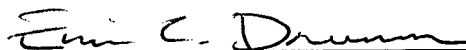
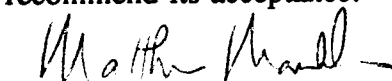
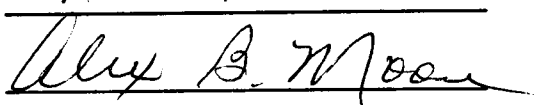


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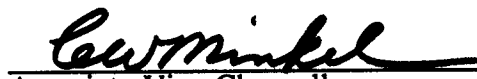
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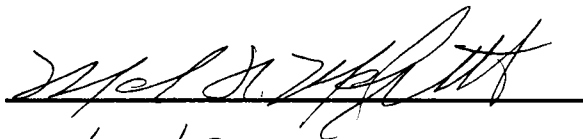

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**Verification Testing and Alternative Test Methods for the Resilient Modulus of
Fine Grained Soils**

A Thesis

Presented for the

Master of Science

Degree

The University of Tennessee, Knoxville

Mark Raulston Madgett

August 1994

DEDICATION

To my parents, Mr. and Mrs. Robert W. Madgett. This would not have been possible without their support and encouragement when the tasks seemed insurmountable.

To my fiance, Ms. Julia C. Ketron, who's love and support gave me the motivation to overcome all adversities and finish this thesis.

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ABSTRACT

Under the new AASHTO (1986) pavement design method, the resilient modulus, M_r , is required to define the elastic modulus for the subgrade soil. This method of designing is mechanistic, which is an improvement over the older empirical method (AASHTO 1972). The cyclic triaxial test has been the standard method of determining the resilient modulus. This method is however, time consuming and does not always produce accurate, reproducible results. Therefore, methods of estimating the resilient response of a subgrade soil are necessary to make the use of resilient modulus in mechanistic design feasible.

A laboratory investigation was conducted on six soils that were obtained from various sites across the state of Tennessee. For each of these six soils, index tests were conducted to determine the properties of each soil. Specimens were compacted at optimum water content and maximum dry density (standard energy), and laboratory measurements of the resilient modulus were performed using the cyclic triaxial method, and an alternative test method (Li 1992). Model parameters were found for two different analytical models. In addition to testing specimens at optimum water content and maximum dry density, three of the soils were compacted at optimum conditions and the degree of saturation was raised to a higher level of saturation before testing. Significant reduction in resilient modulus was observed as the degree of saturation was increased. An alternative test method (Li 1992) based on a falling mass striking a soil specimen compacted in a proctor mold was also used. The ATM detected changes in M_r as measured with the cyclic triaxial method. However, the effects of saturation were not

distinct as in the triaxial test. Furthermore, the response at low values of deviator stress is not measured accurately with the ATM. It is suggested that other variations of hammer weight and drop height be evaluated. However, from this testing it was concluded that while the cyclic triaxial method produces more consistent data, the alternative test method may represent the in-situ stress state better.

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Chapter 1 INTRODUCTION

1.1 Importance of Resilient Modulus Testing

In the design of pavement systems, the mechanical properties of all the materials in the system must be known in order to design a pavement that is both economical and durable. The materials that are used in typical asphaltic concrete pavements systems are an asphaltic concrete layer, a granular base and sometimes subbase layer, and the soil subgrade. Since the performance (or deflection) of a pavement system depends heavily on the subgrade, the properties of the subgrade must be considered in the design process. Older design methods (AASHTO 1972) utilized experience and empirically formed solutions to determine the appropriate stiffness of the subgrade material rather than directly measuring the stiffness in the laboratory. Without a reliable estimation of this stiffness, the pavement system may be designed too thin, resulting in premature degradation, or designed too thick which would be uneconomical. To better describe the subgrade stiffness, the current AASHTO Guide (AASHTO 1986) recommends using the resilient modulus, rather than the structural number, R value, and CBR value which were all empirically related to the subgrade soil stiffness (Huang 1993). Therefore, the use of the resilient modulus to describe the stiffness of the various layers in the pavement systems will make the design mechanistic rather than empirical.

1.2 Definition of Resilient Modulus (M_r)

The Resilient Modulus, M_r , is similar in form to Young's Modulus, E . The

difference between them is that M_r is a dynamic material property, and E is a static or quasi-static material property. M_r is defined as (Elliott and Thornton, 1988):

$$M_r = \frac{\sigma_d}{\epsilon_r} \quad (1-1)$$

Where

M_r = resilient modulus

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

σ_1 = major principal stress

σ_3 = minor principal stress (cell pressure in a triaxial test)

ϵ_r = the resilient, or recoverable axial strain

The deviator stress, σ_d , is applied cyclically to the test specimen, while the corresponding strain is obtained. The resulting value of M_r is the secant modulus of the loading curve. Figure 1-1 shows the load and unload curve for a specimen and the corresponding M_r . The only portion of the strain that is of interest is the recoverable portion which is resilient rather than permanent. This resilient strain corresponds to the modulus which is resilient (Monismith et al., 1967). Under typical traffic loading conditions, it is assumed that the stresses in the subgrade are in the elastic range. Therefore as noted by Monismith et al. (1967), the resilient or instantaneous deformations are responsible for the fatigue failure in a pavement system. The permanent, or plastic, deformations are related to the rutting of the pavement system which is not considered here.

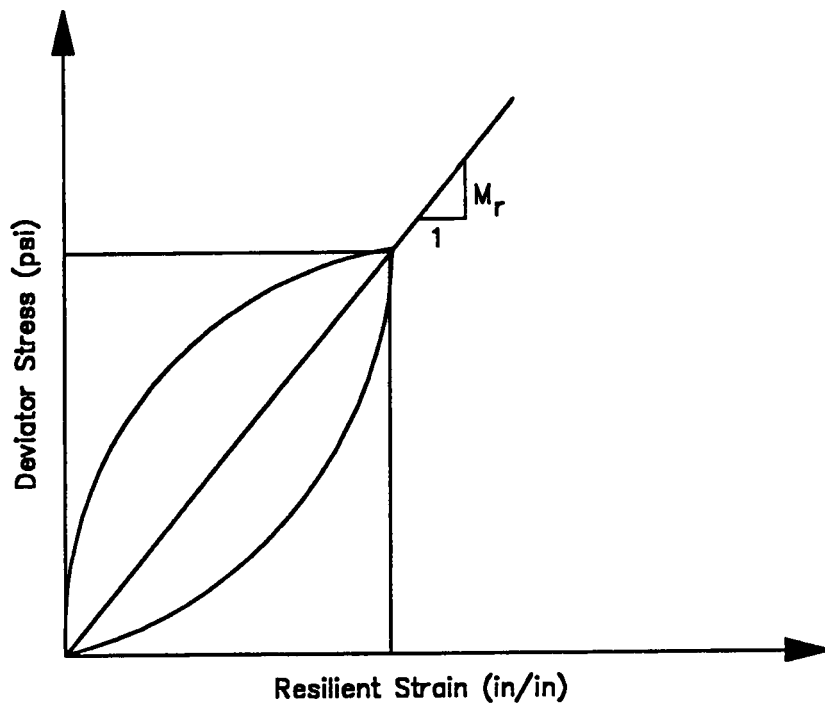


Figure 1-1. Definition of Resilient Modulus on the σ - ϵ Curve.

The resilient modulus is highly dependent upon the magnitude of applied deviator stress. Therefore, the resilient modulus is typically plotted versus the deviator stress to show the change in resilient modulus with varying stress levels (Thompson and Robnett 1976; Hudson 1992; Drumm et al. 1993). The fine-grained soils from Tennessee that were tested by Hudson (1992) exhibit two basic types of behavior. In all tests the value of M_r increased greatly as the deviator stress decreased at deviator stresses below approximately 13.8 kPa (2 psi). As the deviator stress continued to increase above 13.8 kPa (2 psi), the soils either reached a minimum value of M_r , or the value of M_r continually decreased. The degree of increase or decrease in M_r as the deviator stress increased varied for different soils as well as different compaction conditions. These different types of behavior are shown in Figure 1-2. Most of the tests run at optimum

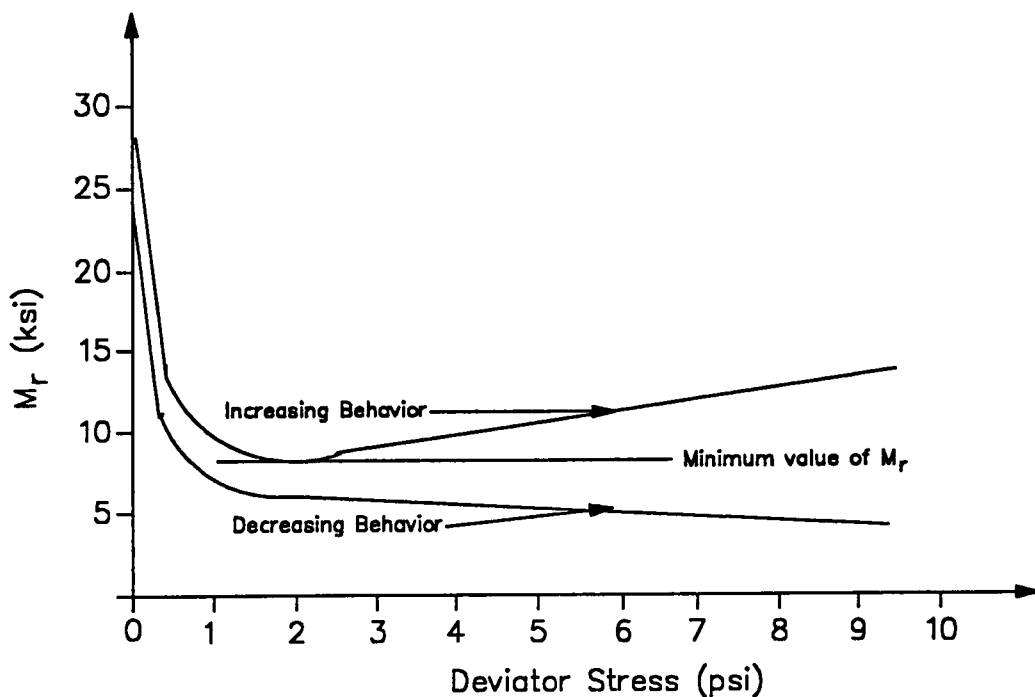


Figure 1-2. M_r vs. Stress Behavior of Previous Samples.

water content and above by Hudson (1992) show the typical decrease or constant value of M_r as σ_d increased. The samples that reached a minimum value of M_r and then increased as σ_d increased were usually at compaction water contents lower than optimum as noted by Hudson (1992). This change in M_r over different stress levels is important due to the different levels of stress that can be expected in a subgrade material. A thick asphaltic concrete layer and granular base layer will result in lower stresses on the subgrade due to vehicular loading. As the thickness of the AC layer and the base layer decreases, the stress in the subgrade increases. The thickness of these layers can be adjusted to control the stress level in the subgrade such that the instantaneous deflections caused by traffic loading are not great enough to cause damage to the pavement system.

1.3 Use of M_r in Pavement Analysis and Design

The resilient modulus is used in design and analysis of pavement systems to define the elastic modulus of the subgrade layer. In analysis procedures, the resilient response of the subgrade should be dependent on stress level in order to accurately depict the system behavior. To accomplish this, some type of model must be used to find the value of M_r at a certain stress level. In design, the resilient modulus that best describes the subgrade must be found directly. This may be accomplished by laboratory procedures or by some approximate method.

1.3.1 Methods of Analysis

Analysis of a pavement system can be done by a variety of methods. Two of the most widely used methods are the layered systems concept (Witczak and Yoder 1975) and the finite element method (FEM) (Huang 1993). The behavior of layered systems, such as an asphaltic concrete system, may be described by elastic theory if the elastic modulus, E , and Poisson's ratio, ν , are known for each of the layers (Huang 1993). The elastic modulus, E , is taken as the resilient modulus. Since M_r varies with the applied stress, it is necessary to use some type of mathematical relationship to describe the resilient response. Four mathematical models were investigated by Hudson (1992) and the coefficients of two models were found from cyclic triaxial tests. From these coefficients, the M_r - σ_d relationship of each of the soils tested can be back-calculated and used in analysis methods. The four models discussed by Hudson (1992) were: Log-Log model, bilinear model, hyperbolic model, and modified hyperbolic model. Models of

this type are used with analysis techniques based on the FEM. The FEM permits the analysis of multi-layered systems which are difficult to describe in closed-form solutions.

1.3.1.1 Log- Log Model

The log-log or exponential model (Moossazadeh and Witczak, 1981) is a simple model which predicts the resilient response of a soil with two constants. The model is of the form:

$$M_r = K_1 (\sigma_d)^{K_2} \quad (1-2)$$

Where: M_r = Resilient Modulus

K_1 and K_2 = Constants

σ_d = Deviator Stress

The constants K_1 and K_2 are parameters which are dependent on the soils properties. K_1 and K_2 are determined by plotting M_r as the ordinate (log-scale) and σ_d as the abscissa (log-scale). From this graph, K_1 is the Y-intercept of the regression line or best fit line through the data points and K_2 is the slope of the line. After K_1 and K_2 have been assigned, the M_r - σ_d response curve can be constructed by simply substituting in values for σ_d and calculating the corresponding M_r value. Hudson (1992) noted that one disadvantage of this model is that it tends to predict high values of M_r at the lower values of deviator stresses and should have some pre-set limit on the magnitude of the resilient modulus at low deviator stresses so that these high predictions could be eliminated. This model is used for reporting M_r data in the SHRP protocol

1.3.1.2 Hyperbolic Model

The hyperbolic model utilizes a normalization procedure to find a linear relation between $M_r\sigma_d$ and σ_d . Introduced by Kondner (1963) for representing stress-strain response, this model can be written as (Drumm et al., 1990):

$$M_r = \frac{(a + b\sigma_d)}{\sigma_d} \quad (1-3)$$

which can be rewritten as:

$$M_r\sigma_d = a + b\sigma_d \quad (1-4)$$

Where:

a = slope intercept, kPa^2 (psi^2)

b = Slope of the $M_r \sigma_d$ vs. σ_d curve, kPa (psi)

If the data from a M_r test are plotted as $M_r\sigma_d$ vs. σ_d , then the resulting line will have a slope and intercept equal to b and a , respectively. The fit of this model to the data will depend on how well the data fits the shape of a hyperbola, but in general, high correlation coefficients should be expected from the regression that yields the coefficients a and b .

1.3.1.3 Bilinear Model

Introduced by Terrel and Award (1972), this model represents the nonlinear resilient response with two straight line segments. The point at which the straight

segments meet is defined as the breakpoint, or breakpoint modulus. Extensive use of this model has been made by Thompson and Robnett (1976), and for the soils that they tested, the breakpoint modulus has been found to occur at $\sigma_d = 41.4$ kPa (6 psi). This model has several parts to describe both segments of the curve and will allow for differences associated with very soft, soft, medium, and stiff soils. These forms are as follows:

$$\begin{array}{ll}
 M_r = c_1 & 0 \leq \sigma_d \leq \sigma_{\min} \\
 M_r = a_1 + k_1 \sigma_d & \sigma_{\min} \leq \sigma_d \leq \sigma_i \\
 M_r = a_2 + k_2 \sigma_d & \sigma_i \leq \sigma_d \leq \sigma_{\max} \\
 M_r = c_2 & \sigma_{\max} \leq \sigma_d
 \end{array}$$

Where:

$$\sigma_{\min} = 2 \text{ psi}$$

c_1 = Upper limit of the resilient modulus

a_1 = Y-intercept of the first straight line segment

k_1 = Slope of the first straight line segment

σ_i = Breakpoint modulus

a_2 = Y-intercept of the second straight line segment

k_2 = Slope of the second straight line segment

Numerous people have used this model to investigate the resilient response of soils in the past (Thompson and Robnett 1976; Thompson and Elliott 1985; Drumm et al. 1992).

1.3.1.4 Modified Hyperbolic Model

As suggested by Drumm et al. (1992), the modified hyperbolic model sets both upper and lower limits on the values of resilient modulus that can be obtained. This is done in order to improve on the limitations of the log-log and hyperbolic models. The form of the model is:

$$M_r = \frac{(E_{\max} - E_{\min})}{[1 + c(\sigma_d)^d]} + E_{\min}$$

Where: $E_{\min}$ = lower limit
 $E_{\max}$ = upper limit
c and d = material constants

1.4 Laboratory Measurement of M_r

Laboratory determinations provide a direct measurement of the resilient modulus and are essential for designing a reliable and economical AC pavement system. Several laboratory methods are available for measuring the resilient modulus. The most popular and widely accepted method is the cyclic triaxial test.

1.4.1 Cyclic Triaxial Method

The cyclic triaxial method uses a loading device, such as an electro-pneumatic device, to apply a dynamic load to a soil specimen in a modified triaxial test cell. Figure 1-3 shows a triaxial cell used for testing at the University of Tennessee with all of the equipment necessary for the cyclic triaxial test. The load is applied through a loading

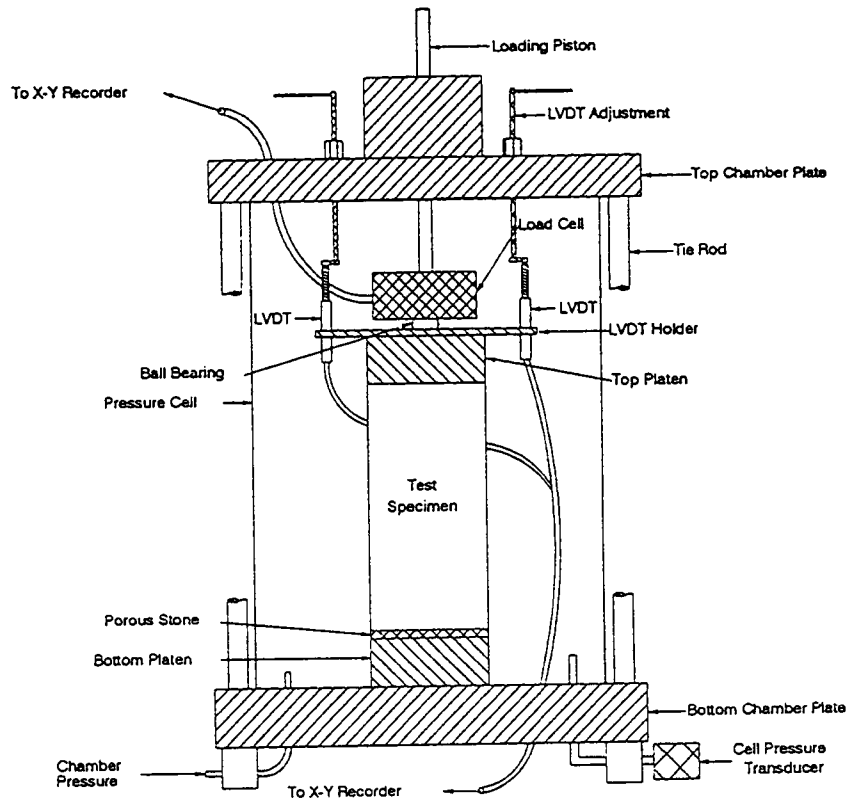


Figure 1-3. Triaxial Cell Setup used for Resilient Modulus Testing

piston which has a load cell attached. The load cell signals are passed through a signal conditioning unit which are then captured by some a recording device. The deflections are measured by use of linear variable displacement transducers, or LVDTs. These LVDTs may be mounted either externally or internally in the triaxial cell and the signals are measured the same way as the load cell. If the LVDTs are mounted internally, then they may also be mounted on the end platens or on the specimen itself. The deflections measured during a cyclic triaxial test at a deviator stress level of 6.9 kPa (1 psi) can be as low as 0.00076 mm (0.00003 in.) for a specimen height of 14.2 cm (5.6 in.). Therefore all LVDTs must be very sensitive in order to accurately measure the strain in the soil specimen.

The major and minor principal stresses are assumed to be oriented vertically and horizontally, respectively, in the cyclic triaxial test. The cell pressure, σ_c , represents the minor principal stress, σ_3 . The load applied to the top of the specimen is then σ_1 . There are however, limitations to this assumption that should be noted. At the top and bottom of the specimen, shear stresses can develop due to friction between the specimen and the platens as the specimen tries to strain radially under the cyclic load. These shear stresses that can develop, cause the major and minor principal stresses to be oriented in unknown directions.

There are several procedures defined for measuring the resilient modulus using the cyclic triaxial method including the AASHTO T274-82 method (AASHTO: "Methods" 1986), the SHRP Protocol P46 (SHRP 1989), the AASHTO T294-92I (AASHTO: "Methods" 1992), and the Simplified method (Elliott and Thornton 1988). A detailed study of the procedures was conducted by Hudson (1992). From this study, a procedure was identified for use in the M_r testing at the University of Tennessee. The SHRP Protocol P46 is the method that is being used for the cyclic triaxial testing (SHRP 1989). This procedure is summarized in Tables 1-1 to 1-3. Equipment setup for cyclic triaxial testing is complex and expensive and regular calibration of the equipment is essential to obtain accurate readings. It is therefore, difficult for smaller agencies to perform resilient modulus testing. The preparation of specimens is also complex and time consuming, but is feasible if numerous specimens are being tested. The test procedure itself requires several load repetitions at different stages in the test as shown in Table 1-3. There are a large amount of repetitions to guarantee that no permanent

Table 1-1. Equipment Setup for SHRP Protocol P46

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 in.

Table 1-2. Specimen Compaction Specifications for SHRP Protocol P46

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

Table 1-3. Testing Procedure for SHRP Protocol P46

TEST PROCEDURE	SHRP PROTOCOL P46		
CONDITIONING (COHESIVE)	σ_c (psi)	σ_d (psi)	Repetitions
	6	4	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

deformations are measured during the test. All of these factors make the resilient modulus test difficult to perform and susceptible to errors.

1.5 Methods of Estimating the Resilient Response

An interlaboratory study on standard specimens was conducted by Lenker (1992). The results of this study show that the resilient response found using the cyclic triaxial method are difficult to duplicate between laboratories and may be easily influenced by equipment setup. Since M_r is difficult to determine in the laboratory and requires sophisticated equipment, there is a need for approximate methods which would allow accurate estimation of M_r . Approximate methods proposed in the past include: a CBR (California Bearing Ratio) to M_r relationship, statistical models, a handbook procedure, and an alternative test method (ATM).

1.5.1 CBR- M_r Relationship

Several researchers have investigated the correlation of CBR with M_r (Barksdale and Leonards 1967; Thompson and Robnett 1973; deMedina and Preussler 1982; Brodsky 1989; and Drumm et al. 1993a). The AASHTO (1986) pavement design guide suggests that estimation of M_r can be made by the following equation:

$$M_r = 1.5 \times CBR \text{ (ksi)} \quad (1-6)$$

In spite of various investigations that do not verify this relationship (Drumm et al. 1993a, Thompson and Robnett 1973), it remains in use. A comparison of soaked CBR data with M_r values obtained from cyclic triaxial tests has shown that this correlation is not reliable

(Drumm et al. 1993a). Figures 1-4 and 1-5 show this comparison. Three different relationships are plotted on each of the figures for M_r equal to $0.75 \times \text{CBR}$, $1.5 \times \text{CBR}$, and $3.0 \times \text{CBR}$ (ksi). The values of M_r fall within this range, but the scatter shows no pattern with which to correlate CBR and M_r , and the range of estimated M_r values is excessive. These figures demonstrate that relationships between CBR data and M_r should be used with appropriate caution.

1.5.2 Statistical models

Statistical models can be highly useful to characterize the resilient response of subgrade soils. There are two main purposes for the development of statistical models.

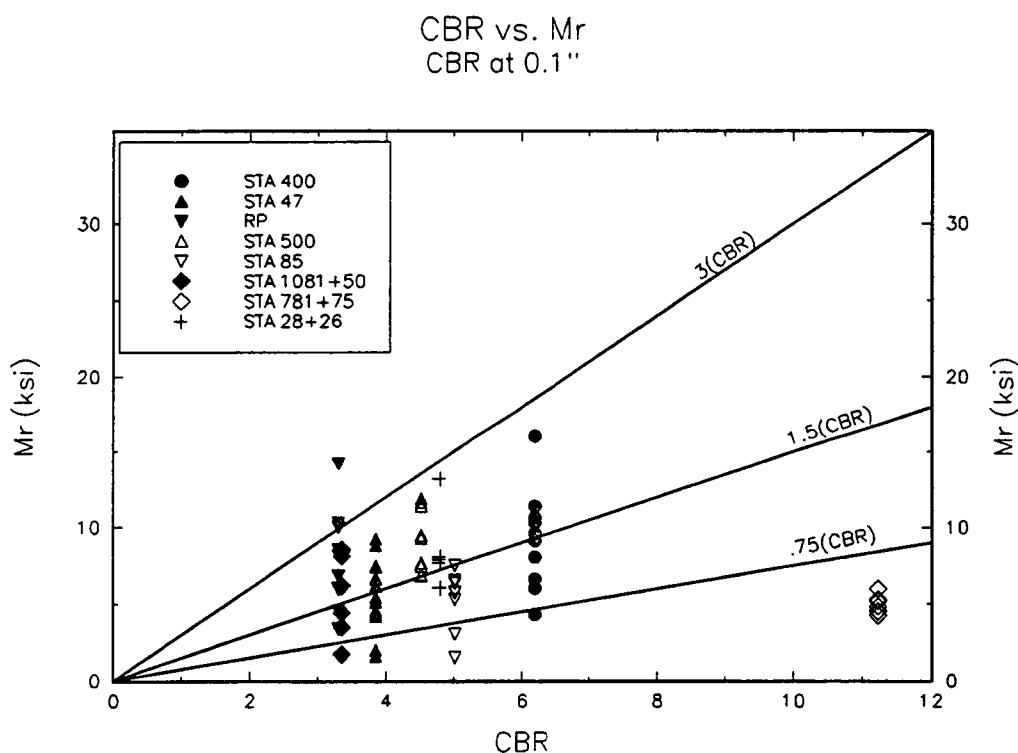


Figure 1-4. Relationship between M_r and CBR at 0.1", after Drumm et al. (1993a).

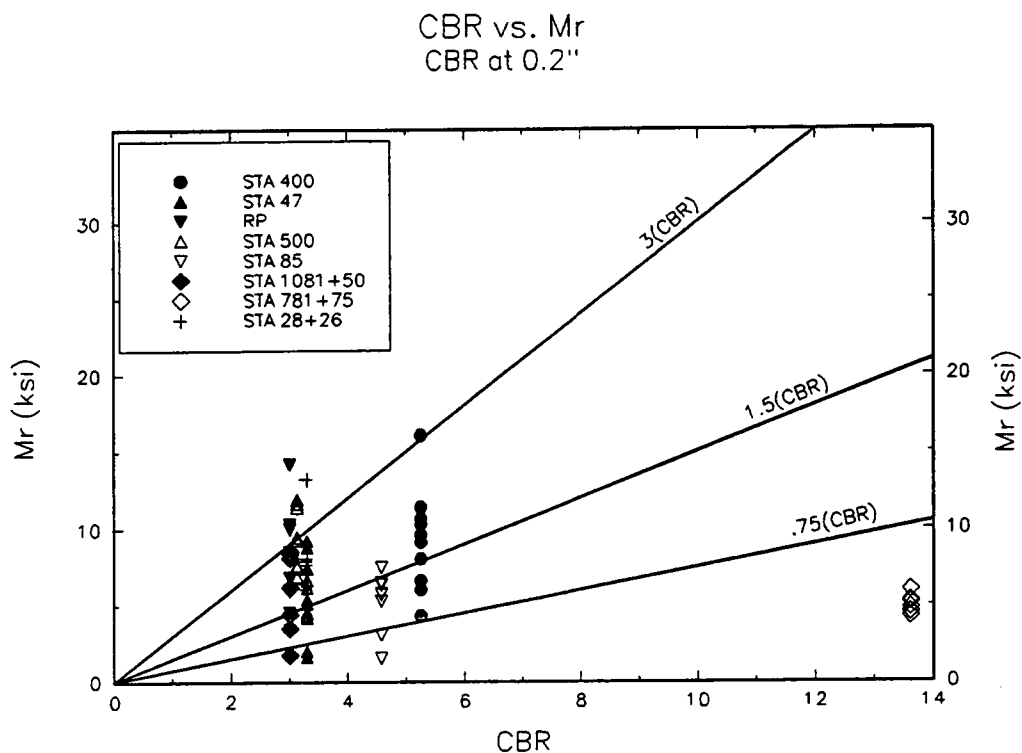


Figure 1-5. Relationship between M_r and CBR at 0.2", after Drumm et al. (1993a).

The first is to help in the identification of significant test parameters. This includes all soil properties (i.e. Atterberg limits, plasticity index, etc.) as well as loading conditions. The identification of the important parameters is essential to the determination of alternative estimating procedures such as the handbook method suggested by Drumm et al. (1993b). The second goal of the statistical model is the direct estimation of the resilient response. The general mathematical form of the model is as follows:

$$M_r = a(A) + b(B) + c(C) \dots\dots\dots (1-7)$$

where: a, b, c = model coefficients
 A, B, C = material and test parameters

The coefficients a , b and c are found using multi-variable regression procedures. From this equation, the resilient response can be calculated directly. Several statistical models have been presented in the past that predict the resilient modulus from soil and test parameters (Drumm et al. 1990; Drumm et al. 1992). Although less reliable than the before mentioned models where actual test data from a particular sample defines the model parameters, these statistical models offer insight as to the importance of various material parameters, while providing an easy method of estimating the resilient modulus with a minimum of laboratory testing.

1.5.3 Handbook Procedure

The handbook is a compilation of cyclic triaxial resilient modulus data and index properties which permits the estimation of the M_r of an unknown soil based on similar soils found in the handbook. Developed with the aid of a statistical model, this handbook procedure allows an estimation of the M_r response based on soils that similar classification, Atterberg limits and grain size distributions (Drumm et al. 1993b). After similar soils are identified, interpolation is done based on the density of the unknown soil versus the handbook soils and the water content of the unknown soil versus the handbook soils. The results of the interpolations are then averaged. The handbook serves as a mechanical database of resilient modulus data and additional handbook pages from this research can be found in Appendix A.

1.5.4 Alternative Test Method

As described by Li (1992), the alternative test method or ATM is based on a falling mass/spring system. The stiffness of the spring can be measured from the peak deceleration of the falling mass. This stiffness is related to the resilient modulus of a soil specimen. For the test device, a soil specimen is compacted using standard proctor procedures. The specimen is then placed onto a pedestal where a falling mass is dropped onto the specimen from a known height. An accelerometer is mounted on the falling mass and the decelerations can be measured with an oscilloscope. This method provides faster acquisition of M_r data than the traditional cyclic triaxial methods. This reduction in testing time makes laboratory measurements of M_r more feasible on a production basis. A more detailed discussion of the ATM setup and procedure will be presented in later sections with laboratory data that has been collected on several soils. Details in the theoretical development of the ATM can be found in Li (1992).

1.6 Scope

There were two primary tasks for this testing program which were an extension of the testing done by Drumm et al. (1993a). The first task was to make improvements to the existing device for the alternative test method (ATM). To do this it was necessary to increase the data acquisition accuracy, precision and speed by integrating an oscilloscope with a personal computer. Additionally, a standard release mechanism was incorporated onto a base plate so that consistent drops could be achieved. Comparisons with cyclic triaxial data are possible with these improvements.

The second task was to verify the prediction methods. This involved adding new soils to the existing data base of Tennessee subgrade soils. This addition will allow the comparison of similar soils already tested. A study on the effects of post-compaction water content changes on M_r was also conducted. These effects were studied for the cyclic triaxial method and the alternative test method (ATM).

Chapter 2 LABORATORY TESTING FOR RESILIENT MODULUS

2.1 Cyclic Triaxial Testing of Tennessee Subgrade Soils

The cyclic triaxial testing procedure was used for two types of specimens. The first type of specimens were compacted in the laboratory using kneading compaction and tested under as compacted conditions, while a second type was compacted identically to the first type and then the water content increased to some higher degree of saturation before testing. The cyclic triaxial tests using SHRP Protocol P46 testing procedure (SHRP 1989) is used as the benchmark for comparison since this method is proven and accepted as the standard method of determining M_r by most agencies. All data was presented in terms of the M_r vs. σ_d curve.

2.1.1 Previous Cyclic Triaxial Testing

A laboratory investigation of Tennessee fine-grained soils was conducted by the University of Tennessee between January 1991 and January 1993 (Drumm et al. 1993a, 1993b). During this period, eight soil samples were located and identified for M_r testing using the cyclic triaxial method. The eight samples were obtained from five locations across the state, including eastern, middle, and western Tennessee. In all, four different AASHTO soil classifications were used. These classifications are as follows: one A-7-5 soil, two A-4 soils, two A-6 soils, and three A-7-6 soils. From this cyclic triaxial testing, a statistical model, handbook method, and the foundation for the ATM testing was established (Drumm et al. 1993a, 1993b). In the previous study, soil specimens were

tested at five different target combinations of dry density and water content. Figure 2-1 shows the different test combinations with respect to the moisture-density curve, and follows a scheme similar to that used by Montalvo et al. (1984). Using these combinations, it was possible to study the affects of soil moisture and dry density at compaction on the resilient properties of the different soils. These different combinations are essential to pavement designers who have to allow for varying moisture conditions as well as different densities at the time of compaction. From this testing, a handbook was developed that showed M_r vs. σ_d curves for all of the soils tested at all five compaction conditions. This handbook also presented a method for estimating the resilient response of a soil directly from its index properties such as maximum dry unit weight, and optimum water content (Drumm et al. 1993b).

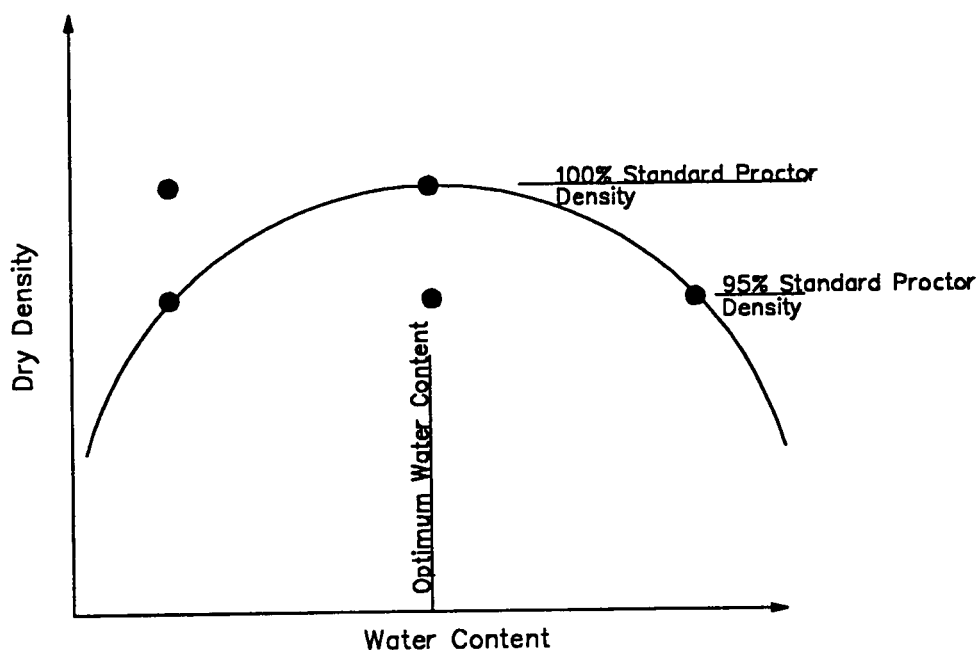


Figure 2-1. Targeting scheme for M_r samples after Montalvo et al 1984, and Hudson 1992

2.1.2 Considerations in M_r Testing

Due to the complexity of resilient modulus testing, there are several factors which may influence the resilient response of a cyclic triaxial test. Differences in test specimens, equipment settings, setup and calibration are all factors that can have a dramatic effect on the results of a resilient modulus test. Attention must be paid to all of these things to ensure that consistent results are achieved.

2.1.2.1 In-Situ vs. Recompacted Specimens

It is the goal of any laboratory to test specimens that most closely represent field or in-situ conditions. This will ensure that engineering behavior of these specimens can be relied on when used in design. Preferably, specimens that are extracted from shelly tubes pushed directly into the subgrade should be used to determine the resilient soil response for the design of the overlying AC and base layers. Sample disturbance associated with these in-situ specimens can effect the measurements. Specimens that have been extracted from shelly tubes are affected by such things as unloading of preexisting stresses, volume change due to sidewall friction, water content changes after acquisition of specimens, and handling and transport of specimens from the sampling site to the laboratory (Bowles, 1988). However, if care is given to sampling procedures and transport, this specimen disturbance is minimal and can be neglected.

Due to difficulties associated with sampling, remolded or laboratory compacted samples are usually used for M_r testing. Therefore, the soil specimens used for the cyclic triaxial testing were prepared using the kneading compaction method under

controlled conditions in the laboratory. This is consistent with the compaction method used by other researchers at the University of Tennessee (Hudson 1992, Boateng-Poku 1988, Johnson 1986). As noted by Seed et al. (1962) and Wahls and Langfelder (1967), kneading compaction produces a soil fabric similar to that seen in the field when rubber-tired rollers are used.

2.1.2.2 Drainage Conditions

SHRP Protocol P46 (SHRP 1989) specifies that all drainage lines leading into the specimen through the platens be open. This is often referred to as a drained condition. The rate at which each specimen tends to drain will largely depend on the composition of that particular soil. Due to the short duration of loading and the structure of the soils being tested (i.e. fine grained), only limited drainage can be expected, and the loading is actually undrained. It would therefore seem to make no difference if the lines leading to the specimen were closed or open. However, this may not be a good assumption because any leaks in the specimen membrane may affect the pore pressure in the specimen. Therefore close attention should therefore be paid to the drainage conditions, and membrane leaks should be avoided. This may require significant care when compacting specimens directly in a membrane.

2.1.2.3 Specimen Grouting

Contact between the platens and the specimen is not always good during the initial cycles of load due to the imperfections in the soil specimen ends. This leads to a non-

uniform distribution of the deviator stress on the specimen. To help remedy this problem, SHRP Protocol P46 suggests conditioning the specimen to improve the contact. Conditioning is accomplished by applying several hundred cycles of load prior to taking measurements. In addition to conditioning, grouting can be used to correct these problems. This is similar to capping concrete cylinders before compression tests. While not commonly used in M_r testing, it has been shown that specimen grouting will allow uniform axial strains and that sample conditioning is not as important (Hudson 1992, Pezo et al. 1992). This leads to resilient modulus tests that have proven to be much more reliable (Kim and Drabkin 1994). With the addition of grout caps on the ends of the specimen, the deformation of the grout cap must be considered to see if it is significant enough to effect the measured M_r values of a specimen. Resilient modulus tests have been conducted by Kim and Drabkin (1994), on specimens made of hydrostone grout to determine the resilient properties of this material. Specimen age was the main consideration in this testing. At an age of 270 minutes, the grout specimen had modulus values that were extremely large compared to the usual range of soil values, and by an age of 100 minutes, the effects of the grout cap are considered negligible.

2.2 ATM Testing of Tennessee Subgrade Soils

Like the cyclic triaxial method, the ATM device was used on two different types of specimens. The first type was compacted at optimum water content and maximum dry density and then tested. After testing, some of the specimens were saturated to a higher degree of saturation and tested again.

2.2.1 Previous ATM Testing

The previous ATM research conducted by Li (1992), concentrated on the development of the ATM itself. Only three soils were tested, each at a different water content. However, the results of this testing compared favorably with standard cyclic triaxial tests at the same compaction conditions. There were several limitations to the test device, such as difficulty with data acquisition, lack of a standard release mechanism and no fixable drop height. Additional refinements in the device and procedure were required before the method could be useful in practice.

2.2.2 Modifications to ATM Device

Since the previous testing was completed by Li (1992), some modifications have been made to the ATM Device. The modifications were made to the release mechanism, base plate and the data acquisition system. The previous research was conducted without a release mechanism, and the mass/accelerometer was dropped by hand. To ensure a consistent release, a "quick connect" pressure fitting was used to release the hammer rod. The male end of the "quick connect" fitting was fitted to the top of the hammer rod while the female end with the release collar was fitted to an adjustable arm on the base plate. The release mechanism is shown in Figure 2-2. The adjustable arm was added to the base plate so that the drop height could be adjusted and the same height of drop could be repeated. An oscilloscope is used to acquire the signal from the accelerometer. An Industrial Computer Source, digital oscilloscope card, model CS220B was added to a computer so that accurate measurements of the deceleration curves could be taken. The

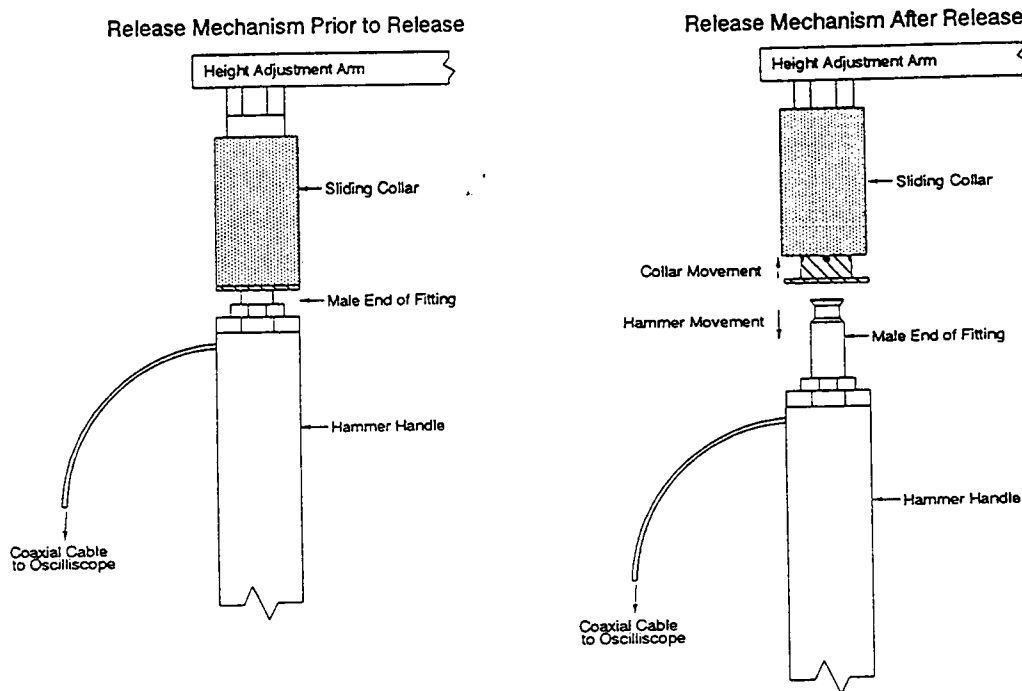


Figure 2-2. Release Mechanism for ATM Device.

addition of this oscilloscope card allows the deceleration data to be stored digitally on the computer for further use. These modifications make the results from the ATM device less user dependent. Figure 2-3 shows the ATM device including all of the modifications.

2.2.3 ATM Test Procedure

Until the time of this research, there was no standard protocol for using the ATM device. Therefore, the following procedure was developed for the ATM device:

1. Place specimen onto the base plate and attach the track to the proctor mold.
2. Make sure that the hammer is lined up correctly and is falling with no resistance.
3. Check to see where true zero falls on the height scale that is on the release mechanism.

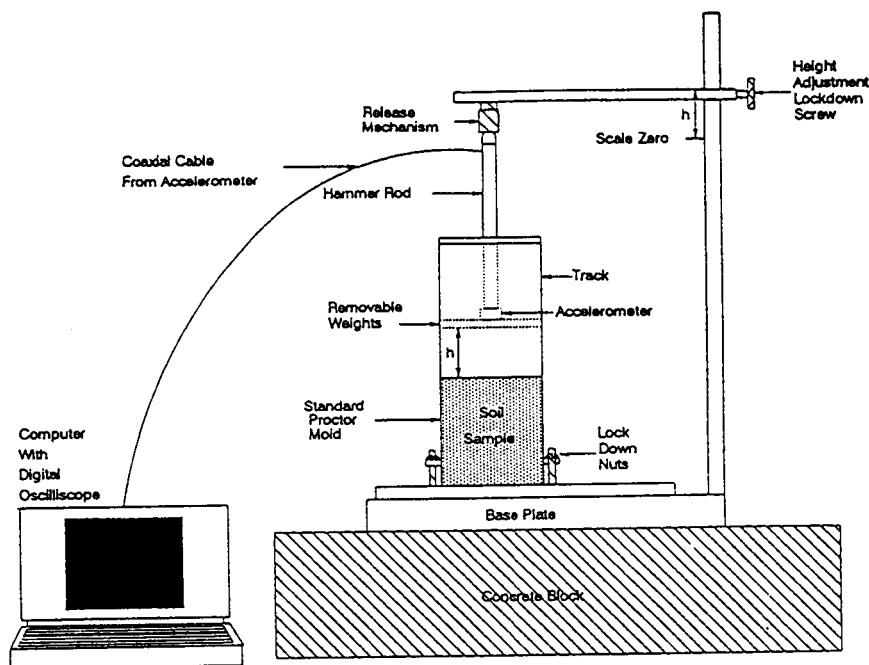


Figure 2-3. Equipment Setup for the ATM Device with Modifications.

