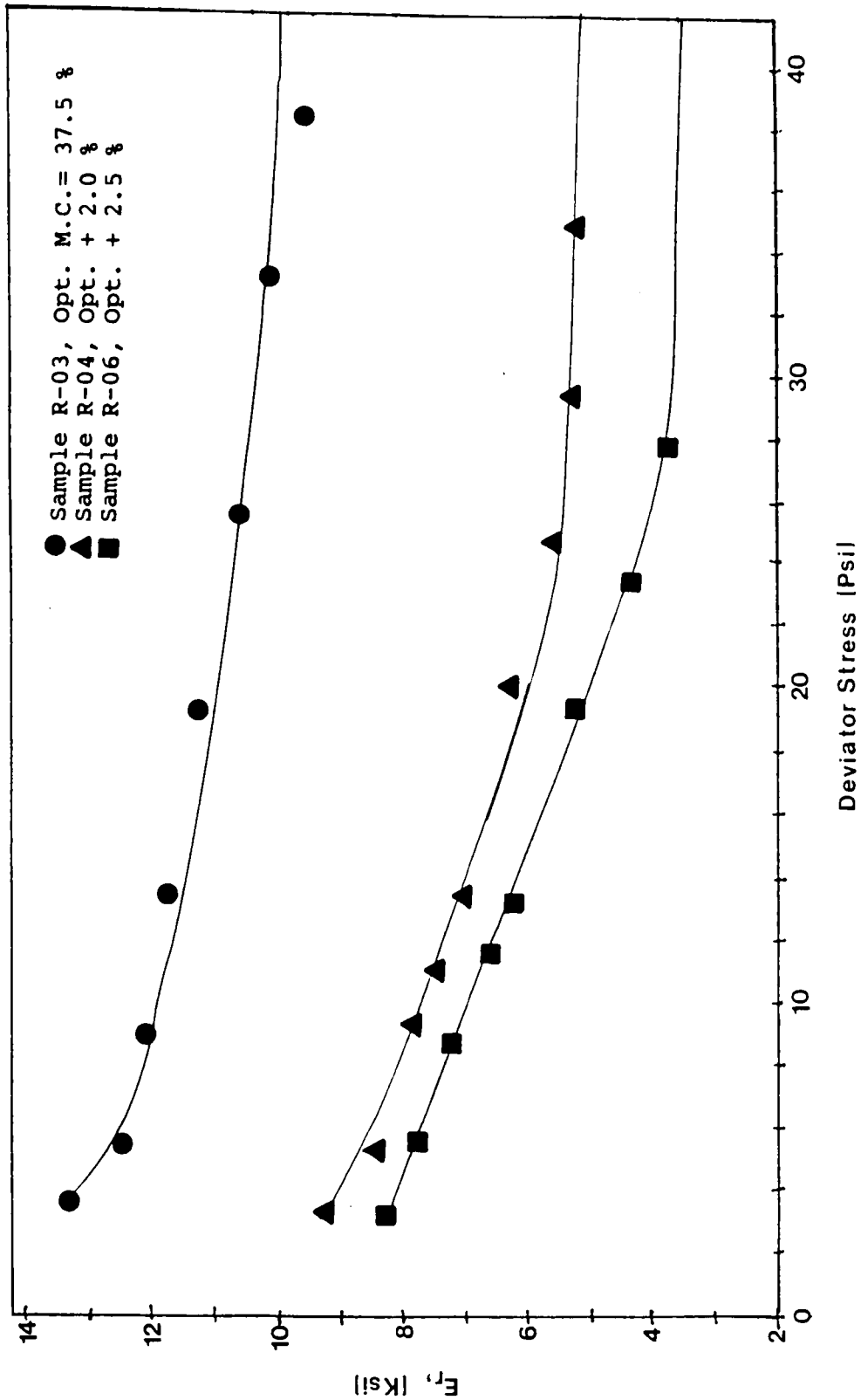


Figure 9. Plot of Resilient Modulus (E_r) versus Deviator Stress for Several Rutledge Pike Samples.

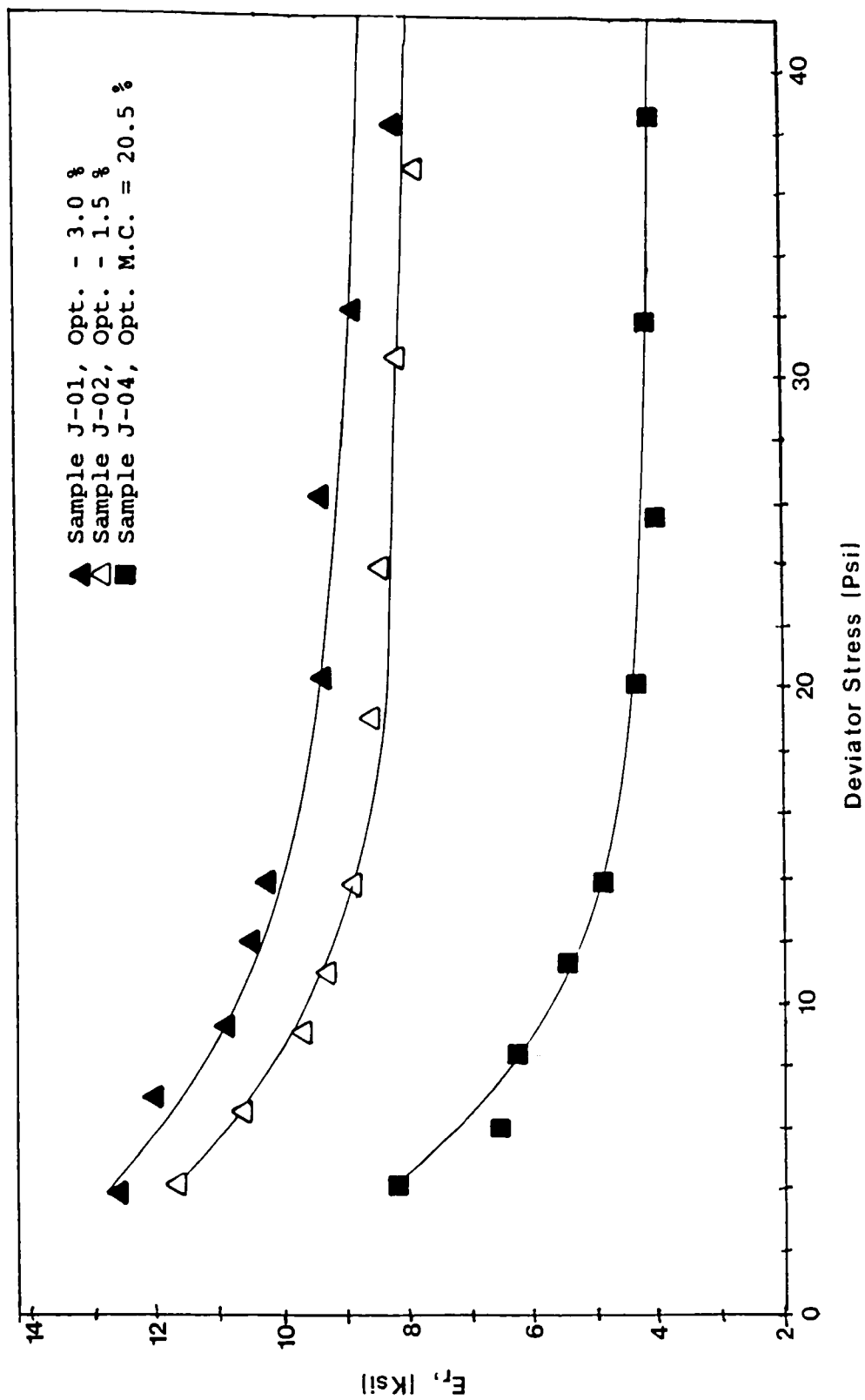


Figure 10. Plot of Resilient Modulus (E_r) versus Deviator Stress for Several Jefferson County Samples.

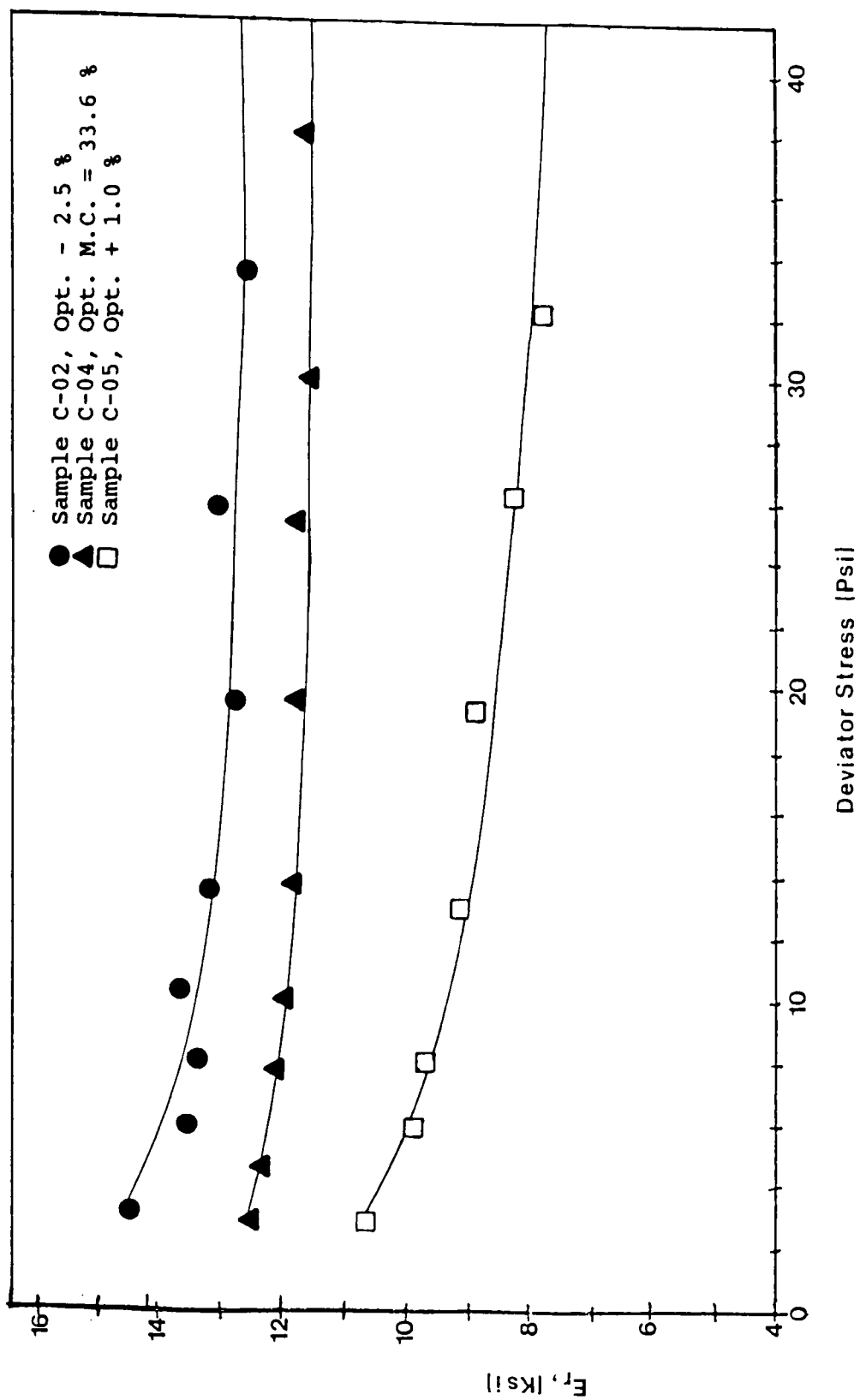


Figure 11. Plot of Resilient Modulus (E_r) versus Deviator Stress for Several Cookeville Samples.

application are desired over a range of stresses.

Several observations can be made from the plots of the resilient modulus versus deviator stress:

1. The resilient modulus decreases as the deviator stress is increased. The more pronounced changes occur at the lower deviator stress values (less than 15 psi). In many samples, the resilient modulus is constant for deviator stresses above 20 psi.

2. Samples with water contents above "optimum", as determined from a Standard Proctor test, have significantly lower resilient modulus values than samples with optimum water contents, while samples with water contents below "optimum" had higher resilient moduli.

3. Although the dry density of test samples was limited to a small range, a trend was noticed. The samples with the greater dry densities for each type of subgrade tested exhibit higher resilient modulus values. (Referring to Figure 22 in Appendix D, sample C-05 has a dry density of + 0.5% above that of sample C-07. Although the water contents and degrees of saturation of the samples are quite similar, the resilient moduli values for sample C-05 are higher at every deviator stress level.)

4. Material characteristics also effect the resilient modulus. The similarities between the Rutledge Pike and Cookeville subgrade materials are reflected in similar resilient modulus values. The Jefferson County subgrade material has significantly different soil index properties (Table III, page 48). The curve of resilient moduli versus deviator stress of the Jefferson County subgrade material at optimum moisture content plots significantly below the Cookeville and Rutledge Pike curves (more than 4 ksi) as shown in Figure 12.

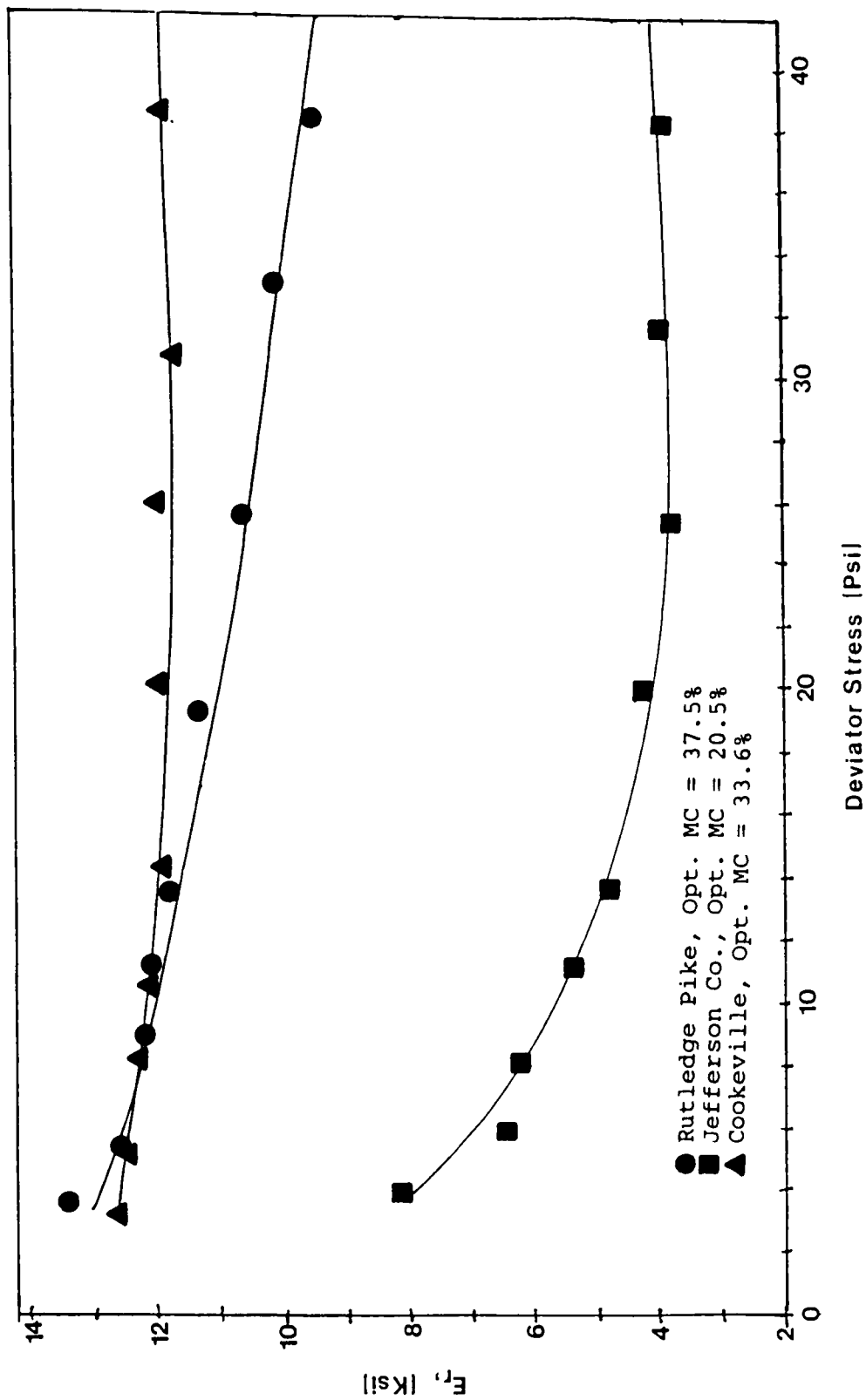


Figure 12. Plot of Resilient Modulus (E_r) versus Deviator Stress for Samples at Optimum Water Content.

CHAPTER VI

CHARACTERIZATION OF RESULTS

1. METHODS OF EVALUATION

A bi-linear relationship has been used to characterize the behavior of subgrade soils in Illinois (Thompson and Robnett, 1976). Although this bi-linear relationship resulted in a simplified relationship for Illinois soils, it did not sufficiently characterize the behavior of subgrade soils in the East Tennessee area as shown in Figure 13. In addition, the bi-linear representation may cause numerical convergence problems during computer modeling at or near the breakpoint.

