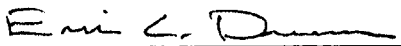
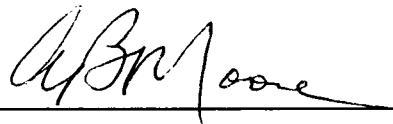


To the Graduate Council:

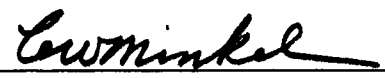
I am submitting herewith a thesis written by Jason Stuart Reeves entitled "Saturation Effects on the Resilient Response of Fine-Grained Soil." I have examined the final copy of this thesis for form and content and recommend that it be accepted in partial fulfillment of the requirements for the degree of Master of Science, with a major in Civil Engineering.


Dr. Eric C. Drumm, Major Professor

We have read this thesis
and recommend its acceptance:

Accepted for the Council:


Associate Vice Chancellor and
Dean of The Graduate School

STATEMENT OF PERMISSION TO USE

In presenting this thesis in partial fulfillment of the requirements for a Master of Science degree in Civil Engineering at the University of Tennessee, I agree that the Library shall make its contents available to borrowers under the rules of the Library. Brief quotations from this thesis are allowable without special permission, provided that accurate acknowledgement of the source is made.

Permission for extensive quotation from or reproduction of this thesis may be granted by my major professor, or in his absence, by the Head of Interlibrary Services when, in the opinion of either, the proposed use of the material is for scholarly purposes. Any duplication or use of the material in this thesis for financial gain shall not be permitted without my written permission.

Signature Jason S. Reeve

Date 4-28-95

Saturation Effects on the Resilient Response of Fine-Grained Soil

A Thesis

Presented for the

Master of Science

Degree

The University of Tennessee, Knoxville

Jason Stuart Reeves

May 1995

DEDICATION

This thesis is dedicated to my grandparents

Mrs. Betty Reeves and the late Mr. M. F. "Jack" Reeves

and

Dr. Donald Vogt and Mrs. Rose Marie Vogt

and to my parents

Mr. James Reeves and Mrs. Elizabeth Gebert

who have given me much support and encouragement.

ACKNOWLEDGEMENTS

I would like to thank Dr. Eric C. Drumm for providing me with the opportunity to further my education. His patience and understanding has won my respect and admiration. Also, I would like to thank Dr. Matthew Mauldon and Professor A.B. Moore for serving on my graduate committee. The Tennessee Department of Transportation provided me with generous financial support.

The laboratory research could not have been completed without the help of Mr. Curtis Allin, who generously allowed me to use the University of Tennessee geotechnical testing laboratory. Special thanks to Mr. Mark Madgett for his patience and advice on all aspects of the laboratory work, and to Mr. Ken Thomas for his modifications to the ATM device. Mr. Lee Spencer and Mr. X.Z. Wang were instrumental in creating the TNDOT program, and they also deserve special recognition.

Lastly, the greatest debt is owed to my fiance, Dawn Currie. Her love and support helped me to get to this moment.

LIST OF SYMBOLS AND ABBREVIATIONS

AASHO - American Association of State and Highway Officials

AASHTO - American Association of State and Highway Transportation Officials

ASTM - American Standards for Testing and Materials

CBR - California Bearing Ratio

D - Constrained modulus

LL - Liquid limit

LVDT - Linear variable differential transducer

M_r - Resilient modulus

PI - Plasticity index

PL - Plastic limit

SHRP - Strategic Highway Research Program

TNDOT - Tennessee Department of Transportation

USCS - Unified Soil Classification System

ϵ_{axial} - resilient or recoverable axial strain

σ_d - deviator stress

σ_c - confining pressure

σ_1 - major principal stress

σ_3 - minor principal stress

ν - Poisson's Ratio

Abstract

The 1993 AASHTO *Guide for the Design of Pavement Structures* uses resilient modulus to characterize the response of subgrade soils under vehicular loading. In order to determine a design resilient modulus, or effective roadbed resilient modulus, the pavement designer must be able to estimate variations in resilient modulus due to seasonal moisture change.

A laboratory investigation was performed on eight different Tennessee soils to determine which types of soils are most susceptible to decreases in resilient modulus with increasing moisture content, and to obtain relationships describing these decreases. The soils were tested in a cyclic triaxial testing device using the SHRP Protocol P46 procedure. Samples were prepared using a kneading compaction techniques at optimum moisture content and maximum dry density. In addition, some of the samples were later subjected to post-compaction saturation. The decrease in resilient modulus with increasing saturation was shown to follow a somewhat linear relationship. Methods to predict reductions in resilient modulus were proposed using the average slopes of these relationships.

The same eight soils were tested using the existing Alternative Test Method (Drumm et al. 1993). Samples were prepared at optimum moisture content and maximum dry density using the Standard Proctor procedure, with slight modifications. The ATM test results were compared with cyclic triaxial test results for the same soils at similar moisture-density conditions. For most of the soils, the difference in ATM test results relative to triaxial results was less than 25%.

Table of Contents

Chapter 1 Introduction

1.1 Importance of Resilient Modulus Testing	1
1.2 Mechanistic-Empirical Design Procedures	1
1.3 Previous Design Procedures	2
1.4 Definition of Resilient Modulus	3
1.5 Scope of the Research	6

Chapter 2 Literature Review

2.1 Factors Affecting Resilient Modulus	8
2.1.1 Effects of Increased Saturation on Resilient Modulus	9
2.2 Effects of the Moisture-Density State on Resilient Modulus	10
2.2.1 Influence of Compaction Method	11
2.2.2 Influence of Soil Structure	11
2.2.3 Influence of Increases in Water Content after Compaction	12
2.3 Triaxial Testing and Alternative Procedures for Estimating Resilient Modulus ...	12
2.3.1 Prediction Models for Samples Molded at Increased Saturation	14
2.3.2 Post-Compaction Saturated Soil Testing	17
2.3.3 Prediction Model for Post-Compaction Saturated Soils	19
2.3.4 Prediction Model Based on Field Seismic Testing	21
2.3.5 Alternative Test Method (ATM)	22
2.4 Previous TNDOT Research	22

2.4.1	Soil Sampling	22
2.4.2	Index Properties and Moisture-Density Relationships	23
2.4.3	Triaxial Testing Procedures	23
2.5	Subgrade Resilient Modulus in Flexible Pavement Structural Design	23
2.5.1	Asphalt Institute Method	28
2.5.2	AASHTO Method	29
 Chapter 3 Triaxial Testing and Effects of Post-Compaction Saturation		
3.1	Cyclic Triaxial Testing	31
3.1.1	Testing at Optimum Moisture Content and Maximum Dry Density	31
3.2	Effects of Seasonal Moisture Changes on Subgrade Resilient Behavior	34
3.2.1	Post-compaction Variation in Moisture Content vs. Compaction at Various Moisture Contents	35
3.3	Cyclic Triaxial Testing of Post-Compaction Saturated Samples	36
3.3.1	Slopes of the Resilient Modulus vs. Degree of Saturation Curves	39
3.3.2	Procedure for Determining Design Values for M_r at Increased Degree of Saturation	41
3.3.3	Statistical Model for M_r vs. Degree of Saturation Slopes	44
3.3.4	Example for Determining Resilient Modulus at Increased Post-Compaction Saturation	45
3.4	Summary	49

Chapter 4 Improvements to the ATM and Verification of ATM Test Results

4.1 Purpose of the ATM Procedure	50
4.2 Improvements to the ATM Procedure	50
4.3 Test Results for Samples Compacted at Optimum Moisture Content and Maximum Dry Density	51
4.3.1 Variability of Resilient Poisson's Ratio with Stress Ratio	52
4.3.2 ATM Resilient Modulus Calculations	53
4.3.3 Testing the Validity of the ATM Procedure	54
4.3.4 Analysis of the Test Results	56

Chapter 5 Summary and Conclusions

5.1 Resilient Modulus Testing and Future Research	60
---	----

References	62
-------------------------	----

Appendices

Appendix A	67
Appendix B	110
Appendix C	136
Vita	141

List of Figures

Figure 1-1. Decrease in M_r with Increasing Deviator Stress	4
Figure 1-2. Typical Variation in Resilient Modulus with Increasing Deviator Stress (Fine-Grained Soil)	5
Figure 2-1. Location of All Soils Used for TNDOT M_r Project	24
Figure 2-2. Moisture-Density Relationship for Phase I and II Soils	25
Figure 3-1. Typical M_r vs. σ_d Response for Optimum Moisture Content and Maximum Dry Density	32
Figure 3-2. Highest Resilient Modulus Values for the Eight Soils Tested	33
Figure 3-3. Lowest Values of Resilient Modulus for the Eight Soils Tested	34
Figure 3-4. Typical Triaxial Results for Post-Compaction Saturation Resilient Modulus	38
Figure 3-5. Typical Effects of Post-Compaction Moisture Change on Resilient Modulus	39
Figure 3-6. Typical Effects of Post-Compaction Saturation on Resilient Modulus	40
Figure 3-7. Resilient Modulus vs. Changes in Degree of Saturation	40
Figure 4-1. Typical ATM Results	52
Figure 4-2. Triaxial and ATM M_r vs. Deviator Stress	55
Figure 4-3. Predicted M_r From ATM Tests (Weight = 2.5 lb)	57
Figure 4-4. Predicted M_r From ATM Tests (Weight = 3.4 lb)	57
Figure 4-5. Predicted M_r From ATM Tests for All Soils	58

List of Tables

Table 2-1. Test Procedures for Index Testing and Moisture-Density	25
Table 2-2. Classification and Index Properties of Phase I and II Soils	26
Table 2-3. Equipment Setup for SHRP Protocol P46 (SHRP 1989)	27
Table 2-4. Specimen Compaction Specifications for SHRP Protocol P46	27
Table 2-5. Testing Procedure for SHRP Protocol P46	28
Table 3-1. Slopes of M_r vs. Degree of Saturation Curves	43
Table 3-2. Atterberg Limits with Slopes of M_r vs. Degree of Saturation Curves	43
Table 3-3. Comparison of Predicted Slopes and Measured Slopes	45
Table 3-4. Comparison of Calculated Resilient Moduli at Increased Saturation	46
Table 4-1. Comparison of ATM and Triaxial Test Results	59

Chapter 1 Introduction

1.1 Importance of Resilient Modulus Testing

The mechanical properties of all the components of a flexible pavement system must be estimated in order to create a design that is both effective and economically feasible. A typical flexible pavement system is composed of several layers, including an asphaltic concrete layer, a granular base layer and possibly subbase layers, and the soil subgrade. The performance, or serviceability, of these pavement systems has been shown to be largely dependent upon the magnitude of surface deflections (Seed et al. 1962; Thompson and Robnett 1976). These "resilient deflections" are responsible for fatigue type failures in asphaltic concrete surface layers. According to Elliot and Thornton (1986), 60 to 80% of these deflections take place in the subgrade. Thus, an understanding of the mechanical properties of the subgrade is a key part of the pavement design process.

A useful way to estimate the performance of the subgrade is to measure its resilient modulus. The resilient modulus is a measure of the elastic stiffness of a material subjected to cyclic loading. A high subgrade resilient modulus will result in lower pavement deflections under cyclic loading, whereas a low subgrade resilient modulus will result in larger pavement deflections. Therefore, the subgrade resilient modulus must be estimated in order to create pavement systems that decrease surface deflections, and to ultimately increase the service life of the pavement.

1.2 Mechanistic-Empirical Design Procedures

Mechanistic methods of design refer to the ability to apply analytical calculations

to pavement design to estimate pavement performance. These design procedures assume that a pavement can be modeled as a multi-layered elastic or visco-elastic structure resting on a elastic or visco-elastic subgrade. Using analytical techniques, it is possible to estimate stress and strain at points within or below the pavement structure. However, all aspects of pavement performance cannot be analyzed with purely analytical techniques. Therefore, it is necessary to calibrate the pavement models with observations of performance, or empirical techniques. Researchers believe that mechanistic-empirical design procedures more accurately describe pavement performance than the purely empirical techniques that were once used. According to the 1993 AASHTO *Guide for the Design of Pavement Structures* (AASHTO 1993), three primary benefits are associated with mechanistic-empirical design procedures:

- 1.) improved reliability for design
- 2.) ability to predict specific types of distress
- 3.) the ability to extrapolate from limited field and laboratory results

1.3 Previous Design Procedures

The 1972 AASHO *Interim Guide for the Design of Pavement Structures* described a "soil support" scale for pavement design that was developed using engineering judgement and data from the AASHO Road Test (Elliot and Thornton 1988). This design procedure was based on pavement tests for a limited number of subgrade soils, and obviously did not address the variability of subgrade stiffnesses (Huang 1993). Since the AASHO Road Test did not address all of the variables that affect pavement performance, it was apparent that a new analytical basis for analysis and a

unified method of testing to further quantify subgrade response was needed (Elliot and Thornton 1988).

With the empirical pavement design procedure, subgrade performance was typically evaluated using California Bearing Ratio (CBR) tests. According to Huang (1993), the CBR-based procedures do not adequately predict the response of subgrade soils under repeated loading. Indeed, Thompson and Robnett (1976) found that no relationship existed between CBR and resilient modulus for a series of tests on Illinois soils, and some soils with similar CBR values exhibited significantly different resilient responses. A similar finding was reported by Drumm et al. (1993). Newer mechanistic-empirical design methods, such as the methods described in the 1986 and 1993 AASHTO *Guide for the Design of Pavement Structures*, require the use of the resilient modulus test to quantify subgrade behavior.

1.4 Definition of Resilient Modulus

The resilient modulus, M_r , is similar to Young's Modulus, E . The difference between the two properties stems from the fact that M_r is determined by cyclic loading, whereas E is usually determined from a constant loading rate. Resilient modulus is defined as the slope of the deviator stress-axial strain curve (AASHTO: "Methods" 1982), or:

$$M_r = \frac{\sigma_d}{\epsilon_{axial}}$$

where:

M_r = resilient modulus

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

σ_1 = major principal stress (axial stress)

σ_3 = minor principal stress (confining pressure)

ϵ_{axial} = the resilient or recoverable axial strain

In cyclic triaxial M_r testing, a specimen of soil is initially subjected to hydrostatic compression ($\sigma_1 = \sigma_3$). The test specimen is then loaded cyclically, during which increments of axial stress, or deviator stress, σ_d , are applied to the specimen. The deviator stress is equal to the difference between the major principal stresses ($\sigma_1 - \sigma_3$). Figure 1-1 is a schematic of the loading and unloading curve for a specimen subjected to cyclic loading. When loading begins, the axial strain is composed of elastic and non-elastic components. However, as the number of load repetitions increases, the non-

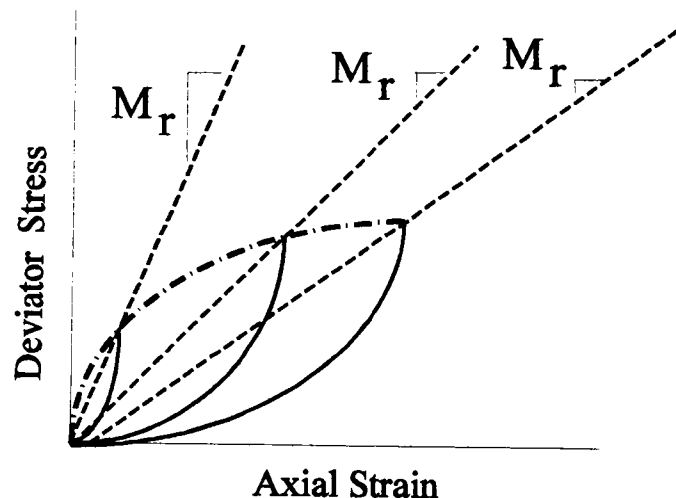


Figure 1-1. Decrease in M_r With Increasing Deviator Stress

elastic strain components become progressively smaller, and the load-unload loop stabilizes. The resulting secant modulus of the axial stress-strain relationship is defined as the resilient modulus, M_r .

The resilient modulus is highly dependent on the magnitude of the deviator stress, as was shown in Figure 1-1. Typically, as the deviator stress increases, the resilient strain increases at an increasing rate. As the magnitude of the resilient strain increases, there is a corresponding decrease in resilient modulus. Resilient modulus results are often plotted versus deviator stress (Thompson and Robnett 1976; Hudson 1992; Drumm et al. 1993; Madgett 1994). Figure 1-2 shows a schematic of the M_r vs. σ_d curve for a typical fine-grained soil. This reduction in resilient modulus, or "stress-softening", is common for fine-grained soils tested under cyclic loading. In contrast, the resilient modulus of coarse-grained materials is typically plotted versus the sum of the principal stresses. Coarse-grained soils typically show an initial decrease in resilient in M_r , and then a

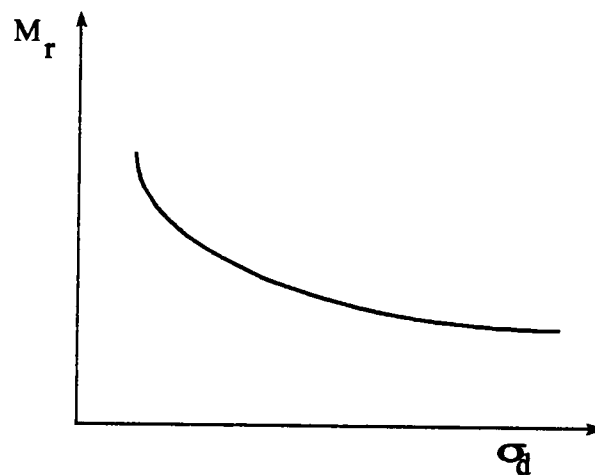


Figure 1-2. Typical Variation in Resilient Modulus with Increasing Deviator Stress (Fine-Grained Soil)

gradual increase in M_r with increasing stress. This apparent increase in stiffness for coarse-grained materials is due to rearrangements and densification of material with increases in the number of load applications, and this type of behavior is referred to as "stress-hardening".

The deviator stress applied to the samples is analogous to the cyclic stresses applied during vehicular loading. If the thickness of an asphalt pavement layer or subbase layers is decreased, the stress applied to the subgrade is increased and larger deflections occur. By increasing the thickness of one or more of these layers, the amount of stress applied to the subgrade can be lowered, thus the magnitude of deflections in the subgrade is reduced (Witzac and Yoder 1975). Elliot and Thornton (1988) stated that for typical flexible pavement designs (using a standard 18-kip single axle load) the deviator stress applied to the subgrade is generally in the range of 13.8 to 55.2 kPa (2 to 8 psi). Therefore, that portion of the resilient modulus curve between these two stress states is of the most interest to pavement designers, and the variation in resilient modulus with stress level should be incorporated into pavement design methods.

1.5 Scope of the Research

The standard method for testing resilient modulus is the cyclic triaxial procedure. Cyclic triaxial testing is an expensive and time-consuming testing procedure. Indeed, many agencies find that they cannot afford to perform resilient modulus testing on a production basis. Further complications arise when all of the factors that affect resilient modulus such as climatic or "environmental" factors are taken into account. The primary environmental factors influencing subgrade resilient modulus are freeze-thaw cycles and

changes in subgrade moisture content, or saturation, which generally lead to a decrease in resilient modulus. It is obvious that there is not only a need for easier procedures to estimate resilient modulus, but also methods to predict changes in resilient modulus due to environmental factors. Researchers have developed various alternative techniques to estimate subgrade resilient response, including predictive models and alternative testing procedures. Some of the more recent techniques are described in the following chapter. However, the primary purpose of this investigation is twofold: to develop methods to predict the reduction in resilient modulus with increasing saturation, and to make improvements and verify the existing Alternative Test Method (ATM) (Drumm et al. 1993). It is hoped that these findings will allow pavement designers to make reasonable estimates of resilient modulus, and thus lead to better road and highway serviceability.

Chapter 2 Literature Review

2.1 Factors Affecting Resilient Modulus

The resilient modulus of fine-grained soils is a highly variable material parameter, because it is dependent upon many different factors. In order to quantify subgrade response, certain service conditions should be considered including: stress state (dependent on loading), environmental conditions (moisture and temperature), and construction conditions (water content and dry density) (Monismith 1991). Elliot and Thornton (1986) discussed several of the factors that effect the resilient response of subgrade soils. These factors include the stress duration, frequency of loading, particle grain-size, degree of saturation, stress state, and freeze-thaw cyles. Seed et al. (1962) discussed the effects of the moisture-density state, the number of load applications, and specimen age after compaction. Thompson and Robnett (1976,1979) investigated the effects of different compactions methods and changes in degree of saturation. Based on the results from these researchers, the following general statements about fine-grained soils can be made:

- 1.) The stress duration and rate of loading have significant effects on resilient modulus.
- 2.) Resilient modulus decreases with increasing deviator stress.
- 3.) Denser samples of the same soil tend to have higher resilient moduli.
- 4.) The compaction moisture content has a significant influence on a soil's structure and therefore the resilient response.
- 5.) The moisture-density state can be described in terms of degree of saturation.

- 6.) Resilient modulus decreases with increasing degree of saturation.
- 7.) Resilient modulus varies with time after compaction due to thixotropic changes in soil structure.
- 8.) The effect of confining pressure varies with soil type, and is less important for high plasticity soils.
- 9.) Freeze-thaw cycles cause a decrease in resilient modulus.

Mechanistic-empirical design procedures, such as the AASHTO *Guide for the Design of Pavement Structures* (AASHTO 1986,1993) provide procedures to account for variations in subgrade resilient modulus due to freeze-thaw cycles and changes in seasonal moisture content, or saturation. The effect of freeze-thaw action varies in different geographical regions, and it is less significant in some of the southeastern states, such as Tennessee (Huang 1993). Typically, freeze-thaw action is accounted for by assuming conservative M_r values for frozen and thawed subgrade conditions. However, over a major portion of the year, subgrades are subjected to changes in resilient modulus due to variations in saturation. Thus, an understanding of how resilient modulus varies with degree of saturation is essential in using these design procedures.

2.1.1 Effects of Increased Saturation on Resilient Modulus

Rainfall that penetrates pavement structures or the roadbed soil influences the properties of pavement materials. Increases in subgrade moisture can be controlled to some degree by good drainage design practices, but these effects cannot be totally eliminated (AASHTO 1993). Several researchers have studied the effects of the moisture-density relationship on resilient modulus (Seed et al. 1962; Thompson and

Robnett 1976,1979; Elliot and Thornton 1988; Hudson 1992; Pezo and Hudson 1994; Li and Selig 1994; Madgett 1994; Thadkamalla and George 1995). The moisture density relationship can also be considered in terms of degree of saturation (Thompson and Robnett 1976,1979). In general, the resilient modulus of fine-grained soils decreases as the degree of saturation increases (Seed et al. 1962; Thompson and Robnett 1976, 1979; Hudson 1993). Traditionally, the effect of increase in saturation on resilient modulus has been determined by molding soil samples wet of optimum.

Another approach for quantifying the effects of increased saturation is post-compaction saturation. Seed et al. (1962) investigated the effects of soaking samples after compaction. Madgett (1994) and Thadmakalla and George (1995) investigated these effects by saturating compacted samples by backpressure saturation.

Researchers continue to test samples both molded at elevated degrees of saturation, and samples subjected to post-compaction saturation. The question is which specimen preparation method best represents the conditions in the field? This question can be answered by considering the influence of soil structure due to the moisture-density state.

2.2 Effects of the Moisture-Density State on Resilient Modulus

Seed et al. (1962) and Thompson and Robnett (1976, 1979) investigated the relationship between resilient modulus and a soil's moisture-density state. Seed et al. (1962) stated, "By the use of an appropriate method of sample preparation and conditioning, based on a thorough understanding of the properties of compacted soils, it should now be possible to closely simulate in the laboratory any desired field condition

of a soil." A discussion of the influence of these sample preparation methods, relating to the moisture-density state and soil structure, is given in the following sections.

2.2.1 Influence of Compaction Method

Seed et al. (1962) found that the method of compaction has a "profound" influence on the arrangement of soil particles, or soil structure, for compacted clays. It was stated that samples compacted by kneading compaction at low saturation, or "dry" of the line of optimums, show a random, or "flocculated", structure. However, when soils are compacted "wet" of the line of optimums, near 85% saturation or higher, the particles take on a parallel structure. For the latter case, soils compacted by kneading compaction are subjected to shear deformations near the compaction foot. Clay particles tend to align themselves parallel to the surfaces of shear, resulting in a parallel particle arrangement (Seed et al. 1962; Monismith 1991).

It is interesting to note that clays compacted by static compaction retain a flocculated structure, regardless of whether the soils are compacted dry or wet of the line of optimums. This is because the soil is compacted within a mold, with a uniform static pressure over the soil surface. Very little lateral displacement is possible in this confined condition, therefore shear deformations are minimal. Because there is essentially no shear deformation, the soil retains its flocculated structure (Seed et al. 1962). This static compaction is recognized by AASHTO (AASHTO "Methods" 1982) as a sample preparation method resilient modulus testing.

2.2.2 Influence of Soil Structure

Seed et al. (1962) found that kneading compacted samples have similar

mechanical responses to "untrafficked" soils compacted by rubber tired rollers in the field. However, Lee et al. (1992) suggested that differences may exist between kneading compacted samples and field rubber tire compacted samples, primarily due to different patterns of porosity, or soil fabric.

2.2.3 Influence of Increases in Water Content after Compaction

In order to investigate the influence of increases in subgrade moisture content, or saturation, the soil structure must be considered. Seed et al. (1962) suggested that soils compacted in the field will retain their flocculated structure even after subsequent increases in saturation. It was found that resilient deformations of kneading compacted samples soaked to a high degree of saturation were much smaller than samples compacted by kneading compaction to the same final saturation.

Monismith (1991) presented data comparing the resilient strain from both of these sample preparation methods. Indeed, the resilient strain for the latter case was approximately 500% higher than post-compaction saturated samples at 200 load applications, and was 400% higher at 10,000 load applications. Therefore, the resilient modulus predicted for samples compacted to a final saturation was significantly lower. Thus, it seems apparent that post-compaction saturated specimens may better represent field subgrade conditions, and molding samples at higher saturations may give overly conservative resilient modulus results.

2.3 Triaxial Testing and Alternative Procedures for Estimating Resilient Modulus

Resilient modulus testing is an expensive and time-consuming process. Research has been focused on finding ways to correlate basic soil parameters with

resilient modulus. One commonly used correlation was based on California Bearing Ratio (CBR) values (Heuklelom and Klomp 1962). It was suggested that resilient modulus could be estimated by the following relationship:

$$M_r = 1500 \times \text{CBR}$$

where:

M_r = resilient modulus (psi)

CBR = California Bearing Ratio (%)

This correlation has been shown to be an unreliable prediction method (Thompson and Robnett 1976; Drumm et al. 1993a). Thus, the need for verification of these prediction methods, and an understanding of the applicability of these methods is apparent.

Since the introduction of the 1986 AASHTO *Guide for the Design of Pavement Structures* (AASHTO 1986), one of the main focuses in resilient modulus research has been on developing testing procedures and modelling techniques that incorporate the effects of increases in subgrade moisture content. As previously mentioned, there are two distinctly different methods for preparing compacted samples: compacting samples to some final degree of saturation or subjecting the samples to post-compaction saturation (Seed et al. 1962). Some of the more recent literature on these methods is described below.

Development of simplified testing procedures to predict resilient modulus is becoming increasingly popular. Two recent methods, one based on field seismic testing (Pezo and Hudson 1994) and the other based on an analytical model with a single degree-

of-freedom lumped mass and spring system (Drumm et al. 1993a), are described below.

2.3.1 Prediction Models for Samples Molded at Increased Saturation

Thompson and Robnett (1979) developed regression equations from laboratory test data to show the decrease in resilient modulus with increasing degree of saturation for various fine-grained Illinois soils. Individual models were developed for various fine-grained soil classifications. The equations take the following general form:

$$E_{Ri} = a + bS_r$$

where:

E_{Ri} = "break point" resilient modulus

a, b = regression constants

S_r = degree of saturation as a percentage

The decrease in M_r with deviator stress was assumed to follow a bilinear relationship.

The slope of the two linear portions of the assumed relationship was assumed to be

identical for all Illinois soils. The "breakpoint" resilient modulus, E_{Ri} , which

corresponds to the junction of the two linear portions at a deviator stress of 6 psi, was

assumed to vary with soil type and consistency. These relationships were shown to

correspond well with a wide range of Illinois soils. It is obvious that the prediction

equations were highly dependent upon the degree of saturation of the soil. The use of the

degree of saturation as a primary input into these equations can be considered a

limitation, since the degree of saturation of a soil is dependent on the soil's specific

gravity, and specific gravity is difficult to measure with certainty.

Pezo and Hudson (1994) conducted a series of M_r tests on Texas soils. Their primary focus of interest was how different parameters such as the PI, the soil, the moisture condition, and specimen age affect M_r values. The soils were divided into subgroups corresponding to different ranges of PI. The specimens were tested using the SHRP Protocol P46 test method. Soil specimens were tested at different water contents and specimen ages. Pezo and Hudson found that the confining pressure appears to not have much effect on M_r for high-PI soils. Specimens that were aged tended to have higher resilient moduli than similar specimens tested soon after preparation. Increases in M_r in the older specimens was most pronounced for high-PI soils. It was also found that the M_r of the high-PI soils has a greater variability with small increases in water content.

Pezo and Hudson (1994) proposed a model to predict values for resilient modulus of nongranular soils. They formed a multilinear regression model that gives an estimate of M_r based on basic properties of the soil. It is given in the following form:

$$M_R = F_0 \cdot F_1 \cdot F_2 \cdot F_3 \cdot F_4 \cdot F_5 \cdot F_6$$

The F factors correspond to correction functions which are dependent on moisture content, percent of maximum dry density, plasticity index, specimen age, confining pressure, and deviator stress, respectively. According to the authors, the model described the experimental data well, but it was pointed out that "its applicability only falls within the ranges of collected data."

Li and Selig (1994) have developed a procedure to estimate the effects of soil physical state, soil stress state, and soil type on resilient modulus. They have proposed that any change in a soil's physical state can be represented by two paths: constant dry

density and constant compactive effort. In order to use this procedure, a value of M_r for the soil in question must be determined at optimum water content. This can be determined from direct testing or from the use of a predictive model. Once a value of M_r at optimum water content is obtained, a value of M_r at another physical state can be determined by use of the following relationships:

$$R_{mi} = \frac{M_r}{M_{r(opt)}}$$

constant dry density: $R_{m1} = f_1(w - w_{opt})$

constant compactive effort: $R_{m2} = f_2(w - w_{opt})$

where:

M_r = predicted resilient modulus at soil state 2

$M_{r(opt)}$ = reference resilient modulus at optimum water content

w = water content for soil state 2

w_{opt} = optimum water content

R_{m1} corresponds to a predicted value of resilient modulus following a horizontal line of constant dry density on a compaction curve, while R_{m2} corresponds to a predicted value of resilient modulus following a curve of constant compactive effort on a compaction curve. The relationships for R_{m1} and R_{m2} are based on polynomial regressions. Li and Selig used this procedure to predict M_r values at higher water contents (higher saturations) from known M_r values at optimum moisture content for Vicksburg clay samples. The predicted values using the new procedure corresponded well with the

actual test data. The researchers pointed out that these models are based on test data from soils compacted to a final degree of saturation.

2.3.2 Post-Compaction Saturated Soil Testing

Elfino and Davidson (1989) conducted resilient modulus testing on a series of four Florida soils. Resilient modulus tests were run on the soils at optimum moisture content and different water retention conditions, representing variations in height above the water table. Specimens were compacted at optimum moisture content and maximum dry density, and later trimmed and placed into Tempe pressure cells, which used air pressure to expel water from the soil pores until static equilibrium was reached with the air pressure. The soil specimens were then saturated from the bottom up, until the daily differences in specimen weight was minimal. Data obtained from this process was plotted to investigate the water retention characteristics of the test soils. After examining the soil retention characteristics of the soils, resilient modulus test specimens were compacted and placed in a device to let water enter from the bottom, simulating a rising water table. The specimens were then placed in a conditioning apparatus until the desired water content was reached. The test specimen was then representative of a subgrade soil at a specific height above the water table. In general, the resilient modulus was shown to increase with increasing height above the water table (decreasing water content).

Thadkamalla and George (1995) investigated the effects of post-compaction saturation on the resilient modulus of coarse and fine-grained soils. Samples were prepared from two coarse-grained and two fine-grained soils, and the samples were

tested using the AASHTO T294/SHRP P46 test procedure, with slight modifications.

The researchers used three different methods for specimen preparation:

vacuum/backpressure saturation of molded samples, capillary saturation of molded samples, and samples molded wet of optimum. An analysis of the test results showed that increases post-compaction saturation led to a decrease in M_r for both coarse and fine-grained soils, but the effects on the fine-grained soils were most pronounced. The resilient modulus was shown to decrease by approximately 20% for coarse-grained soils and 50-75% for fine-grained soils, depending upon the extent of saturation.

An analysis of the three saturation techniques found that samples saturated using the vacuum/ backpressure method showed the largest decreases in resilient modulus. Samples prepared using capillary saturation and samples molded wet of optimum showed similar decreases in resilient modulus. The researchers stated that samples prepared in the latter two manners showed a linear decrease in M_r with increasing saturation, while the samples prepared with vacuum/backpressure saturation showed an exponential decrease in M_r with increasing saturation. The authors proposed explanations for the differences in the test results by describing the probable role of the air-water interface and its effect on soil strength.

Madgett (1994) conducted a similar series of M_r tests to determine the effects of post-compaction saturation on three fine-grained Tennessee subgrade soils. The soils were tested using the SHRP Protocol P46 test procedure. Three samples from each soil were compacted at optimum moisture content and maximum dry density using a kneading compactor, and then two of these samples were later saturated using

backpressure saturation. In contrast to the results found by Thadkamalla and George, the backpressure saturated samples showed a somewhat linear decrease in resilient modulus with increasing saturation. The difference in the results found by the researchers may stem from the amount of time that the samples were allowed to cure. Madgett allowed the test specimens to cure in a humid room for twenty-four hours before saturating the soil samples, while Thadkamalla and George waited only two hours before they beginning saturation. In addition, Madgett allowed all of the soil samples to cure for a minimum of seven days prior to testing. This curing time was allowed to minimize strength gain due to "thixotropy", or rearrangement of soil particles with age after compaction (Thompson and Robnett 1976; Hudson 1992).

Thadkamalla and George (1995) did not mention how long they waited before conducting their triaxial tests, but perhaps a longer curing time was needed to allow the moisture content in the samples to equalize. The distribution of water content shown along the length of the sample suggest that the moisture content was not uniformly distributed at the time of testing, with substantially higher moisture content at the sample ends.

