Final Report

PRECISION AND BIAS OF RESILIENT MODULUS TEST

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FOREWORD

This report documents determination of the precision of test methods AASHTO T 307 and NCHRP 1-28a in measuring the resilient modulus of unbound aggregate base materials and subgrade soils. The report also compares the resilient modulus values measured in accordance with both test methods and defines whether there is a practical difference in determining the design resilient modulus from both procedures. This information can be used to provide inputs to the new AASHTO Pavement ME Design software for the design of flexible and rigid pavement structures.

This final report is intended for use by pavement researchers, as well as by practicing engineers and laboratory technicians involved in measuring and using the resilient modulus of unbound pavement materials and soils.

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16. Abstract
Most agencies use AASHTO T 307 for measuring the resilient modulus of unbound materials and soils. AASHTO T 307 is similar to the LTPP test procedure used for measuring the resilient modulus of unbound materials and soils. To improve on the resilient modulus test, NCHRP sponsored projects 1-28 and 1-28a to recommend details of the repeated load resilient modulus test to reduce its variability. That project recommended a test protocol which is identified as the NCHRP 1-28a test method. The primary difference between NCHRP 1-28a and AASHTO T 307 are in the stress sequencing used and in the location of the deformation measuring devices. However, all versions of the resilient modulus test have been criticized relative to the variability of the test results.

The MEPDG Manual of Practice allows both NCHRP 1-28a and AASHTO T 307 to be used in measuring the resilient modulus primarily because the debate on the precision and bias of these test methods had not been settled at the time of its publication in 2008. As a result, further guidance is needed both in terms of testing the resilient modulus, but more importantly, in applying the test results for designing pavements in accordance with the MEPDG.

The objective of this study was to determine the precision and bias of resilient modulus test methods AASHTO T 307 and NCHRP 1-28a, as well as how to interpret the test results in determining a value to be used in design. The document documents the precision of both test methods in terms of a single operator and between multi-laboratories.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.
(Revised March 2003)

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ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
ASTM	American Society for Testing and Materials
ATB	Asphalt treated base
BCD	Burns Cooley Dennis, Inc.
BRE	Brent Rauhut Engineering, Inc.
CBR	California Bearing Ratio
COV	Coefficient of variation
CRCP	Continuously reinforced concrete pavement
DCP	Dynamic cone penetrometer
CRREL	Cold Regions Research & Engineering Laboratory
DOT	Department of Transportation
ESAL	Equivalent single axle load
FATB	Foamed asphalt treated base
FHWA	Federal Highway Administration
FWD	Falling weight deflectometer
GAB	Granular aggregate base
HMA	Hot mix asphalt
IRI	International Roughness Index
JPCP	Jointed Plain Concrete Pavement
LTPP	Long Term Pavement Performance
LTRC	Louisiana Transportation Research Center
LVDT	Linear variable differential transducer
LWD	Light weight deflectometer
ME	Mechanistic-Empirical
MEPDG	Mechanistic Empirical Pavement Design Guide
NCAT	National Center for Asphalt Technologies
NCHRP	National Cooperative Highway Research Program
NHI	National Highway Institute
NP	Non Plastic
PCC	Portland Cement Concrete
PFS	Pooled fund study
PMIS	Pavement management information system
PSPA	Portable seismic pavement analyzer
RAP	Recycled (Reclaimed) Asphalt Pavement
RCA	Recycled concrete aggregate
RLT	Repeated load triaxial
SASW	Spectral analysis of surface waves
SHA	State Highway Agency
SHRP	Strategic Highway Research Program
SMP	Seasonal Monitoring Pavement
SWCC	Soil water characteristics curve

PRECISION AND BIAS OF RESILIENT MODULUS TEST

CHAPTER 1 INTRODUCTION

1.1 Background

The American Association of State Highway and Transportation Officials (AASHTO) adopted the Mechanistic-Empirical Pavement Design Guide (MEPDG) as an interim pavement design standard in 2008 (AASHTO, 2008). The MEPDG is considered a quantum leap forward from previous pavement design procedures, materials testing, and analysis. Some agencies in the United States (US) have already transitioned to this new method (for example: Arizona, Colorado, Indiana, Missouri, Utah, and Virginia). Many other agencies are evaluating the procedure and creating material input libraries to tailor the AASHTO MEPDG to their local conditions, soils, and materials (for example: Georgia, Maine, Michigan, Mississippi, Nevada, Pennsylvania, Wisconsin Wyoming, etc.). One of the material input libraries is the resilient modulus of unbound materials and soils.

Since the 1970s, various versions of the repeated load resilient modulus test have been used to measure the resilient modulus for mechanistic-empirical (ME) based pavement design procedures. The Long Term Pavement Performance (LTPP) program included this test in characterizing all unbound pavement layers and soils (SHRP Test Protocol P46). Results from this test method are included in the national LTPP database (information regarding this database is published available at:
http://www.fhwa.dot.gov/research/tfhrc/programs/infrastructure/pavements/ltpp/getdata.cfm).
LTPP sponsored an extensive study to determine the precision of the test method, but the study was never completed because issues of sample preparation, displacement measurement location, and others were not resolved.

Eventually, AASHTO adopted test procedure AASHTO T 307. This procedure is similar to the test procedure used in the LTPP program for measuring the resilient modulus of unbound materials and soils. AASHTO T 307, however, has received criticism regarding the amount of variability that has been reported. In actuality, all versions have been criticized relative to the variability of the test results. Boudreau has stated: “nearly every publication regarding the resilient modulus test contains negative language in the introductory paragraphs implying that the test is *too difficult, not repeatable, or not reliable*. Numerous testing laboratories across the nation have overcome the hurdle of *too difficult*, having committed resources to acquire test equipment and train staff on how to operate and maintain the resilient modulus test equipment and data acquisition software.” Many have already found the test to be much easier to perform than the popular California Bearing Ratio (CBR) test in terms of the amount of material to process, effort required to fabricate test specimens, and time required to run the test. Boudreau also states: “while this is certainly a step in the right direction, the issue of not repeatable and not reliable still persists.”

To improve the resilient modulus test method, National Cooperative Highway Research Program (NCHRP) sponsored projects 1-28 and 1-28a to identify and recommend details of the repeated load resilient modulus test to reduce its variability. That project recommended a test protocol which was identified as the NCHRP 1-28a test method/protocol. The primary differences between NCHRP 1-28a and AASHTO T 307 are in the stress sequencing used and in the location of the deformation measuring devices. The MEPDG Manual of Practice allows both NCHRP 1-28a and T 307 to be used in measuring the resilient modulus primarily because the debate on the precision and bias of these test methods had not been settled at the time of its publication in 2008. Few agencies, however, are using the NCHRP 1-28a test method and the variability of both methods has yet to be quantified.

Two other topics or questions related to resilient modulus testing have not been adequately answered, which are: (1) how sensitive a design is with respect to the resilient modulus, and (2) how to determine the design resilient modulus as an input to the MEPDG software (Pavement ME Design). In other words, if the same material (soil or unbound aggregate base material) is tested multiple times by either a single lab with one operator or a single lab with multiple operators or multiple labs how close do the results need to be such as not to affect the final design? As an example, laboratory A yields a 12,000 psi test result while laboratory B yields an 8,500 psi result (more than a 40 percent difference). Does this difference result in a significantly different pavement structure with all other design inputs remaining the same (traffic, climate, distress threshold values, etc.)?

With respect to determination of the design resilient modulus for unbound materials and soils to support the MEPDG, further guidance is needed both in terms of testing the resilient modulus in the laboratory, but more importantly, in applying the test results for designing pavement strategies using the Pavement ME Design software.

This report provides a state-of-the-practice update or synthesis regarding the resilient modulus testing (precision and bias of the test methods) in two areas: (1) the test method itself and variability of the test results, and (2) interpretation of the test results in determining a value to be used for pavement design. The report also provides the precision of both test methods in terms of a single operator and between multiple laboratories.

1.2 Project Objective

The primary objective of this study was to determine the precision of the resilient modulus test methods AASHTO T 307 and NCHRP 1-28a that is an input for designing pavement structures in accordance with the AASHTO Pavement ME Design software. There were two other objectives of the study which were: to determine if there is a significant bias between AASHTO T 307 and NCHRP 1-28a, and to recommend the best method for incorporating resilient modulus into the AASHTO Pavement Design ME software, if different from the 2008 version of the MEPDG Manual of Practice.

1.3 Scope of Report

The report includes five chapters, including Chapter 1—the Introduction. Chapter 2 is a review of the evolution of the resilient modulus test procedure and a summary of the factors that have a significant impact on resilient modulus. This chapter also includes a brief discussion on how the resilient modulus test results have historically been used.

Chapter 3 is a summary of the ruggedness test program in preparation for the round robin test program, while Chapter 4 includes a discussion of the round robin test program and an analysis of the test data for determining the precision of AASHTO T 307 and NCHRP 1-28a. The precision of both test methods in terms of a single operator and between multiple laboratories are included in Chapter 4.

Chapter 5 provides a summary of the recommendations and suggestions for taking the resilient modulus test data and determining the inputs to the Pavement ME Design software. This includes a step by step procedure that is consistent with the current version of the Pavement ME Design software that can be incorporated into the next version of the MEPDG Manual of Practice. Chapter 6 is a summary of the findings from this study, as well as recommendations for more detailed guidance on determining the resilient modulus of unbound materials and soils for use in design in accordance with the MEPDG.

These chapters are followed by four appendices. Appendix A summarizes results from previous studies and documents, while Appendix B is a summary of the sensitivity of the distress predictions from the Pavement ME Design software to the resilient modulus of unbound pavement layers and subgrade. Appendix C summarizes the physical properties of the soils and materials included in the ruggedness and precision and bias test programs, reported herein, and Appendix D is a graphical summary of the test results for the precision and bias or round robin test program.