Several attempts were made to characterize the resilient behavior of the subgrade samples. Although trends in the results indicated that water content, degree of saturation, and dry density affected the resilient modulus, no simple relationships between the resilient moduli of test samples and these three properties could be established. Plots of the resilient modulus versus the log of the applied stress did not reveal a linear relationship. Normalization of (E_r) at a given water content with respect to (E_r) at optimum water content did not reveal a useful relationship. Hyperbolic modeling

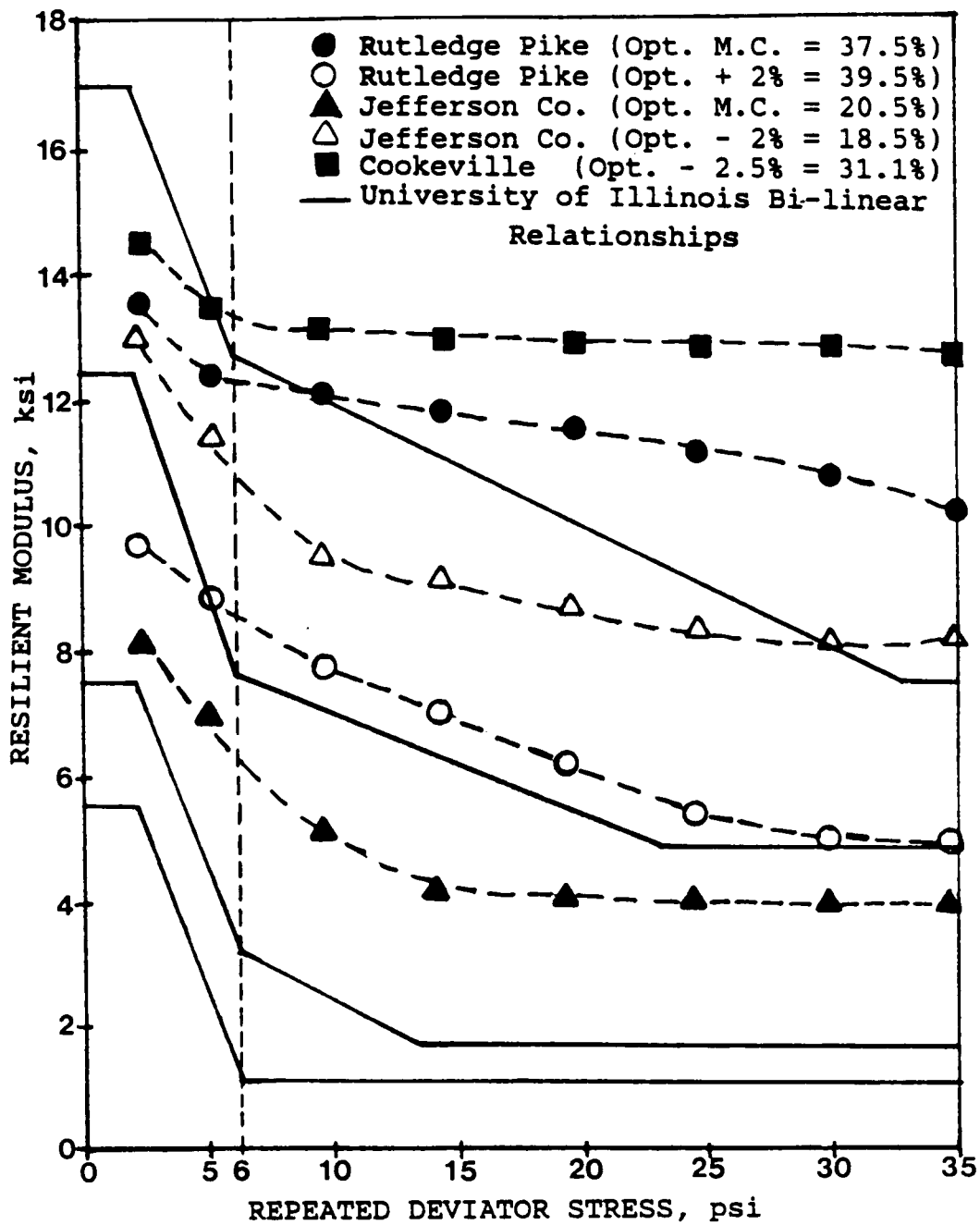


Figure 13. Diagram of the Bi-linear Relationship and Some Typical Tennessee Data.

failed to accurately predict the repeated load behavior of the subgrade soil samples.

A least squares curve fit computer program called "Curvfit" was used to characterize the repeated load behavior of each tested sample (Bennett and Drumm, 1986). Exponential equations with different powers of magnitude and polynomial equations of several orders were evaluated. A second order polynomial equation was chosen as a compromise between accurate prediction and simplicity. The equation follows the form :

$$E_r = A * (\sigma_d)^2 + B * (\sigma_d) + E_o$$

where: σ_d = Deviator Stress, psi

E_r = Resilient Modulus, psi

A = a constant

B = a constant

E_o = a constant, the "initial modulus", psi.

The second order polynomial equation is considered valid over a specified range of stresses (0 - 40 psi). The constants "A", "B", and "Eo" determined for each tested sample from each test site by the least squares computer program are listed in Tables V, VI, and VII. The standard error, determined for each sample, is also given in the tables. Standard error is defined in Appendix E. The

Table V. Curvefit Data for the Rutledge Pike Samples

Sample No.	% Sat.	Curvefit Parameters			Standard Error, psi
		"A"	"B"	"Eo", psi	
R-01	91.8	1.21	-162.9	14,139	± 421.1
R-02	91.9	1.47	- 98.4	12,891	± 227.7
R-03	89.8	0.59	-117.6	13,590	± 424.8
R-04	94.0	3.04	-253.1	9,860	± 147.8
R-05	96.1	2.67	-267.0	10,175	± 96.4
R-06	96.9	3.19	-284.0	9,124	± 100.7
R-07	83.3	7.28	-379.4	14,153	± 113.4
R-08	96.4	5.08	-261.7	12,048	± 389.0
R-09	--	2.11	-163.4	12,130	± 108.2

Table VI. Curvefit Data for the Jefferson Co. Samples

Sample No.	% Sat.	Curvefit Parameters			Standard Error, psi
		"A"	"B"	"Eo", psi	
J-01	79.9	3.32	-255.1	13,174	± 311.7
J-02	79.7	4.22	-272.5	12,117	± 376.8
J-03	78.7	4.80	-280.3	12,384	± 577.1
J-04	87.8	5.89	-350.4	8,600	± 314.9
J-05	87.1	6.93	-479.1	12,737	± 81.9
J-06	89.2	4.38	-258.4	7,266	± 244.1
J-07	87.7	6.04	-325.5	9,461	± 101.8
J-08	89.5	6.03	-312.2	7,404	± 600.5
J-09	83.9	3.35	-248.1	12,240	± 333.4
J-10	92.4	3.79	-246.2	8,162	± 276.5

Table VII. Curvefit Data for the Cookeville Samples

Sample No.	% Sat.	Curvefit Parameters			Standard Error, psi
		"A"	"B"	"Eo", psi	
C-01	90.1	0.19	- 58.3	13,858	± 247.6
C-02	87.8	2.18	-132.8	14,785	± 496.0
C-03	87.4	1.32	- 94.4	14,399	± 261.0
C-04	90.4	1.13	- 66.1	12,722	± 151.2
C-05	95.2	3.07	-195.9	11,110	± 134.2
C-06	94.7	9.63	-483.3	13,895	± 94.9
C-07	94.8	2.50	-213.5	10,988	± 44.7

modulus value determined from the polynomial equation had an average difference of plus or minus the standard error from the actual laboratory determined modulus value. These modulus values using "Curvfit" are within 5% of the laboratory determined values.

A graph of each constant versus degree of saturation was made. The degree of saturation has been proposed as a way of incorporating both water content and dry density in the form of one parameter. Plots of "A" and "B" versus Degree of Saturation, depicted in Figures 14 and 15 respectively, show a scattering of data. Due to the absence of apparent trends, an average value of "A" and an average value of "B" were selected for use with all the soil samples at any given degree of saturation. As shown in Figure 16, the plot of "Eo versus Degree of Saturation" revealed a significant trend.

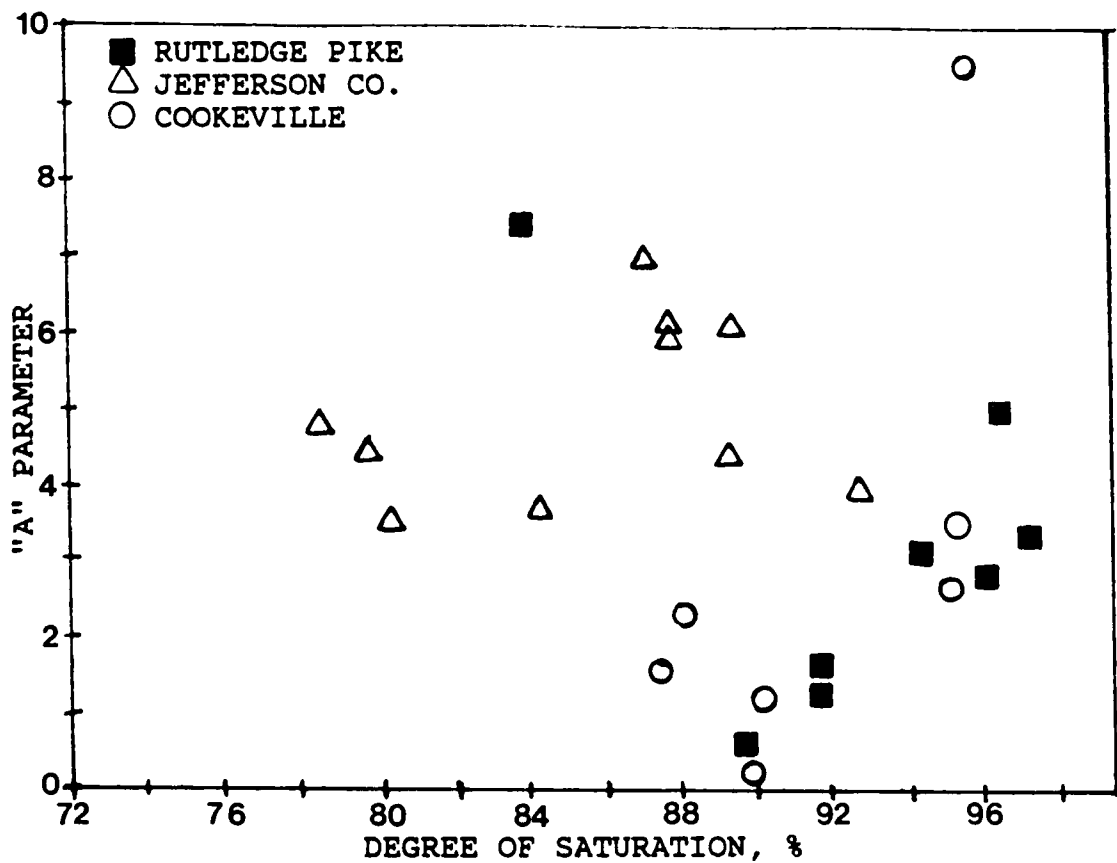


Figure 14. Scatter Plot of "A" versus Degree of Saturation.

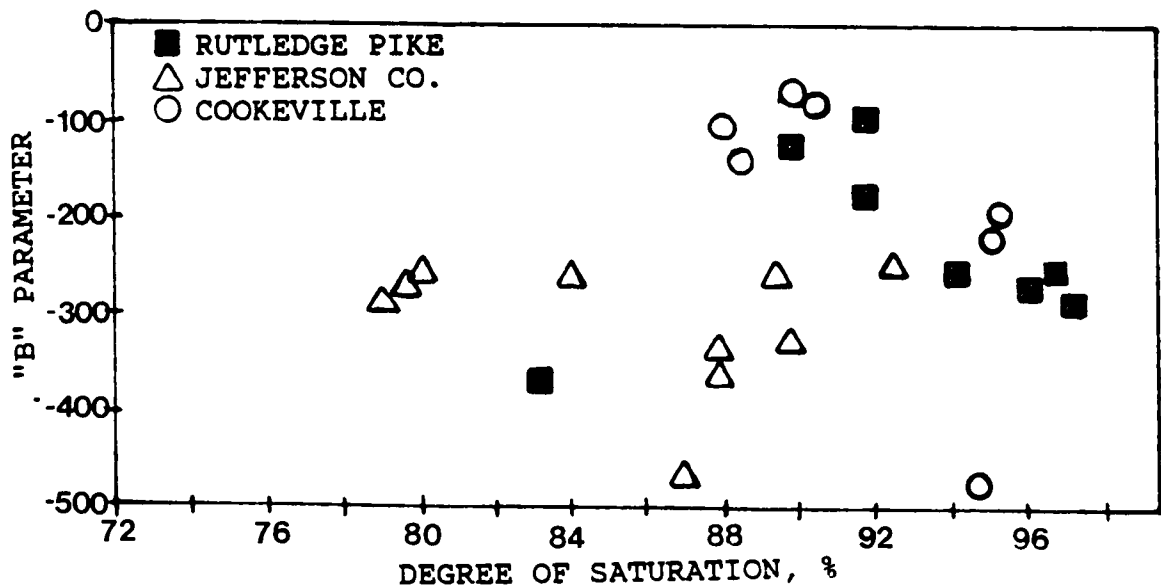


Figure 15. Scatter Plot of "B" versus Degree of Saturation.

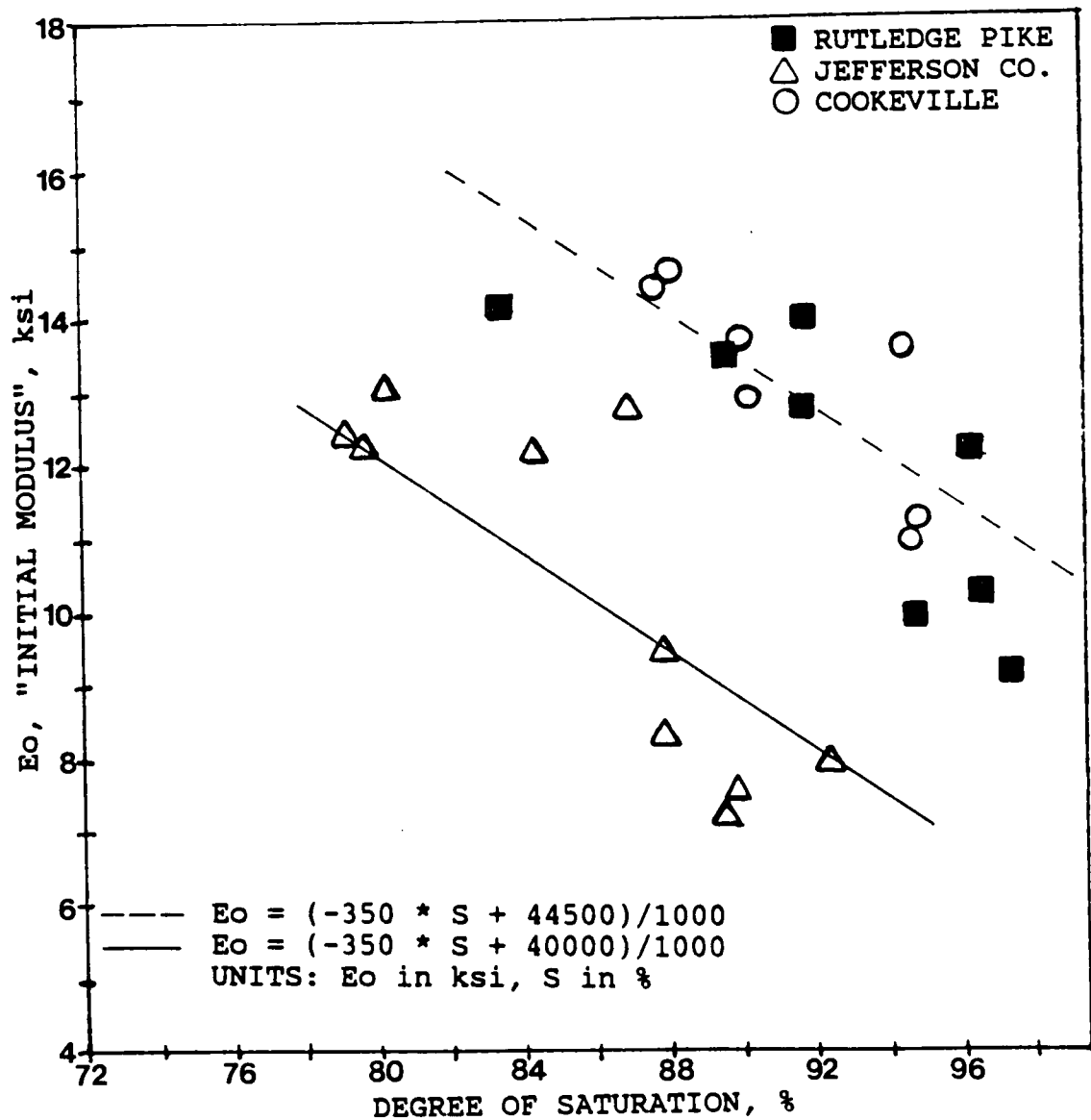


Figure 16. Plot of "Eo" versus Degree of Saturation.

