RESILIENT MODULUS AND PERMANENT DEFORMATION TESTING OF UNBOUND GRANULAR MATERIALS

A Thesis

by

ANUROOPA KANCHERLA

Submitted to the Office of Graduate Studies of
Texas A&M University
in partial fulfillment of the requirements for the degree of

MASTER OF SCIENCE

August 2004

Major Subject: Civil Engineering

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Approved as to style and content by:

Amy Epps Martin (Chair of Committee)	Dallas N. Little (Member)
Christopher Mathewson (Member)	Paul N. Roschke (Head of Department)

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ABSTRACT

Resilient Modulus and Permanent Deformation Testing of Unbound Granular Materials. (August 2004)

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Chair of Advisory Committee: Dr. Amy Epps Martin

Numerous research efforts have been devoted to characterizing the behavior of granular materials, which is one of the main concerns of pavement engineers. For better understanding of this behavior, laboratory tests where in-situ stress conditions and traffic loads are adequately simulated are needed. This study makes use of an expanded test protocol called a performance test that includes resilient modulus as well as permanent deformation testing. This test protocol determines three nonlinear resilient modulus parameters (k_1, k_2, k_3) and two permanent deformation parameters (α,μ). The resilient modulus test results are required inputs in the Level 1 analysis of the proposed American Association of State Highway and Transportation Officials (AASHTO) Pavement Design Guide. In addition, both resilient modulus and permanent deformation test results provide material property inputs to pavement performance prediction models.

This study also evaluated the within laboratory repeatability of the performance test and developed a within laboratory precision statement. Further, a statistical analysis was conducted on the test results to estimate the number of test specimens required for testing for specific reliability levels. Two test specimens are required for a reliability

level of 15%. A within laboratory study was also conducted to investigate the influence of specimen size on test results. The specimen height was reduced from 12 in. (304 mm) to 8 in. (203 mm), and there was no difference in test results at a confidence level of 95%.

The performance test was further used successfully in subsequent studies to evaluate the behavior of granular materials and the influence of various factors on their behavior. As fines content increased, the resilient modulus values decreased and permanent deformation increased. As the moisture content increased, the resilient modulus value decreased and the resistance to permanent deformation decreased.

A simplified laboratory measurement tool that is repeatable, relatively cheap and easy to perform might prompt the use of laboratory measured values of resilient modulus in pavement design and facilitate correlation of these values to field measured values on a large scale. Use of measured data for the base properties rather than estimates would insure improved pavement designs and, in many cases, would save money in construction costs.

DEDICATION

I dedicate this work to my mother, Mrs. B.Vijaya Bharathi.

ACKNOWLEDGMENTS

I would like to acknowledge the assistance of several individuals in conceptualizing my master's thesis. I would like to express my gratitude to my research advisor, Mr. Tom Scullion, for providing invaluable guidance and technical assistance during course of this study. Sincere gratitude is expressed to the chair of my advisory committee, Dr. Amy Epps Martin, for her inspiration and guidance in reviewing the progress of my thesis. Appreciation is given to Dr. Dallas Little and Dr. Christopher Mathewson for participating as members of my graduate committee. Special thanks to Dr. Eyad Masad for substituting at my thesis defense at a short notice. Sincere appreciation is due to Mr. Lee Gustavus, Mr. Tony Barbosa, and Mr. Stephen Kasberg for their help in sample preparation and laboratory testing. Special thanks to Dr. Fujie Zhou and Dr. Jacob Uzan for the discussions that I had with them.

I also wish to thank the faculty, staff and students at the pavements and materials engineering division of the Civil Engineering Department. I thank Mr. Karun Konduru, a friend, for his help and constructive criticism. Finally, I take this opportunity to thank my family for their unconditional love and support.

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INTRODUCTION

OVERVIEW

The successful and economic design of new pavements and prediction of remaining life of existing pavements depend on proper characterization of pavement materials. A conventional flexible pavement consists of a surface layer of hotmix asphalt, base layer of granular materials and subgrade as shown in Figure 1.

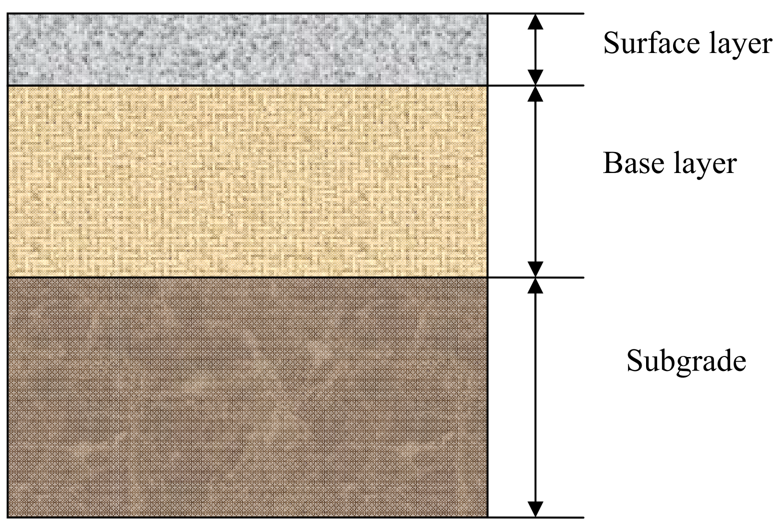

FIGURE 1 Flexible pavement system.

This thesis follows the style and format of the *Transportation Research Record*.

Numerous research efforts have been devoted to characterizing the behavior of granular materials (*1*), which is one of the main concerns of pavement engineers. The major structural function of a granular base layer is to contribute to the distribution of stresses applied to the pavement surface by traffic loading. These stresses must be reduced to levels that do not overstress the underlying base, subbase, and subgrade. Overstressing unbound granular material can produce unacceptable levels of resilient pavement deflections under moving wheel loads or can cause accumulation excessive amounts of permanent deformation, ultimately affecting the pavement performance (*2*). Thus, better understanding of this behavior of base materials by laboratory tests where in-situ stress conditions and traffic loads are adequately simulated are needed. The repeated load triaxial test, also known as the resilient modulus test, is one such method wherein the stiffness characteristics of the material as well as the ability to withstand the accumulation of permanent deformation during repeated loading are evaluated (*3*).

PROBLEM STATEMENT

There has been a significant amount of research in the determination of resilient properties of base materials (*1*). Several agencies have specified different test methods for resilient modulus testing, and some agencies modified the current American Association of State Highway Transportation Officials (AASHTO) test protocol to their need and convenience. Also, different testing equipment is being used at different places for the resilient modulus test. Hence, there is a need to develop a unified test method which would also represent field conditions. The NCHRP project 1-28A "Harmonized Test Methods for Laboratory Determination of Resilient Modulus for Flexible Pavement Design" was initiated to combine the best features of the resilient modulus testing procedures in current usage (*4*).

The literature suggests that the SHRP-46 protocol is one of the methods which closely represents the field stress state conditions (*5, 6*). This SHRP-46 protocol for resilient modulus testing has been deleted from the AASHTO standard specification due to lack of use. The standard test protocol for the resilient modulus test, AASHTO T307, measures only the resilient modulus. The present study will make use of an expanded test protocol including resilient modulus as well as permanent deformation testing. Henceforth, this test will be referred to as performance test.

For a laboratory test method, the variability of experimental responses is one feature that is inherent in the test procedure. In the practical interpretation of the test data, this inherent variability has to be taken into account as the factors that may influence the outcome of the test cannot all be controlled (*7*). In general, the existence of minimum variability of test results from the "true" value or the accepted reference value is defined as accuracy. To be of practical value, standard procedures are required for determining the accuracy of a test method. This is the motivation for this research wherein the main objective is to establish the accuracy of the proposed performance test in terms of its precision and bias in a within laboratory study.

SCOPE

The present study will make use of an expanded test protocol which will include measurement of nonlinear resilient modulus parameters (k_1, k_2, k_3) and permanent deformation parameters (α,μ). The proposed research will evaluate the repeatability of the proposed performance test and evaluate the influence of sample size and level of compaction on test results. The resilient modulus test result is a required input in the level 1 analysis or most sophisticated analysis of the newly proposed 2002 design guide to be released soon. Also, both resilient modulus and permanent deformation test results provide material property input to the VESYS 5 computer model used to predict pavement performance.

A simplified laboratory measurement tool that is repeatable, relatively cheap and easy to perform might prompt the use of laboratory measured values of resilient modulus in pavement design and facilitate correlation of these values to field measured values on a large scale. These measured values of resilient modulus and permanent deformation could then account for variations in moisture and load that would be encountered in the field. Use of measured data for the base properties rather than estimates would insure improved pavement designs and, in many cases, would save money in construction costs.

RESEARCH OBJECTIVES

It is critical to conduct a rigorous evaluation of the permanent deformation and resilient modulus test (proposed performance test) procedure to facilitate Texas Department of Transportation's (TxDOT) implementation efforts of the new 2002 design procedure. In this study the following will be investigated:

1. The precision and bias of the test method: A within laboratory study is conducted to compute the minimum number of samples necessary to test for a reliable level of accuracy.
2. The influence of specimen size on test results: TxDOT wishes to use a 6 in. (152 mm) diameter and 8 in. (203 mm) high samples rather than the recommended 6 in. (152 mm) diameter and 12 in. (304 mm) high samples by the standard procedure. The variation in the test results with the two different specimen sizes will be evaluated.

THESIS ORGANIZATION

The first chapter introduces the reader to the role of granular materials in pavement performance and the necessity for proper characterization of granular bases. It describes the research problem and the research objectives of this study.

The second chapter consists of a literature review of the repeated loading properties of the granular base materials. The resilient modulus and the permanent deformation properties are defined and the factors influencing them are discussed briefly. Further, some test procedures which are widely used are described. Subsequently, the models used for the determination of resilient modulus and permanent deformation properties are presented.

The third chapter discusses the research methodology that is followed in the present study. It consists of a brief explanation of the test matrix and the tests conducted on the granular material. It consists of a detailed description of the performance test procedure. This includes the test apparatus and the test specimen preparation. Salient features that are included in this test sequence are documented.

The fourth chapter presents the performance test results and a discussion of the test results. Further, this chapter presents a statistical analysis conducted on the test results. A within laboratory precision and bias statement has been documented in this chapter. Further, the analysis on the influence of sample size and method of compaction are described.

The fifth chapter consists of a description of case studies of successful applications of performance test procedure. The investigation of various factors influencing the resilient modulus and permanent deformation properties of the granular materials by performance test is documented. Further, case studies of the evaluation of the behavior of granular materials by performance test are described.

The sixth chapter consists of a summary of the research findings. Also, conclusions derived from the study are documented. Recommendations for future research are presented.

CHAPTER II

BACKGROUND

INTRODUCTION

This chapter consists of a literature review conducted on the behavior of granular materials under repeated loading. The repeated loading properties, resilient modulus, and permanent deformation properties are discussed. Further, the factors affecting the determination of these properties are presented. The laboratory determination of these properties and the recent developments in test procedures are described subsequently. The models that are widely used are also documented, and a summary of the literature review is presented at the end of this chapter.

REPEATED LOADING PROPERTIES OF GRANULAR MATERIALS

Granular materials experience some non-recoverable deformation after each load application. After the first few load applications, the resilient (recoverable) deformation increases more than the non-recoverable deformation. If the load is small compared to the strength of the material and is repeated for a large number of times, the deformation under each application is nearly completely recoverable and proportional to the load and can be considered elastic (*8*). This behavior of granular materials is characterized by resilient modulus. The term 'resilient' refers to that portion of the energy that is put into a material while it is being loaded, which is recovered when it is unloaded. The rest of the energy that is not recovered when loaded is capable of doing work on the material. This work results in the accumulation of permanent strain on repeated loading and unloading. This accumulated permanent strain in an aggregate base causes rutting.

The deformational response of granular layers under traffic loading is characterized by a recoverable (resilient) deformation and a residual (permanent) deformation, which is illustrated in Figure 2.

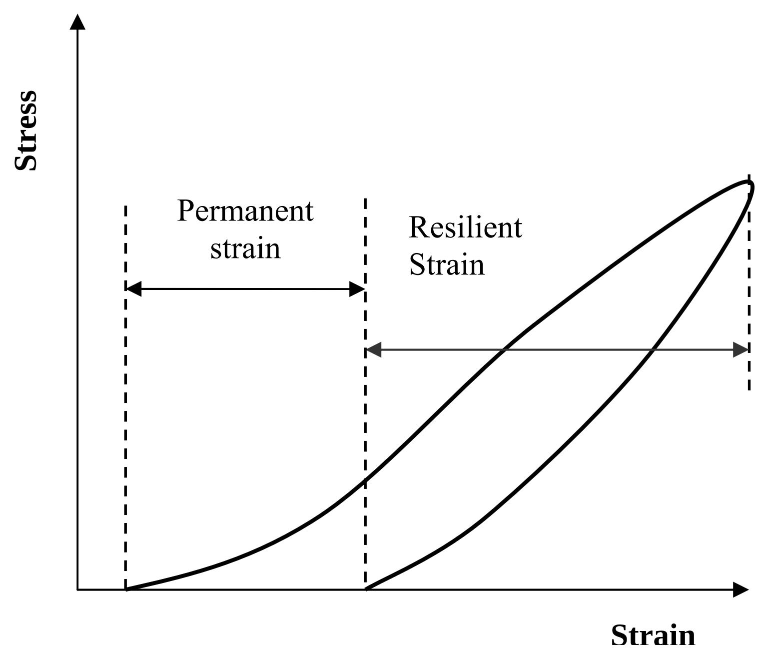

FIGURE 2 Strains in granular materials during one load cycle.

Research conducted previously indicates that repeated loading properties of granular materials like resilient modulus and permanent deformation accumulation are major factors that influence the structural response and performance of conventional flexible pavements. These parameters are typically determined in a repeated load triaxial test. This test is performed by placing a specimen in a triaxial cell and applying repeated axial load. After subjecting the specimen to a confining pressure, measurements are taken of the recoverable axial deformation and the applied load. Both resilient (recoverable) and permanent axial deformation responses of the specimen are recorded and used to calculate the resilient modulus and the permanent deformation, respectively.

Resilient Modulus

The resilient response of granular materials is usually characterized by the resilient modulus. For repeated load triaxial tests with constant confining stress, the resilient modulus is defined as the ratio of the peak axial repeated deviator stress to the peak recoverable axial strain of the specimen.

The resilient modulus (M_r) is expressed as (*1*):

$$M_r = \frac{(\sigma_1 - \sigma_3)}{\varepsilon_1} \tag{1}$$

where

M_r = resilient Modulus,

σ_1 = major principal or axial stress,

σ_3 = minor principal or confining stress, and

ε_1 = major principal or axial resilient strain

Permanent Deformation

Permanent deformation (PD) is the unrecovered deformation during unloading. It accumulates on repeated loading and unloading. The permanent deformation is represented as:

$$PD = \varepsilon_P \tag{2}$$

where

ε_p = permanent axial strain.

There are many factors that affect the repeated loading properties of the material determined by the repeated load triaxial test. A comprehensive literature review was conducted on the resilient and permanent deformation properties of granular materials and is presented in the following section.

FACTORS AFFECTING REPEATED LOADING PROPERTIES

Many factors simultaneously affect both the resilient modulus and permanent deformation properties of granular materials. However, their influence on resilient modulus was not the same as on permanent deformation properties. In this section a brief overview of the factors influencing both the resilient modulus and permanent deformation is presented. Also, the variation in the influence of these properties is described.

Aggregate Type and Particle Shape

Heydinger et al. (*1*) showed that gravel had a higher resilient modulus than crushed limestone. However, many researchers (*9, 10, 11, 12, 13*) have reported that crushed aggregate, having angular to subangular shaped particles, provides better load spreading properties and a higher resilient modulus than uncrushed gravel with subrounded or rounded particles. A rough particle is also said to result in a higher resilient modulus (*1*).

Allen (*10*) argued that angular materials, such as crushed stone undergo smaller plastic deformations compared to materials with rounded particles (*5*). This behavior was said to be a result of a higher angle of shear resistance in angular materials due to better particle interlock. Barksdale and Itani (*13*) investigated the influence of aggregate shape and surface characteristics on aggregate rutting. They concluded that blade shaped crushed aggregate is slightly more susceptible to rutting than other types of crushed aggregate. Moreover, cube-shaped, rounded river gravel with smooth surfaces is much more susceptible to rutting than crushed aggregates (*5*).

Compaction Method

Seed et al. (*14*) recommended the use of two compaction methods for the preparation of test specimens: 1) Kneading or Impact, and 2) Static. The resilient modulus is directly related to the stiffness, which increases with an increase in compactive effort. This increase in stiffness varies with different materials and depends on the water content at which the sample was molded (*6, 14*). Magnusdottir et al. (*15*) reported that there was an increase in stiffness of about 80% when going from standard Proctor energy (593 Kj/m^3) up to a modified Proctor compaction energy (92693 KJ/m^3). Compactive effort (C.E) is calculated by using the following equation (*16*):

$$C.E = \frac{H \cdot W \cdot N_d \cdot N_l}{V} \tag{3}$$

where

H = height of drop in ft,

W = weight of hammer in lb,

N_d = number of drops,

N_l = number of layers, and

V = volume of mold in cubic inch.

TxDOT uses a compactive effort of 13.26 lb-ft/in^3 (10752 KJ/m^3), while the proposed test sequence uses a compactive effort of 32.36 lb-ft/in^3 (26238 KJ/ m^3) as recommended by AASHTO (*16, 17*).

Confining Pressure

The resilient modulus increases considerably with an increase in confining pressure and sum of principal stresses (*1*). Monismith et al. (*18*) reported an increase as great as 500% in resilient modulus for a change in confining pressure from 2.9 psi (20 kPa) to 29 psi (200 kPa). An increase of about 50% in resilient modulus was observed by Smith and

Nair (*19*) when the sum of principal stresses increased from 10 psi (70 kPa) to 20.3 psi (140 kPa). Allen and Thompson (*11*) compared the test results obtained from both constant confining pressure tests (CCP) and variable confining pressure tests (VCP). They reported higher values of resilient modulus computed from the CCP test data. They showed that the CCP tests resulted in larger lateral deformations. Figure 3 illustrates a typical result of their study

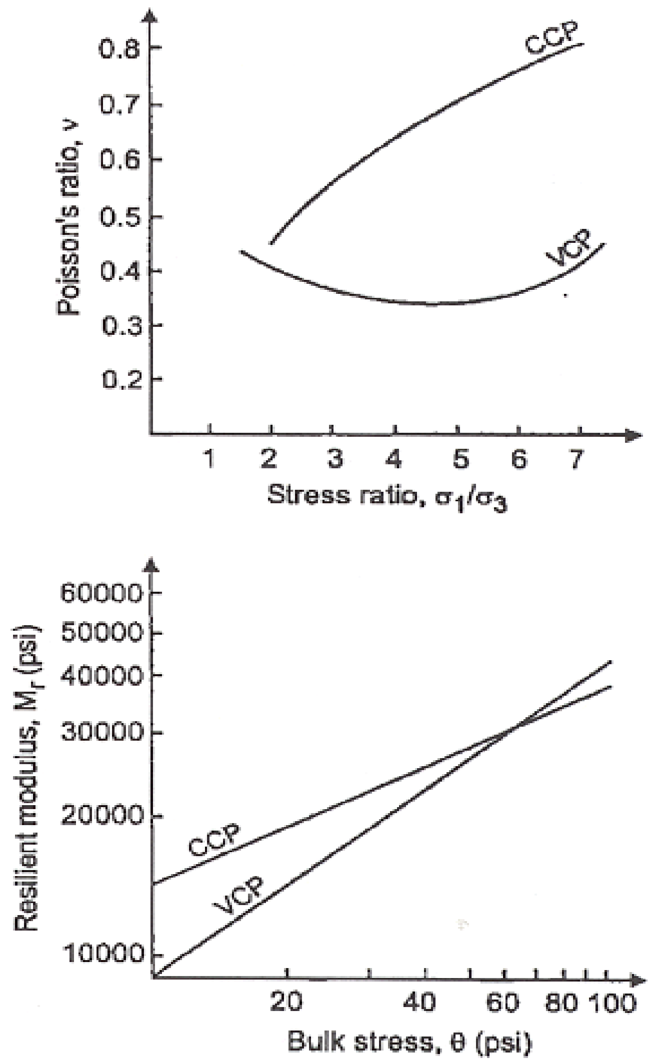

FIGURE 3 Triaxial test results with CCP and VCP (*11*).

Brown and Hyde (*20*) suggested later that VCP and CCP tests yield the same values of resilient modulus provided that the confining pressure in the CCP test is equal to the mean value of the pressure used in the VCP test.

Dry Density

Hicks and Monismith (*9*) found the effect of density to be greater for partially crushed than for fully crushed aggregates. They found that the resilient modulus increased with relative density for the partially crushed aggregate tested, whereas it remained almost unchanged when the aggregate was fully crushed. They further reported that the significance of changes in density decreased as the fines content of the granular material increased.

Barksdale and Itani (*13*) reported that the resilient modulus increased markedly with increasing density only at low values of mean normal stress. At high stress levels, the effect of density was found to be less pronounced. Vuong (*1*) reported test results showing that at densities above the optimum value, the resilient modulus is not very sensitive to density.

Resistance to permanent deformation in granular materials under repetitive loading appears to be highly improved as a result of increased density. Barksdale (*21*) studied the behavior of several granular materials and observed an average of 185% more permanent axial strain when the material was compacted at 95% instead of 100%

of maximum compaction density. Allen (*10*) reported an 80% reduction in total plastic strain in crushed limestone and a 22% reduction in gravel as the specimen density was increased from Proctor to modified Proctor density. For rounded aggregates, this decrease in strain with increasing density is not considered to be significant, as these aggregates are initially of a higher relative density than angular aggregates for the same compactive effort (*2*).

Fines Content

Studies demonstrating the variation in response of granular materials subjected to repeated axial stresses indicate that the fines content (percent passing No.200 sieve) can also affect the resilient behavior (*22*). Hicks and Monismith (*9*) observed some reduction in resilient modulus with increasing fines content for the partially crushed aggregates tested, whereas the effect was reported to be the opposite when the aggregates were fully crushed. The variation of fines content in the range of 2-10% was reported by Hicks to have a minor influence on resilient modulus (*22*). Yet, a dramatic drop of about 60% in resilient modulus was noted by Barksdale and Itani (*13*), when the amount of fines increased from 0 to 10%. Jorenby and Hicks (*1*) showed in a study an initially increasing stiffness and then a considerable reduction as clayey fines were added to a crushed aggregate.

The effect of fines content was investigated by Barksdale (*2, 21*) and Thom and Brown (*12*), who concluded that permanent deformation resistance deformation in granular materials is reduced as the fines content increases.

Gradation and Grain Size

Kolisoja (*23*) showed that for aggregates with similar grain size distribution and the same fines content, the resilient modulus increased with increasing maximum particle size. As the size of the particle increases, the particle to particle contact decreases resulting in less total deformation and consequently higher stiffness. Thom and Brown (*12*) concluded that uniformly graded aggregates were only slightly stiffer than well-graded aggregates. They further indicated that the influence of gradation on the permanent deformation depends on the level of compaction. This is illustrated in Figure 4 (*12*).

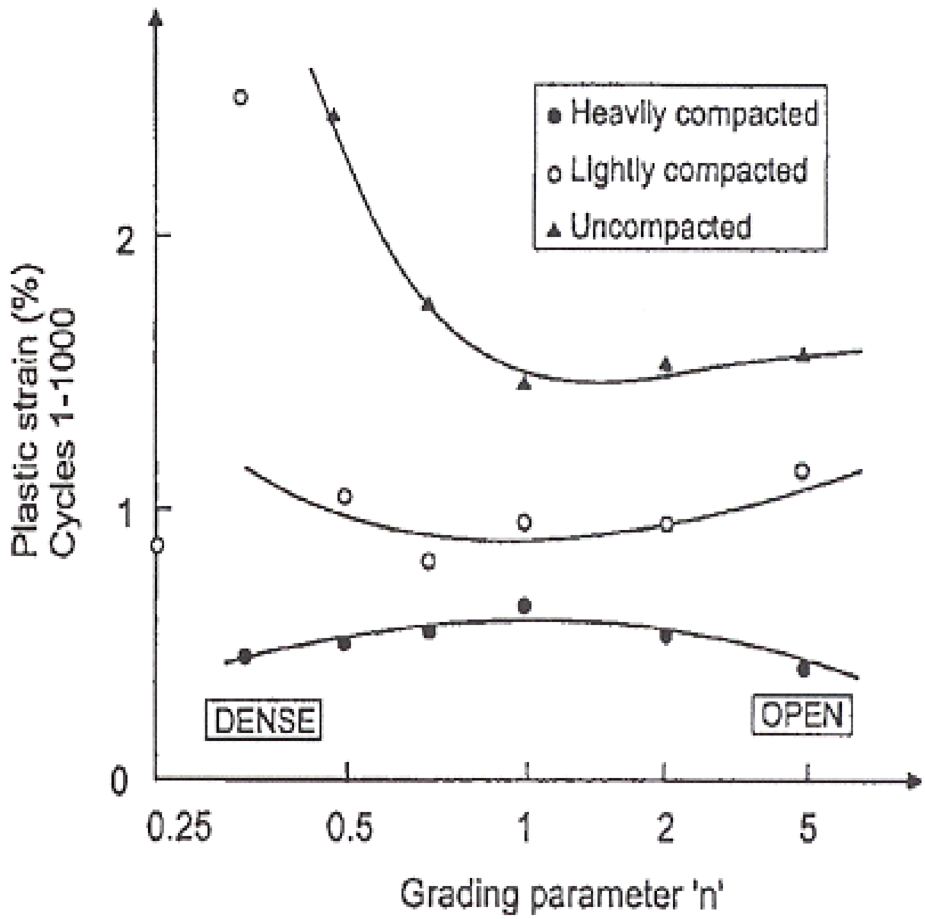

FIGURE 4 Effect of grading and compaction on plastic strain (*12*).

Dawson et al. (*2*) argued that the effect of gradation on permanent deformation was more significant then the degree of compaction, with the highest plastic strain resistance for the densest mix.

Load Duration and Frequency

The load duration and frequency have very little effect on the resilient behavior of granular materials. Seed et al. (*1*) reported a study in which the resilient modulus of sands increased only slightly (from 8700 psi (60 MPa) to 27557 psi (190 MPa)) as the duration of load decreased form 20 min to 0.3 s. Hicks (*22*) conducted tests at stress durations of 0.1, 0.15, and 0.25 s and found no change in the resilient modulus.

Moisture Content

The moisture content of most untreated granular materials has been found to affect the resilient response characteristics of the material in both laboratory and in situ conditions. Researchers who studied the behavior of granular materials at high degrees of saturation have all reported a notable dependence of resilient modulus on moisture content, with the modulus decreasing with growing saturation level (*1*).

Research has shown that the effect of moisture also depends on the analysis. Hicks (*22*) stated that a decrease in the resilient modulus due to saturation is obtained only if the analysis is based on total stresses. Similarly, Pappin (*1*) observed that if the test results are analyzed on the basis of effective stresses, the resilient modulus remains approximately unchanged.

Dawson et al. (*1*) studied a range of well-graded unbound aggregates and found that below the optimum moisture content stiffness tends to increase with increasing moisture level, apparently due to development of suction. Beyond the optimum moisture content, as the material becomes more saturated and excess pore water pressure is developed, the effect changes to the opposite and stiffness starts to decline fairly rapidly.

As moisture content increases and saturation is approached, positive pore pressure may develop under rapid applied loads. Excessive pore pressure reduces the effective stress, resulting in diminishing permanent deformation resistance of the material. Literature available suggests that the combination of a high degree of saturation and low permeability due to poor drainage leads to high pore pressure, low effective stress, and consequently, low stiffness and low deformation resistance (*2*).

In a study conducted by Haynes and Yoder (*24*), the total permanent axial strain rose by more than 100% as the degree of saturation increased from 60 to 80%. Barksdale (21) observed up to 68% greater permanent axial strain in soaked samples compared with those tested in a partially soaked condition.

Thompson (*9*) reported results of repeated load triaxial tests on the crushed stone from the AASHTO Road Test at varying degrees of saturation. In all cases, the samples experienced a substantial increase in permanent deformation after soaking. It was suggested that one reason for the observed increase was development of transient pore pressures in the soaked samples.

Specimen Size

AASHTO specifies that the diameter of the specimen is a function of the maximum size of the aggregate used in the base material. Further, it specifies that the diameter to height ratio is 1:2 (*17*). Thus, according to this for a maximum aggregate size of 1 in. (25.4 mm), the size of the specimen is 6 in. (152 mm) in diameter with a height of 12 in. (304 mm). There are practical problems involved in molding 12 in. (304 mm) height samples and setting up these samples in the triaxial cell due to lack of availability of equipment.

Experimental work done by Taylor indicates that reliable results could be obtained with soil specimens having regular ends provided the slenderness (height to diameter ratio, l/d) is in the range of 1.5 to 3.0 (*1*). According to Lee (*25*) this study established the standard that the slenderness (l/d) of triaxial specimens for soil be limited to 2.0 to 2.5 for tests with regular ends. Since then, many researchers have studied end restraint effects on the shear strength of soils and concluded that sample slenderness can be reduced to 1.0 if frictionless platens are used (*25*). Adu-Osei et al. (*25*) studied the effect of specimen size. They changed the specimen size from a l/d ratio of 2:1 to 1:1. They found that specimens with a l/d ratio of 1:1 gave reliable results when the end platens were lubricated (*25*). These specimens were also more stable and practical.

Stress State

Previous investigations and studies show that stress level has the most significant impact on the resilient properties of granular materials (*1, 2*). Many studies indicated a high degree of dependence on confining pressure and the first stress invariant (sum of principal stresses) for the resilient modulus of untreated granular materials. The resilient modulus is said to increase considerably with an increase in confining pressure and the sum of principal stresses, while the permanent deformation decreases with an increase in confining pressure. Compared to confining pressure, deviator or shear stress is said to be much less influential on resilient modulus of the material. In laboratory triaxial testing, both constant confining pressure and variable confining pressure are used. Brown and Hyde (*20*) suggested that variable confining pressure and constant confining pressure tests yield the same values of resilient modulus, provided that the confining pressure in the constant confining pressure test is equal to the mean value of the pressure used in the variable confining pressure test.

The accumulation of axial permanent strain is directly related to deviator stress and inversely related to confining pressure. Several researchers have reported that permanent deformation in granular materials is principally governed by some form of stress ratio consisting of both deviator and confining stresses (*2*).

Lekarp and Dawson (*26*) argued that failure in granular materials under repeated loading is a gradual process and not a sudden collapse as in static failure tests. Therefore, ultimate shear strength and stress levels that cause sudden failure are of no great interest for analysis of material behavior when the increase in permanent strain is incremental.

Stress History

Studies have indicated that stress history may have some impact on the resilient behavior of granular materials. Boyce et al. (*27*) carried out repeated load triaxial tests on samples of a well-graded crushed limestone, all compacted to the same density in a dry state. The results showed that the material was subjected to stress history effects, but these could be reduced by preloading with a few cycles of the current loading regime and avoiding high stress ratios in tests for resilient response (*1*).

Hicks (*22*) reported that the effect of stress history is almost eliminated, and a steady and stable resilient response is achieved after the application of approximately 100 cycles of the same stress amplitude. Allen (*10*) suggested that specimens should be conditioned for approximately 1,000 cycles prior to repeated load resilient tests. Other researchers (*20*) reported that resilient characteristics of unbound granular materials are basically insensitive to stress history, provided the applied stresses are kept low enough to prevent substantial permanent deformation in the material. Therefore, large numbers of resilient tests can be carried out sequentially on the specimen to determine the resilient parameters of the material (*1*).

Permanent deformation behavior of granular materials is directly related to the stress history. Brown and Hyde (*20*) showed that the higher the stress level higher is the permanent deformation as shown in Figure 5. They also indicated that the permanent strain resulting from a successive increase in the stress level is considerably smaller than the strain that occurs when the highest stress is applied immediately.