4. Start at a drop height of 5 mm (0.2 in.) and make several drops to determine the average peak acceleration.
5. After several drops have been captured using the digital oscilloscope (4 minimum), break down the data to determine if the results are as expected.
6. Repeat step 5. for drop heights of 10 mm (0.4 in.), 15 mm (0.59 in.), 20 mm (0.79 in.), 30 mm (1.18 in.), 40 mm (1.57 in.), 50 mm (1.97 in.) and 60 mm (2.36 in.).

It is necessary to make several drops at every drop height to make certain that the hammer is striking the top of the specimen evenly and in a consistent manner. The higher drop heights were not obtained in all of the tests due to excessive noise in the signal. The noise can be attributed to stiff specimens that cause vibrations as the hammer strikes the top.

2.2.3.1 Calculation of M_r using the ATM

Calculation of the resilient modulus and deviator stress can be accomplished by use of two different methods. These methods are based on the peak acceleration, and on the area under the acceleration vs. time curve from time zero to the time corresponding to the peak acceleration as shown in Figure 2-4. Calculation of M_r from acceleration is based on the derivation by Li (1992) yielding the following relations:

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

and

$$\sigma_d = \frac{1-2\nu}{1-\nu} * \frac{A_p * M_o}{A_{\text{sec}}} \quad (2-2)$$

Where: M_r = Resilient modulus

ν = Poisson's ratio

$M = M_o + M_i$

M_o = Mass of the hammer

M_i = Mass of soil specimen

L = Length of specimen

A_p = Peak deceleration

$h =$ Equivalent drop height $= h_o(M_o/M)$

$h_o =$ Drop height of the hammer

g = Acceleration of gravity

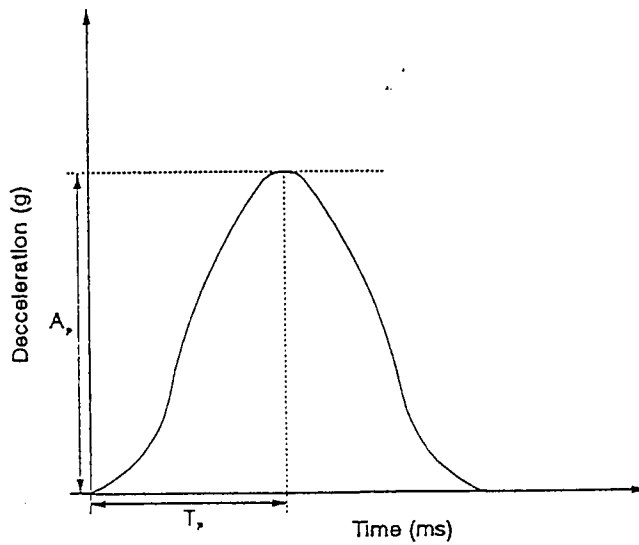


Figure 2-4. Deceleration vs. Time History for the ATM Device.

A_{sec} = Cross sectional area of the specimen

σ_d = Deviator stress

To correct for initial velocity, the resilient modulus and the deviator stress can be calculated based on the area under the deceleration curve from time zero to the time of peak deceleration. This is done by modifying the $2hg$ portion of equation 2-1. As noted by Li (1992), the initial velocity is defined as:

$$v_o = \sqrt{2hg} \quad (2-3)$$

This initial velocity is the area under the deceleration curve from time zero to T_p . Hence the resilient modulus can be calculated as:

$$M_r = \frac{(1+v) * (1-2\nu)}{(1-\nu)} * \frac{M * L}{Area^2 * A_{sec}} * A_p^2 \quad (2-4)$$

Where: $Area^2$ = The area under the deceleration curve squared.

Equations 2-1 and 2-4 then simplify respectively to:

$$M_r = \frac{(1+v) (1-2\nu)}{(1-\nu)} \frac{(M_o + 0.5M_1)^2 L}{Area^2 M_o g A_{sec}} A_p^2 \quad (2-5)$$

and

$$M_r = \frac{(1+v) (1-2\nu)}{(1-\nu)} \frac{(M_o + 0.5M_1)^2 L}{M_o 2h_o g A_{sec}} A_p^2 \quad (2-6)$$

The deviator stress is calculated in the same manner in both cases. This second method was used by Li (1992) except that the area under the deceleration curve was unknown and a sine function was used to approximate this area with a known A_p , and T_p that were read from a standard oscilloscope screen. With the integration of the digital oscilloscope card, the assumption that the deceleration vs. time history follows a sine wave is no longer needed since the data points along this curve are captured in digital format and stored on the computer hard disk. This increased accuracy will improve the estimation of the resilient modulus as calculated from ATM data. At the time of this research, the implementation of the area method was not completed and the software was being developed. A full analysis of this method can be found in the final report to TNDOT.

2.3 Samples

Six soil samples were identified for testing during this research period. These samples were identified at various sampling sites across the state of Tennessee with the help of the Tennessee Department of Transportation (TNDOT) personnel (Trolinger 1993). The goal of the sampling program was to identify soils that were representative of soils commonly used as subgrade material. The search was restricted to fine-grained subgrade soils, with less than 50% passing a #200 sieve.

2.3.1 Sample Locations

In order to get samples that were representative of subgrade soils in Tennessee, it was necessary to identify sampling sites across the state. On-going construction projects were selected for sampling so that the soils could be sampled directly from the compacted subgrade. The first site is a site that was used in the previous research project for TNDOT (Drumm et al. 1993, Hudson 1992). This site is in Crockett County, along SR 20. One sample was acquired from the centerline of the north bound lane at STA 925+20. This sample is referred to as STA 925. The second site was located in Hamilton County, along the new SR 111, that runs between Soddy Daisy and Dunlap. One sample (STA 578) was obtained from the centerline in a fill area at STA 578+50. The third site is in Giles County, along US 64 (SR 15) , which by-passes the city of Pulaski. One sample (STA 270) was obtained from the centerline of the entrance ramp from Highway 31 onto the new SR 15, at STA 270+00. The fourth site is located in Shelby County, along the Nonconnah Parkway (SR 57) in Memphis. One sample (STA

9) was taken at the centerline of the south bound lane at STA 9+00 in a fill. The fifth site is located both in White and Van Buren County, along the new US 70S (SR 1) between Quebec and McMinnville. Two samples were obtained from this site. The first sample (STA 652) was taken from White County in a borrow area at STA 652+00. The second (STA 618) was taken from Van Buren County in a cut slope of a borrow area at STA 618+00. These samples were taken from all across the state of Tennessee to ensure that a diverse group of soils were obtained. Figure 2-5 shows the various counties in which the samples were taken.

2.4 Index Testing and Moisture-Density Relationships

Identification of index properties from the six samples was necessary for proper classification of each soil, and to aid in the statistical analysis. All index tests were performed in the laboratory at the University of Tennessee in accordance with ASTM and AASHTO test specifications. These index tests include: Atterberg limits, specific gravity, and grain size analysis (sieve and hydrometer). The moisture-density relationships were developed using the standard energy proctor. Table 2-1 shows the various test specifications used for each individual test. Using these test procedures, the samples were classified based on both AASHTO and USCS (Unified soil classification system) systems (Holtz and Kovacs, 1981). In all, four different AASHTO classifications were obtained. These different classifications were chosen so that a wide range of properties were obtained and significant variations in resilient behavior could be shown. The range of optimum water contents and maximum dry unit weights is

Sampling Sites for TNDOT M Project

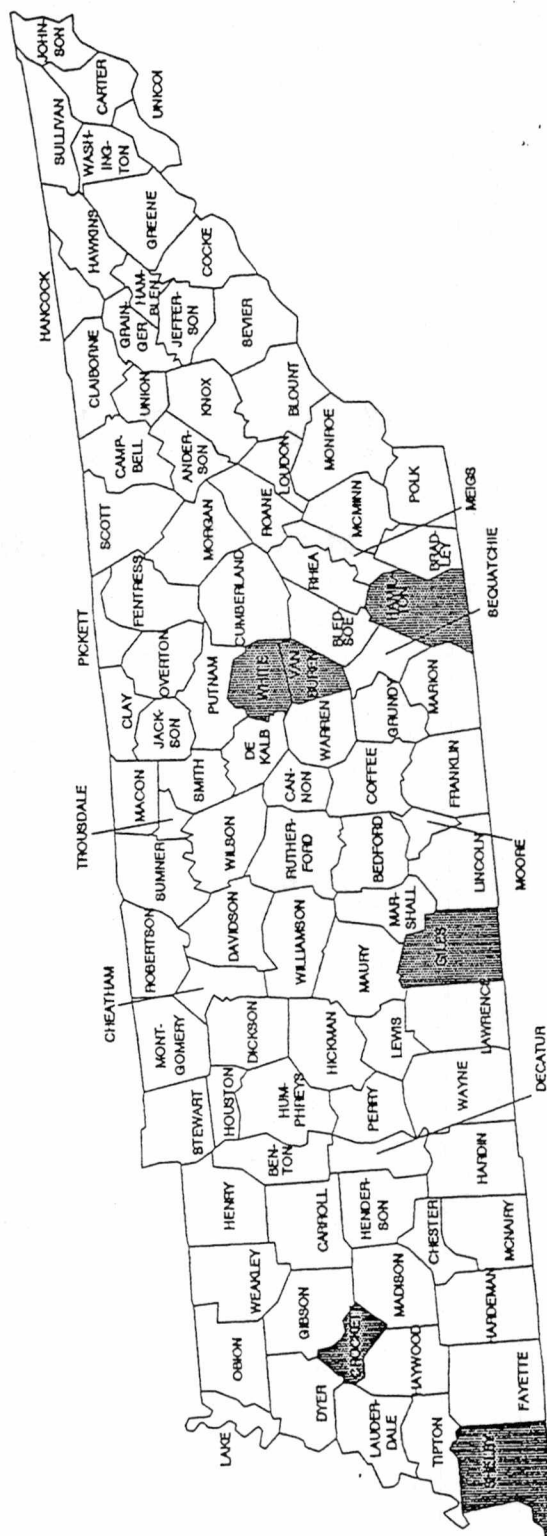


Figure 2-5. Counties Where Samples are Located

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

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

shown in the Moisture-Density relationships in Figure 2-6. Table 2-2 shows the index properties of the soil samples used for this research.

2.5 Relationship of Tested Soils to Previous Study

As with this study, the previous study obtained fine-grained soils from across the state of Tennessee. Figure 2-7 shows the various counties from which samples were taken during each research period. The combined testing program provides a good distribution across the state. The complete range of moisture-density relationships from both investigations are shown in Figure 2-8. All of the soils from this study fall within the range of soils from the previous study as shown. Index properties were determined in the same manner for both of the studies. Table 2-3 shows the index properties for all of the soils used in both studies.

STANDARD PROCTORS TNDOT M, PROJECT

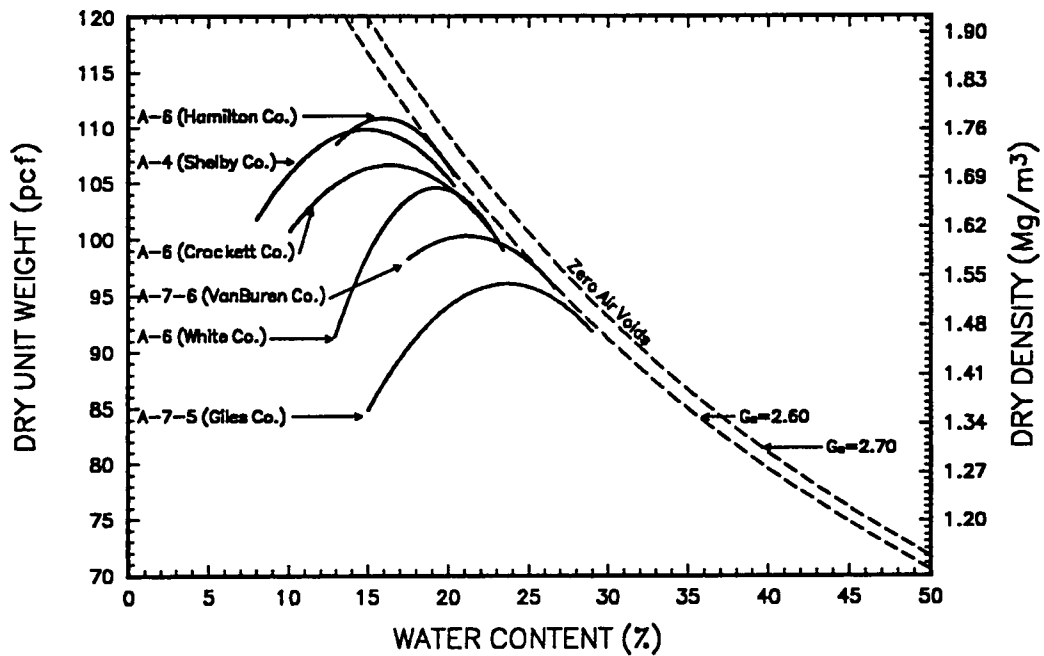


Figure 2-6. Moisture-Density Relationships of Samples at Standard Proctor Energy.

Table 2-2. Classification and Index Properties of Soil Samples

Location	Soil Classification			Atterberg Limits			Percent Clay (%)	Percent Passing #200 Sieve (%)	Specific Gravity	Optimum Water Content (%)	Max. Dry Unit Weight (pcf)
	AASHTO	Group Index	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	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

Sampling Sites for Phase I

Sampling Sites for Phase II

Map of Georgia counties showing sampling sites for Phase I and Phase II. The map includes county names and a legend.

Phase I Sampling Sites (Solid Black):

- Cherokee
- DeKalb
- Wilkes
- Wilkinson
- Colquhoun
- Shelby

Phase II Sampling Sites (Dotted):

- Union
- Greene
- Jefferson
- Knox
- Polk
- Madison
- Franklin
- Lincoln
- Wayne
- Lawrence
- Decatur
- Fayette
- Shelby

* Note- Crockett Co. was a Sampling Site in Phase I and II.

Figure 2-7. Location of All Soil Samples used for TNDOT Testing.

STANDARD PROCTORS TNDOT M, PROJECT

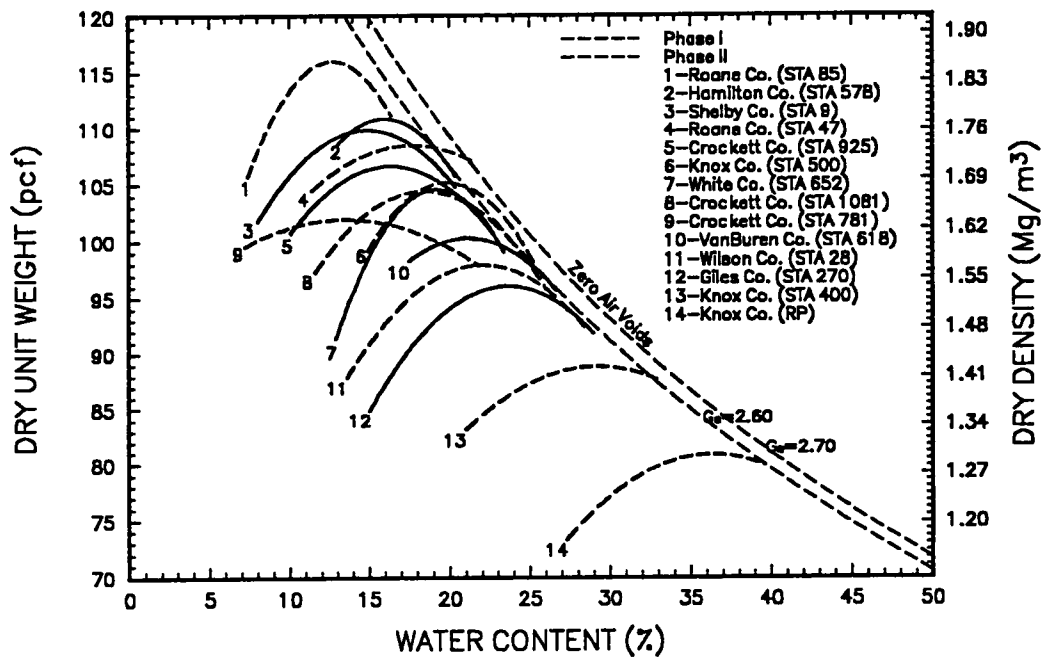


Figure 2-8. Moisture-Density Relationships at Standard Proctor Energy for all Soils used in TNDOT M, Testing.

Table 2-3. Classification and Index Properties of all Soil Samples Collected for TNDOT Research

Location	Soil Classification			Atterberg Limits			Percent Clay (%)	Percent Passing #200 Sieve (%)	Specific Gravity	Optimum Water Content (%)	Max. Dry Unit Weight (pcf)
	AASHTO	Group Index	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	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
Rutledge Pike	A-7.6	50	CH	71	28	43	70.3	97.8	2.60	36.2	82.5
Pellissippi Pky STA 400	A-7.5	36	MH	70	41	29	58.6	94.2	2.61	29.6	90.3
Pellissippi Pky STA 500	A-7.6	15	CL	45	22	23	39.9	70.4	2.70	19.2	106.3
Airport-STA 47	A-6	8	CL	34	21	13	25.4	72.3	2.65	18.2	109.5
Airport- STA 85	A-4	0	CL-ML	21	16	5	16.7	56.1	2.60	13.2	116.0
SR 20 STA 781 +75	A-4	1	ML	24	22	2	22.9	97.2	2.59	16.8	103.3
SR 20 STA 1081 +50	A-6	15	CL	36	22	14	25.4	98.9	2.62	18.4	105.9
I-840 STA 28 +26	A-7.6	36	CH	55	19	36	44.5	92.7	2.84	22.0	97.9

2.6 Laboratory Specimens used for M_r Testing

Laboratory specimens were used for both the cyclic triaxial and ATM testing. Specimens were tested at the maximum dry unit weight and optimum water content (standard proctor) for all of the soil samples. In addition to these specimens, some of the soil samples that were tested at moisture conditions that were different than the moisture conditions at compaction. In the previous research (Drumm et al. 1993), specimens were compacted at moisture conditions greater than, less than and at optimum water content as shown in Figure 2-1. This type of compaction scheme does allow the study of the effects of compaction water contents on the M_r - σ_d response, but it does not tell anything about the variation in the M_r - σ_d response due to post-compaction changes in water content. To investigate post-compaction water content effects on resilient modulus, three samples were tested at various moisture conditions after compaction. The results of the study reflect the seasonal variations in resilient modulus of a subgrade soil.

2.6.1 Cyclic Triaxial Specimen Preparation

Kneading compaction was used in the preparation of the cyclic triaxial specimens. The specimens tested at optimum water content and maximum dry density were compacted in six layers. To ensure that a good bond was obtained between layers, the specimen was scarified and two grooves arranged in a cross pattern were cut into the specimen after each lift. These grooves were approximately 4 mm (0.157 in.) deep and keyed the layers together. After compaction, the specimens were sealed in air-tight bags and stored in a moisture room for seven days to minimize thixotropy.

For the specimens that were saturated after compaction, the same kneading compaction technique was used for the initial compaction of the specimens at optimum water content and maximum dry density. Within 24 hours of compaction, these specimens were placed in a triaxial cell that is used as a flexible wall permeameter for saturation. The triaxial cell was set with a back pressure of 20.7 kPa (3 psi) so the water could fully penetrate the specimen. To eliminate disturbance during this procedure, the effective stress on the specimen never exceeded 41.4 kPa (6 psi) or the maximum effective confining stress exerted on the specimen during cyclic triaxial testing. After the introduction of additional water, the specimen was sealed in an air-tight bag and allowed to hydrate for at least 7 days. This hydration period was to ensure that the specimens achieved a uniform water content across the specimen.

2.6.2 ATM Specimen Preparation

The specimens that were used for the ATM testing were compacted using the AASHTO T-99 standard proctor method with some modifications. To reduce friction between the compaction mold and the soil specimen, vaseline was used to coat the inside of the mold prior to compaction. Another modification was to place a 3.18 mm (1/8 inch) thick disk with the same diameter as the compaction mold into the mold before compacting the specimen. The purpose of this disk was to make the mold extend beyond the end of the specimen during the ATM testing. This is to ensure that the rigid compaction mold does not contact the base plate of the ATM device. The compaction procedures discussed here are consistent with those used by Li (1992) in the previous

testing of the ATM device. These modifications to the compaction procedure do not significantly change the results of the test, but they reduce the noise in the signal recorded by the oscilloscope as noted by Li (1992).

Saturation of ATM specimens was accomplished with the use of a permeameter that mounts directly on the proctor mold. As with the cyclic triaxial specimens, these ATM specimens were compacted at optimum water content and maximum dry density before saturation. However, for the ATM tests the same specimen was used for several different levels of saturation. After the initial compaction, the specimen was tested as compacted. When this testing was completed, the specimen was weighed and then placed in the saturation device. After some predetermined amount of water was allowed to flow into the specimen, the specimen was taken out of the saturation device and weighed to confirm the amount of water added. The specimen was then allowed to hydrate for two days before testing with the ATM. This procedure was then repeated until several different degrees of saturation were reached.

Chapter 3 TEST RESULTS

3.1 Cyclic Triaxial Test Results

Cyclic triaxial testing was conducted in accordance with SHRP Protocol P46 (SHRP 1989). The testing apparatus used for cyclic triaxial testing is the same as used by other researchers at the University of Tennessee (Boateng-Poku 1988, Johnson 1986, and Hudson 1992). The only changes from Johnson (1986) and Boateng-Poku (1988) are the addition of an internal load cell and LVDT's mounted on the top platen. Great effort was made to keep all testing procedures as consistent with previous researchers as possible so that data can be quantitatively compared.

3.1.1 Testing at Optimum Water Content and Maximum Dry Density

All six of the soil samples were tested at optimum water content and maximum dry unit weight (standard proctor energy). Tolerances of ± 0.0320 Mg/cm³ (2 lb/ft³) of maximum dry density and $\pm 1\%$ of optimum water content were used for the preparation of the laboratory specimens. Maximum dry unit weight and optimum water content for standard proctor energy were targeted since construction practice often requires this same compaction energy and water content. Typical results from this testing are shown in figure 3-1. These results show that the resilient modulus increases greatly as the deviator stress approaches zero, reaches a minimum point at low values of deviator stress and changes more slowly as the deviator stress increases up to 69 kPa (10 psi). This behavior is typical of virtually all fine grained soils, as reported by various other

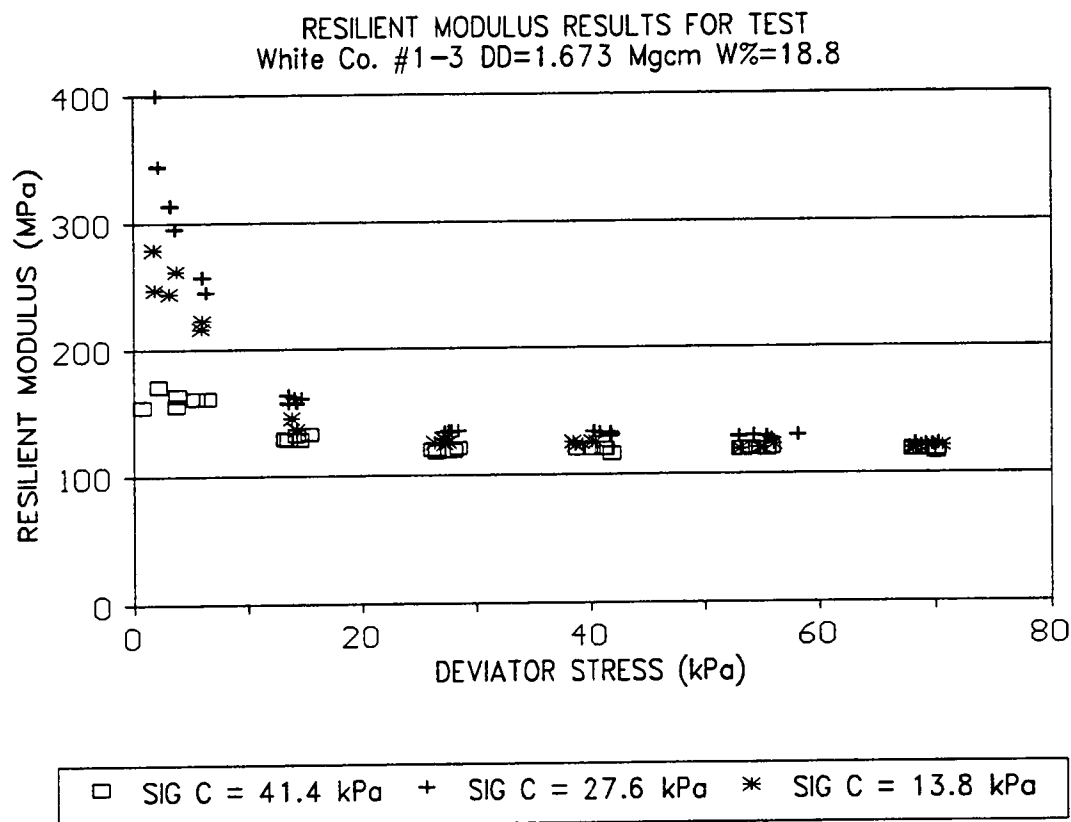


Figure 3-1. Typical M_r vs. σ_d Response for Optimum Water Content and Maximum Dry Density.

researchers (Thompson and Robnett 1976, Seed et al. 1962, Hudson 1992). The range of resilient responses shown by the six soils covered a wide range as expected for the range of soil properties. Figure 3-2 shows the highest resilient modulus values identified during testing, with a minimum value of M_r approximately 170 MPa (24.6 ksi) at a deviator stress of 69 MPa (10 psi). The lowest resilient modulus values were approximately 90 MPa (13.0 ksi) at a deviator stress of 69 MPa (10 psi) as shown in Figure 3-3. The difference between the two samples can be attributed to the difference in percent passing a #200 sieve. The sample with the lower response is classified as an A-4 (CL) and had 98.6% passing a #200 sieve while, the sample with the higher

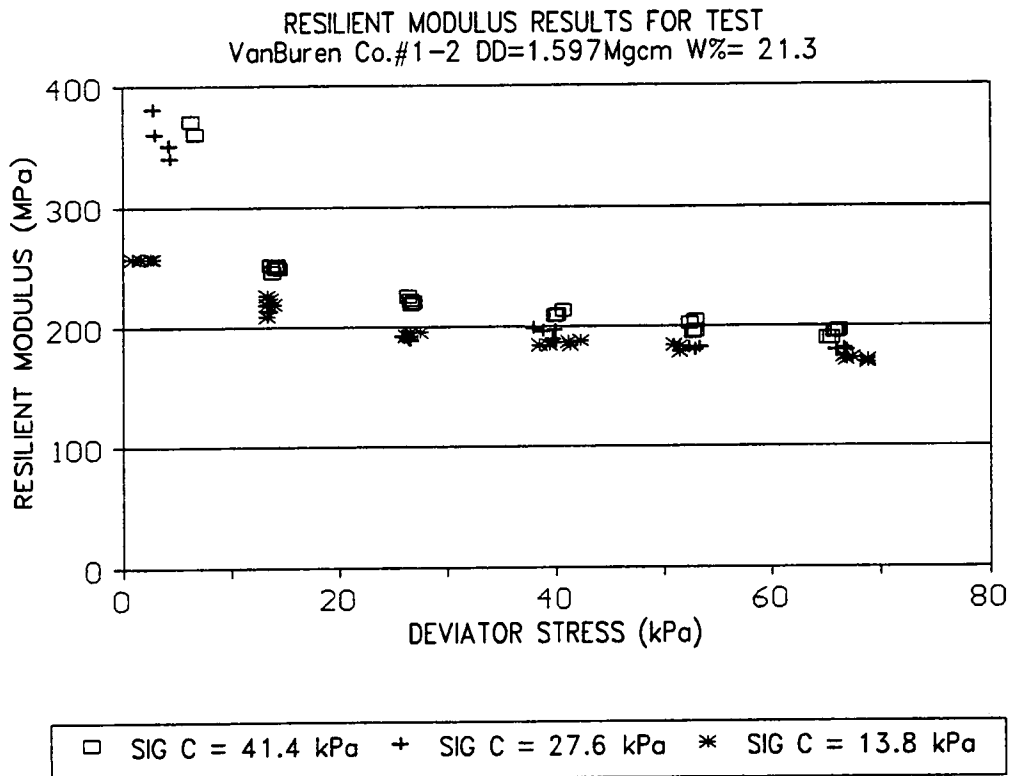


Figure 3-2. Highest Values of Resilient Modulus for the Six Soils Tested

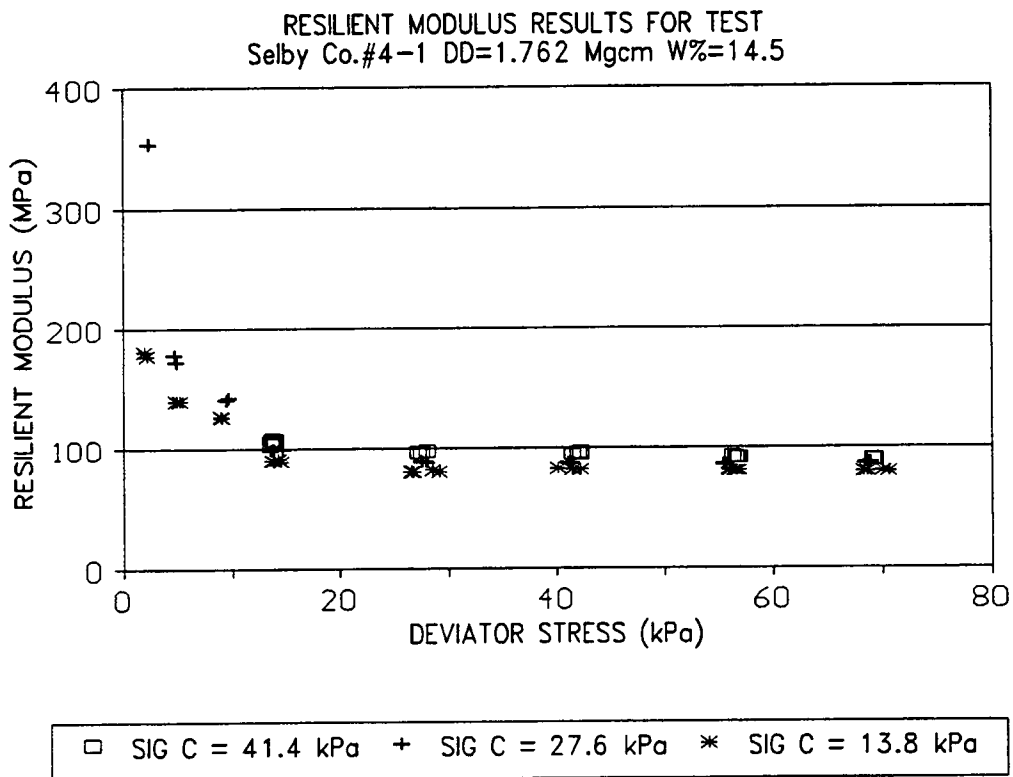


Figure 3-3. Lowest Values of Resilient Modulus for the Six Soils Tested

response classified as an A-7-6 (CH) and had only 58.8% passing a #200 sieve. This comparison is for the soils tested at optimum water content and maximum dry density. A wider range could be expected if the soils were each tested at various water content and dry density combinations as shown by Hudson (1992).

A comparison of M_r data obtained during this research was made with other researchers using similar soils. Table 3-1 shows the results of this comparison grouped by AASHTO soil classification. M_r values at a deviator stress of 41.4 kPa (6 psi) and cell pressures ranging from 34.5 kPa (5 psi) to 0 kPa (0 psi) are shown. Although there is a range of reported results, in general, the M_r values compared well with the values obtained by other researchers. Fredlund et al. (1977) reported resilient modulus values that were low as compared to all others for an A-6 soil. However, Lee et al. (1994)

Table 3-1. Comparison of Results Obtained by Other Researchers for Soils Tested at Optimum Water Content and Maximum Dry Density.

Reference	AASHTO Class.	LL	PI	Opt. W%	Max. γ_d (pcf)	σ_d kPa	σ_c kPa	M_r MPa
Lee et al. (1994)	A-6	28-38	11-19	14.8	114.2	41.4	21	84
Lee et al (1994)	A-4/A-6	19-26	7-14	9.6	130.0	41.4	21	155
Fredlund et al. (1977)	A-6	34	17	16.5	110.3	41.4	21	14
TNDOT STA 578	A-6	34	12	16.0	110.8	41.4	27.6	160
TNDOT STA 925	A-6	33	11	16.5	106.6	41.4	27.6	78
TNDOT STA 652	A-6	39	16	19.3	104.6	41.4	27.6	130
Mohammad et al. (1994)	A-7	-	22	21.2	101.8	42	21	172
Wilson et al (1990)	A-7-6	-	-	11.5	124.3	41.4	34.5	35
TNDOT STA 618	A-7-6	54	28	21.2	100.2	41.4	27.6	190
TNDOT STA 270	A-7-5	52	22	23.7	96.1	41.4	27.6	130
Drumm et al. (1990)	A-4	31	9	18.0	103.5	41.4	0	97
Elliott (1992)	A-4	-	-	-	-	41.4	0	21-35
TNDOT STA 9	A-4	30	11	15.0	109.8	41.4	0	64

described two different A-6 soils that compare well with the A-6 soils tested for TNDOT. The A-7 soil reported by Mohammad et al. (1994) compares very well with the results obtained from the VanBuren County soil (A-7-6) and is somewhat close to the results of the Giles County soil (A-7-5). This difference may be due to a lower maximum dry density and higher optimum moisture content of the Giles County soil. The Shelby County (STA 9) A-4 soil corresponds well with the high range of resilient modulus values obtained by Elliott (1992). The values of the A-4 soil tested by Drumm et al. (1990) are higher than the values obtained for the similar A-4 soil (STA 9), but the difference in these results are thought to be due to improvements in the test device.

This comparison demonstrates the difficulty in detecting trends in reported M_r values from different experimenters. Some soils have M_r values very similar to those reported by others, but no obvious trend can be found when comparing the soil index properties. This suggests that while repeatability of M_r in a particular laboratory is not difficult, repeating this same value of M_r on the same soil in other laboratories is difficult. This conclusion was reached by Lenker (1992) during an interlaboratory comparison of M_r results.

3.1.2 Cyclic Triaxial Testing of Samples Saturated After Compaction

Six specimens that were compacted at optimum water content and maximum dry density were later saturated. This saturation was to simulate the wetting of a subgrade soil as the groundwater table rises during wet seasons, or as rainwater infiltrates into the subgrade. In order to compare the effects of increased saturation on different specimens,

three specimens were compacted at the same initial conditions for each soil. Each specimen came from the same batch of soil to ensure that the water content at compaction was the same in each specimen, with the exception of one of the Shelby County specimens that had to be repeated. The measured water content for this specimen (#4-1) was 0.3% less than the initial water content of the other specimens.

To investigate the effects of post-compaction water content changes, one of these samples was tested as compacted (at optimum water content, and maximum dry density), a second specimen was saturated to some intermediate level of saturation then tested, and the third was saturated to a high level of saturation before testing. Significant softening of specimens was reported by others as the degree of saturation increased (Figuerola and Thompson, 1980, Elliott and Thornton, 1988, Jones and Witczak, 1977). This softening corresponds to the behavior characterized by Monismith et al. (1967) where increased resilience was noted as the water content of a soil increased after placement. Increased resilience is taken to mean increased resilient deflection, or resilient strain (lower values of M_r) in this context.

Specimens that were saturated after compaction for this research showed that as the degree of saturation increased, the resilient modulus decreased. This decrease in modulus, or softening, can be shown in Figure 3-4 for the Giles County samples that were compacted at optimum water content, maximum dry density and saturated to varying degrees of saturation. In this figure, a smooth curve has been fit through all data and the data points have been removed for clarity. Similar results were found for the Crockett County, and Shelby County soils shown in Figures 3-5 and 3-6 respectively. To

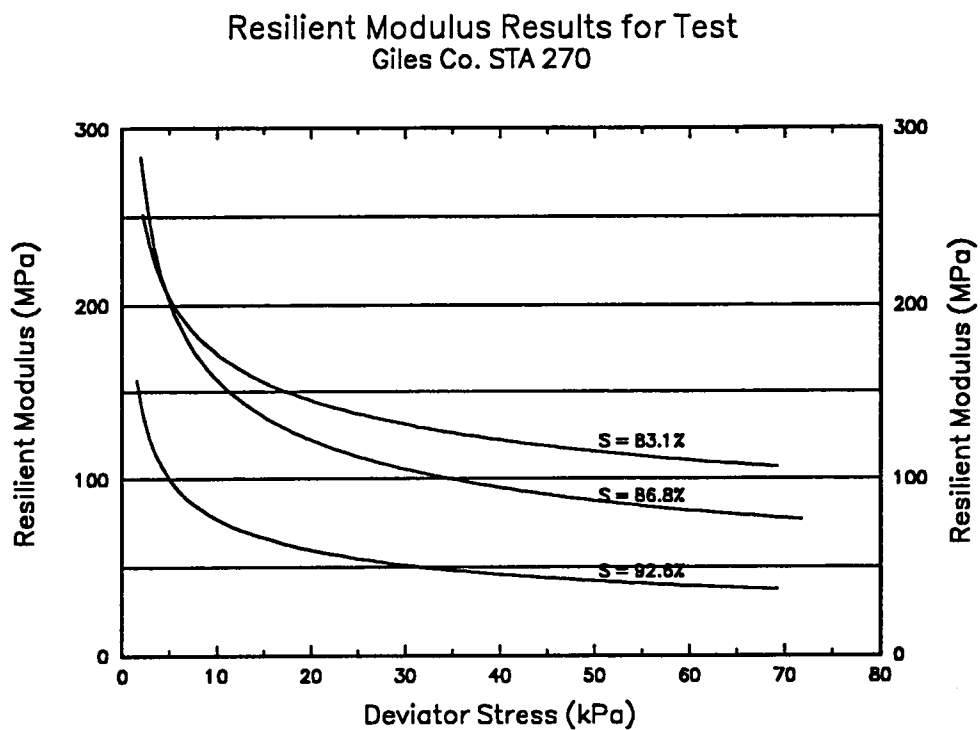


Figure 3-4. Resilient Response of Giles County Specimens Saturated after Compaction.

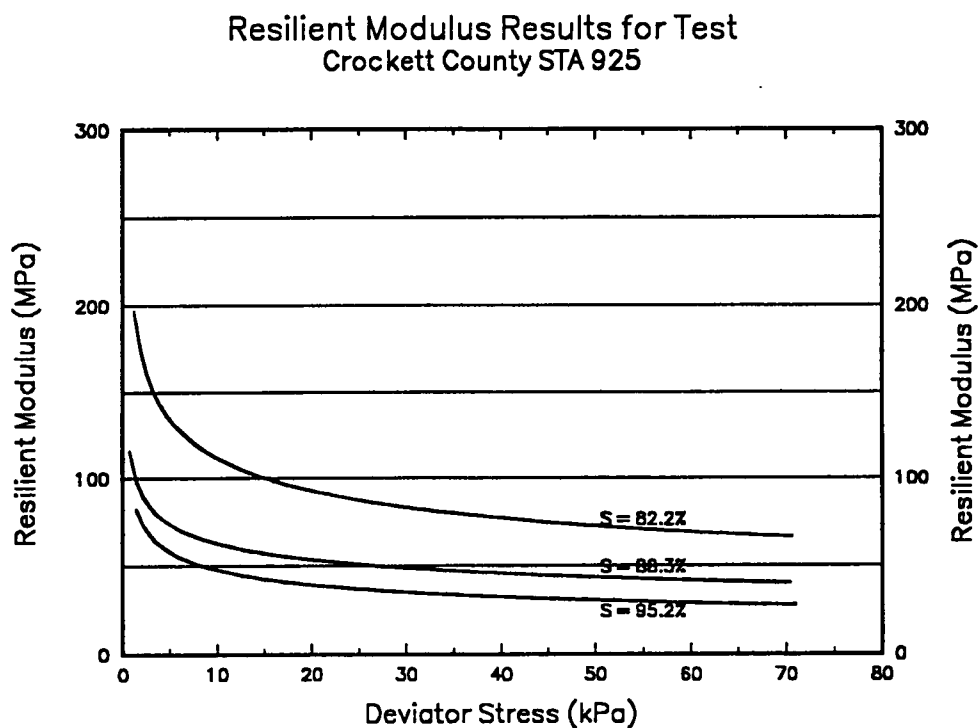


Figure 3-5. Resilient Response of Crockett County Soil Saturated after Compaction.

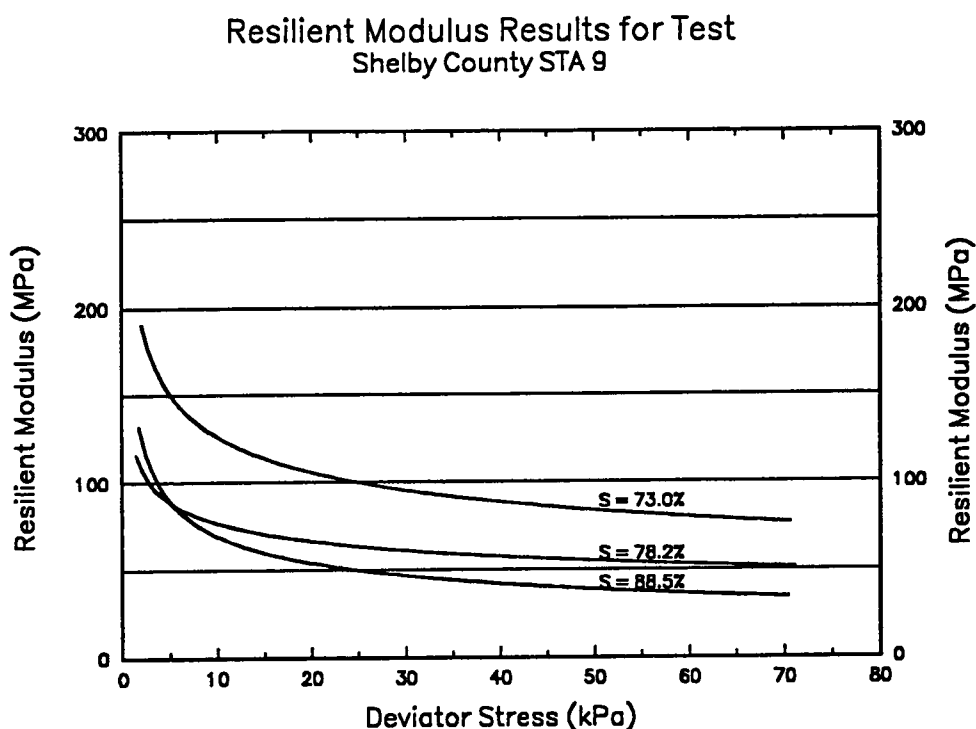


Figure 3-6. Resilient Response of Shelby County Soil Saturated after Compaction

describe how the resilient modulus varies for the samples after saturation, a plot of M_r versus degree of saturation for a deviator stress of 27.6 kPa (4 psi) and a cell pressure of 41.4 kPa (6 psi) shown in Figure 3-7. Curves of this type could be useful to pavement designers to adjust M_r for seasonal variations in water content. Appendix A shows the resilient response of all of the specimens tested using the cyclic triaxial method, and displays the individual data points.