The standard characteristics and index properties of the Rutledge Pike and Cookeville soils, determined in the laboratory, were very similar. The Jefferson County material had significantly different characteristics (Table III page 48). The orientation of the data depicted in Figure 16 suggests a correlation between some material properties (plastic limit, liquid limit, proportion of silt or clay particles, etc.) and the resilient modulus.

In an attempt to characterize the data depicted on the "Eo versus Degree of Saturation" graph, the "Curvfit" program was used again. Both first and second order polynomial equations for determining "Eo" were investigated for each of the three subgrade materials. An evaluation of the second order polynomial equations did not reveal any apparent correlation between soil index properties and the parameters. A larger data base containing a wide range of material properties is needed to determine the significance of the second order polynomial equations. After some modifications, the linear "Curvfit" equations yielded the following functions:

$$\text{Rutledge Pike and Cookeville: } E_o = -350 * S + 45500$$

$$\text{Jefferson County: } E_o = -350 * S + 40000$$

where: E_o = "initial modulus", psi

S = degree of saturation (75% - 100%).

Since the data points for the Rutledge Pike and Cookeville subgrade samples plotted together, the same linear equation was used to determine "Eo". The Jefferson County subgrade material had significantly different data. Only one of the constants in the linear equation was varied to characterize the Jefferson County subgrade material. Additional testing of subgrade samples with different material properties is required to further develop the "family of curves" on the "Eo versus Degree of Saturation" graph and to determine whether there is a correlation between soil classification and the parameter "Eo". (The Rutledge Pike and Cookeville subgrade soils were both given the USCS Classification of "SM" soils, while the Jefferson County subgrade soil was given the USCS Classification "SC-CL".)

2. PREDICTIONS OF THE RESILIENT MODULUS

A primary goal of this research project was the accurate prediction of resilient modulus values with a minimum amount of data input. Predictions of the resilient modulus at different stress levels were made for all of the test samples utilizing a second order polynomial equation of the form:

$$E_r = A * (\sigma_d)^2 + B * (\sigma_d) + E_o$$

where: E_r = the resilient modulus, psi

σ_d = the deviator stress ($\sigma_1 - \sigma_3$), psi

$A = 3.7$

$B = - 240$

E_o = the "initial modulus", psi

(varying according to the following material types and (S), the degree of saturation of the sample, in percent)

For the Rutledge Pike and Cookeville Materials:

$$E_o = -350 * S + 44500$$

For the Jefferson County Material:

$$E_o = -350 * S + 40000$$

The "A" and "B" parameters were kept constant for all of the test sample predictions. The " E_o " parameter chosen for each test prediction was based on the saturation and material type of the test sample. The " E_o " values were determined from the linear relationships depicted previously in Figure 16 on page 63.

Two methods were utilized to compare the predicted resilient modulus values and the resilient modulus values determined from laboratory repeated load tests. A simple correlation of discrete points and statistical analysis were both utilized.

First, a short computer program called "Predict" was created to compare discrete points on the predicted modulus curves with points on the laboratory determined curves. Tables VIII, IX, and X contain this information for three Rutledge Pike samples. Tables XI, XII, and XIII contain the "Predict" output for three Jefferson County samples and the output on three Cookeville samples is given in Tables XIV, XV, and XVI. A listing of the computer program called "Predict" is given in Appendix A.

Table VIII. Predictions of a Rutledge Pike Sample
Repeated Load Test.

Sample: R-02, Saturation = 91.9 %
A = 3.7, B = -240, Eo = 13335 psi

Stress (psi)	Measured Er (psi)	Predicted Er (psi)	Difference (%)
3	13000	12648.3	2.71
5	12400	12227.5	1.39
7	11950	11836.3	0.95
10	11850	11305.0	4.60
15	11750	10567.5	10.06
20	11500	10015.0	12.91
25	11475	9647.5	15.93
30	11400	9465.0	16.97
35	11300	9467.5	16.22
40	11150	9655.0	13.41

Table IX. Predictions of a Rutledge Pike Sample
Repeated Load Test.

Sample: R-03, Saturation = 89.8 %
A = 3.7, B = -240, Eo = 14070 psi

Stress (psi)	Measured Er (psi)	Predicted Er (psi)	Difference (%)
3	14000	13383.3	4.41
5	12800	12962.5	- 1.27
7	12300	12571.3	-2.21
10	12150	12040.0	0.91
15	11800	11302.5	4.21
20	11600	10750.0	7.33
25	11500	10382.5	9.71
30	10600	10200.0	3.77
35	10150	10202.5	- 0.52
40	9700	10390.0	- 7.11

Table X. Predictions of a Rutledge Pike Sample
Repeated Load Test.

Sample: R-08, Saturation = 96.4 %
A = 3.7, B = -240, Eo = 11760 psi

Stress (psi)	Measured Er (psi)	Predicted Er (psi)	Difference (%)
3	11800	11073.3	6.16
5	10900	10652.5	2.27
7	10300	10261.3	0.38
10	9500	9730.0	- 2.42
15	8900	8992.5	- 1.04
20	8800	8440.0	4.09
25	8950	8072.5	9.80
30	9200	7890.0	14.24
35	9300	7892.5	15.13
40	9300	8080.0	13.12

Table XI. Predictions of a Jefferson County Sample
Repeated Load Test.

Sample: J-02 , Saturation = 79.7 %
A = 3.7, B = -240, Eo = 12105 psi

Stress (psi)	Measured Er (psi)	Predicted Er (psi)	Difference (%)
3	11800	11418.3	3.23
5	11000	10997.5	0.03
7	10200	10606.3	- 3.98
10	9200	10075.0	- 9.51
15	8700	9337.5	- 7.33
20	8500	8785.0	- 3.35
25	8250	8417.5	- 2.03
30	8000	8235.0	- 2.94
35	7800	8237.5	- 5.61
40	7700	8425.0	- 9.42

Table XII. Predictions of a Jefferson County Sample
Repeated Load Test.

Sample: J-07, Saturation = 87.7 %
A = 3.7, B = -240, Eo = 9305 psi

Stress (psi)	Measured Er (psi)	Predicted Er (psi)	Difference (%)
3	9500	8618.3	9.28
5	8200	8197.5	0.03
7	6900	7806.3	-13.13
10	5700	7275.0	-27.63
15	5600	6537.5	-16.74
20	5600	5985.0	- 6.88
25	5600	5617.5	- 0.31
30	5600	5435.0	2.95
35	5600	5437.5	2.90
40	5600	5625.0	- 0.45

Table XIII. Predictions of a Jefferson County Sample
Repeated Load Test.

Sample: J-10, Saturation = 92.4%
A = 3.7, B = -240, Eo = 7660 psi

Stress (psi)	Measured Er (psi)	Predicted Er (psi)	Difference (%)
3	7900	6973.3	11.73
5	7000	6552.5	6.39
7	6400	6161.3	3.73
10	5700	5630.0	1.23
15	5200	4892.5	5.91
20	4800	4340.0	9.58
25	4600	3972.5	13.64
30	4400	3790.0	13.86
35	4200	3792.5	9.70
40	4200	3980.0	5.24

Table XIV. Predictions of a Cookeville Sample
Repeated Load Test.

Sample: C-01, Saturation = 90.1 %
A = 3.7, B = -240, Eo = 13965 psi

Stress (psi)	Measured Er (psi)	Predicted Er (psi)	Difference (%)
3	14100	13278.3	5.83
5	13500	12857.5	4.76
7	13200	12466.3	5.56
10	13100	11935.0	8.89
15	12900	11197.5	13.20
20	12800	10645.0	16.84
25	12600	10277.5	18.43
30	12500	10095.0	19.24
35	12200	10097.5	17.23
40	11600	10285.0	11.34

Table XV. Predictions of a Cookeville Sample
Repeated Load Test.

Sample: C-02, Saturation = 87.8 %
A = 3.7, B = -240, Eo = 14770 psi

Stress (psi)	Measured Er (psi)	Predicted Er (psi)	Difference (%)
3	15100	14083.3	6.73
5	14000	13662.5	2.41
7	13500	13271.3	1.69
10	13300	12740.0	4.21
15	13250	12002.5	9.42
20	13150	11450.0	12.93
25	13050	11082.5	15.08
30	12900	10900.0	15.50
35	12800	10902.5	14.82
40	12800	11090.0	13.36

Table XVI. Predictions of a Cookeville Sample
Repeated Load Test.

Sample: C-05, Saturation = 95.2 %
A = 3.7, B = -240, Eo = 12180 psi

Stress (psi)	Measured Er (psi)	Predicted Er (psi)	Difference (%)
3	10700	11493.3	- 7.41
5	10200	11072.5	- 8.55
7	9800	10681.3	- 8.99
10	9300	10150.0	- 9.14
15	8800	9412.5	- 6.96
20	8550	8860.0	- 3.63
25	8300	8492.5	- 2.32
30	7850	8310.0	- 5.86

Secondly, statistical analysis was conducted to determine a "Correlation Coefficient" (Cc) for each of the test samples. The Correlation Coefficient, commonly utilized in engineering, determines one value based on the entire data base rather than making a comparison of discrete points. Graphs depicting the predicted resilient modulus values and the laboratory determined values from several tests are given in Figures 17, 18, and 19 for the Rutledge Pike, Jefferson County, and Cookeville samples respectively. Correlation Coefficients (Cc) are also given on the figures.

The accuracy of the equation was found to be primarily linked to the choice of the "Eo" parameter. The equation was not found to be particularly sensitive to the constants "A" and "B". The predictions were predominately within 15% of the actual recorded test data. Several predicted curves were less than 10% from the actual test data. The Correlation Coefficients ranged from 0.85 to 0.99. (A perfect correlation between predicted resilient modulus values and the actual measured values would be 1.0.) Repeated load laboratory test results on fine-grained soil samples, prepared for a given set of variables, have been found to vary more than 15% between samples (Barksdale, et al., 1975). Therefore, this second order polynomial equation is considered accurate within the limitations of the testing procedure.

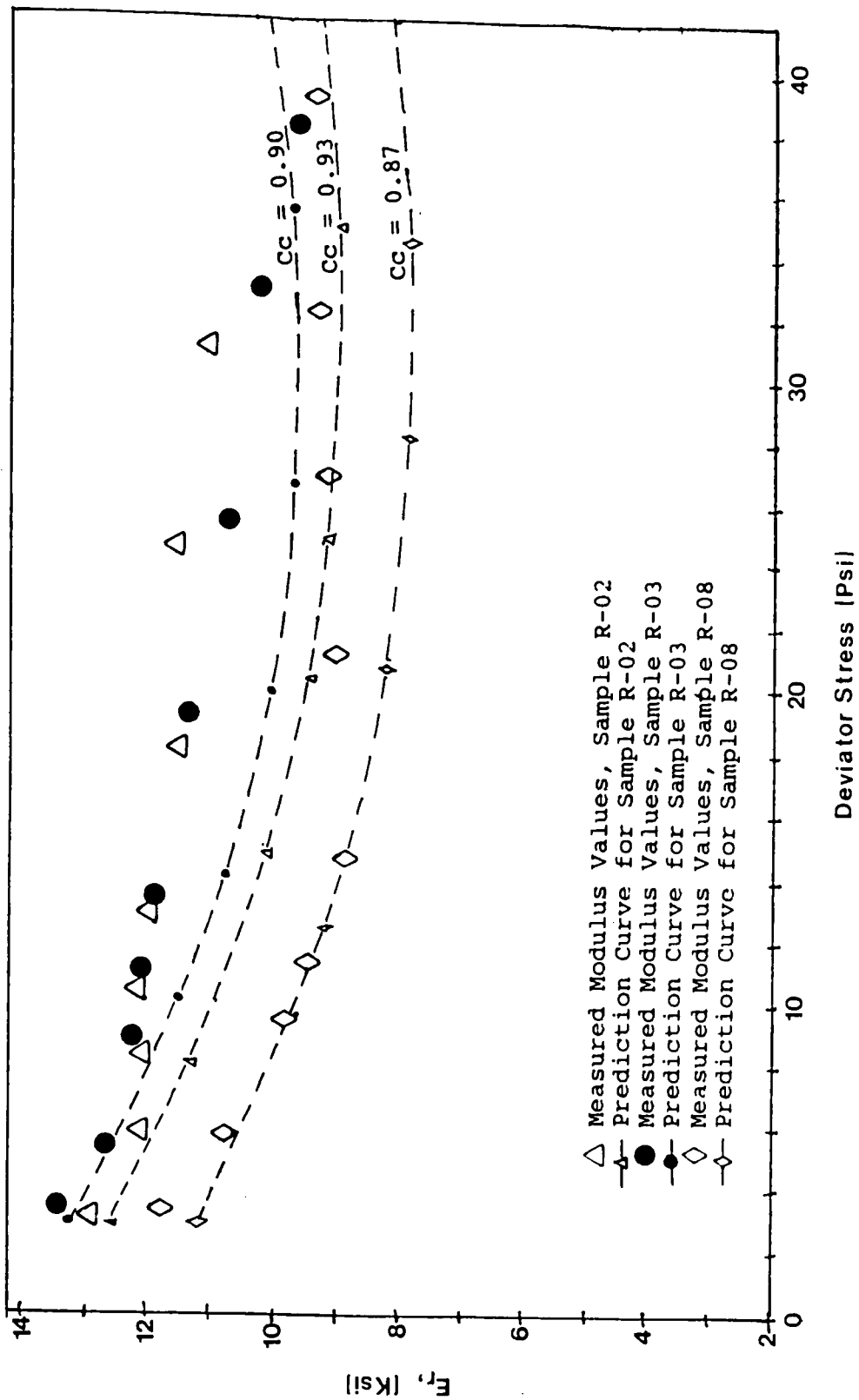


Figure 17. Plot of Predicted Modulus Values for Three Rutledge Pike Samples.

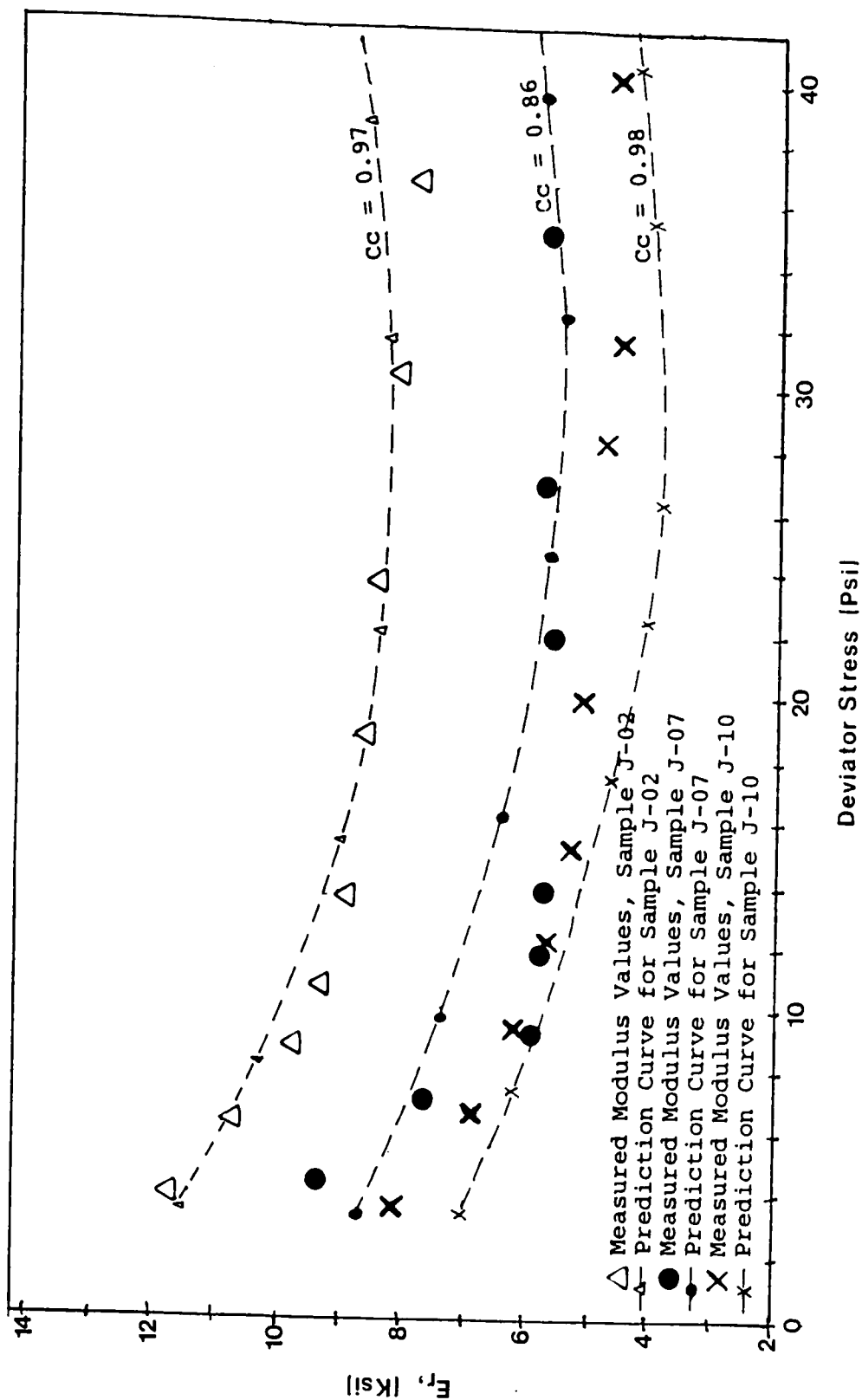


Figure 18. Plot of Predicted Modulus Values for Three Jefferson County Samples.