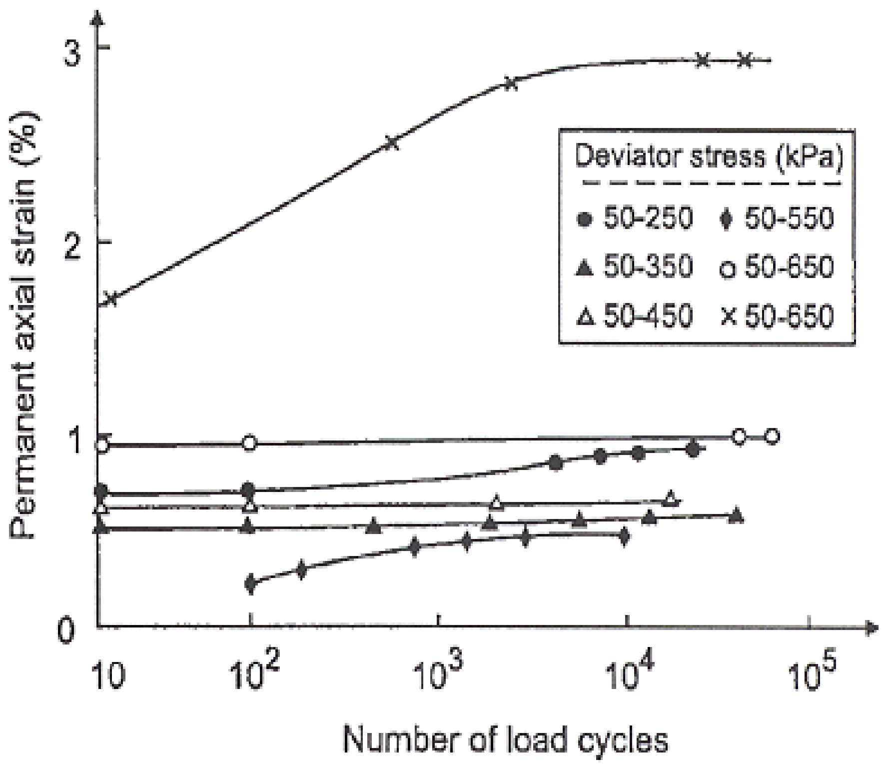

FIGURE 5 Effect of stress history on permanent strain (*20*).

ANISOTROPY OF GRANULAR MATERIALS

The behavior of granular materials, like most geologic materials, depends on particle arrangement which is usually determined by aggregate characteristics, construction methods, and loading conditions. An apparent anisotropy is induced in an unbound granular layer in a pavement system during construction, becoming stiffer in the vertical direction than in the horizontal direction even before traffic loads impose further anisotropy. Barksdale and Itani (*13*) observed that linear cross-anisotropy is equal to or better than a more complicated nonlinear isotropic model for predicting unbound granular layer response to traffic loads.

Adu-Osei et al. (*25, 28*) developed a system identification method to determine the cross-anisotropic elastic properties of unbound aggregates. They found that vertical resilient modulus was higher than the horizontal resilient modulus for the materials that they tested. Also, the horizontal Poisson's ratio always remained greater than the vertical Poisson's ratio. They stated that molding moisture, gradation, and aggregate type have a significant effect on the anisotropic properties. Further, the tensile stresses predicted by the layered linear elastic model within the base layer were reversed and drastically reduced by the nonlinear anisotropic elastic model (*28*).

At present test procedures for the determination of repeated loading properties of granular materials mainly focus on resilient modulus. The determination of anisotropic properties of granular materials is not included in the standards. The following section provides a brief description of the test procedures widely used.

TEST PROCEDURES FOR DETERMINATION OF RESILIENT MODULUS AND PERMANENT DEFORMATION

This section consists of a brief description of the test procedures used to determine the repeated loading properties of granular materials. The AASHTO protocol, European Protocol, ICAR protocol, and the recently released Harmonized protocol are briefly described. This section thus provides an introduction to the testing procedures and the main features of the test protocols. The description focuses only on the test procedures followed for the base materials, and it does not include the description for any other materials which may have been described in the standard protocols.

AASHTO T307 Protocol

AASHTO T307 protocol describes the test procedure for the determination of resilient modulus. A repeated axial cyclic stress of fixed magnitude, load duration (0.1 s), and cycle duration (1.0 s) is applied to a cylindrical test specimen. During testing, the specimen is subjected to a dynamic cyclic stress and a static-confining stress provided by means of a triaxial pressure chamber. The total resilient (recoverable) axial deformation response of the specimen is measured and used to calculate the resilient modulus.

The test is begun by applying a minimum of 500 repetitions of a load equivalent to a maximum axial stress of 15 psi (103.4 kPa) and corresponding cyclic axial stress of 13.5 psi (93.1 kPa) using a haversine shaped load pulse. The confining pressure is set to

15 psi (103.4 kPa). If the sample is still decreasing in height at the end of the conditioning period, stress cycling shall be continued up to 1000 repetitions prior to testing. This is followed by a sequence of loading with varying confining pressure and deviator stress. The confining pressure is set constant, and the deviator stress is increased. Subsequently, the confining pressure is increased, and the deviator stress varied. The resilient modulus values are reported at specified deviator stress and confining pressure values. The stress sequences followed and the detailed procedure can be found in the AASHTO T-307 Protocol (*17*).

European Protocol

A repeated load triaxial test is used to measure material susceptibility to permanent deformation and for assessing the resilient properties. A draft Comité Européen de Normalisation (CEN), Standard (*29*) describes both procedures which use the same repeated load triaxial apparatus.

The draft specifies two procedures, one for the variable confining pressure known as Method A and the other for constant confining pressure known as Method B. In general, test specimens are prepared to the required density and moisture content conditions required.

Test specimens are subjected to a preconditioning test of 20,000 cycles involving an applied loading of mean normal stress (p) = 43.5 psi (300 kPa) and deviatoric stress (q) = 87 psi (600 kPa) (or σ_1 = 101.5 psi (700 kPa) and σ_3 = 14.5 psi (100 kPa)).

Resilient Modulus Stress Stage Test

Following the preconditioning test, the material specimens are subjected to a stress stage resilient modulus test of 100 cycles per stage, with a range of applied loading sequenced according to the CEN procedure with stress ratio loading paths of q/p = 0, 0.5, 1.0, 1.5, 2.0, and 2.5 performed in sequence.

CEN test method B (constant confining pressure) follows a separate set of stress levels compared to Method A, but they are applied to the same specimen. These stress levels are much reduced in magnitude compared to those of Method A, in keeping with material placed in the lower levels of a pavement structure.

In order to directly compare the resilient modulus of different materials, a 'characteristic' stress level has been chosen at which to report the elastic 3D Hooke Law modulus. This stress level of applied loading is p = 36.3 psi (250 kPa) and q = 72.5 psi (500 kPa) (or σ_1 = 84.5 psi (583 kPa) and σ_3 = 12 psi (83 kPa)). The detailed procedure can be found in the CEN standard (*29*)

ICAR Test Protocol

The ICAR test protocol takes into account the anisotropy of granular materials. The stresses used in the triaxial testing were chosen to represent the stress conditions induced in a typical base layer of a flexible pavement by traffic loads. The testing protocol itself involves a programmed loading sequence employing ten static stress states. At each static stress state, small dynamic changes in stresses are applied to obtain three triaxial stress regimes such that the net stress changes represent triaxial compression, triaxial shear, and triaxial extension. The resilient axial and radial strains are determined for each stress regime and implemented in the system identification scheme to back calculate the five anisotropic elastic properties at that particular stress state. The loading sequence is outlined in the following steps:

1. A mounted sample is loaded to a static stress state (axial stress σ_y, and confining stress σ_x). The confining stress is then kept constant while the axial stress is given a small dynamic stress increment of $\Delta\sigma_y$, shown as triaxial compression. The increment loading is applied for 25 repetitions until a stable resilient strain is achieved. A cycle of loading consists of 1.5 seconds loading followed by 1.5 seconds rest period. Since, the RaTT (rapid triaxial test) cell uses air for confinement, the loading cycle was selected to allow for easy application for variable confinement.
2. At the same static stress state (σ_x, σ_y) as in step 1, the axial stress is changed by a small dynamic stress increment of $\Delta\sigma_y$ for 25 repetitions as before, while the radial stress is reduced by $\Delta\sigma_x$ such that the change in the first stress invariant (ΔI_1) is zero in each load cycle. This is shown as triaxial shear.
3. At the same controlled static stress state (σ_x, σ_y) as in step 1, the axial stress is reduced by a small amount, $\Delta\sigma_y$, while the radial stress is increased by $\Delta\sigma_x$. Thus, the net change in stress state is in an extension mode, but the principal stresses are not reversed. The dynamic stresses are applied for 25 repetitions as

before until stable resilient strains are achieved. This is shown as triaxial extension.

4. These steps are repeated for ten different stress states. The measured axial and radial strains at each stress state are used as input to the system identification scheme. A computer program systematically back calculates the five anisotropic elastic properties based on the SID method (*25*).

Permanent Deformation Testing

The permanent deformation behavior of the materials was studied at four stress levels (*25*). At each stress level, static confining stresses were applied to the samples, and deviator stresses were axially cycled 10,000 times. Strains measured are separated into resilient and plastic strains. Plastic strains are then used to characterize the permanent deformation behavior of the material. The deviator stress consists of a haversine pulse-load applied for 0.1 seconds with a 0.9 seconds rest at a frequency of 1 cycle per second. This load cycle was used for permanent deformation testing because the confining pressure was not cycled.

Harmonized Protocol (NCHRP 1-37 A)

The NCHRP project 1-37 A aims at combining the best features of the repeated loading test procedures. However, it can be used for the determination of resilient modulus only.

The test specimen is compacted at optimum moisture content to the required density. The preconditioning of the specimen is carried at 15 psi (103.4 kPa) confining pressure and 18 psi (124.1 kPa) deviator stress and 200 repetitions. If the sample is still decreasing in height at the end of the conditioning process, stress cycling should be continued up to 1000 repetitions prior to testing. If the vertical permanent strain reaches 5% during conditioning then the conditioning process is terminated.

The test is conducted by applying both cyclic and confining stress at a constant stress ratio. Both the cyclic and confining stresses are increased to maintain a constant stress ratio. The stress ratio is then increased. The resilient modulus value is reported at a specified deviator stress and confining pressure. The detailed procedure can be found in the NCHRP 1-37 A report (*4*).

The parameters obtained from the above test procedures are used within the appropriate regression models to determine the regression parameters, k_i, and the resilient and permanent deformation parameters to be used in the pavement performance prediction tools and software. The following section presents an extensive literature on the models used for prediction of these repeated loading properties.

MODELS

A constitutive relationship needs to be established for accurate prediction of long-term behavior of granular materials. A great majority of the models found in the literature are based on simple curve fitting procedures, using the data from laboratory triaxial testing. Some of the modeling techniques found in the literature are presented.

Models for Resilient Modulus

Seed et al. (*1*) suggested the following relationship for resilient modulus:

$$M_r = k_1 \sigma_3^{k_2} \tag{4}$$

where

σ_3 = confining pressure.

Hicks (*22*) suggested the following simple hyperbolic relationship commonly known as the K-θ model:

$$M_r = k_1 \theta^{k_2} \tag{5}$$

where

θ = sum of stresses or bulk stress, and

k_1,k_2 = regression coefficients.

This model assumes a constant Poisson's ratio, while research has proved that the Poisson's ratio varies with applied stresses. Another drawback is that the effect of stress on resilient modulus is accounted for solely by the sum of principal stresses

Uzan (*30*) included deviator stress into the K-θ model and expressed the relationship as follows:

$$M_r = k_1 p_o \left(\frac{\theta}{p_o} \right)^{k_2} \left(\frac{q}{p_o} \right)^{k_3} \tag{6}$$

where

θ = bulk stress,

P_o = atmospheric pressure,

q = deviatoric stress, and

k_1, k_2, k_3 = regression coefficients.

In the three dimensional case, the deviator stress in the Uzan model is replaced by the octahedral stress as follows:

$$M_r = k_1 p_o \left(\frac{\theta}{p_o} \right)^{k_2} \left(\frac{\tau_{oct}}{p_o} \right)^{k_3} \tag{7}$$

where

θ = bulk stress,

P_o = atmospheric pressure,

τ_{oct} = octahedral shear stress, and

k_1, k_2, k_3 = regression coefficients.

This model is further modified by adding a "+1" term to avoid the absurd calculation of modulus when the τ_{oct} tends to zero (the modulus value tends to zero when τ_{oct} tends to zero):

$$M_r = k_1 p_o \left(\frac{\theta}{p_o}\right)^{k_2} \left(\frac{\tau_{oct}}{p_o} + 1\right)^{k_3} \tag{8}$$

This model was proposed for the determination of resilient modulus by NCHRP project 1-37 "Development of the 2002 Guide for the Design of New and Rehabilitated Pavement Structures (*31*).

Kolisoja (*23*) included the effect of material density in both the K-θ and Uzan models. He expressed the following modified formulations:

$$M_r = A(n_{\max} - n) p_o \left(\frac{\theta}{p_o}\right)^{0.5} \tag{9}$$

where

n = porosity of the aggregate, and

A = constant.

The recently completed NCHRP 1-37 A project proposed a model with the best overall goodness of fit statistics based on their study on the development of a harmonized protocol for the determination of resilient modulus (*4*):

$$\log\left(\frac{M_r}{p_a}\right) = k_1 + k_2\left(\frac{x - 3k_6}{p_a}\right) + k_3\left(\frac{y}{p_a} + k_7\right) \tag{10}$$

where

x, y = pair of stress parameters equal to either (σ_3, σ_{cyc}) or (θ, τ_{oct}).

in which σ_3 is the confining stress and σ_{cyc} is the cyclic deviator stress.

k_6 is a material property related to the capillary suction in partially saturated unbound materials

k_7 is the material parameter and all other terms are as defined previously.

Models for Permanent Deformation

Barksdale (*21*) performed a comprehensive study of the behavior of different base course materials using repeated load triaxial tests with 10^5 load applications. For a given stress condition, the accumulation of permanent axial strain was shown to be proportional to the algorithm of the number of load cycles, and the results could be expressed by a log-normal expression of the form:

$$\varepsilon_p = a + b \bullet \log(N) \tag{11}$$

where

ε_p = total permanent axial strain,

N = number of load cycles, and

a, b = constants for a given level of deviator stress and confining pressure.

The long-term behavior of granular materials was also investigated by Sweere (*32*). After applying 10^6 load cycles in repeated load triaxial tests, Sweere observed that the log-normal approach did not fit his test results. He then suggested that for large numbers of load repetitions a log-log approach should be employed and expressed the results by:

$$\varepsilon_p = a \bullet N^b \tag{12}$$

where

a, b = regression parameters.

The applicability of this log-log model was later questioned by Wolff and Visser (*33*) who performed full-scale Heavy Vehicle Simulator (HVS) testing with several million load applications. They suggested a different model given by:

$$\varepsilon_p = (C \bullet N + a) \bullet (1 - e^{-bN}) \tag{13}$$

where

a, b, c = regression parameters.

Kenis (*34*) developed the VESYS model where he considered that the permanent strain is proportional to resilient strain:

$$\varepsilon_p(N) / \varepsilon_r = \mu N^{-\alpha} \tag{14}$$

$$\log \varepsilon_p = \log[\varepsilon_r \mu / 1 - \alpha] + (1 - \alpha) \log N$$

where

s=1-α.

Another model that is used was developed at Ohio state and is known as the Ohio state model developed by Khedr (*2*):

$$\varepsilon_p / N = AN^{-b} \tag{15}$$

Tseng and Lytton developed a three parameter model using a sigmoidal curve fitting and is expressed as:

$$\varepsilon_a = \varepsilon_o \exp[-\rho / N]^{\beta} \tag{16}$$

Where

ε_a = permanent axial strain, and

ε_o, β, ρ = three material parameters.

Lekarp and Dawson (*26*) observed that there was a relationship between accumulated permanent axial strain, the maximum shear-normal stress ratio, q_{max}/p_{max}, and the length of the stress path in p-q space applied to reach this maximum value. A consistent pattern was observed when the ratio of permanent strain at a given number of cycles to the stress path length was plotted against the maximum shear normal stress ratio. The relationship was given by the following expression of the form:

$$\frac{\varepsilon_{1,p}\left(N_{ref}\right)}{L/p_o} = a \cdot \left(\frac{q}{p}\right)_{\max}^{b} \tag{17}$$

where

$\varepsilon_{1,p}(N_{ref})$ = accumulated permanent axial strain after N_{ref} number of cycles,

N_{ref} = any given number of load cycles greater than 100,

L = length of stress path,

P_o = reference stress (e.g. 1 kPa) introduced to ensure non-dimensionality of the equation,

$(q/p)_{max}$ = maximum shear to normal stress ratio, and

a, b = regression parameters.

This equation is applicable for any given number of load cycles greater than 100.

The NCHRP project 1-37 "Development of the 2002 Guide for the Design of New and Rehabilitated Pavement Structures" proposed the following model for the determination of permanent deformation (*31*).

$$\delta_a = \beta_{s1}\ \varepsilon_v\ h\ (\varepsilon_0/\varepsilon_r)\ [e^{-(\rho/N)\beta}] \qquad (18)$$

where

δ_a = permanent strain in unbound aggregate base,

β_{s1} = field calibration factor,

ε_v = average vertical resilient strain obtained from software,

h = layer thickness,

ε_0, ρ, β = regression constants,

ε_r = resilient strain imposed in lab testing, and

$\varepsilon_0/\ \varepsilon_r$ = ratio combined to represent a material property.

Apart from the test procedures, the choice of models also induces variation in the test results. A best fit of the appropriate test procedures and the models would enable determination of the values of the resilient modulus and permanent deformation close to those measured in the field. However, allowance is to be made for variability induced due to the test procedures or the operators and other such factors. Hence, it is necessary to specify the required level of accuracy of measurements which is included as a precision and bias statement in the test procedures. The following section provides a brief discussion on precision and bias.

PRECISION AND BIAS

Apart from the factors mentioned in the previous sections, there is inherent variability within the test method. Many different factors contribute to the variability in the application of a test method. These include: 1) the operator, 2) equipment used, 3) calibration of the equipment, and 4) environment (temperature, humidity, air pollution etc) (*7*).

Precision is expressed in terms of two measurement concepts, repeatability and reproducibility (*7*). In this research only repeatability conditions are considered wherein the same operator conducts the tests under identical environmental conditions using the same equipment. Precision is usually expressed as the standard deviation or some multiple of the standard deviation. A statement on precision provides guidelines for the kind of variability that can be expected between test results when the test method is correctly used for the conditions specified (*35*). A statement of precision allows users of a test method to assess in general terms the test method's usefulness with respect to variability in proposed applications. A statement on precision is not intended to contain values that can be exactly duplicated in every user's laboratory. Instead, the statement provides guidelines as to the kind of variability that can be expected between test results when the method is used in one or more reasonably competent laboratories (*36*).

Bias is a systematic error that contributes to the difference between the mean of a large number of test results and an accepted reference value (*7*). In determining the bias, the effect of the imprecision is averaged out by taking the average of a large set of test results. This average minus the accepted reference value is an estimate of the bias of the test method. For the present study if an accepted reference value is not available, the bias cannot be established. A statement on bias furnishes guidelines on the relationship between a set of typical test results produced by the test method under specific test conditions and a related set of accepted reference values (*36*).

SUMMARY

From the literature review it is evident that the characterization of base materials based on repeated loading testing is quite complex. Several researchers have used the repeated loading triaxial test for the determination of the resilient modulus and the permanent deformation properties. Extensive research has been conducted on the factors that affect these properties. Of these the stress state had the most effect on these properties. Hence, these values are reported at a specified stress state as is observed in the test procedures. However, different test procedures in usage at different locations have made it difficult to ascertain a common classification of the base materials based on resilient and permanent deformation properties. The recently completed NCHRP project takes a big step forward toward a unified test procedure. The harmonized protocol has been developed under the NCHRP project to standardize a testing procedure for the determination of resilient modulus which also includes a model with the best statistical accuracy.

Though permanent deformation properties are important for characterizing the behavior of base material, limited research has been conducted in this area (*37*).Of the test methods described in the literature review based on frequency of use, it is evident that only the ICAR test protocol includes the determination of permanent deformation properties. Furthermore, there was no precision or bias statement for these existing test procedures, which is very important for their standardization.

CHAPTER III

RESEARCH METHODOLOGY

INTRODUCTION

This chapter consists of a description of the research methodology including the experimental setup and test matrix. The tests conducted are briefly described with a sample test result. A new test procedure for the determination of resilient modulus and permanent deformation properties of granular materials is proposed. Further, the salient features of this test procedure are discussed. A description of this procedure is provided in the following sections including the test apparatus and the test specimen preparation.

EXPERIMENTAL DESIGN

A within–laboratory study was conducted for the evaluation of the proposed test procedure. For any test method it is necessary that the inherent variability in the test procedure be minimized. Specimens were compacted to two different sizes: 1) 6 in. (152 mm) diameter by 12 in. (305 mm) height and 2) 6 in. (152 mm) diameter by 8 in. (203 mm) height to be used in the proposed performance test. The present study evaluates the variability between independent test results obtained within a single laboratory in the shortest practical period of time by a single operator with a specific test apparatus using test specimens (or test units) taken at random from a single quantity of homogeneous material obtained or prepared for the study. The repeatability of the test method with the above specified conditions is evaluated. Further, the influence of reducing the specimen height from 12 in. (305 mm) to 8 in. (203 mm) is investigated.

MATERIALS

For the following study, crushed granite that has demonstrated good field performance as a base material was used. The source of the material is Spicewood springs, and it is used on interstate highway I-30 in Oklahoma. It was necessary to conduct a preliminary assessment of the mechanical properties of the materials such as the gradation, dry density and moisture content which influence the compaction and specimen preparation characteristics for the proposed performance test.

PRELIMINARY TESTING

Preliminary testing of the material was conducted and the mechanical properties were determined according to the Texas manual of testing procedures (*38*). After completion of the preliminary tests, the gradation and optimum moisture content results were used in the preparation and compaction of test specimens. The preliminary tests that were conducted are the particle size analysis, determination of liquid limit, determination of plastic limit and determination of plasticity index, and determination of moisture density relationship. The results of these tests are provided in Table 1.

TABLE 1 Preliminary Tests Results

Test	TxDOT Specification	Property measured
Particle size analysis	Tex-101-E	Gradation
Determination of Liquid limit	Tex-104-E	Liquid Limit – 19
Determination of Plastic Limit	Tex-105-E	Plastic Limit – 16
Determination of Plasticity Index	Tex-106-E	Plastic Limit – 16
Laboratory Compaction Characteristics and Moisture-Density Relationship of Base Materials	Tex-113 E	Moisture Density Relationship

Sieve Analysis

Dry and wet sieve analysis was performed on the spicewood material. Sieving of approximately 772 lb (350 kg) of aggregate was accomplished in several batches. Each batch was oven-dried at 110 ºC for 24 hours before being sieved, and the materials retained on each sieve were stored separately in buckets. Following completion of the sieving process, the total weight of each size fraction was measured, and the master gradation was produced. For the wet sieve analysis, a representative sample of 7.7 lb (3.5 kg) of the material was washed with distilled water for separate size fractions. The washed material was dried at 110 ^{o}C until all the material was dry, and the retained material on each sieve size was weighed. Figure 6 shows the gradation of the sample and that of the TxDOT specifications.

FIGURE 6 Gradation of sample.

The dry and wet sieve analysis show that the gradation was within the specification limits for TxDOT. As expected the fines content was more with the wet sieve analysis than the dry sieve analysis. The dry sieve analysis is used as the master gradation for reconstituting the specimens as both the gradations obtained from dry sieve and wet sieve analysis are within the specification limits for TxDOT.

Moisture Density Relationship

Based on dry sieve analysis, four replicate specimens of unbound granular base material were prepared for moisture-density testing using distilled water. A different amount of water was added to each sample, and the moistened aggregate samples were each compacted in four equal lifts inside a cylindrical metal mold to a finished height of about 6 in. (152 mm). Each layer was compacted with 50 blows of a 10 lb (4.5 kg) hammer dropped from a height of 8 in. (203 mm). The molding and compacting equipment is shown in Figure 7.

FIGURE 7 Molding and compacting equipment.

The weights and dimensions of the specimens were measured, and after drying in an oven at 110 ºC, the dry densities corresponding to the different moisture contents were computed. These results are shown in Figure 8. The optimum moisture content was determined to be 5.6 percent with a corresponding maximum dry density of 146.8 lb/ft^3 (2352.6 kg/m^3).

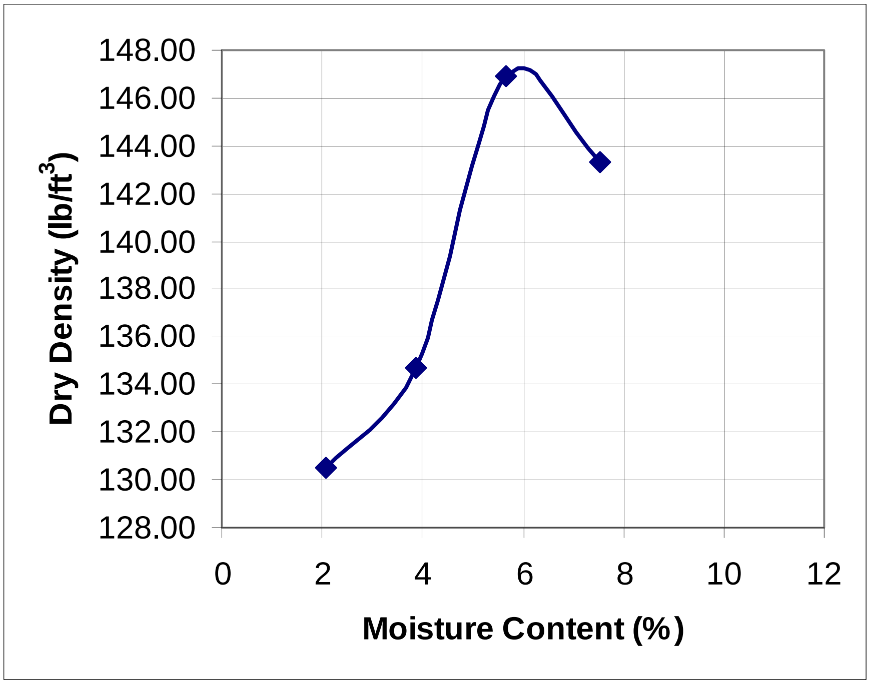

FIGURE 8 Moisture density relationship.

Following determination of the dry weight of aggregate necessary for construction of a single cylindrical specimen, the aggregate was recombined into the required number of replicate samples, based on the master gradation.

PERFORMANCE TEST SEQUENCE

After the completion of the preliminary testing of the material, the proposed performance test was conducted. The proposed performance test sequence is described in detail in this section including the test apparatus, test specimen preparation, and the test sequence.

Proposed Performance Test

Previous research studies focused on the determination of resilient modulus, and relatively little research has been conducted on the determination of permanent deformation properties of granular materials. NCHRP 1-37 A "Development of the 2002 guide for the design of new and rehabilitated pavement structures" also requires the resilient modulus value as an input for level 1 design. It further recommends AASHTO T307 specification for the determination of resilient modulus value which does not include the determination of permanent deformation properties. The proposed performance test procedure integrates the determination of permanent deformation properties along with the determination of resilient modulus values of unbound granular materials.

The test sequence is adapted from the standard test methods given by the VESYS user manual, NCHRP1-28 A report and AASHTO T307, TP46 (*17,34, 39,40*). The data acquisition system is completely automated. The stress sequence follows the recommendations by NCHRP 1-28 A project which is a more rational approach than the stress sequences followed by current standards (*17, 39*) that maintains a constant stress ratio(ratio of maximum axial stress to confining pressure, σ_1/σ_3) by increasing both the principal stresses simultaneously. Since the sequence starts with the minimum stress ratio, the probability of failing the sample is minimized. Then, a similar sequence is performed at a higher stress ratio, producing a higher probability of failure. The method is illustrated in Figure 9 that indicates the sequence of the confining pressure and the deviatoric stress applied. Both the confining pressure and the deviatoric stress should be varied such that increasing stress levels are applied on the specimen while keeping the stress ratio constant. This would prevent the premature failure of the specimen as the specimen is not subjected to high stress ratios in the earlier sequences. Also, this enables for testing the specimen beyond the line of failure and studying the behavior of material as the stress levels increase beyond the line of failure

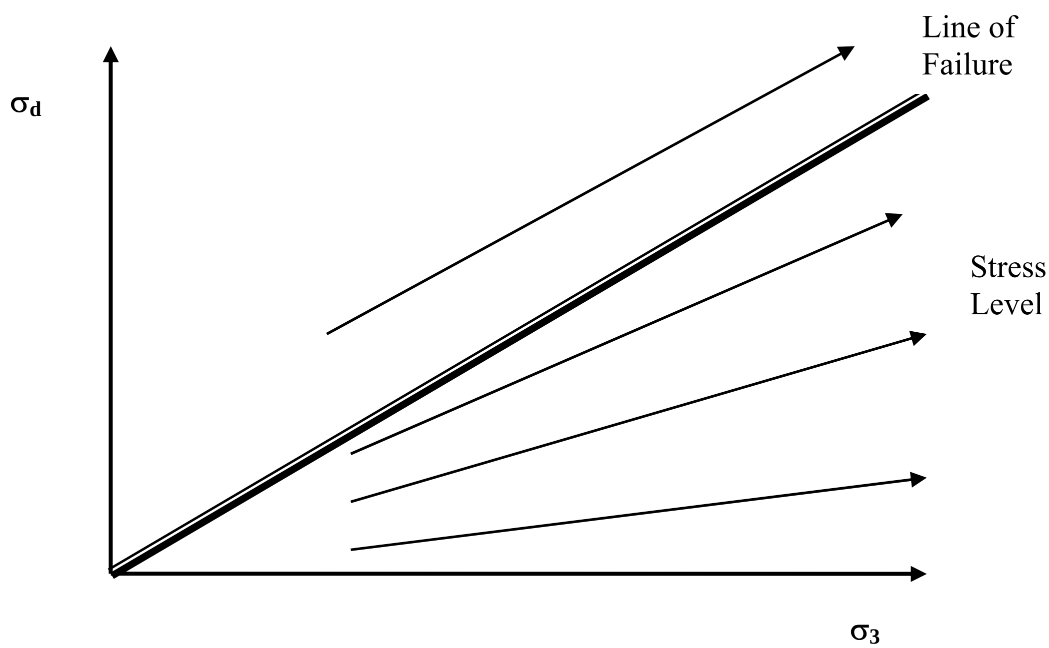

FIGURE 9 Stress sequence compared for granular materials.

Test Apparatus

The test apparatus consists of a triaxial chamber, loading device, response measuring equipment and data acquisition system. The triaxial pressure chamber is used to contain the test specimen and the confining fluid during the test as shown in Figure 10. Air is used in the triaxial chamber as the confining fluid for all testing. The axial deformation is measured internally, directly on the specimen using normal gauges with rubber bands (shown in Figure 11), non-contact sensors and clamps.

FIGURE 10 Test setup for resilient modulus.

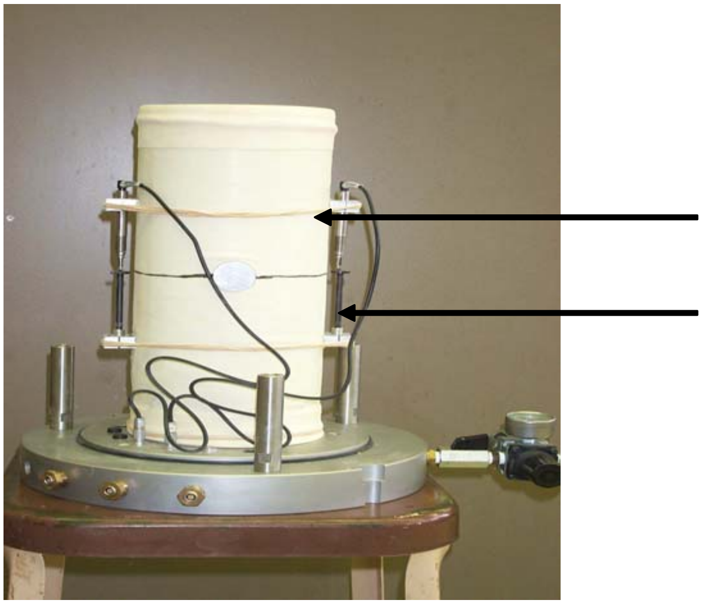

FIGURE 11   Specimen prepared for testing.