3.1.3 Effects of Confining Pressure

Confining pressure in a subgrade varies with depth, soil type, and wheel loading. Therefore, one criterion for the selection of a resilient modulus value to be used in design should be the confining pressure. The level of importance placed on this selection

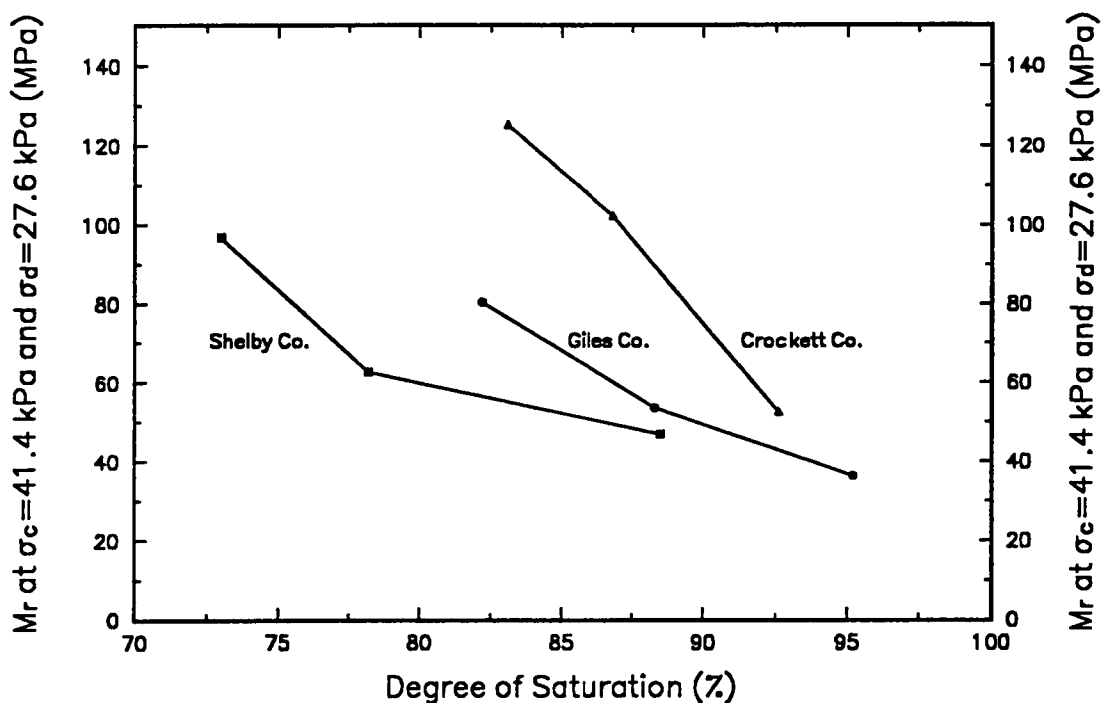


Figure 3-7. Effects of Increased Degree of Saturation at $\sigma_d = 27.6$ kPa (4 psi)

should correspond to the sensitivity of the soil to confining stress. Thornton and Elliott (1986) noted that for the cohesive soils that they had tested, an increase in confining pressure would result in a large increase in resilient modulus at a given stress level. Others have noted that there was relatively no effect of confining pressure on the resilient modulus for the soils that they had tested (Fredlund et al. 1977, Hudson 1992). Thompson and Robnett (1976) suggested that there was little effect of confining stress on the resilient modulus for confining stresses between 20.7 kPa (3 psi) and 34.5 kPa (5 psi) for cohesive soils compacted wet of optimum. During this research program, in addition to the resilient modulus values that were collected for the normal SHRP P46 confining stresses of 41.4 kPa (6 psi), 27.6 kPa (4 psi), and 13.8 kPa (2 psi), data was collected at all levels of deviator stress with no confinement. From this, the effects of

confining stress from 0 kPa (0 psi) up to 41.4 kPa (6 psi) can be studied at 13.8 kPa (2 psi) increments. All tests showed that at any given deviator stress, there was only slight changes in the resilient modulus with confining stresses between 41.4 kPa (6 psi) and 13.8 kPa (2 psi), but the modulus dropped considerably between 13.8 kPa (2 psi) and 0 kPa (0 psi). This nonlinear dependence on confining stress is shown for typical results in Figure 3-8. The selection of a M_r value to use in design for the three levels of confinement greater than zero would be essentially the same value. However, if the confining stress is equal to zero, the M_r values drop considerably. Therefore it should be noted that for the soils tested here, care should be taken when selecting design values of M_r at confining stresses below 13.8 kPa (2 psi).

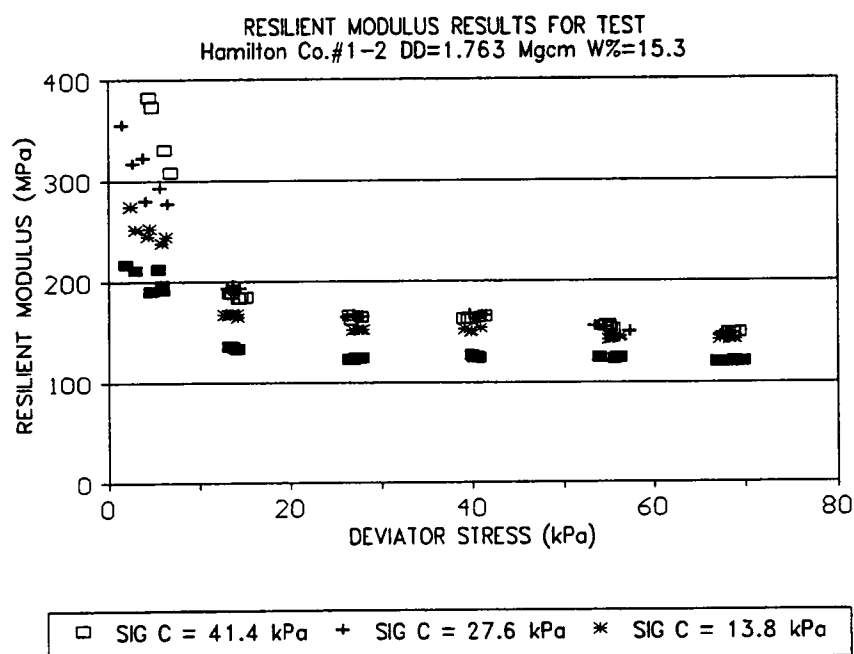


Figure 3-8. Effects of Confining Stress

3.2 ATM Test Results

Since the cyclic triaxial method is time consuming, difficult to reproduce and is not always economically feasible, testing was conducted with the ATM device to evaluate the viability of the method. To compare the ATM and the cyclic triaxial methods, the same soils were used in the ATM testing that were used for the cyclic triaxial testing. From this testing the performance of the ATM can be evaluated.

3.2.1 ATM Results on Specimens Compacted at Optimum W% and Maximum γ_d

Specimens were compacted at optimum water content and maximum dry density, in standard proctor compaction molds, for all six of the soils that were tested using the cyclic triaxial method. Then the ATM was used to determine the resilient modulus. Figure 3-9 shows typical results from the ATM testing. The steady increase in resilient modulus as the deviator stress increases is consistent with the ATM results shown by Li (1992), and is typical of constrained behavior. During a constrained test, the confining stress increases as the vertical stress increases because no horizontal strains are allowed. This results in an axial stress versus axial strain curve which curves upward (Holtz and Kovacs, 1981; Lambe and Whitman, 1969). Since the slope of the stress versus strain line is increasing as the strain increases, the secant modulus is increasing. In a typical triaxial test, as the level of axial stress increases, the axial strain increases, resulting in an axial stress-axial strain curve that curves downward. The decreasing slope of the axial stress-axial strain curve with increasing stress indicates that the secant modulus is decreasing with increasing stress. Figure 3-10 compares the behavior for a

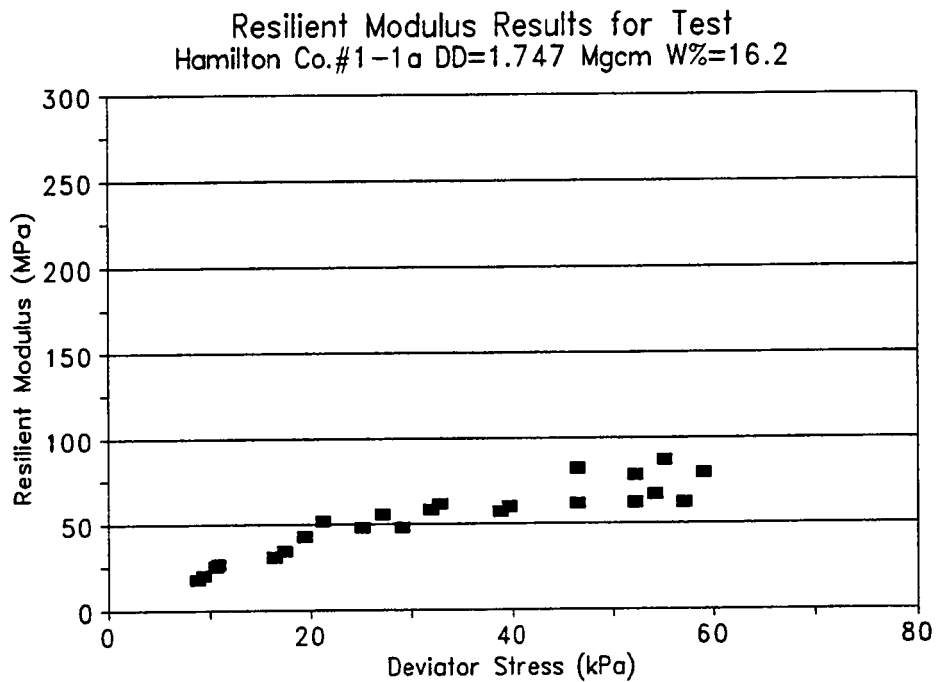


Figure 3-9. Typical ATM Results for a Specimen at Optimum Water Content and Maximum Dry Density

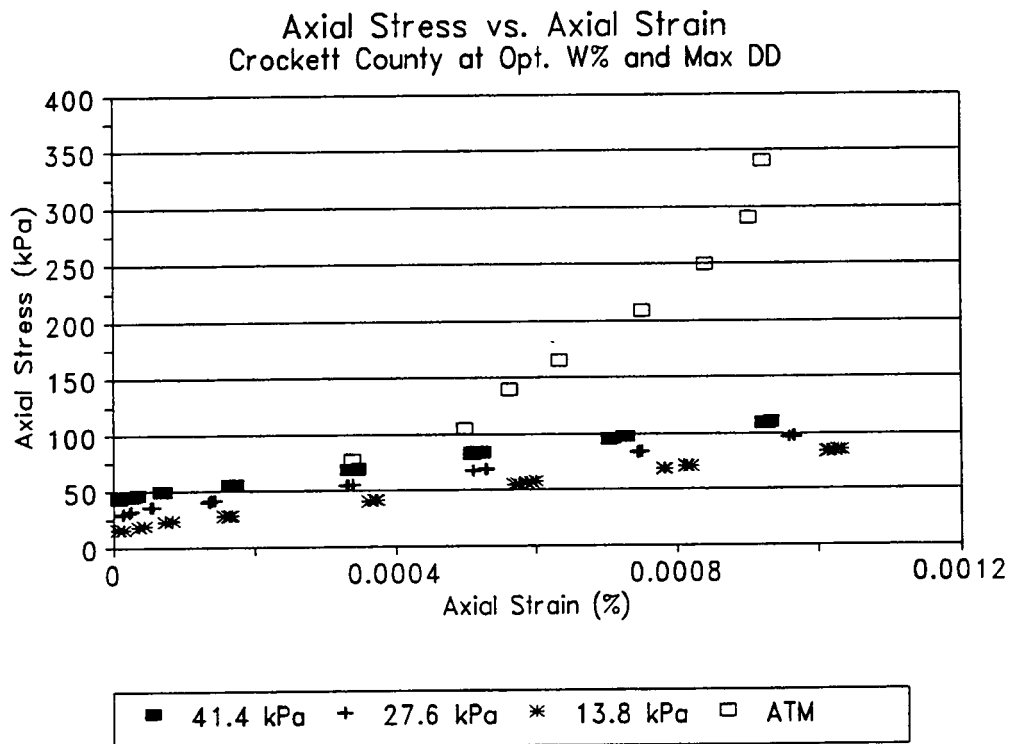


Figure 3-10. Stress Versus Strain Response for Cyclic Triaxial and ATM Tests.

confined compression test (ATM) and a uniaxial compression test (cyclic triaxial). Some of the results showed axial stress versus axial strain curves that had some initial upward curvature but all curves seem to reach a constant slope at higher values of strain. When the slope is constant this means that the modulus is not changing. This results in resilient modulus values that increase at lower deviator stresses and become constant as the deviator stress increases. The constant resilient modulus with increasing deviator stress demonstrates that confining stress becomes less important as the deviator stress increases, since the confining stress increases as the deviator stress increases in a constrained test. Stress versus strain curves and data for all of the tests can be found in Appendix B.

For all of the resilient modulus curves reported here, an assumed value of Poisson's ratio of 0.45 has been used. This value is within the range for a fine grained soils (Huang 1993). If a smaller value of Poisson's ratio is assumed, then the curves shift upward and to the right. This behavior is expected from the equations, Chapter 2, which define the resilient modulus. In general, the ATM values are lower than the values obtained by the cyclic triaxial method, Figure 3-11. The resilient modulus values from the ATM are similar to the cyclic triaxial results at deviator stresses greater than 55.2 kPa (8 psi). However, at low deviator stresses the ATM under predicts the resilient response.

The ATM testing also demonstrated that the device was capable of measuring the difference between soft and stiff soils. The response still shows the increasing resilient modulus with increasing deviator stress, but the rate of increase in M_r changes between soft and stiff specimens as shown in Figure 3-12.

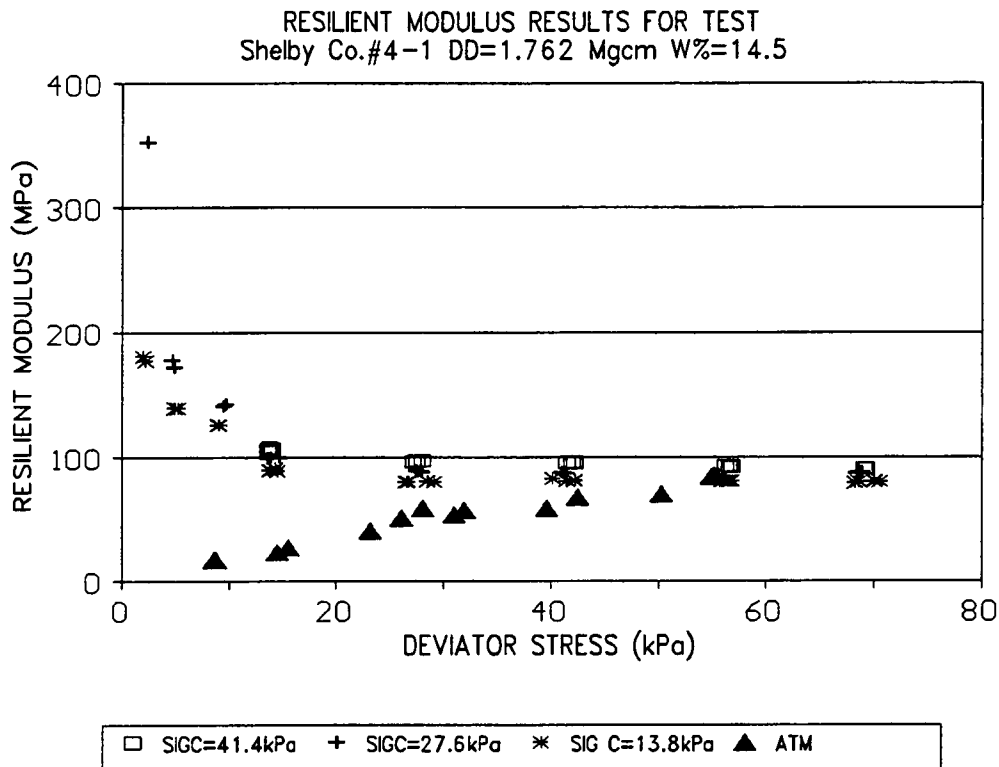


Figure 3-11. Comparison of Cyclic Triaxial and ATM Results

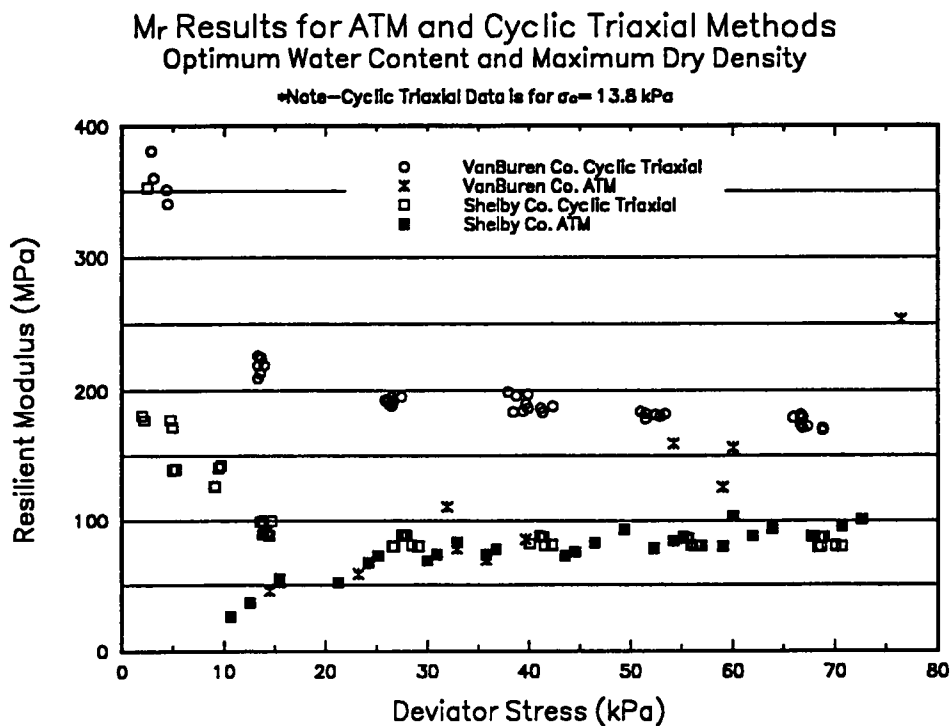


Figure 3-12. ATM Results for Soft and Stiff Specimens

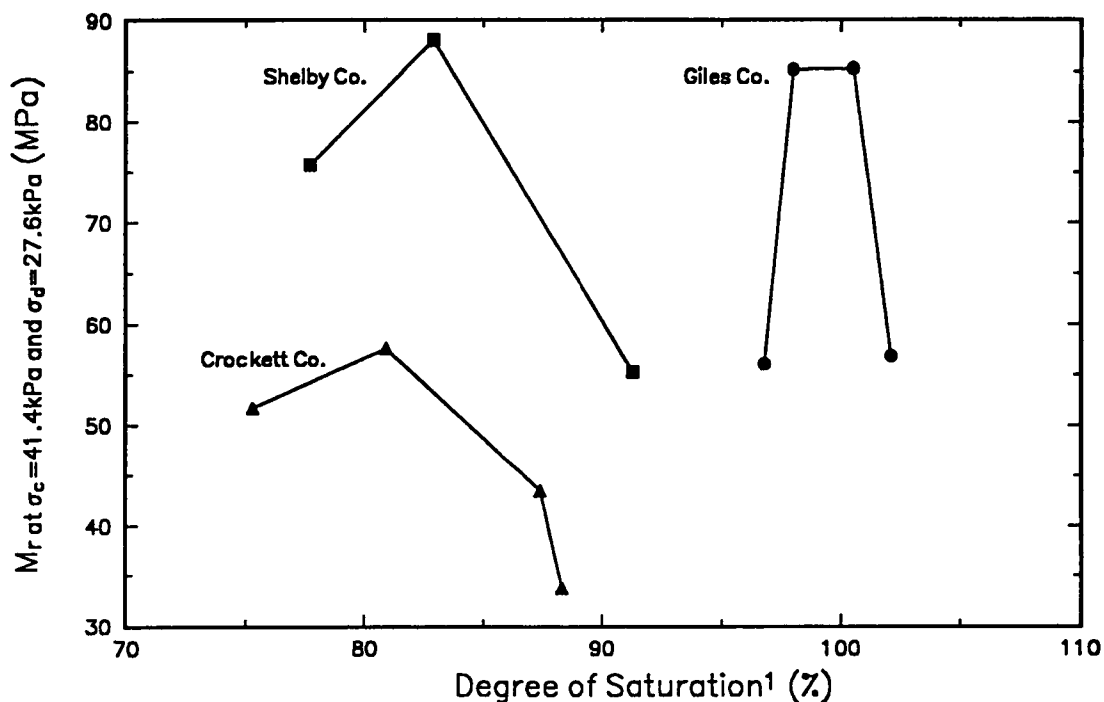
Uniaxial compression tests, such as the triaxial compression test, allow radial strains in the specimen and the horizontal stress remains constant at the cell pressure. As suggested by Li (1992), the confinement in a subgrade is somewhere between the confined and uniaxial confinement cases. This state is referred to as a flexible confinement case. These differences in the types of confinement that are present in each of these tests demonstrate some limitations of these tests for simulating field conditions. However, the ATM test setup is believed to be a better representation of actual confinement in a subgrade (Li 1992).

3.2.2 ATM Results on Specimens Saturated After Compaction

A study was conducted to see if the ATM device could pick up the decrease in resilient modulus due to an increase in the degree of saturation as found in the cyclic triaxial tests. For this study the same three soils that were used in the cyclic triaxial post compaction saturation scheme were used for the ATM device, Crockett, Giles and Shelby Counties. The test procedure was the same as the procedure described for the optimum water content and maximum dry density except that the same specimen was used at each degree of saturation. After the initial compaction at optimum water content, the specimen was placed in a device specially made for saturating specimens in a proctor mold. The specimen was then saturated to some higher degree of saturation. The increase in the degree of saturation was controlled so that three different degrees of saturation (including the initial degree of saturation) could be studied with identical specimens. After each saturation event, or phase, the specimen was taken out of the saturating device and

placed in an air-tight bag and allowed to hydrate for a minimum of four days before testing again.

The results from the specimens that were saturated after compaction showed that as the degree of saturation was increased greatly, the ATM device could pick up a decrease in resilient modulus. At lower degrees of saturation however, the ATM device showed some increase in M_r . Figure 3-13 shows the effects of the increased degree of saturation on the resilient response. The reason for this increase is believed to be an increase in horizontal stresses, and corresponding increased friction along inner walls of the proctor mold. Unlike a cyclic triaxial specimen which is free to swell, an ATM specimen is compacted inside a rigid mold and cannot swell radially, and has some



¹Based on Reported G_s Values in Table 2-2

Figure 3-13. Effects of Increased Degree of Saturation on the Resilient Response

resistance to swelling in the transverse direction due to skin friction. After saturating to high degrees of saturation however, the extra amount of swell that the specimen will develop is minimal and the softening of the specimen can be picked up by the ATM. This behavior was consistent with all three soil samples being used for this study.

3.3 Comparison of Cyclic Triaxial and ATM Test Methods

One objective of this testing was to see if the ATM device could demonstrate the ability to distinguish soft and stiff response. All of the specimens compacted at optimum water content have medium to very stiff responses. The saturated specimens then allow us to evaluate the performance of the ATM with less stiff, or "soft" material. To evaluate this performance, the resilient response was plotted at optimum water content conditions and at high saturation conditions for the same specimen in Figure 3-14. The cyclic triaxial data is also plotted for the same specimen conditions to show how the increase in saturation effects the response for this test method. This plot clearly demonstrates the ability of the ATM device to detect differences in soil stiffness.

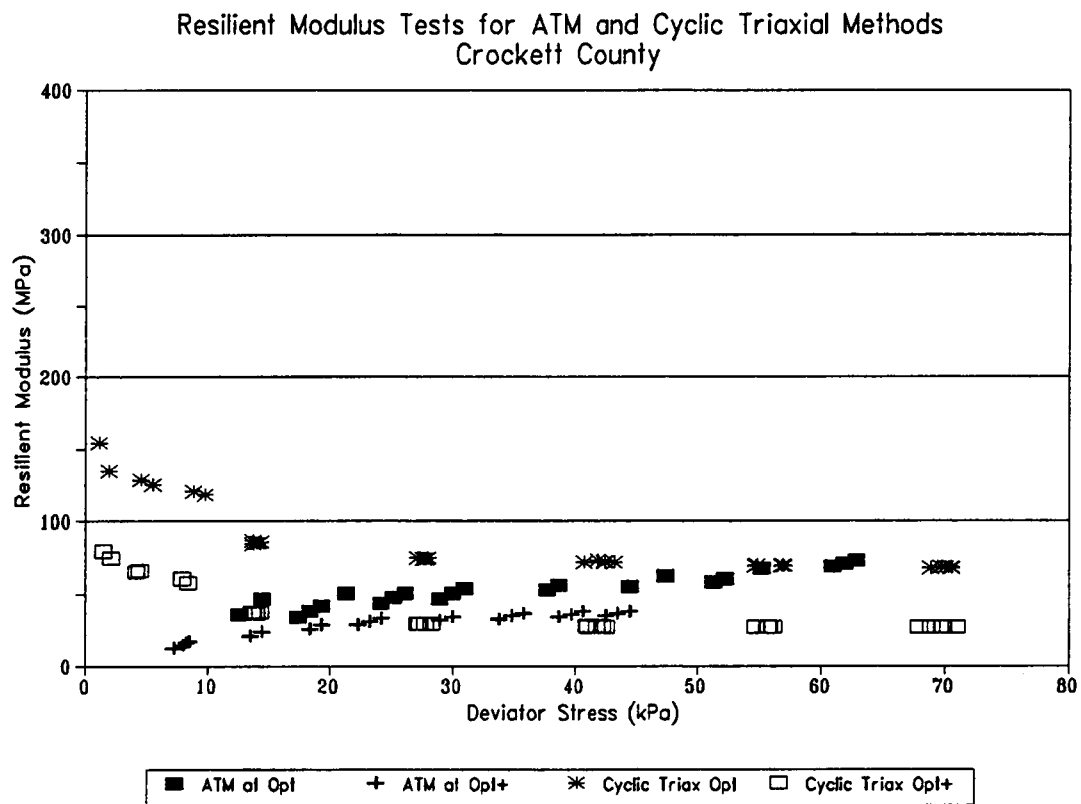


Figure 3-14. Performance of ATM Device on Specimens at Optimum W% and Specimens Saturated to a High Degree of Saturation

Chapter 4 SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

4.1 Summary and Conclusions

A laboratory investigation of six fine-grained soil samples was conducted in order to examine the difference in resilient response for the different soils, and to verify previously gathered data. To determine what types of soils were being tested, standard laboratory index tests were conducted to include: Atterberg limits, specific gravity, grain size distribution and moisture-density relationships. The six soils were collected from across the state of Tennessee to ensure that a good representation of subgrades was obtained. A total of four different AASHTO classifications were selected which include three A-6 soils, one A-4 soil, one A-7-5 soil and one A-7-6 soil.

For this research, two different methods of estimating the resilient modulus were examined on six soils. Two different specimen conditions were investigated: specimens tested at as-compacted conditions at optimum water content and maximum dry density, and specimens compacted at optimum water content and maximum dry density and then saturated to some higher level of saturation before being tested. This saturation after compaction was to simulate the seasonal water content changes in the subgrade. After a subgrade is compacted, the soil structure remains the same but the water content varies. It is this combination of soil structure and degree of saturation that determines the resilient response of a soil. A specimen compacted at optimum water content and later having the degree of saturation increased, will behave differently than a specimen that is originally compacted at the same final water content. Therefore, the soil structure and

appropriate moisture content of the subgrade must be considered when choosing a design modulus value.

The first method of determining the resilient modulus was using the cyclic triaxial method. This is the standard method of determining M_r for use in pavement design. For the specimens that were tested at optimum water content and maximum dry density, the resilient modulus decreased with increasing deviator stress. This behavior is consistent with virtually all other researchers. A comparison with other research found that the responses of the six soils tested compared favorably with the other results. This leads to the conclusion that the data from these six soils was within a reasonable range for the given soils. For three of the soils, two additional specimens were compacted at the same optimum conditions, but the water content was later increased to two different degrees of saturation. The specimens that were saturated after compaction exhibited a reduction in resilient modulus as the water content increased. This reduction in resilient modulus was found to be somewhat linear with the degree of saturation for a given value of deviator stress. From this testing it can be concluded that in the selection of a design value of M_r , care must be taken to select a modulus value that corresponds to the maximum anticipated moisture conditions in the subgrade.

The second method of determining the resilient response was by using the alternative test method (ATM). This method has proven itself to be less time consuming than the traditional cyclic triaxial test. Excluding curing times and soil mixing which are the same for both methods, the ATM method takes about an hour and a half from the beginning of the test until the data has been reduced, while the cyclic triaxial method

takes about five hours. This time savings makes the laboratory measurement of M_r a more feasible goal for agencies which design pavements.

All six of the soil samples were tested using the ATM device, and it was found that while the resilient modulus exhibits an increase with increasing deviator stress, this response is more consistent with conditions in the actual subgrade with respect to the type of confinement. When comparing the specimens compacted at optimum water content and maximum dry density, it was found that the ATM method yields a conservative estimate of the M_r . It was also found that as the level of vertical stress increased, with a corresponding increase in the confining stress, the resilient modulus seemed to be less dependent on the deviator stress level and confining stress. This behavior is typical with the cyclic triaxial tests where the modulus values are closer as the deviator stress increases.

Three of the six samples were tested at optimum water content and maximum dry density and then saturated and tested again. These were the same three soils that were tested at different saturation points in the cyclic triaxial test. The ATM showed an increase in modulus values at the intermediate saturation level and then the typical decrease in modulus at the high levels of saturation. Several factors may have influenced this behavior, such as swelling and increased skin friction between the specimen and the mold, or non uniform distribution of water content. For the higher levels of saturation however, the ATM was able to pick up the decrease in stiffness.

From this testing, there are several conclusions that can be drawn. The ATM device is capable of detecting changes in resilient response but the differences are not as

significant as those measured with the cyclic triaxial device. Also, the results at low stress levels and low drop heights are somewhat independent of the specimen stiffness and seem to be constant for all tests. There are several possible reasons for this behavior. Poisson's ratio is assumed to be the same for all soils and constant for all values of deviator stresses. It is not known what the actual value of Poisson's ratio is for these soils and most likely it is a function of deviator stress. Another consideration is that the ATM device may need to be optimized for a range of typical soil responses. Since the comparisons at low stresses and low drop heights is not good, the hammer weight may need to be decreased and the drop heights increased. Side wall friction in the proctor mold may also be causing problems. To this point the side wall friction has been assumed to be negligible and not considered. However, after several specimens were tested and then pushed out, it is evident that another form of lubricant should be used if this assumption is to remain valid.

4.2 Recommendation for Further Work

Several studies have been completed at the University of Tennessee on resilient modulus of fine-grained soils. The cyclic triaxial method has been used to characterize the response for soils representing most of the state. Recommendations for further work using this method are as follows:

1. Use of a computerized data collection/reduction system that is capable of controlling the test parameters (i.e. load) would make the cyclic triaxial test a much more feasible method of testing. The increased speed in testing would allow much more research to

be conducted in the given time period.

2. Radial strains should be measured for some different soils to determine what typical values of poisson's ratio are during cyclic loading and if poisson's ratio remains constant at different levels of stress.

3. Testing of in-situ specimens from compacted subgrades should be done in order to help correlate field specimens with laboratory specimens. A minimum of nine specimens should be taken from the site. From these nine, three should be tested at the sampling conditions. Three more should be saturated to a medium degree of saturation. While the last three should be saturated to a high degree of saturation. This will help to verify the lab specimens tested during this research.

Further work is needed for the ATM device to be an effective tool to find design resilient modulus values with. Testing that would greatly improve this device is as follows:

1. Test specimens at lower deviator stresses. This testing could be done by removing the weight on the hammer and repeating the procedure. Some of the points will overlap and allow for some fine-tuning to be made with the equations that have been developed.

2. Considerable testing should be conducted using the area method. It is believed that this method will provide a more accurate estimation of the resilient modulus, because energy losses can be included.

3. Some investigations should be conducted to see if fiber-optic strain gauges can be fixed to the mold in order to measure poisson's ratio. This would eliminate some assumptions that have to be made using the current method.

4. Another method of test that should be utilized is the resilient modulus testing with hollow cylinders. This method will allow a better estimation of the resilient modulus since the stress conditions in the specimen are well defined.

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APPENDICES

APPENDIX A

MODEL PARAMETERS, INDEX DATA AND RESILIENT MODULUS CURVES

Table A-1. Model Parameters for all Soils

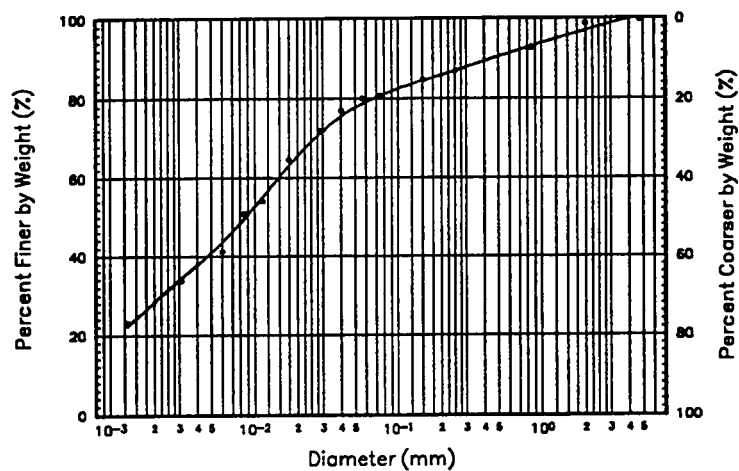
Test No.	γ_d (Mg/m ³)	W% (%)	Hyperbolic Model		Log-Log Model	
			a	b	K ₁	K ₂
Crockett #4-4	1.718	16.3	361.4	66.90	131.3	-1.087
Crockett #4-2	1.702	16.3→18.0	250.2	38.03	74.66	-0.5931
Crockett #4-3	1.709	16.3→19.2	180.0	27.43	55.52	-0.4607
Giles #4-3	1.502	23.8	574.9	106.3	197.8	-1.522
Giles #4-2	1.512	23.8→24.5	791.8	71.93	191.9	-1.956
Giles #4-4	1.510	23.8→26.2	479.0	31.34	96.43	-1.042
Shelby #4-1	1.762	14.5	644.7	74.30	226.6	-2.695
Shelby #3-6	1.746	14.8→15.8	217.1	51.12	86.85	-0.5963
Shelby #3-4	1.763	14.8→17.4	322.3	33.19	98.22	-1.116
Hamilton #1-2	1.763	15.3	696.0	137.6	257.4	-2.025
VanBuren #1-2	1.597	21.3	731.0	173.7	307.7	-2.339
White #1-3	1.673	18.8	410.1	114.6	198.4	-1.452

CROCKETT COUNTY
STA 925
RESILIENT MODULUS PLOTS AND MODEL PARAMETERS

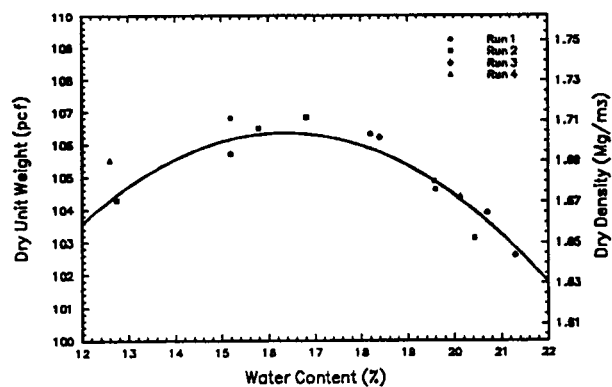
LOCATION: Crockett County STA 925+20	
SOIL CLASSIFICATION	
AASHTO	USCS
A-6 (11)	CL

CLAY CONTENT (%)	PERCENT PASSING #200 SIEVE	ATTERBERG LIMITS			G _s	STANDARD PROCTORS	
		LL	PL	PI		max. γ_d (pcf)	opt. w% (%)
36.0	99.1	33	22	11	2.61	106.6	16.5

Grain Size Distribution
Giles Co. STA 270+00



Standard Proctors
Crockett County STA 925+20



RESILIENT MODULUS RESULTS FOR TEST
Crockett Co.#4-4 DD=1.718 Mgc/m W%=16.3

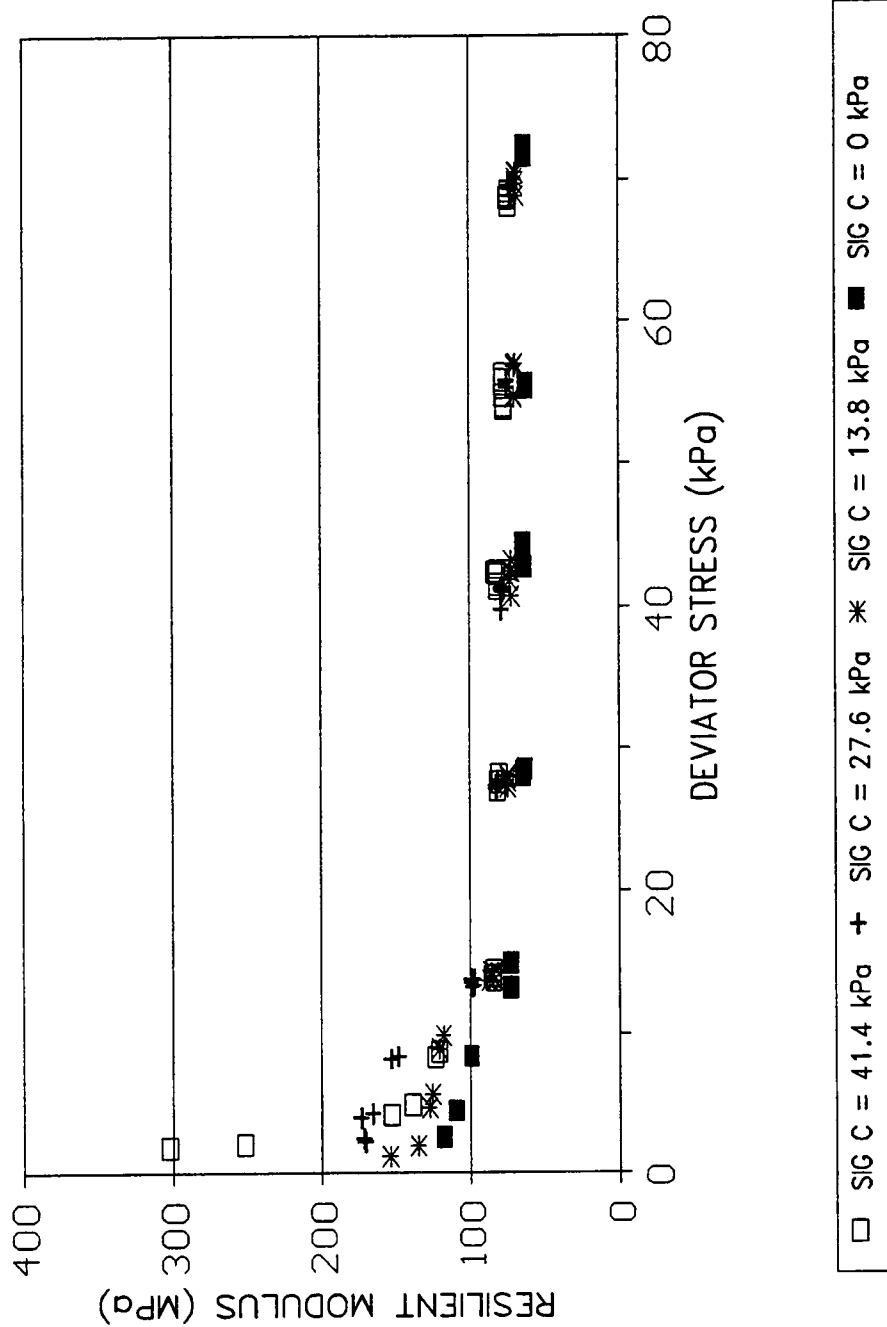


Figure A-1. Resilient Modulus versus Deviator Stress for Crockett County Soil Compacted at Optimum Water Content and Maximum Dry Density (Standard Proctor), with a Degree of Saturation of 82.2%

RESILIENT MODULUS RESULTS FOR TEST
Crockett Co. #4-2 DD=1.702 Mgc_m W%=16.3-18.0

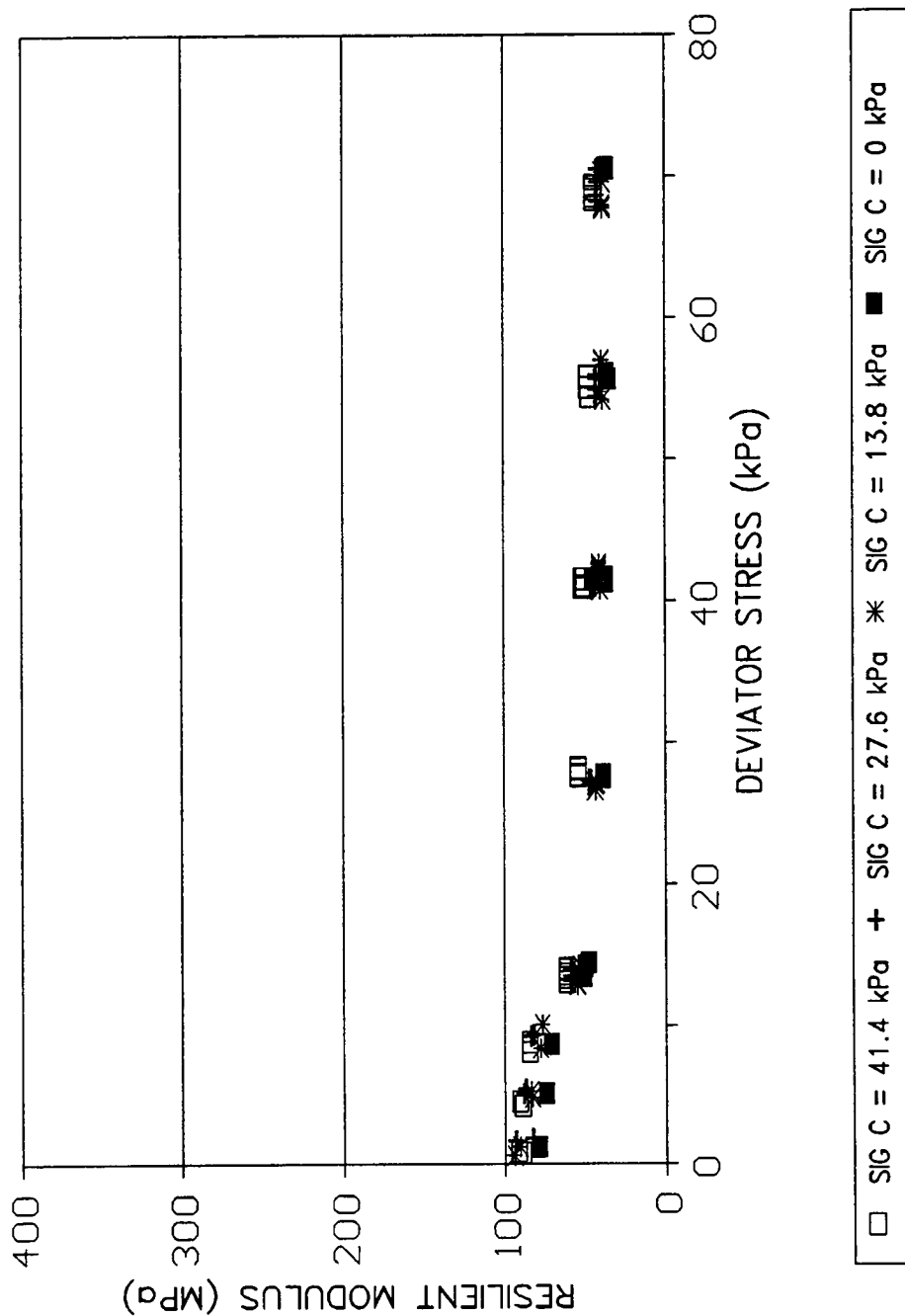


Figure A-2. Resilient Modulus versus Deviator Stress for Crockett County Soil Compacted at Optimum Water Content and Maximum Dry Density, then Increasing the Degree of Saturation from 81.0% to 88.3%

RESILIENT MODULUS RESULTS FOR TEST
Crockett Co. #4-3 DD=1.709 MgcM W%=16.3-19.2

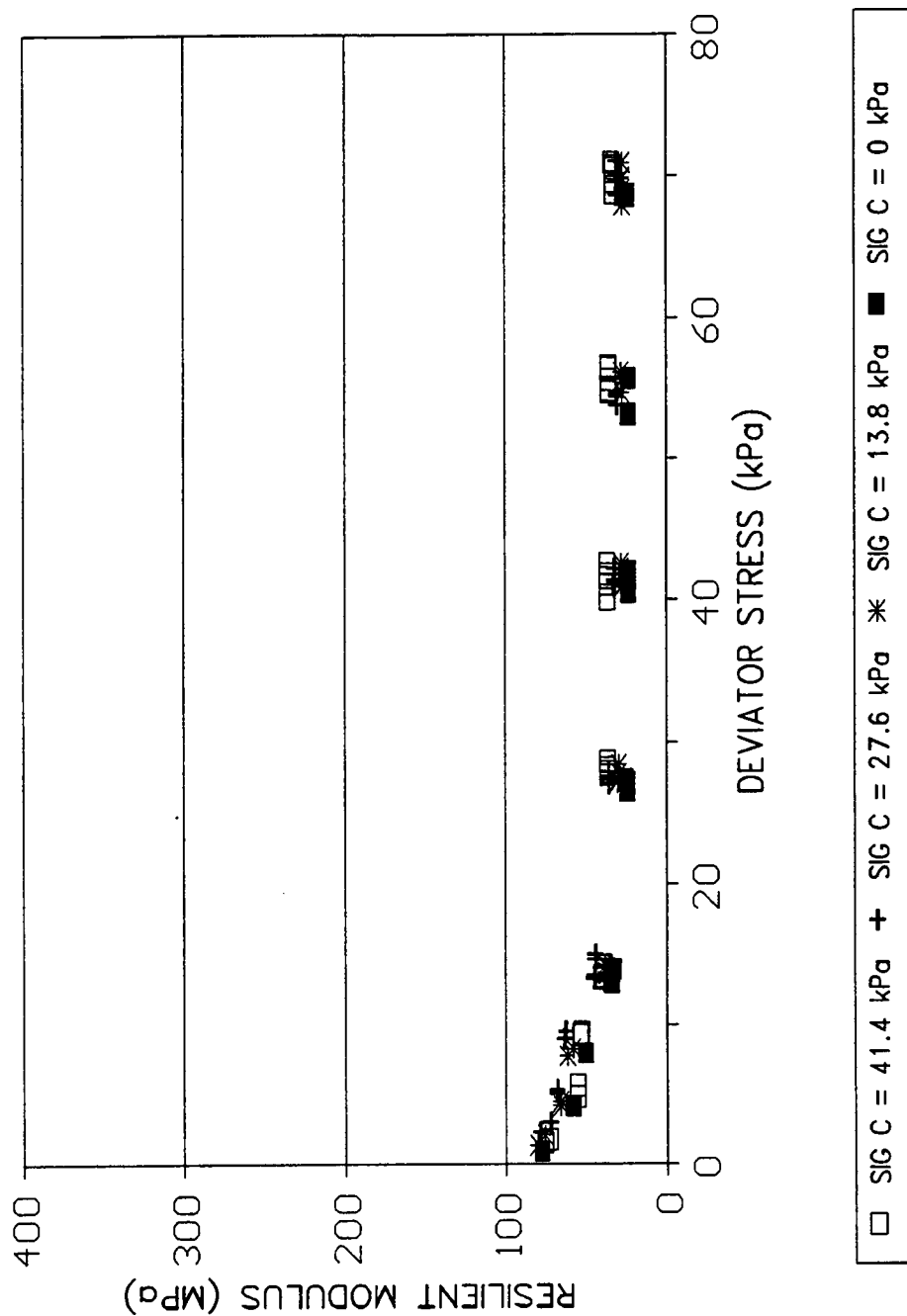


Figure A-3. Resilient Modulus versus Deviator Stress for Crockett County Soil Compacted at Optimum Water Content and Maximum Dry Density, then Increased the Degree of Saturation from 81.3% to 95.2%

Resilient Modulus Results for Test Crockett Co. #1-1a DD=1.670 Mgcmm W%=16.2

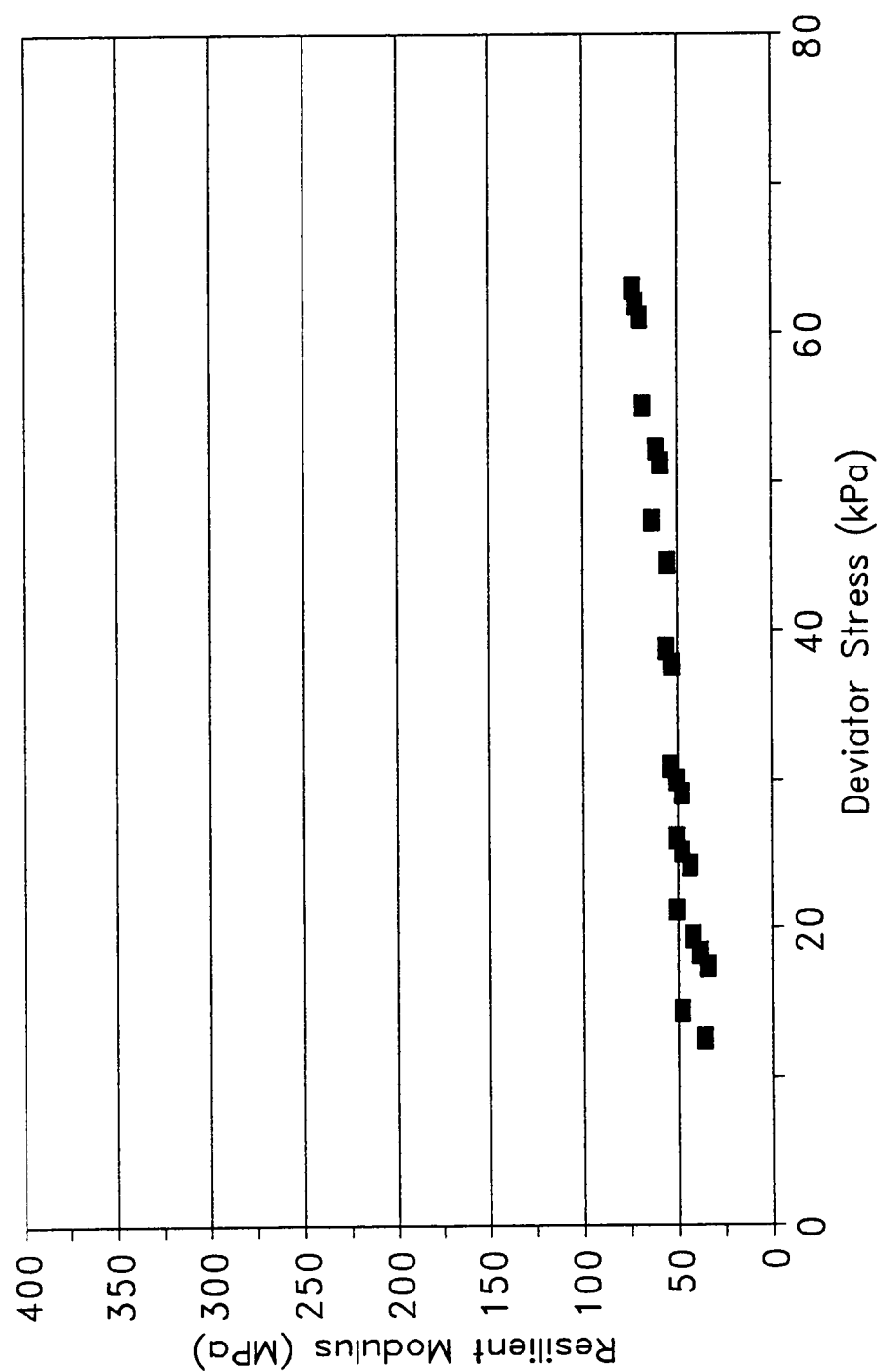


Figure A-4. Resilient Modulus versus Deviator Stress for Crockett County Soil Compacted at Optimum W% and Maximum γ_d (Standard Proctor) at S = 75.3% using the ATM Method.

