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16. Abstract

Many research engineers over the years have reported various problems with the resilient modulus test for soils. Some of these problems are associated with the testing setup, some with the testing procedure. In particular, researchers have observed significant differences in the estimations of the moduli when comparing results from the field with those obtained under laboratory conditions. Thus, the purpose of this study was to develop a reliable resilient modulus test for subgrade and non-granular subbase materials for use in routine pavement design.

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DEVELOPMENT OF A RELIABLE RESILIENT MODULUS TEST FOR SUBGRADE AND NON-GRANULAR SUBBASE MATERIALS FOR USE IN ROUTINE PAVEMENT DESIGN

by

Rafael F. Pezo
Germán Claros
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Research Report 1177-4F

Research Project 2/3/10-8-88/0-1177
Resilient Modulus Testing

conducted for the

Texas Department of Transportation

in cooperation with the

U.S. Department of Transportation
Federal Highway Administration

by the

CENTER FOR TRANSPORTATION RESEARCH
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Research Supervisors

PREFACE

This is the fourth and final report for Research Project 2/3/10-8-88/0-1177, "Resilient Modulus Testing." The study was conducted by the Center for Transportation Research (CTR), The University of Texas at Austin, as part of a research program sponsored by the Texas Department of Transportation.

Many individuals have contributed their time and expertise to the completion of this report. The authors sincerely appreciate the valuable comments provided by Dr. William Spelman, Dr. Virgil Anderson, and Dr. Dong-Soo Kim. In addition, thanks are extended to CTR technical staff members Zhanmin Zhang, Chryssis Papaleontiou, Ray Donley, and Terry Dossey.

Finally, we would like to thank Harold Albers and Bob Mikulin of the Texas Department of Transportation for their generous support and counsel.

Rafael F. Pezo
Germán Claros
W. Ronald Hudson
Kenneth H. Stokoe, II

LIST OF REPORTS

Report 1177-1, "Resilient Modulus of Asphalt Concrete," by D. N. Little, W. W. Crockford, and V. K. R. Gaddam (Texas Transportation Institute, Texas A&M University), documents the development of several approaches to the measurement of asphalt concrete moduli. The test methodology developed in this study provides expedient and flexible testing to quantify moduli for asphalt concrete surface courses and other asphalt-bound materials.

Report 1177-2, "Critical Evaluation of Parameters Affecting Resilient Modulus Tests on Subgrades," by M. Feliberti, S. Nazarian, and T. Srinivasan, is a joint study of The University of Texas at El Paso and the Center for Transportation Research of The University of Texas at Austin. The report details the strengths and limitations of the resilient modulus testing procedure as applied to subgrade soils.

Report 1177-3, "Deformational Characteristics of Soils at Small to Intermediate Strains from Cyclic Tests," by Dong-Soo Kim, Dr. Kenneth H. Stokoe, II, and Dr. W. R. Hudson, presents the results of stiffness measurements made with resonant column and torsional shear equipment on synthetic samples and various soils; these results were compared with moduli determined with resilient modulus equipment. August 1991.

Report 1177-4F, "Development of a Reliable Resilient Modulus Test for Subgrade and Non-granular Subbase Materials for Use in Routine Pavement Design," by Rafael F. Pezo, Dr. Germán Claros, Dr. W. R. Hudson, and Dr. Kenneth H. Stokoe, II, describes the development of a resilient modulus testing method for use in routine pavement design. September 1991.

ABSTRACT

Many research engineers over the years have reported various problems with the resilient modulus test for soils. Some of these problems are associated with the testing setup, some with the testing procedure. In particular, researchers have observed significant differences in the estimations of the moduli when comparing results from the field with those obtained under laboratory conditions. Thus, the purpose of this study was to develop a reliable resilient modulus test for subgrade and non-granular subbase materials for use in routine pavement design.

SUMMARY

In its 1986 guidelines for the design of pavement structures, the American Association of State Highway and Transportation Officials (AASHTO) endorsed the resilient modulus concept as the basis for the characterization of pavement materials, recommending in particular AASHTO T-274-82, the "Standard Method of Testing for Resilient Modulus of Subgrade Soils." But since its introduction, AASHTO T-274 has been widely criticized. Problems in the setup and testing process have prompted concerns regarding the reliability, repeatability, and efficiency of the test method.

This report, in documenting a specific response to these concerns, describes the development of a reliable resilient modulus testing method for subgrade and non-granular subbase materials for use in routine pavement design. In outlining this development, the report documents the state of knowledge regarding the dynamic behavior of soils, as well as the available state-of-the-art equipment used in assessing soil behavior. Equipment used to provide testing configuration guidelines are also described.

After examining available testing procedures, a prototype testing procedure was developed. Then, each aspect and stage of this prototype procedure was thoroughly evaluated in experiments aimed at identifying the most efficient and reliable procedure. To validate this testing procedure and the guidelines to be recommended, moduli results obtained through our experimental programs were compared with results obtained through other laboratory and field tests. Finally, through this extensive investigation, a new resilient modulus testing method has been successfully developed and is herein proposed—one that is reliable, repeatable, and efficient.

Keywords: Resilient modulus, laboratory testing, soil dynamics, subgrade, subbase, flexible pavement, pavement design

IMPLEMENTATION STATEMENT

This project recommends use of the alternative resilient modulus testing method described in Chapter 13. This method, as the report details, is particularly effective in determining, for pavement design purposes, the stiffness characteristics of subgrade and non-granular subbase materials. The report also presents moduli prediction models that can provide the engineer with a quick and early estimate of the resilient moduli for use in pavement design and pavement evaluation.

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CHAPTER 1. INTRODUCTION

BACKGROUND

Years ago engineers had to rely on either experience or some type of index to guide them in the design of pavement thicknesses. In general, the approach chosen attempted to control pavement layer thickness and layer material quality, the assumption being that the primary source of deformation occurs in the subgrade. In this way, allowable deformations were controlled primarily by defining the pavement thickness, with such definitions seeking to reduce the resulting subgrade stress to a level that yielded a permanent deformation representative of a "failed" condition after so many stress repetitions.

However, new design and construction practices recognize that several distress modes (e.g., rutting, shoving, and cracking) contribute to pavement failure. For instance, experimental studies, such as those conducted by the California Division of Highways, have demonstrated that repeated load applications induce repeated deformations that can cause cracking of asphaltic surfacing.

The need to predict the physical response of pavement structures to repeated loads has led to the application of methods based on multi-layered elastic theory. Because they are capable of assessing the magnitude of the strains developed in the subgrade and pavement layers, such methods are now considered necessary for assessing pavement service life. Making use of elastic and/or viscoelastic structural analyses, these methods rely heavily on proper characterization of the stress-strain behavior of the materials comprising the pavement structure. This combined stress-strain behavior is expressed in terms of modulus.

The major component of deformation—or strain—induced into a pavement structure under wheel loading is not associated with either plastic deformation (permanent deformation) or rupture. Rather, it is an elastic deformation referred to as recoverable or *resilient* deformation. The resilient modulus is therefore considered the required input for determining the stresses, strains, and deflections in a pavement structure subjected to traffic loadings.

The other parameter needed for mechanistic analysis is Poisson's ratio. It has been demonstrated that pavement responses are not especially sensitive to variation of this parameter, and that they can be estimated with reasonable accuracy. Therefore, one of the main problems in predicting pavement deflections is the determination of the resilient or elastic moduli of the pavement components.

A successful mechanistic design relies on a proper characterization of the pavement materials, taking into account all the factors affecting their deformational characteristics. This is perhaps the most difficult part in the design and evaluation of pavement structures, since soil properties are likely to differ at each construction site.

Richard and Hall (Ref 44) reported that the dynamic response of a given soil depends not only on the loading conditions, but also on the strain distribution developed in the soil mass. In 1962, Seed (Ref 5) listed the following factors that influence the resilient modulus of soils: (1) the number of stress applications; (2) the age at initial loading; (3) the stress intensity; (4) the method of compaction; and (5) the compaction density and water content. Accordingly, measurements must be taken using samples obtained from the construction site and tested under conditions expected to occur during the service life of the pavement structure.

While field tests can be used to determine the dynamic behavior of soils, most engineers favor laboratory tests. Such a preference is based on the fact that field tests are limited (e.g., constraints associated with relatively small loading magnitudes, accessibility to construction site, an already existing pavement structure, and favorable weather). Laboratory tests, on the other hand, are less constrained because of their carefully controlled conditions. Most researchers agree that laboratory testing is more appropriate for design, while field tests are more appropriately used in the evaluation of pavement structures. Many types of laboratory tests have been developed for a wide variety of materials. One common type is the repeated load triaxial compression test, frequently called the resilient modulus test.

In 1986, the American Association of State Highway and Transportation Officials (AASHTO) adopted in their guide for the design of pavement structures the use of the elastic or resilient modulus as the basis for the characterization of pavement materials (Ref 1). The AASHTO Guide specifies that, for roadbed soils, laboratory tests of resilient modulus should be performed on representative samples under stress and moisture conditions that simulate actual field conditions. For this, the AASHTO Guide suggests using AASHTO T-274-82, the "Standard Method of Testing for Resilient Modulus of Subgrade Soils" (Ref 2). Additionally, the AASHTO Guide suggests using empirical correlations to estimate approximate resilient moduli values based on several soil properties, including fine-grain content, moisture content, plasticity index, and CBR values of the test materials.

REPORTED PROBLEMS

Since its introduction, AASHTO T-274-82 has been the target of widespread criticism. For instance, Vinson (Ref 8), in citing the major disadvantages of this testing procedure, notes that it requires that all specimens be heavily conditioned prior to the actual test; by then, he argues, the sample may have undergone a substantial variety of stress states for both cohesive and cohesionless soils.

Other researchers have criticized the laborious process of sample conditioning and testing. Most of them (Refs 9, 31, 29, 43) questioned the validity of and need for such an extensive process. Ho (Ref 9), in particular, has documented his unsatisfactory experience with AASHTO T-274. In testing several subgrade soils collected across Florida, Ho observed that the resilient modulus (M_R) values seem to be independent of the number of repetitions (up to 10,000), and that the conditioning stage, as suggested by AASHTO T-274, was very severe for many of their soils. In addition, he found that his results had a pronounced variability on the moduli, depending on the position of the transducers within the testing setup. Presently, he is developing a test method and measurement device that he believes will lead to a more reliable system.

In another instance, the Washington experience with AASHTO T-274 has been well-documented by Jackson (Ref 31), who stated that "the description of the preparation procedure is difficult at best, while the description of test sequence is nothing short of a maze." He expressed concern over whether AASHTO T-274 should be adopted as the primary test for characterizing subgrade soil stiffness. He stated that for cohesionless materials, the large number of conditioning and test sequences specified exceeds the stresses expected in any actual pavement section. Moreover, during the testing some of their compacted cohesive samples broke under the unconfined compressive stress. Jackson questioned the need to condition all samples at the full range of deviator and confining pressures. He explained that the Washington DOT operates with a modified testing procedure for both cohesive and cohesionless soils. Nevertheless, he recognized that those modifications were but crude attempts to make the test more rational, and that more effort was needed to refine the test procedures.

Dhamrait (Ref 29), in describing the experience of the Illinois DOT with the resilient modulus test for soils, explained that they used their own testing procedure. That procedure differs from AASHTO T-274 in that the sample conditioning and testing sequences are performed without lateral confining pressures and with a much lower number of stress applications. In addition, Dhamrait explained that they selected the modulus at 6-psi deviator stress as the modulus of the material for design purposes. However, he recognized that some of his results did not follow the "expected" trend of the moduli, which, according to Thompson and Robnett (Ref 19), is influenced by the magnitude of the repeated compression stress (or what is referred to as deviator stress) and characterized by a "presumed" break point at a 6-psi deviator stress. Dhamrait reported that some of his results have little or no break at all, with virtually no downward slope (an upward slope was evident in some results).

Cochran (Ref 43) documented the experience of the Minnesota DOT with the laboratory resilient modulus test. He described comparisons of laboratory with field tests, undertaken by the Minnesota DOT, that led to very disappointing results. He explained that their field values were thought to be the correct ones and their laboratory values the incorrect ones; the fact is, as he later recognized, they were not able to identify which values were really correct. Finally, he reported that the failure of the method used to compare the moduli obtained from *in situ* and laboratory testing has forced the Minnesota DOT to re-structure their laboratory system.

In comparing predicted and measured pavement deflections in two sections of the San Diego Test Road, Dehlen (Ref 21), in 1969, observed that the pavement deflections obtained by his theoretical analyses (linear and non-linear elastic) were much larger than those measured in the field under similar conditions. In listing probable reasons for such a large difference in the results (e.g., material anisotropy not taken into account, laboratory samples with large disturbances, and non-uniform loading system in field tests), he clearly recognized that, whatever the reason, none of his hypotheses could have been verified with his data, and that, for all his efforts, such a discrepancy must unfortunately remain unexplained.

To summarize, many researchers over the years have reported various problems with the resilient modulus test of soils. Some of the problems are associated with the testing setup, some with the testing procedure. In particular, significant differences in the estimations of the moduli have been observed when comparing results obtained from the field with those obtained under laboratory conditions. Such discrepancies do not conduce to accurate pavement evaluations.

OBJECTIVES OF THE STUDY

The purpose of this study is to develop a reliable resilient modulus test for subgrade and non-granular subbase materials for use in routine pavement design. To achieve this broad objective, the following specific tasks were established:

(1) review the state of knowledge regarding the behavior of soils subjected to dynamic loading;
(2) revise resilient modulus testing systems to define limitations and to improve instrumentation and calibration;
(3) develop a prototype resilient modulus testing method for subgrade and subbase materials; evaluate that prototype so that a more reliable and efficient testing procedure can be recommended;
(4) compare the modulus results obtained with resilient modulus tests with other laboratory and in situ tests to validate further the testing procedures and the guidelines that are to be recommended;
(5) evaluate factors (e.g., plasticity index, moisture conditions, density, and age-hardening) that affect the resilient modulus of soils;
(6) formulate more appropriate empirical models that can be used in routine design and periodic evaluation of pavements.

SCOPE OF THE STUDY

This research project is concerned with the development of a reliable and efficient resilient modulus test of subgrade and subbase materials. In recounting this development, this research report has been divided into fourteen chapters. As an introduction to this effort, this first chapter has described a few of the problems associated with the current testing setup and method used in determining the resilient modulus of soils. Chapter 2 presents a literature review that describes both the resilient modulus concept and the state of knowledge regarding the behavior of soils subjected to dynamic loading. The characteristics of the resilient modulus testing system are described in Chapter 3. Also discussed are a revision of the available instrumentation, the characteristics of the testing system used, the calibration process using synthetic samples of constant properties, and the limitations of the testing system.

A prototype resilient modulus test for subgrade and subbase materials is developed and proposed in Chapter 4. The procedures used by several highway agencies are also described in this chapter. The aspects of the materials collected for experimentation and the process followed in the preparation of the test samples are included in Chapter 5.

An experimental assessment of the effect of grouting the specimens to the end platens is described in Chapter 6. Also explained are such concerns as the minimum amount of time necessary for the grout to cure and the proper water-cement ratio of the grout. Chapter 7 presents an experimental evaluation of the effect of sample conditioning, including the design of the experiment, the collection, and the data analysis. An experimental evaluation of the effect of number of stress repetitions is presented in Chapter 8, along with an explanation of the process involved in the collection and analysis of the data. Chapter 9 documents an experimental comparison of resilient moduli of soils obtained by different laboratory tests. The determination of the elastic thresholds of soils are also discussed in that chapter.

A case study comparison of laboratory tests and field measurements of moduli is presented in Chapter 10. A description of the field testing configuration is also included.

An assessment on the importance of testing replicate samples, along with an estimate of the variability of results owing to sample preparation, is described in Chapter 11. An experimental evaluation of several factors affecting the moduli of soils is presented in Chapter 12. The development of empirical equations useful in the design and evaluation of pavements is also described.

The proposed revised resilient modulus testing method for subgrade and subbase materials is presented in Chapter 13, including all aspects of the testing setup and procedures found to be most appropriate. Finally, the summary, conclusions, and recommendations of this research effort are presented in Chapter 14.

CHAPTER 2. LITERATURE REVIEW

INTRODUCTION

This chapter summarizes the results of a literature review of the fundamentals of resilient modulus testing; this review is followed by a general overview of the non-linear stress-strain behavior of dynamically loaded soils. The factors affecting the deformational characteristics of soils are also described.

THE RESILIENT MODULUS CONCEPT

Ideally, to estimate the resilient moduli of pavement materials in the laboratory, one would apply stress-state histories to a specimen simulating a moving wheel load passing over the representative element at some depth in the structure. In such a setup, the elements in a pavement structure are subjected to a series of rapidly applied and rapidly released stresses on vertical and horizontal planes. While the magnitudes of the stress variation will differ between points in the same layer, the basic pattern is similar throughout the pavement structure.

Seed and McNeill (Ref 56) made one of the earliest attempts to duplicate the stress-state history by considering the actual variation in vertical stress on a soil element at a depth of 27 inches below the surface of the pavement at the Stockton test track (see Figure 2.1). Owing to the limitations of their test equipment, they did not use the actual form of the vertical stress that was observed; rather, they chose to use a square wave in their laboratory investigations. Figure 2.1 shows the changes in soil element stress caused by a moving load, as reported by Seed and McNeill in 1958.

Barksdale (Ref 57) observed that vehicle speed and depth beneath the surface of the pavement are of great importance in selecting the appropriate vertical compressive stress pulse time for use in repeated load testing. Using the results of a linear elastic finite element representation of a typical pavement, he established that for full-depth construction with 5 to 12 inches of asphalt concrete and with vehicle speeds of 50 to 60 mph, pulse times of 0.03 to 0.05 seconds are appropriate.

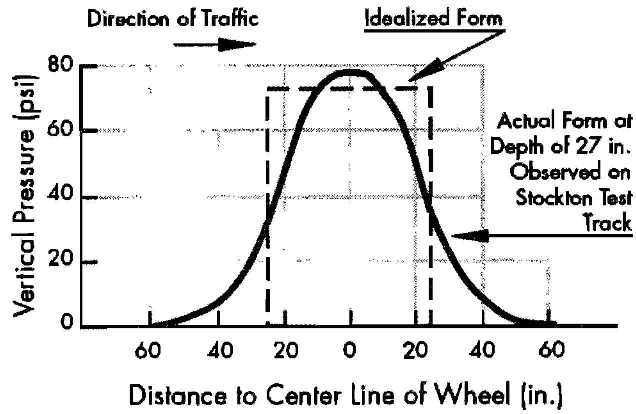


Figure 2.1 Changes in stress on soil element caused by a moving load, as shown by Seed and McNeill (Ref 56)

Terrel (Ref 58) observed that, since asphalt mixes are viscoelastic materials, a computed value of modulus will be dependent upon the rest period between individual pulses, and that the viscoelastic response must be included as a parameter in the material characterization. Terrel concluded that, from the influence of the shape of the wave pulse, either the triangular or the sinusoidal stress pulse produces similar effects on the resilience characteristics of the materials, and that a resting time between the individual pulses of about 0.7 to 2 seconds was a reasonable approximation of the actual conditions within a pavement layer.

Traditionally, the resilient moduli of cohesive and cohesionless materials have been determined in a repeated load triaxial compression test known as the resilient modulus test (or M_R test). The equipment used in this type of test is similar to that used in common triaxial testing, though in this case some modification was required to facilitate the internally mounted load and deformation transducers. Because transducers are located inside the triaxial chamber, air is generally used as the cell fluid to provide confinement to the test samples. A triaxial cell considered suitable for use in repeated load testing of soils is shown in Figure 2.2.

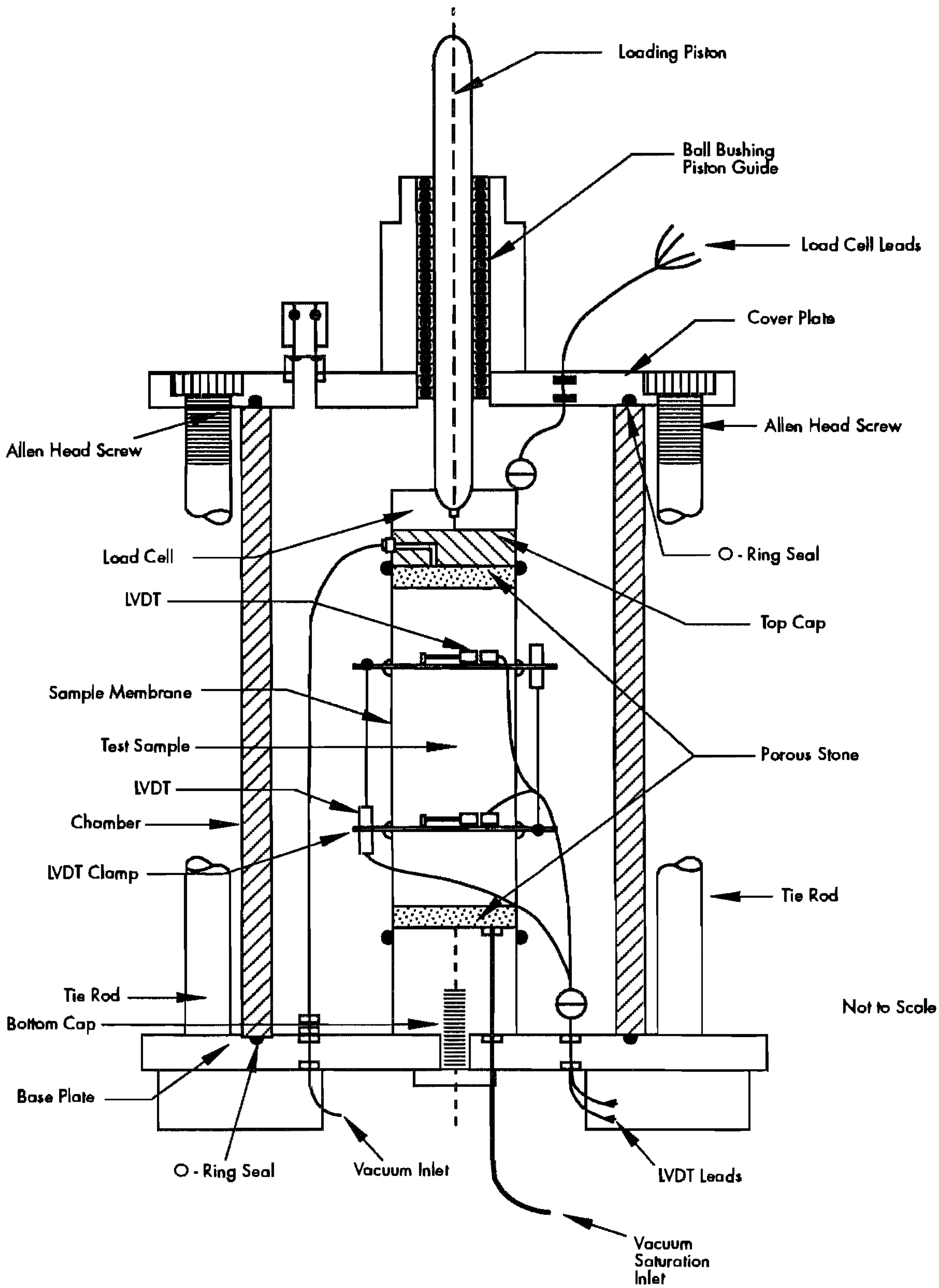


Figure 2.2 Triaxial cell traditionally considered suitable for M_R testing

During the M_R test, specimens are subjected to testing sequences that consist of the application of different repeated axial deviator stresses (σ_d) under different confining pressures (σ_3). Also during the test, the recoverable induced axial strain (ε_a) is determined by measuring the resilient deformations of the sample across a known gauge length.

Figure 2.3 illustrates the typical pattern of soil deformation, with the number of load applications and the sustained confining pressure observed in

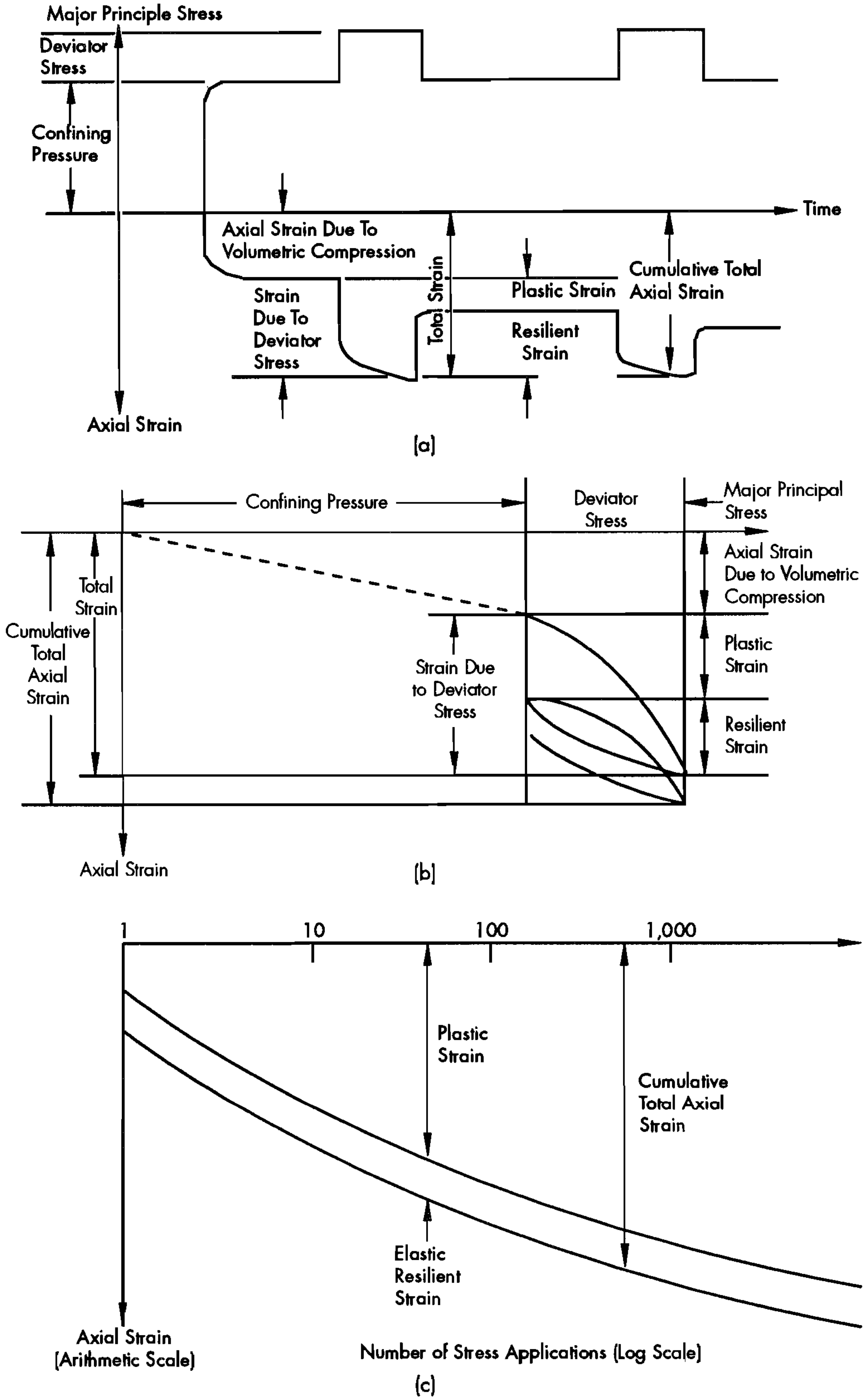


Figure 2.3 Pattern of soil deformation under repeated loading and a sustained confining stress (Refs 4, 8). Shown are: (a) stress-strain-time relationships, (b) stress vs strain relationships, and (c) axial strains vs number of stress repetitions

this type of test documented by Vinson (Ref 8). First, there is a small volumetric compression of the specimen when the confining pressure is first applied. Applying the deviator stresses results in an immediate axial deformation followed by a plastic deformation while the load is sustained, with a rebound occurring once the load is removed. The rebound or resilient deformation remains about the same during the testing process and throughout a large number of applications.

The axial deviator stress is defined as the relation between the applied axial load (P) over the cross-sectional area of the sample (A):

$$\sigma_d = P / A \tag{2.1}$$

The axial strain is defined as the relation between the axial deformation (Δ) over the gauge length (L_g) that such deformation refers to. It is expressed as:

$$\varepsilon_a = \Delta / L_g \tag{2.2}$$

Thus, the resilient modulus (M_R), which is an estimate of the dynamic Young's modulus (the dynamic secant Young's modulus), is defined as the ratio of the applied repetitive axial deviator stress to the recoverable or induced elastic axial strain:

$$M_R = \sigma_d / \varepsilon_a \tag{2.3}$$

Resilient modulus tests made on cohesionless materials have demonstrated the highly significant effect of confining pressure on modulus results. Traditionally, a number of different expressions have been proposed to represent the influence of such stresses on the moduli. These expressions include:

1. Modulus dependent on confining pressure:

$$M_R = K_1 \sigma_3^{K_2} \tag{2.4}$$

2. Modulus dependent on the first stress invariant:

$$M_R = K_1 \theta^{K_2} \tag{2.5}$$

3. Modulus dependent on mean normal stresses:

$$M_R = K_1 (\sigma_o)^{K_2} \tag{2.6}$$

where

M_R = resilient modulus determined from repeated load test,
σ_3 = total confining pressure,
θ = first stress invariant, or sum of principal stress, $\sigma_d + 3\,\sigma_3$,
σ_o = mean total normal stress, $\theta/3$, and
K_1, K_2 = experimental constants determined from a set of test results, with the use of statistical regression tools.

Figure 2.4, taken from Monismith (Ref 27), shows typical test results that illustrate these relationships for cohesionless soils. The relatively high degree of scattering in the testing data observed in this figure generated concerns about the repeatability of the testing approach.

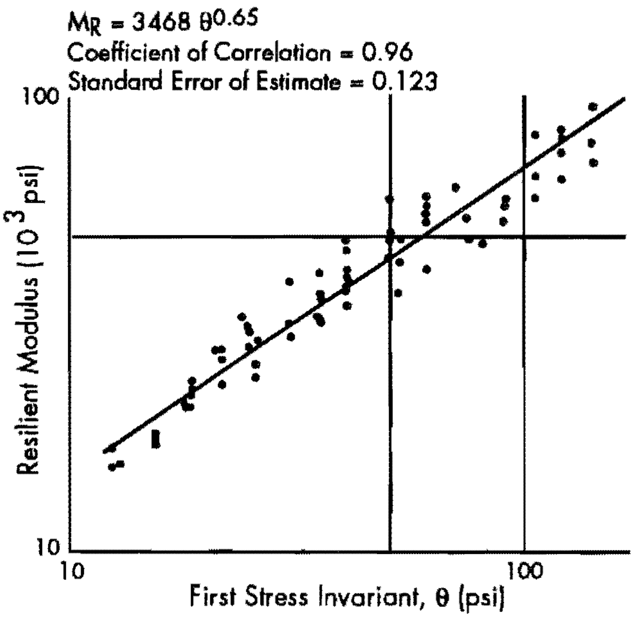


Figure 2.4 Typical test results of modulus versus sum of principal stresses (base course material), as shown by Monismith (Ref 27)

Unlike granular materials, the deformational characteristics for cohesive soils are somewhat independent of the confining pressure (Refs 6, 19, 27, 45); in addition, it has been documented that the most significant effect on the moduli of fine-grained soils is caused by the axial deviator stress applied to the specimen during the test.

To interpret test results for cohesive soils, researchers have used Equation 2.7 (below) to express the resilient moduli obtained in a repeated load triaxial test.

$$M_R = K \sigma_d^{\,n} \tag{2.7}$$

where

K, n = experimental constants determined (using statistical tools) from a set of test results.

Figure 2.5, taken from Thompson (Ref 54), illustrates a typical test result for this type of relationship between the resilient moduli and the applied deviator stresses. As can be noted in this figure, the influence of the deviator stress on the resilient modulus of a subgrade soil is plotted on an arithmetic scale.

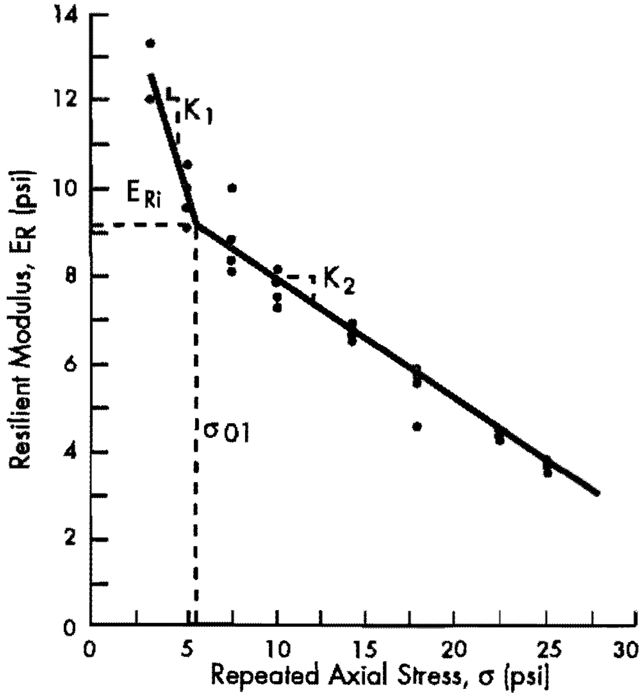


Figure 2.5 Typical variation of modulus versus deviator stress on cohesive soils, as shown by Thompson (Refs 19, 54)

Thompson explained that these graphs were developed based on an extensive resilient testing program carried out at the University of Illinois. He proposed the use of "E_{Ri}" (shown in Figure 2.5) as an effective indicator of a soil's resilience behavior, and added that E_{Ri} (the resilient modulus at interception) is typically associated with a repeated deviator stress of about 6 psi.

However, because the parameter E_{Ri} is not based on any fundamental concept of the behavior of dynamically loaded soils, the introduction of this term by Thompson has met with some opposition. Furthermore, the slopes K_1 and K_2, which have generally been reported by several researchers (Refs 5, 6, 19, 27, and 45), are also highly questionable and deserve a thorough examination.

For instance, the presence of a higher slope (K_1) at lower magnitudes of deviator stresses may be only apparent, since the variability of the M_R values used to determine this slope is extremely high. In addition, it seems that such variability is more likely to be caused by the limitations of the measuring devices and/or by compliances of the testing equipment, rather than by any fundamental behavior of soils. Perhaps the presence of an E_{Ri} corresponding to a 6-psi deviator stress may actually be an indication of the limitations of such a testing device in obtaining reliable measurements of modulus.

From another point of view, the resilient modulus is still observed as a stress-dependent factor, rather than as a strain-dependent parameter. Yet it is now strongly believed that what actually governs the dynamic behavior is the induced elastic strain amplitudes experienced by the materials as responses to applied loads or stresses, and not the magnitudes of such loads or stresses.

Accordingly, it would be useful to include in this report the fundamentals of the non-linear stress-strain behavior of dynamically loaded soils, a topic which is discussed below.

FUNDAMENTALS OF THE NON-LINEAR STRESS STRAIN BEHAVIOR OF SOILS

Whether obtained from triaxial or torsional types of tests, or from cyclic or dynamic tests, the non-linear stress-strain behavior of soils can be observed to have a particular shape, as shown in Figure 2.6. Thus, in dynamic problems, either in compressional or torsional types of motions, this curve is represented by: (1) the initial tangent modulus, E_{max}; (2) the stress at failure, σ_{max}; and (3) the curve linking E_{max} and σ_{max}, which is called the "backbone curve."

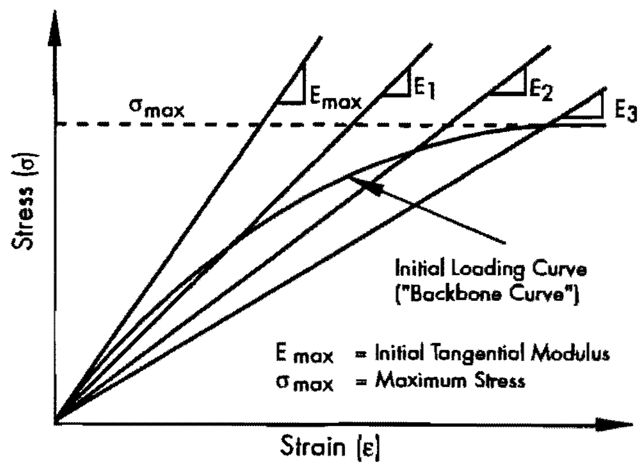


Figure 2.6 Non-linear stress-strain behavior of soils

From the initial loading curve, the initial tangent modulus and the secant moduli of the materials are defined (see Figure 2.6). Then, a plot is developed showing the variation of the secant moduli with the strain amplitudes, ε. For an understanding of the dynamic behavior of soils, the most commonly used plot in geotechnical engineering practice is presented in arithmetic scale for the modulus, and in logarithmic scale for the strains, as illustrated in Figure 2.7.

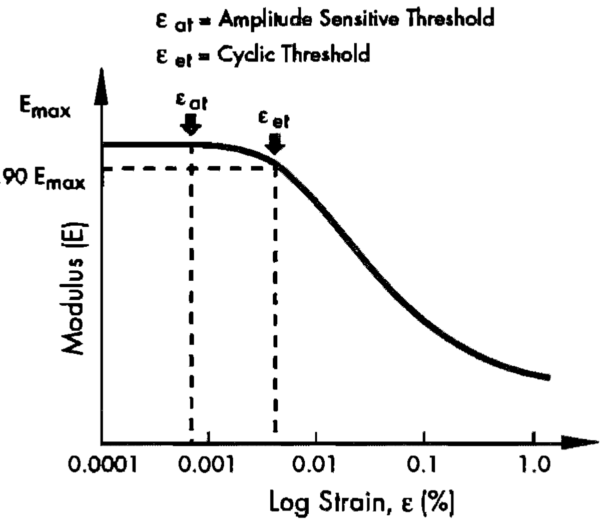


Figure 2.7 Variation of the modulus versus log strain amplitude—the key plot for understanding the dynamic behavior of soils

To understand the dynamic behavior of soils, the plot presented in Figure 2.7 can be divided into the following three ranges: (1) the small-strain range, (2) the non-linear elastic range, and (3) the non-linear range.

(1) The small-strain range is demarcated by a strain threshold called the amplitude sensitive threshold, ε_{at}, as an upper bound. This range is characterized as having a constant value of the modulus equal to E_{max}. Within this scheme, the soil exhibits linear-elastic behavior in which the moduli are independent of the strain amplitudes. In addition, because the induced strains are very small, there is no increase in pore water pressures affecting the stress measurements. Furthermore, field seismic measurements of dynamic soil properties operate best at this specific strain range.

(2) The non-linear "elastic" range is demarcated by the ε_{at} and by a second threshold strain that is related to cyclic loading. This second threshold, which can be seen as the strain at yield of the material, is called the cyclic threshold, or simply the strain-elastic threshold, ε_{et}; however, this threshold is not well defined and depends on several factors and soil characteristics. Within this range, it is expected that neither changes in the material behavior, nor developments of pore water pressures in the soil structure will be observed. In general terms, this strain-elastic threshold is defined when E is approximately 90 to 95 percent of E_{max}, which may occur within strains of roughly 0.001 to 0.01 percent, as shown in Figure 2.7.

(3) The non-linear range is demarcated by the strain-elastic threshold, ε_{et}, as a lower bound. In this range, the material behaves non-linearly, resulting in degradations in the moduli of clays and saturated sands; in addition, pore water pressures are generated, with hardening in dry sands also occurring. This is the range in which most of the severe changes in modulus occur; it is also the range in which the resilient modulus test performs best.

A normalized modulus is another way of presenting the stress-strain behavior. Seed et al (Ref 55) was the first to use this type of plot in which the shear modulus, G, was normalized and plotted against the log of the shear strains, γ, as illustrated in Figure 2.8.

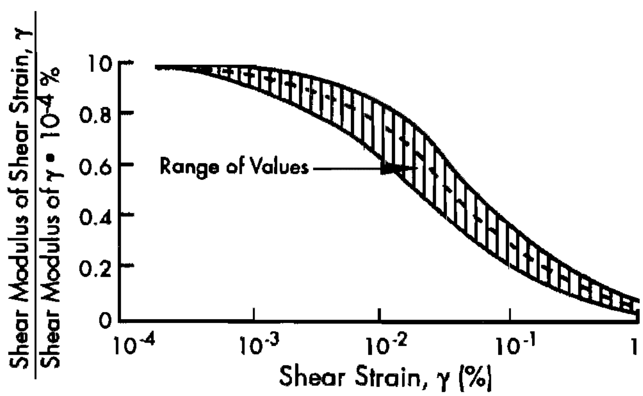


Figure 2.8 Typical variation of the normalized shear modulus with the log of shearing strains, as shown by Seed and Idriss (Ref 55)

This normalized behavior is easily determined using laboratory testing as long as the maximum modulus of the test material is also defined. (In M_R tests, the maximum modulus value is hardly ever detected because testing is carried out in the non-linear range of the material.) The usefulness of this type of information is that, once the maximum modulus of the material is obtained from field (seismic) tests, any modulus at any strain amplitude can be easily estimated.

The deformation characteristics of soil materials will be the same for either dynamic or cyclic loading, as long as they operate within the low-to-intermediate strain amplitudes. And while damping is another important parameter that is involved in cyclic loading, it is of no interest in present resilient modulus tests. For this reason it is excluded from further consideration in this report (although more study of the material damping factor is recommended).

Several researchers (Refs 52 and 55) have proposed analytical methods to predict the non-linear stress-strain behavior of soils. Kim et al (Ref 60) documented the proposed models in terms of the shear modulus.

One of the well-accepted expressions capable of modeling soil behavior precisely is the Ramberg and Osgood expression (Ref 52). This expression was first used in the non-linear analysis of structural frames for modeling the degree of ductility of the elements. Applied to soils, the Ramberg and Osgood expression was first used by Anderson (Ref 18) to describe the variation in normalized shear modulus with shearing strain. The general form of the Ramberg and Osgood relationship is presented as:

$$\frac{G}{G_{max}} = \frac{1}{1 + \alpha * \left[\dfrac{\tau}{\tau_y}\right]^{r-1}} \tag{2.8}$$

where

G = the shear modulus,
G_{max} = the maximum shear modulus at yield,
τ = the applied shearing stress,
τ_y = the shearing stress corresponding to the yield, and
α, r = regression coefficients.

Although Equation 2.8 shows the Ramberg and Osgood expression in terms of the normalized shear modulus, it is quite feasible to formulate a similar expression in terms of a normalized Young's modulus. This will be applicable in cases where the material is subjected to a dynamically axial type of motion, which is the case in the resilient modulus test.

However, in order to apply the Ramberg and Osgood expression it is necessary to identify the maximum modulus and the stress at yield. This is critical in the M_R test because the elastic threshold that defines those parameters is located at very small strain amplitudes strain amplitudes that are beyond the capacities of the measuring devices generally used in M_R systems.

PARAMETERS AFFECTING THE MODULUS OF SOILS

Several researchers have identified factors influencing the modulus. In particular, Seed et al (Ref 5) listed the following: (1) the number of stress applications; (2) the stress intensity; (3) the age at initial loading; (4) the stress intensity; (5) the method of compaction; and (6) the compaction and water content.

A more comprehensive list of the factors affecting the dynamic modulus of soils is the one provided by Richart et al (Ref 44), who explained the dynamic behavior in terms of the shear modulus. The most important factors listed were: (1) strain amplitude; (2) mean effective principal stress; (3) void ratio; (4) number of cycles of loading; (5) degree of saturation; (6) overconsolidation ratio; (7) loading frequency; (8) thixotropy; and (9) natural cementation. These factors, obviously, affect in the same degree the resilient modulus of the material.

Figure 2.9 presents the typical trends of the modulus variation with the logarithmic of the elastic strain amplitude for the main influencing factors. Shown in this figure are the following: (a) as the strain amplitude increases, the modulus of the material decreases; (b) as the mean effective principal stress increases, the modulus increases; (c) as the void ratio of the sample decreases, the modulus increases; (d) as the number of stress repetitions increases, at lower strain amplitudes, there is no effect on the modulus, but at larger strain amplitudes, the modulus values vary uncertainly; (e) as the degree of saturation of the sample increases, its modulus decreases; and (f) as the time increases, the modulus increases as well.

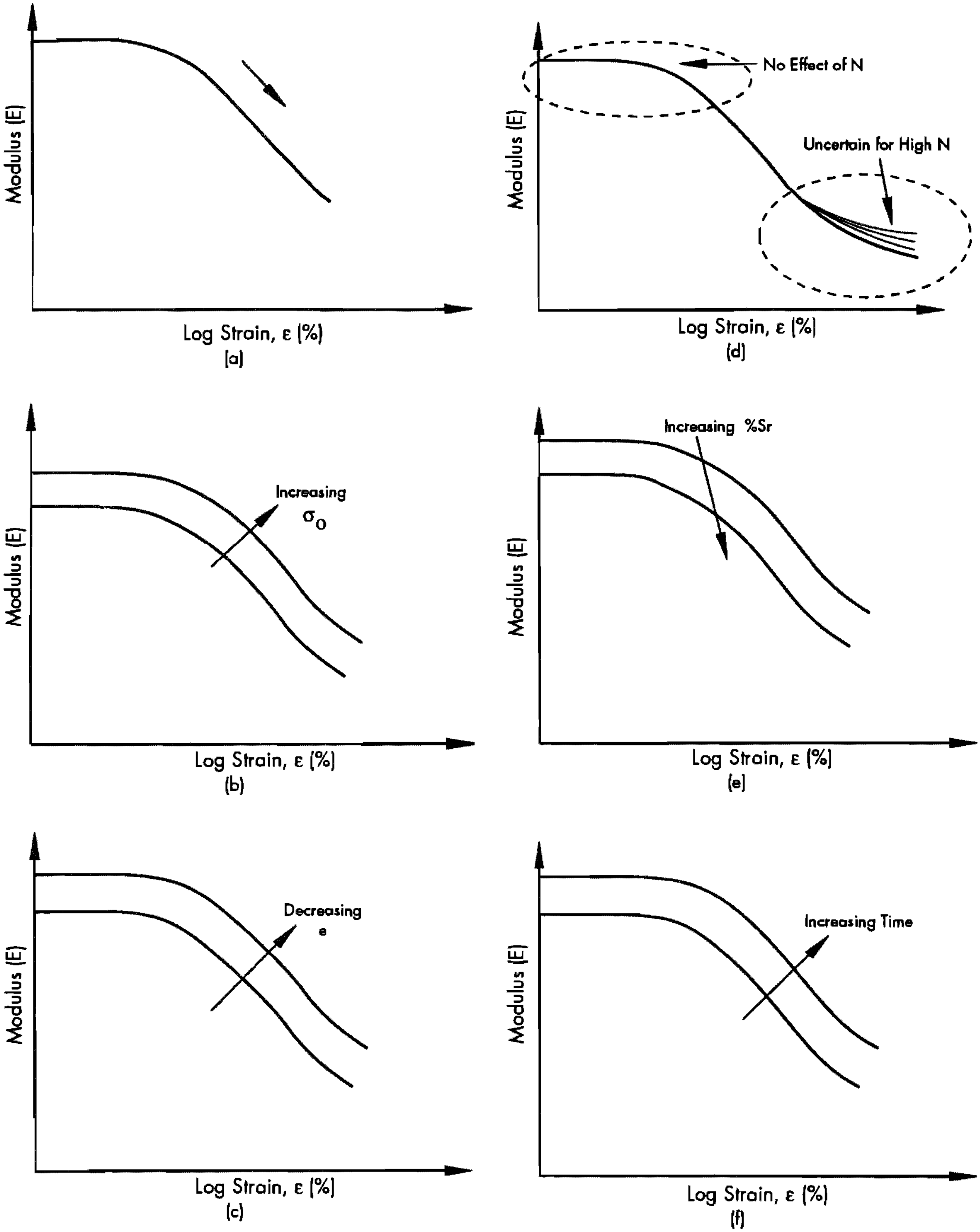


Figure 2.9 **Typical trends of the modulus versus the logarithmic of the elastic strain. Shown are: (a) the effect of elastic strain; (b) the effect of mean effective principal stress; (c) the effect of the void ratio; (d) the effect of number of stress repetitions; (e) the effect of the degree of saturation; and (f) the effect of time**

Regarding the degree of saturation of the material, it can be added that such an effect, observed mainly on cohesive materials, is caused by negative capillary stresses that influence the values of the mean effective principal stresses, even at constant total stress conditions. Elfino (Ref 7), when modeling field moisture conditions in resilient modulus testing, observed that (1) the soil gradation influences the soil-water retention characteristics and the capillary saturation height of the soil materials, and (2) that the greater the height, the higher the capillarity suction and negative pore water pressures, and hence, the stiffer the soil mass. In sands, it has been demonstrated that this factor has very little effect.

Regarding the time effect on the moduli, it can be added that this factor is mainly significant on clayey soils (and can be quite large in soft clays). Anderson (Ref 18), who studied the long-term time effect on stiffness of soils, explained that this effect is caused by the regain in strength and stiffness of the material with time at a constant confining pressure, and that this factor can be quite important when comparing field with laboratory measurements.

Regarding the effect of the overconsolidation ratio, Hardin and Black (Ref 59) reported that the modulus increases as the overconsolidation ratio of the material increases. In addition, they suggested that such a relation is controlled mainly by the plasticity index of the material.

The loading frequency effect, which should be termed more properly the strain rate effect, has been demonstrated to be unimportant for sands; but for clays, it has a minor effect, as explained by Kim (Ref 60). He documented that several researchers have found that an increase in excitation frequency from 1 to 10 Hz caused an increase of the order of 10 percent in modulus, and that the effect increases as the plasticity index and water content of the fine-grained soils increase.

The effect of the number of stress repetitions on the moduli at larger strain amplitudes is generally uncertain. Nevertheless, several researchers (Refs 18, 55, 59) have stated that at those large strain amplitudes, the moduli of cohesionless materials increase with loading cycles. This behavior has been explained by fabric reorientation and particle relocation of these types of materials. In contrast, it has also been observed that the moduli of cohesive soils decrease when induced at large strain amplitudes with number of stress repetitions. The behavior has been explained by the continuous development of excessive pore water pressures in the soil mass during the repetitions of the stress cycles.

Finally, the natural cementation, which apparently causes shifts of the non-linear stress-strain curve, causes drastic reductions of the moduli once the induced strain amplitudes exceed the amplitude-sensitive threshold.

CHAPTER 3. CHARACTERISTICS OF RESILIENT MODULUS TESTING SYSTEMS

INTRODUCTION

This chapter describes (1) current state-of-the-art equipment used for resilient modulus testing, (2) the resilient modulus testing system used in this study, and (3) the development of synthetic samples for equipment evaluation. The chapter concludes with an evaluation of the resilient modulus testing system itself.

EVALUATION OF EQUIPMENT USED FOR RESILIENT MODULUS TESTING

In summarizing and comparing equipment available for resilient modulus testing, this section addresses the following specific items: (1) loading systems, (2) system instrumentation, and (3) data acquisition and control systems. Additional comments on the testing system are also provided.

Loading Systems

AASHTO T-274 prescribes a load waveform that is either a sinusoid or a pulse. The waveform should have a duration of 0.1 to 0.4 seconds and a cyclic period of 1, 2, or 3 seconds. Load magnitudes can range from 10 lb for soft soils in the triaxial test, to over 2,000 lb for stiff bound materials in the diametral test (ASTM Designation 4123). Equipment manufacturers have relied exclusively on fluid power to apply repeated loads in both triaxial and diametral testing.

While suitable for static or slow displacement testing, mechanical testers employing cams, levers, gear or screw drives have proven unsuitable for repeated loads, especially in a load-controlled mode. Similarly, electromagnetic drive systems, while well suited for metal fatigue testing at frequencies higher than 10 Hz, are not suitable for any aspect of M_R testing (the high currents needed to produce repeated loads create an environment that is not only affected by electronic noise, but is disruptive to other nearby electronic instrumentation).

As for fluid power options available for M_R testing, air and hydraulic oil are the most appropriate. And of these, compressed air is the most popular, inasmuch as it is non-toxic, easy to operate, relatively inexpensive, and available in most laboratories. There are, however, certain disadvantages associated with this load source. These disadvantages relate to the compressibility of the air (which limits the quickness of the load application), the large amounts of energy required to cycle high loads continuously, and the need to limit loads to approximately 2,000 lb.

Hydraulic oil, the other source of load power, also has its advantages and disadvantages. The advantages include quick response, almost no limit of load sizes (limit depends only on the size of the actuator, with actuators of different stroke size and load capacity readily available), and the ability to apply and remove the loads at any frequency. Disadvantages include oil-leakage problems, its relatively high cost, its greater complexity (as compared with pneumatic systems), and its requirement for external cooling systems and noise-reduction chambers. Typical plots of a load application in a time domain using compressed air and a hydraulic oil system are presented in Figure 3.1.

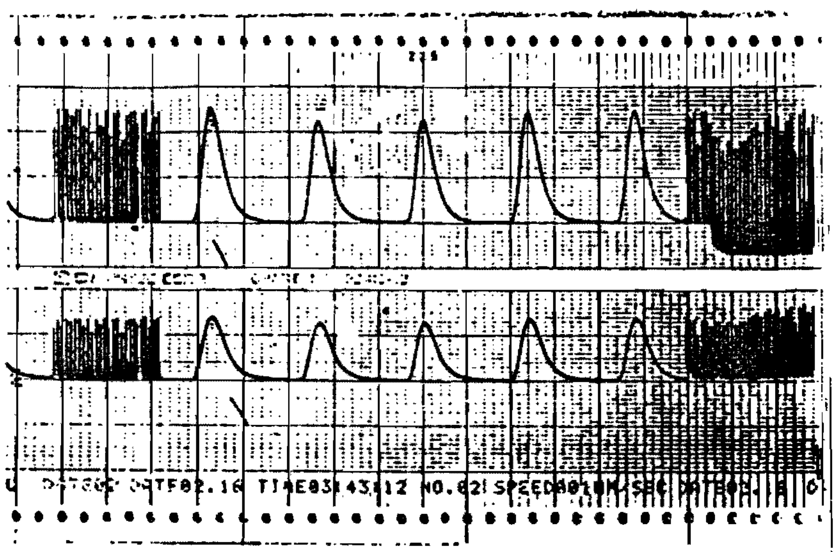
There are two types of control modes for the load application in hydraulic and air systems: open-loop loading and closed-loop loading. Figure 3.2 illustrates schematically the open- and closed-loop loading control systems.

Repeated load modulus systems of the open-loop variety use a source of constant pressure to derive their load pulses. Typically, the actuator cylinder is toggled by a valve between a high pressure source and a low pressure source to gain the desired train of load pulses. Its main advantages include simplicity, reliability, and low cost. The valves used are rugged on/off devices that are easy to service and replace; the actuator can be single acting (unidirectional). Pressure regulators with output gauges (which give the operator a rough idea of applied loads) can supply the high and low pressure.

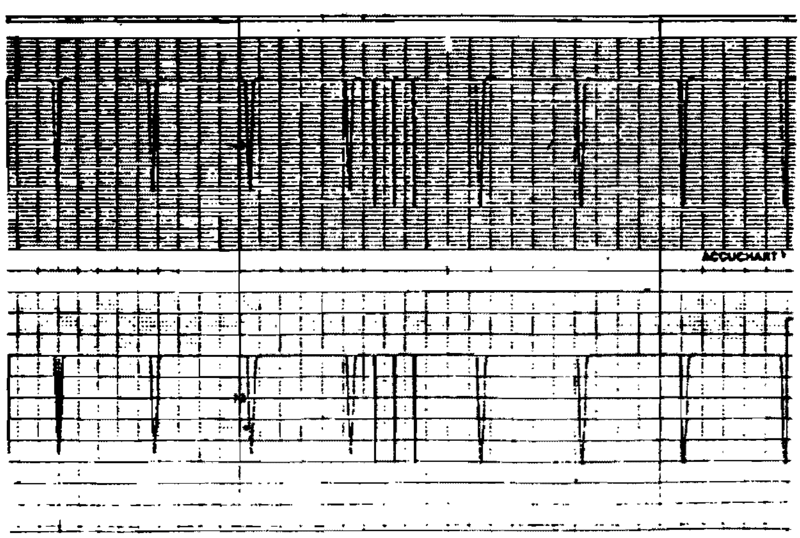
Closed-loop loading systems employ a sensor at the actuator output that can monitor the desired variable, either load or displacement. That signal, which reports the current output status, is called the feedback signal. It is compared to another signal, the input command, at a summing point. The difference between the input command and output status is the error that is used to drive the actuator control valve to minimize error. The main advantage of closed-loop control is its ability to follow command signal input changes within the speed and amplitude capabilities of the actuator. A large industry has evolved in the field of structural-response testing (both destructive and nondestructive) based on the capabilities of these expensive and complex closed-loop systems.

For operation, the actuator of the closed-loop loading system must be double-ended (bidirectional), and the fluid must be ported by a double-acting servo valve (a proportional, electrically driven metering valve manufactured to fine tolerances). A servo-amp drives the servo valve; dynamic response of the complete system with feedback must be optimized or "tuned" for the materials and load frame used. Performance of an improperly adjusted system can range from sluggish to wildly unstable.

An open-loop loading system responds to a command input regardless of either the current



(a)



(b)

Figure 3.1 Load and deformation plots of (a) an open-loop pneumatic device, and (b) a closed-loop electrohydraulic apparatus

output status of the load or the displacement of the actuator. The command input itself is a constant speed setting; once started, the platen moves until shut off, requiring no self-adjusting to maintain speed.

Table 3.1 includes a list of the names and addresses of several U.S. manufacturers of different types of resilient modulus testing equipment.

System Instrumentation

In addition to dynamic load and pressure, resilient modulus testing of diametral and triaxial specimens requires that displacement measurements be recorded electronically. Accordingly, a transducer is used to convert a measurable variable (e.g., load, pressure, deformation) into some sort of electrical signal.

A signal conditioner, used in conjunction with the transducers, is also required in these types of tests. This apparatus first accepts the signal from a transducer and then amplifies it to provide an output voltage signal; this signal varies linearly (with the input measured quantity) and spans a specified full range (e.g., 0 to 10 volts, -5 to +5 volts, 0 to 5 volts).

In resilient modulus systems, load monitoring is most often achieved by strain-gauge load cells. There is a wide selection of load cells, each varying in profile, ruggedness, environment capability, mounting, and, of course, price. Since samples must be stressed axially, it is not difficult for the designer to find space in the "load line" for a load cell.

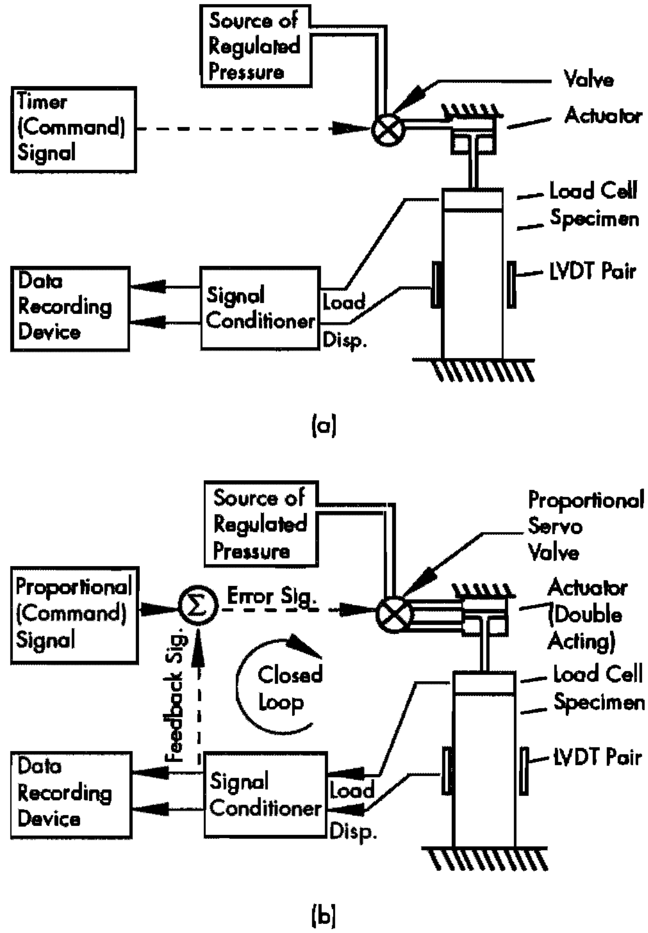


Figure 3.2 Schematic of the load-control modes. Shown are the open-loop system (a), and the closed-loop system (b)

Table 3.1 M_R equipment manufacturers

Name	Mailing Address	Telephone
Interlaken Technology Corporation	6535 Cecilia Circle Minneapolis, MN 55435	(612) 949-1340
James Cox & Sons	P. O. Box 674 Colfax, CA 95713	(916) 346-8322
MTS Systems Corporation	P. O. Box 24012 Minneapolis, MN 55424	(612) 937-4000
Structural Behavior Engineering Laboratories	P. O. Box 23167 Phoenix, AZ 85063	(602) 272-0274
Digital Control Systems	2409 College Ave., Suite 9 Berkeley, CA 94704	(415) 644-3134
H & V Material Research and Development, Inc.	3187 N. W. Seneca Pl. Corvallis, OR 97330	(503) 753-0725

A load cell consists of structures that perform in a predictable and repeatable manner when force is applied. This force is translated into signal voltage by the resistance change of strain gauges applied to the transducer structure. The change in resistance indicates the degree of deformation and, in turn, the load applied. A fixed excitation voltage is applied to the load cell bridge to obtain the changes in resistance.

Displacement measurements are most commonly carried out by linear variable differential transformers (LVDT's); these devices feature little or no hysteresis, "infinite resolution," good stability, and ruggedness. Ordinarily, there is no physical contact between the movable core and the coil structure, thus making the LVDT a frictionless device. The absence of friction and contact between coil and core serves to extend the mechanical life of the LVDT. The frictionless operation, combined with the induction principle by which the LVDT functions, gives the LVDT an "infinite resolution." This means that even the most minute motions of the core can generate output; the readability of the external electronics represents the only limitation on resolution.

Both diametral and triaxial testing invariably use two LVDT's whose outputs may be summed in the signal path; a third or fourth LVDT may be used to read other deflections to estimate the Poisson's ratio or cumulative permanent deformations. The most convenient form of LVDT is the gauge head, which packages body, spring-loaded core, and tip all in one unit, as shown in Figure 3.3. Small gauge heads with precision ball-bearings—ideal for M_R tests—can be found for AC and DC current.

Pressure transducers for triaxial testing may employ either Bourdon gauges or mercury manometers for high or low cell pressures, respectively. Alternatively, transducers of the variable reluctance or strain-gauge type may be employed with suitable signal conditioning.

Signal conditioners are used to condition, amplify, filter, and transmit the signal from the transducer to the data-recording device. Because a signal conditioner should be selected according to the type of transducer to be used, the operator must make electronic adjustments to get meaningful dynamic data. In the case of the LVDT channels, interactive mechanical and electronic adjustments are usually necessary. Any design that blends convenience with operator confidence will increase efficiency. Calibration with laboratory standards should be easy and performed periodically.

Manufacturers can supply conditioning in the following packaging: (1) stand-alone cabinet; (2) multi-channel cabinet with plug-in modules; (3) modules to be installed in users' cabinet; and (4) printed circuit cards requiring mounting and power supply.

Data Acquisition

Data acquisition and control systems are rapidly replacing strip charts and clipboard recorders a result of recent developments in microprocessor technology that have expanded the capabilities of data acquisition units to the extent that they are now highly accurate (with a faster sampling rate), easier to configure for different sampling modes, inexpensive, and have computational and control capabilities. It is this last feature that has enabled the personal computer to become the control center of a very powerful, configurable laboratory data system. Hardware and software have proliferated in recent years, with each year bringing newer developments in the data acquisition field.

The basic elements of an automated data acquisition system include: (1) time-varying signals; (2) an analog-to-digital converter that can digitize the sampled voltages into binary form for all channels simultaneously; (3) a buffer to hold the rapidly sampled set of voltages; and (4) a controller with clock to transmit the necessary commands to the converter and buffer. Figure 3.4 illustrates these elements and their interactions in an automated data acquisition system.

Data acquisition systems, although available in a variety of configurations, are most commonly employed using a host computer (IBM, IBM compatible, Apple, HP, or other). Some of these computers are equipped with a card that fits into an expansion slot, while others have a module cabinet that communicates with the computer via a cable data link.

It should be emphasized here that sampling rates in excess of 1,000 samples per second per channel, which are quite adequate for modulus testing, are widely available in data acquisition add-ons for personal computers at an economical price. Full scale resolution of 12-bits (1 part in 4,096) or 16-bits (1 part in 65,536) provides ample resolution of the sampled signal.

The host computer or microprocessor controls the data acquisition section of the system. Its output includes: a graphic display of sampled dynamic load and displacement waveforms, along with initial data processing to obtain preliminary results; file generation to record and retrieve the testing data; and report generation at the end of the test. In addition, the host computer can be programmed to communicate interactively with the operator at every step of the testing process.

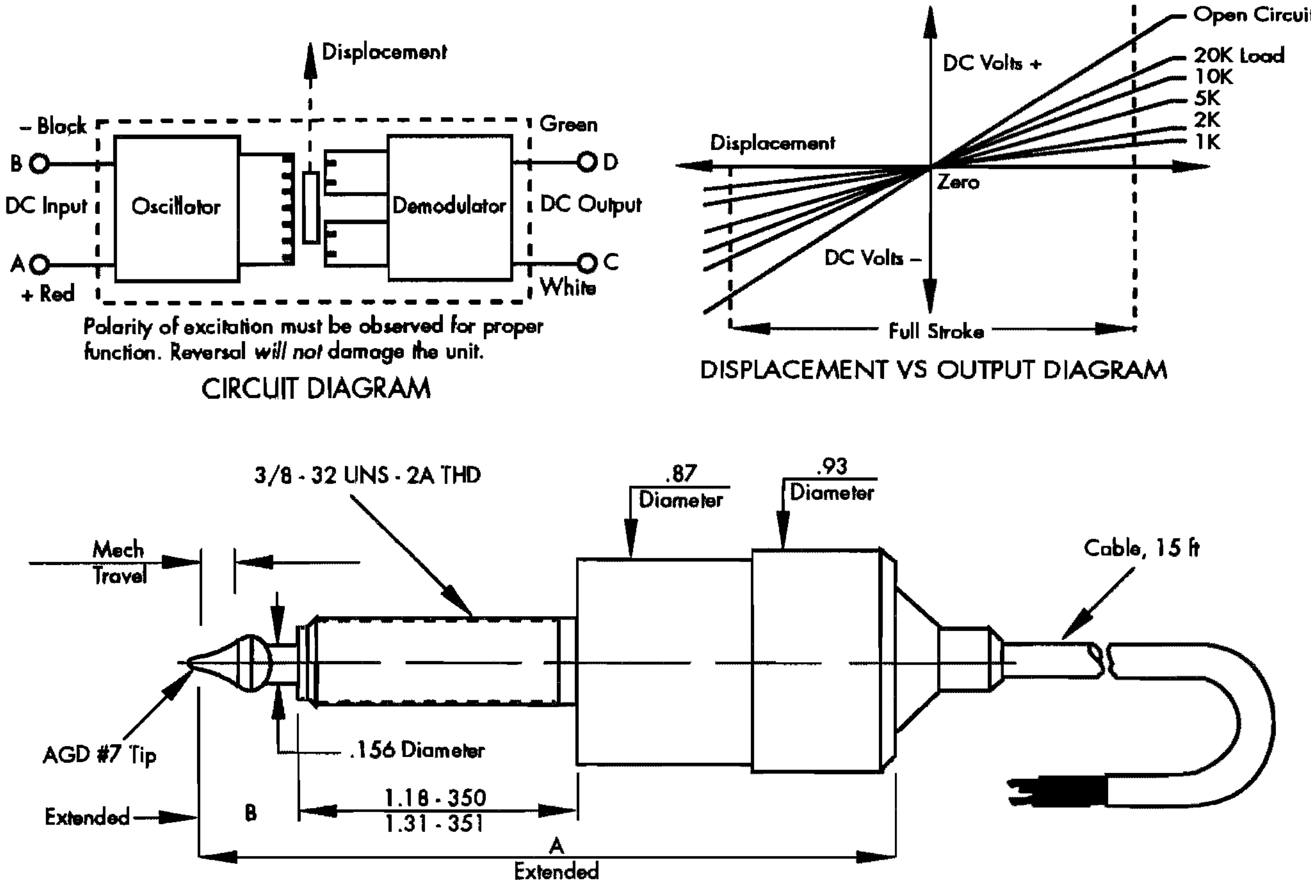


Figure 3.3 Characteristics of a spring-loaded LVDT

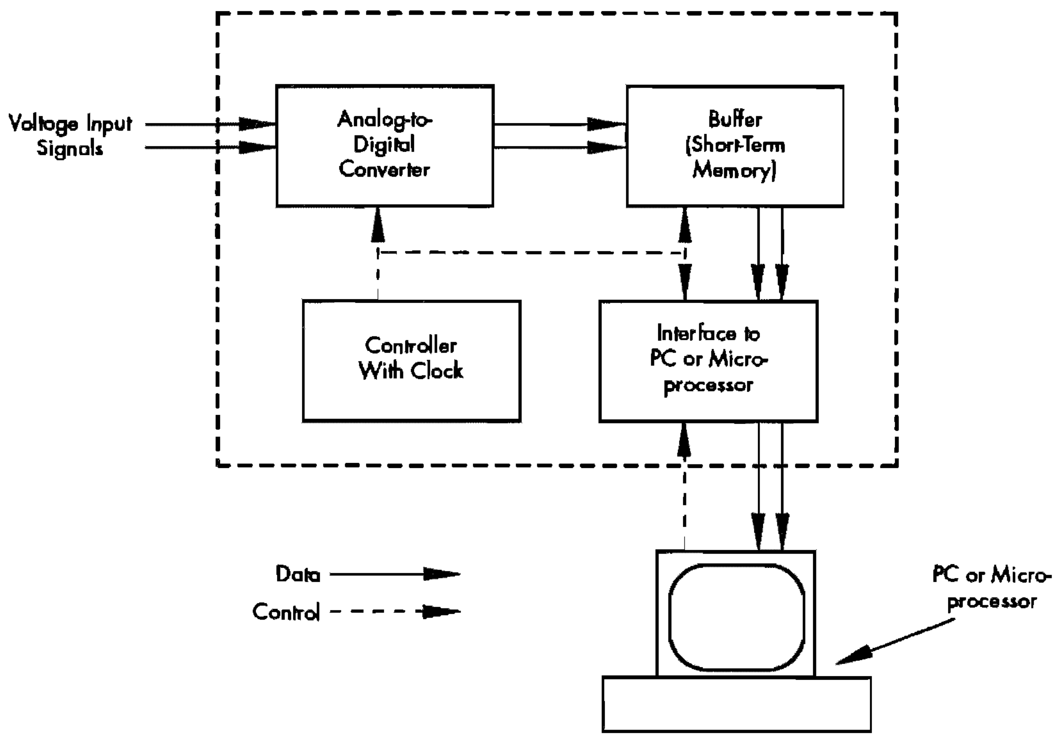


Figure 3.4 Basic elements of an automated data acquisition system

Control Systems

It is now well within the capabilities of the faster personal computers (286- and 386-based) to control closed-loop servo feedback systems. Such high-speed machines can be programmed to (1) scan analog input channels, (2) digitize the signal data, (3) compare the most recent data with the most current value of intended signal, and (4) correct the analog error signal appropriately (output analog from a digital-to-analog converter) in a fraction of a millisecond.

This corrective analog-signal process can be easily used to drive the closed-loop servo valve, as schematically illustrated in Figure 3.5. Several variations of this configuration are possible, with the computer tied either directly to the actuator control duty, or indirectly, commanding and monitoring an analog closed-loop controller.

turn-key operation, and even training, the testing process has been made both more reliable and less complex.

Digital Control Systems, Inc., (DCS) has developed testing control systems that take significant advantage of today's technology. With greater use of menus, graphics, and interactive screen prompting, DCS offers a complete control and data acquisition for servohydraulic systems. Moreover, the DCS control system is designed in such a way that signal functioning, data acquisition, function generation, closed-loop servo-control and hydraulic-pressure control are all provided within a single unit; in addition, the user interacts with the control console entirely through the keyboard of a personal computer. For example, the new DCS software and hardware installed in the laboratories of the Texas Department of Transportation (TxDOT) allow the structuring of a computerized testing

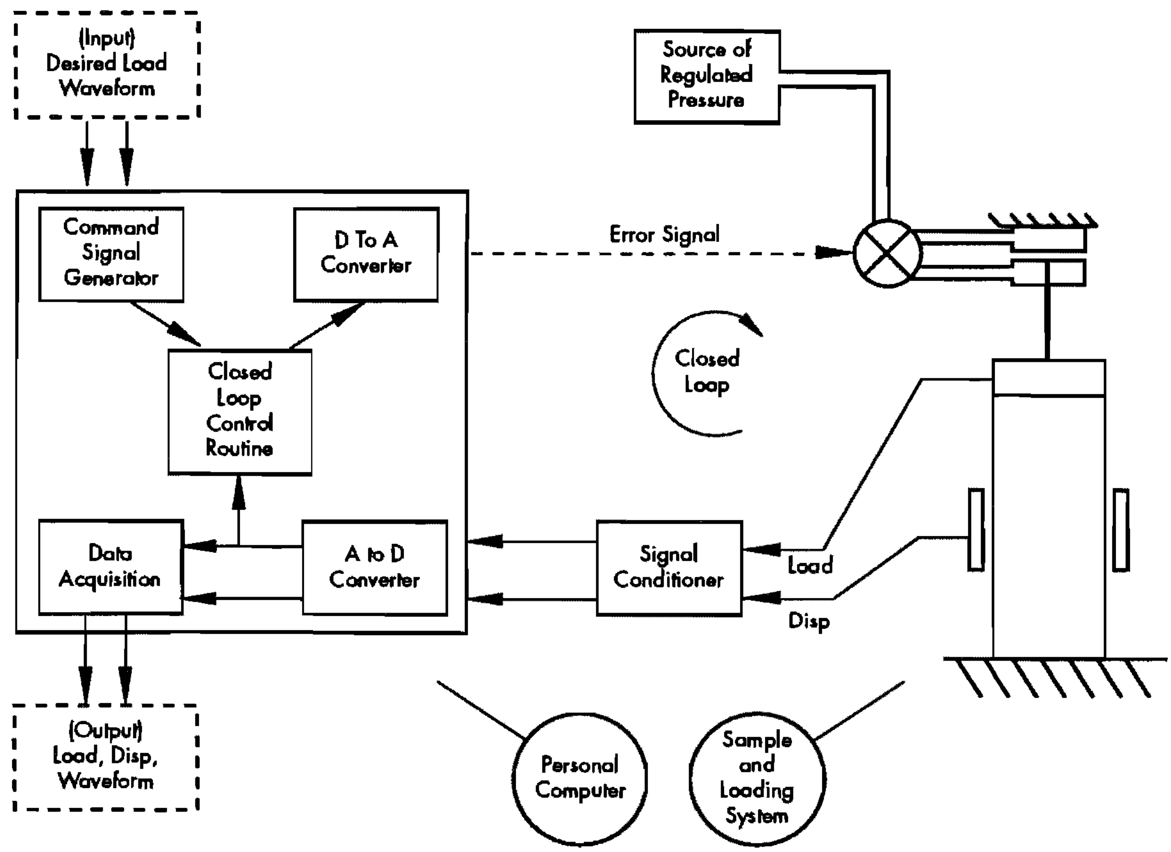


Figure 3.5 Testing configuration using a personal computer directly for closed-loop control

Additional Comments

The complexity of the equipment used for resilient modulus testing, especially in the triaxial setup, can intimidate and frustrate some users. But with personal computers steadily gaining ground as the central instrument of measurement, control, environment that can be used not only for resilient modulus testing, but for many other laboratory tests as well.

Hydraulic resilient modulus equipment (with wholly automated computerized control) ranges in cost from $60,000 to $80,000 and is available from such manufacturers as Interlaken, Cox, and SBEL.

Equipment costs increase according to the number of transducers and other features installed.

Pneumatic equipment with a closed-loop system (a good solution if only soil is to be tested) is generally less expensive. For old or out-of-date hydraulic systems, some manufacturers offer—for \$20,000 to \$30,000—an upgrade package that includes the installation of a computer-based control system.

While recent improvements to the M_R system have mostly involved the data acquisition function, further efforts to refine the system should concentrate on its accuracy and repeatability, ease of use, ruggedness and dependability, and maintainability; the system should also be offered at a reasonable cost. Diligent design and application of the most up-to-date measurement technology can assist manufacturers in achieving these goals.

THE RESILIENT MODULUS TESTING SYSTEM INSTALLED FOR THIS STUDY

Figure 3.6 illustrates the system developed and assembled in the laboratories of the Department of Civil Engineering at The University of Texas at Austin. This resilient modulus testing equipment, set up according to the previous evaluation of state-of-the-art equipment, included the following:

(1) A hydraulic loading system capable of applying repeated dynamic loads controlled under an MTS closed-loop system. The shape and the amplitude of the cyclic loading waveform are set by a function generator, with the loading function continuously monitored by an oscilloscope and a plot-strip chart. The loading pulse duration and the cyclic loading were set at 0.10 and 1.00 second, respectively, with a haversine loading waveform.

(2) Two LVDT's (Lucas Schaevits LBB-375-TR-020, with a calibration range of ±0.02 inches), mounted on opposing sides of the triaxial chamber, were used to monitor axial deformations for the whole length of the specimen (located at the top of the samples). All measurements from the LVDT's are referenced from the base of the triaxial chamber; the average of the two signals is used in the estimation of the strain value, which in turn is used for computing the resilient modulus of the specimen.

(3) A 100-pound load cell (Lebow 3397) mounted inside the triaxial chamber and attached to the loading piston was used to monitor the actual deviatoric force.

(4) An air pressure panel was installed to measure the confining pressures.

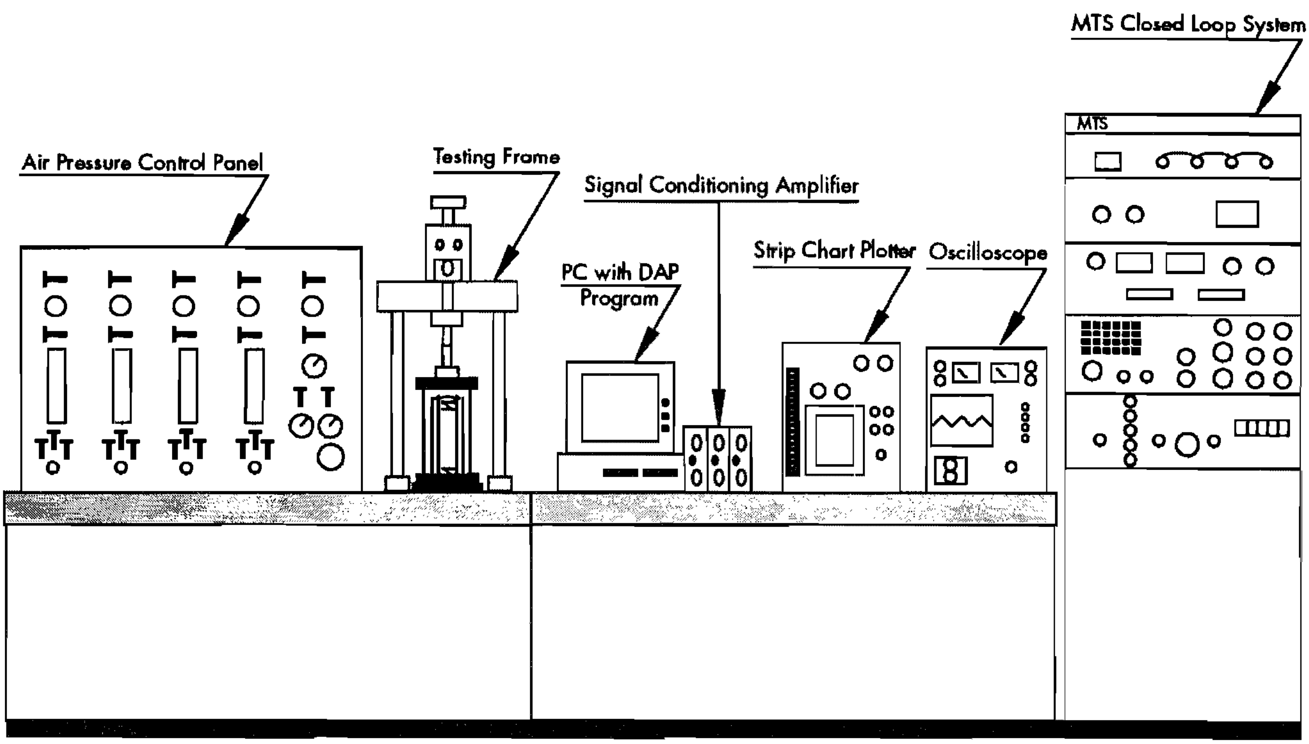


Figure 3.6 Sketch of the resilient modulus testing equipment developed at The University of Texas at Austin

(5) A data acquisition system was developed to record the signals emitted by the transducers. A data acquisition board was mounted inside an IBM XT. This computer was used to host the data acquisition board, which converts the analog signal to digital data for all the transducers (i.e., it was not used to drive the MTS equipment). The software was developed for monitoring, acquiring, plotting, storing, and computing the M_R values of the test samples. To take full advantage of its sampling capabilities, we set this data acquisition system to record 1,000 records per channel per second, so as to improve the accuracy of the results.

It should be noted that higher variability in the results was obtained when the resilient axial strains were smaller than 0.01 percent. Consequently, it was estimated that this system was unable to measure accurately elastic axial strains smaller than 0.01 percent, owing to the resolution limits of the transducers installed and to the particular characteristics of the system itself. This is a factor common to all resilient modulus testing equipment: when the sample undergoes smaller strains, erratic M_R values are calculated.

DEVELOPMENT OF SYNTHETIC SAMPLES FOR EQUIPMENT EVALUATION

One method of evaluating the performance of M_R equipment is to use the equipment to test specimens with known stiffness characteristics. (Such specimens are hereafter referred to as calibration specimens.) Values of M_R determined with the equipment can then be compared with stiffnesses of the calibration specimens that have been established by independent tests. If differences between the measured and calibration stiffnesses are found, then modifications to the equipment and/or procedures can be undertaken.

Synthetic samples were made from urethane elastomers rather than from actual soils. The stiffnesses are conveniently evaluated in terms of Young's modulus, E, which is taken to be equal to M_R for this material. The use of synthetic samples has the following advantages: (1) they are easy to construct and handle; (2) they have stiffness properties that can be determined by independent tests; and (3) they can be tested numerous times by different laboratories.

Calibration Specimens

Calibration specimens were constructed using a two-component urethane elastomer resin system manufactured by Conap, Inc., of Olean, New York. The first component consisted of dicyclohexylmethane-4,4'-diisocyanate for all specimens. The second component consisted of diethyltoluene diamine and methylenedianiline.

One key characteristic of urethane elastomers is their latitude of hardness, which can range from that approximating a very soft subgrade to that approximating a stiff, uncemented base. Other beneficial properties include their toughness, durability, and high resistance to the effects of abrasion, weather, ozone, oxygen, and radiation.

Three individual mixtures were used to create synthetic samples for this research effort. They have been identified (from soft to stiff) as TU-700, TU-900, and TU-960. Following the casting procedures outlined by the manufacturer, each component was measured according to the specified accuracy and mix ratio. After casting, the specimens were cured in the mold for 7 days at atmospheric pressure.

Metal pipe molds having diameters of 1.4, 2.0, and 2.8 inches, with lengths 2 to 3 times the diameter, were used (though for this study the majority of the specimens were 2.8 inches in diameter and 5.6 inches long). These pipes were equipped with an extruder that pushed the specimens out of the molds. The finishing of the urethane specimens consisted of cutting off the top inch and machining the end flat.

Measurements of the Properties of the Calibration Specimens

Several testing methods, including the static unconfined compression, torsional resonant column, and cyclic torsional tests, were used (1) to establish the stiffness characteristics of the three calibration specimens, and (2) to evaluate the variables affecting them.

Static measurements of Young's modulus and Poisson's ratio were determined by applying axial loads on top of each urethane specimen. Axial and radial deformations were measured using proximeters located near the middle and on opposite sides of the specimens. The testing procedure involved simply adding a load and measuring the resulting deformation. The static Young's modulus was calculated by dividing the axial stress by the axial strain; Poisson's ratio was determined from the ratio of the radial strain to the axial strain. Figure 3.7a shows the variation in static Young's modulus with axial strain from the unconfined compression tests. The average values of Young's modulus and Poisson's ratio for the soft (TU-700), medium (TU-900), and hard (TU-960) specimens were:

	Young's Modulus	*Poisson's Ratio*
Soft (TU-700)	1,670 psi	0.48
Medium (TU-900)	6,550 psi	0.50
Hard (TU-960)	32,300 psi	0.47

Dynamic measurements of shear modulus with the shearing strain of the three synthetic samples were determined using resonant column equipment of the torsional fixed-free type. Appendix A includes the basic principles, the characteristics of the equipment used, and the general procedures involved in performing these types of tests. Once the shear modulus, G, and its corresponding shearing strain, γ, were determined, the equivalent dynamic Young's modulus, E, and axial strain, ε_a, were estimated by using the following expressions:

$$E = 2 * G * (1+\nu) \quad (3.1)$$

$$\varepsilon_a = \gamma / (1+n) \quad (3.2)$$

Torsional resonant column and torsional shear tests were performed to determine the effects of: (1) isotropic confining pressure, (2) strain amplitude, (3) loading frequency, and (4) temperature on the dynamic behavior of the three urethane samples. To determine the repeatability of the measurements, we tested each specimen twice; we found that the modulus values for the two test series were within 3 percent—a demonstration of the high degree of repeatability of these tests.

(1) The influence of isotropic confining pressure on small-strain Young's modulus determined by the resonant column tests for the three urethane specimens is shown in Figure 3.7b. All moduli measurements were performed at an equivalent axial strain of about 0.00067 percent after 50 minutes at each pressure. Because the test used log-log plots, moduli corresponding to zero confining pressure are not presented. However, essentially the same moduli were measured at zero confining pressure. Average Young's moduli for the soft, medium, and hard specimens were 2,430 psi, 10,070 psi, and 52,000 psi, respectively. Note that Young's moduli determined by the resonant column are somewhat greater than those determined by the static testing (because of the effect of loading frequency).

(2) The effect of strain amplitude was investigated by testing the specimens at shearing strains ranging from 0.0005 to 0.3 percent. Converting the shearing strains to equivalent axial strains, and shear modulus to equivalent Young's modulus using Equations 3.1 and 3.2, respectively, demonstrated the variation of the Young's modulus with axial strain, as shown in Figure 3.8a. The modulus is observed to be essentially constant over the range of strains tested for all the samples. To obtain a perspective on how the strains used in these tests compare with those generated in M_R testing, the range in strains in the M_R test are also included in this figure for materials, with stiffnesses ranging from 1,000 to 100,000 psi.

(3) The effect of loading frequency was evaluated by using a combination of resonant column and torsional shear tests. Moduli determined by the resonant column test are based on first-mode resonant frequency, which depends on the stiffness of the specimen and on the characteristics of the testing device. For the soft, medium, and hard specimens, resonant frequencies were 27, 56, and 127 Hz, respectively. In the torsional shear test, as in the M_R test, the loading frequency can be varied by changing the input frequency. Moduli determined by the resonant column and torsional shear tests at various loading frequencies and strain amplitudes are plotted in Figure 3.8b. It is interesting to note that while Young's modulus increases with increasing loading frequency, it is independent of strain amplitude. To obtain a perspective on the degree of influence of the loading frequency, all moduli were normalized using the modulus of each specimen (determined at 0.01 Hz) as the basis for normalization, as shown in Figure 3.9a.