The loading device consists of a top loading, closed-loop electro-hydraulic testing machine capable of applying repeated cycles of a haversine shaped load pulse (0.1 sec loading and 0.9 sec unloading).

The axial load measuring device is an electronic load cell, which is located inside the triaxial cell. Axial deformation is measured over approximately the middle ½ of the specimen. Axial deformation is measured at a minimum of two locations 180^{o} apart (in plan view), and a pair of spring loaded LVDTs are placed on the specimen at the 1/4 point. Spring loaded LVDTs are used to maintain a positive contact between the LVDTs and the surface on which the tips of the transducers rest. The distance between the LVDTs is the gauge length used to compute strain. Table 2 summarizes the positioning of spring loaded LVDTs for the two specimen sizes.

TABLE 2 Positioning of Axial LVDTs

Material/ Specimen Size		Distance between the LVDTs
Aggregate Base	6 in. diameter, 12 in. height	6 in
	6 in. diameter, 12 in. height	4 in

The data acquisition system is completely automated. The test apparatus complies with the specifications of AASHTO T307 (*17*).

Test Specimen Preparation

Preparation of the specimens included dry mechanical sieving into various size fractions, determining the optimum moisture content and maximum dry density, recombining the aggregate into replicate samples, and compaction.

The standard method of sample preparation given in AASHTO T307 was followed for the sample preparation (*17*). The optimum moisture content and maximum dry density results are used for the compaction of the specimen for the performance test. The required amount of material is mixed with the optimum amount of water and compacted to the specified dimensions. In this study, the specimen dimensions used are: 6 in. (152 mm) diameter with 12 in. (305 mm) height and 6 in. (152 mm) diameter with 8 in. (203 mm) height. The compaction and molding equipment is shown in Figure 7. After compaction of the specimen, it was extruded from the compaction mold as shown in Figure 12.

FIGURE 12 Extrusion of specimen from the compaction mold.

The specimen was placed on a porous stone/base after extrusion from the compaction mold. The membrane was placed on a membrane stretcher, and a vacuum was applied to the stretcher. The membrane was then carefully placed on the specimen. The membrane stretcher was removed from the membrane by cutting off the vacuum.

After placing the rubber membrane around the specimen, it was kept in the humidity chamber for approximately 16 hours or overnight, to allow for the uniform distribution of the water within the specimen. After preparation of the test specimen, it was subjected to repeated triaxial testing. The compacted specimen was prepared for testing by placing a rubber membrane around it. The membrane was sealed to the top and bottom platens with rubber "O" rings as shown in Figure 13.

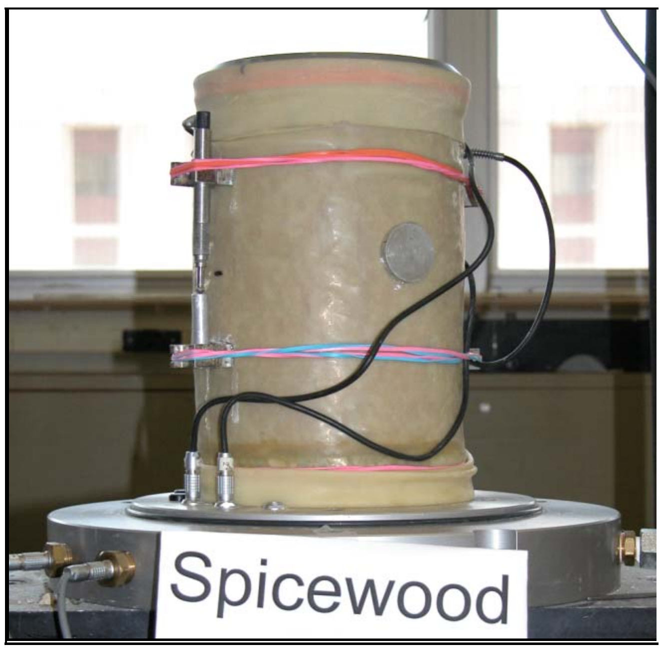

FIGURE 13 Test specimen prepared for testing.

Test Sequence

After the preparation of the test specimen, the following testing sequence was used. The stress sequence follows the recommendations by NCHRP 1-28 A for base/subbase materials that maintains a constant stress ratio by increasing both the principal stresses simultaneously. The test sequence consisted of three stages:

1. Preliminary conditioning
2. Permanent deformation test
3. Resilient modulus test

Conditioning

The specimen was preconditioned before testing by applying 100 repetitions of a load equivalent to a maximum axial stress of 6 psi (41.4 kPa) and a corresponding cyclic stress of 3 psi (20.7 kPa) using a haversine shaped 0.1 second load pulse followed by a 0.9 second rest period. A confining pressure of 15 psi (103.5 kPa) was applied to the test specimen. A schematic representation of the load and the placement of Linear Vertical Displacement Transducers (LVDT) are shown in Figure 14. σ_d is the axial deviatoric stress, and σ_3 is the confining pressure. LVDTs 1 and 2 measure the axial displacement, and LVDTs 3 and 4 measure the radial displacement.

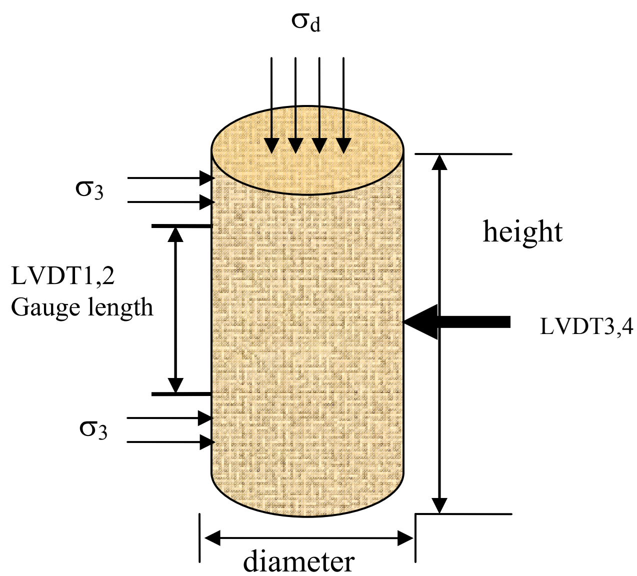

FIGURE 14 Representation of load and position of LVDTs on specimen.

Permanent Deformation Test

A haversine load equivalent to a maximum axial stress of 33 psi (of 227.7 kPa) and a corresponding cyclic stress of 30 psi (207 kPa) with a 0.1 second load pulse followed by a 0.9 second rest period was continued until 10,000 load applications or until the vertical permanent strain reached 3% during the testing, whichever comes first. During load applications, the load applied and the axial deformation measured from two LVDTs through the data acquisition system was recorded. In order to save storage space during data acquisition, the data was recorded at specified intervals shown in Table 3.

TABLE 3 Suggested Data Collection for Permanent Deformation Test

Data Collection During Cycles			
1-15	450	1300	4000
20	500	1400	4500
30	550	1500	5000
40	600	1600	5500
60	650	1700	6000
80	700	1800	6500
100	750	1900	7000
130	800	2000	7500
160	850	2200	8000
200	900	2400	8500
250	950	2600	9000
300	1000	2800	9500
350	1100	3000	10000
400	1200	3500	

Resilient Modulus Test

The same specimen was used to perform the resilient modulus test if the vertical permanent strain did not reach 3%. Otherwise, a new specimen was molded, and the permanent deformation test was performed with the load repetitions reduced to 5,000 from 10,000. If the sample again reached 3% total permanent strain, the test was terminated. If not, the resilient modulus test was performed by initially decreasing the axial stress to 2.1 psi (14.5 kPa) and setting the confining pressure to 3 psi (20.7 kPa). The test was performed by following the sequence of loading at regular intervals shown in Table 4 which was recommended in NCHRP project 1-28 A (*4*).

The test was stopped when the total permanent strain of the sample exceeded 3%, and the result was reported. After the completion of the test, the confining pressure was reduced to zero and the specimen was removed from triaxial chamber. The moisture content of the specimen was determined at the end of the test using AASHTO T265-93 (*41*). The testing sequence is shown schematically in Figure 15.

TABLE 4 Permanent Deformation and Resilient Modulus Test Sequence for Granular Base

Sequence	Confining Pressure		Contact Stress		Cyclic Stress		Maximum Stress		N_{rep}
	kPa	psi	kPa	psi	kPa	psi	kPa	Psi	
Preconditioning									
	103.5	15.0	20.7	3.0	20.7	3.0	41.4	6.0	100
Permanent Deformation									
	103.5	7.0	20.7	3.0	193.0	28.0	213.7	31.0	10000
Resilient Modulus									
1	20.7	3.0	4.1	0.6	10.4	1.5	14.5	2.1	100
2	41.4	6.0	8.3	1.2	20.7	3.0	29.0	4.2	100
3	69.0	10.0	13.8	2.0	34.5	5.0	48.3	7.0	100
4	103.5	15.0	20.7	3.0	51.8	7.5	72.5	10.5	100
5	138.0	20.0	27.6	4.0	69.0	10.0	96.6	14.0	100
6	20.7	3.0	4.1	0.6	20.7	3.0	24.8	3.6	100
7	41.4	6.0	8.3	1.2	41.4	6.0	49.7	7.2	100
8	69.0	10.0	13.8	2.0	69	10.0	82.8	12.0	100
9	103.5	15.0	20.7	3.0	103.5	15.0	124.2	18.0	100
10	138.0	20.0	27.6	4.0	138	20.0	165.6	24.0	100
11	20.7	3.0	4.1	0.6	41.4	6.0	45.5	6.6	100
12	41.4	6.0	8.3	1.2	82.8	12.0	91.1	13.2	100
13	69.0	10.0	13.8	2.0	138	20.0	151.8	22.0	100

TABLE 4 continued

Sequence	Confining Pressure		Contact Stress		Cyclic Stress		Maximum Stress		N_{rep}
	kPa	psi	kPa	psi	kPa	psi	kPa	Psi	
Resilient Modulus									
14	103.5	15.0	20.7	3.0	207	30.0	227.7	33.0	100
15	138.0	20.0	27.6	4.0	276	40.0	303.6	44.0	100
16	20.7	3.0	4.1	0.6	62.1	9.0	66.2	9.6	100
17	41.4	6.0	8.3	1.2	124.4	18.0	132.5	19.2	100
18	69.0	10.0	13.8	2.0	207	30.0	220.8	32.0	100
19	103.5	15.0	20.7	3.0	310.5	45.0	331.2	48.0	100
20	138.0	20.0	27.6	4.0	414.0	60.0	441.6	64.0	100
21	20.7	3.0	4.1	0.6	103.5	15.0	107.6	15.6	100
22	41.4	6.0	8.3	1.2	207	30.0	215.3	31.2	100
23	69.0	10.0	13.8	2.0	345.0	50.0	358.8	52.0	100
24	103.5	15.0	20.7	3.0	517.5	75.0	538.2	78.0	100
25	138.0	20.0	27.6	4.0	690.0	100.0	717.6	104.0	100
26	20.7	3.0	4.1	0.6	144.9	21.0	149.0	21.6	100
27	41.4	6.0	8.3	1.2	289.8	42.0	298.1	43.2	100
28	69.0	10.0	13.8	2.0	483.0	70.0	496.8	72.0	100
29	103.5	15.0	20.7	3.0	724.5	105.0	745.2	108.0	100
30	138.0	20.0	27.6	4.0	966.0	140.0	993.6	144.0	100

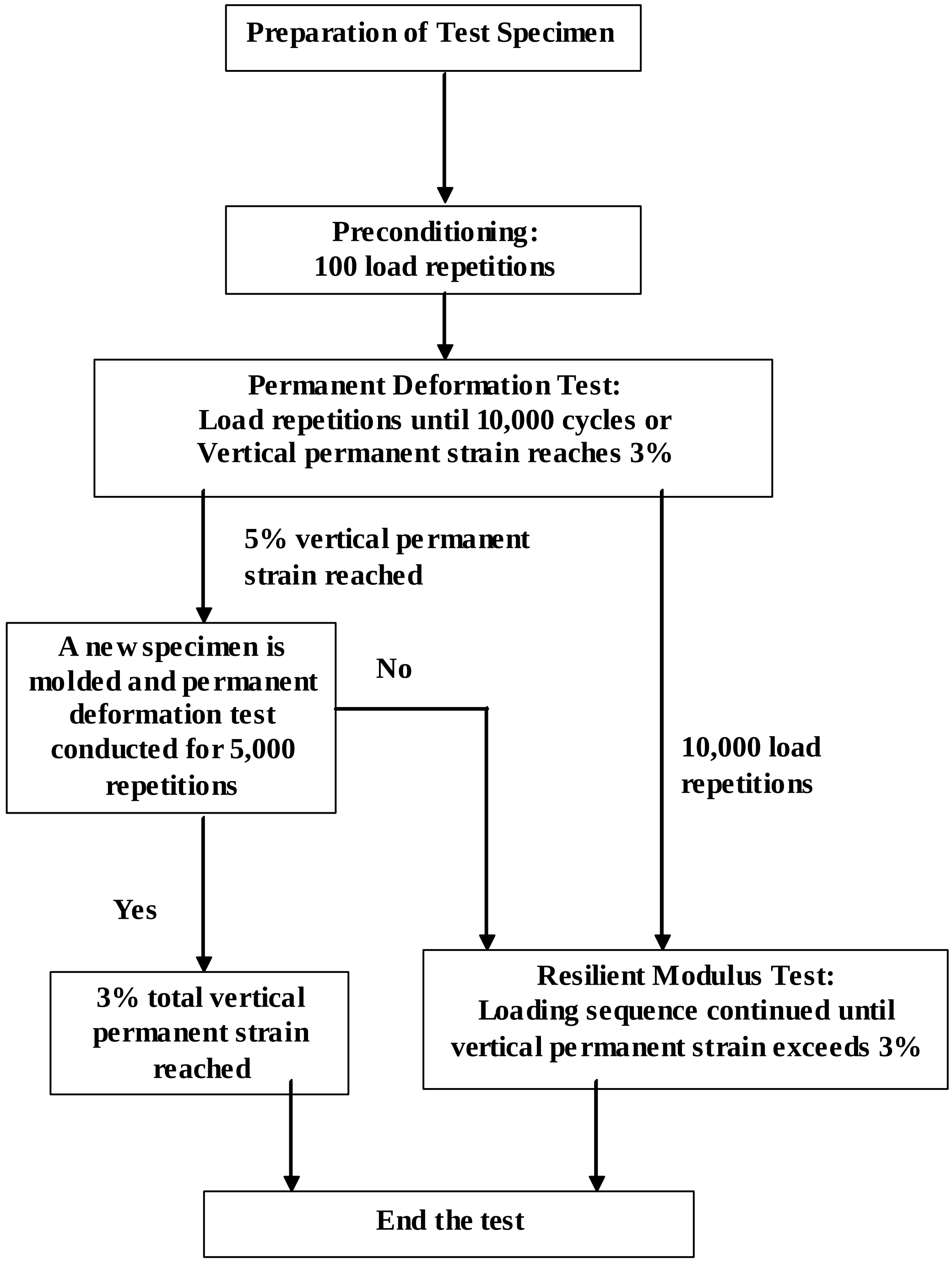

FIGURE 15 Flowchart of the test procedure for permanent deformation and resilient modulus.

Calculations

The following results are computed from the test:

Permanent deformation properties

- Average axial deformation is determined for each specimen by averaging the readings from the two axial LVDTs. The total axial strain is determined by dividing by the gauge length (L). Cumulative axial permanent strain and resilient strain at the 500^{th} load repetition are calculated.
- A graph is plotted between the cumulative axial permanent strain and the number of loading cycles in log space (shown in Figure 16). The permanent deformation parameters, intercept (a) and slope (b), are determined from the linear portion of the permanent strain curve (log-log scale), which is also demonstrated in Figure 16.
  - Rutting parameters α and μ are determined using the following equations as shown in equation (14):

$$\alpha = 1 - b$$

$$\mu = \frac{a \times b}{\varepsilon_r}$$

Resilient Modulus parameter

- The resilient modulus values are computed from each of the last 5 cycles of each load sequence which are then averaged.

- The data obtained from the applied procedure is fit to the following resilient modulus model using nonlinear regression techniques as shown in Figure 17.

The resilient modulus is calculated by the following equation (7) which is being adapted in NCHRP 1-37A project (2002 design guide):

$$M_r = k_1 P_a \left(\frac{\theta}{P_a} \right)^{k_2} \left(\frac{\tau_{oct}}{P_a} + 1 \right)^{k_3}$$

Where:

$k_1, k_2 \geq 0$,

$k_3 \leq 0$,

M_R = resilient modulus,

τ_{oct} = octahedral shear stress $= \frac{1}{3}\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_1 - \sigma_3)^2 + (\sigma_2 - \sigma_3)^2}$,

θ = bulk stress = $\sigma_1 + \sigma_2 + \sigma_3$,

σ_1, σ_2, σ_3 = principal stresses,

k_i = regression constants, and

P_a = atmospheric pressure.

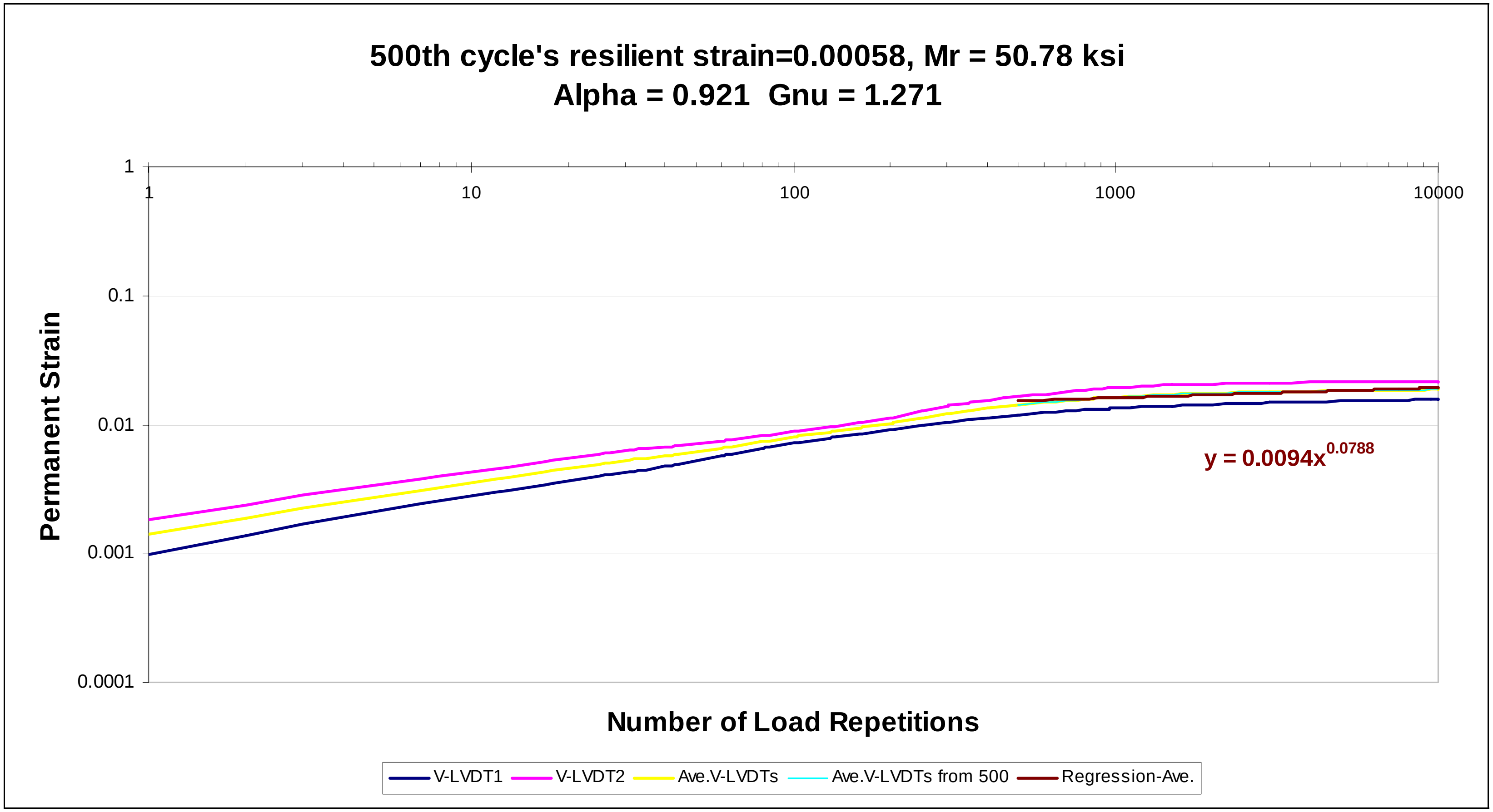

FIGURE 16 Sample plot of permanent strain versus number of load cycles.

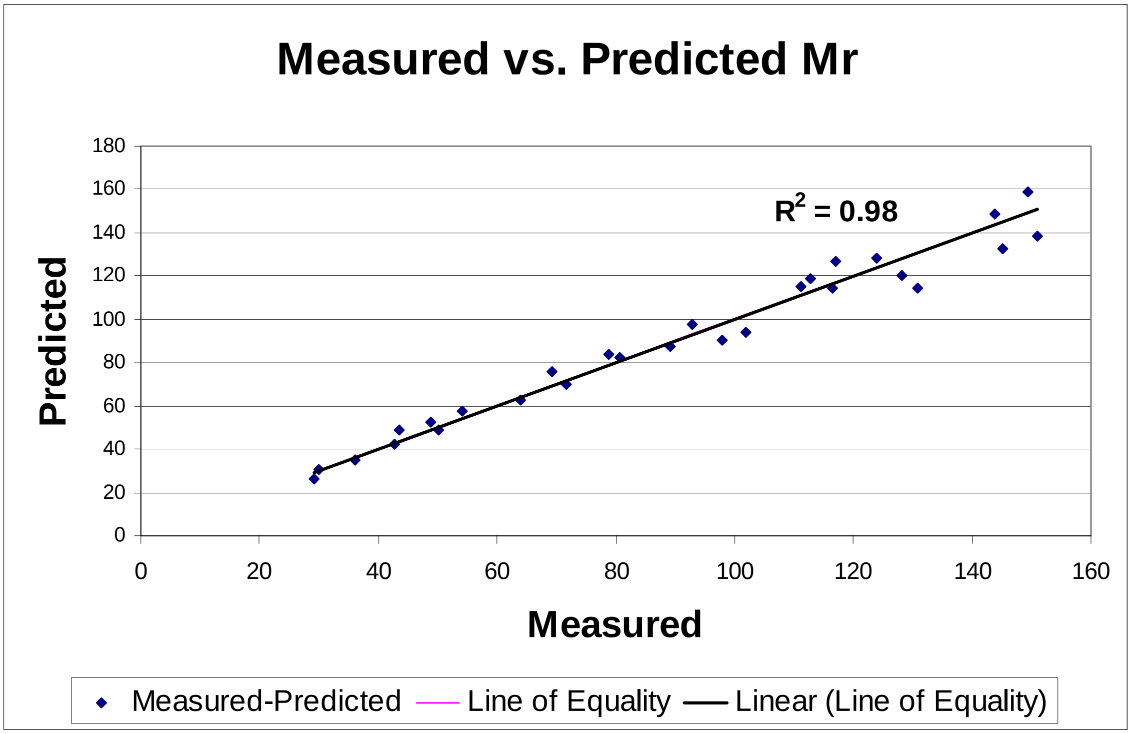

FIGURE 17 Sample regression plot of measured versus predicted values.

The permanent deformation properties were determined at a confining pressure of 7 psi (49.2 kPa) and a deviatoric stress of 28 psi (193 kPa). The resilient modulus values were reported at 5 psi (34.4 kPa) confining pressure and 15 psi (103.4 kPa) deviatoric stress.

PRECISION AND BIAS OF TEST METHOD

Seven specimens were compacted for the determination of resilient modulus and permanent deformation using the proposed performance test. The within laboratory variability analysis was performed on the test results. Based on this analysis the within laboratory precision and bias of the test method is estimated. The precision and bias of test methods was discussed in detail in Chapter II.

INFLUENCE OF SPECIMEN SIZE

The influence of preparing samples with a specimen height of 8 in. (203 mm) instead of the 12 in. (305 mm) height in the proposed performance test is investigated. For the 6 in. (152 mm) by 12 in. (305 mm) (diameter by height) specimens, the gauge length for measuring the axial strains is 6 in. (152 mm). The length to diameter ratio and the length to gauge length ratio used were in accordance with the standard practices (*17, 39*). However, for the use of an 8 in (203 mm) height specimen, the gauge length should also be changed. A gauge length ratio of 0.5 or lower is recommended by researchers (*6*). Hence, for the 8 in. (203 mm) high specimen, a gauge length of 4 in. (102 mm) was used. This enables the placement of the LVDTs and the measurement of the axial deformations at a distance considerably far from the end platens. Measuring the axial deformations closer to the end platens will result in an overestimation of the modulus value of the material. Conversely, measuring the axial deformations closer to the center of the specimen will lead to accurate estimation of the stiffness parameters (*6*). Hence, the configuration of an 8 in specimen with a gauge length of 4 in. (102 mm) is reasonable for the determination of modulus values. Figure 18 shows the specimen with height 8 in. (203 mm) after testing.

FIGURE 18 Specimen after testing.

SUMMARY

This chapter discusses the research methodology of the present study. The proposed performance test sequence is discussed in detail including the test apparatus, test specimen preparation and the stress sequence applied during the test. Replicate specimens were prepared to determine the sample size required for testing. The following were the objectives of the experimental design:

- Evaluate the variability of the test results determined from the test

- Estimate the number of test specimens required for a given tolerance level of the test results
- Investigate the influence of changing the specimen size from 6 in. (152 mm) diameter by 12 in. (305 mm) height to 6 in. (152 mm) diameter by 8 in. (203 mm) height on the test results.

This thesis aims to determine the precision and bias of the proposed testing procedure, which is a combination of several test procedures. This test procedure includes the determination of both resilient modulus and permanent deformation properties. It should give a measure of accuracy for the determination of the resilient and permanent deformation properties in the within laboratory conditions and their appropriate use in the field. Also, the influence of sample size and method of compaction on the test results is studied.

CHAPTER IV

TEST RESULTS AND ANALYSIS

INTRODUCTION

This chapter consists of the description of the results of the performance test conducted in this study. The permanent deformation and resilient modulus results are presented. An analysis on the influence of stress ratios on the test results is conducted. The within laboratory variability of the test results is evaluated. Also, the number of test specimens required for desired reliability of this test method is estimated. Further, the within laboratory precision of the test method is established. Subsequently, the influence of specimen size on test results is investigated.

PERFORMANCE TEST RESULTS

As discussed in Chapter III, the performance test results determine the permanent deformation properties and the resilient modulus values. In this section, the permanent deformation properties are presented followed by the resilient modulus parameters for a 6 in. (152 mm) diameter and 12 in. (305 mm) height (6 in. by 12 in.) specimen.

Permanent Deformation Properties

The accumulation of permanent deformation with number of load cycles is indicated by the graph plotted on a logarithmic scale with permanent strain on the y axis and number of load cycles on the x axis. A typical plot is shown in Figure 19 that indicates that as the number of load cycles increases, the permanent strain increases. The graph shows a linear relationship between the ε_p and number of load cycles on the logarithmic scale. From the graph, permanent deformation parameters α and μ of the VESYS model are determined. The VESYS model is expressed as follows (*34*):

$$\varepsilon_p(N)/\varepsilon_r = \mu N^{-\alpha} \tag{18}$$

The permanent deformation increases at a higher rate for the initial load cycles and at a lower rate as the number of load cycles increases. Thus, the relationship between permanent deformation and number of load cycles is an expression of the form:

$$Y = a\, x^{b} \tag{19}$$

From this expression, the permanent deformation parameters μ and α are determined using the following expressions:

$$\mu = \frac{a \bullet b}{\varepsilon_r}$$

where

ε_r = resilient strain, and

α = 1-b.

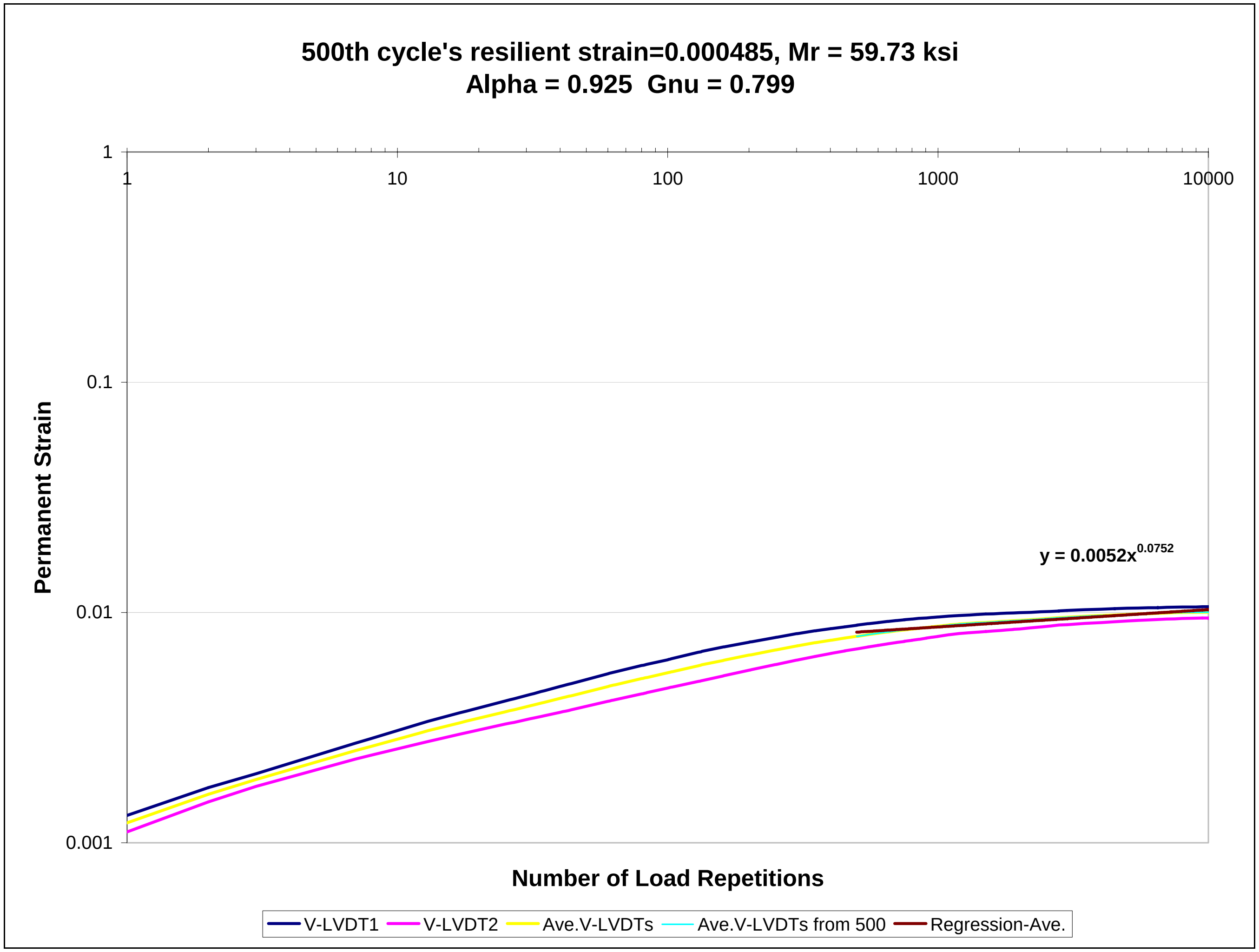

FIGURE 19 Plot of permanent strain versus number of load cycles.