Resilient Modulus Results for Test Crockett Co. #1-1a DD=1.670 MgcM W%=16.2-17.4

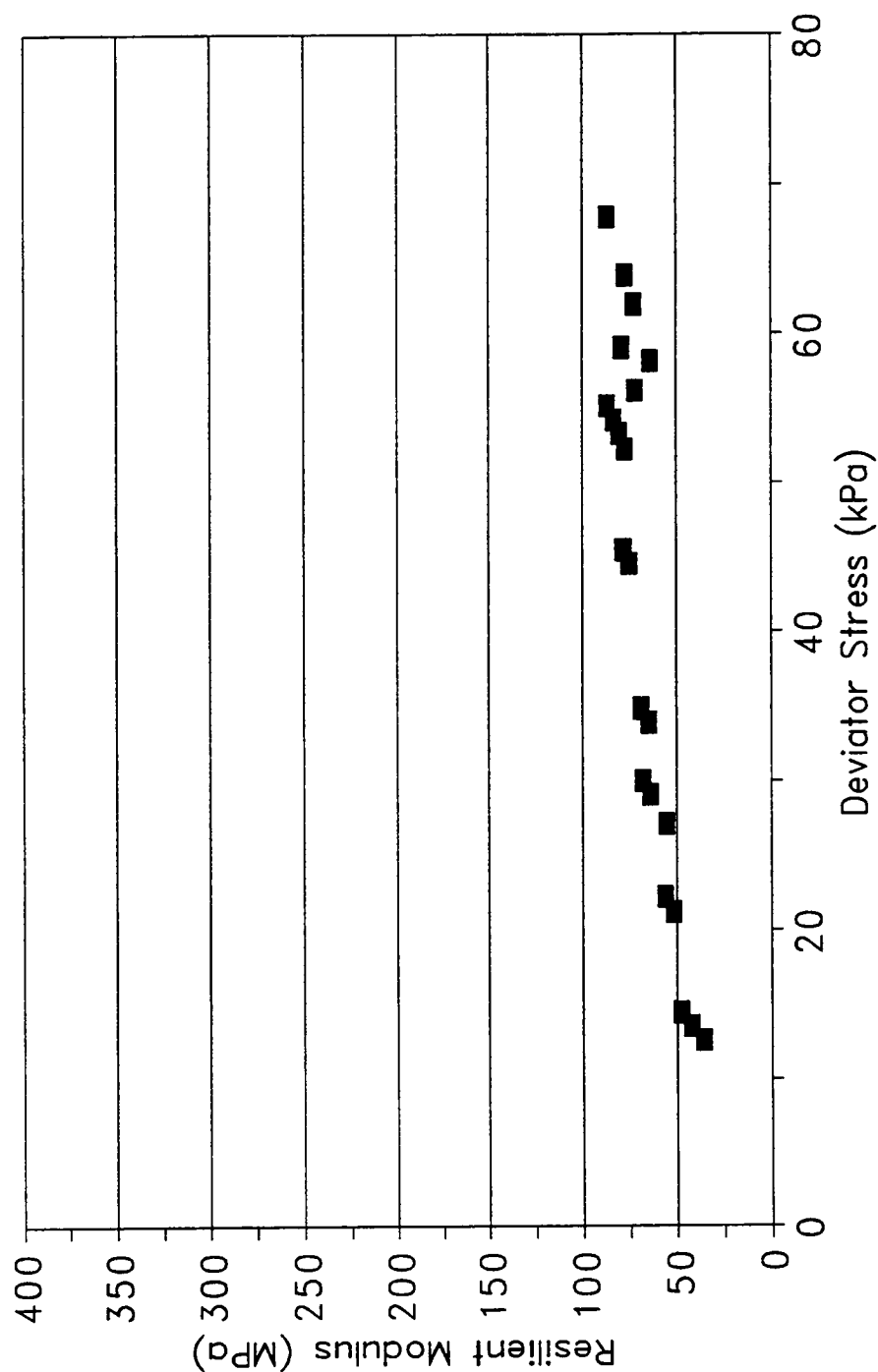


Figure A-5. Resilient Modulus versus Deviator Stress for Crockett County Soil Compacted at Optimum W% and Maximum γ_d at S increasing from 75.3% to 80.9% using the ATM Method.

Resilient Modulus Results for Test Crockett Co. #1-1a DD=1.670 MgcM W%=16.2-18.8

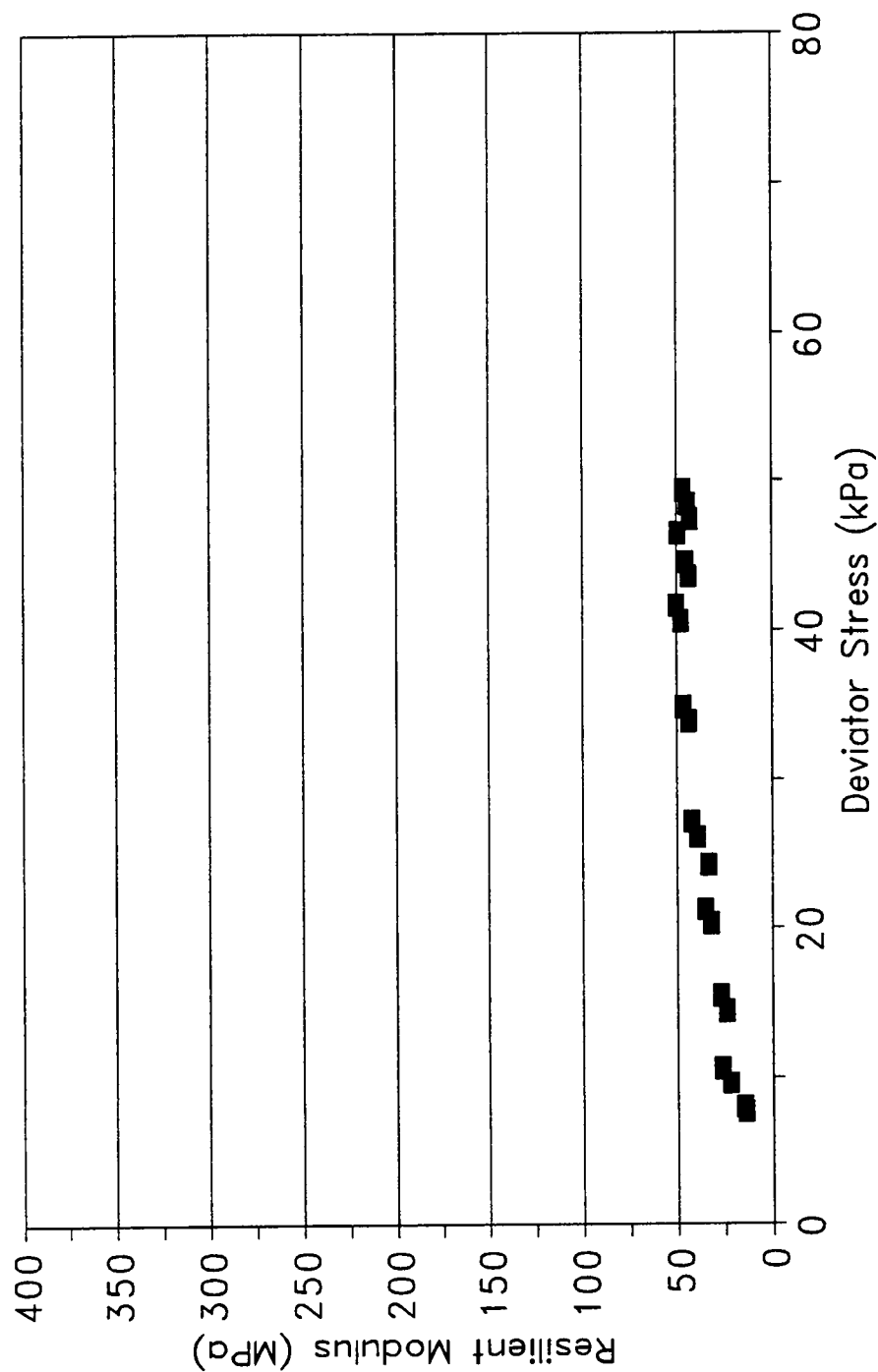


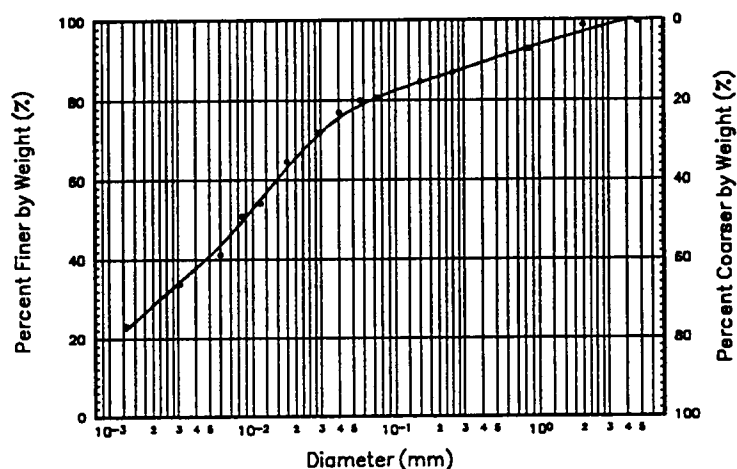
Figure A-6. Resilient Modulus versus Deviator Stress for Crockett County Soil Compacted at Optimum W% and Maximum γ_d at S increasing from 75.3% to 87.3% using the ATM Method.

GILES COUNTY
STA 270
RESILIENT MODULUS PLOTS AND MODEL PARAMETERS

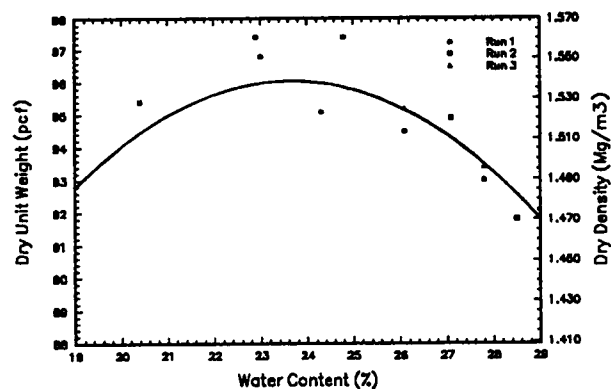
LOCATION: Giles County STA 270+00	
SOIL CLASSIFICATION	
AASHTO	USCS
A-7-5 (19)	CH

CLAY CONTENT (%)	PERCENT PASSING #200 SIEVE	ATTERBERG LIMITS			G_s	STANDARD PROCTORS	
		LL	PL	PI		max. γ_d (pcf)	opt. w% (%)
53.0	80.9	52	30	22	2.64	96.1	23.7

Grain Size Distribution
Giles Co. STA 270+00



Standard Proctors
Giles Co. STA 270+00



RESILIENT MODULUS RESULTS FOR TEST
Giles Co. #4-3 DD=1.502 Mgcmm W%=23.8

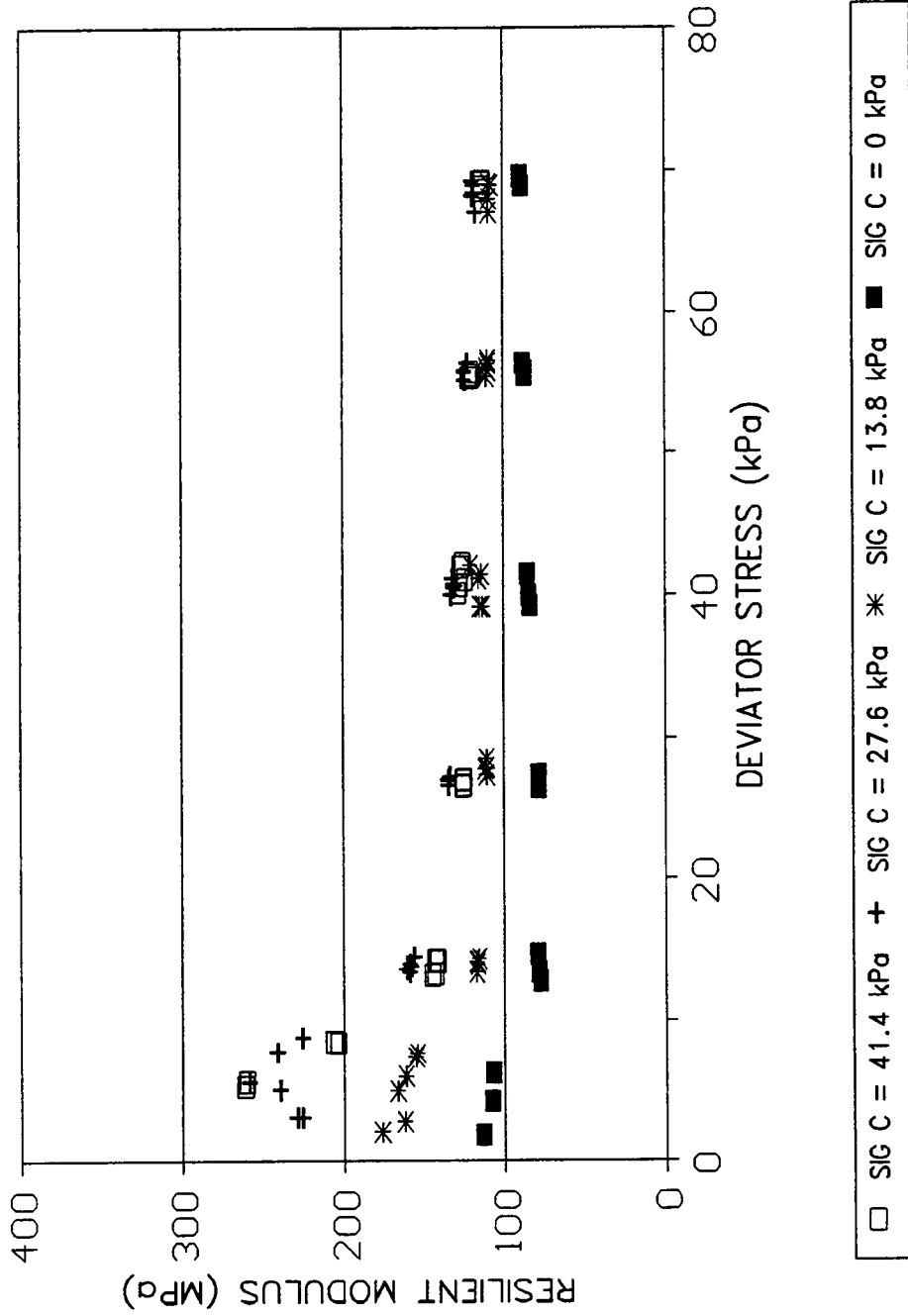


Figure A-7. Resilient Modulus versus Deviator Stress for Giles County Soil Compacted at Optimum Water Content and Maximum Dry Density (Standard Proctor), with a Degree of Saturation of 83.1%

RESILIENT MODULUS RESULTS FOR TEST
Giles Co.#4-4 DD=1.510 MgcM W%=23.8-26.2

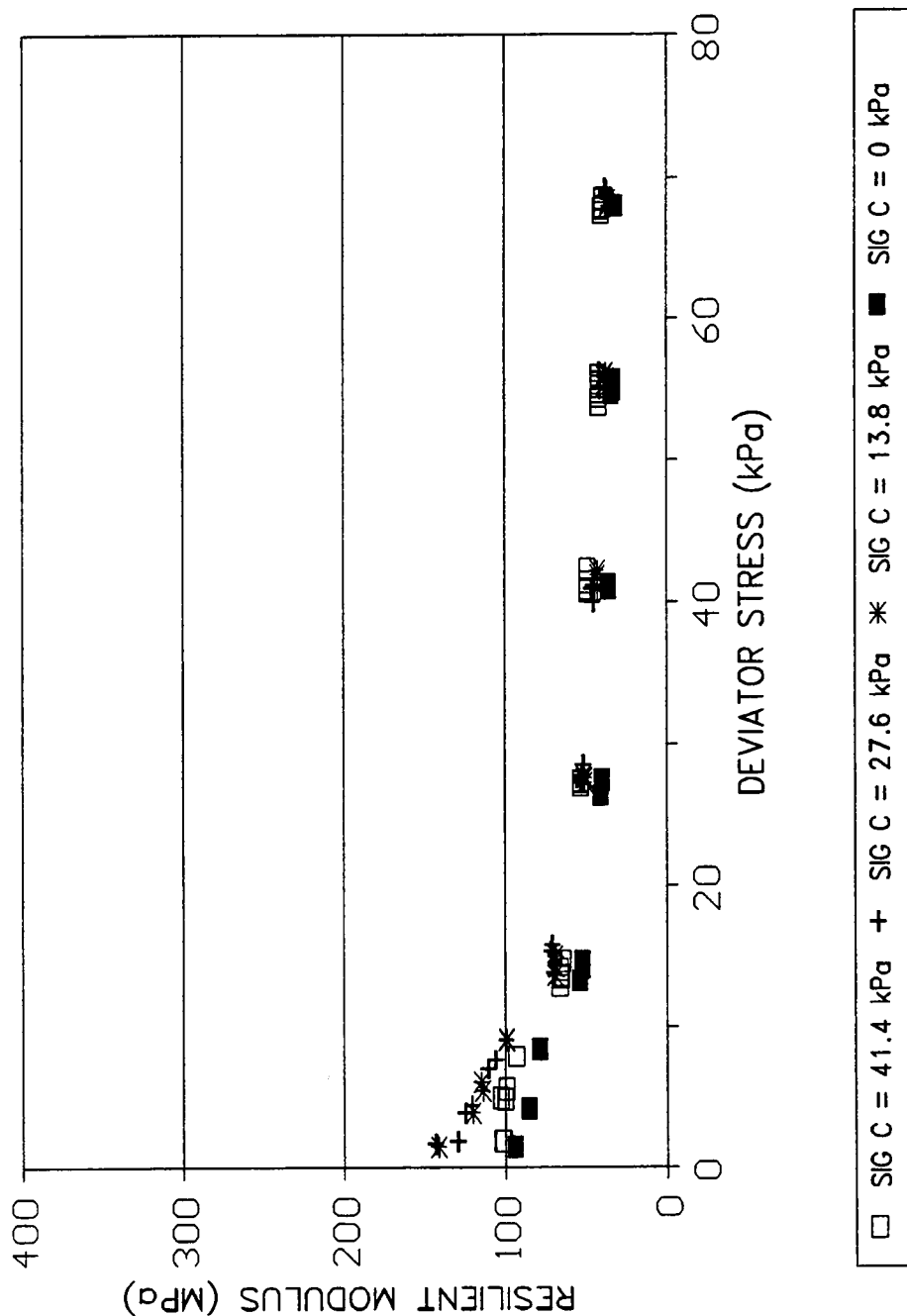


Figure A-8. Resilient Modulus versus Deviator Stress for Giles County Soil Compacted at Optimum Water Content and Maximum Dry Density, then Increasing the Degree of Saturation from 84.7% to 86.8%

RESILIENT MODULUS RESULTS FOR TEST
 Giles Co.#4-2 DD=1.512 MgcM W%=23.8-24.5

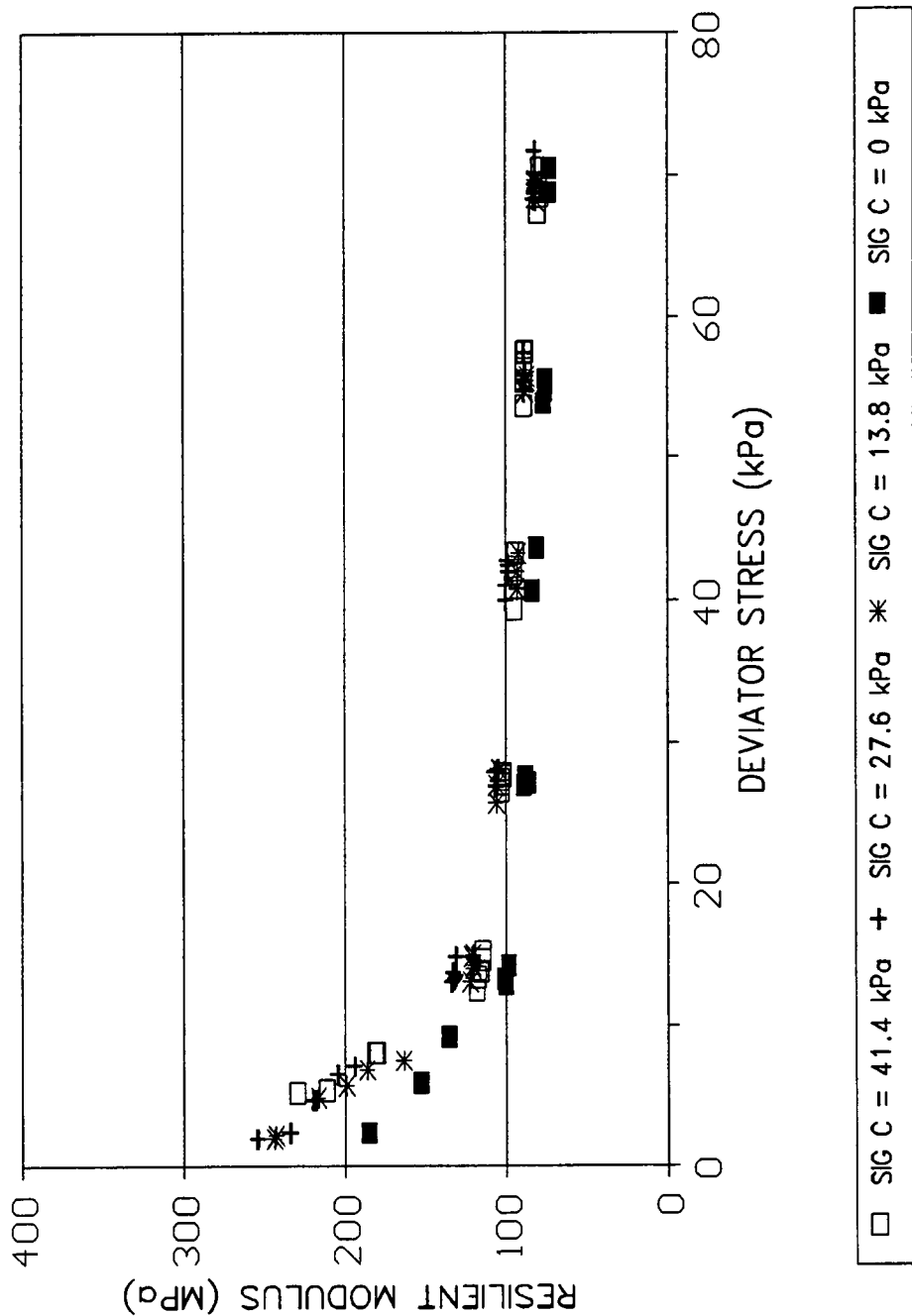


Figure A-9. Resilient Modulus versus Deviator Stress for Giles County Soil Compacted at Optimum Water Content and Maximum Dry Density, then Increasing the Degree of Saturation from 86.5 % to 92.6 %

Resilient Modulus Results for Test Giles Co.#4-1 DD=1.600 Mgcmm W%=23.8

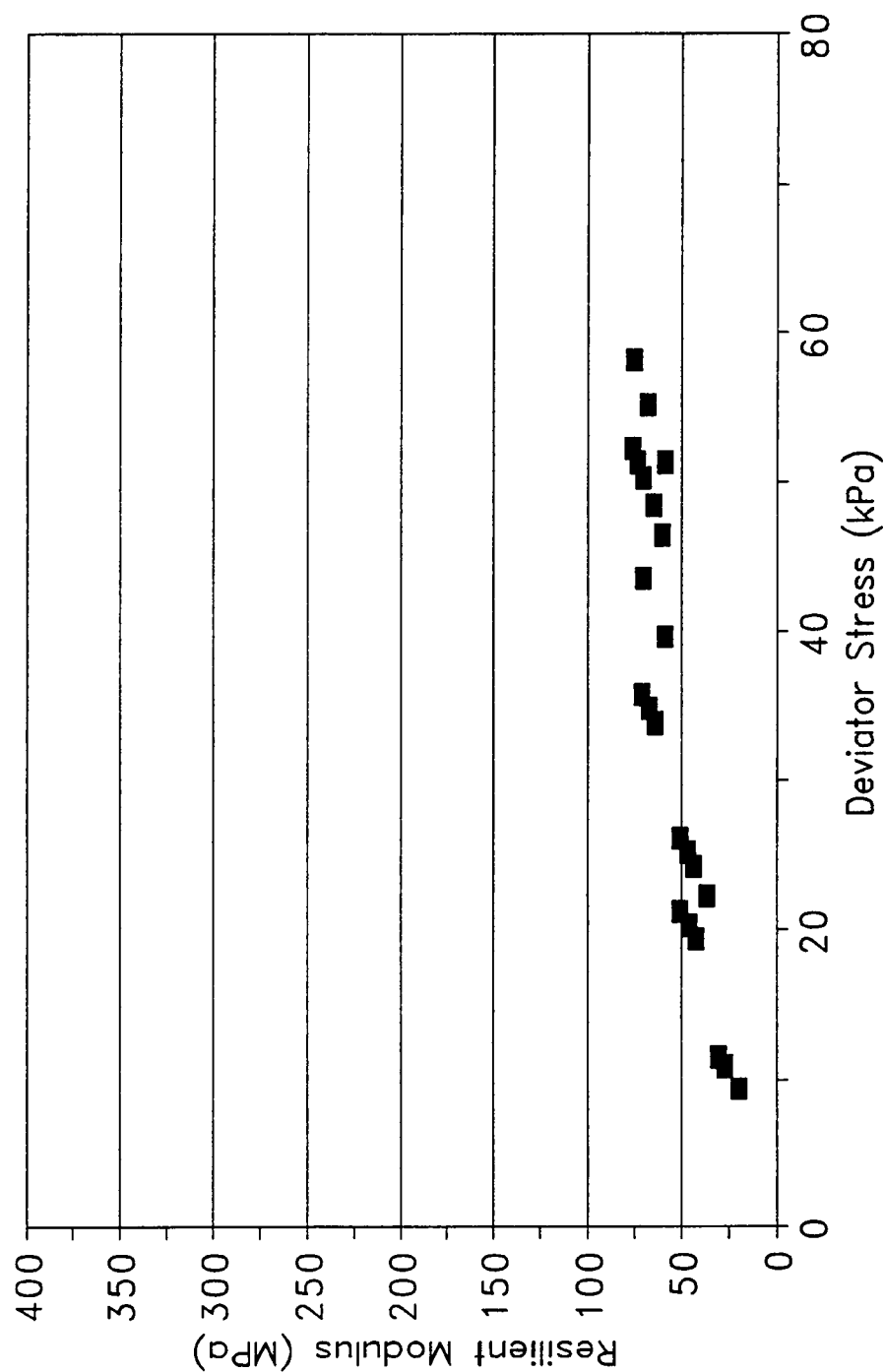


Figure A-10. Resilient Modulus versus Deviator Stress for Giles County Soil Compacted at Optimum W% and Maximum γ_d (Standard Proctor) at S = 95.8% using the ATM Method.

Resilient Modulus Results for Test Giles Co. #4-1 DD=1.600Mgcm W%=23.3-24.1

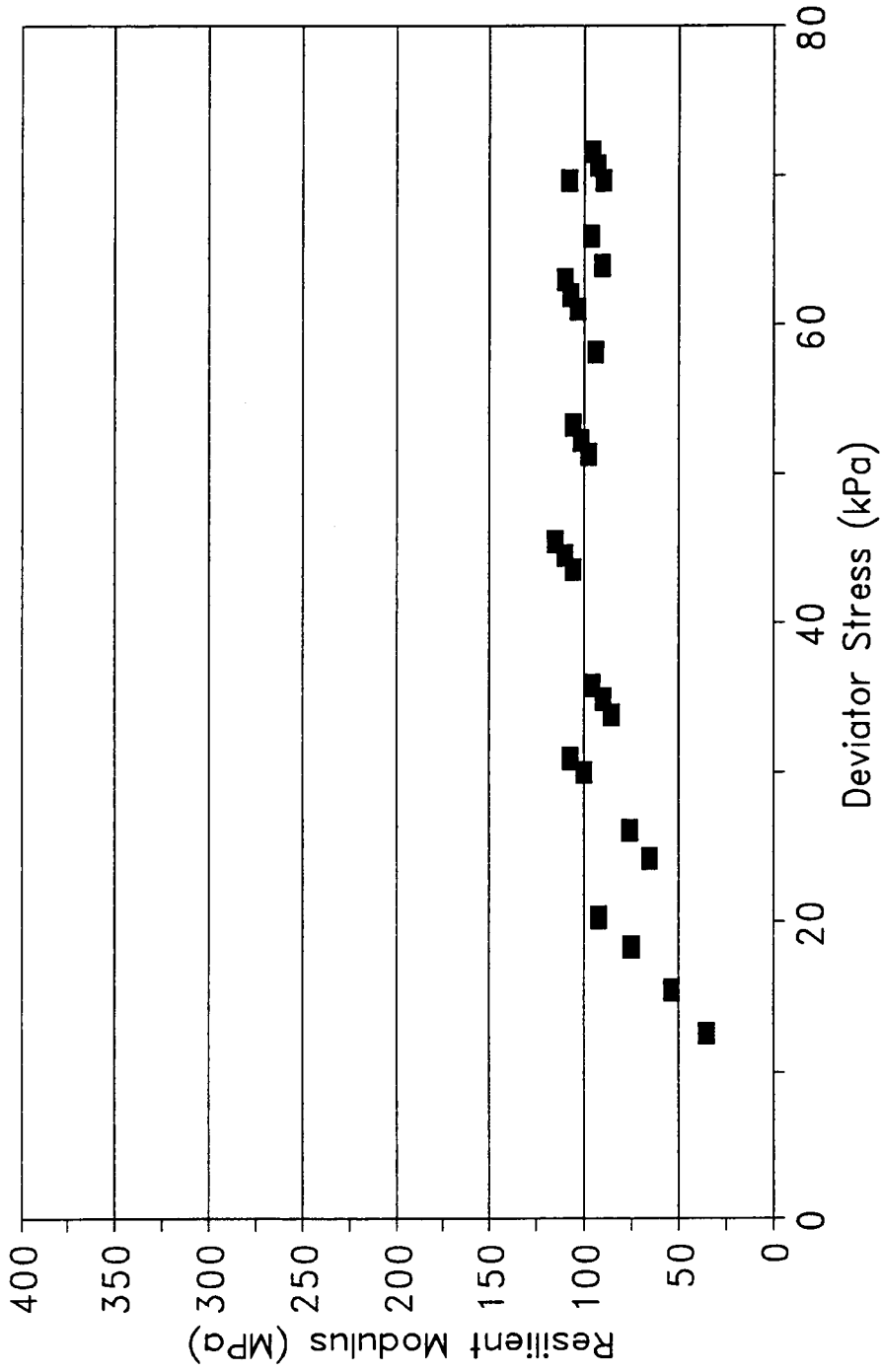


Figure A-11. Resilient Modulus versus Deviator Stress for Giles County Soil Compacted at Optimum W% and Maximum γ_d then increasing S from 95.8% to 97.0% using the ATM Method.

Resilient Modulus Results for Test GilesCo.#4-1a DD=1.600Mgcm W%=23.8-24.7

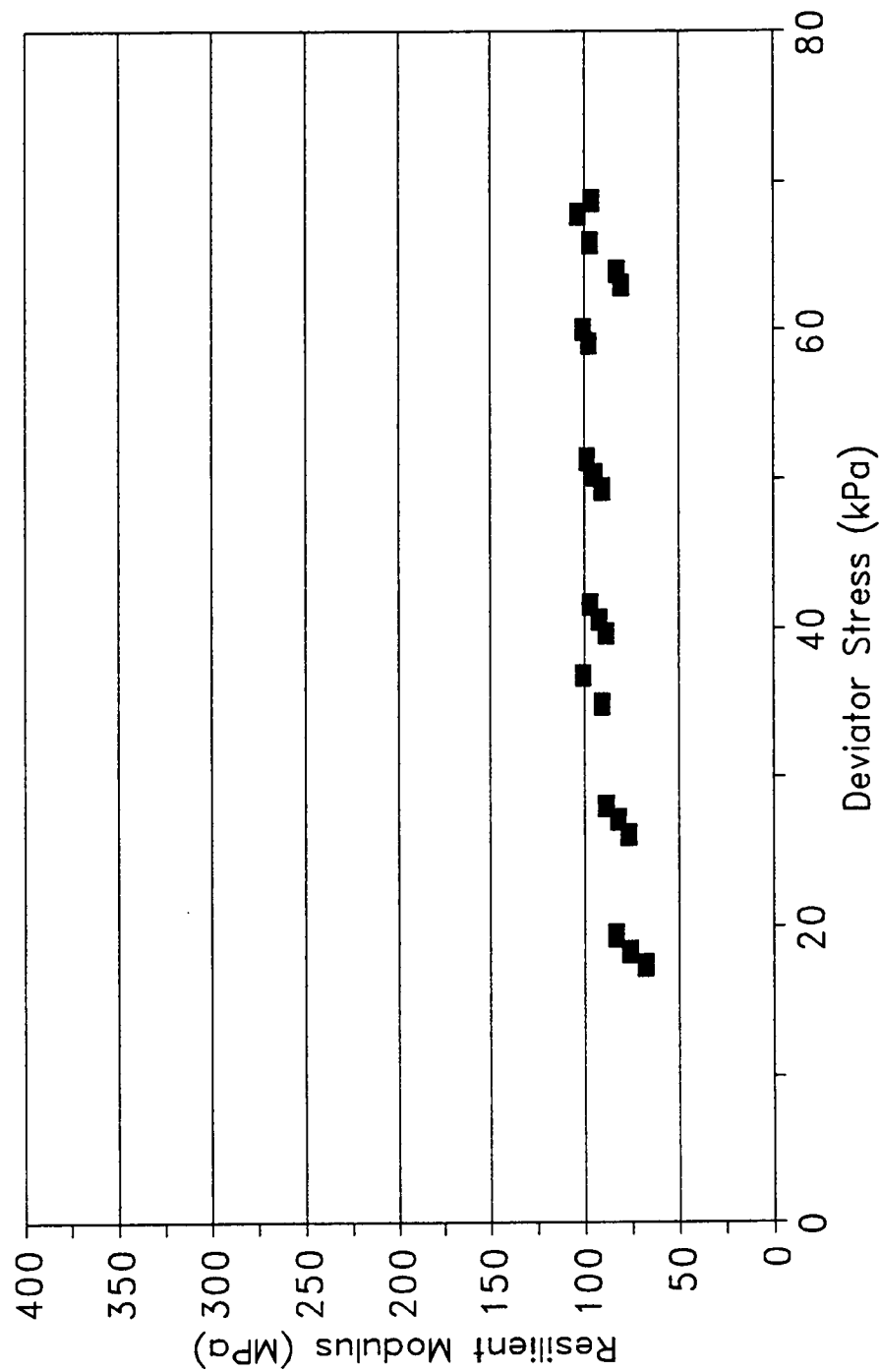


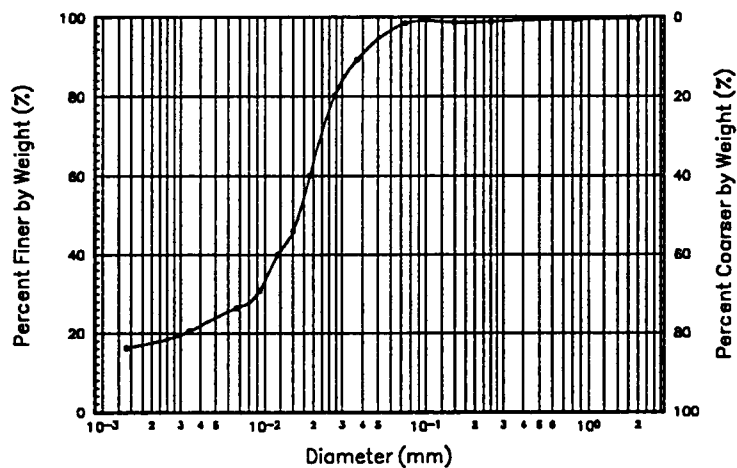
Figure A-12. Resilient Modulus versus Deviator Stress for Giles County Soil Compacted at Optimum W% and Maximum γ_d then increasing S from 95.8% to 99.5% using the ATM Method.