2.3.3 Prediction Model for Post-Compaction Saturated Soils

Drumm et al. (1995) proposed a series of multiple regression models based on triaxial tests conducted on Tennessee subgrade soils. The models are based on triaxial results for samples tested at optimum moisture content and maximum dry density conditions, as well as samples prepared at these conditions and later saturated to a higher degree of saturation. Individual multiple regression models were performed for four

different AASHTO classifications (A-4, A-6, A-7-5, and A-7-6). In addition, a general predictive model was performed including all of these fine-grained classifications. The structure of the models is based on the general power equation form (Moossazadeh and Witczak 1981) as follows:

$$M_r = K_1(\sigma_d)^{K_2}$$

or alternatively:

$$\text{Log}_{10} M_r = \text{Log}_{10} K_1 + K_2 \text{Log}_{10} (\sigma_d)$$

The constants K_1 and K_2 are replaced with the multiple regression coefficients S_1 and S_2 , yielding:

$$\text{Log}_{10} M_r = \text{Log}_{10} S_1 + S_2 (\text{Log}_{10} \sigma_d)$$

where:

M_r = resilient modulus (MPa)

$S_1 = f(\text{CELL, PI, PL, LL, LI, DTS, PCLAY, CLASS})$

S_2 = regression coefficient

σ_d = deviator stress (kPa)

CELL = cell pressure (kPa)

PI = plasticity index (%)

PL = plastic limit (%)

LL = liquid limit (%)

LI = liquidity index

DTS = increase in post-compaction saturation from reference (optimum moisture content and maximum dry density) condition (%)

PCLAY = clay content (%)

CLASS = AASHTO classification number (e.g., 4.0, 6.0, 7.5, 7.6)

2.3.4 Prediction Model Based on Field Seismic Testing

Pezo and Hudson (1994) proposed a predictive model based on the Ramberg and Osgood stress-strain model. This model evaluates the stiffness characteristics of the pavement layers by relating field and laboratory data. The model is presented in the following form:

$$\frac{M_R}{M_{Rmax}} = \frac{1}{1 + \alpha \left[\frac{\sigma_d}{\sigma_{det}} \right]^{r-1}}$$

where:

M_R = resilient modulus

M_{Rmax} = maximum resilient modulus corresponding to the axial strain elastic threshold

σ_d = applied deviator stress

σ_{det} = deviator stress corresponding to the axial strain elastic threshold

α, r = regression coefficients

After performing a multilinear regression based on the preceding equation, Pezo and Hudson presented a set of curves that allow the pavement designer to estimate a design resilient modulus value based on field seismic test results. These curves are only applicable for a confining stress of 41.4 kPa (6 psi).

2.3.5 Alternative Test Method (ATM)

Li (1992) proposed an Alternative Test Procedure (ATM) for determining resilient modulus. The procedure is based on a falling mass striking a mass of soil confined within a Standard Proctor mold. The procedure measures the constrained modulus of the soil, which can be converted to resilient modulus by assuming a value for Poisson's ratio. The relationship between the constrained modulus and resilient modulus is as follows:

$$M_r = \frac{(1+\nu)(1-2\nu)}{(1-\nu)} D$$

where:

M_r = Resilient Modulus

D = Constrained Modulus

ν = Poisson's Ratio

This procedure is described in more detail in Chapter 4 "Improvements to the ATM and Verification of ATM Test Results."

2.4 Previous TNDOT Research

Hudson (1992) and Madgett (1994) conducted a series of triaxial tests on fine-grained Tennessee soils for the Tennessee Department of Transportation. The eight soils tested by Hudson are referred to as Phase I soils, while the six soils tested by Madgett are referred to as Phase II soils, for a total of fourteen soils. The results of the testing were summarized in the form of a *Design Handbook* (Drumm et al. 1993, 1995).

2.4.1 Soil Sampling

The sampling locations for the fourteen Tennessee soils are shown in Figure 2-1. The eight soils tested by Hudson (1992) are given the following designations: Knox Co. Rutledge Pike, Knox Co. Station 400, Knox Co. Station 500, Roane Co. Station 47, Roane Co. Station 85, Crockett Co. Station 781, Crockett Co. Station 1081, and Wilson Co. Station 28. The remaining six soils were tested by Madgett (1994).

2.4.2 Index Properties and Moisture-Density Relationships

Hudson (1992) investigated the effects of compaction at different moisture-density states on resilient modulus, and Madgett tested samples at optimum moisture content conditions and three samples under post-compaction saturation conditions. Testing procedures used by these researchers for moisture-density relationships and index testing are included in Table 2-1, and the moisture-density relationships are shown in Figure 2-2. Index properties and soil classifications for the fourteen Tennessee soils are shown in Table 2-2.

2.4.3 Triaxial Testing Procedures

Triaxial testing conducted by Hudson (1992) and Madgett (1994) followed the SHRP Protocol P46 (SHRP 1989) procedure with a few minor exceptions as described by Hudson (1992). The equipment setup, specimen preparation, and testing procedure for SHRP Protocol P46 are listed in Tables 2-3, 2-4, and 2-5, respectively. The triaxial tests conducted here were performed in the same manner as Hudson and Madgett.

2.5 Subgrade Resilient Modulus in Flexible Pavement Structural Design

The 1991 Manual Series No. 1 (MS-1) Asphalt Institute Method and 1993 AASHTO Method are two of the most commonly used flexible pavement design

Table 2-1. Test Procedures used for Index Testing and Moisture-Density

Property	Test Procedure
Specific Gravity	ASTM D854-83
Atterberg Limits	ASTM D4318-84
Grain Size Distribution	ASTM D422-63
Moisture-Density	AASHTO T-99

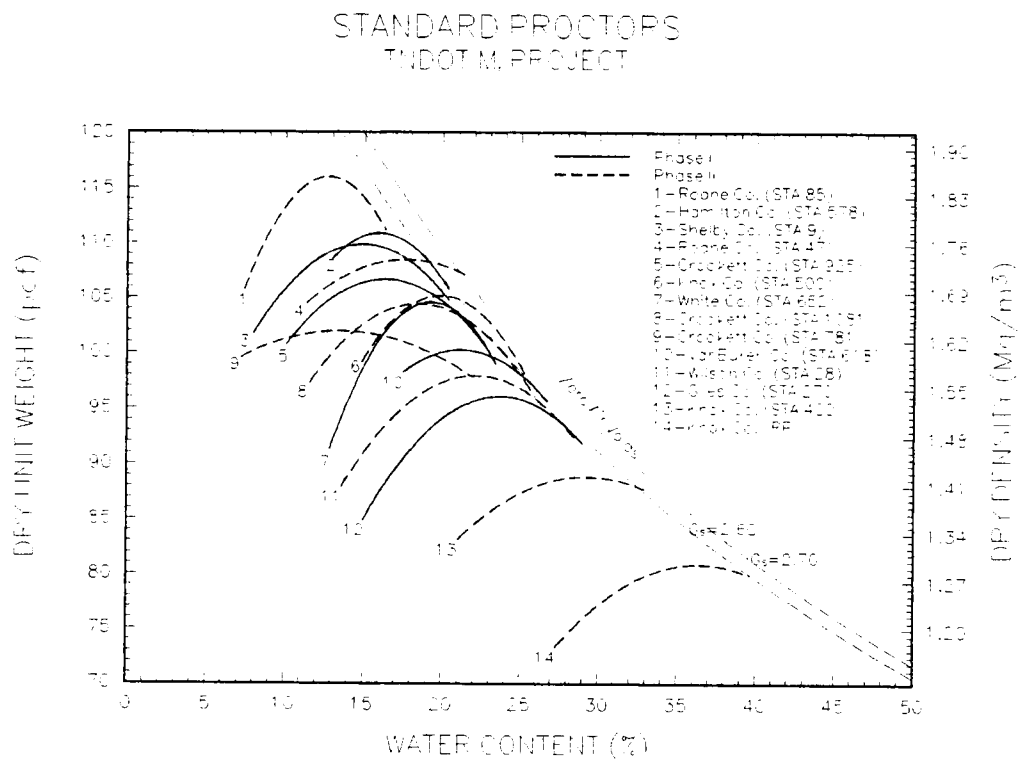


Figure 2-2. Moisture-Density Relationship for Phase I and II Soils (after Madgett, 1994)

Table 2-2. Classification and Index Properties of Phase I and II Soils (after Madgett 1994)

Location	Soil Classification			Atterberg Limits			Percent Clay (%)	Percent Passing #200	Specific Gravity	Optimum Moisture Content (%)	Maximum Dry Density (pcf)
	AASHTO	G.I.	USCS	LL	PL	PI					
Crockett Co. STA 925	A-6	11	CL	33	22	11	36.0	99.1	2.61	16.5	106.6
Giles Co. STA 270	A-7-5	19	CH	52	30	22	53.0	80.9	2.64	23.7	96.1
Shelby Co. STA 9	A-4	10	CL	30	20	10	42.0	98.6	2.70	15.0	109.8
Hamilton Co. STA 578	A-6	10	CL	34	22	12	52.5	84.2	2.66	16.0	110.8
VanBuren Co. STA 618	A-7-6	15	CH	54	26	28	30.0	58.8	2.54	21.2	100.2
White Co. STA 652	A-6	8	CL	39	23	16	40.0	65.0	2.61	19.3	104.6
Knox Co. Rutledge Pike	A-7-6	50	CH	71	28	43	70.3	97.8	2.60	36.2	82.5
Knox Co. STA 400	A-7-5	36	MH	70	41	29	58.6	94.2	2.70	29.6	90.3
Knox Co. STA 500	A-7-6	15	CL	45	22	23	39.9	70.4	2.70	19.2	106.3
Roane Co. STA 47	A-6	8	CL	34	21	13	25.4	72.3	2.65	18.2	109.5
Roane Co. STA 85	A-4	0	CL-	21	16	5	16.7	56.1	2.60	13.2	116.0
Crockett Co. STA 781	A-4	1	ML	24	22	2	22.9	97.2	2.59	16.8	103.3
Crockett Co. STA 1081	A-6	15	CL	36	22	14	25.4	98.9	2.62	18.4	105.9
Wilson Co. STA 28	A-7-6	36	CH	55	19	36	44.5	92.7	2.84	22.0	97.9

Table 2-3. Equipment Setup for SHRP Protocol P46 (SHRP 1989)

APPARATUS	SHRP PROTOCOL P46
LVDT's	2 EXTERNAL LVDT's
CHAMBER FLUID	air, water, or water alcohol mix
LOADING DEVICE	closed-loop electro-hydraulic system required
LOAD DURATION	0.1 seconds
CYCLE DURATION	1 second
STRESS PULSE	haversine
LOAD CELL	internal
MEMBRANE THICKNESS	0.01-0.031

Table 2-4. Specimen Compaction Specifications for SHRP Protocol P46

COMPACTION SPECIFICATIONS	SHRP PROTOCOL P46
SPECIMEN SIZE	minimum diameter = 2.8 in. or 5 x nominal particle size minimum length = 2 x diameter
COHESIVE	AASHTO T99-86
GRANULAR	AASHTO T180-85
CURING TIME	none specified

Table 2-5. Testing Procedure for SHRP Protocol P46

TEST PROCEDURE	SHRP PROTOCOL P46		
CONDITIONING (COHESIVE)	σ_c (psi) 6	σ_d (psi) 4	Repetitions 200
TESTING (COHESIVE)	6	2	100
	6	4	100
	6	6	100
	6	8	100
	6	10	100
	4	2	100
	4	4	100
	4	6	100
	4	8	100
	4	10	100
	2	2	100
	2	4	100
	2	6	100
	2	8	100
	2	10	100

procedures. Both of these methods are based on mechanistic-empirical methodologies, and a value for the subgrade resilient modulus is required in both procedures. A basic overview of these design procedures is given below.

2.5.1 Asphalt Institute Method

In the 1991 MS-1 procedure, a representative field resilient modulus is determined by testing subgrade materials obtained from within 2 ft of the planned subgrade elevation. Usually, six to eight test values are used to determine the subgrade resilient modulus. The seasonal variation in resilient modulus is separated into four distinct period: freeze, thaw, recovery, and normal. The design resilient modulus is taken as the normal resilient modulus value that is smaller than 60, 75, or 87.5% of the

test values, based on traffic level. Full-depth asphalt pavement thicknesses are determined by using design charts with subgrade resilient modulus as an input variable (Huang, 1993).

2.5.2 AASHTO Method

The 1993 AASHTO *Guide for Design of Pavement Structures* describes a relative damage procedure for considering the seasonal variation in subgrade resilient modulus for flexible pavement design. These seasonal moduli typically show a wide variation in values, ranging from very high values for frozen conditions, very low values for thawing conditions, and intermediate values for other times of the year. The seasonal or monthly variation in resilient modulus is estimated by one of the following methods:

- 1.) Obtain a laboratory relationship between resilient modulus and moisture content, and estimate the in situ moisture content of the soil beneath the pavement. Then, estimate the modulus for each season based on the laboratory relationship.
- 2.) Back calculate the resilient modulus using deflections measured on in-service pavements.

Using these M_r estimates, a relative damage value, u_f , is assigned for each half month. The mean of the u_f values is used to determine the effective roadbed resilient modulus. According to the 1993 AASHTO Guide, "the effective subgrade soil resilient modulus determined from this method applies only to flexible pavements designed using serviceability criteria, and is not necessarily applicable to other resilient modulus-based design procedures." A design structural number is determined for the pavement layers overlying the subgrade by using the effective subgrade resilient modulus as an input

variable into design nomographs (AASHTO 1993; Huang 1993).

Chapter 3 Triaxial Testing and Effects of Post-Compaction Saturation

3.1 Cyclic Triaxial Testing

Cyclic triaxial resilient modulus testing was conducted in accordance with SHRP Protocol P46 (SHRP 1989). The eight Phase I soils (Hudson 1992) described in the "Previous TNDOT Research" section of Chapter 2 were retested. The testing equipment used for the triaxial testing is the same used by other researchers at the University of Tennessee (Johnson 1986; Boateng-Poku 1988; Hudson 1992; Madgett 1994). However, testing by Hudson (1992) and Madgett (1994), and that reported here, was conducted with slight modifications to the testing apparatus. An internal load cell and internally mounted LVDT's were added to improve the accuracy of the measured loads and deflections. Care was taken so that the triaxial testing procedures for this thesis are consistent with those of Hudson and Madgett, so that the test results could be quantitatively compared.

3.1.1 Testing at Optimum Moisture Content and Maximum Dry Density

The eight soils were tested at optimum moisture content and maximum dry density (standard proctor energy). Values of maximum dry density and optimum moisture content (standard proctor energy) for each soil were predetermined (Drumm et al. 1993) for each soil using the AASHTO T-99 procedure. These compaction states were used as target values for specimen preparation, since highway construction specifications usually require a 95% degree of compaction (Thompson and Robnett 1976). All specimens were prepared using kneading compaction (Johnson 1986). Tolerances of $\pm 0.0320 \text{ Mg/m}^3$ maximum dry density and $\pm 1\%$ optimum moisture

content were maintained during the preparation of the laboratory specimens. In addition, the specimen ends were grouted with hydrostone grout to ensure that the stress was evenly distributed across the sample.

Typical results from this testing are shown as Figure 3-1. It is evident from these results that at deviator stresses below about 20 kPa, the resilient modulus increased

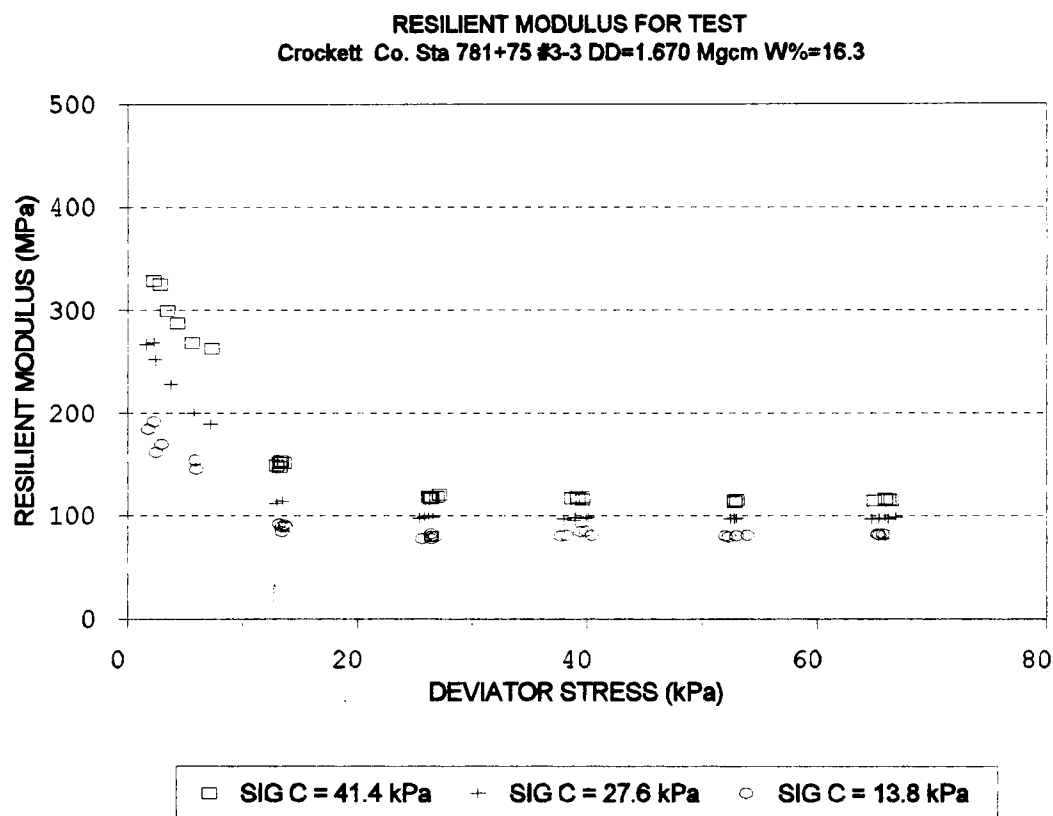


Figure 3-1. Typical M_r vs. σ_d Response for Optimum Moisture Content and Maximum Dry Density

significantly, and at deviator stresses above about 20 kPa the resilient modulus was relatively constant. The resilient modulus increased only slightly with increasing confining stress, and in some cases, not at all. This behavior is typical for fine-grained soils and similar responses have been noted by other researchers (Seed et al. 1962;

Thompson and Robnett 1976; Hudson 1992; Madgett 1994; Pezo and Hudson 1994).

The resilient responses shown by the TNDOT soils cover a wide range, which is expected due to wide range of soil properties. Figure 3-2 shows the highest resilient modulus values identified, with a minimum value of M_r approximately 170 MPa (24.6 ksi) at a deviator stress of 69 MPa (10 psi). For comparison, the lowest measured

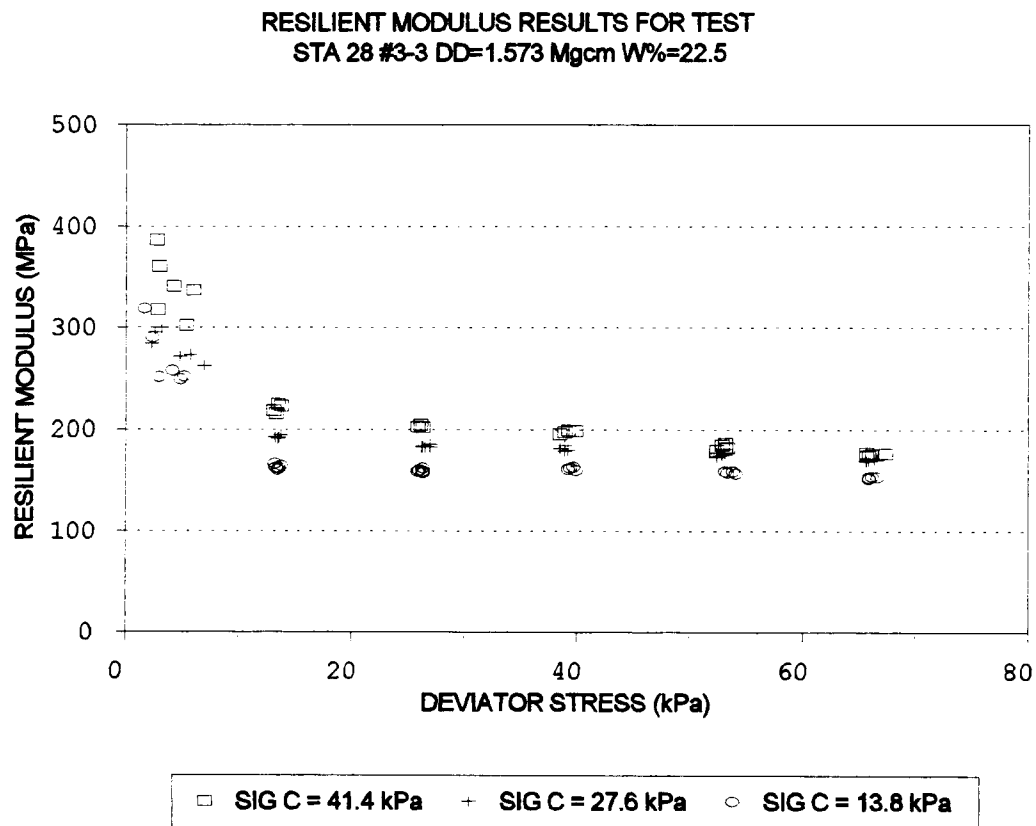


Figure 3-2. Highest Resilient Modulus Values for the Eight Soils Tested

resilient modulus values (Fig. 3-3) were approximately 60 MPa (8.7 ksi) at a deviator stress of 69 MPa (10 psi). Both samples were tested at optimum moisture content and maximum dry density. The soil classification tends to distinguish the differences in resilient response. The sample with the lower response is classified as an

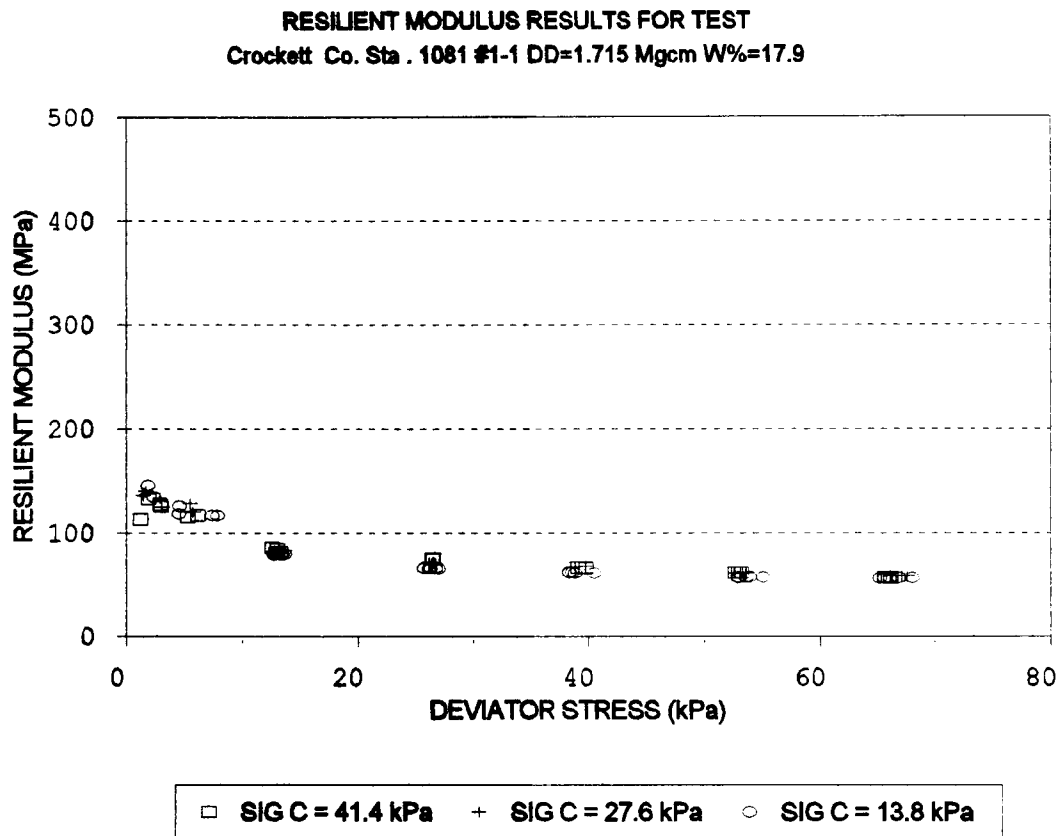


Figure 3-3. Lowest Values of Resilient Modulus for the Eight Soils Tested

A-6 (CL), while the sample with the higher response is classified as an A-7-6 (CH). A greater difference in moduli could be expected if the soils were each tested at various moisture content and dry density combinations as shown by Hudson (1992).

3.2 Effects of Seasonal Moisture Changes on Subgrade Resilient Behavior

A study conducted by the Illinois Department of Transportation showed that deflections in pavement subgrades vary seasonally, with the biggest deflections occurring in the wetter spring months (after Elliot and Thornton 1988). In addition, the magnitude of changes in deflection from season to season was dependent upon the type of soil. In the study, pavements overlying four different types of subgrade soils located near the

AASHTO Road Test site were monitored for deflections. These soil types were as follows: clay, silty clay, poorly-drained silty clay, and well-drained sand. The clay soil showed the most dramatic change in deflection with seasonal moisture changes, while the silty-clay showed less variation, and the well-drained sand showed the least variation in deflections. These results indicate that the clay soil was more susceptible to changes in resilient modulus, thus becoming less resilient (having larger deflections) than the silty or sandy soils.

According to Monismith (1991) increased deflection in subgrades leads to a decrease in pavement serviceability. Thus, the need for an understanding of how the resilient properties of subgrades vary with changes in moisture content is apparent. An understanding of these effects, coupled with knowledge about seasonal subgrade moisture changes, is useful in determining the effective roadbed resilient modulus used in the 1993 AASHTO *Guide for the Design of Pavement Structures* (AASHTO 1993).

3.2.1 Post-compaction Variation in Moisture Content vs. Compaction at Various Moisture Contents

Elliot and Thornton (1988) suggested a procedure similar to the 1993 AASHTO *Guide for the Design of Pavement Structures* by which resilient modulus values from samples molded at different moisture contents could be used to relate to seasonal variations in resilient modulus. These variations could then be used to calculate relative damage factors used to determine one resilient modulus design value. Two methods for determining these relationships were proposed. In the first method, subgrades under nearby pavements could be sampled at different times of the year to determine seasonal

moisture fluctuations in the subgrade. Variations in moisture content could then be correlated with previous M_r vs. Moisture Content curves from triaxial tests to determine seasonal variations in resilient modulus. The second method involved taking representative samples from the subgrade at different times of the year and testing them at their insitu moisture contents. These triaxial test results could then be used to develop M_r vs. moisture content curves for the different subgrade soils.

The procedures recommended by Elliot and Thornton (1988) are a useful way to approach the study of subgrade performance. However, varying the compaction moisture content at the time of compaction may not represent the actual variation in properties under field conditions. Soils compacted wet of optimum or dry of optimum have significant differences in resilient modulus than those compacted at optimum moisture content (Hudson 1992). The moisture content at compaction impacts the strength and stiffness properties of the soil due to the influence of particle orientations during compaction (Seed et al. 1962; Monismith 1991). These soil structure effects have been suggested as important factors governing resilient response (Lee et al. 1992). Therefore, in order to accurately predict how these subgrade soils will react with seasonal moisture changes, the samples should first be compacted near optimum moisture content and maximum dry density, and then the water content should be increased before resilient modulus testing.

3.3 Cyclic Triaxial Testing of Post-Compaction Saturated Samples

Eight different soils were compacted using a kneading compactor at optimum moisture content and maximum dry density, and later tested using SHRP Protocol P46

(SHRP 1989). A total of three samples of each soil were compacted, and two of these samples were later saturated to some increased degree of saturation (three post-compaction saturated samples were tested for Wilson County Sta. 28). After compaction, the specimens that were to be saturated were allowed to rest in a curing room overnight (all the specimens were placed in air-tight bags). The specimens were saturated the following day in a triaxial cell. Water pressure was applied from the bottom of the samples, and the top of the sample was at atmospheric pressure. This created a pressure gradient that allowed water to flow throughout the sample. The back pressure applied to the samples varied from 13.8 kPa to 27.6 kPa depending on the difficulty of saturation, but the effective confining pressure was never allowed to become greater than 41.4 kPa, which was the maximum effective confining pressure during the triaxial test. After the saturation process was completed, the samples were placed in a curing room for seven days, which was consistent with Madgett (1994).

The purpose of the study was to show how the resilient modulus varies with increases in post-compaction moisture content (and thus degree of saturation). Hudson (1992) showed how the resilient modulus varies with different moisture and density states at compaction. However, there is limited data showing how resilient modulus varies with post-compaction changes in moisture content.

Figure 3-4 shows how the resilient modulus typically changes with increases in saturation. Smooth curves have been fit to the data points for clarity (the same individual curves with the data points are shown in Appendix A), and the resulting curve is the average of the results at three different confining pressures of 41.4 kPa (6 psi),

EFFECTS OF POST-COMPACTION SATURATION ON RESILIENT MODULUS
KNOX CO. STA. 400

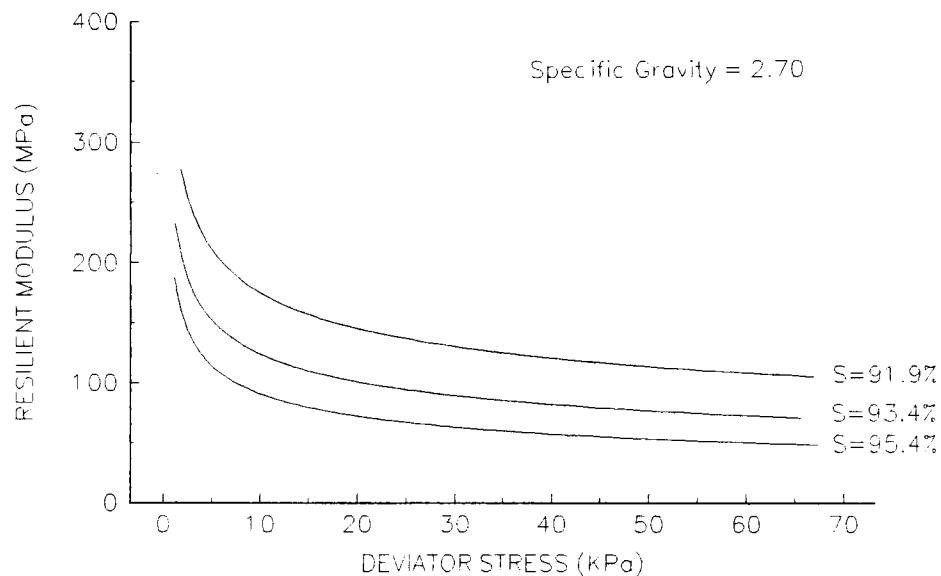


Figure 3-4. Typical Triaxial Results for Post-Compaction Saturation Resilient Modulus

27.6 kPa (4 psi), and 13.8 kPa (2 psi). The upper curve represents triaxial results for a specimen compacted near optimum moisture content at 29.4% and a degree of saturation of 91.9%, the middle curve represents a specimen saturated to a moisture content of 30.1% and 93.4% saturation, and the bottom curve represents a specimen saturated to a moisture content of 30.7% and 95.4 % saturation. It is obvious from Figure 3-4, that as the moisture content and degree of saturation of the soil increase, there is also a corresponding decrease in resilient modulus values. A convenient way to illustrate the magnitude of these changes is to compare values from the triaxial curves at the same deviator stresses and confining pressures. SHRP Protocol (SHRP 1989) suggests reporting M_r values at a deviator stress of 4 psi (27.6 kPa) and a confining

pressure of 6 psi (41.4 kPa). Figure 3-5 shows a plot of M_r vs. moisture content at these stress conditions for the Knox County Station 400 samples. If a M_r vs. degree of saturation curve is plotted, the resulting curve is somewhat linear, as shown in Figure 3-6.

3.3.1 Slopes of the Resilient Modulus vs. Degree of Saturation Curves

Figure 3-7 shows the variation in resilient modulus with degree of saturation for eleven different subgrade soils tested for TNDOT. The soils tested range from A-4 soils to A-7-6 soils. Shelby County Station 9, Crockett County Station 925, and Giles County Station 270 were tested by Madgett (1994). In general, a linear trend is noted in these relationships, with the resilient modulus decreasing as a function of increased saturation.

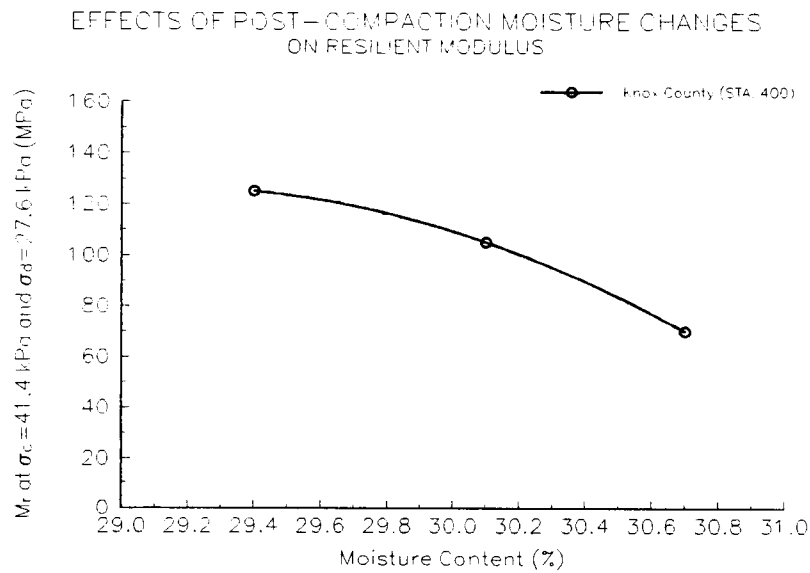


Figure 3-5. Typical Effects of Post-Compaction Moisture Change on Resilient Modulus

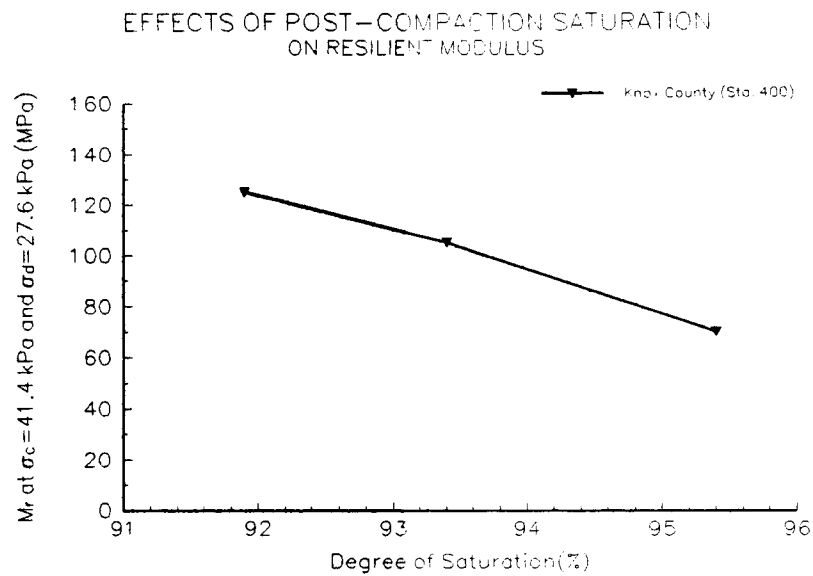


Figure 3-6. Typical Effects of Post-Compaction Saturation on Resilient Modulus

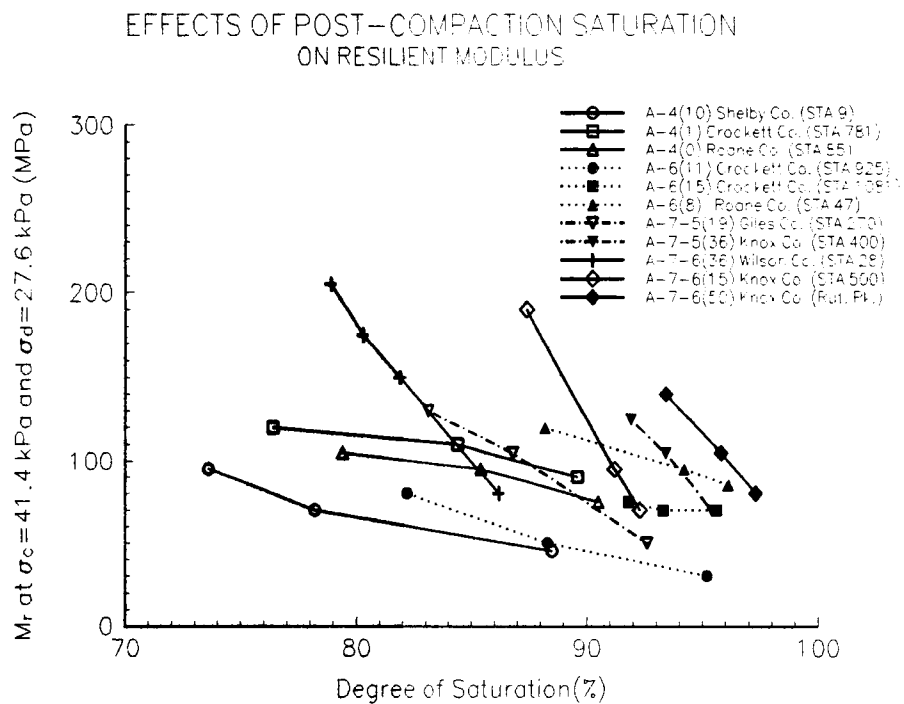


Figure 3-7. Resilient Modulus vs. Changes in Degree of Saturation

The A-7-6 curves have the "steepest" slopes, while the A-4 soil curves have the "flattest" slopes. These trends indicate that the A-7-6 and A-7-5 soils are more susceptible to changes in M_r with changes in saturation, and the A-4 and A-6 soils are less susceptible to M_r changes. The M_r vs. degree of saturation curves can be used to estimate M_r design values at elevated water content or saturation levels if the M_r at optimum moisture content is known for a particular soil. This is of particular value when using the AASHTO relative damage (u_r) procedure.