(4) The effect of temperature on the urethane specimens was also investigated by testing them at different temperatures. Results showed that the deformational characteristics of the three calibration specimens were highly influenced by the temperature, although lesser effects were evident at low loading frequencies. For instance, Figure 3.9b shows the variation in Young's modulus with temperature and loading frequency for specimen TU-900.

Because the urethane specimens showed stiffness characteristics that are independent of confining pressure, strain amplitude, and stress history, they are thus considered appropriate specimens for use in the evaluation of M_R equipment. But because they showed dependency on loading frequency and temperature, frequency and temperature must be selected for comparing these values of Young's modulus with those to be obtained under the M_R method. For complete information on the properties of these calibration specimens, refer to Stokoe (Ref 11).

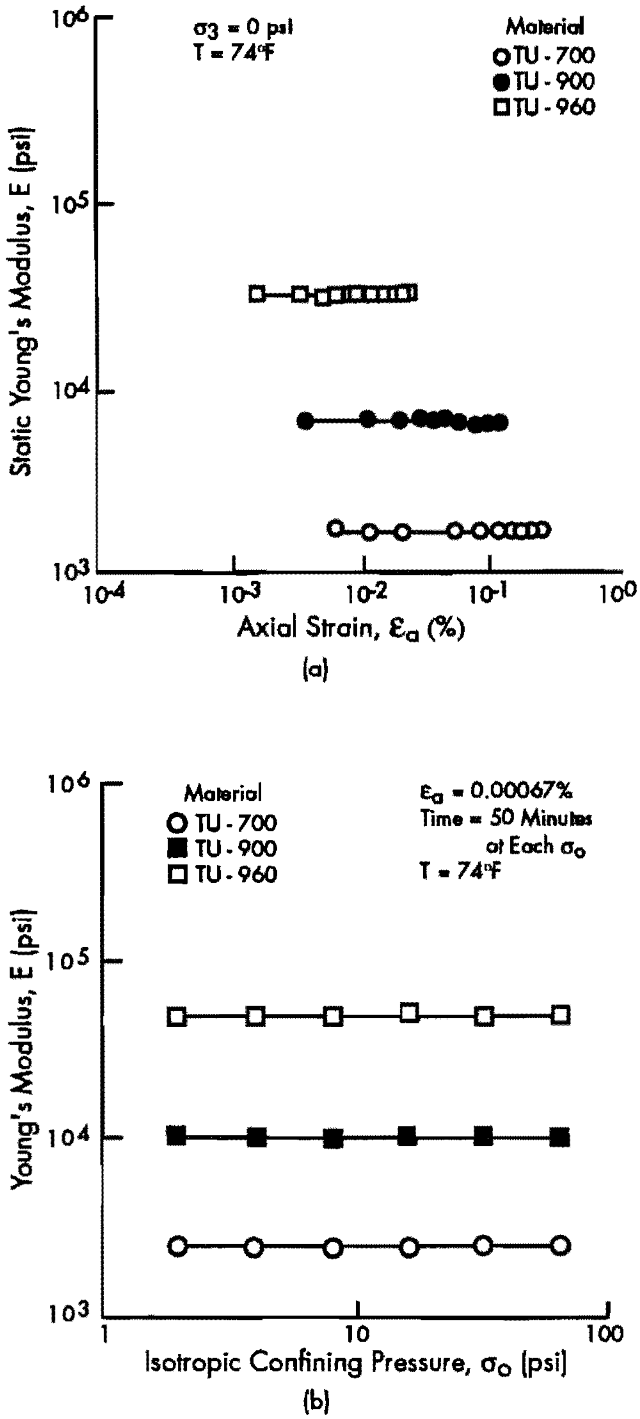


Figure 3.7 Properties of the synthetic samples. Shown are (a) the variation in static Young's modulus with axial strain; and (b) the variation in small-strain Young's modulus with confining stress

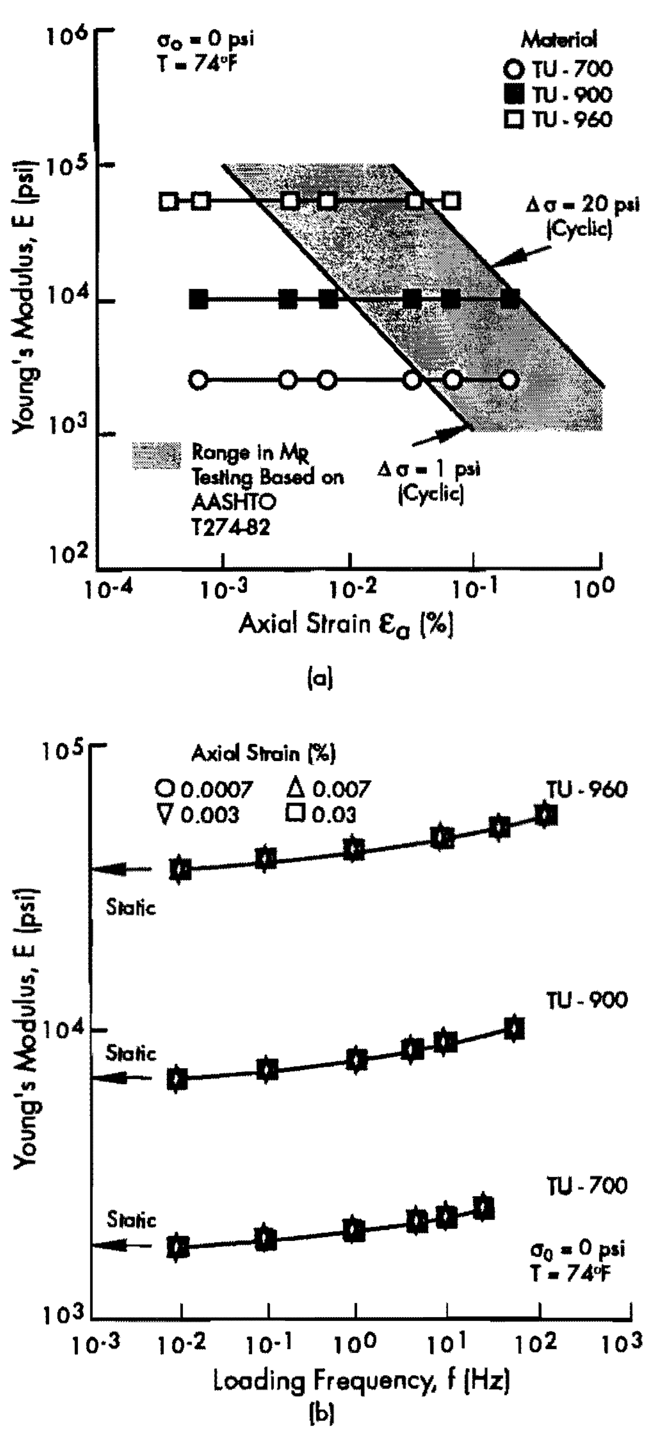


Figure 3.8 Properties of the synthetic samples. Shown are (a) the variation in Young's modulus with axial strain; and (b) the variation in Young's modulus with axial strain and frequency

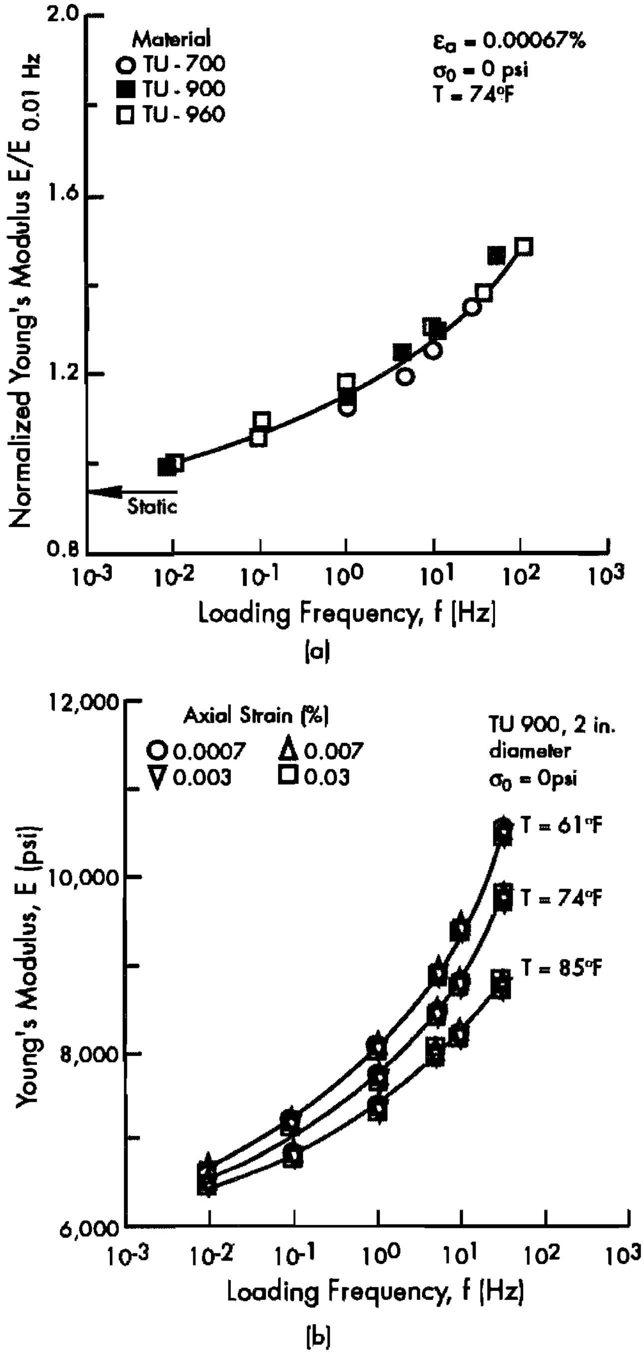


Figure 3.9 Properties of the synthetic samples. Shown are (a) the variation in normalized Young's modulus with frequency; and (b) a typical variation in Young's modulus with temperature

EVALUATION OF THE RESILIENT MODULUS TESTING SYSTEM

As with all cyclic loading equipment, M_R equipment requires careful calibration of each of the deformational and loading transducers. In addition, and equally important, the evaluation of the complete testing system is advisable if accurate results are to be determined. In general, calibrations of the individual transducers are standard procedures, but an evaluation of the entire testing system requires more than routine adjustment of its individual parts. In this study, the evaluation of the M_R testing system was undertaken using the three synthetic samples of known properties previously described.

This evaluation proved to be an involved task, with several problems having to be overcome before a satisfactory state was achieved. Indeed, it was this evaluation that revealed the need for substantial modification of the testing configuration before further testing was possible. (It should be emphasized here that the M_R testing equipment described in the previous section, particularly the configuration of the triaxial chamber, was not the original testing configuration used, but, rather, the final setup suggested by this evaluation.)

To evaluate the equipment performance, the following steps were performed: (1) preliminary testing of the synthetic samples, (2) inspection of the M_R equipment, (3) modification of the M_R configuration, and (4) final testing of the synthetic samples.

Preliminary Testing of the Synthetic Samples

The triaxial chamber for AASHTO T-274 has two different layouts, allowing the measurement of the resilient deformation using internal or externally mounted LVDT's. It has been documented that the use of internal LVDT's clamped to the test specimen increases the variability of results (because of the difficulty in securing such clamps to the specimen). This is further complicated by the fact that the sample has an outer membrane that can slip, inducing small clamp movements that can completely change the estimations of the resilient moduli. On the other hand, soft soils with large permanent deformations make the internal LVDT's go out of range, forcing one to stop the test to readjust the position of the LVDT.

For these reasons, our initial configuration had only one externally mounted LVDT for monitoring the movement of a bracket attached to the piston of the triaxial cell during the action of the loading pulses (see Figure 3.10). Obviously, this configuration assumes that such a movement represents exclusively the axial deformation experienced by the sample.

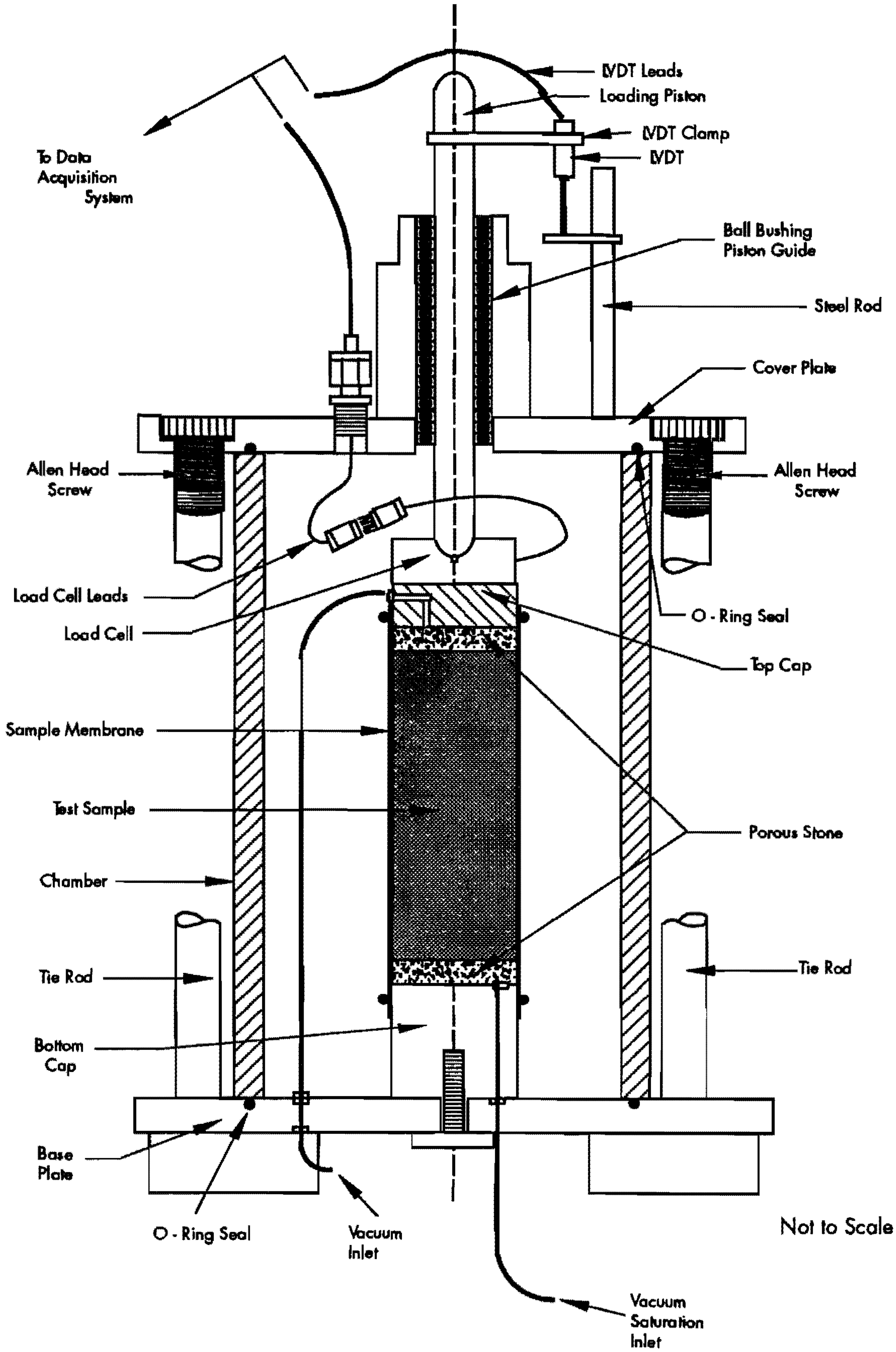


Figure 3.10 Initial configuration of the triaxial cell

The synthetic samples were initially tested by the M_R method to compare results with the resonant column and torsional shear tests. Since the M_R test is set at a loading frequency of 10 Hz and at a laboratory temperature of about 74°F, the expected values of the TU-700 (soft), TU-900 (medium), and TU-960 (hard) were 2,220 psi, 8,921 psi, and 44,197 psi, respectively (see Figure 3.8b).

However, preliminary testing on the three synthetic samples provided unsatisfactory results. For instance, sample TU-960 showed much lower modulus (around 50 percent) than was expected. Samples TU-900 and TU-700 also showed reduced moduli (around 15-20 percent).

It was then concluded that these initial results were not correct because the movements of the LVDT bracket included not only the induced resilient deformation of the test sample, but also some deformations related to deflections of the internal load cell and to movements caused by imperfect contacts between the specimen and the end caps.

Inspection of the Resilient Modulus Setup

The initial results obtained from the testing of the three synthetic samples suggested that better locations of the deformational transducers within the testing configuration were required for reliable estimations of the moduli. Accordingly, we decided to inspect vertical movements at four points within the triaxial chamber while performing the M_R test.

Using the TU-700 synthetic sample (preferred for its low modulus of elasticity), the testing equipment was arranged so that the transducers could be placed at four different locations: (1) the base of the triaxial chamber, (2) the top of the triaxial chamber, (3) the top of the specimen, and (4) the external bracket (LVDT clamp). Figure 3.10 shows these selected locations.

The low-modulus sample was used because it allowed us to record the vibrations within the reliable measurements of the transducers used. These transducers included one proximeter and one microproximeter hooked onto a computerized analyzer that received all the voltage signals sent by the transducers. Under no confining pressure, the TU-700 was subjected to three sets of deviator stresses: 2.42 psi, 5.12 psi, and 8 psi.

The vertical displacements were estimated by first digitizing the signals emitted by the proximeters, and then converting them to absolute displacements using appropriate calibration formulas. Figure 3.11 shows the variation of the vertical movements with deviator stress for the four selected points of the triaxial cell.

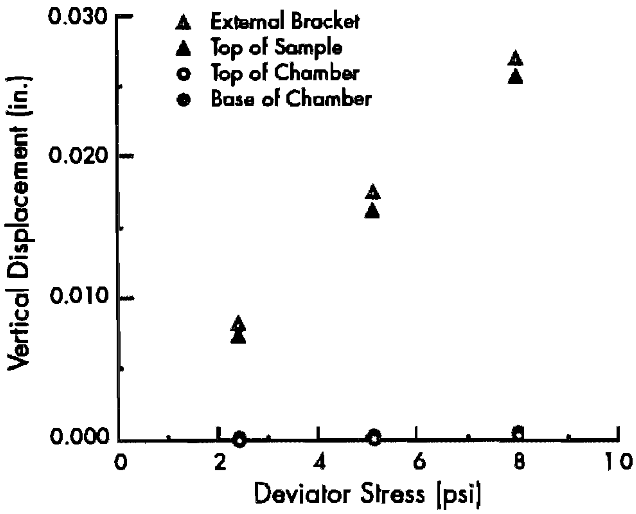


Figure 3.11 Vertical displacements of four selected points in the triaxial chamber observed during the testing of the TU-700 sample

It is interesting to note in Figure 3.11 that more vertical movement is experienced at the LVDT bracket than at the top of the specimen. This observation suggests that the use of an external LVDT can result in misleading estimations of the moduli, and that a more appropriate alternative would be the use of internal LVDT's. Furthermore, it clearly appears that this deviation will be even more significant for samples having higher elastic moduli, since the deformations of the internal load cell and the ones caused by the imperfect contacts of the test sample with the end caps will become more predominant when total movements are smaller.

Figure 3.11 also shows that the variation of the vertical movements of the top and base of the triaxial chamber with deviator stress is almost the same. This clearly suggests that either the base or the top of the triaxial chamber is a good reference point for these measurements. Using these observations, we then modified the initial testing configuration.

Modifications of the Resilient Modulus Setup

Since monitoring the deformations at the top of the specimen eliminates the possibility of including errors caused by deformations of the load cell, the piston, or the connections of the triaxial chamber, we therefore decided to monitor the vertical movements of the sample at this point. In addition, we decided to reference such movements from the base of the triaxial cell.

Two LVDT's (instead of one) were placed inside the triaxial chamber diametrically opposite one another at the top of the specimen. This arrangement allowed the deformation readings to be averaged so as to estimate more reliably the resilient strains and, hence, the moduli of the samples. In addition, each LVDT was then supported by a steel bar attached to the base of the triaxial chamber.

Finally, modifications in the geometry of the top cap were also made to facilitate both the operation of the transducers and the setting of the test sample into the triaxial cell. Figure 3.12 shows the final configuration of the triaxial cell.

Final Testing of the Synthetic Samples

Once the final arrangement was selected, we performed more testing with synthetic samples. The new results, though closer to the moduli than those previously obtained, were still not close enough. In particular, values for the TU-960 specimen (the stiffest sample) were still approximately 50 percent lower. At this point, hydrostone paste was used to improve the connections between the specimen and the top and bottom platens.

By carefully grouting the connections, we were able to achieve an even contact surface—and a

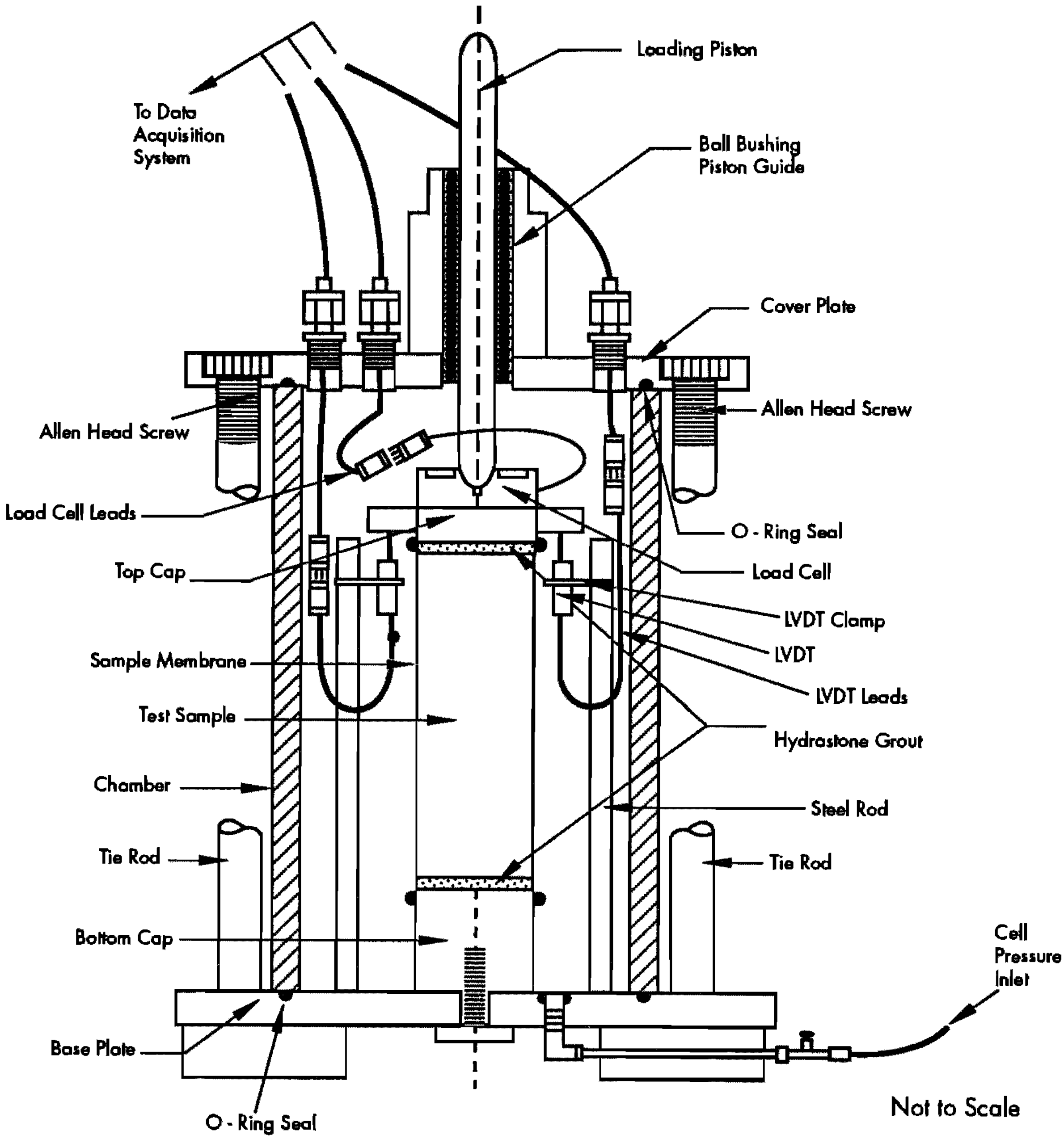


Figure 3.12 Final configuration of the triaxial cell

solid, continuous connection—between the top and bottom steel platens and the synthetic samples. Then, the three synthetic samples of known properties were tested again. [Testing consisted of 200 applications of several levels of deviator stresses at a 10 Hz haversine loading waveform under no confining pressure and a temperature of 74°F.] Several repetitions were performed so as to gain a better statistical representation of the values.

Finally, this arrangement yielded new M_R values that were very close to those expected for the three synthetic samples. Table 3.2 shows the comparison of moduli of synthetic samples determined by both resilient modulus and torsional testing techniques. Figure 3.13 compares modulus means and deviations obtained with ungrouted samples. It is interesting to note in this figure that the deviations in the moduli caused by not grouting the samples to the end platens are significant for materials having a resilient modulus greater than 9,000 psi.

With this calibration, it was felt that there were no significant discrepancies in the comparisons of the resilient modulus with the torsional testing techniques for the synthetic samples, and that this final arrangement of the M_R testing configuration was capable of providing accurate, repeatable, and reliable measurements.

In general, it can be stated that all M_R measurements are sensitive to the location of the deformational transducers; moreover, they are sensitive to the top and bottom cap connections. For stiff materials, these factors are particularly crucial and can lead to erroneous estimates of moduli. Thus, extreme care must be taken to ensure that the hydrostone paste provides a uniform contact between the test specimen and end caps, eliminating additional movement at these points.

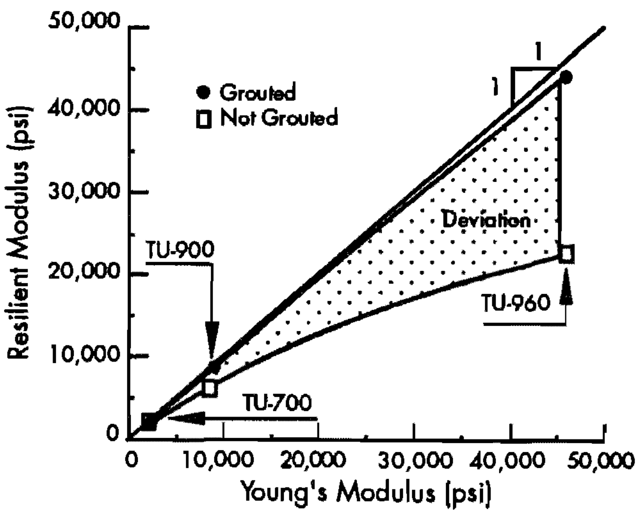


Figure 3.13 Comparison of modulus for synthetic samples tested by the resilient modulus and torsional testing techniques. In addition, shown are the deviations in resilient modulus obtained when the samples were not grouted to the end platens

Table 3.2 Comparison of moduli of synthetic samples

Synthetic Sample	Grouting	Resilient Modulus Test: Mean M_R (psi)	Standard Deviation M_R (psi)	90% C.I. M_R (psi)	Torsional Tests E (psi)	Observations: E within 90% C.I.	Means Ratio M_R/E	Deviation of Means
TU-700	No	1,888	61	[1,788 1,988]	2,220	No	0.850	-0.150
	Yes	2,252	54	[2,163 2,340]		Yes	1.014	+0.014
TU-900	No	6,550	289	[6,076 7,024]	8,921	No	0.734	-0.266
	Yes	8,880	227	[8,507 9,252]		Yes	0.995	+0.005
TU-960	No	22,410	1,223	[20,404 24,415]	45,735	No	0.490	-0.510
	Yes	44,197	931	[42,670 45,724]		Almost Yes	0.966	-0.034

CHAPTER 4. PROTOTYPE RESILIENT MODULUS TESTING PROCEDURE

INTRODUCTION

This chapter describes the development of a prototype resilient modulus testing procedure. First, we survey and discuss the different M_R testing procedures used by the various highway agencies—procedures that include AASHTO T-274, SHRP P-46, ASTM, and other modified methods. Then, we describe the prototype M_R testing procedure completed for use in this study.

As previously noted, AASHTO T-274 has attracted much critical opposition since its introduction in 1986. At issue is its requirement that all specimens be heavily conditioned prior to actual testing. By then, critics argue, the sample is subjected to a substantial variety of stress states. Completely different stress states are specified based on the type of soil (cohesive or cohesionless), but with little consideration of the actual stresses acting on the pavement layer.

The main objective of this test is to simulate field conditions in the laboratory—not to look into the deformational characteristics of soils subjected to much higher stress states than observed in regular pavement structures. Accordingly, several highway agencies, in examining the problems with AASHTO T-274, have developed their own specific testing procedures.

SEVERAL TESTING PROCEDURES

Table 4.1 outlines seven published M_R testing procedures, including (1) AASHTO T-274; (2) SHRP Protocol P-46; (3) the Florida method; (4) the Illinois method; (5) the Washington method; (6) the New York method; and (7) the ASTM method (draft). Specifications of each, including the confining pressure, σ_3, the deviator stress, σ_d, and the number of stress repetitions required on the stress conditioning and the testing sequence stages are also presented in Table 4.1.

The report specifications detailing how to present the testing results of each of the testing procedures are also included in Table 4.1, followed by an estimate of the minimum time required to perform each of the tests specified by the different procedures, and by the maximum principal total stress ratio calculated from the specified stress states.

Stress Conditioning

AASHTO T-274 specifies one stress conditioning for cohesive soils and another for cohesionless soils. For cohesive soils, the highest deviator stress is 10 psi, while the cell pressure specified is 6 psi. For cohesionless soils, the highest deviator stress specified is 20 psi, while the highest confining pressure is 15 psi. For either soil type, samples must be subjected to 200 repetitions at each of the deviator stresses specified. This clearly appears to be excessive, particularly for a process that has a very questionable purpose.

Ho (Ref 9), Jackson (Ref 31), and Seim (Ref 41), in documenting their problems regarding the AASHTO T-274 conditioning stage, reported that their soil samples broke at this stage, and that, consequently, the actual testing sequence had to be discontinued. Ho described the Florida-modified method that is applicable to all types of soils. His method specifies that the conditioning stage consists of static loading of three 10-minute cycles of each of the stress states prior to the dynamic stress state. Although this stage is less severe than that of AASHTO T-274, it cannot be regarded as practical because it delays the testing process, with no guarantee that it is even effective.

The Illinois method, which is also applicable for all types of soils, specifies that the conditioning stage consist of only 200 applications of a 6-psi deviator stress under no confining pressure. This appears to be adequate as long as the material has cohesive properties capable of withstanding extremely high values of principal stress ratios.

Table 4.1 Resilient modulus testing procedures

	Testing Procedure								
	Stress Conditioning			Testing Sequence					
Agency	σ_3 (psi)	σ_d (psi)	Number of Stress Repetitions	σ_3 (psi)	σ_d (psi)	Number of Stress Repetitions	Report	Minimum Time Required to Perform the Test (sec)	Maximum Principal Total Stress Ratio σ_1/σ_3
AASHTO T-274									
(a) Cohesive	6	1, 2, 4, 8, 10	200	6, 3, 0	1, 2, 4, 8, 10	200 each	Plot Log (M_R) vs Log (σ_d)	4,000	∞
(b) Cohesionless	5 10 15	5, 10 10, 15 15, 20	200 each	20 15 10 5 1	1, 2, 5, 10, 15, 20 1, 2, 5, 10, 15, 20 1, 2, 5, 10, 15 1, 2, 5, 10, 15 1, 2, 5, 7.5, 10	200 each	Plot Log (M_R) vs Log (θ) Model: Log (M_R) = a + b* Log (θ)	6,800	11
SHRP P-46									
(a) Soil type 2 (cohesive)	6	4	200	6, 4, 2	2, 4, 6, 8, 10	100 each	Plot Log (M_R) vs Log (σ_d) Model: Log (M_R) = a + b* Log (σ_d)	1,700	6
(b) Soil type 1 (granular)	15	15	200	3 5 10 15 20	3, 6, 9 5, 10, 15 10, 20, 30 10, 15, 30 15, 20, 40	100 each	Plot Log (M_R) vs Log (θ) Model: Log (M_R) = a + b* Log (θ)	1,700	4
Florida DOT (as described by Ho, Ref 9) • For all types of soils It requires one test sample for each stress state	1 2 2 5 5	2 2 4 2 5	static loading 30 minutes each prior to each stress state	1 2 2 5 5	2 2 4 2 5	10,000 maximum each	Plot Log (M_R) vs Log (θ)	59,000	3
Illinois DOT (as described by Dhamrait, Ref 29) • For all types of soils	0	6	200	0	2, 4, 6, 8, 10, 14, 18	10 each	Report M_R (σ_d = 6 psi)	270	∞

Table 4.1 *(continued)*

	Testing Procedure								
	Stress Conditioning			Testing Sequence					
Agency	σ_3 (psi)	σ_d (psi)	Number of Stress Repetitions	σ_3 (psi)	σ_d (psi)	Number of Stress Repetitions	Report	Minimum Time Required to Perform the Test (sec)	Maximum Principal Total Stress Ratio σ_1/σ_3
Washington DOT (as described by Jackson, Ref 31) • For all types of soils	6	8	1,200	1, 2 4 6	1, 2, 4, 6 1, 2, 4, 6, 8 1, 2, 4, 6, 8, 10, 12	200 each	Models: if cohesionless $\text{Log}(M_R) = a + b^* \text{Log}(\theta)$ Report M_R (θ = 25 psi) if cohesive $\text{Log}(M_R) = a + b^* \text{Log}(\sigma_d)$ Report M_R (σ_d = 10 psi)	5,200	7
New York DOT - SMB (as described by Seim, Ref 41) (a) Cohesive	6	1, 2, 3, 4, 5, 6, 7, 8, 9, 10	200 each prior to each stress state	6, 3, 0	1, 2, 3, 4, 5, 6, 7, 8, 9, 10	200 each	Plot $\text{Log}(M_R)$ vs $\text{Log}(\sigma_d)$	8,000	∞
(b) Cohesionless	5 10 15	5, 10 10, 15 15, 20	200 each	20 15 10 5 1	1, 2, 5, 10, 15, 20 1, 2, 5, 10, 15, 20 1, 2, 5, 10, 15 1, 2, 5, 10, 15 1, 2, 5, 7.5, 10	200 each	Plot $\text{Log}(M_R)$ vs $\text{Log}(\theta)$ Model: $\text{Log}(M_R) = a + b^* \text{Log}(\theta)$	6,600	11
ASTM Method (draft) • For all types of soils	6	1	1,000	6, 3, 1	1, 2, 5, 10	200 each	Plot $\text{Log}(M_R)$ vs $\text{Log}(\sigma_d)$	3,400	11

The Washington method, also applicable for all types of soils, specifies that this condition consist of 1,200 applications of an 8-psi deviator stress under a 6-psi confining pressure. In contrast to the Illinois method, this condition appears to be general, in the sense that samples are not driven to high principal stress ratios. Nevertheless, the large number of stress applications makes it less practical.

The New York method specifies one conditioning stage for cohesive soils and another for cohesionless soils (and includes AASHTO T-274). For cohesionless soils, the specifications are similar to those of AASHTO T-274, meaning that it carries the same chronic problems. For cohesive soils, the conditioning stage consists of 200 applications of each of the stress states prior to their particular applications. The highest deviator stress is 10 psi, and the all-around cell pressure specified is 6 psi. Again, this process, plagued by ineffectiveness, fails to demonstrate the validity of its results.

The ASTM method (draft), also applicable for all types of soils, specifies that sample conditioning consist of 1,000 applications of a 1-psi deviator stress under an all-around cell pressure of 6 psi. The same comments are applicable to this method; that is, it appears that the 1,000 applications of a very low deviator stress represent nothing more than wasted time for the machine and the technician. In other words, this type of conditioning stage is unnecessary.

Finally, SHRP Protocol P-46 specifies one conditioning stage for cohesive soils and another for cohesionless soils. For cohesive soils, the conditioning stage consists of 200 applications of a 4-psi deviator stress under a confining stress of 6 psi; for type 1 soils (granular), this stage consists of 200 applications of a 15-psi deviator stress, also under a 15-psi confining stress. Of all the conditioning stages, this appears to be the most adequate, primarily because it is not excessive and because the principal stress ratio specified is relatively low, assuring that the test sample will not fail during the process.

Testing Sequence

The AASHTO T-274 specifies one testing sequence for cohesive soils and another for cohesionless soils. For cohesive soils, the critical state (maximum principal stress ratio) occurs at a 10-psi deviator stress under no confining pressure. For cohesionless soils, there is an extremely large variety of stress states, which appears to be out of perspective. In this case, the critical state occurs at a 10-psi deviator stress under a confining stress of 1 psi. In general, the critical states for both types of materials are quite severe—particularly for the cohesionless material that has to undergo higher values of principal stress ratio—triggering in the process imminent failures of the test samples.

The Florida testing sequence specifies the same state stresses used in conditioning the sample. However, it requires the application of a maximum of 10,000 applications at each of the deviator stresses. This is quite excessive.

The Illinois testing sequence, as described by Dhamrait (Ref 29), specifies that deviator stresses of 2, 4, 6, 8, 10, 14, and 18 psi be applied only 10 times at atmospheric pressure. This specification is practical in the sense that few stress states are applied and repeated; however, it is unrealistic in that it uses no confining pressure and, thus, cannot represent conditions that exist in the lower pavement layers. Such an omission limits the sequence to the testing of materials that have cohesive properties capable of withstanding extremely high values of principal stress ratios.

The Washington testing sequence specifies 200 applications at deviator stresses of 1, 2, 4, 6, 8, 10, and 12 psi. These deviator stresses are applied at different confining pressures (e.g., 1, 2, 4, and 6 psi). While this method avoids subjecting the test material to very high values of principal stress ratios, the process is still somewhat protracted and cumbersome.

The New York method testing sequence specifies that, for cohesive soils, 200 applications of the following deviator stresses be applied under 6, 3, and 0-psi confining pressures: 1, 2, 3, 4, 5, 6, 7, 8, 9, and 10 psi. The use of a 1-psi deviator stress renders the testing sequence impractical.

The ASTM (draft) testing sequence specifies 200 applications at deviator stresses of 1, 2, 5, and 10 psi and at confining pressures of 6, 3, and 1 psi. This is quite practical in the sense that few stress states are used. In addition, the fact that the lowest confining pressure specified is not 0 psi prevents in some degree the failure of samples of reduced cohesive properties.

Finally, the SHRP Protocol P-46 testing sequence specifies that, for cohesive soils, 100 applications of the following deviator stresses be applied under confining stresses of 6, 4, and 2 psi: 2, 4, 6, 8 and 10 psi. This testing sequence appears to be adequate, since stress states are within normal ranges of stresses observed in actual pavements; it is also more efficient because it requires fewer stress applications.

The SHRP P-46 testing sequence for granular materials specifies the application of a substantial variety of stress states, with the critical state occurring when a 30-psi deviator stress is applied to a sample subjected to 10-psi confining pressure. This testing sequence appears to be more appropriate for granular base and subbase materials than for subgrade and non-granular subbase layers.

Testing Report

In general, most of the testing procedures specify that the testing results be reported in a tabular form and in plots of logarithmic graphs that show the variation of the M_R versus the σ_d for a given confining pressure. In some cases, the plots required are logarithmic graphs showing the variation of the M_R versus the sum of principal stresses, θ. The selection of either of these graphs depends highly on the soil type of the test sample. A typical plot is illustrated in Figure 4.1. In this example, the pavement engineer was able to select a particular M_R value for the design of pavements either from the logarithmic plots (Figure 4.1) or from the tabular forms.

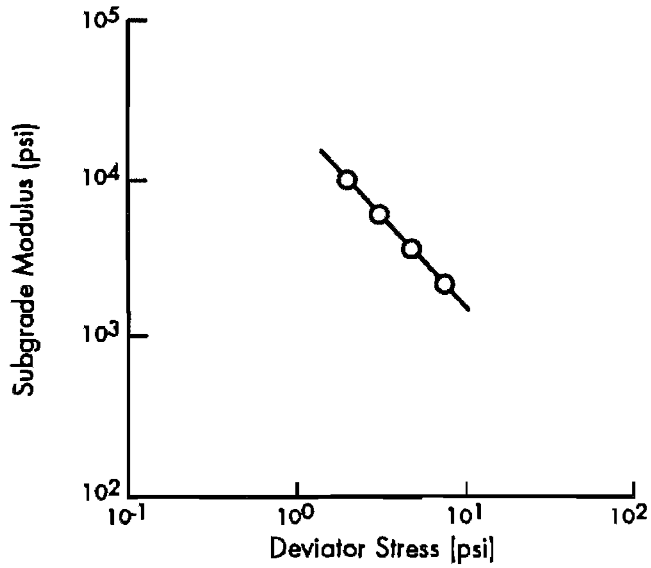


Figure 4.1 Typical plot showing the variation of the resilient modulus with the deviator stress (taken from SHRP P-46, Ref 13)

To refine this selection, some testing procedures have also required the development of regression equations that can predict the moduli. These regression equations consider the moduli the dependent variable and the stress states the regressor factors. Some researchers, including Thompson (Ref 19), Monismith (Ref 27), and Vinson (Ref 8), have suggested that the deviator stress be used as the predictor variable when the material is cohesive, and that the confining pressure (or even the sum of principal stresses) be used as predictors when the material is cohesionless.

We found that AASHTO T-274, SHRP Protocol P-46, the Washington method, and the New York method followed those suggestions to some degree. For cohesive soils, the models can be expressed as follows:

$$\mathrm{Ln}(M_R) = a + b * \mathrm{Ln}(\sigma_d), \text{ or } M_R = e^a * \sigma_d^{\,b} \quad (4.1)$$

where

M_R = the predicted resilient modulus,
σ_d = the applied deviator stress, and
a, b = regression coefficients.

For cohesionless soils, the regression models can be found expressed in terms of the sum of principal stresses, or in terms of the confining pressure:

$$\mathrm{Ln}(M_R) = a + b * \mathrm{Ln}(\theta), \text{ or } M_R = e^a * \theta^b \quad (4.2)$$

$$\mathrm{Ln}(M_R) = a + b * \mathrm{Ln}(\sigma_3), \text{ or } M_R = e^a * \sigma_3^{\,b} \quad (4.3)$$

where

θ = the sum of principal stresses, and
σ_3 = the all-around confining pressure.

Other procedures have gone even further in the specifications. For instance, both the Washington and Illinois methods require that the value of the M_R be calculated, using the σ_d or θ criteria, by applying either one of the developed regression models. The Illinois method specifies that the reported M_R value would correspond to a σ_d equal to 6 psi, while the Washington method specifies that if the material is cohesive, the reported M_R would correspond to a σ_d equal to 10 psi; if it is cohesionless, however, the M_R value would correspond to a θ equal to 25 psi.

All of these reporting techniques appear to be useful. Nonetheless, the fact that the main variation, which is the variation of the moduli versus the resilient axial strains, is not plotted has led to some controversy; that is, we may be overlooking the real behavior of the material. Thus, it is important to include this plot type in the testing reports.

Regarding the specified regression models, they all miss the point in that they do not identify the resilience characteristics of the material, avoiding as they do any mention of their workable strain range. In effect, such regression models are biased and mislead the estimates of the coefficient of determination (R^2) because the resilient modulus (M_R) is calculated and not directly measured. For instance, for cohesive soils, the model suggested in Equation 4.1 actually means the following:

$$\mathrm{Ln}(\sigma_d / \varepsilon_a) = a + b * \mathrm{Ln}(\sigma_d) \tag{4.4}$$

The above equation leads to a situation that ensures that errors associated with the regressor will be directly associated with the predicted values (regressor term in both sides of the regression model).

This situation is not resolved if we follow the recommendations of Boateng-Poku (Ref 17), who suggests developing the following regression model:

$$E_r * \sigma_d = a + b * \sigma_d, \text{ which actually means}$$
$$\sigma_d^2 / \varepsilon_a = a + b * \sigma_d \tag{4.5}$$

From a statistical point of view, this model is biased.

For cohesionless soils, the situation has been somewhat attenuated, since the moduli have been regressed in terms of the sum of principal stresses, which means:

$$\mathrm{Ln}(\sigma_d / \varepsilon_a) = a + b * \mathrm{Ln}(\sigma_d + 3 * \sigma_3) \tag{4.6}$$

ADDITIONAL COMMENTS

Table 4.1 includes estimates of the minimum time required to perform each of the testing procedures. This time requirement, determined by considering the total number of stress states and number of stress repetitions specified by each procedure, is referred to as "minimum" because it represents only the time required for performing the entire test. This minimum does not include any additional time that may be required by the operator for changing the gauge settings and pressures; nor does it include time required for attending to other factors that delay the testing process. In other words, this minimum time can be understood as the time required to perform the test using a fully automated system.

As Table 4.1 shows, the Florida method has the longest minimum time for performing the test, with 59,000 seconds of testing time specified. In contrast, the New York method for cohesive samples requires only about 8,000 seconds, followed by the AASHTO T-274 method for cohesionless samples with 6,600 seconds, and by the Washington method with 5,200 seconds. SHRP P-46 and the Illinois procedure require the shortest test times.

From a practical and economical point of view, it seems reasonable to expect that the method having the shortest duration will be the one favored for use in routine design of pavements. Based on this criterion, either SHRP P-46 or the Illinois method could be used in the development of a prototype testing procedure.

Table 4.1 also includes estimates of the maximum principal total stress ratio that samples experience if subjected to the various testing procedures. As can be noted, many of these testing procedures, including AASHTO T-274 and the Illinois, New York, ASTM, and Washington methods, account for high ratios. Since this ratio controls, to some degree, the strength capacities of the materials, it appears that many of these testing procedures have clearly overlooked the magnitude of this important parameter. Moreover, it seems obvious to expect that samples having few cohesive properties would fail under those critical states with higher ratios. Consequently, from all the testing procedures herein revised, it appears that only SHRP P-46 and the Florida method limit this parameter to a more conservative degree.

PROTOTYPE TESTING PROCEDURE

Since the main objective of this project is to propose an efficient and reliable M_R testing procedure for subgrade and non-granular subbase materials, we decided to assemble a new prototype procedure that can be evaluated through several experiments. Based on the previous discussion, a prototype procedure, consisting of the stress conditioning, the testing sequence, and the testing report, was defined.

Stress Conditioning

Since the subgrade materials are subbases (consisting of locally available compacted materials) and untreated natural or compacted subgrades, the stress conditioning selected was that specified by SHRP P-46 for cohesive soils. Such conditioning subjects the sample first to a confining stress of 6 psi, followed by 200 applications using a 4-psi deviator stress under that confining pressure.

Testing Sequence

The testing sequence selected was also that specified by SHRP P-46 for cohesive soils. This testing sequence consisted of 100 applications at deviator stresses of 2, 4, 6, 8, and 10 psi under 6, 4, and 2-psi confining stresses. The maximum principal stress ratio for this type of material is limited to a value of 6. In addition, the entire procedure would involve only 1,700 seconds of testing time. And finally, the stress states used are the most common stress states observed in traditional pavements, which assures an adequate simulation of the field conditions.

Testing Report

In general, the methods of reporting the testing results do not address completely the deformational characteristics of the materials. Stress-strain behaviors are controlled by the level of strain to which the material is subjected, and not by the level of stress that induces such strain level. Thus, it would be necessary to include plots showing the variation of the resilient modulus with the axial strain and cell pressures.

From a practical point of view, the data sheets should include all the basic properties of the test material, including the plastic index, liquid limit, dry density, moisture content, sample age at testing, and all the information concerning its location, its classification, and its purpose.

Because M_R tests measure resilient axial strains produced under different levels of deviator stresses and confining pressures, a more reliable and general regression model can be developed using the same set of data collected from the test:

$$\begin{aligned} Ln(\varepsilon_a) &= a + b * Ln(\sigma_d) + c * Ln(\sigma_3), \text{ or} \\ \varepsilon_a &= e^a * \sigma_d{}^b * \sigma_3{}^c \end{aligned} \quad (4.7)$$

By definition, we know that the secant resilient modulus is defined as $M_R = \sigma_d / \varepsilon_a$; then, by manipulating these expressions, we can express M_R in terms of either the σ_d or ε_a. Once that is done, the following expressions for M_R can be reported:

$$M_R = e^{-a} * \sigma_d{}^{1-b} * \sigma_3{}^{-c}, \text{ or } K1 * \sigma_d{}^{k2} * \sigma_3{}^{k3} \quad (4.8)$$

$$\begin{aligned} M_R &= e^{-a/b} * \varepsilon_d{}^{1/b-1} * \sigma_3{}^{-c/b}, \text{ or} \\ M_R &= N1 * \varepsilon_a{}^{N2} * \sigma_3{}^{N3} \end{aligned} \quad (4.9)$$

With only one coefficient of determination (R^2) value, Equations 4.8 and 4.9 can be considered the most adequate models for predicting the moduli of subgrade and subbase materials with high or low cohesive properties, or under dry or wet conditions. It should, however, be stated that the workable range of these equations is defined by the strain amplitudes greater than 0.01 percent.

It has been generally found that soils with low plasticity index behave like cohesionless materials, meaning that the confining stress is the main contributor to the explanation of the stiffness behavior of such material, and that for high or even moderate plastic soils, the elastic properties are insensitive to the cell pressure but sensitive to the deviator stress. Consequently, the regression model expressed in Equation 4.7 appears to be the most general and the most adequate for use in this study.

CHAPTER 5. MATERIALS AND PREPARATION

This chapter describes the processes of selecting materials, preparing test samples, and placing those samples into the triaxial chamber of the M_R testing equipment.

SOILS FOR TESTING

This study used fifteen soil samples from across Texas. In collecting the soils, we took care to ensure that the samples represented a wide range of soil characteristics. The Texas county outline map illustrated in Figure 5.1 shows the origin (shaded areas) of the soil samples. The Texas DOT provided the soil samples, which were obtained from compacted subgrades of actual pavement projects that had already been constructed and put in operation.

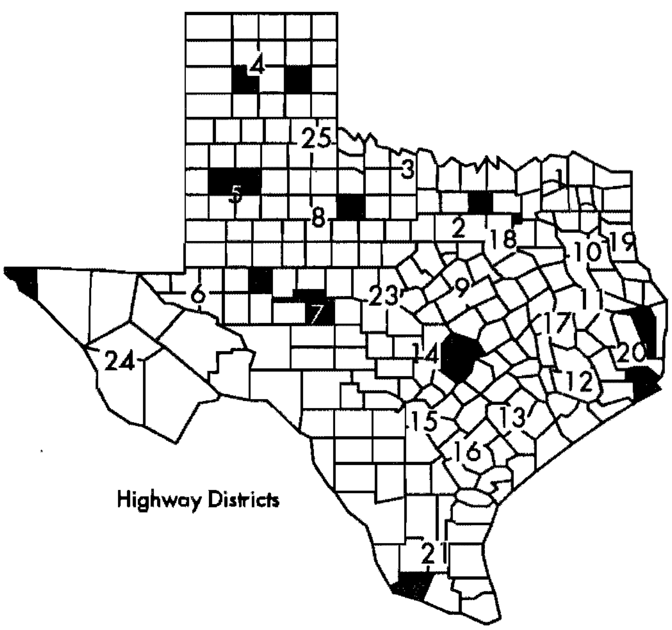


Figure 5.1 Texas county outline map. Shaded counties indicate the origin of the soils used in this study

Soil samples were usually accompanied by a summary of their basic properties, including the Atterberg limits, the fine content, the specified or "actual" field density, and the optimum moisture content. For those samples that did not include a basic properties list, we performed the appropriate tests to determine those basic characteristics. No attempt was made to verify TxDOT's analysis of soil properties.

The plasticity index (PI) of the soils was the other parameter considered during the acquisition of the soil samples. Use of this index could assist in establishing some inferences regarding its effect on the resilient modulus. In this way, soils range from highly plastic to non-plastic materials.

Table 5.1 summarizes the basic properties of these soils. From left to right, they are: (1) Soil ID, which includes the code used both to identify the soils and to indicate the order in which the soils were received; (2) District - County - Highway, which documents the geographic origin of the soils; (3) AASHTO class, which documents the soil's classification according to AASHTO; (4) Pass #200, which includes the soil's fine grain content; (5) Liquid Limit, which reports the soil's liquid limit; (6) Plastic Index, which reports plasticity index; (7) Opt Moisture Content, which documents the value reported as the optimum moisture content to be compacted in the field; and (8) Actual Dry Density, which presents the specified dry density of the compacted soil to be achieved in the field.

Most TxDOT district laboratories use Test Method Tex-114-E for determining desirable densities and moistures. This test method states that, with the specified density and moisture, the material will have adequate strength to support the design wheel load and be in a condition less subject to detrimental volume changes caused by fluctuation of the moisture content during the life of the pavement structure. In addition, this test method is characterized by its use of a compaction ratio that relates loose to dense conditions of the soil. For example, loose density is determined by rodded unit weight or by the soil pat density, while dense densities are determined by dropping a 10-pound hammer 18 inches to effect a total compacting effort of about 30 ft lb per cubic inch. The procedure used to arrive at the optimum moisture content and dry densities is, however, outside the scope of this study; for more information on this test method, the reader should refer to TxDOT's manual of testing procedures (Ref 15).

As shown in Table 5.1, a wide range of PI values is represented. Such a distribution is important because PI is a significant soil parameter in pavement design. It is a common belief, for example, that high PI soils will create many problems in the pavement structure because of the dramatic variations in volume, strength, and stiffness that result from moisture and seasonal changes; low PI soils, on the other hand, present more stable characteristics.

Table 5.1 General characteristics of soils for testing

SOIL ID	District County Highway	AASHTO class	Passing No. 200 (%)	Liquid Limit	Plastic Index	Optimum Moisture Content (%)	Actual Dry Density (pcf)
1	18 Rockwall FM 550	A-7	94.0	85	55	21.6	96.2
2	14 Travis Mopac-183	A-7-6	87.3	56	29	19.3	93.9
3	18 Denton SH 121	A-7-6	99.0	50	33	18.9	104.2
4	14 Travis Mopac-Parmer	A-4	49.0	23.5	4.1	11	122
5	21 Starr FM 755	A-4	34.9	25	9.5	10.6	119.5
6	5 Hockley US 62	A-6	100	30	15	12.7	115.85
7	4 Potter Spur 951	A-6	99.7	37.6	20.4	16.5	106.6
8	7 Glasscock RM 2401	A-6	80	37.1	18.1	14.2	117.58
9	4 Gray SH 70	A-7-6	99.7	52	34	19.2	96
10	5 Lubbock FM 835	A-4	91	20	4	10.6	123.7
11	24 El Paso UTEP	A-7-6	77	44.1	23.6	16	107
12	20 Jasper FM 252	A-7-6	99.7	79.3	52.1	19.9	101.5
13	20 Jefferson US 69	A-7-6	96	54.1	35.9	18	103.5
15	7 Tom Green US 67	A-7-6	98.4	58	40	20.1	102.4
16	8 Haskell Abilene	A-7-6	97	51	29	16.2	109.7

To evaluate the effect of the plasticity index on the dynamic behavior of the materials, we decided to group the soils according to their PI values. This grouping resulted in five PI groups differing from one another in the magnitude of a PI range. The five PI groups were: (1) 0-10, (2) 11-20, (3) 21-30, (4) 31-40, and (5) 41- up. Thus, soils of PI values between 0 to 10 percent are nested (grouped) within the 0-10 PI group, and so on. In this way, three soils are nested within each PI group. Table 5.2 shows the grouping of the soils according to their PI and to their AASHTO classification.

Soils #7 and # 15, though nested, failed to meet the PI criterion. Soil #7 had a PI of 20.4 percent and was nested in the 11-20 PI group; soil #15, having a PI of 40 percent, was nested in the 41-up PI group. These circumstances did not affect the inferences made in this study.

PREPARATION OF THE TEST SAMPLES

Because they were taken directly from the field, most of the soil samples were received in a damp condition. And since all the soil samples were disturbed samples, Test Method Tex-101-E - Part II, "Preparation of Soil and Flexible Base Materials for Testing" (Ref 15) was followed for the preparation of all soil samples used in this study. (This test method is specified by TxDOT for the preparation of disturbed soil samples for mechanical analysis and for physical, moisture-density relations, triaxial, and stabilization tests. For out-of-state readers, Test Method Tex-101-E is in close agreement with AASHTO Designation T 146-86 and T 87-86; see Ref 2.)

Once the soil was air dried and crushed to pass the No. 10 sieve, its moisture content was measured. Then, about 10 kg of the material—enough to prepare four companion specimens per batch—was placed into a 20-rpm mixer. (Companion specimens, which are defined here as samples having similar characteristics, were prepared so that the soil properties could be monitored and evaluated against time and preparation process, and so that they could be tested simultaneously under different laboratory tests for comparison purposes.)

Since the optimum moisture content and the air-dried conditions of the soil sample had been determined, the process of adding the proper amount of distilled water to the soil sample was a straightforward operation. The mixing process continued until a relatively homogeneous material, free of lumps, was achieved. Precautions were taken to prevent any moisture loss.

Compactive Effort

At this point, it is important to mention that Seed et al (Ref 5) recommended the use of two compaction methods for the preparation of the test specimens: (1) kneading or impact, and (2) static. In the past, these methods were considered important for simulating the behavior of materials compacted at water contents below the optimum value. But such materials are far more susceptible to changes in strength and stiffness (resulting from increases in the water content) than materials compacted at optimum water contents and at water contents above the optimum value. Clearly, a more practical approach is necessary. In this study, only one compaction method—the impact compaction method—is used for the preparation of the test specimens.

An impact compactor, Soiltest model CN-4230, was used for densification. This compactor was designed to perform AASHTO Designation T-99 and T-180 test methods (Ref 2). Since a 4-inch diameter mold is used in this test, the test specimens were prepared to that diameter. In addition, because test specimens should be 2.8 inches in diameter and 5.6 inches in height to be tested in our resilient modulus system, an extra piece of the standard mold was used to compact samples 4 inches in diameter and 6 inches in height. Figure 5.2 shows the mold in position ready for material compaction.

To prepare the test specimens, the compactive effort specified in Test Method Tex-113-E was applied (Ref 15). This particular test method, used for determining the relation between the moisture content and density of soils, is actually a modification of ASTM D 1557 (Ref 3) and AASHTO Designation T-180 methods.

Table 5.2 Grouping the soils according to their PI

AASHTO Class	A - 4			A - 6			A - 7 - 6								
PI Group	0-10			11-20			21 - 30			31 - 40			> 40		
Solid ID	4	5	10	6	7	8	2	11	16	3	9	13	1	12	15

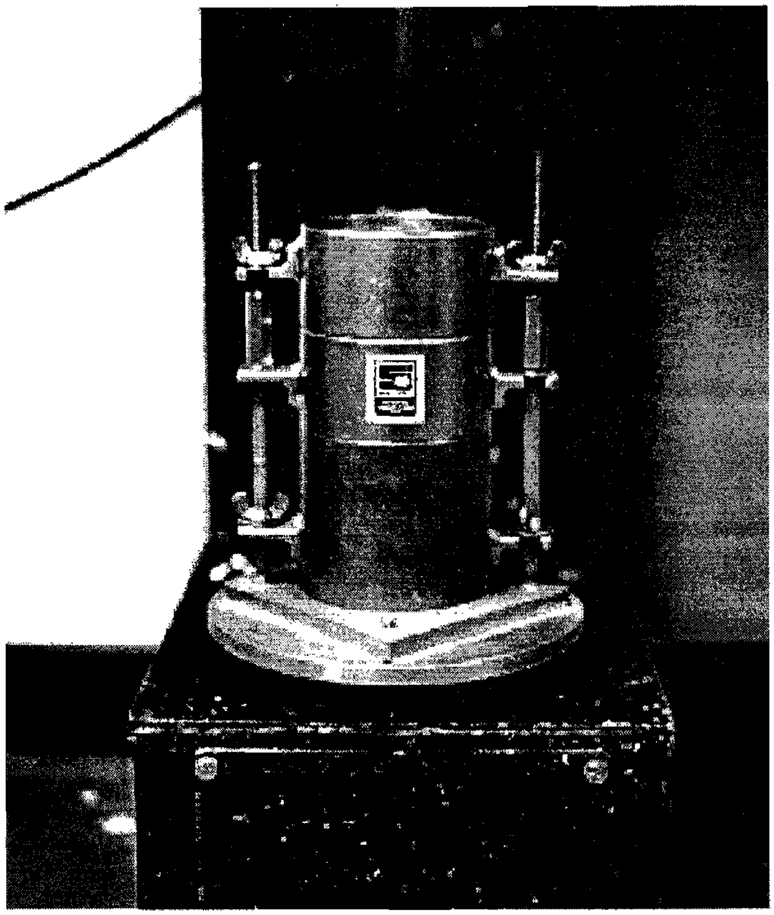


Figure 5.2 Steel mold in position ready for material compaction

To obtain triaxial results with reduced swelling, Test Method Tex-113-E specifies different compactive efforts, depending on the PI of the materials. For instance, for soils having a PI less than 20, it specifies the use of 13.26 ft lb per cubic inch; for soils having a PI from 20 to 35 and a high percentage of soil binder, the use of 6.63 ft lb per cubic inch is specified. In this way, the number of blows per layer was adjusted according to the drop height, number of layers, weight of the hammer, and volume of the specimen, thus assuring that the specified compactive effort was effectively applied.

Moisture Content

For many years, it has been standard practice in design testing to use samples in a soaked or nearly saturated condition. In many cases, certainly, this has led to the overdesign of pavements, since subgrade materials do not always become saturated in practice.

Thus, the selection of representative samples in actual field conditions becomes a challenging undertaking; accordingly, the resilient modulus values to be used in the design of pavements should be based on the results of a thorough analysis of the moisture-density-modulus relationships of the pavement materials. In this study, because most of the factors that contribute to the final *in situ* water content of the material (e.g., level of water table, source of percolating water, soil suction characteristics, *in situ* water content, etc.) are unknown, we decided to prepare the test samples at optimum water contents, which are referred to as *opt*, and at water contents above the optimum value, which are referred to as *wet*, though all were prepared with the same compactive effort.

In general, it was observed that *opt* specimens achieved dry densities similar to those determined by the Test Method Tex-114-E conducted by TxDOT district laboratories.

Dry densities of *opt* samples, which are referred to as the actual dry densities, are actually, in Texas, the densities provided the contractor as target densities for the construction site. On the other hand, *wet* samples were prepared so as to achieve 95 percent of dry densities achieved on the *opt* samples.

Because it deals with just one compaction method, this approach is thought to be more practical than that recommended by Seed (Ref 5). It also appears to be conservative because, as shown by Seed, samples compacted at high degrees of saturation by either the kneading or impact compaction methods had lower modulus values than those samples soaked to a high degree of saturation after being compacted to a low degree of saturation by the static compaction method.

Trimming

Immediately after compacting the soil specimens, we carefully extruded them from the steel mold using a mechanical extruder. Figure 5.3 illustrates the soil specimens just after extrusion from of the mold. But because the soil specimens were prepared at sizes larger than those required for testing in our system, they had to be trimmed.

The trimming process consisted of carefully reducing the dimensions of the samples until they were about 2.8 inches in diameter and 5.6 inches in height. Thus a height-to-diameter (H/D) ratio of 2 was provided for all test specimens used in this study. (This ratio was in accordance with much of the literature on triaxial tests; e.g., see Bishop, Ref 16.)

A trimming frame manufactured by the project team allowed the samples to be manually rotated as they were trimmed. Figure 5.4 shows a soil specimen, along with the resulting soil debris, in the trimming frame. Immediately after trimming, the top and bottom surfaces of the sample were flattened; the test specimen was then weighed, measured for its final dimensions, wrapped, and stored in a special room of constant humidity and temperature. Only on its testing day was the soil specimen taken out of that room.

It is important to mention that the time required for two testers to prepare a test specimen (including compaction, trimming, weighing, wrapping, and storing) was generally 1 hour.

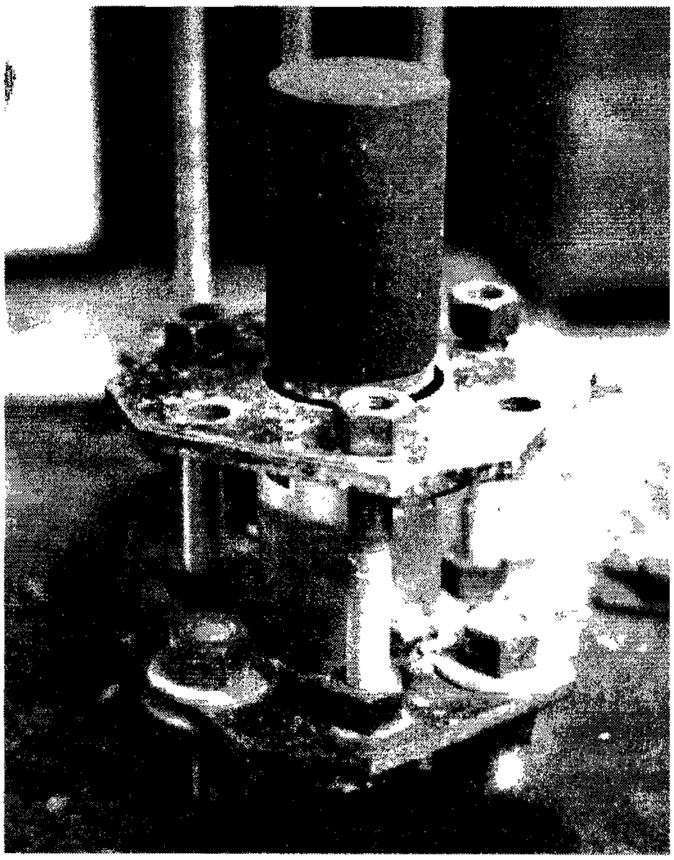


Figure 5.3 Soil specimen after being extruded from the steel mold

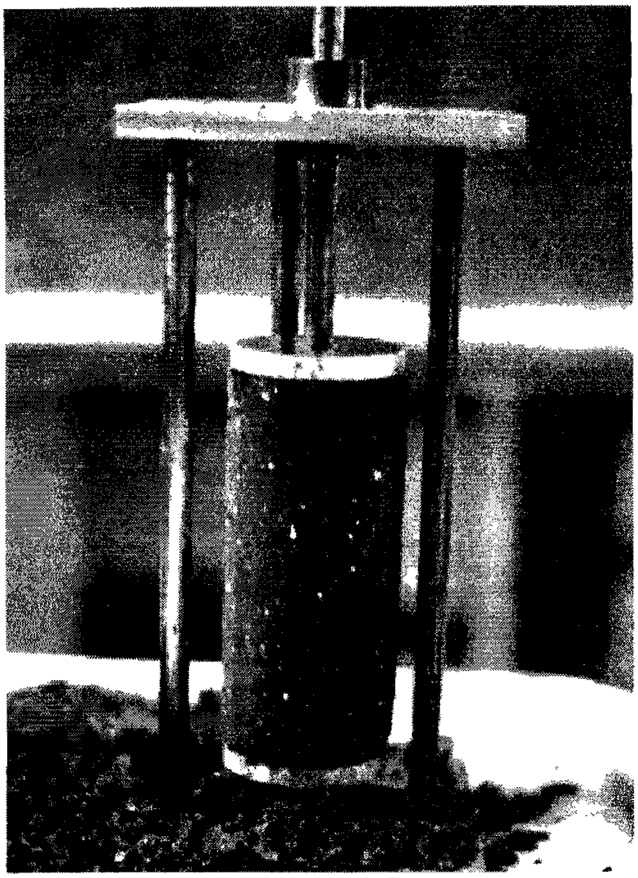


Figure 5.4 Soil specimen being trimmed

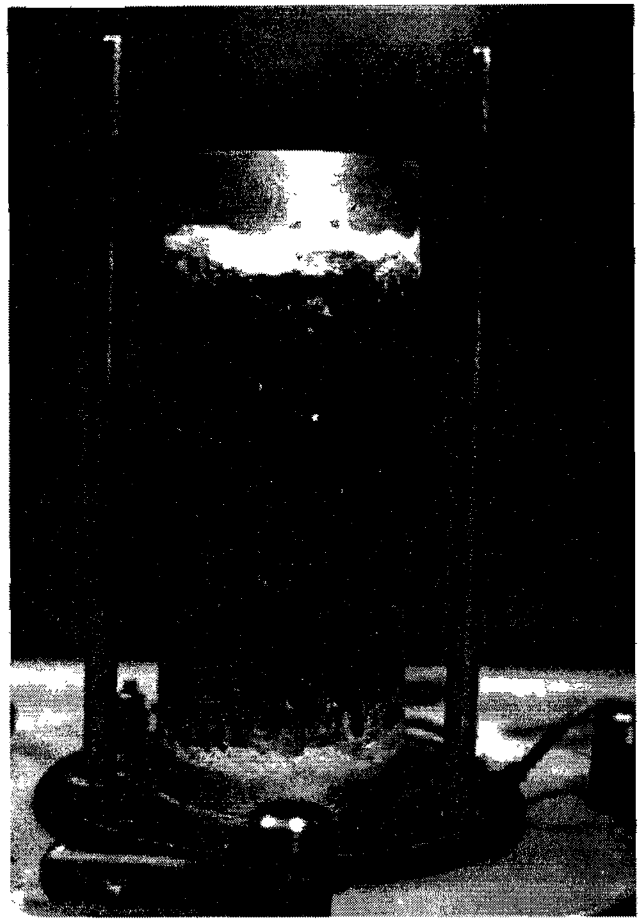


Figure 5.5 Test specimen grouted to the end caps

PLACEMENT OF THE TEST SAMPLES INTO THE TRIAXIAL CELL

Before being placed into the triaxial cell, the test specimen was weighed and its dimensions were again measured to calculate and verify its density. The specimen was then installed in the triaxial cell.

Each specimen was grouted to the top cap and base pedestal of the triaxial chamber using a hydrostone paste. The use of such a hydrostone paste facilitated the sample location process in that the levelness of the top cap and base pedestals could be easily adjusted to accommodate and eliminate any unevenness (imperfections) in the end surfaces of the test specimens.

Grouting was used because we had already demonstrated (during the evaluation of the resilient modulus testing equipment using synthetic samples) that strong contacts between the test specimen and the end caps are required for an accurate and reliable estimation of the M_R values (see Chapter 3).

Test specimens were placed in a manner similar that used by masons in building a brick wall; end caps were leveled and aligned to assure orthogonality in the installation. The joints were then finely arranged so that there were no paste lumps (which could puncture holes in the rubber membranes). Figure 5.5 shows a test specimen grouted to the end caps.

Afterwards, hydrostone debris was removed and the entire setup was cleaned. Vacuum grease was placed on the sides of the end caps so that the rubber membranes could be easily attached to them. Then, two 0.014-inch-thick Soiltest rubber membranes were placed around the test specimens to prevent both moisture loss and gas leakage. Because the water content values of the samples before and after testing were extraordinarily similar, we concluded that the membranes were successful in retaining specimen moisture. (The fact that the room temperature was kept at a constant 74°F perhaps contributed to the similarity in values as well.) Gas leakage was also reduced to a minimum. Kane et al (Ref 50) reported that, for partially saturated soils (using two 0.002-inch-thick membranes and nitrogen gas as the fluid of all-around confining pressure), the pore air pressure changed at a rate of 0.7 psi/min for 100-psi confining pressure. If that relation is directly proportional to the confining pressure and testing time, and inversely proportional to the thickness of the membranes, it would be expected that, in our M_R tests, specimens that were subjected to a 6-psi confining pressure for 30 minutes could have experienced a 0.18-psi change in confining pressure. Yet a 0.18-psi change in the applied confining pressure would only represent 3 percent of the total applied confining pressure. Thus, for all practical purposes, this deviation is negligible and can be tolerated.

It is important to emphasize that because the M_R test is an undrained test performed generally on partially saturated soils, we made no effort to measure pore water pressures or to estimate states of effective stresses; rather, we used the total-state-of-stresses approach to estimate the stress-strain behavior of the test materials.

After the test specimen was installed, its ends grouted, and the membranes secured with O-rings at each end, two linear variable differential transformers (LVDT's) clamped on steel bars fixed to the base of the triaxial cell were installed diametrically opposite one another. Each LVDT was positioned by pointing the steel wings that were clamped to the top cap. In this way, the axial deformations were measured from the total height of the specimen rather than from a small part of the sample. Figure 5.6 presents the top portion of the sample covered by the membranes (the LVDT's are already installed). Once the LVDT's were positioned and the rubber membranes perfectly sealed, the body of the triaxial chamber that provides confinement to the specimens was mounted and assembled (under air-tight conditions).

Finally, after waiting 2 more hours to allow the hydrostone paste (used to grout the test samples to the end caps) to reach its full strength and stiffness properties, we decided to start the test. Figure 5.7 shows the setup of the triaxial chamber during the testing operation.

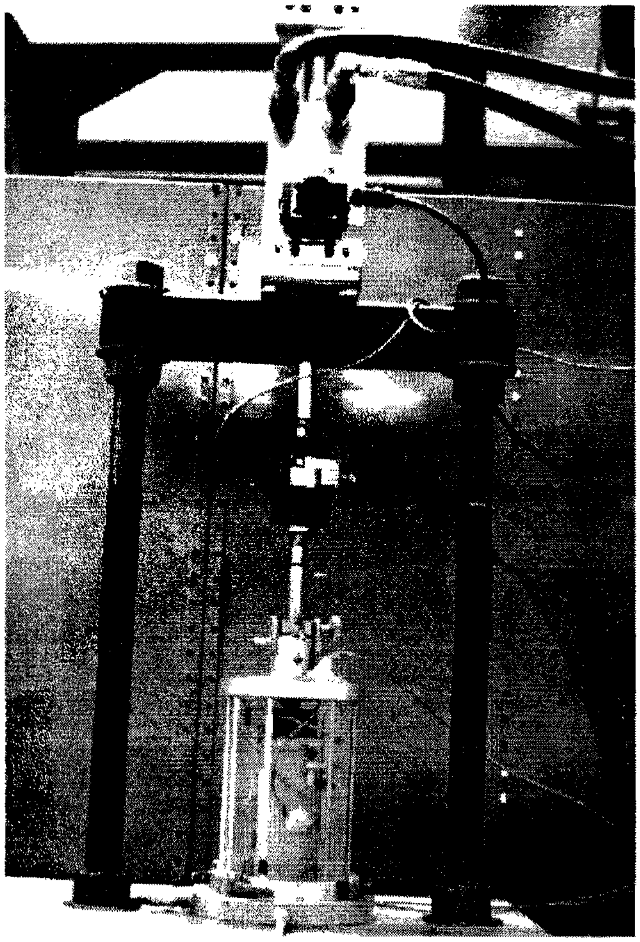


Figure 5.7 Final setup of the triaxial cell as seen during the testing operation

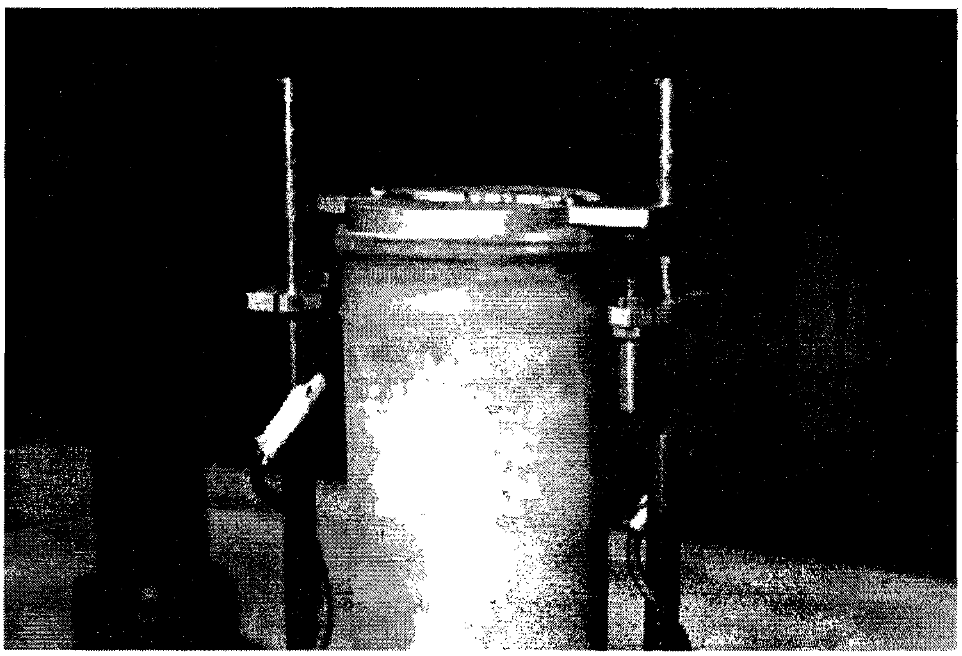


Figure 5.6 Top portion of the setup of the test specimen

CHAPTER 6. IMPORTANCE OF GROUTING TEST SPECIMENS TO THE END CAPS

As discussed in Chapter 3, the use of synthetic samples of known properties is essential in determining the status of M_R testing equipment; additionally, we found that the most reliable method for consistently obtaining the expected values of moduli was to grout the test specimens to the end caps. The objective of this chapter is to verify the importance of this grouting procedure in the M_R test. After providing some background on the subject, this chapter describes and explains the results obtained by testing actual soil samples, with and without grouting.

BACKGROUND

Seed (Ref 5) recommended sample conditioning as a way of improving the contacts between the end caps and the test specimens. In addition, he stated that sample conditioning may also serve to eliminate the time effects created by the interval between compaction and loading, and between loading and reloading. For these reasons, AASHTO T-274 specifies that samples be conditioned prior to testing.

If the end platens or sample ends are not perfectly flat (i.e., the contact is uneven), the normal stresses applied to the ends of the specimen will vary across the core, causing a loss of uniformity in both the applied compressive stress and the induced axial strain. Grouting not only resolves this problem, but reduces the effect of the sample conditioning as well.

In conventional triaxial tests, the cylindrical surfaces of the test samples are subjected to uniform radial stresses (though not to shear stresses). Because the end platens are usually made of materials considerably stiffer than the specimen, researchers assume the test induces equally normal displacements over these end surfaces, which may remain plane. In addition, if those interfaces are frictionless, no shear stresses are applied; in such an ideal circumstance, the normal stresses and strains will be uniform throughout the height of the test specimens.

In the past, it was thought that if the interfaces were rough or grouted, radial displacements at the ends would be restricted, causing the specimen to take on a barrelled shape when loaded. Today we know that this is true only in conventional triaxial tests, where the sample is driven to failure (with axial deformations above 4 percent) in order to estimate its strength capacity. This, however, is not the case with samples used in the M_R test, where test samples are never loaded to failure and where the induced axial strains are much lower (from 0.001 to 0.5 percent). Accordingly, shear contact stresses can be considered negligible, and the normal stresses and axial strains throughout the sample can, for all practical purposes, be considered uniform.

In researching the effects of rigid restraints of triaxial specimens, Dehlen (Ref 21) addressed in particular the effects of (1) using frictional end platens, (2) installing rigid extensometer clamps on the sides of the test specimens, and (3) unevenly trimming the sample ends.

Regarding the effects of frictional end platens, Dehlen documented that many researchers, including Edelman (1949), D'Appolonia and Newmark (1951), and Balla (1960), have theoretically modeled this problem, showing that the effect of end restraint is to reduce the change in length and, except for short cylinders, to increase the change in diameter at mid-height of an axially-loaded specimen. These theoretical studies indicate that an overestimation on the order of 5 percent of the moduli could be obtained in tests where strains are measured between the end plates. Dehlen analyzed this problem using a finite element approach.

Figure 6.1, taken from Dehlen's dissertation, shows his analytical model of the triaxial samples with stiff extensometer rings and frictional caps and bases. His results indicated that an increase in specimen height resulted in an increase in the accuracy of the results for Young's modulus and Poisson's ratio provided by all techniques of measurement. In addition, he showed that for samples with a 2:1 height-diameter ratio, Young's moduli and Poisson's ratios may, because of cap and base friction, be in error by only 1 or 2 percent, and that measuring the strains with bonded strain gauges at mid-height was slightly more accurate, with errors less than 1 percent.

Consequently, his theoretical results showed that the use of frictional end caps will not affect, for all practical purposes, the estimations of the moduli (no matter what the position of strain measurements).

middle-half height of the specimen; in addition, it showed that measurements of the axial strains at the specimen ends are free of this potential problem.

Regarding the effects of unevenly trimming the sample ends, Dehlen explained that the imperfect-contact model had to be axi-symmetric; that is, the load was applied concentrically over a circular area with a radius half that of the sample. His results showed clearly that when the axial strain is computed from the relative displacements of the

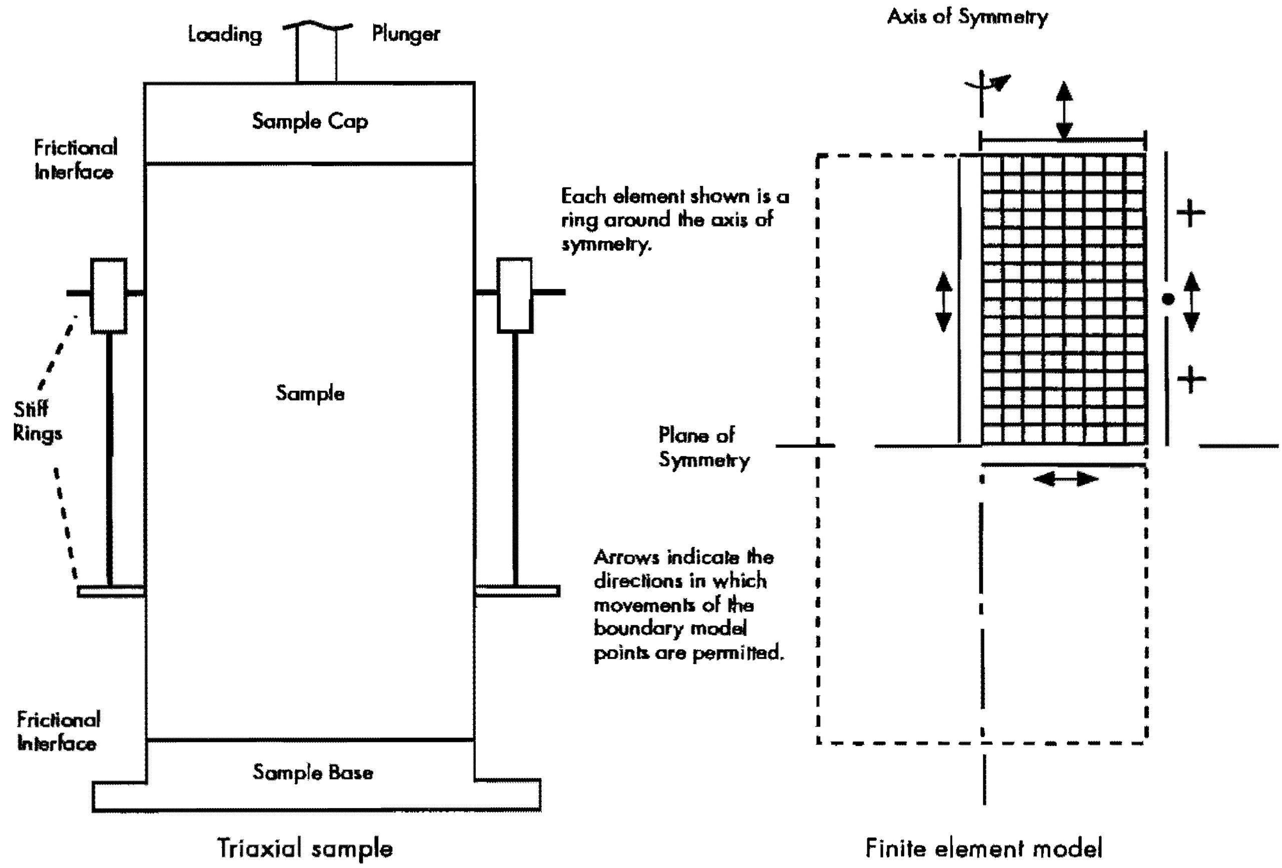


Figure 6.1 Analytical model of the triaxial samples with stiff extensometer rings and frictional caps and bases, as used by Dehlen (Ref 21)

In researching the effects of installing rigid extensometer clamps around the test samples at the quarter and three-quarter height to measure the axial strain, Dehlen used a finite element analysis. His results showed that the errors in Young's modulus caused by rigid clamps are much greater than those caused by cap and base friction, and that the two effects combined would result in an overestimate on the order of 10 percent in a typical test. This particular analysis performed by Dehlen demonstrated the risk of using inappropriate clamps for measuring axial strains at the end platens, imperfect contact could cause an underestimation of Young's modulus and Poisson's ratio by 30 percent, and that errors are much lower when the axial strain is measured at the middle-half height of the specimen.

In summary, Dehlen's theoretical analyses clearly demonstrated the advantages and disadvantages of measuring the relative displacements at different points of the specimen subjected to repetitive axial loading. Two points are particularly relevant: (1) It is evident that the greatest source of error is related to imperfect contact between the test samples and the end platens; and (2) the risk of error is increased if the axial strains are measured at the ends rather than at the half-middle height of the sample.

In assessing the most appropriate alternative for achieving reliable estimations of the moduli, Dehlen recommended measuring the axial strains at the half-middle height of the sample. While this recommendation has found support from Seed and others (Ref 5), several researchers over the past decade (Refs 9, 10, 42) have begun to question this alternative—particularly since during the application of the loading pulses the two reference points (on which the relative displacements are measured) move, thereby losing track of the actual strains. Compounding the resulting uncertainty is the fact that the installation of clamps around the sample can cause disturbances that obstruct the sample's dynamic behavior during the test.

For these reasons, grouting the specimens to the end platens appears to represent the best method for obtaining reliable estimations of the moduli. This was demonstrated experimentally during the calibration of the testing system by using synthetic samples, as explained in Chapter 3. During that calibration (in which axial deformations were recorded at the ends of the samples) the expected or known moduli for the three synthetic samples were consistently achieved only in those cases where the specimens were grouted to the end platens, as illustrated in Figure 3.13 of Chapter 3.

The following testing results show the effect of grouting on the resilient moduli of compacted cohesive samples. These results underscore the importance of grouting when seeking reliable estimations of the resilient modulus.

THE EFFECT OF GROUTING ON THE RESILIENT MODULUS

While calibrating the resilient modulus testing equipment, we learned that strong contacts between the end caps and the specimen are required for an accurate and reliable estimate of the resilient modulus. In this experimental exercise, each sample was first tested ungrouted; then, under the same stress conditions, the sample was tested grouted.

The compacted sample of soil 1 (131 days old, high PI) used for this exercise had a moisture content of 21.2 percent and a dry density of 93.6 pcf. The second sample was a specimen of soil 4 (low PI, compacted 188 days before testing); this sample had a moisture content of 10.2 percent and had 124.4 pcf of dry density. These soils were chosen because they represented a wide range of PI.

The stress conditions applied to the two samples included a confining stress of 6 psi and a deviator stress of 10 psi repeated 2,000 times. These stress conditions were applied to both ungrouted and grouted samples; seating pressure was kept below 1 psi during the entire operation.