CHAPTER 2 OVERIEW OF RESILIENT MODULUS TEST

To date, several AASHTO test methods (T 274, T 292, T 294, and T 307) have been used by state agencies and industry for measuring the resilient modulus of unbound materials and soils. All of these test methods differ from each other in one or more of the following areas:

- Specimen preparation (remolding and compaction procedure).
- Conditioning methodology prior to the actual testing.
- Seating stress application.
- Test sequences of applying confining and cyclic deviatoric stresses.
- Deformation measurements inside/outside of the triaxial cell.

There are literally hundreds of reports on resilient modulus for pavement design, as well as for research purposes. The opinions and outcomes from these studies are diverse in terms of how the test should be performed, as well as how the test results get interpreted for use in design. Few of the documented studies, however, provide a detailed description of the resilient modulus test method, test system or equipment, or explain how the test specimens were prepared in the laboratory.

The Federal Highway Administration (FHWA) and other organizations have sponsored workshops at various points in time to try and find consensus in the standardization and use of the resilient modulus test results. Thus far, different opinions still exist on the "right way" to measure and use resilient modulus test results. The purpose of this chapter is to synthesize the evolution of the resilient modulus test and provide a summary of the factors that have an impact on the resilient modulus, as well as determining the value to be used in designing pavement structures.

2.1 Test Method Development and Use

NCHRP Synthesis 382 report presented the results of two surveys conducted to gather information in two areas: (1) how various agencies determine the resilient modulus of subgrades and unbound bases, and (2) how resilient modulus has been used in the design of pavement structures (Puppala, 2008). The NCHRP Synthesis 382 report also summarized the chronology of the AASHTO test method development for estimation of resilient modulus of subgrades, as shown in Table 1. Table 2 lists the resilient modulus test procedures being used by different agencies, which was prepared from a review of more recent publications and specifications.

The NCHRP Synthesis 382 surveys were completed by various State DOT individuals associated with Geotechnical/Materials groups and Pavement Design groups. The Geotechnical/Materials group survey responses (41 of 50 states) from the study indicated that a few respondents determined resilient modulus from various methods, including laboratory and field methods. The overall satisfaction of the respondents regarding the use of resilient modulus for mechanistic pavement design was found to be low due to constant modification of the test procedures, measurement difficulties, and design-related issues.

Table 1. Chronology of AASHTO Test Procedures for Mr Measurements (Source: NCHRP 382 Synthesis; Puppala, 2008)

Test Procedure	Details
AASHTO T-274-1982	Earliest AASHTO test procedure; No details on the sensitivities of displacement measurement devices were given; Criticisms on test procedure, test duration (5 hours long test) and probable failures of soil sample during conditioning phase; testing stresses are too severe.
AASHTO T-292-1991	AASHTO procedure introduced in 1991; Internal measurement systems are recommended; Testing sequence is criticized owing to the possibility of stiffening effects of cohesive soils
AASHTO T-294-1992	AASHTO modified the T-292 procedure with different sets of confining and deviator stresses and their sequence; Internal measurement system is followed; 2-parameter regression models (bulk stress for granular and deviator stress model for cohesive soils) to analyze test results; Criticism on the analyses models.
Strategic Highway Research Program P-46-1996	Procedural steps of P-46 are similar to T-294 procedure of 1992; External measurement system was allowed for displacement measurement; Soil specimen preparation methods are different from those used in T-292.
AASHTO T 307-1999	T-307-1999 was evolved from P-46 procedure; recommends the use of external displacement measurement system. Different procedures are followed for both cohesive and granular soil specimen preparation.
NCHRP 1-28a: Harmonized Method-2004 (RRD 285)	This recent method recommends a different set of stresses for testing. Also, a new 3-parameter model is recommended for analyzing the resilient properties. The use of internal measurement system is recommended in this method.

The Pavement group survey responses (40 of 50 states) indicate a need to develop simple procedures for resilient modulus determination. The survey results from the NCHRP Synthesis 382 report indicated that 12 of the 41 respondents use laboratory methods to measure resilient modulus. Of these, 9 respondents use repeated load triaxial (RLT) tests to measure resilient moduli of soil samples. In the RLT tests, four respondents followed the AASHTO T-307 procedure while two followed the NCHRP 1-28a Harmonized procedure. The remaining respondents followed AASHTO T 294, TP 46, or some other modified resilient modulus test method.

The NCHRP Synthesis 382 summary, however, did not refer to or identify some of the earlier resilient modulus test methods and work that was used in the late 1970's and early 1980's. As an example, the Cold Regions Laboratory and Waterways Experiment Station of the Corp of Engineers, the American Society for Testing and Materials (ASTM), the Asphalt Institute, Georgia Tech, Brent Rauhut Engineering (BRE), and various Department of Transportation (DOT) laboratories. Some of this earlier knowledge was used in developing the ruggedness and precision and bias testing plans included in Chapters 3 and 4 of this document.

Table 2. State DOT/Other Laboratories Conducting Resilient Modulus Testing

State DOT/Other Laboratories	Test Protocol Followed
Alaska DOT	AASHTO T 307-99
Alabama DOT	AASHTO T 307-99
Arizona DOT/ASU Geotechnical Laboratory	NCHRP 1-28a
Cold Regions Research & Engineering Laboratory (CRREL)	AASHTO T 307-99
Colorado DOT	AASHTO T 307-99
Florida DOT	AASHTO T 307-99
Georgia DOT	AASHTO T 307-99
Iowa DOT	NCHRP 1-28a/AASHTO T307-99
Idaho Transportation Department Laboratory	AASHTO T 307-99
Indiana DOT	AASHTO T 307-99
Kansas DOT	AASHTO T 307-99
Kentucky DOT/University of Kentucky Transportation Center	AASHTO T 307-99
Louisiana DOT/Louisiana Transportation Research Center (LTRC) Laboratory	AASHTO T 307-99
Manitoba Provence, Canada	NCHRP 1-28a
Michigan DOT	AASHTO T 307-99
Minnesota DOT	NCHRP 1-28a
Missouri DOT	AASHTO T 307-99
Mississippi DOT	AASHTO T 307-99
Montana DOT	AASHTO T 307-99
Nebraska DOT/University of Nebraska-Lincoln (UNL) Geomaterials Laboratory	AASHTO T 307-99
North Dakota DOT	NCHRP 1-28a
New Hampshire DOT	AASHTO TP46-94
New Jersey DOT/Rutgers University Asphalt/Pavement Laboratory (RAPL)	AASHTO TP46-94
OH DOT/ORITE Pavement Material Test Laboratory	AASHTO T-274
Oklahoma DOT	AASHTO T 307-99
Rhode Island DOT	AASHTO T 307-99
Tennessee DOT	AASHTO T 307-99
Texas DOT	AASHTO T 307-99
Virginia DOT	AASHTO T 307-99
Wisconsin DOT	AASHTO T 307-99

ASTM Committee D-18.09, Dynamic Properties of Soils, attempted to standardize the test method in the 1980's. At that time consensus could not be reached in different areas, including: specimen preparation, on versus off-specimen deformation measurements, stress states (vertical stress and confinement), as well as type of load application (haversine versus square load pulses). As part of the process to try and standardize the test, there were multiple studies conducted which are not reported in the literature. Many of these were completed in the 1980's and are listed below:

- As part of the ASTM process: Von Quintus measured the resilient modulus on fine and coarse-grained soils using on-specimen compared to off-specimen (outside the triaxial chamber) linear variable differential transducers (LVDTs) for measuring vertical deformations. End effects were obvious and significantly increased the variability in the test results of triplicate samples, in comparison to on-specimen LVDTs. Thus, many of the initial test protocols used on-specimen LVDTs. It was also observed that the variability in test results increased with larger size aggregate particles using on-specimens LVDTs, if not scalped from the sample. Scalping the larger size particles was a debatable issue because the test specimen did not represent the actual gradation.

- The magnitude of the repeated vertical loads was also investigated to prevent damage to an "undisturbed" specimen by simply measuring the compressive strength of the material at different confinements. Establishing an upper limit on the vertical load certainly reduced the variability in results. This step, however, was not included in many of the earlier drafts of the test method because of the time added to the test procedure. Von Quintus used the compressive strength in the 1970's and 1980's, which was removed from some of the latter versions prepared through ASTM. A latter portion of this section discusses the stress state used in more detail (subsection 2.3, Stress States).

- Sample preparation and handling procedures were found to have an effect on the test results and on the variability of test results. Many of the earlier procedures were very descriptive in terms of sample preparation and handling, especially in placing the membranes on the test specimen. This was another area where consensus was difficult to reach because just about every organization involved in resilient modulus testing had a different method for making the test specimens.

- Specimen size was another area of dispute or debate. Both BRE (Von Quintus) and the Corp of Engineers made different size test specimens for both fine and coarse-grained materials/soils. As the ratio of the specimen diameter to aggregate size dropped below 3, variability in the test results increased. Thus, some of the earlier test methods simply required a minimum specimen diameter to aggregate diameter ratio of 4. One of the debates on this topic was how to define the aggregate diameter within the test specimen, which was not resolved. Another aggregate-sample size issue was scalping or removing the larger aggregate particles. Consensus was not reached on this issue, although most users agreed larger particles in a sample need to be removed.

- Other factors studied included:
    - Drained versus undrained conditions.

- Load cell location – outside versus inside the triaxial chamber regarding potential restrictions or friction on the loading rod.
- Number of conditioning cycles required to produce stable results.

Zapata et al. (2009) recommended the use of a modified triaxial system to measure/control soil suction during resilient modulus testing. The reason for measuring soil suction was related to the testing difficulties associated with constraints of existing equipment and limitations of the available protocols. Soil suction was found to be an important factor in measuring resilient modulus, but is excluded from AASHTO T 307 and NCHRP 1-28a.