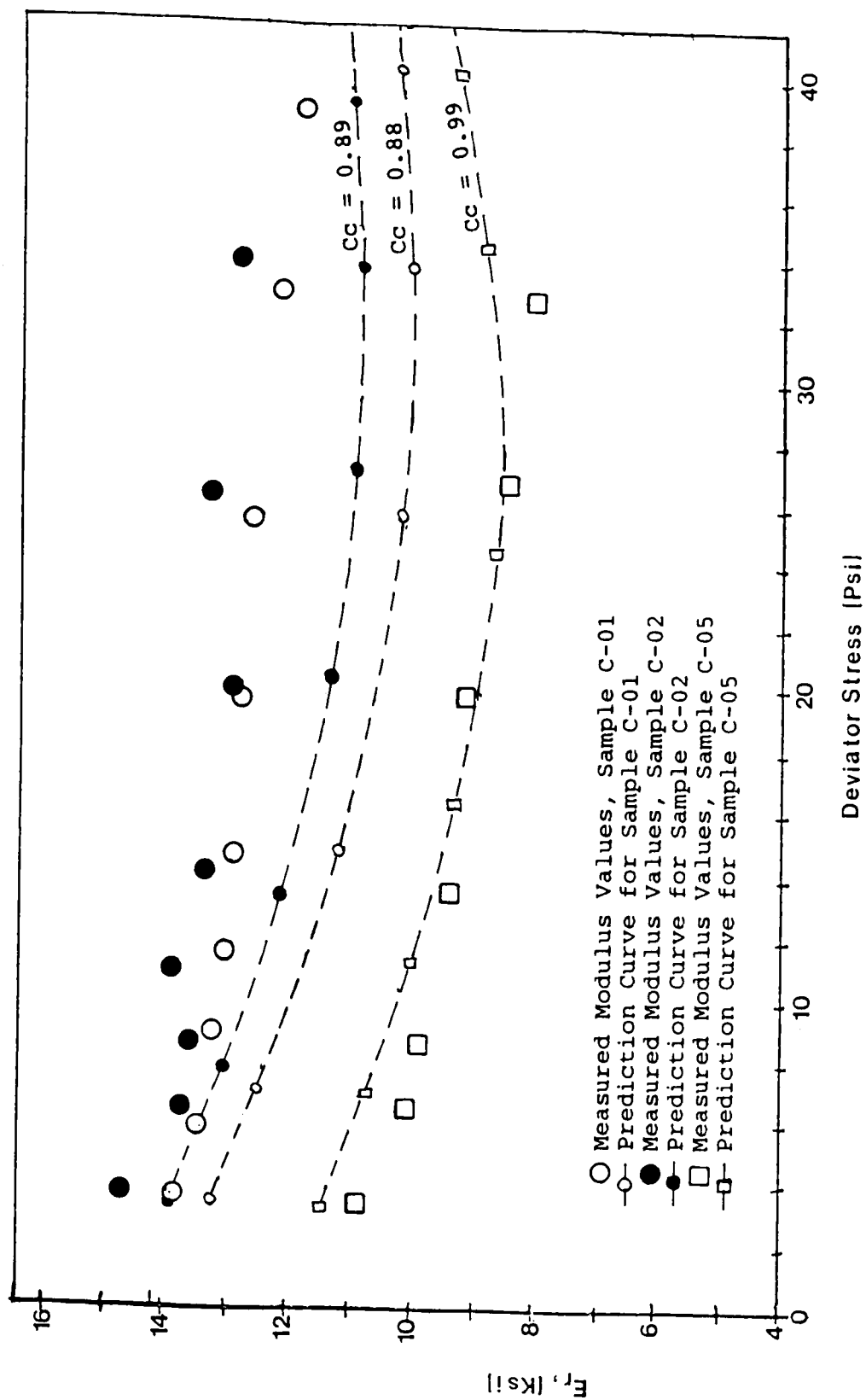


Figure 19. Plot of Predicted Modulus Values for Three Cookeville Samples.

CHAPTER VII

CONCLUSIONS

1. SUMMARY

The first stage of a project to characterize the resilient moduli of subgrade soils in Tennessee was conducted. A review of previous research was conducted. Laboratory equipment was fabricated and a testing procedure was adopted. A computerized data acquisition system was set-up and computer programs were written to reduce the collected data.

Fine-grained subgrade soil samples were collected from three sites in the Eastern Tennessee area for laboratory evaluation. Standard laboratory tests were run on the field samples to determine the soil index properties of each subgrade soil. Samples were prepared at various water contents by a kneading compaction method in the laboratory. Repeated load tests were conducted on the compacted samples at deviator stresses ranging from 2 to 35 psi.

The results of repeated load tests have been shown in plots of deviator stress versus resilient modulus. Preliminary results revealed that the resilient modulus of all three fine-grained subgrade materials was affected by several factors. Reductions in the resilient moduli

resulted from increases in the applied deviator stress. The resilient modulus also increased as the compacted dry density was increased, but decreased when the water content at the time of compaction was above optimum.

Several attempts were made to characterize the results from repeated load tests and to predict the resilient modulus. A second order polynomial equation, which utilizes deviator stress and the degree of saturation of the sample to predict the resilient modulus, was used successfully. Predictions were predominately within 15% of the laboratory measured resilient modulus values. Coefficients of variation ranged between 0.85 and 0.99. In a previous study, the variability of resilient moduli of similar samples prepared under the same conditions on the same day was determined to be as much as 15% for repeated load testing of fine-grained soils (Barksdale, et al., 1975).

2. RECOMMENDATIONS

The testing procedures established for this study provided a starting point for the evaluation of the resilient moduli of subgrade soils throughout the state of Tennessee. Improvements in the accuracy of resilient moduli determinations and verification of the second order

polynomial prediction of E_r are given in the following recommendations:

1. Use two LVDT's, mounted on either side of the loading shaft, to determine axial deformations. The average of the two outputs should be utilized in strain calculations.
2. Recalibrate the LVDTs to increase their accuracy when measuring the deformation of stiffer soil specimens.
3. Increase the resolution of the displacement data by changing the range of voltages that Lab Data Manager (LDM1) reads from ± 5 volts to ± 1 volt. The actual range of testing voltages was found to vary less than 0.5 volts.
4. Install a filter that eliminates the pick-up of "noise" in the system which causes recorded voltage readings to periodically fluctuate as much as 0.03 volts.
5. Modify the existing conversion program so that less input is needed from the keyboard. The program would run with fewer interruptions and some subjectively determined input would be eliminated.
6. Eliminate the tests at deviator stresses above 20 psi. The actual stresses in the subgrade seldom reach these levels. For most of the fine-grained samples tested,

the resilient modulus did not decrease significantly from the 20 psi test to the 35 psi test.

7. Investigate the effects of other material properties, such as density, on the resilient modulus.

8. Perform extensive testing of subgrade soil samples to determine the full range of resilient moduli for Tennessee soils.

9. Check the validity of the second order polynomial predictions. Develop the "family of curves" that relate the degree of saturation to E_o .

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APPENDICES

APPENDIX A

COMPUTER PROGRAMS AND SAMPLE OUTPUT

This appendix contains listings from three basic programs used on an Apple II computer in the laboratory. The program "Set" is a data acquisition program which utilizes a software package called "Lab Data Manager". (A brief explanation of "Lab Data Manager" is given in Appendix C.) The software package stores the data on temporary files. The program "File" creates a permanent data file utilizing the data stored on the temporary files. The data reduction program labeled "Convert" processes load and displacement which has been stored on floppy disc files. The program calculates axial stress and strain and then searches the data for maximum values of stress and strain. These values of maximum stress and strain are used in the calculation of the resilient modulus. Sample output from the program "Convert" also appears in this appendix.

A simple program called "Predict" was created for use on an IBM PC. The program evaluates the second order polynomial equation predictions and compares the predicted modulus values with the actual laboratory determined resilient modulus values. A listing of the program is given in this appendix.

```

1  REM      PROGRAM SET
10 REM      CREATE EXEC FILE: GEO2
20 REM      FOR TWO CHANNEL A/D CONVERSION
30 REM      REQUIRES PROGRAM "FILE"
40 REM      *****
80 REM      UT DEPT OF CIVIL ENGR
90 REM      E.C.DRUMM & T.D.JOHNSON
100 REM     CREATED JANUARY 1985
110 REM     *****
125 DIM ZLOD(2000),ZDSP(2000),INITIAL(50)
130 D$ = CHR$(4): HOME
140 REM     CTRL-D AND CLEAR SCREEN
150 NC = 2
170 PRINT "DO YOU WANT TO RUN A TEST WITH NS=256 & SR=25 (Y=1 & N=2) ?"

172 INPUT ANSWER
175 IF ANSWER = 1 GOTO 182
180 IF ANSWER = 2 GOTO 185
181 GOTO 172
182 NS = 255
183 SR = 25
184 GOTO 190
185 HOME : PRINT " INSERT BLANK DATA DISK IN DRIVE #2 ": PRINT
190 PRINT : INPUT " INPUT THE FILE NAME FOR DATA STORAGE      ";FILE$
200 PRINT : INPUT " ADDITIONAL COMMENTS ";COM$
202 IF ANSWER = 1 GOTO 230
205 PRINT " NUMBER OF SAMPLES-ENTER A MULTIPLE OF 128": PRINT
210 PRINT : INPUT "NUMBER OF SAMPLES (1:2048) = ";NS:NS = NS - 1: PRINT

220 PRINT : INPUT "SAMPLING RATE,PER SECOND (MAX = 1850)";SR
230 TIME = 1.023 / (SR * 10 ^ - 6)
240 PRINT : PRINT : PRINT " FILE NAME = ";FILE$: PRINT
250 PRINT ;COM$: PRINT
260 PRINT " TIME = ";TIME
270 PRINT "NO. OF SAMPLES = ";NS
280 PRINT "SAMPLING RATE = ";SR;" SAMPLES PER SECOND": PRINT : PRINT
290 PRINT : INPUT " O.K.?";ANS$: IF ANS$ = "N" THEN GOTO 190: PRINT
300 HOME
310 PRINT " ENTER 'EXEC GEO2': PRINT " TO RUN DATA ACQISITION ROUT
    INE "
320 REM
330 NI = 4: REM  DIMENSION ON INIT(1)
340 INIT(1) = NS:INIT(2) = SR:INIT(3) = F1:INIT(4) = F2
345 REM  DELETE EXISTING DATA FILES
350 PRINT D$;"CATALOG,D2": PRINT D$;"OPEN ZDISP"
352 PRINT D$;"DELETE ZDISP": PRINT D$;"CLOSE ZDISP"
354 PRINT D$;"OPEN ZLOAD": PRINT D$;"DELETE ZLOAD"
356 PRINT D$;"CLOSE ZLOAD": PRINT D$;"CATALOG,D1"
360 REM
370 PRINT D$;"OPEN GEO2": PRINT D$;"WRITE GEO2"
380 PRINT "RUN LDM1"
390 PRINT "2": REM  ADALAB IN SLOT #2
400 PRINT "3": REM  AI13 IN SLOT #3
410 PRINT NC: REM  NO. OF CHANNELS
420 PRINT CHR$(21); REM  CONTROL U TO GET OUT OF MENU
430 REM
440 REM  AXIAL LOAD:CHANNEL 0, STORED IN BUFFER A
450 PRINT "RAM 0 0 8": REM  ARG 2 IS BUFFER
460 PRINT "FAD 0 4 ";NS;" 1": REM
    G      1 IS CHANNE      L #
470 REM  AXIAL DEFORMATION: CHANNEL 1, STORED IN BUFFER C"

```

AR

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480 PRINT "RAM 0 2 8"
490 PRINT "FAD 1 6 ";NS;" 1"
500 PRINT "STA 0 1"
510 PRINT "STA 1 1"
520 PRINT "STA 18 ";TIME
530 REM ? CHR$ (19)
540 REM ? "ENTER GOTO 998 TO BEG IN SAMPLING "
550 PRINT "STA 17 -1"
560 REM ? "STA 17 -1"; REM STARTSAMPLING (-1 FOR TRIGGER)
570 REM
580 PRINT "CHK 1": REM FLAG F OR SAMPLING COMPLETE
585 PRINT "STA 17 0": REM STOP CLOCK
590 REM MULTIPLY DATA BY CONVERSION FACTORS
620 REM
625 PRINT "CATALOG,D2";D$
630 PRINT "SAV A 1 ZLOAD "
640 PRINT "SAV C 1 ZDISP "
650 PRINT CHR$ (19): REM CTRL S
655 REM CREATE PERMANENT DATA FILE
659 PRINT "CATALOG,D1"
660 PRINT "RUN FILE"
662 PRINT FILE$
663 PRINT COM$
670 PRINT NI
680 FOR I = 0 TO NI: PRINT INIT(I): NEXT I
690 PRINT "RUN SET"
999 REM ? CHR$ (19): REM CTR S
1000 PRINT D$;"CLOSE GEO2"

```

```

10 REM **PROGRAM FILE**
15 REM THIS IS CALLED BY GEO2(SET)
20 REM CREATE DATA FILE FROM DISP AND LOAD FILES
30 REM
35 D$ = CHR$(4)
40 PRINT D$;"CATALOG,D2"
100 INPUT FILE$: INPUT COM$: INPUT NI
160 DIM ZLD(2000),ZDSP(2000),INIT(50)
190 FOR I = 0 TO NI: INPUT INIT(I): NEXT I
200 PRINT D$;"OPEN ZLOAD"
300 PRINT D$;"READ ZLOAD"
400 INPUT N: FOR I = 0 TO N - 1: INPUT ZLD(I)
600 NEXT I
700 PRINT D$;"CLOSE ZLOAD"
750 REM
800 PRINT D$;"OPEN ZDISP"
900 PRINT D$;"READ ZDISP"
1000 INPUT M: FOR I = 0 TO M - 1: INPUT ZDSP(I)
1300 NEXT I
1400 PRINT D$;"CLOSE ZDISP"
1600 PRINT D$;"OPEN";FILE$: PRINT D$;"WRITE";FILE$
1605 NUMB = N + M + NI + 2
1607 PRINT NUMB
1610 PRINT N: FOR I = 0 TO N - 1: PRINT ZLD(I)
1615 NEXT I
1620 PRINT M: FOR I = 0 TO M - 1: PRINT ZDSP(I)
1625 NEXT I
1630 REM FOR I = 0 TO NI
1632 REM PRINT INIT(I)
1635 REM NEXT I
1640 PRINT COM$
1650 PRINT D$;"CLOSE "FILE$
1655 PRINT D$;"CATALOG,D1"
1660 REM ? D$;"LOCK"FILE$

```

```

5 REM *****
10 REM ***** PROGRAM: CONVERT *****
15 REM *** CREATED 9-20-86 BY:T.D.JOHNSON REVISED 1-07-86 ***
20 REM *****
30 REM *THIS PROGRAM TAKES THE ORIGINAL RECORDED TEST DATA *
40 REM *AND MODIFIES IT SO THAT THE DATA CORRESPONDS TO *
50 REM *THE DIGITAL VOLTAGES DISPLAYED ON THE LED READOUTS. *
60 REM *THE DATA IS THEN CONVERTED TO LOAD (LB. AND KN) AND *
70 REM *DISPLACEMENT (IN. AND CM.) VALUES OF STRESS AND *
80 REM *STRAIN ARE CALCULATED NEXT. THE MAXIMUM VALUES OF *

90 REM *STRESS AND STRAIN ARE RECORDED SO THAT THE *
95 REM *MODULUS OF RESILIENCE CAN BE DETERMINED. *
98 REM *****
99 REM
100 DIM ZLD(300),ZDSP(300),INIT(50),LD(300),DISP(300),STRESS(300),ESTRA
    IN(300)
105 REM CONVERSION FACTORS
106 F1 = 1.10
107 F2 = 2.13
108 D$ = CHR$(4)
109 PRINT D$;"CATALOG,D2"
110 PRINT : PRINT
115 PRINT "FILENAME ?"
120 INPUT FILE$
130 PRINT D$;"OPEN";FILE$
135 PRINT D$;"READ";FILE$
140 INPUT NUMB
150 INPUT N
160 FOR I = 1 TO N
170 INPUT ZLD(I)
171 REM SIMPLE ROUTINE FOR ROUNDING
172 REM X = ZLD(I)/10.0
174 REM X1= INT(X)
176 LD(I) = (ZLD(I) - 2046) * F1 / 1000
180 NEXT I
190 INPUT M
200 FOR I = 1 TO M
210 INPUT ZDSP(I)
211 REM ROUTINE FOR ROUNDING
212 REM Y = ZDSP(I)/10.0
214 REM Y1= INT(Y)
216 DISP(I) = (ZDSP(I) - 2046) * F2 / 1000
220 NEXT I
225 REM INSERT THIS SECTION IF INITIAL CONDITIONS ARE GIVEN
230 REM FOR I= 0 TO NI
240 REM INPUT INIT(I)
250 REM NEXT I
260 INPUT COM$
265 PRINT D$;"CLOSE";FILE$
266 PRINT "TO PRINT ALL ORIGINAL DATA & VOLTAGE DATA - TYPE 1 , FOR JUS
    T THE FIRST 75 VALUES - TYPE 2 "
268 INPUT P
270 PR# 1
275 PRINT "=====
    ====="
278 PRINT : PRINT
280 PRINT "FILENAME : ";FILE$
285 PRINT
290 PRINT "COMMENTS BEFORE TESTING : ";COM$
300 PRINT : PRINT
310 PRINT "ORIGINAL DATA",,"VOLTAGE DATA"
320 PRINT "LOAD","DISPLACEMENT",,"LOAD",,"DISPLACEMENT"
330 PRINT "-----"
335 IF P = 2 THEN N = 75
340 FOR I = 1 TO N
342 REM FORMAT FOR OUTPUT
344 LD$ = STR$(LD(I))
346 DISP$ = STR$(DISP(I))
348 Z = LEN(LD$)
350 PRINT ZLD(I),ZDSP(I),, LEFT$(LD$,5); SPC(14 - Z); LEFT$(DISP$,6)