The advantages of plotting M_r vs. degree of saturation over moisture content may not seem apparent at first. The degree of saturation is not only dependent upon the moisture content of the soil, but also any differences in density of the soil (Thompson and Robnett 1976,1979). Therefore, the degree of saturation gives a more complete description of the soil state. Some might argue that a good estimate of the specific gravity of a particular soil is essential in defining the degree of saturation of the soil. Indeed, slight differences in specific value will give different values of degree of saturation, since the degree of saturation is linearly dependent upon specific gravity. This was one of the limitations of the saturation models developed by Thompson and Robnett (1979). However, the slope of M_r vs. degree of saturation curve is not dependent upon specific gravity.

3.3.2 Procedure for Determining Design Values for M_r at Increased Degree of Saturations

It should be noted that to use the relationships shown in Figure 3-7, an appropriate estimate of seasonal moisture variations or changes in saturation is required

before the resilient modulus can be estimated. With an appropriate estimate of seasonal saturation changes, the results from Figure 3-7 can be used with the AASHTO relative damage procedure. Reductions in resilient modulus values due to increases in saturation can also be determined directly from the slope of the M_r vs. degree of saturation curves. Table 3-1 summarizes the change in M_r with increased saturation for the soils tested with their corresponding AASHTO and USCS classifications, and Table 3-2 shows changes in M_r vs. degree of saturation with their respective Atterberg limits. Using these tables, reductions in resilient modulus can be estimated.

First, the engineer or designer must obtain a reference value of M_r at optimum moisture content and maximum dry density from direct testing, or use of one of the previously mentioned estimation techniques. Then, changes in M_r at higher saturations can be estimated by the following relationship:

$$M_{r(sat)} = M_r + m(\Delta S)$$

where:

$M_{r(sat)}$ = resilient modulus at increased post-compaction saturation (MPa)

M_r = resilient modulus at optimum moisture content and maximum dry density (Mpa)

m = slope of the M_r vs. degree of saturation curve (MPa / $\Delta S\%$)

ΔS = change in post-compaction degree of saturation (%)

Table 3-1. Slopes of M_r vs. Degree of Saturation Curves

Sampling Location	Soil Classification		Average Slope ($\Delta MPa / \Delta\%$ Saturation)	Average Slope ($\Delta MPa / \Delta\%$ Saturation)
	AASHTO	USCS		
Shelby Co. Sta. 9	A-4	CL	-3.9	-3
Roane Co. 85	A-4	CL-ML	-2.8	
Crockett Co. Sta. 781	A-4	ML	-2.6	
Crockett Co. Sta. 925	A-6	CL	-3.9	-4
Crockett Co. Sta. 1081	A-6	CL	-3.3	
Roane Co. Sta. 47	A-6	CL	-4.7	
Giles Co. Sta. 270	A-7-5	CH	-8.1	-12
Knox Co. Sta. 400	A-7-5	MH	-15.4	
Wilson Co. Sta. 28	A-7-6	CH	-17.8	-19
Knox Co. Sta. 500	A-7-6	CL	-23.9	
Knox Co. Rut. Pk.	A-7-6	CH	-15.6	

Table 3-2. Atterberg Limits with Slopes of M_r vs. Degree of Saturation Curves

Sampling Location	Atterberg Limits			Average Slope ($\Delta MPa / \Delta\%$ Saturation)
	LL	PL	PI	
Shelby Co. Sta. 9	30	20	10	-3.9
Roane Co. Sta. 85	21	16	5	-2.8
Crockett Co. Sta. 781	24	22	2	-2.6
Crockett Co. Sta. 925	33	22	11	-3.9
Crockett Co. Sta. 1081	36	22	14	-3.3
Roane Co. Sta. 47	34	21	13	-4.7
Giles Co. Sta. 270	52	30	22	-8.1
Knox Co. Sta. 400	70	41	29	-15.4
Wilson Co. Sta. 28	55	19	36	-17.8
Knox Co. Sta. 500	45	22	23	-23.9
Knox Co. Rutledge Pike	71	28	43	-15.6

The slope of the M_r vs. degree of saturation for a particular soil can be estimated based on the post-compaction saturated soil tests in the following ways:

- 1) Estimate changes in M_r for the given soil by choosing the curve for the TNDOT soil from Figure 3-7 or the slope from Table 3-1 or Table 3-2 that most closely matches the soil in question (e.g. similar AASHTO classification, index properties, dry density, etc.).
- 2) Estimate M_r changes by using the average M_r vs. Saturation slope from Table 3-1 for the AASHTO classification for the soil in question.

It should be noted that these relationships begin at a reference value of resilient modulus at optimum moisture content and maximum dry density. These relationships are only applicable to soils compacted at these conditions and later saturated "wet" of optimum moisture content, and cannot be used to estimate increases in resilient modulus "dry" of optimum moisture content.

3.3.3 Statistical Model for M_r vs. Degree of Saturation Slopes

A statistical model was developed to predict the slopes of the M_r vs. degree of saturation slopes. The model was produced with the aid of the SAS statistical software (SAS Institute 1990). A multiple linear regression was performed by treating the following parameters as independent variables: liquid limit, plastic limit, percent clay, AASHTO classification, resilient modulus (at optimum moisture content), degree of saturation (at optimum moisture content), dry density, and liquidity index. Most of these variables did not show statistical significance at a level of 0.15, and the final form of the model is as follows:

$$\text{Slope} = 16.9027 - 1.9367(\text{CLASS}) - 0.1115 (M_r) \quad r^2 = 0.86$$

where:

Slope = slope of the M_r vs. degree of saturation curves

CLASS = AASHTO classification (e.g., 4.0, 6.0, 7.5, 7.6)

M_r = resilient modulus at optimum moisture content and maximum dry
density (MPa)

Table 3-3 shows the predicted slopes and slope angles from the statistical model, along with the measured slopes and slope angles. Table 3-4 is a comparison of resilient modulus values calculated for an increased saturation condition, using the actual and model-predicted slopes from Table 3-3. Although differences exist between the slopes actual and predicted slopes in Table 3-3, it can be seen in Table 3-4 that these differences in slopes do not lead to significant differences in the calculated resilient modulus.

In general, the A-4 and A-6 soils show the smallest differences in calculated M_r values at increased saturation. This is to expected, since these soils have the "flattest" slopes. Drumm et al. (1995) suggested that sample-to-sample variations can limit repeatability of resilient modulus values within only about 35 MPa (5 ksi). Given this fact, the agreement of the calculated M_r values predicted by both slope methods is reasonable, since none of the prediction differences are above 35 MPa. The slopes predicted by the statistical model may be applicable to the previously described "Procedure for Determining Design Values for M_r at Increased Degree of Saturations."

3.3.4 Example for Determining Resilient Modulus at Increased Post-Compaction Saturation

The following example shows a procedure for estimating the resilient modulus

Table 3-3. Comparison of Predicted Slopes and Measured Slopes

Sampling Location	Measured Slope (Δ MPa/ Δ % Saturation)	Predicted Slope (Δ MPa/ Δ % Saturation)	Measured Angle (degrees)	Predicted Angle (degrees)	Difference in Angles Relative to Measured (%)
Shelby Co. Sta. 9	-3.9	-0.9	-75.6	-41.9	+45
Roane Co. Sta. 85	-2.8	-3.1	-70.3	-72.1	-2.6
Crockett Co. Sta. 781	-2.6	-4.2	-69.0	-76.6	-11
Crockett Co. Sta. 925	-3.9	-3.6	-75.6	-74.5	+1.5
Crockett Co. Sta. 1081	-3.3	-3.1	-73.1	-72.1	+1.4
Roane Co. Sta. 47	-4.7	-8.1	-78.0	-83.0	-6.4
Giles Co. Sta. 270	-8.1	-12.1	-83.0	-85.3	-2.8
Knox Co. Sta. 400	-15.4	-13.2	-86.3	-85.7	+0.7
Wilson Co. Sta. 28	-17.8	-20.7	-86.8	-87.2	-0.1
Knox Co. Sta. 500	-23.9	-19	-87.6	-86.9	+0.8
Knox Co. Rutledge Pike	-15.6	-14.5	-86.3	-86.1	+0.2

Table 3-4. Comparison of Calculated Resilient Modulus at Increased Saturation

Predicted Resilient Modulus @ 5% Increase in Degree of Saturation						
Sample Location	Triaxial M_r	M_r Actual	M_r Model	Relative	Relative	Relative
	@ Opt. w% (MPa)	Slopes (MPa)	Slopes (MPa)	Difference (MPa)	Difference (ksi)	Difference (%)
Shelby Co. Sta. 9	90	70	85	+15	+2.2	+21
Roane Co. Sta. 85	110	96	94	-2	-0.3	-2
Crockett Co. Sta. 781	120	107	99	-8	-1.2	-7
Crockett Co. Sta. 925	80	60	62	+2	+0.3	+3
Crockett Co. Sta. 1081	75	58	59	+1	+0.2	+2
Roane Co. Sta. 47	120	96	79	-17	-2.5	-18
Giles Co. Sta. 270	125	84	64	-20	-2.9	-24
Knox Co. Sta. 400	110	33	44	+11	+1.6	+33
Wilson Co. Sta. 28	200	111	96	-15	-2.2	-14
Knox Co. Sta. 500	180	61	85	+24	+3.5	+39
Knox Co. Rutledge Pike	130	52	57	+5	+0.7	+10

of a subgrade soil initially compacted at optimum moisture content and maximum dry density, and later subjected to increases in moisture content. The example shows how the slopes of the M_r vs. degree of saturation curves in Table 3-1 and the slope prediction model can be used to estimate the decrease in resilient modulus.

Example

An estimate of M_r with an increase of 3% moisture content after compaction is desired. It is assumed that $\sigma_d = 27.6$ kPa, and $\sigma_c = 41.4$ kPa.

Known Soil Properties:

AASHTO Classification:	A-7-5
USCS Classification:	CH-MH
Resilient modulus (opt. @opt. w%)	130 MPa (18.9 ksi)
Standard Proctor Maximum Dry Density (Mg/m ³):	1.471 (91.8 pcf)
Optimum Water Content (%):	27.1
Specific Gravity:	2.70
Degree of Saturation @ opt. w% (%):	87.6

The degree of saturation can be calculated using the following relationship:

$$S\% = \frac{G_s \times \rho_d \times w\%}{G_s \times \rho_d} = \frac{G_s \times \gamma_d \times w\%}{(G_s \times \gamma_w) \times \gamma_d}$$

where:

S = degree of saturation (%)

G_s = specific gravity

ρ_d = dry density

w = moisture content (%)

γ_d = dry unit weight

γ_w = unit weight of water

An increase of 3% moisture content raises the degree of saturation of the soil in question from 87.6% to 92.6%. This corresponds to an increase in saturation, ΔS , of 5%. The average AASHTO slope from Table 3-1 and statistical model predicted slopes are used in this case, and the following calculations are used to determine the reduced resilient modulus:

AASHTO slope, $m = -12 \text{ MPa} / \Delta S\%$

$$M_{r(\text{sat})} = 130 \text{ MPa} + (5\%)(-12 \text{ Mpa}/\Delta S\%)$$

$$M_{r(\text{sat})} = 70 \text{ MPa} @w=30.1\%$$

Predicted slope, $m = 16.9027 - 1.9367(7.5) - 0.1115(130) = -12.1 \text{ Mpa} / \Delta S\%$

$$M_{r(\text{sat})} = 130 \text{ MPa} + (5\%)(-12.1 \text{ MPa}/\Delta S\%)$$

$$M_{r(\text{sat})} = 69.5 \text{ Mpa} @w=30.1\%$$

3.4 Summary

Typical resilient modulus data from triaxial testing was shown. The complete set of data from eight soils tested is provided in Appendix A. In addition, two methods for predicting the decrease in resilient modulus with an increase degree of saturation were proposed. This method is consistent with the relative damage guidance provided in the 1993 AASHTO *Guide for Design of Pavement Structures*.

Chapter 4 Improvements to the ATM and Verification of ATM Test Results

4.1 Purpose of the ATM Procedure

The cyclic triaxial test has been the standard method of testing the resilient modulus of fine-grained soils for several years. Since triaxial tests are inherently time-intensive and expensive to run on a production basis, it is desirable to have other methods available to estimate the resilient modulus of subgrade soils. Consequently, an Alternative Test Method (ATM) was developed (Li 1992), and was further refined by Madgett (1994). A series of ATM tests was performed on fine-grained Tennessee soils, and compared to ATM test results by Madgett. Eight of the soils tested by Hudson (1992) in the triaxial test were tested with the ATM device. A comparison was made with the triaxial test in order to evaluate the performance of the ATM procedure.

4.2 Improvements to the ATM Procedure

The ATM procedure requires the use of a digital oscilloscope to capture acceleration vs. time histories of a falling hammer (the relationship between deceleration and ATM calculations are described below). The decelerations are recorded with the use of an accelerometer mounted within the hammer. Previous ATM testing was performed by manually reading peak deceleration values from the oscilloscope output. With the help of Mr. Lee Spencer and Mr. X.Z. Wang, a computer program, TNDOT, was created to read the peak deceleration values and to perform ATM calculations. This eliminated the timely steps required for manual data acquisition. The new ATM procedure is detailed in Appendix C.

4.3 Test Results for Specimens Compacted at Optimum Moisture Content and Maximum Dry Density

Samples from each of the eight soils were compacted in standard proctor molds at optimum moisture content and maximum dry density (standard proctor). Tolerances of $\pm 1\%$ optimum moisture content and $\pm 0.05 \text{ Mg/m}^3$ were maintained. The ATM procedure was used to determine resilient modulus values for each soil. For each test, two different hammer weights were used to determine how differences in hammer weight effect the prediction of M_r .

Typical ATM test results for a soil compacted at optimum moisture content and maximum dry density are shown as Figure 4-1. Unlike the "stress softening" response usually observed in the triaxial resilient modulus tests, a steady increase in M_r with deviator stress was observed. These results are similar to results found by Li (1992) and Madgett (1994). Since the ATM sample is contained in a proctor mold, the soil is in constrained state, and the constrained modulus is measured. During a constrained test, the confining stress increases as the vertical stress increases, because only vertical strains are allowed to take place (Lambe and Whitman 1969). The increase in confining stress is a function of the vertically applied deviator stress and the value of Poisson's ratio. To convert the constrained modulus measured in the ATM to resilient modulus (Young's modulus), a value for Poisson's ratio must be known. Poisson's ratio has been assumed to be 0.45 in all of the ATM tests, which is consistent with common pavement design practices (Huang 1993). If a smaller value of Poisson's ratio is assumed, the calculated M_r and deviator stress both become larger. Individual ATM results for the

eight different soils are included in Appendix B.

4.3.1 Variability of Resilient Poisson's Ratio with Stress Ratio

Poisson's ratio is not actually a constant, but a nonlinear function of the stress state like resilient modulus. Boyce (1980) derived relationships for resilient Poisson's ratio for granular materials, where resilient Poisson's ratio is a function of the stress ratio (σ_1/σ_3). It is likely that if the nonlinear variation of Poisson's ratio for soils tested

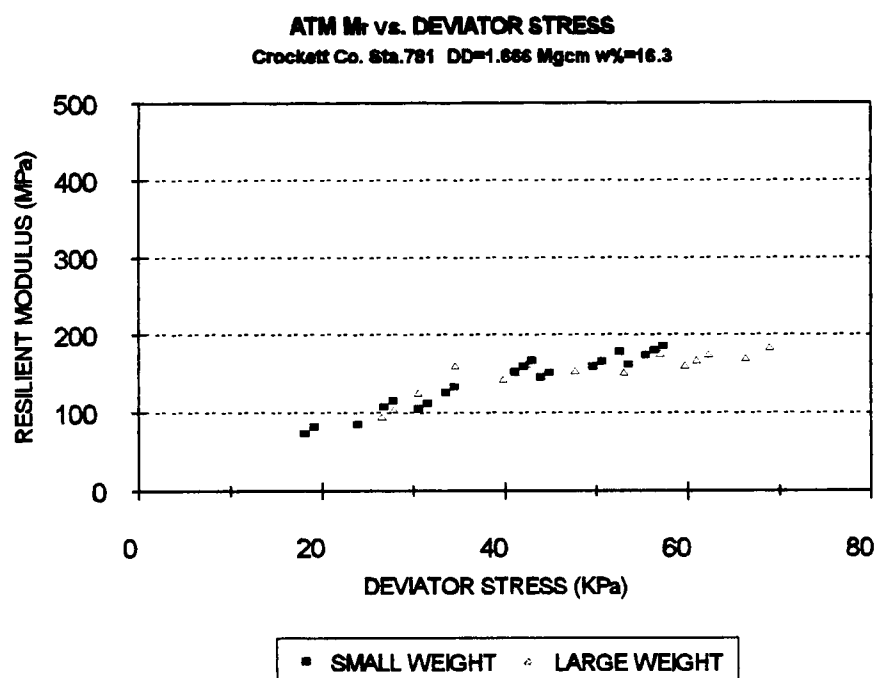


Figure 4-1. Typical ATM Results

with the ATM was known, the agreement between the ATM results and the standard test results might be better.

The need to assume a value for Poisson's ratio is not a limitation of the ATM, and it is only required if ATM results are to be directly compared with standard resilient modulus results. Two material constants are required to describe the elastic behavior of

the subgrade. Typically in design, the resilient (Young's) modulus is determined and a value for Poisson's ratio is assumed. Alternatively, pavement design could be performed with a measured constrained modulus, such as that obtained in the ATM, and an assumed Poisson's ratio. ATM results are presented here in terms of the resilient modulus, since that is the standard approach. When using resilient modulus values from the ATM, it is very important to use the same value for Poisson's ratio in design as was used to convert the ATM constrained modulus, D , to resilient modulus, M_r . These moduli are related by the following equation:

$$M_r = \frac{(1+\nu)(1-2\nu)}{(1-\nu)} D$$

where:

M_r = Resilient Modulus

D = Constrained Modulus

ν = Poisson's Ratio

4.3.2 ATM Resilient Modulus Calculations

Li (1992) developed an analytical model for the ATM based on a single degree-of-freedom lumped mass and spring system. The equations are based on a hammer mass striking a mass of soil. The peak acceleration and drop height of the hammer are used to estimate the resilient modulus and deviator stress in the ATM as:

$$M_r = \frac{(1+\nu)(1-2\nu)}{(1-\nu)} \cdot \frac{M \cdot L}{2hgA_{sec}} \cdot A_p^2$$

$$\sigma_d = \frac{1-2\nu}{1-\nu} \cdot \frac{A_p \cdot M_0}{A_{sec}}$$

where:

M_r = resilient modulus

ν = Poisson's ratio

$M = M_0 + M_1$

M_0 = mass of hammer

M_1 = mass of soil specimen

L = length of specimen

A_p = peak deceleration

h = equivalent drop height = $h_0(M_0/M)$

h_0 = drop height of the hammer

g = acceleration of gravity

4.3.3 Testing the Validity of the ATM Procedure

In order to test the validity of the ATM method, the ATM results were compared with results from cyclic triaxial testing conducted using SHRP Protocol P46 (SHRP, 1989). Figure 4-2 shows ATM and triaxial results for Crockett Co. Station 781. Two different hammer weights of 10.9 N (2.5 lb) and 15.1 N (3.4 lb) were used in this test. These hammer weights were obtained by using two different supplemental hammer weights. Tests using no add-on weight were also performed, but the predicted M_r values were high and are not presented here. The results for tests using the small and large weights are consistent with each other. However, the tests using the larger weight yield higher deviator stresses for a given drop height than tests using the smaller weight. For instance, in Figure 4-1, the smallest deviator stress measured using the larger weight is

Mr BY TRIAXIAL AND ATM TESTS
Crockett County Station 781

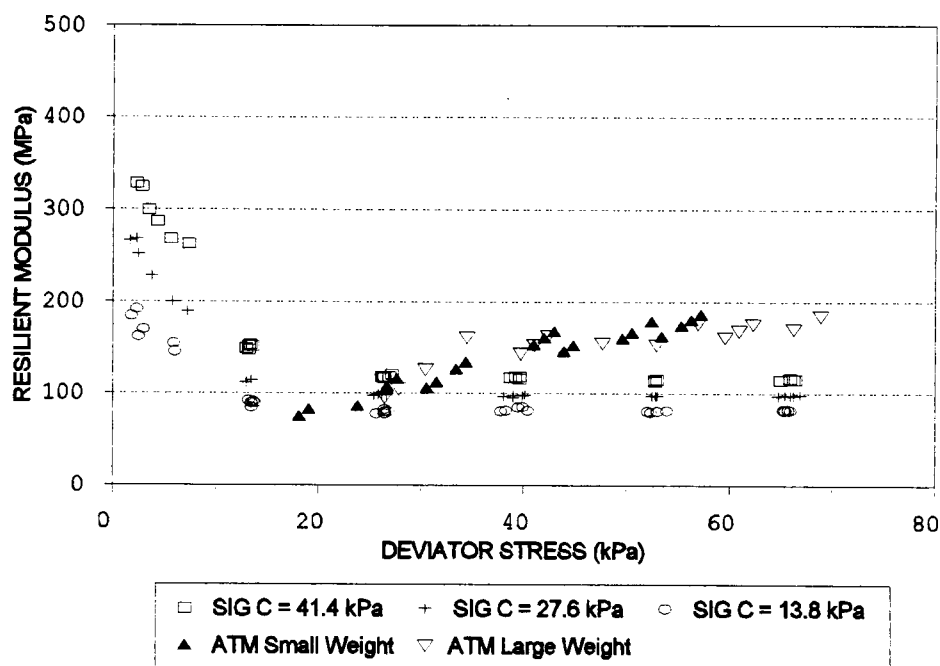


Figure 4-2. Triaxial and ATM M_r vs. Deviator Stress

approximately 25 kPa for a drop height of 10 mm, as opposed to 17.5 kPa for the smaller weight at the same drop height. This is to be expected, since the calculated deviator stress is a function of the mass of the hammer.

For comparison, ATM results were compared with triaxial data for the same moisture-density state. SHRP Protocol P46 (SHRP 1989) suggests reporting triaxial M_r values at a confining stress of 41.4 kPa (6 psi) and a deviator stress of 27.6 kPa (4 psi). This provides a good stress state for comparison with the ATM results. The triaxial values in the comparison curves were interpolated from the M_r vs. deviator stress plots for the suggested stress state. The ATM values were interpolated from the ATM M_r vs. deviator stress plots at a deviator stress of 27.6 kPa (4 psi). All of the samples were

compacted at optimum moisture content and maximum dry density (standard proctor).

Figures 4-3 and 4-4 show the relationship between M_r from cyclic triaxial and ATM tests for the two different hammer weights of 10.9 N (2.5 lb) and 15.1 N (3.4 lb), respectively.

Madgett (1994) conducted a series of ATM tests on 6 of the 14 soils with a hammer weight = 10.9 N. Figure 4-5 shows the combined ATM results for the 14 soils tested with this hammer weight.

4.3.4 Analysis of the Test Results

Table 4-1 is a summary of the triaxial test and ATM results that were shown in Figure 4-5. In addition, the percent difference relative to triaxial tests is shown. With the exception of three cases, all of the ATM results were within 25% of the triaxial test data, which can be considered excellent. However, for the tests that showed differences greater than 25%, the differences were more than 50%. Perhaps some of the difference between the test results for these soils can be attributed to variations in compaction moisture content, since resilient modulus is highly dependent upon moisture content. The ATM results should be cross-checked with triaxial results from similar soils, if the ATM results seem unreasonable.

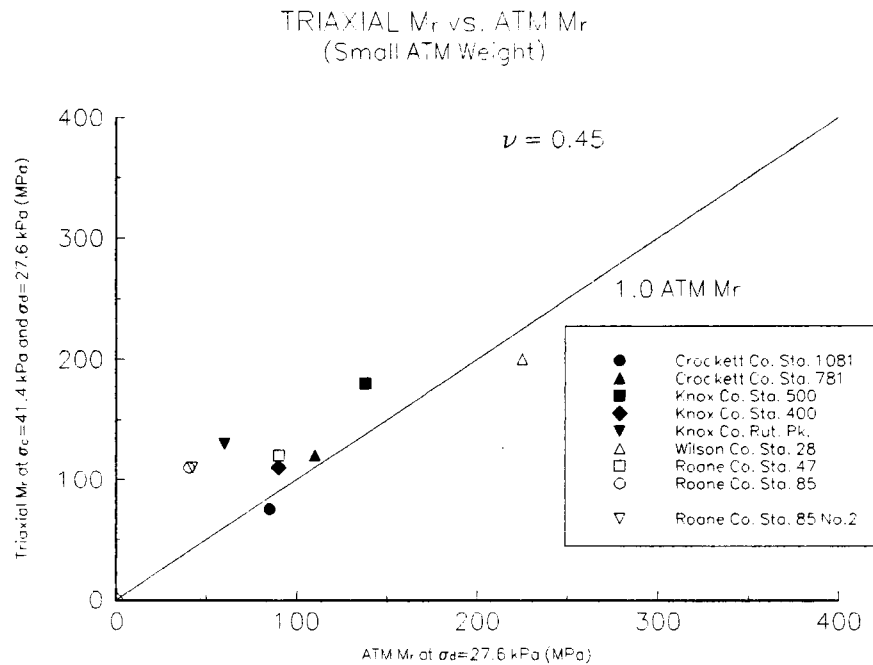


Figure 4-3. Predicted M_r From ATM Tests (Weight = 2.5 lb)

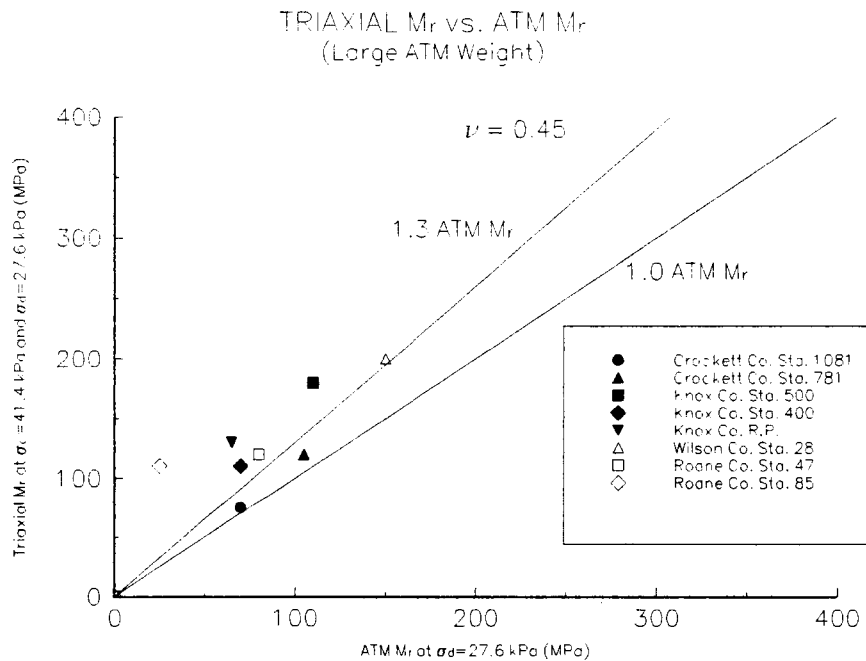


Figure 4-4. Predicted M_r From ATM Tests (Weight = 3.4 lb)

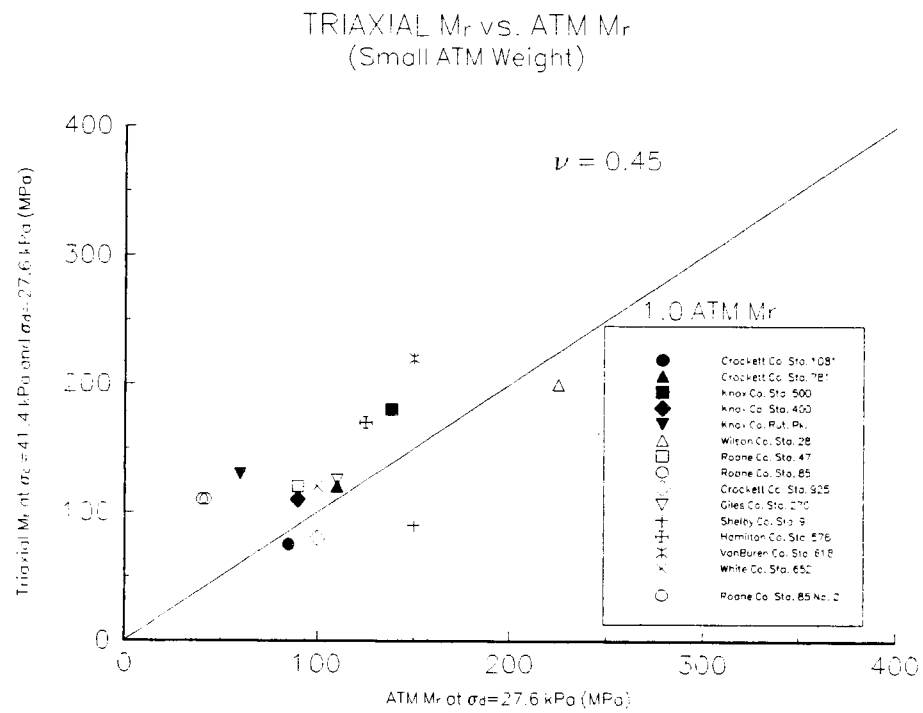


Figure 4-5. Predicted M_r From ATM Tests For All Soils

Table 4-1. Comparison of ATM and Triaxial Test Results

		Resilient Modulus, MPa, at 27.6 kPa Deviator Stress (Percent difference relative to triaxial test)	
Sample Location	AASHTO Classification	Triaxial Resilient Modulus @ Opt. w%	ATM Resilient Modulus@Opt. w%, w/small weight
Crockett Co. Sta. 1081	A-6	75	85 (+13%)
Crockett Co. Sta. 781	A-4	120	110 (-8.3%)
Crockett Co. Sta. 925	A-6	80	100 (+25%)
Giles Co. Sta. 270	A-7-5	125	110 (-12%)
Hamilton Co. Sta. 578	A-6	170	125 (-26%)
Knox Co. Sta. 500	A-7-6	180	140 (-22%)
Knox Co. Sta. 400	A-7-5	110	90 (-18%)
Knox Co. Rutledge Pike	A-7-6	130	60 (-54%)
Roane Co. Sta. 47	A-6	120	90 (-25%)
Roane Co. Sta. 85	A-4	110	40 (-64%)
Shelby Co. Sta. 9	A-4	90	150 (+67)
VanBuren Co. Sta. 618	A-7-6	200	150 (-25%)
White Co. Sta. 652	A-6	120	100 (-17%)
Wilson Co. Sta. 28	A-7-6	200	225 (+13%)

Chapter 5 Summary and Conclusions

5.1 Resilient Modulus Testing

The first part of this study involved the development of methods to estimate the reduction in subgrade resilient modulus with increasing moisture content. Triaxial testing was performed on eight different Tennessee subgrade soils that were compacted using kneading compaction techniques at optimum moisture content and maximum dry density, and later back-pressure saturated. It was explained how this type of sample preparation more closely simulates the soil structure of field compacted subgrades. As expected, the resilient modulus decreased with increasing moisture content. The amount of these reductions was shown to be strongly correlated with soil classifications. The AASHTO A-4 and A-6 soils were shown to be less susceptible to changes in resilient modulus with increases in moisture content, while the A-7-5 and A-7-6 soils were shown to have large reductions in resilient modulus. In all cases, the decrease in resilient modulus was shown to follow a somewhat linear relationship with degree of saturation.

Two different methods were proposed to estimate the reduction in resilient modulus: the average slopes of M_r vs. degree of saturation plots were calculated based on the triaxial test results, and a statistical model was also developed to estimate these slopes. The statistical model was shown to adequately predict these slopes for the range of soils tested. It was pointed out that these relationships are only applicable to soils compacted at optimum moisture content and maximum dry density, and later saturated to "wet" of optimum moisture content. The relationships are not applicable for determining increases in resilient modulus "dry" of optimum moisture content.

The resilient modulus prediction methods are consistent with the procedures provided in the 1993 AASHTO *Guide for the Design of Pavement Structures*. Using these procedures, with adequate knowledge of seasonal subgrade moisture changes, monthly resilient modulus values can be calculated for use in determining the effective roadbed resilient modulus.

The second part of this study dealt with making improvements and verifying the existing ATM procedure. A computer program was created to aid in the data acquisition steps of the ATM procedure, and also to perform ATM calculations. Also, the effects of using different hammer weights was explored. Eight soils were tested using the ATM procedure, and these results were compared with ATM tests performed by Madgett (1994) on six different soils. Comparisons of ATM tests and triaxial tests at similar stress states showed that the ATM procedure predicted the triaxial test results with 25% for most of the soils. However, three of the soils showed differences between the two tests methods of over 50%. Some of this variation was perhaps due to differences in compaction moisture contents. With the exception of those three soils, the ATM resilient modulus predictions can be considered quite good considering the difficulties associated with the determination of geological material properties.