Cary and Zapata (2011) studied the resilient modulus measured on unsaturated unbound materials, and recommended modifications to the stress state conditions of the protocol which were considered necessary due to the axis-translation during the test when measuring soil suction. Although the NCHRP 1-28a protocol defines the contact stress to be 20 percent of the magnitude of the total confining pressure applied in a given sequence, the researchers concluded a contact stress equal to 20 percent of the *net* confining pressure was enough for unsaturated resilient modulus testing conditions. This recommendation was made to ensure that the contact stress does not increase significantly when there was an increased confining pressure due to soil suction control/measurement during the test. Modifications were recommended for incorporation in the current NCHRP 1-28a loading procedure to expand the protocol for unsaturated soil conditions, as shown in Table 3.

Table 3. Changes to the Resilient Modulus Loading Procedures for Unsaturated Soils (Source: Cary and Zapata (2011))

Total Stress Approach	Net Stress Approach for Unsaturated Soils
ψ_m is not applied	ψ_m is applied
$\sigma_{confining} = \sigma_3 = \sigma_{net-confining}$	$\sigma_{confining} = \sigma_3 = \sigma_{net-confining} + u_a$
$\sigma_{contact} = 0.2\,\sigma_{confining}$	$\sigma_{contact} = 0.2\sigma_{net-confining}$
$\sigma_d = \sigma_{max}$	$\sigma_d = \sigma_{max}$
$\sigma_{cyclic} = \sigma_{max} - \sigma_{contact}$	$\sigma_{cyclic} = \sigma_{max} - \sigma_{contact}$

Yau and Von Quintus conducted a study of the resilient modulus test results included in the LTPP database (Yau and Von Quintus, 2002). The purpose of this study was two-fold: (1) identify problems or anomalies in the test results; and (2) determine the materials physical properties that have a significant effect on the measured results, and thus used to estimate resilient modulus. Yau and Von Quintus determined the physical properties statistically affecting resilient modulus were soil type dependent. Obviously, the physical properties included in the study were confined to those included in the LTPP database. Multiple regression equations were developed, but the authors concluded the standard errors of the regression equations were too large and recommended the resilient modulus be measured in the laboratory. These results, however, can be used to identify the soil physical properties that have the greatest impact on resilient modulus.

2.2 Laboratory Data Acquisition Systems

2.2.1 Test Equipment

There are several test systems available on the market today. The so-called high-end equipment (MTS, Interlaken and Instron) is about double the cost of the lower-end equipment (GCTS, GeoComp and IBC). This statement does not imply the high-end equipment is twice as accurate as the lower-end equipment. That observation on accuracy has yet to be determined.

A survey conducted as part of the Resilient Modulus Pool Fund Study (PFS) revealed the participating state agencies only possess the high-end systems, which are listed in Table 4. Many universities and other commercial testing laboratories have measured resilient modulus on a range of soils and aggregate base materials. The specific test system, test procedure, and/or specimen preparation procedure, however, are not always adequately described in the literature. To evaluate the variability or repeatability of the resilient modulus test between different studies, details on the test system and procedure used by an agency or organization must be known to identify factors that can result in different modulus values. Table 5 lists other laboratories not included in the PFS and their test system used for measuring resilient modulus.

2.2.2 Triaxial Cells

There are numerous manufactures of triaxial cells (Trautwein, SoilTest, Humboldt, Karol-Warner, and Durham-Geo to name a few). Regardless of the triaxial cell utilized, the seal friction and drag forces should be measured and held at a minimum criteria (Boudreau and Wang, 2003). Increasing seal friction and drag forces will result in higher resilient modulus and increase variability.

2.2.3 Test Protocols

There are two test procedures currently available that most agencies follow in measuring the resilient modulus. Table 2 listed the test protocol being followed by different agencies. As listed, most laboratories use AASHTO T 307, but there are deviations from that test standard. The Minnesota DOT uses NCHRP 1-28a, while a Mississippi DOT consultant uses a modified NCHRP 1-28a procedure. While both procedures are very similar, the NCHRP 1-28a procedure is very particular with the requirement of internal instrumentation. It also requires more stress sequences and orders these sequences from lowest to highest in terms of stress ratio (vertical to horizontal stress).

A provision included in AASHTO T-307 (paragraph 8.3.2.1) recognizes certain influences acting on the triaxial chamber's loading rod. This provision requires that adjustments be made to account for both the static weight of the rod and deformation measurement system, as well as the uplift force on the load rod due to the confining pressure applied within the triaxial cell. This provision is not restated anywhere in the standard requiring that this adjustment be accounted for again during data reduction and reporting. Further, neither of the references addresses the influences of seal drag forces, alignment of top and bottom platens, or system compliance.

While it is desirable for all test laboratories to use the same procedure and equipment, this is a management policy decision. As such, any systematic difference in results between different procedures and test systems must be explained or understood and statistically analyzed as a potential bias; similar to the bias between the laboratory and field-derived modulus values discussed in subsection 2.1.

Table 4. PFS Participating Laboratories and Their Resilient Modulus Test System

Test Laboratory	Test System
Alabama Department of Transportation [B]	Instron
Boudreau Engineering, Inc. [A]	Instron
Burns Cooley Dennis, Inc. (Mississippi)	Interlaken
Colorado Department of Transportation	IPC
Florida Department of Transportation	Instron
Minnesota Department of Transportation	Interlaken
Missouri Department of Transportation	
Texas Department of Transportation	MTS
Virginia Department of Transportation [A]	Instron
Wyoming Department of Transportation	Interlaken

Note A: These labs have successfully been evaluated using RD-02-034 guidelines
Note B: This lab has been evaluated using RD-02-034 guidelines with inconclusive results.

Table 5. Other Laboratories (Agencies and Commercial Laboratories)

Test Laboratory	Test System [A]
Braun InterTec (consultant-Minnesota)	MTS
Brigham Young University (Dr. Spencer Guthrie)	IPC
Idaho Transportation Department	GeoComp
Indiana Department of Transportation	GeoComp
Kansas Department of Transportation	Interlaken
Maryland State Highway Agency	GeoComp
North Carolina Department of Transportation	Instron
Oklahoma Department of Transportation	MTS
Pennsylvania Department of Transportation	GeoComp
Raba-Kistner (consultant-Texas)	GCTS
Southerm Polytech (Marietta GA)	Interlaken
Southerm Polytech (Marietta GA)	IPC
Terracon (consultant-Oklahoma)	IPC

Note A: Although several states reportedly operate the IBC equipment, it is uncertain whether any of them use the system for material other than asphalt concrete.

2.3 Stress State

As noted in the beginning of this section, the laboratory stress state (vertical stress and confining pressure) has been debated over the years. Both AASHTO T 307 and NCHRP 1-28a procedures use repeated axial stresses and confining pressures that are well beyond the stress-state the material (unbound aggregate base and embankment layers) will feel after construction. One of the issues or debates during the ASTM standardization work in the 1980's was over the magnitude of the cyclic or repeated axial stress. The controversy was focused on the higher axial stresses: specifically, should they be lowered?

More recently, there has been little debate on reducing the magnitude of the cyclic axial stress. A reason for using higher axial stresses and confinement in the laboratory than what the soil actually feels is to reliably determine the coefficients of the material constitutive equation. In the author's opinion, this is an invalid reason for using very high axial stresses. More importantly, constitutive equations are not used by most agencies in estimating the resilient modulus to be used for pavement design purposes. Table 2 listed some of the agencies that use a resilient modulus test protocol, but few agencies actually use the test results (resilient modulus values) in day-to-day practice for pavement design.

For thick flexible and rigid pavement layers, the overburden or at-rest stress state controls or defines the stress state in an unbound aggregate base layer or embankment layer. In other words, the increase in stress state (vertical stress and confinement) from truck loads is minimal, especially for higher volume pavements that are much thicker. Thus, determining the stress sensitivity of the unbound layers is not that important.

An exception to that statement is for low volume or thin surfaced pavements (for example, surface treatments or 2 inches (50.8 mm) of hot mix asphalt [HMA] over an unbound aggregate base layer). Some of the earlier unpublished findings suggest a lower and narrower range of stress states so the material remains undamaged or undisturbed throughout the test to reduce test variability and to reduce the amount of time required to run the test. Thus, the stress states to be used should be selected based on application of the results, rather than a pre-specified set of values; at least in the author's opinion. A latter section in Chapter 5 provides examples of the at-rest stress condition and load-related stresses in determining the resilient modulus to be used for pavement design.

2.4 Variability of Resilient Modulus Test Results

As previously stated by Boudreau, numerous testing laboratories across the nation have overcome the hurdle of *too difficult*, having committed resources to acquire test equipment and train staff on how to operate and maintain such equipment. Many have already found the test to be much easier to perform than the CBR test in terms of amount of material to process, effort required to fabricate test specimen, and time required to perform a test. While this is certainly a step in the right direction, the issue of *not repeatable* and *not reliable* still persists.

Repeatable suggests how well a single operator can repeat a test (i.e., if a test specimen is tested over and over again, does the operator get the same result every time?). In order to perform a resilient modulus test, a test specimen is subjected to cyclic loads which result in elastic (resilient) and plastic (permanent) strains. It is for this reason that a single test specimen should not be tested repeatedly. In order to evaluate how well a single operator can repeat a test, several test specimens consisting of the same material remolded the same way to the same level of density and moisture content are required. Triplicate test specimens have been used in many studies.

The Strategic Highway Research Program (SHRP) initiated a round-robin test program during the beginning of LTPP, in which Boudreau (one of the authors of this report) participated. This program included testing the same synthetic prepared samples to evaluate differences caused by the test system and/or equipment. Test specimen preparation was originally excluded from this part of the round-robin test program. Differences were observed between laboratories and those differences were believed to be significant. A significant bias between the measured values and the assumed elastic modulus value for the synthetic specimen was also found. Extensive work and evaluations were completed to try and explain the bias and differences in resilient modulus values but without resolution.