360 NEXT I
370 PRINT : PRINT
380 PR# 0
390 PRINT "INITIAL LVDT READING ?"
400 INPUT OLVD
410 PRINT "INITIAL LENGTH OF SAMPLE (INCHES) ?"
420 INPUT LNGTH
430 PRINT "INITIAL DIAMETER OF SAMPLE (INCHES) ?"
432 INPUT DMTR
435 PRINT "LVDT READING FOR THIS TEST ?"
440 INPUT NLVD

```

```

443 PRINT "TO PRINT STRESS &    STRAIN DATA - TYPE 1    OTHERWISE
    - TYPE 2"
445 INPUT PS
450 AREA = 3.1415 * DMTR * DMTR / 4
480 PR# 1
490 PRINT "INITIAL LENGTH OF SAMPLE = ";LNGTH;" INCHES"
500 PRINT "INITIAL DIA. OF SAMPLE = ";DMTR;" INCHES"
510 PRINT "LVDT READING FOR THIS TEST = ";NLVDT;" VOLTS"
515 IF PS = 2 GOTO 542
520 PRINT : PRINT
530 PRINT "STRESS(Psi)","STRAIN(%)",,"STRESS(KPa)"
540 PRINT "-----"
542 MAXSTRESS = 0.0
544 LARGESTRAIN = 0.0
550 FOR I = 1 TO 256
555 REM 50.0 LB/VOLT & 0.2225 KN/VOLT
560 STRESS(I) = LD(I) * 50.0 / AREA
565 REM 6.89 KPA/PSI
570 MSTRESS = STRESS(I) * 6.89
575 REM 0.05 IN/VOLT
577 REM REMEMBER TO ADJUST SAMPLE LENGTH
578 LRED = LNGTH - ((NLVDT - OLVD) * 0.05)
580 ESTRAIN(I) = (DISP(I) - NLVDT) * 0.05 / LRED
590 REM STRAIN IS DIMENTIONLESS (IN/IN OR CM/CM)
595 IF I = 1 GOTO 675
600 IF STRESS(I) > STRESS(I - 1) GOTO 620
610 GOTO 640
620 IF STRESS(I) < MAXSTRESS GOTO 640
630 MAXSTRESS = STRESS(I)
640 IF ESTRAIN(I) > ESTRAIN(I - 1) GOTO 660
650 GOTO 675
660 IF ESTRAIN(I) < LARGESTRAIN GOTO 675
670 LARGESTRAIN = ESTRAIN(I)
675 IF PS = 2 GOTO 695
680 REM FORMAT FOR OUTPUT
681 PSTRAIN = ESTRAIN(I) * 100
684 STRESS$ = STR$(STRESS(I))
686 PSTRAIN$ = STR$(PSTRAIN)
688 MSTRESS$ = STR$(MSTRESS)
689 ZZ = LEN(PSTRAIN$)
690 PRINT LEFT$(STRESS$,6), LEFT$(PSTRAIN$,6); SPC(20 - ZZ); LEFT$(
    MSTRESS$,6)
695 NEXT I
700 PRINT : PRINT
710 PRINT "*****"
715 PRINT : PRINT
717 PRINT "MAXIMUM STRESS = ";MAXSTRESS;" PSI"
718 PRINT "MAXIMUM STRAIN = ";LARGESTRAIN
719 PRINT : PRINT
720 MODULUS = MAXSTRESS / LARGESTRAIN
725 REM 6.89 KPA/PSI
730 MODULUS = MODULUS * 6.89
740 PRINT "RESILIENT MODULUS = ";MODULUS;" PSI"
750 PRINT "RESILIENT MODULUS = ";MODULUS;" KPA"
752 PRINT : PRINT
755 PRINT "*****"
760 PR# 0
765 PRINT "ARE THERE ANY ADDITIONAL COMMENTS (Y=1 & N=2)"

766 INPUT C
767 IF C = 1 GOTO 770
768 AC$ = "NONE"
769 GOTO 780
770 PRINT "INPUT COMMENTS"
775 INPUT AC$
780 PR# 1
785 PRINT : PRINT
790 PRINT "ADDITIONAL COMMENTS : ";AC$
795 PRINT : PRINT
797 PRINT "=====
=====
798 PRINT : PRINT : PRINT
799 PRINT : PRINT : PRINT
800 PR# 0
805 PRINT "RUN CONVERT3 AGAIN? (Y=1 & N=2)"
806 INPUT Q
807 IF Q = 2 GOTO 850
810 PRINT D$;"CATALOG,D1"
820 PRINT D$;"RUN CONVERT3"
840 GOTO 100
850 END

```

=====

FILENAME : C-03/10PSI/3/3

COMMENTS BEFORE TESTING : HAD TO REDO 10PSI TEST (THIS IS THE BEST)

ORIGINAL DATA		VOLTAGE DATA	
LOAD	DISPLACEMENT	LOAD	DISPLACEMENT
2082	1939		
2082	1939	.0864	-2.817
2080	1939	.0864	-2.817
2078	1940	.0816	-2.817
2080	1939	.0768	-2.79
2078	1939	.0816	-2.817
2078	1939	.0768	-2.817
2082	1939	.0768	-2.817
2075	1940	.0864	-2.817
2079	1939	.0696	-2.79
2077	1939	.0792	-2.817
2080	1939	.0744	-2.817
2078	1939	.0816	-2.817
2079	1940	.0768	-2.817
2079	1939	.0792	-2.79
2079	1940	.0792	-2.817
2080	1939	.0792	-2.79
2080	1939	.0816	-2.817
2074	1940	.0816	-2.817
2079	1939	.0672	-2.79
2081	1940	.0792	-2.817
2074	1939	.084	-2.79
2079	1939	.0672	-2.817
2078	1939	.0792	-2.817
2079	1938	.0768	-2.817
2065	1940	.0792	-2.845
2066	1939	.0456	-2.79
2072	1939	.048	-2.817
2091	1940	.0624	-2.817
2143	1939	.108	-2.79
2221	1941	.0328	-2.817
2303	1941	.42	-2.752
2374	1942	.6168	-2.762
2413	1943	.7572	-2.734
2401	1942	.8808	-2.706
2332	1943	.852	-2.734
2243	1941	.6864	-2.706
2156	1941	.4728	-2.762
2092	1940	.364	-2.752
2054	1939	.1104	-2.75
2044	1939	.0182	-2.817
2059	1939	-4.68	-2.817
2111	1940	.0312	-2.817
2183	1940	.156	-2.79
2266	1941	.0288	-2.79
2349	1942	.528	-2.762
2401	1942	.7272	-2.734
2414	1943	.852	-2.734
2378	1943	.8932	-2.706
2301	1942	.7968	-2.706
2204	1942	.612	-2.734
2129	1940	.3792	-2.734
2067	1940	.1992	-2.79
		.0504	-2.79

2040	1939	-1.014	-2.817
2047	1939	2.4E-	-2.817
2076	1939	.072	-2.817
2131	1939	.204	-2.817
2207	1941	.2864	-2.762
2288	1941	.5808	-2.762
2366	1942	.768	-2.734
2414	1943	.8832	-2.706
2406	1942	.864	-2.734
2350	1943	.7296	-2.706
2262	1942	.8184	-2.734
2169	1941	.2952	-2.762
2101	1941	.132	-2.762
2056	1939	.024	-2.817
2041	1939	-1.012	-2.817
2053	1938	.0168	-2.845
2097	1939	.1224	-2.817
2159	1940	.2712	-2.79
2245	1940	.4776	-2.79
2327	1942	.6744	-2.734
2383	1942	.8088	-2.734
2416	1943	.888	-2.706

INITIAL LENGTH OF SAMPLE = 4.337 INCHES
 INITIAL DIA. OF SAMPLE = 2 INCHES
 LVDT READING FOR THIS TEST = -2.8 VOLTS

MAXIMUM STRESS = 14.400764 FSI
 MAXIMUM STRAIN = 1.08107485E-03

RESILIENT MODULUS = 13320.7835 FSI
 RESILIENT MODULUS = 91780.1984 KPA

ADDITIONAL COMMENTS : NONE

=====