**SHELBY COUNTY
STA 9
RESILIENT MODULUS PLOTS AND MODEL PARAMETERS**

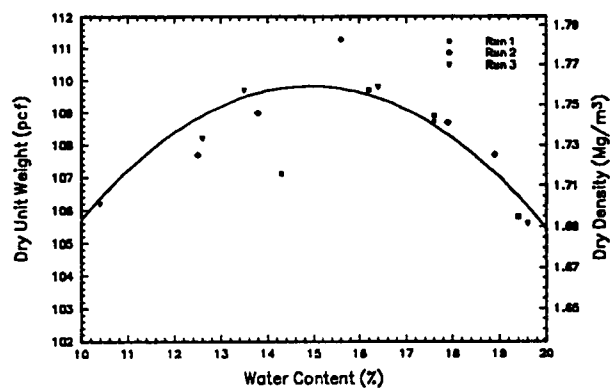
LOCATION: Shelby County STA 9+00	
SOIL CLASSIFICATION	
AASHTO	USCS
A-4 (10)	CL

CLAY CONTENT (%)	PERCENT PASSING #200 SIEVE	ATTERBERG LIMITS			G_s	STANDARD PROCTORS	
		LL	PL	PI		max. γ_d (pcf)	opt. w% (%)
42.0	98.6	30	20	10	2.70	109.8	15.0

Grain Size Distribution
Shelby Co. STA 9+00



Standard Proctors
Shelby Co. STA 9+00



RESILIENT MODULUS RESULTS FOR TEST
Shelby Co.#4-1 DD=1.762 MgcM W%=14.5

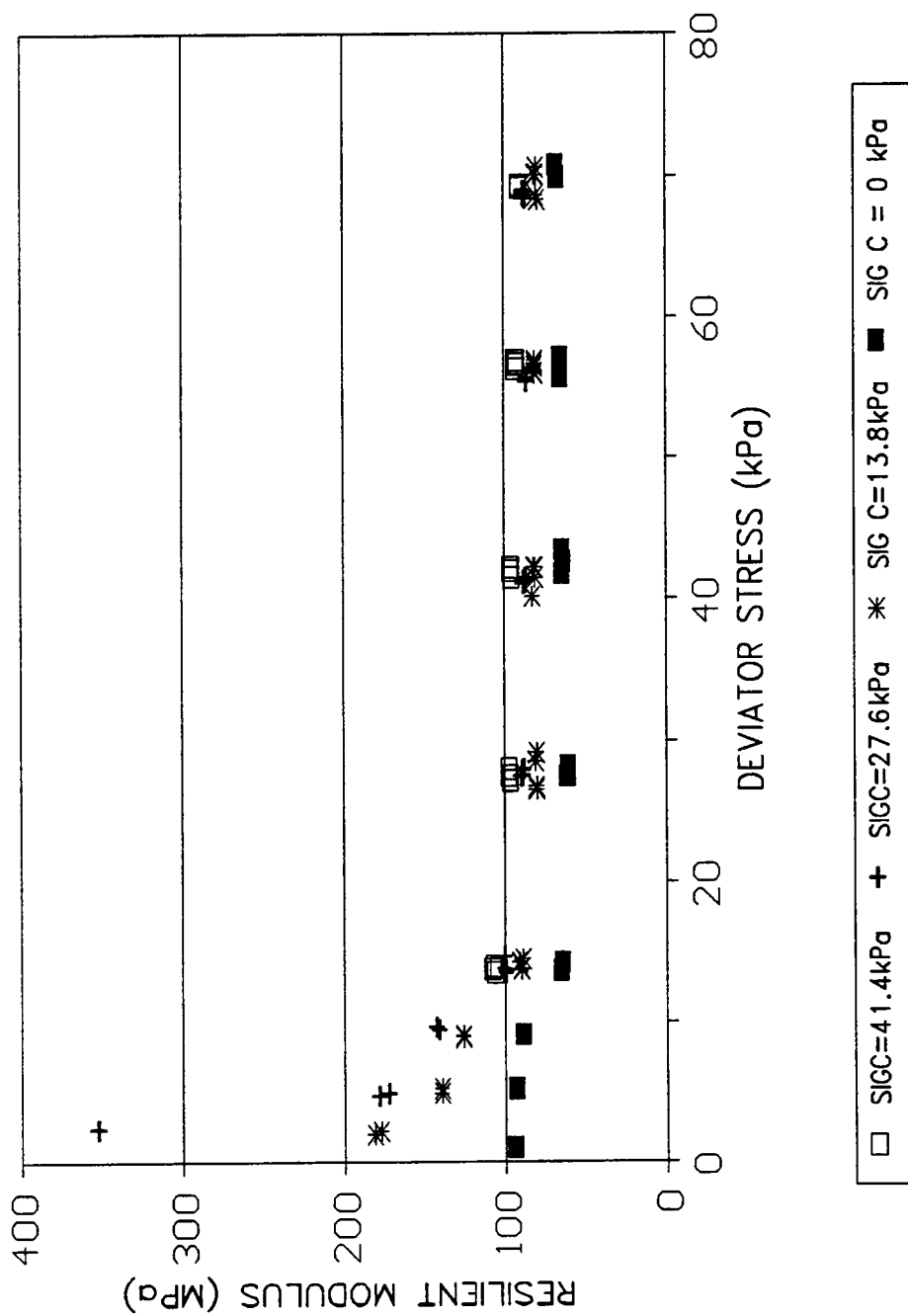


Figure A-13. Resilient Modulus versus Deviator Stress for Shelby County Soil Compacted at Optimum Water Content and Maximum Dry Density (Standard Proctor), with a Degree of Saturation of 73.6%

RESILIENT MODULUS RESULTS FOR TEST
Shelby Co.#3-6 DD=1.746 MgcM W%=14.8-15.8

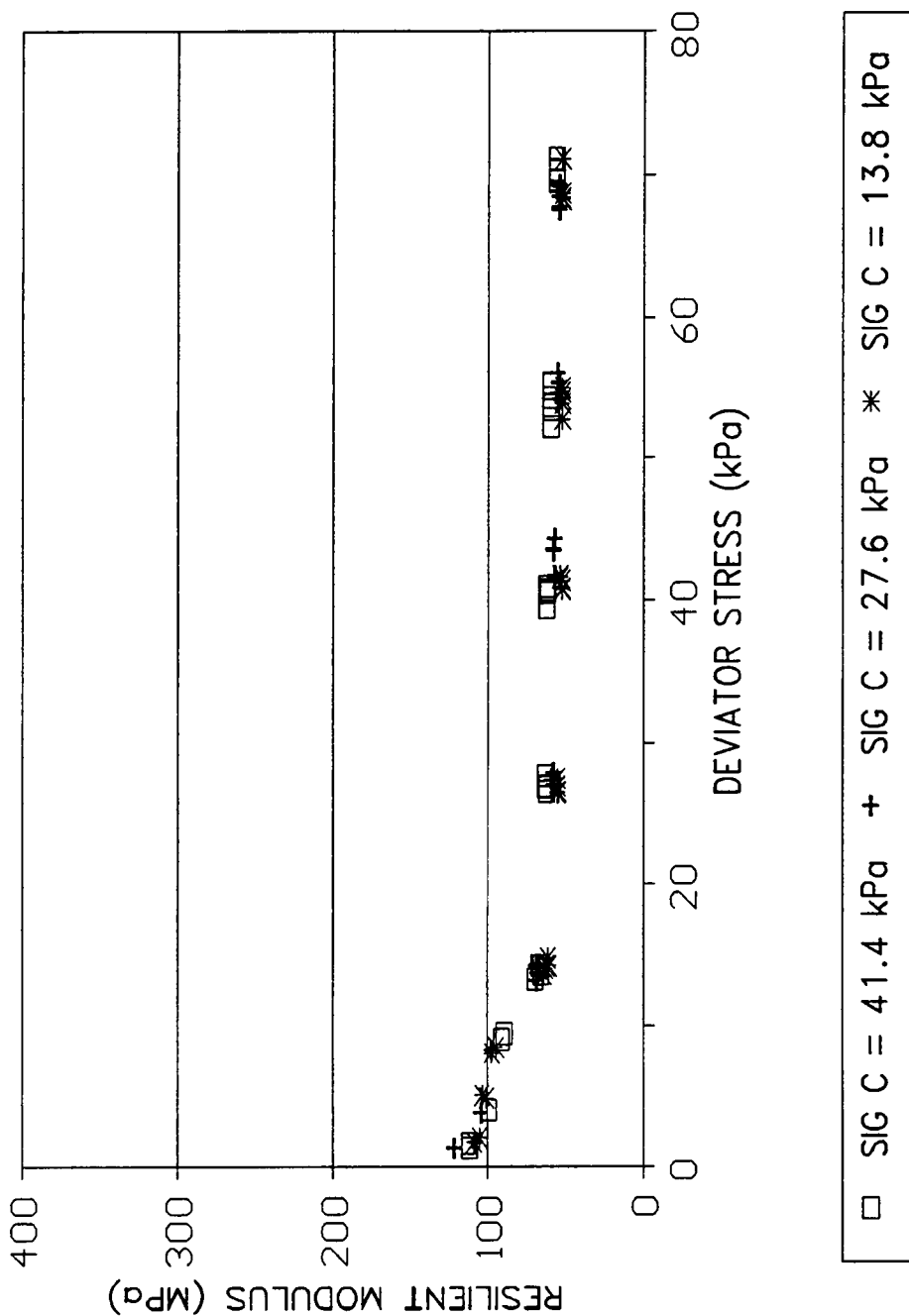


Figure A-14. Resilient Modulus versus Deviator Stress for Shelby County Soil Compacted at Optimum Water Content and Maximum Dry Density, then Increasing Degree of Saturation from 73.4 % to 78.2 %

RESILIENT MODULUS RESULTS FOR TEST
Shelby Co. #3-4 DD=1.763 Mgc/m W%=14.8-17.4

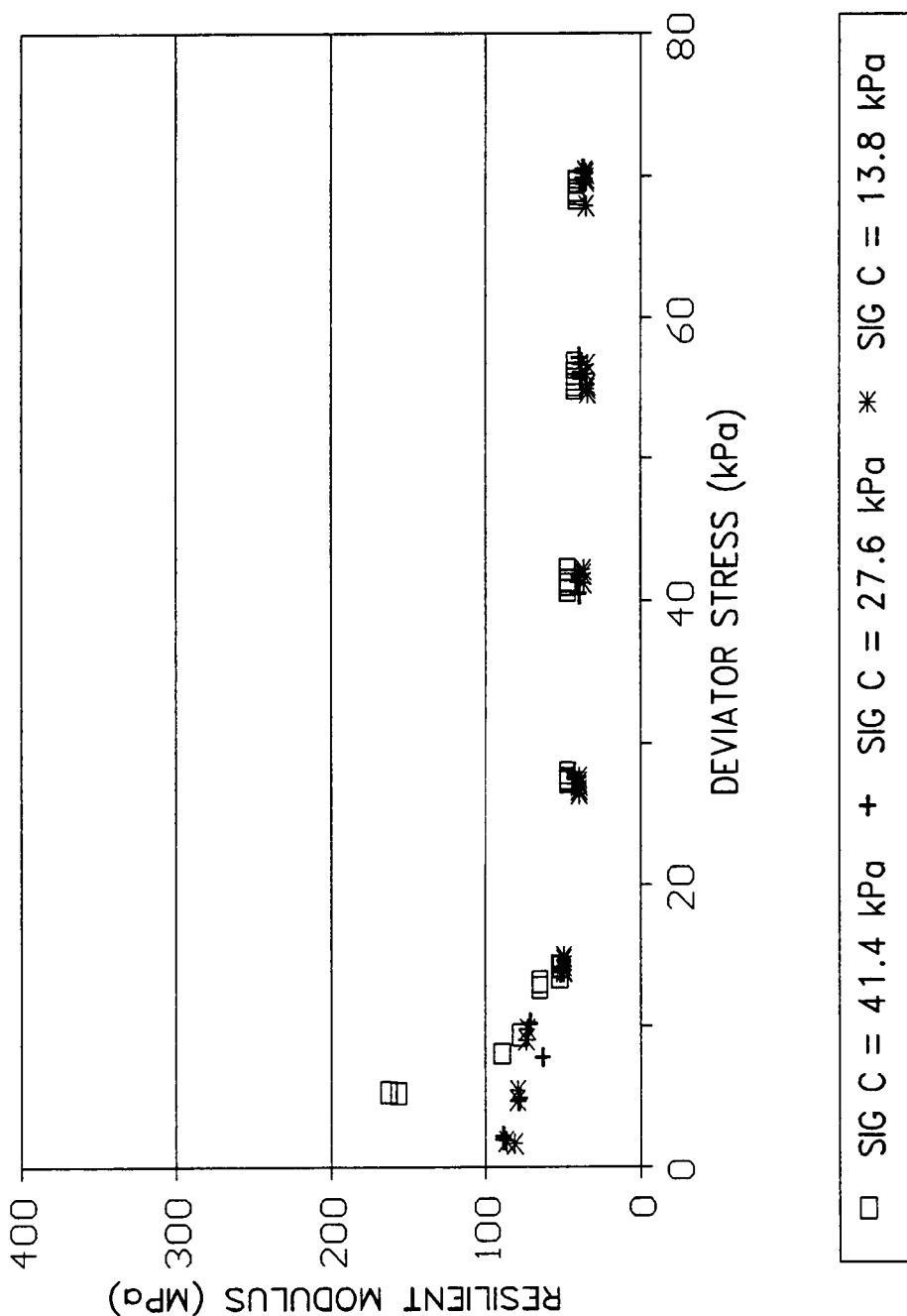


Figure A-15. Resilient Modulus versus Deviator Stress for Shelby County Soil Compacted at Optimum Water Content and Maximum Dry Density, then Increasing the Degree of Saturation from 74.4% to 88.6%

Resilient Modulus Results for Test Shelby Co.#1-2a DD=1.778 Mgcmm W%=14.9

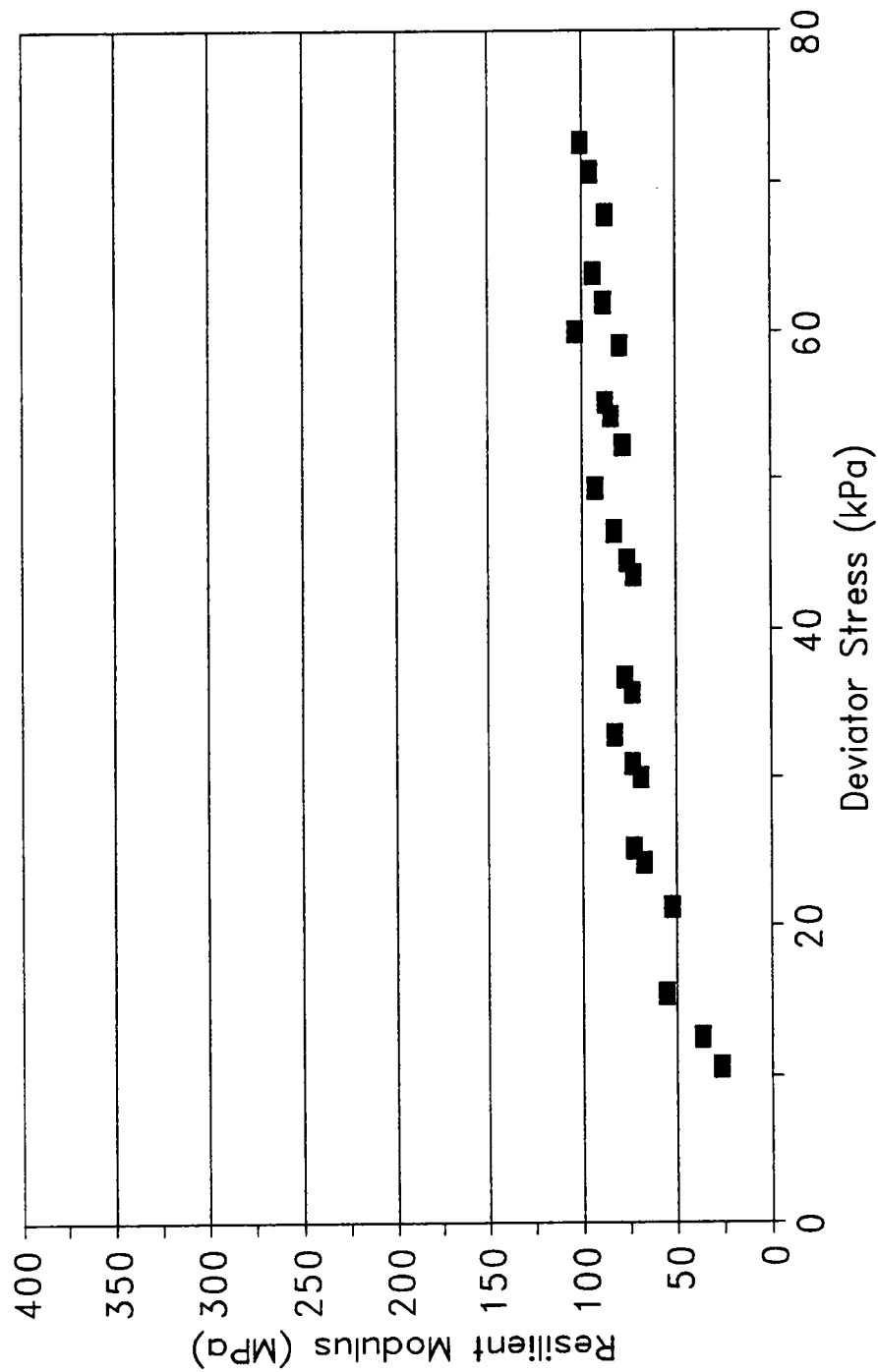


Figure A-16. Resilient Modulus versus Deviator Stress for Shelby County Soil Compacted at Optimum W% and Maximum γ_d then using the ATM Method.

Resilient Modulus Results for Test Shelby Co. #1-2a DD=1.778 MgcM W%=14.9-15.9

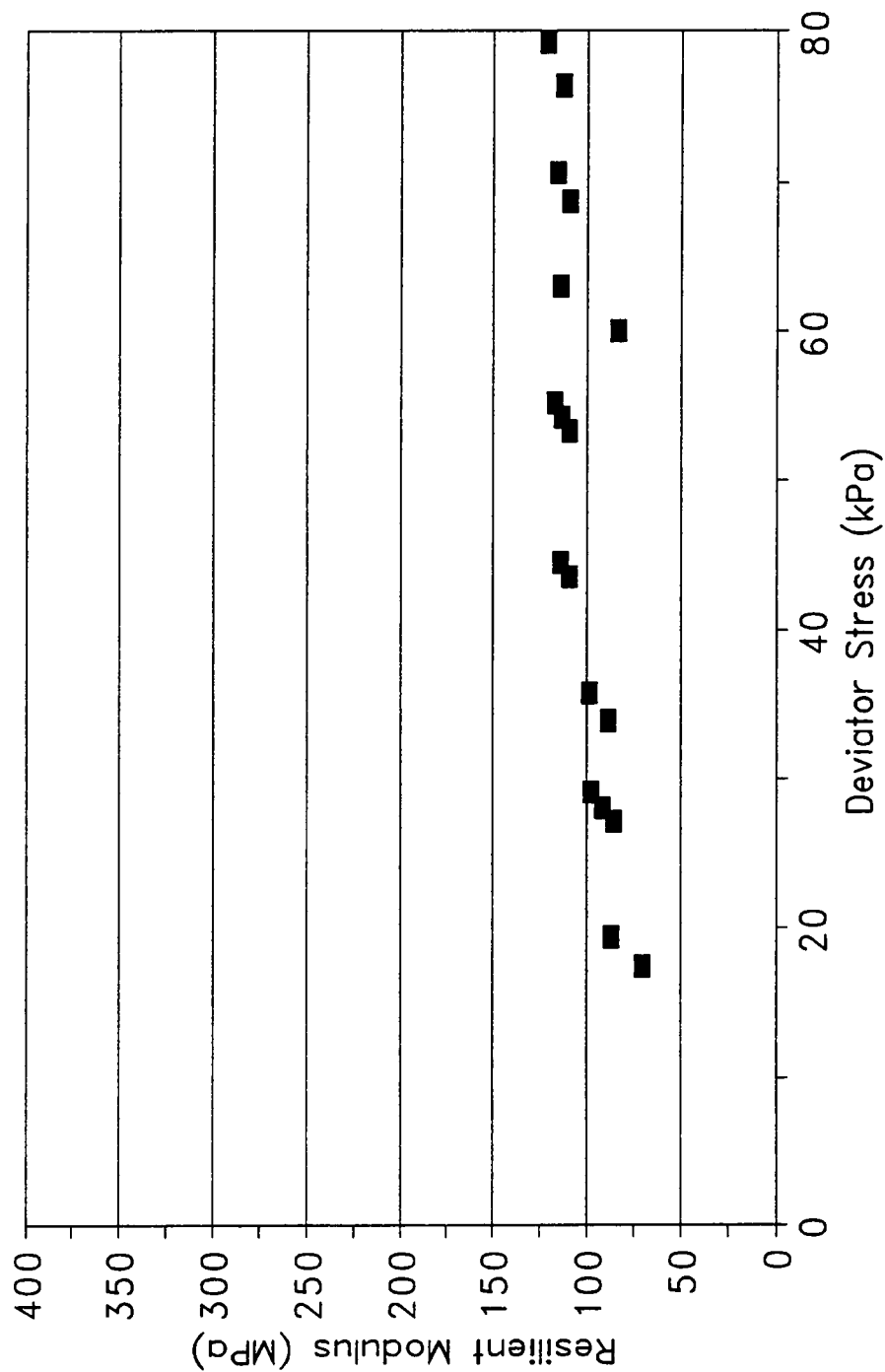


Figure A-17. Resilient Modulus versus Deviator Stress for Shelby County Soil Compacted at Optimum W% and Maximum γ_d then increasing S from 77.7% to 82.9% using the ATM Method

Resilient Modulus Results for Test Shelby Co. 1-2a DD=1.778 Mgc/m W%=14.9-17.5

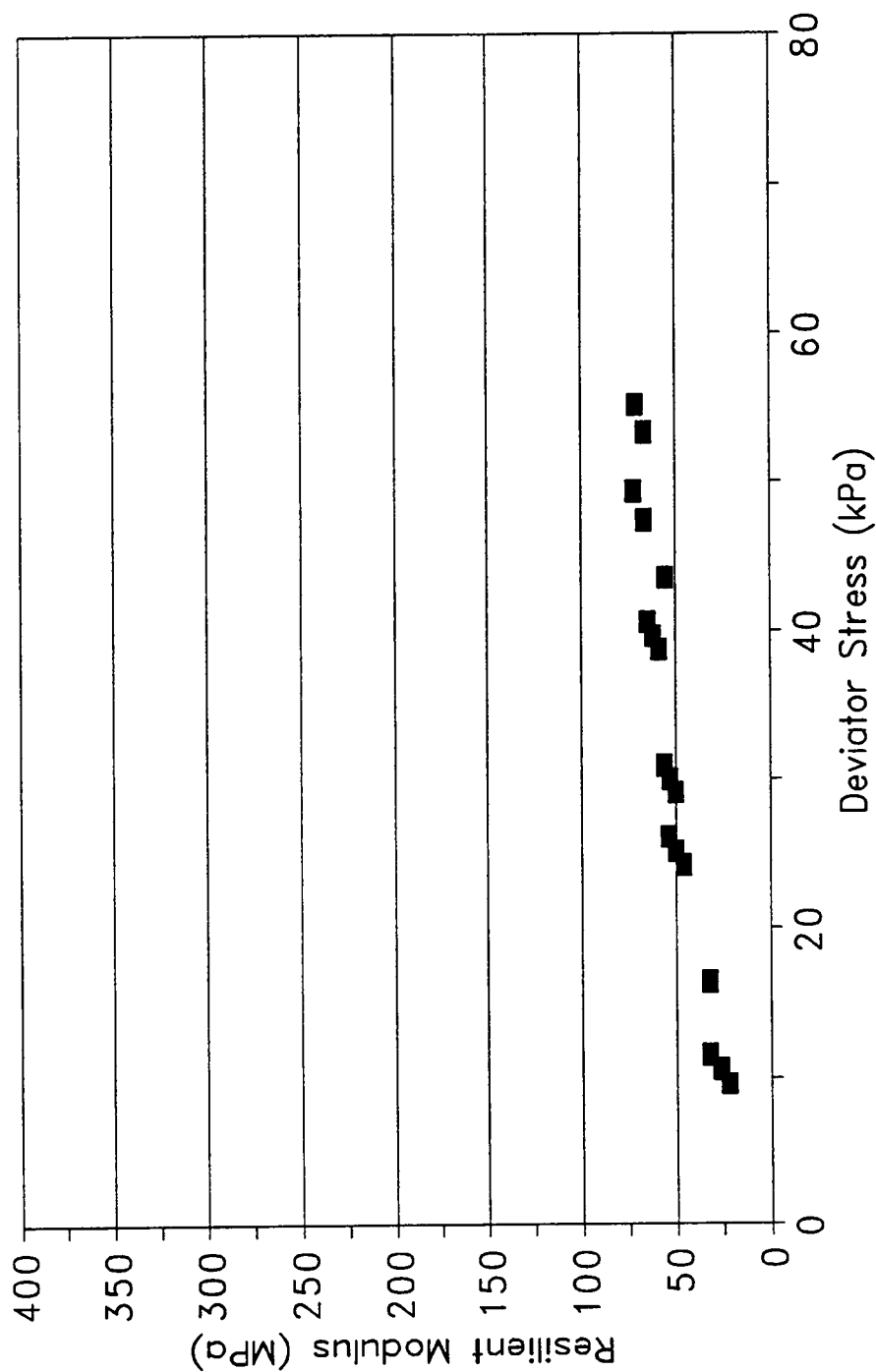


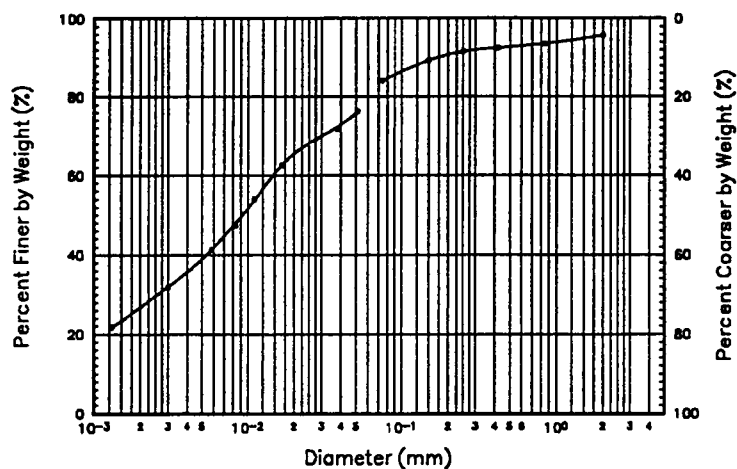
Figure A-18. Resilient Modulus versus Deviator Stress for Shelby County Soil Compacted at Optimum W% and Maximum γ_d then increasing S from 77.7% to 91.3% using the ATM Method

HAMILTON COUNTY
STA 578
RESILIENT MODULUS PLOTS AND MODEL PARAMETERS

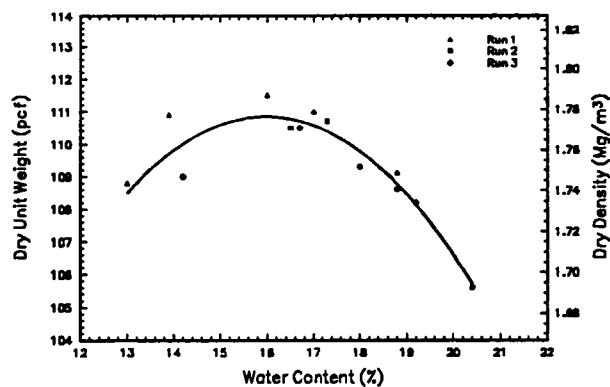
LOCATION: Hamilton County STA 578+00	
SOIL CLASSIFICATION	
AASHTO	USCS
A-6 (10)	CL

CLAY CONTENT (%)	PERCENT PASSING #200 SIEVE	ATTERBERG LIMITS			G _s	STANDARD PROCTORS	
		LL	PL	PI		max. γ_d (pcf)	opt. w% (%)
52.5	84.2	34	22	12	2.66	110.8	16.0

Grain Size Distribution
Hamilton Co. STA 578+00



Standard Proctors
Hamilton Co. Sta. 578+50



RESILIENT MODULUS RESULTS FOR TEST
Hamilton Co.#1-2 DD=1.763 MgcM W%=15.3

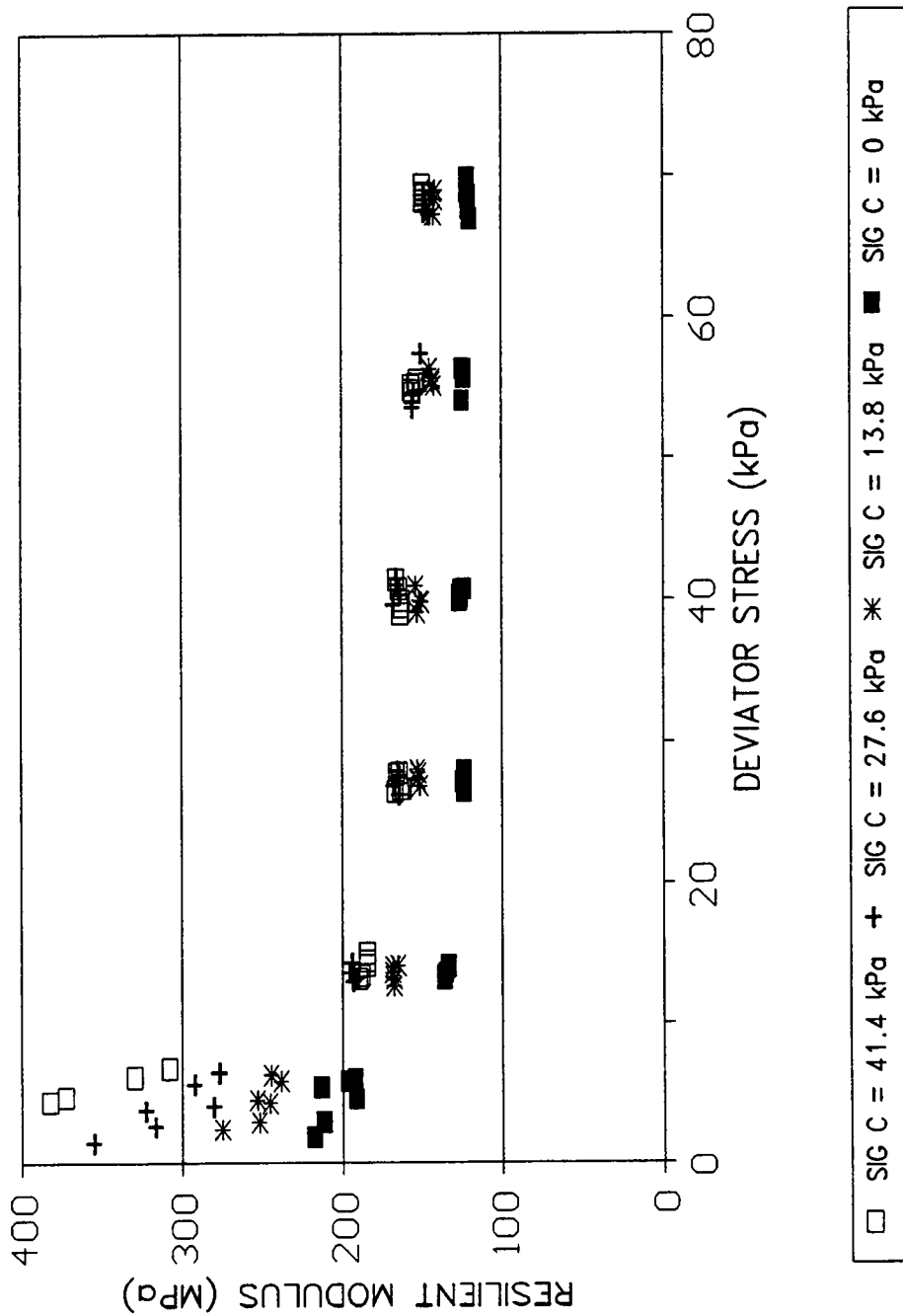


Figure A-19. Resilient Modulus versus Deviator Stress for Hamilton County Soil Compacted at Optimum Water Content and Maximum Dry Density (Standard Proctor Energy)

Resilient Modulus Results for Test Hamilton Co. #1-1a DD=1.747 Mgcmm W%=16.2

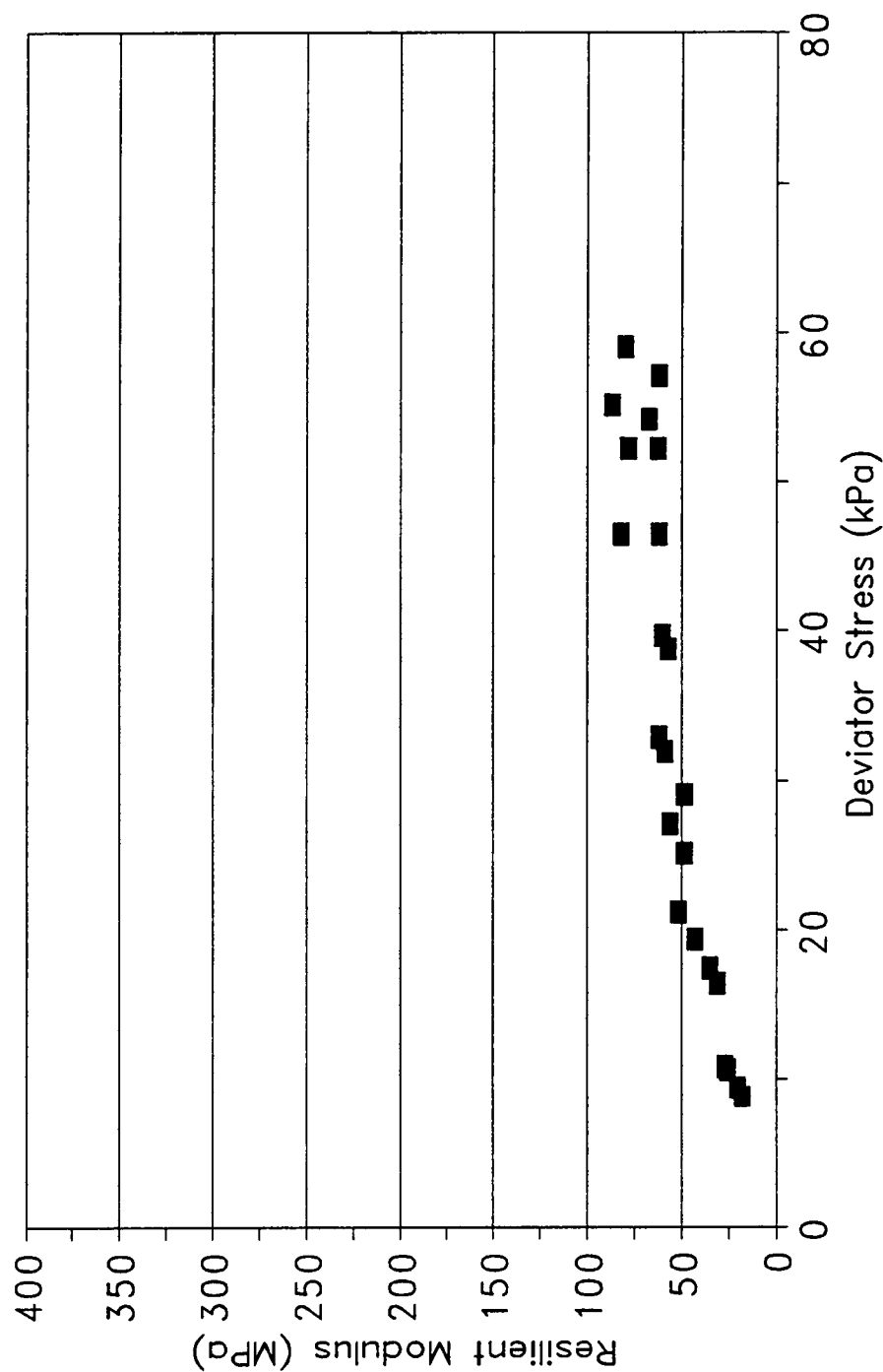


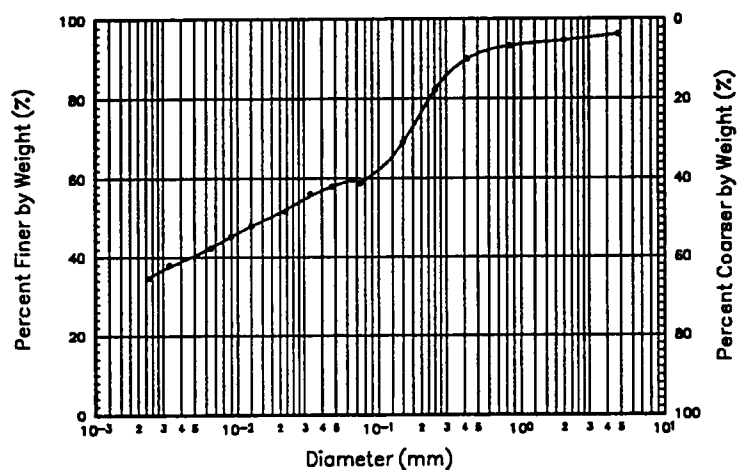
Figure A-20. Resilient Modulus versus Deviator Stress for Hamilton County Soil Compacted at Optimum W% and Maximum γ_d (Standard Proctor), using the ATM Method

VANBUREN COUNTY
STA 618
RESILIENT MODULUS PLOTS AND MODEL PARAMETERS

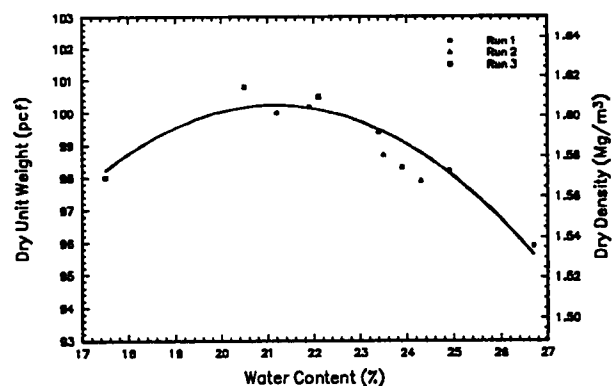
LOCATION: VanBuren County STA 618+00	
SOIL CLASSIFICATION	
AASHTO	USCS
A-7-6 (15)	CH

CLAY CONTENT (%)	PERCENT PASSING #200 SIEVE	ATTERBERG LIMITS			G_s	STANDARD PROCTORS	
		LL	PL	PI		max. γ_d (pcf)	opt. w% (%)
30.0	58.8	54	26	28	2.54	100.2	21.2

Grain Size Distribution
Van Buren Co. Sta. 618+00



STANDARD PROCTORS
VanBuren Co. STA 618+00



RESILIENT MODULUS RESULTS FOR TEST
VanBuren Co.#1-2 DD=1.597Mgcm W%= 21.3

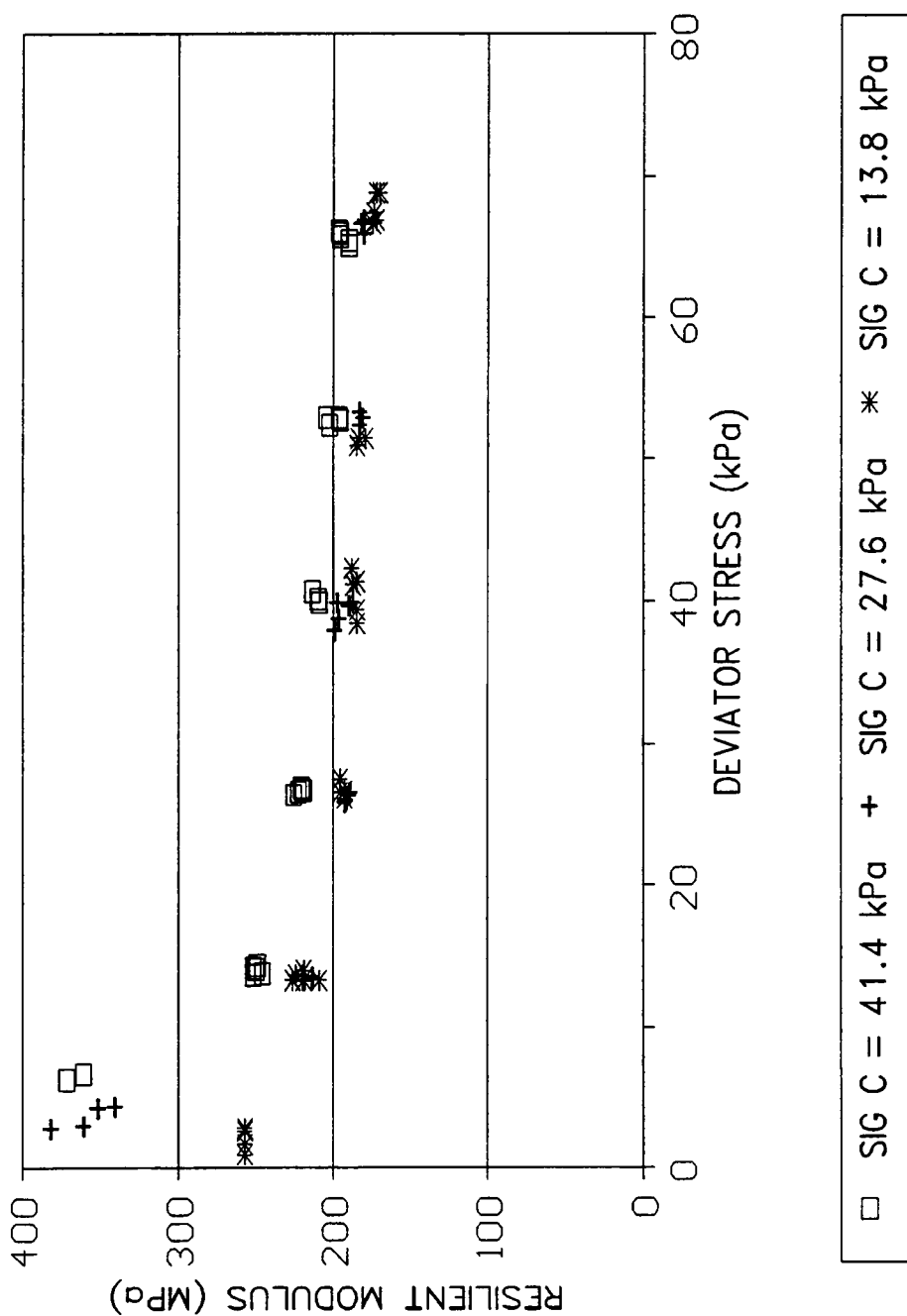


Figure A-21. Resilient Modulus versus Deviator Stress for VanBuren County Soil Compacted at Optimum Water Content and Maximum Dry Density (Standard Proctor)

Resilient Modulus Results for Test
VanBuren Co. #1-1a DD=1.595 Mgcm W%=20.3

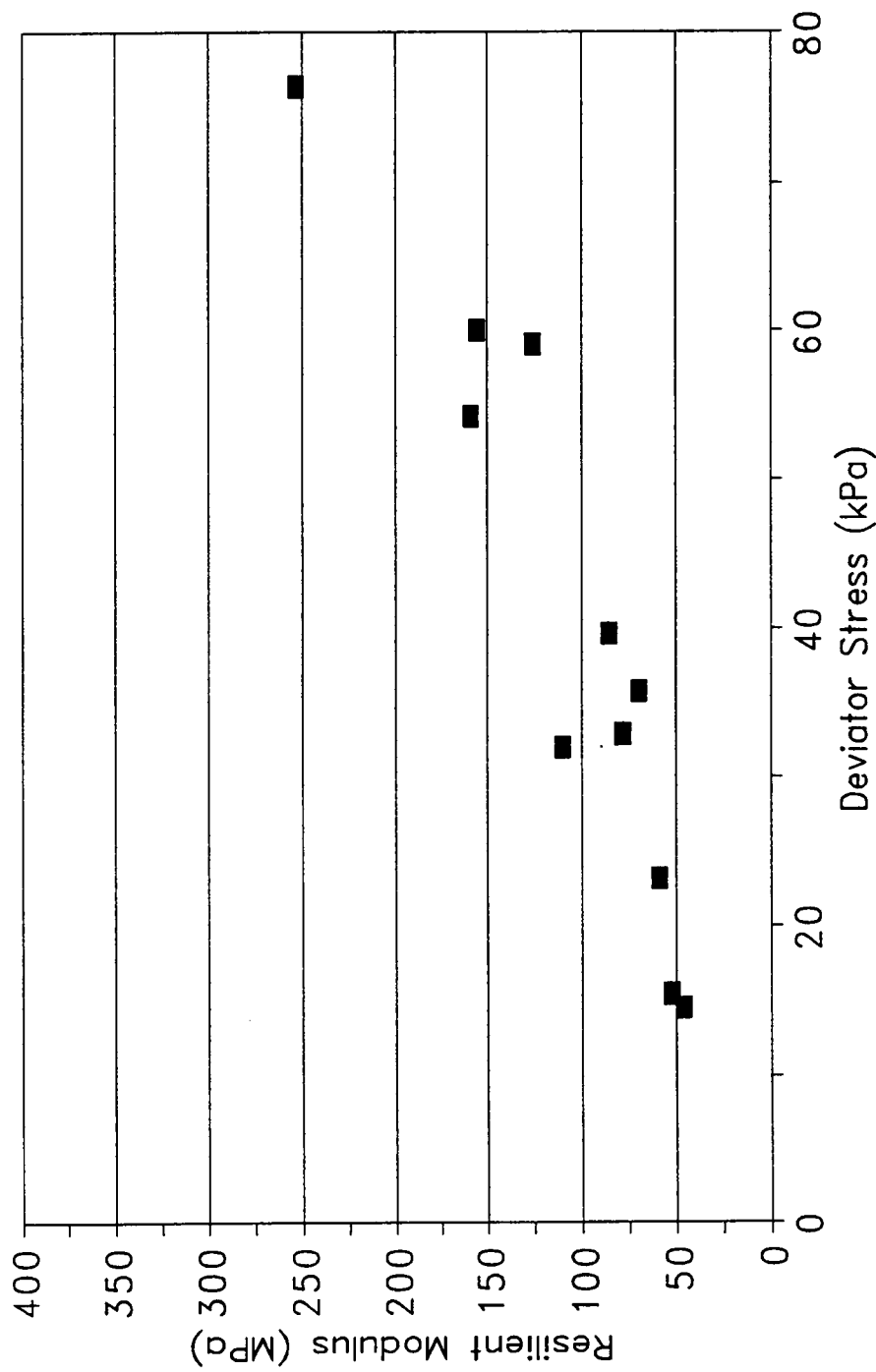


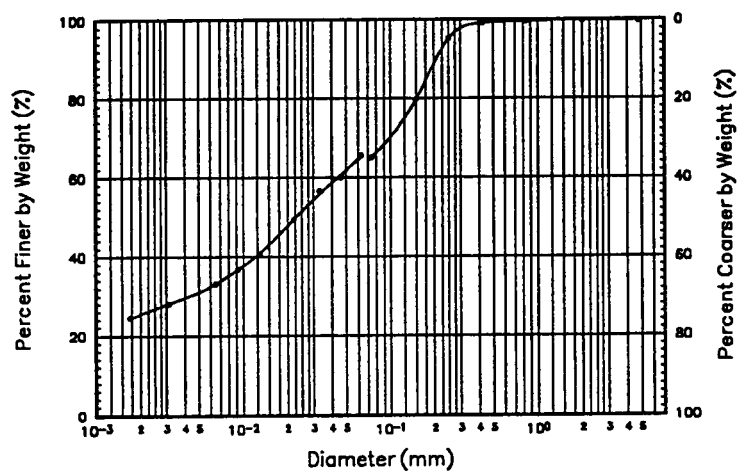
Figure A-22. Resilient Modulus versus Deviator Stress for VanBuren County Soil Compacted at Optimum W% and Maximum γ_d (Standard Proctor), using the ATM Method