Yau and Von Quintus, however, did not detect a statistical bias between the different laboratories used to measure resilient modulus over a range of materials encountered at the LTPP sites (2002). It should be understood the standard deviation of modulus values measured for the same soil type and by the same laboratory was very large. Any bias between the laboratories could be easily hidden in the noise of the data.

A study was initiated in 2001 which included testing 8 replicated test specimens on an Alabama A-4 soil and 19 replicated test specimens on a Georgia A-4 soil (Boudreau, 2003). It was concluded that the test results were considered repeatable (coefficient of variation [COV] being less than 4.5 percent), but were limited to the fact that only A-4 soils were tested. More importantly, the test specimens were prepared using one remolding method, and tested by one operator using a single triaxial chamber for one test system. Experience of the authors has shown variation from any of these factors can lead to variations in results, even if specimens are prepared within tight tolerances of density and moisture content.

Although not published, Boudreau conducted a miniature round-robin test in 2007 with 6 state highway laboratories, each using an Instron test system to conduct the AASHTO T 307 test. The test program included 3 soil types and 4 replicates each (Florida sand, Alabama silt, and Kentucky clay). Results showed an overall COV ranging from 10 to 20 percent, depending on soil type and stress level targeted. This showed good ***within-lab repeatability*** as well as good ***between-lab reproducibility***. (Appendix A includes a tabulation of the results).

It is important to note the COV reported from some of the earlier unpublished work through ASTM also varied between 5 to 15 percent for a single laboratory, single operator and was generally less than 25 percent for multiple laboratories, multiple operators. The lower range of COV values were for fine-grained, low plasticity soils, while the higher range of COV values were for coarse-grained aggregate base materials – all reconstituted and recompacted in the laboratory. In other words, no undisturbed test specimens extracted from Shelby tubes were included in the test program.

2.5 Other Modulus Measurement Methodologies

Prior to the Yau and Von Quintus 2002 LTPP study, Von Quintus and Killingsworth compared resilient modulus included in the LTPP database and backcalculated elastic layer modulus values from falling weight deflectometer (FWD) deflection basins measured in the approach and leave ends of the LTPP test sections. One objective of this study was to confirm or reject the applicability of the c-factor (ratio of laboratory-derived and field-derived or backcalculated modulus values) used in the 1993 AASHTO Design Guide. The authors found there was a difference between the lab and field-derived modulus values, and that difference was pavement structure dependent and suggested continued use of the c-factor. Thus, the c-factor is also included and referred to in the AASHTO MEPDG Manual of Practice (2008).

More importantly, the Von Quintus and Killingsworth study supports two important points: (1) laboratory resilient modulus test conditions and test results do not simulate the soils/materials in place measured response from deflection basins, and (2) the test protocol used in the LTPP program is reasonable because there is a correlation between the laboratory and field-derived modulus values.

A multiple linear regression model was developed in a study conducted by Attoh-Okine and Wiredu (2003) to estimate the resilient modulus of construction materials from basic soil tests. The multiple-linear regression equation developed in this study included the factors that were found to have a significant effect on the CBR value, and hence resilient modulus. Attoh-Okine and Wiredu recommended use of the regression equation in estimating resilient modulus for input levels 2 and 3 of the MEPDG procedure for both new and rehabilitated pavements. The study indicated that the CBR values are sensitive to the compaction moisture content of the sample. More importantly, it was discovered that the moisture content on the wet-side of optimum is associated with reduced shear strengths, as exhibited by the soaked CBR value. The use of low CBR values representing soaked conditions to determine the resilient modulus, however, can be overly conservative and lead to thicker pavements.

Qian et al. (2011) conducted a study to determine the resilient modulus of a weak subgrade, using four cyclic plate loading tests. Their study drew the following conclusions: (1) the vertical permanent deformation of the subgrade increased with the number of cyclic loading, (2) the calculated resilient modulus of the subgrade using the elastic solution first increased, and then decreased and reached a stable value after a certain number of cyclic loading, and (3) the average resilient modulus of the weak subgrade determined form the cyclic plate loading tests in this study was 29.4 Mpa (4,300 psi), which is close to that calculated using three correlations with the CBR value of the subgrade.

Dennis and Bennett (2006) conducted a study that used spectral analysis of surface waves (SASW) testing method for determination of in-situ properties of flexible pavement in the state of Arkansas. This method was developed as an alternative to AASHTO T-307 that requires the use of expensive laboratory equipment, is time consuming, and must be run by highly skilled technicians. SASW testing method used in this study predicted values of resilient modulus for the base course and subgrade soils that were within reasonable ranges for the levels of strain associated with SASW testing. SASW testing, however, estimated a resilient modulus approximately twice that of FWD testing at sites that had stiff subgrade soils (as expected), but estimated resilient modulus values very nearly equal to or slightly less than those estimated by FWD at sites that had a softer subgrade material. It was observed that the resilient moduli of the subgrade soils estimated by SASW testing agreed well with the condition of the subgrade soil observed at the time of testing as evaluated by considering Standard Penetration Test N-values, Atterberg limits, gradation, and in-situ moisture content information. SASW testing had some gaps and the researchers suggested that the SASW method be further refined before implementation.

Von Quintus, et al used the Portable Seismic Pavement Analyzer (PSPA) and other devices to estimate the in place resilient modulus for use in accepting and controlling unbound materials for flexible pavement construction projects (Von Quintus et al., 2009). Other devices used in the study included the Geogauge, light weight deflectometer (LWD; 3 devices were used), dynamic cone penetrometer (DCP), and the FWD. Laboratory resilient modulus tests were performed in accordance with AASHTO T 307 on all of the materials included in the study. The authors found the Geogauge and PSPA were the two devices that had a better correlation to the laboratory measured values. More importantly, the authors reported the elastic modulus measured with the in place methods overestimated the laboratory measured values, similar to Dennis and Bennett. The reported correlations by Von Quintus et al (2009), however, were found to be dependent on the resilient modulus itself; the higher the resilient modulus the greater the difference between in place and laboratory measured values.

In summary, many of the published studies have reported correlations but significant bias between field-measured elastic modulus and resilient modulus measured in the laboratory at equivalent stress states.

2.6 Application of Resilient Modulus Test Results

The 1993 AASHTO Design Gide includes a procedure for determining the effective resilient modulus from resilient modulus values representative of each season or month. Von Quintus and Killingsworth followed that basic concept but developed relative damage values that were based on fatigue cracking and rutting of flexible pavements using the LTPP test sections (1995). The following relative damage relationships were recommended for use with ME-based pavement design methods.

$$u_{f(Base)} = 1885(M_R)^{-0.721} \tag{1}$$

$$u_{f(Subgrade)} = 4.022x10^7 (M_R)^{-1.962} \tag{2}$$

Where:

u_f = Relative damage values for the aggregate base and/or subgrade soil.
M_R = Laboratory measured resilient modulus for a particular season/month, psi.
k = Number of seasons or months.

The equivalent or effective resilient modulus for a particular material/soil, site, and climate is then determined using equation 3.

$$M_R = \frac{\sum_{i=1}^{k} (M_R)_i (u_f)_i}{\sum_{i=1}^{k} (u_f)_i} \tag{3}$$

The difficulty in developing these relative damage values was that only the Seasonal Monitoring Pavement (SMP) sites had deflection basins measured on a monthly basis and only one round or time was used to sample the soils/materials for laboratory resilient modulus testing. Assumptions were made regarding variations in seasonal modulus values in different climates and pavement structures, which were based on the SMP sites. Some agencies simply take the average of all resilient modulus values measured in accordance with AASHTO T 307. For stress-hardening soils, this average value overestimates the in place modulus value, while for stress-softening soils the average value underestimates the in place value.

A study by Janoo et al. (1999) focused on the resilient tests conducted on five subgrade soils found in New Hampshire. The objective of these tests was to determine the effective resilient modulus of the New Hampshire soils for use in the AASHTO design procedure for design and evaluation of pavement structures. The study also provided results from laboratory testing to determine the resilient modulus of the various subgrade soils as a function of temperature. The resilient modulus testing temperatures were +20°, +0.5°, –0.5°, –2°, –5°, and –10° C to evaluate freeze-thaw cycles on the resilient modulus. Most of the resilient modulus tests for this study were conducted at optimum moisture content and a limited number were conducted at the saturated water content to determine the effect of moisture content. The study did not provide a recommended procedure for estimating the design resilient modulus value under different climate conditions on a seasonal basis. Resilient modulus test values were averaged for each temperature or month and the corresponding relative damage values were calculated using the following equation initially recommended for use with the AASHTO 1993 Design Guide by Von Quintus and Killingsworth (refer to equation 1):

$$u_f = 1.18 * 10^8 * Mr^{-2.32} \tag{4}$$

The relative damage values were then averaged and used to calculate the effective resilient modulus using the following equation:

$$M_{eff} = (u_f/1.18 * 10^8)^{-\frac{1}{2.32}} \tag{5}$$

Maher et al. (2000), in their study on the resilient modulus properties of New Jersey Subgrade soils, also used the effective roadbed soil resilient modulus equation provided above to determine the design value for subgrade modulus.

Ping et al. conducted a study to evaluate the effects of base clearance on the resilient modulus of pavement subgrades. A test-pit facility, which simulates the subgrade and base components of a flexible pavement system and permits testing full-scale base–subbase sections constructed on layers of standard subgrade sand under different controlled moisture conditions, was used for this study. The tests were carried out on 10 subgrade materials in Florida by means of the test-pit facility and the laboratory triaxial test under the same density and moisture conditions.