These stress conditions were chosen to reproduce the experience presented by Seed (Ref 5) and to examine the importance of grouting—particularly since Seed concluded that sample conditioning (1) corrected the imperfect contacts between the specimen and end caps and (2) attenuated the effect of time on the moduli of the samples. Seed's results (see Figure 6.2) show that the effect of thixotropy on the resilient deformations was apparently canceled by the deformations induced by the repeated loading; a marked degradation of their resilient moduli for loading repetitions below about 2,000 was evident.

Although these results have been published in several papers and reports (Refs 4, 5, 6, 21), they are nonetheless questionable in that the resilient deformations were measured at the half-middle height of the sample—an approach that has been highly criticized as inefficient and unreliable.

Figures 6.3a and 6.3b compare the resilient modulus with the induced permanent deformations for the ungrouted and grouted sample of soil 1 throughout the 2,000 loading repetitions. Figures 6.4a and 6.4b show the same information for the soil 4 sample.

EXPERIMENTAL OBSERVATIONS

Figures 6.3a and 6.4a indicate the importance of grouting when estimating resilient modulus. For the soil 1 sample, the resilient modulus of the ungrouted sample is about 30,000 psi, while with the grouted sample the modulus is 20 percent higher, or roughly 36,000 psi. This discrepancy is even greater when the sample is stiffer. Figure 6.4a shows the resilient modulus of an ungrouted sample to be about 40,000 psi for soil 4, while with the sample grouted, the modulus is 25 percent higher, or roughly 50,000 psi. This indicates that weak contacts between the test specimen and end platens result in errors in the estimation of the resilient modulus.

Although much greater differences in the moduli were expected (based on experience with the synthetic samples), it appears that top and bottom surface imperfections complicate the task of estimating the moduli of samples. Such imperfections may cause variations on the caps/specimen contact pressure distributions, which can lead to axial deformations that register higher than they actually are, as pointed out by Dehlen.

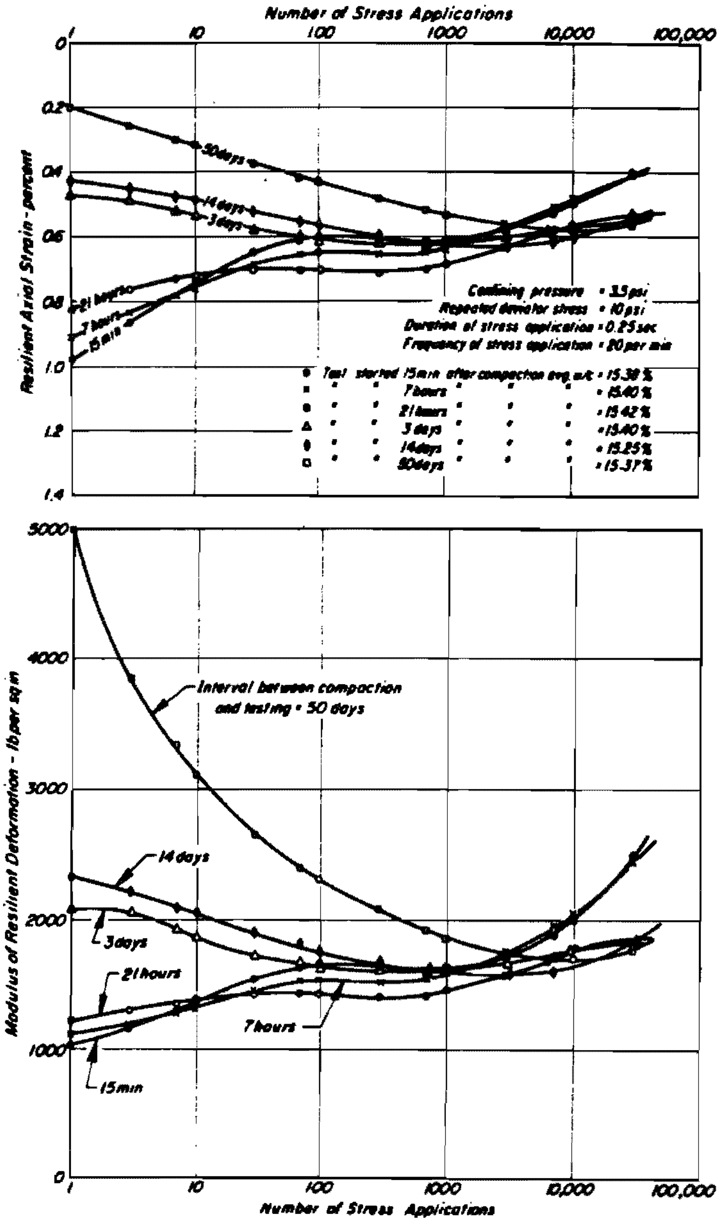


Figure 6.2 Effect of thixotropy on resilient characteristics—AASHO Road Test subgrade soil (Ref 5)

Furthermore, it appears that neither the seating pressure nor the conditioning stage can resolve the problem created by such surface imperfections. This is, in fact, a problem encountered in the testing of other materials. For instance, the standard method for testing the compressive strength of portland cement concrete requires that the top and bottom surfaces of the specimen be capped before any testing takes place. Regarding the incurred permanent deformations, Figures 6.3b and 6.4b show the marked difference between the two conditions. When the specimen is ungrouted, any loading application will tend to compress the specimen, causing larger permanent deformations and, hence, greater changes in the volume and density of the samples. This means that during the test, specimens may change their original properties or control conditions—something that is extremely undesirable from an experimental point of view. Thus this method indicates the importance and necessity of grouting the samples to the end platens. (NOTE: The discontinuity on the permanent deformation observed in Figure 6.4b was caused by the readjustment of the recording LVDT, which was out of the calibration range. This discontinuity is not part of the soil behavior.)

Finally, regarding the sample conditioning suggested by Seed (Ref 5), it appears that such conditioning is ineffective. As shown in Figures 6.3a and 6.4a, there is not a sharp degradation in the moduli; rather, they are constant throughout the 2,000 loading repetitions.

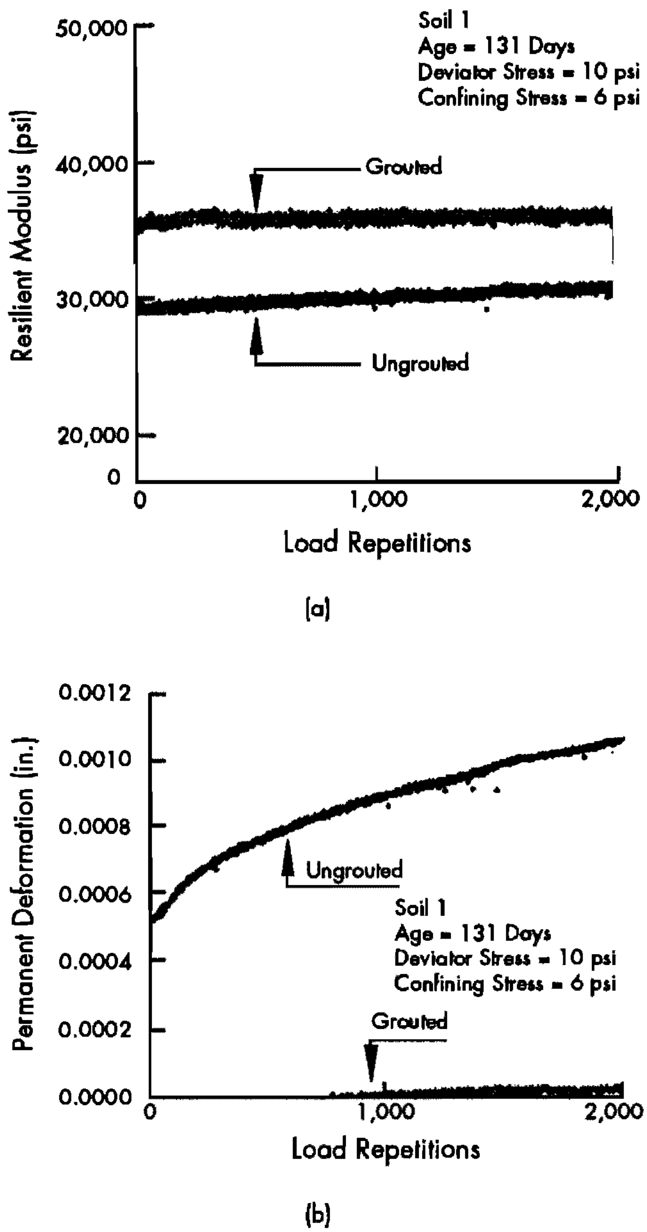


Figure 6.3 **Effect of grouting on (a) the resilient modulus and (b) the permanent deformations of a compacted sample of soil 1 (131 days old) tested under a 6-psi confining stress and 2,000 repeated applications of 10-psi deviator stress**

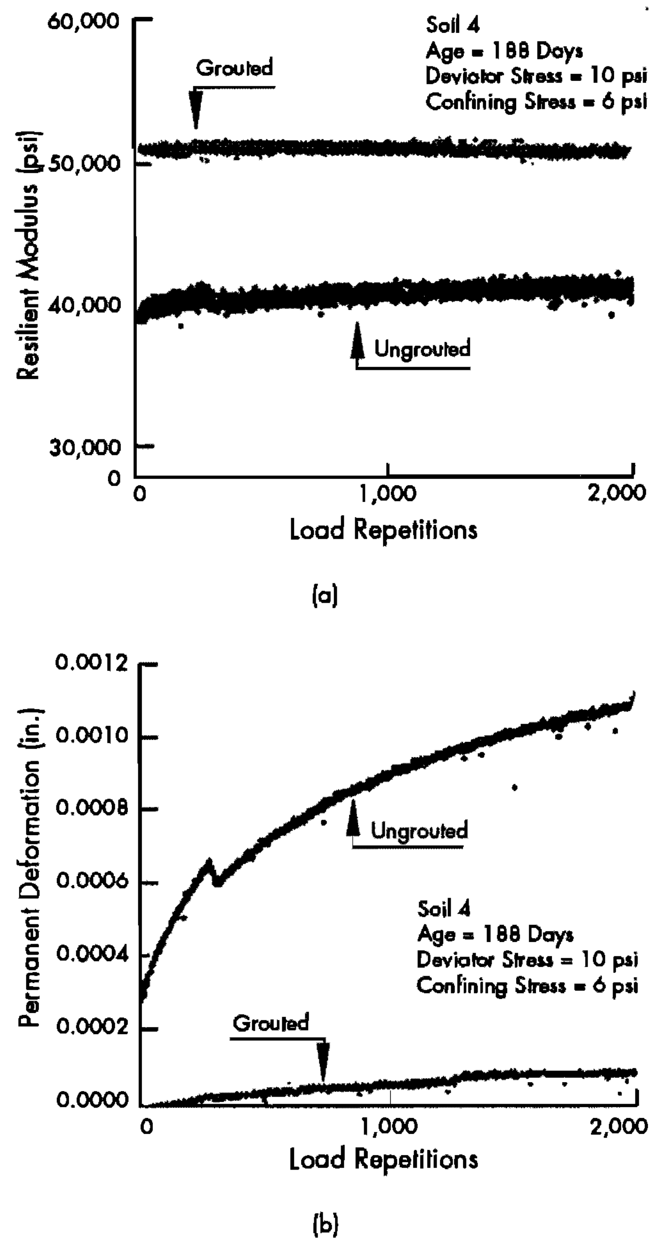


Figure 6.4 **Effect of grouting on (a) the resilient modulus and (b) the permanent deformations of a compacted sample of soil 4 (188 days old) tested under a 6-psi confining stress and 2,000 repeated applications of 10-psi deviator stress. (NOTE: The discontinuity on the permanent deformation observed in Figure 6.4b was caused by the readjustment of the recording LVDT, which was out of the calibration range. This discontinuity is not part of the soil behavior.)**

ADDITIONAL QUESTIONS ABOUT GROUTING

While it has been demonstrated that grouting is necessary in efforts to obtain reliable estimations of the resilient modulus, its use raises further questions: What is the appropriate cement? What is the proper water-cement ratio? What is the minimum amount of time necessary for the grout to cure, assuring that it is strong enough to perform the M_R test? And what is the effect of having a thick grout between the specimen and the end caps? This section will attempt to answer these questions.

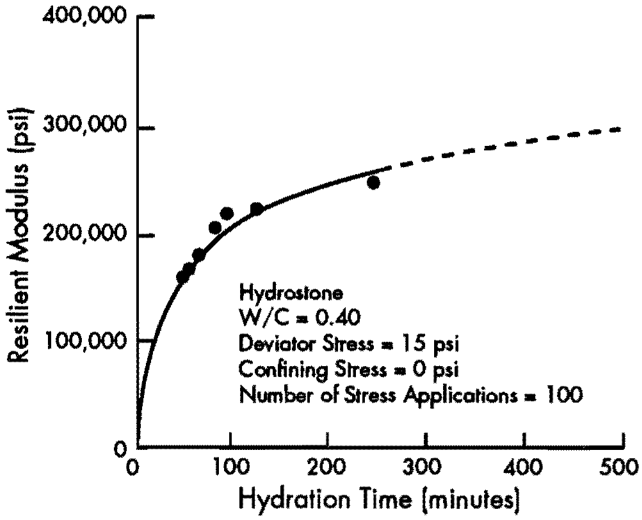
Throughout this experimental study, a hydrostone cement was used to prepare the grout. Hydrostone was considered suitable because its paste is highly workable, it has a rapid setting time, and, once cured, it is very strong. Formulating the specifications to this paste required that we monitor, as in concrete, the water-hydrostone cement (W/C) ratio by weight. Thus, after preparing several pastes of different W/C ratios, and after comparing them in terms of workability and setting time, we concluded that the most suitable W/C for use in the M_R test was 0.40.

A hydrostone paste sample 2.8 inches in diameter and 5.6 inches in height (with W/C of 0.40) was next prepared to: (1) estimate its deformational characteristics in terms of the M_R and unconfined compression tests versus time, and (2) determine the minimum time required for the paste to cure to a point that permitted the application of dynamic loadings. This sample was cast directly into the triaxial chamber with the end caps (to avoid having the same problem of imperfect contacts).

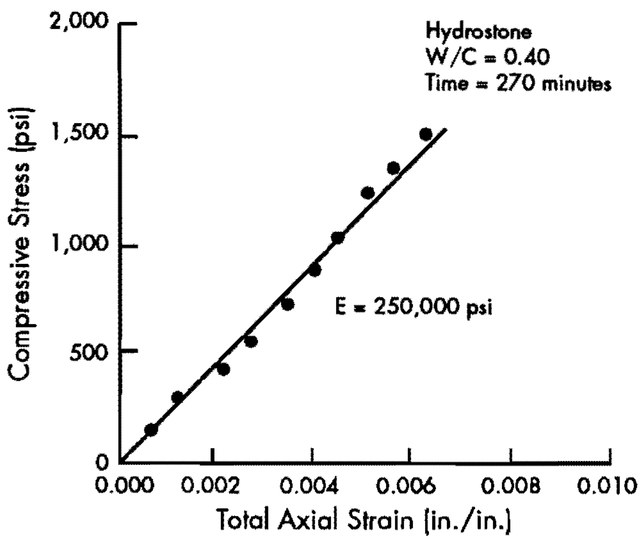
Mixing water with the hydrostone cement induced the hydration that allowed the paste to gain consistency. Fifteen minutes later the paste, now at the proper consistency, was ready for use. We then cast the paste into a steel mold. After another 15 minutes, we stripped the mold from the paste sample. Then, 15 minutes more were required to arrange the testing setup. Thus, with 45 minutes of hydration time, the solid sample made of hydrostone paste was ready for the repetitive loading applications. The same loading pulse specified in the M_R test was used for the 100 applications of a 15-psi deviator stress, and for each application the induced resilient strain was recorded.

By averaging the last five loading applications, the M_R of the hydrostone sample was computed and recorded with its hydration time. At different intervals, this process was repeated to develop the curve M_R versus hydration time for this hydrostone sample. This curve, illustrated in Figure 6.5a, shows the increase of the M_R versus time.

After 250 minutes of hydration time, the sample was then taken out of the triaxial chamber and placed into a standard unconfined compression frame. After 270 minutes of hydration time, the sample was tested by the unconfined compression test. Figure 6.5b shows the results of this test.



(a)



(b)

Figure 6.5 Properties of the hydrostone sample: (a) variation of the resilient modulus versus hydration time, and (b) stress-strain behavior in the unconfined compression test

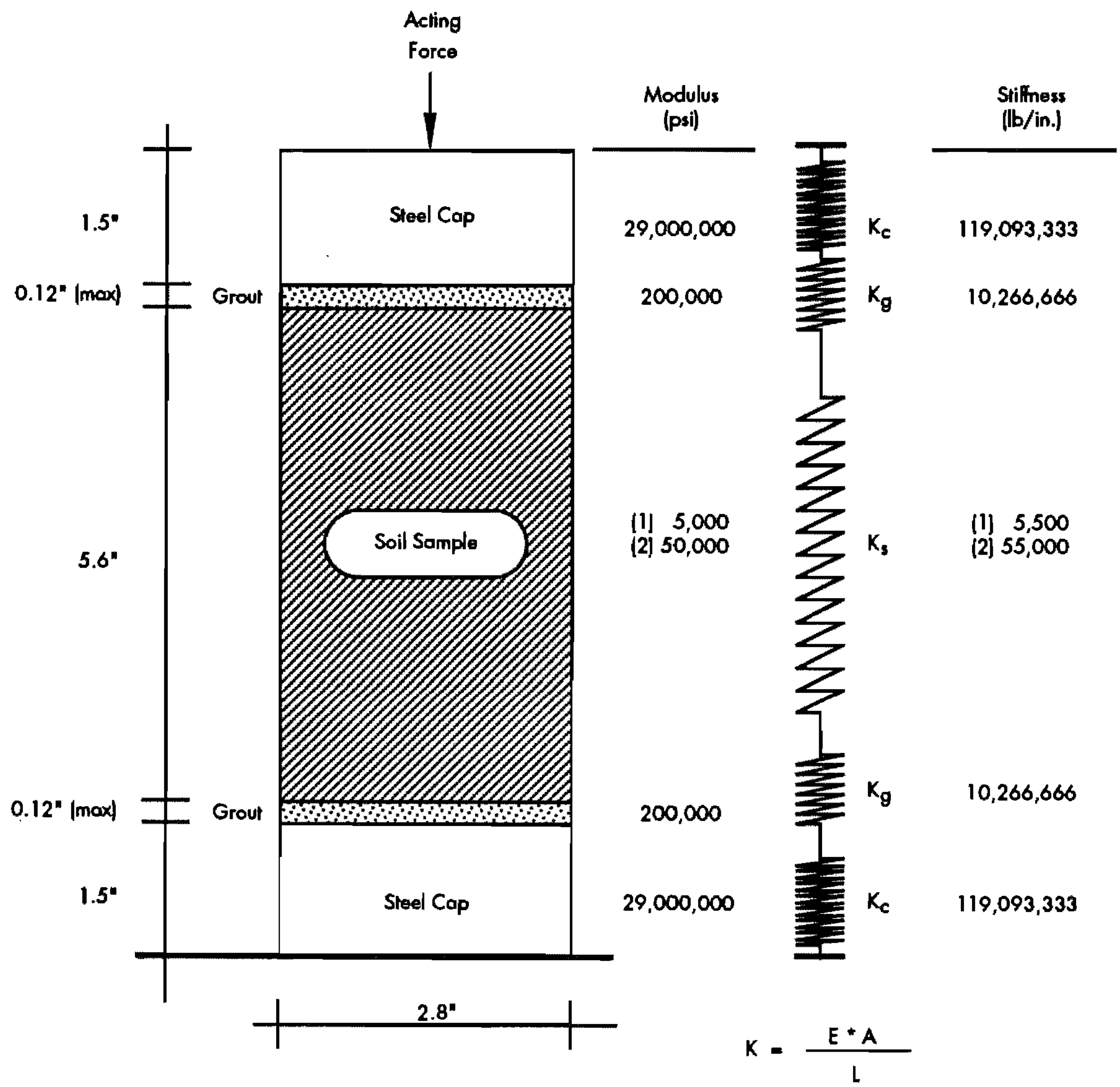
It was then estimated that the minimum hydration time required for the grout to cure to sufficient strength for M_R testing would be 120 minutes, at which time the hydrostone sample had a modulus of about 200,000 psi, as shown in Figure

6.5a. It was then clear that the effect of having the grout between the specimen and the end caps (with that modulus as part of the M_R setup) needed to be analyzed in order to determine if the presence of the grout may be affecting the estimations of the resilient modulus of the soil samples.

An elementary model based on spring stiffnesses was used to check the influence of the grout. Because the direction of the acting force in the M_R test is longitudinal to the sample, this model, illustrated in Figure 6.6, was considered appropriate. In addition, because the induced strain is very small, any shear stress acting orthogonally would be close to negligible.

The equivalent stiffness (K_{eq}) of the system should be similar to the stiffness of the soil sample (K_s) to assure that the estimations of the moduli will be accurate. Two cases were considered: (1) testing a soft soil sample with M_R = 5,000 psi; and (2) testing a stiff soil sample with M_R = 50,000 psi. The analysis of this elementary model was conducted using the equations included in Figure 6.6, where the different modulus and spring stiffness values of the steel caps, the grout layers, and the soil sample are also presented.

In the first case (soft soil) the ratio of the equivalent stiffness (K_{eq}) to the true stiffness of the sample (K_s) was 1.00; in the second case (stiff soil) the ratio was 0.99. These results indicate that after 120 minutes of hydration time, the strength of the grout is such that it can withstand the M_R test without risk of yielding inaccurate measurements.



$$\frac{1}{K_{eq}} = \frac{2}{K_c} + \frac{2}{K_g} + \frac{1}{K_s}$$

Figure 6.6 Analytical model of the grouted soil samples with the end caps

CHAPTER 7. EVALUATION OF THE EFFECT OF SAMPLE CONDITIONING

INTRODUCTION

As mentioned in Chapter 4, several M_R testing procedures specify that the soil samples be first subjected to several stress stages before the actual testing operation is performed. This process is referred to as sample conditioning (also called "stress conditioning" or simply "conditioning"). The variations of the stress-strain behavior experienced by the sample throughout the entire conditioning process are not required to be recorded or reported. In the past, those variations have been understood to represent a researcher's compromise, and not a reflection of the general behavior of the pavement materials.

AASHTO T-274-82 is explicit on the objectives of the conditioning stage: (1) to eliminate the effects of the interval between compaction and loading; (2) to eliminate the effects of initial loading and reloading; and (3) to correct the imperfect contacts between the specimen and end caps.

Figure 6.2 illustrates a specific result obtained in 1962 by Seed (Ref 5), showing what appears to be the main reason for the implementation of the conditioning stage in the different testing procedures. Basically, his results showed that the effect of thixotropy was destroyed by a marked degradation of the resilient moduli for loading repetitions below about 2,000.

Nevertheless, it was also observed in Chapter 6 that such conditioning was ineffective (i.e., no sharp degradations in the moduli, with a constant response throughout the 2,000 loading repetitions). We therefore decided to evaluate experimentally the importance of the conditioning stage in the M_R testing procedure.

OBJECTIVES AND EXPERIMENTAL APPROACH

The objective of this chapter is to present an experimental evaluation of the effect of the conditioning process on the resilient moduli of compacted samples. For this evaluation, an experiment using the soils collected was designed, performed, and statistically analyzed.

Before proceeding to the experimental setup, a data acquisition program capable of monitoring simultaneously and continuously the variation of the deformational parameters of the test sample throughout the entire conditioning process was developed and implemented in the M_R system.

It was also necessary to select the factors and levels to be used in the experiment. Additionally, we defined the type and characteristics of the soil samples to be tested to determine the size and complexity of the experimental design factorial.

DESIGN OF THE EXPERIMENT

In the design of any experiment, the factors and levels to be used, along with the variables to be measured in the experiment, need to be defined. In our case, the factors of interest were: (1) the plasticity index, (2) the soil, and (3) the conditioning state.

Previously, three different soils were grouped into each of the five established PI groups so as to evaluate the effect of the plasticity index of the soils, as shown in Table 5.2. Using the same arrangement, this experiment (see Table 7.1) included the testing of two soils (selected at random) out of the three soils available in each of the five PI groups. Thus, the experiment tested ten different soil samples.

Because the soils are nested within the PI groups, this experiment was treated as a nested factorial with blocking at the soil level. The conditioning state had two levels: initial and final. The initial level corresponded to the state of the sample *prior* to the action of conditioning, while the final level corresponded to the state of the sample *after* the action of conditioning.

Four testing parameters were monitored: deviator stress, axial strain, resilient modulus, and permanent deformations. However, only resilient modulus was used in the analysis, since the objective of this experiment was to determine the effect of conditioning on that parameter. The other three parameters were recorded to check the results and the entire testing operation.

The conditioning process herein considered was the one specified in our prototype testing procedure described in Chapter 4. This procedure specifies that the test sample submit to 200 applications at a deviator stress of 4 psi under a 6-psi confining pressure. It should be pointed out that this particular conditioning is somewhat less severe than the hammering specified by the AASHTO T-274 for cohesive soil samples, or by the ASTM method, or even by the Washington procedure. Therefore, we emphasize here that our conclusions about sample conditioning are framed within our own prototype procedure. Additionally, we point out that, because the collected soils were only subgrade soils (and mainly fine grain), the inference space and, obviously, our conclusions refer only to these soil types.

Blocking is always very important because it removes the variance from the experimental error and helps to detect significant differences (in this case with the conditioning state and the shown interactions). However, there was some confusion regarding the error term and the interactions, as explained by Anderson (Ref 46), because repetitions of the experimental units per treatment combination were not performed.

The model for such an analysis is:

$$M_{R_{ijkl}} = u + PI_i + Soil(PI)_{(i)j} + State_k + PI * State_{ik} + Soil(PI) * State_{(i)jk} + Error_{(ijk)l}$$

where

$M_{R_{ijkl}}$ = resilient modulus of the sample of the j^{th} soil of i^{th} plasticity index at the k^{th} conditioning state,

Table 7.1 Design of the experiment

PI Group	0 - 10		11 - 20		21 - 30		31 - 40		> 40	
Soil ID / Conditioning State	4	10	7	8	2	11	3	9	1	15
Initial	●	●	●	●	●	●	●	●	●	●
Final	●	●	●	●	●	●	●	●	●	●

To define a broader inference space and to permit more general conclusions, we decided that test samples would be prepared under randomly chosen moisture conditions, and that the samples would be tested at randomly chosen sample ages.

This nested factorial experiment with blocking at the soil level has a restriction on randomization. The inferential unit was the soil, and thus the soil is the critical factor in all the tests (i.e., effect of the plasticity index, effect of the conditioning state, and all important interactions as shown in the expected mean square algorithm presented in Table 7.2).

u = overall mean,

PI_i = the effect of the i^{th} plasticity index,

$Soil(PI)_{(i)j}$ = the effect of the j^{th} soil,

$State_k$ = the effect of the conditioning state,

$PI * State_{ik}$ = the effect of the interaction of the i^{th} plasticity index with the k^{th} conditioning state,

$Soil(PI) * State_{(i)jk}$ = the effect of the interaction of the j^{th} soil with the k^{th} conditioning state, and

$Error_{(ijk)l}$ = the experimental error (random).

Table 7.2 Expected mean square algorithm

Item	Expected Mean Squares
PI_i	$\sigma^2 + 10 \cdot \sigma^2$ (Soil (PI)) + $20 \cdot \phi$(PI)
Soil (PI) $_{(i)j}$	$\sigma^2 + 10 \cdot \sigma^2$ (Soil (PI))
State $_k$	$\sigma^2 + 50 \cdot \phi$(State)
PI • State $_{(i)k}$	$\sigma^2 + 5 \cdot \sigma^2$ (Soil (PI)*State) $+ 10 \cdot \sigma^2$ (PI*State)
Soil (PI) • State $_{(i)jk}$	$\sigma^2 + 5 \cdot \sigma^2$ (Soil (PI)*State)
Error $_{(ijk)l}$	σ^2

COLLECTION OF THE DATA

Ten soil samples were first prepared and trimmed according to the sample preparation described in Chapter 5. These samples were then individually placed in the triaxial chamber. Each test sample was grouted to the end caps and, after curing for 2 hours, was subjected to the sample conditioning as specified by our prototype testing procedure.

The test samples were prepared from soils 4 and 10 of PI group 0-10, from soils 7 and 8 of PI group 11-20, from soils 2 and 11 of PI group 21-30, from soils 3 and 9 of PI group 31-40, and from soils 1 and 15 of PI group > 40. Table 7.3 summarizes their basic characteristics.

Table 7.3 Basic characteristics of the test samples

PI Group	Soil ID	Moisture Content (%)	Dry Density (pcf)	Sample Age (days)
0 - 10	4	16.90	117.41	198
	10	10.80	123.05	2
11 - 20	7	17.00	107.00	69
	8	13.70	113.10	96
21 - 30	2	19.30	90.56	2
	11	16.00	110.00	159
31 - 40	3	18.40	101.90	2
	9	25.00	98.50	30
> 40	1	21.20	93.62	131
	15	20.70	105.90	2

The data collected from the testing of these ten soil samples are illustrated in Figures 7.1 through 7.10, which present the variation of the deviator stress, the resilient axial strain, the resilient modulus, and the permanent deformations of the samples throughout the entire conditioning stage. This was considered important, since in that way the magnitude of the applied stress and induced strains were continuously checked (machine malfunctioning may cause an irregular loading application, creating apparent degradations or changes in the moduli).

EXPERIMENTAL OBSERVATIONS

Figure 7.1 illustrates the results obtained from the testing of the compacted sample of soil 4. A constant 4-psi deviator stress applied 200 times and a consistent induced resilient axial strain can be observed in Figures 7.1a and 7.1b, respectively. While the calculated resilient modulus (see Figure 7.1c) oscillates slightly owing to the small value of the axial strain (close to the axial strain limitation of the equipment), it shows a uniform pattern—that is, no degradation, but rather a consistent modulus throughout the 200 loading repetitions. Finally, Figure 7.1d shows that only the permanent deformation changes increasingly with the number of stress repetitions.

Results obtained from the testing of the compacted sample of soil 10 (see Figure 7.2) revealed the same behavior: the value of the resilient modulus remains constant throughout the 200 loading repetitions; only the permanent deformation changes.

The same can be said for Figure 7.3, emphasizing that the resilient modulus value varies somewhat periodically. Such variation is explained by the fact that the induced resilient axial strain (quite low) bordered on the measuring limits of the equipment (0.01 percent of axial strain); but again, the permanent deformation appears to be the only parameter that varies throughout the entire conditioning stage.

Figure 7.4 shows the results of soil 8, a very strong sample. With an even higher deviator stress (5 psi) the moduli were obviously oscillatory owing to the small level of strain induced; but the moduli did not change and no permanent deformation was detected.

Figure 7.5 shows a typical instance of machine malfunction in which the magnitude of the applied deviator stress did not remain constant. As shown in (a), a 6-psi deviator stress was initially applied to a compacted sample of soil 8; over the conditioning stage, that load, unexpectedly, was gradually reduced. That situation induced a low axial strain, as shown in (b), causing the slight upward tendency of the magnitude of the resilient modulus, as shown in (c). This is explained as being related to the non-linear stress-strain behavior of the material, and not to the conditioning stage itself.

Figure 7.6 shows the same constant loading application, the same response, and the same resilient modulus of the compacted sample of soil 11 throughout the entire conditioning stage. This time, however, high permanent deformation values were recorded.

In general, these experimental observations are reinforced by the other testing results of compacted samples of soils 3, 9, 1, and 15, as shown in Figures 7.7, 7.8, 7.9, and 7.10, respectively. These results indicate that the 200 loading applications of the stress conditioning specified by our prototype procedure have no effect on the magnitudes of the resilient modulus; rather, they cause unnecessary permanent deformations to the test samples.

ANALYSIS OF THE EXPERIMENT

The analysis of the experiment was performed using the personal computer version of the statistical analysis software (SAS), with all the experimental data to be analyzed arranged and processed as required by SAS.

Because a large amount of information was collected, we selected only the most representative resilient modulus values from the test results for analysis. Accordingly, the initial state of the sample was defined from the first five computed resilient modulus values of the conditioning process, with the final state defined from the last five resilient modulus values.

Tests for homogeneity of variance and normality were first performed, as suggested by Anderson (Ref 46). Because these tests demonstrated that there was no need for transforming the data, the data were therefore analyzed in their original units (i.e., psi).

Table 7.4, which includes the factors and their interactions, their degrees of freedom, sum of squares, mean squares, and "F" values, summarizes the results of the analysis of variances (ANOVA).

The effect of the soil type is reflected in the "F" value of the Soil (PI) factor. While an "F" value of 5147.81 is quite high, this was expected, since the soil samples were not only compacted at different densities and moisture contents, but were tested at different times as well. And because the soil samples were different (even very different within the PI group), the effect of the plasticity index could not be estimated.

The effect of the conditioning was evaluated, with the effect expressed in terms of an "F" value of the State factor in the ANOVA analysis. An extremely low "F" value of 0.11 was computed to measure this effect. The F tests at 5 and 25 percent significance levels revealed that the conditioning process had no effect on the resilient modulus of compacted samples—even when test samples were prepared at different moisture conditions and tested at different sample ages.

This analysis indicates that the effect of thixotropy on the resilient deformations of the compacted samples is neither canceled nor destroyed by such a conditioning stage. Thus, we can conclude that the conditioning stage is unnecessary and can therefore be eliminated from the M_R testing specifications.

SUMMARY

The objective of this chapter was to describe the experimental evaluation of the effect of the conditioning process on the resilient moduli of compacted samples. Conclusions drawn from this evaluation are as follows:

1. Where strong contacts exist between the test samples and end caps, the conditioning process specified in the prototype testing procedure has no effect on the resilient modulus of compacted samples of cohesive soils, even when test samples were prepared under different moisture conditions and tested at different sample ages. Therefore, it can be concluded that the effect of thixotropy on the resilient deformations of the compacted cohesive samples is neither canceled nor destroyed by such a conditioning stage.
2. Although these conclusions appear to be framed within the conditioning type used, they clearly reflect a general pattern in the material in which no degradation of the moduli is detected. Appendix B, which presents the test results of a sample of soil 2 under three conditioning types, serves to reinforce further the observations in this evaluation.
3. The conditioning stage specified by AASHTO T-274 for cohesive samples requires higher magnitudes and many more deviator stress applications than the used prototype procedure. Of course, it might be argued that the conditioning stage used was insufficient for reproducing Seed's behavior; that issue is fully addressed in the next chapter.

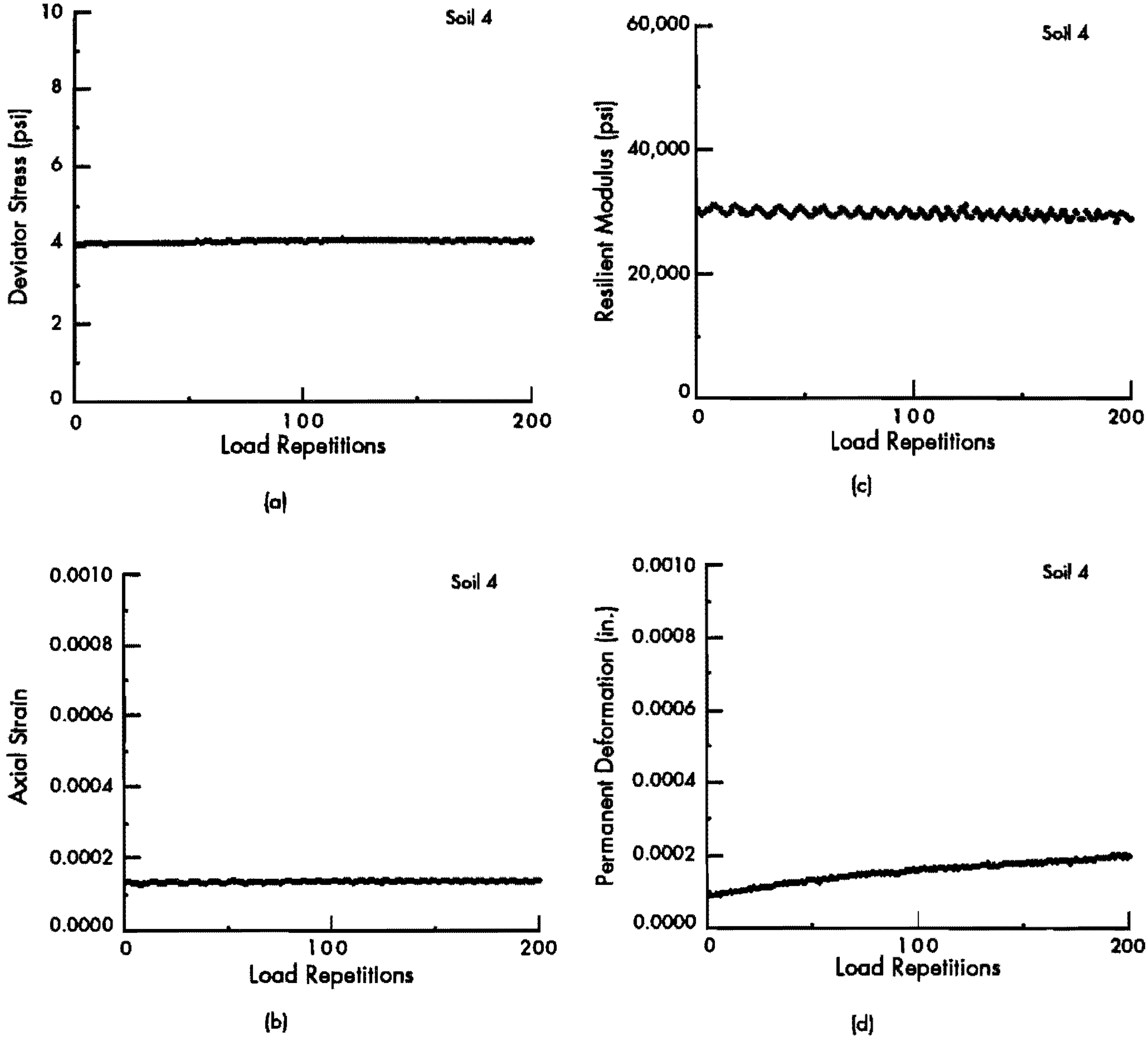


Figure 7.1 **Deformational characteristics of a compacted sample of soil 4 (198 days) tested under a 6-psi confining stress and 200 repeated applications of about 4-psi deviator stress. Shown are: (a) the applied deviator stress, (b) the induced axial strain, (c) the resilient modulus, and (d) the permanent deformation of the sample**

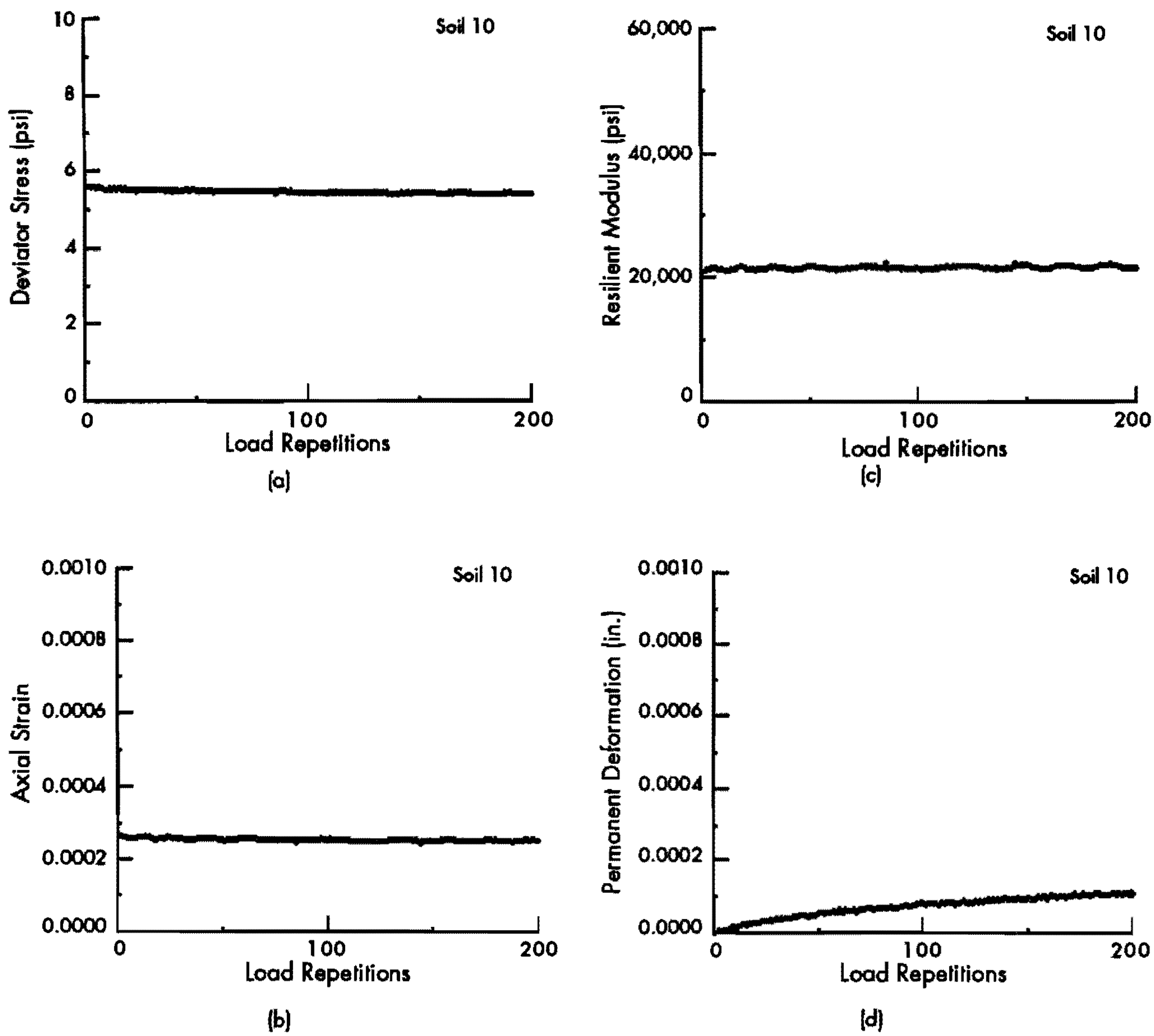


Figure 7.2 Deformational characteristics of a compacted sample of soil 10 (2 days) tested under a 6-psi confining stress and 200 repeated applications of about 5-psi deviator stress. Shown are: (a) the applied deviator stress, (b) the induced axial strain, (c) the resilient modulus, and (d) the permanent deformation of the sample

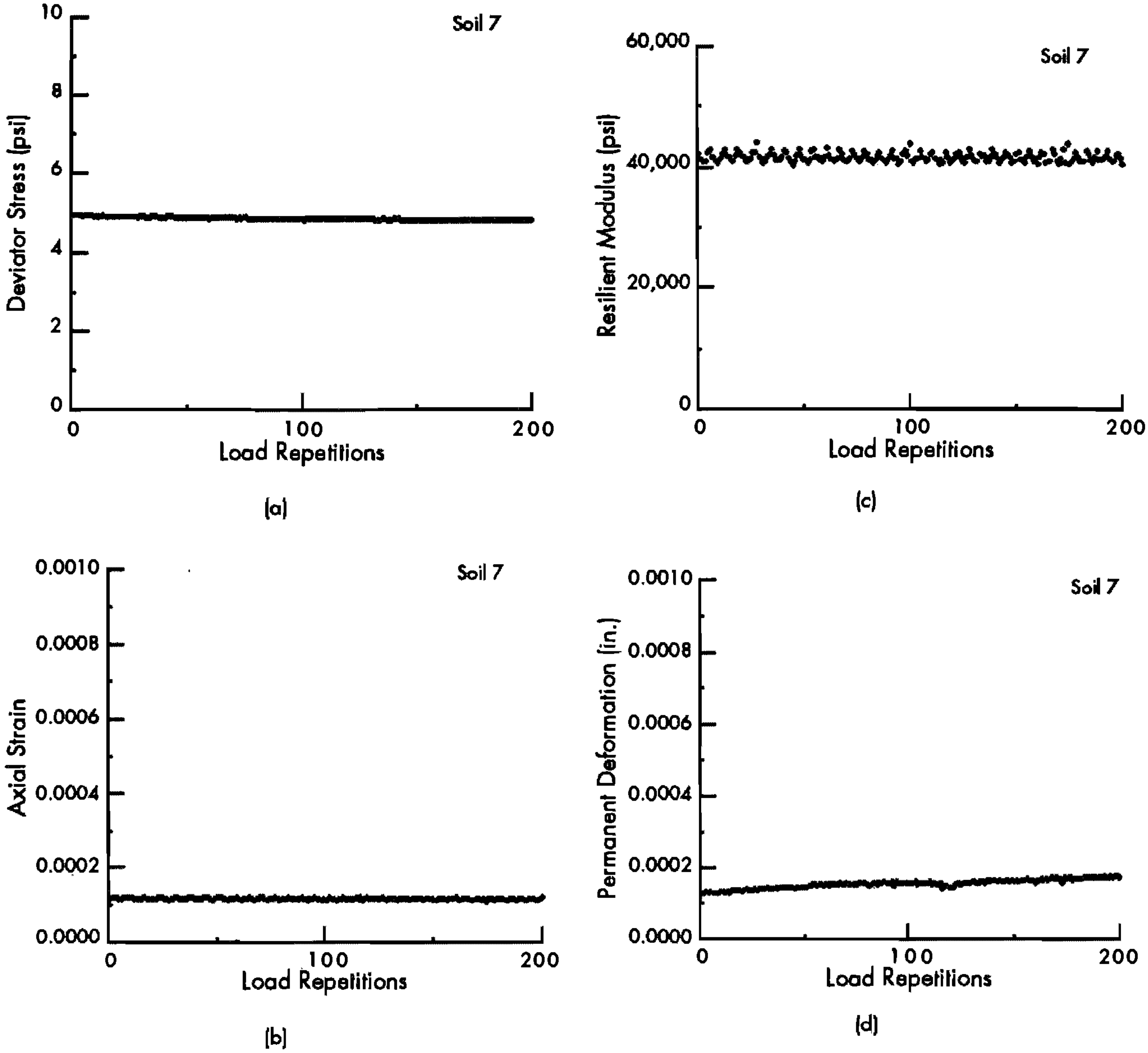


Figure 7.3 **Deformational characteristics of a compacted sample of soil 7 (69 days) tested under a 6-psi confining stress and 200 repeated applications of about 5-psi deviator stress. Shown are: (a) the applied deviator stress, (b) the induced axial strain, (c) the resilient modulus, and (d) the permanent deformation of the sample**

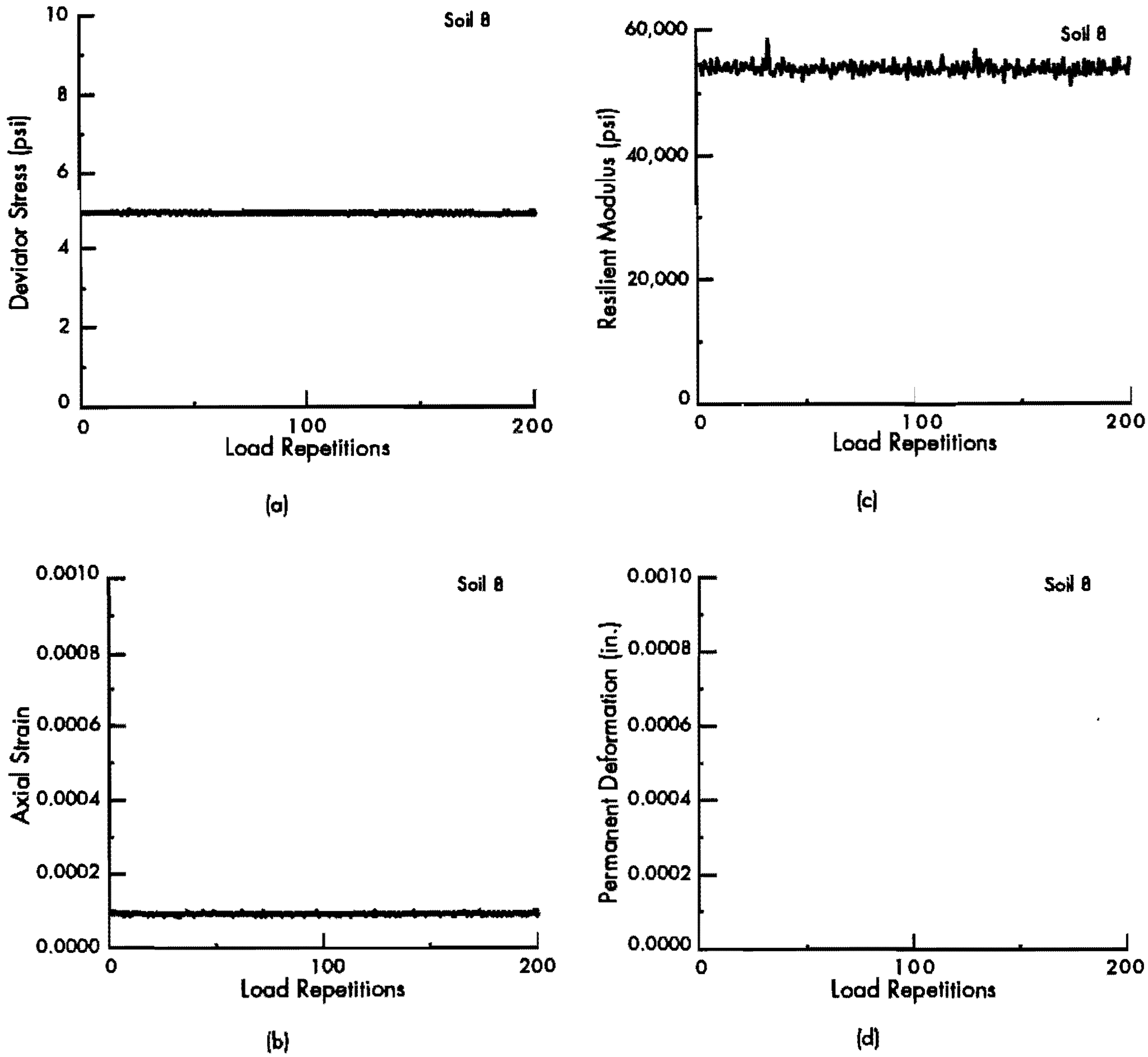


Figure 7.4 Deformational characteristics of a compacted sample of soil 8 (96 days) tested under a 6-psi confining stress and 200 repeated applications of about 5-psi deviator stress. Shown are: (a) the applied deviator stress, (b) the induced axial strain, (c) the resilient modulus, and (d) the permanent deformation of the sample

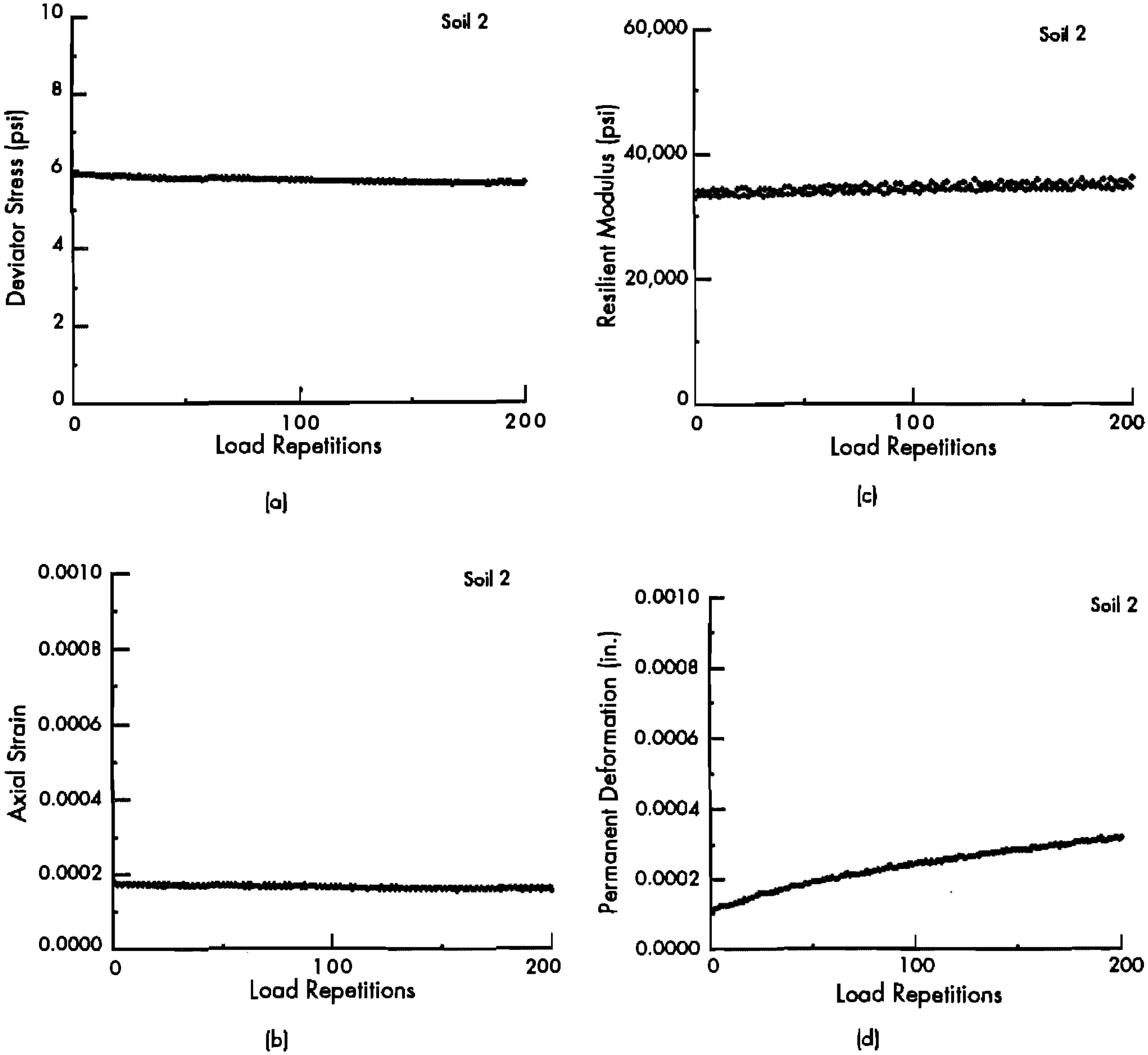


Figure 7.5 Deformational characteristics of a compacted sample of soil 2 (2 days) tested under a 6-psi confining stress and 200 repeated applications of about 6-psi deviator stress. Shown are: (a) the applied deviator stress, (b) the induced axial strain, (c) the resilient modulus, and (d) the permanent deformation of the sample

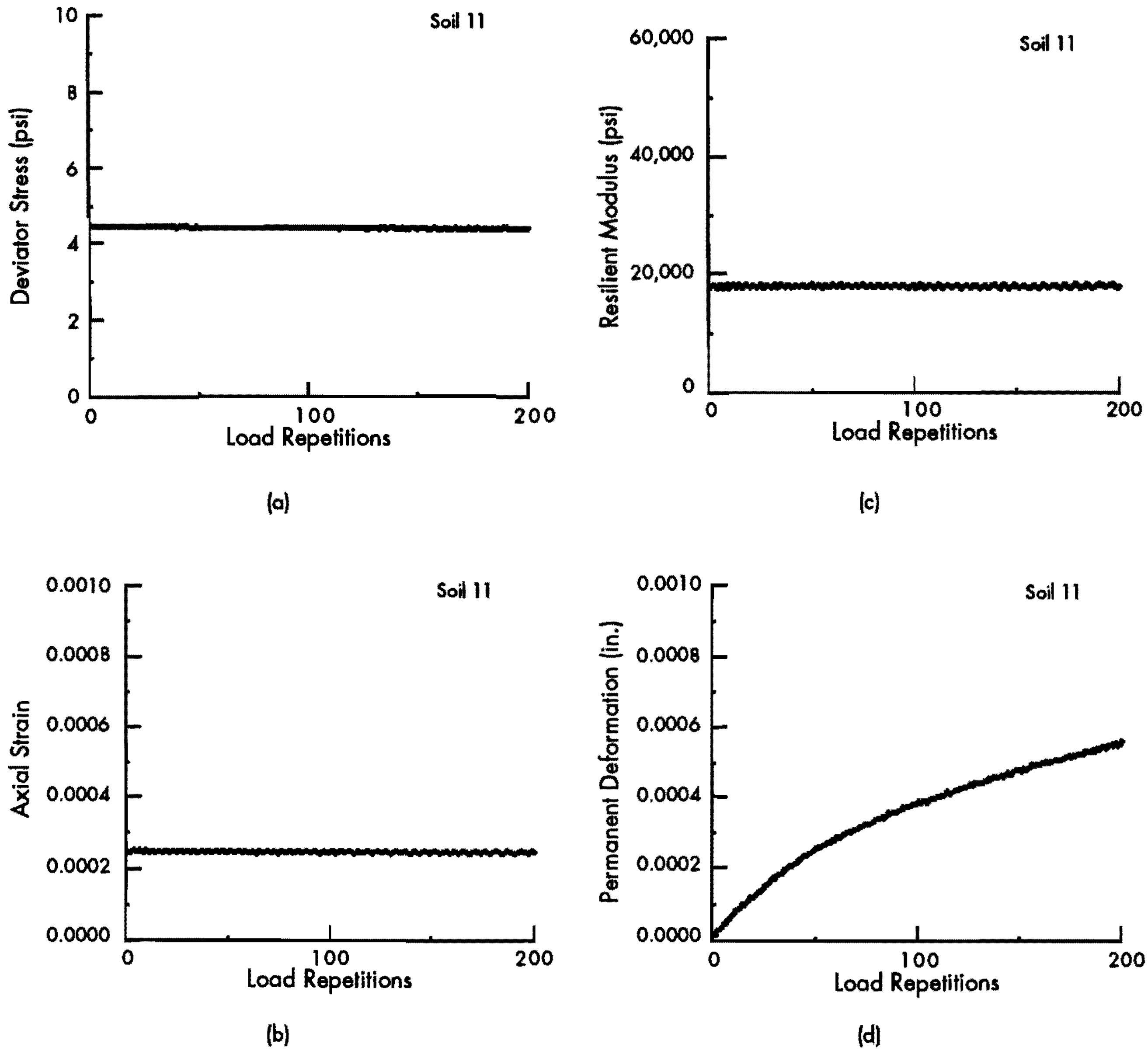


Figure 7.6 Deformational characteristics of a compacted sample of soil 11 (159 days) tested under a 6-psi confining stress and 200 repeated applications of about 4-psi deviator stress. Shown are: (a) the applied deviator stress, (b) the induced axial strain, (c) the resilient modulus, and (d) the permanent deformation of the sample

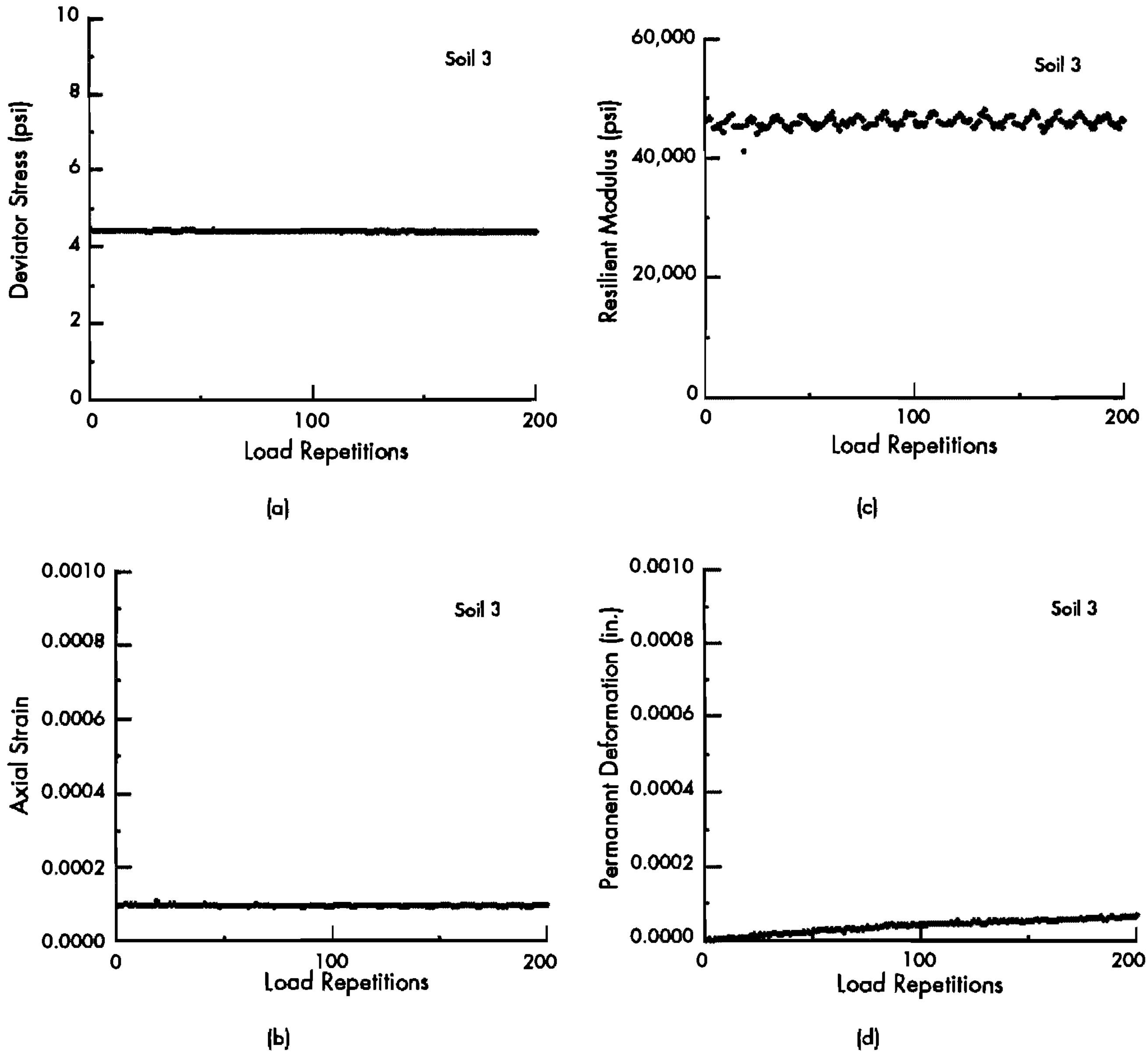


Figure 7.7 Deformational characteristics of a compacted sample of soil 3 (2 days) tested under a 6-psi confining stress and 200 repeated applications of about 4-psi deviator stress. Shown are: (a) the applied deviator stress, (b) the induced axial strain, (c) the resilient modulus, and (d) the permanent deformation of the sample

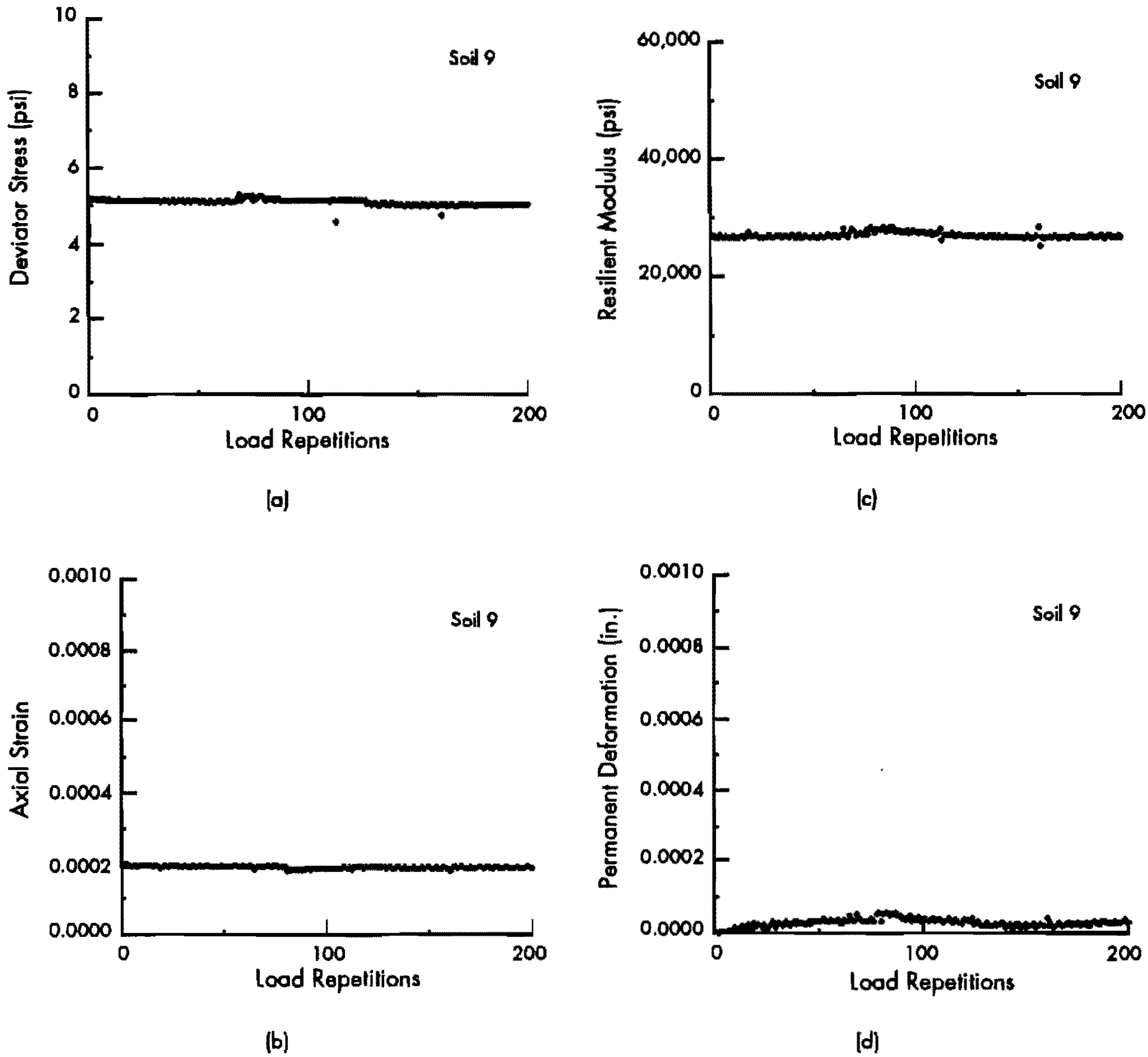


Figure 7.8 **Deformational characteristics of a compacted sample of soil 9 (30 days) tested under a 6-psi confining stress and 200 repeated applications of about 5-psi deviator stress. Shown are: (a) the applied deviator stress, (b) the induced axial strain, (c) the resilient modulus, and (d) the permanent deformation of the sample**

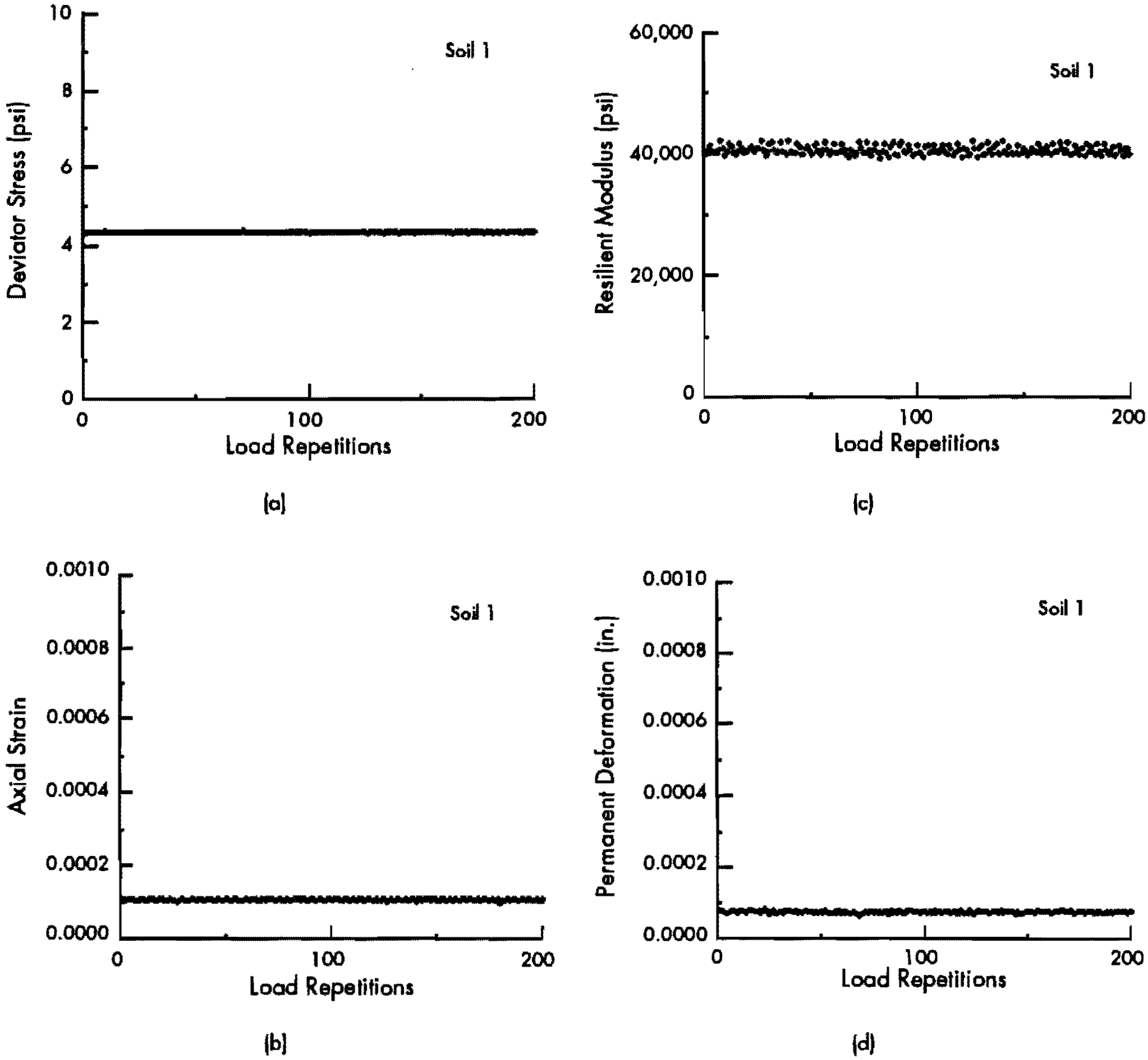


Figure 7.9 Deformational characteristics of a compacted sample of soil 1 (131 days) tested under a 6-psi confining stress and 200 repeated applications of about 4-psi deviator stress. Shown are: (a) the applied deviator stress, (b) the induced axial strain, (c) the resilient modulus, and (d) the permanent deformation of the sample

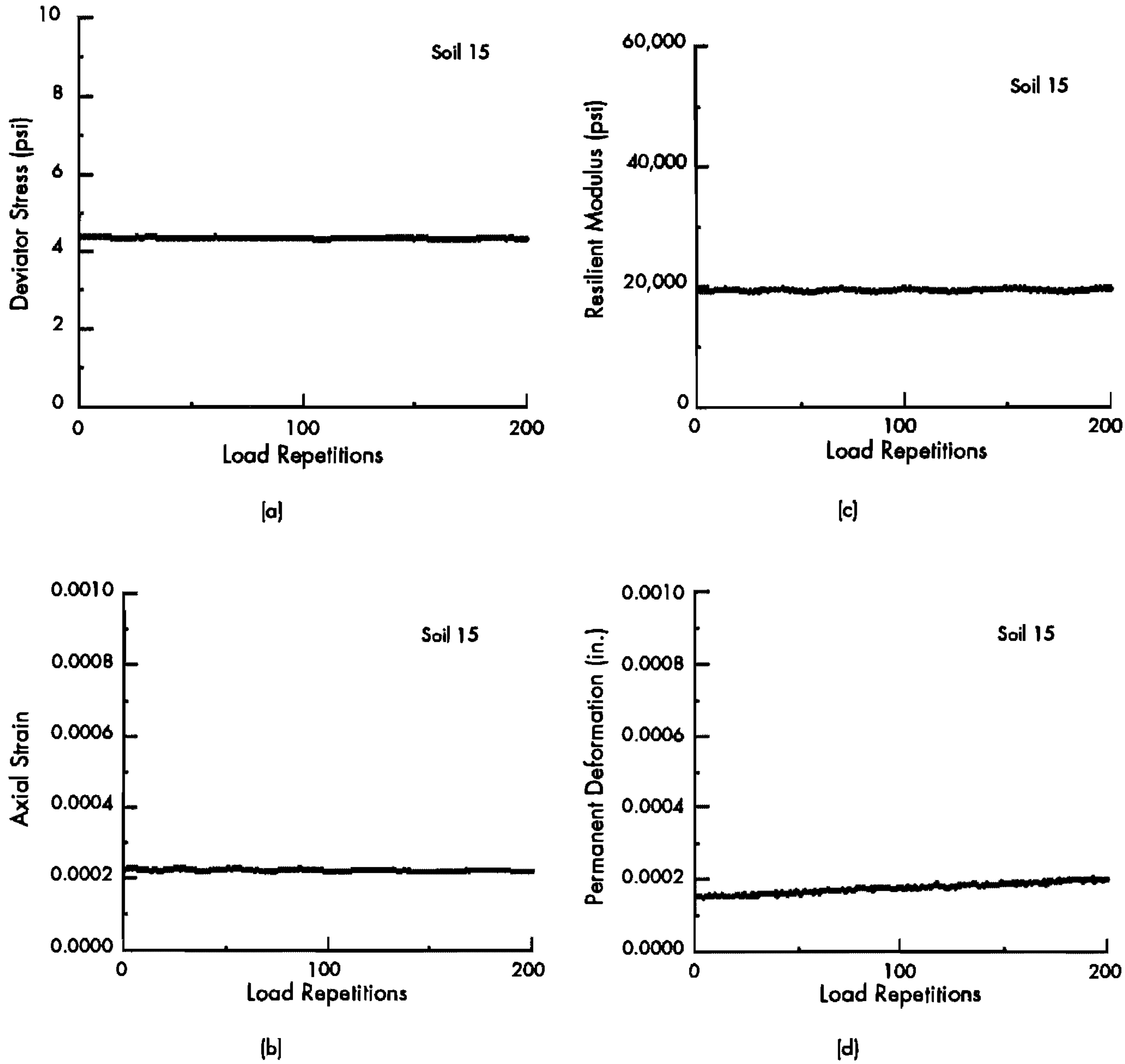


Figure 7.10 Deformational characteristics of a compacted sample of soil 15 (2 days) tested under a 6-psi confining stress and 200 repeated applications of about 4-psi deviator stress. Shown are: (a) the applied deviator stress, (b) the induced axial strain, (c) the resilient modulus, and (d) the permanent deformation of the sample

Table 7.4 Analysis of variance

Item	DF	Sum of Squares	Mean Squares	F
PI_i	4	7180570444	1795142611	1.22
Soil $(PI)_{(i)j}$	5	7341803362	1468360672	5147.81
$State_k$	1	32400	32400	0.11
PI * State $_{(i)k}$	4	3596951	899238	1.06
Soil (PI) * State $_{(i)jk}$	5	4221044	844209	2.96
$Error_{(ijk)l}$	80	22819012	285238	-

CHAPTER 8. EXPERIMENTAL EVALUATION OF THE EFFECT OF NUMBER OF STRESS REPETITIONS

This chapter evaluates the effect of the number of stress repetitions specified in the M_R testing procedure. The same methodology used in the previous chapter to evaluate the effect of conditioning was applied in this evaluation.

INTRODUCTION

As described in Chapter 4, several M_R testing procedures specify that soil samples be subjected to a wide variety of stress states and stress repetitions. For instance, the AASHTO T-274 testing sequence for cohesive soils requires the application of 6, 3, and 0 psi confining pressures at each of the 200 repetitions of 1, 2, 4, 8 and 10-psi deviator stresses. The procedure further specifies that the axial resilient deformation at the 200th repetition be recorded to compute the resilient modulus of that specific stress state.

In contrast, the testing sequence of the prototype procedure consists of the application of 100 repetitions at deviator stresses of 2, 4, 6, 8, and 10 psi at each of the confining pressures of 6, 4, and 2 psi. In all cases, the strain values of the last 5 cycles of the 100 repetitions are recorded and averaged to calculate the resilient modulus values.

Why does AASHTO T-274 specify 200 stress repetitions? And why do other testing procedures (e.g., Washington procedure, Florida method) specify other varieties of stress states and, again, a different number of stress repetitions? Probably because it was thought in the past that, after so many stress repetitions, the material somehow stabilizes. Beyond that, there are no real answers to these questions. These specifications thus appear to be based more on hypothetical conditions than on an experimental evaluation.

The objective of this chapter is to evaluate the effect the number of stress repetitions has on the resilient modulus, with such an evaluation hopefully determining precisely the necessary number of loading applications to be specified in the M_R test.

DESIGN OF THE EXPERIMENT

As in the previous experimental evaluation, this experiment is treated as a nested factorial with blocking at the soil level, since the soils are nested into the PI groups (as explained by Anderson, Ref 46). The factors of interest were: (1) the plasticity index, (2) the soil, (3) the deviator stress, and (4) the number of stress repetitions. Table 8.1 presents the arrangement of this particular experiment.

The plasticity index, PI, had five levels (the five PI groups). The soil factor, expressed as Soil (PI), had two different soils (selected at random) in each of the PI groups. The deviator stress, Dev, had five levels; the number of stress repetitions, Rep, had five levels also.

The testing sequence used in this evaluation consisted of applying, 200 times, 5 different deviator stresses under a single confining pressure (to reduce the number of units within the experiment). The deviator stresses used were 2, 4, 6, 8, and 10 psi, as specified in our prototype method, under a 6-psi confining pressure.

The four testing parameters (deviator stress, resilient axial strain, resilient modulus, and permanent deformation) were monitored throughout the testing sequence. This meant that the total testing data would have 200 records per deviator stress, per deformational parameter, and per test sample; in other words, an ample amount of information. Consequently, only resilient modulus values were used in the analysis, since the objective was to determine the effect of the number of stress repetitions on that particular parameter. The other three parameters were recorded to check the results and the testing operation, as was described in Chapter 7.

Five stress repetition levels were defined: the 5th, the 25th, the 50th, the 100th, and the 200th loading applications. This is the essence of this experiment, insofar as the effect of the number of stress repetitions on the moduli can be evaluated and the number of stress repetitions actually necessary in the M_R test can be determined.

Table 8.1 Design of the experiment

PI Group	0 - 10		11 - 20		21 - 30		31 - 40		> 40	
Soil ID / Number of Stress Repetitions	4	10	6	7	11	16	9	13	1	15
5	●	●	●	◍	●	◍	●	●	●	●
25	●	●	◍	◍	◍	◍	◍	●	◍	◍
50	●	◍	●	◍	●	◍	●	◍	●	◍
100	●	◍	●	◍	●	◍	●	◍	●	◍
200	●	●	●	●	●	◍	◍	●	◍	●

The model of this experiment is:

$$
\begin{aligned}
M_{R_{ijklm}} = {} & u + PI_i + Soil(PI)_{(i)j} + Dev_k + PI * Dev_{ik} \\
& + Soil(PI) * Dev_{(i)jk} + Rep_l + PI * Rep_l \\
& + Dev * Rep_{kl} + Soil(PI) * Rep_{(i)jl} + PI \\
& * Dev * Rep_{ikl} + Soil(PI) * Dev \\
& * Rep_{(i)jkl} + Error_{(ijkl)m}
\end{aligned}
$$

where

$M_{R_{ijklm}}$ = resilient modulus of the sample of the j^{th} soil of i^{th} plasticity index at the k^{th} deviator stress and l^{th} loading application,

u = overall mean,

PI_i = the effect of the i^{th} plasticity index,

$Soil(PI)_{(i)j}$ = the effect of the j^{th} soil,

Dev_k = the effect of the deviator stress,

$PI \cdot Dev_{ik}$ = the effect of the interaction of the i^{th} plasticity index with the k^{th} deviator stress,

Rep_l = the effect of the l^{th} loading application (stress repetition),

$PI \cdot Rep_{il}$ = the effect of the interaction of the i^{th} plasticity index with the l^{th} loading application,

$Soil(PI) \cdot Rep_{(i)jl}$
= the effect of the interaction of the j^{th} soil with the l^{th} loading application,

$Dev \cdot Rep_{kl}$ = the effect of the interaction of the k^{th} deviator stress with the l^{th} loading application,

$PI \cdot Dev \cdot Rep_{ikl}$
= the effect of the interaction of the i^{th} plasticity index with the k^{th} deviator stress and with the l^{th} loading application,

$Soil(PI) \cdot Dev \cdot Rep_{(i)jkl}$ = the effect of the interaction of the j^{th} soil with the k^{th} deviator stress and with the l^{th} loading application, and

$Error_{(ijkl)m}$ = the experimental error.

To define a broader inference space and to permit more general conclusions, we prepared test samples under randomly chosen moisture conditions and tested them at randomly chosen sample ages. However, it should be pointed out that, because the soil types used were only fine-grain soils, the inference space of our conclusions refers only to this soil type.

It should also be recognized that this complete nested factorial experiment with two- and three-factor interactions has a restriction on randomization, since the soil is nested into the PI group. And because replicates of the experimental units were not considered, the error term and the interactions may be affected to some degree. Accordingly, the expected mean square algorithm of this experiment was developed (as shown in Table 8.2).

COLLECTION OF THE DATA

All data collected in this experiment were obtained from the testing of samples that were grouted to the end caps. After these samples were subjected to the sample conditioning, they were subjected to a sequence of stress states. Based on the conclusions drawn from the previous evaluation, it is clear that such a conditioning stage does not affect the quality of the data collected in this particular experiment.

The testing sequence consisted of 200 applications of 2-, 4-, 6-, 8-, and 10-psi deviator stresses to the test sample that is subjected to an all-around confining pressure of 6 psi during the entire testing operation.

Ten soil samples were used in this experiment: soils 4 and 10 of PI group 0-10, soils 6 and 7 of PI group 11-20, soils 11 and 16 of PI group 21-30, soils 9 and 13 of PI group 31-40, and soils 1 and 15 of PI group >40. Table 8.3 summarizes their basic characteristics.

EXPERIMENTAL OBSERVATIONS

Figures 8.1 through 8.10 present the results obtained from the testing of 10 different compacted samples. In all cases the results show a well-defined, non-linear stress-strain behavior of the material: as the deviator stress and resilient strain increase, the resilient modulus decreases; and as the number of stress applications increases, the cumulative permanent deformation also increases.

Figure 8.1 shows the results obtained from the testing of the compacted sample of soil 4, with Figure 8.1a illustrating each of the five deviator stresses at 200 applications. It should be emphasized that in some cases the magnitude of first deviator stress applied was higher than 2 psi, since the induced resilient axial strains caused by such a deviator stress were much lower than the minimum reliable value that can be recorded by our M_R system, as shown in Figure 8.1b. Thus, higher variations in the resilient modulus values computed from resilient axial strains close to 0.01 percent can be observed; but once the value of the axial strain is greater than such a limit, the variability of the resilient modulus values is reduced and appears to remain constant, as shown in Figure 8.1c. Finally, Figure 8.1d shows that only the permanent deformation generated during the entire testing sequence changes with the number of stress repetitions. Figures 8.2 and 8.3 show similar results obtained from the testing of compacted samples of soil 10 and soil 6, respectively.

Table 8.2 Expected mean square algorithm

Item	Expected Mean Squares
PI_i	$\sigma^2 + 25 \cdot \sigma^2$ (Soil (PI)) + $50 \cdot \Phi$ (PI)
Soil $(PI)_{(i)j}$	$\sigma^2 + 25 \cdot \sigma^2$ (Soil (PI))
Dev_k	$\sigma^2 + 50 \cdot \phi$ (Dev)
PI * $Dev_{(i)k}$	$\sigma^2 + 5 \cdot \sigma^2$ (Soil (PI)*Dev) + $10 \cdot \sigma^2$ (PI*Dev)
Soil (PI) * $Dev_{(i)jk}$	$\sigma^2 + 5 \cdot \sigma^2$ (Soil (PI)*Dev)
Rep_l	$\sigma^2 + 50 \cdot \phi$ (Rep)
PI * Rep_{il}	$\sigma^2 + 5 \cdot \sigma^2$ (Soil (PI)*Rep) + $10 \cdot \sigma^2$ (PI*Rep)
Soil(PI) * $Rep_{(i)jl}$	$\sigma^2 + 5 \cdot \sigma^2$ (Soil (PI)*Rep)
Dev * Rep_{kl}	$\sigma^2 + 10 \cdot \phi$(Rep)
PI * Dev * Rep_{ikl}	$\sigma^2 + \sigma^2$ (Soil (PI)*Dev*Rep) + $2 \cdot \sigma^2$ (PI*Dev*Rep)
Soil(PI) * Dev * $Rep_{(i)jkl}$	$\sigma^2 + \sigma^2$ (Soil (PI)*Dev*Rep)
$Error_{(ijkl)m}$	σ^2

Table 8.3 Basic characteristics of the test sample

PI Group	Soil ID	Moisture Content (%)	Dry Density (pcf)	Sample Age (days)
0 - 10	4	14.10	122.41	188
	10	10.10	125.20	6
11-20	6	11.80	117.30	2
	7	20.10	104.40	2
21-30	11	16.00	110.10	159
	16	20.10	106.89	64
31-40	9	20.10	103.04	6
	13	18.00	100.20	2
> 40	1	21.20	93.62	131
	15	21.20	104.10	69

Figure 8.4 illustrates the testing results of the compacted sample of soil 7. Again, a well-defined stress-strain behavior is observed, with negligible variations of the responses to the applications of the different deviator stresses evident. The same comments are applicable to the results obtained in the testing of compacted samples of soils 11, 16, 9, 13, 1, and 15, as shown in Figures 8.5, 8.6, 8.7, 8.8, and 8.9, respectively.

The results thus indicate that the number of applications (200) of the different deviator stresses required to compute the resilient modulus at the 200th repetition (or even at the 100th repetition) may be unnecessarily high.