References

References

- American Association of State and Highway Transportation Officials. (1993). "AASHTO Guide for the Design of Pavement Structures," AASHTO, Washington D.C., 403 pp.
- Boateng-Poku, Y. (1988). "Testing and Modeling of Subgrade Resilient Modulus for Tennessee Soils," Thesis presented in partial fulfillment for the Master of Science Degree, Department of Civil Engineering, The University of Tennessee, Knoxville, TN. 143 pp.
- Boyce, H.R. (1980). "A Non-Linear Model for the Elastic Behaviour of Granular Materials Under Repeated Loading," *International Symposium on Soils under Cyclic and Transient Loading*, Swansea, January 7-11, pp. 285-294.
- Drumm, E.C., Hudson, J., Li, Z., Madgett, M., and Baker, D. (1993a) "Resilient Response of Tennessee Subgrades Volume 1: Laboratory Testing, Modeling, and Alternative Test Method," Department of Civil and Environmental Engineering, The University of Tennessee, Knoxville, TN.
- Drumm, E.C., Reeves, J.S., and Madgett, M.R. (1995). "Resilient Response of Tennessee Subgrades Phase II: Verification of Prediction Methods," Department of Civil and Environmental Engineering, The University of Tennessee, Knoxville, TN. 89 pp.
- Elfino, M.K. and Davidson, J.L. (1989). "Modeling Field Moisture Content in Resilient Moduli Testing," *Resilient Moduli of Soils: Laboratory Conditions*, Geotechnical Special publication No. 24, American Society of Civil Engineers, York, N.Y., pp 31-51.
- Elliot, R.P. and Thornton, S.I. (1986). "Resilient Modulus - What Is It?" *Proceedings of the 37th Annual Highway Geology Symposium*, Helena, Montana, pp. 267-282.
- Elliot, R.P. and Thornton, S.I. (1988). "Resilient Modulus and AASHTO Pavement Design," *Transportation Research Board Record 1196*, Transportation Research Board, Washington D.C., pp. 116-124.
- Holtz, R.D. and Kovacs, W.D. (1981). *An Introduction to Geotechnical Engineering*, Prentice Hall, Englewood Cliffs, New Jersey, 733 pp.
- Huang, Y.H. (1993). *Pavement Analysis and Design*, Prentice-Hall, Inc. Englewood Cliffs, New Jersey, 805 pp.

- Hudson, J.M. (1992). "The Resilient Response of Fine-Grained Tennessee Subgrade Soil," Thesis presented in partial fulfillment of the Master of Science Degree, The University of Tennessee, Knoxville, TN, 186 pp.
- Heuklelom W. and Klomp, A.J.G. (1962). "Dynamic Testing as a Means of Controlling Pavement During and After Construction," Proceedings, First International Conference on Structural Design of Asphalt Pavement, University of Michigan.
- Johnson, T.D. (1986). "The Resilient Moduli of Subgrade Soil in the East Tennessee Area," Thesis presented in partial fulfillment of the Master of Science Degree, Department of Civil Engineering, The University of Tennessee, Knoxville, TN. 124 pp.
- Lamb, T.W. and Whitman, R.V. (1969). *Soil Mechanics*, John Wiley and Sons, Inc. 553 pp.
- Lee, W. , Bohra, N., and Altschaeffl, A.G. (1992) Discussion of "Estimation of Subgrade Resilient Modulus from Standard Tests," *Journal of Geotechnical Engineering*, ASCE, Vol. 118, No. 2, pp. 352-354.
- Li, D. and Selig, E.T. (1994). "Resilient Modulus for Fine-Grained Subgrade Soil," *Journal of Geotechnical Engineering*, American Society of Civil Engineers, Vol. 120, No. 6, June, pp. 939.
- Li, Z. (1992). "An Alternative Test Method for Resilient Modulus of Fine Grained Subgrades," Thesis presented in partial fulfillment of the requirements for the Master of Science Degree, Department of Civil Engineering, The University of Tennessee, Knoxville, TN, 90 pp.
- Madgett, M. R. (1994). "Verification Testing and Alternative Test Methods for the Resilient Modulus of Fine Grained Soils," Thesis presented in partial fulfillment of the requirements for the Master of Science Degree, The University of Tennessee, Knoxville, TN, 155 pp.
- Monismith, C. L. (1992). "Analytically-Based Asphalt Pavement Design and Rehabilitation- Theory to Practice (1962-1992)," TRB Distinguished Lecture, *Transportation Research Board, 71st Annual Meeting*, Washington, D.C., January 12-16.
- Pezo, R. and Hudson, W.R. (1994). "Prediction Models of Resilient Modulus for Nongranular Materials," *ASTM Geotechnical Testing Journal*, GTODJ, Vol. 17, No. 3, September, pp. 349-355.

- Prapahran, S., White, D.M. and Altschaeffl, A.G. (1991). "Fabric of Field- and Laboratory-Compacted Clay," *Journal of Geotechnical Engineering*, American Society of Civil Engineers, Vol. 117, No. 12, December, pp. 1934-1940.
- Seed, H.B., Chan, C.K., and Lee, C.E. (1962). "Resilience Characteristics of Subgrade Soils and Their Relation to Fatigue Failures in Asphalt Pavements," *Proceedings, International Conference on the Structural Design of Asphalt Pavements*, University of Michigan, Ann Arbor, August 20-24, pp. 611-636.
- Strategic Highway Research Program (1989). "Resilient Modulus of Unbound Granular Base/Subbase Materials and Subgrade Soils," SHRP Protocol P-46, UG07, SS07.
- Thadmakalla, G.B. and George, K.P. (1995). "Characterization of Subgrade Soils at Simulated Field Moisture," *Preprint, Transportation Research Board, 74th Annual Meeting*, January, Washington, D.C., pp. 1-26.
- Thompson, M.R. and Robnett, Q.L. (1973). "Interim Report, Resilient Properties of Subgrade Soils, Phase I - Development of Testing Procedure," *Civil Engineering Series, Transportation Engineering Series No. 5*, Illinois Cooperative Highway Program, Series No. 139, May, 25 pp.
- Thompson, M.R. and Robnett, Q.L. (1976). "Final Report, Resilient Properties of Subgrade Soils," *Civil Engineering Series, Civil Engineering Studies Transportation Engineering Series No. 14*, Illinois Cooperative Highway and Transportation Series No. 160, 59 pp.
- Thompson, M.R. and Robnett, Q.L. (1979). "Resilient Properties of Subgrade Soils," *Transportation Engineering Journal*, American Society of Civil Engineers, Vol 105, No. TE1, January. pp. 71-89.
- Yoder, E.J. and Witczak, M.W. (1975). *Principles of Pavement Design*, John Wiley & Sons Inc., New York, 711 pp.

Appendices

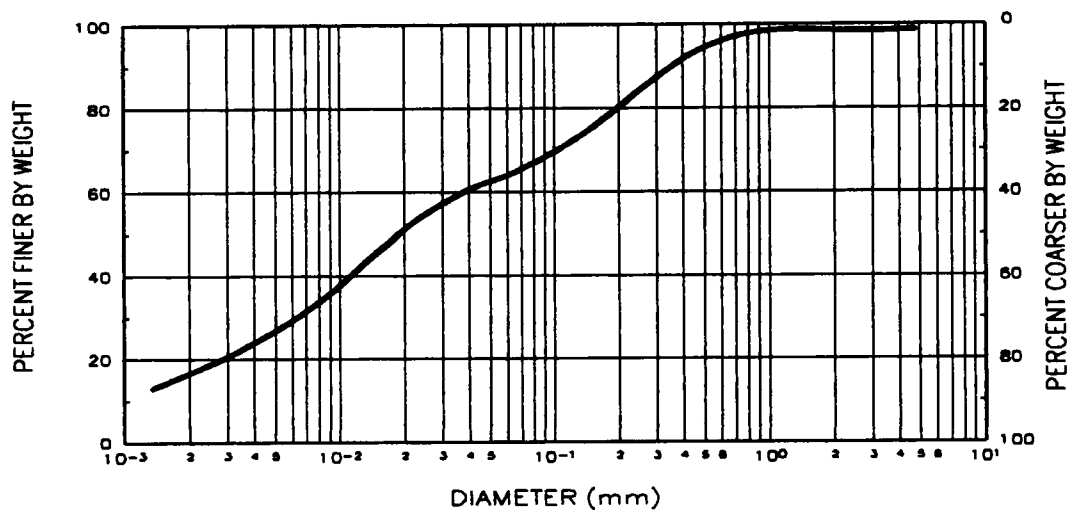
Appendix A

A-4 (0): Roane County Station 85

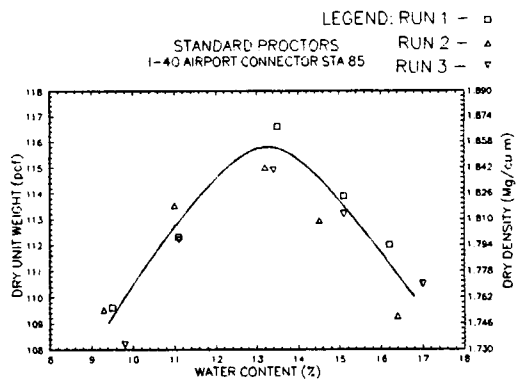
LOCATION: ROANE COUNTY STATION 85	
SOIL CLASSIFICATION	
AASHTO	USCS
A-4 (0)	CL-ML

CLAY CONTENT (%)	PASSING #200 SIEVE (%)	ATTERBERG LIMITS			SPECIFIC GRAVITY OF SOLIDS	STANDARD PROCTORS	
		LL	PL	PI		max. ρ_d (Mg/m ³)	opt. w (%)
16.7	56.1	21	16	5	2.60	1.859	13.2

GRAIN SIZE DISTRIBUTION
AIRPORT CONNECTOR STATION 85



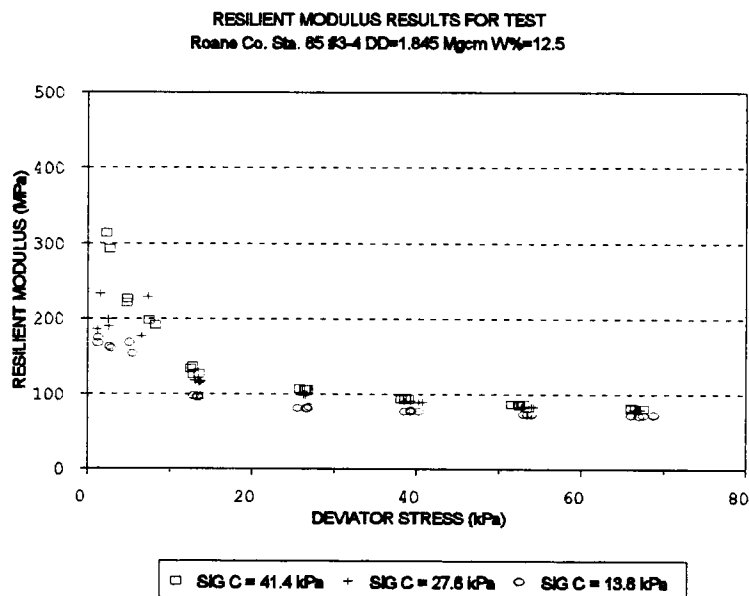
Grain Size Distribution



Standard Proctor Moisture-Density Relationship

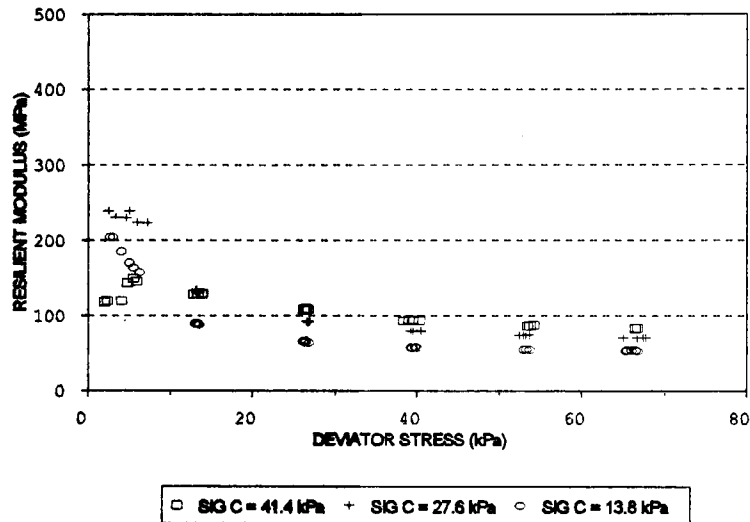
Hyperbolic and Power Model Parameters

TEST NO.	DRY DENSITY (Mg/m ³)	WATER CONTENT (%)	HYPERBOLIC MODEL			POWER MODEL		
			a (10 ⁹ Pa)	b (Mpa)	r ²	K ₁	K ₂	r ²
3-4	1.845	12.5	580.5	70.57	.97	258.6	-0.2918	.82
1-8	1.843	13.1-13.5	659.0	60.49	.85	276.2	-0.3399	.70
1-7	1.873	13.0-13.5	298.5	44.46	.86	157.6	-0.2990	.70



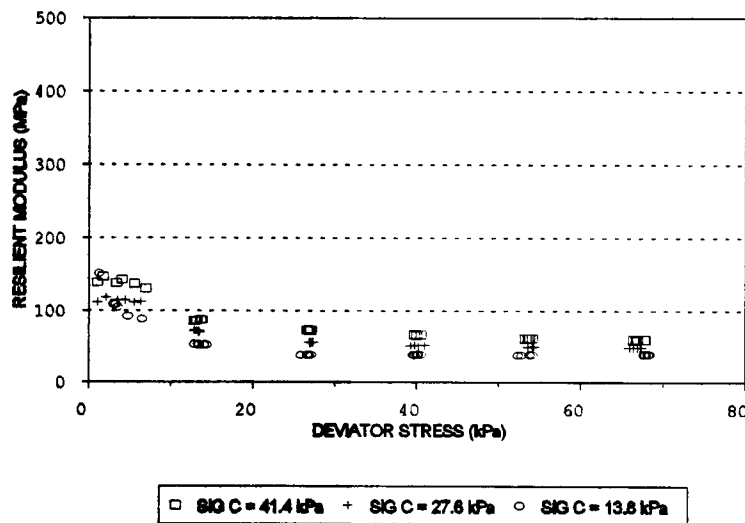
Resilient Modulus vs. Deviator Stress for Roane County Sta. 85
Soil Compacted at Optimum Moisture Content and Maximum Dry
Density (Standard Proctor), with a Degree of Saturation of 79.4%.

RESILIENT MODULUS RESULTS FOR TEST
Roane Co. Sta 85 #1-8 DD=1.843 Mgcm W%=13.5



Resilient Modulus vs. Deviator Stress for Roane County Sta. 85 Soil Compacted at Optimum Moisture Content and Maximum Dry Density (Standard Proctor), the Increasing the Degree of Saturation from 83.7% to 85.4%.

RESILIENT MODULUS RESULTS FOR TEST
Roane Co. Sta. 85 #1-7 DD=1.873 Mgcm W%=13.5



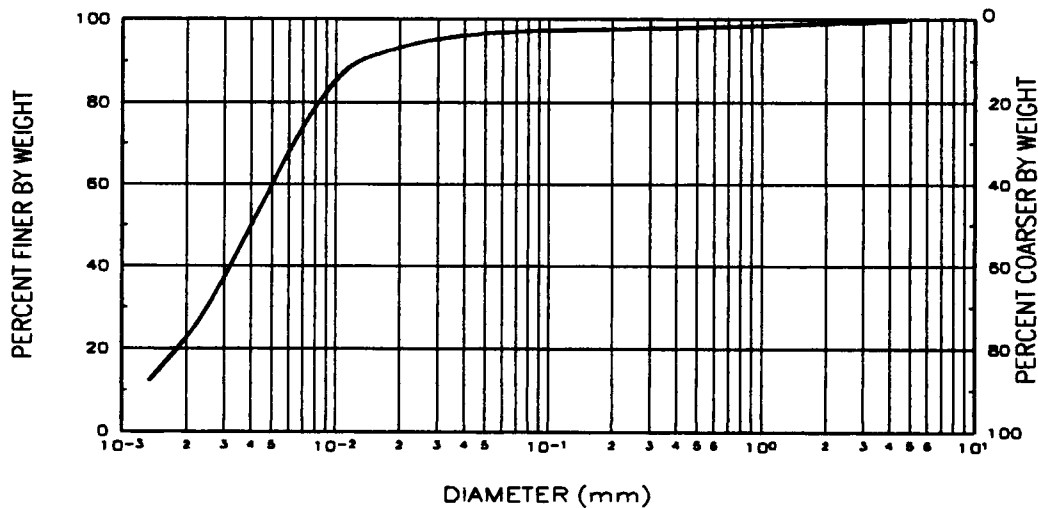
Resilient Modulus vs. Deviator Stress for Roane County Sta. 85 Soil Compacted at Optimum Moisture Content and Maximum Dry Density (Standard Proctor), then Increasing the Degree of Saturation from 86.4% to 90.5%.

A-4 (0): Crockett County Station 781

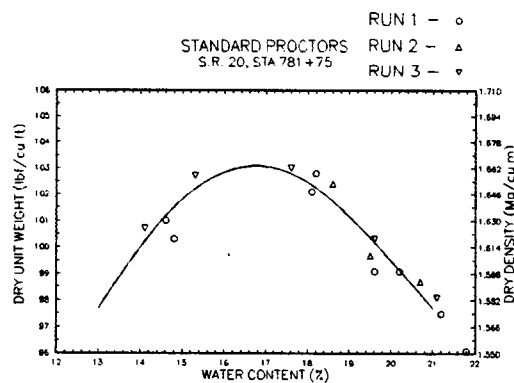
LOCATION: CROCKETT COUNTY STATION 781+75	
SOIL CLASSIFICATION	
AASHTO	USCS
A-4 (0)	ML

CLAY CONTENT (%)	PASSING #200 SIEVE (%)	ATTERBERG LIMITS			SPECIFIC GRAVITY OF SOLIDS	STANDARD	
		LL	PL	PI		max. ρ_d (Mg/m ³)	opt. w (%)
22.9	97.2	24	22	2	2.59	1.655	16.8

GRAIN SIZE DISTRIBUTION
S.R. 20 STA 781 + 75



Grain Size Distribution

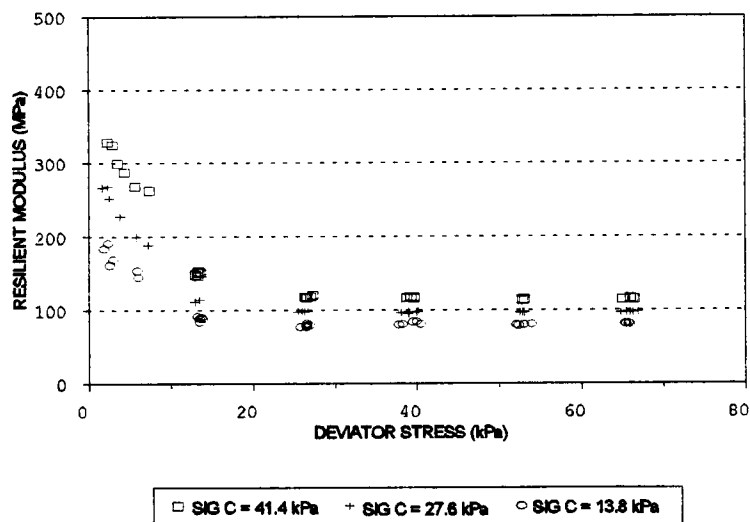


Standard Proctor Moisture-Density Relationship

Hyperbolic and Power Model Parameters

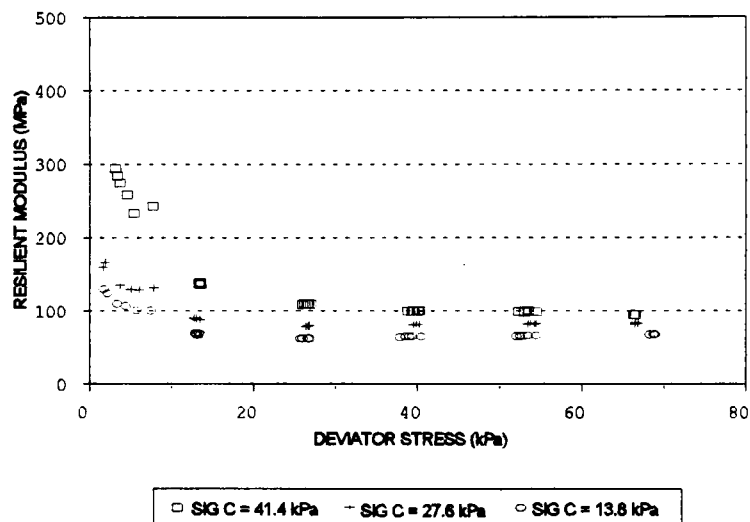
TEST NO.	DRY DENSITY (Mg/m ³)	WATER CONTENT (%)	HYPERBOLIC MODEL			POWER MODEL		
			a (10 ⁹ Pa)	b (Mpa)	r ²	K ₁	K ₂	r ²
3-3	1.668	16.3	384.1	90.99	.92	285.6	-.2860	.65
3-4	1.668	16.6-18.0	311.4	76.95	.90	198.3	-.2335	.44
3-2	1.675	16.7-18.9	55.09	71.95	.92	113.7	-.0124	.32

RESILIENT MODULUS FOR TEST
Crockett County Sta. 781 #3-3 DD=1.668 Mg/m³ Wt%=16.3



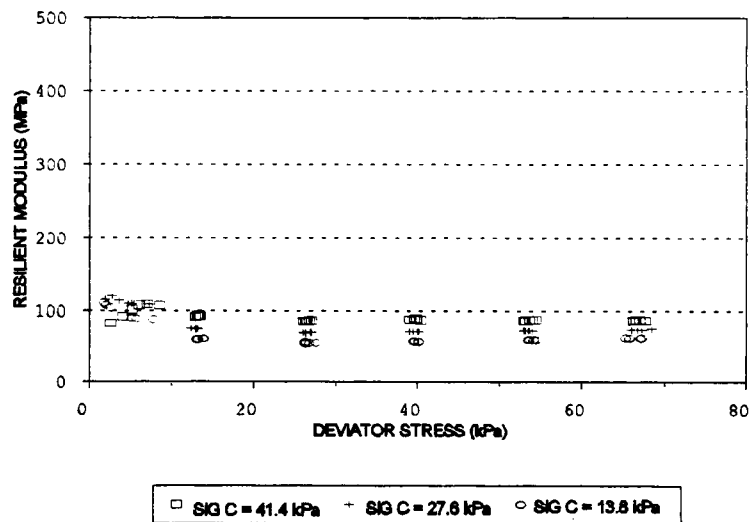
Resilient Modulus vs. Deviator Stress for Crockett County Sta. 781+75 Soil Compacted at Optimum Moisture Content and Maximum Dry Density (Standard Proctor), with a Degree of Saturation of 76.4%.

RESILIENT MODULUS RESULTS FOR TEST
Crockett Co. Sta 781 #3-4 DD=1.688 Mg/cm W%= 18.0



Resilient Modulus vs. Deviator Stress for Crockett County Sta. 781+75 Soil Compacted at Optimum Moisture Content and Maximum Dry Density (Standard Proctor), then Increasing the Degree of Saturation from 78.7% to 84.4%.

RESILIENT MODULUS RESULTS FOR TEST
Crockett Co. Sta 781 #3-2 DD=1.689 Mg/cm W%=18.9



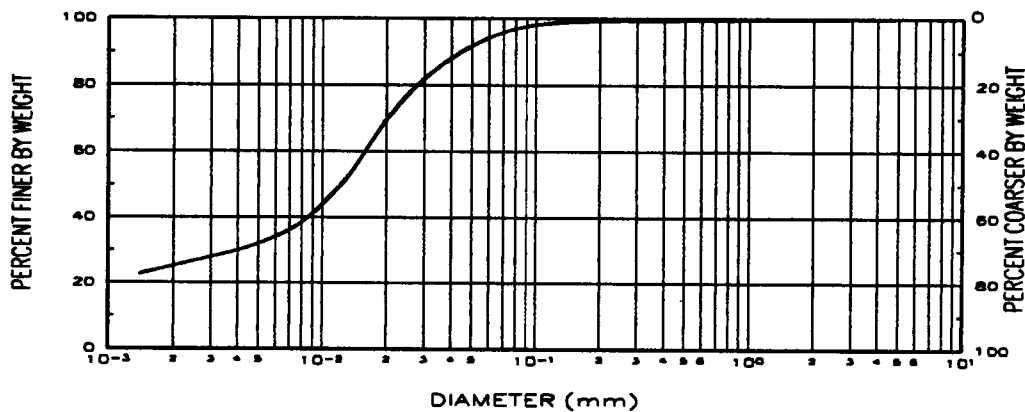
Resilient Modulus vs. Deviator Stress for Crockett County Sta. 781+75 Soil Compacted at Optimum Moisture Content and Maximum Dry Density (Standard Proctor), then Increasing the Degree of Saturation from 79.4% to 89.6%.

A-6 (15): Crockett County Station 1081

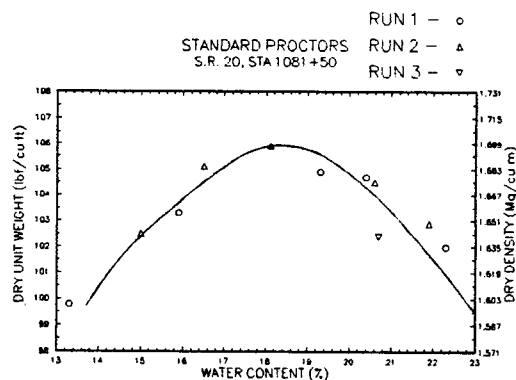
LOCATION: CROCKETT COUNTY STATION 1081+50	
SOIL CLASSIFICATION	
AASHTO	USCS
A-6 (15)	CL

CLAY CONTENT (%)	PASSING #200 SIEVE (%)	ATTERBERG LIMITS			SPECIFIC GRAVITY OF SOLIDS	STANDARD	
		LL	PL	PI		max. ρ_d (Mg/m ³)	opt. w (%)
25.4	98.9	36	22	14	2.62	1.697	18.4

GRAIN SIZE DISTRIBUTION
S.R. 20 STA 1081 +50



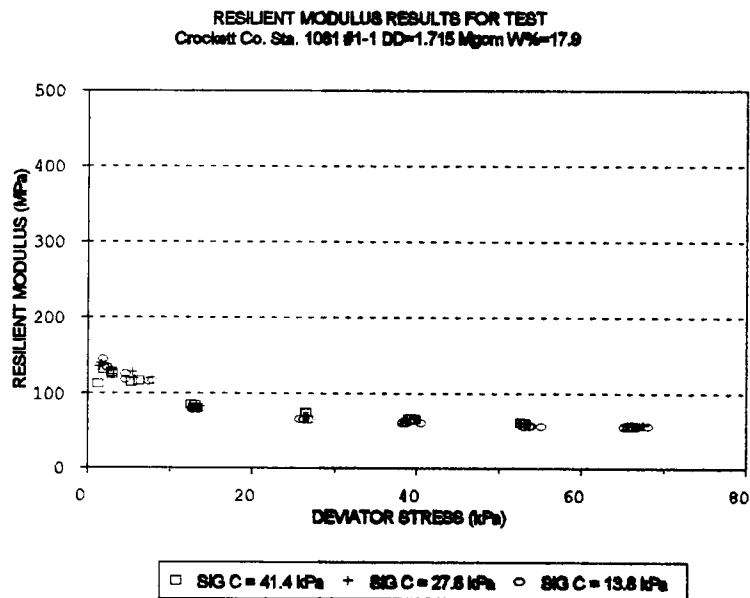
Grain Size Distribution



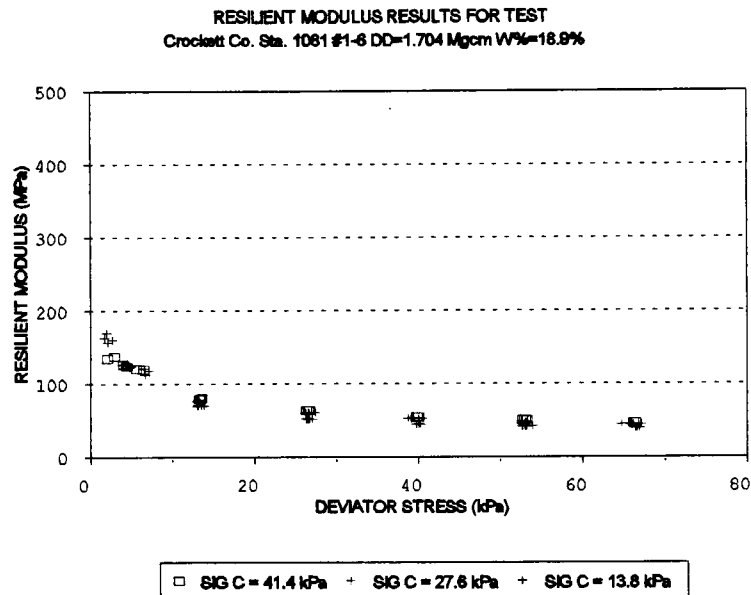
Standard Proctor Moisture-Density Relationship

Hyperbolic and Power Model Parameters

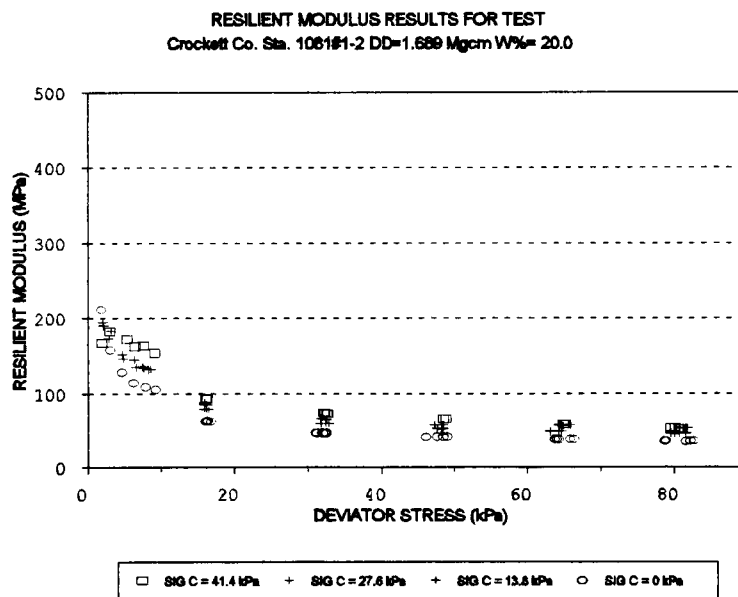
TEST NO.	DRY DENSITY (Mg/m ³)	WATER CONTENT (%)	HYPERBOLIC MODEL			POWER MODEL		
			a (10 ⁹ Pa)	b (Mpa)	r ²	K ₁	K ₂	r ²
1-1	1.715	17.9	363.6	53.43	.99	163.5	-.2511	.95
1-6	1.704	18.5-18.9	468.9	38.44	.96	214.5	-.3836	.96
1-2	1.689	18.6-20.0	663.7	43.49	.96	274.9	-.3951	.94



Resilient Modulus vs. Deviator Stress for Crockett County Sta. 1081+50 Soil Compacted at Optimum Moisture Content and Maximum Dry Density (Standard Proctor), with a Degree of Saturation of 91.8%.



Resilient Modulus vs. Deviator Stress for Crockett County Sta. 1081+50 Soil Compacted at Optimum Moisture Content and Maximum Dry Density (Standard Proctor), then Increasing the Degree of Saturation from 90.1% to 93.3%.



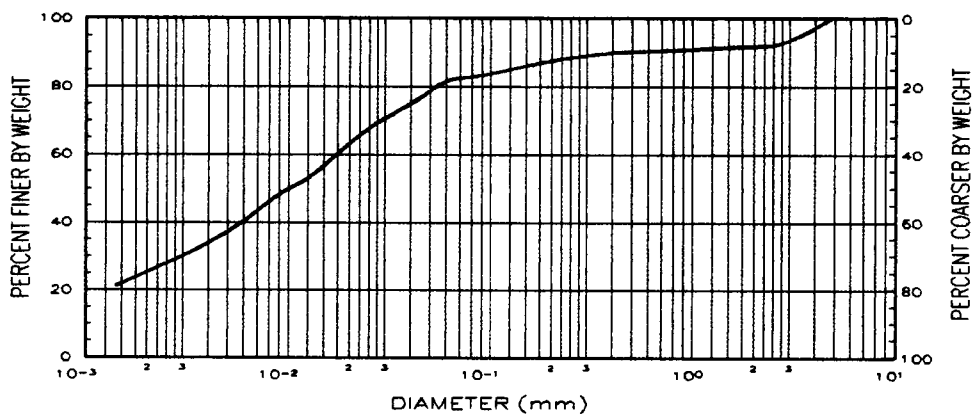
Resilient Modulus vs. Deviator Stress for Crockett County Sta. 1081+50 Soil Compacted at Optimum Moisture Content and Maximum Dry Density (Standard Proctor), then Increasing the Degree of Saturation from 88.4% to 95.6%.