The individual plots for all the specimens tested are presented in Appendix A. Table 5 provide a summary of the test results of the permanent deformation parameters. Thompson stated that for reasonable stress states, the “b” term in equation (19) for soils and granular materials is generally within the range of 0.12 to 0.2. The lower values are for the soils. He also indicated that the “a” term was quite variable and is dependent on material type, repeated stress state and factors influencing material shear strength (*42*).

TABLE 5 Permanent Deformation Test Result for Specimen Size 6 in. by 12 in.

Specimen	ε_r at 500th load cycle	M_r (ksi)	Rutting parameters	
			μ	α
1	0.000546	54.18	1.478	0.860
2	0.000485	59.73	0.799	0.925
3	0.000580	50.78	1.271	0.921
4	0.000619	48.71	1.280	0.912
5	0.000641	45.51	1.470	0.911
6	0.000687	43.33	0.844	0.933
7	0.000589	50.07	1.544	0.926

From Table 5, the μ values range from 0.79 to 1.5, and α values range from 0.86 to 0.93. Also, the resilient strain at the 500th repetition is used to compute the resilient modulus values at 7 psi (48 kPa) confining pressure and 28 psi (193 kPa) deviator stress.

Bonaquist and Witzack (*43*) indicated that the typical values of α and μ range between 0.85 to 0.95 and 0.1 to 0.4, respectively. The higher the value of α, the lower the slope of the curve and the lower the rate of accumulated strain. The values of μ are high compared to the values reported by Bonaquist and Witzack (*43*). This is due to the high stress level at which the testing was conducted. These properties depend on the ratio of maximum axial stress to the confining pressure (σ_1/σ_3), termed the stress ratio. The higher the stress ratio, the higher is the accumulation of permanent strain.

Figure 20 shows the relationship between the ratio of permanent strain to resilient strain ($\varepsilon p/\varepsilon_r$) and the number of load cycles for all specimens. The results indicate a linear relationship. Thus as the number of load cycles increases, the ratio $\varepsilon_p/\varepsilon_r$ increased.

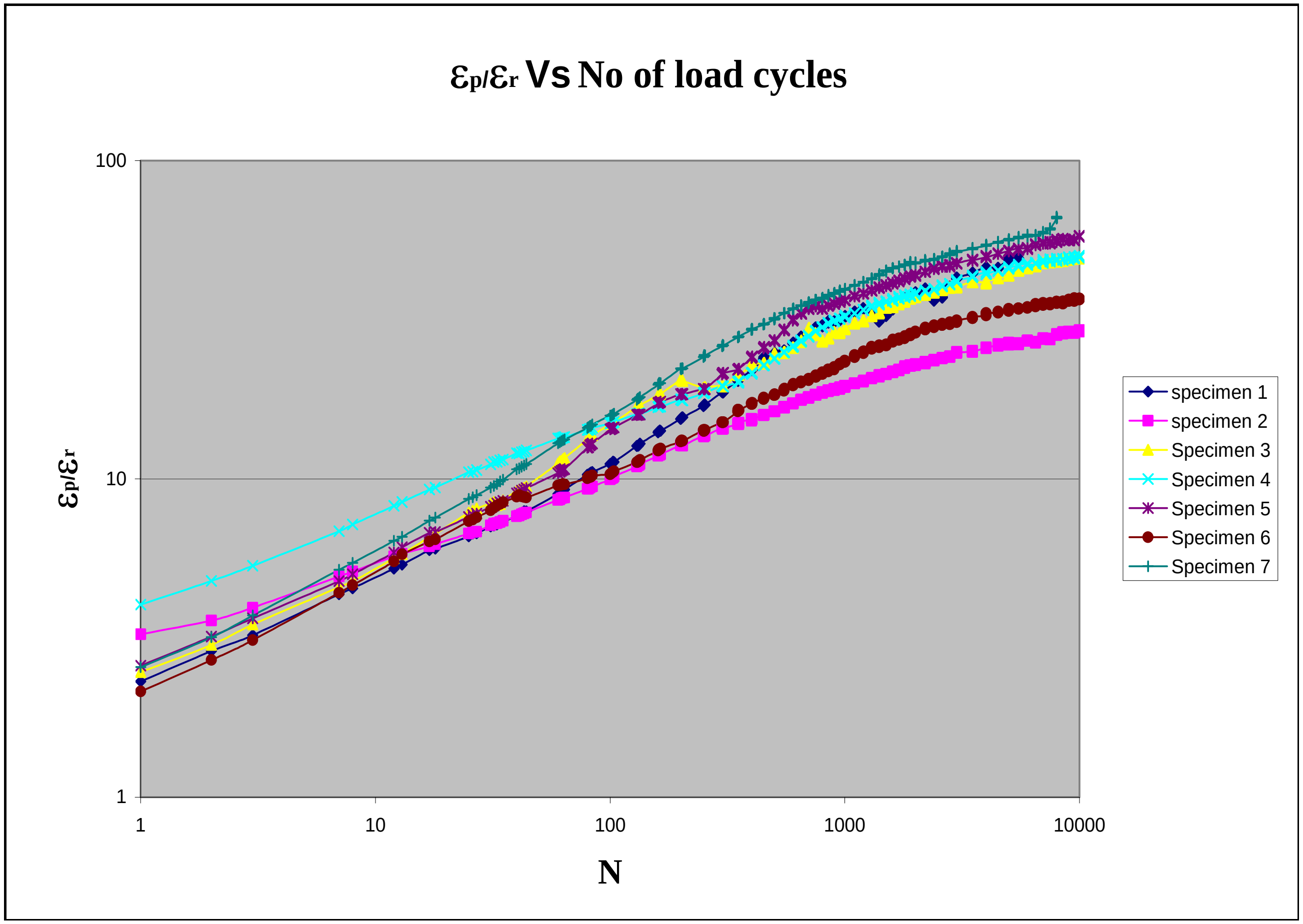

FIGURE 20 Plot of $\varepsilon_p/\varepsilon_r$ with number of load cycles.

Resilient Modulus Test Results

The resilient modulus value is computed from the model described in Chapter II. The model with k_1, k_2 and k_3 parameters is chosen for the determination of the resilient modulus. This would also provide input parameters for the characterization of unbound granular base material in the proposed 2002 design guide (*31*). The measured resilient modulus values are compared to the predicted values, and the parameters are determined by regression analysis.

The resilient modulus is calculated by the following equation (7) which is being adapted in NCHRP 1-37A (*31*):

$$M_r = k_1 P_a \left(\frac{\theta}{P_a} \right)^{k_2} \left(\frac{\tau_{oct}}{P_a} + 1 \right)^{k_3}$$

The regression parameters k_1, k_2, and k_3 computed by this model for each of the specimen are presented in Table 6. These parameters are used to calculate the resilient modulus value at a specific confining pressure and deviatoric stress. Here, the resilient modulus value at 5 psi (34.5 kPa) confining pressure and 15 psi (103.4 kPa) deviatoric stress are computed for comparison. The individual regression plots for each of the seven specimens are provided in Appendix B.

TABLE 6 Resilient Modulus Test Results for Specimen Size 6 in. by 12 in. at 5 psi Confining Pressure and 15 psi Deviatoric Stress

Specimen	k_1	k_2	k_3	M_r (ksi)
1	1699.49	0.71	0.04	42.15
2	2424.13	1.13	-0.99	54.21
3	2591.25	1.02	-0.98	53.63
4	2406.69	0.81	-0.55	50.73
5	2321.15	1.05	-0.83	51.97
6	2002.96	0.74	-0.45	41.94
7	2057.65	1.25	-1.26	45.02

The results of Table 6 are used to estimate the precision and bias of the test method which will be discussed subsequently.

From Table 6, the average resilient modulus value was 48.5 ksi (334.4 MPa), which is typical of a good unbound granular base material. Figure 21 shows the resilient modulus values for each of the seven specimens tested. It indicates that there is not much variation among the results for resilient modulus values.

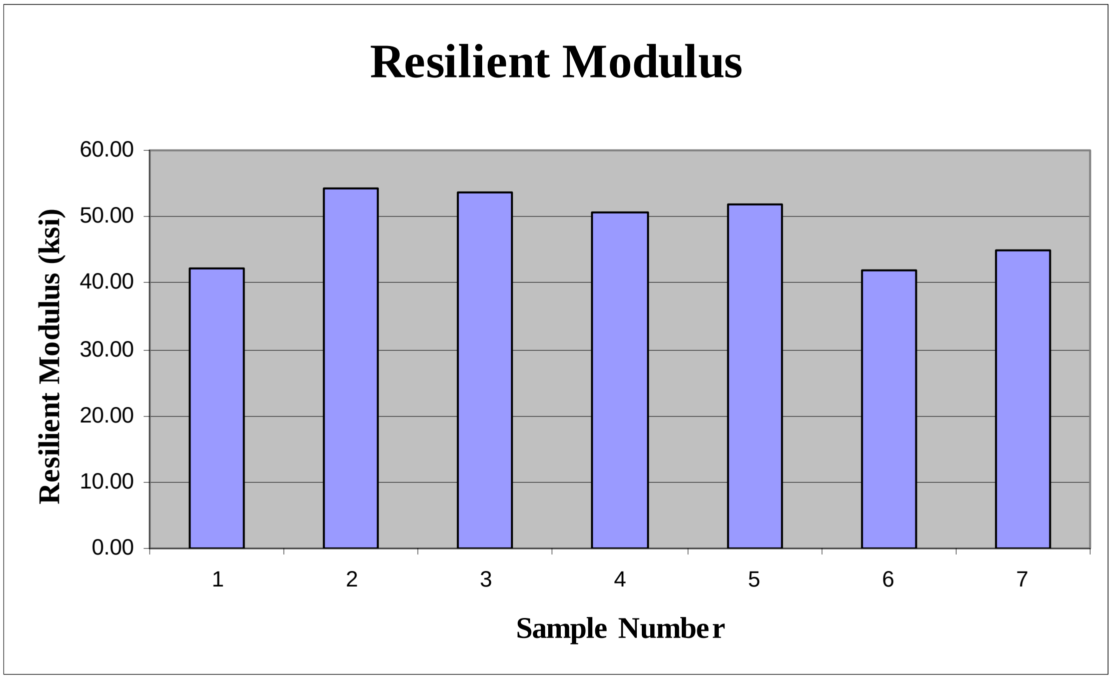

FIGURE 21 Resilient modulus value at 5 psi confining pressure and 15 psi deviatoric stress.

EFFECT OF STRESS STATE ON TEST RESULTS

The resilient modulus value at any specified confining pressure and deviatoric stress can be calculated from the regression parameters k_1, k_2, and k_3. The resilient modulus values are reported at a confining pressure of 5 psi (34.5 kPa) and a deviatoric stress of 15 psi (103.4 kPa), while the permanent deformation properties are computed at confining pressure of 7 psi (48.3 kPa) and a deviatoric stress of 28 psi (193 kPa) or ($\sigma_1/\sigma_3 = 5$). Here, the resilient modulus values are computed at a confining pressure of 7 psi (48.3 kPa) and a deviatoric stress of 28 psi (193 kPa) to compare the effect of higher stress levels on the resilient modulus values. The resilient modulus values computed at a confining pressure of 7 psi (48.3 kPa) and a deviatoric stress of 28 psi (193 kPa) are shown in Table 7.

TABLE 7 Resilient Modulus Values for a 6 in. by 12 in. at 7 psi Confining Pressure and 28 psi Deviatoric Stress

Specimen	k_1	k_2	k_3	M_r (ksi)
1	1699.49	0.71	0.04	60.25747272
2	2424.13	1.13	-0.99	73.66015037
3	2591.25	1.02	-0.98	69.41480562
4	2406.69	0.81	-0.55	65.95037353
5	2321.15	1.05	-0.83	70.96983963
6	2002.96	0.74	-0.45	53.78925249
7	2057.65	1.25	-1.26	60.76577268

The resilient modulus values obtained from Table 6 and Table 7 are plotted as shown in Figure 22. It indicates that as the stress level increases, the resilient modulus value increases. This result reaffirms the stress dependency of the resilient modulus values.

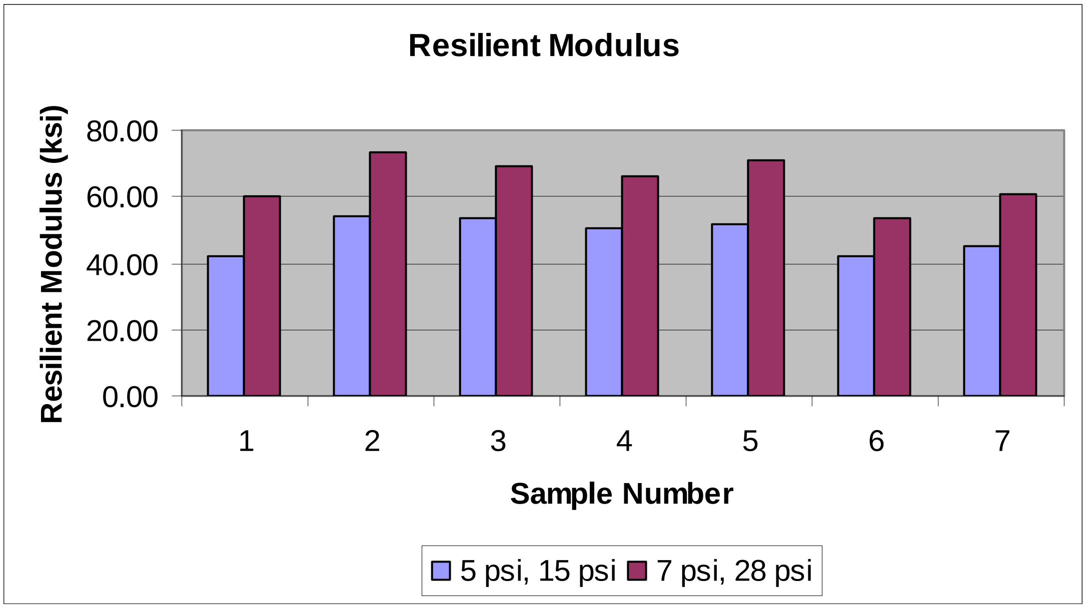

FIGURE 22 Resilient modulus values for different stress levels.

Also, from the resilient modulus test data, the induced ε_p is calculated for the different combinations of confining pressure and deviatoric stress. A graph is plotted between induced ε_p and confining pressure for the different stress ratios (1.5, 2.0, 3.0, 4.0, 6.0, and 8.0). The graph indicates that induced ε_p increases with increasing stress levels for the same stress ratios (σ_1/σ_3). For example, the stress ratio of 1.5 may have stress levels of 1.5 psi (10.3 kPa) deviatoric stress and 3 psi (20.6 kPa) confining pressure and 3 psi (20.6 kPa) deviatoric stress and 6 psi confining pressure. Hence, the induced permanent deformation values are computed at various stress levels for each of the stress ratios at which measurements were made during testing. There was no apparent increase in the induced ε_p for stress ratios 1.5, 2, 3, 4. For these stress ratios, there was no apparent change in induced ε_p for increasing stress levels. However, when the stress ratio increased, induced ε_p increased with increasing stress levels. The higher the stress ratio, higher was the rate of increase in induced ε_p for increasing stress levels. It should be noted here that the increase in induced ε_p was not due to increase in confining pressure but mainly due to the increase in deviatoric stresses. The same pattern was noticed for all of the specimens tested. Figure 23 shows a typical plot of induced ε_p versus confining pressure at different stress ratios.

Similarly, the resilient modulus values are plotted for different stress ratios as shown in Figure 24. It should be noted here that the graphs in Figure 23 and Figure 24 are plotted with confining pressure for clarity while the modulus values depicted are for a combination of confining pressure and deviatoric stresses. Figure 24 indicates that higher the stress level, higher is the modulus value. The same result was also demonstrated by Figure 22. The resilient modulus increases linearly with stress level for the same stress ratios. However, at higher stress ratios of 6 and 8, the modulus values decrease at higher stress levels. This may be due to the high stress levels causing the dilation of the material. Also, this may be due to the stress levels exceeding the failure line of the specimen at higher stress levels under repeated loading. Figure 24 shows that the difference in resilient modulus values for different stress ratios was not significant. It was the increase in deviatoric stress which resulted in increased resilient modulus values.

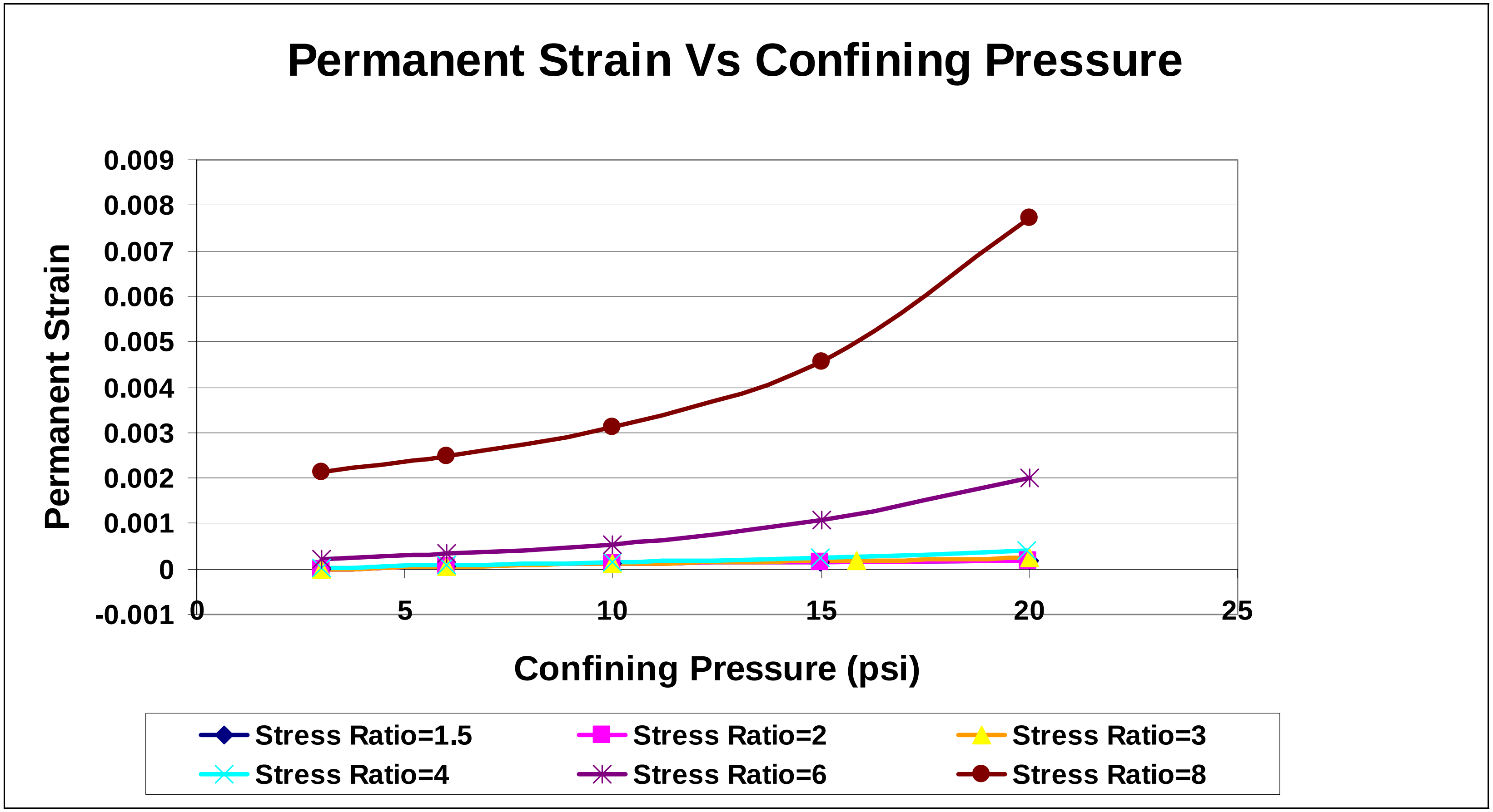

FIGURE 23 Plot of induced permanent strain during resilient modulus sequence at different stress ratios.

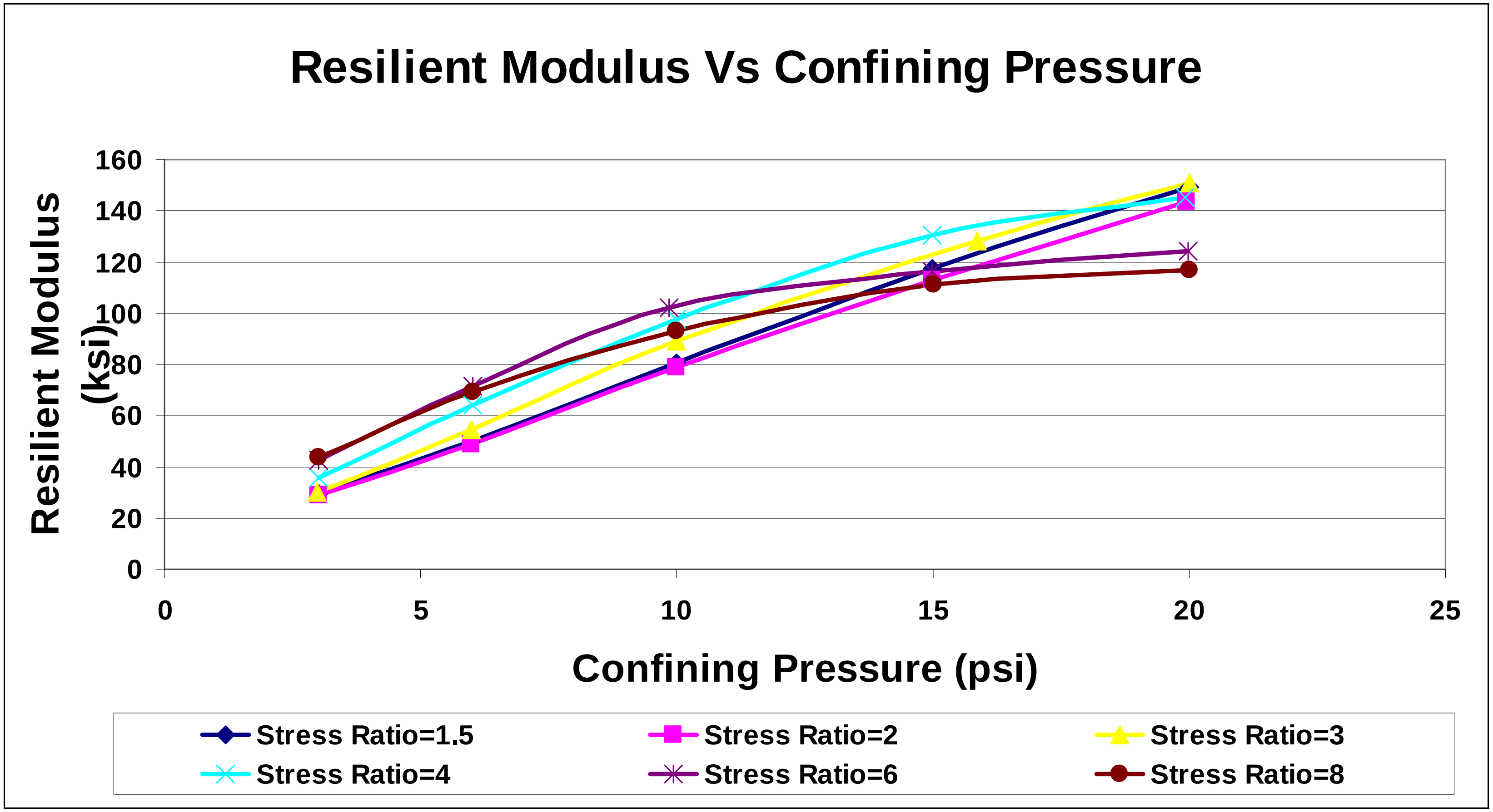

FIGURE 24 Resilient modulus for different stress ratios.

STATISTICAL ANALYSIS OF TEST RESULTS

Analysis of the test results obtained from the seven replicate specimens is presented in this section. As the present study was completed in a single laboratory, the precision statement is expressed in terms of the repeatability within the laboratory. Repeatability concerns the variability between independent test results obtained within a single laboratory, in the shortest practical period of time, by a single operator, with a specific set of test apparatus using test specimens (or test units) taken at random from a single quantity of homogeneous material obtained or prepared for the laboratory study (*36*). Seven replicate specimens were prepared and tested with the same equipment by the same operator. The repeatability is expressed in terms of the standard deviation of test results (*7*). These values for the test results are shown in Table 8. From these values the variability within the test results is estimated.

From standard practice ASTM E 691, the repeatability limit for the result is calculated as 2.8 times the standard deviation for a confidence level of 95% in test results. Thus, the repeatability limits for the results for resilient modulus and permanent deformation are as shown below in Table 9.

TABLE 8 Average and Standard Deviations of Test Results

Specimen	Resilient Modulus (ksi)	Permanent Strain at 5000 cycles
1	42.15	0.01873
2	54.21	0.01800
3	53.63	0.01830
4	50.73	0.01910
5	51.97	0.02200
6	41.94	0.01790
7	45.02	0.02300
Average	48.52	0.01958
Std Dev	5.34	0.00206
coeff of var	11.02	10.52349

TABLE 9 Repeatability Limits for Resilient Modulus and Permanent Strain

Test property	Average	Standard deviation (std dev)	95% repeatability limit= 2.8* std dev
Resilient Modulus	48.52 ksi	5.345	15 ksi
Permanent Strain	0.01958	0.00206	0.0057

Sample Size Calculations

Statistical methods were used to estimate the number of specimens required for a desired tolerance level in the test results. The number of observations to be included in the sample will be a compromise between the desired accuracy of the sample statistic as an estimate of the population parameter and the required time and cost to achieve this degree of accuracy. There are two considerations in determining the appropriate sample size for estimating an average using a confidence interval. First, the tolerable error establishes the desired width of the interval. The second consideration is the desired level of confidence (*35*). The tolerable error is the width of the confidence interval from the average value which is also expressed as percentage error (% error).

The sample size is determined by the following equation (*35*):

$$n = \frac{(z_{\alpha/2})^2 \sigma^2}{E^2} \tag{20}$$

Where

n= sample size,

$Z_{\alpha/2}$ = Z value used for a desired confidence level,

σ = standard deviation, and

E = half of the width of the confidence interval.

At a confidence level of 95%, the Z value is 1.96 from a statistical table of standard normal curve areas (*35*). The standard deviation values are obtained from Table 8. The sample size calculations are made for different tolerable errors from the mean of the resilient modulus values and the permanent deformation values which are shown in Tables 10 and Table 11, respectively. Graphs plotted between the sample size and the % errors of the results are presented in Figure 25 and Figure 26.

TABLE 10 Sample Size Calculation Based on Resilient Modulus Values

Error (M_r)	No of Samples
8%	7.28
9%	5.75
10%	4.66
11%	3.85
12%	3.24
13%	2.76
14%	2.38
15%	2.07
16%	1.82

TABLE 11 Sample Size Calculation Based on Permanent Strain at 5000th Load Cycle

Error (PD)	No of Samples
8%	6.65
9%	5.25
10%	4.25
11%	3.52
12%	2.95
13%	2.52
14%	2.17
15%	1.89
16%	1.66

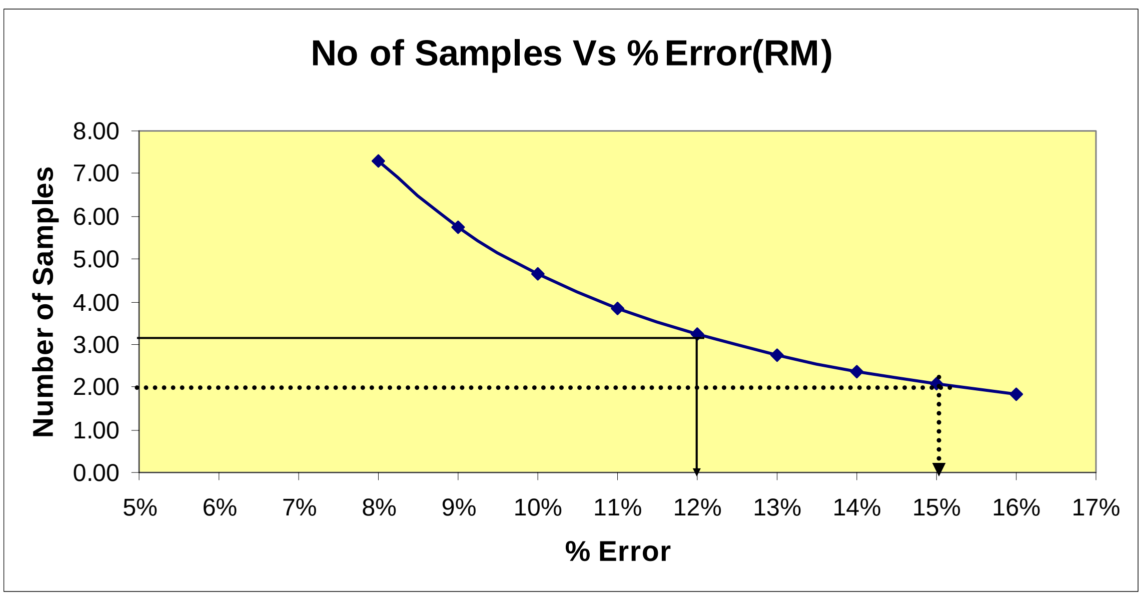

FIGURE 25 Plot of number of samples versus % error of resilient modulus value.

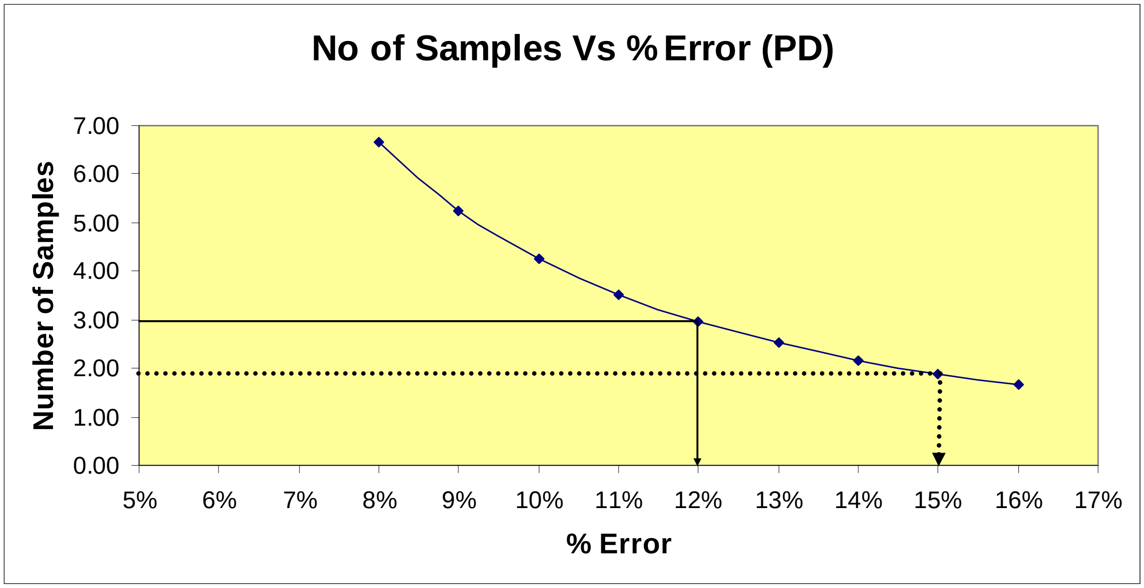

FIGURE 26 Plot of number of samples versus % error of permanent deformation values.

Table 10, Table 11, Figure 25 and Figure 26 indicate that for the determination of the resilient modulus and the permanent deformation properties using the proposed performance test requires a sample size of 3 for a tolerance level of 12%.