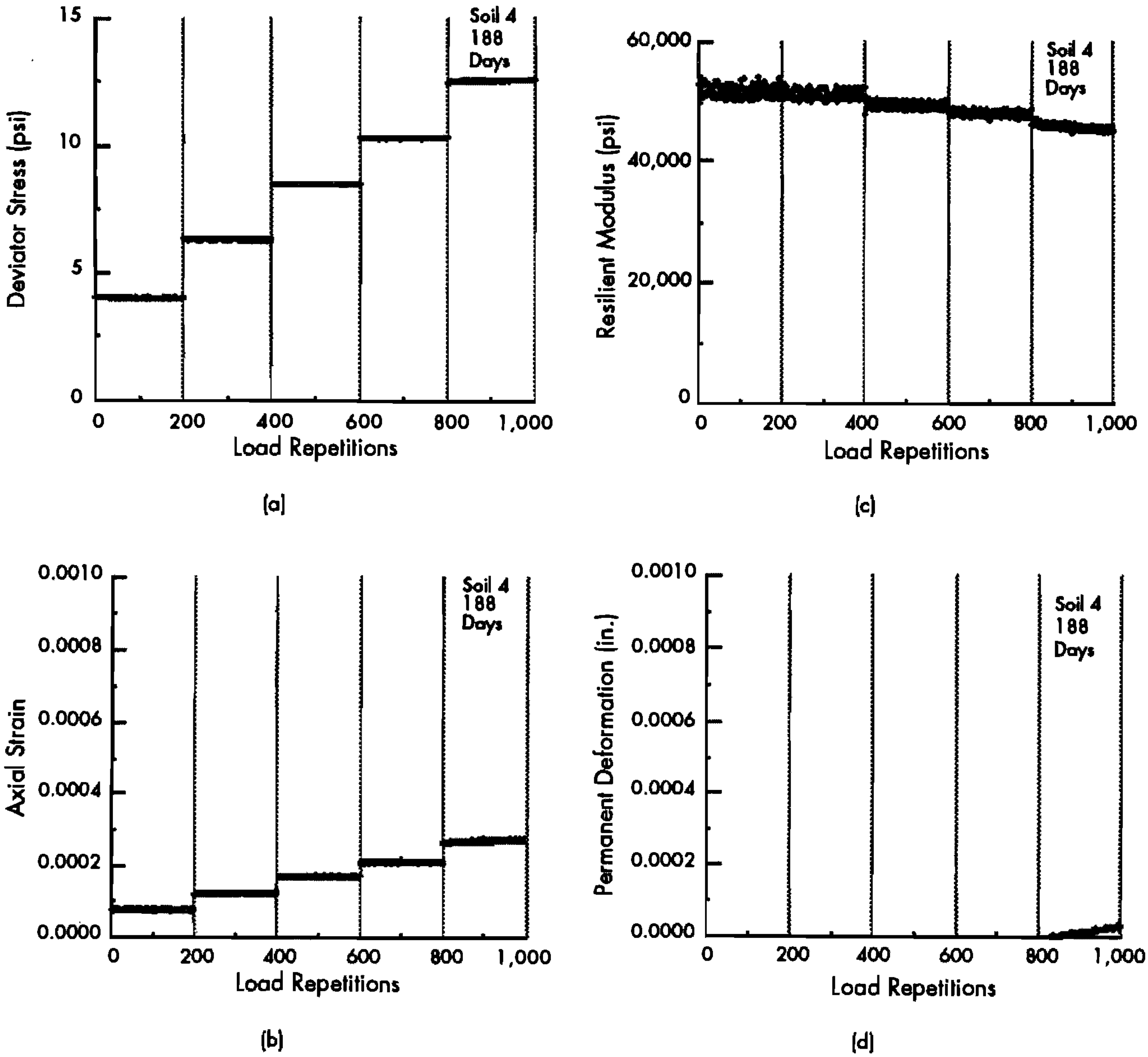


Figure 8.1 Deformational characteristics of a compacted sample of soil 4 (188 days) tested under a 6-psi confining stress and under 200 applications of a 2-psi, 4-psi, 6-psi, 8-psi, and 10-psi deviator stress. Shown are: (a) the applied deviator stresses, (b) the resilient axial strains, (c) the computed resilient moduli, and (d) the permanent deformations

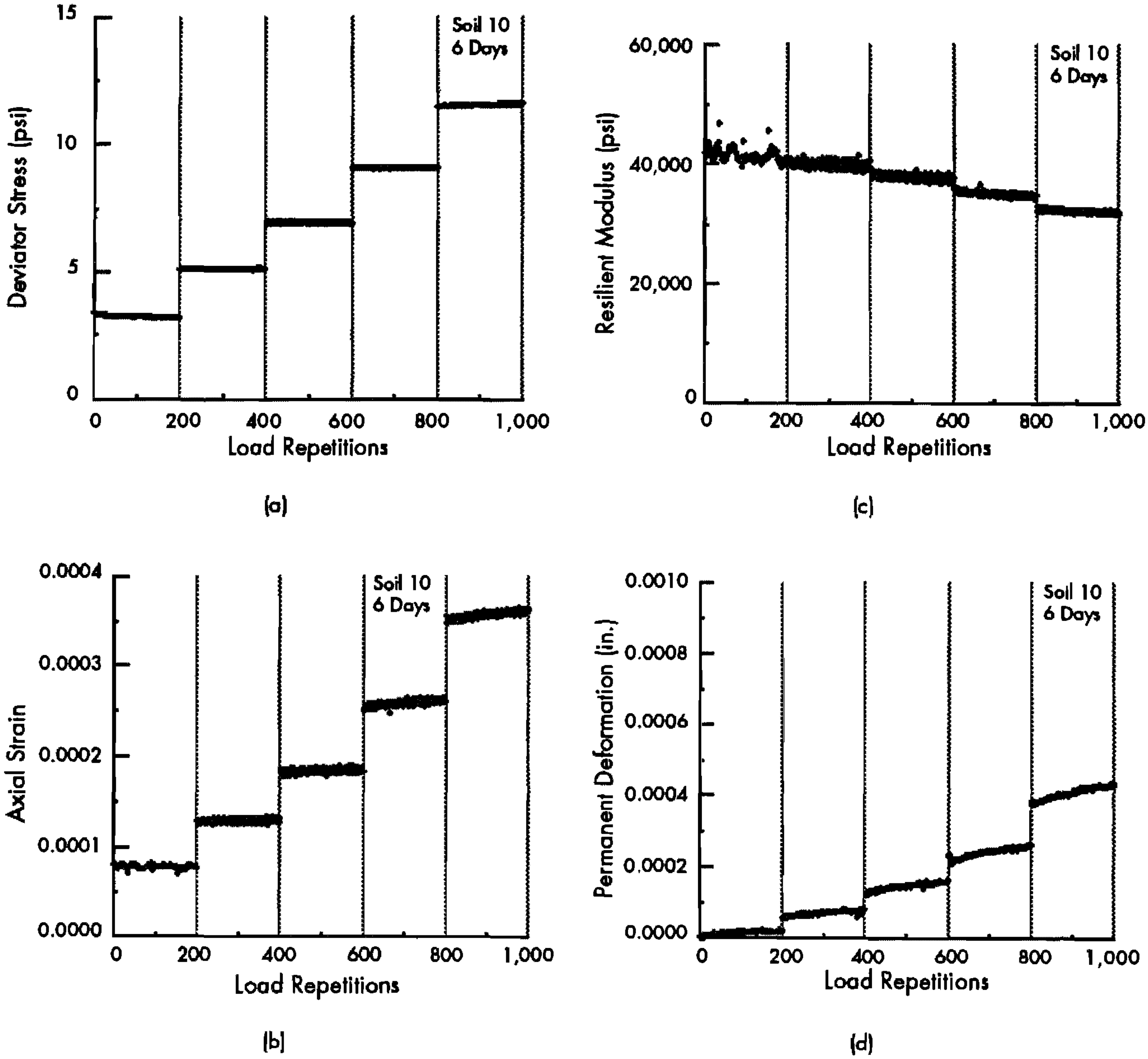


Figure 8.2 Deformational characteristics of a compacted sample of soil 10 (6 days) tested under a 6-psi confining stress and under 200 applications of a 2-psi, 4-psi, 6-psi, 8-psi, and 10-psi deviator stress. Shown are: (a) the applied deviator stresses, (b) the resilient axial strains, (c) the computed resilient moduli, and (d) the permanent deformations

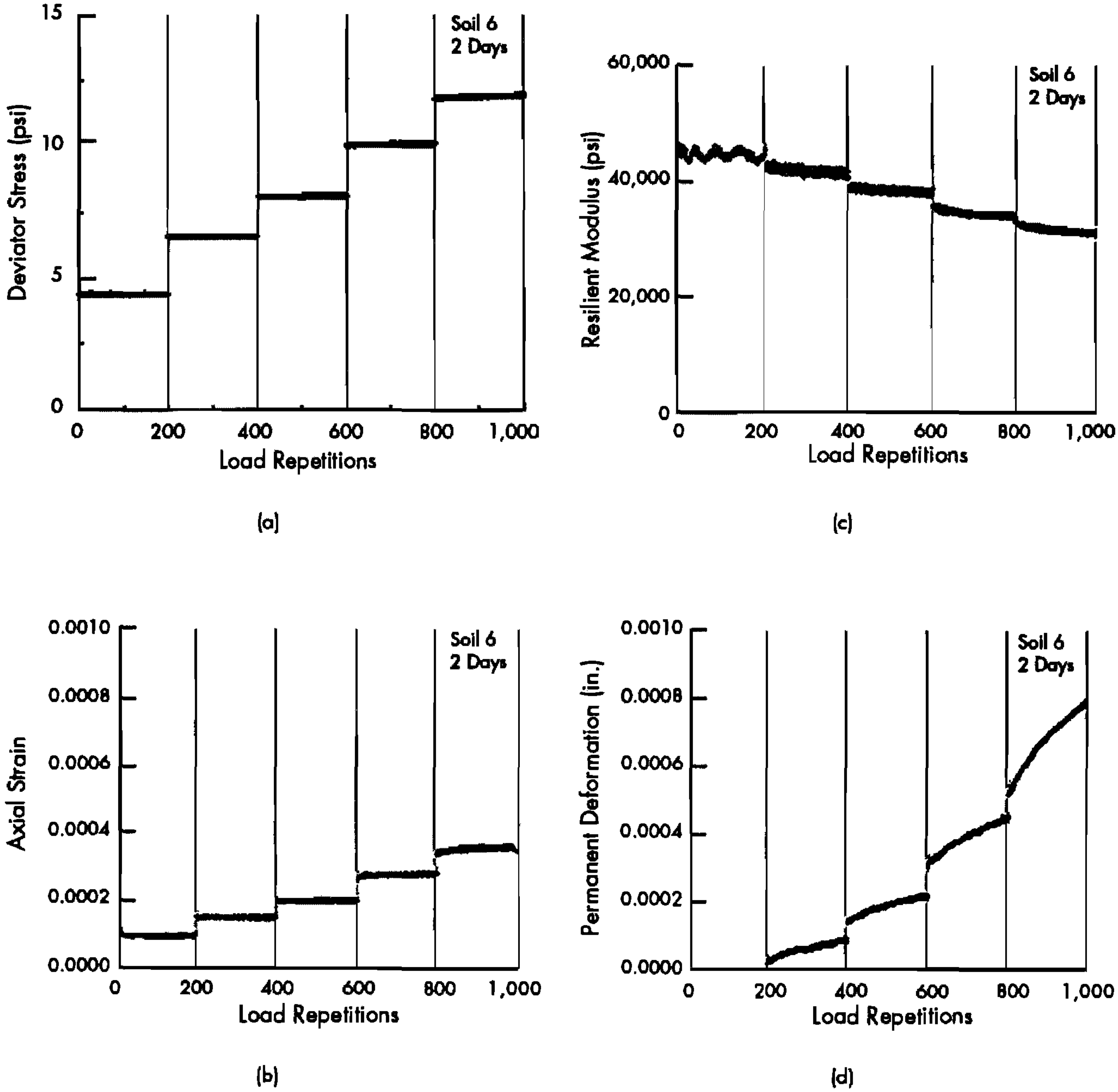


Figure 8.3 Deformational characteristics of a compacted sample of soil 6 (2 days) tested under a 6-psi confining stress and under 200 applications of a 2-psi, 4-psi, 6-psi, 8-psi, and 10-psi deviator stress. Shown are: (a) the applied deviator stresses, (b) the resilient axial strains, (c) the computed resilient moduli, and (d) the permanent deformations

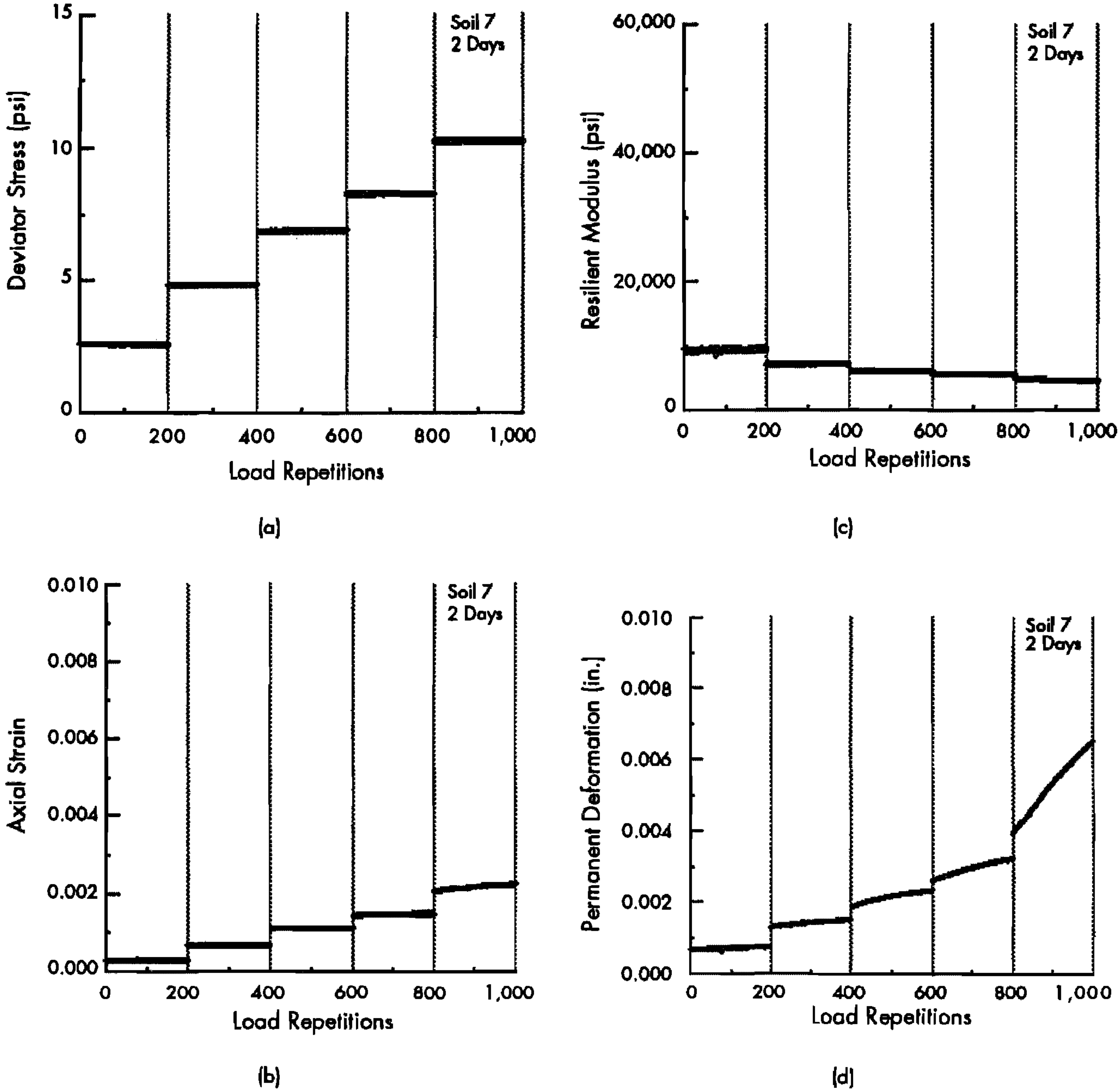


Figure 8.4 Deformational characteristics of a compacted sample of soil 7 (2 days) tested under a 6-psi confining stress and under 200 applications of a 2-psi, 4-psi, 6-psi, 8-psi, and 10-psi deviator stress. Shown are: (a) the applied deviator stresses, (b) the resilient axial strains, (c) the computed resilient moduli, and (d) the permanent deformations

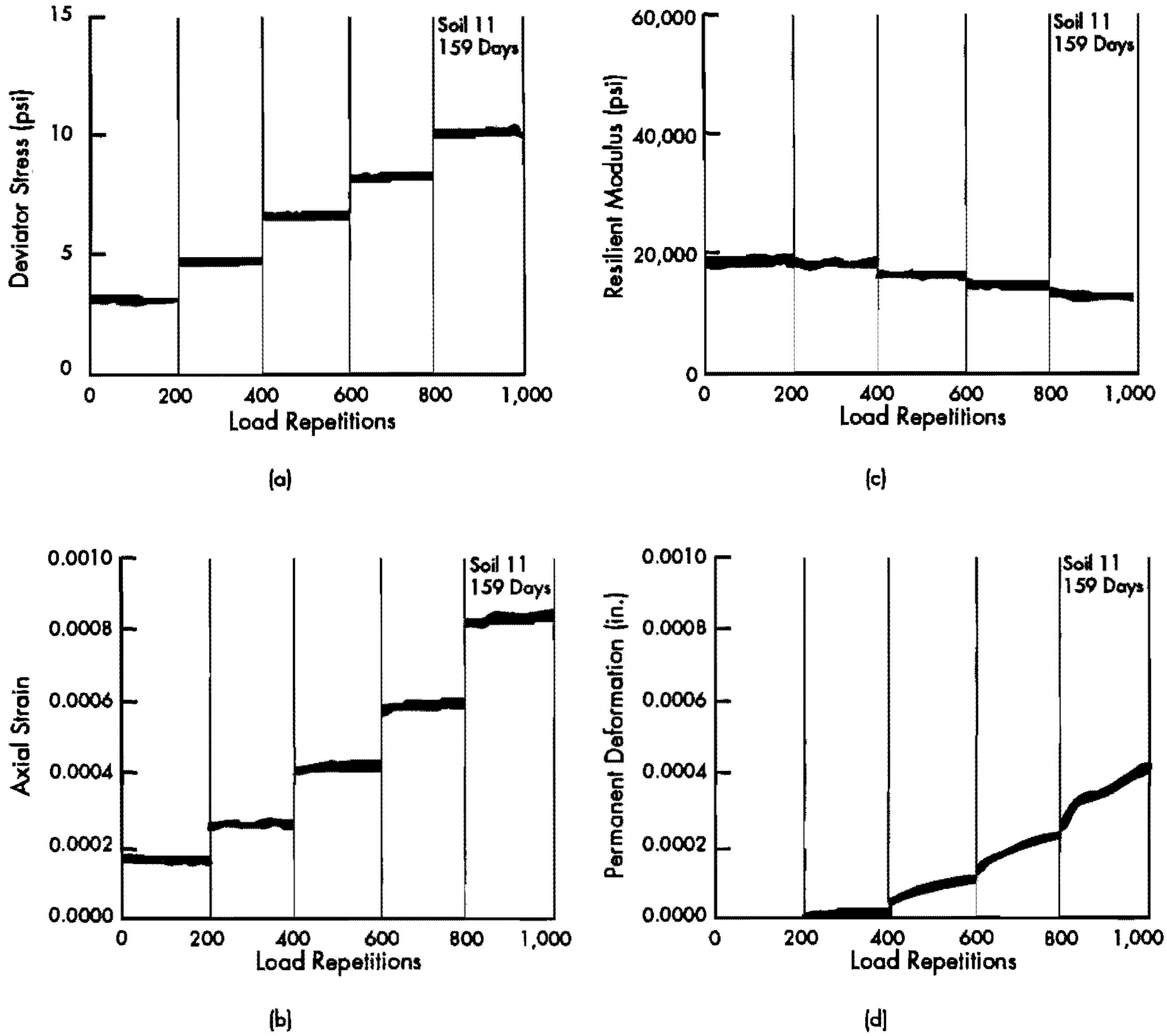


Figure 8.5 Deformational characteristics of a compacted sample of soil 11 (159 days) tested under a 6-psi confining stress and under 200 applications of a 2-psi, 4-psi, 6-psi, 8-psi, and 10-psi deviator stress. Shown are: (a) the applied deviator stresses, (b) the resilient axial strains, (c) the computed resilient moduli, and (d) the permanent deformations

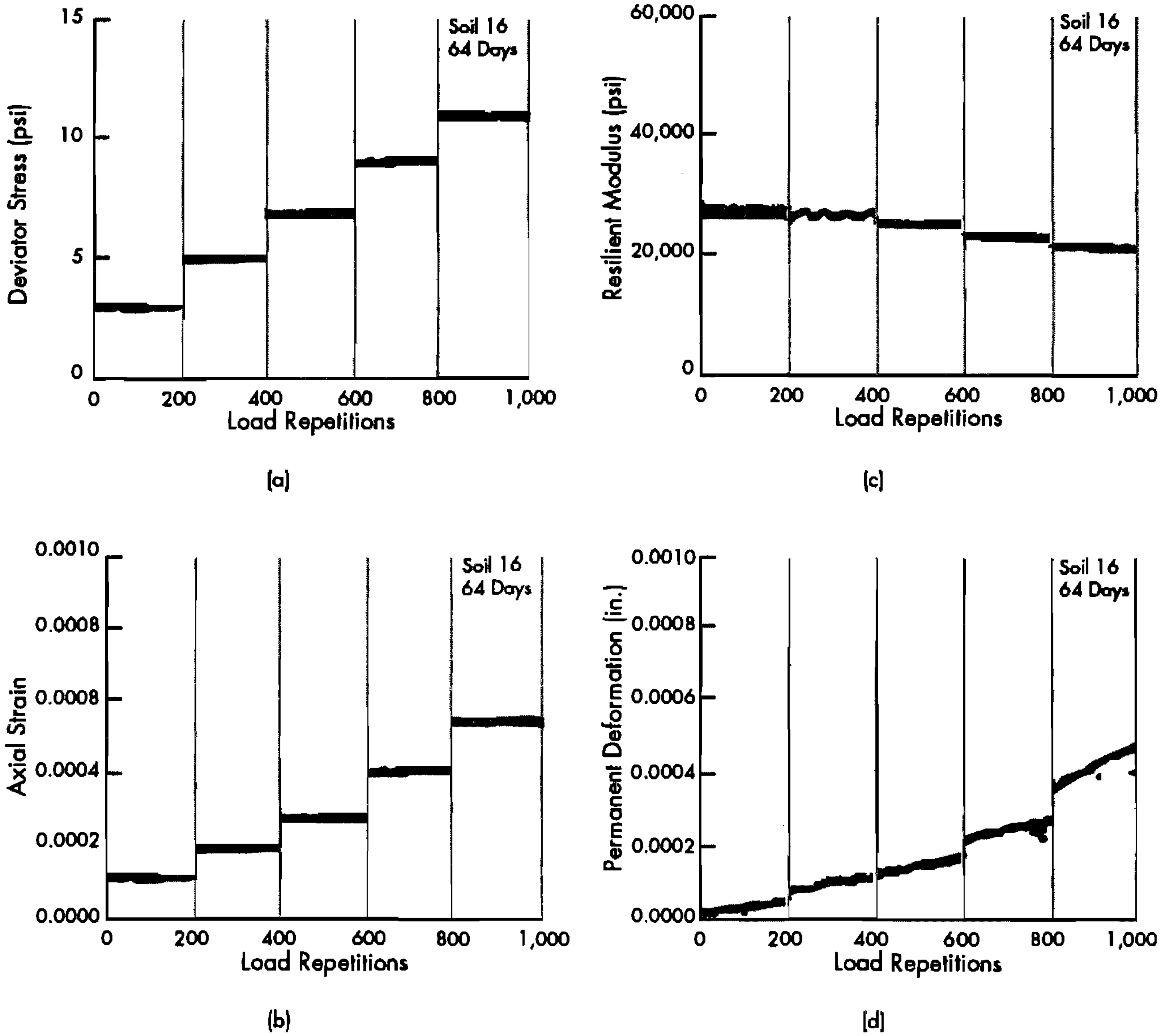


Figure 8.6 Deformational characteristics of a compacted sample of soil 16 (64 days) tested under a 6-psi confining stress and under 200 applications of a 2-psi, 4-psi, 6-psi, 8-psi, and 10-psi deviator stress. Shown are: (a) the applied deviator stresses, (b) the resilient axial strains, (c) the computed resilient moduli, and (d) the permanent deformations

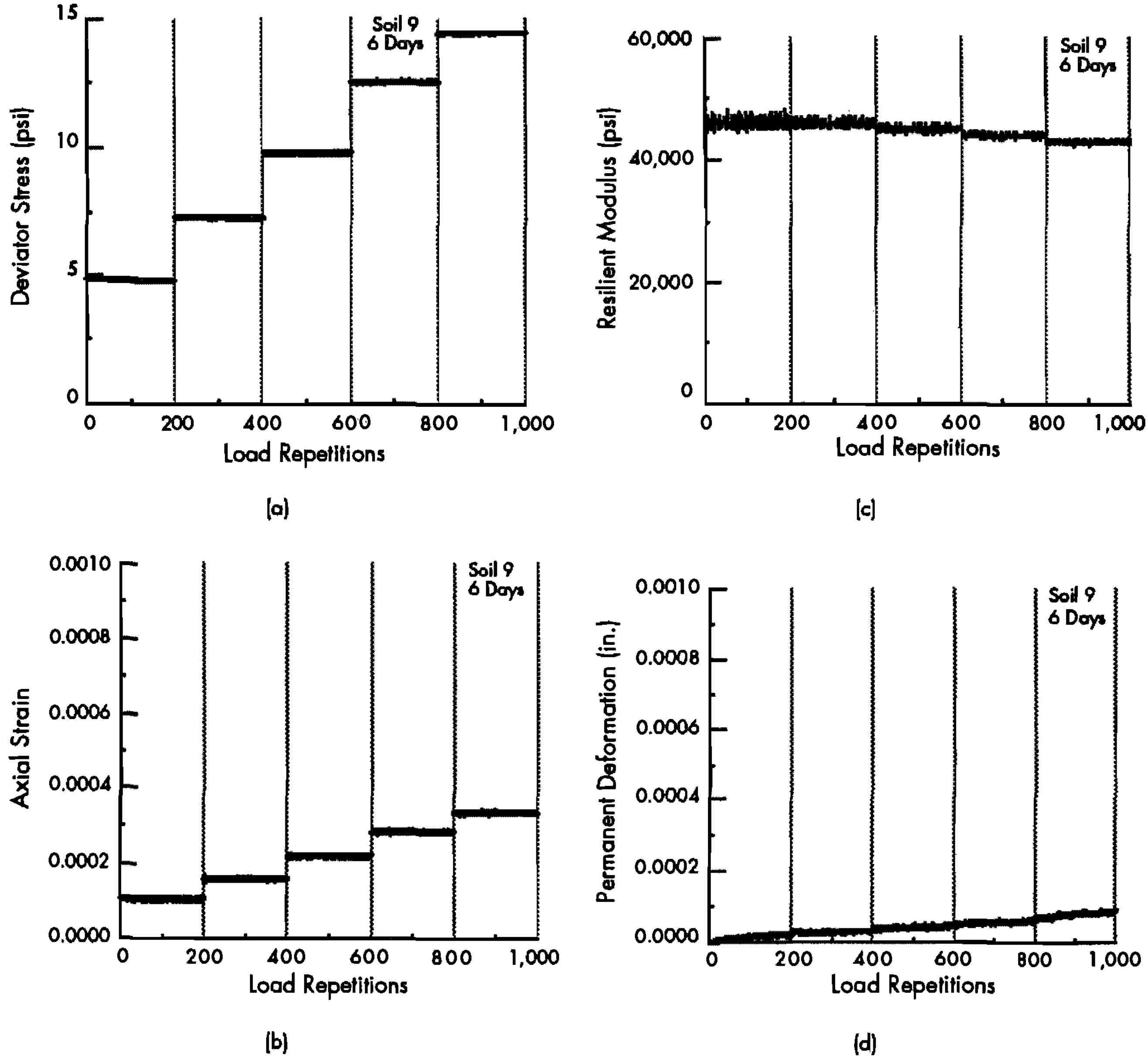


Figure 8.7 **Deformational characteristics of a compacted sample of soil 9 (6 days) tested under a 6-psi confining stress and under 200 applications of a 2-psi, 4-psi, 6-psi, 8-psi, and 10-psi deviator stress. Shown are: (a) the applied deviator stresses, (b) the resilient axial strains, (c) the computed resilient moduli, and (d) the permanent deformations**

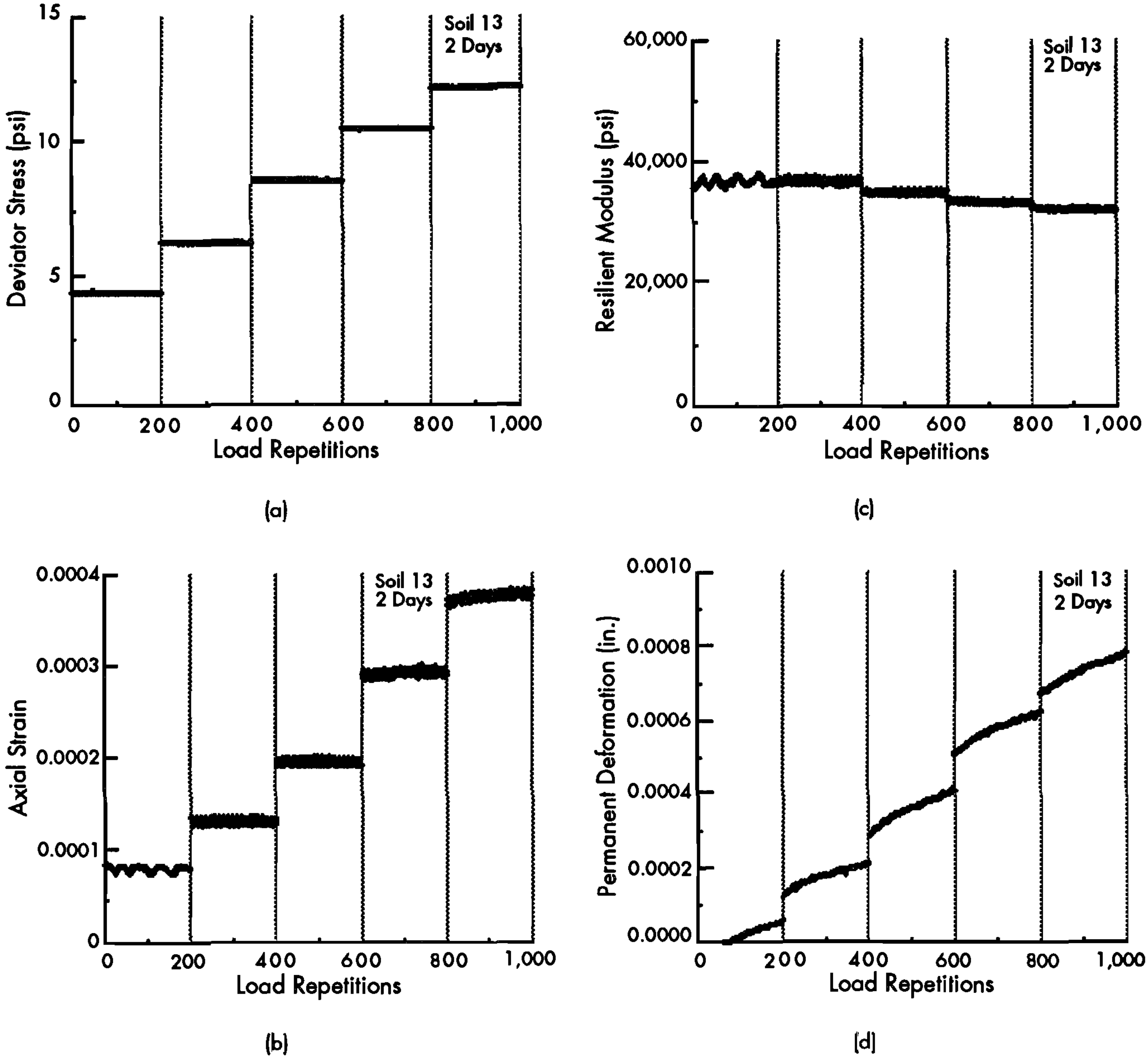


Figure 8.8 Deformational characteristics of a compacted sample of soil 13 (2 days) tested under a 6-psi confining stress and under 200 applications of a 2-psi, 4-psi, 6-psi, 8-psi, and 10-psi deviator stress. Shown are: (a) the applied deviator stresses, (b) the resilient axial strains, (c) the computed resilient moduli, and (d) the permanent deformations

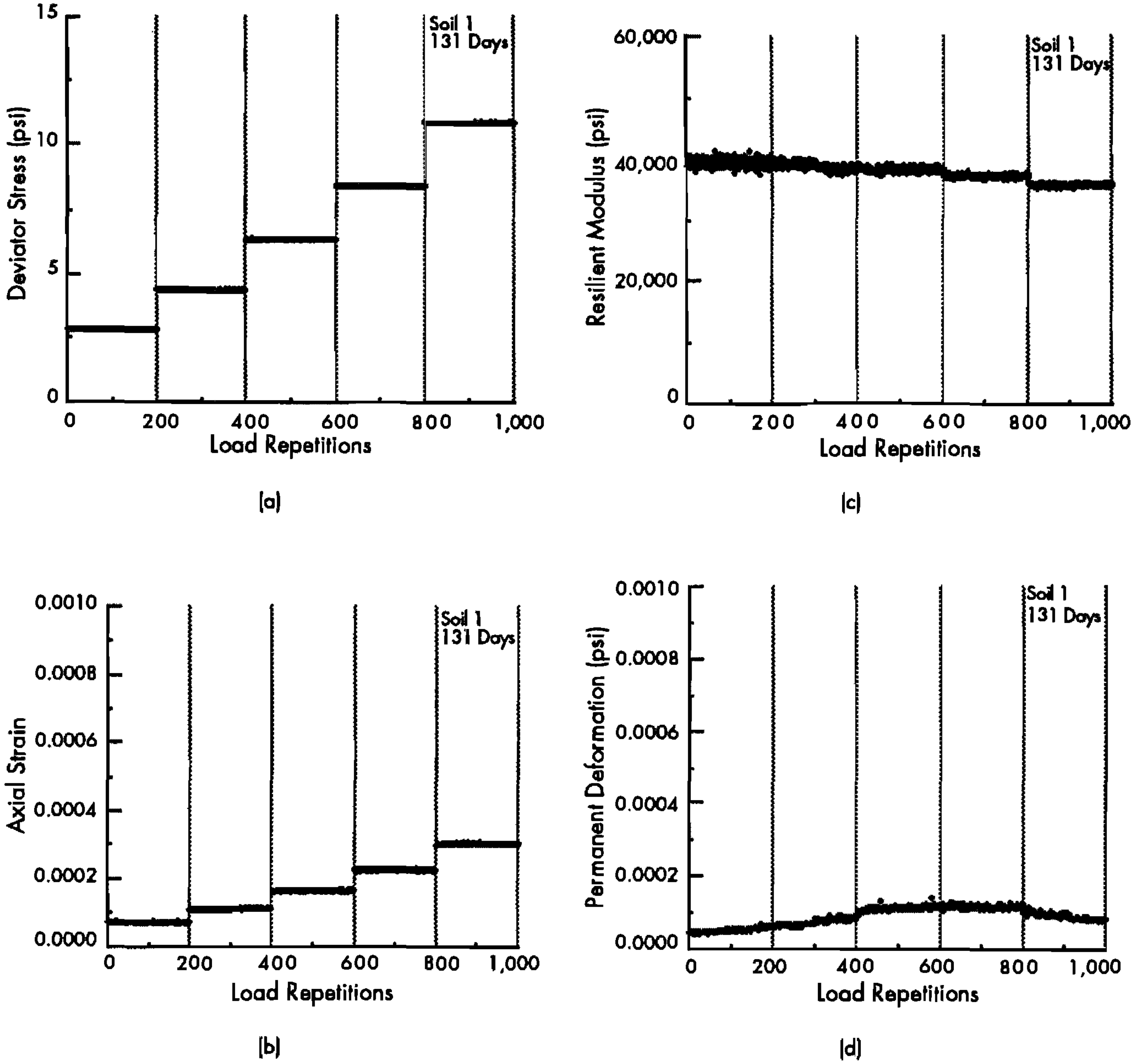


Figure 8.9 Deformational characteristics of a compacted sample of soil 1 (131 days) tested under a 6-psi confining stress and under 200 applications of a 2-psi, 4-psi, 6-psi, 8-psi, and 10-psi deviator stress. Shown are: (a) the applied deviator stresses, (b) the resilient axial strains, (c) the computed resilient moduli, and (d) the permanent deformations

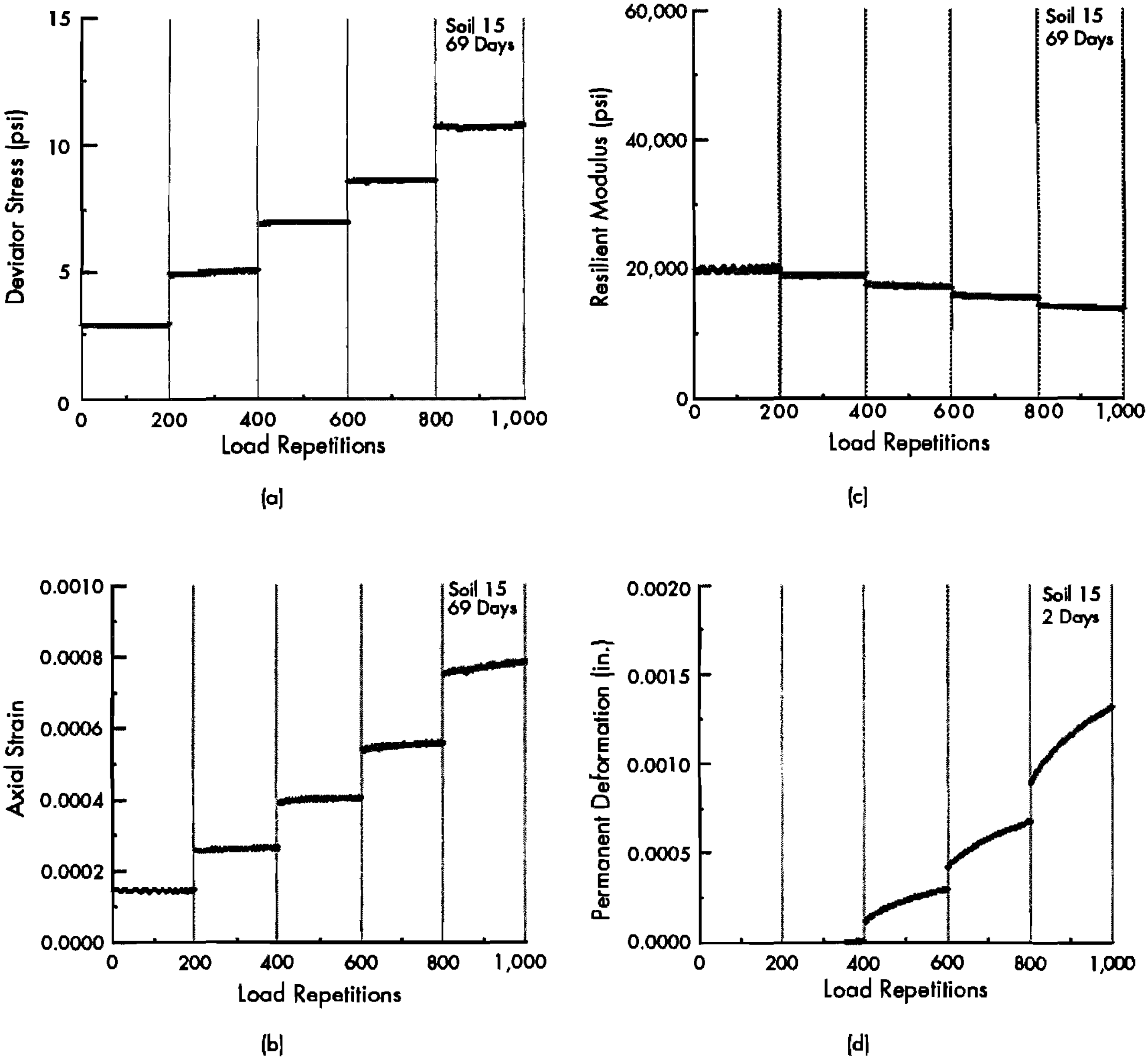


Figure 8.10 Deformational characteristics of a compacted sample of soil 15 (69 days) tested under a 6-psi confining stress and under 200 applications of a 2-psi, 4-psi, 6-psi, 8-psi, and 10-psi deviator stress. Shown are: (a) the applied deviator stresses, (b) the resilient axial strains, (c) the computed resilient moduli, and (d) the permanent deformations

ANALYSIS OF THE EXPERIMENT

The analysis of this experiment was also performed using the statistical analysis software (SAS). However, in this case the mainframe version was used to perform the analysis of variances of our experimental model of four main effects, five 2-factor interactions, and two 3-factor interactions. Because there was an overwhelming amount of information, we decided to select only the most representative resilient modulus values for analysis. Consequently, the levels of two factors were reconsidered: (1) the deviator stress factor, and (2) the number-of-stress-repetitions factor.

Deviator stresses were, in reality, numerically different from one test to another; therefore, to identify properly the levels of this factor, it was necessary to define the first applied deviator stress as "Dev 1" level, the second applied deviator stress as "Dev 2" level, the third applied deviator stress as "Dev 3" level, and so on.

Since 200 stress repetitions of each deviator stress were applied, five levels of this factor were defined: the first five repetitions as "Rep 5," the second set of five repetitions (21-25) as "Rep 25," the third set of five repetitions (46-50) as "Rep 50," the fourth set of five repetitions (96-100) as "Rep 100," and the fifth set of five repetitions (196-200) as "Rep 200."

First, we performed tests for homogeneity of variance and normality, as suggested by Anderson (Ref 46) and Wonacott (Ref 47). We found from those tests that there was a need for transforming the data. The logarithmic function was the transforming function selected; thus the data were analyzed using such transformed units (log, psi).

Table 8.4 summarizes the results of the analysis of variances obtained on a mainframe computer (such analysis required 40 minutes). Table 8.3 includes the main factors, the 2- and 3-factor interactions of the model, their degree of freedom, sum of squares, mean squares, "F" values, and their "F" tests at 5 and 25 percent significance levels.

Table 8.4 Analysis of variances

Item	DF	Sum of Squares	Mean Squares	F Computed	F $\alpha = 0.05$	F $\alpha = 0.25$
PI_i	4	190.811	47.702	0.895	5.19	1.89
$Soil(PI)_{(i)j}$	5	266.404	53.281	4,440.01	2.21	1.33
Dev_k	4	8.825	2.208	185.61	2.37	1.35
$PI * Dev_{ik}$	16	7.111	0.442	4.89	2.19	1.37
$Soil(PI) * Dev_{(i)jk}$	20	1.888	0.089	7.58	1.57	1.19
Rep_l	4	0.041	0.010	0.84	2.37	1.35
$PI * Rep_{il}$	16	0.134	0.008	0.57	2.19	1.37
$Soil (PI) * Rep_{(i)jl}$	20	0.288	0.014	1.18	1.57	1.19
$Dev * Rep_{kl}$	16	0.226	0.014	1.18	1.65	1.22
$PI * Dev * Rep_{ikl}$	64	0.607	0.010	0.70	1.45	1.17
$Soil (PI) * Dev * Rep_{(i)kl}$	80	1.136	0.014	1.195	1.28	1.11
$Error_{(ijkl)m}$	1,000	11.880	0.012	-	-	-

The effect of the soil type is reflected in the "F" value of the Soil(PI) factor. Its "F" value of 4,440 indicates that the soil type significantly affects the resilient modulus. This was expected, since the soil samples were prepared differently and tested at different times.

The effect of the deviator stress is reflected in the "F" value of the Dev factor. Its "F" value of 185.61 indicates that there is, as expected, a clear and significant effect of the deviator stress level on the resilient modulus. This analysis, however, does not allow us to say anything about which deviator stress has the greatest effect on the modulus, though intuitively it appears that the highest stress has the greatest effect. In any case, the Newman-Keuls test could have been used for this particular purpose.

The effect of the number of stress repetitions is reflected in the "F" value of the Rep factor. Its "F" value of 0.84 indicates that the number of stress repetitions does not have a significant effect on the resilient modulus at any deviator stress level used. This is a very important finding, one that can be used to modify the M_R testing procedures. Thus, there is no need for 200, 100, 50, or even 25 applications of the deviator stresses in computing the M_R values; rather, our tests indicate that a reliable resilient modulus can be estimated from only the first five loading cycles.

Nevertheless, in looking at the actual testing operation, its practicality, and at the steps required in performing the test, we recommend using 25 stress repetitions would not result in a significant increase in machine and operator time costs. Additionally, it is interesting to note that all the higher factor interactions that include the number of repetitions present low "F" values. This indicates that, in general, there is not a significant effect of such interaction on the resilient moduli of the test samples. Similar conclusions are found in the analysis of the effect of the other high factor interactions.

SUMMARY

This section discussed an experimental evaluation of the effect of the number of stress repetitions on the resilient moduli of compacted samples. The number of stress repetitions has little effect on the resilient moduli of compacted soils, provided strong contacts between the test samples and end caps are obtained.

From a practical point of view, however, it should be recognized that during the testing operation an initial checking is required to establish the proper level of stress and/or strain to be applied. Therefore, it is estimated that 25 cycles are sufficient for accurate measurements of moduli. Moreover, 25 stress repetitions are sufficient to permit the strain values of the last 5 cycles to be recorded and averaged in the calculation of the M_R values at the different stress states of the test.

Since the testing sequence used in this experimental evaluation is similar to the conditioning stage specified by AASHTO T-274 for cohesive soils, these findings also demonstrate that the AASHTO T-274 conditioning stage has little effect on the moduli of compacted samples, provided, again, strong contacts are obtained.

Finally, it should be pointed out that, depending on the level of strain amplitude, the number of stress repetitions can be important, as explained in Chapter 2. Accordingly, this experiment was carried out on samples that experienced a wide range of elastic axial strains (0.01 percent to 0.20 percent) subjected to stress states commonly found in existing pavement layers. Consequently, the conclusions obtained from this investigation are also framed within the range of the operating strain amplitude, which is actually the workable strain range of the M_R test.

CHAPTER 9. EXPERIMENTAL COMPARISON OF RESILIENT MODULI OF SOILS OBTAINED BY DIFFERENT LABORATORY TESTS

This chapter documents the experimental comparisons made using various laboratory tests. Following a brief introduction, the objectives and the experimental approach of this comparison are presented. An explanation of the design of the experiment, the collection of the data, and the experimental observations are also provided. Finally, the chapter presents the determination of the elastic thresholds of the test materials, followed by a summary of this experimental comparison.

INTRODUCTION

Laboratory measurements of the deformational characteristics of subgrade materials can be quite complex, owing to the smallness of the strains that are typically involved in such pavement components. Moreover, experience has shown that extreme care must be exercised when evaluating the deformational characteristics of soils, particularly at small to intermediate strain levels (e.g., 0.001 to 0.1 percent).

Other popular techniques used to measure the dynamic properties of soils in this strain range include the torsional resonant column test and the torsional shear test—herein referred to collectively as torsional testing techniques.

Because torsional testing techniques differ from the resilient modulus test in the way they characterize materials, certain assumptions were made to make this experimental comparison possible. For example, it was assumed that the test material is homogeneous, isotropic, and behaves elastically across the range of strain amplitudes. Such assumptions, frequently used in geotechnical engineering, were felt to be equally applicable in the comparison of the results of this study.

It is therefore our conviction that this experimental comparison is important, particularly insofar as the M_R testing procedures developed herein can be validated and the guidelines and recommendations supported.

OBJECTIVES AND EXPERIMENTAL APPROACH

The objective of this chapter is to compare the results obtained under the resilient modulus (M_R) tests, the torsional resonant column (RC) tests, and the torsional shear (TS) tests, with such a comparison seeking to validate the M_R testing procedures to be proposed in this study. In addition, this chapter presents a rational approach that focuses on the characterization of materials, in the sense that the complete dynamic behavior of the material expressed in terms of modulus versus strain amplitudes is determined by the overlapping of results obtained from the three laboratory tests. The importance of this is such that the axial-strain-elastic thresholds of the test materials can also be defined.

The experimental approach of this study covers the testing procedures, the preparation of the test specimens, and the process involved in comparing the test results obtained by the three laboratory tests. At the time this experiment was programmed, we decided that our prototype testing procedure, as detailed in Chapter 4, would be the procedure used in performing all the M_R tests. This meant that the test samples used in this technique were subjected first to a conditioning stress and then to a sequence of different stress states. Most of the test data obtained under the M_R test are included in Appendix C.

The basic principles of the torsional testing techniques, RC/TS, have been extensively documented over the years. For instance, in *Vibrations of Soils and Foundations*, authors Richart, Woods, and Hall (Ref 44) document the basic principles and applications of these types of tests. Appendix A includes a brief summary of their basic principles, some characteristics of the equipment used, the procedures involved in performing these types of tests, and the computational process followed in relating the cyclic triaxial and resonant column results.

As previously mentioned in the discussion of M_R testing with synthetic samples (Chapter 3), good agreement was found between the moduli of the synthetic samples determined by both types of equipment. The synthetic specimens were easy to work with and test because their properties remained constant with time, were independent of strain amplitude and confining pressure, and could be repeatedly tested. In contrast, the comparison of the testing results for actual subgrade soils was far more complicated.

In the comparison (for which it was assumed that the material is homogeneous and isotropic), the shear moduli (G) obtained under the RC/TS tests were converted to equivalent Young's moduli (E)—called in this case *resilient moduli*— as follows:

$$E = 2(1+\nu)G \qquad (9.1)$$

Similarly, the axial strain (ε_a) compatible with the shearing strain (γ) of the RC/TS tests was estimated as follows:

$$\varepsilon_a = \frac{\gamma}{(1+\nu)} \qquad (9.2)$$

where

ε_a = the axial strain amplitude,
ν = the Poisson's ratio which was assumed to be 0.45 in all cases,
E = the Young's modulus, and
G = the shear modulus.

In addition, because all three laboratory tests operate at different frequencies, their results had to be adjusted to one particular excitation frequency in order to make a proper comparison. Therefore, we decided to adjust the modulus values of the RC/TS tests to an excitation frequency of 10 Hz, since this is the frequency established in M_R measurements.

But because of the availability constraints of the RC/TS testing apparatus, it was not possible to perform as many tests as we would have liked. It was for this reason that RC/TS tests were performed using only a 6-psi confining pressure. In addition, because the apparatus lacked sufficient power to subject stiffer samples to higher strain amplitudes, few TS tests were perfomed. Consequently, the comparison of results between the M_R test and the torsional testing techniques sometimes includes the three modulus results, while on other occasions the comparison includes only the results obtained by the M_R and the RC tests.

In comparing the resilient modulus values obtained from different testing equipment, it was necessary to prepare two specimens with identical characteristics (i.e., similar moisture content, compacting effort, and density). Both specimens were tested simultaneously, one with the M_R testing equipment, and the other with the RC/TS testing apparatus. These specimens with identical characteristics are also referred to in this report as companion specimens.

These companion specimens were prepared following each of the steps detailed in Chapter 5, "Materials and Preparation." It should be emphasized that in both the M_R test and in the RC/TS tests the test specimens were grouted to the end caps to assure strong contacts and to eliminate any slippage (a chronic problem encountered in RC/TS tests performed at low confining pressures).

DESIGN OF THE EXPERIMENT

Because the selection of the soil types, along with their particular characteristics, needed to be defined for this experimental comparison, we again applied the design-of-experiments concepts.

This particular experiment uses the same arrangement of soils shown in Table 5.2, in which three different soils were grouped into each of the five PI groups. To reduce the amount of testing, we decided to select at random only two of the three different soils available in each of the PI groups; thus only ten different soils were to be used in this experiment. Table 9.1 presents the design of this experiment.

To extend our inferences, we decided that the companion specimens would be prepared under randomly chosen moisture conditions and tested at randomly chosen sample ages.

Finally, our experiment involved testing one companion specimen under the M_R test, while the other was tested under the RC/TS test (to compare the results without concern for time effects).

Since the primary purpose of this chapter is to present the experimental comparisons, neither the model nor a statistical analysis was programmed for this study.

COLLECTION OF THE DATA

Ten pairs of soil samples were prepared and trimmed according to the sample preparation section in Chapter 5. Each pair of soil samples corresponded to the companion specimens as previously referred to. Companion specimens were prepared from soils 5 and 10 of PI group 0-10, from soils 6 and 7 of PI group 11-20, from soils 2

Table 9.1 Design of the experiment

PI Group	0 - 10		11 - 20		21 - 30		31 - 40		> 40	
Soil ID	5	10	6	7	2	16	9	13	12	15
Testing System										
MR	●	●	●	●	●	●	●	●	●	●
RC/TS	●	●	●	●	●	●	●	●	●	●

and 16 of PI group 21-30, from soils 9 and 13 of PI group 31-40, and from soils 12 and 15 of PI group >40.

The basic characteristics of these companion samples were recorded. Unfortunately, the list of characteristics of samples tested by the RC/TS technique was lost. For that reason, only the list of characteristics of samples tested by the M_R approach is herein reported and included in Table 9.2.

Table 9.2 Basic characteristics of the test samples

PI Group	Soil ID	Moisture Content (%)	Dry Density (pcf)	Sample Age (days)
0 - 10	5	10.40	120.74	99
	10	14.00	118.20	36
11 - 20	6	11.80	117.30	2
	7	20.10	104.40	6
21 - 30	2	39.80	77.90	6
	16	20.10	106.89	64
31 - 40	9	25.30	97.15	6
	13	18.00	100.20	6
> 40	12	20.60	85.60	2
	15	20.70	105.90	6

The experimental comparison collected from the testing of these ten companion specimens is illustrated in Figures 9.1 through 9.5, all of which present the variation of the resilient modulus to the axial strain amplitude determined by the different testing methods.

EXPERIMENTAL OBSERVATIONS

Figure 9.1 presents the test results of soils having a very low plasticity index (PI group 0-10), with soil 5 in (a) and soil 10 in (b). Figure 9.2 presents the test comparison corresponding to soils of low plasticity index (PI group 11-20), with soil 6 in (a) and soil 7 in (b). Figure 9.3 shows the test results of soils with intermediate plasticity index (PI group 21-30), with soil 2 in (a) and soil 16 in (b). Figure 9.4 shows the test results of soils with high plasticity index (PI group 31-40), with soil 9 in (a) and soil 13 in (b). And finally, Figure 9.5 presents the test comparison corresponding to soils of very high plasticity index (PI group >40), with soil 12 in (a) and soil 15 in (b).

It is interesting to note that, in general, all the M_R testing results fall into a range of small to intermediate resilient axial strain amplitudes (0.01 to 0.1 percent), while the experiment performed using the RC/TS test falls into a much wider range of very small to intermediate strain amplitudes (0.001 to 0.1 percent). This is because the M_R test is set up as a stress-controlled system, while the RC/TS is operated as a strain-controlled system.

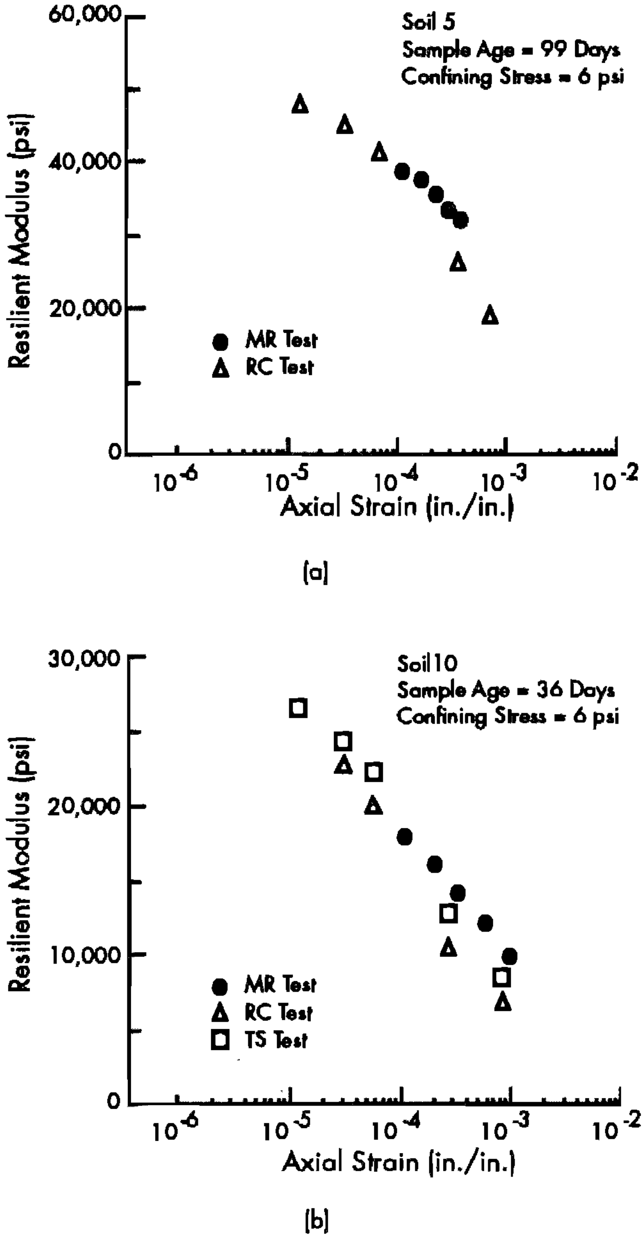


Figure 9.1 Variation in resilient modulus with axial strain amplitude under a 6-psi confining stress. Shown are the testing results of companion samples of (a) soil 5 at 99 days and (b) soil 10 at 36 days after compaction

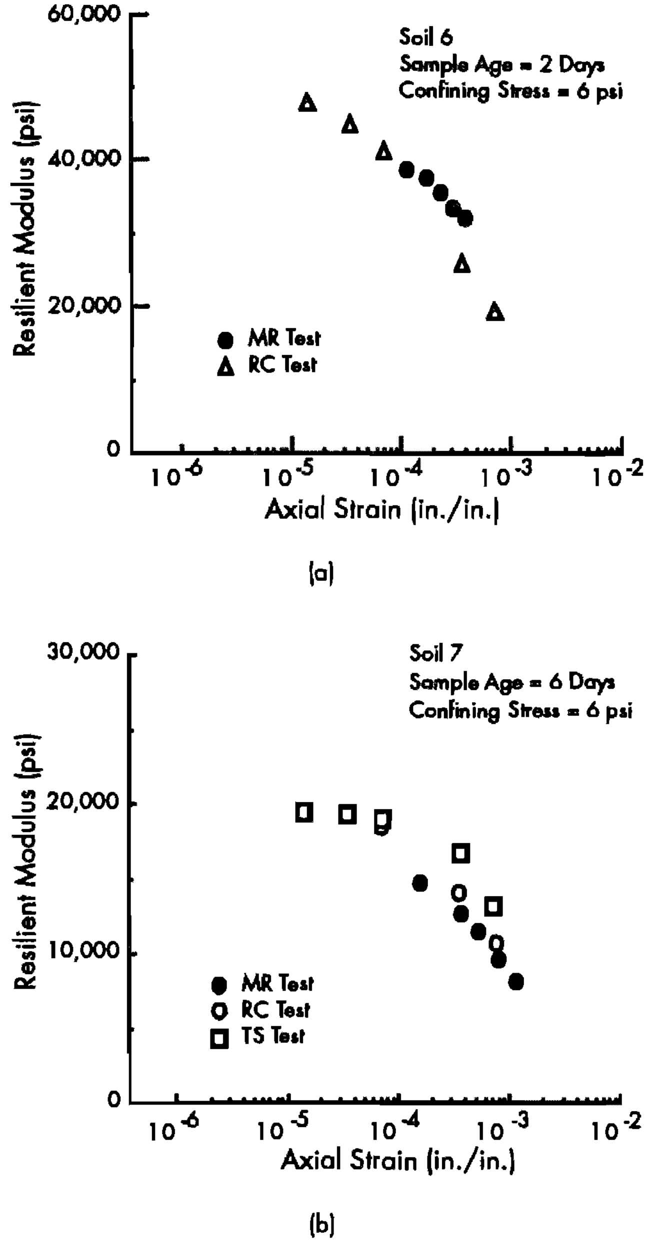


Figure 9.2 Variation in resilient modulus with axial strain amplitude under a 6-psi confining stress. Shown are the testing results of companion samples of (a) soil 6 at 2 days and (b) soil 7 at 6 days after compaction

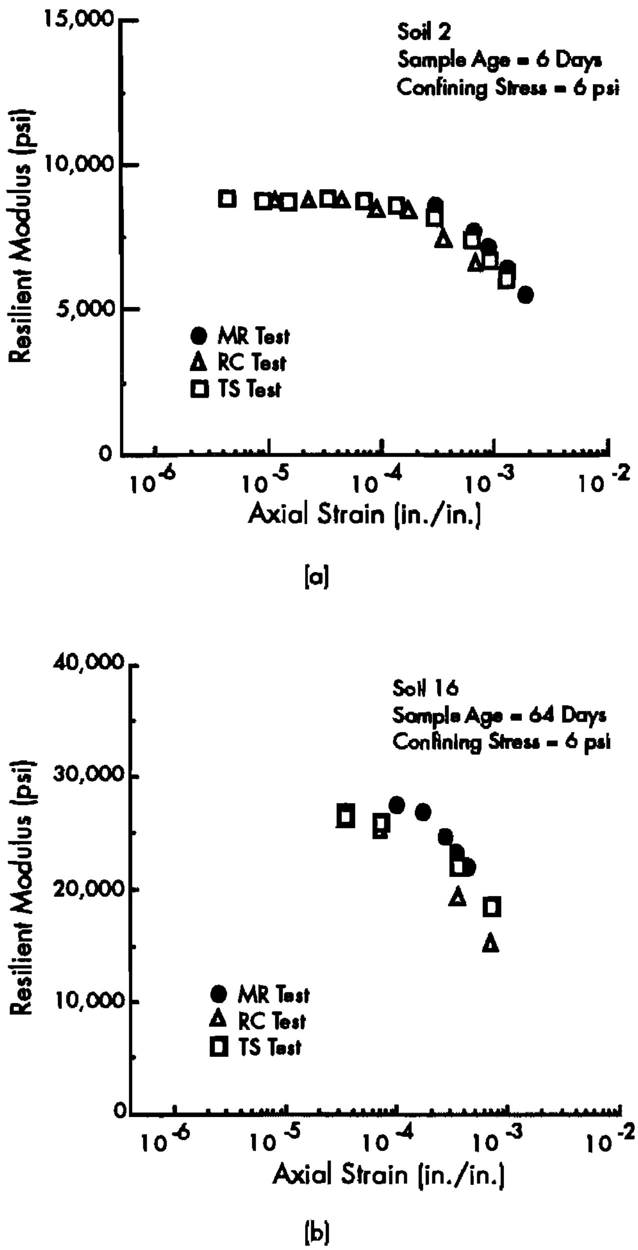


Figure 9.3 Variation in resilient modulus with axial strain amplitude under a 6-psi confining stress. Shown are the testing results of companion samples of (a) soil 2 at 6 days and (b) soil 16 at 64 days after compaction

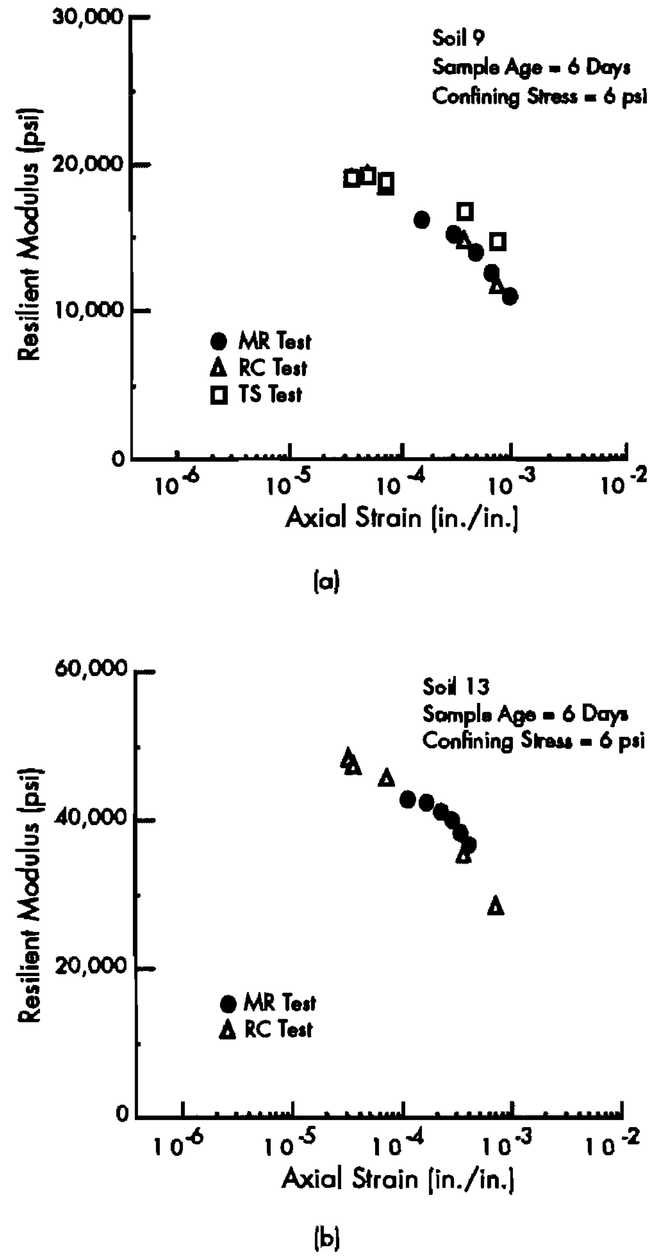


Figure 9.4 Variation in resilient modulus with axial strain amplitude under a 6-psi confining stress. Shown are the testing results of companion samples of (a) soil 9 at 6 days and (b) soil 13 at 6 days after compaction

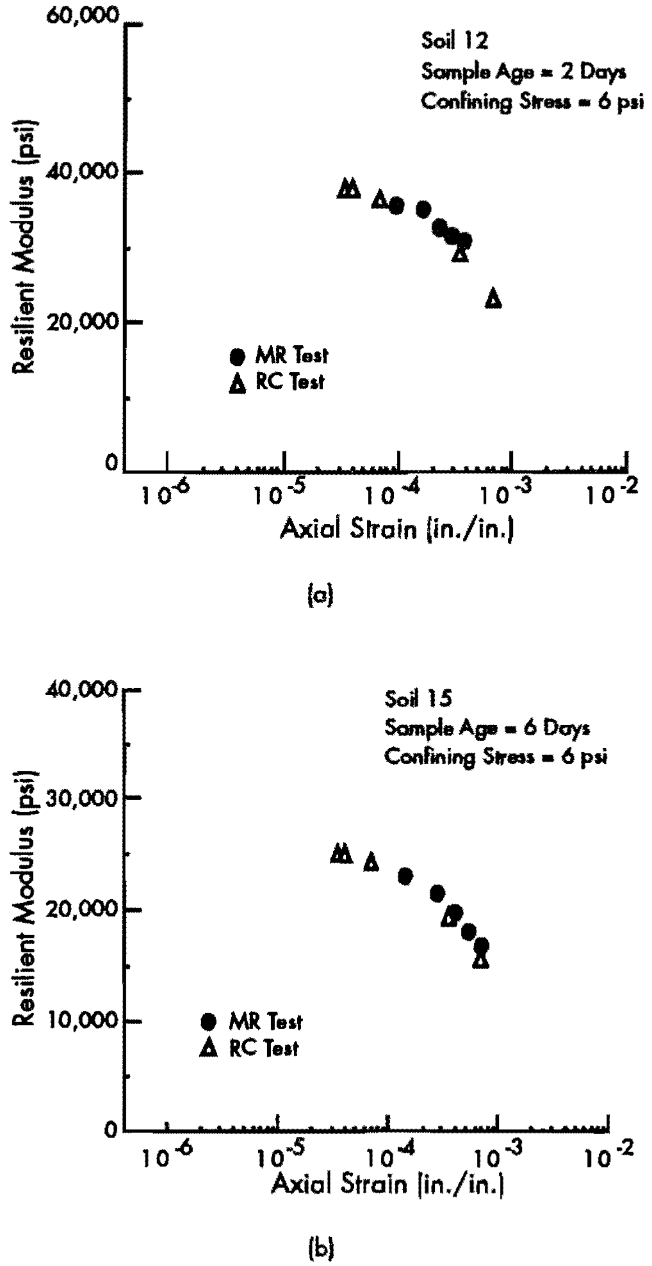


Figure 9.5 Variation in resilient modulus with axial strain amplitude under a 6-psi confining stress. Shown are the testing results of companion samples of (a) soil 12 at 2 days and (b) soil 15 at 6 days after compaction

The fact that the M_R test is based on magnitudes of stress applications rather than on induced resilient axial strain measurements means that only part of the stress-strain behavior of the material can be determined under this type of test.

Nevertheless, it seems that if the M_R testing method were a strain-controlled test, the test could not simulate properly the actual field conditions. Thus, to define the complete deformational characteristics of the material, the measurements would have to be taken under stresses that induce resilient axial strain amplitudes that cannot be recorded accurately ($\varepsilon_a < 0.01$ percent). Accordingly, it is our recommendation that the M_R testing procedure remain a stress-controlled test, and that any modulus value obtained from axial strain measurements lower than 0.01 percent be ignored.

Figures 9.1 through 9.5 show an encouraging overlap of the moduli for all the companion samples tested under both the M_R method and the RC/TS techniques. Based on this type of comparison, we felt that there was sufficient evidence to conclude that a reliable resilient modulus system for measuring the elastic properties of subgrade materials had been developed.

The key elements for such encouraging overlaps of moduli rely on the facts that: (1) the companion specimens were grouted to the end platens in the M_R equipment and in the RC/TS apparatus; (2) extreme care was taken during the preparation and handling of the companion specimens; and (3) M_R tests and RC/TS tests on companion specimens were performed simultaneously.

Although Figures 9.1 through 9.5 present the same type of information, each is very useful in that they permit estimations of the axial-strain-elastic threshold, which can also be related to basic properties of the soils tested.

DETERMINATION OF THE ELASTIC THRESHOLDS

The axial-strain-elastic threshold, as explained in Chapter 2, defines the limit at which the material passes from a linear elastic behavior to a non-linear elastic one. In other words, this threshold is the point at which the modulus of a material changes from a non-strain (nor stress) dependent to a strain (or stress) dependent.

Because good agreement was found between the moduli of the compacted soils with M_R tests and RC/TS tests, the complete stress-strain behaviors of each of the samples tested were defined (as illustrated in Figures 9.1 through 9.5), with the axial-strain-elastic thresholds of each of the test samples then estimated.

Table 9.3 includes the PI group in which the tested soils are nested, the soil identification numbers with their plasticity index values, and the axial-strain-elastic thresholds estimated from Figures 9.1 through 9.5.

Using these data, we attempted to observe trends of the axial-strain-elastic thresholds in terms of soil properties through a correlation analysis. This analysis was performed with the following factors: (1) axial-strain-elastic threshold, ε_{aet}; (2) plasticity index, PI; (3) the moisture content, ω; (4) sample age, η; and (5) dry density, γ_d. Based on the results obtained from this analysis, we found that the PI factor was highly correlated with the ε_{aet}, while the other factors presented very poor correlations. The implications of these results are that neither variations in the sample's moisture content and soil density nor its increase of age may influence the position of the ε_{aet}; the plasticity index is the only factor that appears to influence significantly the position of the ε_{aet} of the soil samples.

Table 9.3 Axial-strain-elastic thresholds

PI Group	Soil ID	PI (%)	Axial Strain Elastic Threshold (%)
0- 10	5	10	0.0011
	10	4	0.0008
11 - 20	6	15	0.0014
	7	20	0.0020
21 - 30	2	27	0.0030
	16	29	0.0034
31 - 40	9	34	0.0048
	13	36	0.0031
> 40	12	52	0.0048
	15	40	0.0043

Accordingly, a regression model was then developed to estimate this fundamental parameter based on the plasticity index of the soil. The regression analysis, performed using SAS in a personal computer, had (once transformed) the following output:

$$\begin{aligned} \varepsilon_{aet} &= e^{-8.45} * PI^{0.79} \\ SEE &= 0.0006 \\ R^2 &= 0.916 \end{aligned} \tag{9.3}$$

where

ε_{aet} = the predicted axial-strain-elastic threshold, in percent,

PI = the plasticity index of the soil, in percent,

SEE = the standard error of the estimate of the model, and

R^2 = the coefficient of determination.

With a high coefficient of determination value, this regression model appears to be statistically sound. Thus, its use in the analysis of pavement materials is recommended.

SUMMARY

This chapter presented an experimental comparison of the results obtained with resilient modulus (M_R) tests, torsional resonant column (RC) tests, and torsional shear (TS) tests.

Ten different soils were used in the preparation of ten pairs of test samples. Each pair of samples having identical characteristics were tested at the same time, one with the M_R equipment and the other with the RC/TS testing apparatus.

In making that comparison, moduli obtained under the resonant column and torsional shear tests were converted to the equivalent resilient moduli by assuming that the materials were homogeneous, isotropic, and had a Poisson's ratio of 0.45. In addition, the moduli were further adjusted to a frequency of 10 Hz, which is the frequency of the M_R test.

In general, all the results show extraordinary overlaps of the moduli obtained with the different testing techniques. Based on this type of comparison, we believe strongly that there is sufficient evidence to suggest that our M_R testing system is very reliable in measuring the deformational characteristics of the compacted soils within the range of small to intermediate strain amplitudes (0.01 to 0.1 percent).

The key elements for such comparisons rely on the facts that: (1) the test specimens were grouted to the end platens in the M_R equipment and in the RC/TS apparatus; (2) extreme care was taken in the preparation and handling of the test specimens; and (3) the tests were performed simultaneously in order to exclude time effects in the results.

Because these comparisons presented the complete stress-strain behavior of the test samples, it was possible to define their axial-strain-elastic thresholds. This fundamental parameter was found to be predominantly related to the plasticity index of the soils tested. In general, it can be expected that soils with high PI's will also have higher ε_{aet} values. Consequently, a regression equation was developed aimed at predicting the position of this elastic threshold.

Nevertheless, it should also be recognized that because this empirical model was developed from tests performed at a 6-psi confining stress only, the applicability of this model is therefore limited to that confining stress.

CHAPTER 10. COMPARISON OF LABORATORY AND FIELD MEASUREMENTS

This chapter describes a case study in which the moduli of several laboratory and field measurements were compared. The objectives and the experimental approach are discussed, along with the actual data collection and comparisons.

INTRODUCTION

Several reports have documented comparisons of the theoretical and experimental responses of pavements subjected to traffic loading (Refs 21, 43). In one such report, Dehlen (Ref 21) compared strains and deflections measured on sections of a San Diego test road with analytical computed values. Field measurements were performed under normal passing traffic using LVDT's and a Benkelman beam. For his theoretical analyses, Dehlen first collected soil samples for laboratory testing in order to assess the stiffness characteristics of the pavement components; the values obtained were then used for linear and non-linear analyses of the pavement structure. By comparing the results obtained from the linear and non-linear analyses with the field measurements, he found that his analyses predicted higher deflections than the deflections recorded in the field. Figure 10.1 shows the comparison between the analytical and measured vertical deflections presented by Dehlen (Ref 21).

In his attempt to explain the large discrepancy, Dehlen suggested that there were: (1) non-uniform normal stresses imposed by the tire in the field tests; (2) anisotropy effects not considered in his theoretical analyses; or (3) some effects related to the method of measurement in the field tests, e.g., disturbances near the holes where the transducers were located. Whatever the reason, he recognized that none of these hypotheses could have been verified from the data collected, and that such a difference had to remain unexplained.

From the experience gained in the present study, it appears that Dehlen's discrepancy may have been caused by erroneous modulus assessments of each of the pavement components tested in the laboratory; that is, the moduli obtained by Dehlen in the laboratory could have been underestimated because of disturbances to the samples or because of a lack of testing system compliance.

In contrast to Dehlen's approach, this study compared only the moduli obtained by laboratory tests with those obtained by field tests (since

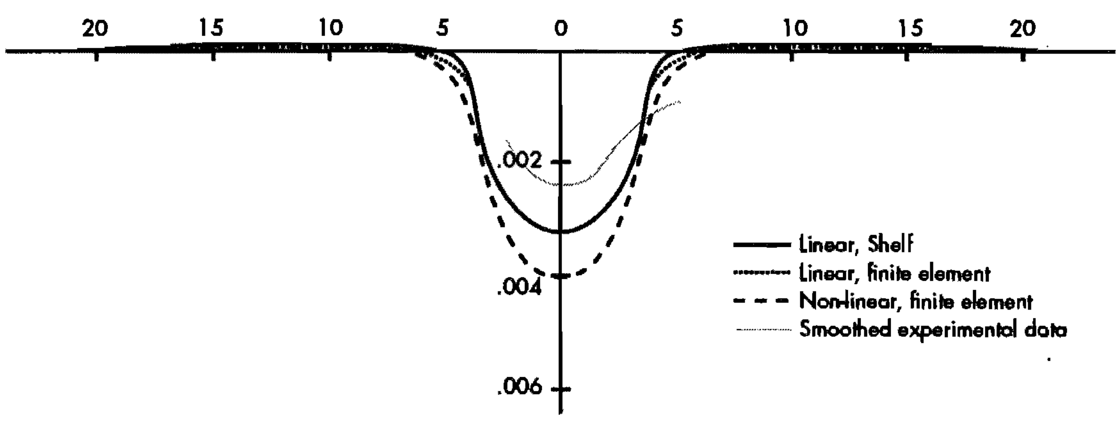


Figure 10.1 Theoretical and measured vertical compression within asphalt concrete layers of the San Diego road test reported by Dehlen (Ref 21)

comparing actual pavement responses to analytical predictions of such responses constitutes in itself a large and complicated study).

On October 30, 1990, our field team took measurements on FM-971, located 5 miles from Granger, Texas (about 60 miles northeast of Austin). Because the site had been periodically monitored for variation of the moduli of the pavement layers at this location for more than 13 years, it was considered an ideal site.

OBJECTIVES AND EXPERIMENTAL APPROACH

The objective of this exercise was to test the validity of the laboratory M_R test by comparing—through both field and laboratory tests—the modulus of a subgrade and subbase layer of an in-service road. For the field measurements, we decided to use the crosshole method (Ref 53), since that method tests the material using a very small strain amplitude, and because only one modulus value (generally the maximum one) is estimated.

Laboratory tests, characterized by larger strain amplitudes, were performed to determine the nonlinear behavior of the material. Additionally, in comparing the moduli, we took care to consider the proper strain amplitudes and confining-pressure levels.

CROSSHOLE MEASUREMENTS

The crosshole method, ASTM Designation 4428M-84, was used to measure the time required for compression and shear waves to travel between several points located at similar depths from the surface within a soil mass. Once the travel times were determined, wave velocities were calculated.

Next, two boreholes, one for the source and one for the receiver, were constructed and spaced about 7.38 feet apart on the surface of the road. Soil samples at several profile depths were taken from the boreholes for laboratory testing using 3-inch-diameter shelby tubes. Field testing began once the equipment setup was installed and the transducers were placed in their proper orientation. Compression and shear waves in the soil mass were generated by a hand-held hammer that was used to strike the source system placed inside the source borehole. Measurements for a given depth were taken and travel paths determined down to a depth of 20.3 feet.

The source system consisted of steel rods connected to one another and to an end element. The number of steel rods used depended on the profile depth of measurements; the type of end element used depended on the wave type selected for measurements.

A solid steel rod with a diameter of 3 inches and a height of 6 inches was the end element used in the source system to generate compression waves that could be clearly defined by the recording system; in addition, a shelby tube having dimensions similar to those of the solid-steel rod was the end element used to generate shear waves. A layout of the soil profile and the testing configuration is presented in Figure 10.2 (the soil profile is taken from Stokoe; see Ref 53).

Figure 10.3 shows a typical record of travel times collected from the vertical velocity transducer for the initial wave arrivals of the shear wave and the compression wave. Only direct travel times, t_d, were recorded, since only one receiver was used. Each direct travel time represents the time elapsed between the triggering and the arrival at the receiver in the borehole of either the shear wave or the compression wave. In addition, the striking time was monitored (time zero) by a transducer that sent signals to an analyzer, as shown in Figure 10.2.

Total travel times, t, for the S-waves and P-waves at each measurement depth were determined using information similar to that presented in Figure 10.3. Proper calibration factors and plot scales were considered to determine such travel times. The total travel times recorded are associated with total travel distances that include the length of the steel rod at each depth of measurements (S_r) plus the travel distance into the soil media (S_s) measured from the end element of that steel rod to the position of the receiver located inside the receiver borehole.

Furthermore, because of the inclinations of the boreholes, travel distances into the soil media (S_s) had to be corrected using simple principles of geometry. In fact, that travel distance turned out to be different at each measurement depth; thus, it was not equal to the distance measured at the surface.

The time traveled by the waves through the source system and the soil media had to be accurately defined. Since the compressional wave velocity of the steel rod (V_c) was known to have a value of about 16,400 ft/sec, the time traveled by the waves in the rod (t_r) was therefore determined by using the equation:

$$t_r = S_r / V_c \qquad (10.1)$$

In this way, the velocities of either the compression waves or shear waves at each profile depth

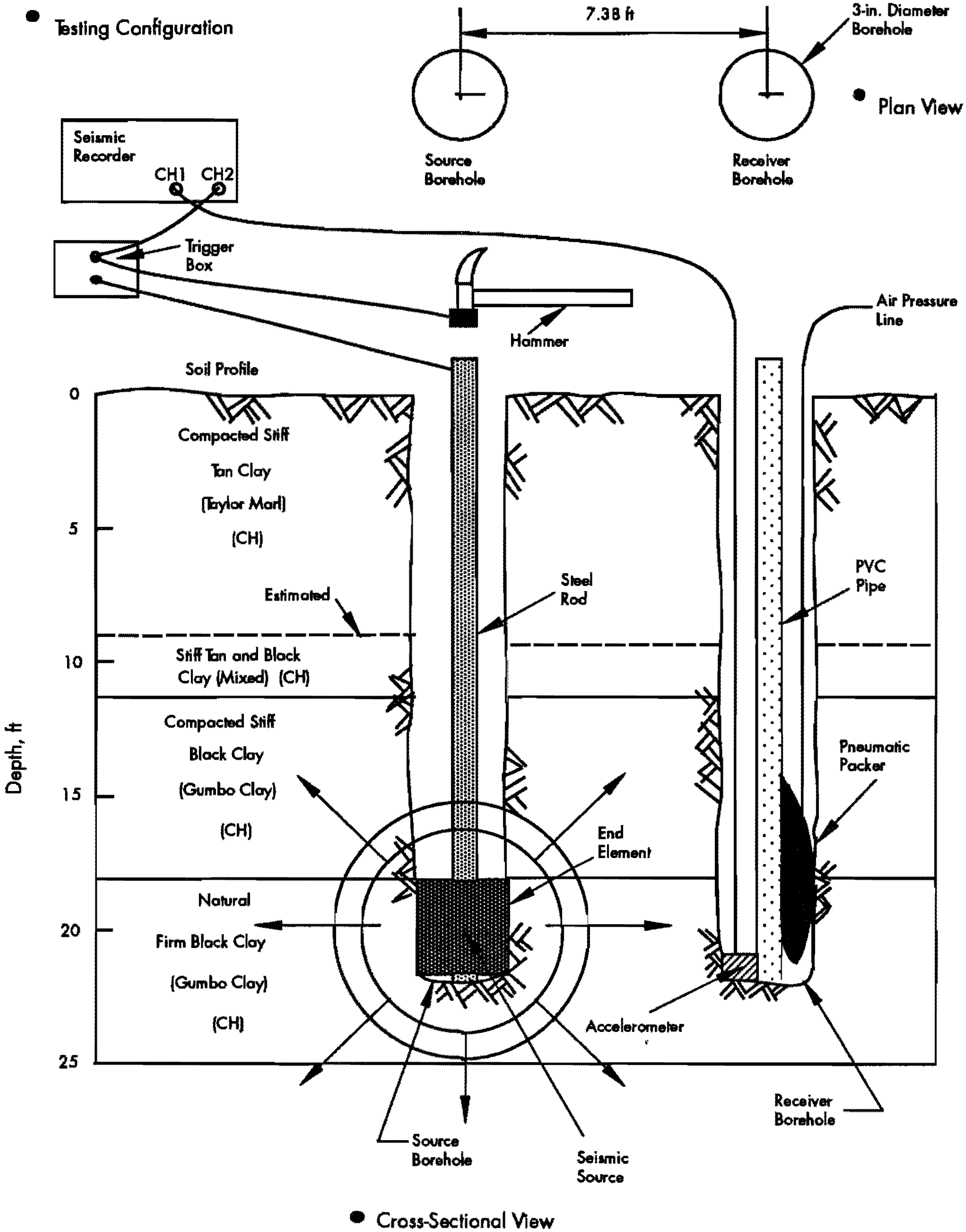


Figure 10.2 Soil profile and testing configuration of the Granger site

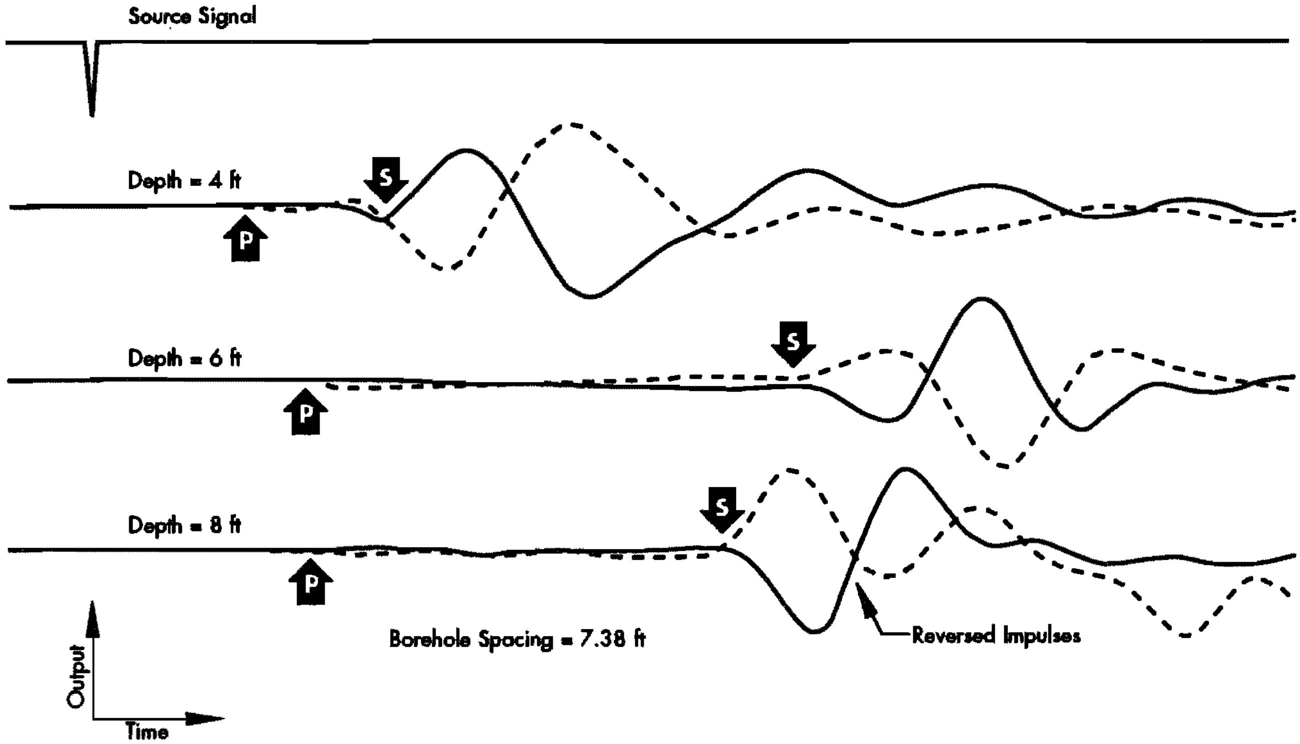


Figure 10.3 Typical travel time record of the S-wave and the P-wave

of measurements were determined by using the following equation:

$$V = S_s / (t - t_r) \qquad (10.2)$$

Additionally, Poisson's ratio (v) at each measurement depth was also determined by applying the following equation (assuming that the materials are isotropic):

$$v = 0.5 * \frac{1 - V_s^2}{V_p^2 - V_s^2} \qquad (10.3)$$

Figure 10.4 shows the variation of the velocities of the compression and shear waves along the profile depth. As shown in that figure, the compression wave had lower values than 5,000 ft/sec, which indicates that the soil profile measured was partially saturated, as explained by Stokoe (Ref 53). Moreover, it is interesting to note that the band of shear wave velocities, ranging from 400 to 600 ft/sec, was narrower than the band of compressional wave velocities.