WHITE COUNTY
STA 652
RESILIENT MODULUS PLOTS AND MODEL PARAMETERS

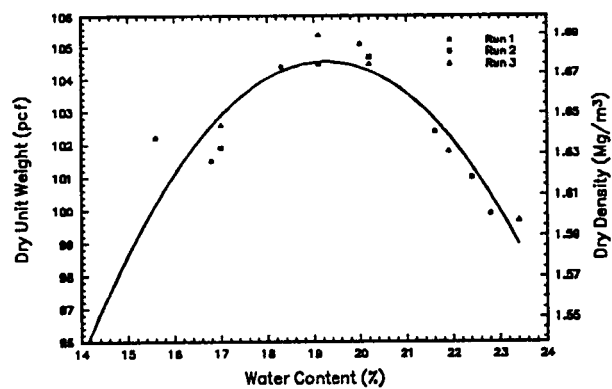
LOCATION: White County STA 652+00	
SOIL CLASSIFICATION	
AASHTO	USCS
A-6 (8)	CL

CLAY CONTENT (%)	PERCENT PASSING #200 SIEVE	ATTERBERG LIMITS			G_s	STANDARD PROCTORS	
		LL	PL	PI		max. γ_d (pcf)	opt. w% (%)
40.0	65.0	39	23	16	2.61	104.6	19.3

Grain Size Distribution
White Co. STA 652+00



STANDARD PROCTORS
White Co. STA 652+00



RESILIENT MODULUS RESULTS FOR TEST
White Co. #1-3 DD=1.673 Mgcm W%=18.8

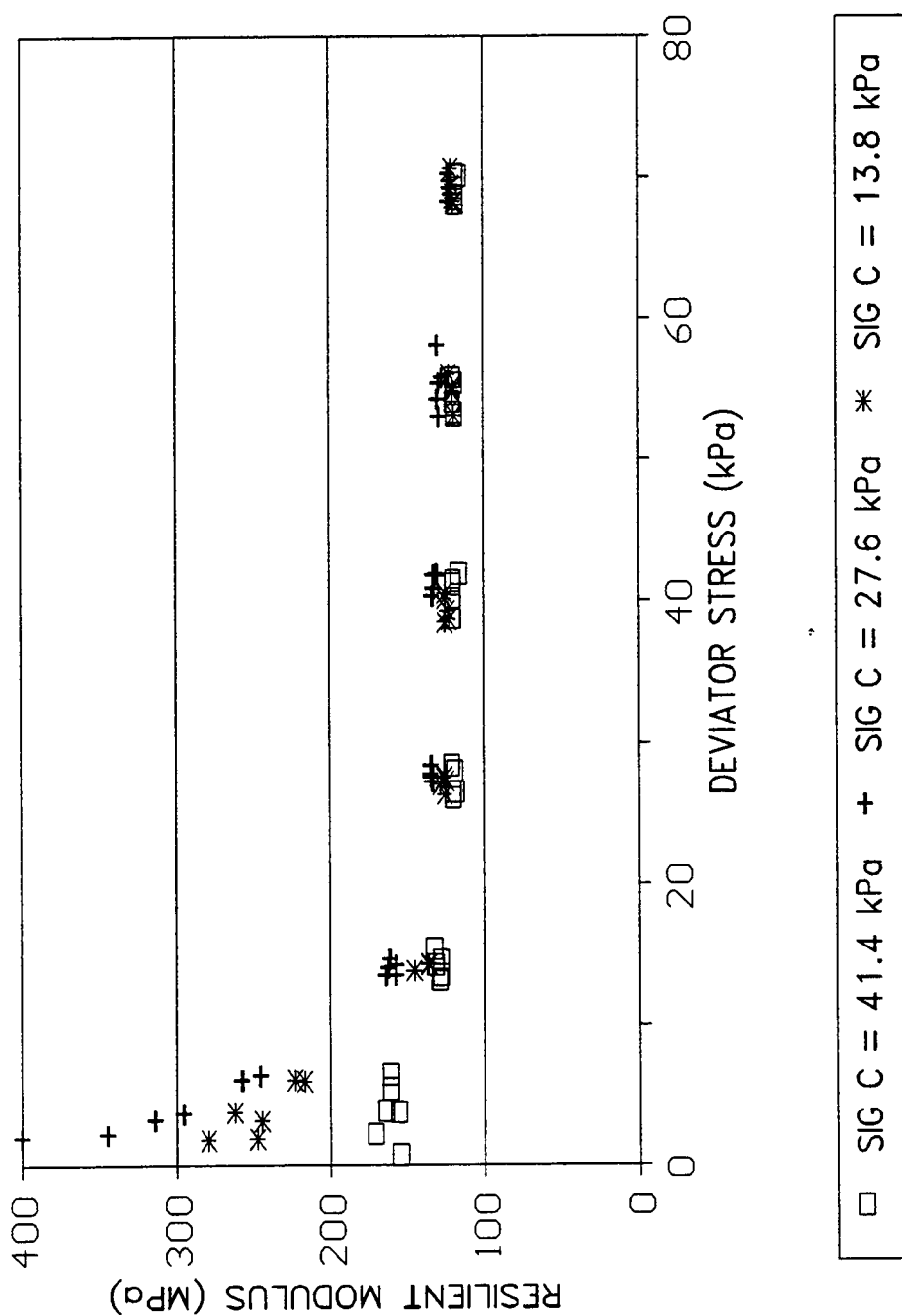


Figure A-23. Resilient Modulus versus Deviator Stress for White County Soil Compacted at Optimum Water Content and Maximum Dry Density (Standard Proctor)

Resilient Modulus Results for Test White Co.#1-1a DD=1.683 MgcM W%=19.4

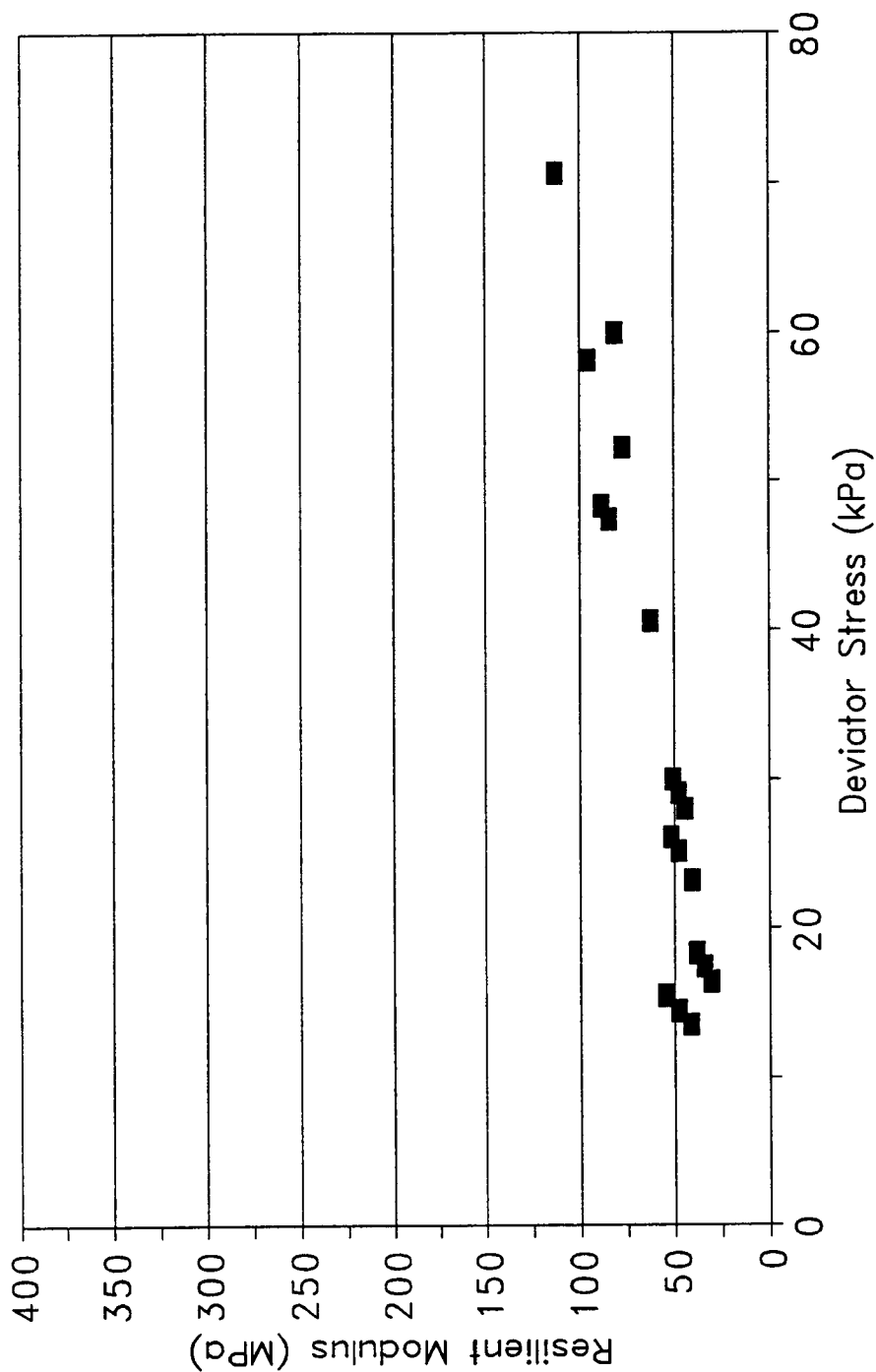


Figure A-24. Resilient Modulus versus Deviator Stress for White County Soil Compacted at Optimum W% and Maximum γ_d (Standard Proctor), using the ATM Method.

APPENDIX B

ATM DATA

CROCKETT COUNTY
STA 925

CROCKETT COUNTY #1-1A AT COMPACTION

Peak Acceleration Method

Peak Accel. g	Mass of Soil lb	Time to Peak $\times 10^{-4}$ sec	Poisson's Ratio	Drop Height mm	Falling Mass Mo (lb)	Resilient Modulus MPa	Deviator Stress kPa
58.59	4.03	3.50	0.45	5.00	2.48	47.11	14.52
58.59	4.03	3.00	0.45	5.00	2.48	47.11	14.52
50.78	4.03	5.00	0.45	5.00	2.48	35.38	12.58
58.59	4.03	4.00	0.45	5.00	2.48	47.11	14.52
70.31	4.03	5.50	0.45	10.00	2.48	33.92	17.42
74.22	4.03	4.50	0.45	10.00	2.48	37.80	18.39
78.13	4.03	5.50	0.45	10.00	2.48	41.88	19.36
85.94	4.03	5.00	0.45	10.00	2.48	50.67	21.29
97.66	4.03	4.75	0.45	15.00	2.48	43.63	24.20
105.47	4.03	4.50	0.45	15.00	2.48	50.88	26.13
101.56	4.03	5.50	0.45	15.00	2.48	47.18	25.16
105.47	4.03	3.50	0.45	15.00	2.48	50.88	26.13
125.00	4.03	5.50	0.45	20.00	2.48	53.60	30.97
117.19	4.03	4.50	0.45	20.00	2.48	47.11	29.04
121.09	4.03	4.50	0.45	20.00	2.48	50.30	30.00
121.09	4.03	4.50	0.45	20.00	2.48	50.30	30.00
152.34	4.03	4.50	0.45	30.00	2.48	53.08	37.75
156.25	4.03	5.00	0.45	30.00	2.48	55.84	38.71
152.34	4.03	5.00	0.45	30.00	2.48	53.08	37.75
152.34	4.03	5.50	0.45	30.00	2.48	53.08	37.75
179.69	4.03	4.75	0.45	40.00	2.48	55.38	44.52
179.69	4.03	5.00	0.45	40.00	2.48	55.38	44.52
179.69	4.03	4.50	0.45	40.00	2.48	55.38	44.52
191.41	4.03	4.00	0.45	40.00	2.48	62.84	47.43
210.94	4.03	4.50	0.45	50.00	2.48	61.06	52.27
222.66	4.03	5.00	0.45	50.00	2.48	68.03	55.17
207.03	4.03	3.75	0.45	50.00	2.48	58.82	51.30
210.94	4.03	5.50	0.45	50.00	2.48	61.06	52.27
250.00	4.03	3.50	0.45	60.00	2.48	71.47	61.94
250.00	4.03	4.50	0.45	60.00	2.48	71.47	61.94
253.91	4.03	3.50	0.45	60.00	2.48	73.72	62.91
246.09	4.03	5.00	0.45	60.00	2.48	69.25	60.97

Resilient Modulus Results for Test Crockett Co. #1-1a DD=1.670 Mgc/m W%=16.2

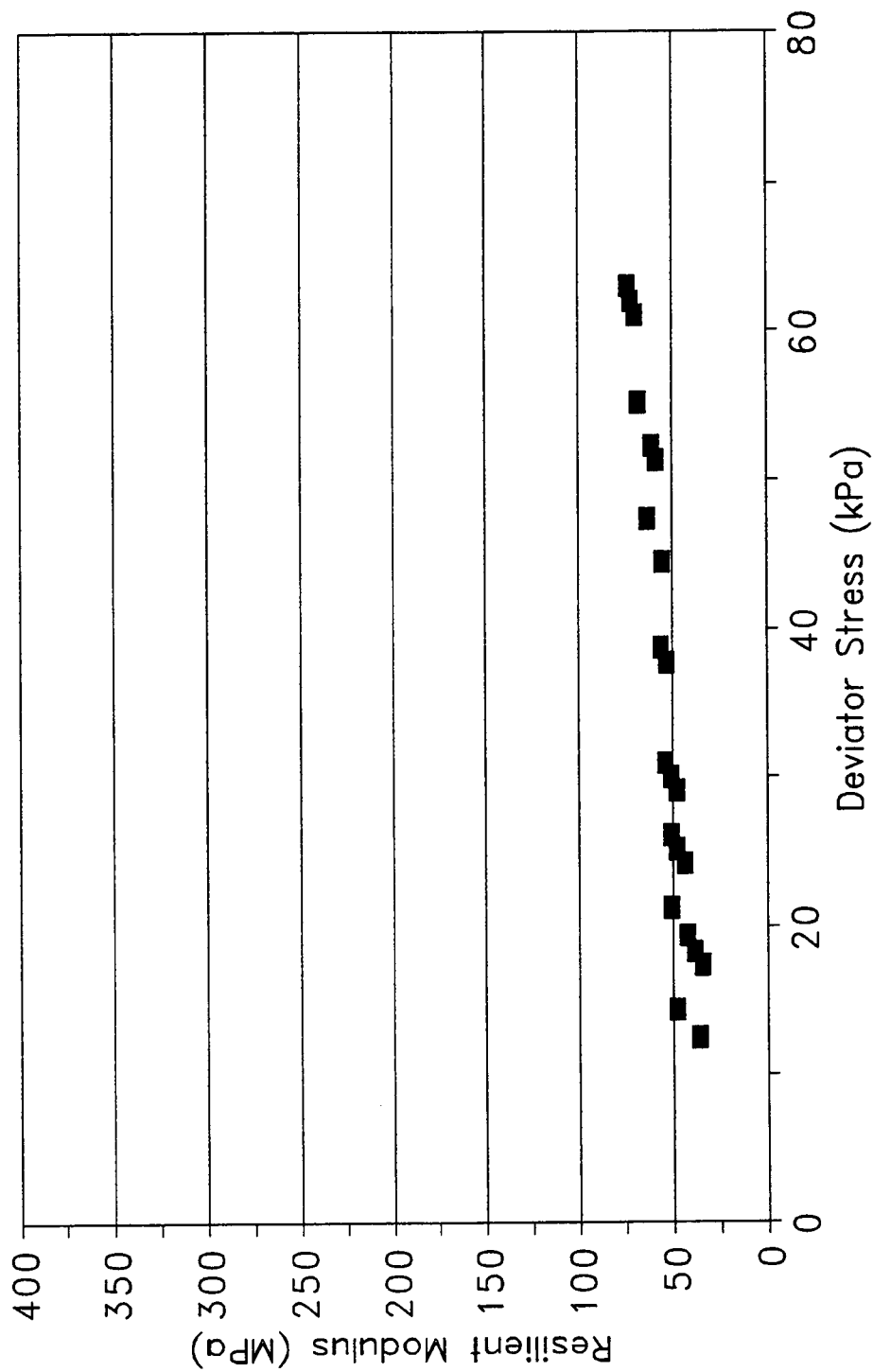


Figure B-1. ATM Data for Crockett County at Optimum Water Content and Maximum Dry Density

Axial Stress vs. Axial Strain Crockett County Soil at Opt. W% and Max DD

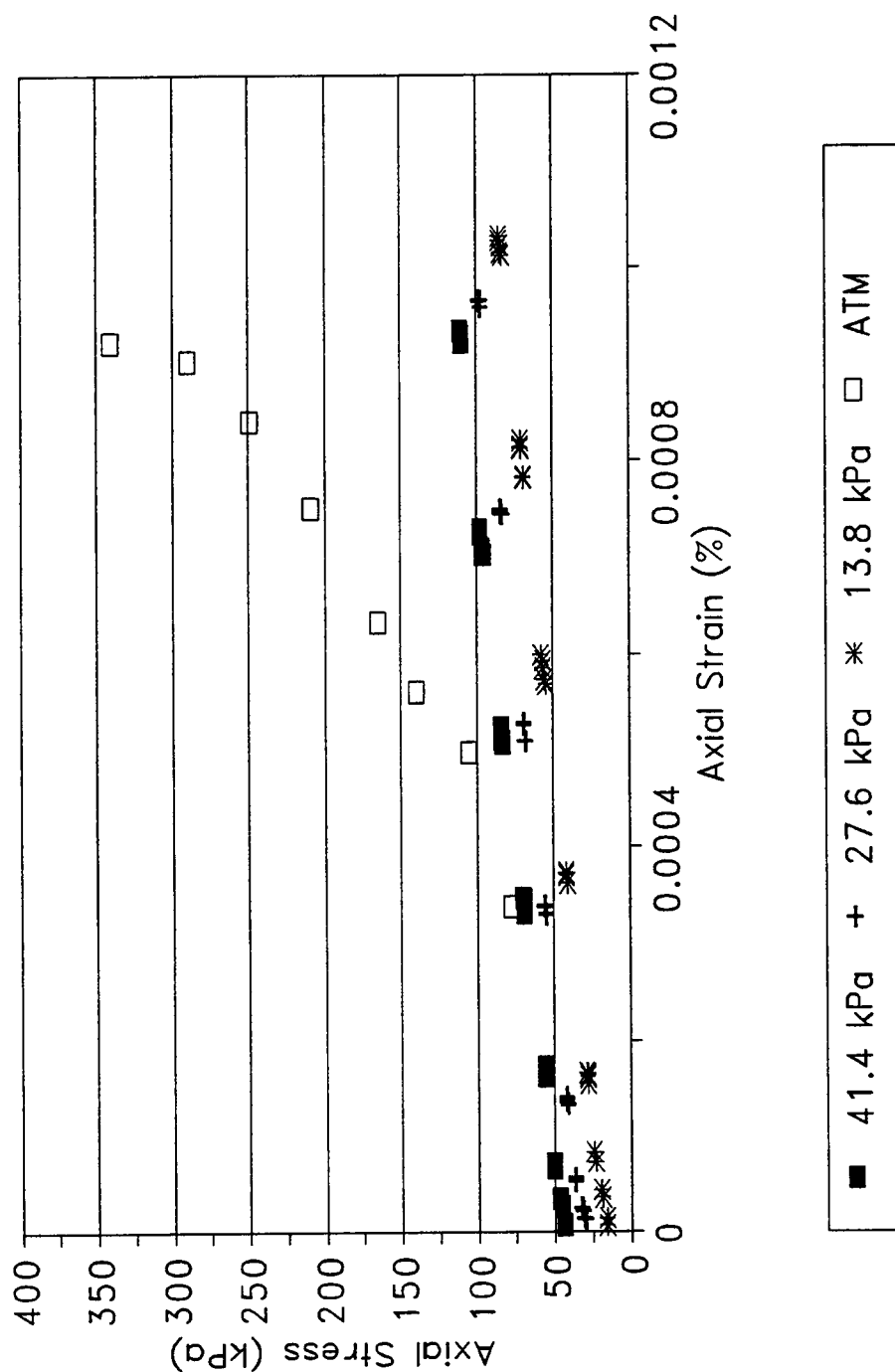


Figure B-2. Stress versus Strain Curves for ATM and Cyclic Triaxial on Crockett County Soil at Optimum Water Content and Maximum Dry Density

CROCKETT COUNTY #1-1A AFTER FIRST SATURATION

Peak Acceleration Method

Peak Accel. g	Mass of Soil lb	Time to Peak x10 ⁻⁴ sec	Poisson's Ratio	Drop Height mm	Falling Mass Mo (lb)	Resilient Modulus MPa	Deviator Stress kPa
54.69	4.09	5.20	0.45	5.00	2.48	41.64	13.55
50.78	4.09	4.60	0.45	5.00	2.48	35.90	12.58
58.59	4.09	5.00	0.45	5.00	2.48	47.79	14.52
54.69	4.09	4.40	0.45	5.00	2.48	41.64	13.55
89.84	4.09	4.60	0.45	10.00	2.48	56.18	22.26
85.94	4.09	4.80	0.45	10.00	2.48	51.41	21.29
89.84	4.09	4.60	0.45	10.00	2.48	56.18	22.26
89.84	4.09	4.40	0.45	10.00	2.48	56.18	22.26
109.38	4.09	4.40	0.45	15.00	2.48	55.52	27.10
109.38	4.09	4.20	0.45	15.00	2.48	55.52	27.10
117.19	4.09	4.40	0.45	15.00	2.48	63.73	29.04
121.09	4.09	4.60	0.45	15.00	2.48	68.04	30.00
136.72	4.09	4.40	0.45	20.00	2.48	65.06	33.88
136.72	4.09	4.60	0.45	20.00	2.48	65.06	33.88
140.63	4.09	4.20	0.45	20.00	2.48	68.83	34.84
136.72	4.09	4.60	0.45	20.00	2.48	65.06	33.88
183.59	4.09	4.20	0.45	30.00	2.48	78.21	45.49
183.59	4.09	4.00	0.45	30.00	2.48	78.21	45.49
179.69	4.09	4.80	0.45	30.00	2.48	74.92	44.52
183.59	4.09	4.40	0.45	30.00	2.48	78.21	45.49
218.75	4.09	4.00	0.45	40.00	2.48	83.27	54.20
210.94	4.09	4.20	0.45	40.00	2.48	77.43	52.27
222.66	4.09	4.60	0.45	40.00	2.48	86.27	55.17
214.84	4.09	4.20	0.45	40.00	2.48	80.32	53.23
238.28	4.09	4.20	0.45	50.00	2.48	79.04	59.04
238.28	4.09	4.20	0.45	50.00	2.48	79.04	59.04
238.28	4.09	4.20	0.45	50.00	2.48	79.04	59.04
226.56	4.09	4.20	0.45	50.00	2.48	71.46	56.14
234.38	4.09	3.80	0.45	60.00	2.48	63.73	58.07
257.81	4.09	4.40	0.45	60.00	2.48	77.11	63.88
273.44	4.09	4.20	0.45	60.00	2.48	86.74	67.75
250.00	4.09	4.50	0.45	60.00	2.48	72.51	61.94

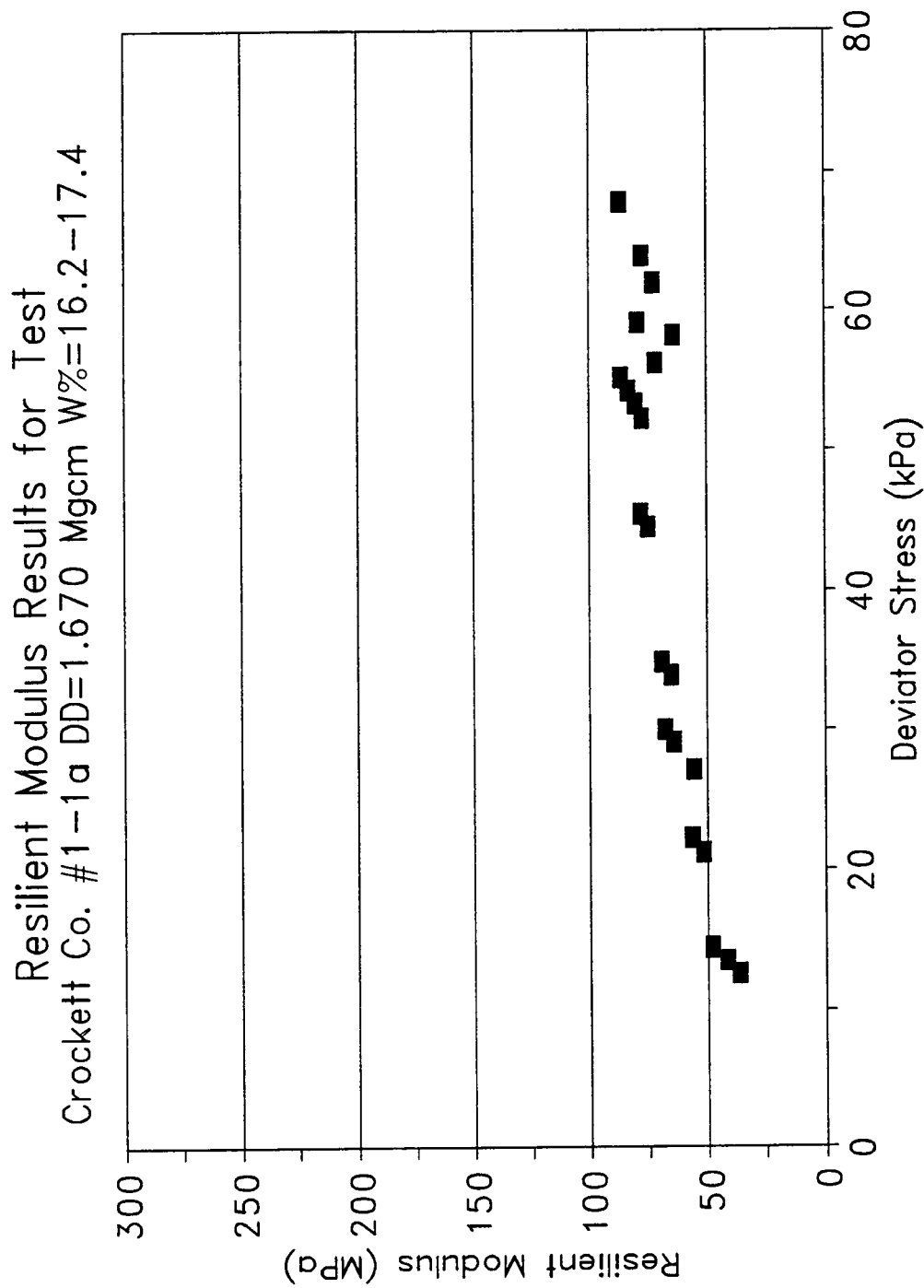


Figure B-3. ATM Data for Crockett County at Compacted at Optimum Water Content and Maximum Dry Density, then Saturated from S=75.3% to 80.9%.

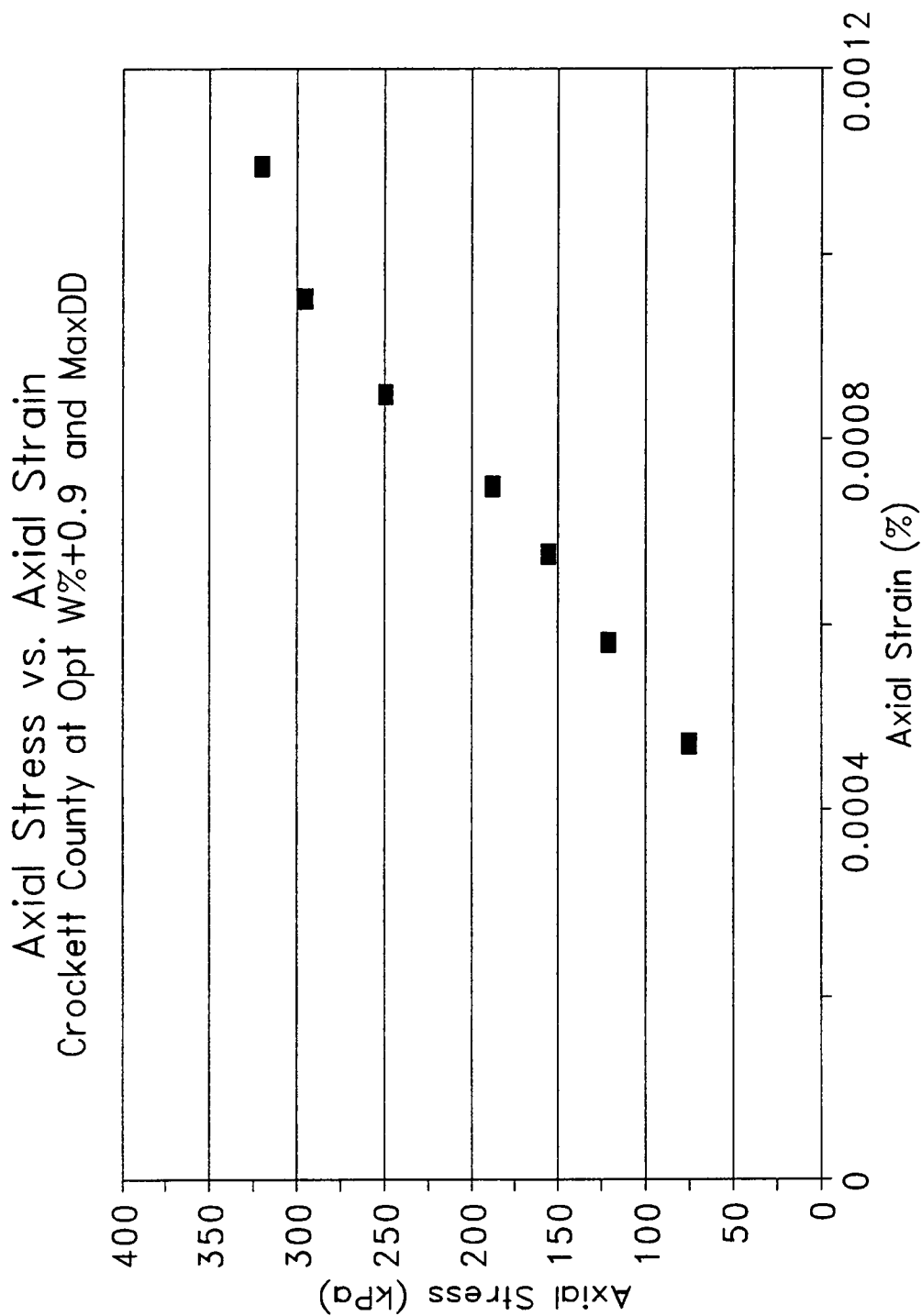


Figure B-4. Stress versus Strain Curves for Crockett County at Compacted at Optimum Water Content and Maximum Dry Density, then Saturated from S=75.3% to 80.9%.

CROCKETT COUNTY #1-1A AFTER SECOND SATURATION

Peak Acceleration Method

Peak Accel. g	Mass of Soil lb	Time to Peak $\times 10^{-4}$ sec	Poisson's Ratio	Drop Height mm	Falling Mass Mo (lb)	Resilient Modulus MPa	Deviator Stress kPa
39.06	4.13	6.80	0.45	5.00	2.48	21.43	9.68
42.97	4.13	5.40	0.45	5.00	2.48	25.93	10.65
30.86	4.13		0.45	5.00	2.48	13.38	7.65
32.03	4.13		0.45	5.00	2.48	14.41	7.94
62.50	4.13	7.20	0.45	10.00	2.48	27.43	15.49
58.59	4.13	6.60	0.45	10.00	2.48	24.11	14.52
58.59	4.13	6.40	0.45	10.00	2.48	24.11	14.52
62.50	4.13	7.40	0.45	10.00	2.48	27.43	15.49
85.94	4.13	6.20	0.45	15.00	2.48	34.58	21.29
85.94	4.13	6.20	0.45	15.00	2.48	34.58	21.29
82.03	4.13	6.60	0.45	15.00	2.48	31.50	20.32
82.03	4.13	6.00	0.45	15.00	2.48	31.50	20.32
97.66	4.13	6.40	0.45	20.00	2.48	33.49	24.20
109.38	4.13	6.60	0.45	20.00	2.48	42.01	27.10
105.47	4.13	6.20	0.45	20.00	2.48	39.06	26.13
109.38	4.13	6.40	0.45	20.00	2.48	42.01	27.10
140.63	4.13	5.60	0.45	30.00	2.48	46.29	34.84
136.72	4.13	5.60	0.45	30.00	2.48	43.76	33.88
140.63	4.13	6.40	0.45	30.00	2.48	46.29	34.84
136.72	4.13	5.80	0.45	30.00	2.48	43.76	33.88
164.06	4.13	5.00	0.45	40.00	2.48	47.25	40.65
167.97	4.13	5.60	0.45	40.00	2.48	49.53	41.62
164.06	4.13	5.70	0.45	40.00	2.48	47.25	40.65
167.97	4.13	5.60	0.45	40.00	2.48	49.53	41.62
179.69	4.13	5.60	0.45	50.00	2.48	45.35	44.52
187.50	4.13	5.80	0.45	50.00	2.48	49.38	46.46
187.50	4.13	5.40	0.45	50.00	2.48	49.38	46.46
175.78	4.13	5.80	0.45	50.00	2.48	43.40	43.55
199.22	4.13	5.20	0.45	60.00	2.48	46.45	49.36
195.31	4.13	5.00	0.45	60.00	2.48	44.65	48.39
191.41	4.13	5.00	0.45	60.00	2.48	42.88	47.43
191.41	4.13	5.30	0.45	60.00	2.48	42.88	47.43

Resilient Modulus Results for Test Crockett Co. #1-1a DD=1.670 Mgcmm W%=16.2-18.8

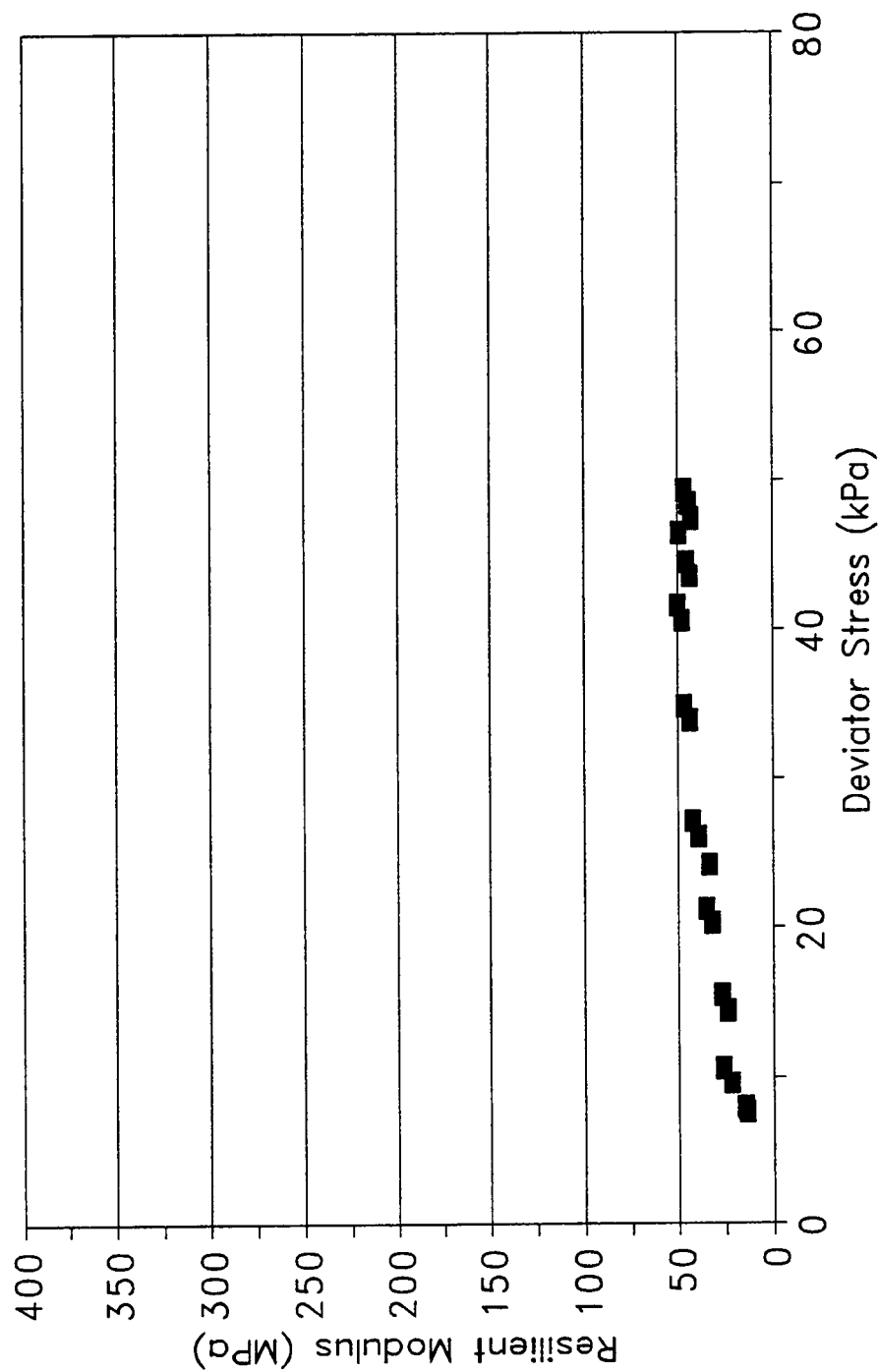


Figure B-5. ATM Data for Crockett County at Compacted at Optimum Water Content and Maximum Dry Density, then Saturated from S=75.3% to 87.4%

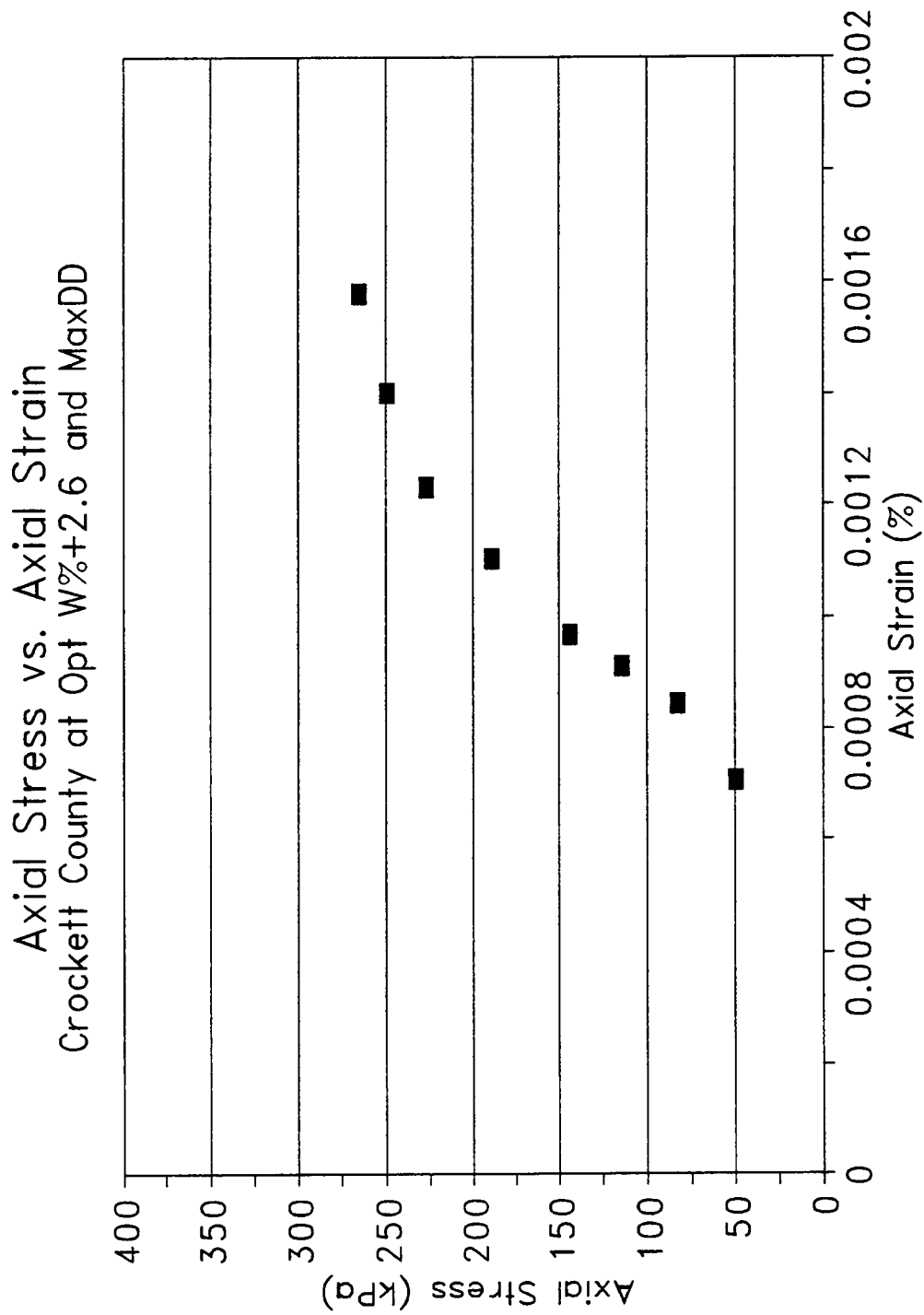


Figure B-6. Stress versus Strain Curves for Crockett County at Compacted at Optimum Water Content and Maximum Dry Density, then Saturated from S=75.3% to 80.9%.