```

0
                                PREDICT
5 REM *****
10 REM This program uses curvefit data to predict the resilient modulus
20 REM at different stress levels. The predicted data is compared to
30 REM the actual data and the percent difference is calculated.
40 REM *****
50 DIM ERP(10),ERA(10),DIFF(10),S(10)
60 PRINT "Input the sample name"
70 INPUT SAMPLE$
80 PRINT "Input A"
90 INPUT A
100 PRINT "Input B"
110 INPUT B
120 PRINT "Input E0"
130 INPUT E0
140 A$ = STRING$(70,45)
150 FOR X = 1 TO 10
155 S(1) = 3
160 S(2) = 5
165 S(3) = 7
170 S(4) = 10
175 S(5) = 15
180 S(6) = 20
185 S(7) = 25
190 S(8) = 30
195 S(9) = 35
200 S(10) = 40
210 PRINT "Input Actual Er Data "
220 PRINT "What is Er when stress = " S(X) " psi?"
230 INPUT ERA(X)
240 NEXT X
250 LPRINT
260 LPRINT "SAMPLE : " SAMPLE$
262 LPRINT " A = " A " B = " B " E0 = " E0
270 LPRINT
280 LPRINT "STRESS      " "ACTUAL Er VALUES      " "PREDICTED Er VALUES      " "DIFF
DIFFERENCE"
290 LPRINT A$
300 LPRINT
310 FOR X = 1 TO 10
320 ERP(X) = A*S(X)*S(X) + B*S(X) + E0
330 DIFF(X) = ((ERA(X) - ERP(X))/ERA(X))* 100
340 LPRINT TAB(2) S(X) TAB(12) ERA(X) "      " ERP(X) "      " DIFF
F(X)
350 NEXT X
360 END

```

APPENDIX B

CALIBRATION OF THE TESTING EQUIPMENT

The electronic data acquisition equipment, consisting of the load cell, Brush amplifier, LVDT and panel of Schavitz digital displays was taken to the Tennessee Valley Authority's Singleton Laboratory for calibration. (The Brush amplifier was used to boost the signal coming out of the load cell.)

The load cell was placed in a load frame which was connected to electronic equipment capable of measuring applied load to the thousandths of a pound. Specified loads were applied to the load cell and the Brush amplifier was adjusted to reach the desired calibration value of 50 lb./volt. After the adjustments were made, a dial on the Brush amplifier was turned to calibrate and a calibration voltage equal to + 3.23 was recorded.

The Linear Variable Differential Transducer (LVDT) was placed in the zero position in a clamping device. The LVDT was displaced + 0.5 inches and the output voltage was adjusted to read +10.00 volts, then the LVDT was displaced 0.5 inches in the negative direction to make sure the output voltage was - 10.00 volts. For this calibration value of 0.05 inches/volt, a value of + 7.58 volts was recorded when the LVDT was in the zero position and the calibration switch was turned on.

The calibrated equipment was then checked to verify the accuracy. Loads from 0 to 200 lbs. were applied and the voltage output from the load cell at each level was recorded. A plot of load versus output voltage revealed a straight line. The LVDT was placed in the zero position and then the voltage output was checked with calibrated stainless steel gauge blocks.

The calibration digital voltage values for both the load cell and LVDT were checked and minor adjustments were made (when necessary) before each laboratory repeated load test was conducted. The equipment was also taken back to the Singleton Laboratory to check the calibration periodically.

APPENDIX C

LAB DATA MANAGER

"Lab Data Manager" is a general purpose data acquisition system designed for laboratory use. It has a variety of features including multiple inputs, fast sampling rate, and data processing (Warne, 1982). With the fast A/D (Analog to Digital) option, up to sixteen voltage inputs may be measured simultaneously or sequentially. "Lab Data Manager" assigns an integer value between 0 and 4095 to the output voltage. Several voltage ranges are available including -5.0 to +5.0 volts and -0.1 to +1.0 volts. Over 300 samples per second can be made from these sixteen inputs. A number of math calculations can be made including addition, subtraction, multiplication, division, averaging, and normalizing (Warne, 1981).

"Lab Data Manager" was used for two channel sampling of the load cell and LVDT voltage outputs. An external trigger was used to initiate sampling. Ranges of -5.0 to +5.0 volts were chosen for the load cell and LVDT. The integer values output from the "Lab Data Manager" program were stored as data files on floppy disks. The internal mathematical operations were not utilized. The computer program "Convert" was used to reduce the stored integer values into stress and strain data.

APPENDIX D

ADDITIONAL LABORATORY TEST DATA

The laboratory data sheets from several types of standard laboratory tests are given in this appendix for the three fine-grained subgrade materials that were investigated. Standard Proctor test data, specific gravity of solids determinations, Atterburg Limit data, and washed sieve analyses data are presented.

The results from repeated load tests on all of the samples are given in three plots of the resilient modulus versus deviator stress. Moisture content and degree of saturation are given for each sample.

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COMPACTION TEST-MOISTURE DENSITY RELATIONSHIP

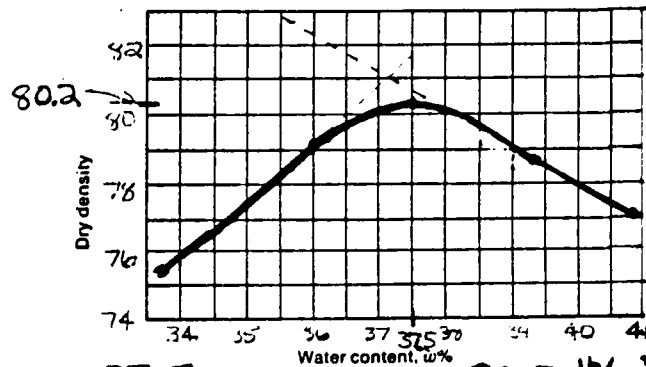
Project Rutledge Pike Job No. Job Location State Route #1
Boring No. Sample No. Depth
Description of Soil USCS classification = SM
Tested By T. Johnson Date 9/30/85 Calculated By T. Johnson Date 10/1/85
Test Description: Blows/Layer 25 No. of Layers 3
Hammer Mass (wt) kg (lbs) Volume of Mold 1/30 m³(ft³)

Density

Determination No.	1	2	3	4	5	6	7	8
Mass of Soil + Mold	13.05	13.27	13.34	13.30				
Mass of Mold	9.64	9.69	9.64	9.67				
Mass of Soil	3.37	3.59	3.66	3.62				
Wet Density	101.1	107.7	109.8	108.6				
Dry Density	75.62	79.19	78.82	77.08				

Water Content

Determination No.	1	2	3	4	5	6	7	8
Tare No.	C11	*200	*202	*216				
Mass of Tare + Wet Soil	32.5	32.9	29.0	29.3				
Mass of Tare + Dry Soil	29.5	27.9	24.8	24.8				
Mass of water	3.3	5.0	4.2	4.5				
Mass of Tare	19.7	14.0	14.1	13.8				
Mass of Dry Soil	9.8	13.9	10.7	11.0				
Water Content	33.7	36.0	39.3	40.9				



Optimum moisture = 37.5 % Maximum dry density = 80.2 lb/ft³

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COMPACTION TEST-MOISTURE DENSITY RELATIONSHIP

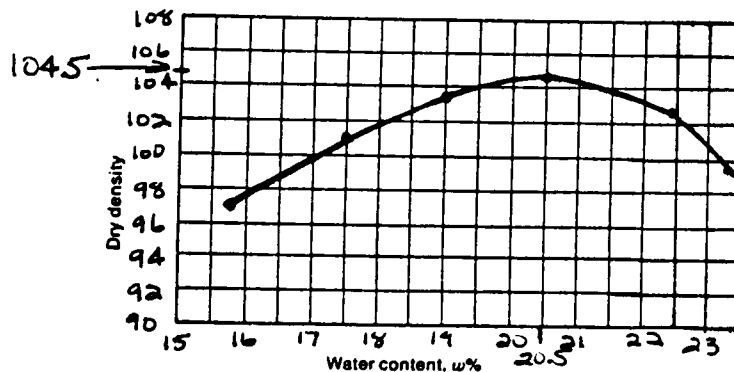
Project Jefferson Co. Job No. _____ Job Location I-40
Boring No. _____ Sample No. _____ Depth _____
Description of Soil USCS Classification = SC-CL
Tested By T. Johnson Date 4/7/86 Calculated By T. Johnson Date 4/8/86
Test Description: Blows/layer 25 No. of Layers 3
Hammer Mass (wt) _____ kg (lbs) Volume of Mold 1/30 m³ (ft³)

Density

Determination No.	1	2	3	4	5	6	7	8
Mass of Soil + Mold	13.205	13.41	13.565	13.645	13.635	13.545		
Mass of Mold	9.85	9.85	9.46	9.45	9.46	9.45		
Mass of Soil	3.255	3.760	4.115	4.195	4.185	4.095		
Wet Density	112.65	118.80	123.85	125.86	125.55	122.85		
Dry Density	97.4	101.1	103.7	104.5	102.6	99.6		

Water Content

Determination No.	1	2	3	4	5	6	7	8
Tare No.	C9	C10	C11	C15	D1	D12		