PRECISION

The precision of a test method is expressed as the standard deviation (*7*).The precision information given below is for average resilient modulus (ksi) for the comparison of two test results, each of which is the average of seven test results.

Figure 27 shows a statement of within laboratory precision expressed according to the ASTM E 177.

Average test value:

Resilient Modulus = 48.52 ksi

95% repeatability limit = 2.8* std dev = 2.8* 5.3 = 15 ksi

Permanent Deformation = 0.0198 in

95% repeatability limit = 2.8* std dev = 2.8* 0.00206 = 0.00576 in

FIGURE 27 Within laboratory precision statement.

INFLUENCE OF SPECIMEN SIZE ON TEST RESULTS

The number of specimens for a tolerance level of 12% is three from Table 10 and Table 11. Thus, three specimens were tested with a specimen size of 6 in. (152 mm) diameter and 8 in. (203 mm) height. The results of permanent deformation properties and resilient modulus values are shown in Table 12, Table 13 and Figure 28.

TABLE 12 Permanent Deformation Test Results for 6 in. by 8 in.

Specimen	ε_r	Mr	Rutting parameters	
			μ	α
1	0.000525	55.2381	0.5750	0.8560
2	0.000527	55.0285	1.2459	0.9294
3	0.000535	54.2056	0.8401	0.6337

TABLE 13 Resilient Modulus Test Results for 6 in. by 8 in.

Specimen	k_1	k_2	k_3	M_r (ksi)
1	2740.55	0.97	-0.67	61.59
2	1880.56	0.98	-0.38	47.86
3	2377.63	0.82	-0.42	53.38

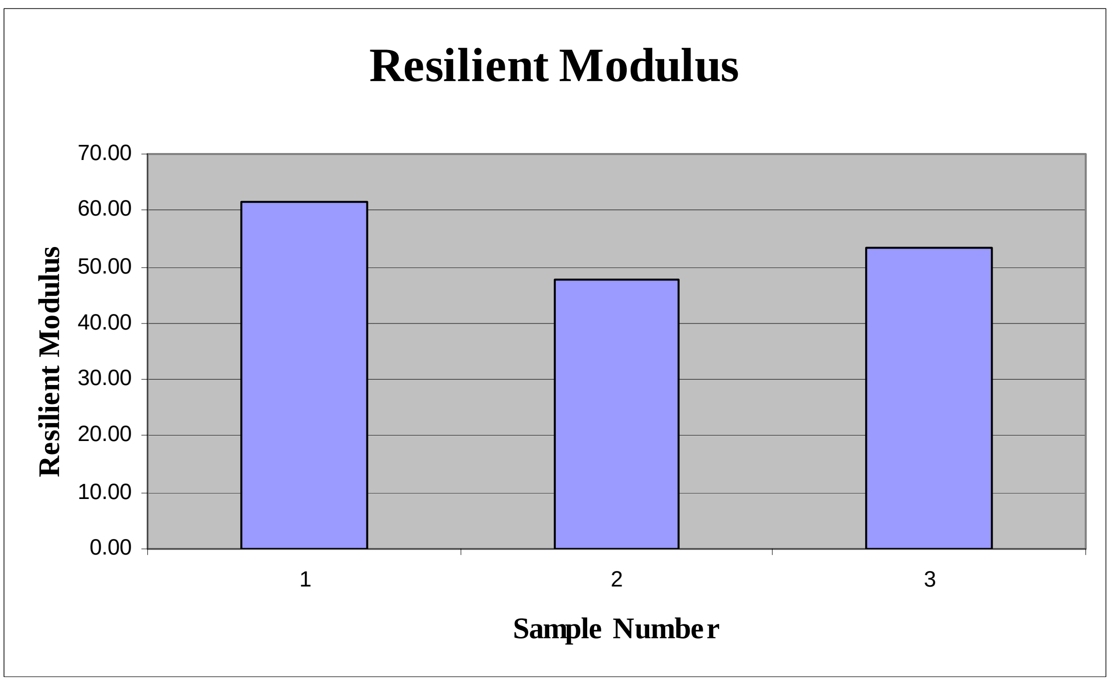

FIGURE 28 Resilient modulus values for 6 in. by 8 in.

The maximum aggregate size is 1-2 in. (25 – 50 mm) requiring a large specimen size of at least 6 in. (152 mm) to maintain a 1: 6 ratio of maximum aggregate size to diameter of specimen as recommended by NCHRP 1-37 A. Further, a diameter to height ratio of 1: 2 is recommended by literature to reduce the end effects on the deformation measurements made on the full length of the sample (*44*).

A statistical analysis of the test results was conducted to determine the influence of the specimen size. The student's t test was used to compare the test results and to estimate the difference between the test results with different specimen sizes. The following assumptions are made for this test:

- The data is normally distributed
- The samples are independent. That is the testing of one specimen size does not affect the testing of the other specimen size.
- The two populations, specimen height 12 in. (304 mm) and specimen height 8 in. (203 mm) have the same population standard deviation.

Since it was assumed that the standard deviations of the populations are the same, the sample standard deviations are pooled using the formula:

$$S_p = \sqrt{\frac{(n_1 - 1)s_1^{\ 2} + (n_2 - 1)s_2^{\ 2}}{n_1 + n_2 - 2}} \tag{21}$$

where

S_p = pooled standard deviation,

n_1 = sample size of population one,

n_2 = sample size of population two,

s_1 = standard deviation for population one, and

s_2 = standard deviation for population two.

The null hypothesis for the comparison between the two specimen sizes is that there is no difference between the test results when the specimen height is reduced to 8 in. (152 mm) from 12 in. (304 mm)

The test statistic for the above hypothesis testing is:

$$t = \frac{\left(\bar{y}_1 - \bar{y}_2\right) - 0}{S_p \sqrt{\frac{1}{n_1} + \frac{1}{n_2}}} \tag{22}$$

where

$\bar{y}_1$ = sample mean of population one (specimen height 12 in.),

$\bar{y}_2$ = sample mean of population two (specimen height 8 in.),

n_1 = sample size of population one (specimen height 12 in.), and

n_2 = sample size of population two (specimen height 8 in.).

For a level α, Type I error rate and with degrees of freedom df, the hypothesis is rejected if $|t| \geq t_{\alpha/2}$.

where

$t_{\alpha/2}$ is determined from a table of critical values for the student's t distribution

(*35*).

Sample sizes and standard deviations for the two populations for the calculation of the pooled standard deviation based on resilient modulus and permanent deformation values are shown in Table 14 and Table 15. Using the average and standard deviation values of the samples from Table 8, the t statistic is calculated.

TABLE 14 Calculation of Pooled Standard Deviation for Resilient Modulus Values

population one, height 12 in.		population two, height 8 in.	
n_1	7	n_2	3
y_1	48.52	y_2	54.28
s_1	5.345	s_2	6.9
s_1^2	28.56903	s_2^2	47.61
S_p	5.773150678		
df	8		
t`	1.445836679		

TABLE 15 Calculation of Pooled Standard Deviation for Permanent Deformation Values at 5000 Load Cycles

population one, height 12 in.		population two, height 8 in.	
n_1	7	n_2	3
y_1	0.000592	y_2	0.0048
s_1	0.000066	s_2	8.27E-05
s_1^2	4.356E-09	s_2^2	6.84E-09
S_P	0.008377052		
df	8		
t`	0.727937626		

For an α value of 5 % and df = 8, the $t_{\alpha/2}$ determined from table of critical values for the student's t distribution is 2.3. Since $|t| \leq t_{\alpha/2}$, for both the permanent deformation and resilient modulus values the null hypothesis cannot be rejected. Thus, it can be concluded that there is no difference in the test results when the specimen size is reduced to 8 in. (203 mm) from 12 in. (304 mm) for the resilient modulus value.

SUMMARY

This chapter described the determination of the resilient modulus and permanent deformation test results. The 6 in. (152 mm) diameter with 12 in. (304 mm) height specimen was tested to evaluate the test procedure. Further, the influence of stress ratios on these properties is discussed. Statistical procedures were followed to estimate the number of test specimens necessary for a desired level of tolerance. After the estimation of the samples size, it was found that for a tolerance level of 12% three replicate specimens are required to be tested. Based on this three specimens of 6 in. (152 mm) diameter and 8 in. (304 mm) height specimens were prepared for conducting the performance test. The Student's t test was conducted to investigate the influence of the specimen size on the test results. It was found that there was no statistically significant difference for a confidence level of 95% between the test results for both resilient modulus and permanent deformation properties.

CHAPTER V

APPLICATIONS OF PERFORMANCE TEST

INTRODUCTION

This chapter consists of a documentation of studies wherein the performance test was applied successfully for the evaluation of unbound granular base materials. This test was applied to evaluate the behavior of granular materials based on the resilient modulus and permanent deformation properties. Further, this test was also used for the investigation of influence of fines content and the moisture content on the resilient modulus and permanent deformation properties. The results of these studies are presented in this chapter. Further, case studies of materials tested using the performance test are documented.

INFLUENCE OF FINES CONTENT

The performance test was applied to investigate the influence of fines content on the permanent deformation and resilient modulus values of Texas crushed stone material. The details of this study are in research report 4358-2, "Impact of Aggregate Gradation on Base Material Performance" (*45*). The gradation of the material was within the specifications for granular base materials.

Test specimens were prepared to three different fines content, 5 percent, 10 percent and 17 percent. The performance test was conducted at varying fines content to determine their influence on the permanent deformation and resilient modulus properties of the material. The results of the performance test are provided in Table 16.

TABLE 16 Performance Test Results for Varying Fines Content

Fines Content	Resilient Modulus (ksi)	Resilient strain at 500th load cycle	alpha (a)	gnu (m)
5%	76.63	0.0006195	0.9005	0.5125
10%	64.055	0.0006845	0.838	1.435
17%	56.862	0.0004428	0.8388	0.9792

Figure 29 presents the resilient modulus values obtained at the varying fines content.

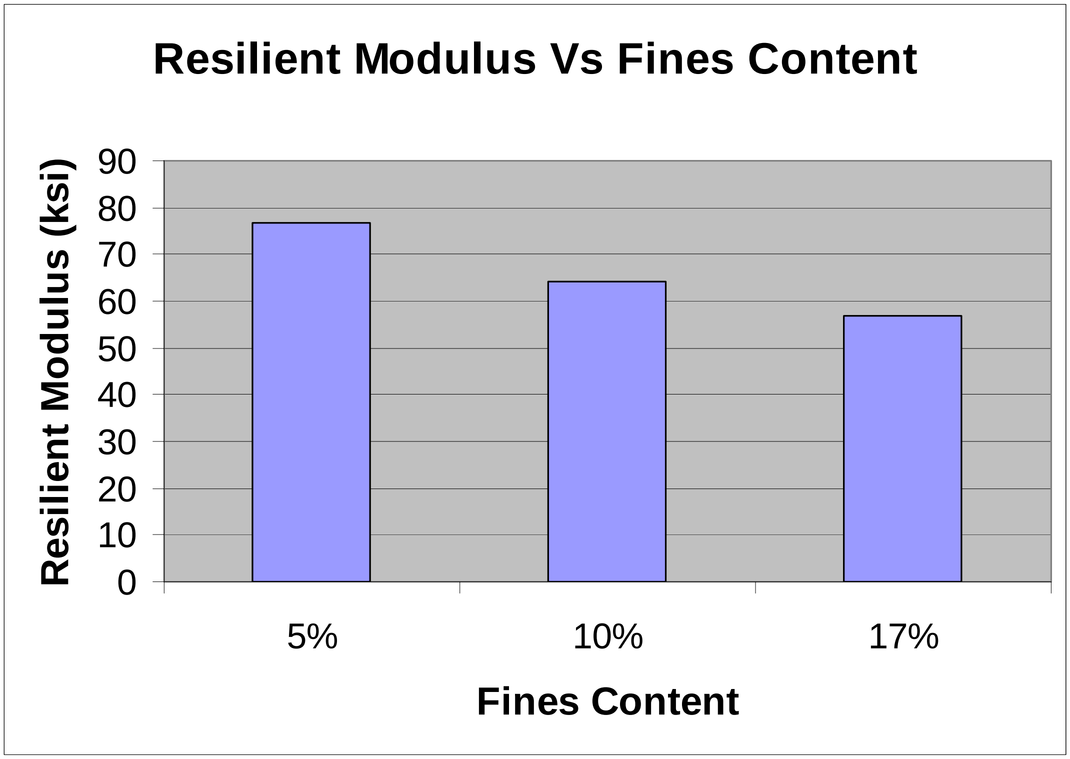

FIGURE 29 Resilient modulus values at varying fines content.

Figure 30 shows the permanent deformation at 5000 load cycles. It indicates that the sample with 5 percent fines content had better tendency to recover the deformation it underwent under the load. While, the samples with 10 percent and 17 percent fines had more permanent deformation. Also, the alpha value decreased as the fines content increased. As the alpha value is reduced, the resistance to permanent deformation also reduces. It should be noted here that though the difference in alpha values is less for the three fines content, this value is an exponent when used in the equation. Thus, small variances in alpha value result in larger variances in the permanent deformation values.

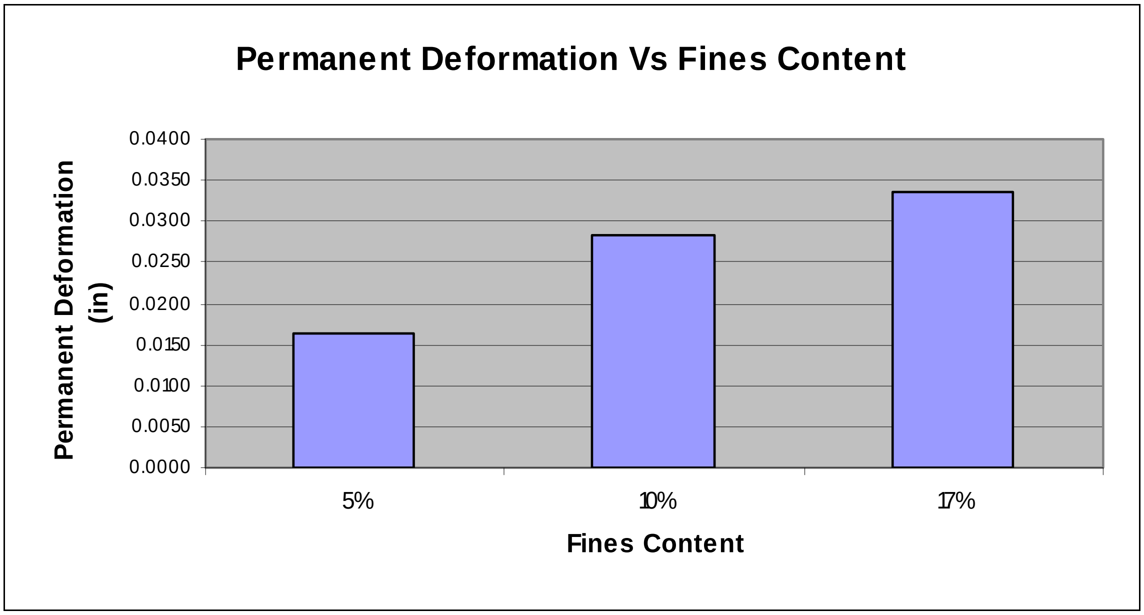

FIGURE 30 Plot of permanent deformation at varying fines content.

Thus, from this study they concluded that as the fines content decreases, the resilient modulus value and the resistance to permanent deformation increases. The higher the fines content, higher is the permanent deformation which causes rutting, ultimately affecting the performance of the pavement.

INFLUENCE OF MOISTURE CONTENT ON TEST RESULTS

A study was conducted by the performance test on the influence of the moisture content on the granular material behavior. A flex base material from Interstate 35 was tested at optimum moisture content, 1% below optimum moisture content and 1% above optimum moisture content. The optimum moisture content and the maximum dry density values were 7.7 % and 134.8 lb/ft^3 (2159 kg/m^3). Test specimens were prepared at optimum moisture content, 1% below optimum moisture content and 1% above the optimum moisture content. The results of the performance test are provided in Figure 31 and Figure 32.

The results indicate that as the moisture content decreased, the resilient modulus value increased. Further, the permanent deformation increased as the moisture content increased. For the material compacted at 1 % above the optimum moisture content, the resilient modulus test sequence could not be completed as the material was too wet and excessive deformation occurred which could not be measured.

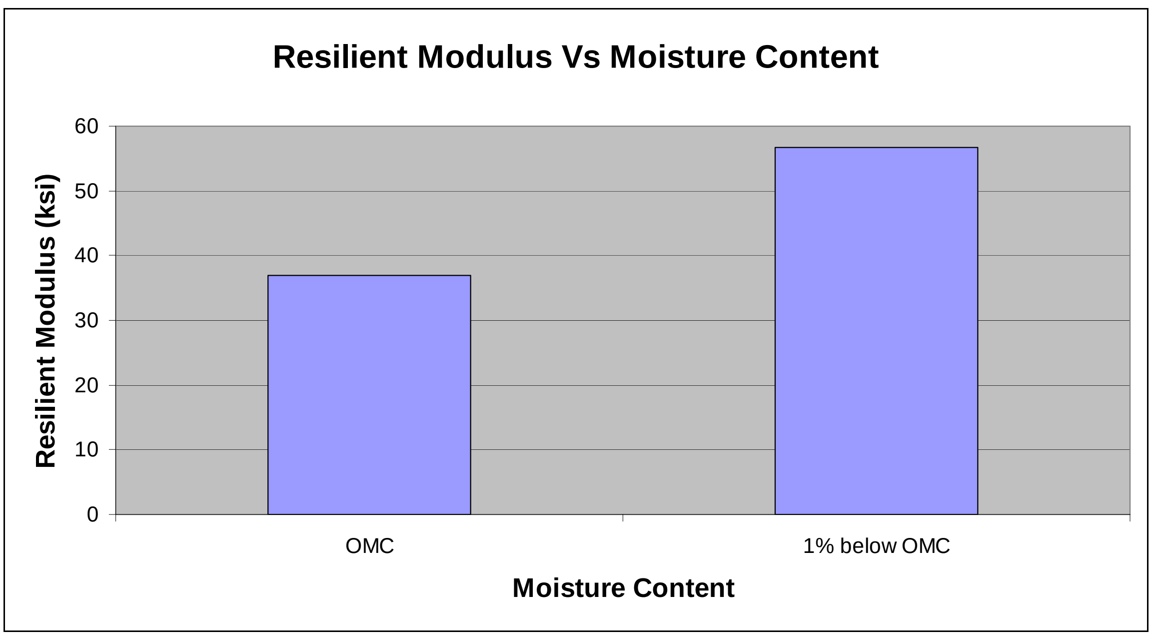

FIGURE 31 Comparison of resilient modulus with varying moisture content

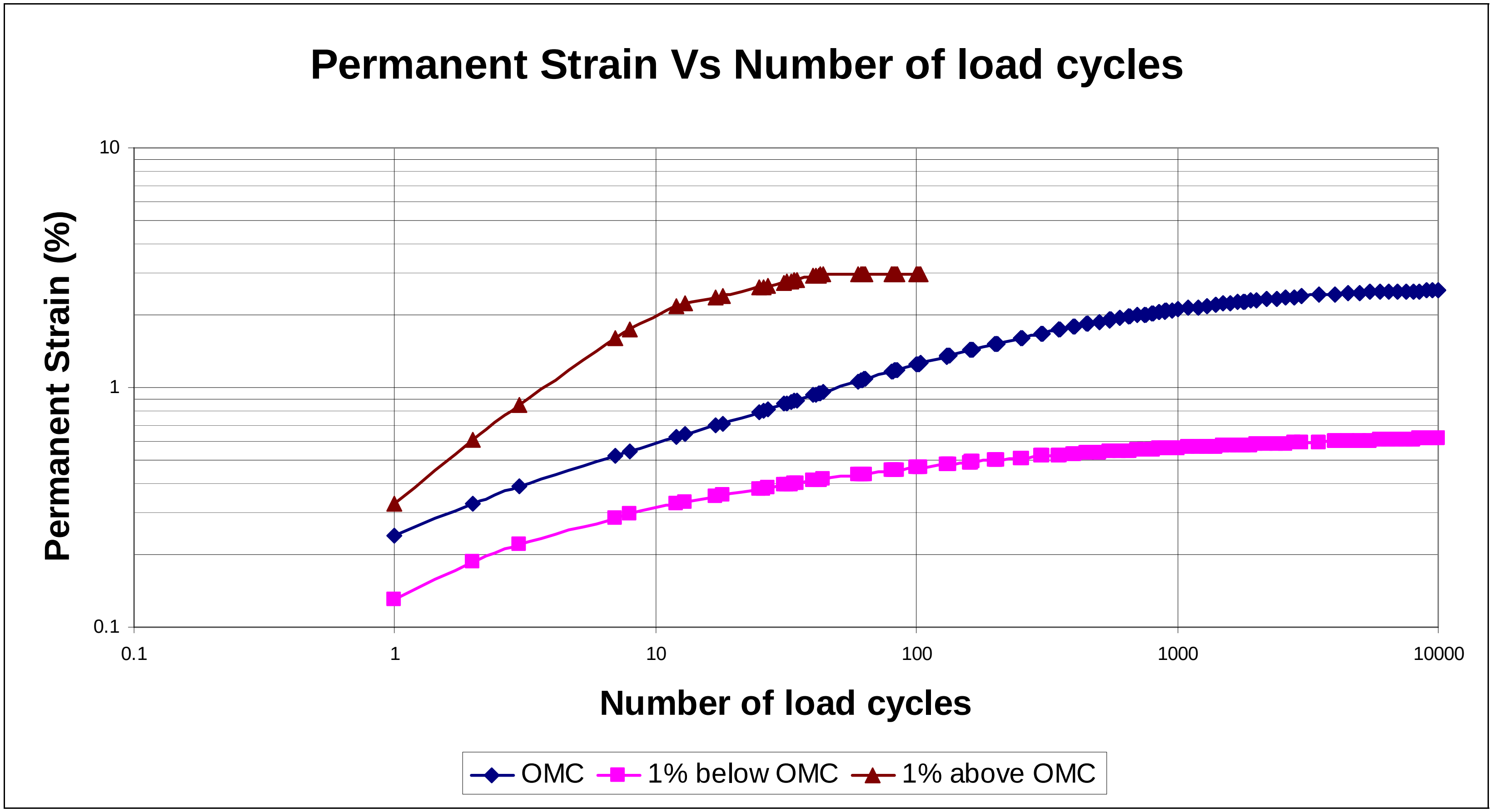

FIGURE 32 Comparison of permanent strain with varying moisture content.

Another study was conducted to evaluate New York base material by the performance test at two different moisture contents, at optimum moisture content and at moisture content reached at the end of the tube suction test (TST) (*46*). TST is used to assessing the moisture susceptibility of granular base materials (*46*). The gradation and the moisture content results are provided in Appendix C. The specimens were compacted at optimum moisture content. When the performance test sequence was applied on this material, it deformed excessively. Hence, only a resilient modulus stress sequence was applied on the test specimen. The results of the resilient modulus values for the two specimens tested are shown in Table 17.

TABLE 17 Resilient Modulus Test Results

Specimen	Resilient Modulus (ksi)	K1	K2	K3
New York Base Type 1- Specimen #2 (4.3% wc)	30.75	1381.41	.65	-.12
New York Base Type 1- Specimen #1 (5.9 % wc)	20.62	1063.62	.55	-.29

From Table 17, the modulus value at optimum moisture content was lower than the moisture content at the end of TST. TST shows the maximum moisture that can be imbibed under favorable conditions in the field (*46*). Figure 33 shows that the material had a better resilient modulus value at the moisture content at the end of TST than at optimum moisture content. This further enunciates the adverse effect of increasing moisture content on resilient modulus.

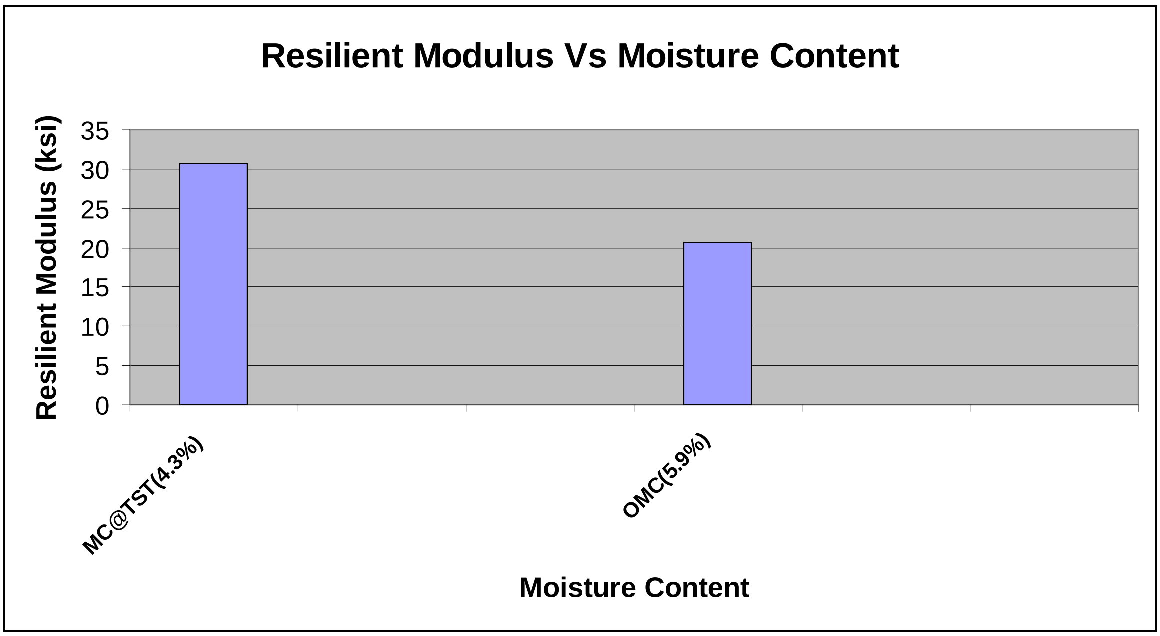

FIGURE 33 Resilient modulus at OMC and at end of TST.

Figure 34 and Figure 35 show the specimens at the end of testing. These figures indicate that the specimens deformed excessively during testing and the LVDTs went out of range. That is, the LVDTs were not calibrated to such deformation

FIGURE 34   Type 1 specimen #1 at 5.9 % water content at the end of testing.

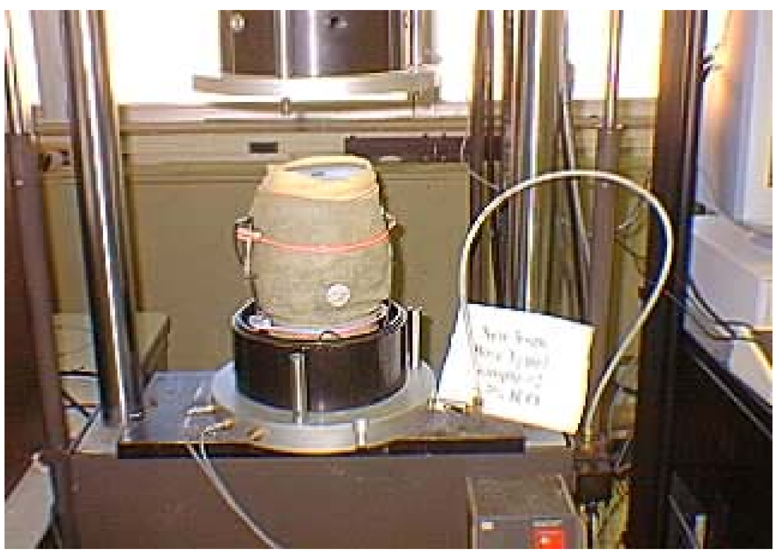

FIGURE 35   Type 1 specimen # 2 at 4.3 % water content at the end of testing.

The resilient modulus value obtained at the two moisture contents as discussed earlier indicate that the value at optimum moisture content results in under-predicting the modulus. Hence, conducting the test at the moisture content at the end of TST may give a better representation of the resilient modulus value.

This study draws our attention to a major concern of sample preparation for resilient modulus testing. The moisture content at which the test is conducted needs to be reviewed. The inclusion of the testing at the moisture content at the end of TST may help in better predicting the resilient modulus value.

EVALUATION OF GRANULAR BASE MATERIALS

Two samples of base course aggregates from different locations; State Highway 6 (SH 6) at Waco, US 59 at Lufkin were tested to evaluate their engineering properties. Performance test is conducted on these materials to determine the permanent deformation properties and the resilient modulus based on stress dependency. The gradation and moisture content test results for both these samples are provided in Appendix D. Two test specimens were prepared for each sample of material at optimum moisture content for conducting the performance test. The results of the resilient modulus and permanent deformation test are presented in Table 18. The rutting parameters μ and α are also shown.

TABLE 18 Results of Resilient Modulus and Permanent Deformation Test

Specimen	Water content %	Resilient Modulus (ksi)	Permanent Deformation	ε_r	Rutting parameters μ	Rutting parameters α
Highway6	7.9	68.44	0.0337	0.000815	1.0133	0.9667
US 59	7.8	50.97	0.0061	0.000455	0.4234	0.9493

The Aggregate US 59 had more permanent deformation than the sample Highway 6. Based on the permanent deformation values it can be concluded that base material US 59 was the better of the two materials. Also, the resilient modulus value for US 59 and SH 6 were 50.97 ksi and 68.44 ksi, typical for a granular base material. Here, this test provides for comparison of the two base materials. It should be noted that while US 59 material was better in terms of resistance to permanent deformation while SH 6 was better in terms of the resilient modulus values. However, both these materials should prove to be good base materials based on their permanent deformation and resilient modulus values, to be used in Texas where problems due to frost susceptibility are considerably less.

SUMMARY

This chapter provides a description of the application of the performance test for the evaluation of granular base materials. Case studies were documented which applied the performance test to investigate the influence of factors like moisture content and aggregate gradation on the behavior of granular materials. It was found that the fines content and the moisture content had an adverse affect on the resilient modulus and resistance to permanent deformation properties. Further, a study conducted to evaluate the effect of the moisture content at which the specimen was compacted on the test results. A comparative study on two base materials by using the performance test was also documented.

CHAPTER VI

SUMMARY, CONCLUSIONS AND FUTURE RESEARCH

SUMMARY

This thesis presents an integrated test procedure which determines both the resilient modulus and permanent deformation properties of unbound granular materials. Testing was conducted on a specimen of 6 in. (152 mm) diameter and 12 in. (304 mm) height. The sample size was estimated for determining the permanent deformation and the resilient modulus properties for different reliability levels of the test results. The following parameters were obtained from the test:

- The permanent deformation parameters α and μ of the VESYS model
- The resilient modulus value and the parameters k1,k2 and k3 for use in level 1 analysis of the 2002 design guide

The results obtained were documented, and an analysis was made of the influence of the stress levels on the test results. Granular material is stress sensitive, and it was confirmed that as the stress level increased both the permanent deformation and the resilient modulus values increased.

A statistical analysis of the test procedure was carried out to estimate the sample size required for a desired tolerance level. A of 6 in. (152 mm) diameter and 12 in. (304 mm) height specimen was used for this testing. The variability of the test results was estimated, and the repeatability limits were specified for the within laboratory conditions. Based on this analysis, a sample of 3 specimens was chosen with a specimen size of 6 in. (152 mm) diameter and 8 in. (203 mm) height. A student's t -test was conducted on the test results for the two specimen sizes to estimate the influence of specimen size on the test results. It was found that there was no difference in the test results for the two specimen sizes. Further, case studies of application of performance test for the evaluation of granular material behavior was documented. Further, studies wherein the performance test was used to investigate the factors influencing the resilient modulus and permanent deformation properties of granular materials are described.

CONCLUSIONS

The following are the conclusions of the work that is presented in this thesis:

- It presented an integrated test procedure for the determination of both resilient modulus and permanent deformation properties of unbound granular materials. Both properties were determined by testing one specimen.
- The proposed performance test was repeatable, and the variability of individual results for different reliability levels was estimated.
- The stress sequence for the resilient modulus testing was such that the material is subjected to stresses above the line of failure.
- The stress levels may be high for the permanent deformation testing and the sequence may be carried out at a lower stress ratio which would prevent the premature failure of specimens.