GILES COUTY
STA 270

GILES COUNTY #4-1 AT COMPACTION

Peak Acceleration Method

Peak Accel. g	Mass of Soil lb	Time to Peak x10 ⁻⁴ sec	Poisson's Ratio	Drop Height mm	Falling Mass Mo (lb)	Resilient Modulus MPa	Deviator Stress kPa
37.89	4.00	0.00	0.45	5.00	2.48	19.57	9.39
44.30	4.00	0.00	0.45	5.00	2.48	26.75	10.98
46.88	4.00	0.00	0.45	5.00	2.48	29.96	11.62
82.03	4.00	0.00	0.45	10.00	2.48	45.86	20.32
78.13	4.00	0.00	0.45	10.00	2.48	41.60	19.36
78.13	4.00	0.00	0.45	10.00	2.48	41.60	19.36
85.94	4.00	0.00	0.45	10.00	2.48	50.34	21.29
78.13	4.00	0.00	0.45	10.00	2.48	41.60	19.36
105.50	4.00	0.00	0.45	15.00	2.48	50.57	26.14
101.56	4.00	0.00	0.45	15.00	2.48	46.87	25.16
97.66	4.00	0.00	0.45	15.00	2.48	43.33	24.20
89.84	4.00	0.00	0.45	15.00	2.48	36.67	22.26
136.70	4.00	0.00	0.45	20.00	2.48	63.68	33.87
144.50	4.00	0.00	0.45	20.00	2.48	71.15	35.80
140.63	4.00	0.00	0.45	20.00	2.48	67.39	34.84
136.72	4.00	0.00	0.45	20.00	2.48	63.70	33.88
160.16	4.00	0.00	0.45	30.00	2.48	58.28	39.68
175.78	4.00	0.00	0.45	30.00	2.48	70.20	43.55
195.31	4.00	0.00	0.45	40.00	2.48	65.00	48.39
187.50	4.00	0.00	0.45	40.00	2.48	59.90	46.46
210.94	4.00	0.00	0.45	40.00	2.48	75.81	52.27
207.03	4.00	0.00	0.45	40.00	2.48	73.03	51.30
203.13	4.00	0.00	0.45	40.00	2.48	70.30	50.33
222.66	4.00	0.00	0.45	50.00	2.48	67.58	55.17
207.03	4.00	0.00	0.45	50.00	2.48	58.42	51.30
234.38	4.00	0.00	0.45	50.00	2.48	74.88	58.07

Resilient Modulus Results for Test Giles Co.#4-1 DD=1.600 Mgcmm W%=23.8

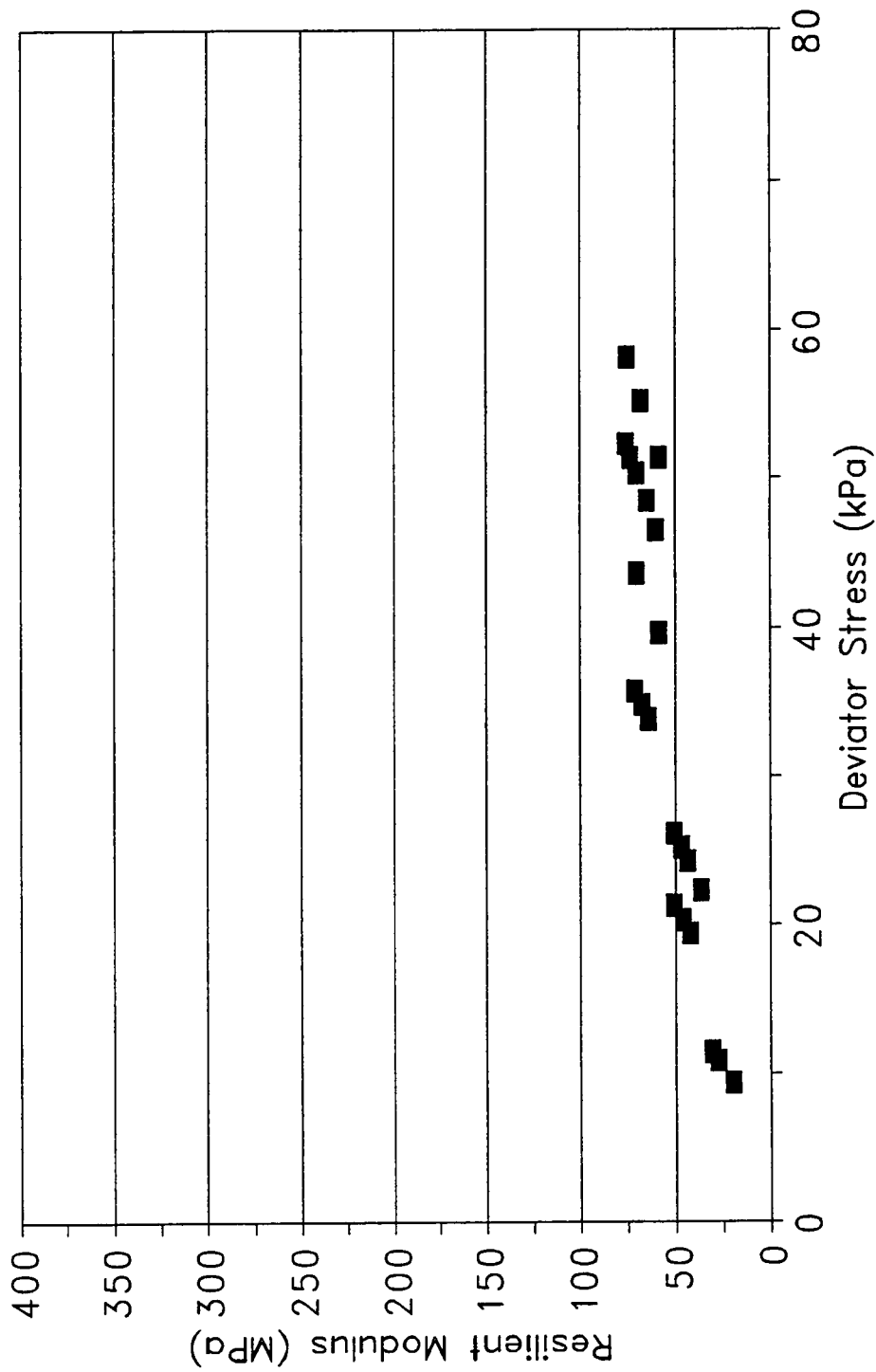


Figure B-7. ATM Data for Giles County at Optimum Water Content and Maximum Dry Density

Axial Stress vs. Axial Strain Giles County at Opt. W% and Max DD

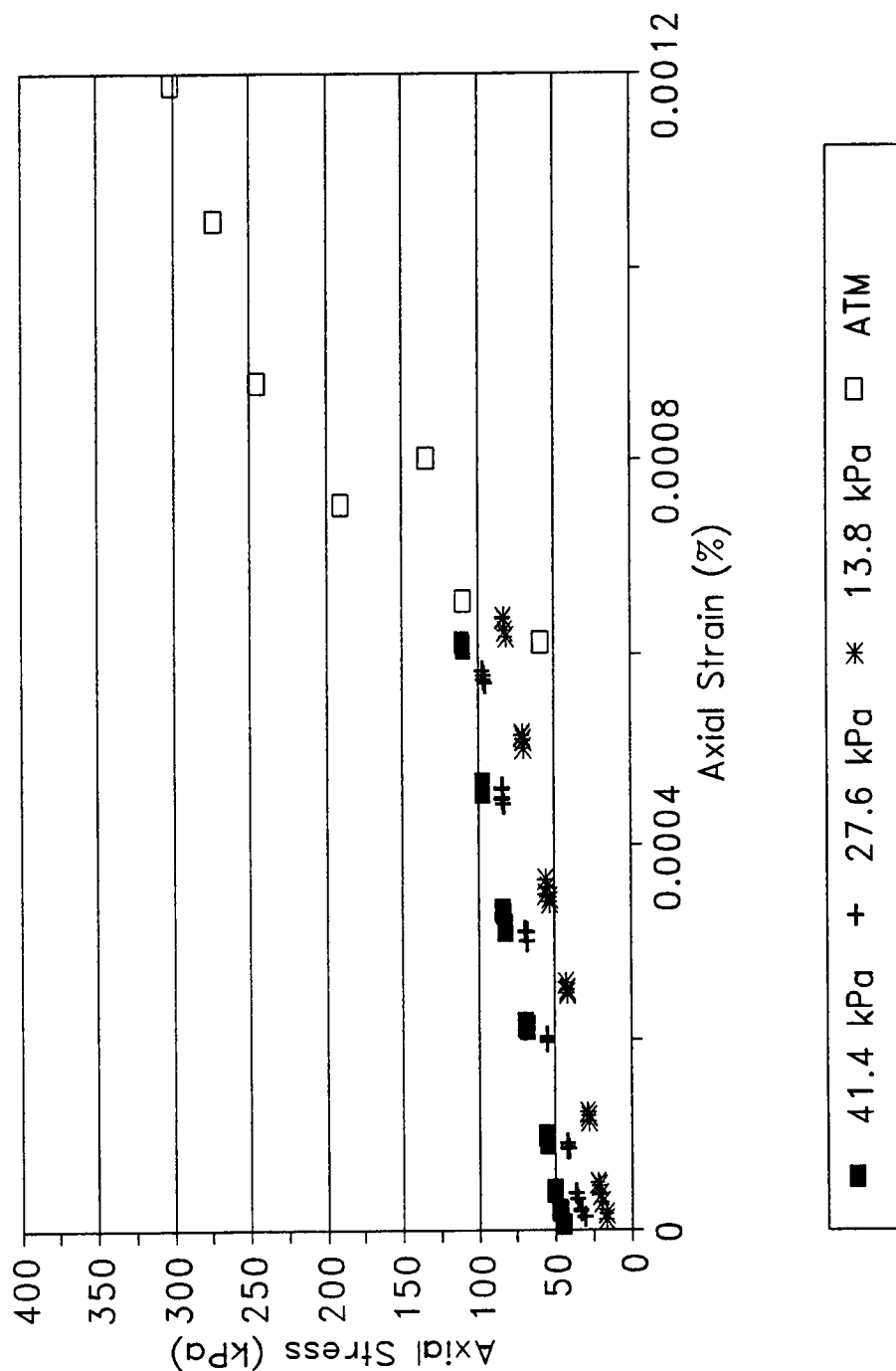


Figure B-8. Stress versus Strain Curves for ATM and Cyclic Triaxial on Giles County Soil at Optimum Water Content and Maximum Dry Density

GILES COUNTY #4-1 AFTER FIRST SATURATION

Peak Acceleration Method

Peak Accel. g	Mass of Soil lb	Time to Peak $\times 10^{-4}$ sec	Poisson's Ratio	Drop Height mm	Falling Mass Mo (lb)	Resilient Modulus MPa	Deviator Stress kPa
74.20	4.01	4.00	0.45	5.00	2.48	75.27	18.38
50.78	4.01	4.25	0.45	5.00	2.48	35.25	12.58
82.03	4.01	4.50	0.45	5.00	2.48	91.99	20.32
62.50	4.01	4.50	0.45	5.00	2.48	53.40	15.49
105.47	4.01	4.50	0.45	10.00	2.48	76.03	26.13
97.66	4.01	5.00	0.45	10.00	2.48	65.19	24.20
121.09	4.01	4.50	0.45	10.00	2.48	100.22	30.00
125.00	4.01	3.50	0.45	10.00	2.48	106.80	30.97
144.53	4.01	5.50	0.45	15.00	2.48	95.19	35.81
140.60	4.01	4.50	0.45	15.00	2.48	90.08	34.84
136.70	4.01	4.00	0.45	15.00	2.48	85.15	33.87
140.63	4.01	3.75	0.45	15.00	2.48	90.12	34.84
179.69	4.01	3.50	0.45	20.00	2.48	110.35	44.52
183.59	4.01	4.50	0.45	20.00	2.48	115.19	45.49
183.59	4.01	5.50	0.45	20.00	2.48	115.19	45.49
175.78	4.01	5.25	0.45	20.00	2.48	105.60	43.55
214.84	4.01	3.75	0.45	30.00	2.48	105.16	53.23
207.03	4.01	5.00	0.45	30.00	2.48	97.66	51.30
214.84	4.01	4.50	0.45	30.00	2.48	105.16	53.23
210.94	4.01	5.00	0.45	30.00	2.48	101.38	52.27
246.09	4.01	3.50	0.45	40.00	2.48	103.49	60.97
234.38	4.01	4.00	0.45	40.00	2.48	93.87	58.07
253.91	4.01	3.50	0.45	40.00	2.48	110.17	62.91
250.00	4.01	5.50	0.45	40.00	2.48	106.80	61.94
281.25	4.01	3.00	0.45	50.00	2.48	108.14	69.69
257.81	4.01	3.50	0.45	50.00	2.48	90.86	63.88
265.62	4.01	4.50	0.45	50.00	2.48	96.45	65.81
281.25	4.01	3.50	0.45	50.00	2.48	108.14	69.69
281.25	4.01	4.50	0.45	60.00	2.48	90.11	69.69
285.16	4.01	3.75	0.45	60.00	2.48	92.64	70.65
289.06	4.01	3.00	0.45	60.00	2.48	95.19	71.62
285.16	4.01	4.50	0.45	60.00	2.48	92.64	70.65

Resilient Modulus Results for Test
Giles Co. #4-1 DD=1.600Mgcm W%=23.3-24.1

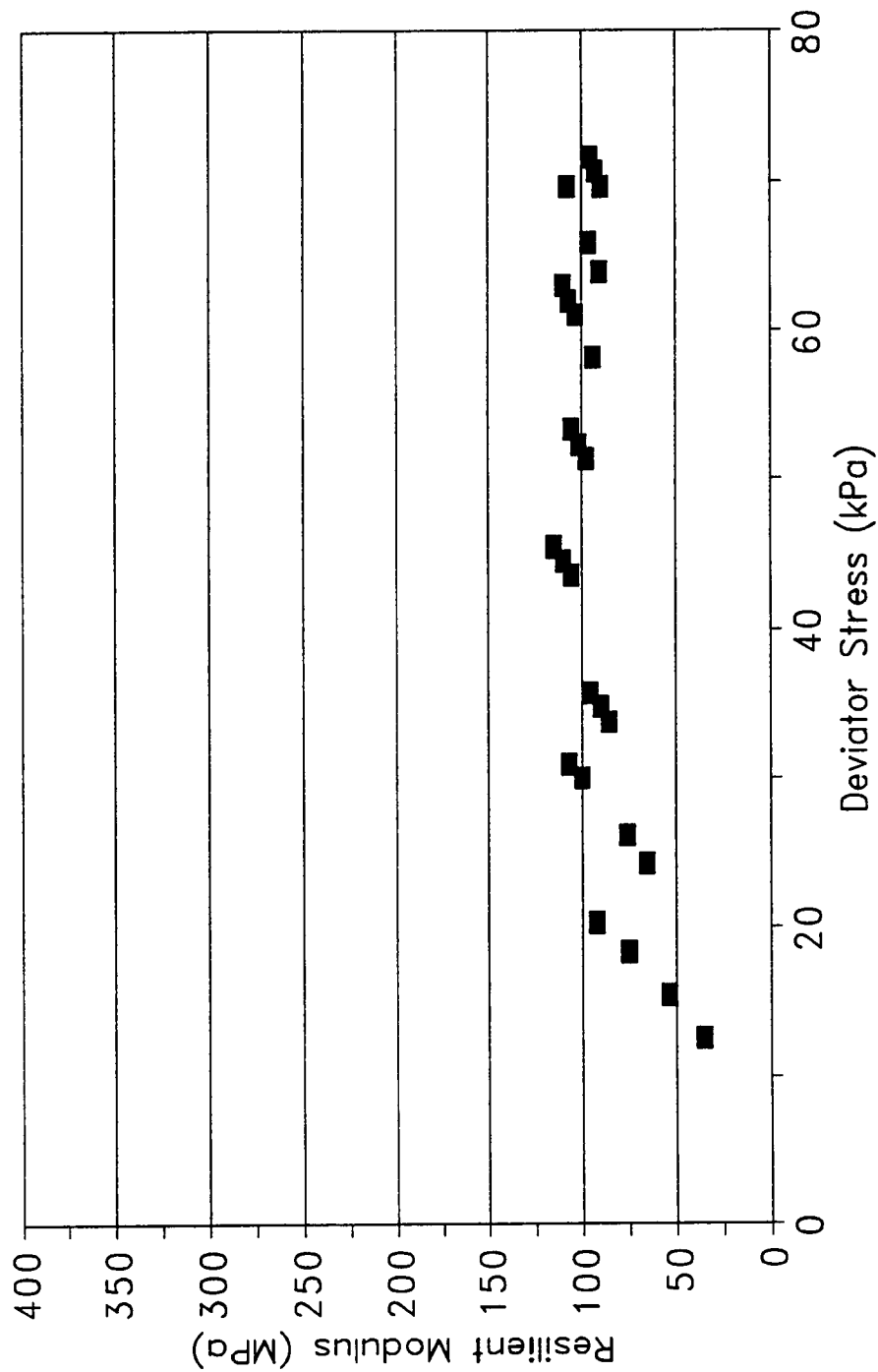


Figure B-9. ATM Data for Giles County at Compacted at Optimum Water Content and Maximum Dry Density, then Saturated from W% = 23.8% to 24.1%

Axial Stress vs. Axial Strain Giles County at Opt. W%+0.3% and Max DD

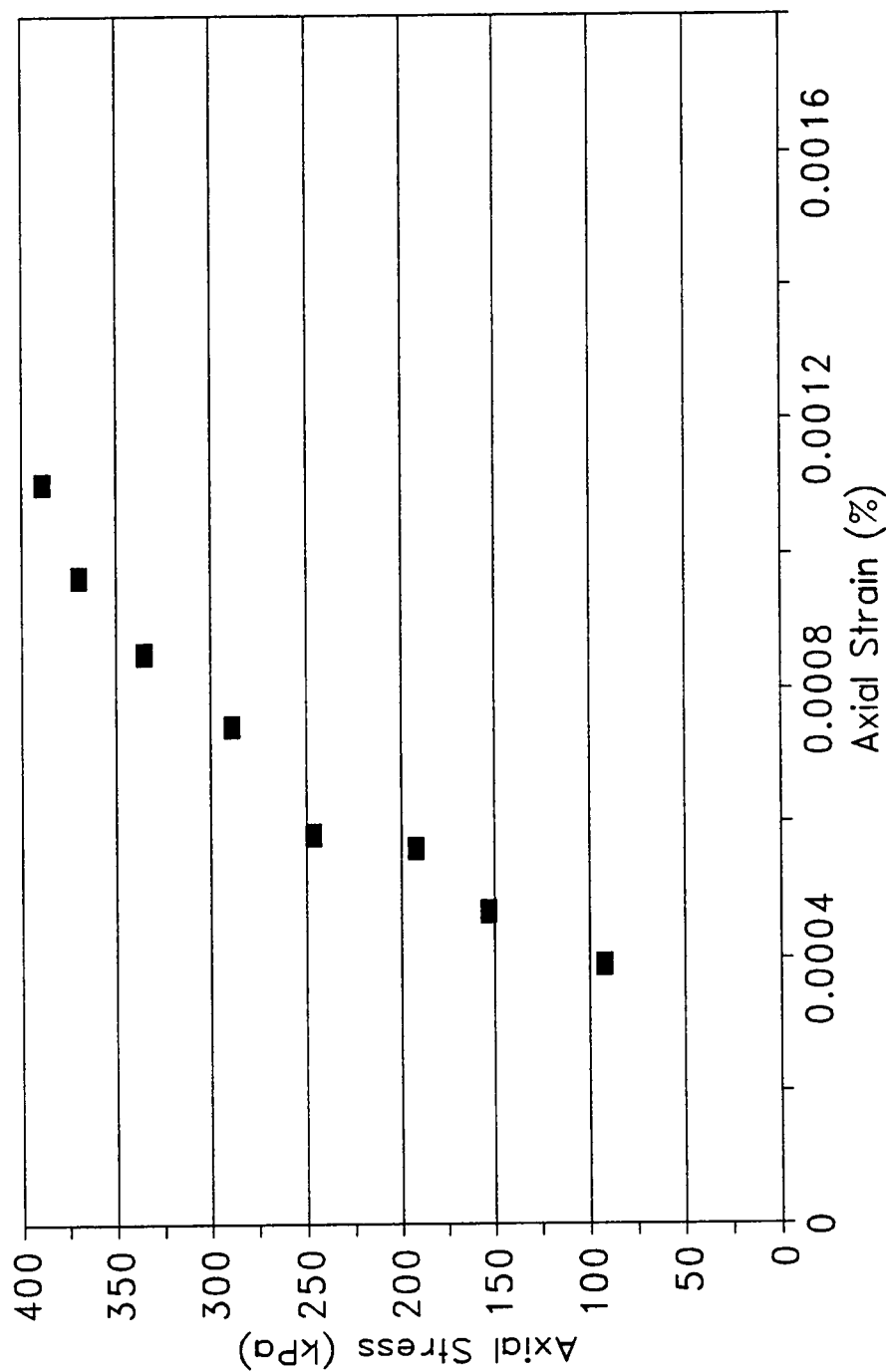


Figure B-10. Stress versus Strain Curves for Giles County at Compacted at Optimum Water Content and Maximum Dry Density, then Saturated from W% = 23.8% to 24.1%

GILES COUNTY #4-1 AFTER SECOND SATURATION

Peak Acceleration Method

Peak Accel. g	Mass of Soil lb	Time to Peak $\times 10^{-4}$ sec	Poisson's Ratio	Drop Height mm	Falling Mass Mo (lb)	Resilient Modulus MPa	Deviator Stress kPa
78.13	4.03	4.80	0.45	5.00	2.48	83.91	19.36
74.22	4.03	4.20	0.45	5.00	2.48	75.72	18.39
70.31	4.03	4.00	0.45	5.00	2.48	67.95	17.42
70.31	4.03	4.40	0.45	5.00	2.48	67.95	17.42
109.38	4.03	4.60	0.45	10.00	2.48	82.23	27.10
113.28	4.03	5.00	0.45	10.00	2.48	88.20	28.07
105.47	4.03	4.60	0.45	10.00	2.48	76.45	26.13
109.38	4.03	5.40	0.45	10.00	2.48	82.23	27.10
148.44	4.03	4.60	0.45	15.00	2.48	100.96	36.78
148.44	4.03	4.20	0.45	15.00	2.48	100.96	36.78
140.63	4.03	4.60	0.45	15.00	2.48	90.62	34.84
140.63	4.03	4.60	0.45	15.00	2.48	90.62	34.84
160.16	4.03	4.10	0.45	20.00	2.48	88.15	39.68
164.06	4.03	4.00	0.45	20.00	2.48	92.50	40.65
167.97	4.03	3.90	0.45	20.00	2.48	96.96	41.62
167.97	4.03	4.60	0.45	20.00	2.48	96.96	41.62
203.13	4.03	4.40	0.45	30.00	2.48	94.53	50.33
207.03	4.03	4.20	0.45	30.00	2.48	98.19	51.30
199.22	4.03	4.40	0.45	30.00	2.48	90.93	49.36
203.13	4.03	4.30	0.45	30.00	2.48	94.53	50.33
242.18	4.03	4.00	0.45	40.00	2.48	100.78	60.01
238.28	4.03	4.40	0.45	40.00	2.48	97.56	59.04
238.28	4.03	4.80	0.45	40.00	2.48	97.56	59.04
242.19	4.03	4.00	0.45	40.00	2.48	100.79	60.01
265.63	4.03	3.90	0.45	50.00	2.48	96.99	65.82
265.63	4.03	4.20	0.45	50.00	2.48	96.99	65.82
273.44	4.03	4.00	0.45	50.00	2.48	102.78	67.75
273.44	4.03	3.80	0.45	50.00	2.48	102.78	67.75
253.91	4.03	3.80	0.45	55.00	2.48	80.56	62.91
257.81	4.03	5.00	0.45	55.00	2.48	83.06	63.88
257.81	4.03	4.60	0.45	55.00	2.48	83.06	63.88
277.34	4.03	4.40	0.45	55.00	2.48	96.12	68.72

Resilient Modulus Results for Test
GilesCo. #4-1a DD=1.600Mgcm W%=23.8-24.7

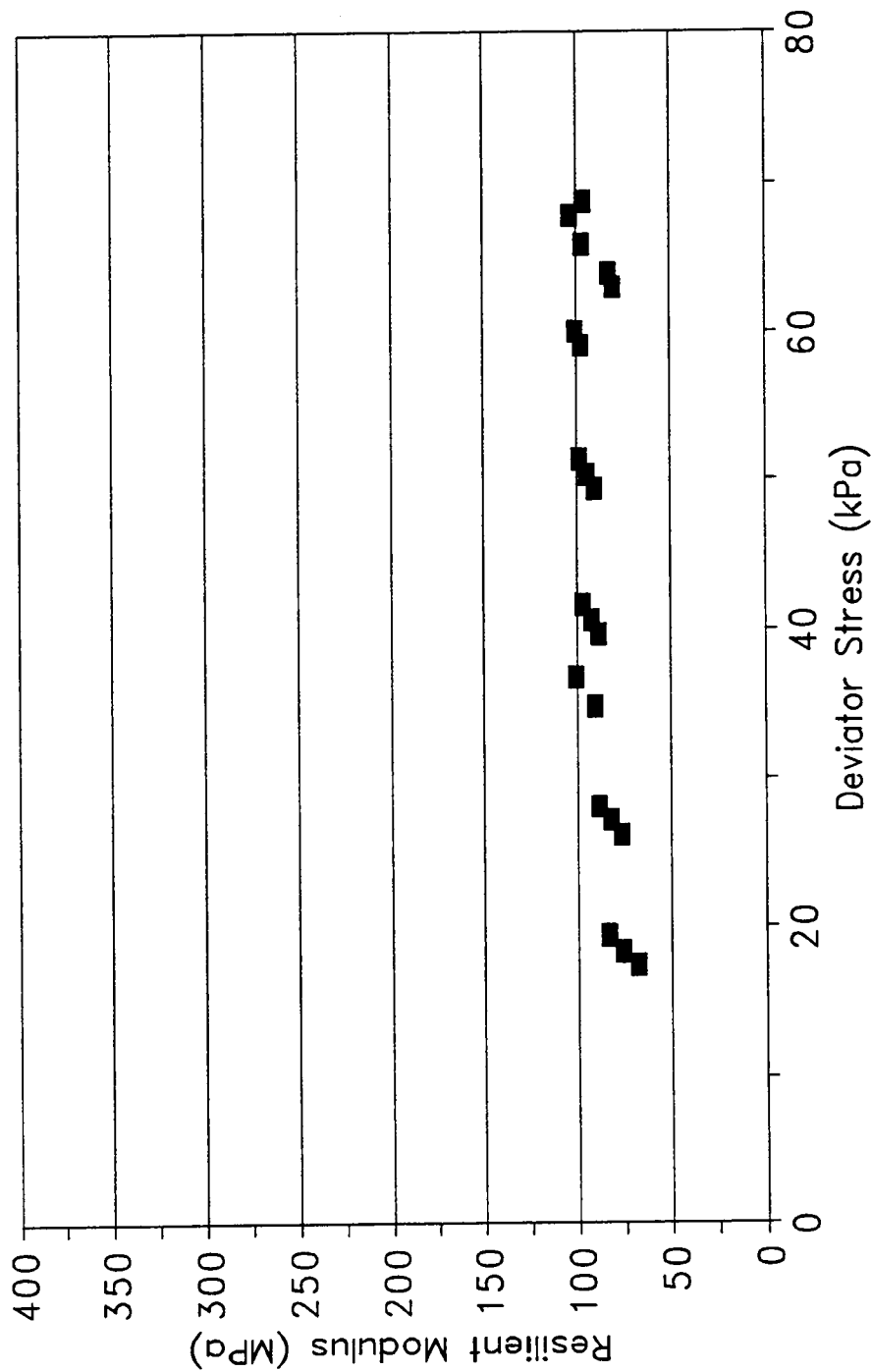


Figure B-11. ATM Data for Giles County at Compacted at Optimum Water Content and Maximum Dry Density, then Saturated from W%=23.8% to 24.7%

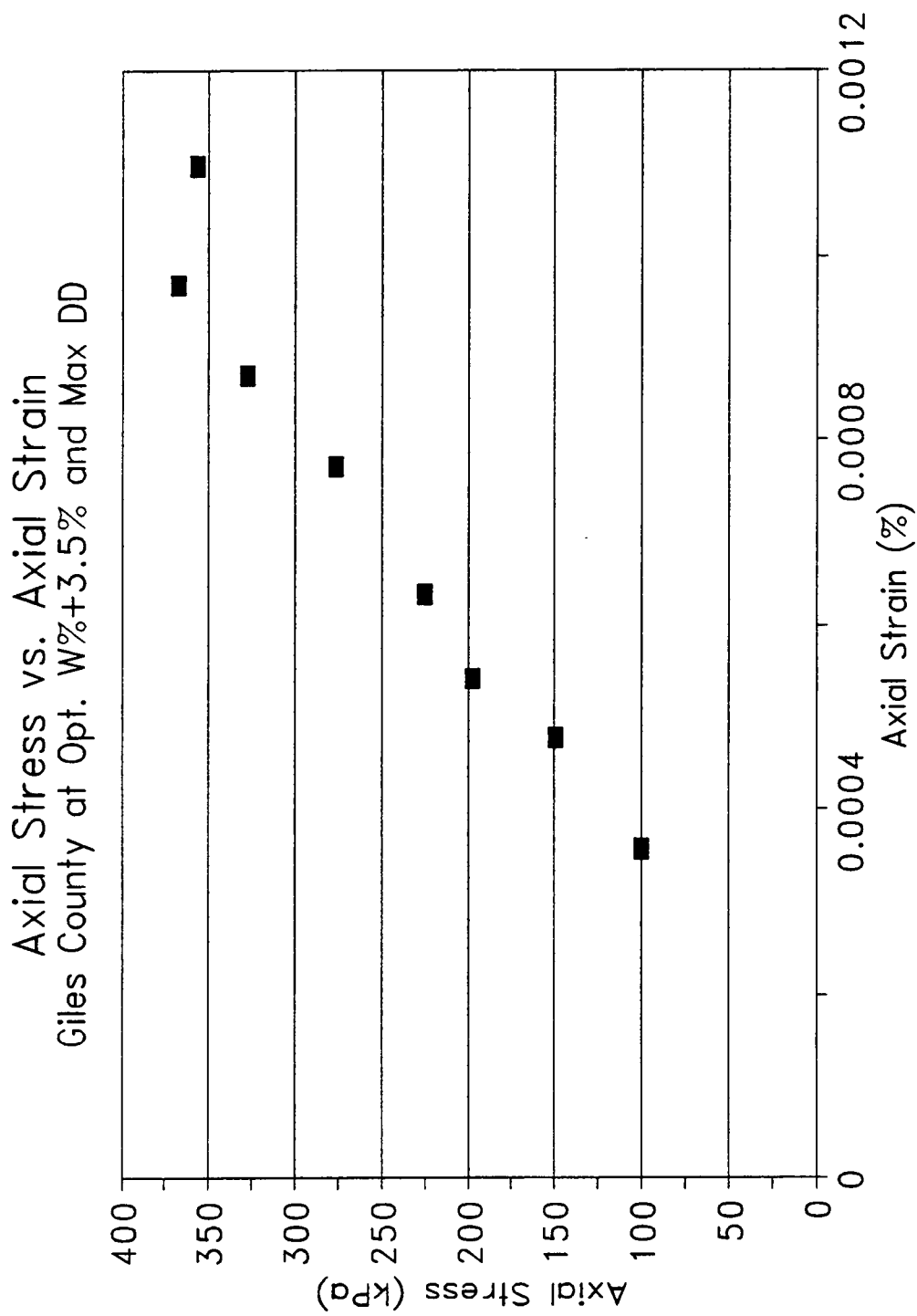


Figure B-12. Stress versus Strain Curves for Giles County at Compacted at Optimum Water Content and Maximum Dry Density, then Saturated from W% = 23.8% to 24.7%

SHELBY COUNTY

STA 9

SHELBY COUNTY #1-2A AT COMPACTION

Peak Acceleration Method

Peak Accel. g	Mass of Soil lb	Time to Peak $\times 10^{-4}$ sec	Poisson's Ratio	Drop Height mm	Falling Mass Mo (lb)	Resilient Modulus MPa	Deviator Stress kPa
42.97	4.14	5.00	0.45	5.00	2.48	25.9613	10.65
62.50	4.14	3.50	0.45	5.00	2.48	54.9232	15.49
50.78	4.14	4.50	0.45	5.00	2.48	36.2561	12.58
85.94	4.14	4.00	0.45	10.00	2.48	51.9226	21.29
97.66	4.14	4.50	0.45	10.00	2.48	67.0500	24.20
97.66	4.14	5.25	0.45	10.00	2.48	67.0500	24.20
101.56	4.14	4.00	0.45	10.00	2.48	72.5122	25.16
132.81	4.14	3.50	0.45	15.00	2.48	82.6677	32.91
125.00	4.14	3.50	0.45	15.00	2.48	73.2309	30.97
121.09	4.14	4.50	0.45	15.00	2.48	68.7212	30.00
125.00	4.14	5.50	0.45	15.00	2.48	73.2309	30.97
144.50	4.14	4.50	0.45	20.00	2.48	73.3958	35.80
148.44	4.14	4.00	0.45	20.00	2.48	77.4529	36.78
144.53	4.14	4.50	0.45	20.00	2.48	73.4263	35.81
148.44	4.14	3.50	0.45	20.00	2.48	77.4529	36.78
179.69	4.14	4.00	0.45	30.00	2.48	75.6645	44.52
175.78	4.14	5.00	0.45	30.00	2.48	72.4075	43.55
187.50	4.14	3.50	0.45	30.00	2.48	82.3848	46.46
199.22	4.14	4.00	0.45	30.00	2.48	93.0058	49.36
210.94	4.14	4.25	0.45	40.00	2.48	78.2030	52.27
242.19	4.14	4.50	0.45	40.00	2.48	103.0904	60.01
222.66	4.14	5.00	0.45	40.00	2.48	87.1345	55.17
218.75	4.14	4.75	0.45	40.00	2.48	84.1011	54.20
250.00	4.14	4.50	0.45	50.00	2.48	87.8771	61.94
238.28	4.14	4.00	0.45	50.00	2.48	79.8309	59.04
238.28	4.14	5.00	0.45	50.00	2.48	79.8309	59.04
257.81	4.14	5.00	0.45	50.00	2.48	93.4534	63.88
273.44	4.14	3.50	0.45	60.00	2.48	87.6069	67.75
285.16	4.14	4.00	0.45	60.00	2.48	95.2778	70.65
285.16	4.14	3.00	0.45	60.00	2.48	95.2778	70.65
292.97	4.14	4.00	0.45	60.00	2.48	100.5682	72.59

Resilient Modulus Results for Test Shelby Co. #1-2a DD=1.778 Mgcmm W%=14.9

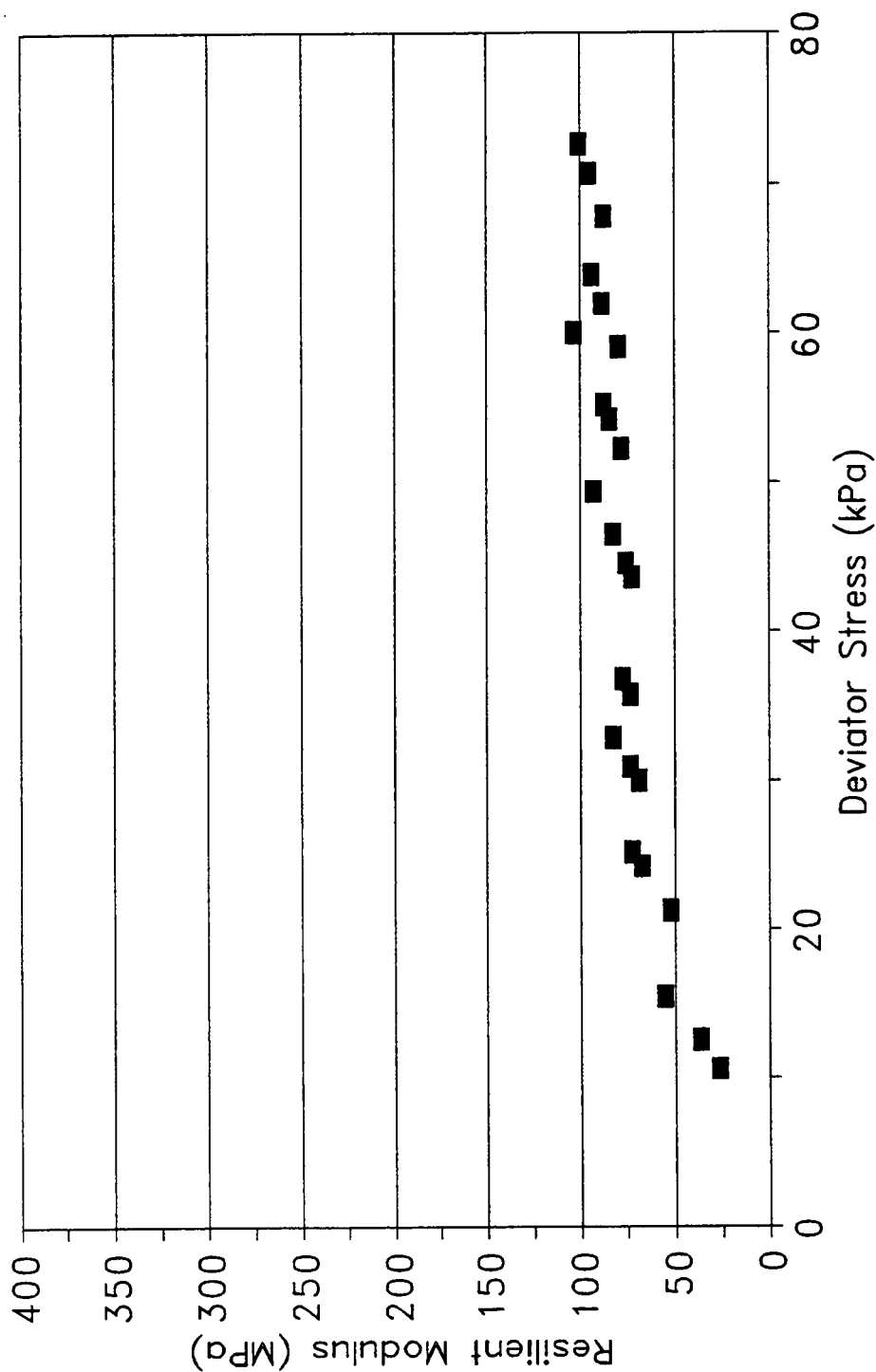


Figure B-13. ATM Data for Shelby County at Compacted at Optimum Water Content and Maximum Dry Density

Axial Stress vs. Axial Strain Shelby County at Opt. W% and Max DD

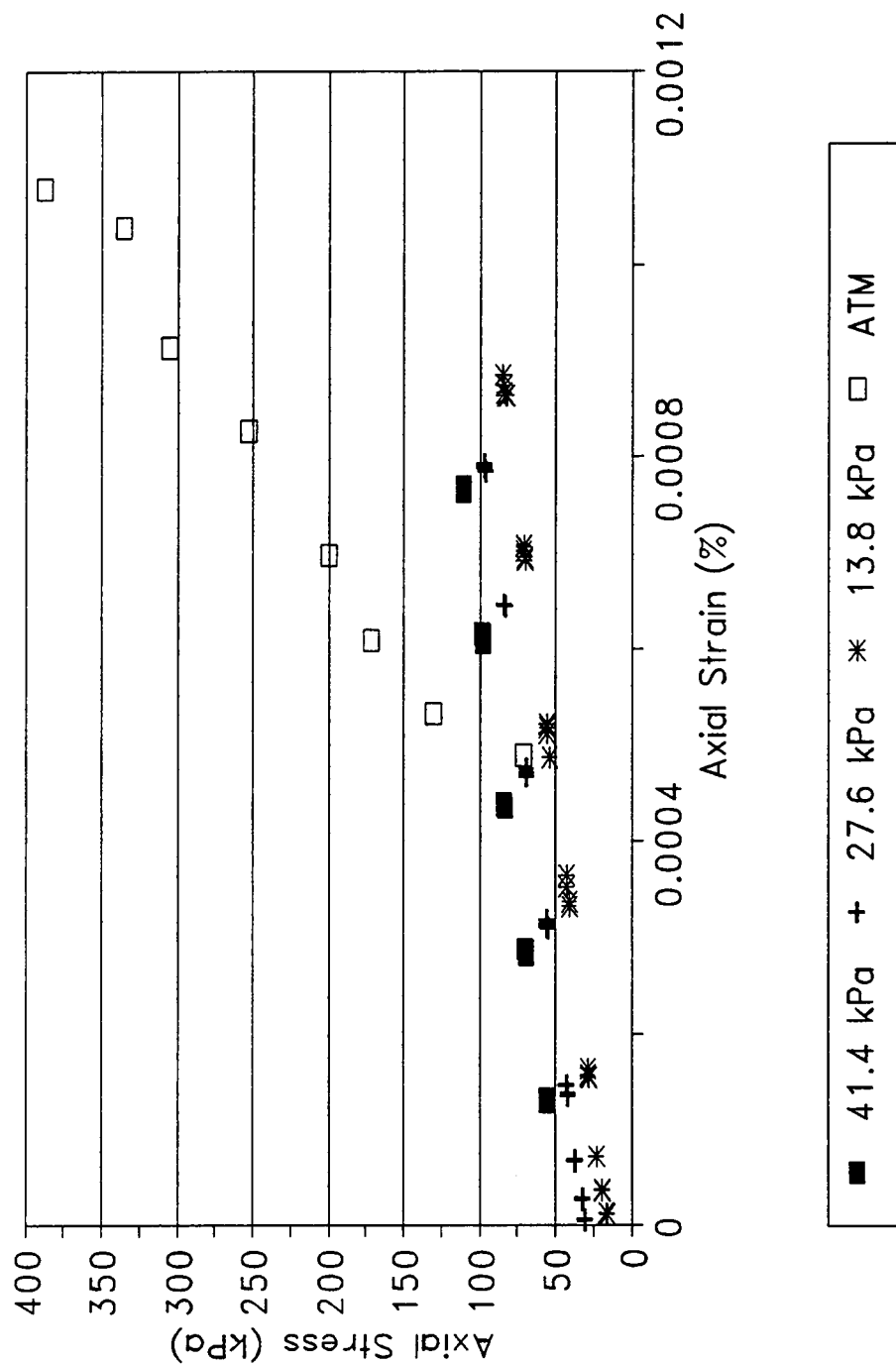


Figure B-14. Stress versus Strain Curves for ATM and Cyclic Triaxial on Shelby County Soil at Optimum Water Content and Maximum Dry Density

SHELBY COUNTY #1-2A AFTER FIRST SATURATION

Peak Acceleration Method

Peak Accel. g	Mass of Soil lb	Time to Peak $\times 10^{-4}$ sec	Poisson's Ratio	Drop Height mm	Falling Mass Mo (lb)	Resilient Modulus MPa	Deviator Stress kPa
70.31	4.17	4.60	0.45	5.00	2.48	70.10	17.42
78.13	4.17	4.20	0.45	5.00	2.48	86.57	19.36
78.13	4.17	4.40	0.45	5.00	2.48	86.57	19.36
70.31	4.17	4.60	0.45	5.00	2.48	70.10	17.42
113.28	4.17	4.20	0.45	10.00	2.48	90.99	28.07
113.28	4.17	4.00	0.45	10.00	2.48	90.99	28.07
117.19	4.17	4.20	0.45	10.00	2.48	97.38	29.04
109.38	4.17	4.40	0.45	10.00	2.48	84.83	27.10
136.72	4.17	4.20	0.45	15.00	2.48	88.36	33.88
144.53	4.17	4.00	0.45	15.00	2.48	98.74	35.81
136.72	4.17	4.20	0.45	15.00	2.48	88.36	33.88
144.53	4.17	4.20	0.45	15.00	2.48	98.74	35.81
175.78	4.17	4.00	0.45	20.00	2.48	109.54	43.55
175.78	4.17	4.60	0.45	20.00	2.48	109.54	43.55
179.69	4.17	4.20	0.45	20.00	2.48	114.47	44.52
175.78	4.17	4.20	0.45	20.00	2.48	109.54	43.55
222.66	4.17	4.60	0.45	30.00	2.48	117.18	55.17
222.66	4.17	3.80	0.45	30.00	2.48	117.18	55.17
214.84	4.17	3.80	0.45	30.00	2.48	109.09	53.23
218.75	4.17	4.00	0.45	30.00	2.48	113.10	54.20
253.91	4.17	3.90	0.45	40.00	2.48	114.28	62.91
253.91	4.17	3.80	0.45	40.00	2.48	114.28	62.91
253.91	4.17	4.20	0.45	40.00	2.48	114.28	62.91
253.91	4.17	4.00	0.45	40.00	2.48	114.28	62.91
242.17	4.17	4.50	0.45	50.00	2.48	83.17	60.00
285.16	4.17	3.60	0.45	50.00	2.48	115.32	70.65
277.34	4.17	4.10	0.45	50.00	2.48	109.08	68.72
285.16	4.17	4.00	0.45	50.00	2.48	115.32	70.65
320.31	4.17	3.80	0.45	60.00	2.48	121.25	79.36
320.31	4.17	3.60	0.45	60.00	2.48	121.25	79.36
308.59	4.17	3.40	0.45	60.00	2.48	112.54	76.46
308.59	4.17	3.40	0.45	60.00	2.48	112.54	76.46

Resilient Modulus Results for Test
Shelby Co. #1-2a DD=1.778 Mgcmm W%=14.9-15.9

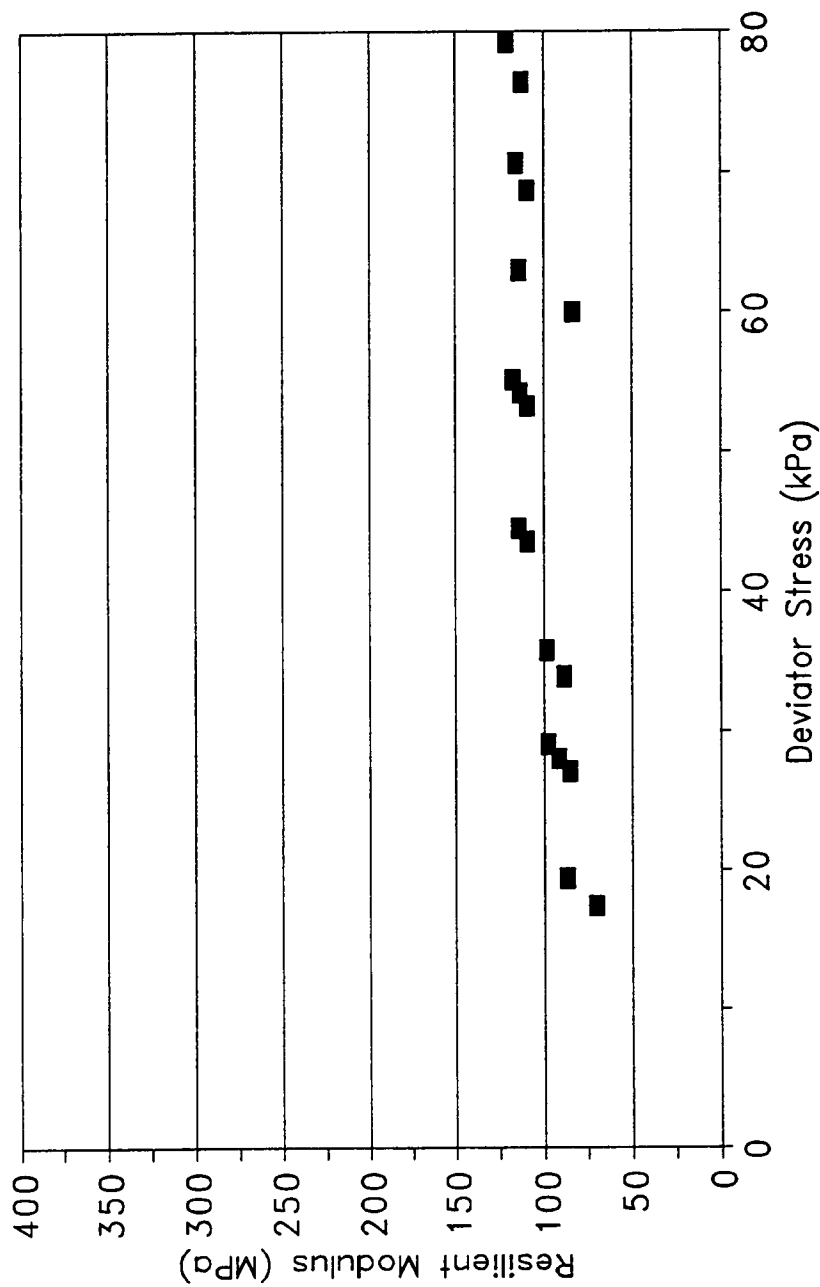


Figure B-15. ATM Data for Shelby County at Compacted at Optimum Water Content and Maximum Dry Density, then Saturated from S=77.7% to 82.9%