LABORATORY MEASUREMENTS

Field work concluded with the collection of the soil samples from the two boreholes inside the shelby tubes; once brought to the surface, these tubes were sealed with wax to prevent any moisture loss in the soil samples. Samples were then transported to the laboratory, where they were extruded from the shelby tubes. Unfortunately, some soils crumbled, losing their consistency and shape. This occurred especially in the samples obtained from the upper layers of the pavement (i.e., the granular base). No attempt was made to reconstruct this sample in the laboratory—its granulometry would have required samples too large to be tested in our M_R testing system.

The samples that withstood extrusion with the fewest problems were the clayey specimens obtained from depths of 7 and 12 feet. These robust samples were coded in this project as soil 14. The 7-foot sample was a compacted stiff clay having a moisture content of 30 percent, a dry density of 93.2 pcf, a total unit weight of 121.2 pcf, a liquid limit of 66 percent, and a plasticity index of 43 percent.

The 12-foot sample was also a compacted clay having a moisture content of 23.1 percent, a dry density of 98.3 percent, a total unit weight of 121 pcf, a liquid limit of 66.7 percent, and a plasticity index of 43.6 percent. Then, the *in situ* confining pressure was considered so as to reproduce the field conditions in the laboratory. The *in situ* confining pressure is routinely estimated as follows:

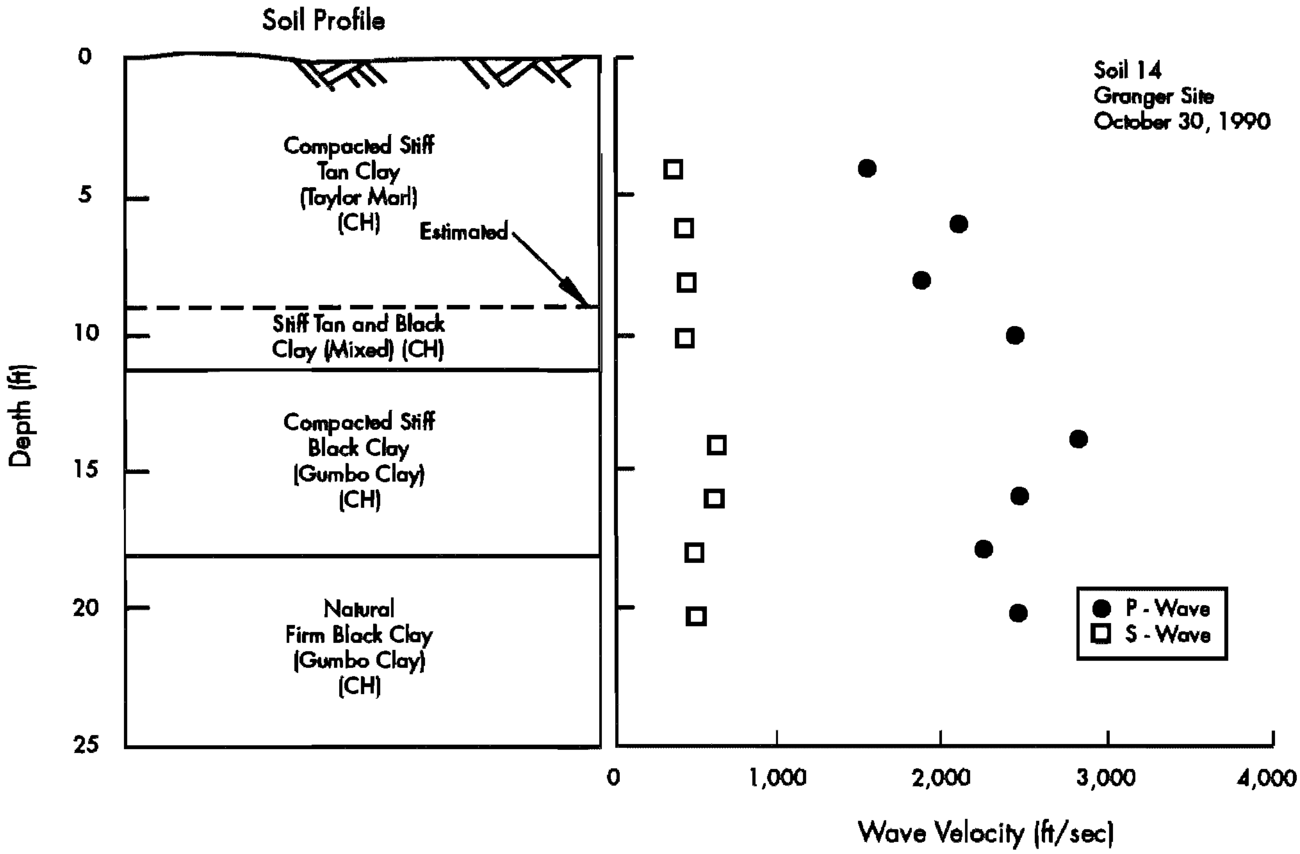


Figure 10.4 Variation of the P-wave and S-wave velocities along the soil profile

$$\sigma_c = \frac{\sigma_1 * [1 + 2 * K_o]}{3} \tag{10.4}$$

where

σ_c = the confining pressure,
σ_1 = the total vertical stress at the depth of measurements, and
K_o = the coefficient of earth pressure at rest.

By assuming that the material had an isotropic confining pressure, $K_o = 1$, the confining pressure was then estimated as the overburden stress at the measurement depth. In this way, it was estimated that the 7-foot sample had a confining pressure of about 6 psi, and the 12-foot sample had a confining pressure of about 10 psi. Although it is recognized that confining pressure time affects the modulus of soils, as demonstrated by Anderson et al (Ref 18), this factor was not considered in this study because Anderson's results showed that the effect is significant only over long periods of time and for confining pressures higher than 10 psi.

Prior to laboratory testing, the two samples were grouted to the end platens to assure strong contacts. The M_R tests were then performed by subjecting the samples to their corresponding *in situ* confining stress and by applying deviator stresses of 2, 4, 6, 8, 10, and 12 psi 100 times. The testing results of the two samples are included in Appendix C.

COMPARISONS OF LABORATORY WITH FIELD MEASUREMENTS

While water in the soil mass has little effect on the shear wave velocity, it can have a significant effect on the compressional wave velocity determined from first time arrivals, as explained by Stokoe (Ref 53). Consequently, only the S-waves measured in the field tests were used in this comparison.

Using Figure 10.4, we estimated the shear wave velocities, V_s, of the soil profile at 7 feet and 12 feet. With the total unit weight, γ_t, of the material, the shear modulus at each depth was determined using the following equation:

$$G = \frac{V_s^2 * \gamma_t}{g} \tag{10.5}$$

where

G = the shear modulus,
V_s = the shear wave velocity,
γ_t = the total unit weight, and
g = the acceleration of gravity.

Then, in the comparison of moduli, the material was assumed to be homogeneous and isotropic; as in Chapter 9, the shear modulus was converted to an equivalent Young's modulus, E, as follows:

$$E = 2(1+\nu)G \tag{10.6}$$

The value of the Poisson's ratio used, 0.46, was the one determined by the field tests (though the analysis is not sensitive to the Poisson's ratio). The shearing strains in field tests were determined using such figures as 10.3, in which the amplitude of the first shear wave arrival was measured and then related to the calibration factors of the transducers. Once the shearing strain, γ, was obtained, the axial strain, ε_a, was estimated as follows:

$$\varepsilon_a = \frac{\gamma}{1+\nu} \tag{10.7}$$

Although the M_R test works at the 10 Hz frequency and the crosshole method works at a variable frequency, we decided to omit consideration of the loading frequency in these comparisons, principally because this factor has little effect on the modulus of clayey soils, as explained by Stokoe (Ref 53).

Figures 10.5 and 10.6 compare the results obtained by field and laboratory tests of the soils located 7 feet and 12 feet below the surface of the Granger site, respectively. As shown in the two figures, the modulus values obtained by the laboratory tests are lower and within the 0.01 to 0.1 percent of axial strain amplitude, while the modulus values obtained by the field tests are higher and within strain amplitudes below 0.001 percent.

Using the regression equation developed in Chapter 9, and considering the PI of this soil, we estimated the value of its axial-strain-elastic threshold, ε_{aet}, to be 0.00417 percent. Defining this important factor helps to clarify this comparison, inasmuch as the results from the two approaches differ in strain range and in magnitude.

Figures 10.5 and 10.6 also show the trend, represented by dashed lines, of the modulus estimated from the M_R test. Each dashed line is intended to represent the non-linear stress-strain behavior of the material over the entire range of axial strain amplitudes. Accordingly, at strain amplitudes lower than the ε_{aet}, the dashed lines are horizontal lines, representing the linear elastic variation of the modulus; at strain amplitudes higher than the ε_{aet}, the dashed lines are curvilinear, representing the variation of the modulus and its dependency on the strain amplitudes.

Based on that observation, the maximum M_R that can be measured in the laboratory will be lower than the modulus measured in the field. This discrepancy may be caused by disturbances affecting the soil specimens during sampling, or even by the effect of confinement time, neither of which was considered in this comparison. Because the boreholes were only 3 inches in diameter (leaving barely enough material for trimming), it is possible that the soil specimens were disturbed.

The effect of time of confinement, though not considered here, is certainly an influencing factor. As reported by Anderson (Ref 18), the testing site embankment was built in 1977, making the pavement structure 13 years old at the time of our field testing. It is very likely that the drilling for, and the collection of, the soil samples destroyed the effect of 13 years of confining pressure. Thus, samples taken for laboratory testing were, in effect, 1-day-old samples.

Whatever the causes of such discrepancies, laboratory tests underestimate the modulus of existing pavement layers, as Dehlen concluded in 1969. There is therefore a need to (1) develop a laboratory technique that can take into account the time of confining pressure of the soil samples in the field, to compensate for their losses of stiffness caused by the sampling process; or (2) improve the sampling technique.

SUMMARY

This study compared the moduli of soils determined by laboratory and field testing methods. The laboratory testing method used was the M_R test; field testing employed the crosshole method. Because the objective of this exercise was to test the validity of our laboratory M_R test, it was interesting to note that the moduli obtained in field tests were generally higher than those obtained in laboratory tests. This was explained by the different strain amplitude levels at which field and laboratory tests operate.

Nonetheless, the M_R test demonstrated typical trends of the non-linear elastic behavior of soils at low to intermediate strain amplitudes (0.01 to 0.1 percent). Conversely, the crosshole method proved to be a useful technique for the *in situ* determination of the elastic properties of soils at very low strain amplitudes (below 0.001 percent).

Though the pattern of the variation of the modulus versus strain was clearly identified, some discrepancies were found in the comparison of moduli determined by the laboratory and field techniques. As this situation suggests, much work is still required in the effort to provide a correct estimation of the actual field conditions, specifically with respect to the process of sampling and preparing truly representative specimens for testing.

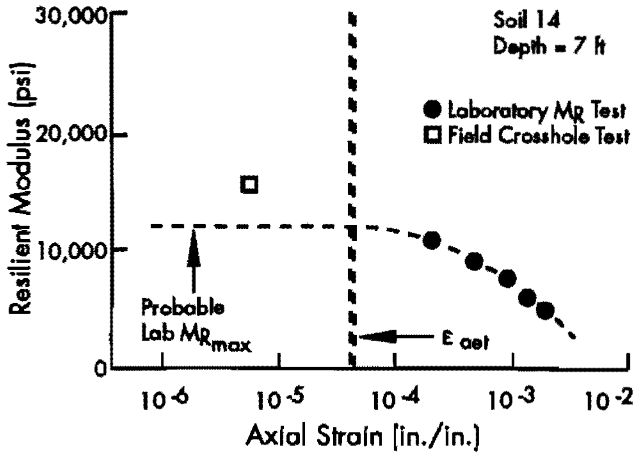


Figure 10.5 Comparison of the resilient modulus obtained by field and laboratory tests of soil 14 (from a depth of 7 feet)

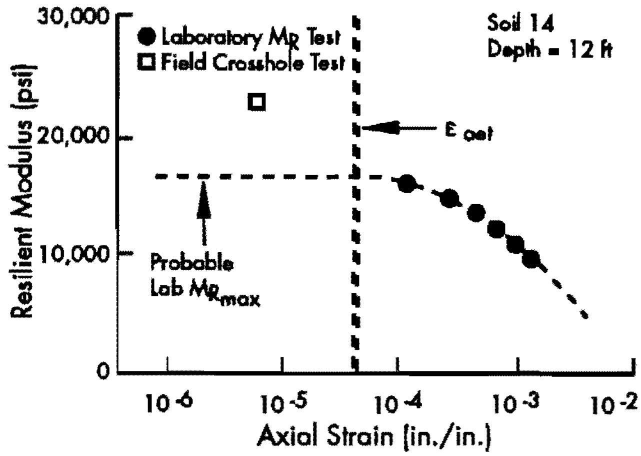


Figure 10.6 Comparison of the resilient modulus obtained by field and laboratory tests of soil 14 (from a depth of 12 feet)

CHAPTER 11. IMPORTANCE OF TESTING REPLICATE SAMPLES

INTRODUCTION

This study has investigated several aspects of the M_R soil test, including the effect of equipment compliances and sample setting in the triaxial cell, the effect of stress conditioning, and the effect of number of stress repetitions. In addition, we have demonstrated the benefits of comparing field with laboratory tests. This chapter discusses the results obtained in testing replicate samples used in the estimation of M_R values of compacted soils.

EXPERIMENTAL APPROACH

First, it must be understood that the process used to remold (reconstitute) in-laboratory soil samples representing field conditions influences to some degree the deformational characteristics of the compacted soils. For that reason, we decided in this investigation to use two different soils: soil 1, having a high PI, and soil 10, having a low PI. From each of these soils, three test samples were prepared at one time using the same method (Tex-113-E) so as to control other influencing factors of the moduli (e.g., age-hardening and moisture content). Thus, a total of six M_R tests were performed to assess the importance of testing replicate specimens.

Table 11.1 contains the basic properties of the three compacted specimens of soils 1 and 10, including their moisture content, density, dry density, and degree of saturation. As can be seen, their properties differ somewhat. In addition, it should be stated that for samples of soil 1, their densities were 12 percent lower than the maximum density reported by TxDOT, while for samples of soil 10, each was near the maximum value. All three samples of soil 1 were tested 8 days after compaction; samples of soil 10 were tested after 9 days of compaction.

All samples were grouted to the end caps of the triaxial cell prior to testing. The M_R test consisted of delivering 100 applications of 2-, 4-, 6-, 8-, and 10-psi deviator stress to a sample subjected to cell pressures of 6, 4, and 2 psi.

EXPERIMENTAL RESULTS

An M_R testing report was obtained from each of the M_R tests performed on each of the test specimens. Using this information, plots were developed to compare the testing results of the three samples of each of the soils used in this study.

Figure 11.1 presents the comparison obtained from the testing of the three companion samples of soil 1 in three plots corresponding to the different confining stresses used. It is interesting to note in this figure that the M_R values range from 16,000 to 10,000 psi within 10^{-4} to 10^{-3} of resilient axial strain range.

Table 11.1 Basic characteristics of the compacted samples

Soil ID	PI (%)	Sample No.	Moisture (%)	Density (pcf)	Dry Density (pcf)	Degree of Saturation (%)
1	55	1	18.83	100.78	84.81	53.8
		2	19.36	101.71	85.21	55.9
		3	x	103.20	86.65*	57.0
		pot	19.09			
10	4	1	10.92	138.75	125.09	94.6
		2	11.30	138.56	124.49	97.9
		3	11.13	138.84	124.93	96.4
		pot	11.40			

* Dry density was calculated using moisture content of the pot.

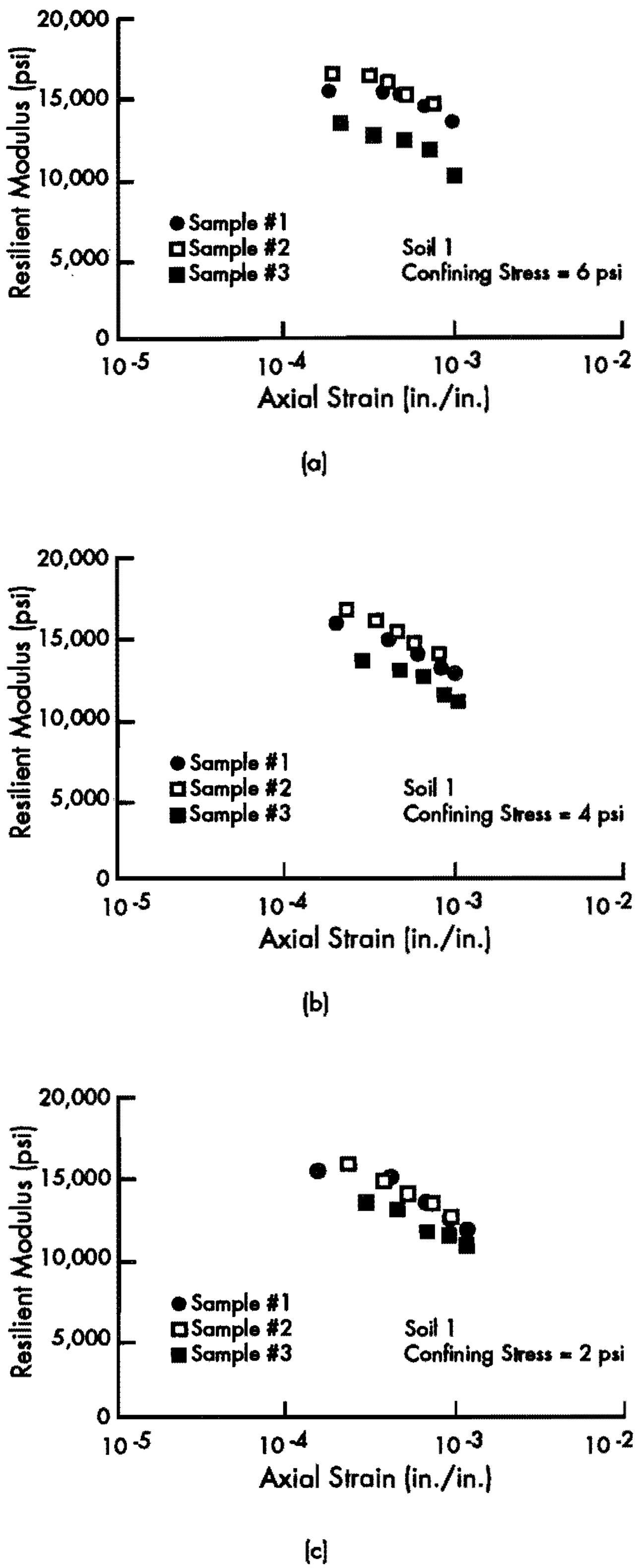


Figure 11.1 Comparison of results obtained from the testing at confining stresses of (a) 6 psi, (b) 4 psi, and (c) 2 psi of three companion samples of soil 1 tested 8 days after their compaction

In general, it can be observed that the M_R values corresponding to samples #1 and #2 were quite similar, but not those for sample #3. Sample #3, though having a higher density than the other two companion samples, had the highest degree of saturation among the three samples. Thus sample #3 demonstrated lower M_R values. In any event, it is estimated that the variations observed among the properties of the replicate samples may have been associated with variabilities inherent in the process used for preparing the test samples.

Figure 11.2 compares results obtained from the testing of the three companion samples of soil 10 (as does Figure 11.1). It can be noted in this case that the M_R values ranged more widely from 15,000 to 6,000 psi within the 10^{-4} to 10^{-3} strain range. Moreover, it is encouraging to note that the M_R values corresponding to the three test samples were quite similar at all levels of confining pressure and axial strain amplitude.

ANALYSIS OF RESULTS

A multi-linear regression analysis, using SAS on a personal computer, demonstrated the degree of variability within this data set. Because each of the six M_R test reports included fifteen different stress conditions, a total of ninety stress-strain states were merged into one data file for the subsequent statistical analysis. The induced axial strain, ε_a, was taken as the dependent variable, while the deviator, σ_d, and confining, σ_c, stresses were taken as the independent parameters. The other factor included in the model was the soil identification, S, which was used to differentiate the soil types. Consequently, the regression model had the following form:

$$\ln(\varepsilon_a) = a + b * \ln(\sigma_d) + c * \ln(\sigma_c) + d * S \qquad (11.1)$$

Because they are of no interest in this investigation, the regression coefficients (a, b, c, and d) are excluded here. Because the purpose of the coefficient of determination, R^2, is to indicate how well the model fits the test data, having a high R^2 ensures the effectiveness of the model. But the main parameter in this investigation is the standard error of the estimate, SEE. This is the case because SEE indicates the variability of the model and allows the development of confidence intervals of the measurements at a given significance level. Thus, the output of interest obtained from SAS was:

$$SEE = 0.139$$
$$R^2 = 0.958$$

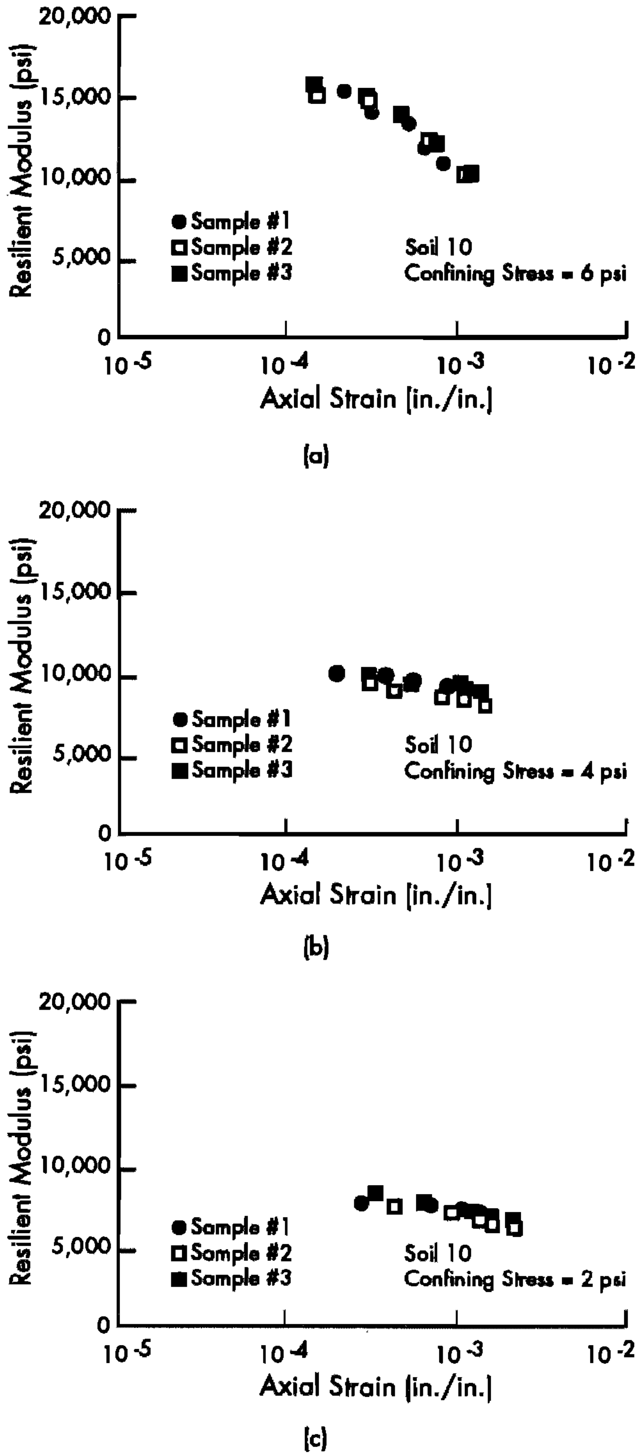


Figure 11.2 Comparison of results obtained from the testing at confining stresses of (a) 6 psi, (b) 4 psi, and (c) 2 psi of three companion samples of soil 10 tested 9 days after their compaction

Obviously, this SEE reflects the variability of the transformed measurements of the resilient axial strains. Moreover, it is believed that this error corresponds mainly to the pure error of the experiment, and not to any other factor that was left out of the regression model (e.g., lack of fit).

It is believed that the variabilities inherent in the test sample preparation process are the main factors responsible for this pure error. Errors of measurements, though probable, are not believed to be relevant in this case, inasmuch as the M_R testing system had demonstrated its ability to produce accurate, repeatable, and consistent results during its calibration process, as described in Chapter 3.

The value obtained in the SEE of the model represents a variability in the estimations of the M_R values. With the proper conversions, this SEE corresponded to a coefficient of variation of about 13 percent in terms of moduli. This indicates that, for an individual sample tested, the M_R values reported are likely to have a variability of ±22 percent, with a 90 percent confidence interval.

This analysis underscores two points: (1) the importance of testing replicate samples, and (2) the fact that, no matter how accurate our testing setup, there will always be some variability in the estimations of the moduli.

Formulating policy on this subject is very difficult. Obviously, the higher the number of replicate samples tested, the more reliable the estimations of the moduli. For instance, if our tolerance is ±5 percent and our confidence interval is 90 percent, we will be required to test at least 20 replicate samples under the same conditions and at the same time. At the moment, such an approach is unfeasible and excessive. Even if we are required to test three replicate samples, our tolerance must be equal to the coefficient of variation (13 percent) for the same 90-percent confidence interval. We do not therefore consider even this approach worth the effort. However, if we test only two replicate samples—an approach that is more reasonable—our tolerance will be on the order of ±15 percent.

Thus, it is more reasonable to specify the testing of at least two replicate samples in the M_R method, and to acknowledge a variability (tolerance) in the estimations of the M_R values on the order of ±15 percent.

CHAPTER 12. EXPERIMENTAL EVALUATION OF SEVERAL FACTORS INFLUENCING RESILIENT MODULI

INTRODUCTION

This chapter presents an experimental evaluation of several factors that influence the resilient moduli. Specifically, the effects of plasticity index, percent of fines, time, moisture content, and dry density on the resilient modulus of compacted soil samples are discussed. In addition, this chapter will present empirical equations developed using the testing data obtained from this experiment. Because these equations are based on reliable testing data, we believe they represent an improvement over those previously published.

EXPERIMENTAL APPROACH

To determine the variation of the M_R values of samples tested at different times, two samples with identical characteristics (companion specimens) were prepared for each of the treatment combinations. All test samples were prepared as described in Chapter 5.

In the first case, a sample—tested 2 days after compaction—was trimmed and measured for final dimensions, water content, and density. The specimen was then placed in the triaxial cell with its ends grouted to the end caps; only when the grout reached its full strength and stiffness did the testing of the sample begin. Following the test, the sample was retained for 4 more days in the triaxial chamber at 0-psi confining pressure. Six days after compaction, the sample was again tested.

In the meantime, the second sample was stored in a constant temperature and humidity chamber. More than 30 days after compaction, this sample was tested under the prototype M_R procedure. In this way, the thixotropy effect of the soils could be assessed quantitatively.

The prototype testing procedure, as detailed in Chapter 4, was the procedure used in performing all M_R tests. This meant that all test samples were first subjected to a conditioning stress of 200 applications of a 4-psi deviator stress under a 6-psi confining pressure. The specimens were then subjected to 100 applications of 2-, 4-, 6-, 8-, and 10-psi deviator stress at each of the confining pressures of 6, 4, and 2 psi.

To evaluate the effect of moisture content changes on the resilient moduli of compacted samples, companion specimens were prepared at their optimum moisture content and at a moisture content higher than their optimum.

DESIGN OF THE EXPERIMENT

As with the previous experimental evaluations, this experiment was treated as a nested factorial with blocking at the soil level. The factors of interest were (1) the plasticity index, (2) the soil, (3) the moisture condition, and (4) the sample age.

Table 12.1, illustrating the arrangement of this particular experiment, shows that all soils were used, that test samples were prepared at their optimum moisture content (opt), and that they were tested 2, 6, and 30-plus days after compaction. Test samples compacted at moisture contents higher than optimum (wet) were also prepared, though from only one of the three different soils available in each of the PI groups. A total of 60 M_R tests were thus performed.

Because a partial number of treatment combinations were used, the statistical analysis of this experiment differs in some degree from the analyses performed in previous chapters. Accordingly, this experiment was treated as a fractional factorial experiment.

COLLECTION OF THE DATA

Test samples were prepared at optimum moisture conditions from soils 4, 5, and 10 of PI group 0-10; from soils 6, 7, and 8 of PI groups 11-20; from soils 2, 11, and 16 of PI group 21-30; from soils 3, 9, and 13 of PI group 31-40; and from soils 1, 12, and 15 of PI group > 40.

Test samples prepared at wet moisture conditions were from soil 10 of PI group 0-10, from soil

Table 12.1 Design of the experiment

Sample Age	PI Group →	0 - 10			11 - 20			21 - 30			31 - 40			> 40		
	Soil ID → Moisture Condition ↓	4	5	10	6	7	8	2	11	16	3	9	13	1	12	15
2	OPT	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●
	WET			●		●		●				●		●		
6	OPT	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●
	WET			●		●		●				●		●		
> 30	OPT	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●
	WET			●		●		●				●		●		

7 of PI group 11-20, from soil 2 of PI group of 21-30, from soil 9 of PI group 31-40, and from soil 1 of PI group > 40. Thus, as indicated earlier, a total of 60 M_R tests were performed in this experiment. All testing data obtained from each of the treatment combinations were collected and stored properly for later analysis (testing results are included in Appendix C).

Typical variations of the resilient modulus versus the axial strain amplitudes are illustrated in Figures 12.1 through 12.8. In this case, the log of the induced resilient axial strains was used instead of the applied deviator stress (as commonly used in the past), so that the dynamic behavior of soil samples could be better represented.

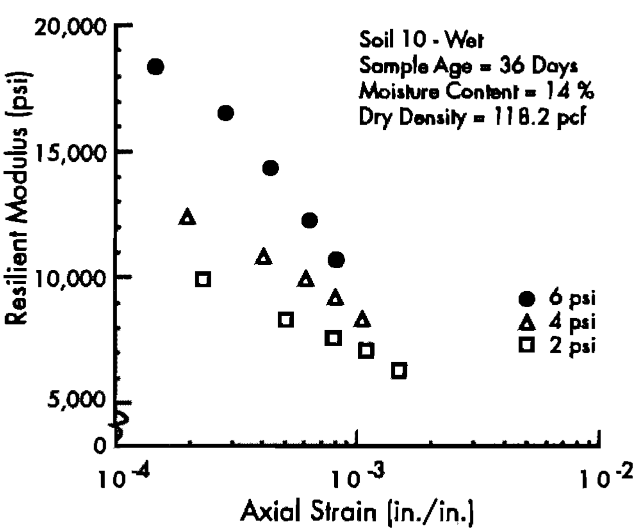


Figure 12.1 Variation of the resilient modulus with the induced resilient axial strain of compacted sample of soil 10 tested 36 days after compaction

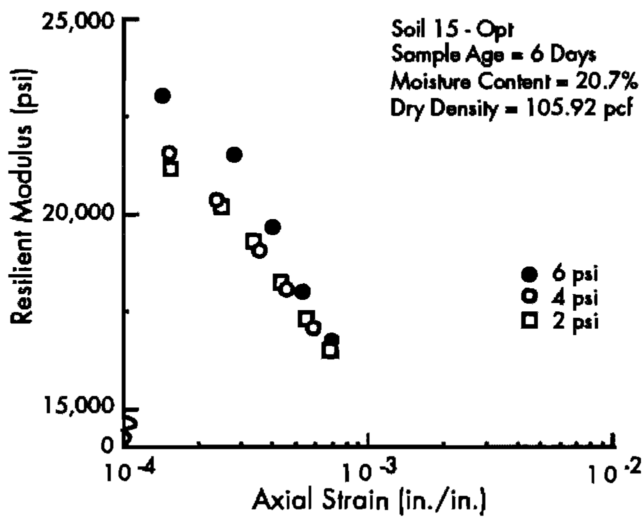


Figure 12.2 Variation of the resilient modulus with the induced resilient axial strain of compacted sample of soil 15 tested 6 days after compaction

EXPERIMENTAL OBSERVATIONS

Figure 12.1 illustrates the results obtained from the testing of a compacted sample of soil 10. This specimen had a moisture content of 14 percent (wet of optimum), a dry density of 118.20 pcf, and was tested 36 days after compaction. The testing, which showed that the confining pressure acting on this low-PI soil had a significant effect on the resilient moduli, suggested a trend: as the confining pressure increases, the resilient modulus of the soil sample also increases. This observation also indicates that we cannot eliminate different confining pressures from the testing procedures, in contrast to what Thompson suggests (Ref 19).

Figure 12.2 illustrates similar results obtained from the testing of a sample of soil 15. This

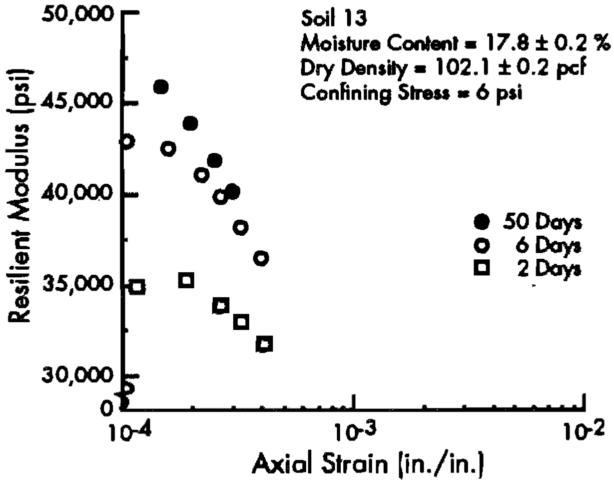


Figure 12.3 Variation of the resilient modulus with the induced resilient axial strain of companion samples of soil 13 tested 2, 6, and 50 days after compaction

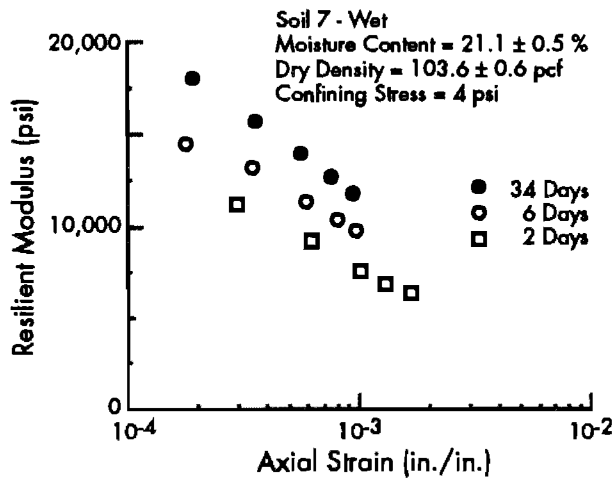


Figure 12.4 Variation of the resilient modulus with the induced resilient axial strain of companion samples of soil 7 tested 2, 6, and 34 days after compaction

sample had a moisture content of 20.7 percent (optimum), a dry density of 105.92 pcf, and was tested 6 days after compaction. Unlike the above case, the confining pressure did not have an effect on the resilient moduli of this high-PI-value soil.

Figure 12.3 illustrates the results obtained from the testing of compacted samples of soil 13 at a confining stress of 6 psi. These companion samples had a moisture content of 17.8 ± 0.2 percent (optimum), a dry density of 102.1 ± 0.2 pcf, and were tested 2, 6, and 50 days after compaction. Although the companion specimens were subjected to the same level of stress states, their dynamic responses

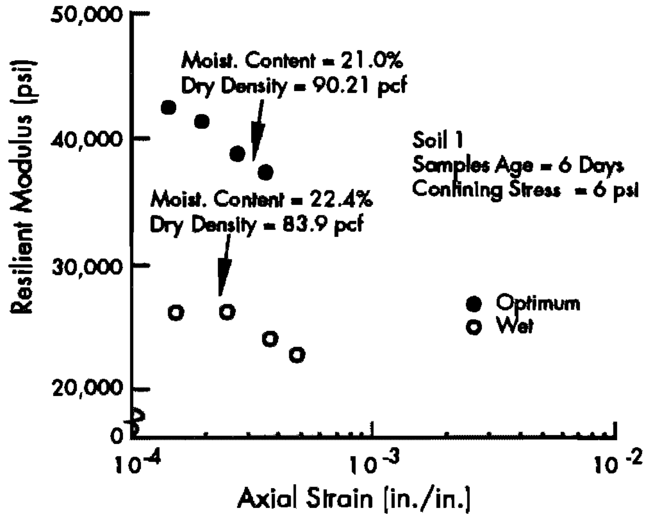


Figure 12.5 Variation of the resilient modulus with the induced resilient axial strain of samples of soil 1 compacted at optimum and wet of optimum moisture contents, and tested 6 days after compaction and at 6-psi confining pressure

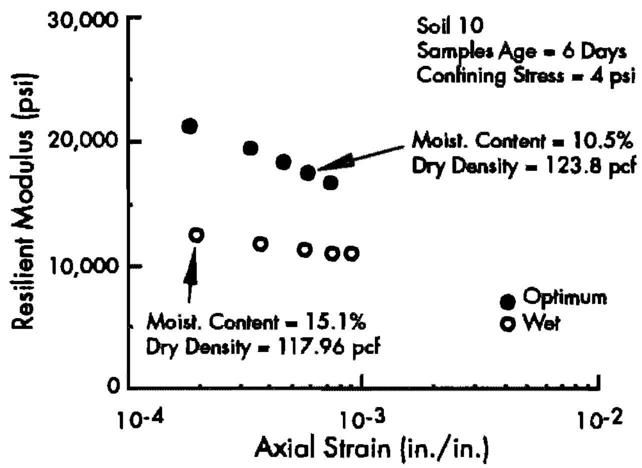


Figure 12.6 Variation of the resilient modulus with the induced resilient axial strain of samples of soil 10 compacted at optimum and wet of optimum moisture contents, and tested 6 days after compaction and at 4-psi confining pressure

differed. Our tests showed that time has an effect on the resilient moduli; that is, as time increases, the resilient modulus of compacted samples (under control conditions) also increases. This tendency, however, was more pronounced at early ages; after about 6 days the effect loses its significance.

Another example of this effect is shown in Figure 12.4, which presents results obtained from the testing of compacted samples of soil 7 at a 4-psi confining stress. These companion samples had a

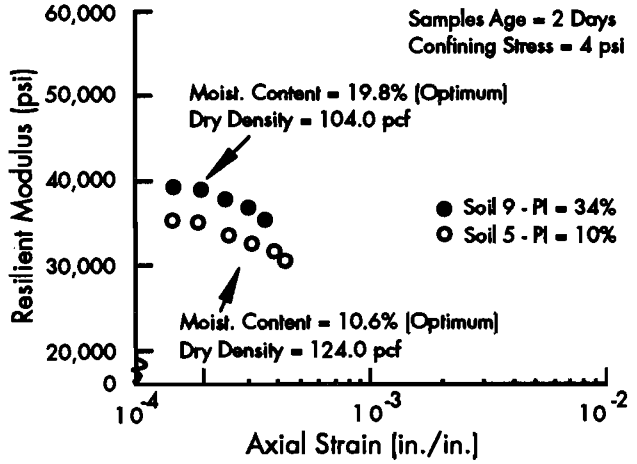


Figure 12.7 Variation of the resilient modulus with the induced resilient axial strain of compacted samples of soil 5 and 9 tested 2 days after compaction and at a 4-psi confining stress

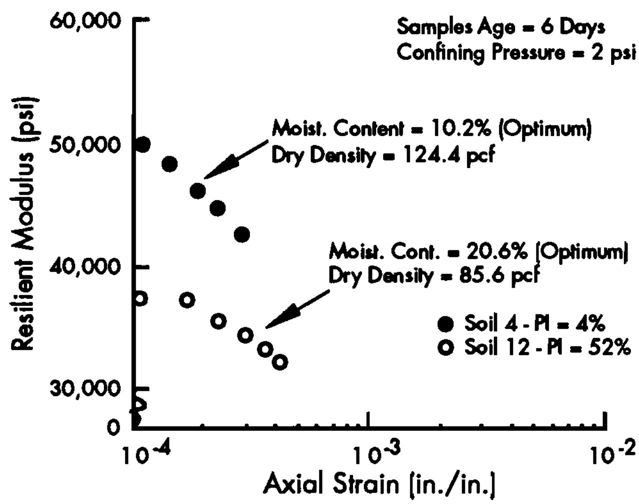


Figure 12.8 Variation of the resilient modulus with the induced resilient axial strain of compacted samples of soil 4 and 12 tested 6 days after compaction and at a 2-psi confining stress

moisture content of 21.1 ± 0.5 percent (optimum), a dry density of 103.6 ± 0.6 pcf, and were tested 2, 6, and 34 days after compaction. In this case, it is interesting to note that the effect of time is also significant on the resilient moduli of this soil type. However, the increase of the moduli is not as pronounced as in the case shown in Figure 12.3 (perhaps a consequence of this soil's relatively low PI value). These results indicate that time effects are more significant for soils having a high PI than for soils having low PI.

It should be emphasized that throughout the testing program, all test samples prepared from the 15 different soils collected from across Texas were indeed affected, to some degree, by the phenomenon known as thixotropy.

Figure 12.5 illustrates the results obtained from the testing of compacted samples of soil 1 at a confining stress of 6 psi 6 days after compaction. The first sample had a moisture content of 21 percent (optimum) and a dry density of 90.21 pcf. The second sample had a moisture content of 22.4 percent (wet of optimum) and a dry density of 83.9 pcf. It was observed that a small increase in the moisture content of the soil sample caused high reductions in the dry density of the soil sample, as well as a significant decrease of resilient moduli. While this trend was expected, the situation nonetheless demonstrated that the moduli of high-PI-value soils have greater variability as a result of small increases in the water content.

Figure 12.6 illustrates the results obtained from the testing of compacted samples of soil 10 at a 4-psi confining stress after 6 days of compaction. The first sample had a moisture content of 10.5 percent (optimum) and a dry density of 123.8 pcf.

The second sample had a moisture content of 15.1 percent (wet of optimum) and a dry density of 118 pcf. It is interesting to observe in this case that a large increase in the water content of the soil specimen caused a relatively small reduction of the density of the sample, and, hence, a moderate decrease of its resilient moduli. Although this trend was, again, expected, the results indicate that, for soils of low PI, an increase of the water content causes a reduction of the resilient modulus—though not as great a reduction as would occur in soils of high PI experiencing the same increase in water content. Indeed, this observation confirms the common perception that soils of low PI are more stable than soils of high PI.

Figure 12.7 illustrates the results obtained from the testing of compacted samples of soils 5 and 9 at a 4-psi confining stress 2 days after compaction. The sample of soil 5 had a moisture content of 10.6 percent (optimum) and a dry density of 124 pcf, while the sample of soil 9 had a moisture content of 19.8 percent (optimum) and a dry density of 104 pcf. Although the two soils were compacted according to specifications, and the samples were at "optimum," their resilient moduli differ, as shown in Figure 12.7. However, it is interesting to note that in this case, the soil with the high PI has a higher modulus, even with its much lower density.

Figure 12.8 illustrates results obtained from the testing of compacted samples of soils 4 and 12 tested 6 days after compaction and at a 2-psi confining stress. The sample of soil 4 had a moisture content of 10.2 percent (optimum) and a dry density of 124.4 pcf, while the soil 12 sample had a moisture content of 20.6 percent (optimum) and a dry density of 85.6 pcf. In contrast to the findings presented in Figure 12.7, the sample of soil 4, having a low PI, appears to have higher resilient moduli than the sample of soil 12, which has a high PI.

In general, Figures 12.1 through 12.8 demonstrate that the dynamic behavior of materials cannot be explained by simply recording and comparing test results randomly. Thus, it was determined that the effects of the influencing factors and their interactions would require a more in-depth evaluation.

ANALYSIS OF THE EXPERIMENT

The analysis of this experiment was performed using the mainframe version of the statistical analysis software, SAS, available at The University of Texas. A data file was created containing all 60 M_R testing reports collected in this experiment, with each report including the 15 different stress states applied to the specimen (as specified in our prototype testing procedure). In addition to specific information on each of the soil specimens and their corresponding testing reports, characteristics such as the moisture content, dry density, and age at testing of the samples—as well as the AASHTO classification, plasticity index, and percent of fines of the soils—were included.

Thus the factors considered in the analysis of this experiment were: (1) the AASHTO classification; (2) the plasticity index, PI, in percent; (3) the percent of fines, Φ, in percent; (4) the moisture content, ω, in percent; (5) the dry density, γ_d, in pcf; (6) the percent of the sample density with respect to the maximum specified density, λ, in percent; (7) the sample age at testing, η, in days; (8) the confining pressure, σ_c, in psi; (9) the seating pressure, σ_a, in psi; (10) the deviator stress, σ_d, in psi; (11) the axial strains, ε_a, in inch/inch; (12) the permanent deformations, δ, in inches; and (13) the resilient moduli, M_R, in psi.

After performing the tests for homogeneity of variances and normality, we determined that there was a need for transforming the data. Accordingly, we selected the logarithmic function as the transforming function; thus, all data are analyzed in transformed units.

We next performed a correlation analysis in which all numeric factors having high correlations with the axial strains (resilient) were searched. Axial strains were selected because they are actual measured values, unlike resilient modulus values that are calculated from two measured values (the deviator stress and the axial strain). The entire analysis followed the same principle, in which the resilient axial strain was the main factor under study.

This correlation analysis proved to be useful in determining several trends in the dynamic behavior of the test materials. From the analysis of signs of the correlation values and their level of probability, the following conclusions were drawn:

(1) As the plasticity index of the soils increases, the induced axial strain (resilient) appears to be somewhat lower. This means that we may expect higher values of M_R for soils of high PI.
(2) As the moisture content of the test samples increases, the induced axial strain definitely decreases. This means that we should expect lower M_R values in soils that have high moisture contents (a well-known fact).
(3) As the dry density of the test samples increases, the induced axial strain decreases. This means that we should expect higher M_R values in soils that have high dry densities.
(4) As the percent of the sample density with respect to the maximum specified density increases, the induced axial strain definitely decreases. This parameter was found to have the highest correlation.
(5) The older the sample at the time of testing, the lower the axial strain. This means that we should expect higher M_R values in older soil samples.
(6) As the applied confining pressure increases, the axial strain definitely decreases. This means that we may expect higher M_R values when testing the samples at higher confining pressures.
(7) As the applied deviator stress increases, the axial strain definitely increases; we should consequently expect lower M_R values (another well-known fact).
(8) The other factors (the dry density, the AASHTO classification, the permanent deformations, the seating pressures, and the percent of fines) were found not to correlate with the resilient axial strain. This means that they do not contribute significantly to the explanation of the dynamic behavior of the materials. Therefore, they were not considered in further analyses.

Resilient Modulus Prediction Model

The model selected to represent the dynamic behavior of the subgrade and non-granular subbase materials was a multi-linear regression model containing all the factors studied. Thus, the most significant factors correlating with the resilient strain were used in this analysis, including: (1) the plasticity index, PI; (2) the moisture content, ω;

(3) the dry density, γ_d; (4) the confining pressure, σ_c; (5) the deviator stress, σ_d; (6) sample age, η; and (7) the percent of the sample density with respect to the maximum specified density, λ.

Many models were developed using different factor combinations. We then evaluated the models for their coefficient of determination, R^2, in an attempt to identify the most efficient. Because we found that dry density contributed the least to the regression models, we decided to drop this factor. Thus, the regression model, once transformed, had the following form:

$$\varepsilon_a = e^a * (\sigma_d)^b * (\sigma_c)^c * (\omega)^d * (\lambda)^e * (\eta)^f * (PI)^g \tag{12.1}$$

$$SEE = 0.106$$
$$R^2 = 0.803$$

Since we know by definition that $M_R = \sigma_d/\varepsilon_a$, a secant M_R model can be formulated as follows:

$$M_R = e^{-a} * (\sigma_d)^{1-b} * (\sigma_c)^{-c} * (\omega)^{-d} * (\lambda)^{-e} * (\eta)^{-f} * (PI)^{-g} \tag{12.2}$$

To facilitate its use, we arranged this equation in a way such that some of the terms of the model would be expressed as correction factors. Such correction factors were tabulated and are included in Table 12.2. Thus, the final form of the model was:

$$M_R = 9800 * F_1 * F_2 * F_3 * F_4 * F_5 * F_6 \tag{12.3}$$

where

- M_R = the predicted resilient modulus, in psi,
- F_1 = the correction factor function of the moisture content,
- F_2 = the correction factor function of the percent of dry density, with respect to the maximum density,
- F_3 = the correction factor function of the plasticity index,
- F_4 = the correction factor function of the sample age,
- F_5 = the correction factor function of the confining pressure, and
- F_6 = the correction factor function of the repeating deviator stress.

The correction factors shown in Table 12.2 were developed in a form that facilitated identifying within the model the effects of each of the factors on the resilient moduli of soils. Thus we determined that, within the model, moisture content is the factor having the greatest effect on the moduli, followed by the plasticity index, percentage of dry density (with respect to the maximum density), age of the sample, and the confining stress. The factor having the least influence within the model, with respect to the overall modulus spectrum, is the deviator stress.

Table 12.2 Correction factors

Moisture Content (%)	F_1	$\frac{\gamma_d}{\gamma_d\,(max)}$ (%)	F_2	Plasticity Index (%)	F_3
10	4.00	100	1.00	10	1.00
15	2.00	95	0.90	20	1.50
20	1.00	90	0.80	30	2.00
25	0.50	85	0.70	≥40	2.50

Sample Age (days)	F_4	σ_c (psi)	F_5	σ_d (psi)	F_6
2	1.00	2	1.00	2	1.00
10	1.10	4	1.05	4	0.98
20	1.15	6	1.10	6	0.96
≥30	1.20			8	0.94
				10	0.92

The value obtained in the SEE, which reflects the variability of the transformed measurements of the axial strains, also represents the variability of the M_R values estimated from this model. Thus, making the proper conversions, the SEE obtained for the model corresponded to a coefficient of variation of about 11 percent in terms of the moduli. This indicates that any engineer who uses Equation 12.3 must be aware that at a 90 percent confidence interval, his/her M_R prediction falls within a range of ±17 percent of tolerance.

The coefficient of variation obtained in this experiment was, throughout its many observations, only 2 points lower than that obtained in Chapter 11. Unquestionably, this experiment had some hidden replications. But the fact that the coefficients of variation of the last two experiments were very similar indicates that such variability is related to the test sample preparation process. Consequently, obtaining reliable estimations of the moduli requires testing replicate samples.

Although Equation 12.3 fits the experimental data remarkably well, it should be mentioned that its applicability falls also within the ranges where such data were collected. These ranges were: (1) from 10 to 35 percent of moisture content; (2) from 100 to 80 percent of percent of dry density with respect to the maximum density; (3) from 4 to 52 percent of plasticity index; (4) from 2 to 188 days of sample age; (5) from 1.6 to 14.9 psi of deviator stress; and (6) from 2 to 6 psi of confining stress.

Thus, the results indicate that Equation 12.3 will provide to the engineer a reliable preliminary estimate of the M_R of compacted soils. But because the model was developed using laboratory data collected under controlled conditions, it cannot provide precise assessments of actual field conditions.

Ramberg and Osgood Prediction Model

Laboratory tests alone cannot account for all variables affecting pavements during their service lives. In addition to lab tests, field tests must be undertaken. Some field tests commonly used to evaluate pavement structures involve the application of relatively small loading magnitudes that induce in the soil mass very small strain amplitudes. Examples of such tests include the Dynaflect deflection, SASW, and crosshole techniques. Other test methods involve the application of loads comparable to actual traffic loads. An example of this method is the falling weight deflectometer (FWD).

When comparing laboratory with field measurements, we must make some kind of judgment about the modulus to use in the mechanistic evaluation of pavements. (A typical example of this was presented in Chapter 10.) Yet because field methods mainly work in very small strain amplitudes, only one modulus value is determined, which is the maximum modulus for that particular condition. On the other hand, the M_R test works in small-to-intermediate strain amplitudes. It is therefore necessary to relate field testing data with laboratory measurements using some kind of master curves. Thus we decided to develop those master curves by analyzing, in different ways, all testing data obtained in this experiment.

Ramberg and Osgood (Ref 52) presented a normalized curve for London clay, expressing its general non-linear, stress-strain behavior in the form of a hyperbola. Because their testing data came from the torsional resonant column test, the model had the following form:

$$\frac{G}{G_{max}} = \frac{1}{1+\alpha * \left[\frac{\tau}{\tau_g}\right]^{r-1}} \qquad (12.4)$$

where

G = the shear modulus,
G_{max} = the maximum shear modulus at yield,
τ = the applied shearing stress,
τ_y = the shearing stress corresponding to the yield, and
α, r = regression coefficients.

Applying the same principle in our case, the general non-linear stress-strain behavior should have the following form:

$$\frac{M_R}{M_{R\max}} = \frac{1}{1+\alpha * \left[\frac{\sigma_d}{\sigma_{det}}\right]^{r-1}} \qquad (12.5)$$

where

M_R = the resilient modulus,
M_{Rmax} = the maximum resilient modulus corresponding to the axial-strain-elastic threshold,
σ_d = the applied deviator stress,
σ_{det} = the deviator stress corresponding to the axial-strain-elastic threshold, and
α, r = regression coefficients.

It should be pointed out here that the M_R test rarely defines M_{Rmax}, inasmuch as most of our measurements fall within axial strains of 0.01 to 0.1 percent. We therefore decided to perform the following:

1. Estimate the axial-strain-elastic threshold, ε_{aet}, for each of the soils using the regression equation "ε_{aet} vs PI" developed in Chapter 9 (recognizing, however, that the equation was developed for those cases in which soils are subjected to a 6-psi confining stress).
2. Develop $M_R = N_1 \cdot \varepsilon_a^{N2}$ models using only data at 6-psi confining stress for each of the 60 M_R tests.
3. Estimate the M_{Rmax} of each of the 60 M_R tests by using those models with the corresponding ε_{aet} of the soils.
4. Estimate the σ_{det} of each of the 60 M_R tests by using $\sigma_d = M_R \cdot \varepsilon_a$.
5. Finally, normalize each of the 5 axial strains and 5 deviator stresses recorded in each of the 60 M_R tests at 6-psi confining stress with their corresponding ε_{aet} and σ_{det}. In addition, we included in that data file the ten normalized testing results performed under the resonant column tests described in Chapter 9 in order to mitigate possible problems of heteroskedasticity.

The Ramberg-Osgood curve, which measured the degree of ductility of the material, is generally modeled as follows:

$$\ln\left[\frac{\varepsilon_a}{\varepsilon_{aet}} - \frac{\sigma_d}{\sigma_{det}}\right] = a + b * \ln\left[\frac{\sigma_d}{\sigma_{det}}\right] \qquad (12.6)$$

Because our materials were prepared and tested under different conditions, our model is a multi-linear model having the following form:

$$\ln\left[\frac{\varepsilon_a}{\varepsilon_{aet}} - \frac{\sigma_d}{\sigma_{det}}\right] = a + b * \ln\left[\frac{\sigma_d}{\sigma_{det}}\right] + c * \ln[\omega] + d * \ln[\lambda] + e * \ln[PI] + f * \ln[\eta] + g * \ln[\sigma_c]$$

First, different combinations with these factors were developed. Then, an evaluation was made in terms of their corresponding coefficients of determination, R^2, in order to select the most efficient one. It was found that only the plasticity index factor contributes significantly to the explanation of the dependent variable. The other factors (such as moisture content, percent of dry density with respect to the maximum density, and sample age) were found to have negligible contributions. This is a very important finding in the sense that those factors appear to have no effect on the normalized modulus. Thus, the regression has the following form:

$$\ln\left[\frac{\varepsilon_a}{\varepsilon_{aet}} - \frac{\sigma_d}{\sigma_{det}}\right] = a + b * \ln\left[\frac{\sigma_d}{\sigma_{det}}\right] + c * \ln[PI] \tag{12.7}$$

SEE = 0.036
R^2 = 0.872

In order to predict the normalized resilient modulus, M_R/M_{Rmax}, the equation had to be arranged as follows:

$$\left[\frac{\varepsilon_a}{\varepsilon_{aet}} - \frac{\sigma_d}{\sigma_{det}}\right] = e^a * \left[\frac{\sigma_d}{\sigma_{det}}\right]^b * [PI]^c \tag{12.8}$$

Since we know that by definition $M_R = \sigma_d / \varepsilon_a$, manipulating those equations yields the regression model:

$$\frac{M_R}{M_{Rmax}} = \frac{1}{1 + e^{-7.07} * \left[\frac{\sigma_d}{\sigma_{det}}\right]^{1.45} * PI^{0.7}} \tag{12.9}$$

Developing functional graphs that can be used efficiently to relate field data with laboratory testing data required additional work on the equations. Since we have determined (Equation 9.3) that

$$\varepsilon_{aet} = \frac{e^{-8.45} * PI^{0.79}}{100}$$

then

$$\frac{M_R}{M_{Rmax}} = \frac{1}{1 + e^{-7.07} * \left[\frac{\varepsilon_a * M_R}{\varepsilon_{aet} * M_{Rmax}}\right]^{1.45} * PI^{0.7}} \tag{12.10}$$

Finally, after several manipulations, the equation became

$$\varepsilon_a = \frac{e^{-8.17} * PI^{0.30} * \left[1 - \frac{M_R}{M_{Rmax}}\right]^{0.69}}{\left[\frac{M_R}{M_{Rmax}}\right]^{1.69}} \tag{12.11}$$

which facilitates the development of an empirical family of curves, as illustrated in Figure 12.9. Figure 12.9 presents the influence of both the axial strain amplitude and the plasticity index on the normalized resilient modulus of the soils subjected to a 6-psi confining stress.

SUMMARY

Several factors that influence the resilient moduli, such as the plasticity index, the percent of fines, moisture content, dry density, and age of the soil sample at the time of testing were investigated. An experiment was designed and undertaken in which a total of 60 M_R tests were performed on samples prepared from 15 different soils, compacted at different moisture conditions, and tested at three different sample ages.

Our analysis of this experimental data indicated the following:

(1) As the PI increases, the M_R value slightly increases for samples tested shortly after compaction (see Figures 12.7 and 12.8);
(2) As the moisture content increases, the M_R value decreases (this was true for those cases in which the moisture content was greater than optimum; see Figures 12.5 and 12.6);
(3) As the dry density increases, the M_R value increases;
(4) The longer the sample ages, the more the M_R value increases;
(5) As the confining pressure increases, the M_R value increases; and
(6) As the deviator stress increases, the M_R value decreases.

The other factors considered in this analysis—dry density, the AASHTO classification, the permanent deformations, the seating pressures, and the percent of fines—were found not to correlate with the moduli.

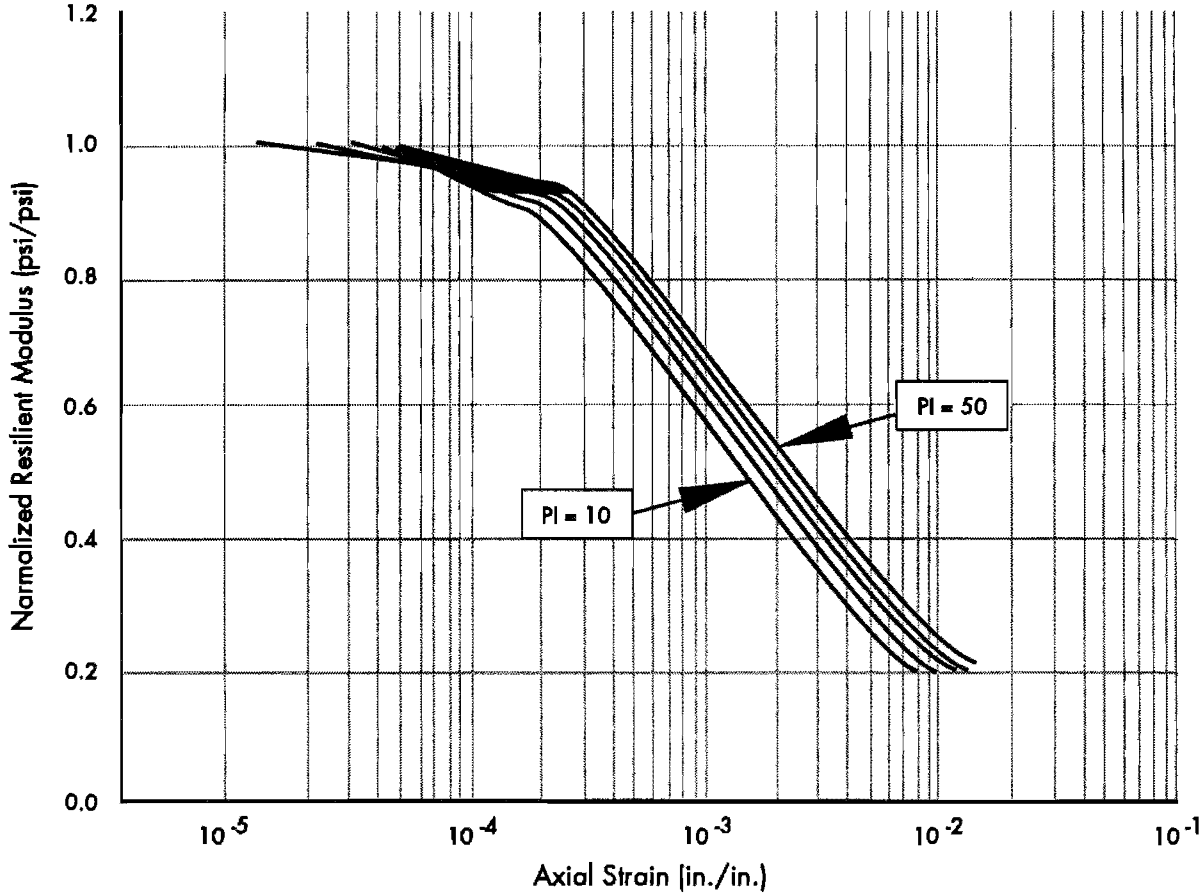


Figure 12.9 Influence of the axial strain amplitude and the plasticity index on the normalized resilient modulus of the soils subjected to a 6-psi confining stress

The implications of determining significant effects of time on the resilient moduli are controversial. Many researchers argue that this effect exists only in the laboratory, i.e., it could never be expected in the field (because laboratory conditions do not have major changes in temperature and humidity, while field conditions may be variable). Nevertheless, the significant increase in stiffness in young samples (between 2 and 6 days), and the not-so-significant increase in older samples (after 6 days) that our tests revealed cause us to question this belief.

Empirical regression equations were developed for use in the design of pavements, while other equations were developed for use in the evaluation of pavement structures. The first provides to the pavement engineer a reliable and quick estimation of the resilient modulus of compacted materials, based on such properties as the moisture content, the percent of dry density with respect to the maximum density, the plasticity index, and the age of the sample at the time of testing. In this equation, the most significant parameter is the moisture content of the sample.

Other equations developed in this chapter include a family of curves that provide the pavement engineer with a powerful tool for evaluating the stiffness characteristics of the pavement layers by relating field with laboratory testing data. This family of curves was defined by applying the same principle of non-linear stress-strain behavior (characterized by a normalized modulus) presented by Ramberg and Osgood. Empirical models were studied to identify the influences of moisture content, density, plasticity index, and age. It was found that only the plasticity index factor contributes significantly to the explanation of the normalized behavior.

In addition, the finding that neither the moisture content nor the age of the sample affects the normalized behavior indicates that the empirical equations can be used independent of the moisture condition and age of the samples, and that by defining the PI of the soil—and, through field tests, the *in situ* elastic modulus (which is actually M_{Rmax})—any M_R at any axial strain amplitude can be easily estimated.

CHAPTER 13. PROPOSED RESILIENT MODULUS TESTING METHOD

This chapter presents the laboratory testing method proposed for determination of the resilient modulus, M_R, of subgrade soils and non-granular subbase materials. This test method, which is a modification of AASHTO T 274-82, features a testing setup and procedure we have found to be more reliable.

This modified test method, then, specifically outlines the procedures for preparing and testing untreated soils used for determining dynamic elastic modulus. Most importantly, this determination is made under conditions that represent a reasonable simulation of the physical conditions and stress states of subgrade materials placed beneath flexible pavements subjected to moving wheel loads. The test method is applicable to undisturbed samples of natural and compacted soils and to disturbed samples prepared for testing by compaction in the laboratory. Finally, the values of resilient modulus determined with these procedures can be used in the available linear-elastic and nonlinear elastic layered system theories used to calculate the physical response of pavement structures.

SUMMARY OF THE TEST METHOD

A repeated axial deviator stress of fixed magnitude, duration, and frequency is applied to an appropriately prepared cylindrical test specimen. During and between the dynamic deviator stress applications, the specimen is subjected to a static all-around stress provided by a triaxial pressure chamber. The induced resilient axial strain is measured and used to calculate the dynamic secant resilient moduli.

SIGNIFICANCE AND USE

The resilient modulus test reveals the basic constitutive relationship between stress and deformation of flexible pavement construction materials—information which is necessary for a structural analysis of layered pavement systems. It also provides a means for characterizing pavement materials under a variety of environmental and stress conditions that simulate the field conditions of pavements subjected to moving wheel loads.

BASIC DEFINITIONS

(1) σ_1 is the total axial stress (major principal stress).
(2) σ_3 is the total radial stress; that is, the applied confining pressure in the triaxial chamber (minor and intermediate principal stresses).
(3) $\sigma_d = \sigma_1 - \sigma_3$ is the deviator stress; that is, the repeated axial stress for this procedure.
(4) ε_a is the resilient axial strain induced by σ_d.
(5) $M_R = \sigma_d / \varepsilon_a$ is the secant resilient modulus.
(6) Load duration is the time interval during which the specimen is subjected to a deviator stress.
(7) Cycle duration is the time interval between applications of a deviator stress.
(8) Subgrade material consists of the natural or compacted soils on which the pavement structure rests.
(9) Subbase material consists of locally available compacted materials (non-aggregate) comprising a layer between the base and the subgrade layers of a flexible pavement.

APPARATUS

(1) Triaxial pressure chamber: The pressure chamber is used to contain the test specimen and the confining fluid during the test (air is used as the chamber fluid). A triaxial chamber suitable for use in resilience testing of soils is shown in Figure 13.1. The chamber is similar to most standard triaxial cells except that it is somewhat larger (so as to facilitate the internally mounted transducers) and has additional outlets (for the electrical leads of those transducers).

(2) Loading device: The external loading device must be one capable of providing varying repeated loads in fixed cycles of load and

release. A closed-loop electro-hydraulic system is required for this operation. A haversine loading waveform consisting of a load duration of 0.10 seconds and a cycle duration of 1 second is used.

(3) Load and specimen response measuring equipment:

a. The axial load measuring device should be an electronic load cell placed between the sample cap and the loading piston, as shown in Figure 13.1. The following load cell capacities are recommended:

Sample Diameter (inches)	Maximum Load (lbs)
2.80	100
4.00	200

b. Test chamber pressures are monitored with conventional pressure gauges, manometers, or pressure transducers having an accuracy within 0.1 psi.

c. The deformation measuring device consists of two linear variable differential transformers (LVDT's) clamped to steel bars inside the triaxial chamber, as shown in Figure 13.1. The LVDT's will have a linearity of ±0.20 percent of full range output, a repeatability of 0.000004 inch, and a minimum sensitivity of 2mv/v(AC) or 5 mv/v(DC). The following LVDT ranges are recommended:

Sample Diameter (inches)	Maximum Load (inches)
2.80	±0.04
4.00	±0.06

d. The characteristics of the deformational transducers limit the capabilities of the testing system. For such characteristics, in general, resilient axial strains below 0.01 percent are not measured accurately.

e. Suitable signal excitation should be maintained so that recording equipment and measuring devices can be used for simultaneous recording of axial load and deformation. The signal should be free of noise. The LVDT's should be wired separately so each LVDT signal can be monitored independently.

f. To minimize errors in testing, the transducers, along with the entire testing system, should be calibrated periodically. The use of synthetic samples of known properties is recommended to assess the accuracy of measurements.

g. A data acquisition system is required to record the signals emitted by the transducers. A data acquisition board mounted inside a personal computer having computational and control capabilities (with a test sampling rate of at least 1,000 records per channel per second) is recommended.

(4) Specimen preparation equipment: A variety of test specimen preparation equipment is required to prepare undisturbed samples for testing and to obtain compacted specimens that are representative of field conditions. Such equipment typically includes:

a. Equipment for trimming test specimens from undisturbed thin-walled tube samples of subgrade material as described in AASHTO T-234-85.

b. Split molds used to provide either 2.8 or 4.0-inch-diameter samples, with heights of about 5.6 and 8.0 inches, respectively. For compaction, an automatic tamper (as specified in Tex-113-E, which is in close agreement with ASTM D 1557 and AASHTO T-180) can be used—provided that the area of the rammer's striking face represents no more than 30 percent of the specimen area.

c. Miscellaneous: calipers, micrometer gauge, steel rule, rubber membranes, rubber O-rings, membrane expander, scales, moisture content cans, and hydrostone. In addition, a pedestal for grouting can be used to expedite the entire testing process.

PREPARATION OF TEST SPECIMENS

(1) Specimen size: Specimen length should not be less than twice the diameter. Minimum specimen diameter is 2.8 inches, or 5 times the nominal size (nominal size is the particle size of the material corresponding to the 95 percent passing size). The following guidelines should be used to determine the specimen size:

a. Use 2.8-inch-diameter samples from the thin-walled-tube undisturbed samples for cohesive subgrade soils, and from disturbed samples with higher than 70 percent passing sieve No. 10. Use only the portion of the material passing sieve No. 10.

b. Use 4.0-inch-diameter samples for all subgrade and subbase material types with a nominal particle size of 3/4 inches.

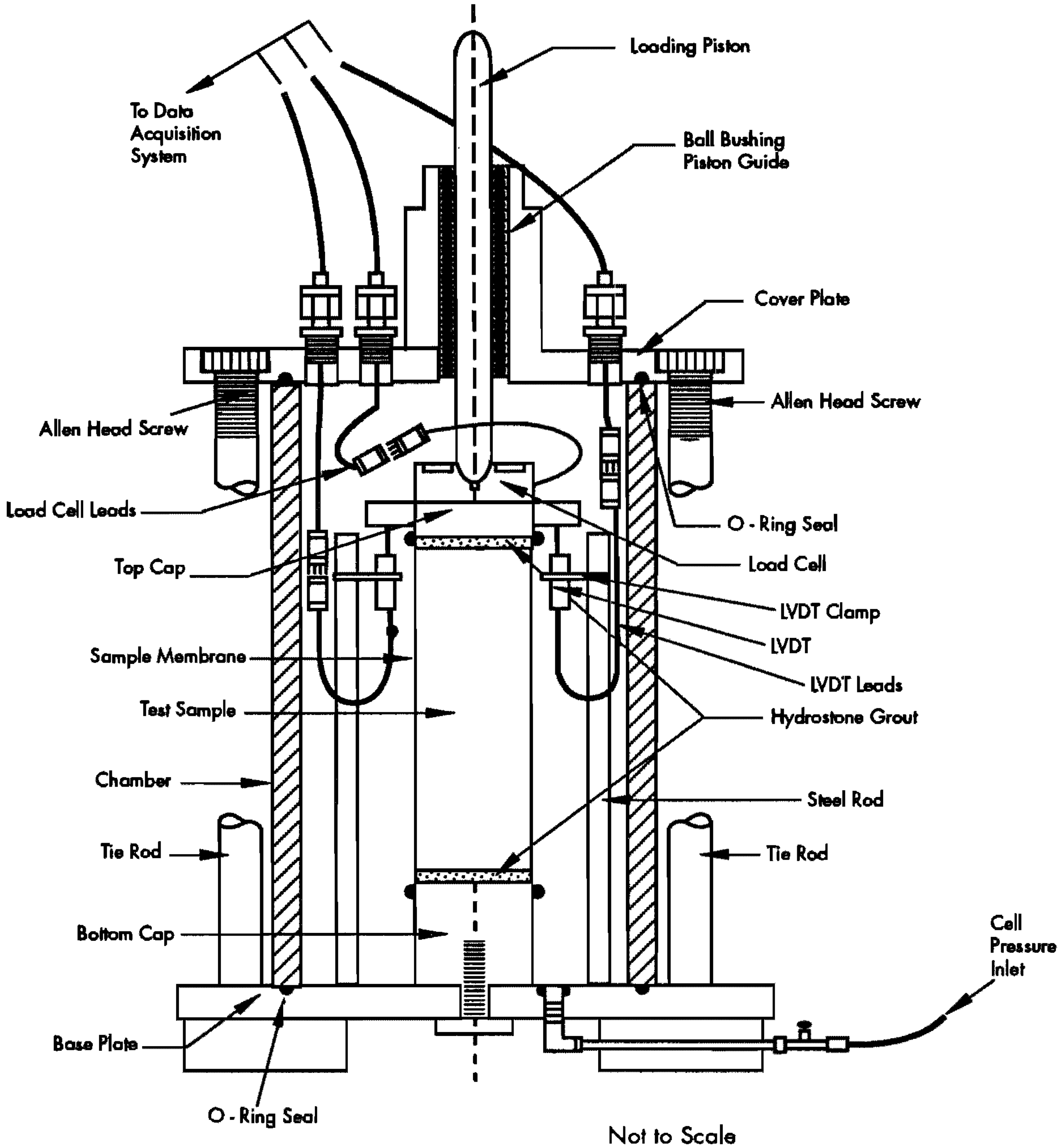


Figure 13.1 Triaxial chamber used in M_R tests of subgrade and subbase soils

(2) Undisturbed specimens: Undisturbed subgrade and subbase specimens are trimmed and prepared as described in AASHTO T-234-85. Determine the natural water content and in-place density of the soils according to Tex-115-E (Tex-101-E is in close agreement with AASHTO T-146-82), and record the values in the test report. If thin-walled tube samples do not provide a good sample for testing, then reconstitute the specimen as described in item 3.

(3) Disturbed specimens: All disturbed specimens shall be first prepared according to Tex-101-E, which is in close agreement with AASHTO T-146-82. Then, laboratory-compacted specimens should be prepared at *in situ* dry density and at *in situ* water content. The compacting effort specified in Tex-113-E should be used to compact the samples.

a. The moisture content and the dry density of the laboratory compacted specimens should not vary more than ±0.5 percent and ±2 percent from the *in situ* water content and *in situ* dry density determined in the field for that layer, respectively. In case the field data are not available, the actual dry density and optimum water content of the material should be determined according to Tex-113-E or Tex-114-E.

b. At least two replicate specimens that represent actual *in situ* conditions should be prepared for testing.

c. If the pavement engineer feels it is necessary, more than two replicate specimens can be used; these should be prepared at water contents that differ from the optimum water content, using the same compacting effort specified in Tex-113-E. This may be required by the pavement engineer who aims at simulating more reliably the different seasonal conditions of the pavement materials.

(4) Compaction method: Tex-113-E is the method of compaction recommended. However, the plasticity index of the soil should first be determined in order to select the appropriate compacting effort, CE, to be applied in compacting the test samples.

a. To compact the total volume of the soil, V, five layers are recommended to obtain a more uniform sample. The surface of each layer should be scarified before placing the next layer. Knowing the weight of the hammer, W, and the height of drop, H, the number of blows, N, per layer can be determined as follows:

$$N = \frac{CE * V}{5 * W * H} \qquad (13.1)$$

b. After specimen compaction has been completed, verify the compaction water content of the remaining soil and carefully remove the specimen from the mold. If the compacted specimen does not have the desired dimensions, trim the test sample (in accordance with the procedures described in AASHTO T-234) and square the end surfaces.

c. Weigh the test specimen to the nearest gram. Determine the average height and diameter to the nearest 0.02 inches and compute its wet density. The excess material trimmed from the sample can also be used to verify its water content.

d. Wrap the test samples with plastic sheets or bags to prevent moisture loss; store them in a humidity room of constant temperature for 2 days.

(5) Placement of the test samples into the triaxial chamber: Undisturbed and compacted samples shall be weighed and their dimensions measured to calculate their initial density and to prepare them for installation in the triaxial cell.

a. All test specimens shall be grouted to the top cap and base pedestal of the triaxial chamber using a hydrostone paste having a thickness no greater than 0.12 inches. The hydrostone paste is useful in that it allows adjustment of the level of the top cap and base pedestals to accommodate or eliminate any imperfections in the end surfaces of the test specimens. It also helps to improve both the uniformity of the applied repeated stress and the accuracy of the deformational measurements of the sample. Figure 13.1 shows a test specimen grouted to the end caps.

b. The hydrostone paste consists of potable water and hydrostone cement mixed in a 0.40 ratio. Once the water is mixed with the hydrostone cement, the hydration of the paste begins, with consistency rapidly obtained. A minimum of 120 minutes (counting from the moment water is added to the hydrostone cement) is recommended as a curing time; this assures that the grout will be strong enough to withstand the M_R test without risking the accuracy and reliability of the measurements.

c. It is not necessary to grout the test sample directly in the triaxial chamber. To expedite this operation, the grouting process can be performed on a pedestal frame, similar to that used in capping concrete cylinders, with additional steel caps that can be bolted to the original end caps of the triaxial chamber.

d. After the specimen is installed and its ends grouted, place vacuum grease at the sides of the end platens to facilitate the adherence of the membranes to the end platens.

e. Two rubber membranes, 0.014-inches thick and secured with O-rings at each end, should be used in order to eliminate probable gas leakage problems. Seal the membrane to the top and bottom platens.

f. Clamp the LVDT's on steel bars fixed inside to the base or to the top of the triaxial cell. The LVDT's should be installed diametrically opposite to one another and positioned so that they point to the top of the sample. In this way, axial deformations can be measured from the total height of the sample. Figure 13.1 illustrates the final configuration on which the LVDT's are finally installed.

g. Once the LVDT's are positioned, the body of the triaxial chamber can be mounted. Tighten the chamber tie rods firmly.
h. Slide the triaxial chamber into position under the axial loading device. Bring the loading device down and couple it to the triaxial chamber piston; apply a seating pressure of no more than 2 psi to the sample.

TESTING PROCEDURE

The following procedure used for undisturbed and laboratory-compacted specimens requires a minimum of 375 seconds (6 minutes and 15 seconds) of testing time; at least two replicate specimens should be tested 2 days after their compaction in the laboratory.

(1) Apply a confining pressure of 6 psi to the test specimen.
(2) Apply 25 repetitions of each of the following deviator stresses: 2, 4, 6, 8, and 10 psi. During the application of each deviator stress, record and average the actual applied compressional force and the induced resilient axial deformation of the last 5 cycles of the 25 cycles. Report (on a testing form similar to the one shown in Figure 13.2) the actual confining pressure, the actual applied deviator stress, the induced resilient axial strain, and the calculated resilient modulus. Other parameters—including the seating pressure and the cumulative permanent deformations—can also be reported.
(3) Apply a confining pressure of 4 psi to the test specimen and repeat item 2.
(4) Apply a confining pressure of 2 psi to the test specimen and repeat item 2.
(5) If the axial strain (resilient) is below the 0.01 percent (minimum reliable strain measurement), ignore that particular testing result in further analysis. If the axial strain (resilient) is greater than 1 percent, or if the permanent deformations exceed 1 percent of the sample height, stop the test.
(6) Upon completion of the test, reduce the confining pressure to zero and disassemble the triaxial cell.
(7) Removing the membranes from the specimen, take a piece from the core of the specimen and determine the water content of the sample after testing; compare this value with the initial water content.

REPORT

The M_R testing report consists of three parts: (1) the basic information of the material and test samples; (2) the testing results and plots of the variations of the moduli versus deviator stress and moduli versus axial strain (resilient); and (3) an analysis of results. Figure 13.2 illustrates a typical M_R testing report.

(1) Data sheets shall include the basic information of the material (e.g., its origin and Atterberg limits) as well as information related to the test sample (e.g., the age of the sample at the time of testing, its dimensions, its water content, and its dry density). In addition, the following testing results should be included: the confining pressures, the seating pressures, the deviator stresses, the axial strains, the permanent deformations, and the calculated secant resilient moduli of the sample at each of the stress states of the test.
(2) Two plots are required per test. One arithmetic plot showing the variation of the resilient modulus with deviator stress for a given confining pressure, and one semi-logarithmic plot showing the variation of the resilient modulus with logarithmic of the resilient axial strain for a given confining pressure.
(3) The analysis of results consists in developing a linear regression equation to predict the deformational characteristics of the material, suggesting one M_R value for design. Use all the results obtained from the testing of the replicate samples in the statistical analysis
    a. A regression model accompanied by both its coefficient of determination, R^2, and the standard error of the estimate, SEE, should have the following form:

$$\ln(\varepsilon_a) = a + b * \ln(\sigma_d) + c * \ln(\sigma_3), \text{ or}$$
$$\varepsilon_a = \varepsilon_a * \sigma_d{}^b * \sigma_3{}^c \qquad (13.2)$$

By definition: $M_R = \sigma_d / \varepsilon_a$

Thus, the modulus can be expressed in two similar equations, in terms of either the deviator stress or the axial strain:

$$M_R = e^{-a} * \sigma_d^{(1-b)} * \sigma_3^{-c}, \text{ or}$$
$$M_R = K1 * \sigma_d^{K2} * \sigma_3^{K3} \qquad (13.3)$$

$$M_R = e^{-a/b} * \sigma_d^{(1-b)/b} * \sigma_3^{-c/b}, \text{ or}$$
$$M_R = N1 * e_a^{N2} * \sigma_3^{N3} \qquad (13.4)$$

b. Based on either stress or strain criteria, the pavement engineer can estimate a unique resilient modulus value for use as an input in the AASHTO pavement design guide. For example, using the report illustrated in Figure 13.2, if the σ_d were 6 psi, and the σ_3 were 2 psi, then the design M_R would be 36,612 psi.

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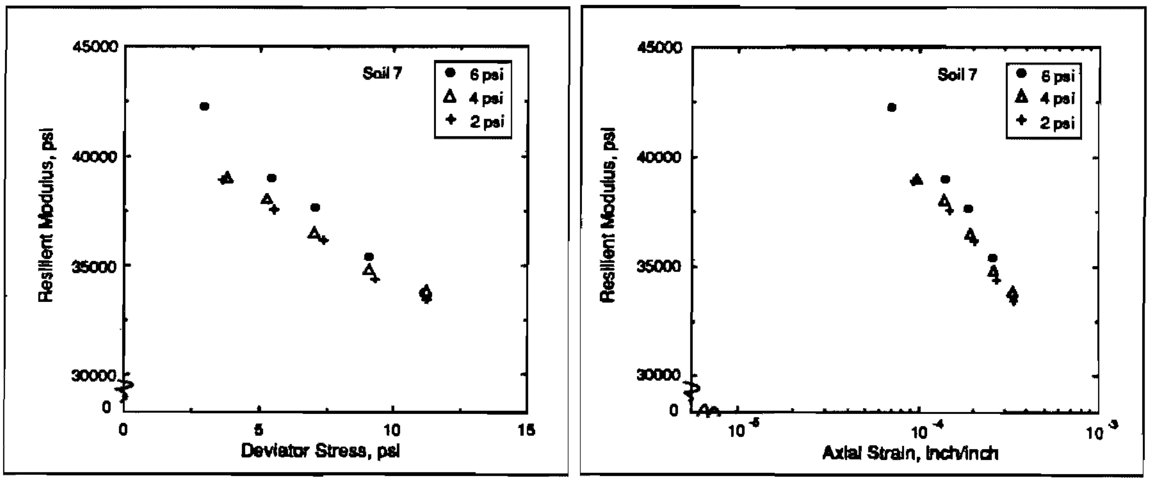
Example.out RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = 7
DESCRIPTION = Dist 4 - Potter - Spur 951 - 2 days
MOISTURE CONTENT = 16.50 percent
DRY DENSITY = 106.40 pcf.
LIQUID LIMIT = 37.60 percent
SAMPLE HEIGHT = 5.665 inches
SAMPLE DIAMETER = 2.840 inches

CONFIN. (psi)	SEATING (psi)	DEV. STRESS (psi)	PERM DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	0.946	2.937594	0.00000596	0.000209	0.000578	0.000070	42256.336
6.000	0.778	5.466032	0.00005450	0.000431	0.001157	0.000140	38984.285
6.000	0.674	7.085805	0.00009558	0.000586	0.001546	0.000188	37651.320
6.000	0.492	9.087582	0.00013200	0.000819	0.002090	0.000257	35397.219
6.000	0.324	11.139918	0.00018613	0.001070	0.002670	0.000330	33748.883
4.000	0.905	3.807870	0.00020498	0.000334	0.000772	0.000098	39007.582
4.000	0.871	5.291881	0.00021393	0.000474	0.001103	0.000139	38026.734
4.000	0.762	7.032435	0.00022162	0.000653	0.001530	0.000193	36505.316
4.000	0.616	9.082899	0.00022540	0.000876	0.002075	0.000260	34872.164
4.000	0.436	11.221374	0.00023128	0.001126	0.002630	0.000332	33847.223
2.000	1.140	3.644021	0.00024341	0.000313	0.000748	0.000094	38913.883
2.000	1.034	5.539998	0.00027566	0.000490	0.001182	0.000148	37549.988
2.000	0.945	7.367626	0.00029344	0.000677	0.001633	0.000204	36150.348
2.000	0.787	9.302927	0.00032233	0.000903	0.002162	0.000271	34388.422
2.000	0.622	11.255548	0.00033430	0.001143	0.002667	0.000336	33474.125



ANALYSIS OF RESULTS		
EXPRESSIONS	STATISTICS	APPLICATION
WHEN $\varepsilon_a > 0.0001$ (1) $MR = K1 \cdot \sigma d^{K2} \cdot \sigma_3^{K3}$ or (2) $MR = N1 \cdot \varepsilon_a^{N2} \cdot \sigma_3^{N3}$	MODEL: $Ln(\varepsilon a) = A + B \cdot Ln(\sigma d) + C \cdot Ln(\sigma 3)$ R^2 = 0.999 AND SEE = 0.008 (1) K1 = 46594 , K2 = -0.145 AND K3 = 0.027 (2) N1 = 11941 , N2 = -0.127 AND N3 = 0.024	SAY σd = 6 psi and $\sigma 3$ = 2psi USING Eq. (1): **MR(design) = 36,612 psi**

Figure 13.2 A typical M_R testing report

CHAPTER 14. SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

SUMMARY

In 1986, the American Association of State Highway and Transportation Officials (AASHTO) adopted for use in the design of pavement structures the resilient modulus test for determining properties of roadbed soil and pavement components. For roadbed soils, the AASHTO pavement design guide specifies that laboratory resilient modulus tests be performed on representative samples under stress and moisture conditions that simulate the actual field conditions. The testing method they endorsed was AASHTO T-274-82, the "Standard Method of Testing for Resilient Modulus of Subgrade Soils." Since its introduction, however, AASHTO T-274 has been widely criticized. Problems in the setup and testing process generated deep concerns regarding the reliability, repeatability, and efficiency of the test method. And while a variety of alternatives have been developed in response, none of the proposed methods have been subjected to rigorous evaluation.

The purpose of this research effort was to evaluate and, if possible, develop a reliable resilient modulus test for subgrade and non-granular subbase materials for use in routine pavement design. In undertaking these tasks, we investigated not only the state of knowledge regarding the dynamic behavior of soils, but also the characteristics and limitations of the resilient modulus testing system. As a result, guidelines on required instrumentation and calibration of the testing system were developed. A major breakthrough was the use of synthetic samples of known elastic properties to evaluate and calibrate the test equipment. In addition, alternate testing procedures were examined to develop a prototype procedure that was then thoroughly evaluated under many different conditions. To validate the testing procedure and the guidelines on equipment configuration to be recommended, we compared modulus results obtained with other laboratory and field tests. Based on this extensive investigation, a new resilient modulus testing method has been developed.