The confining pressure at subgrade layers in actual field conditions was found to be about 2.0 psi (13.8 kPa). Some researcher's state 2.0 psi represents the average stress and pressure that occurs in the subgrade under traffic loading and surcharge. This opinion in conjunction with the recommendations from previous studies was used to determine the selection of the resilient modulus of roadbed soils for pavement design. The resilient modulus value from laboratory testing obtained at a deviator stress of 5.0 psi (34.5 kPa) under the confining pressure 2.0 psi was considered representative of the in-situ subgrade modulus. Similarly, another study by Watson et al on comparing the Georgia DOT Design Procedure with the MEPDG, a cyclic deviator stress of 5.5 psi (37.9 kPa) and a confining stress of 2 psi was used to determine the subgrade resilient modulus values (2009).

Ceylan et al. (2009) conducted a study on characterization of unbound materials (soils and aggregate bases) for the MEPDG. A total of three soil types commonly found and used in Iowa, categorized as select, class 10 or suitable soil, and unsuitable soil, were sampled and tested for this study. In this study, all soils investigated were categorized as Type 2 and the one type of aggregate considered was categorized as Type 1. Two data sets used for the calculation of average resilient modulus in this study are as follows:

1. Resilient modulus results of the standard 15 stress combinations without zero-confining stress conditions (i.e., standard 15 load sequences according to AASHTO T 307).
2. Resilient modulus results of the 20 stress combination with zero-confining stress conditions (i.e., standard 15 load sequences followed by 5 load sequences under zero-confining stress conditions).

For MEPDG input level 3, resilient modulus results without zero confining stress conditions (standard test procedure) for three types of soil at optimum moisture content conditions and one type of aggregate with 10 percent moisture condition were considered.

Gupta et al. (2007) in their study on development of a pavement design method based on the principles of unsaturated soil mechanics consistent with MnPAVE design framework, used design resilient modulus values based on NCHRP 1-28a recommendations for typical pavement stress conditions. As recommended by NCHRP 1-28a, the resilient modulus value was calculated at a bulk stress (θ_B) of 12 psi (83 kPa) and octahedral shear stress (T_{oct}) of 2.8 psi (19 kPa).

Hossain et al. conducted a study to evaluate the resilient response of unbound aggregates toward implementation of the MEPDG in Oklahoma. In this study, resilient modulus data and other routine properties (gradation, LA Abrasion loss, standard Proctor, and unconfined compressive strength) of 105 samples of two different types of aggregate (limestone and sandstone) were analyzed. The researchers used four nonlinear regression models in this study. The default MEPDG resilient modulus values (input level 3) were calculated using the average material constants (regression constants) obtained from regression modeling.

Khoury et al. (2010) conducted a study on stability and permeability of proposed aggregate bases in Oklahoma. The study focused on the effect of gradation and compaction energy on resilient modulus and permeability (k) of aggregates from three commonly used sources in Oklahoma, namely Anchor Stone, Dolese and Martin Marietta. The design resilient modulus values for this study were calculated at a deviator stress of 6.0 psi (41.4 kPa) and a confining pressure of 4.0 psi (27.6 kPa).

Hossain (2008) conducted a study on characterization of subgrade resilient modulus for Virginia soils and its correlation with the results of other soil tests. The study involved testing more than 100 soils sampled across Virginia representing every physiographic region for resilient modulus, soil index properties, standard Proctor, and CBR testing. Computations were carried out for resilient modulus values and regression coefficients (*k*-values) of constitutive models for resilient modulus for typical Virginia soils. The MEPDG input level 3 design values of resilient modulus for Virginia soils were determined using average regression coefficients at confining and deviator stresses of 2 and 6 psi (13.8 and 41.4 kPa), respectively.

Titi and English (2011) conducted a study on determination of resilient modulus values for typical plastic soils in Wisconsin. In this study, a laboratory testing program was conducted on 13 fine-grained soil samples to evaluate their physical and compaction properties. Titi and English conducted several laboratory tests to determine physical properties such as grain size distribution (hydrometer and sieve analysis), Atterberg limits (liquid limit, *LL* and plastic limit, *PL*), specific gravity (*Gs*), moisture unit weight relationship, and resilient modulus for each soil. To determine the resilient modulus model parameters k1, k2 and k3, statistical analysis based on multiple linear regressions was performed. To establish MEPDG input level 3 values, further analyses were conducted for all soils together and for each of the soil categories according to the AASHTO soil classification system: A-4, A-6, and A-7 (A-7-5 and A-7-6). The resilient modulus values corresponding to the average minus one and two standard deviations (μ-σ and μ-2σ) were then calculated. Based on the analysis, average resilient modulus values minus one standard deviation (μ-σ) on the wet category and a confining pressure of 4 psi (27.6 kPa) was recommended for use as a representative value for the specific soil type.

As stated previously, Von Quintus and Killingsworth conducted an FHWA sponsored study to compare the use of backcalculated elastic modulus values from FWD deflection basin data and laboratory measured resilient modulus values for rehabilitation design and pavement evaluation (1995). This study included soils and aggregate base materials in the LTPP program. The most important observation from this study was that the overburden or at-rest stress condition controls determination of the in place resilient modulus for most of the LTPP sites. Another observation was the in place stress state was significantly less than the values used to measure the resilient modulus in the laboratory. In addition, laboratory resilient modulus values were found to be significantly less than but correlated to the backcalculated elastic modulus values. The correlation or adjustment to equate the two modulus values was reported to be pavement structure dependent. The c-factor included in the 1993 AASHTO Design Guide for conventional flexible pavements was confirmed to be 0.35. The other important observation from the study was the coefficients of the laboratory resilient modulus regression equation were dependent on the type of soil and physical properties of the soil.

Rusell and Hossain (2000) analyzed subgrade elastic modulus values backcalculated from FWD test results, on nine projects located in Kansas. This study was sponsored by the Kansas DOT. They also reviewed the laboratory resilient modulus test results for the fine-grained soils on the referenced projects. For the 2000 project, four samples were taken from each project of which three samples were tested. The resilient modulus test data from the resilient modulus testing was used to create Quattro Pro graphical plots of the resilient modulus versus deviator stress for each soil sample at each confining pressure (3.0 and 5.0 psi [21 and 34 kPa], respectively). For three samples tested, six different plots were generated, and a breakpoint value was calculated for each individual plot in order to determine the design resilient modulus for that soil sample. For all sample design resilient moduli determined, the average was used to determine the design resilient modulus. It was noted that this design practice appeared to be sound for soil samples that clearly behaved like the typical AASHTO soils. However, for soil samples that displayed non-bilinear behavior, the design soil resilient modulus value for the project was subjectively estimated.

2.7 Summary of Previous Studies

Table 2 listed the agencies/organizations conducting laboratory resilient modulus tests and the test protocols being followed, while Appendix A presents a tabular summary of the material types, test protocols, and compaction methods used in some of the documented studies reviewed. An important observation from this review is that most agencies are determining the resilient modulus at higher confinements and deviators stresses than the in place stress state than the soil actually feels. More importantly, few studies have focused on determining the design resilient modulus values for crushed aggregate base materials.

The following provides a summary of the more important findings relative to determining the precision and bias of the resilient modulus test methods, as well as how sensitive distress predictions are to resilient modulus and how the test results are used to determine the design resilient modulus.

<u>Test Systems and Protocols:</u>

- Most studies focused on resilient modulus testing were found to follow the AASHTO T 307-99 test protocol, compared to the more recent NCHRP 1-28a procedure. A list of state DOT, University, and other laboratories was summarized in Table 2 and Appendix A. Systematic differences, if any, between these two test protocols have not been adequately defined. As such, AASHTO T 307 and NCHRP 1-28a need to be evaluated for potential bias and whether that bias is significant in terms of pavement design for flexible and rigid pavements.

- There are several test systems available on the market today. The so-called high-end equipment (MTS, Interlaken and Instron) is about double the cost of the lower-end equipment (GCTS, GeoComp and IBC). This statement does not imply the high-end equipment is twice as accurate as the lower-end equipment. Few studies have focused on determining if there is a bias between these different systems, as well as defining the precision of the test system.

<u>Testing Conditions:</u>

- The end effects for off-specimen LVDTs were obvious and significantly increased the variability in the test results of triplicate samples, in comparison to on-specimen LVDTs. Different studies, however, have reported opposite results in comparing the resilient modulus values between on-specimen and off-specimen displacement measurements for calculating resilient modulus. As such, both of these conditions should be included and evaluated as part of a ruggedness test program before measuring the precision and bias of the test methods.

- Both AASHTO T 307 and NCHRP 1-28a procedures use repeated axial stresses and confining pressures that are well beyond the stress-state the material (unbound aggregate base and embankment layers) will feel after construction. The precision and bias of the two methods should be focused on the lower axial stress. More importantly, constitutive equations are not used by most agencies in estimating the resilient modulus to be used for pavement design purposes. Thus, determining or evaluating the stress sensitivity of the unbound layers is not that important.

<u>Test Specimen Preparation:</u>

- Some of the earlier test methods simply required a minimum specimen diameter to aggregate diameter ratio of 4 to reduce test variability.

- Few if any of the studies identified allowable deviations in the water content-density or degree of saturation which does have a significant effect on resilient modulus. It was found that all soils exhibited a decrease in resilient modulus with an increase in saturation, but the magnitude of the decrease in resilient modulus was found to depend on the soil type. It was observed and reported a 3 to 5 percent increase in moisture content from optimum conditions can result in a 50 to 70 percent reduction in resilient modulus. The drying of the test specimens can also result in a significant increase in resilient modulus, in some cases ten-fold. Thus, moisture content and dry density are important in measuring the resilient modulus. The allowable deviation stated in AASHTO T 307 and NCHRP 1-28a procedures should be checked as part of a ruggedness test program to ensure differences between laboratories and test specimens are not a result of too large differences in the volumetric properties of the specimen.

<u>Test Specimen Replication:</u>

- The COV for multiple laboratories and operators varies between 15 to 25 percent, and for single operators that value is around 5 to 10 percent. These values were used in setting up a ruggedness or precision and bias test program to determine the number of samples at specific confidence intervals.