Mass of Tare + Wet Soil	45.8	44.4	49.5	40.6	44.1	50.7		
Mass of Tare + Dry Soil	42.3	40.7	44.7	37.1	39.8	44.8		
Mass of water	3.6	3.7	4.8	3.6	4.3	5.9		
Mass of Tare	19.2	19.6	19.5	20.0	20.6	19.5		
Mass of Dry Soil	23.0	21.1	25.2	17.1	19.2	25.3		
Water Content	15.7	17.5	19.0	20.5	22.4	23.3		



Optimum moisture = 20.5 % Maximum dry density = 104.5 lb/ft³

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COMPACTION TEST-MOISTURE DENSITY RELATIONSHIP

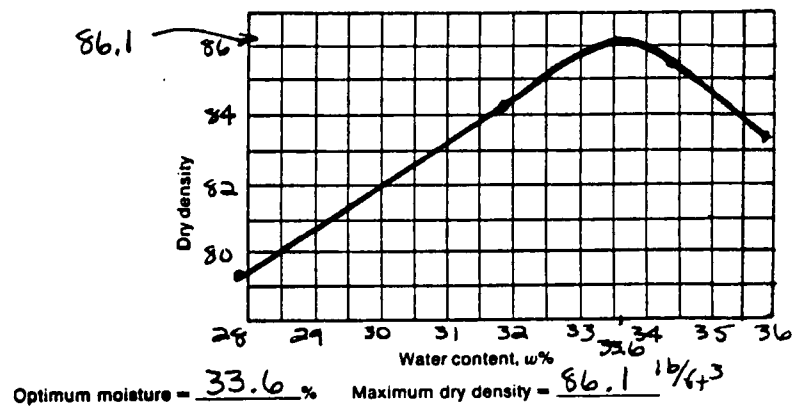
Project Cookeville Job No. _____ Job Location State Route #111
Boring No. _____ Sample No. _____ Depth _____
Description of Soil USCS Classification = SM
Tested By T. Johnson Date _____ Calculated By T. Johnson Date _____
Test Description: Blows/Layer _____ No. of Layers _____
Hammer Mass (wt) _____ kg (lbs) Volume of Mold _____ $M^3 (ft^3)$

Density

Determination No.	1	2	3	4	5	6	7	8
Mass of Soil + Mold	12.82	13.14	13.265	13.22				
Mass of Mold	9.44	9.44	9.44	9.44				
Mass of Soil	3.38	3.70	3.825	3.78				
Wet Density	101.4	111.0	114.75	113.4				
Dry Density	79.3	84.2	85.4	83.5				

Water Content

Determination No.	1	2	3	4	5	6	7	8
Tare No.	A1	A10	A12	A13				
Mass of Tare + Wet Soil	45.7	42.1	44.7	41.5				
Mass of Tare + Dry Soil	39.9	36.6	38.4	35.8				
Mass of water	5.8	5.5	6.3	5.75				
Mass of Tare	19.1	19.3	20.1	19.7				
Mass of Dry Soil	20.8	17.3	18.3	16.1				
Water Content	27.9	31.8	34.4	35.8				



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SPECIFIC GRAVITY DETERMINATION

Project Rutledge Pike Job No. _____ Job Location State Route #1

Boring No. _____ Sample No. _____ Depth _____

Description of Soil U.S.C.S Classification = SM

Tested By T. Johnson Date 3/31/86 Calculated By T. Johnson Date 4/1/86
C. Allin

$K =$	0.99646	0.99651		
Flask No.	#1	#1		
Method of Air Removal	aspiration	aspiration		
M_{bws}	701.6	702.0		
Temperature $T^{\circ}C$	27.3°	27.1°		
M_{bw}	671.26	671.28		
Evap. Dish No.	#38	#35		
Mass of Sample + Dish Dry	483.4	517.8		
Tare	435.1	469.4		
M_s	48.3	48.4		
G_s	2.68	2.73		

M_{bws} = Mass of Flask + Water + Solids at $T^{\circ}C$

Average $G_s = 2.71$

M_{bw} = Mass of Flask + Water at $T^{\circ}C$ (From Calibration Curve for Flask)

M_s = Mass of Dry Soil

K = Correction factor based on the density of water at $20^{\circ}C$ (see Table IV-1 Lab Manual)

G_s = Specific Gravity of Solids =
$$\frac{M_s K}{M_s + M_{bw} - M_{bws}}$$

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SPECIFIC GRAVITY DETERMINATION

Project Jeff. Co. Job No. _____ Job Location I-40
Boring No. _____ Sample No. _____ Depth _____
Description of Soil USCS Classification = SC-CL
Tested By T. Johnson Date 3/31/86 Calculated By T. Johnson Date 4/2/86
C. Allin

K =	0.99626	0.99626		
Flask No.	# 2	# 2		
Method of Air Removal	aspiration	aspiration		
M _{bws}	698.0	698.0		
Temperature T °C	28°C	28°C		
M _{bw}	667.27	667.27		
Evap. Dish No.	HR 5	HR 6		
Mass of Sample + Dish Dry	477.8	471.7		
Tare	429.0	422.9		
M _s	48.8	48.8		
G _s	2.69	2.69		

- M_{bws} = Mass of Flask + Water + Solids at T °C
M_{bw} = Mass of Flask + Water at T °C (From Calibration Curve for Flask)
M_s = Mass of Dry Soil
K = Correction factor based on the density of water at 20°C (see Table IV-1 Lab Manual)
G_s = Specific Gravity of Solids = $\frac{M_s K}{M_s + M_{bw} - M_{bws}}$

Average G_s = 2.69

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SPECIFIC GRAVITY DETERMINATION

Project Cookeville Job No. _____ Job Location State Route #111
Boring No. _____ Sample No. _____ Depth _____
Description of Soil U.S.C.S. Classification = SM
Tested By T. Johnson Date 3/31/86 Calculated By T. Johnson Date 4/2/86
C. Allin

$K =$	0.99626	0.99611	0.99626	
Flask No.	# 3	# 6	# 3	
Method of Air Removal	aspiration	aspiration	aspiration	
M_{bws}	702.9	700.6	704.0	
Temperature T °C	28°C	28.5°C	28°C	
M_{bw}	669.2	666.6	669.2	
Evap. Dish No.	A-10	E-222	#172	
Mass of Sample + Dish Dry	478.2	465.6	480.5	
Tare	425.4	412.9	426.4	
M_s	52.8	52.7	54.1	
G_s	2.75	2.81	2.79	

- M_{bws} = Mass of Flask + Water + Solids at T °C
 M_{bw} = Mass of Flask + Water at T °C (From Calibration Curve for Flask)
 M_s = Mass of Dry Soil
 K = Correction factor based on the density of water at 20°C (see Table IV-1 Lab Manual)
 G_s = Specific Gravity of Solids = $\frac{M_s K}{M_s + M_{bw} - M_{bws}}$

Average $G_s = 2.78$

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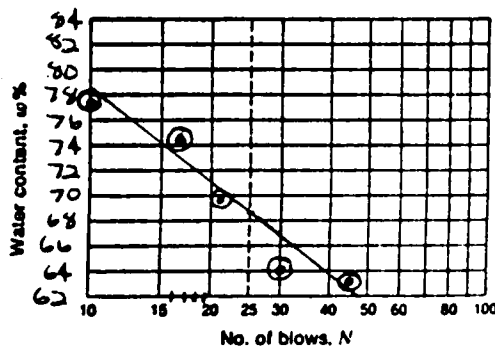
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Geotechnical Engineering Laboratory

ATTERBERG LIMITS-PLASTIC AND LIQUID

Project Rutledge Pike Job No. _____ Job Location State Route # 1
Doring No. _____ Sample No. _____ Depth _____
Description of Soil USCS Classification = SM
Tested By T. Johnson Date 2/21/86 Calculated By T. Johnson Date 2/22/86

Liquid Limit Determination

Determination No.	1	2	3	4	5
Tare No.	A7	C5	C7	C11	D9
Mass of Wet Soil + Tare	38.2	40.0	37.1	31.3	43.6
Mass of Dry Soil + Tare	30.9	31.8	30.3	26.3	35.2
Mass of Water	7.3	8.2	6.8	5.0	18.4
Mass of Tare	19.5	20.0	19.5	19.6	19.8
Mass of Dry Soil	11.4	11.8	10.8	6.7	13.4
Water Content (%)	64.0%	69.5%	63.0%	74.6%	77.6%
No. of Blows	30	21	45	17	10



Flow Index $F_i =$ _____
Liquid limit = 68.5
Plastic limit = 39.2
Plasticity index $I_p =$ 29.3

MH

Plastic Limit Determination

Determination No.	1	2	3	4
Tare No.	188	212	216	180
Mass of Wet Soil + Tare	22.5	19.7	19.3	18.4
Mass of Dry Soil + Tare	20.1	18.1	17.8	17.1
Mass of Water	2.4	1.6	1.5	1.3
Mass of Tare	14.1	14.1	13.8	13.8
Mass of Dry Soil	6.0	4.0	4.0	3.3
Water Content (w_{pl})	40.0%	40.0%	37.5%	39.4%

Average Plastic Limits 39.2%

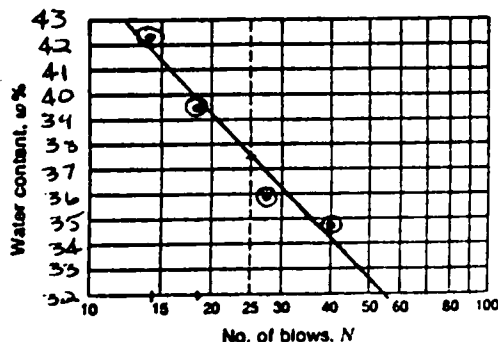
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ATTERBERG LIMITS-PLASTIC AND LIQUID

Project Jeff. Co. Job No. _____ Job Location I-40
 Boring No. _____ Sample No. _____ Depth _____
 Description of Soil USCS classification = SC-CL
 Tested By T. Johnson Date 2/21/86 Calculated By T. Johnson Date 2/22/86

Liquid Limit Determination

Determination No.	1	2	3	4	5
Tare No.	204	205	206	198	
Mass of Wet Soil + Tare	37.3	40.3	37.9	40.1	
Mass of Dry Soil + Tare	31.3	32.9	30.8	33.2	
Mass of Water	6.0	7.3	7.1	6.9	
Mass of Tare	14.0	14.1	14.0	14.0	
Mass of Dry Soil	17.3	18.8	16.8	19.2	
Water Content (%)	34.7%	39.4%	42.3%	35.9%	
No. of Blows	40	18	14	27	



Flow Index F_i = _____
 Liquid limit = 37.5
 Plastic limit = 18.3
 Plasticity index I_p = 19.2

(CL)

Plastic Limit Determination

Determination No.	1	2	3	4
Tare No.	170	174	177	
Mass of Wet Soil + Tare	20.0	20.6	21.8	
Mass of Dry Soil + Tare	19.1	19.6	20.5	
Mass of Water	0.9	1.0	1.3	
Mass of Tare	13.9	14.0	13.9	
Mass of Dry Soil	5.2	5.6	6.6	
Water Content (w_p)	17.3%	17.9%	19.7%	

Average Plastic Limits 18.3%

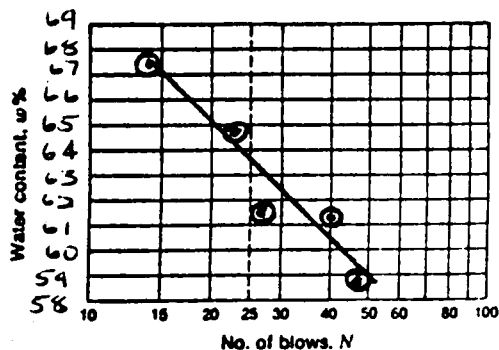
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ATTERBERG LIMITS-PLASTIC AND LIQUID