Thus, the major contribution of this investigation is a new resilient modulus testing method, the application of which will ensure fast, accurate, and reliable modulus estimates of subgrade and non-granular subbase materials. Accordingly, the method represents a testing procedure far more efficient and reliable than any other alternative procedure, including AASHTO T-274. Moreover, this new approach can be used in the evaluation of several factors (e.g., plasticity index, moisture conditions, density, among others) affecting the resilient modulus of soils. These investigations permitted the formulation of modulus prediction models that can be used to obtain preliminary modulus estimates of these pavement materials for use in the design and evaluation of pavements.

A few caveats are in order, however: Despite the positive contributions of this study, it should be recognized that, since this test is a laboratory test, much effort—specifically in the selection, sampling, and preparation of truly representative specimens for testing—is still required by the pavement engineer attempting to provide a correct assessment of field conditions. Additionally, a point of concern in the application of this new testing method is the 1986 AASHTO pavement design guide, insofar as the guide includes fatigue equations that were developed using resilient modulus estimates obtained from questionable approaches. Based on the strong evidence presented in this study, we believe those modulus estimates are inaccurate. Consequently, there is a need for revising those equations using reliable modulus estimates that can be obtained through the application of this testing method.

CONCLUSIONS

To conclude: As long as the guidelines proposed in this report are followed, the laboratory resilient modulus test can now be used to determine accurately and reliably the stiffness characteristics of subgrade and non-granular subbase materials.

From the investigations performed on the different aspects of the resilient modulus test, specific conclusions are also drawn. These conclusions are grouped according to: (1) equipment configuration, (2) testing procedure, and (3) material characteristics.

From the aspect of equipment configuration, the following conclusions are drawn from this study:

(1) Diligent effort is required in the design, installation, and use of a resilient modulus testing system. Loading systems, system instrumentation, and data acquisition and control systems must be carefully designed if they are to have the capabilities and accuracy required in the resilient modulus test.
(2) Locating two LVDT's inside the triaxial chamber—oriented in the direction of the loading motion and at the top of the sample, and clamped to either the top or base of the triaxial chamber—is the most effective method for monitoring accurate and reliable resilient axial deformations.
(3) The entire resilient modulus testing system—and not merely the individual transducers—requires calibration. For such calibration, the testing of synthetic samples of known properties can be useful in assessing equipment compliances and system reliability.
(4) Strong contacts between the specimen and the caps (top and bottom) are very important. This factor can be particularly crucial for stiff materials, where poor contact can result in erroneous modulus values. Hydrostone paste, or similar material that provides a uniform and strong contact, can be used to grout the specimen to the end caps, thus eliminating the risk of movement and incompatibility at these points.

From the aspect of testing procedure, the following conclusions are drawn:

(1) For properly grouted specimens, sample conditioning is an unnecessary process and can be eliminated from the testing procedure. The study found that sample conditioning neither corrects the imperfect contacts between the specimen and end caps, nor destroys the effect of thixotropy of the compacted soils.
(2) For properly grouted specimens, fewer stress repetitions than are customarily used are sufficient for reliable modulus estimates. A maximum of 25 loading repetitions at the different stress states in the testing procedure is proposed.
(3) Because the variabilities inherent in the preparation process of the test samples can affect the modulus estimates, at least two replicate samples are necessary so as to increase the reliability of the modulus estimates.

From the aspect of material characteristics, the following conclusions are drawn:

(1) The aging of laboratory-compacted soils is an important factor in laboratory modulus measurements and should therefore be considered in routine testing. Testing the samples 2 days after their preparation is proposed.
(2) Based on the evaluation of several factors that influence the overall modulus spectrum of compacted soils, moisture content was identified as the factor that has the largest effect on the moduli, followed by the plasticity index, percentage of dry density with respect to the maximum density, age of the sample, confining stress, and deviator stress.
(3) The plasticity index—rather than the moisture content or the age of the samples—is the factor that contributes most significantly to the explanation of the normalized modulus-strain behavior. This implies that the normalized behavior is independent of the age and moisture condition of the samples.
(4) Axial-strain-elastic thresholds were found to be highly related to the plasticity index of the material.
(5) Good comparisons were found between the moduli of compacted soils measured with resilient modulus and torsional resonant equipment. An important point in the comparisons was that moduli had to be compared at the same frequency and strain amplitude.

RECOMMENDATIONS

This study recommends that any testing laboratory that performs or plans to perform the laboratory resilient modulus test consider adopting the resilient modulus testing method described in this report.

For application, the resilient modulus tests should, when feasible, be performed using the new approach described in Chapter 13. If this approach is not feasible, or when there is a need for a quick and preliminary modulus estimate of subgrade and non-granular subbase materials, use of the resilient modulus prediction models presented in Chapter 12 is recommended.

Finally, the following are suggested as areas for future research:

(1) More comparisons between laboratory and field modulus measurements should be performed to determine the most effective approach for selecting, sampling, and preparing truly representative specimens for laboratory testing.
(2) The AASHTO fatigue equations should be revised using reliable modulus estimates that can be obtained through the application of the resilient modulus testing method described in this report.
(3) Investigations should be conducted on granular base and subbase materials in order to develop a reliable testing method for these types of materials. Such investigations will further our understanding of the stiffness characteristics of pavement components.

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APPENDIX A. DESCRIPTION OF THE TORSIONAL TESTING TECHNIQUES

Torsional testing techniques are popular laboratory techniques used to measure the deformational characteristics of materials. Such techniques used in this study have included the torsional resonant column test and the torsional shear test. In general, they operate best in the shearing strain range of approximately 0.0001 to 0.1 percent.

TORSIONAL RESONANT COLUMN

Torsional resonant column equipment of the fixed-free type was used in these tests. In the fixed-free configuration, the bottom of the test specimen is held fixed while the top (free end) is connected to a drive system used to excite and monitor torsional motion, as illustrated in Fig A.1(a).

The basic operational principle is to vibrate the cylindrical specimen in first-mode torsional motion. Once first mode is established, measurements of the resonant frequency and amplitude of vibration are made, as shown in Fig A.1(b). These measurements are then combined with equipment characteristics and specimen size to calculate shear wave velocity, V_s, shear modulus, G, and shearing strain amplitude, γ (Ref 14).

One-dimensional wave propagation in a circular rod is used to analyze the dynamic response of the specimen. The basic data-reduction equation is expressed as follows:

$$I / I_o = \omega_r * L / V_s * \tan(\omega_r * L / V_s) \qquad (A.1)$$

where I is the mass moment of inertia, I_o is the mass moment of inertia of drive system, ω_r is the resonant circular frequency, and L is the length of the specimen. Once the value of shear wave velocity is determined from Eq. A.1, and knowing the density of the specimen, ρ, its shear modulus can be calculated from:

$$G = \rho * V_s^2 \qquad (A.2)$$

The shearing strain, γ, is calculated from the peak rotation of the top of the specimen at 0.67 times the radius of the solid sample (Ref 14).

TORSIONAL SHEAR TEST

The torsional shear test is another method for determining shear moduli using the same resonant column equipment but operating it in a different fashion. In this test, a cyclic torsional force with a given frequency, generally below 10 Hz, is applied at the top of the specimen while the bottom is held fixed, as shown in Fig A.2(a). Instead of determining a resonant frequency, the stress-strain hysteresis loop is determined from measuring the torque-twist response of the specimen. Proximetors are used to measure the twist while the current applied to the coils is calibrated to yield torque. Shear modulus corresponds to the slope of a line through the end points of the hysteresis loop as shown in Fig A.2(b). Using this technique, the shear modulus defined as the ratio of shearing stress, τ , to shearing strain, is calculated from:

$$G = \tau / \gamma \qquad (A.3)$$

Values of shearing strain are presented as single-amplitude values and are calculated at 0.67 times the radius of the specimen, just as in resonant column tests.

TORSIONAL TESTING PROCEDURES

Before testing in either the resonant or torsional shear mode, each specimen was fixed to the base pedestal and top cap using hydrostone paste and allowed to cure overnight. This approach was meant to eliminate any slippage problem that might occur at low confining pressures. A rubber membrane was placed around each specimen to prevent moisture loss or air migration during testing.

Samples 2.8 inches in diameter were tested under similar confining pressures used in M_R tests. At each confining stress, low-amplitude resonant column tests (γ < 0.001 percent) were performed at 10-minute intervals for 1 hour. Upon completion of the low-amplitude resonant column tests, a series of torsional shear tests was also performed, mainly at a loading frequency of 5 Hz, with varying shearing strain amplitudes. To check the effect of frequency on stiffness, loading frequencies of 0.05, 0.1, 0.5, 1, and 5 Hz were used.

High-amplitude resonant column tests were then performed at each confining stress, changing the strain amplitude. Finally, low-amplitude resonant column tests were again performed to determine if any changes had occurred in the low-amplitude modulus from the torsional shear and the high-amplitude resonant column tests. In general, variation in the low amplitude moduli measured before and after high-amplitude testing was less than 5 percent. Thus, it was determined that any changes in the soil skeleton due to high-amplitude testing were negligible.

On some occasions, these samples (particularly those presenting higher stiffness characteristics) were trimmed to 1.5 inches in diameter to facilitate the testing and measuring of the moduli at higher shearing strains (due to the capacity limit of the equipment) of up to 0.1 percent; in this way, the moduli between torsional and M_R testing were easily compared.

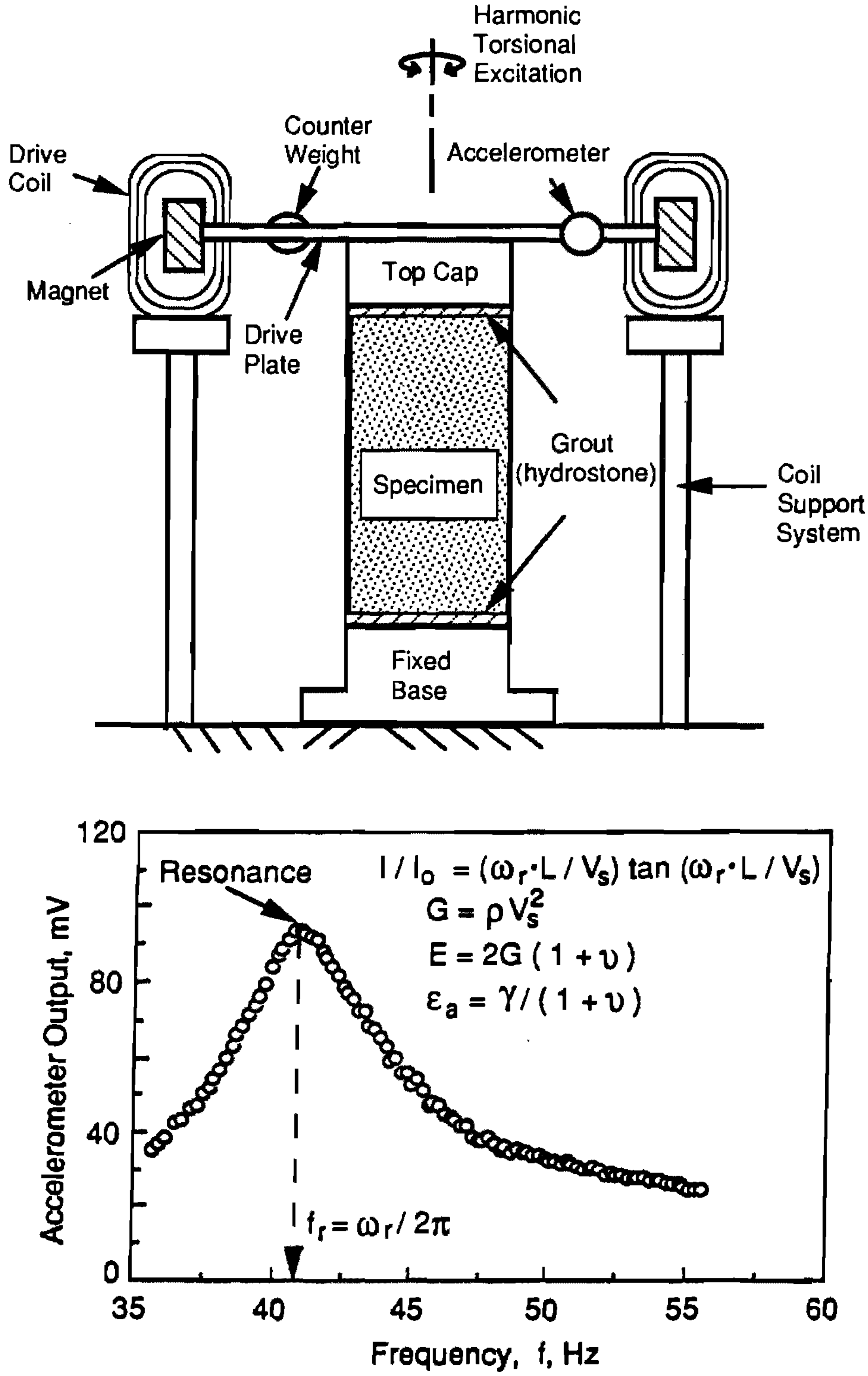

Figure A.1 Configuration of fixed-free resonant column test

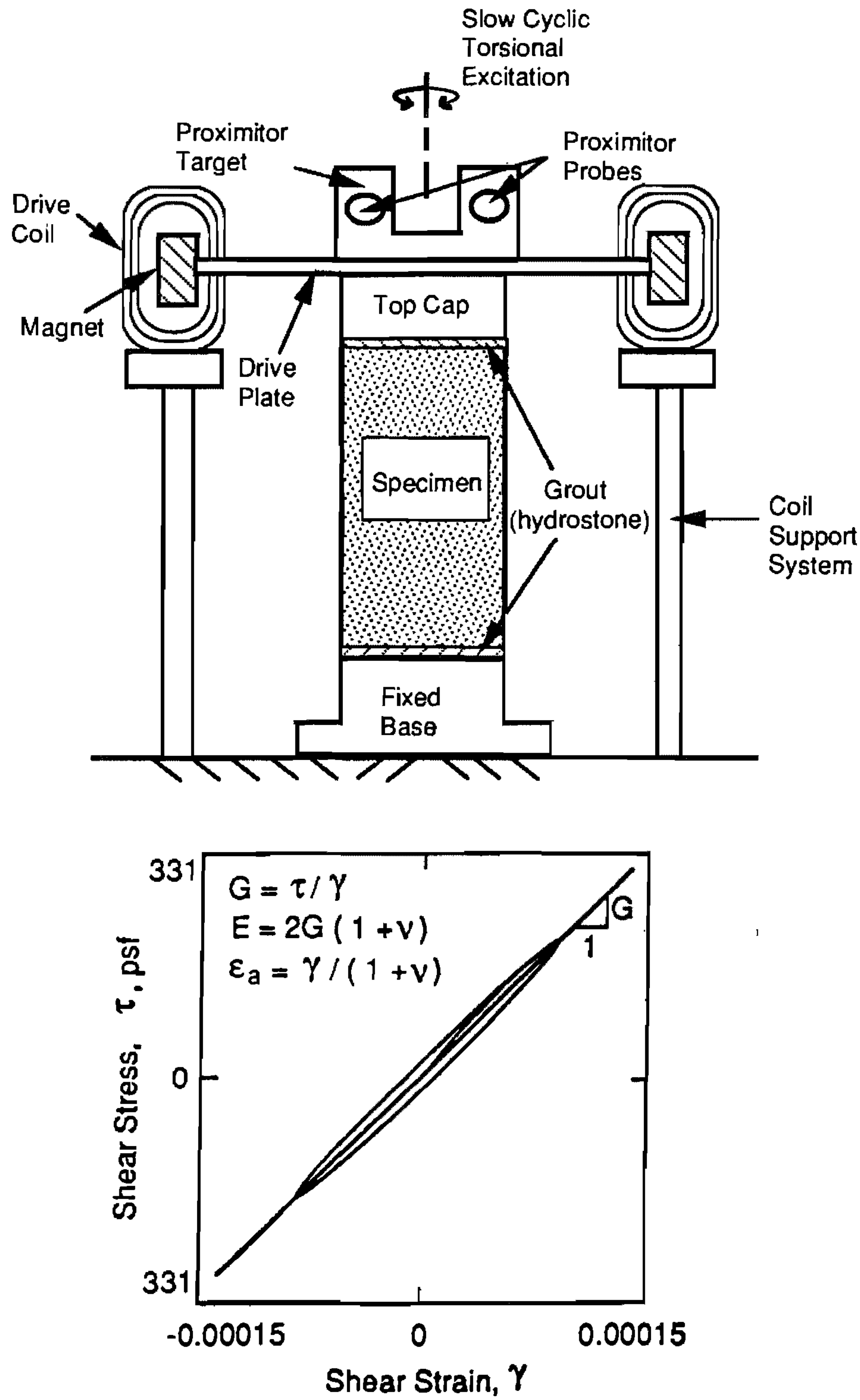

Figure A.2 Configuration of torsional shear test

APPENDIX B. TESTING A SAMPLE UNDER THREE CONDITIONING TYPES

This section shows the testing results of a compacted sample of soil 2. This sample had a moisture content of 39.8 percent (wet of optimum), a dry density of 77 pcf, and was tested 288 days after compaction.

After grouting the sample to the end caps, we first subjected the sample to the conditioning stage specified by our prototype testing procedure, followed by one specified by AASHTO T-274, and finally to the stress states used by Seed et al. (Ref 5) in 1962. The testing results of this particular investigation are presented in Figures B.1, B.2, and B.3, respectively.

Figure B.1(a) shows a 4-psi deviator stress applied 200 times; Figure B.1(b) shows the recorded axial strain induced by the 200 applications of such a deviator stress; Figure B.1(c) illustrates a consistent resilient modulus along those 200 stress repetitions; and Figure B.1(d) shows the increasing permanent deformation induced by such loadings. Similar observations are made in Figures B.2 and B.3, in which the only parameter that really varies throughout the different conditioning stages is the permanent deformation.

These observations demonstrate that none of the conditioning stages used had an effect on the resilient modulus of compacted samples, and that the effect of thixotropy on the resilient deformations is neither cancelled nor destroyed by such conditioning types. Thus, it appears that the conditioning stage is unnecessary and should be eliminated from the M_R testing specifications when grouting is used.

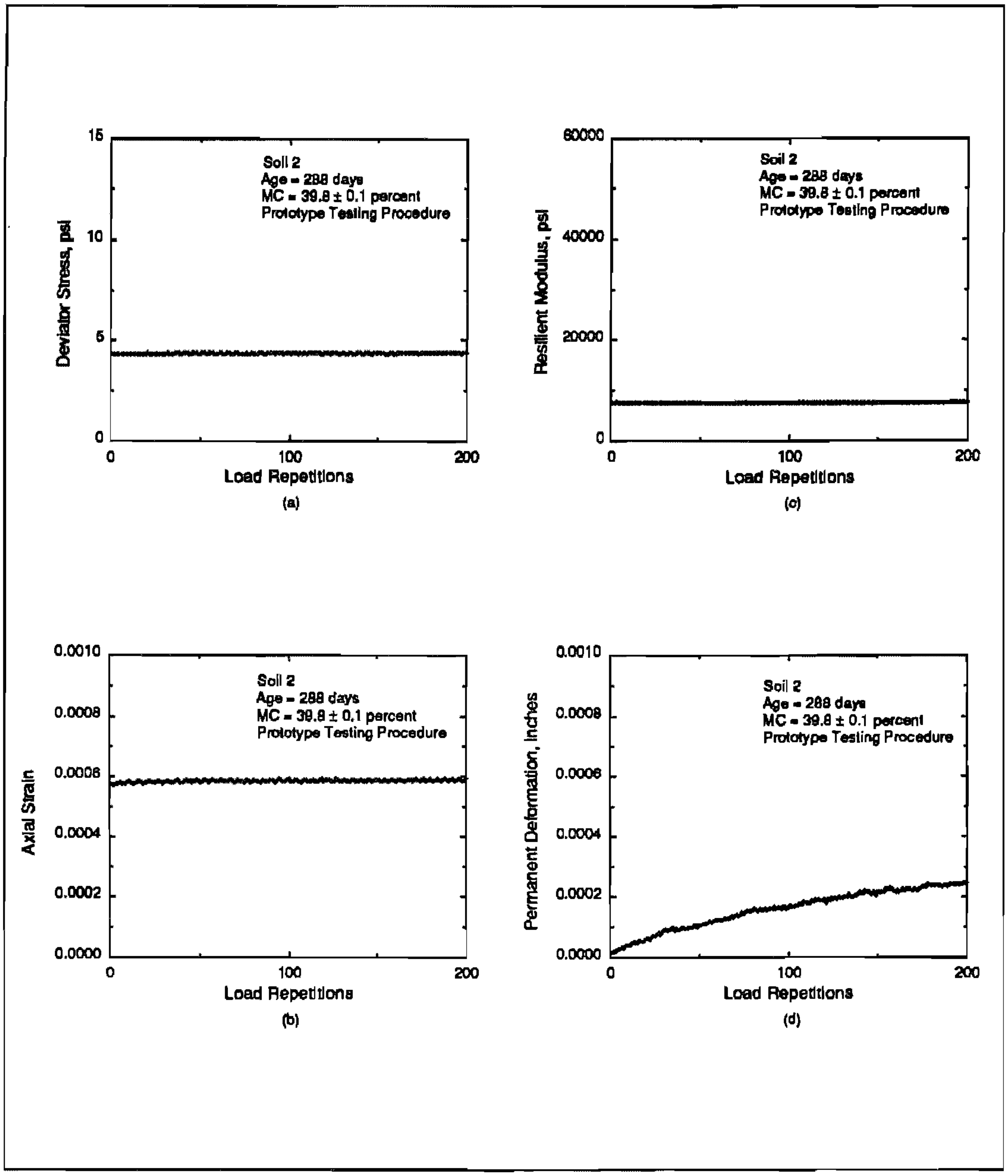

Figure B.1 Deformational characteristics of a compacted sample of soil 2 (288 days) tested under the conditioning stage specified by the prototype testing procedure. Shown are: (a) the applied deviator stress, (b) the induced resilient axial strain, (c) the resilient modulus, and (d) the permanent deformation

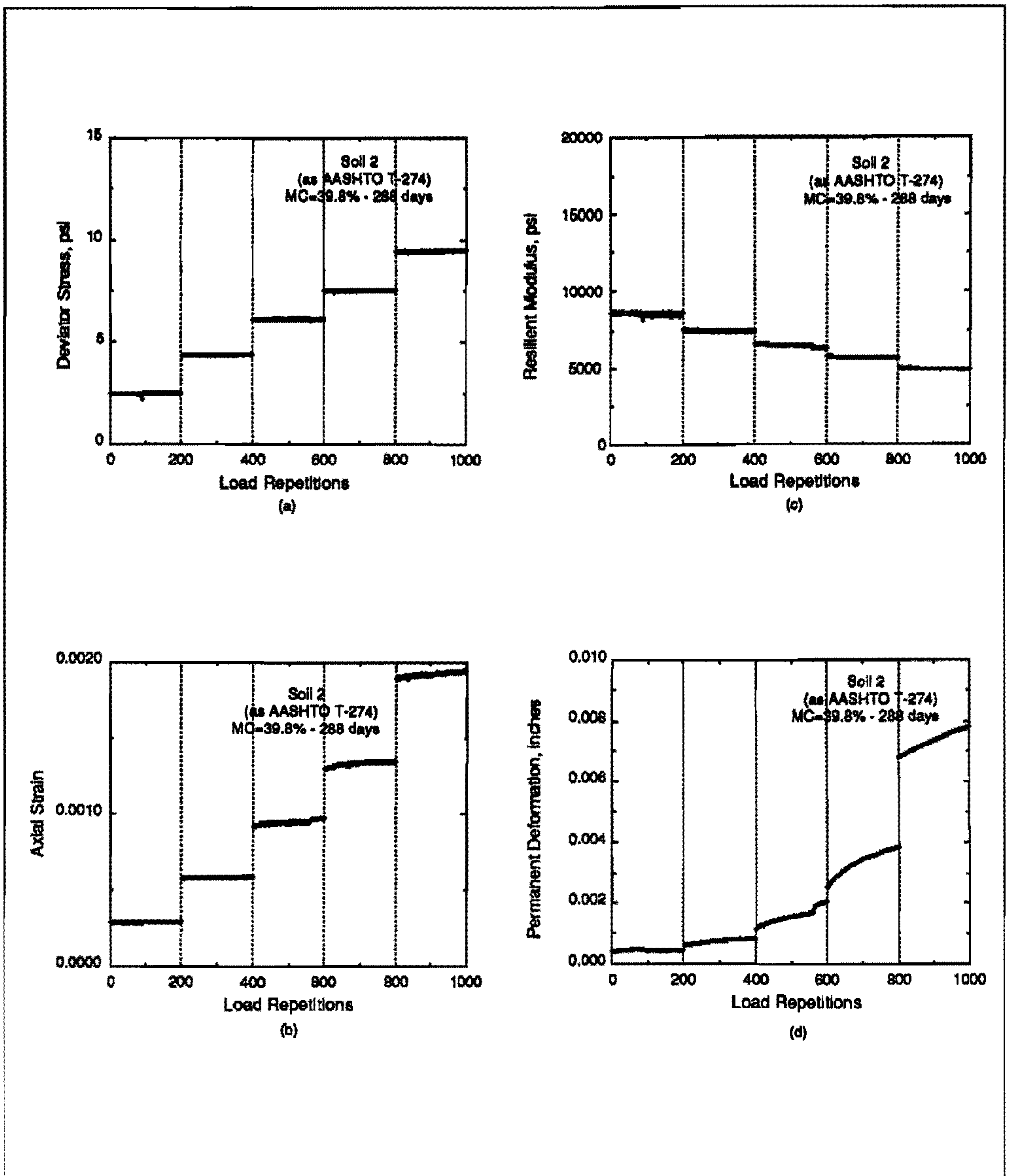

Figure B.2 Deformational characteristics of a compacted sample of soil 2 (288 days) tested under the conditioning stage specified by AASHTO T-274. Shown are: (a) the applied deviator stress, (b) the induced resilient axial strain, (c) the resilient modulus, and (d) the permanent deformation

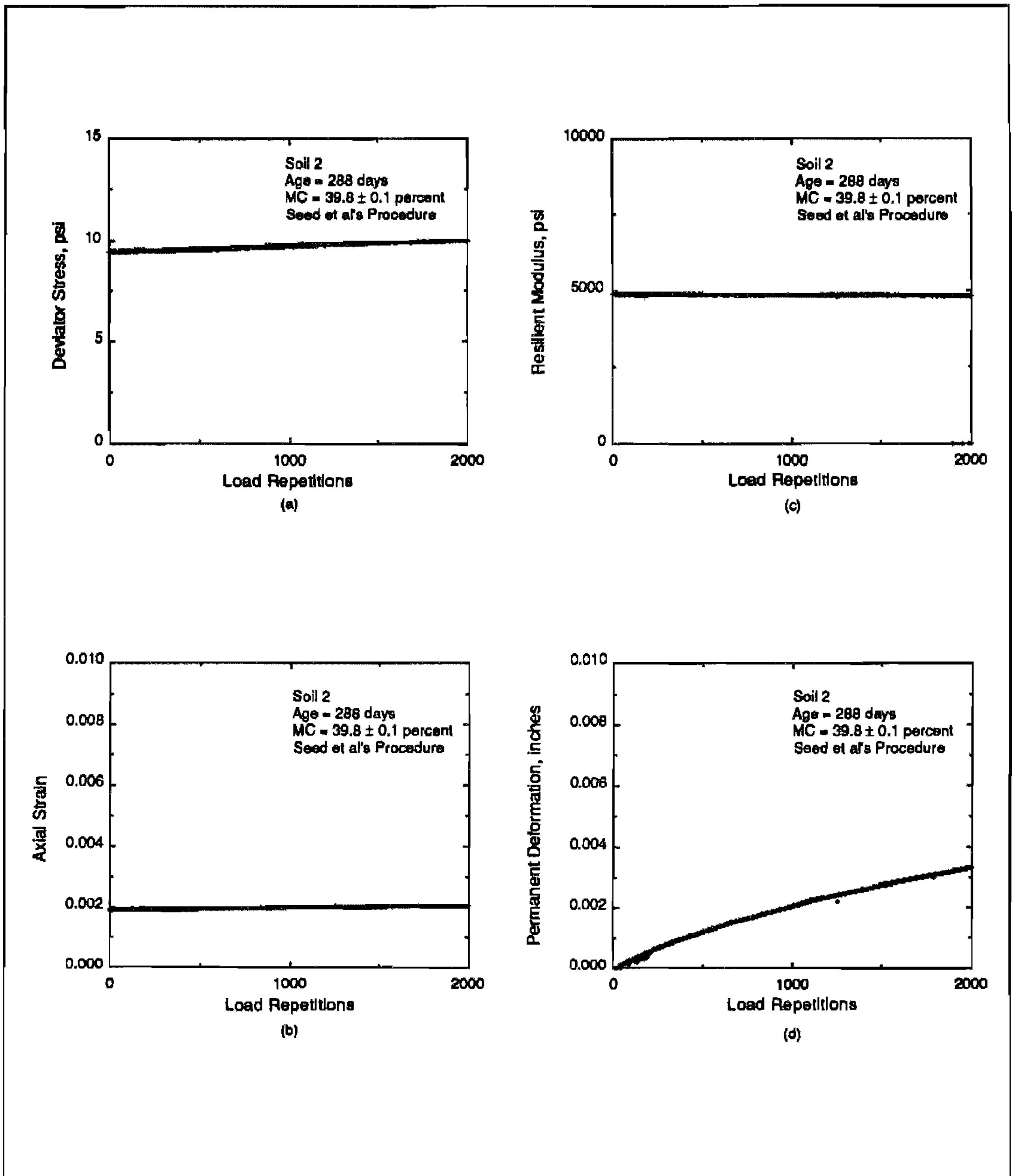

Figure B.3 Deformational characteristics of a compacted sample of soil 2 (288 days) tested under the stress state used by Seed et al. (Ref 5) in 1962. Shown are: (a) the applied deviator stress, (b) the induced resilient axial strain, (c) the resilient modulus, and (d) the permanent deformation

APPENDIX C. EXPERIMENTAL RESULTS

This section includes the results obtained from the resilient modulus testing of compacted samples prepared for the experiment described in Chapter 12, and from the laboratory testing of "undisturbed" samples collected for the case study described in Chapter 10.

It should be emphasized that all these results were obtained from samples that were previously grouted to the end caps in the triaxial chamber before resilient modulus testing. Such grouting sought to ensure strong contacts and to eliminate the probability of movement at these points during the test.

These testing results include the basic information of the material (e.g., its origin and Atterberg limits), as well as information related to the test sample (e.g., the age of the sample at the time of testing, its dimensions, its water content, and its dry density). Also included in tabular form was such testing information as the confining pressures, seating pressures, deviator stresses, axial strains, permanent deformations, and the calculated secant resilient moduli at each of the stress states of the test.

The testing results present two plots. One arithmetic plot shows the variation of the moduli with deviator stress for a given confining stress, while the other shows the semi-logarithmic plot of the variation of the moduli (with logarithmic) of the resilient axial strain for a given confining pressure.

Finally, these results include their testing reports, which consisted of a linear regression equation for predicting the modulus of these materials. Using that model, a unique resilient modulus value is then estimated as an example for use in pavement design. However, it should be recognized that at the time the experiment was set up, the deviator stress was the only regressor variable thought to be important in the moduli prediction models. For this reason, the confining pressure was omitted from such models.

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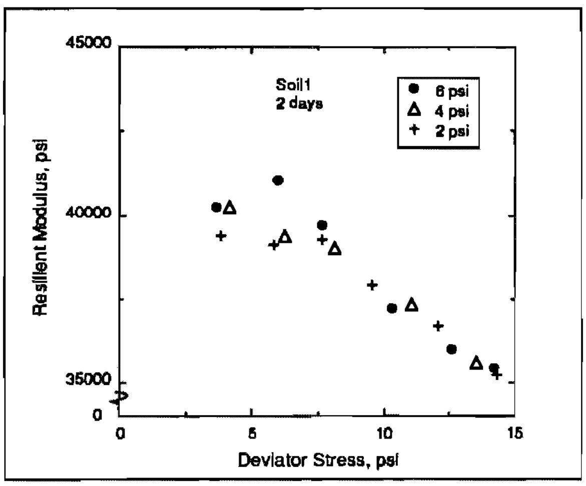
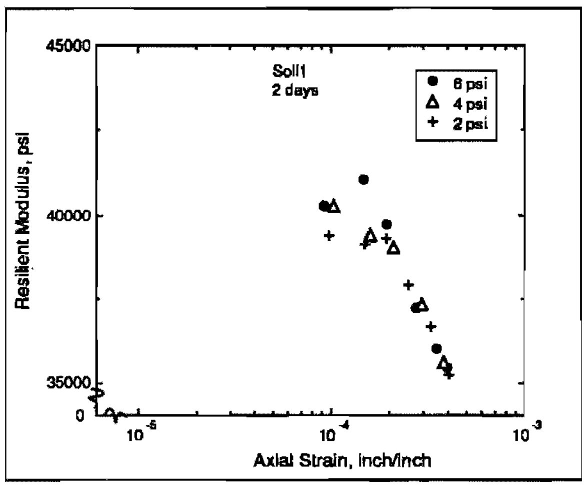
soil-1a.out — RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = Soil 1-opt
DESCRIPTION = Dist 18 - Rockwall - FM550 - 2 days
MOISTURE CONTENT = 21.00 percent
DRY DENSITY = 90.21 pcf.
PLASTICITY INDEX = 55.00 percent
LIQUID LIMIT = 85.00 percent
SAMPLE HEIGHT = 5.630 inches
SAMPLE DIAMETER = 2.840 inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	0.031	3.688494	0.002393323	0.000544	0.000487	0.000092	40264.730
6.000	0.032	5.976307	0.002393807	0.000874	0.000766	0.000146	41032.555
6.000	0.031	7.682685	0.002394440	0.001176	0.001002	0.000193	39717.473
6.000	0.032	10.318327	0.002396105	0.001697	0.001425	0.000277	37218.676
6.000	0.032	12.565413	0.002397944	0.002132	0.001797	0.000349	36015.098
6.000	0.032	14.231531	0.002399781	0.002445	0.002074	0.000401	35457.699
4.000	0.059	4.147274	0.002370030	0.000619	0.000541	0.000103	40257.301
4.000	0.029	6.241275	0.002368497	0.000951	0.000835	0.000159	39357.574
4.000	0.022	8.143806	0.002366729	0.001279	0.001071	0.000209	39013.996
4.000	0.021	11.045819	0.002365927	0.001824	0.001507	0.000296	37345.410
4.000	0.021	13.530722	0.002365966	0.002321	0.001956	0.000380	35620.176
2.000	0.199	3.803188	0.002350900	0.000574	0.000514	0.000097	39384.242
2.000	0.163	5.803562	0.002348939	0.000895	0.000777	0.000149	39079.926
2.000	0.140	7.670982	0.002348200	0.001194	0.001006	0.000195	39262.543
2.000	0.103	9.549638	0.002347625	0.001539	0.001300	0.000252	37875.844
2.000	0.095	12.073393	0.002347522	0.002020	0.001686	0.000329	36686.320
2.000	0.123	14.344352	0.002347617	0.002490	0.002094	0.000407	35235.098





ANALYSIS OF RESULTS		
EXPRESSIONS	STATISTICS	APPLICATION
$MR = MR_{max}$ WHEN $\varepsilon_a \le 0.0001$ (1) $MR = K1 \cdot \sigma_d^{K2}$ or WHEN $\varepsilon_a > 0.0001$ (2) $MR = N1 \cdot \varepsilon_a^{N2}$	MODEL: LOG (ε_a) = A + B * LOG (σ_d) R^2 = 0.998 AND SEE = 0.005 (1) K1 = 49310 AND K2 = -0.12 (2) N1 = 15486 AND N2 = -0.107 MRmax = 41032 psi	SAY σ_d = 6 psi USING Eq. (1): MR = 39770 psi Q: MR < MRmax ? ... No **MR(design) = 39770 psi**

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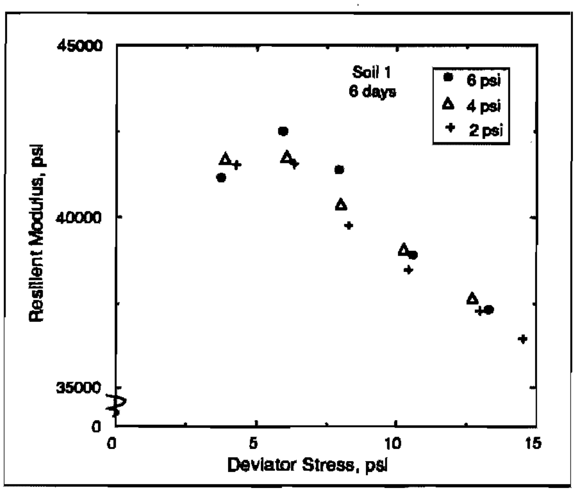
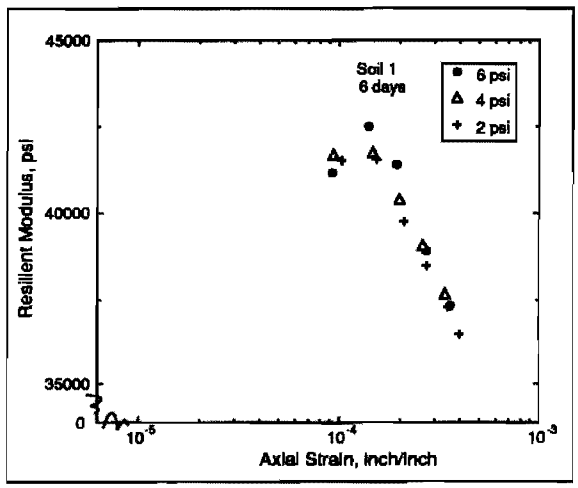
soil-1b-out — RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = Soil 1 - opt
DESCRIPTION = Dist 18 - Rockwall - FM550 - 6 days
MOISTURE CONTENT = 21.00 percent
DRY DENSITY = 90.21 pcf.
PLASTICITY INDEX = 55.00 percent
LIQUID LIMIT = 85.00 percent
SAMPLE HEIGHT = 5.620 Inches
SAMPLE DIAMETER = 2.840 Inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (Inch)	AXIAL DEFORMATION A (Inch)	AXIAL DEFORMATION B (Inch)	STRAIN (in/in)	M r. (psi)
6.000	0.135	3.777910	-.00004904	0.000547	0.000485	0.000092	41157.238
6.000	0.010	5.944005	-.00003769	0.000830	0.000742	0.000140	42512.387
6.000	0.033	7.943908	-.00003234	0.001160	0.000997	0.000192	41386.965
6.000	0.036	10.558954	-.00004407	0.001678	0.001373	0.000271	38897.738
6.000	0.036	13.293372	-.00001084	0.002190	0.001812	0.000356	37339.172
4.000	0.254	3.898222	-.00016983	0.000563	0.000487	0.000093	41708.629
4.000	0.120	6.085383	-.00019733	0.000883	0.000755	0.000146	41756.430
4.000	0.090	8.023026	-.00018410	0.001216	0.001015	0.000199	40416.176
4.000	0.049	10.251382	-.00018216	0.001619	0.001331	0.000262	39069.992
4.000	0.002	12.711473	-.00018527	0.002058	0.001735	0.000337	37676.199
2.000	0.297	4.296612	-.00020765	0.000623	0.000540	0.000103	41548.113
2.000	0.243	6.362992	-.00020590	0.000932	0.000789	0.000153	41556.973
2.000	0.224	8.291271	-.00020234	0.001277	0.001067	0.000209	39754.832
2.000	0.187	10.445195	-.00020101	0.001660	0.001391	0.000271	38479.980
2.000	0.145	12.968950	-.00019831	0.002118	0.001790	0.000348	37299.074
2.000	0.141	14.496497	-.00019104	0.002408	0.002058	0.000397	36479.965





ANALYSIS OF RESULTS		
EXPRESSIONS	STATISTICS	APPLICATION
MR = MRmax WHEN $\varepsilon a \le 0.0001$ (1) $MR = K1 \cdot \sigma d^{K2}$ or WHEN $\varepsilon a > 0.0001$ (2) $MR = N1 \cdot \varepsilon a^{N2}$	MODEL: LOG (εa) = A + B * LOG (σd) R^2 = 0.998 AND SEE = 0.005 (1) K1 = 52343 AND K2 = -0.128 (2) N1 = 15200 AND N2 = -0.114 MRmax = 42512 psi	SAY σd = 6 psi USING Eq. (1): MR = 41615 psi Q: MR < MRmax ? ... No **MR(design) = 41615 psi**

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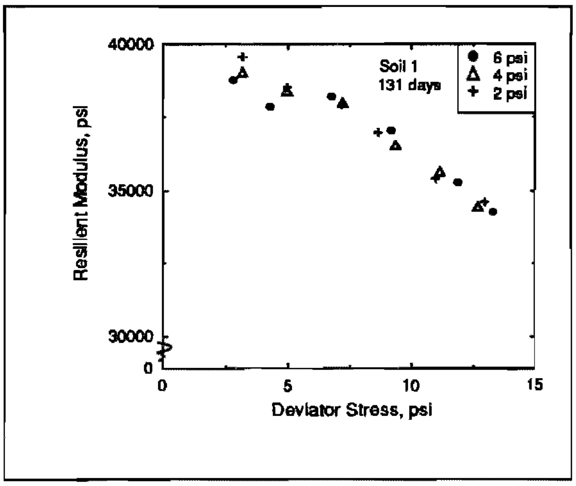
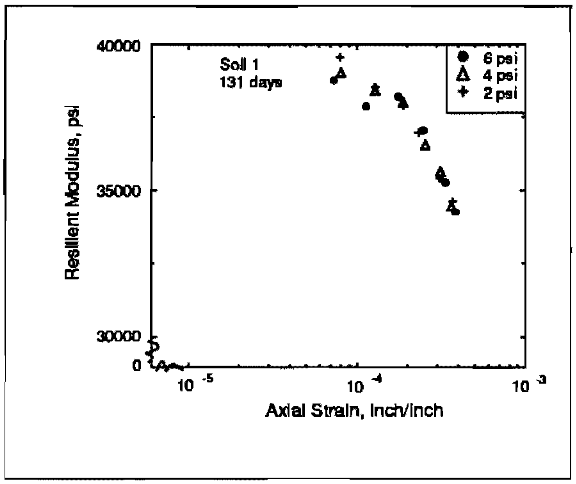
soil-1reg.out RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = Soil 1 - opt - 131 days - grouts
DESCRIPTION = Rockwall
MOISTURE CONTENT = 21.20 percent
DRY DENSITY = 93.62 pcf.
PLASTICITY INDEX = 52.00 percent
LIQUID LIMIT = 73.00 percent
SAMPLE HEIGHT = 5.770 Inches
SAMPLE DIAMETER = 2.840 Inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (Inch)	AXIAL DEFORMATION A (Inch)	B (Inch)	STRAIN (in/in)	M r. (psi)
6.000	0.442	2.836007	-.00005087	0.000391	0.000453	0.000073	38760.090
6.000	0.398	4.308314	-.00005905	0.000613	0.000700	0.000114	37875.480
6.000	0.353	6.760912	-.00006749	0.000992	0.001049	0.000177	38210.410
6.000	0.303	9.187764	-.00005875	0.001458	0.001404	0.000248	37042.691
6.000	0.270	11.861792	-.00002016	0.002042	0.001840	0.000336	35257.105
6.000	0.271	13.303204	0.00004730	0.002450	0.002031	0.000388	34265.313
4.000	0.338	3.162302	-.00002893	0.000436	0.000499	0.000081	39042.105
4.000	0.323	4.966055	-.00002979	0.000705	0.000788	0.000129	38394.094
4.000	0.299	7.211265	-.00003523	0.001083	0.001107	0.000190	37998.516
4.000	0.280	9.353486	-.00002507	0.001522	0.001432	0.000256	36541.805
4.000	0.264	11.148810	-.00001172	0.001908	0.001703	0.000313	35634.648
4.000	0.265	12.678234	0.00000604	0.002273	0.001972	0.000368	34466.422
2.000	0.167	3.176347	-.00004901	0.000427	0.000500	0.000080	39552.852
2.000	0.150	4.958097	-.00006874	0.000708	0.000777	0.000129	38526.871
2.000	0.120	7.185986	-.00007495	0.001083	0.001105	0.000190	37899.641
2.000	0.100	8.662040	-.00008033	0.001368	0.001337	0.000234	36964.223
2.000	0.085	10.986835	-.00007995	0.001874	0.001708	0.000310	35397.887
2.000	0.078	12.943671	-.00004231	0.002328	0.001984	0.000374	34643.145





ANALYSIS OF RESULTS		
EXPRESSIONS	STATISTICS	APPLICATION
$MR = MR_{max}$ WHEN $\varepsilon a \leq 0.0001$ (1) $MR = K1 \cdot \sigma d^{K2}$ or (2) $MR = N1 \cdot \varepsilon a^{N2}$ WHEN $\varepsilon a > 0.0001$	MODEL: LOG (εa) = A + B * LOG (σd) R^2 = 0.998 AND SEE = 0.005 (1) K1 = 45545 AND K2 = -0.103 (2) N1 = 16796 AND N2 = -0.093 MRmax = 39556 psi	SAY σd = 6 psi USING Eq. (1): MR = 37902 psi Q: MR < MRmax ? ... No **MR(design) = 37,902 psi**

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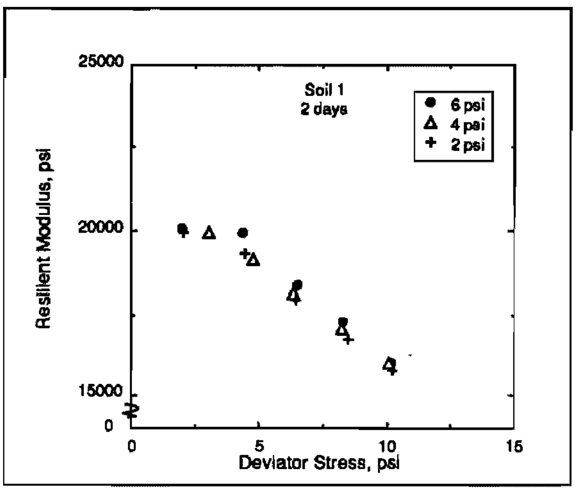
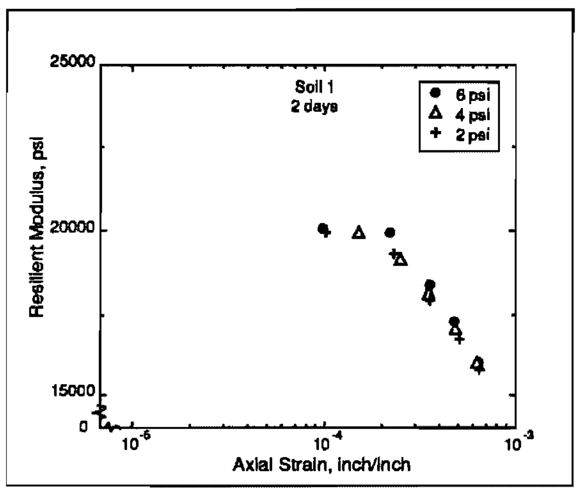
soil-1wa.out — RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = Soil1- wet
DESCRIPTION = Dist 18 - Rockwall - FM550-2 days
MOISTURE CONTENT = 22.37 percent
DRY DENSITY = 83.90 pcf.
PLASTICITY INDEX = 55.00 percent
SAMPLE HEIGHT = 5.610 inches
SAMPLE DIAMETER = 2.810 inches

CONFIN. (psi)	SEATING (psi)	DEV. STRESS (psi)	PERM DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	1.221	1.982104	0.02108869	0.000505	0.000603	0.000099	20069.531
6.000	1.017	4.376410	0.02117746	0.001072	0.001393	0.000220	19920.678
6.000	0.832	6.504841	0.02148174	0.001676	0.002298	0.000354	18367.367
6.000	0.706	8.262197	0.02190501	0.002257	0.003104	0.000478	17292.477
6.000	0.578	10.163486	0.02240719	0.003004	0.004129	0.000636	15985.466
4.000	1.348	3.017868	0.02202779	0.000744	0.000953	0.000151	19951.057
4.000	1.187	4.761354	0.02197952	0.001199	0.001593	0.000249	19137.486
4.000	1.054	6.350864	0.02196768	0.001675	0.002252	0.000350	18145.689
4.000	0.898	8.222983	0.02197091	0.002301	0.003110	0.000482	17052.553
4.000	0.759	10.089368	0.02203560	0.003005	0.004063	0.000630	16015.291
2.000	1.618	2.034227	0.02162095	0.000496	0.000648	0.000102	19939.043
2.000	1.422	4.449574	0.02153825	0.001107	0.001482	0.000231	19284.924
2.000	1.227	6.442198	0.02146340	0.001721	0.002310	0.000359	17934.658
2.000	1.040	8.521376	0.02141506	0.002451	0.003267	0.000510	16720.977
2.000	0.892	10.209870	0.02141645	0.003119	0.004143	0.000647	15775.114





ANALYSIS OF RESULTS		
EXPRESSIONS	STATISTICS	APPLICATION
MR = MRmax WHEN $\varepsilon a \le 0.0001$ (1) $MR = K1 * \sigma d^{K2}$ or WHEN $\varepsilon a > 0.0001$ (2) $MR = N1 * \varepsilon a^{N2}$	MODEL: LOG (εa) = A + B * LOG (σd) R^2 = 0.997 AND SEE = 0.010 (1) K1 = 23985 AND K2 = -0.162 (2) N1 = 5870 AND N2 = -0.140 MRmax = 20069 psi	SAY σd = 6 psi USING Eq. (1): MR = 17936 psi Q: MR < MRmax ? ... No **MR(design) = 17936 psi**

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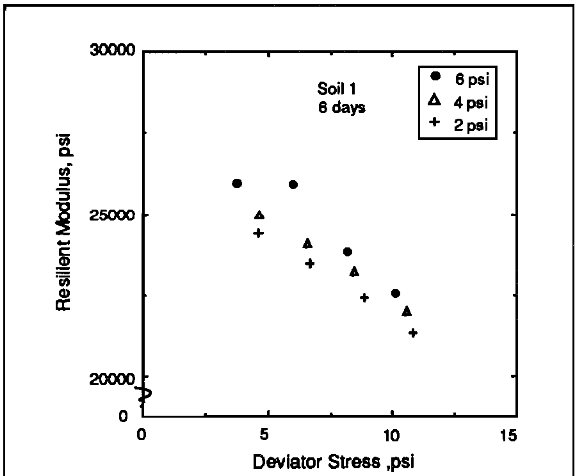
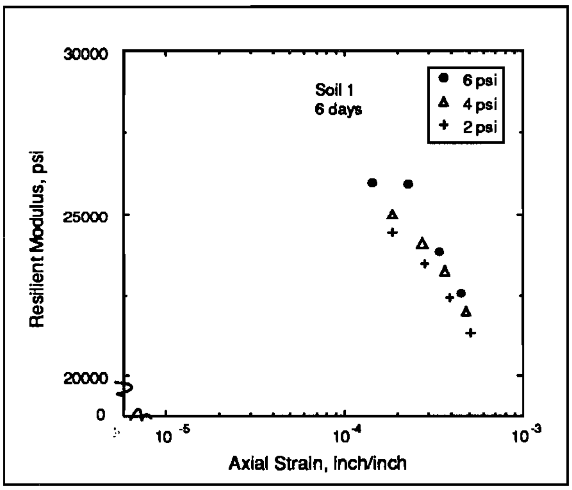
soil-1wb.out RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = Soil 1-wst
DESCRIPTION = Dist.18-Rockwall-FM550-6 days
MOISTURE CONTENT = 22.37 percent
DRY DENSITY = 83.90 pcf.
PLASTICITY INDEX = 55.00 percent
SAMPLE HEIGHT = 5.610 Inches
SAMPLE DIAMETER = 2.810 Inches

CONFIN. (psi)	SEATING (psi)	DEV. STRESS (psi)	PERM DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	1.696	1.907985	0.00042157	0.000410	0.000471	0.000079	24292.949
6.000	1.540	3.739937	0.00041743	0.000700	0.000916	0.000144	25968.781
6.000	1.358	5.995568	0.00044423	0.001072	0.001524	0.000231	25916.803
6.000	1.172	8.172297	0.00048940	0.001564	0.002285	0.000343	23824.488
6.000	1.009	10.098453	0.00056159	0.002023	0.003009	0.000448	22518.273
4.000	1.764	1.869251	0.00034130	0.000389	0.000480	0.000077	24138.768
4.000	1.550	4.660934	0.00028136	0.000866	0.001227	0.000186	24991.814
4.000	1.455	6.587569	0.00026478	0.001248	0.001819	0.000273	24103.068
4.000	1.329	8.470687	0.00024971	0.001654	0.002436	0.000365	23232.324
4.000	1.116	10.567558	0.00022295	0.002192	0.003197	0.000480	22002.209
2.000	1.959	2.282887	-.00013052	0.000462	0.000600	0.000095	24130.955
2.000	1.850	4.590162	-.00015763	0.000868	0.001239	0.000188	24447.381
2.000	1.643	6.670297	-.00022836	0.001305	0.001886	0.000284	23453.939
2.000	1.454	8.870934	-.00034076	0.001815	0.002620	0.000395	22446.207
2.000	1.286	10.816219	-.00037046	0.002342	0.003342	0.000507	21348.730





ANALYSIS OF RESULTS		
EXPRESSIONS	STATISTICS	APPLICATION
MR = MRmax WHEN $\varepsilon a \leq 0.0001$ (1) $MR = K1 * \sigma d^{K2}$ or WHEN $\varepsilon a > 0.0001$ (2) $MR = N1 * \varepsilon a^{N2}$	MODEL: LOG (εa) = A + B * LOG (σd) R^2 = 0.998 AND SEE = 0.004 (1) K1 = 32218 AND K2 = -0.158 (2) N1 = 7804 AND N2 = -0.137 MRmax = 25968 psi	SAY σd = 6 psi USING Eq. (1): MR = 24274 psi Q: MR < MRmax ? ... No **MR(design) = 24274 psi**

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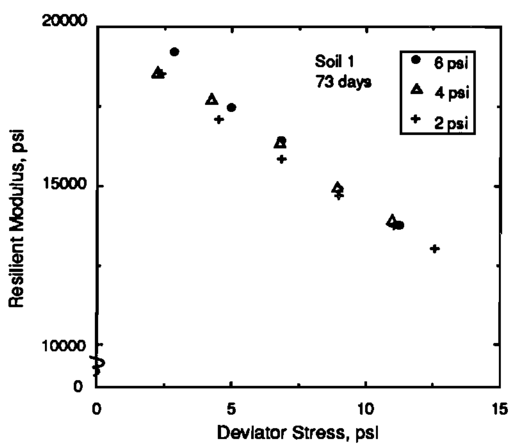
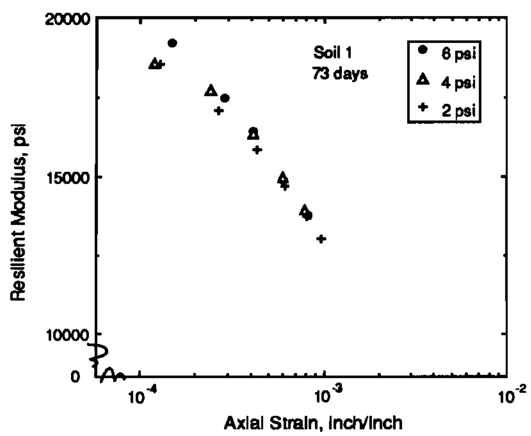
Soil-1wc.out — RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = 1
DESCRIPTION = Dist 18 - Rockwall - FM550 - 73 days
MOISTURE CONTENT = 22.00 percent
DRY DENSITY = 82.79 pcf.
PLASTICITY INDEX = 55.00 percent
LIQUID LIMIT = 85.00 percent
SAMPLE HEIGHT = 5.500 inches
SAMPLE DIAMETEA = 2.810 inches

CONFINE (psi)	SEATING (psi)	DEVIA STAESS (psi)	PEA DEFOAMATION (inch)	AXIAL DEFOAMATION A (inch)	AXIAL DEFOAMATION B (inch)	STAAIN (in/in)	M r. (psi)
6.000	0.362	2.850023	-.00011311	0.000783	0.000849	0.000148	19204.766
6.000	0.106	5.021491	-.00016170	0.001511	0.001653	0.00028B	17455.480
6.000	0.024	6.808972	-.00016469	0.002160	0.002395	0.000414	16444.951
6.000	0.030	8.964660	-.00013559	0.003107	0.003507	D.000601	14909.583
6.000	0.029	11.259024	0.00002087	0.004205	0.004801	0.000819	13752.392
4.000	0.575	2.217375	-.00061436	0.000637	0.000677	0.000120	18554.855
4.000	0.351	4.249690	-.00073240	0.001251	0.001387	0.000240	17718.957
4.000	0.120	6.782671	-.00080006	0.002148	0.002410	0.000414	16368.308
4.000	0.082	8.938360	-.00079446	0.003067	0.003509	0.000598	14952.522
4.000	0.078	11.002715	-.00077360	0.004044	0.004647	0.000790	13926.043
2.000	0.661	2.372787	-.00122451	0.000668	0.000740	0.00012B	18548.518
2.000	0.410	4.513174	-.00136509	0.001382	0.001520	D.000264	17107.480
2.000	0.230	6.829534	-.00146927	0.002243	0.002492	0.000430	15867.818
2.000	0.211	8.988091	-.00144502	0.003153	0.003565	0.000611	14716.401
2.000	0.200	11.024233	-.00144590	0.004127	0.004707	0.000803	13728.510
2.000	0.194	12.523842	-.00144457	0.004926	0.005645	0.000961	13031.458





ANALYSIS OF RESULTS		
EXPRESSIONS	**STATISTICS**	**APPLICATION**
MR = MRmax WHEN $\varepsilon a \le 0.0001$ (1) MR = K1 * σd^{K2} or (2) MR = N1 * εa^{N2} WHEN $\varepsilon a > 0.0001$	MODEL: LOG (εa) = A + B * LOG (σd) R^2 = 0.997 AND SEE = 0.010 (1) K1 = 23195 AND K2 = -0.206 (2) N1 = 4175 AND N2 = -0.171 MRmax = 19205 psi	SAY σd = 6 psi USING Eq. (1): MR = 16036 psi Q: MR < MRmax ? ... No **MR(design) = 16036 psi**

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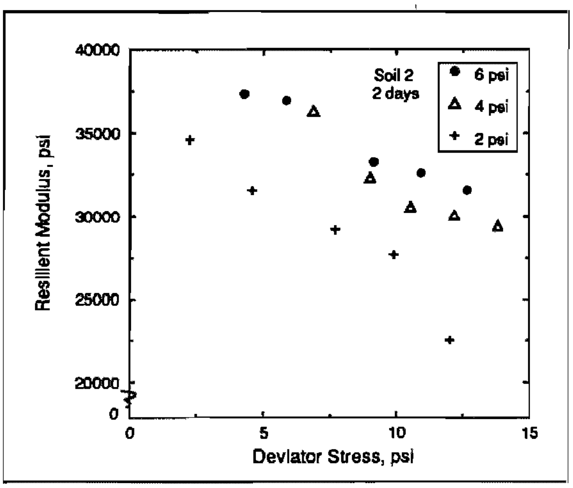
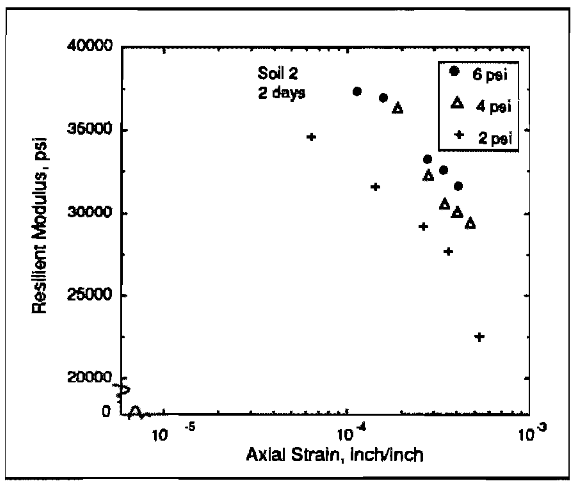
soil-2ar.out — RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = Soil 2 - opt
DESCRIPTION = Dist 14 - Travis - Mopac & 183 - 2 days - rep
MOISTURE CONTENT = 19.30 percent
DRY DENSITY = 90.56 pcf.
PLASTICITY INDEX = 29.00 percent
LIQUID LIMIT = 56.00 percent
SAMPLE HEIGHT = 5.610 inches
SAMPLE DIAMETER = 2.840 inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	0.575	4.264777	0.00004135	0.000380	0.000901	0.000114	37364.477
6.000	0.541	5.836331	0.00009724	0.000504	0.001269	0.000158	36928.105
6.000	0.545	9.152184	0.00009724	0.000935	0.002151	0.000275	33260.375
6.000	0.536	10.955001	0.00030055	0.001120	0.002650	0.000336	32599.709
6.000	0.541	12.697428	0.00049311	0.001327	0.003177	0.000401	31628.307
4.000	0.697	6.891057	0.00000326	0.000457	0.001669	0.000190	36364.293
4.000	0.646	9.004720	-.00006854	0.000876	0.002249	0.000279	32327.619
4.000	0.624	10.524310	-.00009649	0.001168	0.002689	0.000344	30613.484
4.000	0.623	12.199793	-.00006961	0.001376	0.003181	0.000406	30041.084
4.000	0.619	13.838289	-.00001754	0.001592	0.003691	0.000471	29392.264
2.000	1.508	2.216186	0.00641043	0.000164	0.000555	0.000064	34586.398
2.000	1.337	4.534896	0.00612605	0.000435	0.001176	0.000144	31588.422
2.000	1.082	7.691580	0.00600551	0.000873	0.002086	0.000264	29169.320
2.000	0.910	9.889977	0.00585158	0.001237	0.002773	0.000357	27674.303
2.000	0.840	11.987722	0.00435400	0.002079	0.003886	0.000532	22549.207





ANALYSIS OF RESULTS		
EXPRESSIONS	STATISTICS	APPLICATION
MR = MRmax WHEN $\varepsilon a \le 0.0001$ (1) $MR = K1 \cdot \sigma d^{K2}$ or WHEN $\varepsilon a > 0.0001$ (2) $MR = N1 \cdot \varepsilon a^{N2}$	MODEL: LOG (εa) = A + B * LOG (σd) R^2 = 0.948 AND SEE = 0.032 (1) K1 = 48456 AND K2 = -0.202 (2) N1 = 7896 AND N2 = -0.168 MRmax = 37103 psi	SAY σd = 6 psi USING Eq. (1): MR = 33741 psi Q: MR < MRmax ? ... No **MR(design) = 33741 psi**

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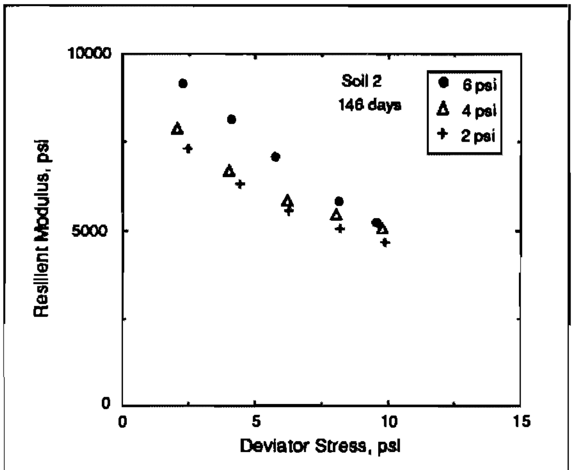
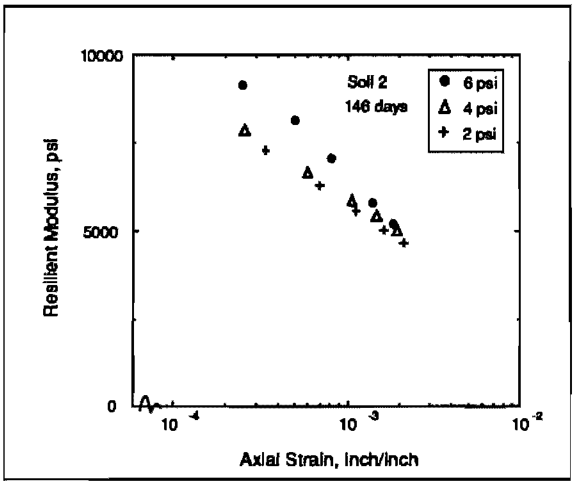
soil-2wc.out RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = Soil2 - very wet
DESCRIPTION = Dist 14 - Travis - Mopac & 183 - 146 days
MOISTURE CONTENT = 39.80 percent
DRY DENSITY = 78.00 pcf.
PLASTICITY INDEX = 27.00 percent
SAMPLE HEIGHT = 5.640 inches
SAMPLE DIAMETER = 2.830 inches

CONFIN. (psi)	SEATING (psi)	DEV. STRESS (psi)	PERM DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	0.631	2.288923	0.01706416	0.001992	0.000839	0.000251	9122.781
6.000	0.462	4.091301	0.01702280	0.003885	0.001794	0.000503	8126.982
6.000	0.345	5.761672	0.01759677	0.006151	0.003050	0.000816	7063.540
6.000	0.254	8.111413	0.01939608	0.010119	0.005613	0.001395	5815.615
6.000	0.216	9.574344	0.02166712	0.012725	0.007992	0.001837	5212.952
4.000	0.968	2.041879	-.00027657	0.001995	0.000928	0.000259	7880.683
4.000	0.708	4.000310	-.00054752	0.004534	0.002221	0.000599	6680.025
4.000	0.559	6.210028	-.00059585	0.007727	0.004228	0.001060	5859.306
4.000	0.388	8.049652	-.00062848	0.010345	0.006341	0.001479	5441.796
4.000	0.318	9.778482	-.00011451	0.013161	0.008723	0.001940	5040.333
2.000	0.502	2.476091	-.00206424	0.002649	0.001176	0.000339	7301.465
2.000	0.314	4.399162	-.00233255	0.005321	0.002552	0.000698	6302.884
2.000	0.289	6.232658	-.00239675	0.008148	0.004519	0.001123	5550.014
2.000	0.261	8.160444	-.00240265	0.011212	0.007039	0.001618	5043.648
2.000	0.264	9.864761	-.00221147	0.014295	0.009589	0.002117	4658.817





ANALYSIS OF RESULTS		
EXPRESSIONS	STATISTICS	APPLICATION
MR = MRmax WHEN $\varepsilon a \leq 0.0001$ (1) $MR = K1 \cdot \sigma d^{K2}$ or (2) $MR = N1 \cdot \varepsilon a^{N2}$ WHEN $\varepsilon a > 0.0001$	MODEL: LOG (εa) = A + B * LOG (σd) R^2 = 0.986 AND SEE = 0.025 (1) K1 = 10924 AND K2 = -0.335 (2) N1 = 1061 AND N2 = -0.251 MRmax = 10708 psi	SAY σd = 6 psi USING Eq. (1): MR = 5994 psi Q: MR < MRmax ? ... No **MR(design) = 5994 psi**

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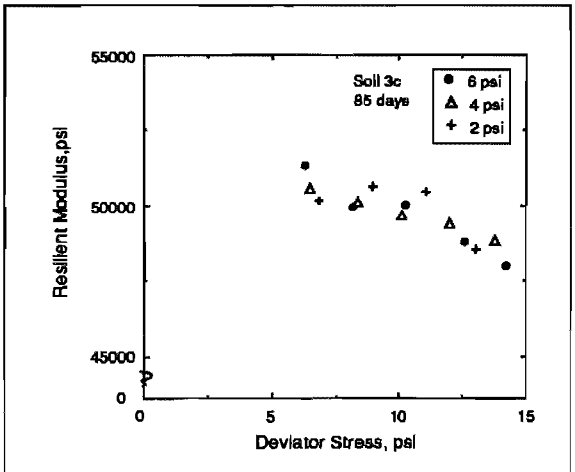
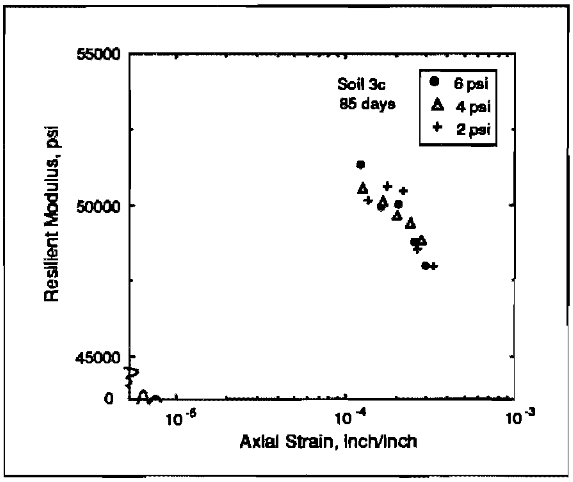
soil-3c.out — RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = Soil 3 - opt
DESCRIPTION = Dist 18 - Denton - SH121 - 85 days
MOISTURE CONTENT = 18.00 percent
DRY DENSITY = 102.85 pcf.
PLASTICITY INDEX = 33.00 percent
LIQUID LIMIT = 50.00 percent
SAMPLE HEIGHT = 5.670 inches
SAMPLE DIAMETER = 2.870 inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	0.023	6.314093	0.00074369	0.000863	0.000533	0.000123	51321.980
6.000	0.027	8.205019	0.00075136	0.001166	0.000696	0.000164	49968.727
6.000	0.026	10.235762	0.00078504	0.001454	0.000866	0.000205	50026.859
6.000	0.026	12.576386	0.00083148	0.001801	0.001123	0.000258	48776.258
6.000	0.027	14.259198	0.00086934	0.002068	0.001303	0.000297	47972.484
4.000	0.061	6.477286	0.00072360	0.000894	0.000558	0.000128	50578.473
4.000	0.036	8.358587	0.00072567	0.001164	0.000727	0.000167	50137.914
4.000	0.020	10.139956	0.00072078	0.001433	0.000882	0.000204	49675.180
4.000	0.008	11.991919	0.00072052	0.001710	0.001042	0.000243	49397.031
4.000	0.020	13.811334	0.00072474	0.001988	0.001220	0.000283	48822.309
2.000	0.157	6.834842	0.00057430	0.000951	0.000594	0.000136	50165.691
2.000	0.148	8.959100	0.00057480	0.001249	0.000757	0.000177	50637.719
2.000	0.097	11.041642	0.00056827	0.001545	0.000937	0.000219	50463.477
2.000	0.122	13.050378	0.00056435	0.001880	0.001170	0.000269	48524.645
2.000	0.119	15.906712	0.00056079	0.002280	0.001482	0.000332	47953.992





ANALYSIS OF RESULTS		
EXPRESSIONS	**STATISTICS**	**APPLICATION**
MR = MRmax WHEN $\varepsilon_a \le 0.0001$ (1) $MR = K1 \cdot \sigma_d^{K2}$ or WHEN $\varepsilon_a > 0.0001$ (2) $MR = N1 \cdot \varepsilon_a^{N2}$	MODEL: LOG (ε_a) = A + B * LOG (σ_d) R^2 = 0.999 AND SEE = 0.003 (1) K1 = 57286 AND K2 = -0.062 (2) N1 = 30177 AND N2 = -0.058 MRmax = 51322 psi	SAY σ_d = 6 psi USING Eq. (1): MR = 51263 psi Q: MR < MRmax ? ... No **MR(design) = 51263 psi**

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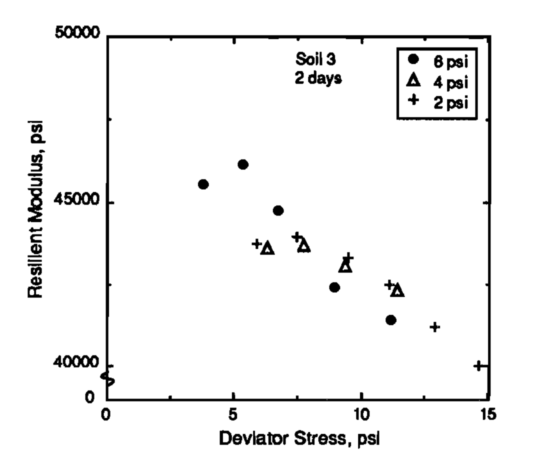
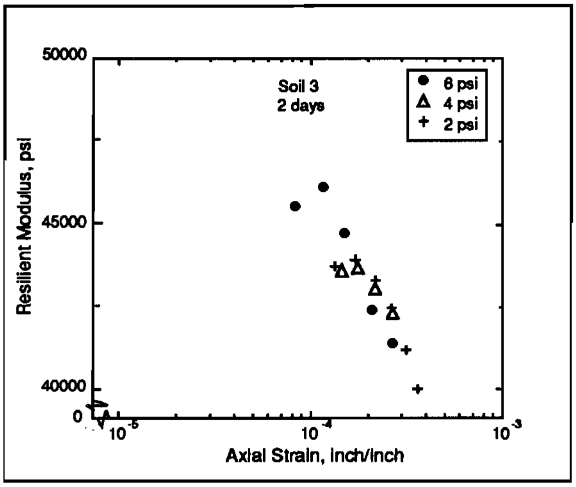
soil-3o.out RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = 3 - opt
DESCRIPTION = Dist 18 - Denton - SH121 - 2 days
MOISTURE CONTENT = 18.40 percent
DRY DENSITY = 101.90 pcf.
PLASTICITY INDEX = 33.00 percent
SAMPLE HEIGHT = 5.640 Inches
SAMPLE DIAMETER = 2.820 Inches

CONFIN. (psi)	SEATING (psi)	DEV. STRESS (psi)	PERM DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	0.039	3.754293	0.00004856	0.000155	0.000775	0.000082	45547.391
6.000	0.030	5.370534	0.00006546	0.000229	0.001084	0.000116	46153.063
6.000	0.016	6.706165	0.00006154	0.000311	0.001380	0.000150	44721.164
6.000	0.057	8.892647	0.00003926	0.000428	0.001937	0.000210	42407.387
6.000	0.060	11.095273	0.00004891	0.000691	0.002332	0.000268	41392.371
4.000	0.112	4.604671	0.00191989	0.000146	0.001060	0.000107	43070.949
4.000	0.102	6.319197	0.00192187	0.000277	0.001357	0.000145	43623.395
4.000	0.096	7.705157	0.00191950	0.000407	0.001583	0.000176	43682.965
4.000	0.092	9.347987	0.00192507	0.000574	0.001874	0.000217	43080.680
4.000	0.092	11.397249	0.00193454	0.000812	0.002225	0.000269	42335.914
2.000	0.325	3.925224	0.00175043	0.000146	0.000903	0.000093	42227.816
2.000	0.308	5.867656	0.00174717	0.000282	0.001232	0.000134	43720.754
2.000	0.297	7.431669	0.00175363	0.000428	0.001480	0.000169	43936.156
2.000	0.289	9.446273	0.00175301	0.000637	0.001822	0.000218	43332.199
2.000	0.275	11.070111	0.00175208	0.000823	0.002117	0.000261	42473.512
2.000	0.252	12.869151	0.00175020	0.001060	0.002462	0.000312	41218.199
2.000	0.218	14.573706	0.00176284	0.001300	0.002809	0.000364	40005.863





ANALYSIS OF RESULTS		
EXPRESSIONS	STATISTICS	APPLICATION
MR = MRmax WHEN $\varepsilon_a \le 0.0001$ (1) $MR = K1 * \sigma_d^{K2}$ or WHEN $\varepsilon_a > 0.0001$ (2) $MR = N1 * \varepsilon_a^{N2}$	MODEL: LOG (ε_a) = A + B * LOG (σ_d) R^2 = 0.997 AND SEE = 0.006 (1) K1 = 51267 AND K2 = -0.083 (2) N1 = 22405 AND N2 = -0.076 MRmax = 45117 psi	SAY σ_d = 6 psi USING Eq. (1): MR = 44182 psi Q: MR < MRmax ? ... No **MR(design) = 44,182 psi**

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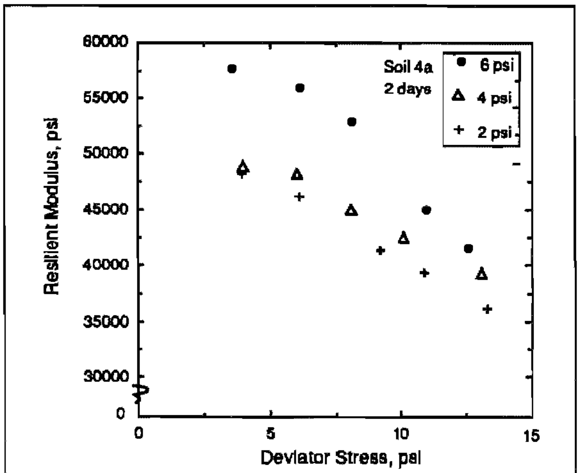
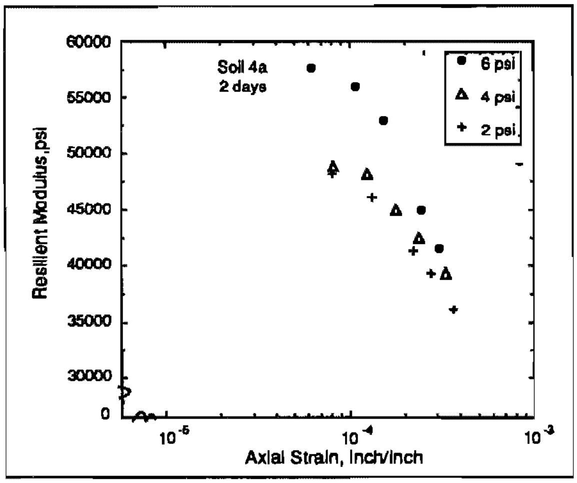
soil-4a.out — RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = Soil4 - opt
DESCRIPTION = Distr 14 - Travis - Mopac & Parmer - 2 days
MOISTURE CONTENT = 10.20 percent
DRY DENSITY = 124.36 pcf.
PLASTICITY INDEX = 6.00 percent
SAMPLE HEIGHT = 5.640 inches
SAMPLE DIAMETER = 2.840 inches

CONFIN. (psi)	SEATING (psi)	DEV. STRESS (psi)	PERM DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	0.060	3.565971	-.00003026	0.000414	0.000286	0.000062	57738.059
6.000	0.061	6.161352	-.00001777	0.000733	0.000512	0.000110	56030.715
6.000	0.061	8.143806	0.00001100	0.001027	0.000708	0.000154	52945.109
6.000	0.061	11.012565	0.00008416	0.001628	0.001136	0.000245	44956.914
6.000	0.061	12.639378	0.00020869	0.002025	0.001410	0.000305	41505.480
4.000	0.061	4.002150	0.00008032	0.000541	0.000382	0.000082	48900.805
4.000	0.061	6.031079	0.00008878	0.000822	0.000589	0.000125	48205.871
4.000	0.061	8.097460	0.00008895	0.001196	0.000836	0.000180	44952.719
4.000	0.061	10.124516	0.00010411	0.001584	0.001106	0.000238	42459.227
4.000	0.058	13.124374	0.00019809	0.002223	0.001546	0.000334	39269.602
2.000	0.148	3.922565	-.00007834	0.000524	0.000393	0.000081	48237.301
2.000	0.114	6.149988	-.00009557	0.000866	0.000635	0.000133	46233.289
2.000	0.139	9.227087	-.00009088	0.001476	0.001044	0.000223	41288.609
2.000	0.098	10.935338	-.00007691	0.001827	0.001312	0.000278	39299.406
2.000	0.107	13.349081	0.00002575	0.002427	0.001730	0.000369	36223.211





ANALYSIS OF RESULTS		
EXPRESSIONS	STATISTICS	APPLICATION
MR = MRmax WHEN $\varepsilon a \leq 0.0001$ (1) MR = K1 $\cdot$ σd^{K2} or (2) MR = N1 $\cdot$ εa^{N2} WHEN $\varepsilon a > 0.0001$	MODEL: LOG (εa) = A + B $\cdot$ LOG (σd) R^2 = 0.978 AND SEE = 0.024 (1) K1 = 73959 AND K2 = -0.237 (2) N1 = 8619 AND N2 = -0.192 MRmax = 50394 psi	SAY σd = 6 psi USING Eq. (1): MR = 48350 psi Q: MR < MRmax ? ... No **MR(design) = 48,350 psi**

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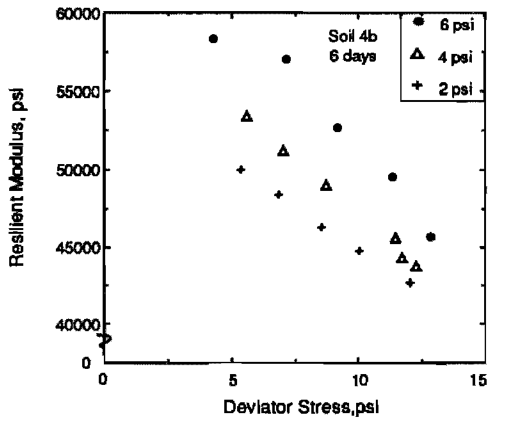
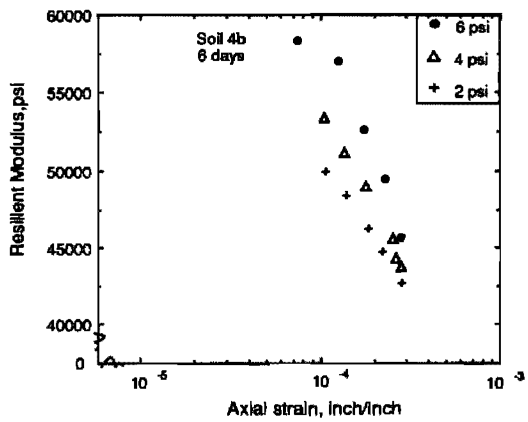
soil-4b.out RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = 4
DESCRIPTION = Dist 14 - Travis - Mopac & Parmer - 6 days
MOISTURE CONTENT = 10.20 percent
DRY DENSITY = 124.36 pcf.
PLASTICITY INDEX = 6.00 percent
SAMPLE HEIGHT = 5.640 inches
SAMPLE DIAMETER = 2.840 inches

CONFIN. (psi)	SEATING (psi)	DEV. STRESS (psi)	PEAK DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	0.060	4.293334	.00003641	0.000472	0.000358	0.000074	58350.238
6.000	0.061	7.172878	.00010923	0.000811	0.000609	0.000126	56984.801
6.000	0.061	9.199936	.00018127	0.001127	0.000844	0.000175	52641.605
6.000	0.061	11.351987	.00029090	0.001497	0.001093	0.000230	49449.727
6.000	0.061	12.841615	.00041359	0.001831	0.001341	0.000281	45652.922
4.000	0.018	5.583535	.00026290	0.000663	0.000517	0.000105	53378.531
4.000	0.016	7.035714	.00026540	0.000878	0.000671	0.000137	51220.641
4.000	0.027	8.753327	.00026932	0.001149	0.000866	0.000179	48988.074
4.000	0.040	11.471362	.00025241	0.001633	0.001204	0.000252	45609.762
4.000	0.023	11.714327	.00021610	0.001722	0.001258	0.000264	44341.668
4.000	0.015	12.289208	.00019925	0.001821	0.001343	0.000281	43800.898
2.000	0.141	5.374744	.00037846	0.000687	0.000526	0.000107	50007.770
2.000	0.130	6.845180	.00038232	0.000910	0.000685	0.000141	48420.176
2.000	0.123	8.572625	.00038189	0.001199	0.000892	0.000185	46254.977
2.000	0.114	10.049613	.00038047	0.001460	0.001073	0.000225	44757.703
2.000	0.108	12.044370	.00037308	0.001828	0.001353	0.000282	42704.027





ANALYSIS OF RESULTS		
EXPRESSIONS	**STATISTICS**	**APPLICATION**
$MR = MR_{max}$ WHEN $\varepsilon_a \leq 0.0001$ (1) $MR = K1 * \sigma_d^{K2}$ or (2) $MR = N1 * \varepsilon_a^{N2}$ WHEN $\varepsilon_a > 0.0001$	MODEL: $LOG(\varepsilon_a) = A + B * LOG(\sigma_d)$ R^2 = 0.980 AND SEE = 0.017 (1) K1 = 77018 AND K2 = -0.212 (2) N1 = 10745 AND N2 = -0.175 MR_{max} = 53876 psi	SAY σ_d = 6 psi USING Eq. (1): MR = 52660 psi Q: MR < MRmax ? ... No **MR(design) = 52,660 psi**

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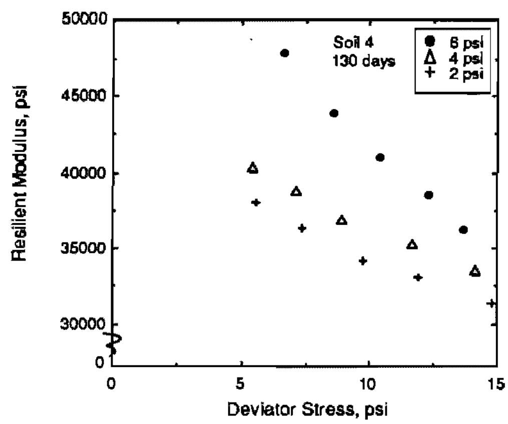
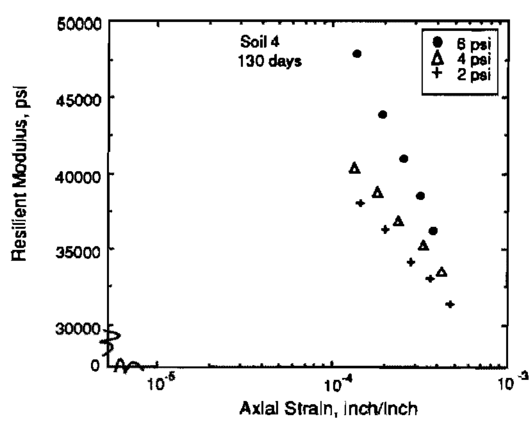
soil-4mc.out — RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = 4-wet
DESCRIPTION = Dist 14 - Travis - Mopac & Parmer - 188 days
MOISTURE CONTENT = 14.1 percent
DRY DENSITY = 122 pcf.
PLASTICITY INDEX = 6.00 percent
SAMPLE HEIGHT = 5.640 inches
SAMPLE DIAMETER = 2.820 inches

CONFIN. (psi)	SEATING (psi)	DEV. STRESS (psi)	PERM DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	-0.057	6.669604	-.00005090	0.000529	0.001043	0.000139	47852.418
6.000	-0.057	8.617259	-.00005020	0.000789	0.001426	0.000196	43899.316
6.000	-0.057	10.450962	-.00004148	0.001065	0.001809	0.000255	41017.961
6.000	-0.057	12.333570	0.00000558	0.001383	0.002226	0.000320	38549.324
6.000	-0.057	13.649731	0.00009222	0.001680	0.002573	0.000377	36199.980
4.000	-0.042	5.417066	-.00010593	0.000529	0.000984	0.000134	40391.898
4.000	-0.051	7.111175	-.00011794	0.000749	0.001319	0.000183	38786.582
4.000	-0.054	8.893597	-.00012519	0.001026	0.001694	0.000241	36882.527
4.000	-0.053	11.672163	-.00010795	0.001458	0.002273	0.000331	35283.711
4.000	-0.054	14.170596	-.00002927	0.001909	0.002858	0.000423	33531.934
2.000	0.146	5.537192	-.00015005	0.000631	0.001009	0.000145	38097.227
2.000	0.139	7.362347	-.00015611	0.000911	0.001377	0.000203	36300.035
2.000	0.127	9.755371	-.00015348	0.001312	0.001907	0.000285	34190.289
2.000	0.124	11.929507	-.00013717	0.001675	0.002393	0.000361	33082.898
2.000	0.131	14.764577	-.00001865	0.002235	0.003072	0.000470	31383.604