<u>Impact of Soil Physical Properties on Resilient Modulus:</u>

- The studies reviewed indicated that the resilient modulus values were impacted by moisture content, soil suction, Atterberg limits, gradation, source lithology, stress-strain levels, degree of saturation, seasonal variation, aggregate angularity, and surface texture.

- The resilient modulus values due to wetting are lower compared to the corresponding values after drying. It was also found that the initial compaction moisture content followed by drying or wetting affect the hysterics loop of both soil water characteristics curve (SWCC) and the resilient modulus-moisture variation curve.

- Multiple regression equations have been developed that can be used to estimate the importance and effect of varying physical properties of the soil on the resilient modulus for planning different experimental programs and sampling matrices.

<u>Impact of Resilient Modulus on Pavement Service Life or Predicted Distress:</u>

- The precision of the resilient modulus test methods estimated from existing literature was found to be sufficient as to not result in significant deviations in the pavement structure. Appendix B includes a summary of pavement distresses to changes in resilient modulus that has been documented in the literature. The potential bias between the different test systems and procedures and resulting pavement designs, however, has not been clearly defined or estimated.

- The review of published papers and reports indicate resilient modulus of unbound materials and soils have an impact on pavement performance (see Appendix B). The following is a general summary of the impact levels of the subgrade resilient modulus on pavement performance indicators:
    - HMA
        - Longitudinal Cracking – Moderate to High Impact
        - Alligator Cracking – Low to Moderate Impact
        - Transverse Cracking – None to Low Impact
        - Rutting – Low to Moderate Impact
        - IRI – Variable
    - JPCP
        - Faulting – Low Impact
        - Transverse Cracking – Moderate to High Impact
        - IRI – None to Low Impact

<u>Determination of the Design Resilient Modulus for Use in Pavement Design:</u>

- Many agencies have published default design resilient moduli that are used within their pavement design procedure, but little explanation is provided on how the design default values were derived. In reviewing actual design projects, more than a few agencies simply use the average resilient modulus across all stress states used in AASHTO T 307. Most agencies, however, do consider stress state (confining pressure and cyclic axial stress) in determining the design resilient modulus. The stress state used and how it is determined varies from agency to agency. Few details are provided in terms of where and under what conditions (seasons, axle load, etc.) the stresses are calculated. Even the MEPDG Manual of Practice provides little guidance on calculating the stress states. References documenting the step by step process, however, are included in the Manual of Practice. Chapter 5 of this report provides a detailed procedure for determining the resilient modulus, as well as, some examples demonstrating use of that procedure.

CHAPTER 3 RUGGEDNESS TESTING

Multiple resilient modulus test programs have been completed to develop and improve the AASHTO T 307 and NCHRP 1-28a test methods. Some of this testing was completed within the SHRP program prior to production-level testing of the unbound materials and soils. A lot of testing, however, has been completed post-SHRP to determine the effect of test parameters and equipment over time. Some ruggedness testing has been completed in evaluating the two test methods (AASHTO T 307 and NCHRP 1-28a), but by one laboratory and/or in a fragmented manner.

Boudreau conducted a round robin test program that included several State agency laboratories, with promising results. This test program, although defined as a round robin test program, also looked into the effect of compaction method, and to a lesser extent, test specimen size. Results from that study, however, were not formally published.

Burns Cooley Dennis (BCD) conducted a limited internal study to evaluate selected specimen preparation parameters relative to the variability of the resilient modulus test (James, et al., 2010). This study looked at specimen condition in terms of density, water content, and mellowing time on resilient modulus. To accurately define the precision and bias of AASTHO T 307 and NCHRP 1-28a, the effect of selected key factors needs to be determined to define the allowable deviations which do not increase the variability of test results for each method.

Many variables affect resilient modulus such as the testing system, compaction method, water content, dry density, deformation measuring system, triaxial cell, and testing procedure. The Synthesis – Literature Review Summary Report prepared as part of this project identified the parameters and soil properties that have a significant effect on resilient modulus.[1] Chapter 2 and Appendix A are a summary of the Synthesis. This chapter of the report presents the test results and data analyses of selected factors associated with resilient modulus testing.

3.1 Ruggedness Testing Plan

Two laboratories agreed to participate in the ruggedness testing for the resilient modulus test: BCD located in Jackson, Mississippi and Boudreau Engineering located outside of Atlanta, Georgia. One of these laboratories was defined as the alpha laboratory and the other defined as the beta laboratory. Other pool fund study laboratories were asked to participate, but the amount of testing was beyond their day to day time availability. Both laboratories were checked and evaluated using RD-02-034. Thus, the impact of only two laboratories participating in the ruggedness test program should not have a detrimental impact on the outcome and findings. Two materials selected for the ruggedness testing plan, which were (see Appendix C for the physical properties of these soils and base material):

1. A cohesionless aggregate base material sampled from a Georgia quarry and defined as a Granular Aggregate Base (GAB).
2. A highly plasticity cohesive A-7-6 clay sampled from the Mississippi river delta.

[1] The Synthesis – Literature Review Summary Report is an unpublished document and was used in setting up the ruggedness and precision testing plans.

Five factors were initially identified to be included and evaluated within the ruggedness testing plan: (1) location of the LVDTs, (2) seating or contact load, (3) curing time, (4) water content around the optimum value, and (5) compaction method. Two of these factors (location of LVDTs on the test specimen and compaction method) were dropped because the value obtained was considered minimal for the effort required. In addition, previous testing had already quantified the effect of these two factors (James, et al, 2010; and other unpublished test data by the authors). The remaining three factors are explained below and shown in Table 6.

1. Seating or contact load; three levels: 5, 10, and 20 percent of the confining pressure (low and high confinement with the upper deviator stress; use one set of the compaction method specimens for evaluating the effect of this factor). Two specimens for each soil type using on-specimen LVDTs.

2. Curing time, three levels: compaction and testing of test specimens 0, 24, and 48 hours after curing in a plastic bag (see Note 2 in Table 6). AASHTO T 307 and NCHRP 1-28a provide a minimum and maximum time to allow the moisture content of the soil to stabilize for fine and coarse-grained soils. This factor has been reported to be an issue when testing some soils within the range specified by both test methods. Thus, the curing time in a plastic bag (time between adding moisture and test specimen compaction) was included in the ruggedness test program to determine what time delay, if any, is needed.

3. Degree of saturation, four levels: -1.0, -0.5, +0.5. +1.0 percent from the optimum water content and maximum dry unit weight (2 specimens at each degree of saturation or a total of 16 specimens for each laboratory). Impact compaction was used to prepare test specimens.

Factors that required a change in the data acquisition system by the equipment manufacturer were excluded from the ruggedness test plan (see Note 1 in Table 6). As an example, three factors that required a change in the data acquisition for measuring resilient modulus included:

- Load application time, two levels: 0.1 and 0.2 seconds with a 1 second repeated load. Previous experience of the authors suggests load application time does not increase the variability of the test results or cause a bias in the test results between different laboratories.[2]
- Cyclic loading time; two levels: 0.5 and 1.0 seconds. Previous experience of the authors suggest cyclic loading time does not increase the variability of the test results or cause a bias in the test results between different testing laboratories.
- Contact load; three levels: 5, 10, and 20 percent of the confining pressure. This factor was left in the ruggedness test program (see Table 1), because the alpha laboratory was able to vary the contact load without having to make a change to the data acquisition or system controls.

[2] Different load application and cycling loading times were included in the earlier ASTM studies to standardize the resilient modulus test. Results from the testing to evaluate different factors were unpublished. The statements for the effect of those parameters are based on the experience of the authors that participated in this testing.

Table 6. Ruggedness Test Plan

<table>
<tr><th colspan="2" rowspan="2">Factor</th><th rowspan="2">Levels</th><th colspan="2">Number of Test Specimens</th><th rowspan="2">Specimen Total Number</th><th rowspan="2">Test Specimen Condition</th><th rowspan="2">Comments</th></tr>
<tr><th>Fine-Grained Soil</th><th>Coarse-Grained Soil</th></tr>
<tr><td rowspan="3">1</td><td rowspan="3">Contact Load; % of confinement (See Note 1)</td><td>5</td><td rowspan="3">2</td><td rowspan="3">2</td><td rowspan="3">4</td><td rowspan="3">Impact compaction at optimum conditions.</td><td rowspan="3">The same specimen can be used for each contact load level</td></tr>
<tr><td>10</td></tr>
<tr><td>20</td></tr>
<tr><td rowspan="3">2</td><td rowspan="3">Curing Time, hrs. (See Note 2)</td><td>0 or same day</td><td>2</td><td>2</td><td rowspan="3">12</td><td rowspan="3">Impact compaction wet side of optimum</td><td rowspan="3">Assumed water content on wet side is sufficient to define factor effects.</td></tr>
<tr><td>24 or next day</td><td>2</td><td>2</td></tr>
<tr><td>48 or two days later</td><td>2</td><td>2</td></tr>
<tr><td rowspan="4">3</td><td rowspan="4">Degree of Saturation, %</td><td>-1.0</td><td>2</td><td>2</td><td rowspan="4">16</td><td rowspan="4">Impact compaction</td><td rowspan="4">Use curing condition defined from Factor #3.</td></tr>
<tr><td>-0.5</td><td>2</td><td>2</td></tr>
<tr><td>+0.5</td><td>2</td><td>2</td></tr>
<tr><td>+1.0</td><td>2</td><td>2</td></tr>
<tr><td colspan="8">Note 1: The above factors or deviations selected were those factors that did not require a change in the data acquisition software that a manufacturer would have to make.
Note 2: Three times are listed for curing. Most agencies, but not all, compact and test the specimen one-day after adding water to the specimen which can vary between 16 to 30 hours. Rather than be specific (+ or – 2 hours for example) on compacting and testing the specimen using only two curing times, three are requested after adding water to the soil and placing in a plastic bag for comparing the time difference effect between different labs.</td></tr>
</table>

3.2 Experimental Hypothesis and Data Analysis

The ruggedness test plan was designed to answer specific questions regarding three factors: does a change in the factor from the standard required in the test protocol result in a significant bias or difference in resilient modulus? The null hypothesis for this part of the experimental plan is as follows:

- The change in the factor does not significantly affect the resilient modulus of the test specimen.