- The permanent deformation and resilient modulus test results were stress sensitive. The variation of these results depended on the stress level. A higher stress level resulted in the estimation of higher values for resilient modulus and permanent deformation properties.
- It was proved statistically that the determination of both the resilient modulus and the permanent deformation properties required a sample size of three for a tolerance level of 12% for the testing of granular materials.
- A within laboratory precision statement was given for the within laboratory conditions at which the study was conducted.
- It was found that there was no influence on the test results when the specimen height was reduced from 12 in. (305 mm) to 8in. (152 mm) at a confidence level of 95%. Thus, a 6 in. (152 mm) diameter by 8 in. (203 mm) height specimen can be used for testing using the proposed performance test procedure.

FUTURE RESEARCH

The following recommendations are made for future research and continuation of the present study:

- Investigate the influence of method of compaction on test procedure
- Study the influence of the permanent deformation testing of specimens prior to resilient modulus testing, if any

- Evaluation of the models for the permanent deformation properties for obtaining statistically accurate results
- Improvement of the specimen preparation procedure by introducing gyratory preparation of specimens

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APPENDIX A

RESILIENT MODULUS RESULTS FOR

SPICEWOOD SPRINGS

SPECIMEN # 1

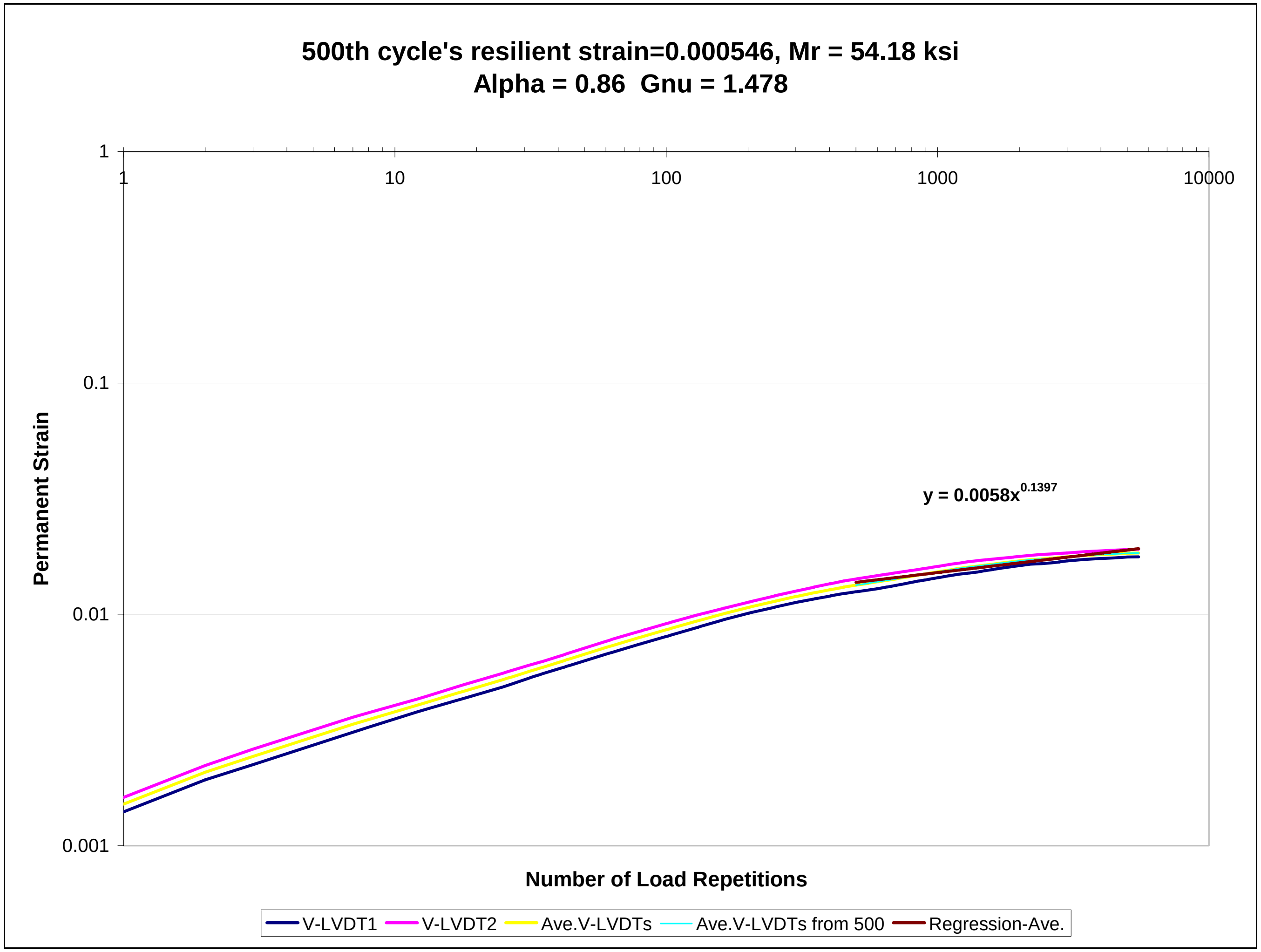

Figure A- 1 Permanent deformation result for specimen #1

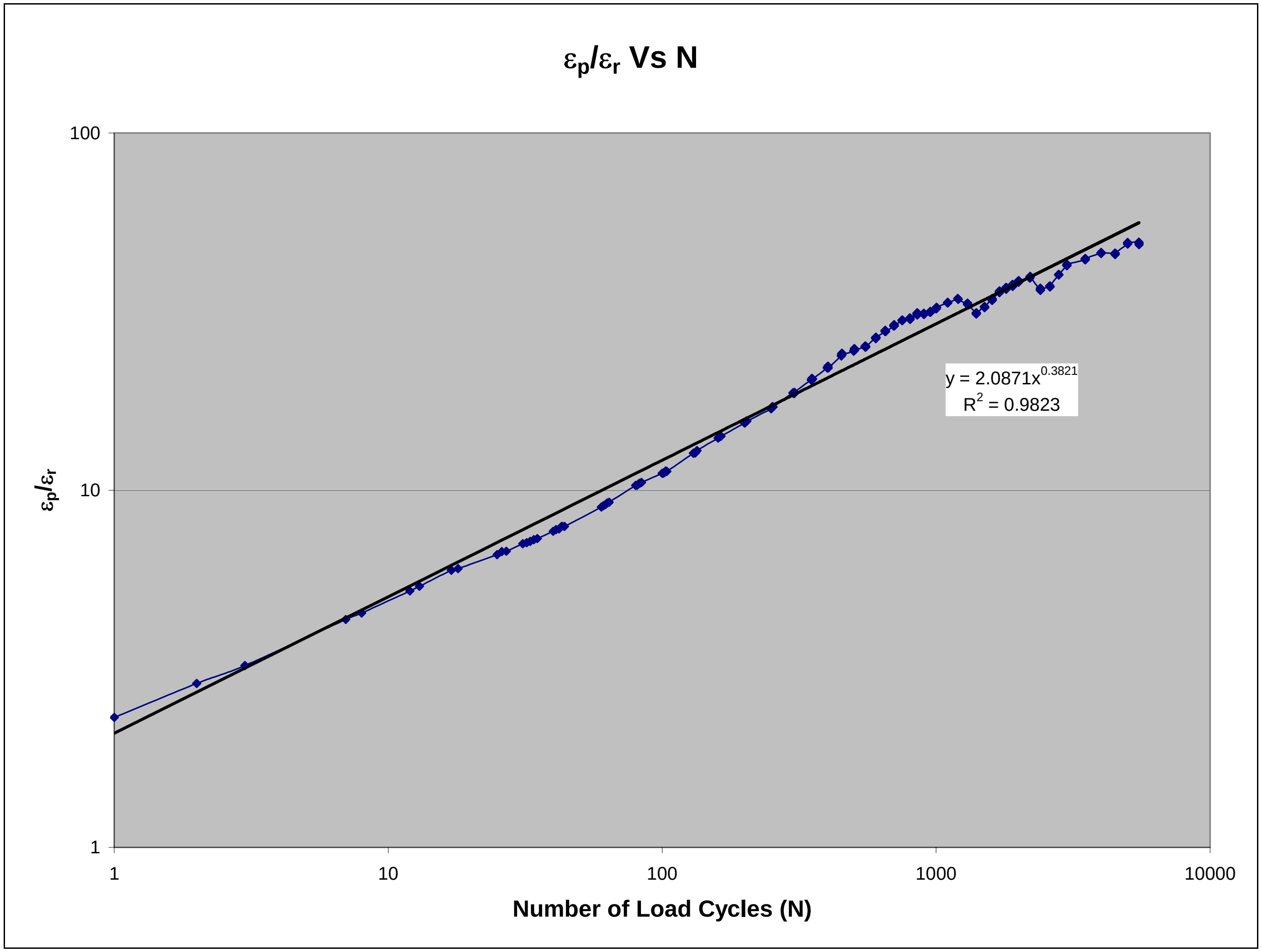

Figure A- 2 **$\varepsilon_p/\varepsilon_r$ Vs Number of load cycles for specimen #1**

SPECIMEN # 2

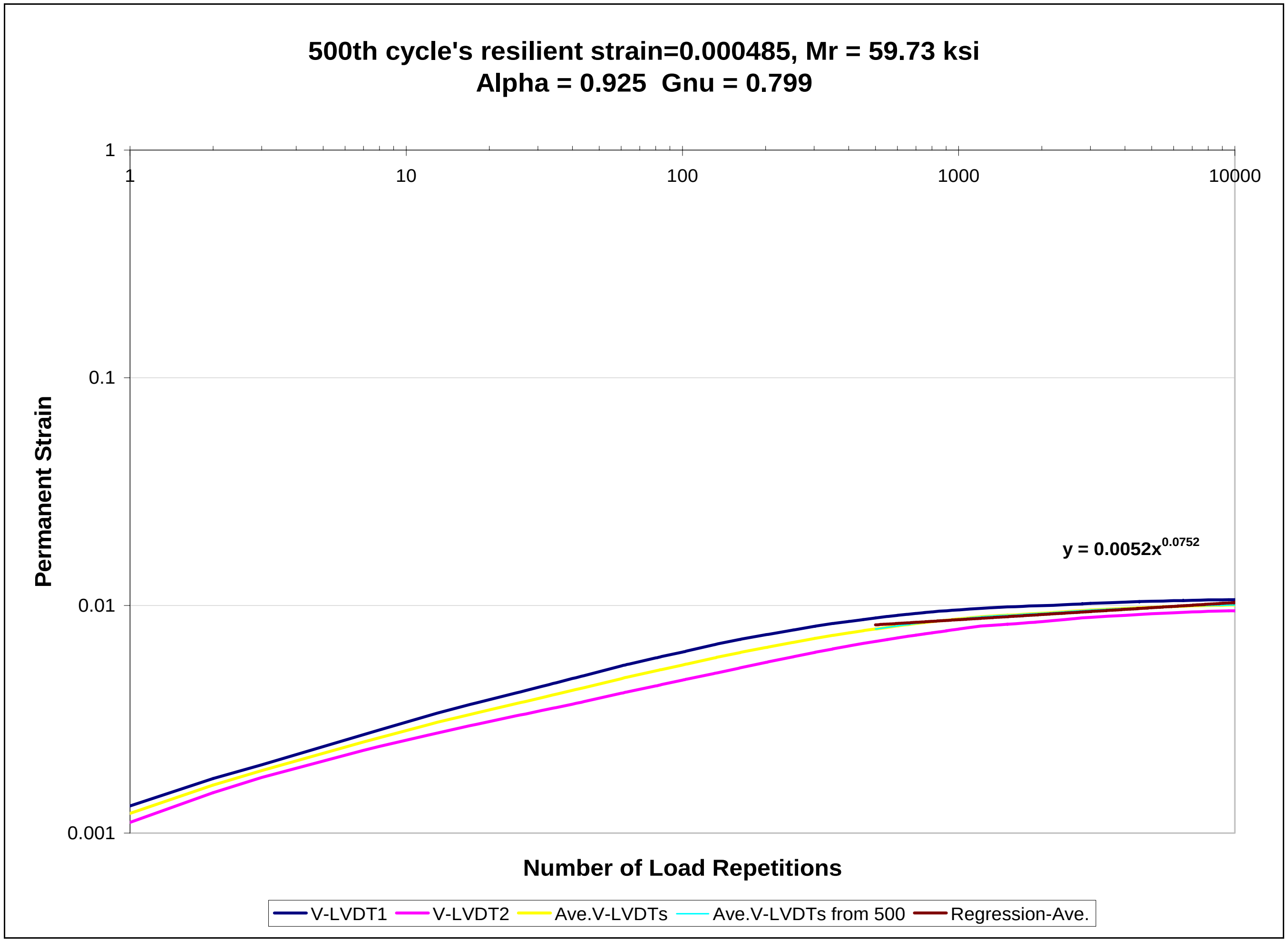

Figure A- 3 Permanent deformation result for specimen #2

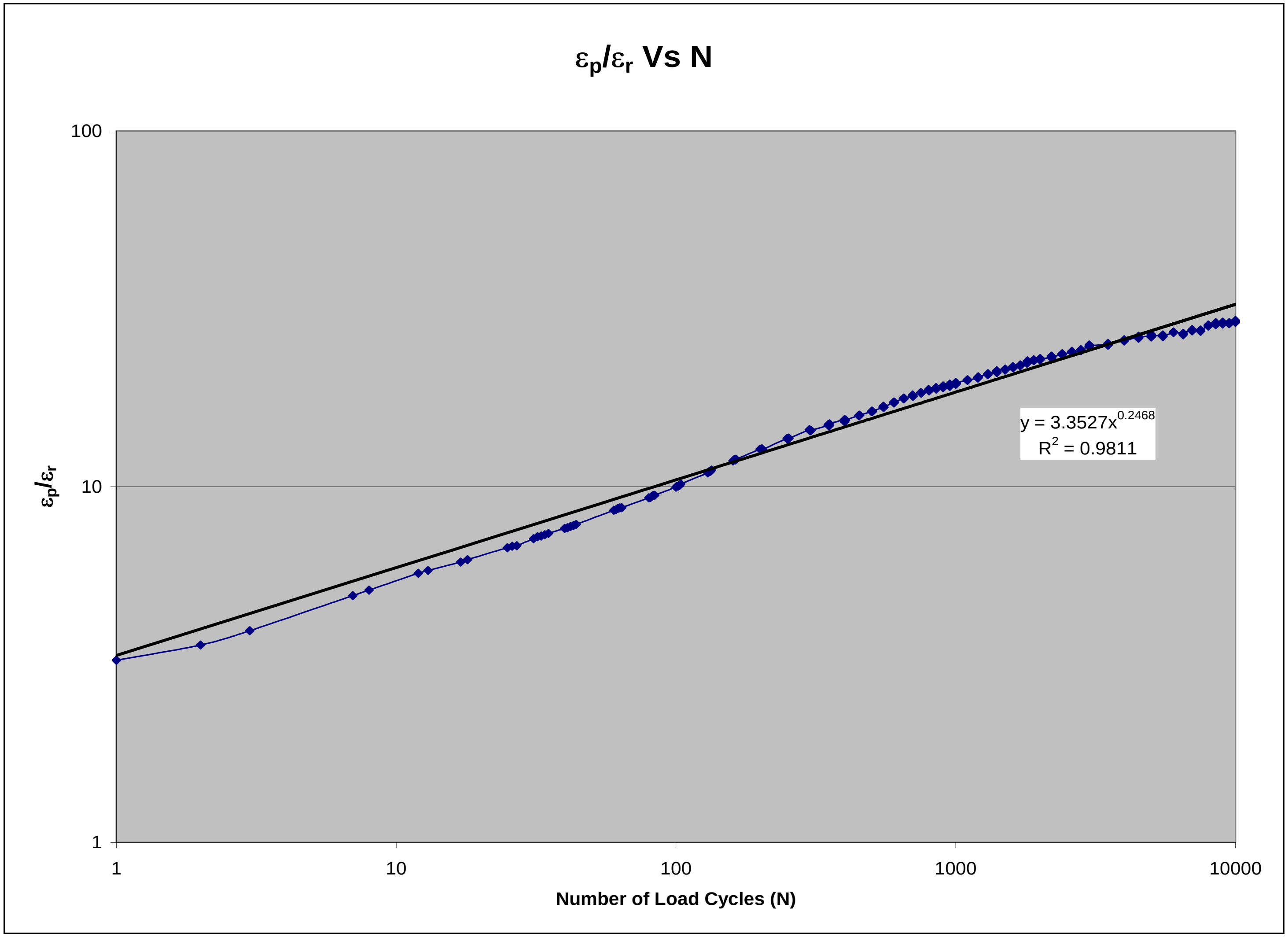

Figure A- 4 $\varepsilon_p/\varepsilon_r$ Vs Number of load cycles for specimen #2

SPECIMEN # 3

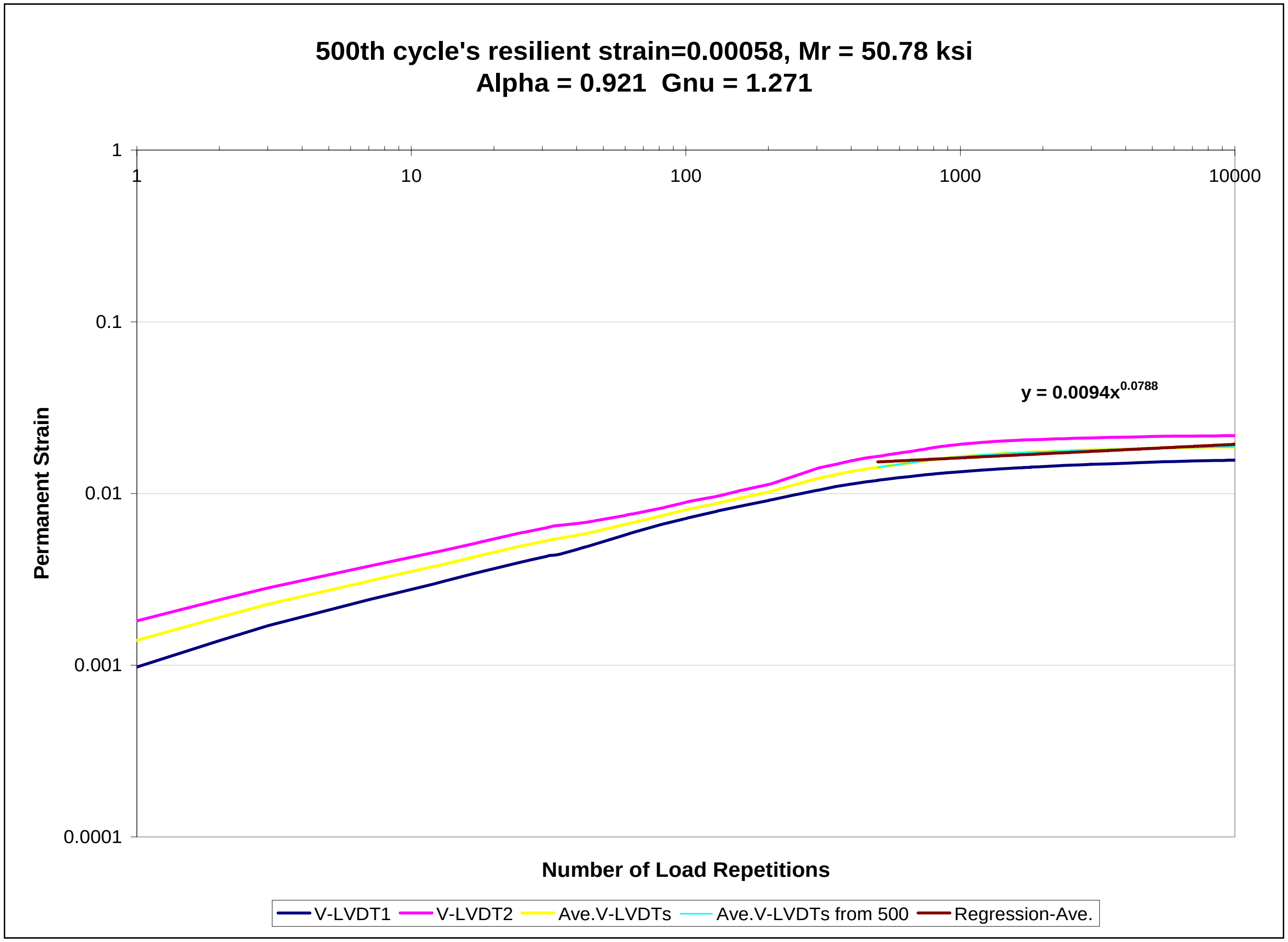

Figure A- 5 Permanent deformation result for specimen #3

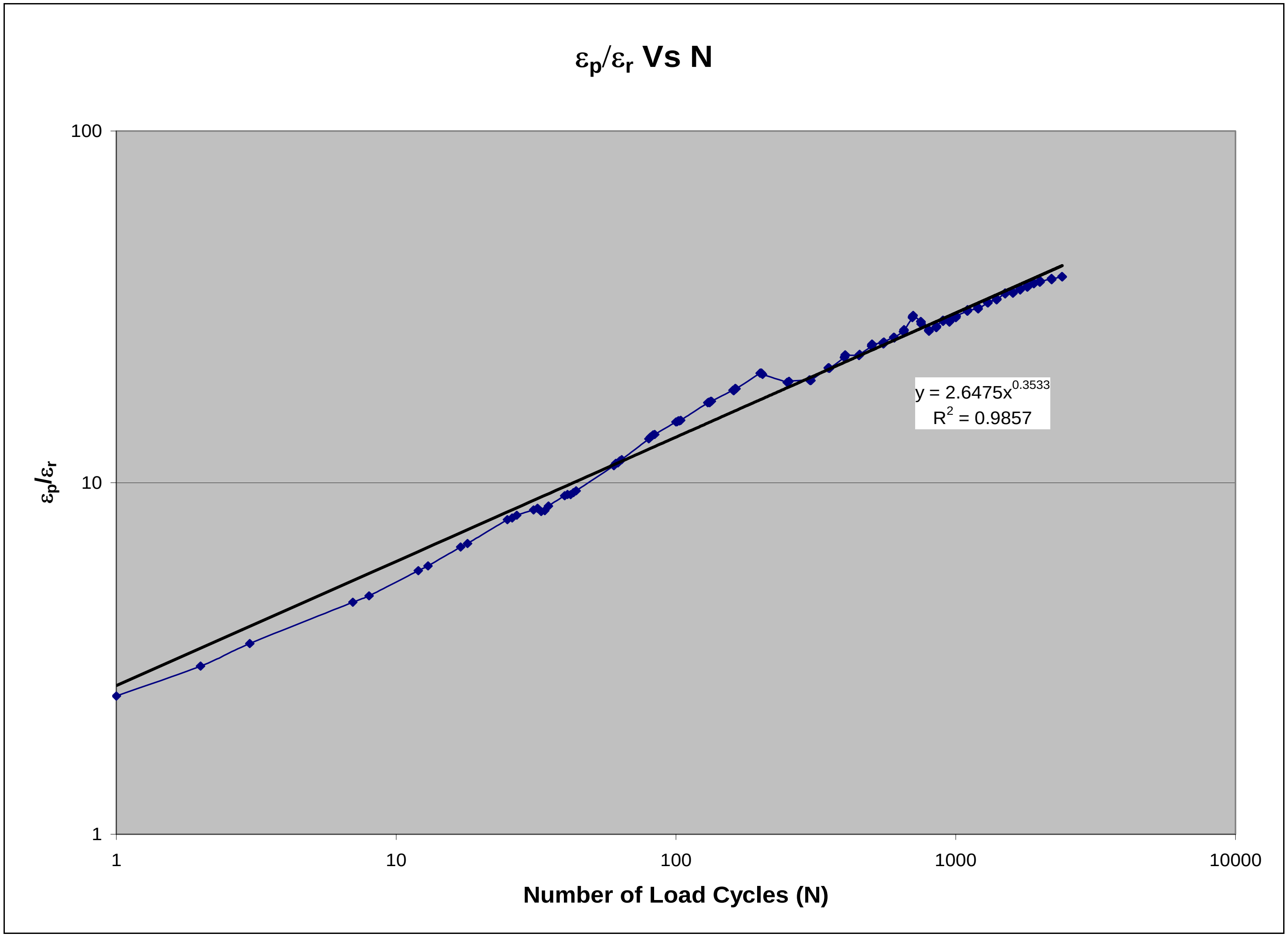

Figure A- 6 $\varepsilon_p/\varepsilon_r$ Vs Number of load cycles for specimen #3

SPECIMEN # 4

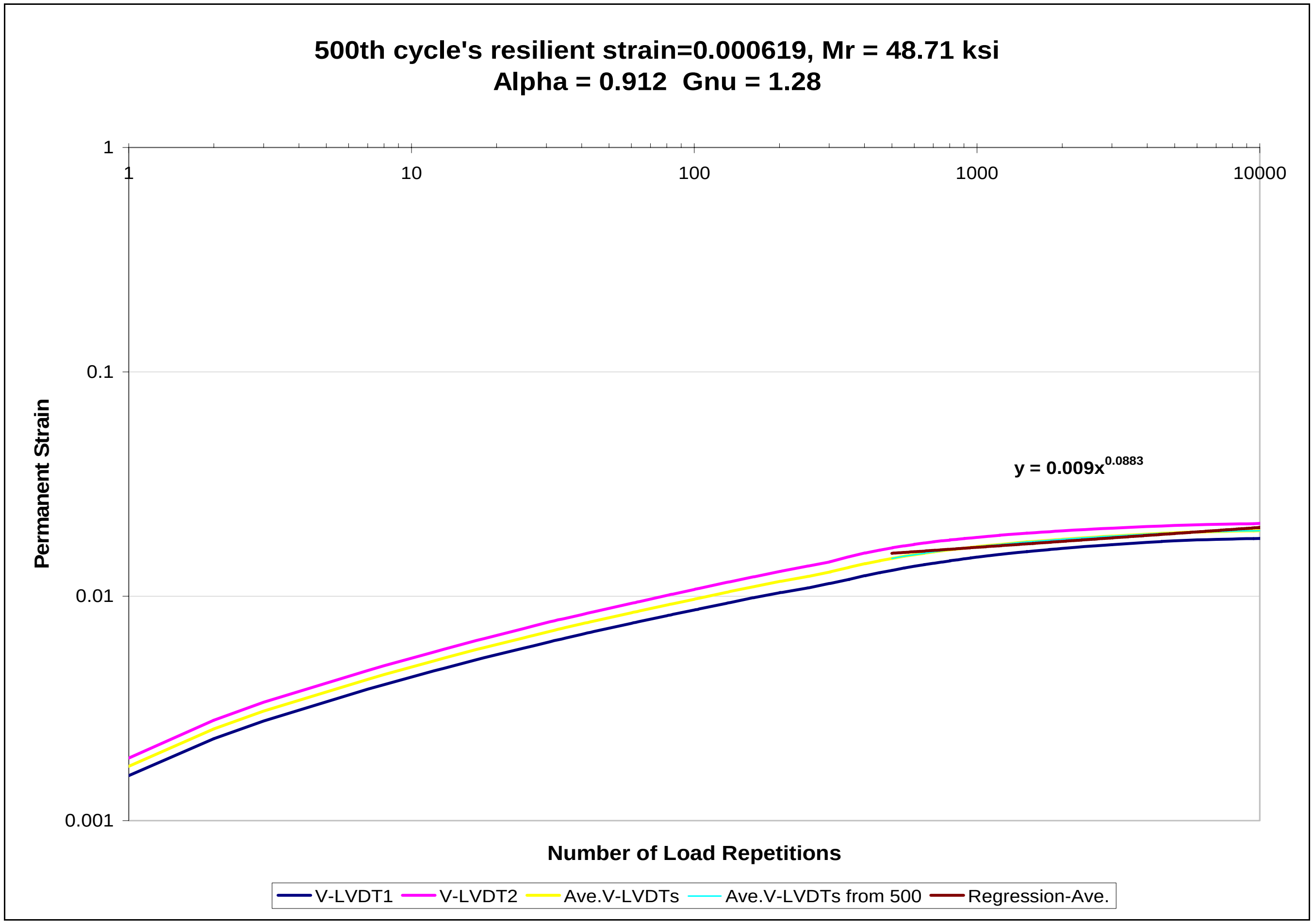

Figure A- 7 Permanent deformation result for specimen #4

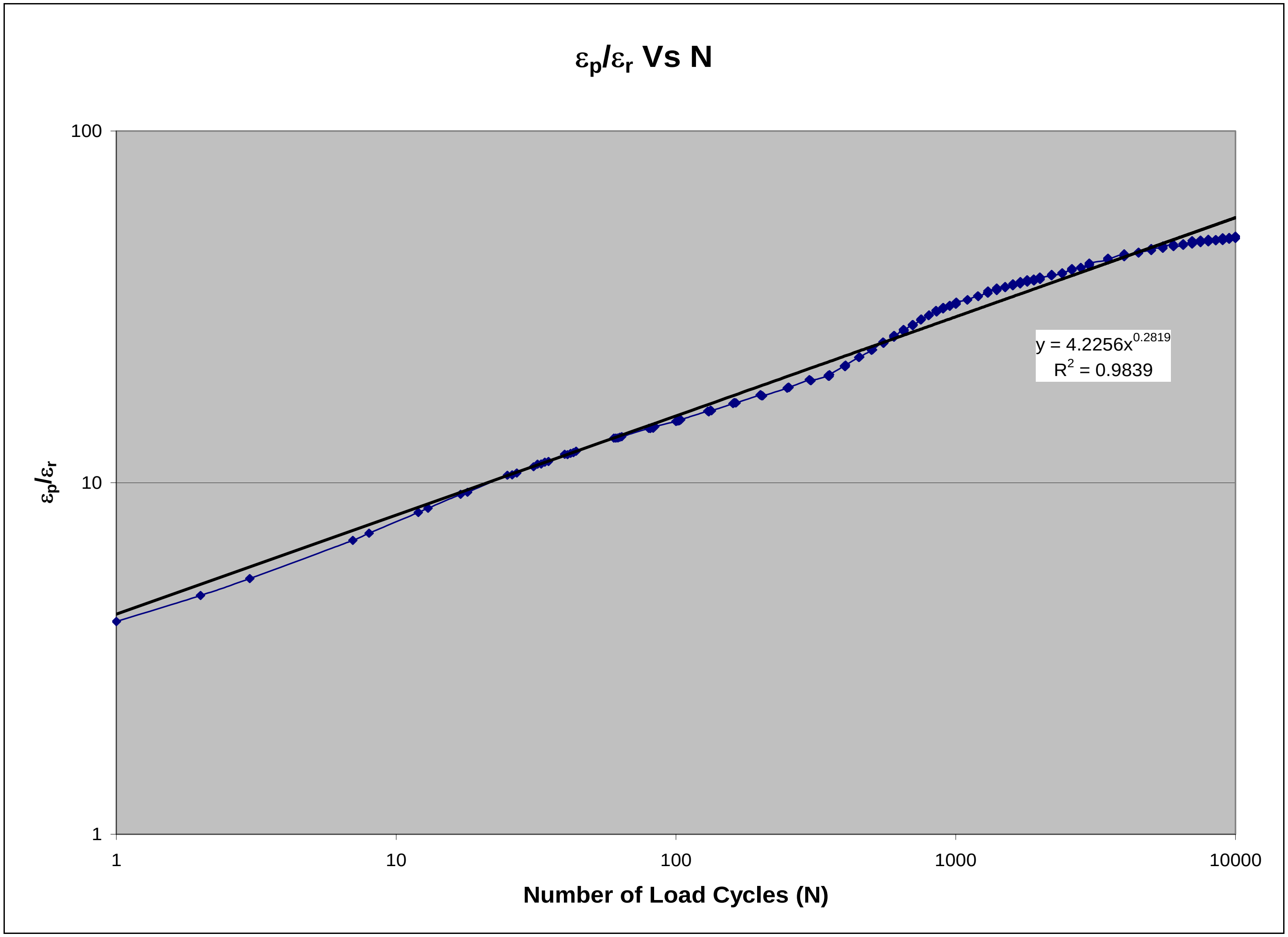

Figure A- 8 $\varepsilon_p/\varepsilon_r$ **Vs Number of load cycles for specimen #4**