A-6 (8): Roane County Station 47

LOCATION: ROANE COUNTY STATION 47	
SOIL CLASSIFICATION	
AASHTO	USCS
A-6 (8)	CL

CLAY CONTENT (%)	PASSING #200 SIEVE (%)	ATTERBERG LIMITS			SPECIFIC GRAVITY OF SOLIDS	STANDARD PROCTORS	
		LL	PL	PI		max. ρ_d (Mg/m ³)	opt. w (%)
25.4	72.3	34	21	13	2.65	1.755	18.2

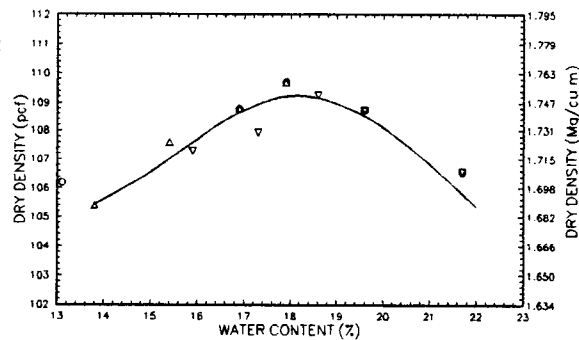
GRAIN SIZE DISTRIBUTION
AIRPORT CONNECTOR STATION 47



STANDARD PROCTORS
1-40 AIRPORT CONNECTOR STA 47

RUN 1 - ○
RUN 2 - △
RUN 3 - ▽

Grain Size Distribution

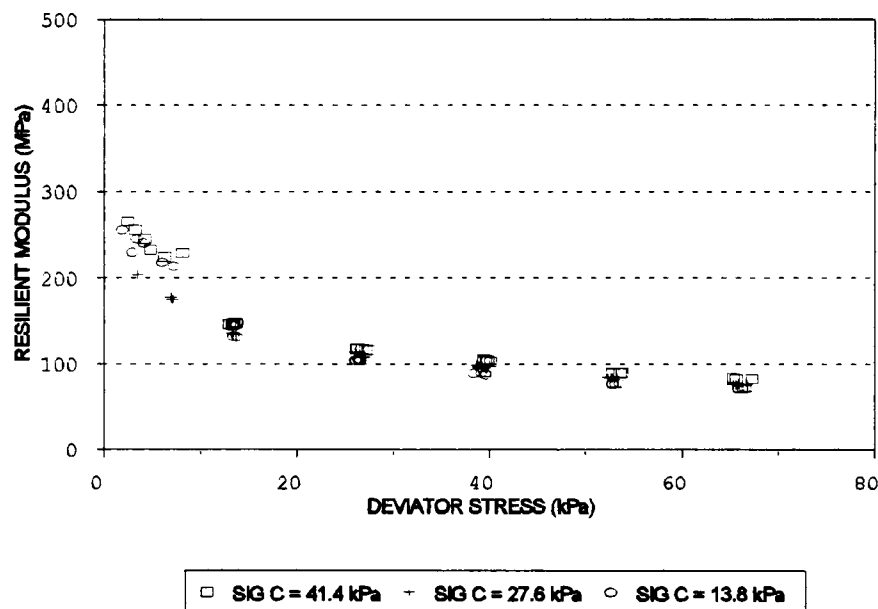


Standard Proctor Moisture-Density
Relationship

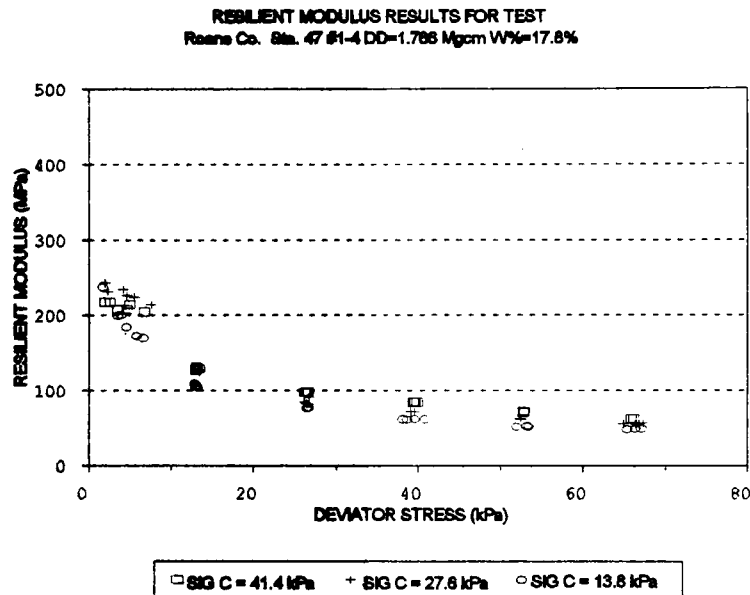
Hyperbolic and Power Model Parameters

TEST NO.	DRY DENSITY Y (Mg/m ³)	WATER CONTENT (%)	HYPERBOLIC MODEL			POWER MODEL		
			a (10 ⁹ Pa)	b (Mpa)	r ²	K ₁	K ₂	r ²
1-3	1.747	17.3	961.4	66.33	.96	383.4	-.3796	.96
1-4	1.766	17.4-17.	898.3	45.27	.99	378.9	-.4528	.92
1-2	1.777	17.4-17.	868.4	38.57	.88	383.7	-.4870	.93

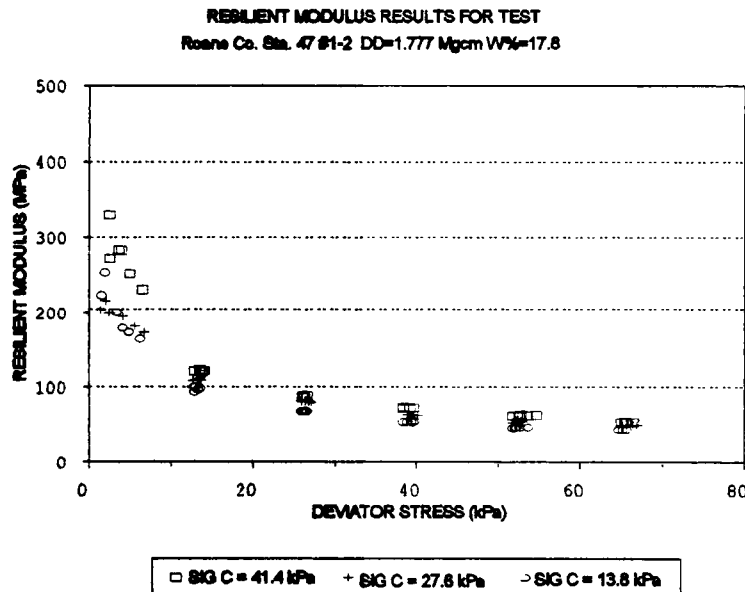
RESILIENT MODULUS RESULTS FOR TEST
Roane Co. Sta. 47 #1-3 DD=1.747 Mgcm W%=17.3



Resilient Modulus vs. Deviator Stress for Roane County Sta. 47 Soil Compacted at Optimum Moisture Content and Maximum Dry Density (Standard Proctor), with a Degree of Saturation of 88.2%.



Resilient Modulus vs. Deviator Stress for Roane County Sta. 47 Soil Compacted at Optimum Moisture Content and Maximum Dry Density (Standard Proctor), then Increasing the Degree of Saturation from 93.9% to 94.2%.



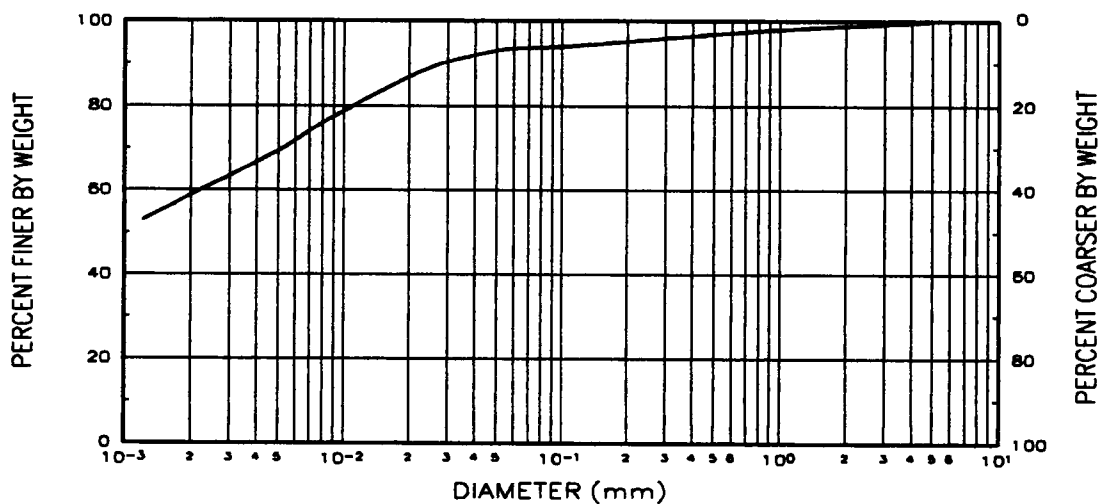
Resilient Modulus vs. Deviator Stress for Roane County Sta. 47 Soil Compacted at Optimum Moisture Content and Maximum Dry Density (Standard Proctor), then Increasing the Degree of Saturation from 92.6% to 96.1%.

A-7-5 (36) Knox County Station 400

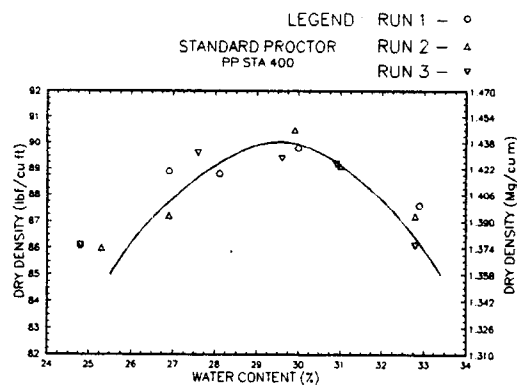
LOCATION: KNOX COUNTY STATION 400	
SOIL CLASSIFICATION	
AASHTO	USCS
A-7-5 (36)	MH

CLAY CONTENT (%)	PASSING #200 SIEVE (%)	ATTERBERG LIMITS			SPECIFIC GRAVITY OF SOLIDS	STANDARD	
		LL	PL	PI		max. ρ_d (Mg/m ³)	opt. w (%)
58.6	94.2	70	41	29	2.61	1.447	29.6

GRAIN SIZE DISTRIBUTION
PELLISSIPPI PARKWAY STA 400



Grain Size Distribution

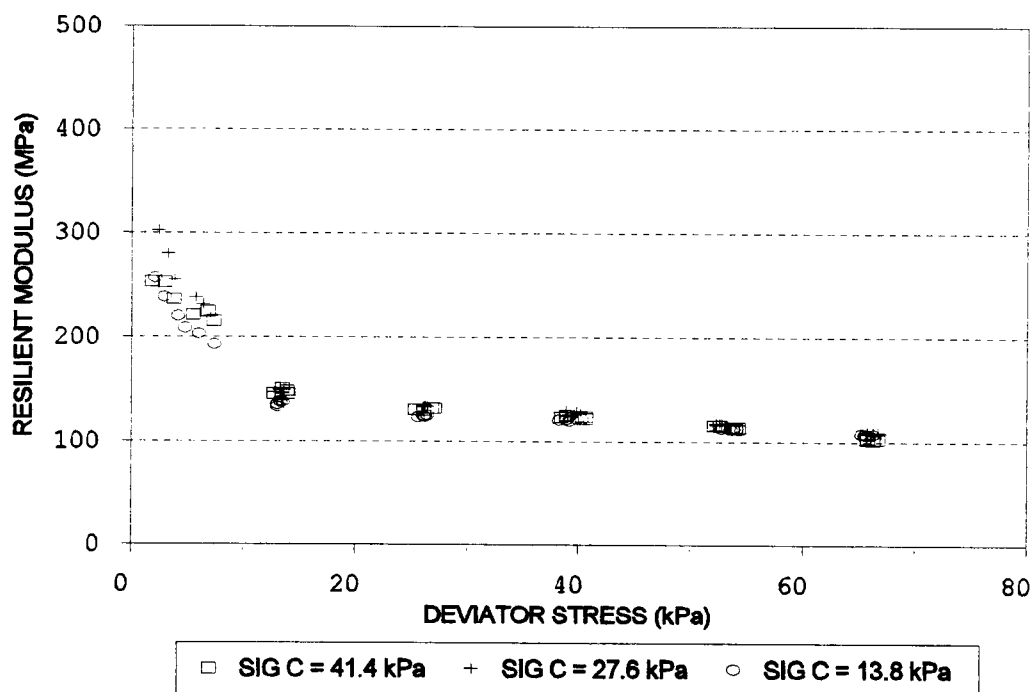


Standard Proctor Moisture-Density Relationship

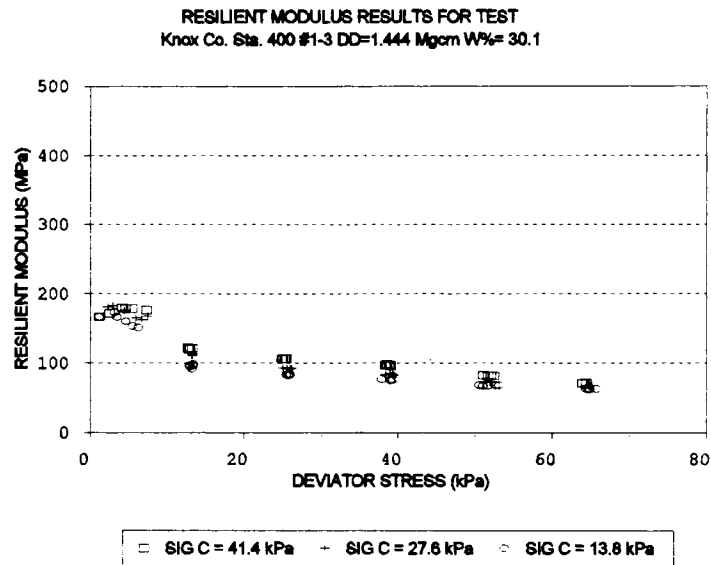
Hyperbolic and Power Model Parameters

TEST NO.	DRY DENSITY (Mg/m ³)	WATER CONTENT (%)	HYPERBOLIC MODEL			POWER MODEL		
			a (10 ⁹ Pa)	b (Mpa)	r ²	K ₁	K ₂	r ²
1-4	1.449	29.4	687.6	100.2	.99	325.3	-.2695	.92
1-3	1.444	29.9-30.1	663.2	61.64	.95	246.2	-.2987	.89
2-2	1.446	29.5-30.6	487.1	42.03	.97	194.7	-.3324	.95

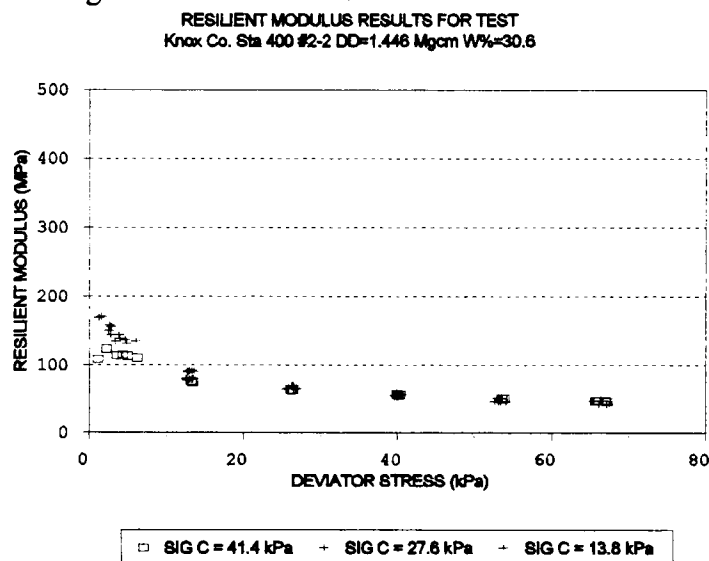
RESILIENT MODULUS RESULTS FOR TEST
Knox Co. Sta 400 #1-4 DD=1.449 Mgcm W%=29.4



Resilient Modulus vs. Deviator Stress for Knox County Sta. 400 Soil Compacted at Optimum Moisture Content and Maximum Dry Density (Standard Proctor), with a Degree of Saturation of 91.9%.



Resilient Modulus vs. Deviator Stress for Knox County Sta. 400 Soil Compacted at Optimum Moisture Content and Maximum Dry Density (Standard Proctor), then Increasing the Degree of Saturation to 93.4%.



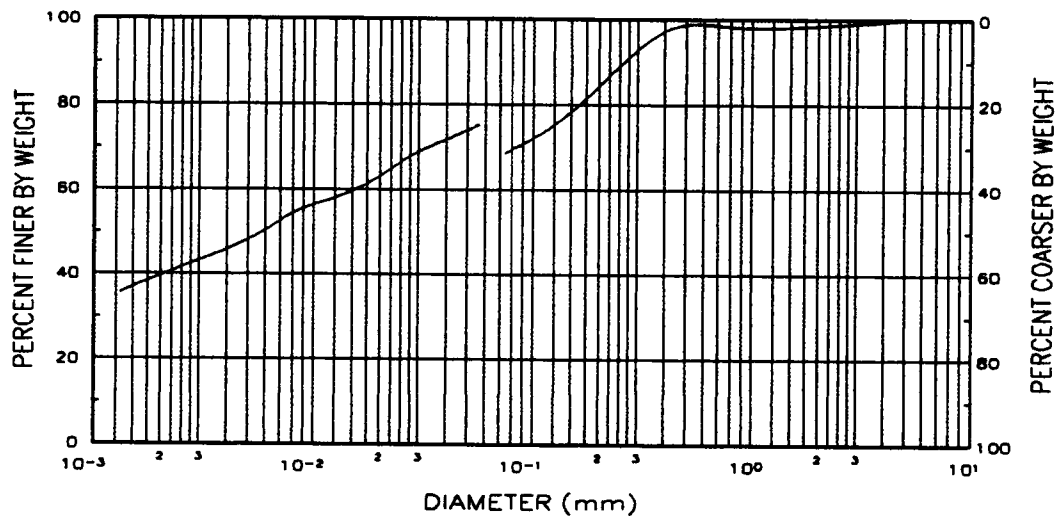
Resilient Modulus vs. Deviator Stress for Knox County Sta. 400 Soil Compacted at Optimum Moisture Content and Maximum Dry Density (Standard Proctor), then Increasing the Degree of Saturation from 91.3% to 95.4%.

A-7-6 (15): Knox County Station 500

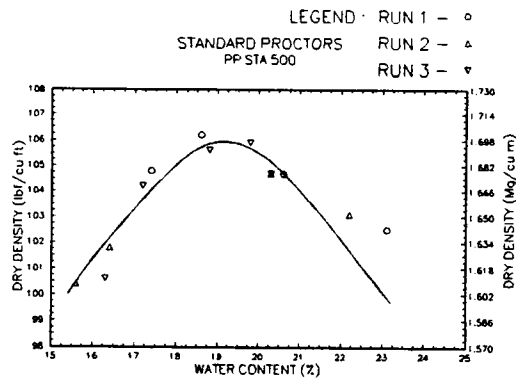
LOCATION: KNOX COUNTY STATION 500	
SOIL CLASSIFICATION	
AASHTO	USCS
A-7-6 (15)	CL

CLAY CONTENT (%)	PASSING #200 SIEVE (%)	ATTERBERG LIMITS			SPECIFIC GRAVITY OF SOLIDS	STANDARD PROCTORS	
		LL	PL	PI		max. ρ_d (Mg/m ³)	opt. w (%)
39.9	70.4	45	22	23	2.70	1.704	19.2

PELLISSIPPI PARKWAY STA 500



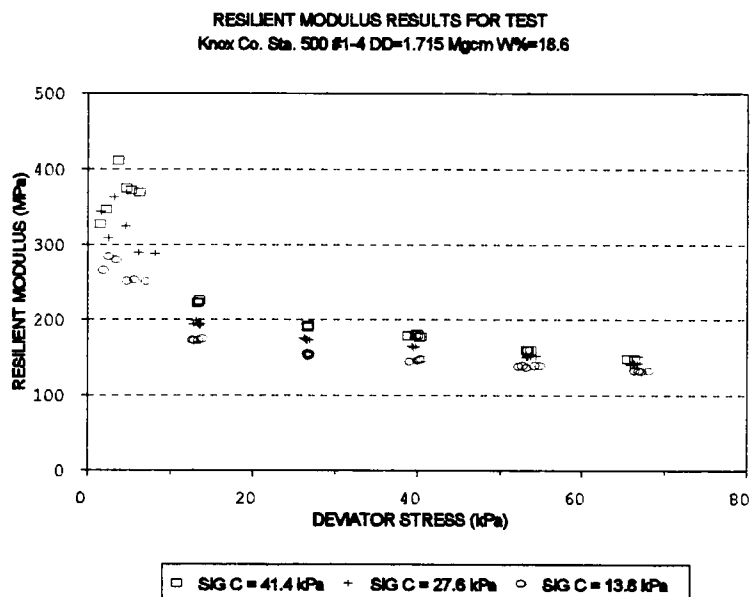
Grain Size Distribution



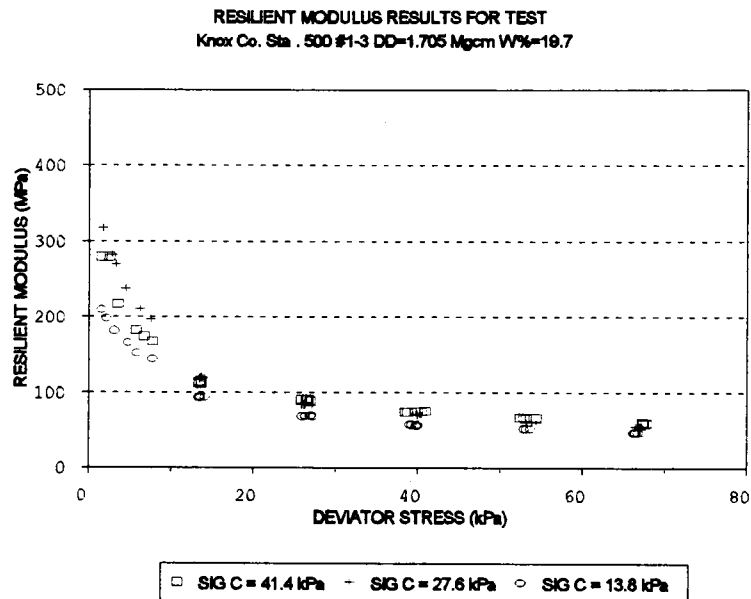
Standard Proctor Moisture-Density Relationship

Hyperbolic and Power Model Parameters

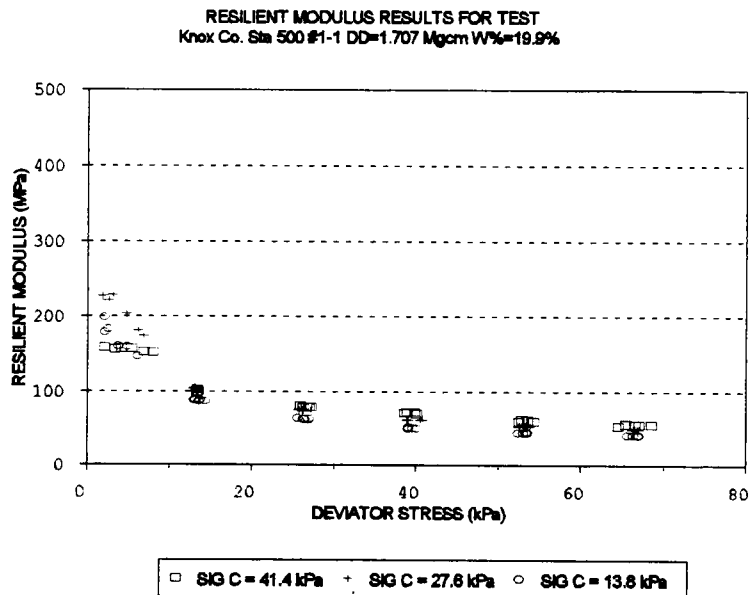
TEST NO.	DRY DENSITY (Mg/m ³)	WATER CONTENT (%)	HYPERBOLIC			POWER MODEL		
			a (10 ⁹ Pa)	b (MPa)	r ²	K ₁	K ₂	r ²
1-4	1.715	18.6	935.5	131.8	.97	414.3	-.2577	.85
1-3	1.705	19.2-19.7	834.3	44.08	.86	374.1	-.4620	.93
1-1	1.707	19.2-19.9	712.8	39.31	.89	310.1	-.4459	.93



Resilient Modulus vs. Deviator Stress for Knox County Sta. 500
Soil Compacted at Optimum Moisture Content and Maximum Dry
Density (Standard Proctor), with a Degree of Saturation of 87.4%.



Resilient Modulus vs. Deviator Stress for Knox County Sta. 500 Soil Compacted at Optimum Moisture Content and Maximum Dry Density (Standard Proctor), then Increasing the Degree of Saturation from 87.7% to 91.2%.

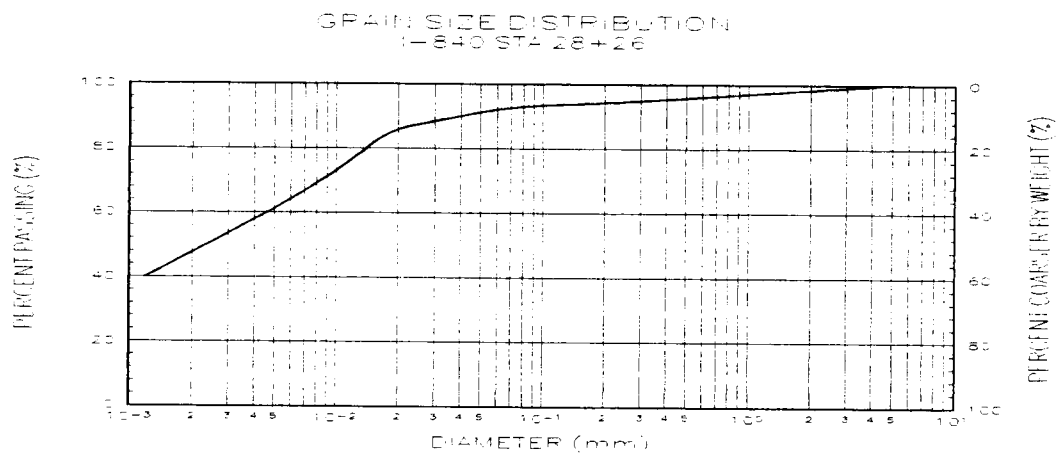


Resilient Modulus vs. Deviator Stress for Knox County Sta. 500 Soil Compacted at Optimum Moisture Content and Maximum Dry Density (Standard Proctor), then Increasing the Degree of Saturation from 85.7% to 92.3%.

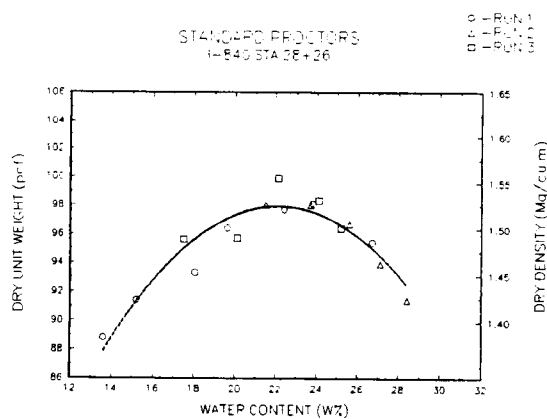
A-7-6 (36): Wilson County Station 28

LOCATION: WILSON COUNTY STATION 28	
SOIL CLASSIFICATION	
AASHTO	USCS
A-7-6 (36)	CH

CLAY CONTENT (%)	PASSING #200 SIEVE (%)	ATTERBERG LIMITS			SPECIFIC GRAVITY OF SOLIDS	STANDARD PROCTORS	
		LL	PL	PI		max. ρ_d (Mg/m ³)	opt. w (%)
44.5	92.7	55	19	36	2.84	1.569	22.0



Grain Size Distribution

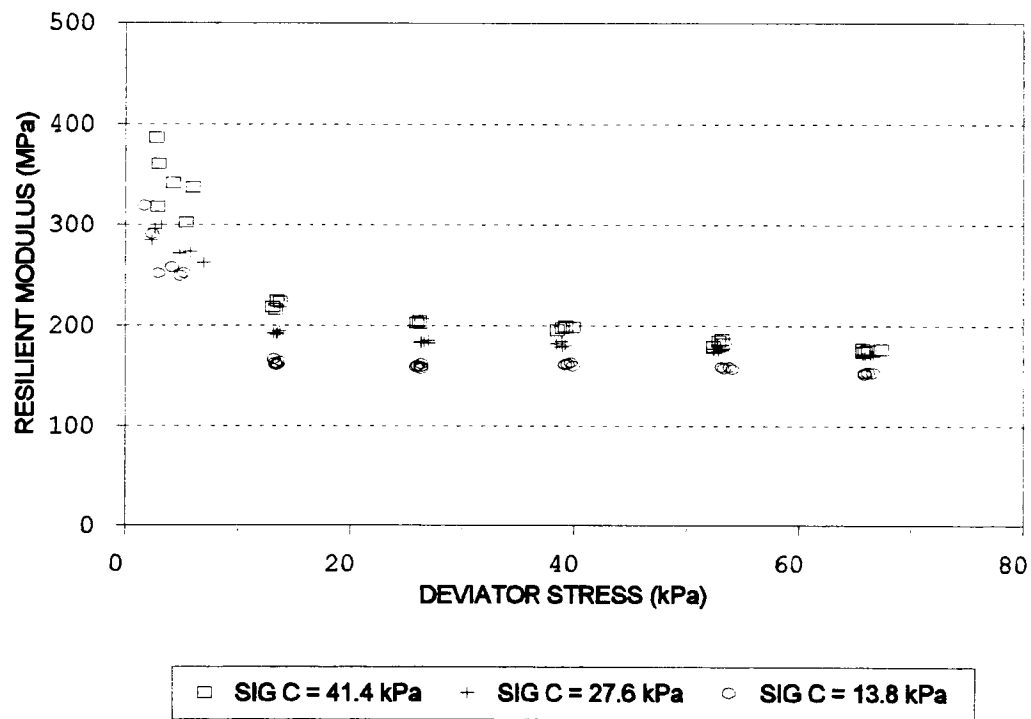


Standard Proctor Moisture-Density Relationship

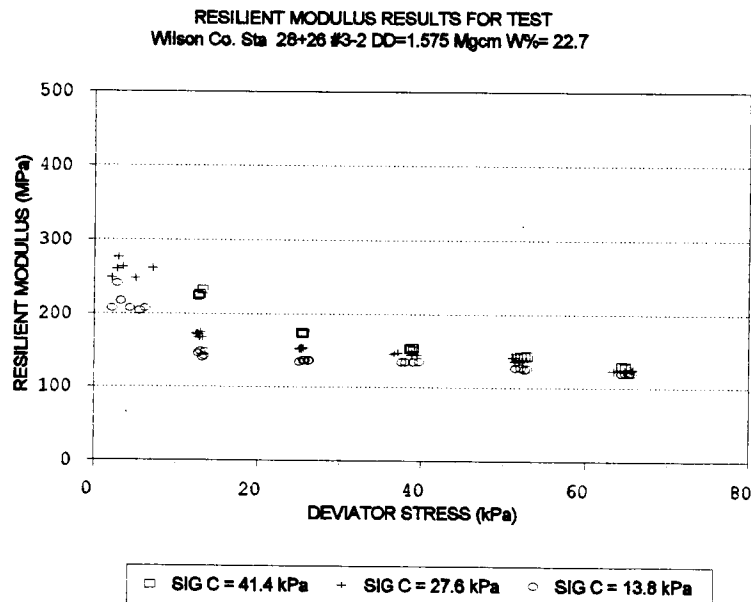
Hyperbolic and Power Model Parameters

TEST NO.	DRY DENSITY (Mg/m ³)	WATER CONTENT (%)	HYPERBOLIC MODEL			POWER MODEL		
			a (10 ⁹ Pa)	b (MPa)	r ²	K ₁	K ₂	r ²
3-3	1.580	22.2	535.7	160.9	.98	354.8	-.1900	.74
3-2	1.575	22.3-22.7	793.4	117.5	.97	306.9	-.2108	.78
1-6	1.550	20.7-24.0	656.3	110.6	.98	304.8	-.2308	.79
1-7	1.532	20.7-25.9	500.2	60.46	.97	185.8	-.2414	.88

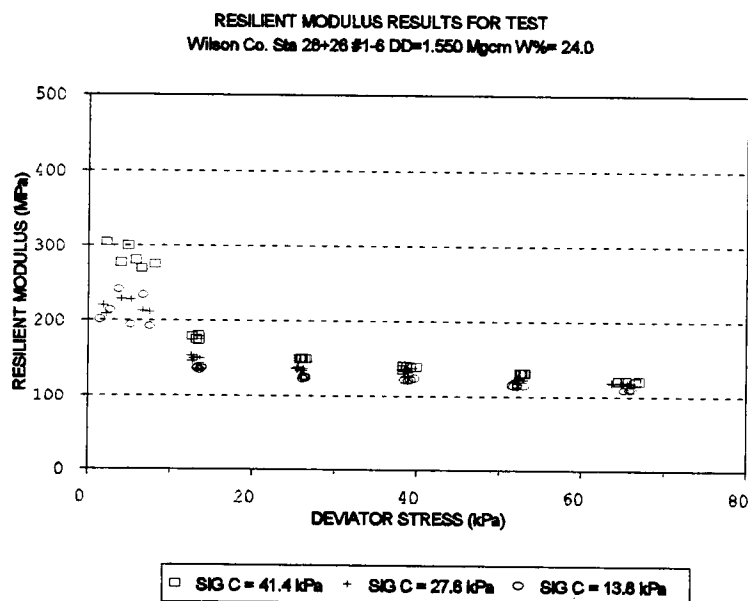
RESILIENT MODULUS RESULTS FOR TEST
Wilson Co. Sta. 28 #3-3 DD=1.573 Mgcm W%=22.5



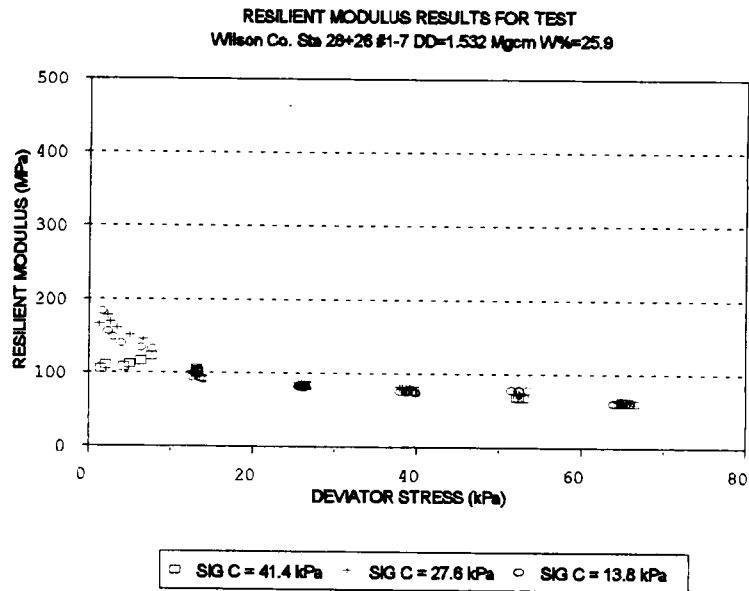
Resilient Modulus vs. Deviator Stress for Wilson County Station 28+26 Soil Compacted at Optimum Moisture Content and Maximum Dry Density (Standard Proctor), with a Degree of Saturation of 78.9%.



Resilient Modulus vs. Deviator Stress for Wilson County Station 28+26 Soil Compacted at Optimum Moisture Content and Maximum Dry Density (Standard Proctor), then Increasing the Degree of Saturation to 80.3%.



Resilient Modulus vs. Deviator Stress for Wilson County Sta. 28+26 Soil Compacted at Optimum Moisture Content and Maximum Dry Density (Standard Proctor), the Increasing the Degree of Saturation from 74.4% to 81.9%.



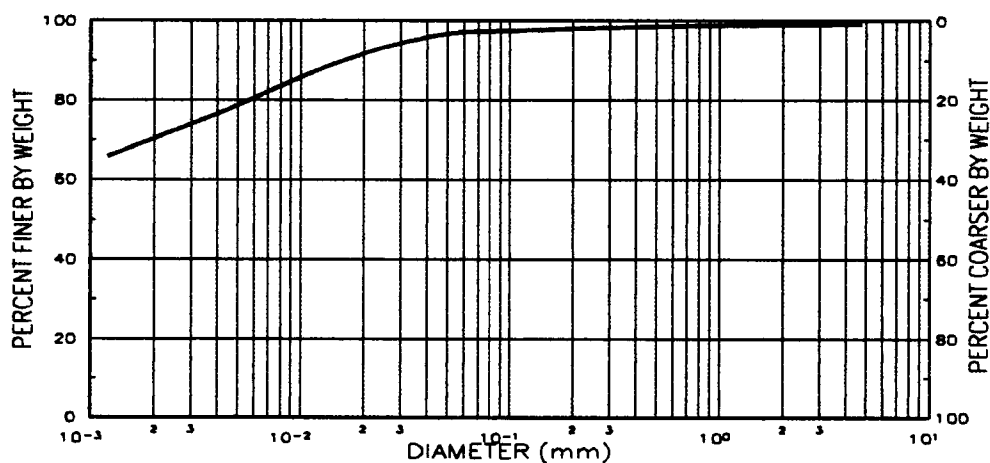
Resilient Modulus vs. Deviator Stress for Wilson County Sta. 28+26 Soil Compacted at Optimum Moisture Content and Maximum Dry Density (Standard Proctor), then Increasing the Degree of Saturation from 74.9% to 86.2%.