<table>
<tr><th colspan="3">ANALYSIS OF RESULTS</th></tr>
<tr><th>EXPRESSIONS</th><th>STATISTICS</th><th>APPLICATION</th></tr>
<tr>
<td>MR = MRmax WHEN $\varepsilon_a \le 0.0001$
(1) $MR = K1 \cdot \sigma_d^{K2}$
or WHEN $\varepsilon_a > 0.0001$
(2) $MR = N1 \cdot \varepsilon_a^{N2}$</td>
<td>MODEL: $LOG(\varepsilon_a) = A + B \cdot LOG(\sigma_d)$
R^2 = 0.997 AND SEE = 0.008
(1) K1 = 55953 AND K2 = -0.087
(2) N1 = 23316 AND N2 = -0.080
MRmax = 48747 psi</td>
<td>SAY σ_d = 6 psi
USING Eq. (1): MR = 47873 psi
Q: MR < MRmax ? ... No
MR(design) = 47,873 psi</td>
</tr>
</table>

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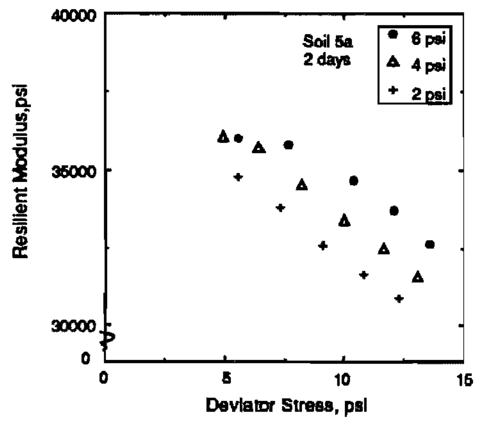
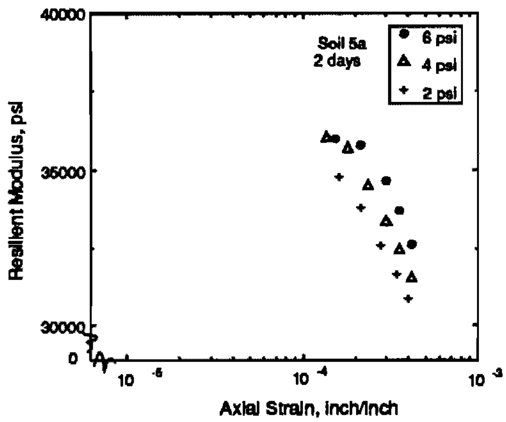
soil-5a.out — RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = 5-opt
DESCRIPTION = Dist 21 - Starr - FM755 - 2 days
MOISTURE CONTENT = 10.60 percent
DRY DENSITY = 124.02 pcf.
PLASTICITY INDEX = 9.50 percent
SAMPLE HEIGHT = 5.640 inches
SAMPLE DIAMETER = 2.820 inches

CONFIN. (psi)	SEATING (psi)	DEV. STRESS (psi)	PERM DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	0.046	5.556183	0.00060483	0.001425	0.000317	0.000154	35985.512
6.000	0.049	7.650079	0.00065606	0.001836	0.000575	0.000214	35789.414
6.000	0.050	10.368821	0.00069416	0.002397	0.000977	0.000299	34662.352
6.000	0.050	12.108036	0.00076158	0.002779	0.001273	0.000359	33704.309
6.000	0.050	13.552399	0.00086438	0.003141	0.001541	0.000415	32648.197
4.000	0.011	4.941309	0.00055306	0.001270	0.000275	0.000137	36065.207
4.000	0.008	6.402288	0.00053150	0.001569	0.000451	0.000179	35737.527
4.000	0.005	8.188511	0.00053081	0.001963	0.000712	0.000237	34538.051
4.000	0.014	9.964760	0.00054873	0.002360	0.001007	0.000298	33383.449
4.000	0.008	11.645100	0.00054229	0.002740	0.001304	0.000358	32483.945
4.000	0.006	13.052425	0.00056395	0.003077	0.001503	0.000413	31594.023
2.000	0.182	5.557608	0.00027666	0.001388	0.000414	0.000160	34794.934
2.000	0.171	7.277831	0.00025487	0.001763	0.000665	0.000215	33805.570
2.000	0.151	9.077822	0.00023381	0.002181	0.000961	0.000279	32587.551
2.000	0.146	10.777154	0.00022491	0.002580	0.001257	0.000340	31682.359
2.000	0.148	12.259974	0.00022765	0.002936	0.001546	0.000397	30852.799
2.000	0.143	13.704811	0.00024894	0.003319	0.001844	0.000458	29939.533





ANALYSIS OF RESULTS

EXPRESSIONS	STATISTICS	APPLICATION
MR = MRmax WHEN $\varepsilon a \leq 0.0001$ (1) $MR = K1 \cdot \sigma d^{K2}$ or WHEN $\varepsilon a > 0.0001$ (2) $MR = N1 \cdot \varepsilon a^{N2}$	MODEL: LOG (εa) = A + B * LOG (σd) R^2 = 0.992 AND SEE = 0.009 (1) K1 = 44415 AND K2 = -0.126 (2) N1 = 13421 AND N2 = -0.1118 MRmax = 37594 psi	SAY σd = 6 psi USING Eq. (1): MR = 35445 psi Q: MR < MRmax ? ... No **MR(design) = 35,445 psi**

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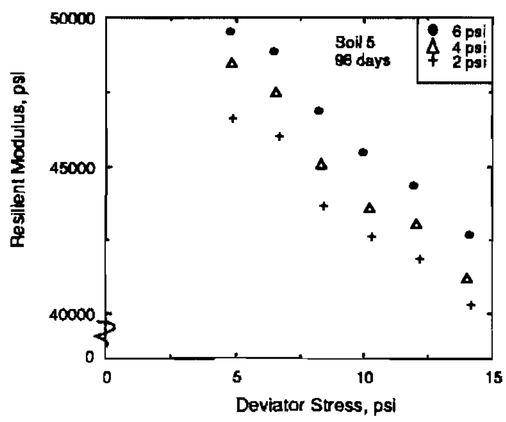
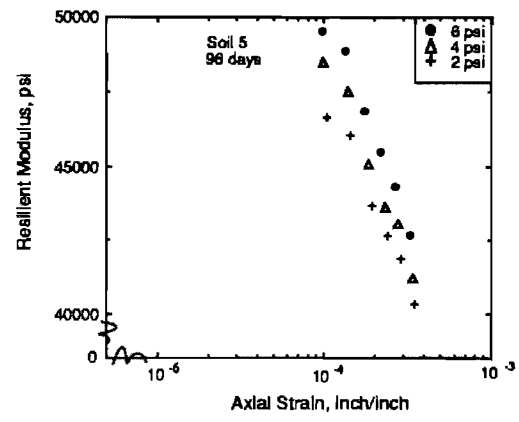
soil-5c1.out

RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = 5 - 96 days
DESCRIPTION = Dist 21 - Starr - FM755 - opt - 1/9
MOISTURE CONTENT = 10.40 percent
DRY DENSITY = 120.74 pcf.
PLASTICITY INDEX = 9.50 percent
LIQUID LIMIT = 25.00 percent
SAMPLE HEIGHT = 5.640 inches
SAMPLE DIAMETER = 2.855 inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	0.156	4.789862	0.00007345	0.000435	0.000656	0.000097	49522.973
6.000	0.132	6.482990	0.00010405	0.000607	0.000889	0.000133	48877.254
6.000	0.104	8.216419	0.00013092	0.000803	0.001174	0.000175	46872.293
6.000	0.050	9.971157	0.00015829	0.000999	0.001472	0.000219	45511.625
6.000	-0.020	11.941765	0.00019583	0.001242	0.001795	0.000269	44351.797
6.000	-0.025	14.111565	0.00029420	0.001544	0.002187	0.000331	42669.168
4.000	0.370	4.819045	0.00009202	0.000444	0.000677	0.000099	48502.035
4.000	0.350	6.593239	0.00008440	0.000631	0.000936	0.000139	47485.082
4.000	0.310	8.335935	0.00008136	0.000841	0.001243	0.000185	45104.488
4.000	0.224	10.187490	0.00007107	0.001070	0.001566	0.000234	43595.117
4.000	0.143	12.038582	0.00006679	0.001285	0.001868	0.000280	43068.820
4.000	0.086	14.028643	0.00009521	0.001585	0.002256	0.000341	41195.496
2.000	0.538	4.859347	-.00004964	0.000464	0.000712	0.000104	46627.141
2.000	0.510	6.659483	-.00006576	0.000658	0.000974	0.000145	46036.941
2.000	0.462	8.409126	-.00008170	0.000874	0.001299	0.000193	43659.109
2.000	0.365	10.300056	-.00009931	0.001103	0.001623	0.000242	42620.949
2.000	0.278	12.197933	-.00010495	0.001343	0.001943	0.000291	41866.410
2.000	0.226	14.156959	-.00008863	0.001629	0.002334	0.000351	40293.656





ANALYSIS OF RESULTS		
EXPRESSIONS	**STATISTICS**	**APPLICATION**
MR = MRmax WHEN $\varepsilon a \leq 0.0001$ (1) $MR = K1 * \sigma d^{K2}$ or WHEN $\varepsilon a > 0.0001$ (2) $MR = N1 * \varepsilon a^{N2}$	MODEL: LOG (εa) = A + B * LOG (σd) R^2 = 0.973 AND SEE = 0.016 (1) K1 = 65825 AND K2 = -0.138 (2) N1 = 17149 AND N2 = -0.121 MRmax = 52381 psi	SAY σd = 6 psi USING Eq. (1): MR = 51409 psi Q: MR < MRmax ? ... No **MR(design) = 51,409 psi**

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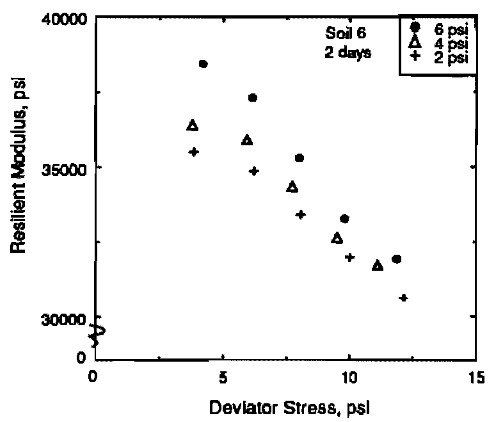
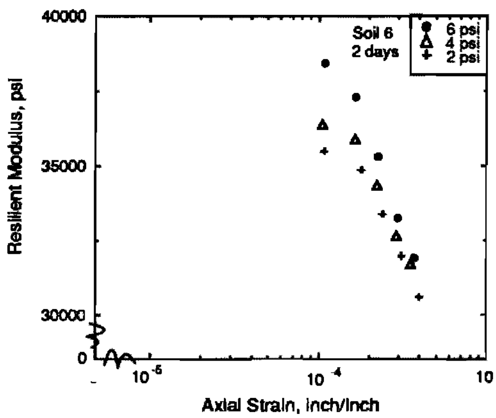
soil-6ra.out — RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = Soil 6 - opt - replicate
DESCRIPTION = Dist 5 - Hockley - US62 - 2 days
MOISTURE CONTENT = 11.60 percent
DRY DENSITY = 120.61 pcf.
PLASTICITY INDEX = 15.00 percent
LIQUID LIMIT = 30.00 percent
SAMPLE HEIGHT = 5.710 inches
SAMPLE DIAMETER = 2.840 inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	-0.009	4.186598	-.00006168	0.000529	0.000715	0.000109	38448.344
6.000	-0.009	6.200547	-.00004760	0.000857	0.001042	0.000166	37299.586
6.000	-0.011	8.007109	-.00002838	0.001210	0.001380	0.000227	35307.371
6.000	-0.014	9.792603	-.00003811	0.001594	0.001766	0.000294	33281.465
6.000	-0.013	11.879113	0.00004325	0.002042	0.002205	0.000372	31943.820
4.000	0.126	3.806465	-.00014906	0.000523	0.000671	0.000105	36396.387
4.000	0.083	5.961326	-.00016887	0.000877	0.001020	0.000166	35891.250
4.000	0.073	7.745416	-.00016992	0.001216	0.001356	0.000225	34379.781
4.000	0.067	9.509377	-.00016530	0.001603	0.001723	0.000291	32655.176
4.000	0.064	11.109956	-.00016970	0.001943	0.002055	0.000350	31728.877
2.000	0.306	3.843450	-.00036223	0.000569	0.000667	0.000108	35502.574
2.000	0.265	6.242211	-.00037153	0.000981	0.001063	0.000179	34866.707
2.000	0.243	8.045964	-.00037170	0.001342	0.001409	0.000241	33403.738
2.000	0.222	9.993905	-.00036343	0.001755	0.001813	0.000312	31987.176
2.000	0.211	12.131911	-.00034601	0.002238	0.002283	0.000396	30644.773





ANALYSIS OF RESULTS		
EXPRESSIONS	STATISTICS	APPLICATION
MR = MRmax WHEN $\varepsilon a \leq 0.0001$ (1) MR = K1 * σd^{K2} or WHEN $\varepsilon a > 0.0001$ (2) MR = N1 * εa^{N2}	MODEL: LOG (εa) = A + B * LOG (σd) R^2 = 0.975 AND SEE = 0.020 (1) K1 = 42812 AND K2 = -0.062 (2) N1 = 22891 AND N2 = -0.059 MRmax = 39308 psi	SAY σd = 6 psi USING Eq. (1): MR = 38285 psi Q: MR < MRmax ? ... No **MR(design) = 38,285 psi**

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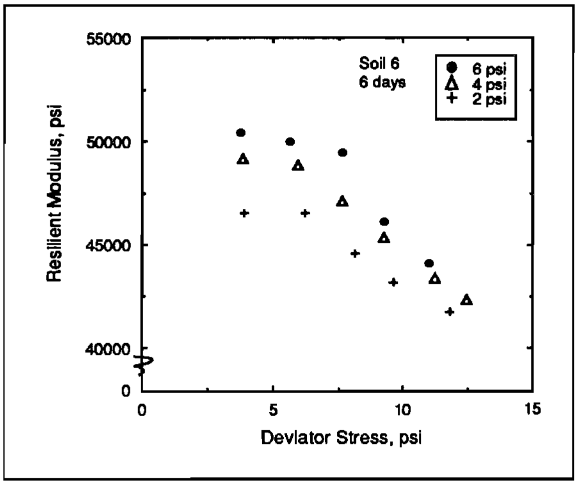
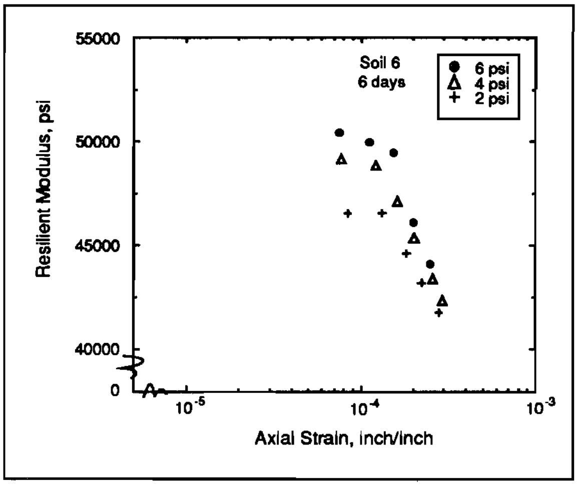
soil-6rb.out — RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = soil 6 - opt - replicote
DESCRIPTION = Dist 5 - Hockley - US62 - 6 doys
MOISTURE CONTENT = 11.60 percent
DRY DENSITY = 120.61 pcf.
PLASTICITY INDEX = 15.00 percent
LIQUID LIMIT = 30.00 percent
SAMPLE HEIGHT = 5.700 inches
SAMPLE DIAMETER = 2.840 inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PEA DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	-0.015	3.756375	0.00000883	0.000349	0.000500	0.000075	50412.559
6.000	-0.017	5.647670	0.00007981	0.000561	0.000728	0.000113	49953.820
6.000	-0.017	7.666301	0.00020070	0.000809	0.000959	0.000155	49429.676
6.000	-0.017	9.285137	0.00031151	0.001069	0.001228	0.000201	46097.098
6.000	-0.019	11.033180	0.00045322	0.001350	0.001503	0.000250	44091.918
4.000	0.140	3.831278	0.00020479	0.000390	0.000499	0.000078	49141.965
4.000	0.114	5.953367	0.00020311	0.000646	0.000744	0.000122	48832.102
4.000	0.093	7.655065	0.00020453	0.000878	0.000973	0.000162	47136.355
4.000	0.083	9.258453	0.00020888	0.001116	0.001213	0.000204	45329.027
4.000	0.071	11.253207	0.00022992	0.001434	0.001524	0.000259	43377.641
4.000	0.066	12.449780	0.00026883	0.001634	0.001719	0.000294	42328.844
2.000	0.278	3.906181	0.00002539	0.000439	0.000518	0.000084	46536.871
2.000	0.255	6.203825	0.00000587	0.000727	0.000792	0.000133	46559.422
2.000	0.234	8.138187	-.00000688	0.001011	0.001069	0.000183	44588.051
2.000	0.216	9.641863	-.00000321	0.001251	0.001292	0.000223	43220.277
2.000	0.198	11.807957	0.00000576	0.001600	0.001624	0.000283	41762.879





ANALYSIS OF RESULTS		
EXPRESSIONS	STATISTICS	APPLICATION
$MR = MR_{max}$ WHEN $\varepsilon a \leq 0.0001$ (1) $MR = K1 * \sigma d^{K2}$ or WHEN $\varepsilon a > 0.0001$ (2) $MR = N1 * \varepsilon a^{N2}$	MODEL: LOG (εa) = A + B * LOG (σd) R^2 = 0.974 AND SEE = 0.020 (1) K1 = 61855 AND K2 = -0.097 (2) N1 = 23333 AND N2 = -0.088 MRmax = 52655 psi	SAY σd = 6 psi USING Eq. (1): MR = 51993 psi Q: MR < MRmax ? ... No **MR(design) = 51,993 psi**

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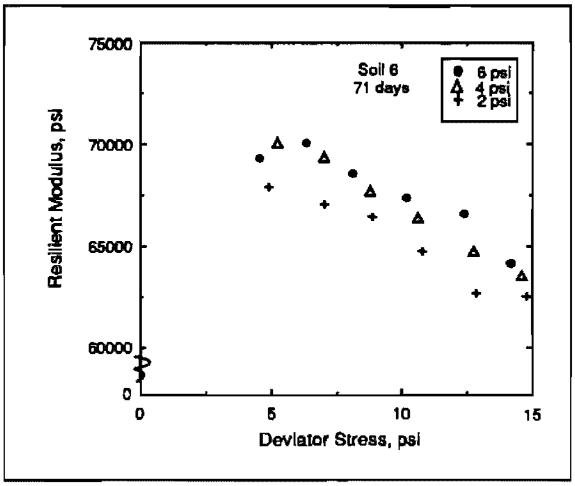
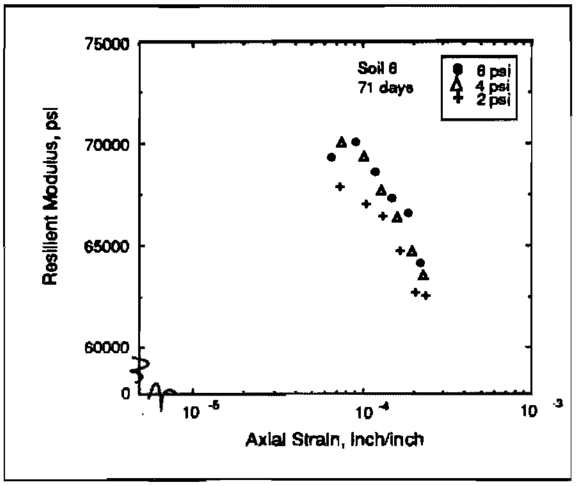
soil-6c.out

RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = 6 - 71 days
DESCRIPTION = Dist 9 - Hockley - US62 - opt
MOISTURE CONTENT = 12.10 percent
DRY DENSITY = 118.95 pcf.
PLASTICITY INDEX = 15.00 percent
LIQUID LIMIT = 30.00 percent
SAMPLE HEIGHT = 5.640 inches
SAMPLE DIAMETER = 2.850 inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	0.481	4.557518	-.00003979	0.000377	0.000364	0.000066	69301.352
6.000	0.398	6.363509	0.00001604	0.000521	0.000504	0.000091	70043.727
6.000	0.335	8.107211	0.00009263	0.000680	0.000653	0.000118	68586.227
6.000	0.211	10.182361	0.00019046	0.000863	0.000843	0.000151	67363.711
6.000	-0.022	12.422998	0.00029388	0.001069	0.001036	0.000187	68596.516
6.000	-0.023	14.180181	0.00038600	0.001240	0.001253	0.000221	64150.998
4.000	0.263	5.229709	0.00021416	0.000431	0.000411	0.000075	70051.148
4.000	0.223	7.045929	0.00021327	0.000591	0.000554	0.000102	69361.742
4.000	0.180	8.754300	0.00021501	0.000749	0.000709	0.000129	67717.734
4.000	0.095	10.625372	0.00021053	0.000927	0.000878	0.000160	66377.852
4.000	0.034	12.744219	0.00021996	0.001132	0.001087	0.000197	64768.559
4.000	0.018	14.599488	0.00023590	0.001309	0.001282	0.000230	63546.289
2.000	0.385	4.927083	0.00012274	0.000426	0.000393	0.000073	67885.102
2.000	0.336	7.030123	0.00010375	0.000616	0.000566	0.000105	67068.188
2.000	0.283	8.848202	0.00009307	0.000789	0.000714	0.000133	66432.422
2.000	0.208	10.783426	0.00008394	0.000973	0.000905	0.000167	64759.414
2.000	0.167	12.867409	0.00008252	0.001192	0.001123	0.000205	62710.172
2.000	0.151	14.797053	0.00008859	0.001369	0.001299	0.000237	62566.484





ANALYSIS OF RESULTS

EXPRESSIONS	STATISTICS	APPLICATION
MR = MRmax WHEN $\varepsilon a \le 0.0001$ (1) $MR = K1 * \sigma d^{K2}$ or (2) $MR = N1 * \varepsilon a^{N2}$ WHEN $\varepsilon a > 0.0001$	MODEL: LOG (εa) = A + B * LOG (σd) R^2 = 0.997 AND SEE = 0.005 (1) K1 = 85520 AND K2 = -0.111 (2) N1 = 27505 AND N2 = -0.100 MRmax = 69019 psi	SAY σd = 6 psi USING Eq. (1): MR = 70100 psi Q: MR < MRmax ? ... No **MR(design) = 69,019 psi**

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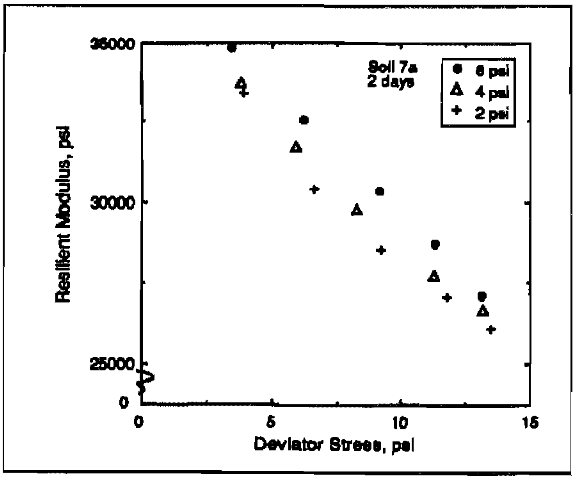
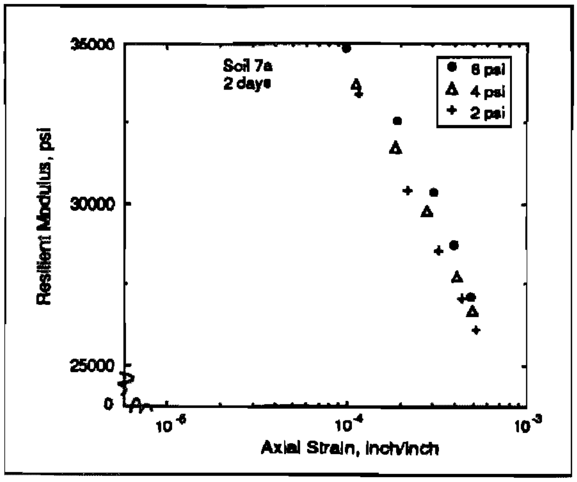
soil-7a.out RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = 7-opt
DESCRIPTION = Dist 4 - Potter - Spur 951 - 2 days
MOISTURE CONTENT = 16.50 percent
DRY DENSITY = 106.40 pcf.
PLASTICITY INDEX = 20.40 percent
SAMPLE HEIGHT = 5.665 inches
SAMPLE DIAMETER = 2.840 inches

CONFIN. (psi)	SEATING (psi)	DEV. STRESS (psi)	PERM DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	1.112	3.451614	-.00005124	0.000328	0.000794	0.000099	34848.293
6.000	0.927	6.204291	-.00008780	0.000633	0.001529	0.000191	32521.572
6.000	0.632	9.162016	-.00013598	0.001006	0.002418	0.000302	30321.242
6.000	0.452	11.333261	-.00010694	0.001316	0.003157	0.000398	28704.621
6.000	0.309	13.104242	-.00005144	0.001641	0.003835	0.000483	27111.254
4.000	1.277	3.803169	-.00017847	0.000373	0.000906	0.000113	33693.617
4.000	1.078	5.914043	-.00023062	0.000612	0.001502	0.000187	31704.469
4.000	0.870	8.277695	-.00027623	0.000912	0.002239	0.000278	29761.832
4.000	0.618	11.285042	-.00033563	0.001364	0.003246	0.000407	27731.533
4.000	0.454	13.146844	-.00034867	0.001685	0.003904	0.000493	26654.311
2.000	1.379	3.873411	-.00044119	0.000379	0.000937	0.000116	33368.055
2.000	1.154	6.638260	-.00049544	0.000715	0.001759	0.000218	30401.693
2.000	0.929	9.219131	-.00054532	0.001071	0.002591	0.000323	28525.350
2.000	0.694	11.770971	-.00060482	0.001466	0.003462	0.000435	27060.646
2.000	0.571	13.419305	-.00061786	0.001774	0.004051	0.000514	26099.244





ANALYSIS OF RESULTS		
EXPRESSIONS	STATISTICS	APPLICATION
MR = MRmax WHEN $\varepsilon a \le 0.0001$ (1) MR = K1 $\cdot$ σd^{K2} or WHEN $\varepsilon a > 0.0001$ (2) MR = N1 $\cdot$ εa^{N2}	MODEL: LOG (εa) = A + B $\cdot$ LOG (σd) R^2 = 0.998 AND SEE = 0.007 (1) K1 = 44044 AND K2 = -0.189 (2) N1 = 8040 AND N2 = -0.159 MRmax = 34792 psi	SAY σd = 6 psi USING Eq. (1): MR = 31384 psi Q: MR < MRmax ? ... No MR(design) = 31,384 psi

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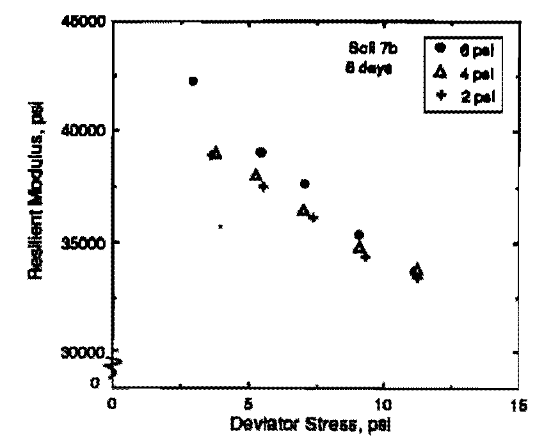
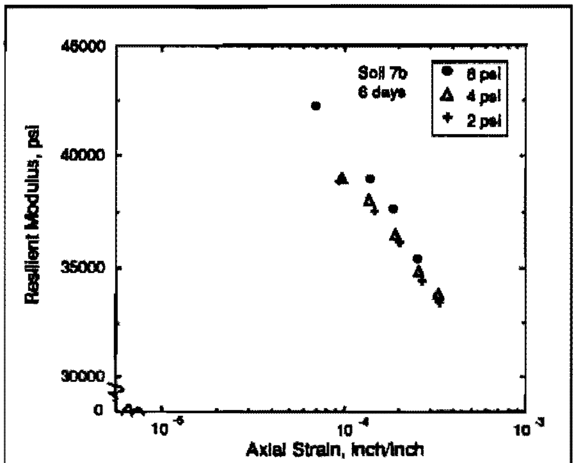
soil-7b.out — RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = 7
DESCRIPTION = Dist 4 - Potter - Spur 951 - 6 days
MOISTURE CONTENT = 16.50 percent
DRY DENSITY = 106.40 pcf.
PLASTICITY INDEX = 20.40 percent
SAMPLE HEIGHT = 5.665 inches
SAMPLE DIAMETER = 2.840 inches

CONFIN. (psi)	SEATING (psi)	DEV. STRESS (psi)	PERM DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	0.946	2.937594	0.00020596	0.000209	0.000578	0.000070	42256.336
6.000	0.778	5.466032	0.00025450	0.000431	0.001157	0.000140	38984.285
6.000	0.674	7.085805	0.00029558	0.000586	0.001546	0.000188	37651.320
6.000	0.492	9.087582	0.00033200	0.000819	0.002090	0.000257	35397.219
6.000	0.324	11.139918	0.00038613	0.001070	0.002670	0.000330	33748.883
4.000	0.905	3.807870	0.00002498	0.000334	0.000772	0.000098	39007.582
4.000	0.871	5.291881	0.00001393	0.000474	0.001103	0.000139	38026.734
4.000	0.762	7.032135	0.00000162	0.000653	0.001530	0.000193	36508.316
4.000	0.616	9.082699	-.00001540	0.000876	0.002075	0.000260	34872.164
4.000	0.436	11.221374	-.00002128	0.001126	0.002630	0.000332	33847.223
2.000	1.140	3.644021	-.00024341	0.000313	0.000748	0.000094	38913.883
2.000	1.034	5.539998	-.00027566	0.000490	0.001182	0.000148	37549.988
2.000	0.945	7.367626	-.00029344	0.000677	0.001633	0.000204	36150.348
2.000	0.787	9.302927	-.00032233	0.000903	0.002162	0.000271	34388.422
2.000	0.622	11.255548	-.00033430	0.001143	0.002667	0.000336	33474.125





ANALYSIS OF RESULTS		
EXPRESSIONS	**STATISTICS**	**APPLICATION**
MR = MRmax WHEN $\varepsilon a \le 0.0001$ (1) $MR = K1 * \sigma d^{K2}$ or (2) $MR = N1 * \varepsilon a^{N2}$ WHEN $\varepsilon a > 0.0001$	MODEL: LOG (εa) = A + B * LOG (σd) R^2 = 0.999 AND SEE = 0.006 (1) K1 = 48395 AND K2 = -0.147 (2) N1 = 12178 AND N2 = -0.128 MRmax = 39556 psi	SAY σd = 6 psi USING Eq. (1): MR = 37211 psi Q: MR < MRmax ? ... No **MR(design) = 37,211 psi**

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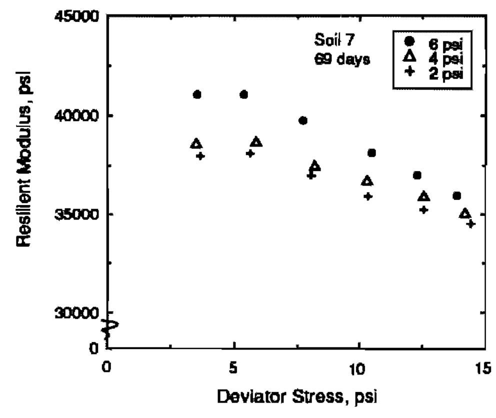
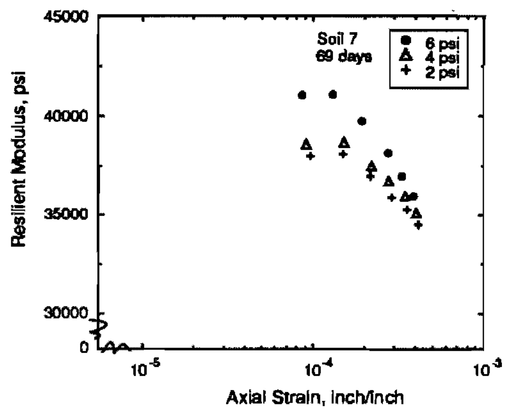
soil-7c.out

RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = 7 - 69 days
DESCRIPTION = Dist 4 - Potter - Spur951 - opt
MOISTURE CONTENT = 17.00 percent
DRY DENSITY = 107.00 pcf.
PLASTICITY INDEX = 20.40 percent
LIQUID LIMIT = 37.60 percent
SAMPLE HEIGHT = 5.630 Inches
SAMPLE DIAMETER = 2.855 Inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (Inch)	AXIAL DEFORMATION A (Inch)	AXIAL DEFORMATION B (Inch)	STRAIN (In/In)	M r. (psi)
6.000	0.792	3.528470	-.00002941	0.000547	0.000422	0.000086	41027.840
6.000	0.770	5.361495	-.00000603	0.000817	0.000653	0.000131	41068.238
6.000	0.552	7.763375	0.00002737	0.001196	0.001002	0.000195	39766.500
6.000	0.310	10.477940	0.00008213	0.001649	0.001444	0.000275	38140.941
6.000	0.148	12.257691	0.00013392	0.001967	0.001765	0.000331	36981.359
6.000	0.032	13.860951	0.00019189	0.002238	0.002101	0.000385	35971.059
4.000	0.943	3.509940	0.00000326	0.000572	0.000451	0.000091	38601.480
4.000	0.863	5.887732	-.00000763	0.000927	0.000785	0.000152	38711.223
4.000	0.737	8.205301	-.00002688	0.001304	0.001158	0.000219	37524.313
4.000	0.592	10.293571	-.00004667	0.001630	0.001524	0.000280	36755.047
4.000	0.392	12.552774	-.00006728	0.001994	0.001938	0.000349	35944.156
4.000	0.258	14.205601	-.00007126	0.002283	0.002277	0.000405	35078.508
2.000	1.042	3.665587	-.00015398	0.000589	0.000497	0.000096	37999.336
2.000	1.002	5.661208	-.00015665	0.000896	0.000777	0.000149	38095.781
2.000	0.900	8.064014	-.00017004	0.001267	0.001188	0.000218	36987.379
2.000	0.788	10.363519	-.00019517	0.001635	0.001614	0.000289	35914.961
2.000	0.570	12.533782	-.00022576	0.001987	0.002018	0.000356	35240.219
2.000	0.436	14.398769	-.00023843	0.002303	0.002396	0.000417	34503.973





ANALYSIS OF RESULTS

EXPRESSIONS	STATISTICS	APPLICATION
MR = MRmax WHEN $\varepsilon a \le 0.0001$ (1) $MR = K1 \cdot \sigma d^{K2}$ or WHEN $\varepsilon a > 0.0001$ (2) $MR = N1 \cdot \varepsilon a^{N2}$	MODEL: LOG (εa) = A + B * LOG (σd) R^2 = 0.995 AND SEE = 0.007 (1) K1 = 48414 AND K2 = -0.118 (2) N1 = 15497 AND N2 = -0.106 MRmax = 40986 psi	SAY σd = 6 psi USING Eq. (1): MR = 39183 psi Q: MR < MRmax ? ... No **MR(design) = 39,183 psi**

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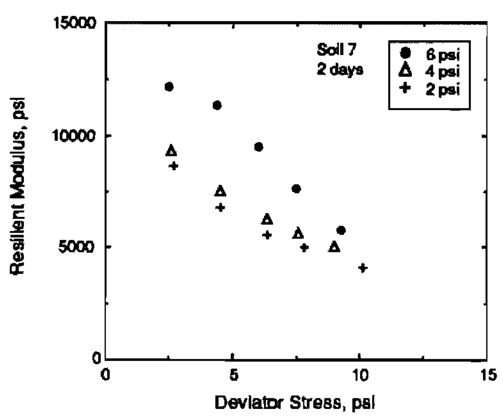
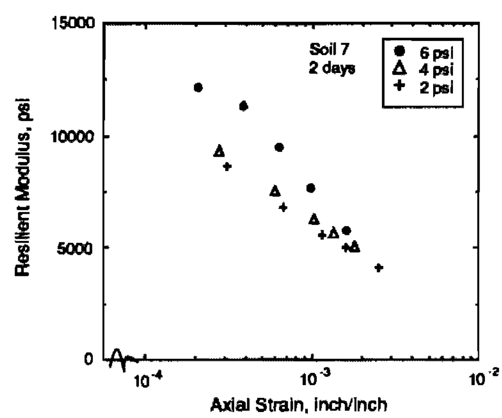
soil-7wa.out — RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = Soil 7 - wet
DESCRIPTION = Dist 4 - Potter - Spur 951 - 2 days
MOISTURE CONTENT = 22.10 percent
DRY DENSITY = 103.24 pcf.
PLASTICITY INDEX = 20.40 percent
LIQUID LIMIT = 37.60 percent
SAMPLE HEIGHT = 5.670 inches
SAMPLE DIAMETER = 2.840 inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	-0.017	2.501285	0.00125516	0.001145	0.001193	0.000206	12130.272
6.000	-0.018	4.388367	0.00144351	0.002114	0.002277	0.000387	11332.345
6.000	-0.018	6.041848	0.00240127	0.003372	0.003829	0.000635	9515.168
6.000	-0.018	7.508069	0.00411021	0.005113	0.005982	0.000978	7673.385
6.000	-0.019	9.250027	0.00816527	0.008261	0.009974	0.001608	5752.234
4.000	0.137	2.578997	0.00569135	0.001473	0.001658	0.000276	9339.885
4.000	0.079	4.524598	0.00564585	0.003114	0.003666	0.000598	7568.167
4.000	0.044	6.371887	0.00569712	0.005223	0.006349	0.001020	6244.565
4.000	0.042	7.558160	0.00584011	0.006847	0.008324	0.001338	5649.743
4.000	0.037	9.009402	0.00637538	0.009121	0.011133	0.001786	5044.364
2.000	0.254	2.677307	0.00497507	0.001647	0.001866	0.000310	8642.844
2.000	0.183	4.538641	0.00491984	0.003450	0.004106	0.000666	6811.406
2.000	0.160	6.370950	0.00493535	0.005836	0.007155	0.001146	5561.124
2.000	0.149	7.857771	0.00504014	0.008017	0.009843	0.001575	4988.989
2.000	0.139	10.131069	0.00657587	0.012577	0.015538	0.002479	4086.319





ANALYSIS OF RESULTS		
EXPRESSIONS	**STATISTICS**	**APPLICATION**
MR = MRmax WHEN $\varepsilon a \le 0.0001$ (1) MR = K1 $\cdot\ \sigma d^{K2}$ or WHEN $\varepsilon a > 0.0001$ (2) MR = N1 $\cdot\ \varepsilon a^{N2}$	MODEL: LOG (εa) = A + B $\cdot$ LOG (σd) R^2 = 0.936 AND SEE = 0.055 (1) K1 = 17899 AND K2 = -0.547 (2) N1 = 561 AND N2 = -0.354 MRmax = 14568 psi	SAY σd = 6 psi USING Eq. (1): MR = 6714 psi Q: MR < MRmax ? ... No **MR(design) = 6,714 psi**

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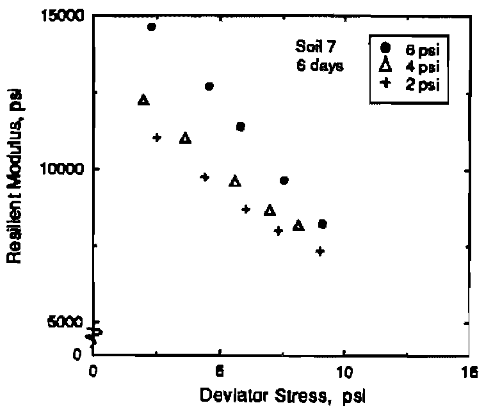
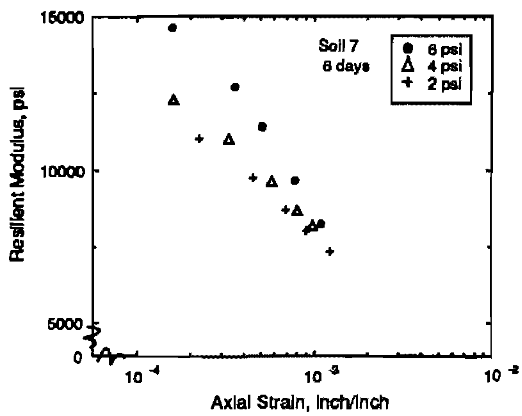
soil-7wb.out — RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = Soil 7 - wet
DESCRIPTION = Dist 4 - Potter - Spur 951 - 6 days
MOISTURE CONTENT = 20.00 percent
DRY DENSITY = 101.10 pcf.
PLASTICITY INDEX = 20.10 percent
LIQUID LIMIT = 37.60 percent
SAMPLE HEIGHT = 5.800 inches
SAMPLE DIAMETER = 2.840 inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	0.304	2.299516	0.00009584	0.000948	0.000873	0.000157	14649.418
6.000	0.204	4.573284	0.00123197	0.002247	0.001928	0.000360	12707.851
6.000	0.176	5.820884	0.00241356	0.003215	0.002704	0.000510	11408.521
6.000	0.148	7.579694	0.00488025	0.005018	0.004064	0.000783	9680.844
6.000	0.095	9.154525	0.00894778	0.007231	0.005655	0.001111	8240.875
4.000	0.556	1.971816	0.00801774	0.001005	0.000861	0.000161	12253.563
4.000	0.489	3.646361	0.00794883	0.002100	0.001737	0.000331	11021.468
4.000	0.384	5.574641	0.00795239	0.003716	0.003007	0.000580	9618.191
4.000	0.362	6.965192	0.00804956	0.005186	0.004148	0.000805	8656.047
4.000	0.310	8.151765	0.00827597	0.006458	0.005096	0.000996	8184.355
2.000	0.610	2.488177	0.00708018	0.001414	0.001198	0.000225	11049.598
2.000	0.530	4.398199	0.00699281	0.002868	0.002359	0.000451	9761.967
2.000	0.472	6.026397	0.00698431	0.004444	0.003589	0.000693	8702.351
2.000	0.123	7.327834	0.00700261	0.005884	0.004703	0.000913	8029.597
2.000	0.347	9.010298	0.00719614	0.007984	0.006299	0.001231	7342.176





ANALYSIS OF RESULTS		
EXPRESSIONS	**STATISTICS**	**APPLICATION**
$MR = MR_{max}$ WHEN $\varepsilon_a \le 0.0001$ (1) $MR = K1 \cdot \sigma_d^{K2}$ or WHEN $\varepsilon_a > 0.0001$ (2) $MR = N1 \cdot \varepsilon_a^{N2}$	MODEL: $LOG(\varepsilon_a) = A + B \cdot LOG(\sigma_d)$ R^2 = 0.974 AND SEE = 0.031 (1) K1 = 16883 AND K2 = -0.327 (2) N1 = 1532 AND N2 = -0.247 MRmax = 14838 psi	SAY σ_d = 6 psi USING Eq. (1): MR = 9393 psi Q: MR < MRmax ? ... No **MR(design) = 9,393 psi**

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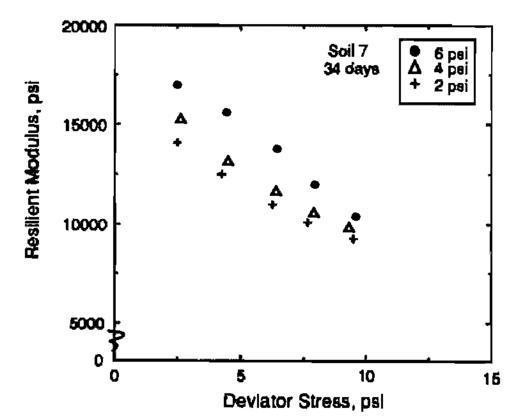
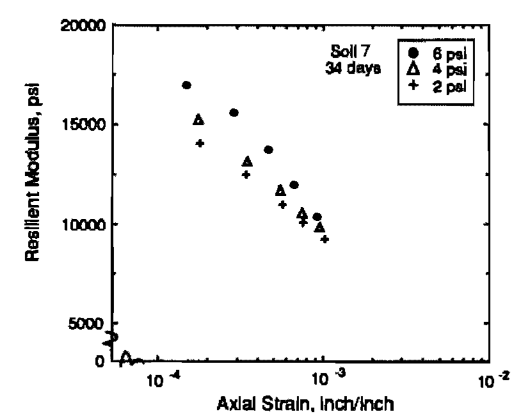
soil-7wc.out — RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = Soil 7 - wet
DESCRIPTION = Dist 4 - Potter - Spur 951 - 34 days
MOISTURE CONTENT = 22.10 percent
DRY DENSITY = 103.24 pcf.
PLASTICITY INDEX = 20.40 percent
LIQUID LIMIT = 37.60 percent
SAMPLE HEIGHT = 5.660 inches
SAMPLE DIAMETER = 2.840 inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	-0.024	2.528437	0.00012518	0.000802	0.000887	0.000149	16945.205
6.000	-0.024	4.492763	0.00035631	0.001533	0.001735	0.000289	15562.438
6.000	-0.024	6.439767	0.00069308	0.002442	0.002858	0.000468	13754.056
6.000	-0.024	7.978552	0.00110119	0.003394	0.004138	0.000665	11990.584
6.000	-0.024	9.589430	0.00161583	0.004696	0.005756	0.000923	10386.138
4.000	-0.022	2.662794	0.00020299	0.000926	0.001053	0.000175	15225.631
4.000	-0.022	4.530215	0.00016204	0.001769	0.002137	0.000345	13128.457
4.000	-0.023	6.425255	0.00017676	0.002801	0.003427	0.000550	11676.750
4.000	-0.022	7.940164	0.00019671	0.003803	0.004675	0.000749	10602.135
4.000	-0.024	9.331952	0.00022705	0.004812	0.005934	0.000949	9830.095
2.000	0.074	2.503158	-.00067567	0.000943	0.001075	0.000178	14039.219
2.000	0.066	4.242307	-.00071516	0.001753	0.002084	0.000339	12515.656
2.000	0.050	6.261874	-.00073601	0.002884	0.003555	0.000569	11007.661
2.000	0.054	7.662087	-.00075021	0.003837	0.004735	0.000757	10118.639
2.000	0.059	9.488780	-.00073698	0.005215	0.006393	0.001025	9253.368





ANALYSIS OF RESULTS		
EXPRESSIONS	STATISTICS	APPLICATION
$MR = MR_{max}$ WHEN $\varepsilon a \le 0.0001$ (1) $MR = K1 \cdot \sigma d^{K2}$ or WHEN $\varepsilon a > 0.0001$ (2) $MR = N1 \cdot \varepsilon a^{N2}$	MODEL: $LOG(\varepsilon a) = A + B \cdot LOG(\sigma d)$ R^2 = 0.980 AND SEE = 0.026 (1) K1 = 21615 AND K2 = -0.332 (2) N1 = 1791 AND N2 = -0.250 MR_{max} = 17833 psi	SAY σd = 6 psi USING Eq. (1): MR = 11914 psi Q: MR < MR_{max} ? ... No **MR(design) = 11,914 psi**

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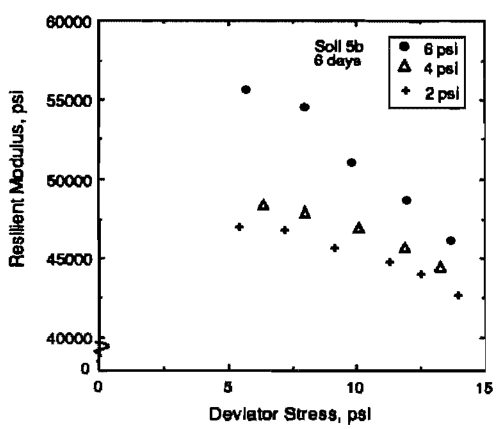
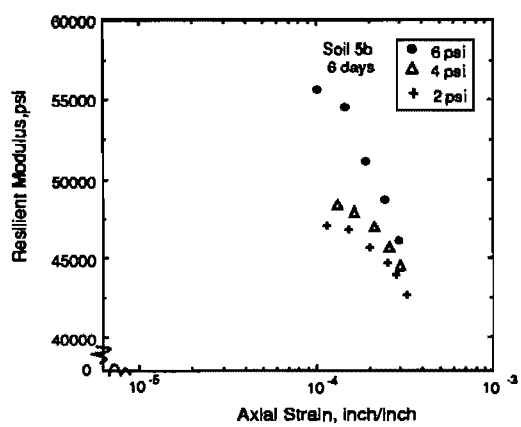
soil-5b.out

RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = 5-opt
DESCRIPTION = Dist 21 - Starr - FM755 - 6 days
MOISTURE CONTENT = 10.60 percent
DRY DENSITY = 124.02 pcf.
PLASTICITY INDEX = 9.50 percent
SAMPLE HEIGHT = 5.640 inches
SAMPLE DIAMETER = 2.820 inches

CONFIN. (psi)	SEATING (psi)	DEV. STRESS (psi)	PERM DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	0.054	5.700525	0.00007746	0.000933	0.000223	0.000102	55632.012
6.000	0.055	7.983867	0.00015125	0.001250	0.000401	0.000146	54546.012
6.000	0.055	9.826592	0.00023990	0.001556	0.000615	0.000192	51063.766
6.000	0.054	11.955623	0.00034940	0.001902	0.000867	0.000246	48698.434
6.000	0.055	13.635013	0.00045455	0.002220	0.001115	0.000296	46119.121
4.000	0.044	6.370951	0.00020333	0.001139	0.000347	0.000132	48370.691
4.000	0.037	7.960603	0.00020059	0.001370	0.000504	0.000166	47919.930
4.000	0.023	10.087260	0.00020333	0.001682	0.000741	0.000215	46960.059
4.000	0.019	11.843568	0.00020916	0.001967	0.000957	0.000259	45684.023
4.000	0.019	13.263715	0.00022917	0.002213	0.001149	0.000298	44490.605
2.000	0.216	5.422763	0.00005418	0.000990	0.000311	0.000115	47013.906
4.000	0.204	7.158654	0.00004531	0.001247	0.000479	0.000153	46786.836
2.000	0.190	9.144294	0.00004145	0.001558	0.000699	0.000200	45699.461
2.000	0.177	11.279499	0.00003816	0.001891	0.000953	0.000252	44734.086
2.000	0.177	12.485507	0.00004251	0.002091	0.001116	0.000284	43914.176
2.000	0.167	13.913727	0.00004815	0.002357	0.001322	0.000326	42666.348





ANALYSIS OF RESULTS		
EXPRESSIONS	STATISTICS	APPLICATION
MR = MRmax WHEN $\varepsilon a \le 0.0001$ (1) $MR = K1 \cdot \sigma d^{K2}$ or WHEN $\varepsilon a > 0.0001$ (2) $MR = N1 \cdot \varepsilon a^{N2}$	MODEL: LOG (εa) = A + B * LOG (σd) R^2 = 0.972 AND SEE = 0.017 (1) K1 = 63495 AND K2 = -0.128 (2) N1 = 18072 AND N2 = -0.114 MRmax = 51466 psi	SAY σd = 6 psi USING Eq. (1): MR = 50464 psi Q: MR < MRmax ? ... No **MR(design) = 50,464 psi**

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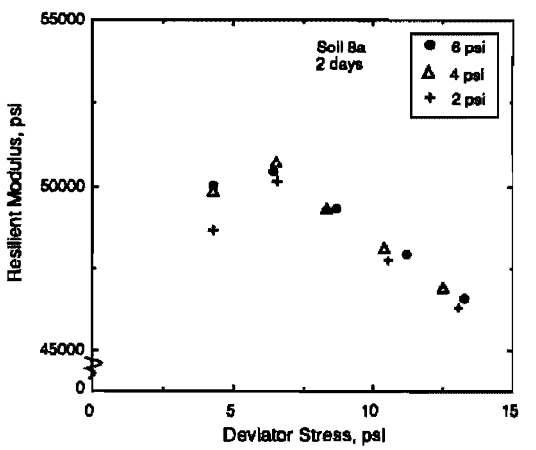
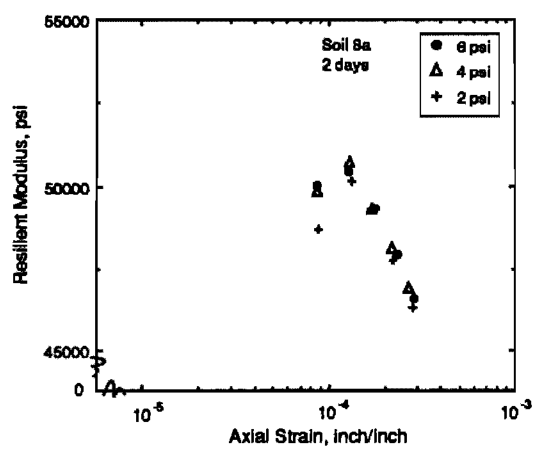
soil-8a.out

RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = 8-opt
DESCRIPTION = Dist 7 - Glasscock - RM2401 - 2 days
MOISTURE CONTENT = 14.20 percent
DRY DENSITY = 113.17 pcf.
PLASTICITY INDEX = 18.10 percent
SAMPLE HEIGHT = 5.600 inches
SAMPLE DIAMETER = 2.840 inches

CONFIN. (psi)	SEATING (psi)	DEV. STRESS (psi)	PERM DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	0.035	4.292866	.00001463	0.000319	0.000642	0.000086	50042.863
6.000	0.037	6.466453	.00000942	0.000528	0.000909	0.000128	50419.531
6.000	0.038	8.698555	.00000025	0.000768	0.001206	0.000176	49346.758
6.000	0.039	11.197029	.00001488	0.001059	0.001557	0.000234	47941.160
6.000	0.040	13.261540	.00003560	0.001326	0.001861	0.000285	46597.465
4.000	0.051	4.283036	.00021598	0.000311	0.000650	0.000086	49872.957
4.000	0.035	6.520756	.00022778	0.000532	0.000908	0.000129	50719.887
4.000	0.034	8.343703	.00023636	0.000734	0.001160	0.000169	49339.625
4.000	0.035	10.402125	.00023662	0.000982	0.001438	0.000216	48144.387
4.000	0.042	12.507361	.00022960	0.001249	0.001736	0.000267	46921.492
2.000	0.174	4.276481	.00034692	0.000329	0.000654	0.000088	48688.852
2.000	0.161	6.600808	.00033533	0.000551	0.000923	0.000132	50161.590
2.000	0.158	8.337617	.00033638	0.000739	0.001153	0.000169	49338.324
2.000	0.185	10.526619	.00035395	0.001011	0.001457	0.000220	47755.301
2.000	0.176	13.033554	.00035068	0.001343	0.001811	0.000282	46285.594





ANALYSIS OF RESULTS		
EXPRESSIONS	**STATISTICS**	**APPLICATION**
$MR = MR_{max}$ WHEN $\varepsilon_a \le 0.0001$ (1) $MR = K1 \cdot \sigma_d^{K2}$ or WHEN $\varepsilon_a > 0.0001$ (2) $MR = N1 \cdot \varepsilon_a^{N2}$	MODEL: LOG (ε_a) = A + B * LOG (σ_d) R^2 = 0.998 AND SEE = 0.006 (1) K1 = 54702 AND K2 = -0.056 (2) N1 = 30739 AND N2 = -0.053 MRmax = 50005 psi	SAY σ_d = 6 psi USING Eq. (1): MR = 49499 psi Q: MR < MRmax ? ... No **MR(design) = 49,499 psi**

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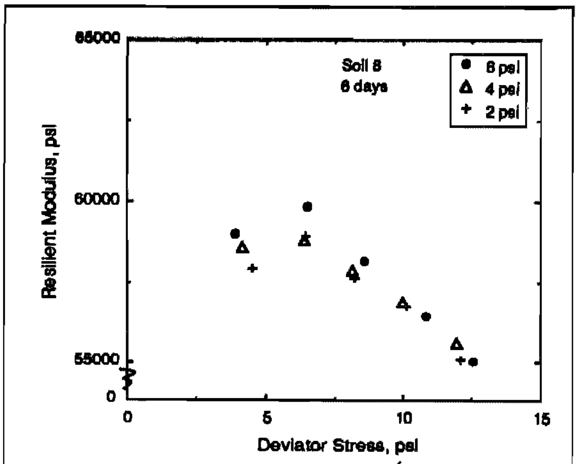
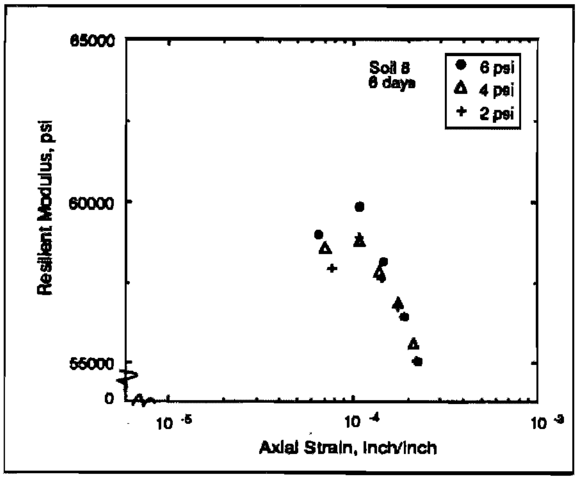
soil-6b.out — RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = 6-opt
DESCRIPTION = Dist 7 - Glasscock - AM2401 - 6 days
MOISTURE CONTENT = 14.20 percent
DRY DENSITY = 113.17 pcf.
PLASTICITY INDEX = 18.10 percent
SAMPLE HEIGHT = 5.600 inches
SAMPLE DIAMETER = 2.840 inches

CONFIN. (psi)	SEATING (psi)	DEV. STRESS (psi)	PERM DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	0.039	3.898690	.00000937	0.000217	0.000523	0.000066	59003.707
6.000	0.042	6.528437	.00003050	0.000444	0.000777	0.000109	59868.957
6.000	0.043	8.609609	.00007008	0.000644	0.001014	0.000148	58163.703
6.000	0.044	10.848264	.00011961	0.000675	0.001278	0.000192	56434.219
6.000	0.045	12.517661	.00017570	0.001063	0.001485	0.000228	55021.441
4.000	0.087	4.181448	.00002648	0.000249	0.000550	0.000071	58603.961
4.000	0.073	6.404655	.00002971	0.000446	0.000774	0.000109	58796.164
4.000	0.067	8.170022	.00002588	0.000610	0.000971	0.000141	57852.223
4.000	0.060	10.004670	.00002657	0.000791	0.001180	0.000176	56852.559
4.000	0.051	11.959167	.00002255	0.000998	0.001411	0.000215	55604.766
2.000	0.239	4.523660	.00016734	0.000293	0.000561	0.000078	57928.016
2.000	0.225	6.443514	.00014323	0.000453	0.000772	0.000109	58923.848
2.000	0.217	8.234625	.00014541	0.000621	0.000980	0.000143	57617.988
2.000	0.206	10.117962	.00014198	0.000803	0.001194	0.000178	56742.539
2.000	0.200	12.108036	.00014474	0.001020	0.001441	0.000220	55082.910





ANALYSIS OF RESULTS		
EXPRESSIONS	STATISTICS	APPLICATION
MR = MRmax WHEN $\varepsilon a \le 0.0001$ (1) $MR = K1 \cdot \sigma d^{K2}$ or WHEN $\varepsilon a > 0.0001$ (2) $MR = N1 \cdot \varepsilon a^{N2}$	MODEL: $LOG(\varepsilon a) = A + B \cdot LOG(\sigma d)$ R^2 = 0.999 AND SEE = 0.004 (1) K1 = 64605 AND K2 = -0.057 (2) N1 = 35691 AND N2 = -0.054 MRmax = 58460 psi	SAY σd = 6 psi USING Eq. (1): MR = 58374 psi Q: MR < MRmax ? ... No **MR(design) = 58,374 psi**

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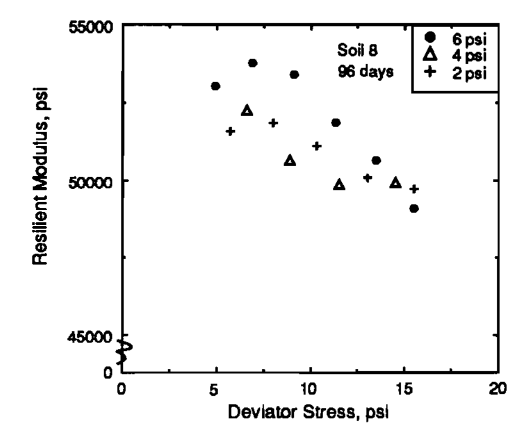
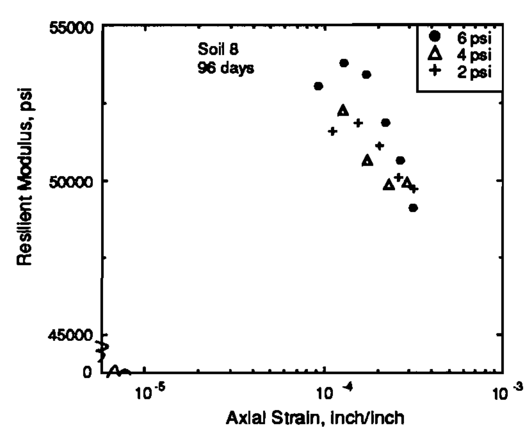
soil-8c.out — RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = Soil 8 - opt
DESCRIPTION = Dist 7 - Glasscock - RM2401 - 96 days
MOISTURE CONTENT = 13.70 percent
DRY DENSITY = 113.10 pcf.
PLASTICITY INDEX = 18.10 percent
LIQUID LIMIT = 37.10 percent
SAMPLE HEIGHT = 5.640 inches
SAMPLE DIAMETER = 2.840 inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	0.356	4.952011	0.01840947	0.000295	0.000758	0.000093	53049.250
6.000	0.263	6.919145	0.01847118	0.000414	0.001038	0.000129	53752.145
6.000	-0.020	9.107711	0.01852671	0.000532	0.001392	0.000171	53395.504
6.000	-0.024	11.342155	0.01863125	0.000692	0.001775	0.000219	51852.746
6.000	-0.025	13.486245	0.01873822	0.000839	0.002163	0.000266	50671.305
6.000	-0.024	15.503472	0.01885159	0.000985	0.002576	0.000316	49114.480
4.000	0.039	6.612980	0.01862594	0.000381	0.001046	0.000127	52262.461
4.000	0.025	8.886747	0.01860414	0.000528	0.001451	0.000175	50654.141
4.000	0.013	11.540646	0.01860483	0.000698	0.001913	0.000231	49865.852
4.000	-0.009	14.504925	0.01863327	0.000905	0.002372	0.000291	49925.871
2.000	0.116	5.716019	0.01849229	0.000334	0.000916	0.000111	51598.934
2.000	0.126	8.025366	0.01844906	0.000466	0.001279	0.000155	51874.766
2.000	0.134	10.357654	0.01845304	0.000608	0.001676	0.000203	51145.508
2.000	0.143	13.007807	0.01844977	0.000791	0.002137	0.000260	50107.199
2.000	0.152	15.551224	0.01847302	0.000971	0.002556	0.000313	49729.938





ANALYSIS OF RESULTS

EXPRESSIONS	STATISTICS	APPLICATION
MR = MRmax WHEN $\varepsilon a \le 0.0001$ (1) MR = K1 * σd^{K2} or (2) MR = N1 * εa^{N2} WHEN $\varepsilon a > 0.0001$	MODEL: LOG (εa) = A + B * LOG (σd) R^2 = 0.998 AND SEE = 0.005 (1) K1 = 58660 AND K2 = -0.060 (2) N1 = 31622 AND N2 = -0.056 MRmax = 53101 psi	SAY σd = 6 psi USING Eq. (1): MR = 52715 psi Q: MR < MRmax ? ... No **MR(design) = 52,715 psi**

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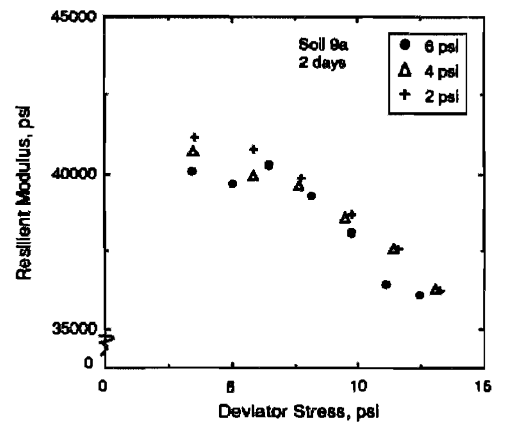
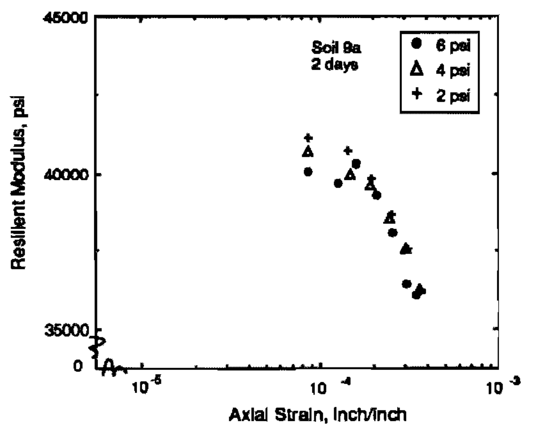
soil-9a.out — RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = 9
DESCRIPTION = Dist 4 - Gray - SH70 - 2 days
MOISTURE CONTENT = 19.80 percent
DRY DENSITY = 104.03 pcf.
PLASTICITY INDEX = 34.00 percent
SAMPLE HEIGHT = 5.600 Inches
SAMPLE DIAMETER = 2.840 Inches

CONFIN. (psi)	SEATING (psi)	DEV. STRESS (psi)	PERM DEFORMATION (Inch)	AXIAL DEFORMATION A (Inch)	AXIAL DEFORMATION B (Inch)	STRAIN (In/In)	M r. (psi)
6.000	0.544	6.173174	0.01901865	0.000719	0.001080	0.000161	40296.184
6.000	0.559	8.148487	0.01908635	0.000953	0.001371	0.000208	39262.578
6.000	0.564	9.754215	0.01915678	0.001163	0.001707	0.000256	38071.668
6.000	0.719	11.122595	0.01934066	0.001397	0.002022	0.000305	36441.375
6.000	0.755	12.446801	0.01947871	0.001587	0.002279	0.000345	36055.551
4.000	1.091	5.877527	-.00150809	0.000654	0.000993	0.000147	39964.712
4.000	1.029	7.617108	-.00150947	0.000875	0.001288	0.000193	39599.289
4.000	0.971	9.472395	-.00149378	0.001127	0.001626	0.000246	38513.680
4.000	0.908	11.402079	-.00147717	0.001398	0.002002	0.000304	37562.363
4.000	0.877	13.056962	-.00143595	0.001666	0.002367	0.000360	36262.125
2.000	1.221	5.883146	-.00157906	0.000612	0.000975	0.000144	40748.703
2.000	1.166	7.745416	-.00157751	0.000878	0.001299	0.000194	39845.891
2.000	1.112	9.780900	-.00157372	0.001155	0.001678	0.000253	38668.410
2.000	1.066	11.615550	-.00157006	0.001428	0.002035	0.000309	37559.582
2.000	1.006	13.240005	-.00157138	0.001690	0.002403	0.000365	36228.875





ANALYSIS OF RESULTS		
EXPRESSIONS	**STATISTICS**	**APPLICATION**
MR = MRmax WHEN $\varepsilon a \leq 0.0001$ (1) $MR = K1 \cdot \sigma d^{K2}$ or (2) $MR = N1 \cdot \varepsilon a^{N2}$ WHEN $\varepsilon a > 0.0001$	MODEL: LOG (εa) = A + B * LOG (σd) R^2 = 0.998 AND SEE = 0.004 (1) K1 = 52641 AND K2 = -0.143 (2) N1 = 13534 AND N2 = -0.125 MRmax = 42776 psi	SAY σd = 6 psi USING Eq. (1): MR = 40758 psi Q: MR < MRmax ? ... No **MR(design) = 40,758 psi**

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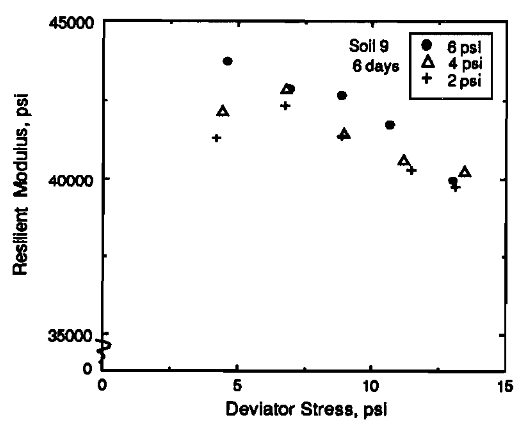
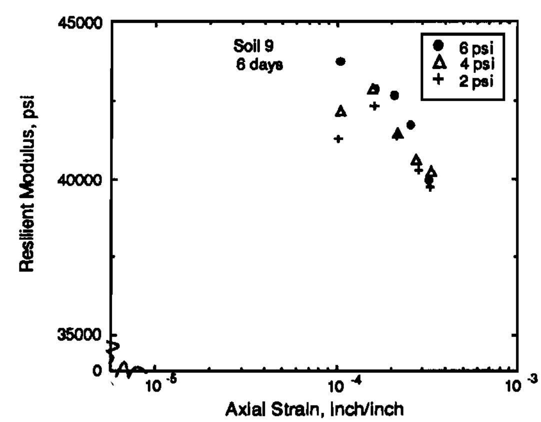
soil-9b.out — RESILIENT MODULUS (MR) TEST RESULTS

- SAMPLE IDENTIFICATION = Soil 9b - opt
- DESCRIPTION = Diet 4 - Gray - SH70 - 6 days
- MOISTURE CONTENT = 20.00 percent
- DRY DENSITY = 103.04 pcf.
- PLASTICITY INDEX = 34.00 percent
- LIQUID LIMIT = 52.00 percent
- SAMPLE HEIGHT = 5.630 inches
- SAMPLE DIAMETER = 2.840 inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	0.112	4.608862	0.01408551	0.000752	0.000435	0.000105	43748.121
6.000	-0.001	6.949575	0.01414203	0.001164	0.000661	0.000162	42873.898
6.000	-0.023	8.867556	0.01422280	0.001504	0.000836	0.000208	42671.461
6.000	-0.028	10.651177	0.01426209	0.001847	0.001027	0.000255	41733.262
6.000	-0.026	13.038704	0.01435462	0.002278	0.001396	0.000326	39957.316
4.000	0.122	4.417861	0.01421965	0.000755	0.000425	0.000105	42167.500
4.000	0.021	6.796960	0.01420147	0.001130	0.000655	0.000159	42861.020
4.000	0.006	8.945734	0.01420359	0.001517	0.000912	0.000216	41477.965
4.000	-0.001	11.186733	0.01422142	0.001910	0.001189	0.000275	40647.891
4.000	-0.010	13.455818	0.01424370	0.002300	0.001464	0.000334	40253.988
2.000	0.285	4.193620	0.01413255	0.000720	0.000423	0.000102	41312.578
2.000	0.220	6.755297	0.01414052	0.001124	0.000673	0.000160	42335.094
2.000	0.195	8.875045	0.01414612	0.001485	0.000931	0.000215	41364.023
2.000	0.173	11.442337	0.01415638	0.001938	0.001259	0.000284	40298.387
2.000	0.158	13.100966	0.01416840	0.002225	0.001484	0.000329	39776.305





ANALYSIS OF RESULTS

EXPRESSIONS	STATISTICS	APPLICATION
$MR = MR_{max}$ WHEN $\varepsilon a \leq 0.0001$ (1) $MR = K1 \cdot \sigma d^{K2}$ or (2) $MR = N1 \cdot \varepsilon a^{N2}$ WHEN $\varepsilon a > 0.0001$	MODEL: $LOG(\varepsilon a) = A + B \cdot LOG(\sigma d)$ $R^2 = 0.998$ AND SEE = 0.006 (1) K1 = 46208 AND K2 = -0.051 (2) N1 = 27568 AND N2 = -0.048 MR_{max} = 42929 psi	SAY $\sigma d = 6$ psi USING Eq. (1): MR = 42209 psi Q: MR < MRmax ? ... No **MR(design) = 42,209 psi**

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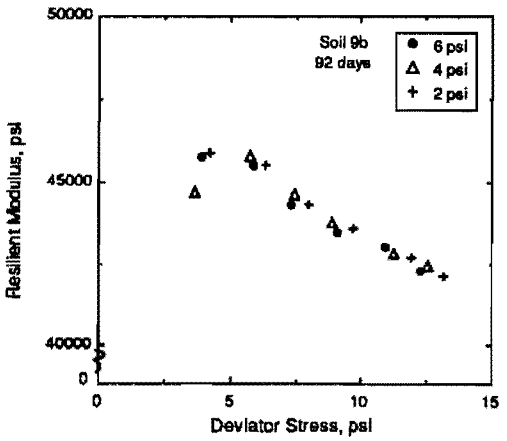
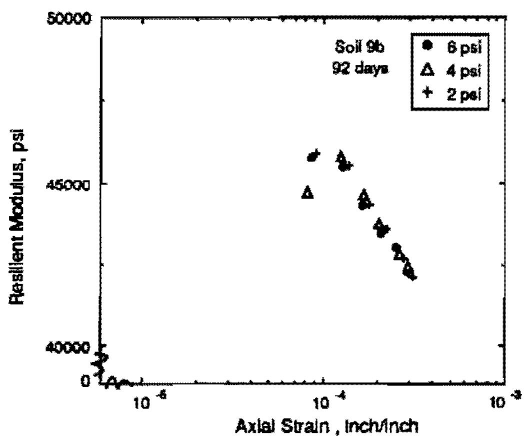
soil-9c.out — RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = 9-opt
DESCRIPTION = Dist4 - Gray - SH70 - 92 days
MOISTURE CONTENT = 19.80 percent
DRY DENSITY = 104.03 pcf.
PLASTICITY INDEX = 34.00 percent
SAMPLE HEIGHT = 5.600 inches
SAMPLE DIAMETER = 2.840 inches

CONFIN. (psi)	SEATING (psi)	DEV. STRESS (psi)	PERM DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	0.252	5.868633	0.00004887	0.000516	0.000929	0.000129	45486.070
6.000	0.221	7.301149	0.00007466	0.000677	0.001169	0.000165	44302.414
6.000	0.230	9.110988	0.00013314	0.000873	0.001474	0.000210	43482.984
6.000	0.223	10.942830	0.00016375	0.001075	0.001772	0.000254	43041.438
6.000	0.227	12.254564	0.00019142	0.001237	0.002009	0.000290	42291.016
4.000	0.552	5.713210	0.00002961	0.000501	0.000896	0.000125	45814.375
4.000	0.512	7.453765	0.00002371	0.000686	0.001165	0.000167	44628.938
4.000	0.489	8.881599	0.00002526	0.000847	0.001427	0.000203	43754.539
4.000	0.457	11.247122	0.00003094	0.001121	0.001821	0.000263	42818.223
4.000	0.445	12.516726	0.00003792	0.001267	0.002034	0.000295	42463.176
2.000	0.734	6.302602	-.00009800	0.000562	0.000968	0.000138	45530.922
2.000	0.687	7.990723	-.00010693	0.000748	0.001270	0.000180	44338.945
2.000	0.655	9.693826	-.00010651	0.000941	0.001549	0.000222	43597.355
2.000	0.622	11.890348	-.00010446	0.001201	0.001918	0.000278	42704.266
2.000	0.608	13.105180	-.00009874	0.001346	0.002138	0.000311	42118.680





ANALYSIS OF RESULTS

EXPRESSIONS	STATISTICS	APPLICATION
$MR = MR_{max}$ WHEN $\varepsilon_a \leq 0.0001$ (1) $MR = K1 \cdot \sigma_d^{K2}$ or (2) $MR = N1 \cdot \varepsilon_a^{N2}$ WHEN $\varepsilon_a > 0.0001$	MODEL: $LOG(\varepsilon_a) = A + B \cdot LOG(\sigma_d)$ R^2 = 1.000 AND SEE = 0.001 (1) K1 = 54153 AND K2 = -0.087 (2) N1 = 20644 AND N2 = -0.089 MR_{max} = 46835 psi	SAY σ_d = 6 psi USING Eq. (1): MR = 45508 psi Q: MR < MR_{max} ? ... No MR(design) = 45,508 psi

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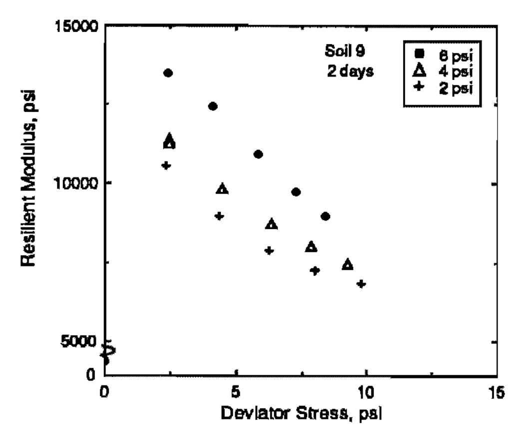
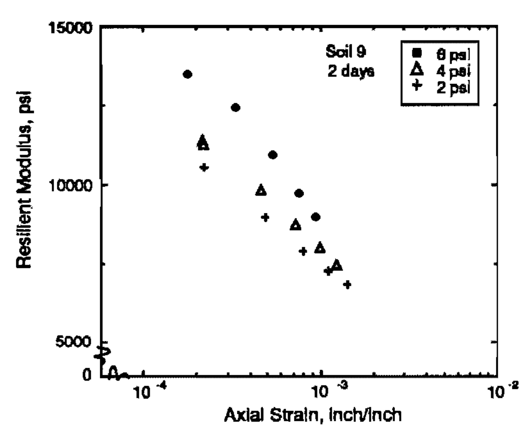
soil-9wa.out — RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = Soil 9 - wet
DESCRIPTION = Diet 4 - Gray - SH70 - 2 days
MOISTURE CONTENT = 26.10 percent
DRY DENSITY = 97.92 pcf.
PLASTICITY INDEX = 34.00 percent
LIQUID LIMIT = 52.00 percent
SAMPLE HEIGHT = 5.700 Inches
SAMPLE DIAMETER = 2.820 Inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	-0.011	2.420088	-.00008523	0.000990	0.001054	0.000179	13493.696
6.000	-0.011	4.123693	0.00001198	0.001874	0.001903	0.000331	12446.955
6.000	-0.014	5.859111	0.00078192	0.003076	0.003026	0.000535	10947.229
6.000	-0.012	7.304896	0.00189097	0.004318	0.004231	0.000750	9740.392
6.000	-0.014	8.422114	0.00301556	0.005424	0.005295	0.000940	8957.682
4.000	0.224	2.465669	0.00051683	0.001196	0.001299	0.000219	11264.957
4.000	0.232	2.450950	0.00044254	0.001174	0.001277	0.000215	11399.549
4.000	0.162	4.489769	0.00040848	0.002552	0.002659	0.000457	9822.364
4.000	0.129	6.352908	0.00047555	0.004098	0.004193	0.000727	8735.934
4.000	0.116	7.888432	0.00062707	0.005600	0.005657	0.000987	7988.917
4.000	0.101	9.265846	0.00106893	0.007065	0.007093	0.001242	7461.210
2.000	0.359	2.310407	-.00015351	0.001185	0.001311	0.000219	10552.725
2.000	0.271	4.344953	-.00018981	0.002676	0.002844	0.000484	8972.674
2.000	0.254	6.288810	-.00016217	0.004472	0.004634	0.000799	7873.162
2.000	0.236	8.028974	-.00008039	0.006246	0.006349	0.001105	7267.212
2.000	0.189	9.779586	0.00023094	0.008121	0.008151	0.001427	6851.507





ANALYSIS OF RESULTS		
EXPRESSIONS	**STATISTICS**	**APPLICATION**
MR = MRmax WHEN $\varepsilon_a \le 0.0001$ (1) $MR = K1 * \sigma_d^{K2}$ or WHEN $\varepsilon_a > 0.0001$ (2) $MR = N1 * \varepsilon_a^{N2}$	MODEL: LOG (ε_a) = A + B * LOG (σ_d) R^2 = 0.968 AND SEE = 0.035 (1) K1 = 16216 AND K2 = -0.329 (2) N1 = 1471 AND N2 = -0.248 MRmax = 14387 psi	SAY σ_d = 6 psi USING Eq. (1): MR = 8992 psi Q: MR < MRmax ? ... No **MR(design) = 8,992 psi**

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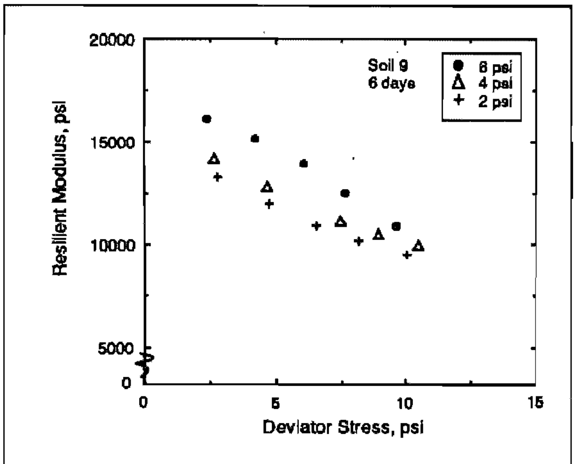
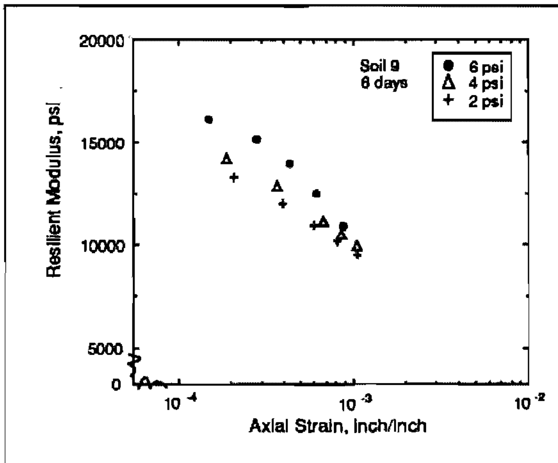
soil-9wb.out — RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = Soil 9 - wet
DESCRIPTION = Dist 4 - Gray - SH70 - 6 days
MOISTURE CONTENT = 25.30 percent
DRY DENSITY = 97.15 pcf.
PLASTICITY INDEX = 34.00 percent
LIQUID LIMIT = 52.00 percent
SAMPLE HEIGHT = 5.720 Inches
SAMPLE DIAMETER = 2.870 Inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (Inch)	AXIAL DEFORMATION A (Inch)	AXIAL DEFORMATION B (Inch)	STRAIN (in/in)	M r. (psi)
6.000	-0.019	2.393800	-.00002894	0.000748	0.000953	0.000149	16093.214
6.000	-0.020	4.227884	0.00007126	0.001438	0.001751	0.000279	15163.885
6.000	-0.019	6.060135	0.00059562	0.002282	0.002696	0.000435	13928.062
6.000	-0.019	7.659057	0.00138687	0.003213	0.003790	0.000612	12512.804
6.000	-0.020	9.634333	0.00336084	0.004646	0.005450	0.000883	10916.992
4.000	-0.018	2.661509	0.00177593	0.000977	0.001168	0.000188	14191.018
4.000	-0.018	4.664288	0.00174438	0.001924	0.002228	0.000363	12853.345
4.000	-0.020	7.491280	0.00179159	0.003556	0.004136	0.000672	11141.051
4.000	-0.018	8.921968	0.00186766	0.004511	0.005193	0.000848	10517.991
4.000	-0.019	10.462214	0.00205220	0.005590	0.006410	0.001049	9974.031
2.000	0.068	2.760067	0.00098419	0.001078	0.001299	0.000208	13280.520
2.000	0.053	4.767888	0.00093971	0.002103	0.002423	0.000396	12050.971
2.000	0.050	6.535045	0.00093988	0.003167	0.003656	0.000596	10957.390
2.000	0.058	8.147719	0.00095931	0.004287	0.004908	0.000804	10136.569
2.000	0.064	10.034062	0.00104131	0.005668	0.006413	0.001056	9501.806





ANALYSIS OF RESULTS		
EXPRESSIONS	STATISTICS	APPLICATION
$MR = MRmax$ WHEN $\varepsilon a \le 0.0001$ (1) $MR = K1 \cdot \sigma d^{K2}$ or WHEN $\varepsilon a > 0.0001$ (2) $MR = N1 \cdot \varepsilon a^{N2}$	MODEL: LOG (εa) = A + B * LOG (σd) R^2 = 0.983 AND SEE = 0.024 (1) K1 = 18560 AND K2 = -0.275 (2) N1 = 2317 AND N2 = -0.216 MRmax = 16923 psi	SAY σd = 6 psi USING Eq. (1): MR = 11944 psi Q: MR < MRmax ? ... No **MR(design) = 11,944 psi**

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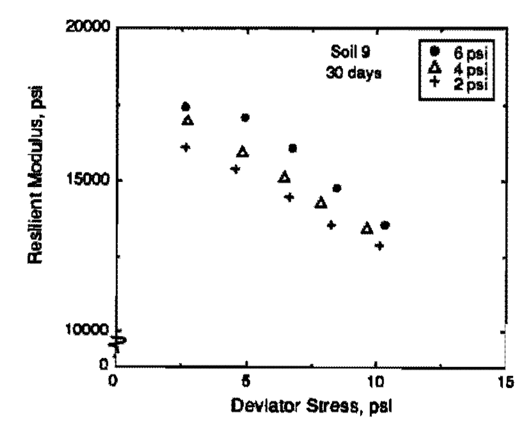
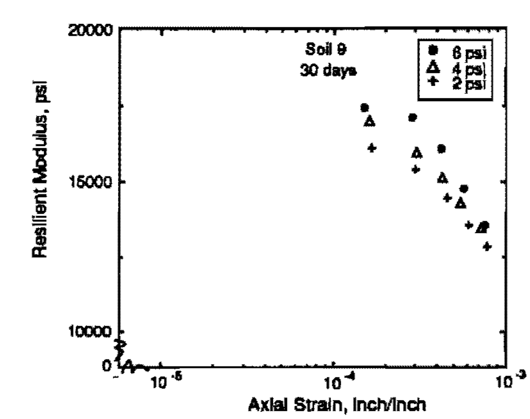
soil-9wc.out — RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = Soil 9 - wet
DESCRIPTION = Dist 4 - Gray - SH70 - 30 days
MOISTURE CONTENT = 25.30 percent
DRY DENSITY = 97.15 pcf.
PLASTICITY INDEX = 34.00 percent
LIQUID LIMIT = 52.00 percent
SAMPLE HEIGHT = 5.720 Inches
SAMPLE DIAMETER = 2.870 Inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	-0.019	2.638130	0.00011162	0.000762	0.000970	0.000151	17425.818
6.000	-0.019	4.926038	0.00030450	0.001481	0.001812	0.000288	17112.516
6.000	-0.020	6.764249	0.00057628	0.002193	0.002629	0.000421	16050.570
6.000	-0.021	8.463103	0.00084911	0.002977	0.003576	0.000573	14774.025
6.000	-0.024	10.307732	0.00129560	0.003982	0.004712	0.000760	13562.445
4.000	-0.016	2.720185	0.00016734	0.000812	0.001020	0.000160	16987.783
4.000	-0.016	4.830231	0.00019290	0.001557	0.001903	0.000302	15968.974
4.000	-0.018	6.463534	0.00022267	0.002218	0.002666	0.000427	15141.273
4.000	-0.017	7.847922	0.00026650	0.002842	0.003438	0.000549	14295.693
4.000	-0.018	9.645334	0.00034289	0.003739	0.004448	0.000716	13479.141
2.000	0.065	2.657383	-.00061411	0.000835	0.001052	0.000165	16115.490
2.000	0.071	4.584525	-.00063925	0.001535	0.001869	0.000297	15410.744
2.000	0.064	6.619392	-.00065185	0.002386	0.002846	0.000457	14472.947
2.000	0.070	8.256362	-.00066611	0.003198	0.003770	0.000609	13556.136
2.000	0.075	10.102366	-.00063515	0.004147	0.004831	0.000785	12871.743