SPECIMEN # 5

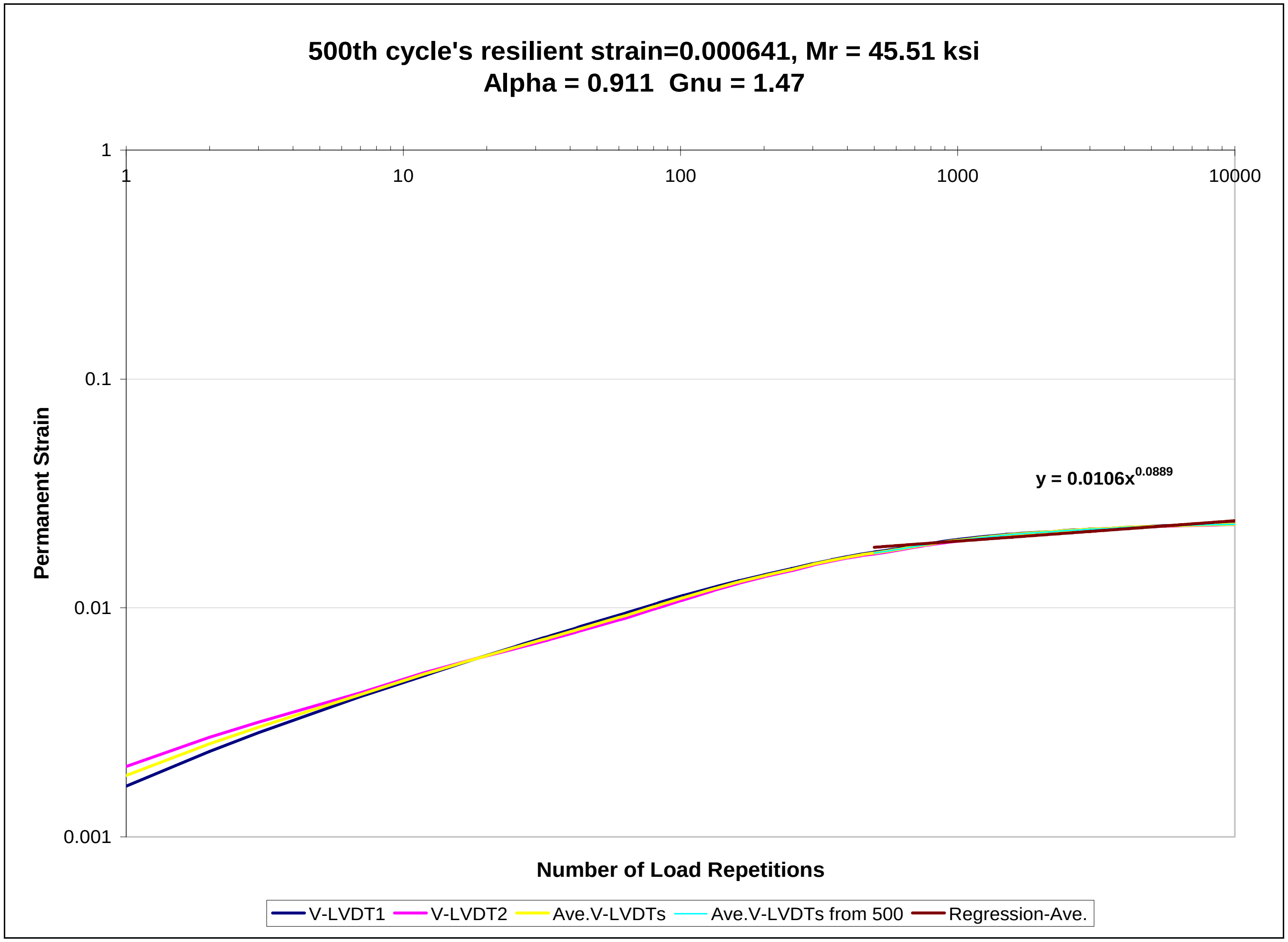

Figure A- 9 Permanent deformation result for specimen #5

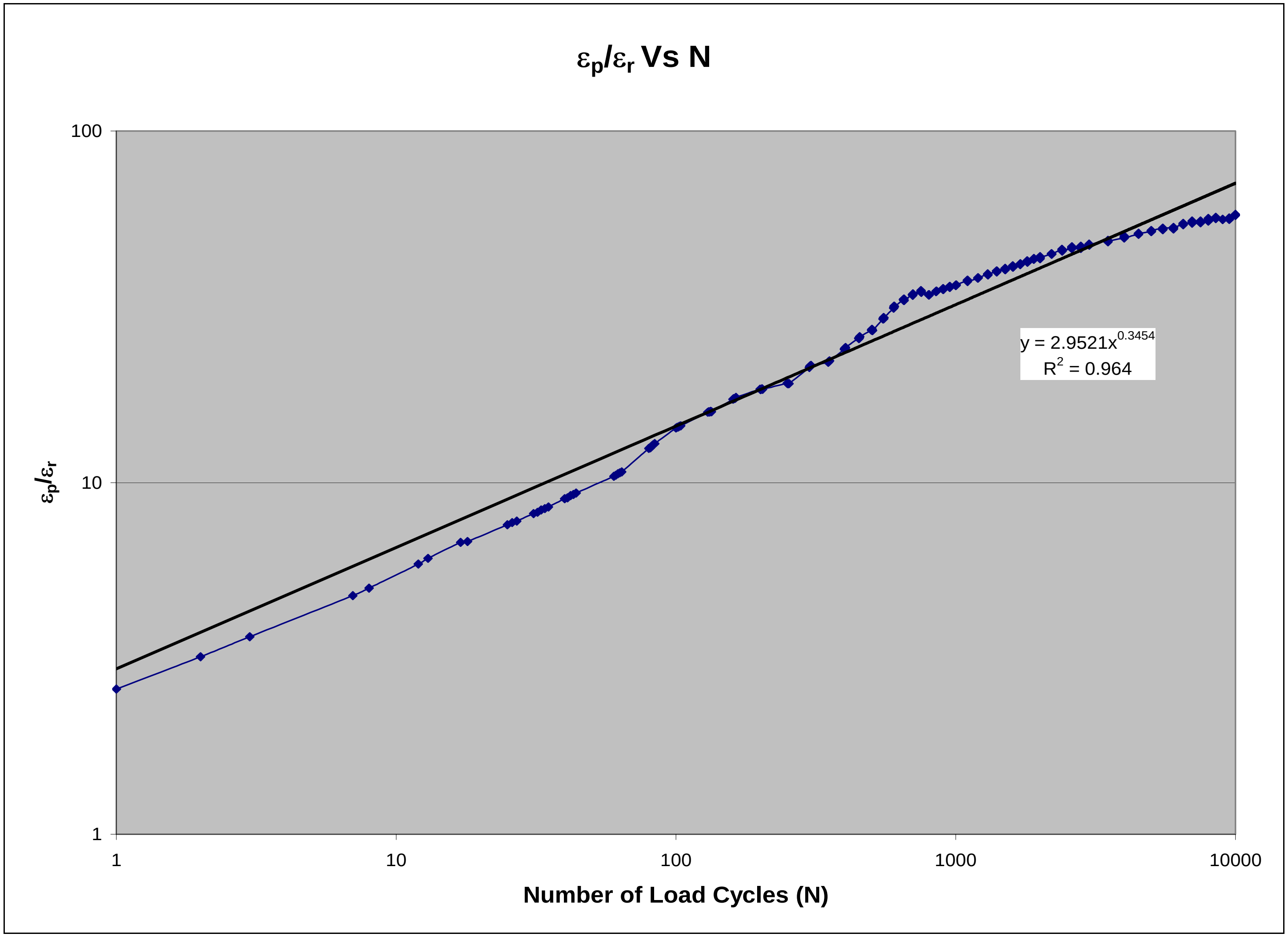

Figure A- 10 $\varepsilon_p/\varepsilon_r$ Vs Number of load cycles for specimen #5

SPECIMEN # 6

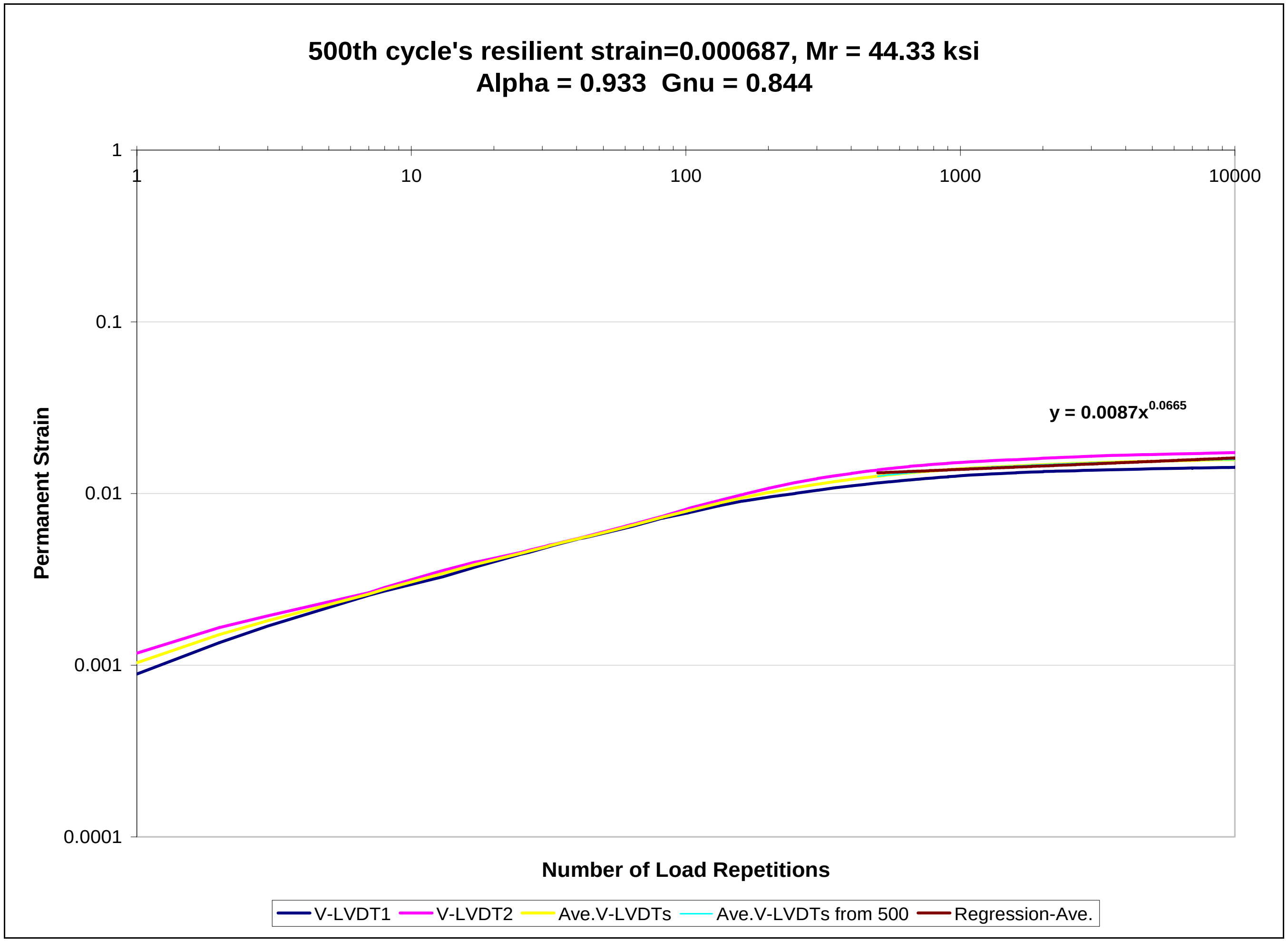

Figure A- 11 Permanent deformation result for specimen #6

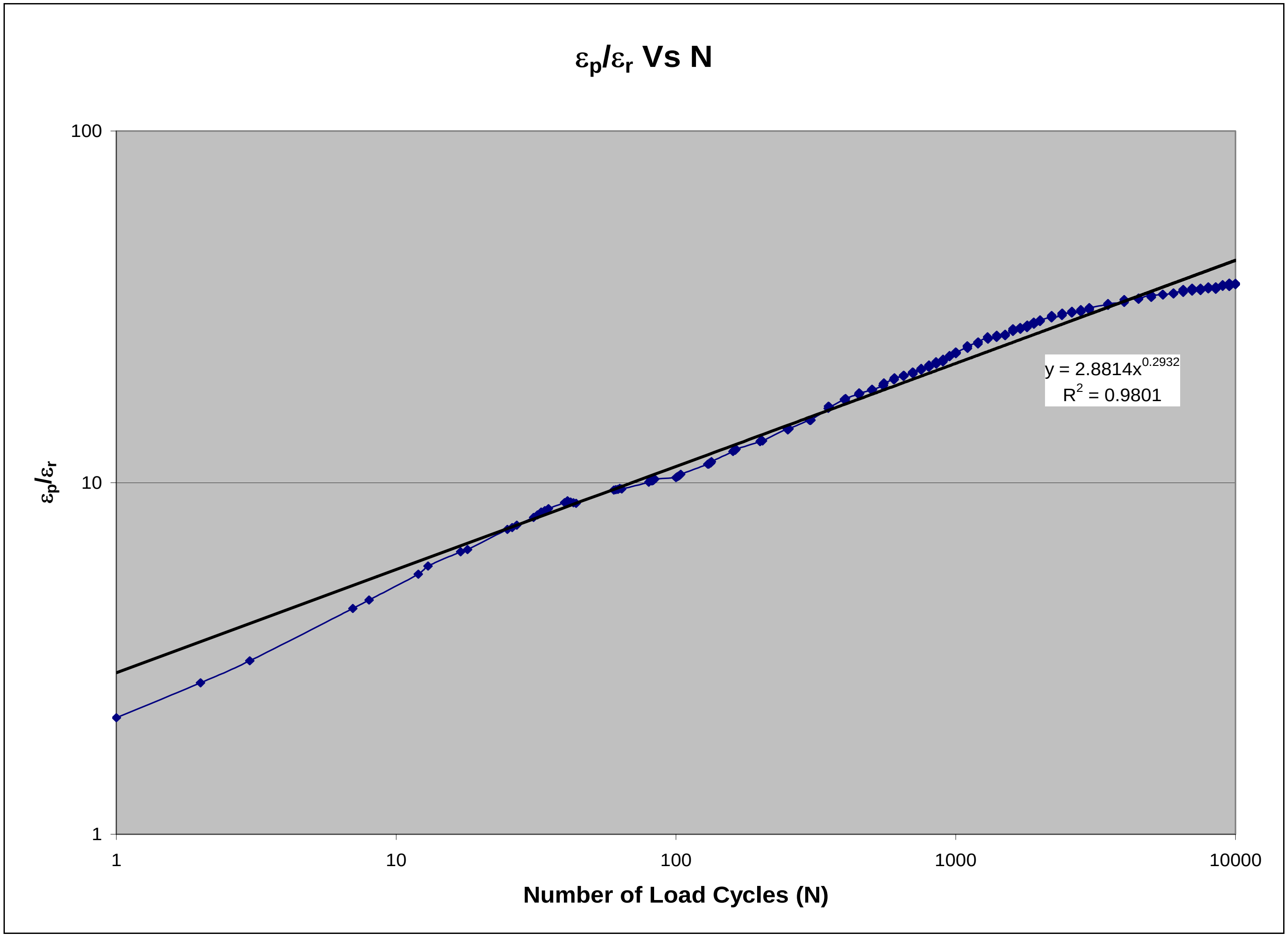

Figure A- 12 $\varepsilon_p/\varepsilon_r$ Vs Number of load cycles for specimen #6

SPECIMEN # 7

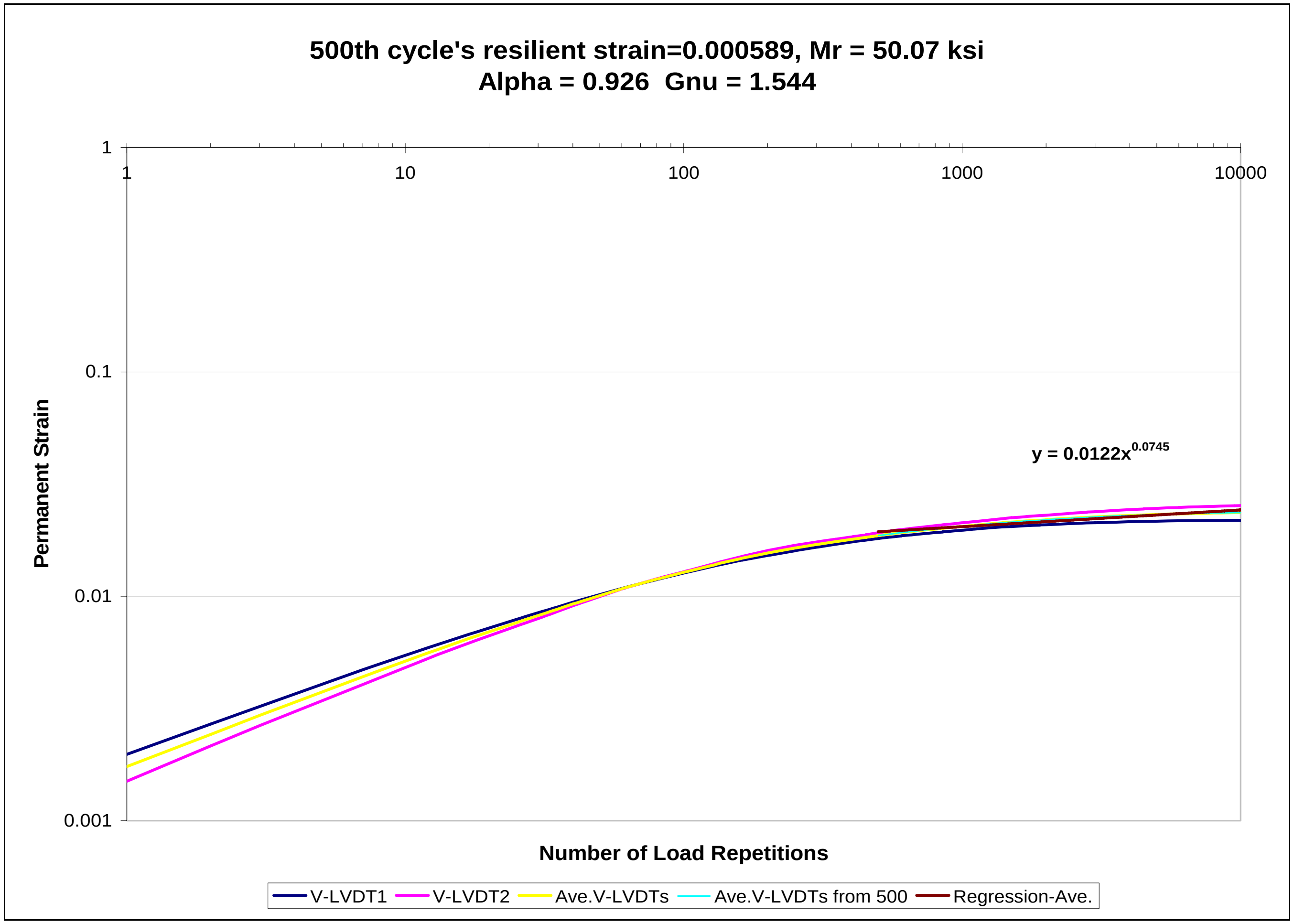

Figure A- 13 Permanent deformation result for specimen #7

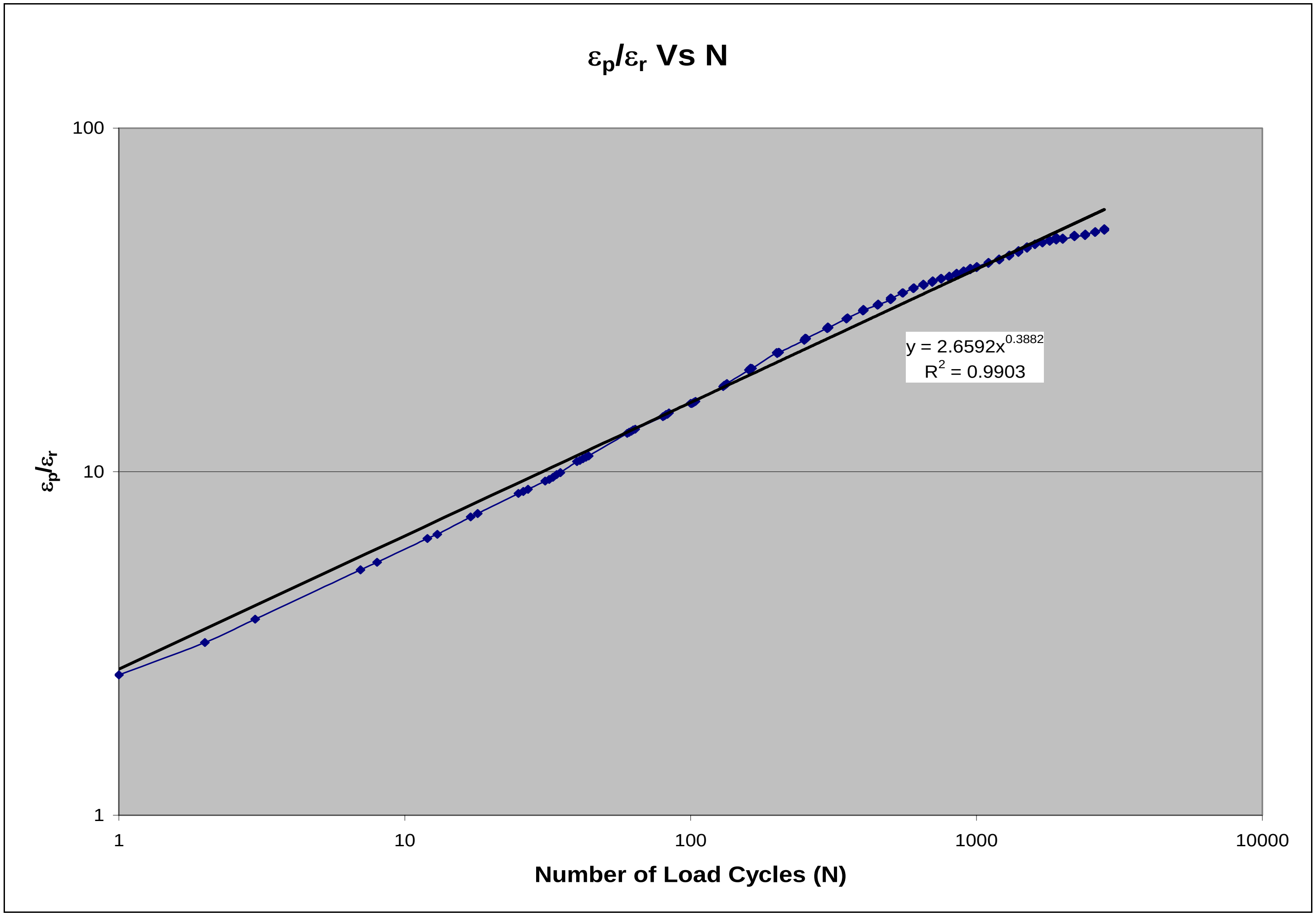

Figure A- 14 $\varepsilon_p/\varepsilon_r$ Vs Number of load cycles for specimen #7

APPENDIX B

PERMANENT DEFORMATION RESULTS FOR

SPICEWOOD SPRINGS

SPECIMEN # 1

2002 Design Guide, Granular Base Resilient Modulus Mr-v, for Level I analysis

Regression Equation:

$$M_r = k_1 P_a \left(\frac{\theta}{P_a}\right)^{k_2} \left(\frac{\tau_{oct}}{P_a} + 1\right)^{k_3}$$

K1=1699.46
K2=.71
K3=.04

Resilient Vertical Modulus at Confining Pressure=5 psi and Deviator Stress =15 psi

Mr-v=42.15 ksi

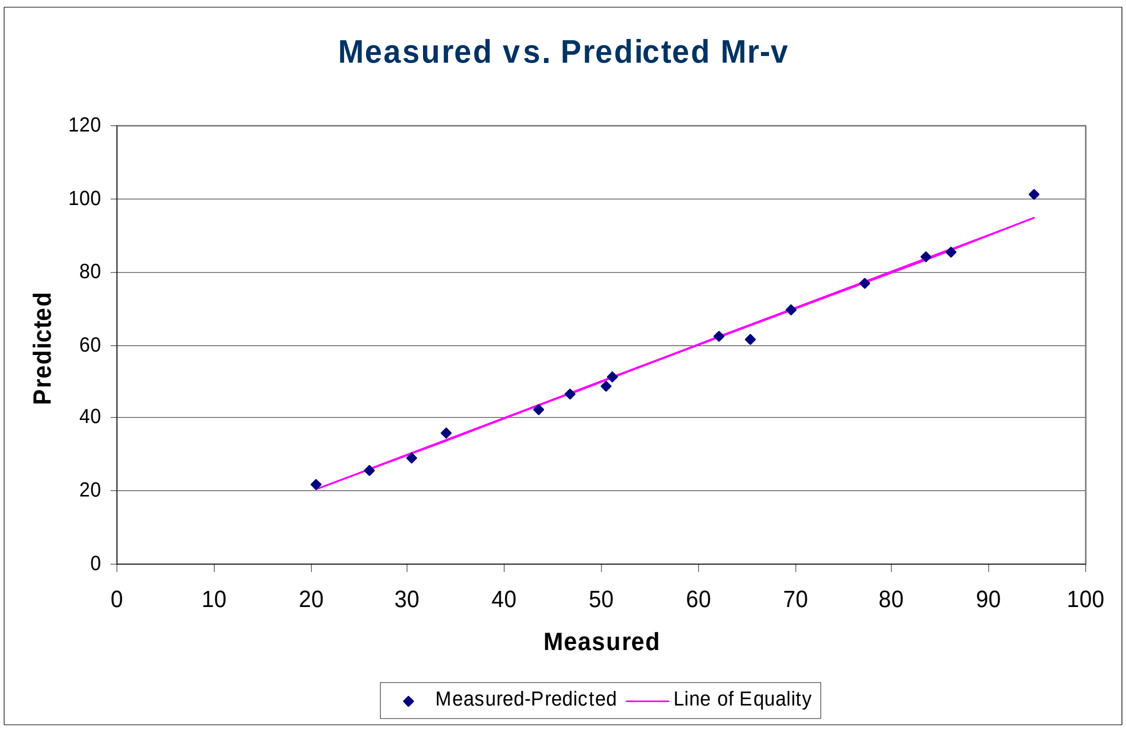

Figure B- 1 Regression plot of resilient modulus results for specimen #1

SPECIMEN # 2

2002 Design Guide, Granular Base Resilient Modulus Mr-v, for Level I analysis

Regression Equation:

$$M_r = k_1 P_a \left(\frac{\theta}{P_a}\right)^{k_2} \left(\frac{\tau_{oct}}{P_a} + 1\right)^{k_3}$$

K1=2424.13
K2=1.13
K3=-.99

Resilient Vertical Modulus at Confining Pressure=5 psi and Deviator Stress =15 psi

Mr-v=54.21 ksi

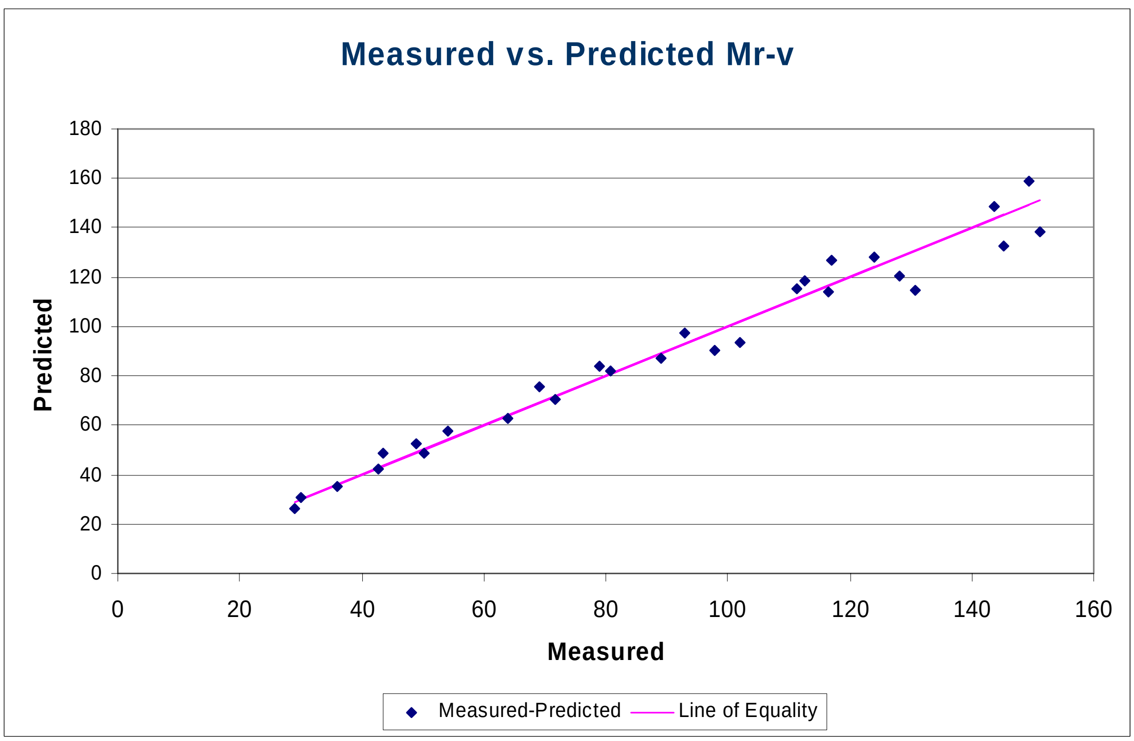

Figure B- 2   Regression plot of resilient modulus results for specimen #2

SPECIMEN # 3

2002 Design Guide, Granular Base Resilient Modulus Mr-v, for Level I analysis

Regression Equation:

$$M_r = k_1 P_a \left(\frac{\theta}{P_a}\right)^{k_2} \left(\frac{\tau_{oct}}{P_a} + 1\right)^{k_3}$$

K1=2591.25
K2=1.02
K3=-.98

Resilient Vertical Modulus at Confining Pressure=5 psi and Deviator Stress =15 psi