A-7-6 (50): Knox County Rutledge Pike

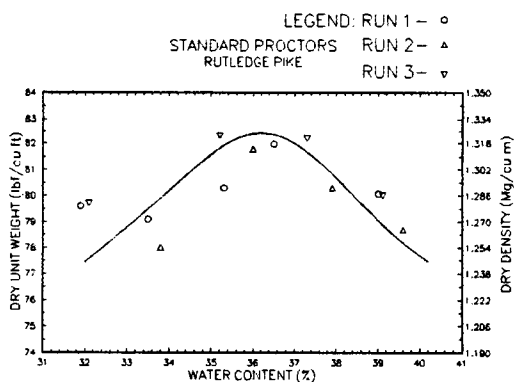
LOCATION: KNOX COUNTY RUTLEDGE PIKE	
SOIL CLASSIFICATION	
AASHTO	USCS
A-7-6 (50)	CH

CLAY CONTENT (%)	PASSING #200 SIEVE (%)	ATTERBERG LIMITS			SPECIFIC GRAVITY OF SOLIDS	STANDARD	
		LL	PL	PI		max. ρ_d (Mg/m ³)	opt. w (%)
70.3	97.8	71	28	43	2.60	1.322	36.2

GRAIN SIZE DISTRIBUTION
RUTLEDGE PIKE



Grain Size Distribution

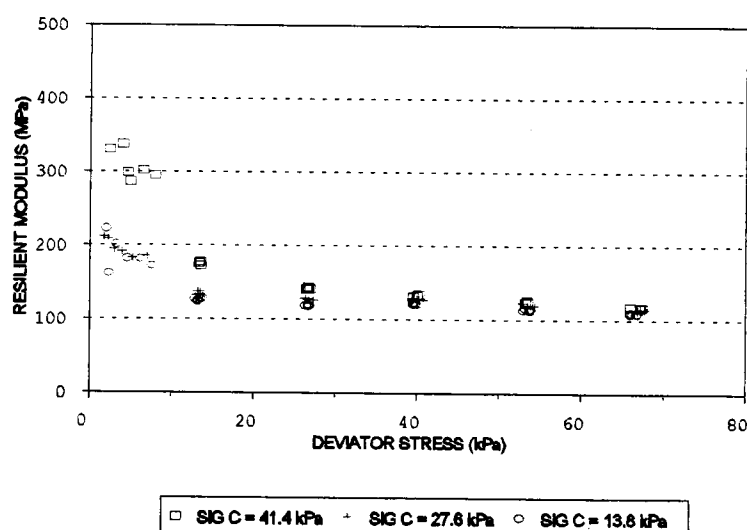


Standard Proctor Moisture-Density Relationship

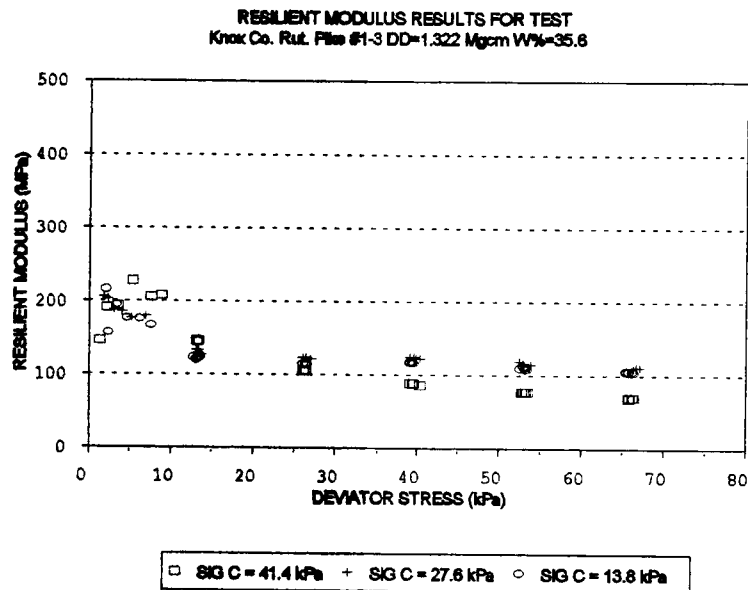
Hyperbolic and Power Model Parameters

TEST NO.	DRY DENSITY (Mg/m ³)	WATER CONTENT (%)	HYPERBOLIC MODEL			POWER MODEL		
			a (10 ³ Pa)	b (Mpa)	r ²	K ₁	K ₂	r ²
1-5	1.306	35.6	578.1	108.1	.98	281.3	-.2194	.70
1-3	1.322	34.6-35.6	555.7	90.72	.90	239.4	-.2180	.69
1-2	1.329	34.6-35.8	496.1	54.25	.97	191.3	-.2746	.95

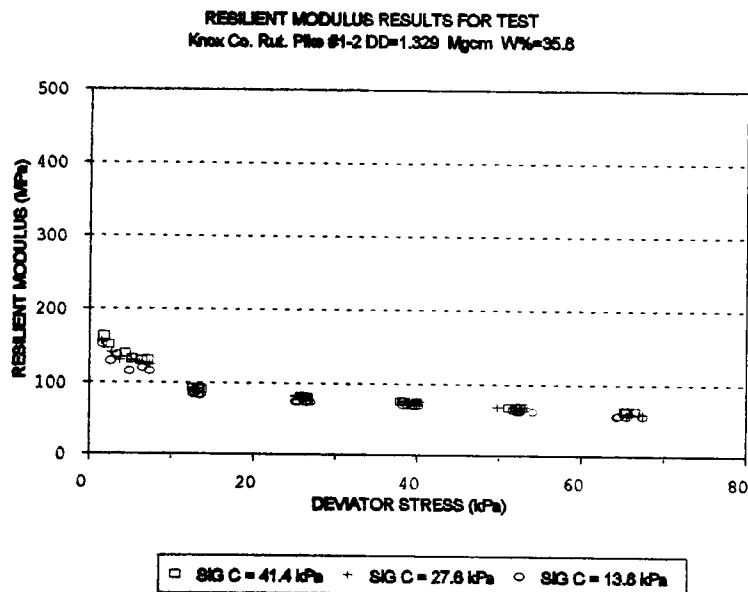
RESILIENT MODULUS RESULTS FOR TEST
Knox Co. Rut. Pike #1-5 DD=1.306 Mg/m³ W%=35.6



Resilient Modulus vs. Deviator Stress for Knox County Rutledge Pike Soil Compacted at Optimum Moisture Content and Maximum Dry Density (Standard Proctor), with a Degree of Saturation of 93.4%.



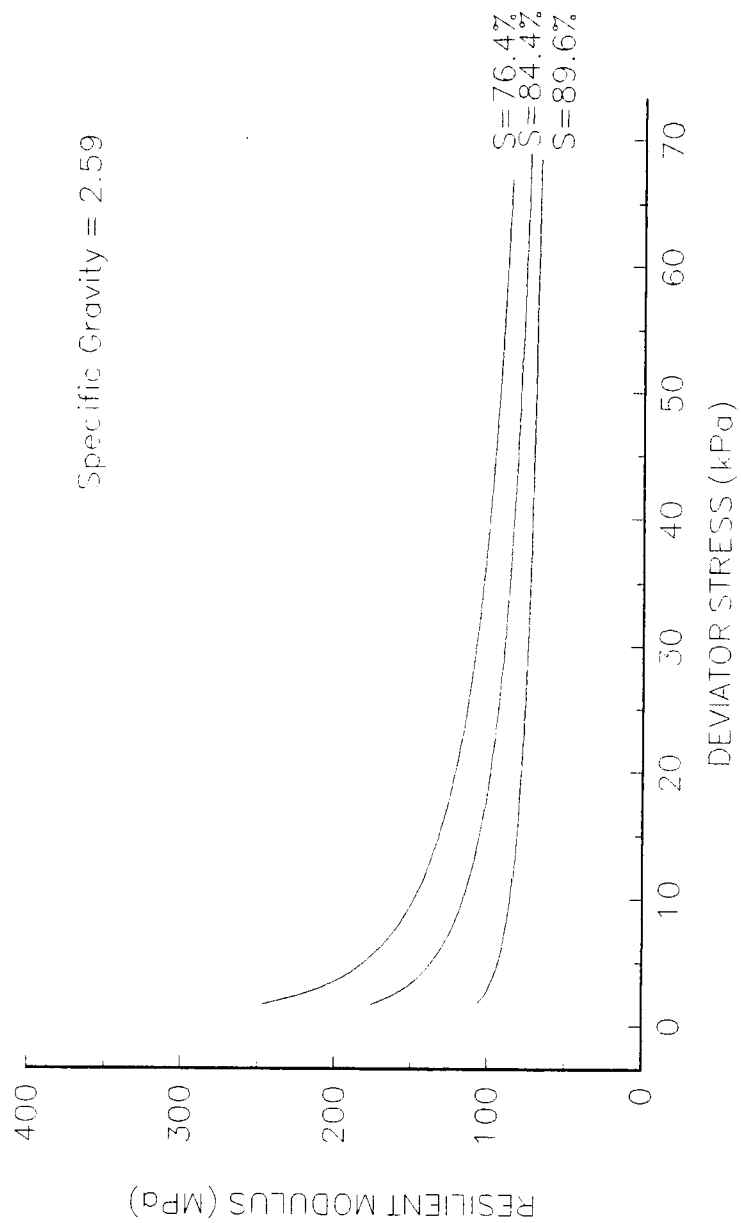
Resilient Modulus vs. Deviator Stress for Knox County Rutledge Pike Soil Compacted at Optimum Moisture Content and Maximum Dry Density (Standard Proctor), then Increasing the Degree of Saturation from 95.4% to 95.8%.



Resilient Modulus vs. Deviator Stress for Knox County Rutledge Pike Soil Compacted at Optimum Moisture Content and Maximum Dry Density (Standard Proctor), then Increasing the Degree of Saturation from 93.5% to 97.3%.

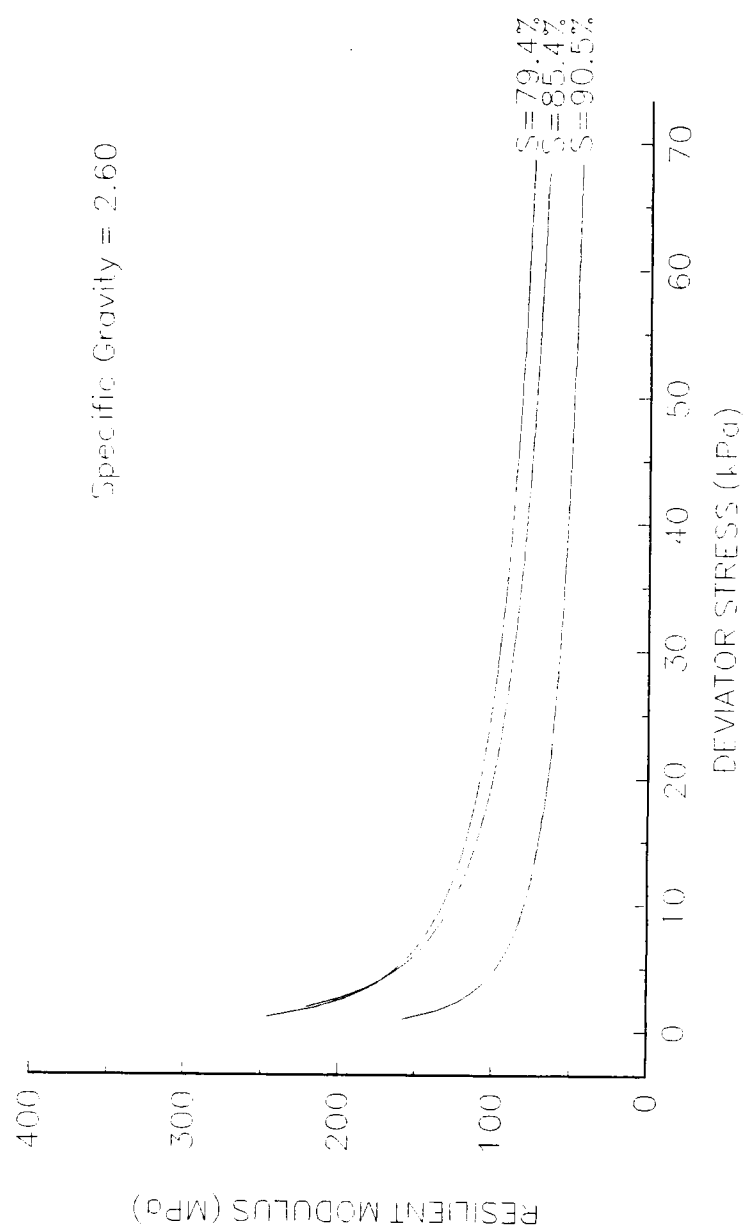
Post-Compaction Triaxial Results

EFFECTS OF POST-COMPACTION SATURATION ON RESILIENT MODULUS
CROCKETT CO. STA. 781



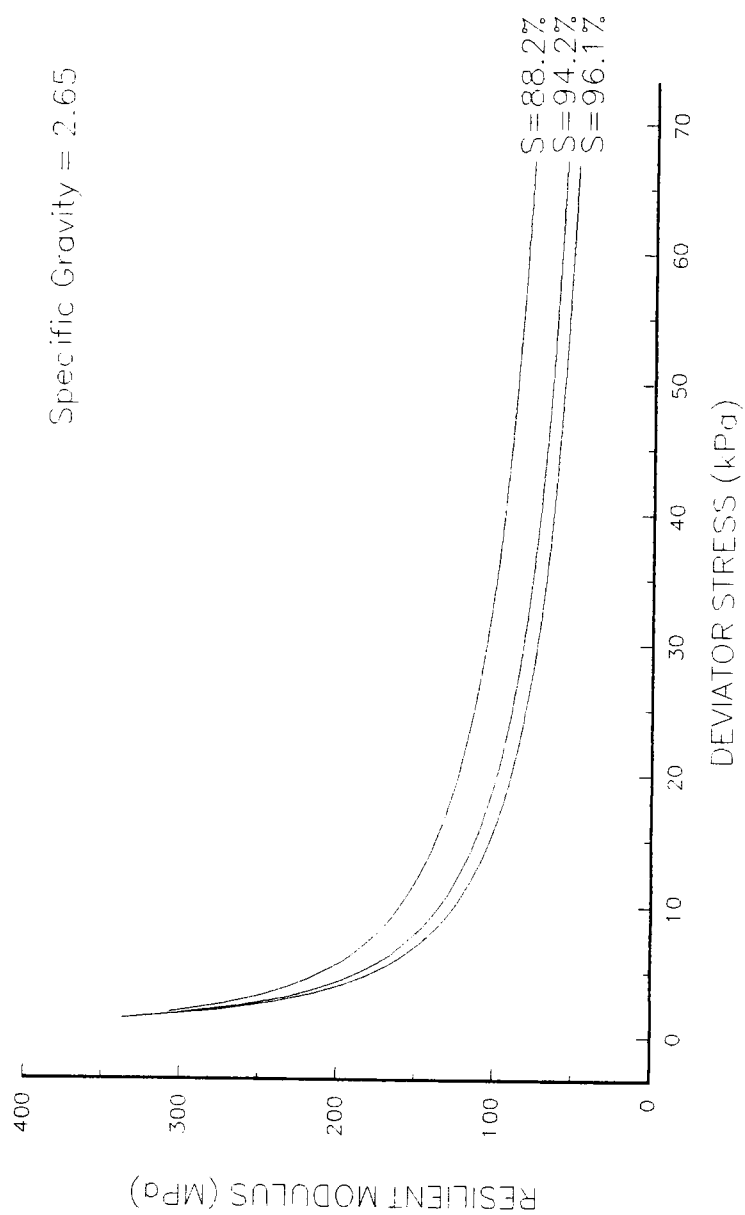
EFFECTS OF POST-COMPACTION SATURATION ON RESILIENT MODULUS
 ROANE CO. STA. 85

Specific Gravity = 2.60



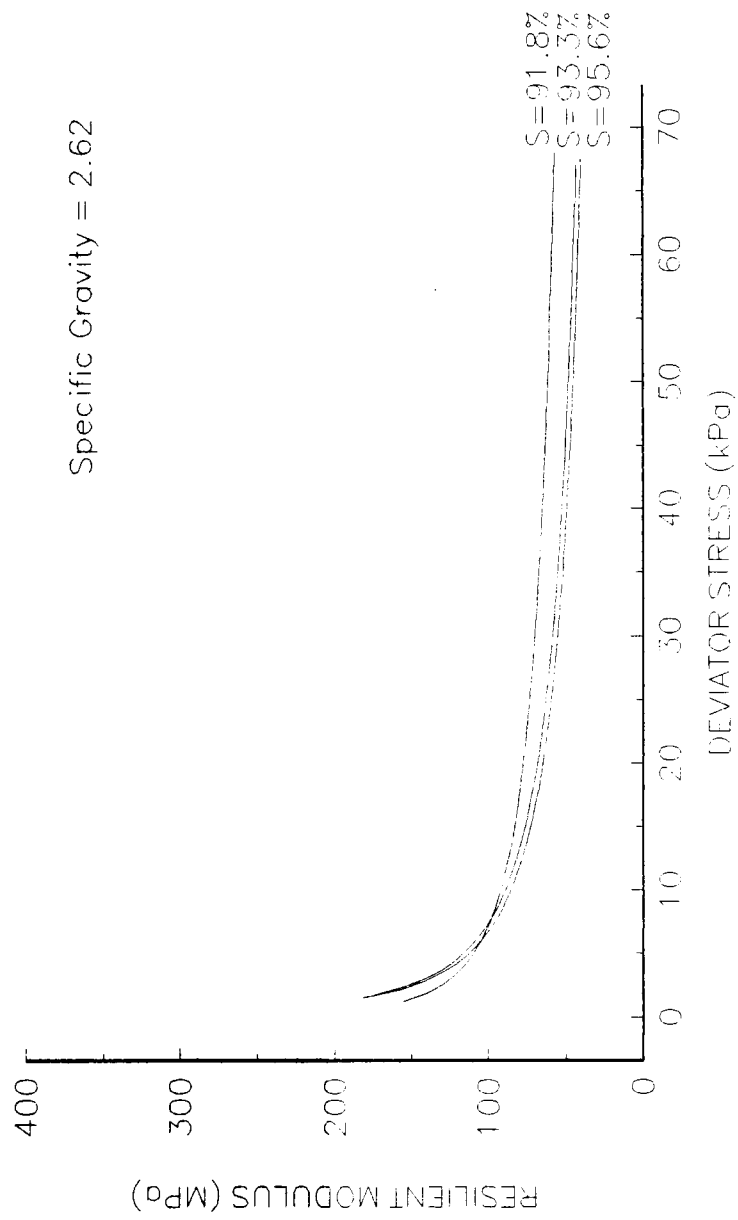
EFFECTS OF POST-COMPACTION SATURATION ON RESILIENT MODULUS
ROANE CO. STA. 47

Specific Gravity = 2.65

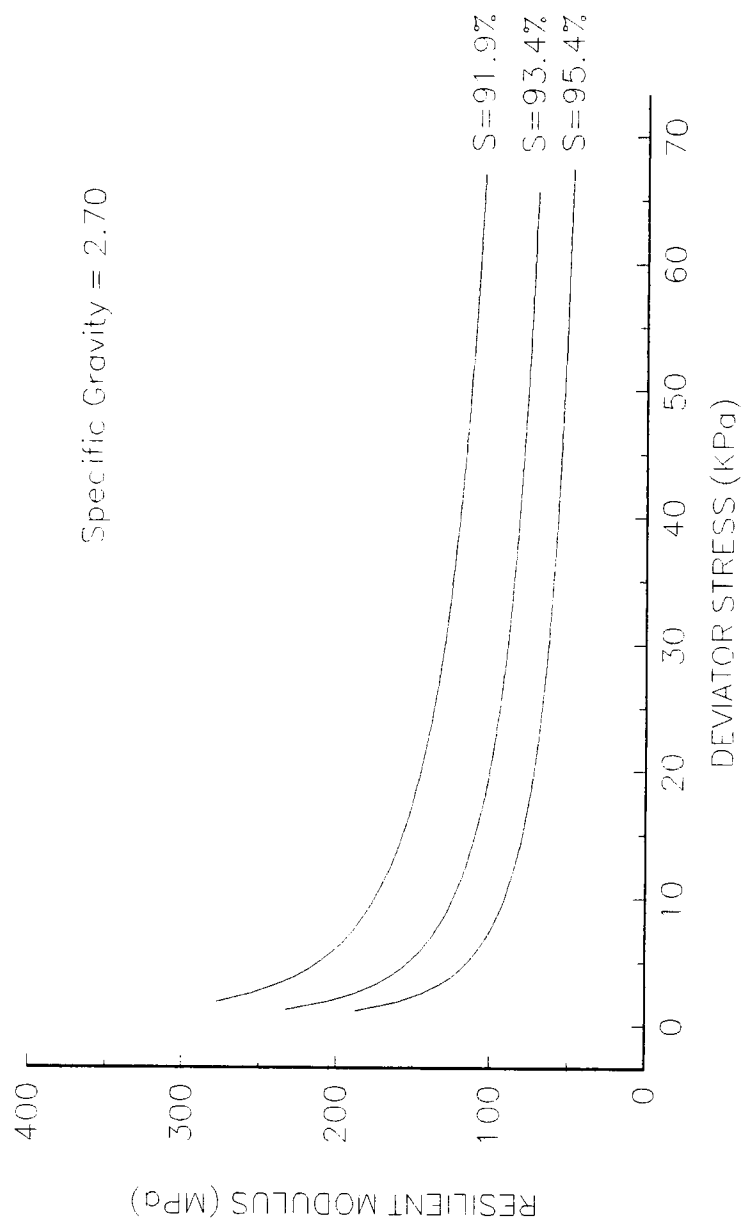


EFFECTS OF POST-COMPACTION SATURATION ON RESILIENT MODULUS
CROCKETT CO. STA. 1081

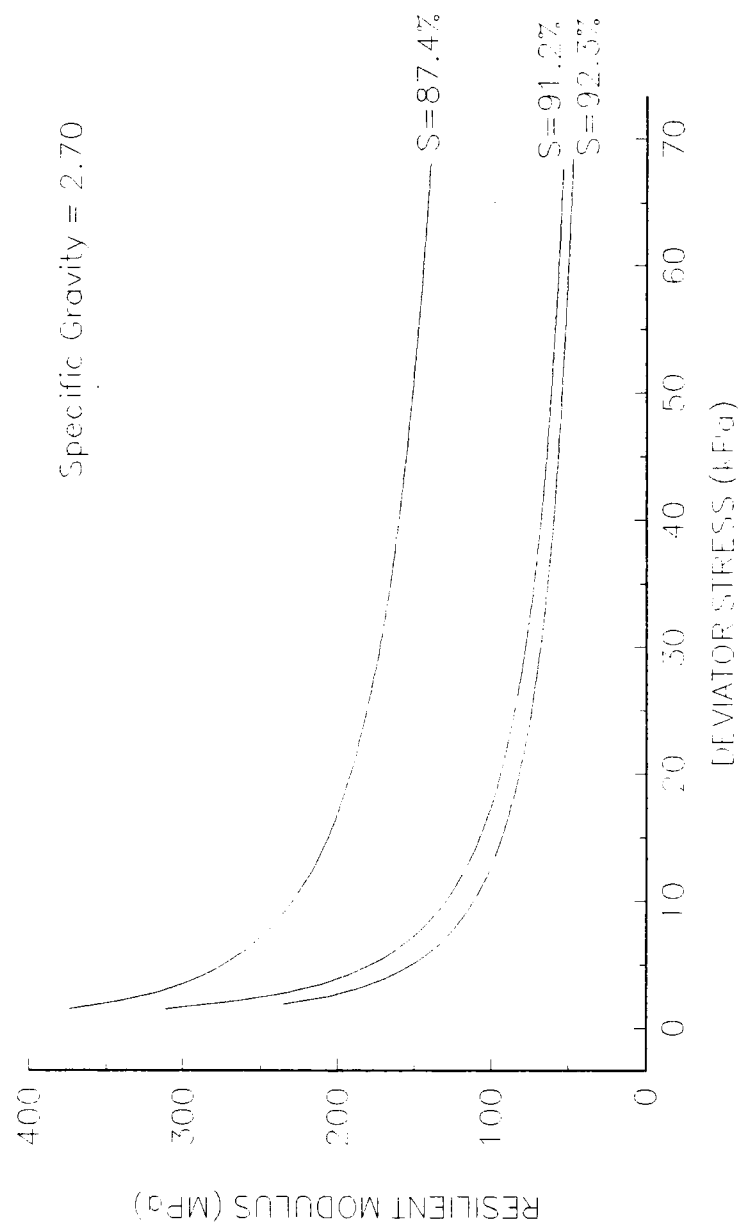
Specific Gravity = 2.62



EFFECTS OF POST-COMPACTION SATURATION ON RESILIENT MODULUS
KNOX CO. STA. 400

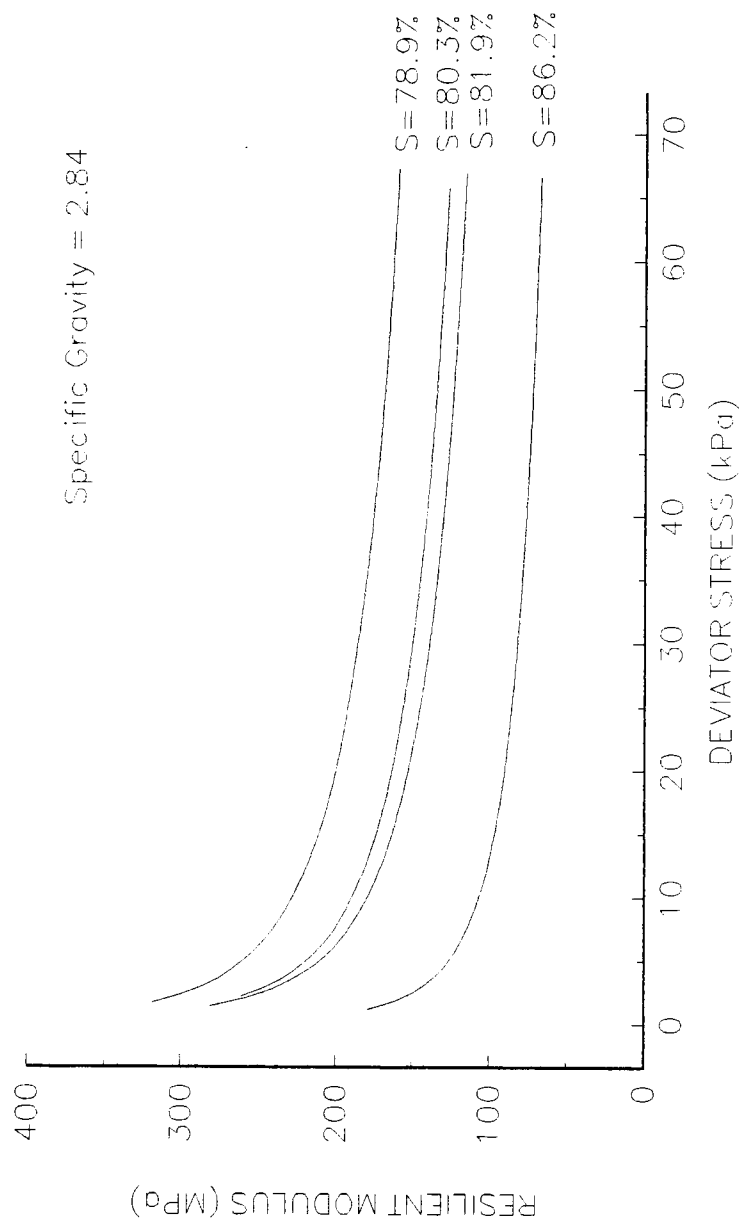


EFFECTS OF POST-COMPACTION SATURATION ON RESILIENT MODULUS
FNOX, CO., STA. 500



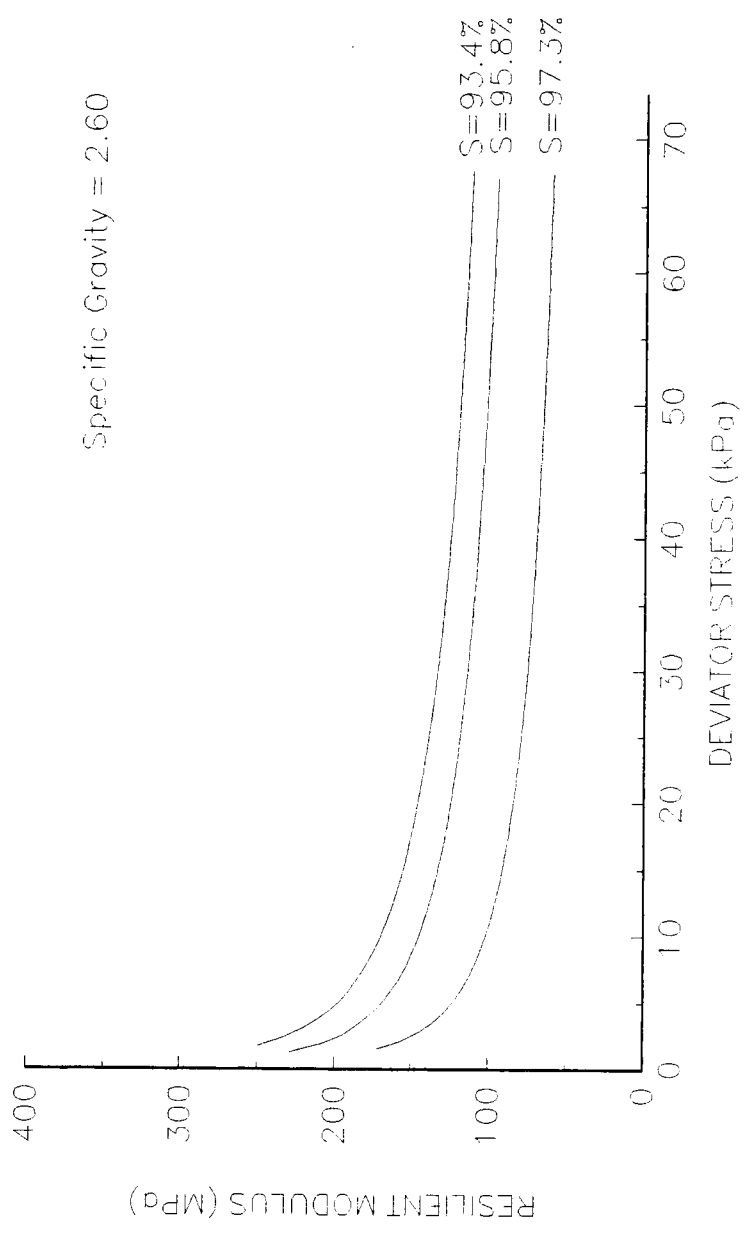
EFFECTS OF POST-COMPACTION SATURATION ON RESILIENT MODULUS
WILSON CO. STA. 28

Specific Gravity = 2.84



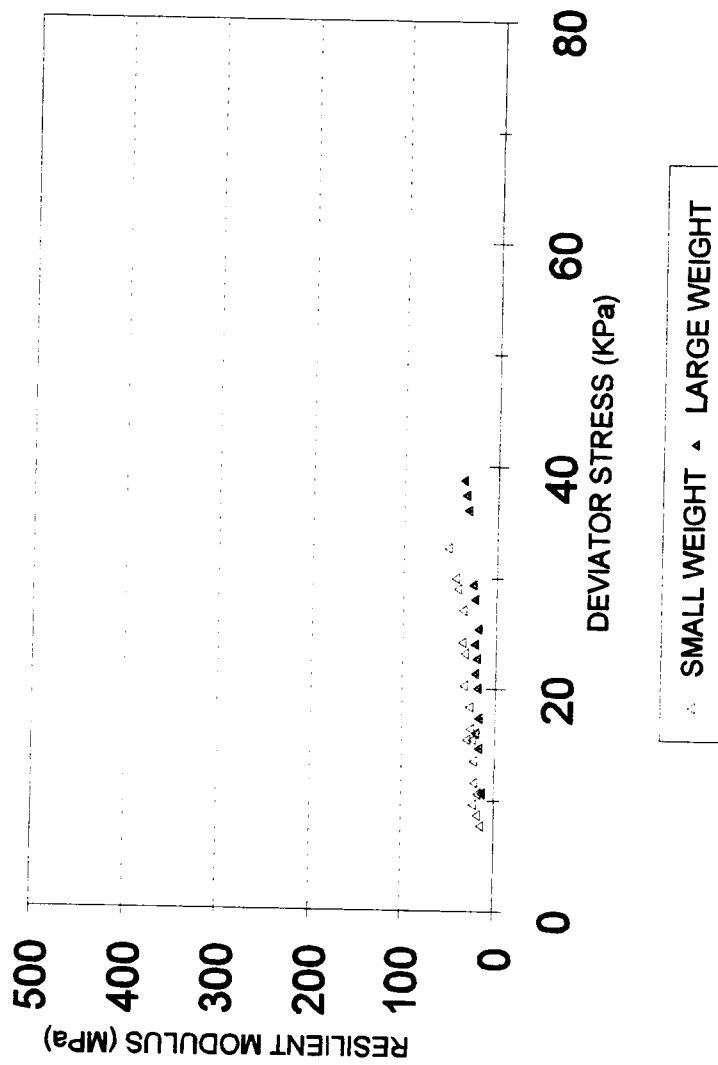
EFFECTS OF POST-COMPACTION SATURATION ON RESILIENT MODULUS
KNOX CO. RUTLEDGE PIKE