The hypothesis was evaluated over different stress states to determine if stress state is an important factor evaluating the null hypothesis. If the null hypothesis is found to be true, the test protocol can be relaxed without having a detrimental effect on measuring the resilient modulus of a test specimen. Conversely, rejection of the null hypothesis requires more detail or a tighter tolerance to reduce the amount of variation between the testing laboratories included in the precision and bias part of this project.

3.3 Factor 1: Seating or Contact Load

AASHTO T 307 and NCHRP 1-28a require different seating or contact loads, which could be a source of bias or increased variability between the two test procedures. For the NCHRP 1-28a procedure, 20 percent of the confining pressure is the standard seating load, while 10 percent of the applied vertical load is the standard for the AASHTO T 307 procedure.

As noted above, three seating loads were used in the testing program to determine if variation in the contact load causes a significant deviation in the resilient modulus (see Table 6). The seating loads selected were a percentage of the confinement pressure, because of the effect of confinement can cause some soils to increase in height as the confinement is increased throughout the test. This effect has been observed on some of the softer soils when the LVDTs are placed outside the test chamber under which the entire length of the test specimen is the gauge length.

Figures 1 and 2 show the test results measured by the Alpha Lab on the A-7-6 soil in accordance with AASHTO T 307 and NCHRP 1-28a test procedures for the different seating loads. Figure 3 shows similar results but for the GAB material. As shown, the higher seating load results in higher resilient modulus values measured in accordance with AASHTO T 307 for both the A-7-6 soil and GAB material, while there was no consistent change in resilient modulus for the A-7-6 soil measured in accordance with the NCHRP 1-28a procedure (see Figure 2).

Figures 4 and 5 shows a comparison between the resilient modulus values at equivalent stress states measured using the standard seating load designated in AASHTO T 307 to the other three seating loads based on a percentage of the confining pressure. The higher the seating load, the higher the resilient modulus in comparison to the standard seating load. Figure 6 shows a similar comparison but for the NCHRP 1-28a procedure. As shown, seating load does not have a significant effect on the measured resilient modulus using the NCHRP 1-28a procedure. Figures 7 and 8 show the resilient modulus ratio between the standard seating load and other seating loads used in the ruggedness test program.

In summary, seating load has a biased effect on resilient modulus measured using the AASHTO T 307 procedure, while there is no bias generated from the NCHRP 1-28a procedure. Some of the earlier versions of the resilient modulus test procedures designated the seating load as a function or percentage of the confining pressure to reduce the amount of variability, which is supported by Figures 7 and 8. Figures 7 and 8 include a graph of the resilient modulus ratio for different seating loads. The resilient modulus ratio is defined as the resilient modulus measured for the standard seating load divided by the resilient modulus measured for the other seating loads. This observation suggests that the seating load be defined as 10 to 20 percent of the confining pressure to reduce the variation in resilient modulus test results with slight variations in the seating load.

The t-test and paired t-test were used to determine if the resilient modulus values were statistically different or indifferent between use of the standard seating load and value based on the confining pressure. Table 7 summarizes the effect of seating load variation in comparison to the standard value for the two test protocols in terms of whether there is a significant effect on resilient modulus.

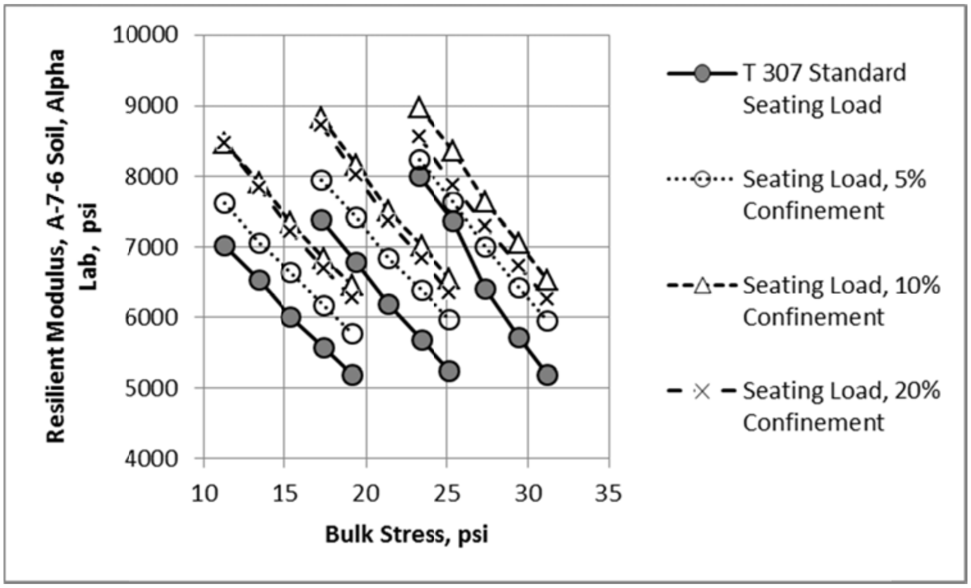

Figure 1. Resilient Modulus for the A-7-6 Soil Measured by the Alpha Laboratory for different Seating Loads, AASHTO T 307

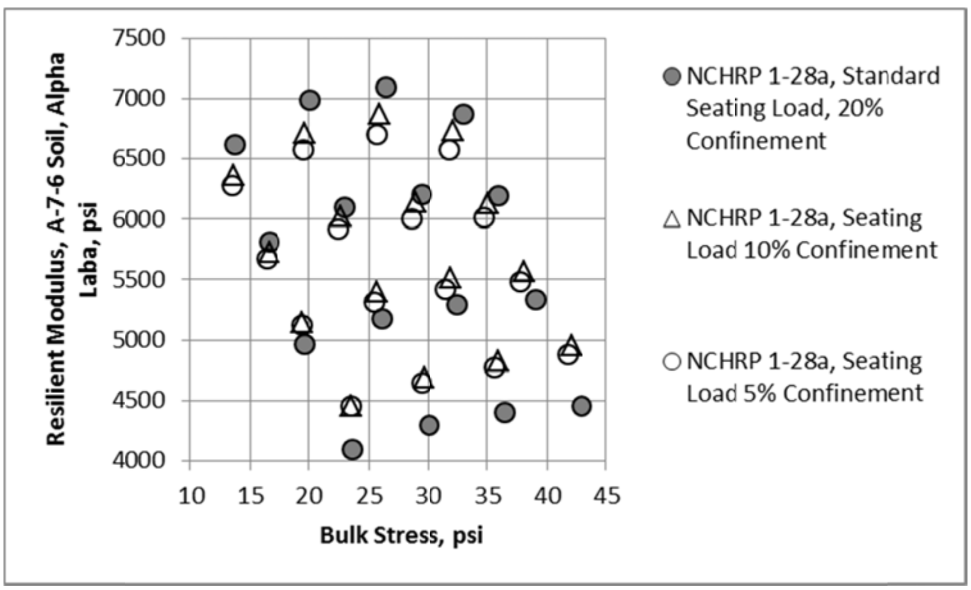

Figure 2. Resilient Modulus for the A-7-6 Soil Measured by the Alpha Laboratory for different Seating Loads, NCHRP 1-28a

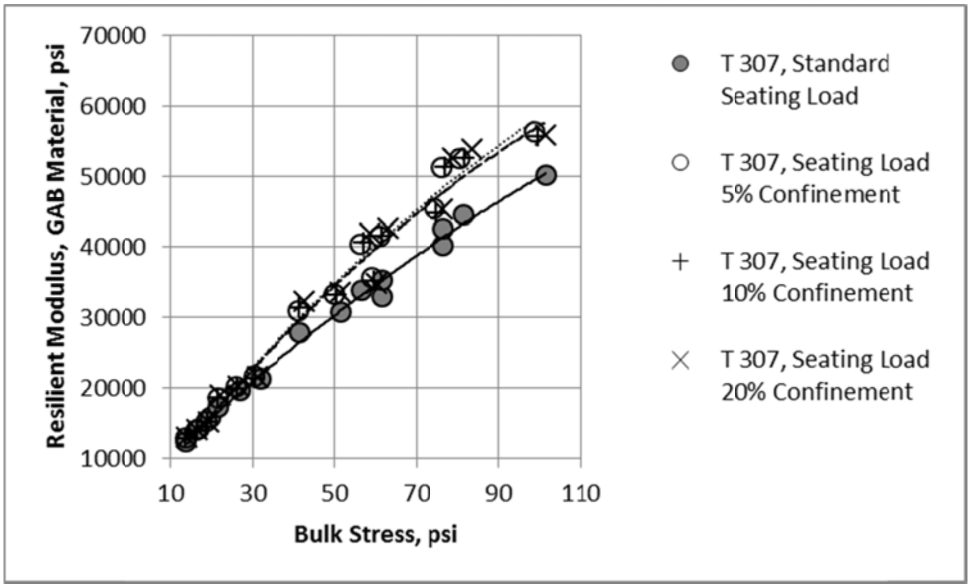

Figure 3. Resilient Modulus for the GAB Material Measured by the Alpha Laboratory for different Seating Loads, AASHTO T 307