Mr-v=53.63 ksi

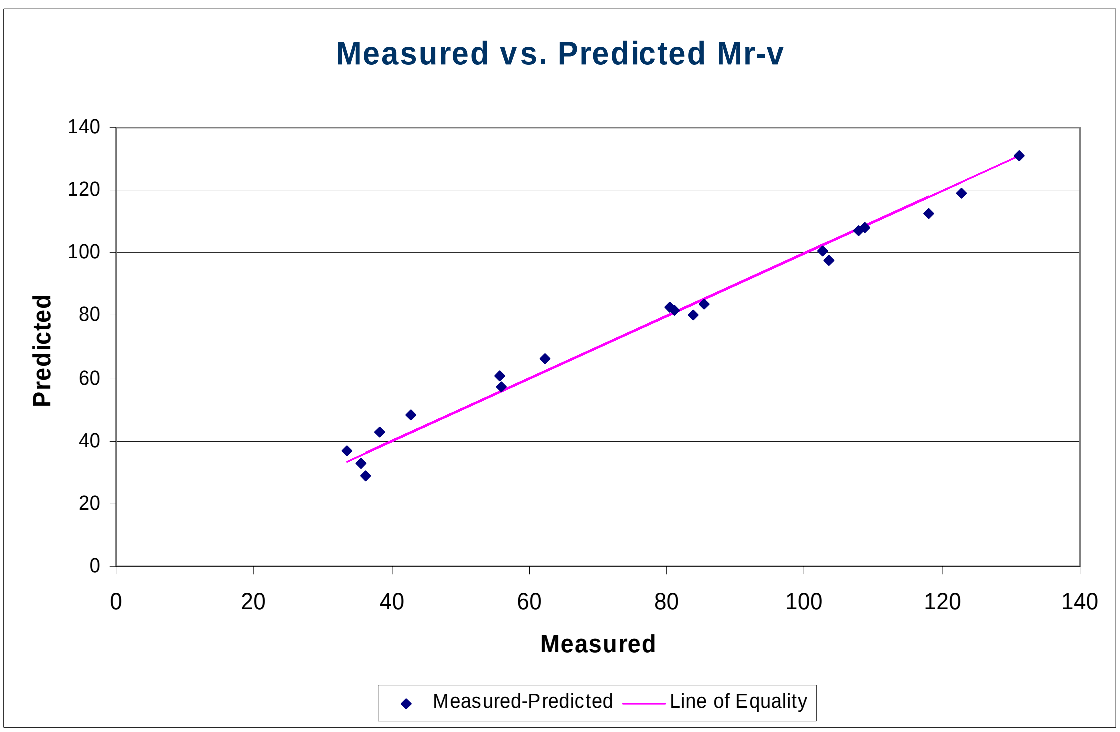

Figure B- 3   Regression plot of resilient modulus results for specimen #3

SPECIMEN # 4

2002 Design Guide, Granular Base Resilient Modulus Mr-v, for Level I analysis

Regression Equation:

$$M_r = k_1 P_a \left(\frac{\theta}{P_a} \right)^{k_2} \left(\frac{\tau_{oct}}{P_a} + 1 \right)^{k_3}$$

K1=2406.69
K2=.81
K3=-.55

Resilient Vertical Modulus at Confining Pressure=5 psi and Deviator Stress =15 psi

Mr-v=50.73 ksi

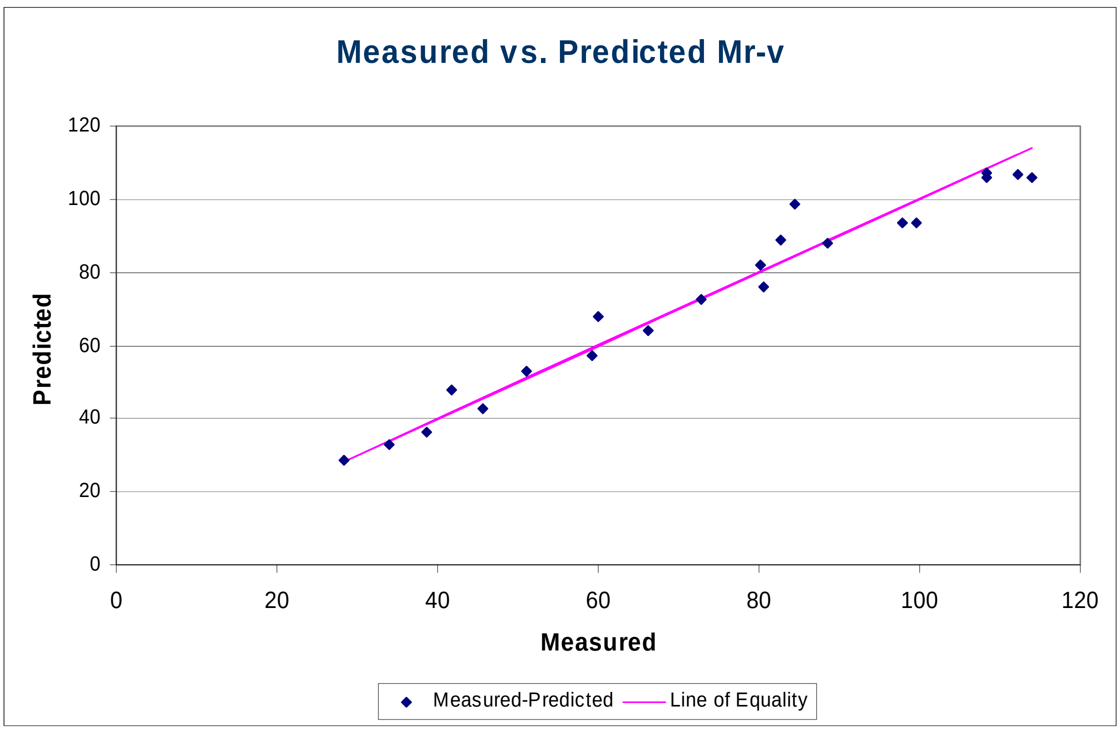

Figure B- 4 Regression plot of resilient modulus results for specimen #4

SPECIMEN # 5

2002 Design Guide, Granular Base Resilient Modulus Mr-v, for Level I analysis

Regression Equation:

$$M_r = k_1 P_a \left(\frac{\theta}{P_a} \right)^{k_2} \left(\frac{\tau_{oct}}{P_a} + 1 \right)^{k_3}$$

K1=2321.15
K2=1.05
K3=-.83

Resilient Vertical Modulus at Confining Pressure=5 psi and Deviator Stress =15 psi

Mr-v=51.97 ksi

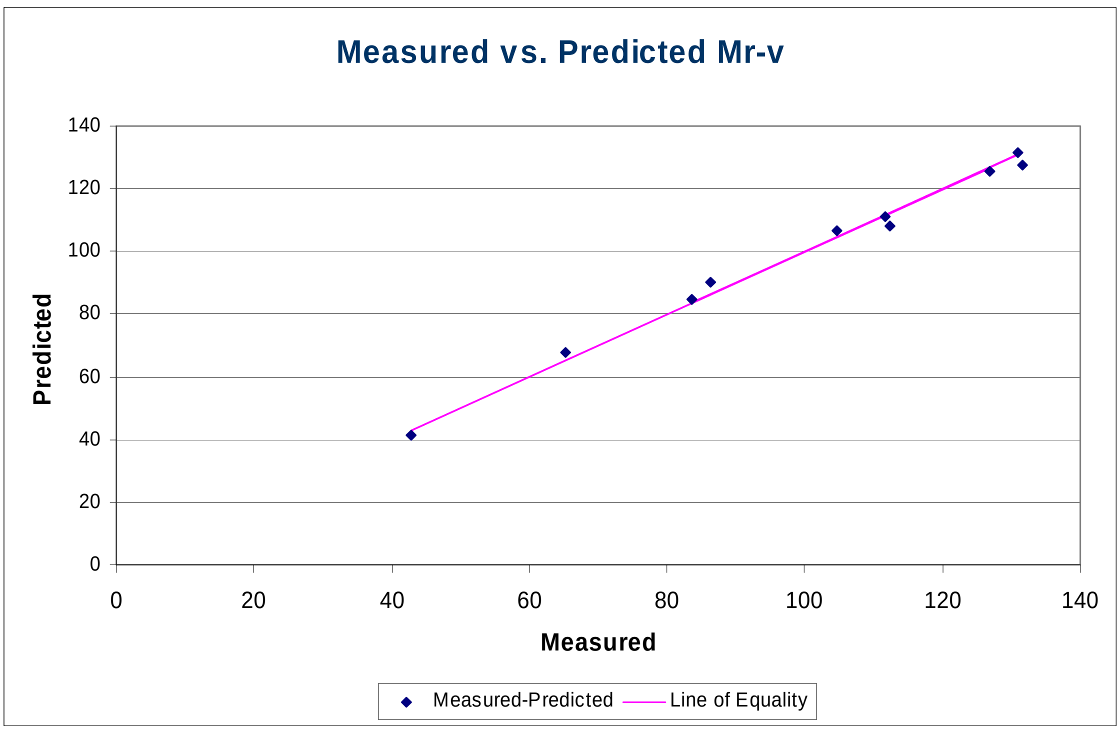

Figure B- 5 Regression plot of resilient modulus results for specimen #5

SPECIMEN # 6

2002 Design Guide, Granular Base Resilient Modulus Mr-v, for Level I analysis

Regression Equation:

$$M_r = k_1 P_a \left(\frac{\theta}{P_a}\right)^{k_2} \left(\frac{\tau_{oct}}{P_a} + 1\right)^{k_3}$$

K1=2002.96
K2=.74
K3=-.45

Resilient Vertical Modulus at Confining Pressure=5 psi and Deviator Stress =15 psi

Mr-v=41.94 ksi

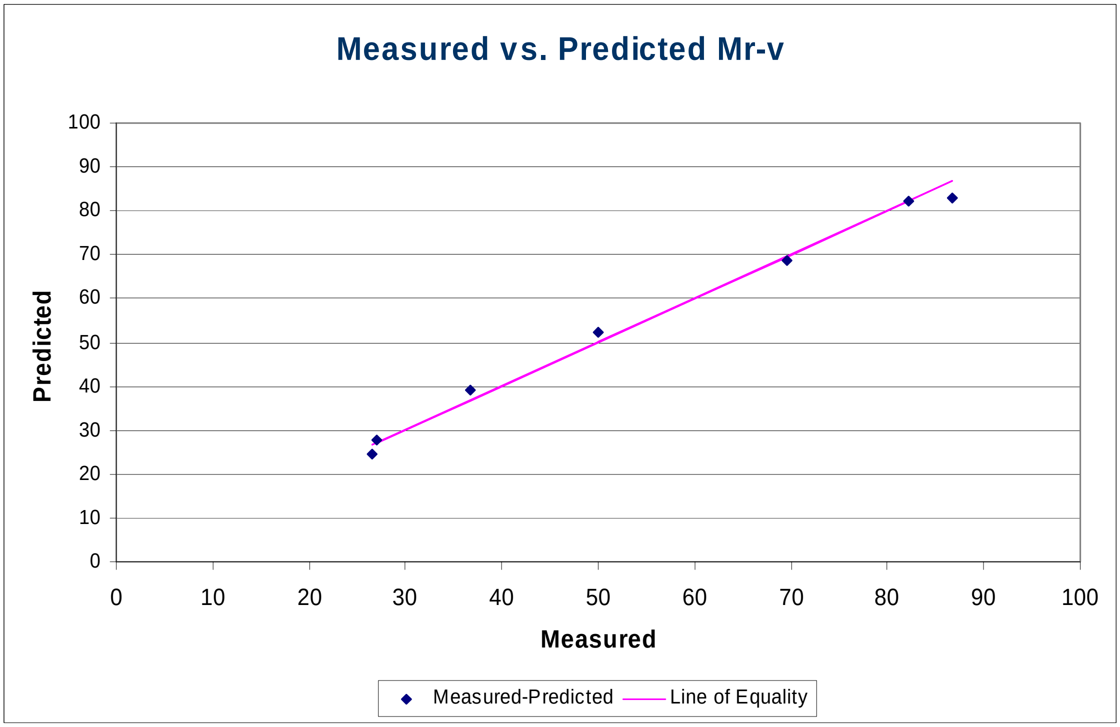

Figure B- 6 Regression plot of resilient modulus results for specimen #6

SPECIMEN # 7

2002 Design Guide, Granular Base Resilient Modulus Mr-v, for Level I analysis

Regression Equation:

$$M_r = k_1 P_a \left(\frac{\theta}{P_a} \right)^{k_2} \left(\frac{\tau_{oct}}{P_a} + 1 \right)^{k_3}$$

K1=2057.65
K2=1.25
K3=-1.26
Recommended Resilient Vertical Modulus at Confining Pressure=5 psi and Deviator Stress =15 psi

Mr-v=45.02 ksi

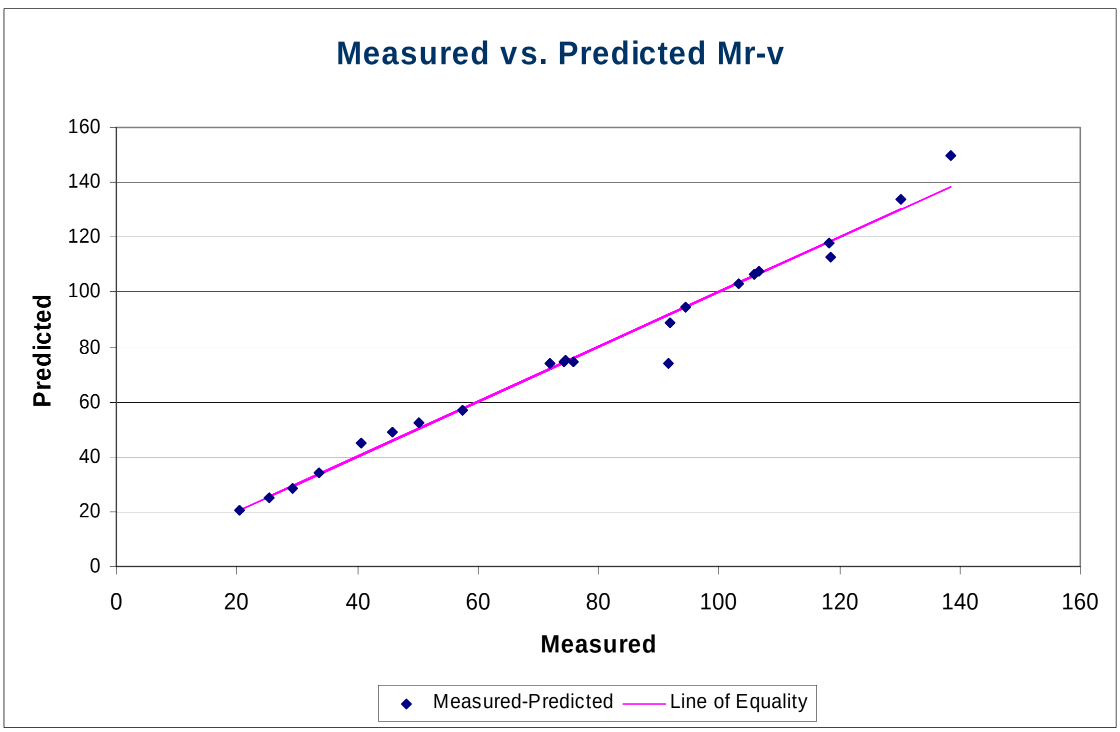

Figure B- 7 Regression plot of resilient modulus results for specimen #7

APPENDIX C

NEW YORK BASE MATERIAL PROPERTIES

Gradation (wash) for New York Type 1

Date: 12/18/2003
Initial Wt: 4000

Sieve Size				
Standard	Metric	Weight Retained	% Passing	% Retained
1 1/4	31.75	431	89.23	10.78
7/8	22.225	628	73.53	15.70
5/8	15.875	149	69.80	3.73
3/8	9.525	394	59.95	9.85
#4	4.76	402	49.90	10.05
#10	2	329	41.68	8.23
#40	0.42	936	18.28	23.40
#200	0.074	328	10.08	8.20
-200	< 0.075	403	0.00	10.08
				100.00

Total: 4000
Dust: 0

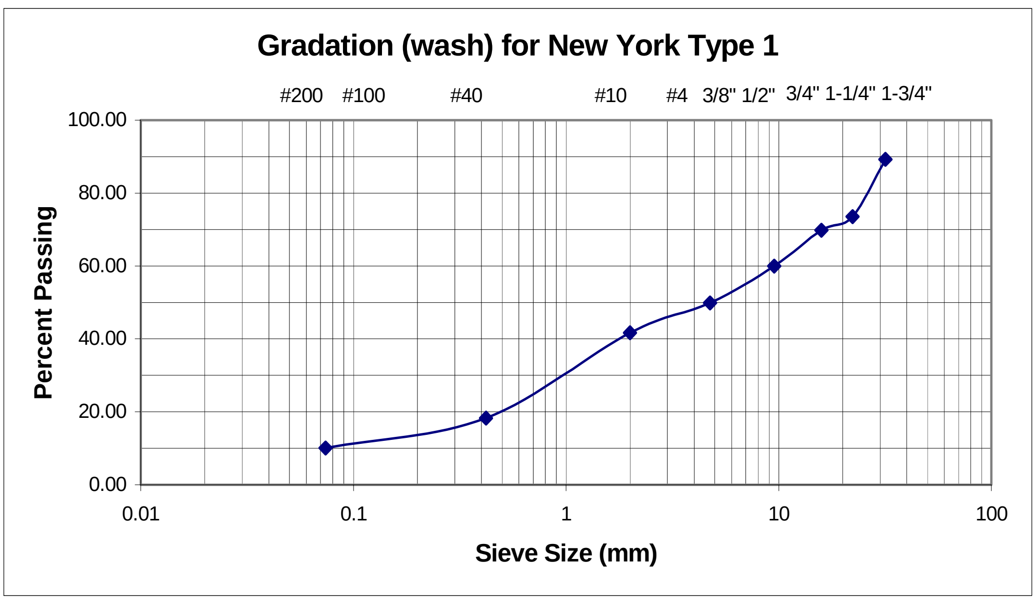

Figure C- 1 Gradation plots for New York base

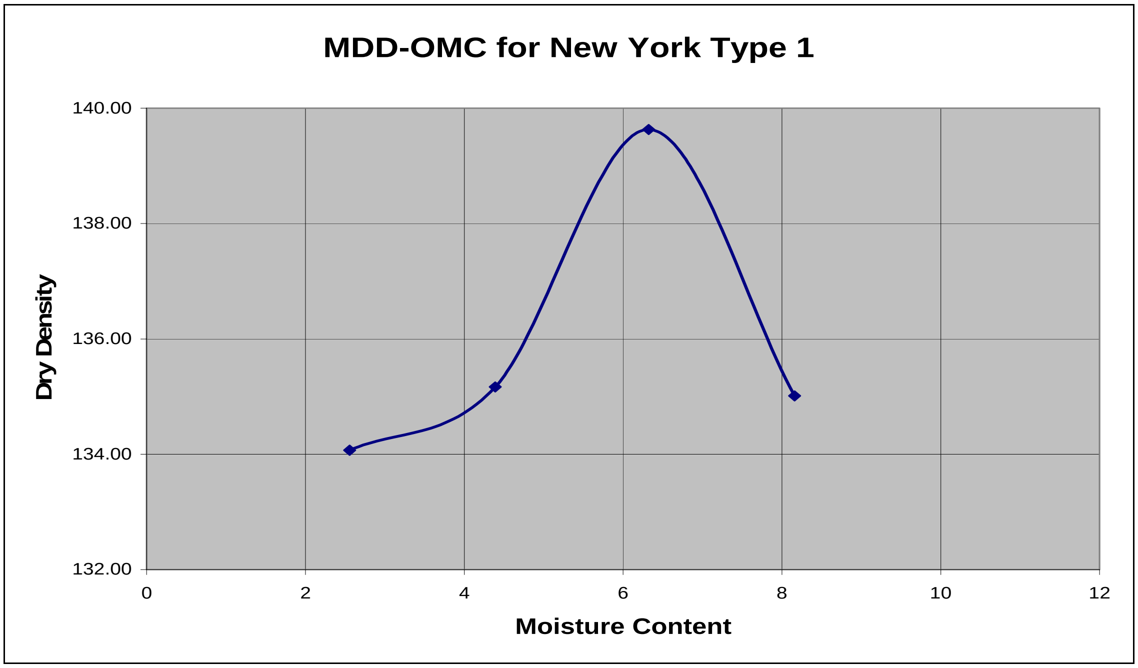

Figure C- 2 Moisture density relationship for specimen compacted with proctor compaction

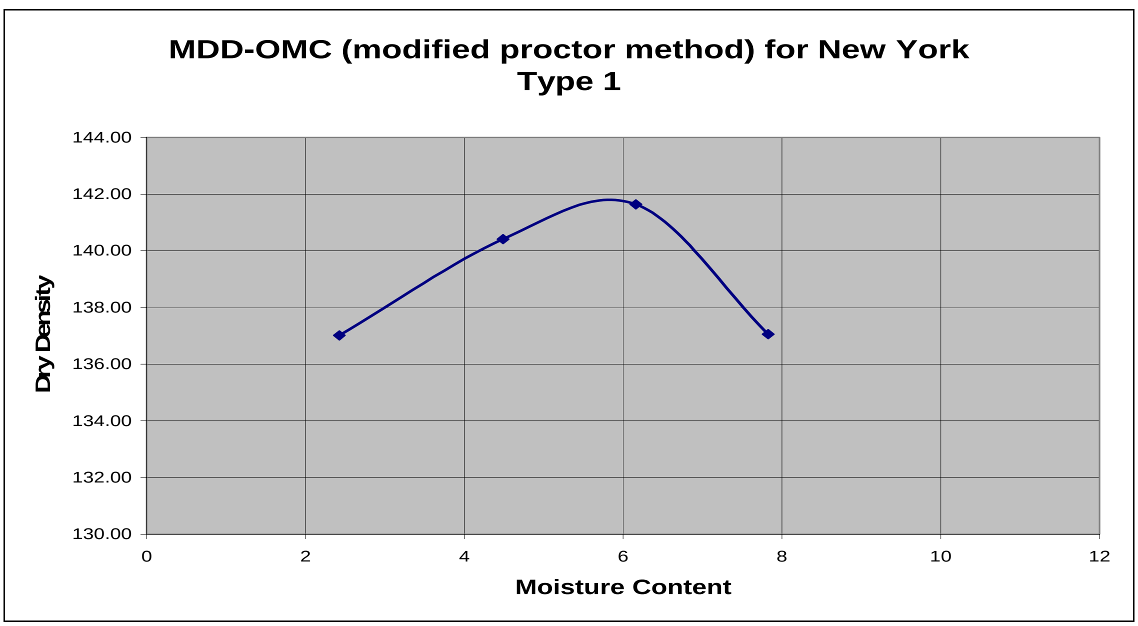

Figure C- 3 Moisture density relationship for specimen compacted with modified proctor compaction

APPENDIX D

HIGHWAY 6 AND US 59 MATERIAL PROPERTIES

Gradation on Highway 6 and US 59

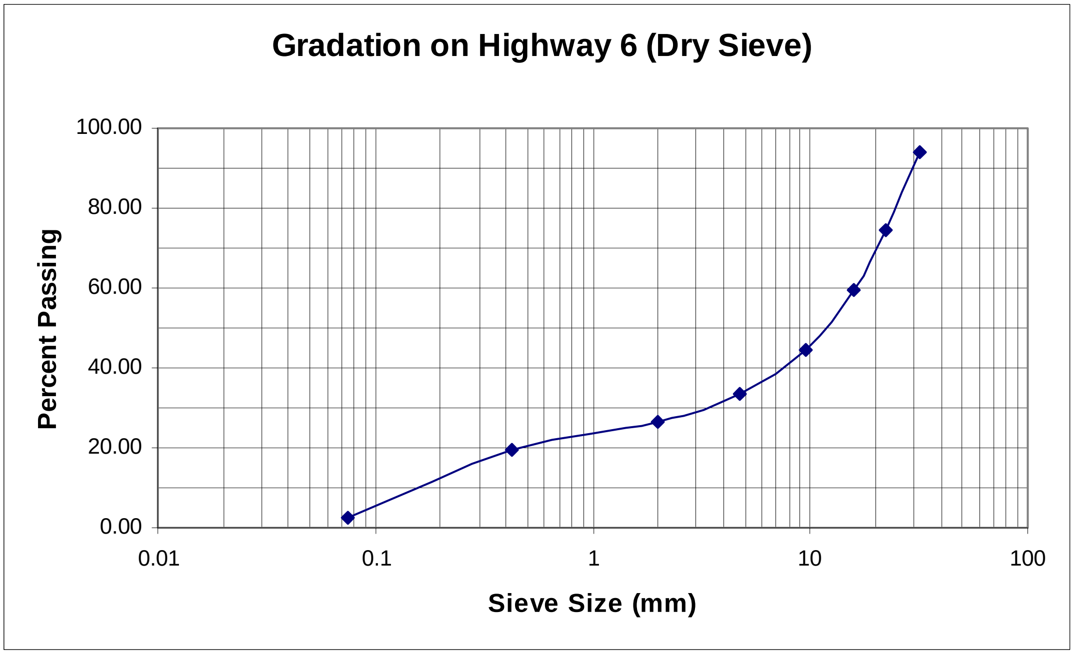

Figure D- 1 Dry sieve analysis on Highway 6

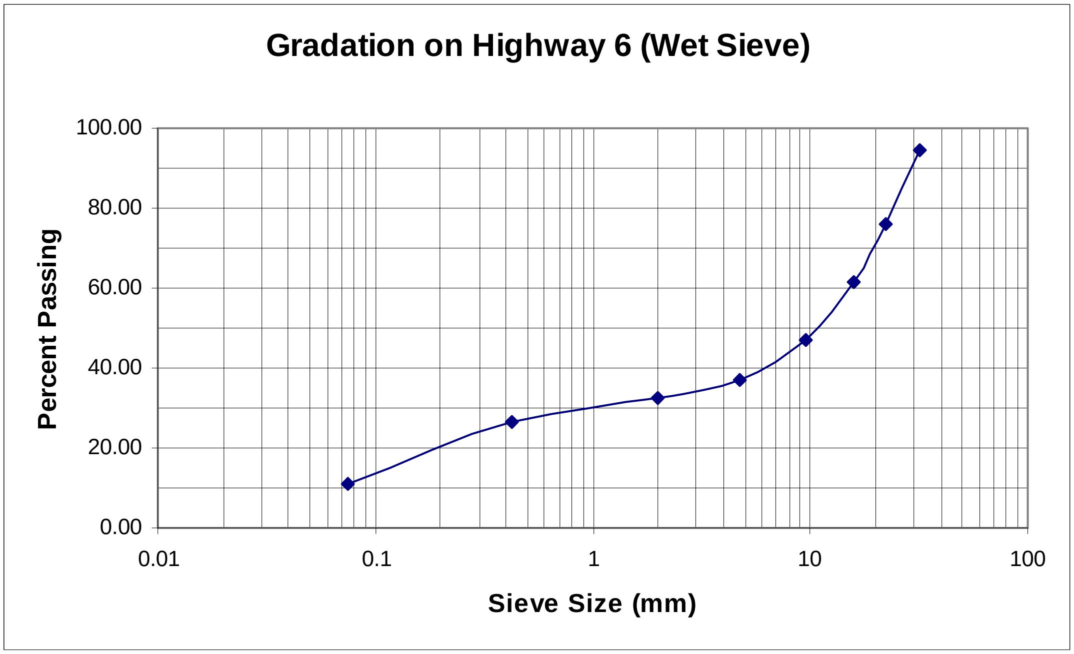

Figure D- 2 Wet sieve analysis on Highway 6

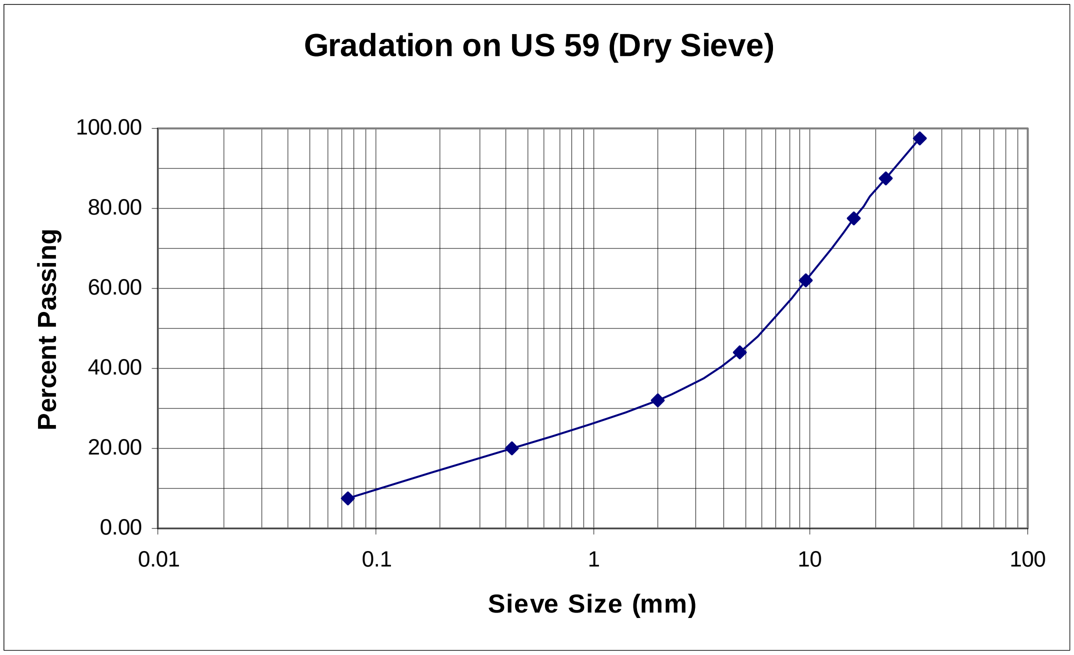

Figure D- 3 Dry sieve analysis on US 59

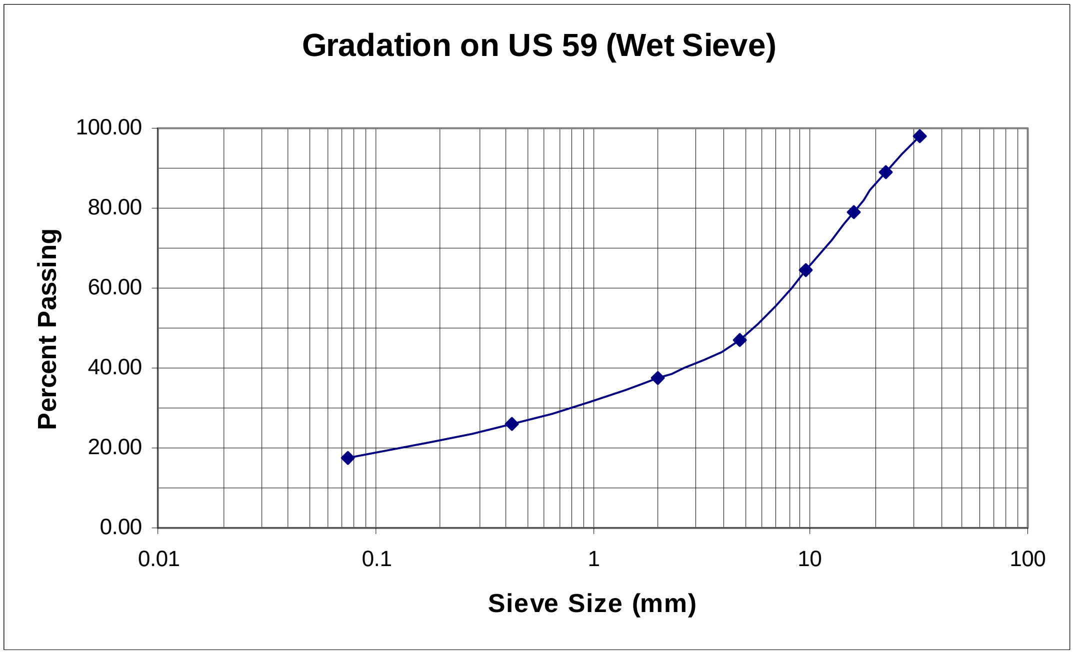

Figure D- 4 Wet sieve analysis on US 59

Table D- 1 Primary Properties of the Highway 6 and US 59 Base Materials

Test Result	**Highway 6**	**US 59**
Linear Shrinkage	2	2
Liquid limit	18	17
Plastic limit	16	12
Plasticity Index	2	4
% percent of fines	2.57	7.34
Optimum Moisture Content	7.9	7.8
Maximum Dry Density	137.33	137.59

VITA

Anuroopa Kancherla was born in Hyderabad, India on February 23, 1981. She received Bachelor of Science degree in civil engineering in July 2002 from Osmania University, Hyderabad, India. Miss. Kancherla enrolled in graduate studies at Texas A&M University in September 2002 and received her Master of Science degree in civil engineering in August 2004. While attending graduate school at Texas A&M, she was employed as a graduate research assistant at Texas Transportation Institute. She received her Engineer in Training Certificate from Texas in December 2003.

After graduation, Miss Kancherla has intentions of pursuing a Ph.D. degree in civil engineering and plans to work in the transportation materials consulting field. Miss Kancherla can be reached through her parents at MIG-232, Balaji Nagar Housing Board Colony, Kukatpally, Hyderabad, Andhra Pradesh, India –500072.