Specific Gravity = 2.60

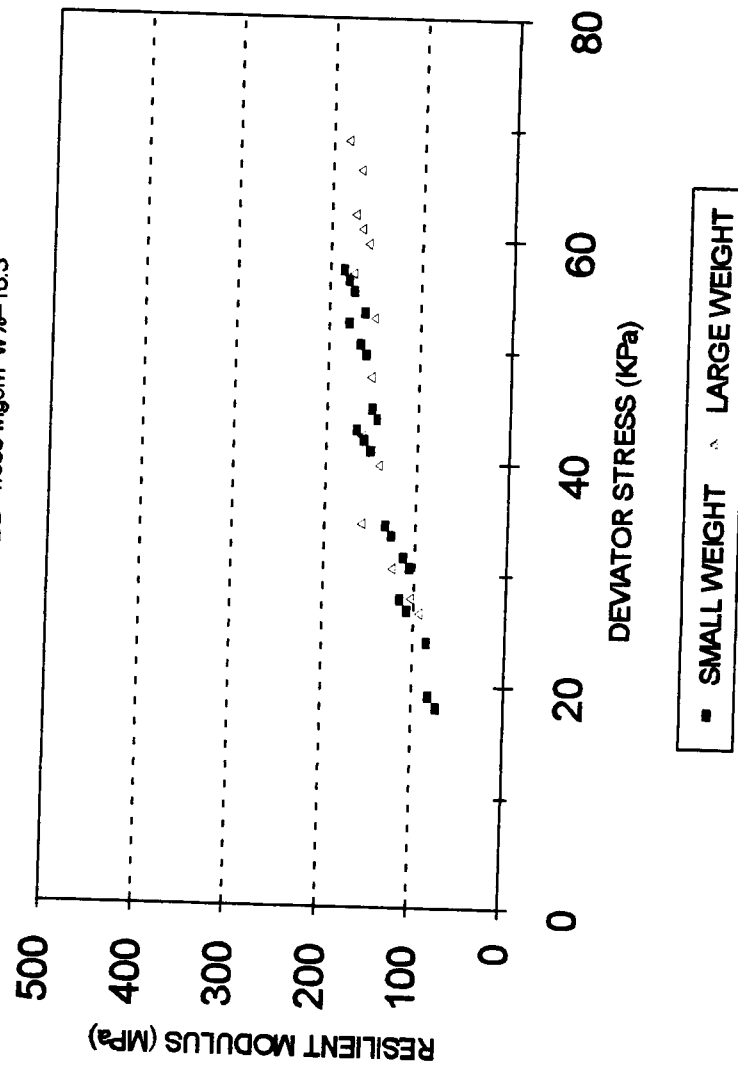


Appendix B

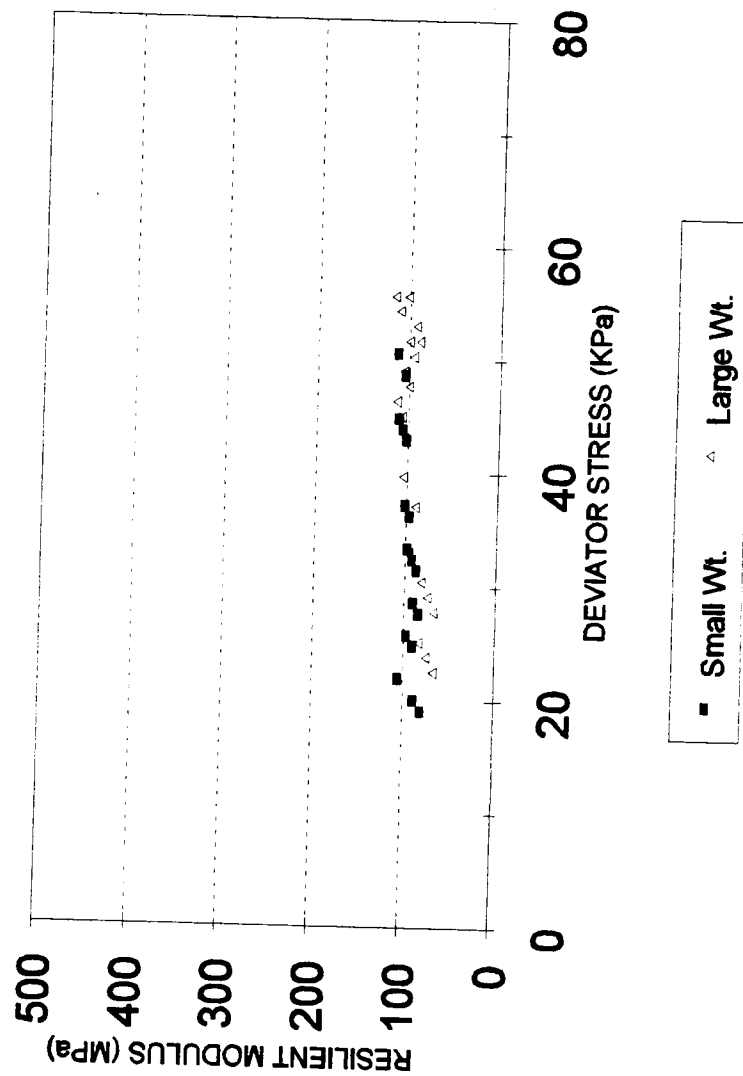
ATM Mr vs. DEVIATOR STRESS Roane Co. St a. 85 DD=1.889 Mpcm w%=13.0%



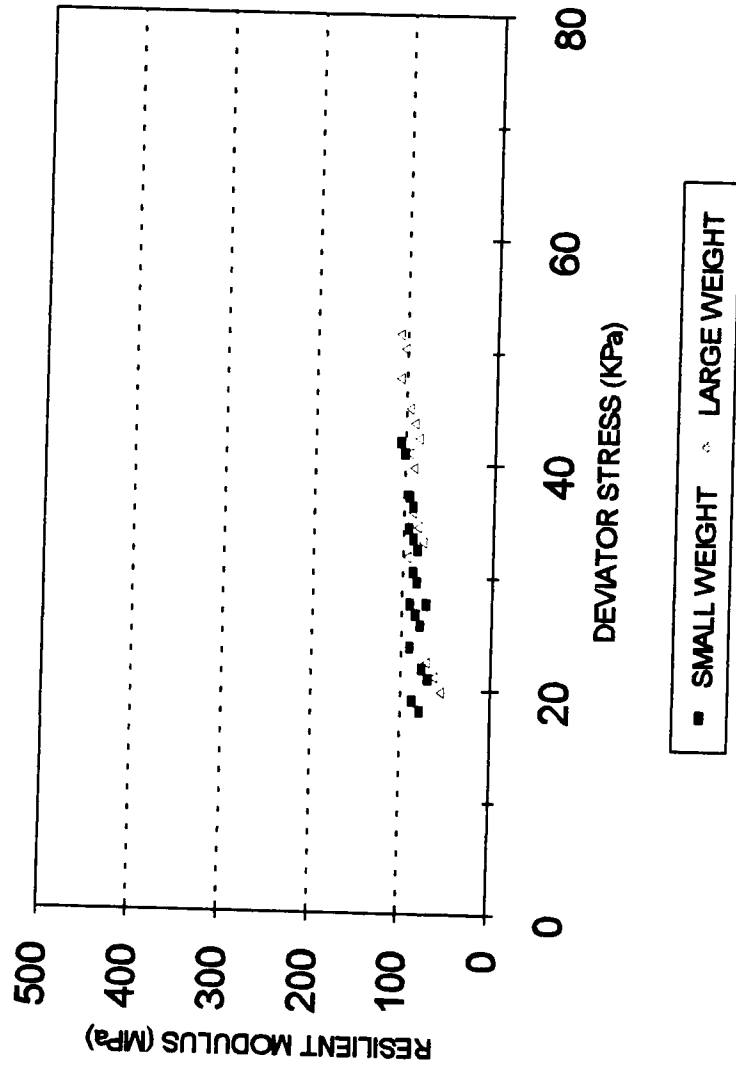
ATMM vs. DEVIATOR STRESS
 Crockett Co. Sta. 781 DD=1.686 Mgcm w%=16.3



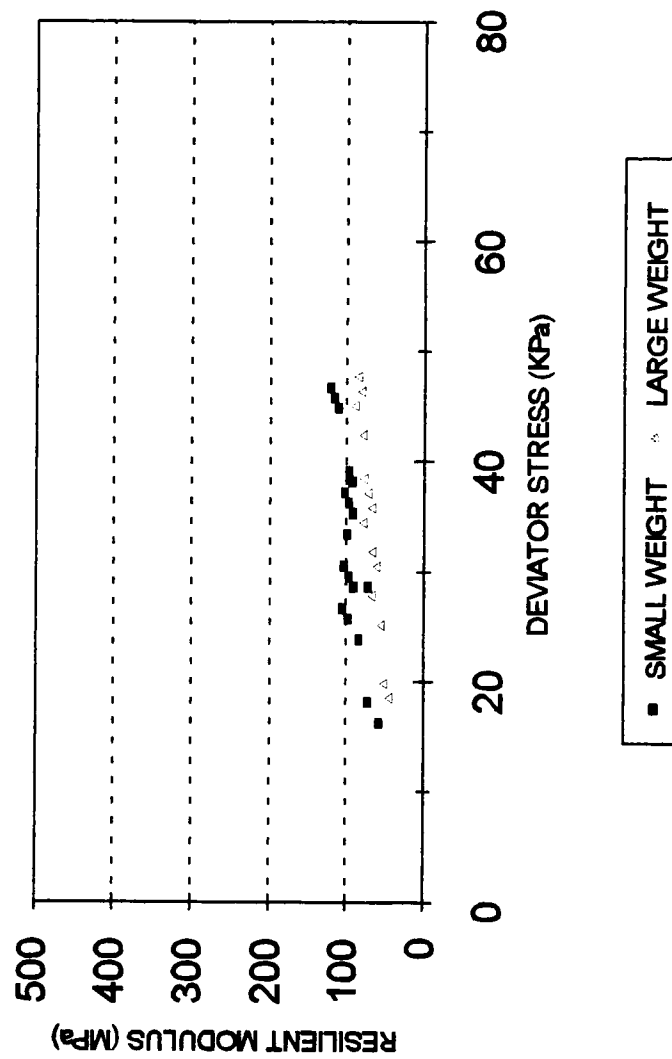
ATM Mr vs. DEVIATOR STRESS
 Roane Co. Sta . 47 DD=1.762 Mpcm w%=17.7



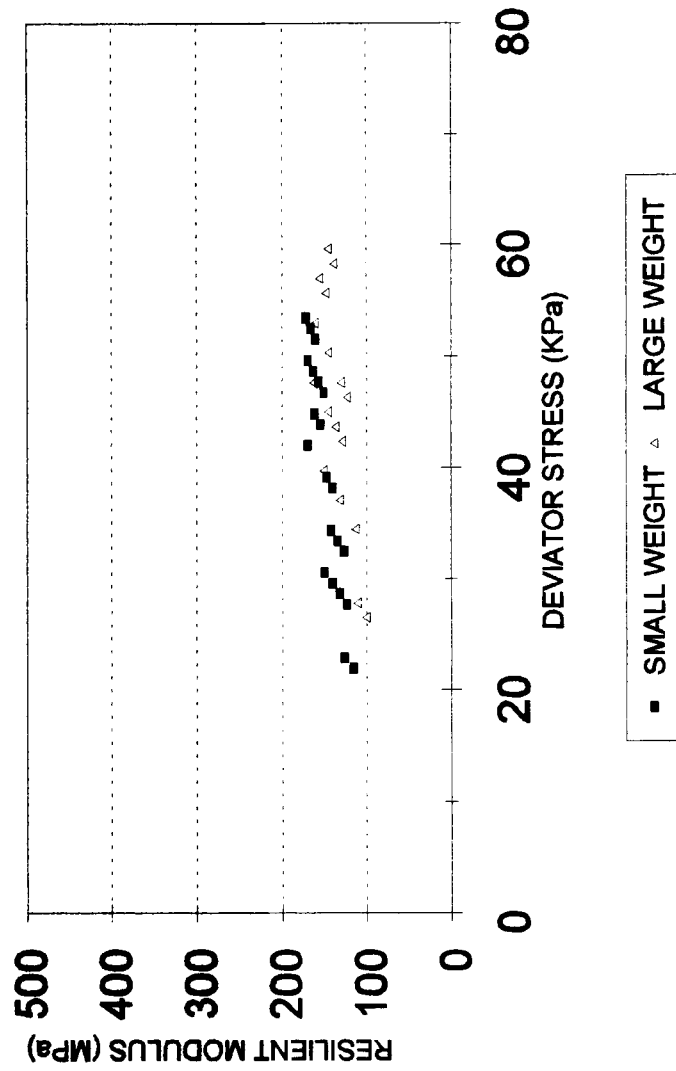
ATM Mr vs. DEVIATOR STRESS
Crockett Co. Sta. 1081 DD=1.724Mgcm w%=17.5



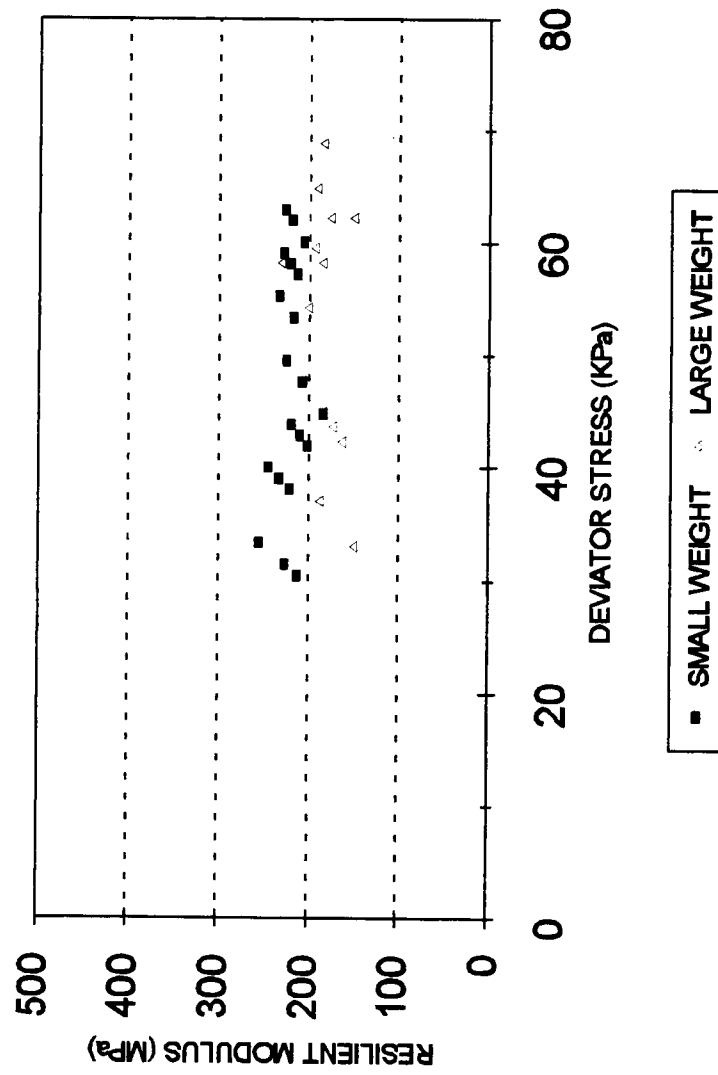
ATM Mr vs. DEVIATOR STRESS
 Knox Co. Sta 400 DD=1.463 Mgcm w%=30.0



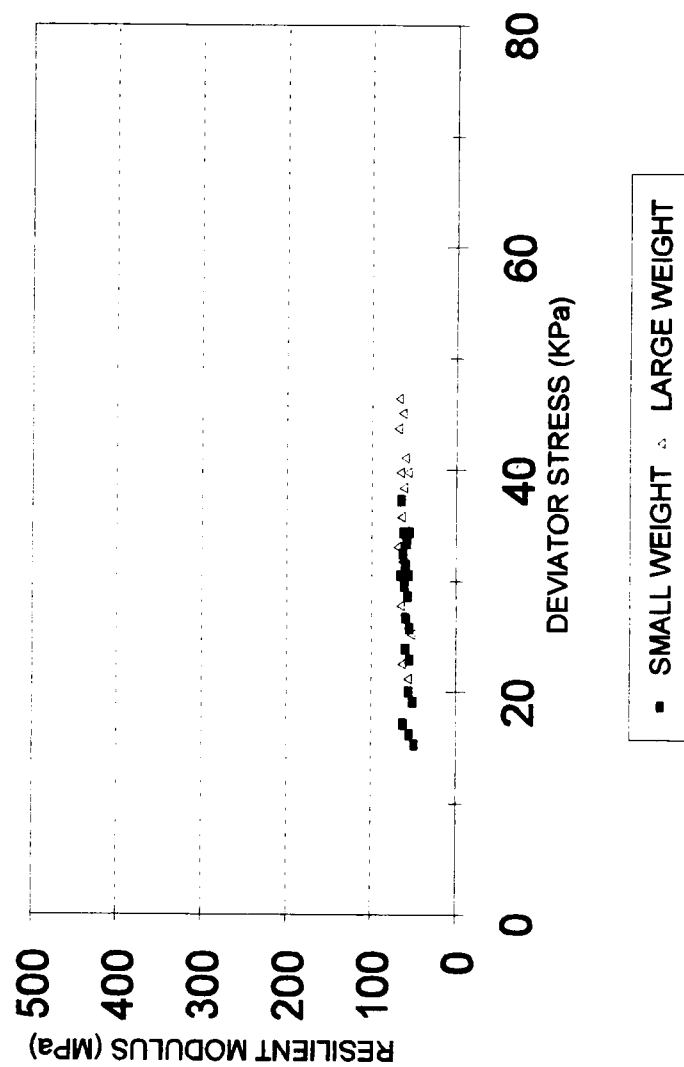
ATM Mr vs. DEVIATOR STRESS
 Knox Co Sta 500 DD=1.697 Mgem w%=19.7



ATM Mr vs. DEVIATOR STRESS
Wilson Co. Sta. 28 DD=1.587Mgcm w%=22.7



ATM Mr vs. DEVIATOR STRESS
 Knox Co. Rutledge Pike DD=1.330 Mgcw w%=36.7



ATM Data and Calculations

Roane Co. Station 85 Small Weight w%= 13.0% Dry Density = 1.889 Mg/m³

Mo	Ms	M	L	Vp	Ap	ho	h	Mr	Dev. Stress
(kg)	(kg)	(kg)	(m)	(V)	(m/s^2)	(m)	(m)	(MPa)	(kPa)
1.13	1.96	3.08	0.114	0.31	306.3	0.010	0.004	14.9	7.7
1.13	1.96	3.08	0.114	0.31	306.3	0.010	0.004	14.9	7.7
1.13	1.96	3.08	0.114	0.39	382.8	0.010	0.004	23.3	9.7
1.13	1.96	3.08	0.114	0.35	344.6	0.010	0.004	18.9	8.7
1.13	1.96	3.08	0.114	0.47	459.4	0.015	0.005	22.4	11.6
1.13	1.96	3.08	0.114	0.43	421.1	0.015	0.005	18.8	10.6
1.13	1.96	3.08	0.114	0.43	421.1	0.015	0.005	18.8	10.6
1.13	1.96	3.08	0.114	0.47	459.4	0.015	0.005	22.4	11.6
1.13	1.96	3.08	0.114	0.55	536.0	0.020	0.007	22.9	13.5
1.13	1.96	3.08	0.114	0.55	536.0	0.020	0.007	22.9	13.5
1.13	1.96	3.08	0.114	0.63	612.5	0.020	0.007	29.9	15.5
1.13	1.96	3.08	0.114	0.55	536.0	0.020	0.007	22.9	13.5
1.13	1.96	3.08	0.114	0.66	650.8	0.025	0.009	27.0	16.4
1.13	1.96	3.08	0.114	0.63	612.5	0.025	0.009	23.9	15.5
1.13	1.96	3.08	0.114	0.63	612.5	0.025	0.009	23.9	15.5
1.13	1.96	3.08	0.114	0.63	612.5	0.025	0.009	23.9	15.5
1.13	1.96	3.08	0.114	0.74	727.4	0.030	0.011	28.1	18.4
1.13	1.96	3.08	0.114	0.74	727.4	0.030	0.011	28.1	18.4
1.13	1.96	3.08	0.114	0.82	803.9	0.030	0.011	34.3	20.3
1.13	1.96	3.08	0.114	0.82	803.9	0.030	0.011	34.3	20.3
1.13	1.96	3.08	0.114	0.94	918.8	0.040	0.015	33.6	23.2
1.13	1.96	3.08	0.114	0.98	957.1	0.040	0.015	36.5	24.2
1.13	1.96	3.08	0.114	0.98	957.1	0.040	0.015	36.5	24.2
1.13	1.96	3.08	0.114	0.98	957.1	0.040	0.015	36.5	24.2
1.13	1.96	3.08	0.114	1.21	1186.7	0.050	0.018	44.8	30.0
1.13	1.96	3.08	0.114	1.32	1294.4	0.050	0.018	53.3	32.7
1.13	1.96	3.08	0.114	1.17	1148.5	0.050	0.018	42.0	29.0
1.13	1.96	3.08	0.114	1.09	1071.9	0.050	0.018	36.6	27.1

Roane Co. Station 85 Large Weight w%= 13.0% Dry Density = 1.889 Mg/m³

Mo	Ms	M	L	Vp	Ap	ho	h	Mr	Dev. Stress
(kg)	(kg)	(kg)	(m)	(V)	(m/s^2)	(m)	(m)	(MPa)	(kPa)
1.56	1.96	3.51	0.114	0.31	306.3	0.010	0.004	14.0	10.7
1.56	1.96	3.51	0.114	0.31	306.3	0.010	0.004	14.0	10.7
1.56	1.96	3.51	0.114	0.31	306.3	0.010	0.004	14.0	10.7
1.56	1.96	3.51	0.114	0.31	306.3	0.010	0.004	14.0	10.7
1.56	1.96	3.51	0.114	0.43	421.1	0.015	0.007	17.7	14.7
1.56	1.96	3.51	0.114	0.43	421.1	0.015	0.007	17.7	14.7
1.56	1.96	3.51	0.114	0.47	459.4	0.015	0.007	21.0	16.0
1.56	1.96	3.51	0.114	0.43	421.1	0.015	0.007	17.7	14.7
1.56	1.96	3.51	0.114	0.51	497.7	0.020	0.009	18.5	17.4
1.56	1.96	3.51	0.114	0.51	497.7	0.020	0.009	18.5	17.4
1.56	1.96	3.51	0.114	0.51	497.7	0.020	0.009	18.5	17.4
1.56	1.96	3.51	0.114	0.51	497.7	0.020	0.009	18.5	17.4
1.56	1.96	3.51	0.114	0.59	574.2	0.025	0.011	19.7	20.1
1.56	1.96	3.51	0.114	0.59	574.2	0.025	0.011	19.7	20.1
1.56	1.96	3.51	0.114	0.63	612.5	0.025	0.011	22.4	21.4
1.56	1.96	3.51	0.114	0.59	574.2	0.025	0.011	19.7	20.1
1.56	1.96	3.51	0.114	0.66	650.8	0.030	0.013	21.1	22.7
1.56	1.96	3.51	0.114	0.66	650.8	0.030	0.013	21.1	22.7
1.56	1.96	3.51	0.114	0.66	650.8	0.030	0.013	21.1	22.7
1.56	1.96	3.51	0.114	0.70	689.1	0.030	0.013	23.7	24.1
1.56	1.96	3.51	0.114	0.82	803.9	0.040	0.018	24.2	28.1
1.56	1.96	3.51	0.114	0.74	727.4	0.040	0.018	19.8	25.4
1.56	1.96	3.51	0.114	0.86	842.2	0.040	0.018	26.5	29.4
1.56	1.96	3.51	0.114	0.82	803.9	0.040	0.018	24.2	28.1
1.56	1.96	3.51	0.114	1.13	1110.2	0.050	0.022	36.9	38.8
1.56	1.96	3.51	0.114	1.05	1033.6	0.050	0.022	32.0	36.1
1.56	1.96	3.51	0.114	1.09	1071.9	0.050	0.022	34.4	37.4
1.56	1.96	3.51	0.114	1.13	1110.2	0.050	0.022	36.9	38.8

Crockett Co. Station 781 Small Weight w%=16.3 Dry Density = 1.666 Mg/m³

Mo	Ms	M	L	Vp	Ap	ho	h	Mr	Dev. Stress
(kg)	(kg)	(kg)	(m)	(V)	(m/s^2)	(m)	(m)	(MPa)	(kPa)
1.11	1.78	2.89	0.114	0.74	727.4	0.010	0.004	74.9	18.1
1.11	1.78	2.89	0.114	0.74	727.4	0.010	0.004	74.9	18.1
1.11	1.78	2.89	0.114	0.74	727.4	0.010	0.004	74.9	18.1
1.11	1.78	2.89	0.114	0.78	765.7	0.010	0.004	83.0	19.1
1.11	1.78	2.89	0.114	0.98	957.1	0.015	0.006	86.4	23.9
1.11	1.78	2.89	0.114	1.13	1110.2	0.015	0.006	116.3	27.7
1.11	1.78	2.89	0.114	1.13	1110.2	0.015	0.006	116.3	27.7
1.11	1.78	2.89	0.114	1.09	1071.9	0.015	0.006	108.4	26.7
1.11	1.78	2.89	0.114	1.25	1225.0	0.020	0.008	106.2	30.5
1.11	1.78	2.89	0.114	1.29	1263.3	0.020	0.008	113.0	31.5
1.11	1.78	2.89	0.114	1.37	1339.9	0.020	0.008	127.1	33.4
1.11	1.78	2.89	0.114	1.41	1378.2	0.020	0.008	134.4	34.4
1.11	1.78	2.89	0.114	1.68	1646.1	0.025	0.010	153.4	41.0
1.11	1.78	2.89	0.114	1.76	1722.7	0.025	0.010	168.0	43.0
1.11	1.78	2.89	0.114	1.72	1684.5	0.025	0.010	160.7	42.0
1.11	1.78	2.89	0.114	1.68	1646.1	0.025	0.010	153.4	41.0
1.11	1.78	2.89	0.114	1.84	1799.2	0.030	0.012	152.8	44.9
1.11	1.78	2.89	0.114	1.84	1799.2	0.030	0.012	152.8	44.9
1.11	1.78	2.89	0.114	1.80	1761.0	0.030	0.012	146.3	43.9
1.11	1.78	2.89	0.114	1.84	1799.2	0.030	0.012	152.8	44.9
1.11	1.78	2.89	0.114	2.15	2105.5	0.035	0.013	179.3	52.5
1.11	1.78	2.89	0.114	2.03	1990.6	0.035	0.013	160.3	49.6
1.11	1.78	2.89	0.114	2.07	2028.9	0.035	0.013	166.5	50.6
1.11	1.78	2.89	0.114	2.07	2028.9	0.035	0.013	166.5	50.6
1.11	1.78	2.89	0.114	2.34	2296.9	0.040	0.015	186.7	57.3
1.11	1.78	2.89	0.114	2.27	2220.3	0.040	0.015	174.5	55.4
1.11	1.78	2.89	0.114	2.19	2143.8	0.040	0.015	162.6	53.5
1.11	1.78	2.89	0.114	2.30	2258.7	0.040	0.015	180.5	56.3

Crockett Co. Station 781 Large Weight w%=16.3 Dry Density = 1.666 Mg/m³

Mo	Ms	M	L	Vp	Ap	ho	h	Mr	Dev. Stress
(kg)	(kg)	(kg)	(m)	(V)	(m/s^2)	(m)	(m)	(MPa)	(kPa)
1.54	1.78	3.32	0.114	1.02	995.3	0.010	0.004	161.2	34.4
1.54	1.78	3.32	0.114	0.90	880.4	0.010	0.004	126.1	30.5
1.54	1.78	3.32	0.114	0.82	803.9	0.010	0.004	105.1	27.8
1.54	1.78	3.32	0.114	0.78	765.7	0.010	0.004	95.4	26.5
1.54	1.78	3.32	0.114	1.25	1225.0	0.015	0.006	162.8	42.4
1.54	1.78	3.32	0.114	1.17	1148.5	0.015	0.006	143.1	39.7
1.54	1.78	3.32	0.114	1.17	1148.5	0.015	0.006	143.1	39.7
1.54	1.78	3.32	0.114	1.21	1186.7	0.015	0.006	152.7	41.1
1.54	1.78	3.32	0.114	1.41	1378.2	0.020	0.008	154.5	47.7
1.54	1.78	3.32	0.114	1.41	1378.2	0.020	0.008	154.5	47.7
1.54	1.78	3.32	0.114	1.41	1378.2	0.020	0.008	154.5	47.7
1.54	1.78	3.32	0.114	1.41	1378.2	0.020	0.008	154.5	47.7
1.54	1.78	3.32	0.114	1.68	1646.1	0.025	0.010	176.3	57.0
1.54	1.78	3.32	0.114	1.56	1531.3	0.025	0.010	152.6	53.0
1.54	1.78	3.32	0.114	1.56	1531.3	0.025	0.010	152.6	53.0
1.54	1.78	3.32	0.114	1.56	1531.3	0.025	0.010	152.6	53.0
1.54	1.78	3.32	0.114	1.76	1722.7	0.030	0.012	160.9	59.6
1.54	1.78	3.32	0.114	1.76	1722.7	0.030	0.012	160.9	59.6
1.54	1.78	3.32	0.114	1.80	1761.0	0.030	0.012	168.2	60.9
1.54	1.78	3.32	0.114	1.84	1799.2	0.030	0.012	175.6	62.3
1.54	1.78	3.32	0.114	1.95	1914.1	0.035	0.013	170.3	66.2
1.54	1.78	3.32	0.114	1.95	1914.1	0.035	0.013	170.3	66.2
1.54	1.78	3.32	0.114	2.03	1990.6	0.035	0.013	184.2	68.9
1.54	1.78	3.32	0.114	2.03	1990.6	0.035	0.013	184.2	68.9

Roane Co. Station 47 Small Weight w%=17.7 Dry Density = 1.762 Mg/m³

Mo	Ms	M	L	Vp	Ap	ho	h	Mr	Dev. Stress
(kg)	(kg)	(kg)	(m)	(V)	(m/s^2)	(m)	(m)	(MPa)	(kPa)
1.11	1.73	2.84	0.114	0.82	803.9	0.010	0.004	88.6	20.0
1.11	1.73	2.84	0.114	0.78	765.7	0.010	0.004	80.4	19.1
1.11	1.73	2.84	0.114	0.78	765.7	0.010	0.004	80.4	19.1
1.11	1.73	2.84	0.114	0.90	880.4	0.010	0.004	106.3	22.0
1.11	1.73	2.84	0.114	1.02	995.3	0.015	0.006	90.6	24.8
1.11	1.73	2.84	0.114	1.02	995.3	0.015	0.006	90.6	24.8
1.11	1.73	2.84	0.114	1.05	1033.6	0.015	0.006	97.7	25.8
1.11	1.73	2.84	0.114	1.02	995.3	0.015	0.006	90.6	24.8
1.11	1.73	2.84	0.114	1.17	1148.5	0.020	0.008	90.4	28.6
1.11	1.73	2.84	0.114	1.13	1110.2	0.020	0.008	84.5	27.7
1.11	1.73	2.84	0.114	1.17	1148.5	0.020	0.008	90.4	28.6
1.11	1.73	2.84	0.114	1.17	1148.5	0.020	0.008	90.4	28.6
1.11	1.73	2.84	0.114	1.29	1263.3	0.025	0.010	87.5	31.5
1.11	1.73	2.84	0.114	1.33	1301.6	0.025	0.010	92.9	32.5
1.11	1.73	2.84	0.114	1.37	1339.9	0.025	0.010	98.5	33.4
1.11	1.73	2.84	0.114	1.37	1339.9	0.025	0.010	98.5	33.4
1.11	1.73	2.84	0.114	1.52	1493.0	0.030	0.012	101.9	37.2
1.11	1.73	2.84	0.114	1.52	1493.0	0.030	0.012	101.9	37.2
1.11	1.73	2.84	0.114	1.48	1454.7	0.030	0.012	96.7	36.3
1.11	1.73	2.84	0.114	1.48	1454.7	0.030	0.012	96.7	36.3
1.11	1.73	2.84	0.114	1.76	1722.7	0.040	0.016	101.7	43.0
1.11	1.73	2.84	0.114	1.84	1799.2	0.040	0.016	111.0	44.9
1.11	1.73	2.84	0.114	1.80	1761.0	0.040	0.016	106.3	43.9
1.11	1.73	2.84	0.114	1.80	1761.0	0.040	0.016	106.3	43.9
1.11	1.73	2.84	0.114	2.07	2028.9	0.050	0.020	112.9	50.6
1.11	1.73	2.84	0.114	2.07	2028.9	0.050	0.020	112.9	50.6
1.11	1.73	2.84	0.114	2.07	2028.9	0.050	0.020	112.9	50.6
1.11	1.73	2.84	0.114	1.99	1952.4	0.050	0.020	104.5	48.7

Roane Co. Station 47 Large Weight w%=17.7 Dry Density = 1.762 Mg/m³

Mo	Ms	M	L	Vp	Ap	ho	h	Mr	Dev. Stress
(kg)	(kg)	(kg)	(m)	(V)	(m/s^2)	(m)	(m)	(MPa)	(kPa)
1.54	1.73	3.28	0.114	0.74	727.4	0.010	0.004	83.5	25.2
1.54	1.73	3.28	0.114	0.66	650.8	0.010	0.004	66.9	22.5
1.54	1.73	3.28	0.114	0.70	689.1	0.010	0.004	75.0	23.8
1.54	1.73	3.28	0.114	0.70	689.1	0.010	0.004	75.0	23.8
1.54	1.73	3.28	0.114	0.90	880.4	0.015	0.006	81.6	30.5
1.54	1.73	3.28	0.114	0.86	842.2	0.015	0.006	74.7	29.1
1.54	1.73	3.28	0.114	0.98	957.1	0.015	0.006	96.4	33.1
1.54	1.73	3.28	0.114	0.82	803.9	0.015	0.006	68.0	27.8
1.54	1.73	3.28	0.114	1.09	1071.9	0.020	0.008	90.7	37.1
1.54	1.73	3.28	0.114	1.17	1148.5	0.020	0.008	104.1	39.7
1.54	1.73	3.28	0.114	1.17	1148.5	0.020	0.008	104.1	39.7
1.54	1.73	3.28	0.114	1.17	1148.5	0.020	0.008	104.1	39.7
1.54	1.73	3.28	0.114	1.37	1339.9	0.025	0.010	113.4	46.4
1.54	1.73	3.28	0.114	1.33	1301.6	0.025	0.010	107.0	45.0
1.54	1.73	3.28	0.114	1.37	1339.9	0.025	0.010	113.4	46.4
1.54	1.73	3.28	0.114	1.37	1339.9	0.025	0.010	113.4	46.4
1.54	1.73	3.28	0.114	1.41	1377.9	0.030	0.012	99.9	47.7
1.54	1.73	3.28	0.114	1.41	1377.9	0.030	0.012	99.9	47.7
1.54	1.73	3.28	0.114	1.41	1377.9	0.030	0.012	99.9	47.7
1.54	1.73	3.28	0.114	1.45	1416.4	0.030	0.012	105.6	49.0
1.54	1.73	3.28	0.114	1.48	1454.7	0.035	0.014	95.5	50.3
1.54	1.73	3.28	0.114	1.52	1493.0	0.035	0.014	100.6	51.7
1.54	1.73	3.28	0.114	1.64	1607.8	0.035	0.014	116.6	55.6
1.54	1.73	3.28	0.114	1.60	1569.6	0.035	0.014	111.1	54.3
1.54	1.73	3.28	0.114	1.64	1607.8	0.040	0.016	102.0	55.6
1.54	1.73	3.28	0.114	1.64	1607.8	0.040	0.016	102.0	55.6
1.54	1.73	3.28	0.114	1.52	1493.0	0.040	0.016	88.0	51.7
1.54	1.73	3.28	0.114	1.56	1531.3	0.040	0.016	92.6	53.0