ANALYSIS OF RESULTS		
EXPRESSIONS	**STATISTICS**	**APPLICATION**
MR = MRmax WHEN $\varepsilon_a \le 0.0001$ (1) $MR = K1 \cdot \sigma_d^{K2}$ or WHEN $\varepsilon_a > 0.0001$ (2) $MR = N1 \cdot \varepsilon_a^{N2}$	MODEL: LOG (ε_a) = A + B * LOG (σ_d) R^2 = 0.997 AND SEE = 0.014 (1) K1 = 20491 AND K2 = -0.173 (2) N1 = 4739 AND N2 = -0.147 MRmax = 18434 psi	SAY σ_d = 6 psi USING Eq. (1): MR = 15029 psi Q: MR < MRmax ? ... No **MR(design) = 15,029 psi**

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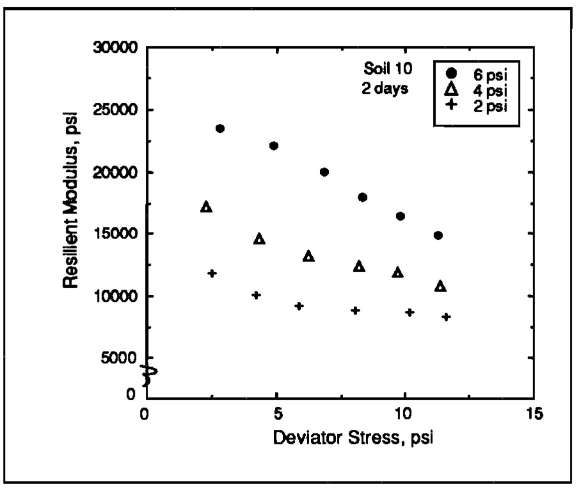
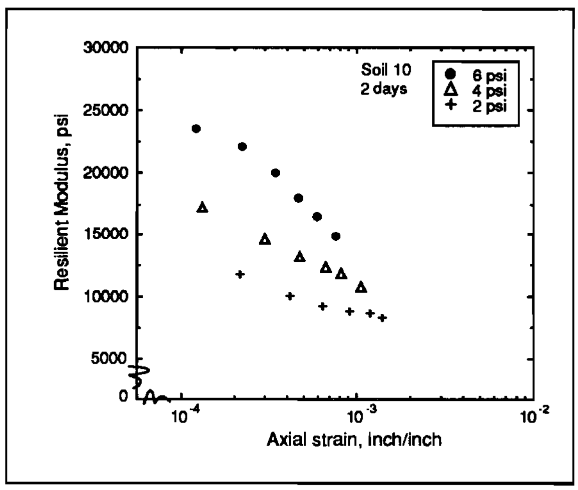
soil-10a.out — RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = 10
DESCRIPTION = Dist 5 - Lubbock - FM835 - 2 days
MOISTURE CONTENT = 10.80 percent
DRY DENSITY = 123.05 pcf.
PLASTICITY INDEX = 4.00 percent
LIQUID LIMIT = 20.00 percent
SAMPLE HEIGHT = 5.630 inches
SAMPLE DIAMETER = 2.840 inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	0.536	2.840220	-.00005703	0.000651	0.000712	0.000121	23469.918
6.000	0.383	4.900983	-.00008179	0.001216	0.001283	0.000222	22078.014
6.000	0.182	6.850327	0.00001027	0.001889	0.001964	0.000342	20020.533
6.000	0.071	8.340895	0.00046402	0.002636	0.002585	0.000464	17989.764
6.000	0.030	9.798692	0.00143382	0.003457	0.003274	0.000598	16392.324
6.000	-0.015	11.283636	0.00307487	0.004469	0.004110	0.000762	14809.893
4.000	0.841	2.267682	0.00264344	0.000768	0.000709	0.000131	17284.596
4.000	0.622	4.362619	0.00253538	0.001768	0.001603	0.000299	14574.675
4.000	0.459	6.211314	0.00248748	0.002788	0.002508	0.000470	13206.640
4.000	0.314	8.185471	0.00262107	0.003938	0.003523	0.000663	12351.814
4.000	0.235	9.708807	0.00318429	0.004882	0.004340	0.000819	11854.553
4.000	0.022	11.366032	0.00424444	0.006309	0.005564	0.001054	10779.563
2.000	0.279	2.512052	0.00347013	0.001281	0.001132	0.000214	11722.003
2.000	0.182	4.220304	0.00331599	0.002516	0.002191	0.000418	10096.228
2.000	0.158	5.855525	0.00331567	0.003832	0.003287	0.000632	9261.705
2.000	0.145	8.048305	0.00347160	0.005546	0.004741	0.000914	8809.491
2.000	0.147	10.141838	0.00429193	0.007130	0.006079	0.001173	8645.734
2.000	0.147	11.607124	0.00600685	0.008467	0.007178	0.001389	8353.932





ANALYSIS OF RESULTS		
EXPRESSIONS	STATISTICS	APPLICATION
MR = MRmax WHEN $\varepsilon a \le 0.0001$ (1) $MR = K1 \cdot \sigma d^{K2}$ or WHEN $\varepsilon a > 0.0001$ (2) $MR = N1 \cdot \varepsilon a^{N2}$	MODEL: LOG (εa) = A + B * LOG (σd) R^2 = 0.789 AND SEE = 0.091 (1) K1 = 19788 AND K2 = -0.208 (2) N1 = 3598 AND N2 = -0.172 MRmax = 17593 psi	SAY σd = 6 psi USING Eq. (1): MR = 13627 psi Q: MR < MRmax ? ... No **MR(design) = 13,627 psi**

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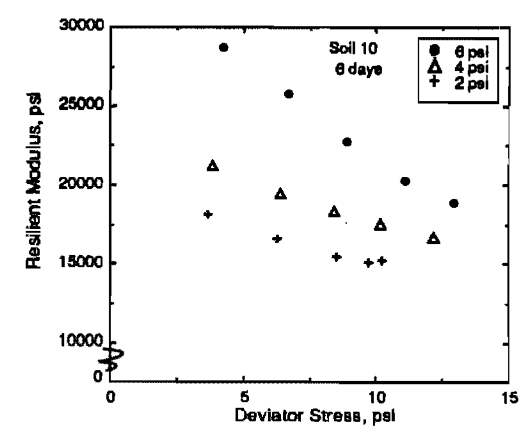
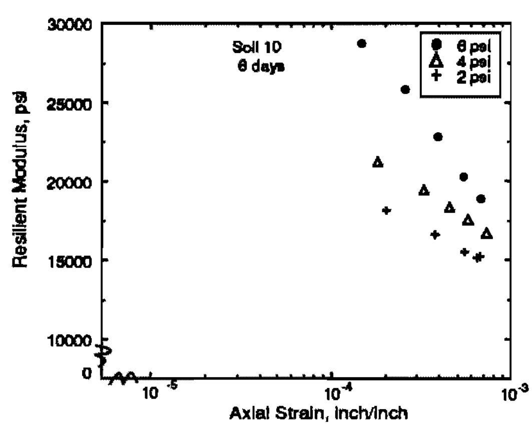
soil-10b.out — RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = Soil #10 - opt
DESCRIPTION = Dist 5 - Lubbock - FM835 - 6 days
MOISTURE CONTENT = 10.50 percent
DRY DENSITY = 123.80 pcf.
PLASTICITY INDEX = 4.00 percent
LIQUID LIMIT = 20.00 percent
SAMPLE HEIGHT = 5.620 Inches
SAMPLE DIAMETER = 2.840 Inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (Inch)	AXIAL DEFORMATION A (Inch)	AXIAL DEFORMATION B (Inch)	STRAIN (In/In)	M r. (psi)
6.000	-0.023	4.264778	0.00006086	0.001028	0.000640	0.000148	28741.197
6.000	-0.025	6.732825	0.00103082	0.001812	0.001124	0.000261	25774.855
6.000	-0.026	8.932158	0.00296231	0.002697	0.001703	0.000391	22816.771
6.000	-0.025	11.092166	0.00527744	0.003686	0.002458	0.000547	20290.186
6.000	-0.026	12.922605	0.00788726	0.004568	0.003121	0.000684	18891.811
4.000	0.032	3.872942	0.00742219	0.001265	0.000779	0.000182	21299.471
4.000	0.018	6.386400	0.00741305	0.002254	0.001430	0.000328	19488.619
4.000	0.005	8.398008	0.00746490	0.003149	0.002007	0.000459	18309.244
4.000	-0.004	10.150733	0.00757451	0.003927	0.002569	0.000578	17562.805
4.000	-0.015	12.186683	0.00809041	0.004883	0.003332	0.000731	16673.865
2.000	0.138	3.660874	0.00754595	0.001396	0.000873	0.000202	18135.721
2.000	0.126	6.263278	0.00750708	0.002577	0.001663	0.000377	16600.828
2.000	0.118	8.518788	0.00752495	0.003719	0.002464	0.000550	15486.023
2.000	0.115	10.228911	0.00759366	0.004496	0.003074	0.000674	15187.359
2.000	0.110	9.709743	0.00763504	0.004287	0.002917	0.000641	15150.793





ANALYSIS OF RESULTS		
EXPRESSIONS	STATISTICS	APPLICATION
MR = MRmax WHEN $\varepsilon_a \le 0.0001$ (1) $MR = K1 \cdot \sigma_d^{K2}$ or (2) $MR = N1 \cdot \varepsilon_a^{N2}$ WHEN $\varepsilon_a > 0.0001$	MODEL: LOG (ε_a) = A + B * LOG (σ_d) R^2 = 0.893 AND SEE = 0.049 (1) K1 = 28940 AND K2 = -0.206 (2) N1 = 5004 AND N2 = -0.171 MRmax = 24136 psi	SAY σ_d = 6 psi USING Eq. (1): MR = 20006 psi Q: MR < MRmax ? ... No **MR(design) = 20,006 psi**

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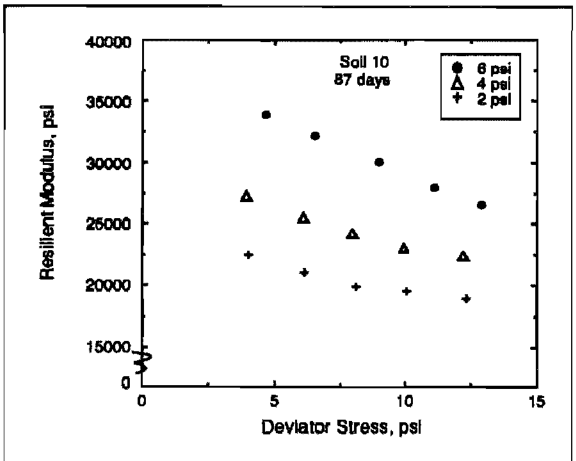
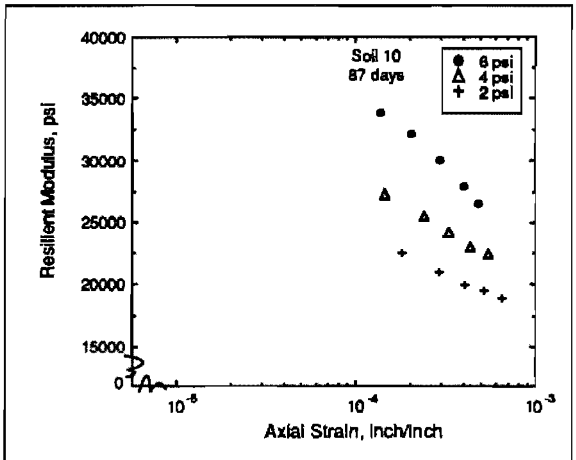
Soil-10c.out — RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = 10
DESCRIPTION = Dist 5 - Lubbock - FM835 - 87 days
MOISTURE CONTENT = 10.80 percent
DRY DENSITY = 123.05 pcf.
PLASTICITY INDEX = 4.00 percent
LIQUID LIMIT = 20.00 percent
SAMPLE HEIGHT = 5.630 inches
SAMPLE DIAMETER = 2.840 inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	-0.048	4.685637	0.00015824	0.000719	0.000810	0.000138	33837.688
6.000	-0.049	6.549314	0.00019194	0.001108	0.001184	0.000204	32176.543
6.000	-0.054	8.939180	0.00028428	0.001640	0.001702	0.000297	30120.570
6.000	-0.053	11.121192	0.00042196	0.002215	0.002259	0.000397	27990.861
6.000	-0.053	12.896855	0.00059371	0.002706	0.002751	0.000485	26608.750
4.000	-0.002	3.952526	0.00007499	0.000781	0.000847	0.000145	27334.082
4.000	-0.012	6.090065	0.00005431	0.001315	0.001371	0.000239	25528.000
4.000	-0.011	7.951400	0.00004456	0.001837	0.001870	0.000329	24151.057
4.000	-0.015	9.932578	0.00005463	0.002430	0.002427	0.000431	23030.777
4.000	-0.023	12.232561	0.00009816	0.003088	0.003045	0.000545	22456.557
2.000	0.156	4.039134	-.00037591	0.000999	0.001023	0.000180	22494.371
2.000	0.142	6.119559	-.00040922	0.001646	0.001638	0.000292	20982.506
2.000	0.136	8.111972	-.00042037	0.002314	0.002267	0.000407	19937.412
2.000	0.133	10.034165	-.00042907	0.002912	0.002853	0.000515	19497.756
2.000	0.123	12.291547	-.00036962	0.003721	0.003573	0.000648	18974.559





ANALYSIS OF RESULTS		
EXPRESSIONS	**STATISTICS**	**APPLICATION**
$MR = MR_{max}$ WHEN $\varepsilon_a \le 0.0001$ (1) $MR = K1 \cdot \sigma_d^{K2}$ or WHEN $\varepsilon_a > 0.0001$ (2) $MR = N1 \cdot \varepsilon_a^{N2}$	MODEL: $LOG(\varepsilon_a) = A + B \cdot LOG(\sigma_d)$ R^2 = 0.875 AND SEE = 0.050 (1) K1 = 32992 AND K2 = -0.143 (2) N1 = 6977 AND N2 = -0.125 MR_{max} = 28415 psi	SAY σ_d = 6 psi USING Eq. (1): MR = 25535 psi Q: MR < MRmax ? ... No **MR(design) = 25,535 psi**

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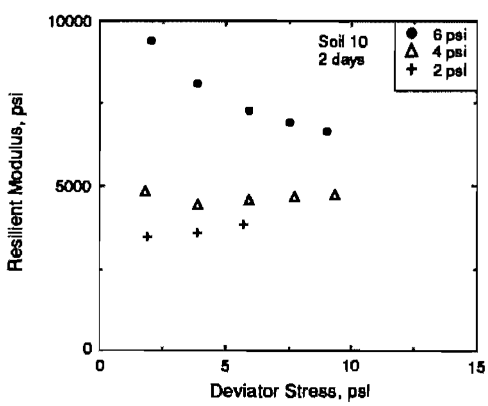
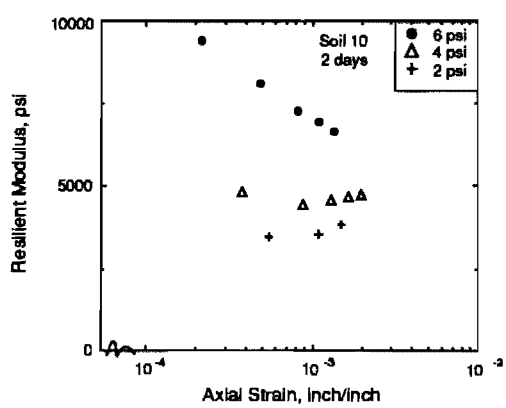
soil-10wa.out RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = Soil 10 - wet
DESCRIPTION = Dist 5 - Lubbock - FM835 - 2 days
MOISTURE CONTENT = 15.10 percent
DRY DENSITY = 117.96 pcf.
PLASTICITY INDEX = 4.00 percent
LIQUID LIMIT = 20.00 percent
SAMPLE HEIGHT = 5.680 Inches
SAMPLE DIAMETER = 2.840 Inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (Inch)	AXIAL DEFORMATION A (Inch)	AXIAL DEFORMATION B (Inch)	STRAIN (In/In)	M r. (psi)
6.000	0.203	2.040633	0.04816192	0.001084	0.001386	0.000217	9386.131
6.000	0.113	3.916948	0.04795244	0.002403	0.003086	0.000483	8106.327
6.000	0.048	5.947750	0.04808686	0.004128	0.005178	0.000819	7260.868
6.000	-0.002	7.583908	0.04846220	0.005595	0.006834	0.001094	6931.754
6.000	-0.011	9.049192	0.04888056	0.007046	0.008482	0.001367	6620.329
4.000	0.421	1.847759	0.04758357	0.001938	0.002398	0.000382	4840.783
4.000	0.347	3.909458	0.04757119	0.004548	0.005467	0.000882	4434.403
4.000	0.284	5.950091	0.04765140	0.006819	0.008007	0.001305	4559.177
4.000	0.198	7.746820	0.04784948	0.008748	0.010098	0.001659	4669.485
4.000	0.156	9.358169	0.04885074	0.010513	0.011982	0.001980	4726.023
2.000	0.572	1.911425	0.04845643	0.002834	0.003401	0.000549	3482.728
2.000	0.536	3.909457	0.04904523	0.005730	0.006732	0.001097	3563.832
2.000	0.479	5.747853	0.04987992	0.007861	0.009113	0.001494	3846.847





ANALYSIS OF RESULTS		
EXPRESSIONS	**STATISTICS**	**APPLICATION**
MR = MRmax WHEN $\varepsilon a \le 0.0001$ (1) $MR = K1 \cdot \sigma d^{K2}$ or WHEN $\varepsilon a > 0.0001$ (2) $MR = N1 \cdot \varepsilon a^{N2}$	MODEL: LOG (εa) = A + B * LOG (σd) R^2 = 0.741 AND SEE = 0.093 (1) K1 = 5129 AND K2 = -0.008 (2) N1 = 4783 AND N2 = -0.008 MRmax = 5157 psi	SAY σd = 6 psi USING Eq. (1): MR = 5054 psi Q: MR < MRmax ? ... No **MR(design) = 5,054 psi**

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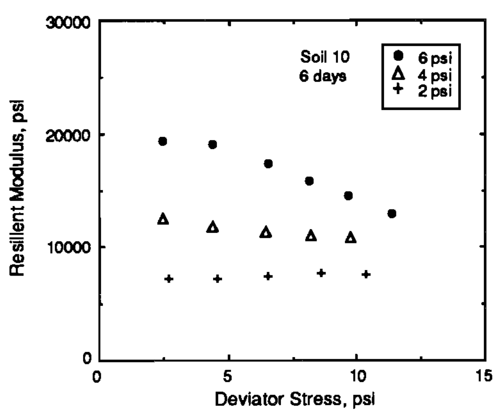
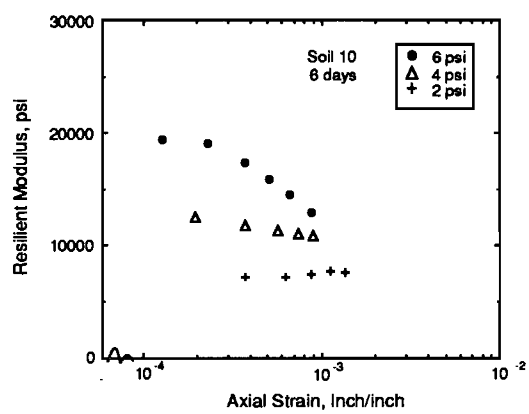
soil-10wb.out — RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = Soil 10 - wet
DESCRIPTION = Dist 5 - Lubbock - FM835 - 6 days
MOISTURE CONTENT = 15.10 percent
DRY DENSITY = 117.96 pcf.
PLASTICITY INDEX = 4.00 percent
LIQUID LIMIT = 20.00 percent
SAMPLE HEIGHT = 5.670 inches
SAMPLE DIAMETER = 2.840 inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	0.293	2.477409	-.00006021	0.000613	0.000834	0.000128	19410.178
6.000	0.169	4.401007	-.00007760	0.001106	0.001505	0.000230	19114.455
6.000	0.093	6.528247	0.00003453	0.001815	0.002443	0.000375	17388.152
6.000	0.028	8.155978	0.00016798	0.002512	0.003319	0.000514	15861.619
6.000	-0.012	9.689143	0.00036126	0.003321	0.004277	0.000670	14460.776
6.000	-0.014	11.362286	0.00064223	0.004447	0.005532	0.000880	12912.276
4.000	0.333	2.460556	0.00032088	0.000955	0.001272	0.000196	12528.710
4.000	0.247	4.384154	0.00028428	0.001845	0.002381	0.000373	11763.127
4.000	0.195	6.455216	0.00026474	0.002868	0.003628	0.000573	11268.783
4.000	0.120	8.202323	0.00024520	0.003761	0.004685	0.000745	11012.599
4.000	0.096	9.774815	0.00029515	0.004607	0.005613	0.000901	10845.989
2.000	0.482	2.674966	-.00029797	0.001901	0.002339	0.000374	7153.640
2.000	0.403	4.587328	-.00036805	0.003262	0.003940	0.000635	7222.311
2.000	0.366	6.526842	-.00040068	0.004548	0.005401	0.000877	7439.849
2.000	0.293	8.594627	-.00044347	0.005853	0.006866	0.001122	7663.042
2.000	0.248	10.347820	-.00034340	0.007211	0.008319	0.001369	7556.157





ANALYSIS OF RESULTS		
EXPRESSIONS	STATISTICS	APPLICATION
MR = MRmax WHEN $\varepsilon a \le 0.0001$ (1) MR = K1 * σd^{K2} or (2) MR = N1 * εa^{N2} WHEN $\varepsilon a > 0.0001$	MODEL: LOG (εa) = A + B * LOG (σd) R^2 = 0.713 AND SEE = 0.103 (1) K1 = 13895 AND K2 = -0.117 (2) N1 = 5117 AND N2 = -0.105 MRmax = 13425 psi	SAY σd = 6 psi USING Eq. (1): MR = 11268 psi Q: MR < MRmax ? ... No **MR(design) = 11,268 psi**

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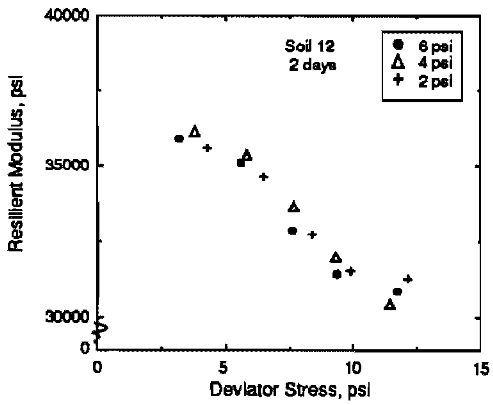
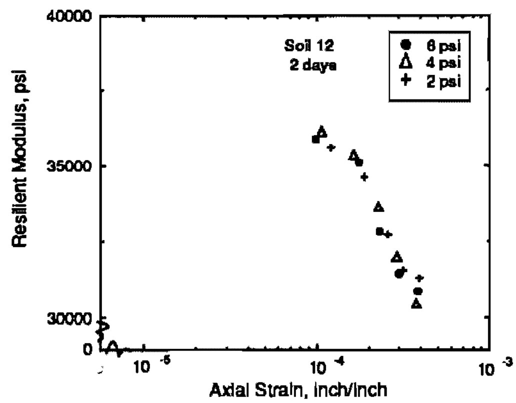
soil-12a.out RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = Soil 12 - opt
DESCRIPTION = Dist 20 - Jasper - FM252 - 2 days
MOISTURE CONTENT = 20.70 percent
DRY DENSITY = 83.97 pcf.
PLASTICITY INDEX = 52.10 percent
LIQUID LIMIT = 79.30 percent
SAMPLE HEIGHT = 5.790 Inches
SAMPLE DIAMETER = 2.770 Inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (Inch)	AXIAL DEFORMATION A (Inch)	AXIAL DEFORMATION B (Inch)	STRAIN (In/In)	M r. (psi)
6.000	0.517	3.133603	-.00006504	0.000285	0.000825	0.000096	35758.457
6.000	0.386	5.930937	-.00002833	0.000447	0.001505	0.000169	35109.141
6.000	-0.016	7.599531	0.00195415	0.000578	0.002102	0.000231	32839.109
6.000	-0.017	9.364699	0.00219219	0.000730	0.002707	0.000297	31479.414
6.000	-0.018	11.741554	0.00270020	0.000990	0.003410	0.000380	30902.736
4.000	0.091	3.815759	0.00243271	0.000258	0.000965	0.000106	36132.238
4.000	0.058	5.804343	0.00242382	0.000389	0.001512	0.000164	35367.512
4.000	0.039	7.672363	0.00242682	0.000539	0.002104	0.000228	33620.852
4.000	0.036	9.301220	0.00245348	0.000693	0.002671	0.000291	32013.598
4.000	0.024	11.450230	0.00249285	0.000931	0.003420	0.000376	30478.631
2.000	0.271	4.285717	0.00217203	0.000301	0.001093	0.000120	35593.828
2.000	0.214	6.500175	0.00215163	0.000446	0.001727	0.000188	34649.082
2.000	0.192	8.352940	0.00214243	0.000600	0.002354	0.000255	32737.012
2.000	0.180	9.881900	0.00212174	0.000755	0.002869	0.000313	31575.785
2.000	0.163	12.139666	0.00207412	0.000990	0.003496	0.000387	31337.080





ANALYSIS OF RESULTS		
EXPRESSIONS	STATISTICS	APPLICATION
MR = MRmax WHEN $\varepsilon a \le 0.0001$ (1) MR = K1 * σd^{K2} or WHEN $\varepsilon a > 0.0001$ (2) MR = N1 * εa^{N2}	MODEL: LOG (εa) = A + B * LOG (σd) R^2 = 0.996 AND SEE = 0.008 (1) K1 = 40865 AND K2 = -0.108 (2) N1 = 14480 AND N2 = -0.098 MRmax = 35613 psi	SAY σd = 6 psi USING Eq. (1): MR = 33657 psi Q: MR < MRmax ? ... No **MR(design) = 33,657 psi**

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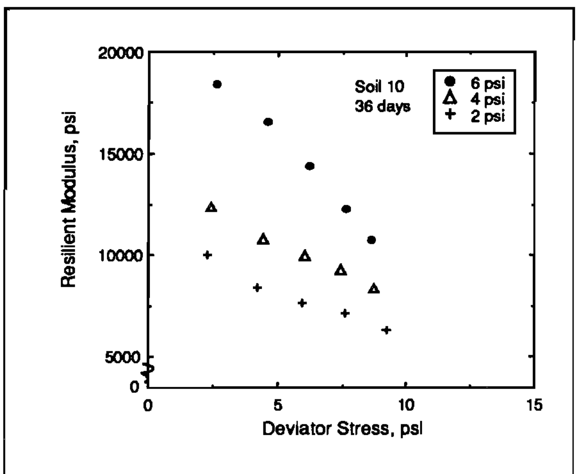
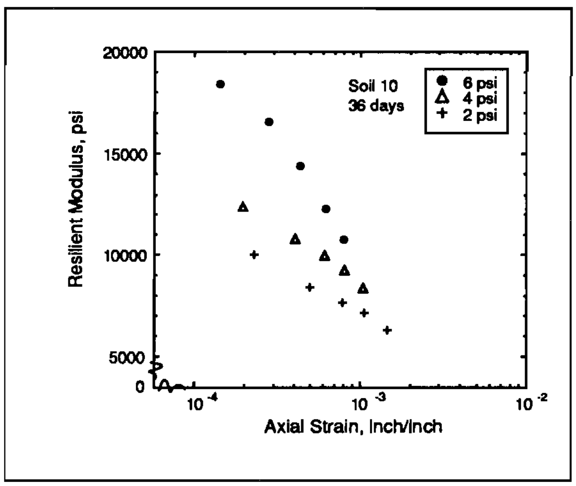
soil-10wc.out — RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = Soil 10 - wet
DESCRIPTION = Dist 5 - Lubbock - FM835 - 36 days
MOISTURE CONTENT = 14.00 percent
DRY DENSITY = 118.20 pcf.
PLASTICITY INDEX = 4.00 percent
LIQUID LIMIT = 20.00 percent
SAMPLE HEIGHT = 5.810 inches
SAMPLE DIAMETER = 2.840 inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	-0.027	2.627684	0.00005033	0.000883	0.000778	0.000143	18384.518
6.000	-0.027	4.614948	0.00067925	0.001789	0.001448	0.000279	16565.039
6.000	-0.028	6.236125	0.00211860	0.002802	0.002232	0.000433	14397.710
6.000	-0.027	7.643830	0.00514529	0.004066	0.003158	0.000622	12294.777
6.000	-0.026	8.630673	0.00804631	0.005280	0.004034	0.000802	10766.646
4.000	0.135	2.439022	0.00707100	0.001294	0.000994	0.000197	12388.317
4.000	0.084	4.421137	0.00707391	0.002696	0.002050	0.000408	10823.392
4.000	0.053	6.064317	0.00715333	0.004055	0.003027	0.000610	9949.600
4.000	0.034	7.426611	0.00749302	0.005324	0.004011	0.000803	9244.823
4.000	-0.005	8.743964	0.00857603	0.006926	0.005218	0.001045	8366.469
2.000	0.263	2.278449	0.00793365	0.001526	0.001126	0.000228	9985.898
2.000	0.218	4.205791	0.00778463	0.003355	0.002477	0.000502	8380.019
2.000	0.198	5.959453	0.00774792	0.005217	0.003864	0.000781	7626.025
2.000	0.167	7.596080	0.00807752	0.007078	0.005260	0.001062	7153.906
2.000	0.121	9.203212	0.00978673	0.009675	0.007241	0.001456	6321.994





ANALYSIS OF RESULTS		
EXPRESSIONS	STATISTICS	APPLICATION
MR = MRmax WHEN $\varepsilon a \le 0.0001$ (1) $MR = K1 * \sigma d^{K2}$ or WHEN $\varepsilon a > 0.0001$ (2) $MR = N1 * \varepsilon a^{N2}$	MODEL: LOG (εa) = A + B * LOG (σd) R^2 = 0.846 AND SEE = 0.077 (1) K1 = 17616 AND K2 = -0.316 (2) N1 = 1685 AND N2 = -0.240 MRmax = 15377 psi	SAY σd = 6 psi USING Eq. (1): MR = 10003 psi Q: MR < MRmax ? ... No **MR(design) = 10,003 psi**

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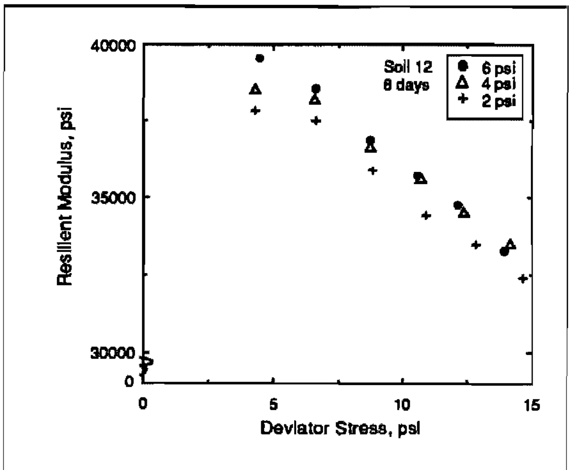
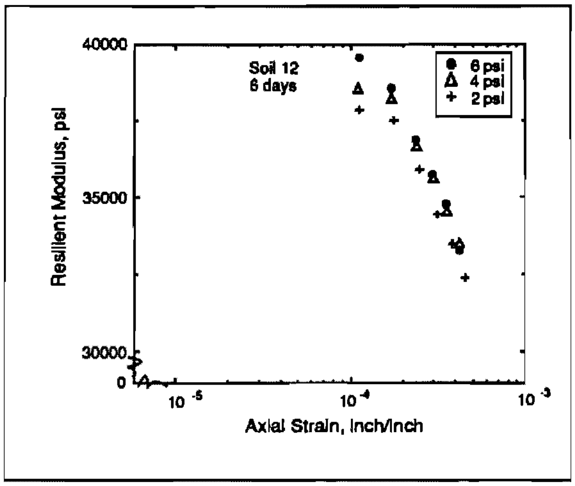
soil-12b.out

RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = Soil 12 - opt
DESCRIPTION = Dist 20 - Jasper - FM252 - 6 days
MOISTURE CONTENT = 20.60 percent
DRY DENSITY = 85.60 pcf.
PLASTICITY INDEX = 52.10 percent
LIQUID LIMIT = 79.30 percent
SAMPLE HEIGHT = 5.650 inches
SAMPLE DIAMETER = 2.850 inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	0.278	4.480814	-.00010202	0.000773	0.000508	0.000113	39532.984
6.000	0.182	6.613141	0.00000995	0.001156	0.000783	0.000172	38537.738
6.000	0.116	8.717575	0.00020536	0.001594	0.001078	0.000236	36865.527
6.000	0.009	10.566801	0.00036830	0.001976	0.001366	0.000296	35726.672
6.000	-0.017	12.155237	0.00058602	0.002284	0.001667	0.000350	34762.793
6.000	-0.019	13.943100	0.00087097	0.002650	0.002085	0.000419	33273.145
4.000	0.533	4.286036	0.00048291	0.000756	0.000501	0.000111	38528.391
4.000	0.438	6.589433	0.00045900	0.001178	0.000770	0.000172	38211.664
4.000	0.334	8.727339	0.00044789	0.001607	0.001081	0.000238	36681.512
4.000	0.226	10.701612	0.00044559	0.001991	0.001404	0.000300	35626.281
4.000	0.166	12.362102	0.00044994	0.002329	0.001716	0.000358	34538.711
4.000	0.138	14.151359	0.00047815	0.002687	0.002088	0.000423	33487.707
2.000	0.700	4.279064	0.00022929	0.000758	0.000520	0.000113	37832.172
2.000	0.576	6.604773	0.00020186	0.001194	0.000797	0.000176	37484.758
2.000	0.456	8.803576	0.00018329	0.001635	0.001135	0.000245	35913.707
2.000	0.322	10.898249	0.00014974	0.002076	0.001502	0.000317	34423.516
2.000	0.282	12.817203	0.00015637	0.002464	0.001862	0.000383	33475.629
2.000	0.258	14.633888	0.00016126	0.002856	0.002245	0.000451	32412.975





ANALYSIS OF RESULTS		
EXPRESSIONS	**STATISTICS**	**APPLICATION**
MR = MRmax WHEN $\varepsilon_a \leq 0.0001$ (1) $MR = K1 \cdot \sigma_d^{K2}$ or WHEN $\varepsilon_a > 0.0001$ (2) $MR = N1 \cdot \varepsilon_a^{N2}$	MODEL: $LOG(\varepsilon_a) = A + B \cdot LOG(\sigma_d)$ R^2 = 0.998 AND SEE = 0.006 (1) K1 = 46738 AND K2 = -0.118 (2) N1 = 14975 AND N2 = -0.106 MRmax = 39699 psi	SAY σ_d = 6 psi USING Eq. (1): MR = 37805 psi Q: MR < MRmax ? ... No **MR(design) = 37,805 psi**

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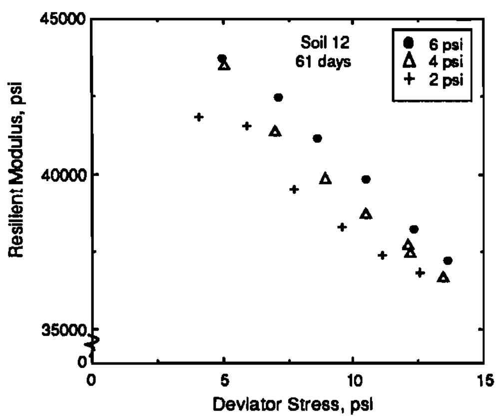
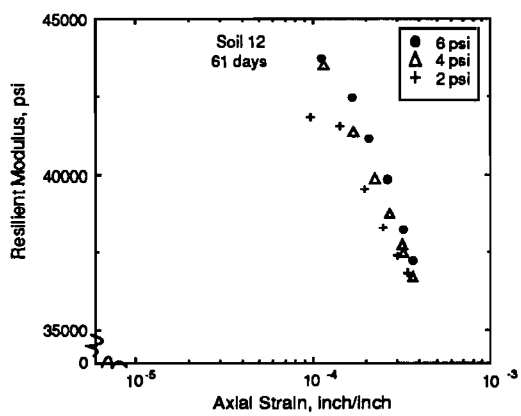
soil-12o.out

RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = Soil 12 - opt
DESCRIPTION = Dist 20 - Jasper - FM252 - 61 days
MOISTURE CONTENT = 20.60 percent
DRY DENSITY = 85.53 pcf.
PLASTICITY INDEX = 52.10 percent
LIQUID LIMIT = 79.30 percent
SAMPLE HEIGHT = 5.650 Inches
SAMPLE DIAMETER = 2.850 Inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (Inch)	AXIAL DEFORMATION A (Inch)	AXIAL DEFORMATION B (Inch)	STRAIN (In/In)	M r. (psi)
6.000	0.286	4.962413	-.00012541	0.000759	0.000523	0.000113	43733.949
6.000	0.177	7.117517	-.00008748	0.001111	0.000782	0.000168	42480.133
6.000	0.075	8.618561	-.00004645	0.001373	0.000992	0.000209	41174.250
6.000	0.008	10.482195	0.00001332	0.001695	0.001275	0.000263	39882.645
6.000	-0.014	12.332351	0.00010855	0.002040	0.001604	0.000322	38246.363
6.000	-0.018	13.631177	0.00018648	0.002288	0.001850	0.000366	37216.496
4.000	0.384	5.040045	0.00002587	0.000792	0.000517	0.000116	43517.141
4.000	0.309	6.997118	0.00002772	0.001145	0.000764	0.000169	41416.313
4.000	0.195	8.892829	0.00002994	0.001480	0.001039	0.000223	39904.305
4.000	0.152	10.475688	0.00003769	0.001758	0.001300	0.000271	38717.430
4.000	0.131	12.079465	0.00005140	0.002041	0.001577	0.000320	37727.285
4.000	0.074	12.191961	0.00001979	0.002075	0.001604	0.000326	37454.051
4.000	0.038	13.442443	0.00003301	0.002305	0.001835	0.000366	36683.938
2.000	0.641	4.077313	-.00014506	0.000662	0.000439	0.000097	41862.723
2.000	0.519	5.903295	-.00017010	0.000975	0.000631	0.000142	41557.625
2.000	0.427	7.682326	-.00017936	0.001307	0.000888	0.000194	39544.363
2.000	0.266	9.529225	-.00020112	0.001648	0.001164	0.000249	38292.965
2.000	0.221	11.108832	-.00020256	0.001932	0.001428	0.000297	37361.504
2.000	0.190	12.543398	-.00020108	0.002179	0.001671	0.000341	36817.125





ANALYSIS OF RESULTS		
EXPRESSIONS	**STATISTICS**	**APPLICATION**
MR = MRmax WHEN $\varepsilon a \le 0.0001$ (1) MR = K1 * σd^{K2} or WHEN $\varepsilon a > 0.0001$ (2) MR = N1 * εa^{N2}	MODEL: LOG (εa) = A + B * LOG (σd) R^2 = 0.997 AND SEE = 0.006 (1) K1 = 56792 AND K2 = -0.164 (2) N1 = 12172 AND N2 = -0.141 MRmax = 44480 psi	SAY σd = 6 psi USING Eq. (1): MR = 42352 psi Q: MR < MRmax ? ... No **MR(design) = 42,352 psi**

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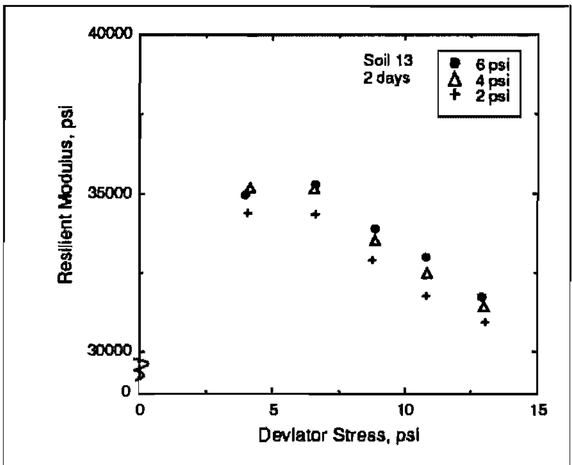
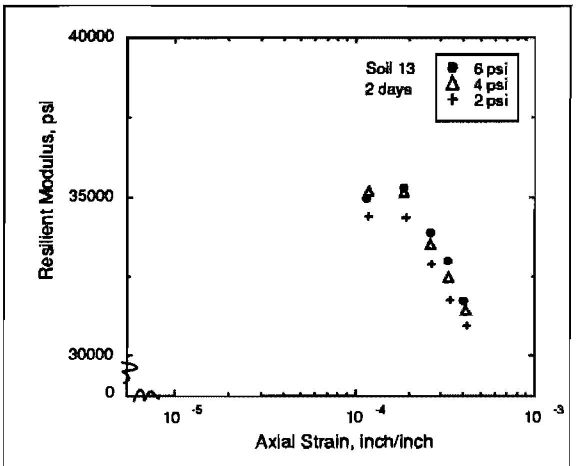
soil-13a.out — RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = Soil 13 - opt
DESCRIPTION = Dist 20 - Jefferson - US69 - 2 days
MOISTURE CONTENT = 18.00 percent
DRY DENSITY = 100.20 pcf.
PLASTICITY INDEX = 35.90 percent
LIQUID LIMIT = 54.10 percent
SAMPLE HEIGHT = 5.610 inches
SAMPLE DIAMETER = 2.840 inches

CONFINE (psi)	SEATING (psi)	DEVIR STRESS (psi)	PER DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	0.065	4.006831	-.00003857	0.000737	0.000548	0.000115	34971.297
6.000	0.041	6.628897	-.00001995	0.001174	0.000933	0.000188	35307.637
6.000	0.035	8.882068	0.00000708	0.001611	0.001329	0.000262	33903.949
6.000	0.025	10.797236	0.00004865	0.001995	0.001677	0.000327	32990.789
6.000	0.011	12.888431	0.00010637	0.002446	0.002107	0.000406	31759.867
4.000	0.134	4.152424	-.00002667	0.000757	0.000566	0.000118	35201.852
4.000	0.124	6.579274	-.00002549	0.001174	0.000926	0.000187	35158.961
4.000	0.117	8.876450	-.00001909	0.001635	0.001334	0.000265	33544.773
4.000	0.106	10.829070	-.00000587	0.002031	0.001708	0.000333	32499.207
4.000	0.080	13.006400	0.00001444	0.002488	0.002151	0.000413	31456.441
2.000	0.239	4.056922	-.00018675	0.000746	0.000577	0.000118	34400.422
2.000	0.220	6.623746	-.00019512	0.001194	0.000968	0.000193	34369.773
2.000	0.216	8.779077	-.00019239	0.001633	0.001361	0.000267	32894.520
2.000	0.203	10.784129	-.00018655	0.002051	0.001752	0.000339	31813.662
2.000	0.186	13.038705	-.00017103	0.002518	0.002205	0.000421	30975.678





ANALYSIS OF RESULTS		
EXPRESSIONS	STATISTICS	APPLICATION
$MR = MR_{max}$ WHEN $\varepsilon_a \le 0.0001$ (1) $MR = K1 \cdot \sigma_d^{K2}$ or WHEN $\varepsilon_a > 0.0001$ (2) $MR = N1 \cdot \varepsilon_a^{N2}$	MODEL: $\text{LOG}(\varepsilon_a) = A + B \cdot \text{LOG}(\sigma_d)$ R^2 = 0.998 AND SEE = 0.006 (1) K1 = 40272 AND K2 = -0.090 (2) N1 = 16744 AND N2 = -0.083 MRmax = 35886 psi	SAY σ_d = 6 psi USING Eq. (1): MR = 34260 psi Q: MR < MRmax ? ... No **MR(design) = 34,260 psi**

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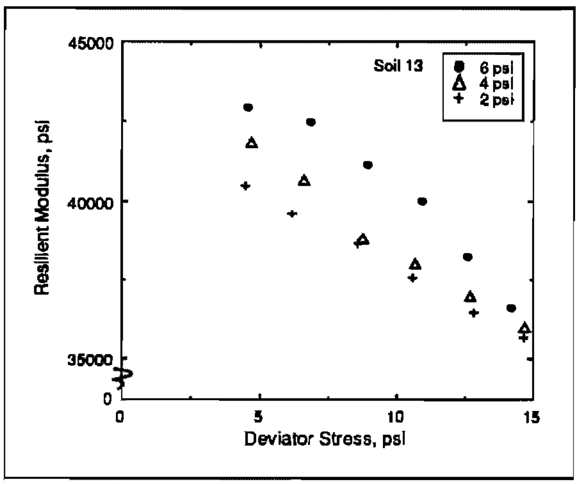
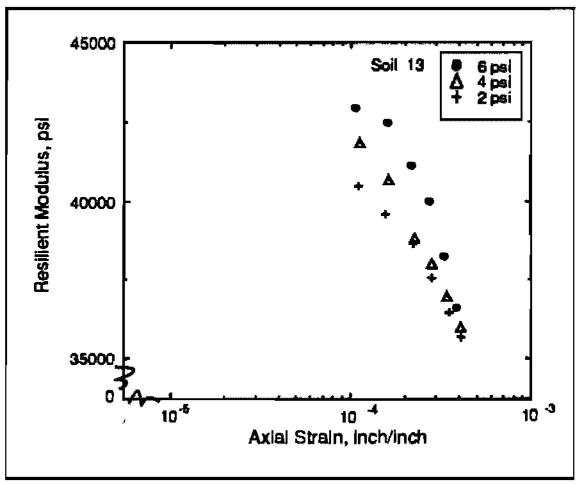
soil-13b.out — RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = Soil 13 - opt
DESCRIPTION = Dist. 20 - Jefferson - US 69 - 6 days
MOISTURE CONTENT = 18.00 percent
DRY DENSITY = 100.20 pcf.
PLASTICITY INDEX = 35.90 percent
LIQUID LIMIT = 54.10 percent
SAMPLE HEIGHT = 5.610 inches
SAMPLE DIAMETER = 2.840 inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	0.301	4.540514	-.00001609	0.000661	0.000526	0.000106	42933.563
6.000	0.204	6.831135	0.00001071	0.000983	0.000822	0.000161	42457.840
6.000	0.096	8.935435	0.00006920	0.001325	0.001112	0.000217	41140.145
6.000	0.067	10.916146	0.00012267	0.001680	0.001382	0.000273	39995.852
6.000	0.044	12.603800	0.00016785	0.002013	0.001685	0.000330	38241.625
6.000	0.030	14.186119	0.00023107	0.002339	0.002005	0.000387	36643.730
4.000	0.158	4.667849	0.00010114	0.000706	0.000546	0.000112	41825.480
4.000	0.144	6.589572	0.00010609	0.001010	0.000808	0.000162	40663.125
4.000	0.126	8.750519	0.00011012	0.001389	0.001141	0.000226	38799.289
4.000	0.113	10.664285	0.00011965	0.001715	0.001434	0.000281	38003.266
4.000	0.102	12.670276	0.00013113	0.002062	0.001780	0.000342	36999.215
4.000	0.084	14.673456	0.00014732	0.002430	0.002142	0.000407	36009.574
2.000	0.301	4.479187	0.00000425	0.000690	0.000550	0.000111	40514.195
2.000	0.280	6.166371	0.00000136	0.000962	0.000785	0.000156	39596.367
2.000	0.248	8.562325	-.00000122	0.001346	0.001140	0.000222	38645.082
2.000	0.218	10.368784	0.00000182	0.001689	0.001466	0.000281	37580.453
2.000	0.194	12.815868	-.00000585	0.002090	0.001854	0.000352	36454.250
2.000	0.178	14.659879	-.00001277	0.002424	0.002187	0.000411	35672.340





ANALYSIS OF RESULTS		
EXPRESSIONS	STATISTICS	APPLICATION
MR = MRmax WHEN $\varepsilon_a \le 0.0001$ (1) $MR = K1 \cdot \sigma_d^{K2}$ or (2) $MR = N1 \cdot \varepsilon_a^{N2}$ WHEN $\varepsilon_a > 0.0001$	MODEL: LOG (ε_a) = A + B * LOG (σ_d) R^2 = 0.995 AND SEE = 0.009 (1) K1 = 49120 AND K2 = -0.104 (2) N1 = 17746 AND N2 = -0.094 MRmax = 42277 psi	SAY σ_d = 6 psi USING Eq. (1): MR = 40765 psi Q: MR < MRmax ? ... No **MR(design) = 40,765 psi**

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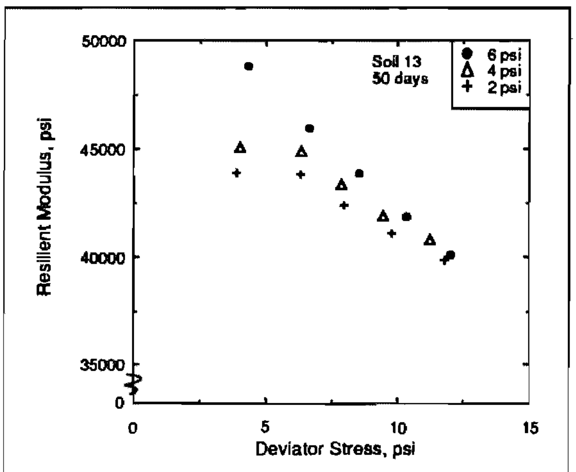
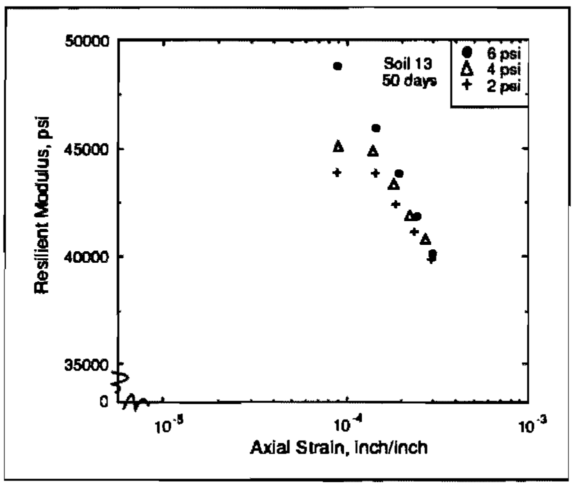
soil-13c.out — RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = Soil 13 - opt
DESCRIPTION = Dist 20 - Jefferson - US69 - 50 days
MOISTURE CONTENT = 17.00 percent
DRY DENSITY = 105.50 pcf.
PLASTICITY INDEX = 35.90 percent
LIQUID LIMIT = 54.10 percent
SAMPLE HEIGHT = 5.760 inches
SAMPLE DIAMETER = 2.840 inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	0.146	4.340617	0.01086257	0.000643	0.000382	0.000089	48778.840
6.000	0.092	6.676648	0.01092253	0.001096	0.000578	0.000145	45955.574
6.000	0.043	8.554834	0.01100363	0.001513	0.000734	0.000195	43860.594
6.000	-0.021	10.332372	0.01108046	0.001955	0.000888	0.000247	41871.613
6.000	-0.021	11.994277	0.01116085	0.002418	0.001029	0.000299	40084.875
4.000	0.292	4.036325	0.01097170	0.000662	0.000369	0.000090	45087.777
4.000	0.246	6.349416	0.01097035	0.001109	0.000521	0.000141	44882.652
4.000	0.213	7.899436	0.01097222	0.001448	0.000650	0.000182	43358.852
4.000	0.163	9.437752	0.01097651	0.001812	0.000782	0.000225	41904.781
4.000	0.081	11.234482	0.01098461	0.002231	0.000942	0.000276	40777.473
2.000	0.501	3.890731	0.01072796	0.000663	0.000357	0.000089	43919.586
2.000	0.421	6.323200	0.01072165	0.001136	0.000526	0.000144	43840.926
2.000	0.396	7.965912	0.01070533	0.001504	0.000660	0.000188	42392.484
2.000	0.326	9.745790	0.01070441	0.001915	0.000816	0.000237	41106.496
2.000	0.238	11.782208	0.01070941	0.002393	0.001013	0.000296	39857.184





ANALYSIS OF RESULTS		
EXPRESSIONS	STATISTICS	APPLICATION
MR = MRmax WHEN $\varepsilon a \le 0.0001$ (1) $MR = K1 \cdot \sigma d^{K2}$ or WHEN $\varepsilon a > 0.0001$ (2) $MR = N1 \cdot \varepsilon a^{N2}$	MODEL: LOG (εa) = A + B * LOG (σd) R^2 = 0.997 AND SEE = 0.008 (1) K1 = 52034 AND K2 = -0.122 (2) N1 = 16016 AND N2 = -0.109 MRmax = 43508 psi	SAY σd = 6 psi USING Eq. (1): MR = 41839 psi Q: MR < MRmax ? ... No **MR(design) = 41,839 psi**

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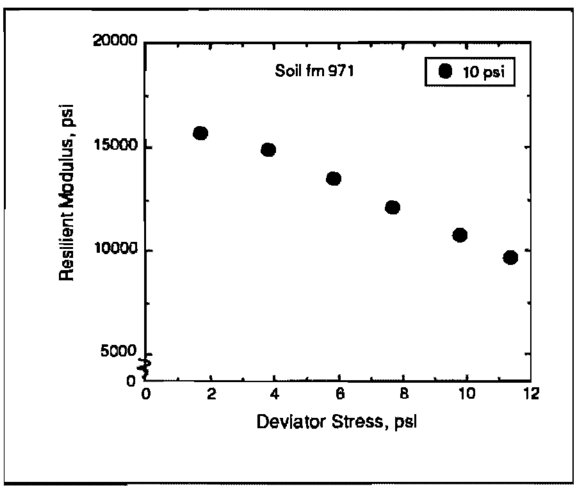
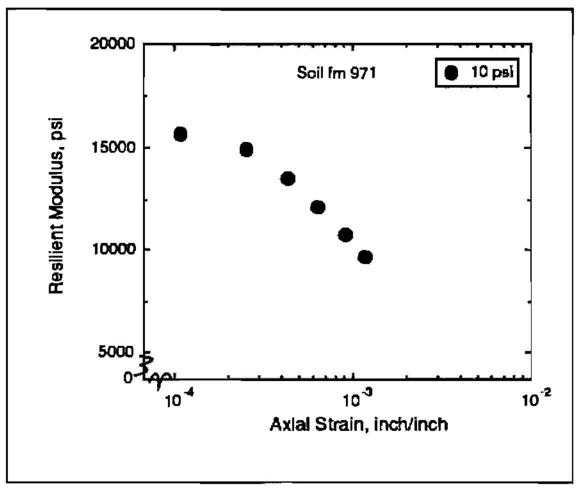
fm971-10.out RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = 14
DESCRIPTION = Dist 14 - Williamson - FM971 - undi - 10 psi
MOISTURE CONTENT = 30.00 percent
DRY DENSITY = 93.20 pcf.
PLASTICITY INDEX = 43.00 percent
LIQUID LIMIT = 66.00 percent
SAMPLE HEIGHT = 5.740 inches
SAMPLE DIAMETER = 2.840 inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
10.000	0.125	1.714806	-.00004919	0.000733	0.000524	0.000109	15669.786
10.000	-0.022	3.811616	-.00008936	0.001737	0.001200	0.000256	14897.340
10.000	-0.028	5.848504	-.00004435	0.002966	0.002009	0.000433	13495.869
10.000	-0.027	7.687834	0.00005342	0.004323	0.002978	0.000636	12088.836
10.000	-0.027	9.771069	0.00019740	0.006155	0.004326	0.000913	10701.768
10.000	-0.026	11.386629	0.00036358	0.007868	0.005658	0.001178	9664.305





ANALYSIS OF RESULTS		
EXPRESSIONS	STATISTICS	APPLICATION
MR = MRmax WHEN $\varepsilon a \le 0.0001$ (1) $MR = K1 * \sigma d^{K2}$ or WHEN $\varepsilon a > 0.0001$ (2) $MR = N1 * \varepsilon a^{N2}$		

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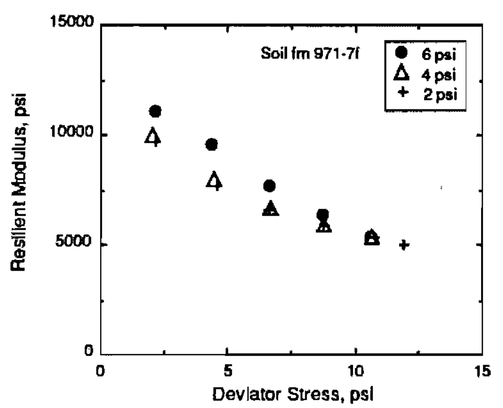
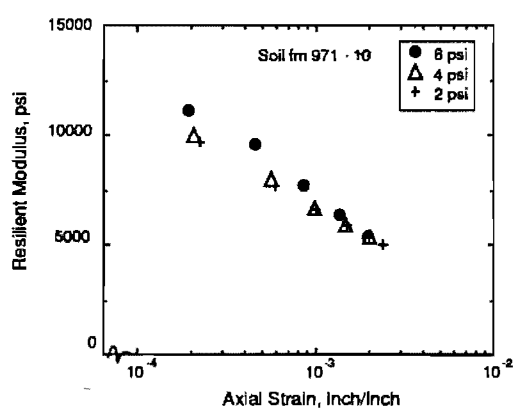
fm971-7f.out

RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = fm971 - 7 feet deep
DESCRIPTION = Dist 14 - Williamson - FM971 - 7 feet - undist
MOISTURE CONTENT = 30.00 percent
DRY DENSITY = 93.20 pcf.
PLASTICITY INDEX = 43.00 percent
LIQUID LIMIT = 66.00 percent
SAMPLE HEIGHT = 5.750 inches
SAMPLE DIAMETER = 2.840 inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	0.385	2.140815	-.00002358	0.001272	0.000949	0.000193	11080.229
6.000	0.292	4.357938	0.00001439	0.002992	0.002227	0.000454	9603.618
6.000	0.112	6.607831	0.00069132	0.005791	0.004058	0.000856	7715.293
6.000	-0.019	8.686383	0.00208044	0.009079	0.006505	0.001355	6409.894
6.000	-0.027	10.595936	0.00538040	0.013103	0.009524	0.001968	5385.312
4.000	0.524	2.054209	0.00546037	0.001378	0.000983	0.000205	10008.308
4.000	0.406	4.476378	0.00543119	0.003781	0.002661	0.000560	7992.054
4.000	0.252	6.652305	0.00542500	0.006665	0.004737	0.000991	6709.393
4.000	0.135	8.784694	0.00557073	0.009894	0.007136	0.001481	5932.359
4.000	0.112	10.671307	0.00603708	0.013176	0.009635	0.001984	5379.826
2.000	0.554	2.145965	0.00593720	0.001495	0.001054	0.000222	9678.741
2.000	0.449	4.537237	0.00588552	0.003951	0.002797	0.000587	7732.232
2.000	0.321	6.618130	0.00590366	0.006687	0.004784	0.000998	6634.527
2.000	0.213	8.734601	0.00601278	0.009905	0.007180	0.001486	5879.283
2.000	0.190	10.741060	0.00632188	0.013279	0.009737	0.002001	5366.739
2.000	0.182	11.905332	0.00718149	0.015724	0.011680	0.002383	4996.053





ANALYSIS OF RESULTS		
EXPRESSIONS	STATISTICS	APPLICATION
MR = MRmax WHEN $\varepsilon a \le 0.0001$ (1) $MR = K1 \cdot \sigma d^{K2}$ or WHEN $\varepsilon a > 0.0001$ (2) $MR = N1 \cdot \varepsilon a^{N2}$		

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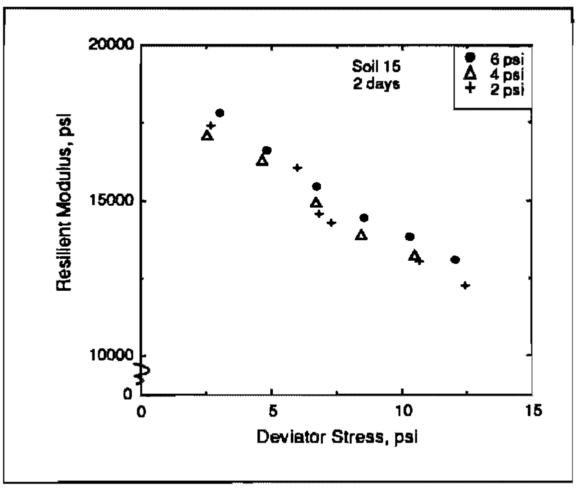
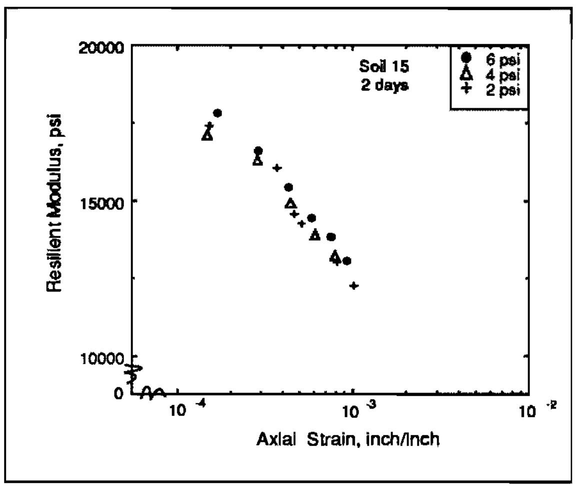
soil-15a.out

RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = Soil 15 - opt
DESCRIPTION = Dist 7 - Tom Green - US67 - 2 days
MOISTURE CONTENT = 20.70 percent
DRY DENSITY = 105.92 pcf.
PLASTICITY INDEX = 40.00 percent
LIQUID LIMIT = 58.00 percent
SAMPLE HEIGHT = 5.610 inches
SAMPLE DIAMETER = 2.840 inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	-0.032	3.028414	-.00010718	0.000828	0.001082	0.000170	17791.016
6.000	-0.033	4.816249	-.00008601	0.001465	0.001792	0.000290	16590.961
6.000	-0.033	6.724865	-.00005317	0.002259	0.002632	0.000436	15424.354
6.000	-0.035	8.515979	-.00000309	0.003101	0.003506	0.000589	14463.488
6.000	-0.037	10.322073	0.00010034	0.003977	0.004405	0.000747	13818.371
6.000	-0.041	12.068710	0.00037527	0.004966	0.005408	0.000925	13052.772
4.000	-0.012	2.546695	-.00065522	0.000730	0.000940	0.000149	17111.486
4.000	-0.024	4.664104	-.00067809	0.001466	0.001746	0.000286	16288.982
4.000	-0.024	6.702863	-.00068171	0.002351	0.002673	0.000448	14968.617
4.000	-0.015	8.453716	-.00067954	0.003233	0.003586	0.000608	13910.563
4.000	-0.011	10.469068	-.00064972	0.004244	0.004634	0.000791	13229.986
2.000	0.086	2.669816	-.00123045	0.000749	0.000971	0.000153	17409.900
2.000	0.109	5.965071	-.00122849	0.001940	0.002224	0.000371	16074.242
2.000	0.118	6.787130	-.00122559	0.002449	0.002769	0.000465	14594.757
2.000	0.116	7.265570	-.00121922	0.002697	0.003019	0.000509	14261.929
2.000	0.111	10.658668	-.00112353	0.004399	0.004787	0.000819	13019.340
2.000	0.118	12.452123	-.00105228	0.005480	0.005898	0.001014	12279.926





ANALYSIS OF RESULTS		
EXPRESSIONS	STATISTICS	APPLICATION
MR = MRmax WHEN $\varepsilon a \le 0.0001$ (1) $MR = K1 * \sigma d^{K2}$ or WHEN $\varepsilon a > 0.0001$ (2) $MR = N1 * \varepsilon a^{N2}$	MODEL: LOG (εa) = A + B * LOG (σd) R^2 = 0.997 AND SEE = 0.009 (1) K1 = 21736 AND K2 = -0.199 (2) N1 = 4138 AND N2 = -0.166 MRmax = 19106 psi	SAY σd = 6 psi USING Eq. (1): MR = 15212 psi Q: MR < MRmax ? ... No **MR(design) = 15,212 psi**

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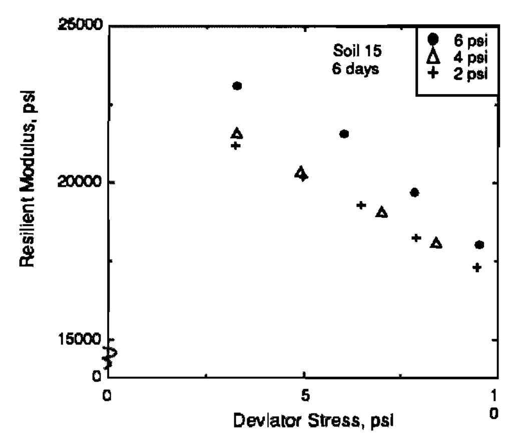
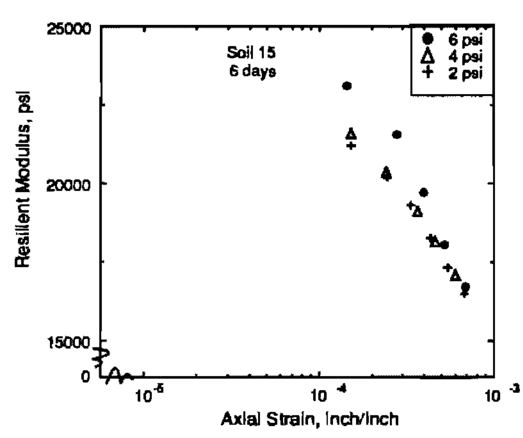
soil-15b.out RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = Soil 15 - opt
DESCRIPTION = Dist 7 - Tom Green - US67 - 6 days
MOISTURE CONTENT = 20.70 percent
DRY DENSITY = 105.92 pcf.
PLASTICITY INDEX = 40.00 percent
LIQUID LIMIT = 58.00 percent
SAMPLE HEIGHT = 5.610 inches
SAMPLE DIAMETER = 2.840 inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	-0.028	3.292914	0.00002876	0.000701	0.000899	0.000143	23089.340
6.000	-0.034	6.027334	0.00015085	0.001444	0.001690	0.000279	21579.477
6.000	-0.033	7.845599	0.00026338	0.002094	0.002375	0.000398	19699.072
6.000	-0.036	9.508442	0.00038306	0.002822	0.003096	0.000527	18028.814
6.000	-0.037	11.554222	0.00055001	0.003729	0.004017	0.000690	16737.254
4.000	-0.028	3.295722	0.00015563	0.000751	0.000960	0.000152	21614.189
4.000	-0.028	4.917368	0.00014413	0.001230	0.001480	0.000242	20358.953
4.000	-0.029	7.000602	0.00014828	0.001914	0.002201	0.000367	19087.193
4.000	-0.030	8.429841	0.00015366	0.002463	0.002757	0.000465	18117.854
4.000	-0.031	10.301473	0.00016757	0.003226	0.003529	0.000602	17109.512
2.000	0.099	3.252654	-.00022554	0.000754	0.000966	0.000153	21216.389
2.000	0.099	4.983845	-.00023665	0.001259	0.001508	0.000247	20210.236
2.000	0.076	6.479559	-.00022201	0.001747	0.002017	0.000335	19314.180
2.000	0.082	7.916756	-.00021499	0.002293	0.002572	0.000434	18258.084
2.000	0.087	9.452265	-.00021331	0.002910	0.003204	0.000545	17346.561
2.000	0.098	11.298149	-.00020582	0.003688	0.003991	0.000684	16509.523





ANALYSIS OF RESULTS		
EXPRESSIONS	STATISTICS	APPLICATION
$MR = MR_{max}$ WHEN $\varepsilon a \le 0.0001$ (1) $MR = K1 \cdot \sigma d^{K2}$ or (2) $MR = N1 \cdot \varepsilon a^{N2}$ WHEN $\varepsilon a > 0.0001$	MODEL: LOG (εa) = A + B * LOG (σd) R^2 = 0.995 AND SEE = 0.011 (1) K1 = 28542 AND K2 = -0.208 (2) N1 = 4868 AND N2 = -0.172 MRmax = 23821 psi	SAY σd = 6 psi USING Eq. (1): MR = 19651 psi Q: MR < MRmax ? ... No **MR(design) = 19,651 psi**

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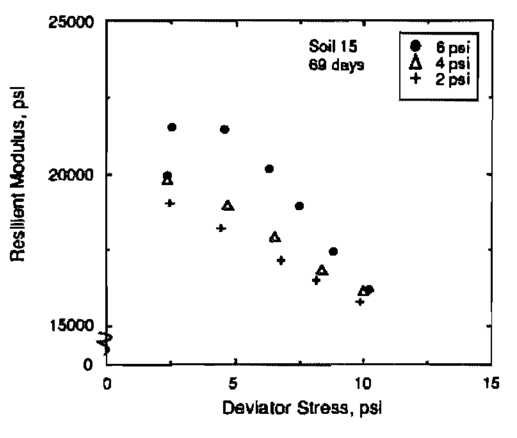
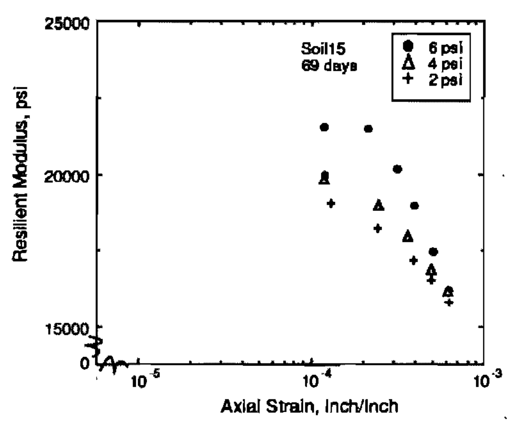
soil-15c.out — RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = Soil 15 - opt
DESCRIPTION = Dist 7 - Tom Green - US67 - 69 days
MOISTURE CONTENT = 21.20 percent
DRY DENSITY = 104.10 pcf.
PLASTICITY INDEX = 40.00 percent
LIQUID LIMIT = 58.00 percent
SAMPLE HEIGHT = 5.390 Inches
SAMPLE DIAMETER = 2.840 Inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (Inch)	AXIAL DEFORMATION A (Inch)	AXIAL DEFORMATION B (Inch)	STRAIN (In/In)	M r. (psi)
6.000	-0.020	2.537801	-.00003328	0.000700	0.000570	0.000118	21535.457
6.000	-0.021	4.574220	-.00001723	0.001313	0.000985	0.000213	21458.895
6.000	-0.021	6.330221	-.00000181	0.001988	0.001396	0.000314	20164.213
6.000	-0.021	7.498706	-.00002048	0.002539	0.001731	0.000396	18931.430
6.000	-0.023	8.840870	-.00000926	0.003309	0.002154	0.000507	17444.781
6.000	-0.024	10.188652	0.00002746	0.004184	0.002602	0.000629	16186.434
6.000	0.029	2.386590	-.00024169	0.000729	0.000560	0.000120	19950.389
4.000	0.040	2.360842	-.00025145	0.000727	0.000555	0.000119	19845.801
4.000	0.009	4.689363	-.00025948	0.001587	0.001080	0.000247	18951.875
4.000	-0.010	6.520756	-.00025379	0.002387	0.001535	0.000364	17924.865
4.000	-0.015	8.370387	-.00024453	0.003289	0.002060	0.000496	16869.111
4.000	-0.017	9.974710	-.00023310	0.004115	0.002537	0.000617	16162.580
2.000	0.252	2.464302	-.00050567	0.000794	0.000602	0.000130	19027.865
2.000	0.221	4.431436	-.00050888	0.001556	0.001067	0.000243	18211.393
2.000	0.185	6.740783	-.00050532	0.002571	0.001659	0.000392	17178.545
2.000	0.175	8.146146	-.00049488	0.003248	0.002067	0.000493	16520.314
2.000	0.174	9.837077	-.00048008	0.004119	0.002598	0.000623	15788.277





ANALYSIS OF RESULTS

EXPRESSIONS	STATISTICS	APPLICATION
MR = MRmax WHEN $\varepsilon a \leq 0.0001$ (1) $MR = K1 \cdot \sigma d^{K2}$ or (2) $MR = N1 \cdot \varepsilon a^{N2}$ WHEN $\varepsilon a > 0.0001$	MODEL: LOG (εa) = A + B * LOG (σd) R^2 = 0.989 AND SEE = 0.018 (1) K1 = 23788 AND K2 = -0.156 (2) N1 = 6123 AND N2 = -0.135 MRmax = 21168 psi	SAY σd = 6 psi USING Eq. (1): MR = 17999 psi Q: MR < MRmax ? ... No **MR(design) = 17,999 psi**

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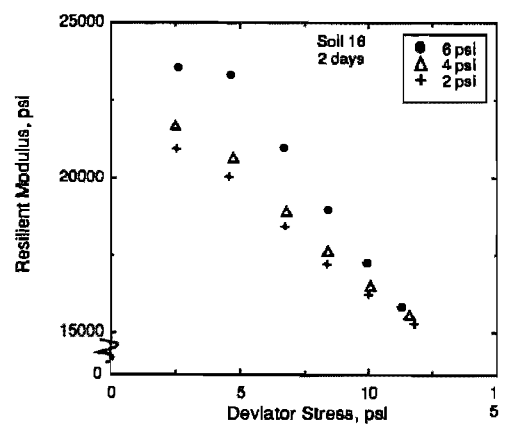
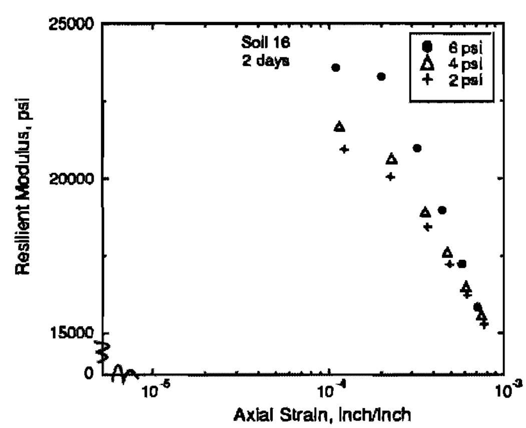
soil-16a.out

RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = Soil 16 - opt
DESCRIPTION = Dist 8 - Haskell - Abilene - 2 days
MOISTURE CONTENT = 20.00 percent
DRY DENSITY = 108.96 pcf.
PLASTICITY INDEX = 29.00 percent
LIQUID LIMIT = 51.00 percent
SAMPLE HEIGHT = 5.600 Inches
SAMPLE DIAMETER = 2.820 Inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (Inch)	AXIAL DEFORMATION A (Inch)	AXIAL DEFORMATION B (Inch)	STRAIN (in/in)	M r. (psi)
6.000	0.293	2.622830	-.00005176	0.000493	0.000753	0.000111	23570.830
6.000	0.229	4.678267	0.00002649	0.000930	0.001320	0.000201	23293.352
6.000	0.146	6.700468	0.00033267	0.001499	0.002079	0.000319	20975.211
6.000	0.122	8.403597	0.00082020	0.002077	0.002882	0.000443	18980.809
6.000	0.107	9.941020	0.00147870	0.002692	0.003759	0.000576	17259.699
6.000	0.096	11.261930	0.00219665	0.003311	0.004649	0.000711	15845.328
4.000	0.372	2.488460	0.00197144	0.000505	0.000782	0.000115	21650.467
4.000	0.314	4.737143	0.00196964	0.001047	0.001523	0.000229	20644.404
4.000	0.252	6.788780	0.00199513	0.001642	0.002374	0.000359	18931.859
4.000	0.197	8.396001	0.00203631	0.002191	0.003144	0.000476	17625.027
4.000	0.184	10.056398	0.00210090	0.002818	0.003996	0.000608	16529.396
4.000	0.175	11.614712	0.00228243	0.003481	0.004882	0.000747	15554.382
2.000	0.487	2.550184	0.00196992	0.000540	0.000825	0.000122	20930.711
2.000	0.423	4.580932	0.00195958	0.001043	0.001515	0.000228	20060.441
2.000	0.356	6.774537	0.00197202	0.001688	0.002429	0.000368	18431.471
2.000	0.301	8.371311	0.00198749	0.002238	0.003208	0.000486	17216.924
2.000	0.273	9.980904	0.00201283	0.002850	0.004038	0.000615	16229.212
2.000	0.258	11.797038	0.00211123	0.003605	0.005043	0.000772	15278.646





ANALYSIS OF RESULTS		
EXPRESSIONS	STATISTICS	APPLICATION
MR = MRmax WHEN $\varepsilon a \le 0.0001$ (1) $MR = K1 \cdot \sigma d^{K2}$ or WHEN $\varepsilon a > 0.0001$ (2) $MR = N1 \cdot \varepsilon a^{N2}$	MODEL: LOG (εa) = A + B * LOG (σd) R^2 = 0.989 AND SEE = 0.019 (1) K1 = 27553 AND K2 = -0.200 (2) N1 = 5013 AND N2 = -0.167 MRmax = 23270 psi	SAY σd = 6 psi USING Eq. (1): MR = 19255 psi Q: MR < MRmax ? ... No **MR(design) = 19,255 psi**

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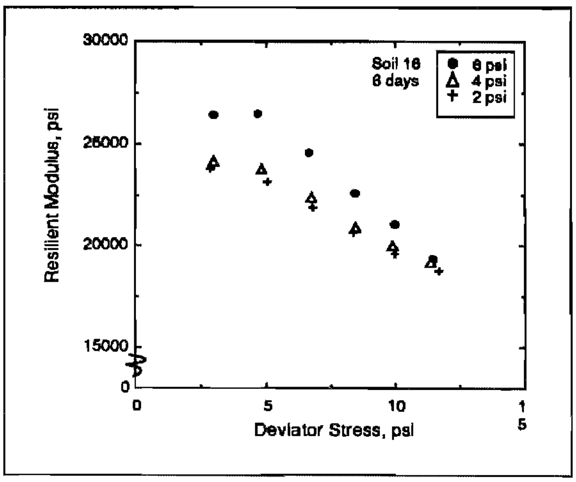
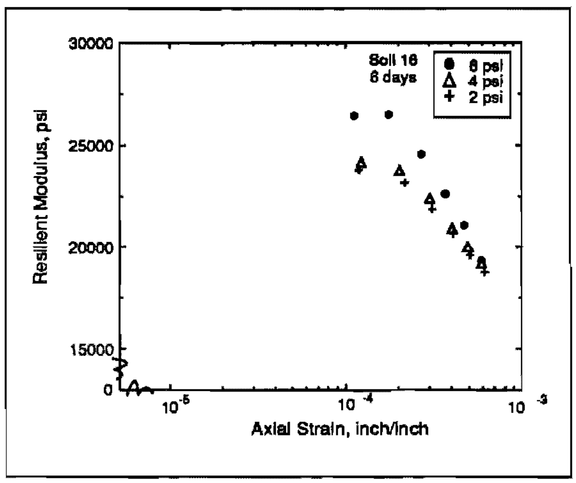
soil-16b.out

RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = Soil 16 - 6 days
DESCRIPTION = Dist 8 - Haskell - Abilene - opt
MOISTURE CONTENT = 20.00 percent
DRY DENSITY = 108.96 pcf.
PLASTICITY INDEX = 29.00 percent
LIQUID LIMIT = 51.00 percent
SAMPLE HEIGHT = 5.600 inches
SAMPLE DIAMETER = 2.820 inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PER DEFORMATION (inch)	AXIAL DEFORMATION A (inch)	AXIAL DEFORMATION B (inch)	STRAIN (in/in)	M r. (psi)
6.000	0.151	2.987481	0.00000653	0.000502	0.000764	0.000113	26426.951
6.000	0.104	4.722898	0.00006376	0.000819	0.001178	0.000178	26488.559
6.000	0.089	6.656785	0.00016676	0.001268	0.001766	0.000271	24571.828
6.000	0.085	8.419266	0.00029033	0.001749	0.002424	0.000373	22601.344
6.000	0.082	9.979480	0.00045482	0.002236	0.003074	0.000474	21049.643
6.000	0.082	11.447580	0.00068326	0.002803	0.003820	0.000591	19358.420
4.000	0.276	2.983683	0.00049467	0.000546	0.000838	0.000123	24161.652
4.000	0.220	4.818810	0.00046924	0.000931	0.001341	0.000203	23751.281
4.000	0.188	6.758393	0.00047478	0.001401	0.001976	0.000302	22411.998
4.000	0.173	8.441563	0.00049513	0.001693	0.002627	0.000404	20919.148
4.000	0.167	9.904460	0.00052395	0.002330	0.003219	0.000495	19992.879
4.000	0.160	11.358316	0.00057353	0.002800	0.003832	0.000592	19179.555
2.000	0.321	2.875427	0.00032268	0.000529	0.000826	0.000121	23778.127
2.000	0.284	5.040070	0.00031603	0.000997	0.001441	0.000218	23149.857
2.000	0.258	6.814895	0.00032027	0.001447	0.002034	0.000311	21922.102
2.000	0.245	8.395051	0.00032479	0.001903	0.002654	0.000407	20631.263
2.000	0.232	10.005594	0.00033049	0.002400	0.003319	0.000511	19594.682
2.000	0.216	11.681166	0.00035803	0.002939	0.004031	0.000622	18760.627





ANALYSIS OF RESULTS		
EXPRESSIONS	STATISTICS	APPLICATION
MR = MRmax WHEN $\varepsilon_a \leq 0.0001$ (1) $MR = K1 * \sigma_d^{K2}$ or WHEN $\varepsilon_a > 0.0001$ (2) $MR = N1 * \varepsilon_a^{N2}$	MODEL: LOG (ε_a) = A + B * LOG (σ_d) R^2 = 0.989 AND SEE = 0.016 (1) K1 = 30722 AND K2 = -0.169 (2) N1 = 6904 AND N2 = -0.145 MRmax = 26123 psi	SAY σ_d = 6 psi USING Eq. (1): MR = 22700 psi Q: MR < MRmax ? ... No **MR(design) = 22,700 psi**

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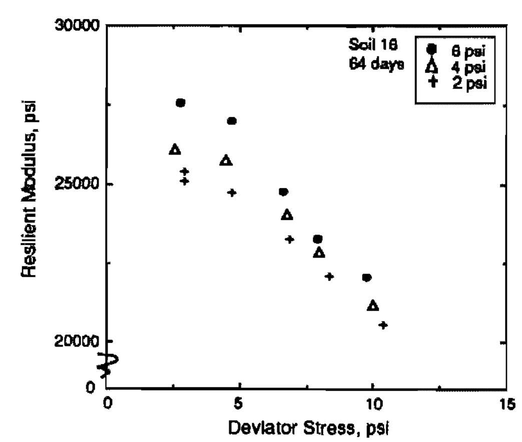
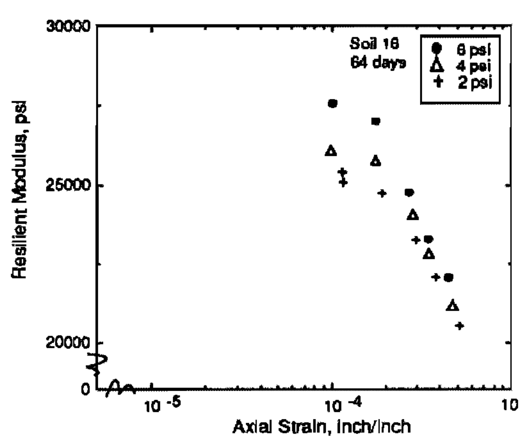
soil-16c.out — RESILIENT MODULUS (MR) TEST RESULTS

SAMPLE IDENTIFICATION = Soil 16 - opt
DESCRIPTION = Dist 8 - Haskell - Abilene - 64 days
MOISTURE CONTENT = 20.1 percent
DRY DENSITY = 106.89 pcf.
PLASTICITY INDEX = 29.00 percent
LIQUID LIMIT = 51.00 percent
SAMPLE HEIGHT = 5.700 Inches
SAMPLE DIAMETER = 2.840 Inches

CONFINE (psi)	SEATING (psi)	DEVIA STRESS (psi)	PEA DEFORMATION (Inch)	AXIAL DEFORMATION A (Inch)	AXIAL DEFORMATION B (Inch)	STRAIN (in/in)	M r. (psi)
6.000	0.557	2.791534	-.00003149	0.000540	0.000614	0.000101	27583.250
6.000	0.477	4.710449	-.00002326	0.000970	0.001019	0.000174	27001.000
6.000	0.380	6.641537	0.00003760	0.001544	0.001513	0.000268	24772.770
6.000	0.343	7.919097	0.00009542	0.001991	0.001886	0.000340	23288.459
6.000	-0.020	9.757962	0.00011868	0.002583	0.002454	0.000442	22083.271
4.000	0.228	2.552312	-.00024879	0.000518	0.000597	0.000098	26107.299
4.000	0.168	4.486209	-.00025305	0.000964	0.001020	0.000174	25775.396
4.000	0.118	6.781043	-.00025671	0.001603	0.001611	0.000282	24054.455
4.000	0.093	7.989786	-.00025520	0.002004	0.001978	0.000349	22873.756
4.000	0.049	9.974710	-.00023829	0.002732	0.002633	0.000471	21193.096
2.000	0.374	2.916995	-.00047258	0.000631	0.000695	0.000116	25085.066
2.000	0.384	2.894994	-.00048574	0.000618	0.000681	0.000114	25397.443
2.000	0.321	4.693596	-.00048419	0.001064	0.001098	0.000190	24747.977
2.000	0.271	6.837689	-.00048195	0.001684	0.001667	0.000294	23263.119
2.000	0.226	8.330127	-.00047691	0.002182	0.002118	0.000377	22085.998
2.000	0.180	10.408212	-.00045912	0.002964	0.002812	0.000507	20544.016





ANALYSIS OF RESULTS		
EXPRESSIONS	STATISTICS	APPLICATION
$MR = MR_{max}$ WHEN $\varepsilon_a \leq 0.0001$ (1) $MR = K1 \cdot \sigma_d^{K2}$ or (2) $MR = N1 \cdot \varepsilon_a^{N2}$ WHEN $\varepsilon_a > 0.0001$	MODEL: LOG (ε_a) = A + B * LOG (σ_d) R^2 = 0.994 AND SEE = 0.012 (1) K1 = 32029 AND K2 = -0.164 (2) N1 = 7430 AND N2 = -0.141 MR_{max} = 27186 psi	SAY σ_d = 6 psi USING Eq. (1): MR = 23878 psi Q: MR < MR_{max} ? ... No **MR(design) = 23,878 psi**