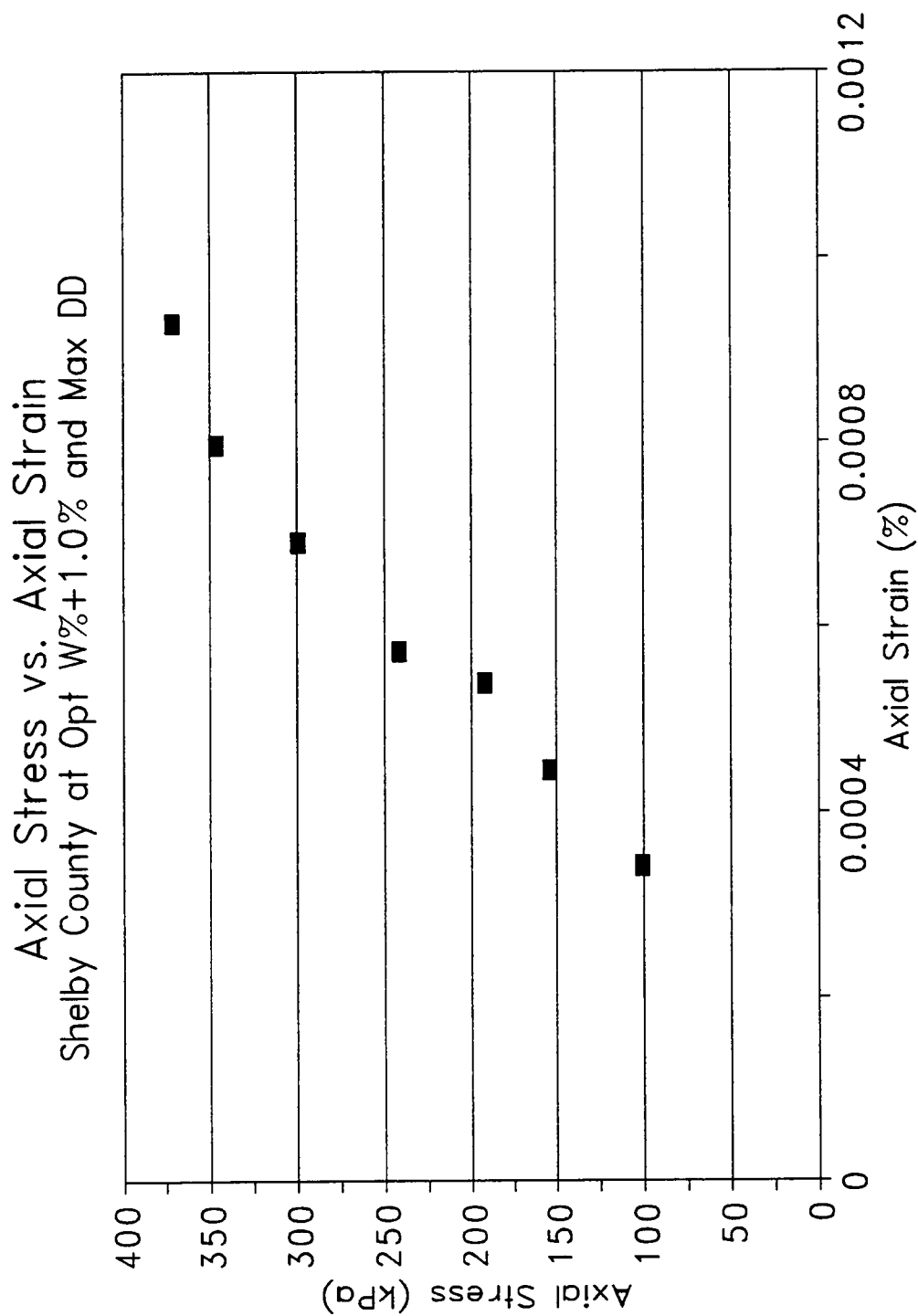


Figure B-16. Stress versus Strain Curves for Shelby County at Compacted at Optimum Water Content and Maximum Dry Density, then Saturated from $S=77.7\%$ to 82.9%

SHELBY COUNTY #1-2A AFTER SECOND SATURATION

Peak Acceleration Method

Peak Accel. g	Mass of Soil lb	Time to Peak $\times 10^{-4}$ sec	Poisson's Ratio	Drop Height mm	Falling Mass Mo (lb)	Resilient Modulus MPa	Deviator Stress kPa
42.97	4.24	6.00	0.45	5.00	2.48	26.57	10.65
39.06	4.24	6.00	0.45	5.00	2.48	21.96	9.68
39.06	4.24	6.20	0.45	5.00	2.48	21.96	9.68
46.87	4.24	6.20	0.45	5.00	2.48	31.62	11.61
66.41	4.24	5.80	0.45	10.00	2.48	31.74	16.45
66.41	4.24	6.00	0.45	10.00	2.48	31.74	16.45
66.41	4.24	6.60	0.45	10.00	2.48	31.74	16.45
66.41	4.24	7.00	0.45	10.00	2.48	31.74	16.45
101.56	4.24	4.90	0.45	15.00	2.48	49.48	25.16
105.47	4.24	5.20	0.45	15.00	2.48	53.36	26.13
97.66	4.24	6.00	0.45	15.00	2.48	45.75	24.20
105.47	4.24	5.80	0.45	15.00	2.48	53.36	26.13
117.19	4.24	6.00	0.45	20.00	2.48	49.41	29.04
125.00	4.24	6.00	0.45	20.00	2.48	56.22	30.97
121.09	4.24	6.00	0.45	20.00	2.48	52.76	30.00
117.19	4.24	6.20	0.45	20.00	2.48	49.41	29.04
160.16	4.24	6.90	0.45	30.00	2.48	61.53	39.68
164.06	4.24	6.00	0.45	30.00	2.48	64.56	40.65
156.25	4.24	5.80	0.45	30.00	2.48	58.56	38.71
156.25	4.24	5.80	0.45	30.00	2.48	58.56	38.71
199.22	4.24	5.00	0.45	40.00	2.48	71.40	49.36
191.41	4.24	5.00	0.45	40.00	2.48	65.91	47.43
191.41	4.24	5.20	0.45	40.00	2.48	65.91	47.43
175.78	4.24	5.40	0.45	40.00	2.48	55.59	43.55
214.84	4.24	5.40	0.45	50.00	2.48	66.43	53.23
222.66	4.24	5.00	0.45	50.00	2.48	71.35	55.17
222.66	4.24	4.60	0.45	50.00	2.48	71.35	55.17
222.66	4.24	5.00	0.45	50.00	2.48	71.35	55.17

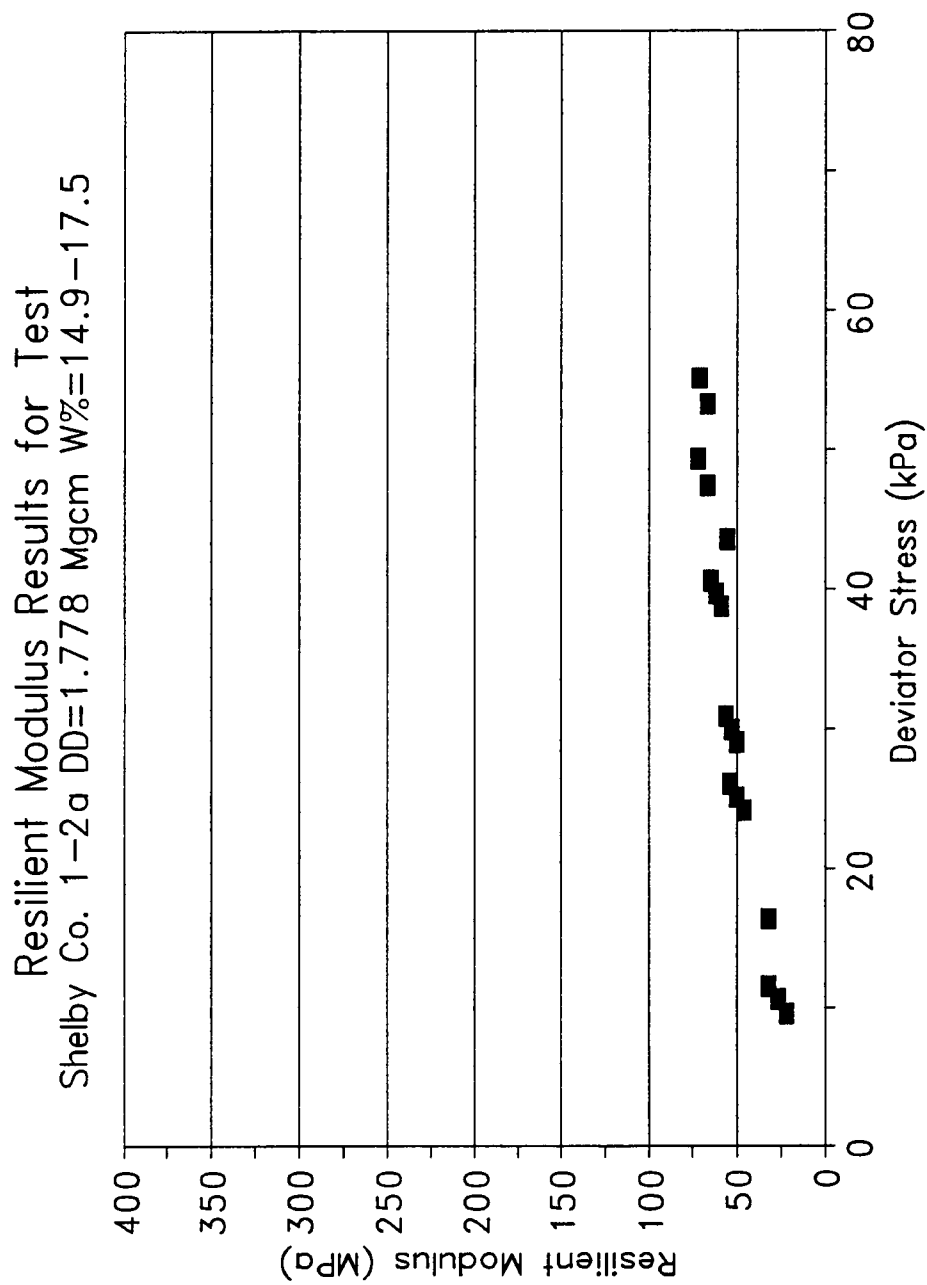


Figure B-17. ATM Data for Shelby County at Compacted at Optimum Water Content and Maximum Dry Density, then Saturated from S=77.7% to 91.2%

Axial Stress vs. Axial Strain Shelby County at Opt W%+2.6% and Max DD

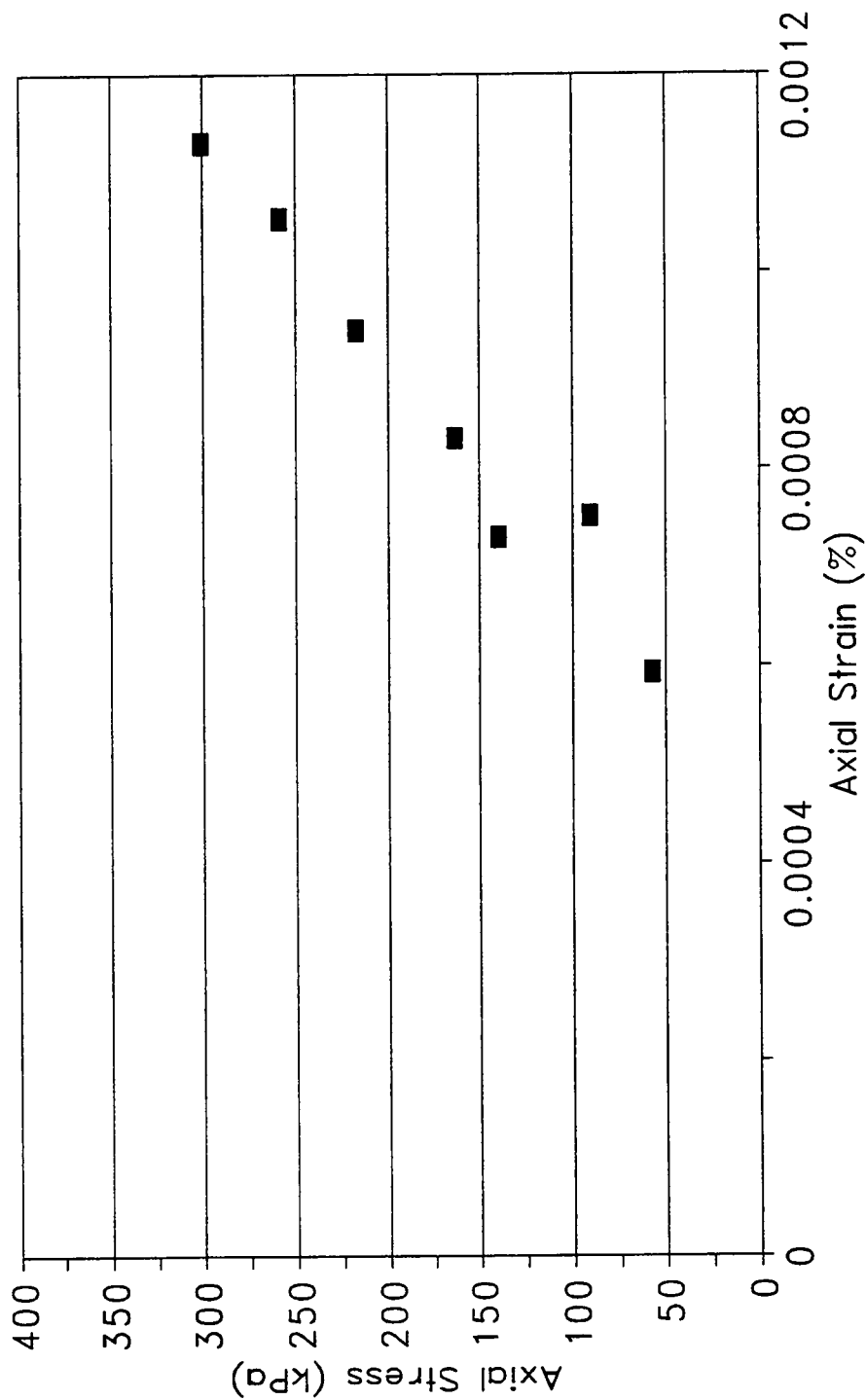


Figure B-18. Stress versus Strain Curves for Shelby County at Compacted at Optimum Water Content and Maximum Dry Density, then Saturated from S=77.7% to 91.2%

HAMILTON COUNTY

STA 578

HAMILTON COUNTY #1-1A

Peak Acceleration Method

Peak Accel. g	Mass of Soil lb	Time to Peak x10 ⁻⁴ sec	Poisson's Ratio	Drop Height mm	Falling Mass Mo (lb)	Resilient Modulus MPa	Deviator Stress kPa
42.97	4.11	8.00	0.45	5.00	2.48	25.82	10.65
37.89	4.11	7.00	0.45	5.00	2.48	20.07	9.39
43.75	4.11	9.00	0.45	5.00	2.48	26.76	10.84
35.55	4.11		0.45	5.00	2.48	17.67	8.81
66.41	4.11	5.00	0.45	10.00	2.48	30.83	16.45
70.31	4.11	6.00	0.45	10.00	2.48	34.56	17.42
78.13	4.11	5.00	0.45	10.00	2.48	42.68	19.36
85.94	4.11	5.00	0.45	10.00	2.48	51.64	21.29
109.34	4.11	6.00	0.45	15.00	2.48	55.72	27.09
101.56	4.11	4.00	0.45	15.00	2.48	48.08	25.16
109.38	4.11	5.00	0.45	15.00	2.48	55.76	27.10
117.19	4.11	6.00	0.45	20.00	2.48	48.01	29.04
128.91	4.11	5.50	0.45	20.00	2.48	58.09	31.94
117.19	4.11	5.00	0.45	20.00	2.48	48.01	29.04
132.81	4.11	5.00	0.45	20.00	2.48	61.66	32.91
156.25	4.11	5.00	0.45	30.00	2.48	56.90	38.71
156.25	4.11	6.00	0.45	30.00	2.48	56.90	38.71
160.06	4.11	5.00	0.45	30.00	2.48	59.71	39.66
164.06	4.11	6.00	0.45	30.00	2.48	81.93	46.46
187.50	4.11	6.00	0.45	40.00	2.48	77.77	52.27
210.94	4.11	5.00	0.45	40.00	2.48	61.45	46.46
187.50	4.11	5.00	0.45	40.00	2.48	77.77	52.27
210.94	4.11	5.50	0.45	40.00	2.48	86.66	55.17
222.66	4.11	4.50	0.45	50.00	2.48	62.22	52.27
210.94	4.11	5.50	0.45	50.00	2.48	66.91	54.20
218.75	4.11	5.00	0.45	50.00	2.48	66.91	54.20
218.75	4.11	5.00	0.45	50.00	2.48	79.39	59.04
238.28	4.11	4.00	0.45	60.00	2.48	61.89	57.10
230.47	4.11	4.50	0.45	60.00	2.48	61.89	57.10
230.47	4.11	4.00	0.45	60.00	2.48	ERR	ERR

Resilient Modulus Results for Test Hamilton Co.#1-1a DD=1.747 Mgcmm W%=16.2

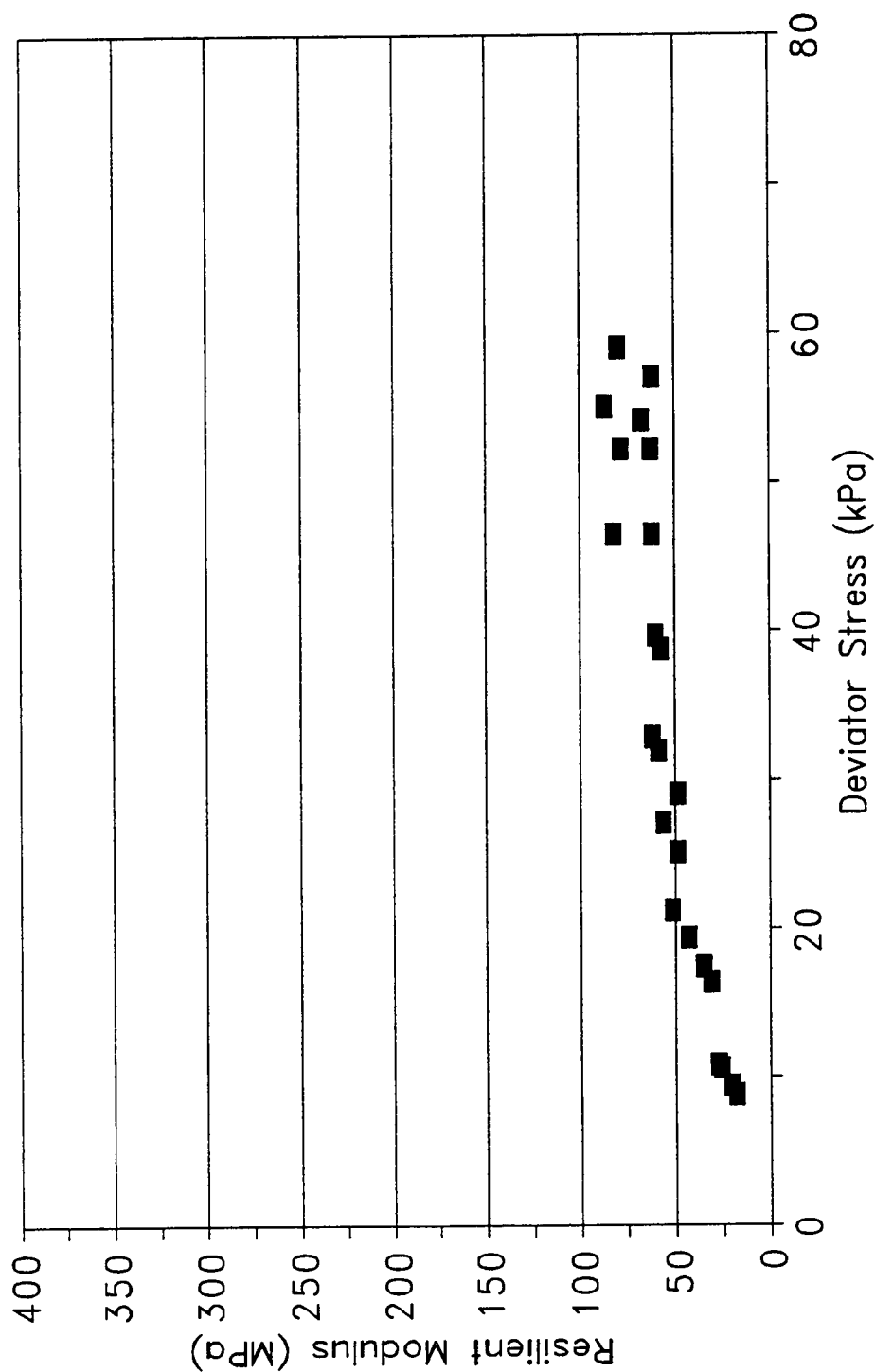


Figure B-19. ATM Data for Hamilton County at Compacted at Optimum Water Content and Maximum Dry Density

Axial Stress vs. Axial Strain Hamilton County at Opt. W% and Max DD

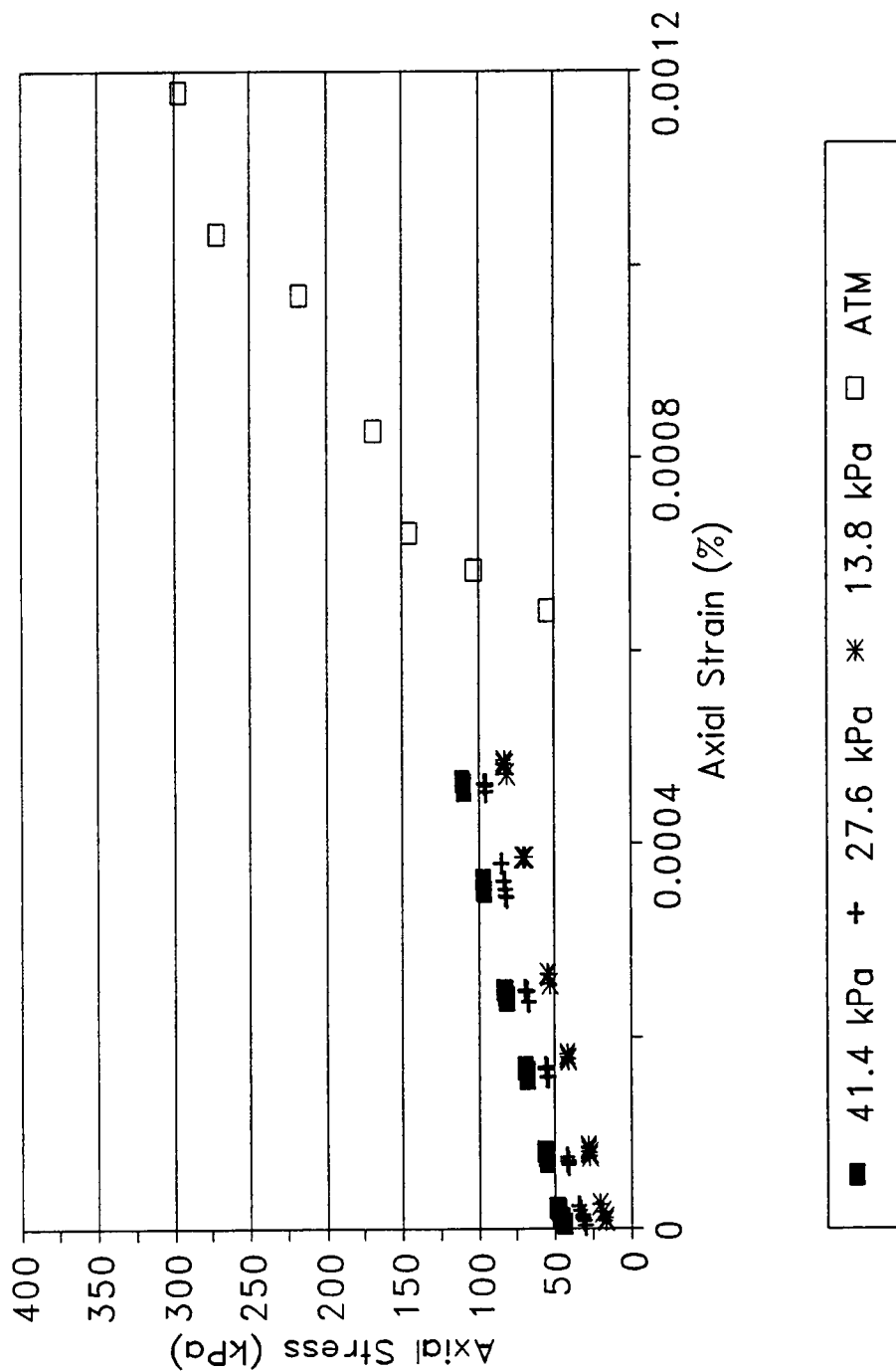


Figure B-20. Stress versus Strain Curves for ATM and Cyclic Triaxial on Shelby County Soil at Optimum Water Content and Maximum Dry Density

VANBUREN COUNTY

STA 618

VANBUREN COUNTY #1-1A

Peak Acceleration Method

Peak Accel. g	Mass of Soil lb	Poisson's Ratio	Drop Height mm	Falling Mass Mo (lb)	Resilient Modulus MPa	Deviator Stress kPa
62.50	3.88	0.45	5.00	2.48	51.92	15.49
58.60	3.88	0.45	5.00	2.48	45.64	14.52
93.80	3.88	0.45	10.00	2.48	58.48	23.24
128.90	3.88	0.45	10.00	2.48	110.43	31.94
132.80	3.88	0.45	15.00	2.48	78.14	32.90
144.50	3.88	0.45	20.00	2.48	69.39	35.80
160.20	3.88	0.45	20.00	2.48	85.28	39.69
218.80	3.88	0.45	20.00	2.48	159.09	54.21
242.20	3.88	0.45	25.00	2.48	155.95	60.01
308.60	3.88	0.45	25.00	2.48	253.17	76.46
238.20	3.88	0.45	30.00	2.48	125.70	59.02
238.30	3.88	0.45	30.00	2.48	125.80	59.04
328.10	3.88	0.45	50.00	2.48	143.09	81.29
363.30	3.88	0.45	50.00	2.48	175.44	90.02
328.10	3.88	0.45	50.00	2.48	143.09	81.29
500.00	3.88	0.45	60.00	2.48	276.92	123.89

Resilient Modulus Results for Test
VanBuren Co. #1-1a DD=1.595 Mgc/m W%=20.3

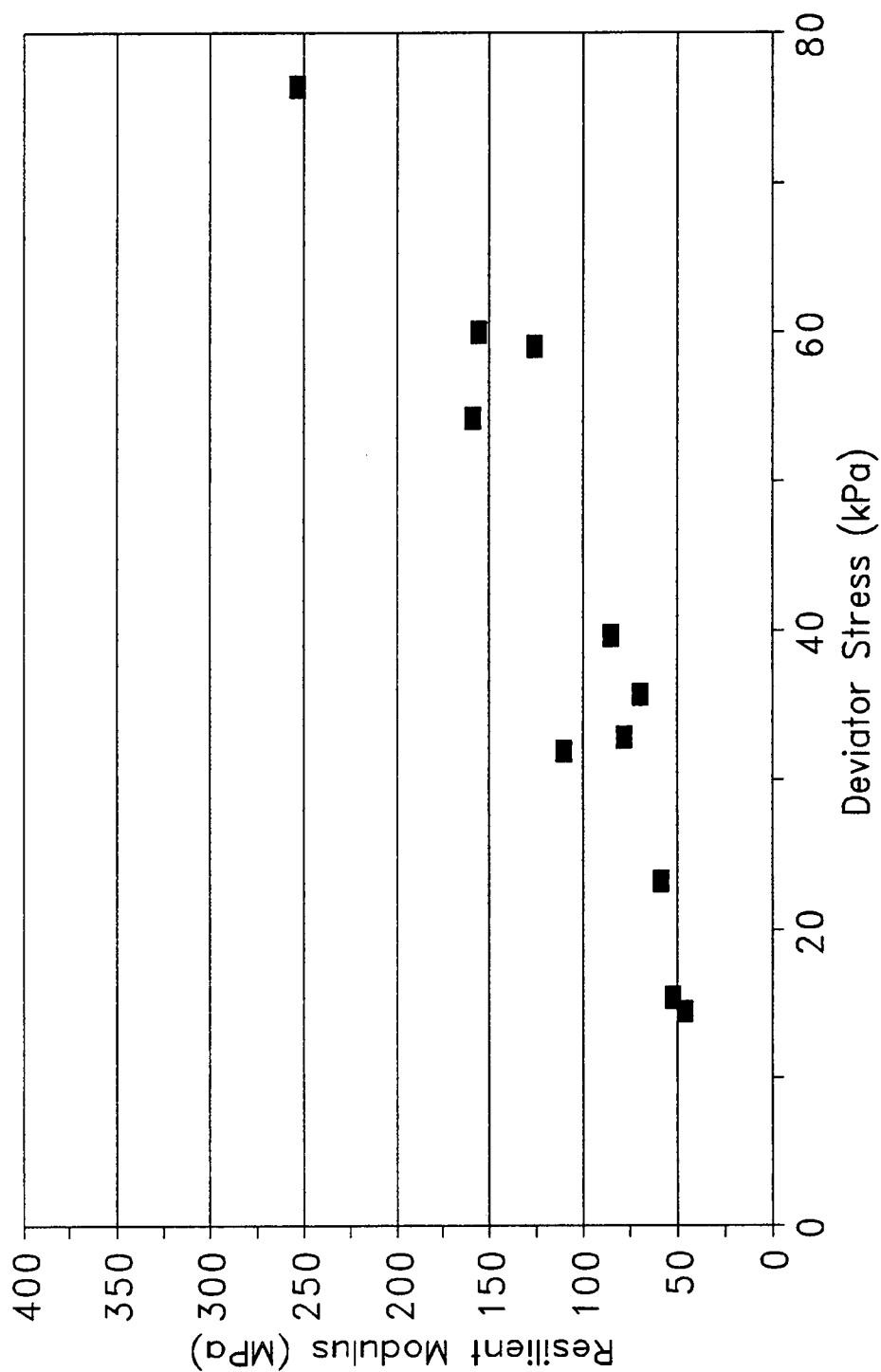


Figure B-21. ATM Data for VanBuren County at Compacted at Optimum Water Content and Maximum Dry Density

Axial Stress vs. Axial Strain VanBuren County at Opt. W% and Max DD

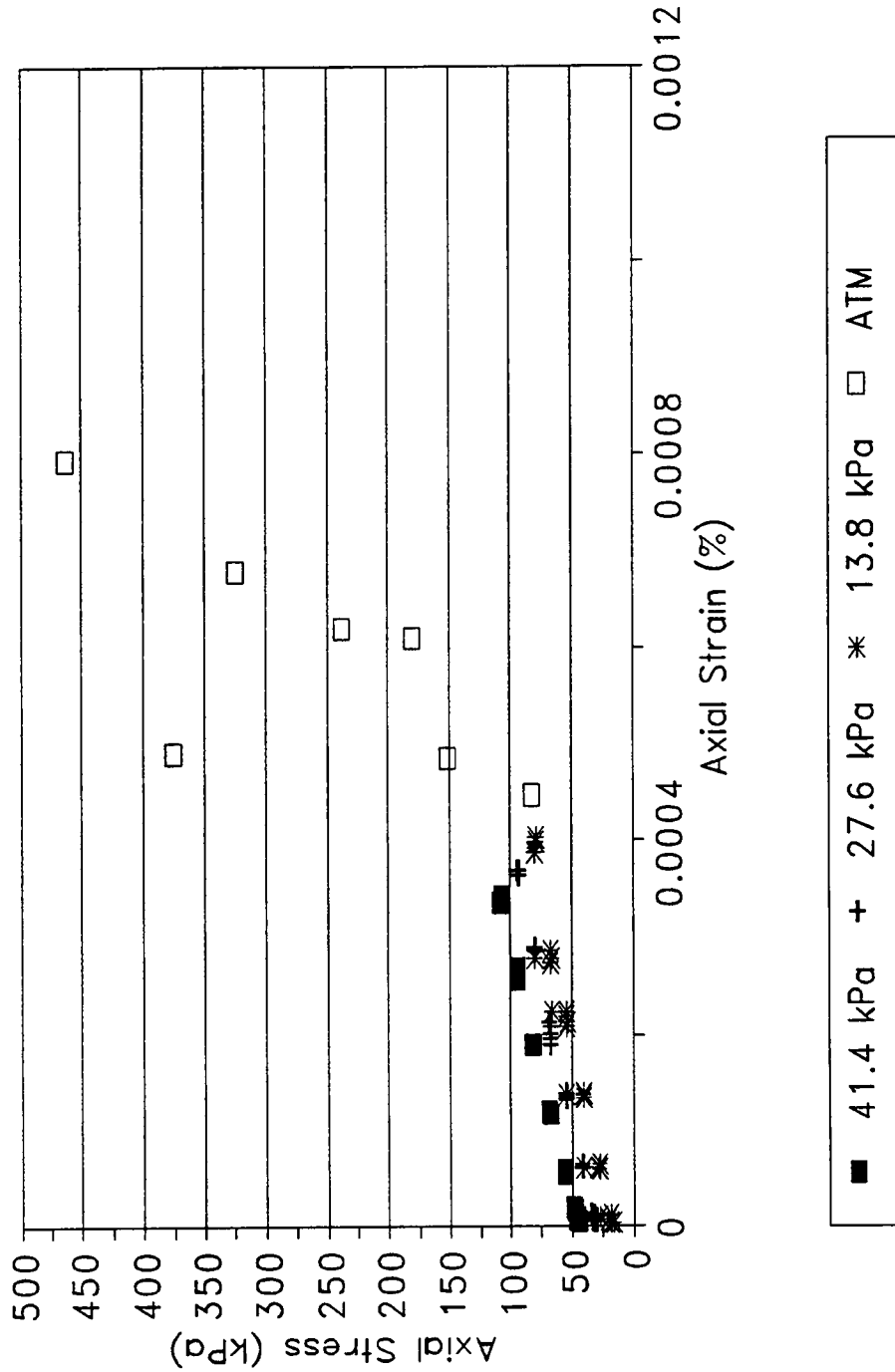


Figure B-22. Stress versus Strain Curves for ATM and Cyclic Triaxial on VanBuren County Soil at Optimum Water Content and Maximum Dry Density

WHITE COUNTY

STA 652

WHITE COUNTY #1-1A

Peak Acceleration Method

Peak Accel. g	Mass of Soil lb	Time to Peak $\times 10^{-4}$ sec	Poisson's Ratio	Drop Height mm	Falling Mass Mo (lb)	Resilient Modulus MPa	Deviator Stress kPa
62.50	4.07	4.00	0.45	5.00	2.48	54.14	15.49
58.59	4.07	4.00	0.45	5.00	2.48	47.58	14.52
58.59	4.07	5.00	0.45	5.00	2.48	47.58	14.52
54.69	4.07	4.00	0.45	5.00	2.48	41.46	13.55
66.41	4.07	5.00	0.45	10.00	2.48	30.56	16.45
70.31	4.07	4.00	0.45	10.00	2.48	34.26	17.42
70.31	4.07	5.00	0.45	10.00	2.48	34.26	17.42
74.22	4.07	5.00	0.45	10.00	2.48	38.18	18.39
101.56	4.07	4.00	0.45	15.00	2.48	47.65	25.16
93.75	4.07	5.00	0.45	15.00	2.48	40.61	23.23
93.75	4.07	4.00	0.45	15.00	2.48	40.61	23.23
105.47	4.07	3.50	0.45	15.00	2.48	51.39	26.13
117.19	4.07	4.00	0.45	20.00	2.48	47.59	29.04
113.28	4.07	4.00	0.45	20.00	2.48	44.46	28.07
121.09	4.07	3.50	0.45	20.00	2.48	50.81	30.00
117.19	4.07	4.00	0.45	20.00	2.48	47.59	29.04
164.06	4.07	6.50	0.45	30.00	2.48	62.18	40.65
191.41	4.07	4.50	0.45	30.00	2.48	84.63	47.43
195.30	4.07	4.50	0.45	30.00	2.48	88.11	48.39
210.94	4.07	3.50	0.45	40.00	2.48	77.09	52.27
234.38	4.07	3.50	0.45	40.00	2.48	95.17	58.07
242.19	4.07	4.50	0.45	50.00	2.48	81.30	60.01
285.16	4.07	4.50	0.45	50.00	2.48	112.71	70.65

Resilient Modulus Results for Test
White Co.#1-1a DD=1.683 Mgcmm W%=19.4

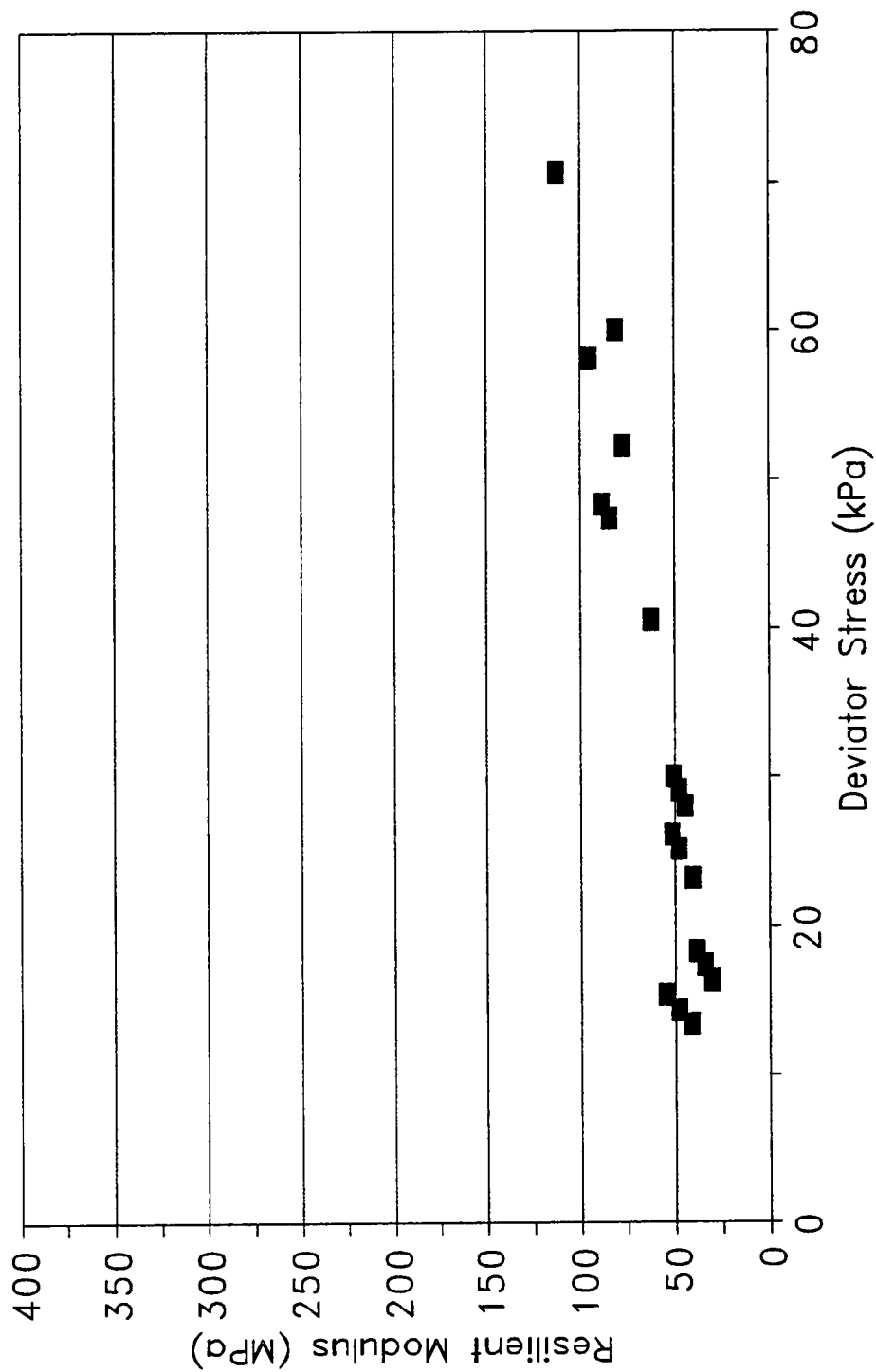


Figure B-23. ATM Data for White County at Compacted at Optimum Water Content and Maximum Dry Density

Axial Stress vs. Axial Strain White County at Opt. W% and Max DD

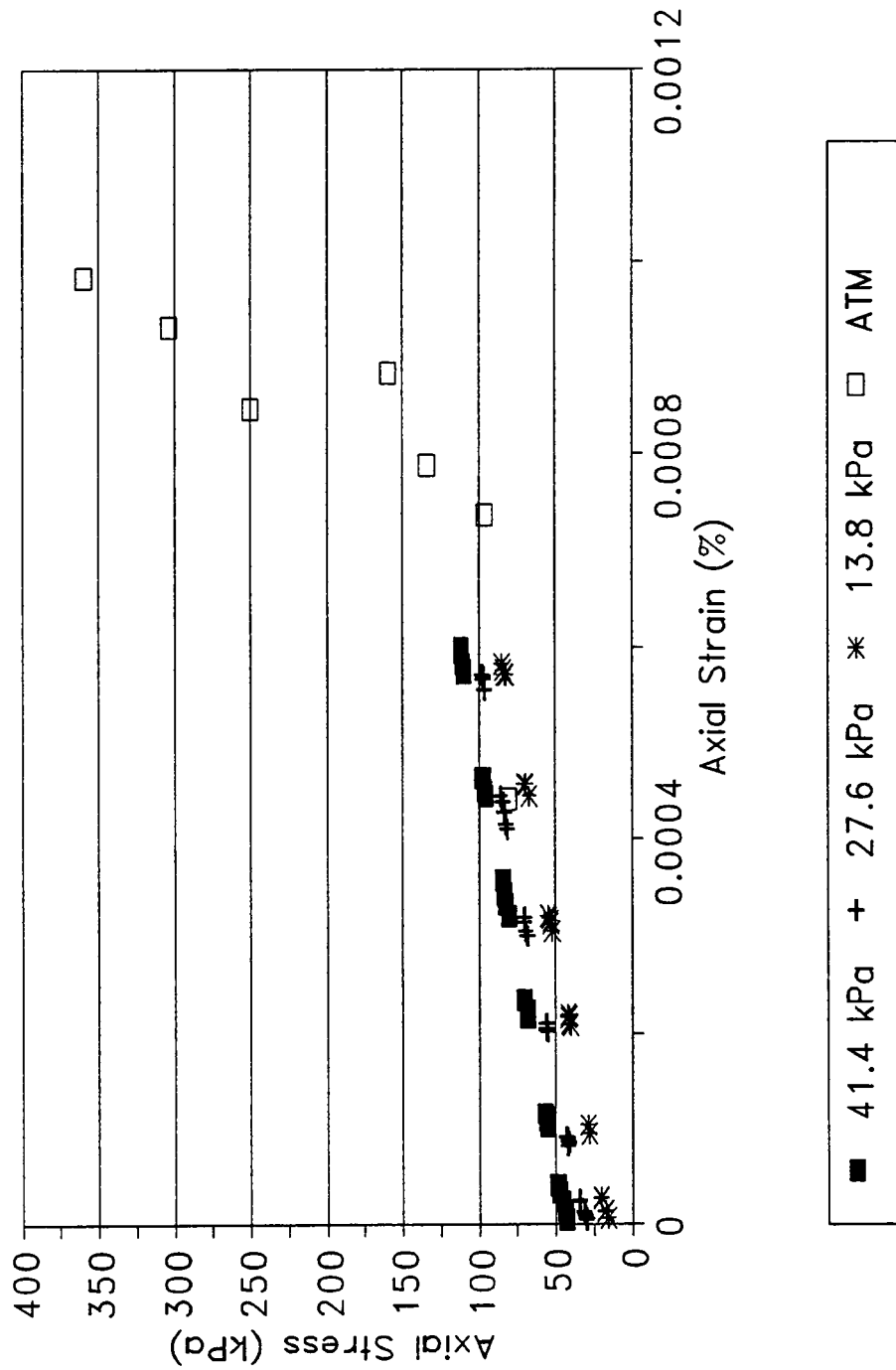


Figure B-24. Stress versus Strain Curves for ATM and Cyclic Triaxial on White County Soil at Optimum Water Content and Maximum Dry Density

VITA

Mark Raulston Madgett was born November 9, 1967, in Elizabethton, Tennessee. He attended West Side Elementary and T. A. Dugger Jr. High School before graduating from Elizabethton High School in May of 1986. After one year of attending East Tennessee State University, he transferred to the University of Tennessee in the fall of 1989 where he received his Bachelor of Science Degree in Civil Engineering in May 1992. He then began graduate school at the University of Tennessee in June of 1992 where he worked towards a Master of Science in Civil Engineering with an emphasis in Geotechnical Engineering which was completed in August 1994. The author received his Engineer-In-Training certificate in December of 1992 and is a member of the American Society of Civil Engineers. Prior to graduating he received employment with S&ME, Inc. in Knoxville, Tennessee.