Crockett Co. Station 1081 Small Weight w%=17.5 Dry Density = 1.724 Mg/m³

Mo	Ms	M	L	Vp	Ap	ho	h	Mr	Dev. Stress
(kg)	(kg)	(kg)	(m)	(V)	(m/s^2)	(m)	(m)	(MPa)	(kPa)
1.11	1.86	2.97	0.114	0.781	765.7	0.01	0.004	87.8	19.1
1.11	1.86	2.97	0.114	0.781	765.7	0.01	0.004	87.8	19.1
1.11	1.86	2.97	0.114	0.742	727.4	0.01	0.004	79.2	18.1
1.11	1.86	2.97	0.114	0.742	727.4	0.01	0.004	79.2	18.1
1.11	1.86	2.97	0.114	0.898	880.4	0.015	0.006	77.4	22.0
1.11	1.86	2.97	0.114	0.898	880.4	0.015	0.006	77.4	22.0
1.11	1.86	2.97	0.114	0.977	957.1	0.015	0.006	91.4	23.9
1.11	1.86	2.97	0.114	0.859	842.2	0.015	0.006	70.8	21.0
1.11	1.86	2.97	0.114	1.055	1033.6	0.02	0.007	80.0	25.8
1.11	1.86	2.97	0.114	1.133	1110.2	0.02	0.007	92.2	27.7
1.11	1.86	2.97	0.114	1.133	1110.2	0.02	0.007	92.2	27.7
1.11	1.86	2.97	0.114	1.094	1071.9	0.02	0.007	86.0	26.7
1.11	1.86	2.97	0.114	1.250	1225.0	0.025	0.009	89.9	30.5
1.11	1.86	2.97	0.114	1.211	1186.7	0.025	0.009	84.3	29.6
1.11	1.86	2.97	0.114	1.133	1110.2	0.025	0.009	73.8	27.7
1.11	1.86	2.97	0.114	1.250	1225.0	0.025	0.009	89.9	30.5
1.11	1.86	2.97	0.114	1.367	1339.9	0.03	0.011	89.6	33.4
1.11	1.86	2.97	0.114	1.328	1301.6	0.03	0.011	84.5	32.5
1.11	1.86	2.97	0.114	1.406	1378.2	0.03	0.011	94.8	34.4
1.11	1.86	2.97	0.114	1.367	1339.9	0.03	0.011	89.6	33.4
1.11	1.86	2.97	0.114	1.484	1454.7	0.035	0.013	90.5	36.3
1.11	1.86	2.97	0.114	1.523	1493.0	0.035	0.013	95.3	37.2
1.11	1.86	2.97	0.114	1.523	1493.0	0.035	0.013	95.3	37.2
1.11	1.86	2.97	0.114	1.523	1493.0	0.035	0.013	95.3	37.2
1.11	1.86	2.97	0.114	1.719	1684.5	0.04	0.015	106.2	42.0
1.11	1.86	2.97	0.114	1.680	1646.1	0.04	0.015	101.4	41.0
1.11	1.86	2.97	0.114	1.719	1684.5	0.04	0.015	106.2	42.0
1.11	1.86	2.97	0.114	1.719	1684.5	0.04	0.015	106.2	42.0

Crockett Co. Station 1081 Large Weight w%=17.5 Dry Density = 1.724 Mg/m³

Mo	Ms	M	L	Vp	Ap	ho	h	Mr	Dev. Stress
(kg)	(kg)	(kg)	(m)	(V)	(m/s^2)	(m)	(m)	(MPa)	(kPa)
1.54	1.86	3.40	0.114	0.625	612.5	0.01	0.004	64.3	21.2
1.54	1.86	3.40	0.114	0.664	650.8	0.01	0.004	72.6	22.5
1.54	1.86	3.40	0.114	0.625	612.5	0.01	0.004	64.3	21.2
1.54	1.86	3.40	0.114	0.586	574.2	0.01	0.004	56.5	19.9
1.54	1.86	3.40	0.114	0.898	880.4	0.015	0.006	88.6	30.5
1.54	1.86	3.40	0.114	0.938	918.8	0.015	0.006	96.5	31.8
1.54	1.86	3.40	0.114	0.898	880.4	0.015	0.006	88.6	30.5
1.54	1.86	3.40	0.114	0.898	880.4	0.015	0.006	88.6	30.5
1.54	1.86	3.40	0.114	1.016	995.3	0.02	0.007	84.9	34.4
1.54	1.86	3.40	0.114	1.094	1071.9	0.02	0.007	98.5	37.1
1.54	1.86	3.40	0.114	1.055	1033.6	0.02	0.007	91.6	35.8
1.54	1.86	3.40	0.114	0.977	957.1	0.02	0.007	78.5	33.1
1.54	1.86	3.40	0.114	1.211	1186.7	0.025	0.009	96.6	41.1
1.54	1.86	3.40	0.114	1.172	1148.5	0.025	0.009	90.4	39.7
1.54	1.86	3.40	0.114	1.172	1148.5	0.025	0.009	90.4	39.7
1.54	1.86	3.40	0.114	1.172	1148.5	0.025	0.009	90.4	39.7
1.54	1.86	3.40	0.114	1.289	1263.3	0.03	0.011	91.2	43.7
1.54	1.86	3.40	0.114	1.250	1225.0	0.03	0.011	85.8	42.4
1.54	1.86	3.40	0.114	1.406	1378.2	0.03	0.011	108.5	47.7
1.54	1.86	3.40	0.114	1.328	1301.6	0.03	0.011	96.8	45.0
1.54	1.86	3.40	0.114	1.484	1454.7	0.035	0.013	103.7	50.3
1.54	1.86	3.40	0.114	1.523	1493.0	0.035	0.013	109.2	51.7
1.54	1.86	3.40	0.114	1.523	1493.0	0.035	0.013	109.2	51.7
1.54	1.86	3.40	0.114	1.523	1493.0	0.035	0.013	109.2	51.7

Knox Co. Station 400 Small Weight w%=30.0 Dry Density = 1.463 Mg/m³

Mo	Ms	M	L	Vp	Ap	ho	h	Mr	Dev. Stress
(kg)	(kg)	(kg)	(m)	(V)	(m/s ²)	(m)	(m)	(MPa)	(kPa)
1.11	1.75	2.86	0.114	0.74	727.4	0.010	0.004	73.2	18.1
1.11	1.75	2.86	0.114	0.66	650.8	0.010	0.004	58.6	16.2
1.11	1.75	2.86	0.114	0.74	727.4	0.010	0.004	73.2	18.1
1.11	1.75	2.86	0.114	0.66	650.8	0.010	0.004	58.6	16.2
1.11	1.75	2.86	0.114	1.05	1033.6	0.015	0.006	98.5	25.8
1.11	1.75	2.86	0.114	0.98	957.1	0.015	0.006	84.5	23.9
1.11	1.75	2.86	0.114	1.05	1033.6	0.015	0.006	98.5	25.8
1.11	1.75	2.86	0.114	1.09	1071.9	0.015	0.006	106.0	26.7
1.11	1.75	2.86	0.114	1.21	1186.7	0.020	0.008	97.4	29.6
1.11	1.75	2.86	0.114	1.25	1225.0	0.020	0.008	103.8	30.5
1.11	1.75	2.86	0.114	1.25	1225.0	0.020	0.008	103.8	30.5
1.11	1.75	2.86	0.114	1.17	1148.5	0.020	0.008	91.2	28.6
1.11	1.75	2.86	0.114	1.37	1339.9	0.025	0.010	99.4	33.4
1.11	1.75	2.86	0.114	1.37	1339.9	0.025	0.010	99.4	33.4
1.11	1.75	2.86	0.114	1.37	1339.9	0.025	0.010	99.4	33.4
1.11	1.75	2.86	0.114	1.17	1148.5	0.025	0.010	73.0	28.6
1.11	1.75	2.86	0.114	1.45	1416.4	0.030	0.012	92.5	35.3
1.11	1.75	2.86	0.114	1.48	1454.7	0.030	0.012	97.6	36.3
1.11	1.75	2.86	0.114	1.52	1493.0	0.030	0.012	102.8	37.2
1.11	1.75	2.86	0.114	1.48	1454.7	0.030	0.012	97.6	36.3
1.11	1.75	2.86	0.114	1.60	1569.6	0.035	0.014	97.4	39.1
1.11	1.75	2.86	0.114	1.60	1569.6	0.035	0.014	97.4	39.1
1.11	1.75	2.86	0.114	1.60	1569.6	0.035	0.014	97.4	39.1
1.11	1.75	2.86	0.114	1.56	1531.3	0.035	0.014	92.7	38.2
1.11	1.75	2.86	0.114	1.91	1875.9	0.040	0.016	121.7	46.8
1.11	1.75	2.86	0.114	1.88	1837.5	0.040	0.016	116.8	45.8
1.11	1.75	2.86	0.114	1.88	1837.5	0.040	0.016	116.8	45.8
1.11	1.75	2.86	0.114	1.84	1799.2	0.040	0.016	112.0	44.9

Knox Co. Station 400 Large Weight w%=30.0 Dry Density = 1.463 Mg/m³

Mo	Ms	M	L	Vp	Ap	ho	h	Mr	Dev. Stress
(kg)	(kg)	(kg)	(m)	(V)	(m/s^2)	(m)	(m)	(MPa)	(kPa)
1.54	1.75	3.29	0.114	0.55	536.0	0.010	0.004	45.7	18.5
1.54	1.75	3.29	0.114	0.55	536.0	0.010	0.004	45.7	18.5
1.54	1.75	3.29	0.114	0.55	536.0	0.010	0.004	45.7	18.5
1.54	1.75	3.29	0.114	0.59	574.2	0.010	0.004	52.5	19.9
1.54	1.75	3.29	0.114	0.74	727.4	0.015	0.006	56.2	25.2
1.54	1.75	3.29	0.114	0.82	803.9	0.015	0.006	68.6	27.8
1.54	1.75	3.29	0.114	0.74	727.4	0.015	0.006	56.2	25.2
1.54	1.75	3.29	0.114	0.74	727.4	0.015	0.006	56.2	25.2
1.54	1.75	3.29	0.114	1.02	995.3	0.020	0.008	78.9	34.4
1.54	1.75	3.29	0.114	1.02	995.3	0.020	0.008	78.9	34.4
1.54	1.75	3.29	0.114	0.94	918.8	0.020	0.008	67.2	31.8
1.54	1.75	3.29	0.114	0.90	880.4	0.020	0.008	61.7	30.5
1.54	1.75	3.29	0.114	1.09	1071.9	0.025	0.010	73.2	37.1
1.54	1.75	3.29	0.114	1.13	1110.2	0.025	0.010	78.5	38.4
1.54	1.75	3.29	0.114	1.13	1110.2	0.025	0.010	78.5	38.4
1.54	1.75	3.29	0.114	1.05	1033.6	0.025	0.010	68.1	35.8
1.54	1.75	3.29	0.114	1.25	1225.0	0.030	0.012	79.7	42.4
1.54	1.75	3.29	0.114	1.33	1301.6	0.030	0.012	89.9	45.0
1.54	1.75	3.29	0.114	1.33	1301.6	0.030	0.012	89.9	45.0
1.54	1.75	3.29	0.114	1.33	1301.6	0.030	0.012	89.9	45.0
1.54	1.75	3.29	0.114	1.37	1339.9	0.035	0.014	81.7	46.4
1.54	1.75	3.29	0.114	1.41	1378.2	0.035	0.014	86.4	47.7
1.54	1.75	3.29	0.114	1.37	1339.9	0.035	0.014	81.7	46.4
1.54	1.75	3.29	0.114	1.41	1378.2	0.035	0.014	86.4	47.7

Knox Co. Station 500 Small Weight w%=19.7 Dry Density = 1.697 Mg/m³

Mo	Ms	M	L	Vp	Ap	ho	h	Mr	Dev. Stress
(kg)	(kg)	(kg)	(m)	(V)	(m/s^2)	(m)	(m)	(MPa)	(kPa)
1.11	1.86	2.98	0.114	0.94	918.8	0.010	0.004	126.7	22.9
1.11	1.86	2.98	0.114	0.90	880.4	0.010	0.004	116.3	22.0
1.11	1.86	2.98	0.114	0.94	918.8	0.010	0.004	126.7	22.9
1.11	1.86	2.98	0.114	0.90	880.4	0.010	0.004	116.3	22.0
1.11	1.86	2.98	0.114	1.17	1148.5	0.015	0.006	131.9	28.6
1.11	1.86	2.98	0.114	1.21	1186.7	0.015	0.006	140.9	29.6
1.11	1.86	2.98	0.114	1.13	1110.2	0.015	0.006	123.3	27.7
1.11	1.86	2.98	0.114	1.25	1225.0	0.015	0.006	150.1	30.5
1.11	1.86	2.98	0.114	1.37	1339.9	0.020	0.007	134.7	33.4
1.11	1.86	2.98	0.114	1.37	1339.9	0.020	0.007	134.7	33.4
1.11	1.86	2.98	0.114	1.41	1378.2	0.020	0.007	142.5	34.4
1.11	1.86	2.98	0.114	1.33	1301.6	0.020	0.007	127.1	32.5
1.11	1.86	2.98	0.114	1.56	1531.3	0.025	0.009	140.7	38.2
1.11	1.86	2.98	0.114	1.56	1531.3	0.025	0.009	140.7	38.2
1.11	1.86	2.98	0.114	1.60	1569.6	0.025	0.009	147.9	39.1
1.11	1.86	2.98	0.114	1.72	1684.5	0.025	0.009	170.3	42.0
1.11	1.86	2.98	0.114	1.84	1799.2	0.030	0.011	161.9	44.9
1.11	1.86	2.98	0.114	1.84	1799.2	0.030	0.011	161.9	44.9
1.11	1.86	2.98	0.114	1.84	1799.2	0.030	0.011	161.9	44.9
1.11	1.86	2.98	0.114	1.80	1761.0	0.030	0.011	155.1	43.9
1.11	1.86	2.98	0.114	2.03	1990.6	0.035	0.013	169.9	49.6
1.11	1.86	2.98	0.114	1.91	1875.9	0.035	0.013	150.9	46.8
1.11	1.86	2.98	0.114	1.99	1952.4	0.035	0.013	163.4	48.7
1.11	1.86	2.98	0.114	1.95	1914.1	0.035	0.013	157.1	47.7
1.11	1.86	2.98	0.114	2.15	2105.5	0.040	0.015	166.3	52.5
1.11	1.86	2.98	0.114	2.15	2105.5	0.040	0.015	166.3	52.5
1.11	1.86	2.98	0.114	2.19	2143.8	0.040	0.015	172.4	53.5
1.11	1.86	2.98	0.114	2.11	2067.3	0.040	0.015	160.3	51.5

Knox Co. Station 500 Large Weight w%=19.7 Dry Density = 1.697 Mg/m³

Mo	Ms	M	L	Vp	Ap	ho	h	Mr	Dev. Stress
(kg)	(kg)	(kg)	(m)	(V)	(m/s^2)	(m)	(m)	(MPa)	(kPa)
1.54	1.86	3.41	0.114	0.82	803.9	0.010	0.004	111.0	27.8
1.54	1.86	3.41	0.114	0.78	765.7	0.010	0.004	100.7	26.5
1.54	1.86	3.41	0.114	0.78	765.7	0.010	0.004	100.7	26.5
1.54	1.86	3.41	0.114	0.82	803.9	0.010	0.004	111.0	27.8
1.54	1.86	3.41	0.114	1.17	1148.5	0.015	0.006	151.1	39.7
1.54	1.86	3.41	0.114	1.02	995.3	0.015	0.006	113.5	34.4
1.54	1.86	3.41	0.114	1.09	1071.9	0.015	0.006	131.6	37.1
1.54	1.86	3.41	0.114	1.17	1148.5	0.015	0.006	151.1	39.7
1.54	1.86	3.41	0.114	1.41	1378.2	0.020	0.007	163.2	47.7
1.54	1.86	3.41	0.114	1.33	1301.6	0.020	0.007	145.5	45.0
1.54	1.86	3.41	0.114	1.25	1225.0	0.020	0.007	128.9	42.4
1.54	1.86	3.41	0.114	1.29	1263.3	0.020	0.007	137.1	43.7
1.54	1.86	3.41	0.114	1.56	1531.3	0.025	0.009	161.1	53.0
1.54	1.86	3.41	0.114	1.48	1454.7	0.025	0.009	145.4	50.3
1.54	1.86	3.41	0.114	1.37	1339.9	0.025	0.009	123.4	46.4
1.54	1.86	3.41	0.114	1.41	1378.2	0.025	0.009	130.5	47.7
1.54	1.86	3.41	0.114	1.68	1646.1	0.030	0.011	155.2	57.0
1.54	1.86	3.41	0.114	1.64	1607.8	0.030	0.011	148.0	55.6
1.54	1.86	3.41	0.114	1.64	1607.8	0.030	0.011	148.0	55.6
1.54	1.86	3.41	0.114	1.68	1646.1	0.030	0.011	155.2	57.0
1.54	1.86	3.41	0.114	1.72	1684.5	0.035	0.013	139.3	58.3
1.54	1.86	3.41	0.114	1.76	1722.7	0.035	0.013	145.7	59.6
1.54	1.86	3.41	0.114	1.72	1684.5	0.035	0.013	139.3	58.3
1.54	1.86	3.41	0.114	1.76	1722.7	0.035	0.013	145.7	59.6

Wilson Co. Station 28 Small Weight w%=22.7 Dry Density = 1.587 Mg/m³

Mo	Ms	M	L	Vp	Ap	ho	h	Mr	Dev. Stress
(kg)	(kg)	(kg)	(m)	(V)	(m/s^2)	(m)	(m)	(MPa)	(kPa)
1.11	1.79	2.90	0.114	1.29	1263.3	0.010	0.004	227.4	31.5
1.11	1.79	2.90	0.114	1.25	1225.0	0.010	0.004	213.8	30.5
1.11	1.79	2.90	0.114	1.29	1263.3	0.010	0.004	227.4	31.5
1.11	1.79	2.90	0.114	1.37	1339.9	0.010	0.004	255.8	33.4
1.11	1.79	2.90	0.114	1.56	1531.3	0.015	0.006	222.7	38.2
1.11	1.79	2.90	0.114	1.60	1569.6	0.015	0.006	234.0	39.1
1.11	1.79	2.90	0.114	1.64	1607.8	0.015	0.006	245.6	40.1
1.11	1.79	2.90	0.114	1.60	1569.6	0.015	0.006	234.0	39.1
1.11	1.79	2.90	0.114	1.76	1722.7	0.020	0.008	211.4	43.0
1.11	1.79	2.90	0.114	1.80	1761.0	0.020	0.008	220.9	43.9
1.11	1.79	2.90	0.114	1.76	1722.7	0.020	0.008	211.4	43.0
1.11	1.79	2.90	0.114	1.72	1684.5	0.020	0.008	202.1	42.0
1.11	1.79	2.90	0.114	2.03	1990.6	0.025	0.010	225.8	49.6
1.11	1.79	2.90	0.114	1.95	1914.1	0.025	0.010	208.8	47.7
1.11	1.79	2.90	0.114	1.84	1799.2	0.025	0.010	184.5	44.9
1.11	1.79	2.90	0.114	2.03	1990.6	0.025	0.010	225.8	49.6
1.11	1.79	2.90	0.114	2.27	2220.3	0.030	0.012	234.1	55.4
1.11	1.79	2.90	0.114	2.19	2143.8	0.030	0.012	218.3	53.5
1.11	1.79	2.90	0.114	2.19	2143.8	0.030	0.012	218.3	53.5
1.11	1.79	2.90	0.114	2.27	2220.3	0.030	0.012	234.1	55.4
1.11	1.79	2.90	0.114	2.34	2296.9	0.035	0.013	214.8	57.3
1.11	1.79	2.90	0.114	2.42	2373.5	0.035	0.013	229.3	59.2
1.11	1.79	2.90	0.114	2.34	2296.9	0.035	0.013	214.8	57.3
1.11	1.79	2.90	0.114	2.38	2335.2	0.035	0.013	222.0	58.2
1.11	1.79	2.90	0.114	2.58	2526.6	0.040	0.015	227.4	63.0
1.11	1.79	2.90	0.114	2.58	2526.6	0.040	0.015	227.4	63.0
1.11	1.79	2.90	0.114	2.46	2411.7	0.040	0.015	207.2	60.1
1.11	1.79	2.90	0.114	2.54	2488.4	0.040	0.015	220.6	62.0

Wilson Co. Station 28 Large Weight $w\%=22.7$ Dry Density = 1.587 Mg/m^3

Mo	Ms	M	L	Vp	Ap	ho	h	Mr	Dev. Stress
(kg)	(kg)	(kg)	(m)	(V)	(m/s ²)	(m)	(m)	(MPa)	(kPa)
1.54	1.79	3.33	0.114	1.09	1071.9	0.010	0.004	188.1	37.1
1.54	1.79	3.33	0.114	0.98	957.1	0.010	0.004	149.9	33.1
1.54	1.79	3.33	0.114	0.98	957.1	0.010	0.004	149.9	33.1
1.54	1.79	3.33	0.114	1.09	1071.9	0.010	0.004	188.1	37.1
1.54	1.79	3.33	0.114	1.25	1225.0	0.015	0.006	163.8	42.4
1.54	1.79	3.33	0.114	1.41	1378.2	0.015	0.006	207.3	47.7
1.54	1.79	3.33	0.114	1.29	1263.3	0.015	0.006	174.2	43.7
1.54	1.79	3.33	0.114	1.33	1301.6	0.015	0.006	184.9	45.0
1.54	1.79	3.33	0.114	1.72	1684.5	0.020	0.008	232.2	58.3
1.54	1.79	3.33	0.114	1.72	1684.5	0.020	0.008	232.2	58.3
1.54	1.79	3.33	0.114	1.60	1569.6	0.020	0.008	201.6	54.3
1.54	1.79	3.33	0.114	1.60	1569.6	0.020	0.008	201.6	54.3
1.54	1.79	3.33	0.114	1.72	1684.5	0.025	0.010	185.8	58.3
1.54	1.79	3.33	0.114	1.72	1684.5	0.025	0.010	185.8	58.3
1.54	1.79	3.33	0.114	1.76	1722.7	0.025	0.010	194.3	59.6
1.54	1.79	3.33	0.114	1.76	1722.7	0.025	0.010	194.3	59.6
1.54	1.79	3.33	0.114	1.91	1875.9	0.030	0.012	192.0	64.9
1.54	1.79	3.33	0.114	1.84	1799.2	0.030	0.012	176.6	62.3
1.54	1.79	3.33	0.114	1.84	1799.2	0.030	0.012	176.6	62.3
1.54	1.79	3.33	0.114	1.91	1875.9	0.030	0.012	192.0	64.9
1.54	1.79	3.33	0.114	2.03	1990.6	0.035	0.013	185.3	68.9
1.54	1.79	3.33	0.114	1.84	1799.2	0.035	0.013	151.4	62.3
1.54	1.79	3.33	0.114	2.03	1990.6	0.035	0.013	185.3	68.9
1.54	1.79	3.33	0.114	2.03	1990.6	0.035	0.013	185.3	68.9

Knox Co. Rutledge Pike Small Weight w%=36.7 Dry Density = 1.330 Mg/m³

Mo	Ms	M	L	Vp	Ap	ho	h	Mr	Dev. Stress
(kg)	(kg)	(kg)	(m)	(V)	(m/s^2)	(m)	(m)	(MPa)	(kPa)
1.11	1.67	2.78	0.114	0.70	689.1	0.010	0.004	62.1	17.2
1.11	1.67	2.78	0.114	0.63	612.5	0.010	0.004	49.1	15.3
1.11	1.67	2.78	0.114	0.66	650.8	0.010	0.004	55.4	16.2
1.11	1.67	2.78	0.114	0.70	689.1	0.010	0.004	62.1	17.2
1.11	1.67	2.78	0.114	0.78	765.7	0.015	0.006	51.2	19.1
1.11	1.67	2.78	0.114	0.78	765.7	0.015	0.006	51.2	19.1
1.11	1.67	2.78	0.114	0.82	803.9	0.015	0.006	56.4	20.0
1.11	1.67	2.78	0.114	0.82	803.9	0.015	0.006	56.4	20.0
1.11	1.67	2.78	0.114	0.98	957.1	0.020	0.008	59.9	23.9
1.11	1.67	2.78	0.114	0.94	918.8	0.020	0.008	55.2	22.9
1.11	1.67	2.78	0.114	0.98	957.1	0.020	0.008	59.9	23.9
1.11	1.67	2.78	0.114	0.98	957.1	0.020	0.008	59.9	23.9
1.11	1.67	2.78	0.114	1.09	1071.9	0.025	0.010	60.2	26.7
1.11	1.67	2.78	0.114	1.09	1071.9	0.025	0.010	60.2	26.7
1.11	1.67	2.78	0.114	1.05	1033.6	0.025	0.010	55.9	25.8
1.11	1.67	2.78	0.114	1.05	1033.6	0.025	0.010	55.9	25.8
1.11	1.67	2.78	0.114	1.17	1149.5	0.030	0.012	57.6	28.7
1.11	1.67	2.78	0.114	1.21	1186.7	0.030	0.012	61.4	29.6
1.11	1.67	2.78	0.114	1.17	1148.5	0.030	0.012	57.5	28.6
1.11	1.67	2.78	0.114	1.25	1225.0	0.030	0.012	65.5	30.5
1.11	1.67	2.78	0.114	1.29	1263.3	0.035	0.014	59.7	31.5
1.11	1.67	2.78	0.114	1.33	1301.6	0.035	0.014	63.4	32.5
1.11	1.67	2.78	0.114	1.25	1225.0	0.035	0.014	56.1	30.5
1.11	1.67	2.78	0.114	1.29	1263.3	0.035	0.014	59.7	31.5
1.11	1.67	2.78	0.114	1.37	1339.9	0.040	0.016	58.7	33.4
1.11	1.67	2.78	0.114	1.41	1378.2	0.040	0.016	62.2	34.4
1.11	1.67	2.78	0.114	1.41	1378.2	0.040	0.016	62.2	34.4
1.11	1.67	2.78	0.114	1.37	1339.9	0.040	0.016	58.7	33.4
1.11	1.67	2.78	0.114	1.52	1493.0	0.045	0.018	64.8	37.2
1.11	1.67	2.78	0.114	1.41	1378.2	0.045	0.018	55.2	34.4
1.11	1.67	2.78	0.114	1.52	1493.0	0.045	0.018	64.8	37.2
1.11	1.67	2.78	0.114	1.41	1378.2	0.045	0.018	55.2	34.4

Knox Co. Rutledge Pike Large Weight w%=36.7 Dry Density = 1.330 Mg/m³

Mo	Ms	M	L	Vp	Ap	ho	h	Mr	Dev. Stress
(kg)	(kg)	(kg)	(m)	(V)	(m/s^2)	(m)	(m)	(MPa)	(kPa)
1.54	1.67	3.21	0.114	0.66	650.8	0.010	0.004	64.0	22.5
1.54	1.67	3.21	0.114	0.66	650.8	0.010	0.004	64.0	22.5
1.54	1.67	3.21	0.114	0.66	650.8	0.010	0.004	64.0	22.5
1.54	1.67	3.21	0.114	0.63	612.5	0.010	0.004	56.7	21.2
1.54	1.67	3.21	0.114	0.82	803.9	0.015	0.006	65.1	27.8
1.54	1.67	3.21	0.114	0.82	803.9	0.015	0.006	65.1	27.8
1.54	1.67	3.21	0.114	0.78	765.7	0.015	0.006	59.1	26.5
1.54	1.67	3.21	0.114	0.74	727.4	0.015	0.006	53.3	25.2
1.54	1.67	3.21	0.114	0.98	957.1	0.020	0.008	69.3	33.1
1.54	1.67	3.21	0.114	0.90	880.4	0.020	0.008	58.6	30.5
1.54	1.67	3.21	0.114	0.94	918.8	0.020	0.008	63.8	31.8
1.54	1.67	3.21	0.114	0.94	918.8	0.020	0.008	63.8	31.8
1.54	1.67	3.21	0.114	1.05	1033.6	0.025	0.010	64.6	35.8
1.54	1.67	3.21	0.114	1.05	1033.6	0.025	0.010	64.6	35.8
1.54	1.67	3.21	0.114	1.05	1033.6	0.025	0.010	64.6	35.8
1.54	1.67	3.21	0.114	1.02	995.3	0.025	0.010	59.9	34.4
1.54	1.67	3.21	0.114	1.13	1110.2	0.030	0.012	62.1	38.4
1.54	1.67	3.21	0.114	1.13	1110.2	0.030	0.012	62.1	38.4
1.54	1.67	3.21	0.114	1.17	1148.5	0.030	0.012	66.5	39.7
1.54	1.67	3.21	0.114	1.13	1110.2	0.030	0.012	62.1	38.4
1.54	1.67	3.21	0.114	1.29	1263.3	0.035	0.014	68.9	43.7
1.54	1.67	3.21	0.114	1.29	1263.3	0.035	0.014	68.9	43.7
1.54	1.67	3.21	0.114	1.21	1186.7	0.035	0.014	60.8	41.1
1.54	1.67	3.21	0.114	1.17	1148.5	0.035	0.014	57.0	39.7
1.54	1.67	3.21	0.114	1.33	1301.6	0.040	0.016	64.0	45.0
1.54	1.67	3.21	0.114	1.33	1301.6	0.040	0.016	64.0	45.0
1.54	1.67	3.21	0.114	1.37	1339.9	0.040	0.016	67.9	46.4
1.54	1.67	3.21	0.114	1.33	1301.6	0.040	0.016	64.0	45.0

Appendix C

Using the ATM Procedure

Setting Up Gagescope

Before turning on the computer to begin ATM testing, plug the cable into channel 2 into the back of the computer. This cable connects the computer to the signal conditioner (amp/battery pack). The ATM test hammer cable should be plugged into the input side of the signal conditioner and the output side of the conditioner is linked with the computer. The signal conditioner setting can usually be kept on the gain setting 1 (on the signal conditioner)..

1. Select the TNDOT setup option. Get into Gagescope directory by typing "cd gagescope" at the c:\ prompt and press "enter".
2. Type "TNDOT".
3. You are now in the TNDOT program.
4. The program will show "Use the last group file?(y/n)." You can input the classification information for the soil, or simply type "y" when prompted to bypass this step.
5. The TNDOT program is linked to Gagescope. Gagescope is the digital oscilloscope software that records signals from the accelerometer in the test hammer. Signals are recorded by Gagescope and further manipulated by the TNDOT program.
6. The F10 key will allow you to get into Gagescope from TNDOT. When gagescope is entered you are prompted for a user name (press "enter" to bypass this step). A message will appear that not all of the channels are enabled and it will ask if you want to enable these channels. Type "n". Make sure that channel 2 is turned on within the Gagescope software. The channel corresponds to the set of outlet plugs on the back of the computer. (Channel 1 should be turned off, but the master channel and channel 2 must stay on).
7. Now you are ready to get a signal from the accelerometer. The ATM hammer is raised to an initial height. Hit the left arrow (←) until you come to the "capture" menu. Hit "enter" to see the pull-down menu. Hit the down arrow (↓) until the "get data" line is highlighted. Press "enter".
8. The computer will tell you that it is waiting on a trigger. Release the ATM hammer using the drop mechanism. A curve should appear on the screen. Hit the left arrow to get to the signal menu. Hit "enter" and then "save signal". Gagescope asks for a channel number. Enter "2" and press "enter". Save the signal file as "####.sig" (e.g. soil1.sig). It is important to develop a signal naming convention. Since each

signal is stored as a file, and a typical ATM test takes 20 to 25 files, the files can become cluttered. If the files are cluttered, it is easy to forget which files went with a particular soil. The time base for the screen should be set at 500 us/div. and the gain should be at 2 volts/div.

8. Now arrow over to the file menu and highlight "quit" and press "enter" to get out of Gagescope. (Several signals for one drop height could be captured and saved before going to the TNDOT program).

TNDOT Program

9. The TNDOT program will appear on the screen. Type the name of the signal file you want to manipulate (e.g. soil1.sig) and press "enter".

10. Arrow down and input the appropriate gain setting (1,10,100), poisson's ratio (assumed to be 0.45 in tests conducted at UT), and soil weight in lbs. (2 decimal places). Press "enter" after each value is entered. It is important to note, that each value should be input in order from top to bottom of the page. Arrow down to the hammer weight spot. The type of hammer corresponding to the test can be selected with the left or right arrow. The hammers are referred to as "n", "s", or "l" (none, small, large). The small weight is considered the ATM hammer with the small add-on weight, while the large weight is the hammer plus the large weight. Select the hammer configuration that is being used to test the soil and press "enter". Finally, enter the drop height (10mm or greater) and press "enter".

11. After the input sequence is complete, hit the F1 key to calculate the value for resilient modulus. Resilient modulus values tested with ATM typically range from about 50 Mpa to 200 Mpa.

12. After the F1 key is used, the values can be saved into a data file. Hit the F3 key to save the file with an appropriate file name and extension (e.g. soil1.dat).

13. Several data files can be saved into a group file. Figure A-1 shows a chart of the file naming process. Hit the F4 key to create a group file. You will be prompted for the name of the new group (e.g. soils) with no file extension. Enter the number of data files that you want to group and press "enter". Type the name of the first data file (e.g. soil1.dat) and press "enter". Then enter the name of the second data file (e.g. soil2.dat) and press "enter", and so on.

14. The F5 key will allow the user to see the ATM results for a particular group. Enter the group name (e.g. soils) and indicate whether you want a hard copy of the report with "n" or "y".

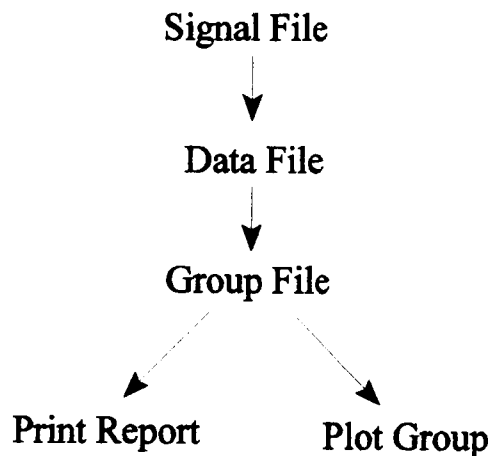
15. The F6 key allows the user to see a plot of the group file results.

Exiting the TNDOT Program

After you are finished using the TNDOT program, go back to the main program screen. Press F10 to go back into gagescope to get more signal files or you can exit the program. Press "escape" to exit the program. The program will ask you if you really want to quit. Type "y" for yes.

Getting Out of Trouble

If the TNDOT program becomes "locked up" hit "escape". If "escape" does not work, press "control" and "c" at the same time. Then type "cls" (the cursor may be covered up by the TNDOT screen, but "cls" will clear the screen). This will return you to the gagescope subdirectory prompt.



Process for Naming and Viewing Files

Setting Up the ATM Device

In order to get good ATM results, all components of the system must completely vertical. The ATM frame should be placed on a firm levelling table. An ideal supporting table would be made of a poured-in-place concrete block with a level surface, since the base must be rigid to receive good signals.

The alignment of the frame in relation to the mold can be checked (prior to placing the proctor mold on the frame) with the use of a small plumb bob. The plumb bob can be suspended from a string that hangs through the center of a detachable

pressure fitting. This ensures that the plumbob is completely centered. If the frame is level, the plumb bob tip should hang directly over the small indentation at the center of the bottom of the frame.

The proctor mold is placed upon the circular pedestal on the ATM frame. The mold can be levelled with the use of a small carpenter's square and adjusting the mold until it is square on at least two sides. There are three levelling screws that can be tightened to ensure that the mold does not move. It is important that the mold be levelled before tightening the screws, or else the sample will become rounded on one bottom edge and will tend to wobble during testing. The screws should only be tightened enough to secure the sample. Excessive tightening will force part of the soil sample out of the mold.

Sample Preparation

The samples are compacted using standard procedures in a proctor mold. The sides of the mold should be sprayed with a light coating of canola oil-based vegetable spray to minimize sidewall friction. The sample should be covered with plastic in between lapses in testing. Samples left in the mold for long periods of time (e.g. 24 hrs.) tend to swell. This swelling tends to cause higher resilient modulus values and should be avoided.

Vita

Vita

Jason Stuart Reeves was born on April 15, 1971, in Lexington, Kentucky. He lived in Grayson, Kentucky until 1982, after which his mother moved his family to Greeneville, Tennessee. The author graduated from Chuckey-Doak High School in May of 1989, and later enrolled at the University of Tennessee, Knoxville in the following fall. After a brief stint as an architecture student, he changed his major to civil engineering. Jason received a Bachelor of Science degree in civil engineering in December of 1993. He later enrolled in the University of Tennessee Graduate School with an emphasis in geotechnical engineering. While attending graduate school, he became engaged to wed Miss Dawn Currie. The two of them plan to reside in Charlotte, North Carolina.