Project Cookeville Job No. _____ Job Location State Route #111
Boring No. _____ Sample No. _____ Depth _____
Description of Soil U.S.C.S Classification = SM
Tested By T. Johnson Date 2/20/86 Calculated By T. Johnson Date 2/21/86

Liquid Limit Determination

Determination No.	1	2	3	4	5
Tare No.	176	173	189	210	214
Mass of Wet Soil + Tare	30.2	29.0	29.0	34.3	34.8
Mass of Dry Soil + Tare	24.2	23.1	23.0	26.6	25.0
Mass of Water	6.0	5.9	6.0	7.7	6.8
Mass of Tare	14.0	14.0	14.1	14.7	13.9
Mass of Dry Soil	10.2	9.1	8.9	12.5	11.1
Water Content (%)	58.8%	64.8%	67.4%	61.6%	61.3%
No. of Blows	46	23	14	26	40



Flow Index $F_i = \text{---}$
Liquid limit = 63.6
Plastic limit = 36.5
Plasticity index $I_p = \text{---}$

(MH)

Plastic Limit Determination

Determination No.	1	2	3	4
Tare No.	176	173	189	210
Mass of Wet Soil + Tare	16.3	16.4	20.1	21.6
Mass of Dry Soil + Tare	15.7	15.3	15.5	19.5
Mass of Water	0.6	0.6	1.6	2.1
Mass of Tare	14.1	14.0	14.1	14.1
Mass of Dry Soil	1.6	1.3	4.4	5.4
Water Content (w_{pl})	37.5%	33.3%	36.4%	38.9%

Average Plastic Limits 36.5%

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WASHED
GRAIN SIZE ANALYSIS-MECHANICAL

Project Rutledge Pike Job No. _____ Job Location State Route #1

Boring No. _____ Sample No. _____ Depth _____

Description of Soil USCS classification = SM

Tested By T. Johnson Date 2/21/86 Calculated By T. Johnson Date 2/23/86

Sieve No. or Size	Size of Opening (mm)	Mass Retained	Percent Retained		Percent Finer By Mass
			Partial	Cumulative	
3 in.	76.2				
2 in.	50.8				
1 in.	25.4				
3/4 in.	19.1				
1/2 in.	9.52				
No. 4	4.75				
6	3.36				
8	2.38				
10 *	2.00	0.6 g	0.6%	0.6%	99.4%
20 *	0.84	0.8 g	0.8%	1.4%	98.6%
30	0.59				
40 *	0.42	28.9 g	28.9%	30.3%	
50	0.297				
70	0.210				
100 *	0.149	26.3 g	26.3%	56.6%	43.4%
140	0.105				
200 *	0.074	14.7 g	14.7%	71.3%	28.7%
Pan		28.7 g	28.7%	100.0%	

100.0 g

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WASHED
GRAIN SIZE ANALYSIS-MECHANICAL

Project Jeff. Co Job No. _____ Job Location I-40

Boring No. _____ Sample No. _____ Depth _____

Description of Soil USCS Classification = SC-CL

Tested By T. Johnson Date 2/22/86 Calculated By T. Johnson Date 2/24/86

Sieve No. or Size	Size of Opening (mm)	Mass Retained	Percent Retained		Percent Finer By Mass
			Partial	Cumulative	
3 in.	76.2				
2 in.	50.8				
1 in.	25.4				
3/4 in.	19.1				
1/2 in.	9.52				
No. 4	4.75				
6	3.36				
8	2.38				
10 *	2.00	0.1 g	0.170	0.170	99.970
20 *	0.84	2.1 g	2.170	2.270	97.870
30	0.59				
40 *	0.42	12.0 g	12.070	14.270	85.870
50	0.297				
70	0.210				
100 *	0.149	28.6 g	28.670	42.870	57.270
140	0.105				
200 *	0.074	11.3 g	11.370	54.170	45.970
Pan		45.9 g	45.970	100.070	

100.0 g

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WASHEO
GRAIN SIZE ANALYSIS-MECHANICAL

Project Cookeville Job No. _____ Job Location State Route #111

Boring No. _____ Sample No. _____ Depth _____

Description of Soil USCS classification = SM

Tested By T. Johnson Date 2/22/86 Calculated By T. Johnson Date 2/24/86

Sieve No. or Size	Size of Opening (mm)	Mass Retained	Percent Retained		Percent Finer By Mass
			Partial	Cumulative	
3 in.	76.2				
2 in.	50.8				
1 in.	25.4				
3/4 in.	19.1				
1/2 in.	9.52				
No. 4	4.76				
6	3.36				
8	2.38				
10 *	2.00	0.8 g	0.8%	0.8%	99.2%
20 *	0.84	1.9 g	1.9%	2.7%	97.3%
30	0.59				
40 *	0.42	12.3 g	12.3%	15.0%	85.0%
50	0.297				
70	0.210				
100 *	0.149	31.3 g	31.3%	46.3%	53.7%
140	0.105				
200 *	0.074	20.8 g	20.8%	67.1%	32.9%
Pan		32.9 g	32.9%	100.0%	

100.0 g

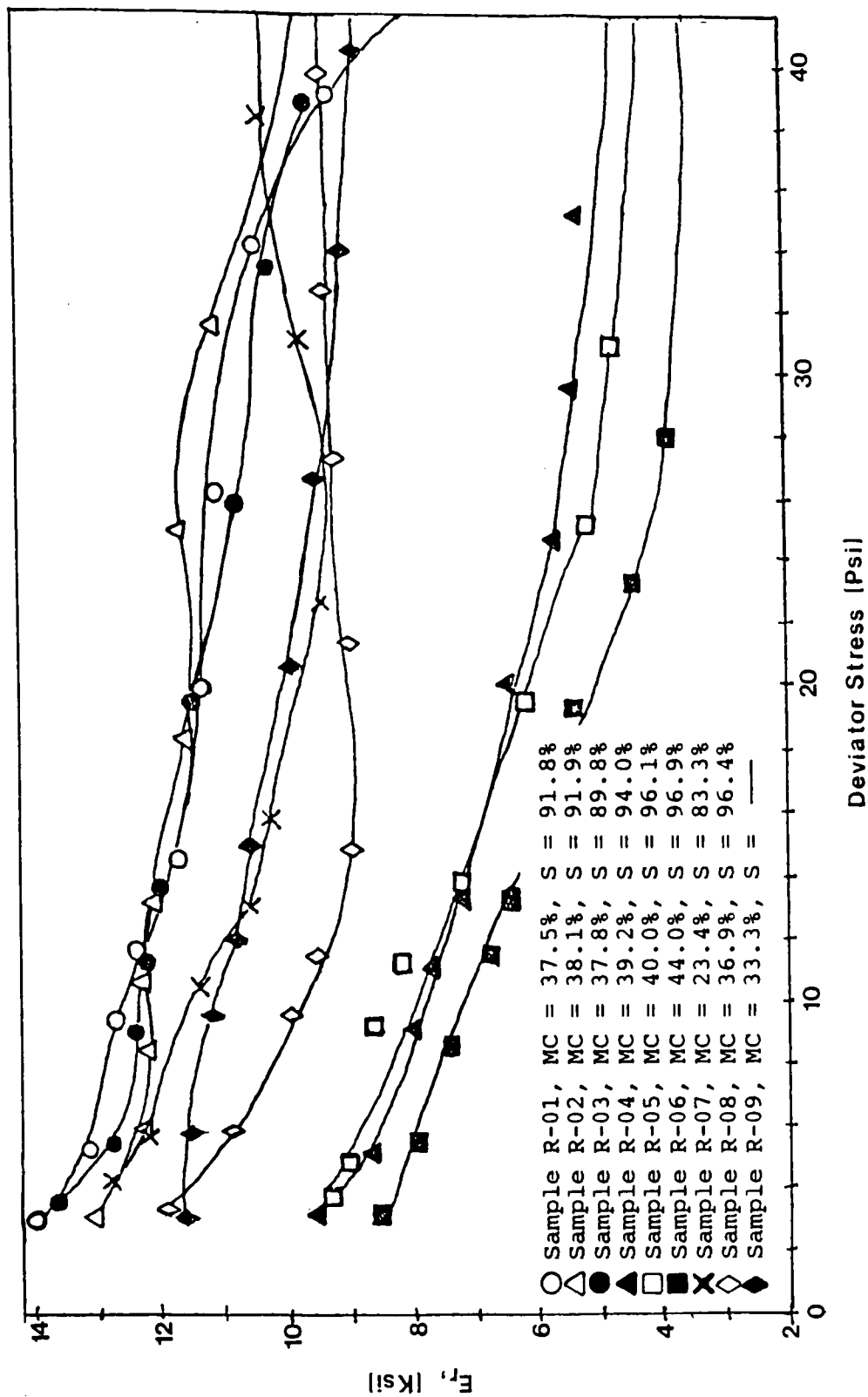


Figure 20. Plot of Resilient Modulus (E_r) versus Deviator Stress for All of the Rutledge Pike Samples.

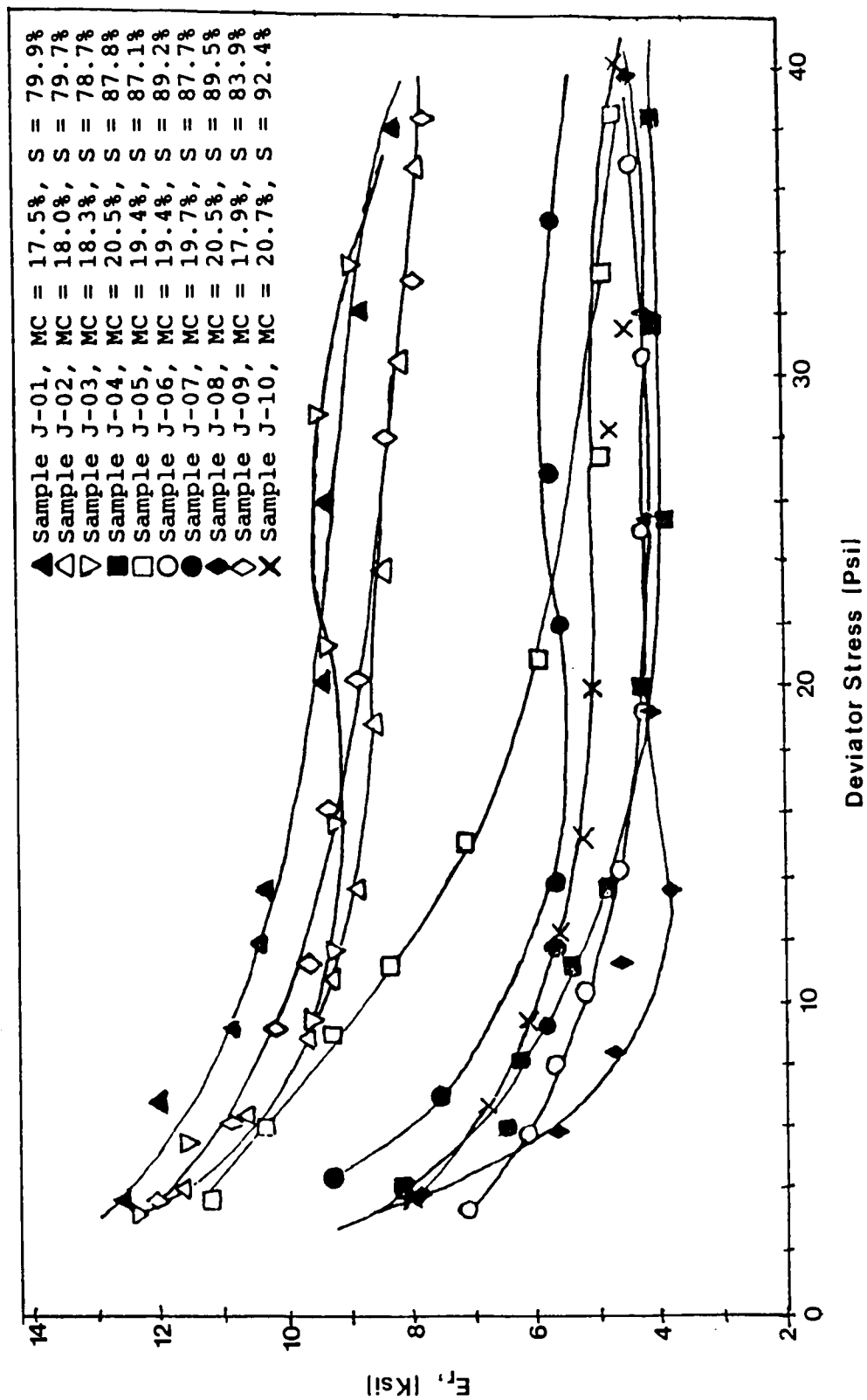


Figure 21. Plot of Resilient Modulus (E_r) versus Deviator Stress for All of the Jefferson County Samples.

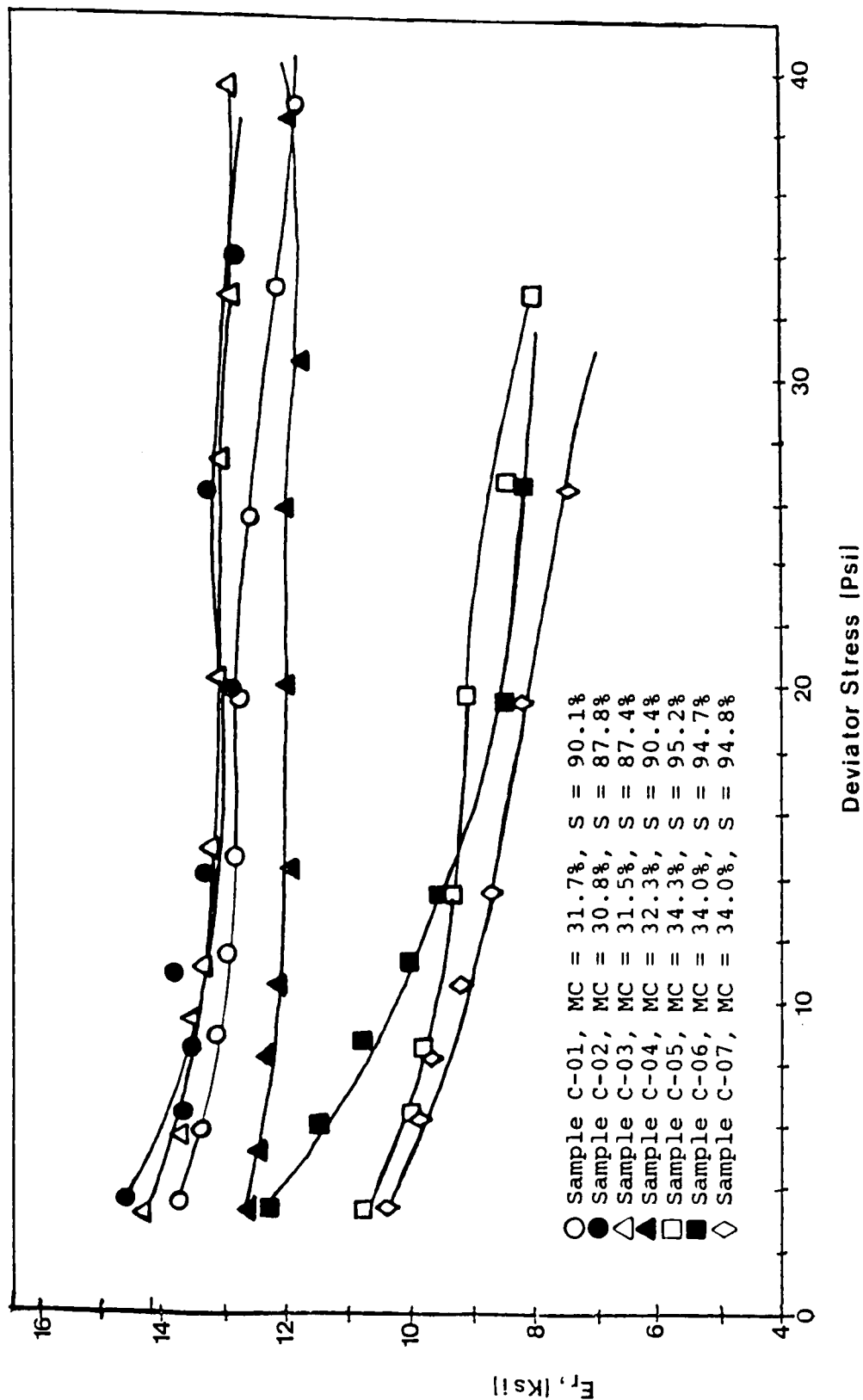


Figure 22. Plot of Resilient Modulus (E_r) versus Deviator Stress for All of the Cookeville Samples.

APPENDIX E

STANDARD ERROR CALCULATION FOR THE CURVFIT PROGRAM

Standard error is a measure of how well a predicted curve fits the actual data. Average standard error is calculated using the following equation:

$$\text{Standard Error} = \pm \sqrt{\frac{\sum (r^2)}{(N - M)}}$$

where: r = vertical distance from the predicted curve to the actual data point

M = the number of data points

N = the number of parameters.

The average standard error values obtained in this study of resilient modulus versus deviator stress data indicate a "good fit" of the predicted curves to the measured resilient modulus data. The calculated standard error values shown in Tables V, VI, and VII on pages 60 and 61 in Chapter VI of the text translate to variations of less than 5%.

APPENDIX F

PLATES

Photographs of the Rutledge Pike Permanent Test Site are given in this appendix. The wheel-mounted core driller is pictured while in operation at the Rutledge Pike site. Several pictures of the laboratory equipment including the kneading compactor, repeated load testing apparatus, and computer monitoring system are also included in this appendix.

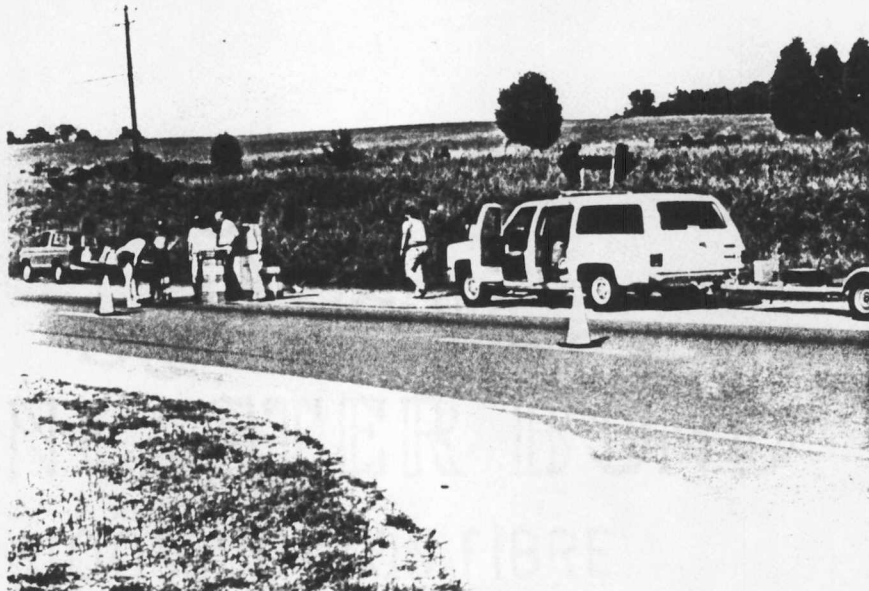


Plate 1. Permanent Test Site in Knox County on State Route 1 (Rutledge Pike).

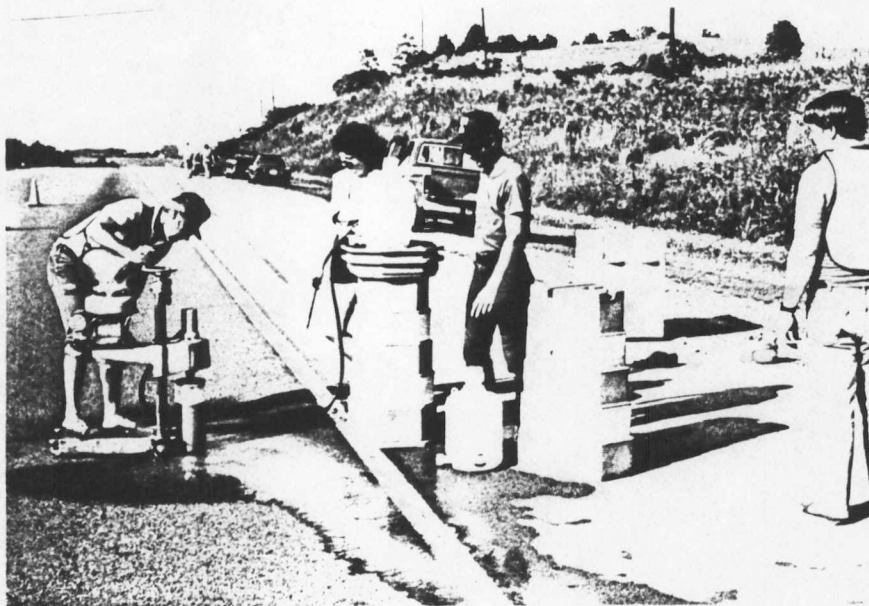


Plate 2. Core Driller Set-Up on Wheel Path Number One.

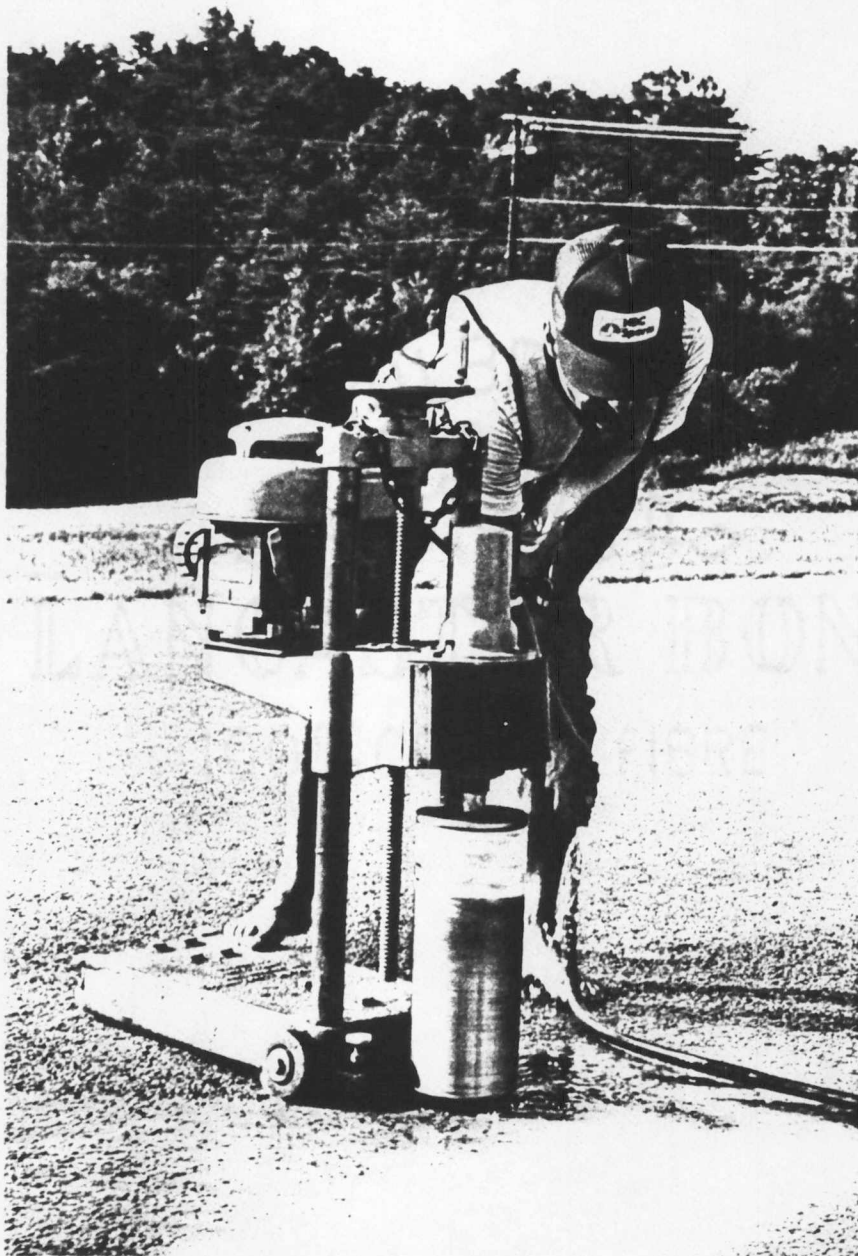


Plate 3. Wheel-Mounted Core Driller with Six-Inch Diameter Bit (Shown in Operation),

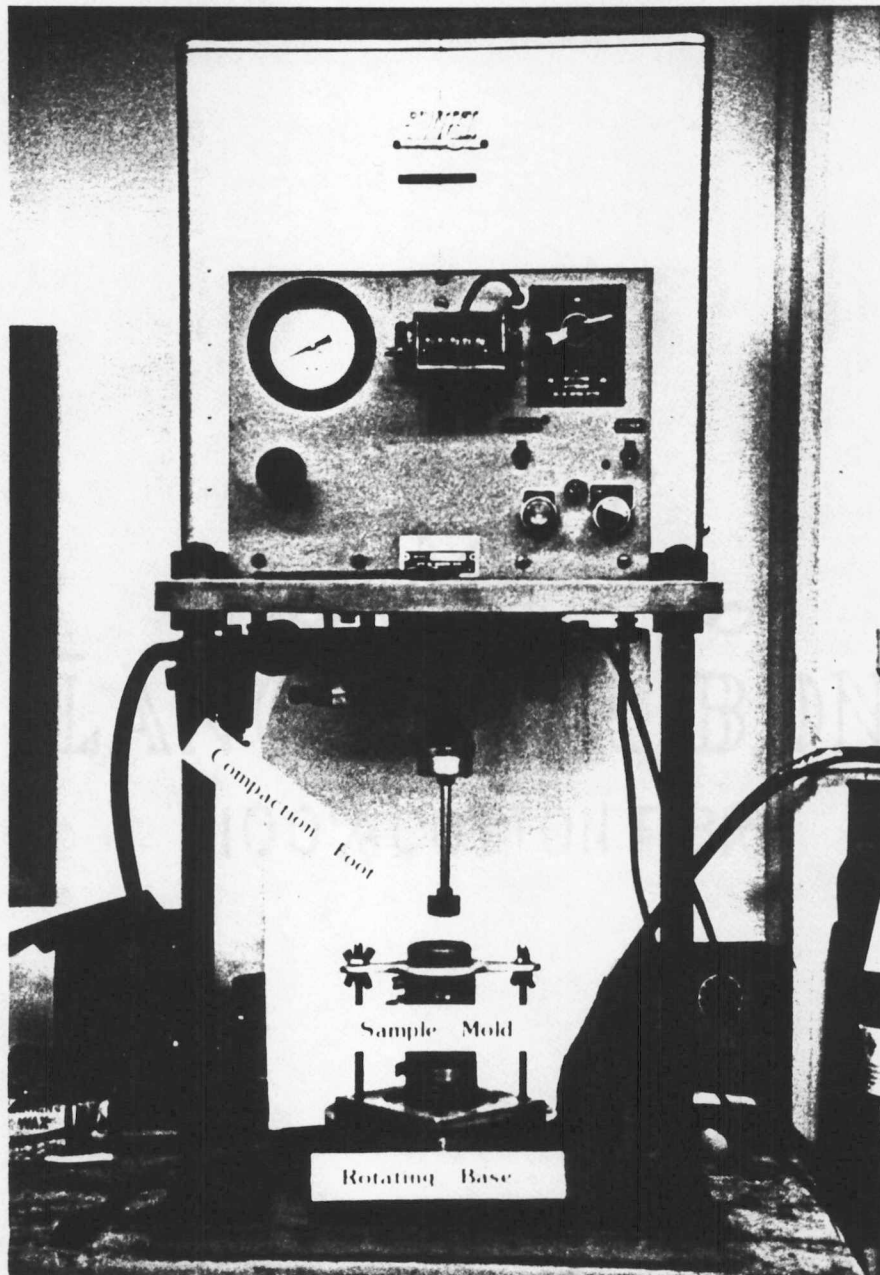


Plate 4. Kneading Compactor Used to Prepare Soil Samples.

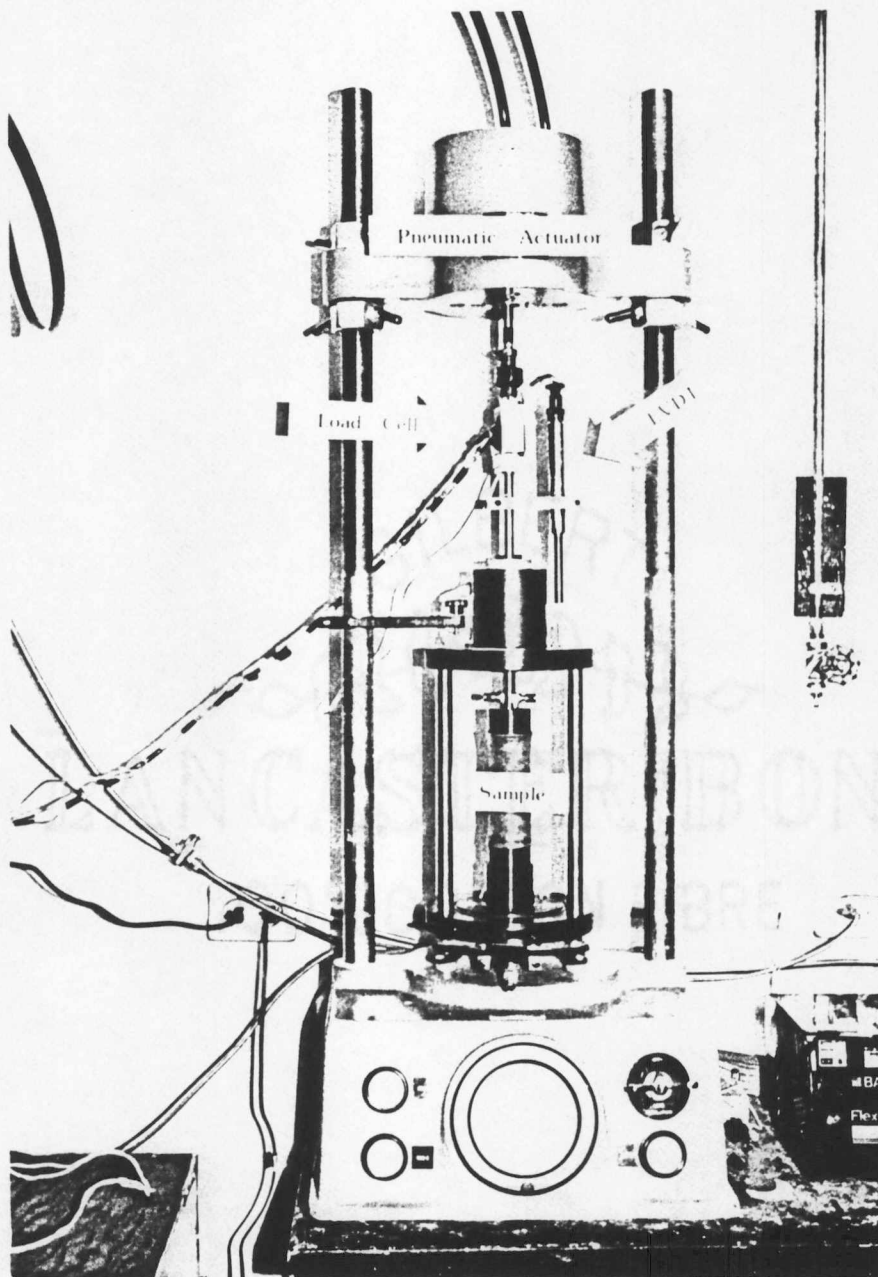


Plate 5. Repeated Load Testing Apparatus.

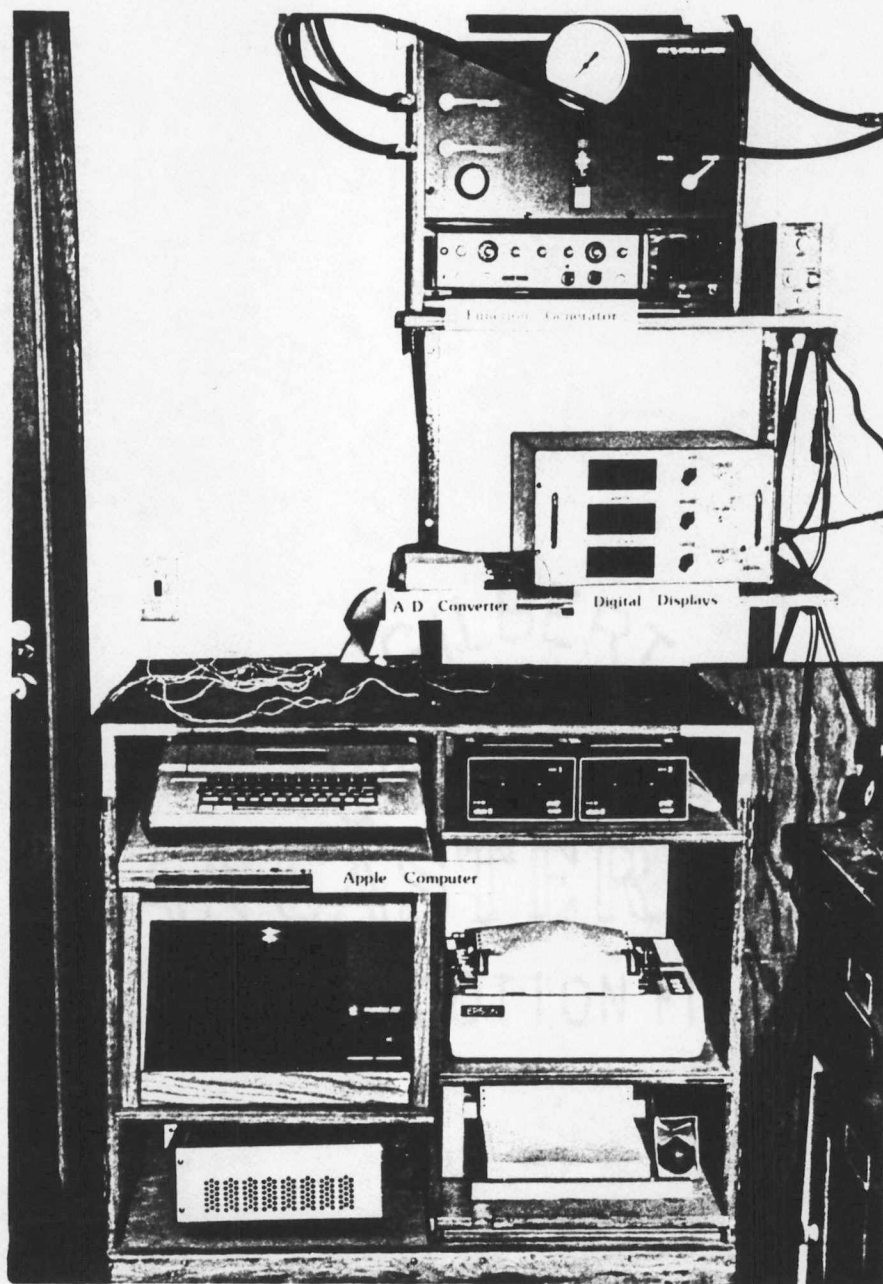


Plate 6. Computer monitoring System.

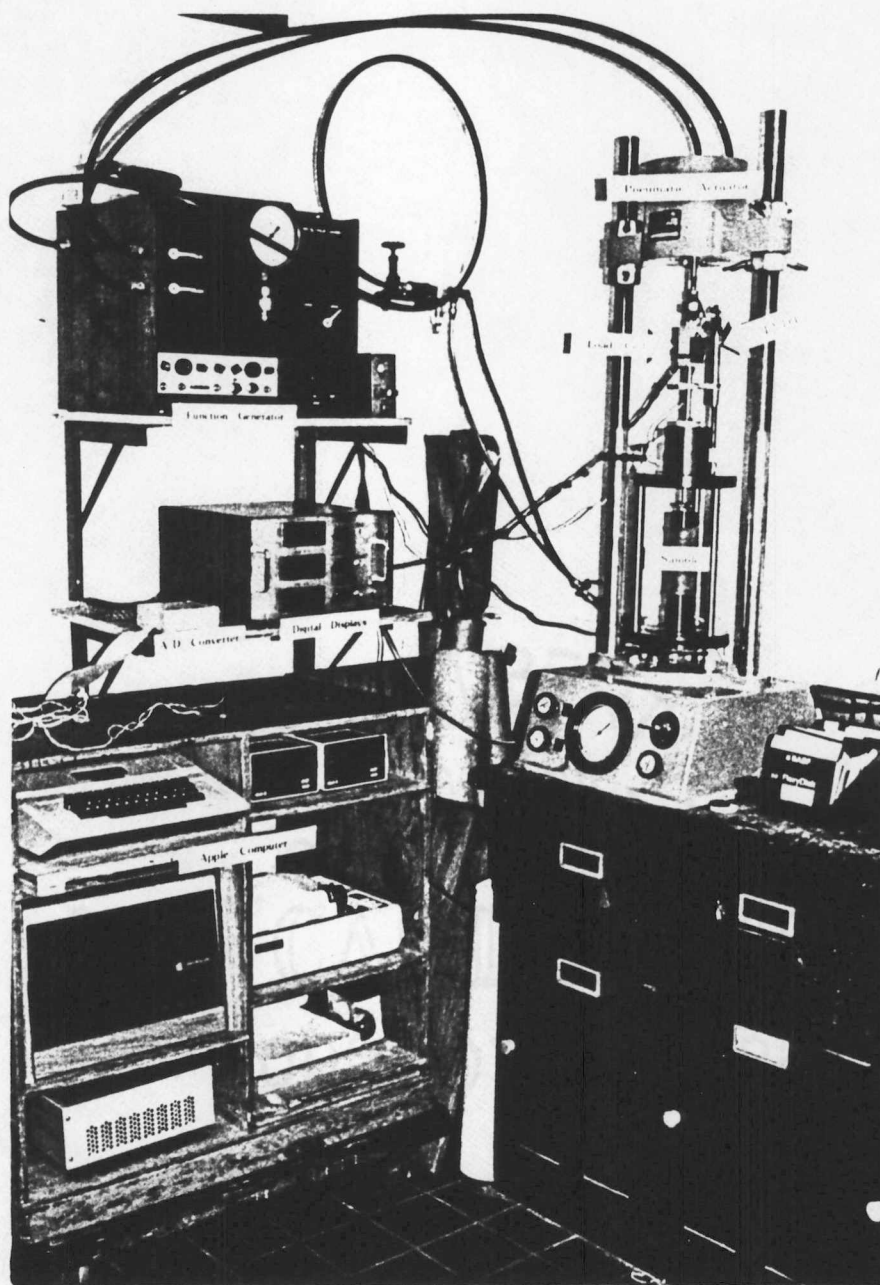


Plate 7. Complete Lab Set-Up with Repeated Load Testing Apparatus and Computer Monitoring System.

VITA

Teresa Dawn Johnson was born in Knoxville, Tennessee on May 1, 1961. She attended elementary schools in that city and graduated from Farragut High School in June 1979. The following September she entered Roane State Community College as a pre-engineering student. She transferred to the University of Tennessee, Knoxville in January 1982 and in June 1984 she received a Bachelor of Science degree in Civil Engineering. In June of 1984 she accepted a graduate research assistantship at the University of Tennessee and began study toward a Master's degree in Geotechnical Engineering. This degree was awarded in December 1986.

The author is a member of Tau Beta Pi, Chi Epsilon, and the American Society of Civil Engineers. As an undergraduate, she was elected secretary/treasurer of Chi Epsilon and vice-president of the A.S.C.E. student chapter. While attending graduate school, the author held the position of graduate research assistant on a research project funded by the Tennessee Department of Transportation.