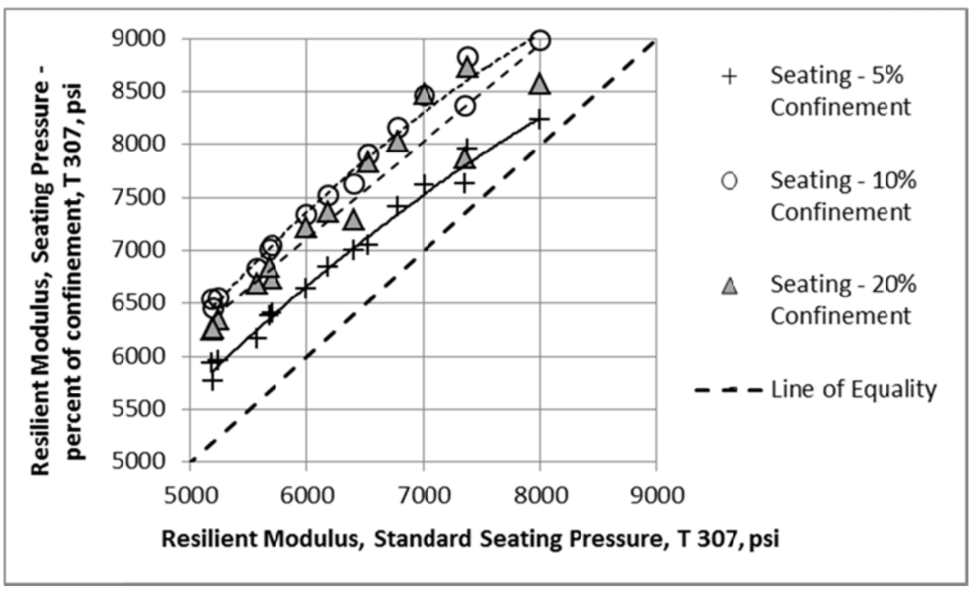

Figure 4. Effect of Seating Load on Resilient Modulus for the A-7-6 Soil, AASHTO T 307

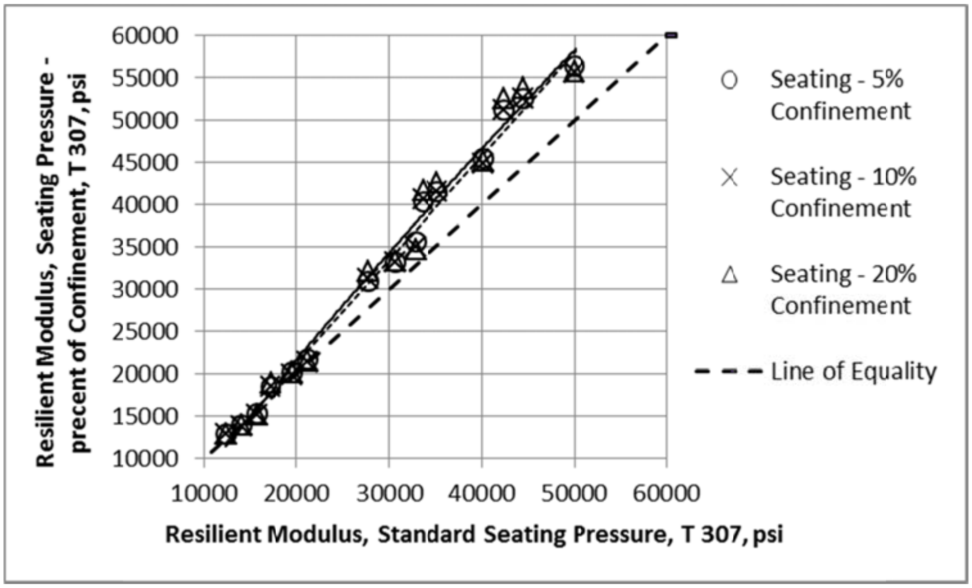

Figure 5. Effect of Seating Load on Resilient Modulus for the GAB Material, AASHTO T 307

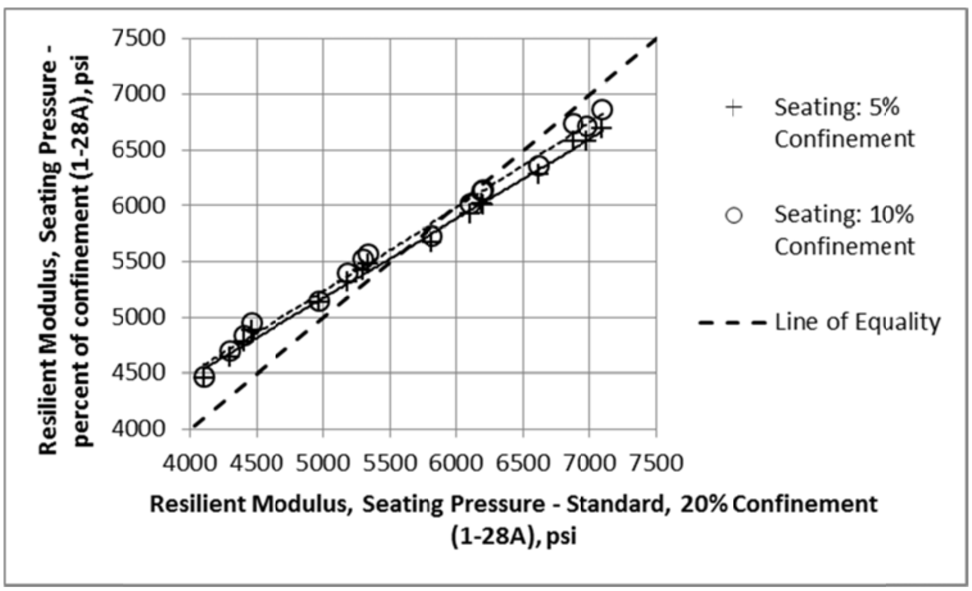

Figure 6. Effect of Seating Load on Resilient Modulus, NCHRP 1-28a

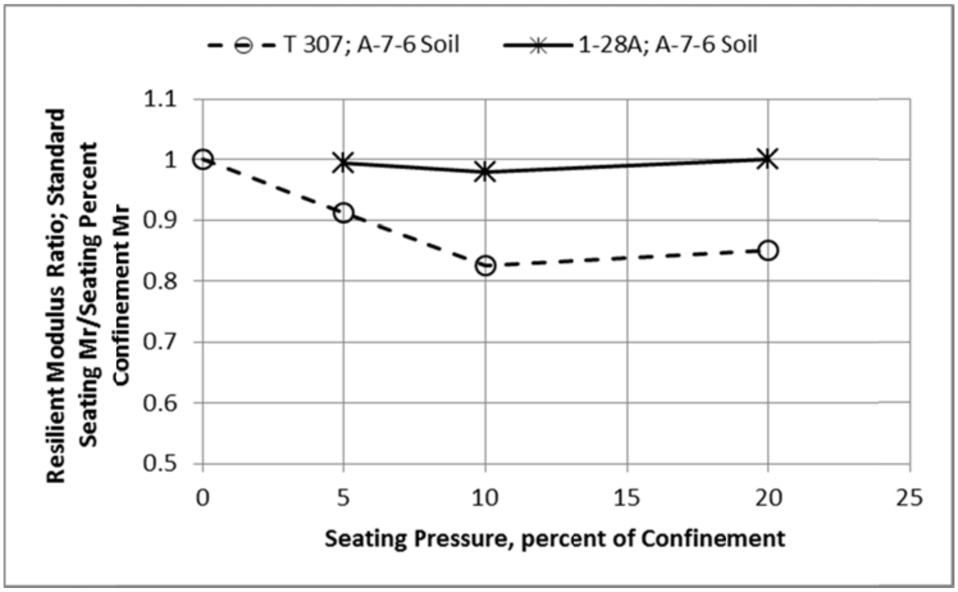

Figure 7. Effect of Seating Load on the Resilient Modulus Ratio; A-7-6 Soil (for AASHTO T 307, "0" is the standard seating load)

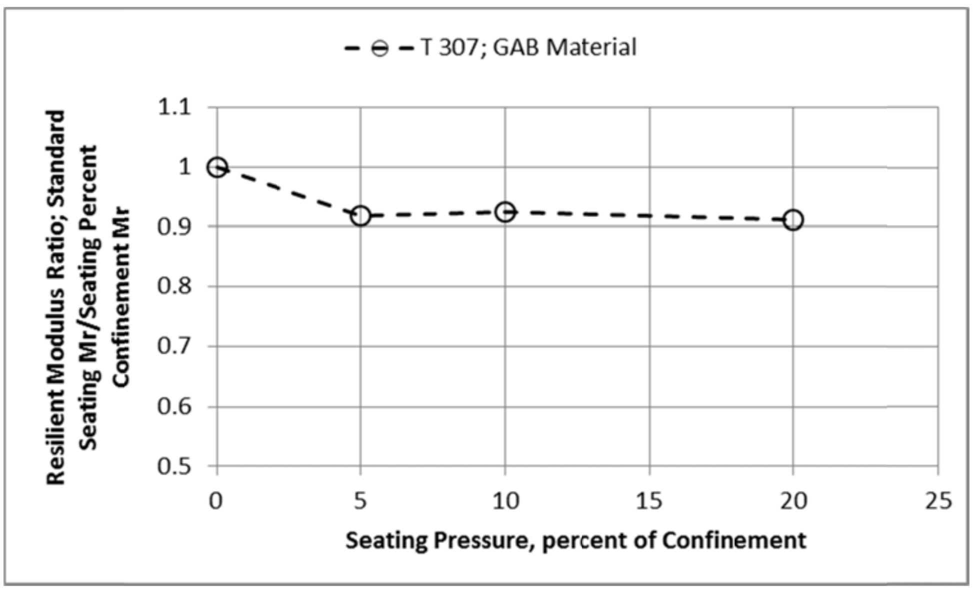

Figure 8. Effect of Seating Load on the Resilient Modulus Ratio, GAB Material (for AASHTO T 307, "0" is the standard seating load)

Table 7. Summary of Seating Load Variation on Resilient Modulus

Test Protocol	Confining Pressure, psi	Cyclic Stress Level, psi	Seating/Contact Load, % of Confinement		
			5	10	20
AASHTO T 307	6.1	5.0 (Low)	Not Significant	Significant	Not Significant
	6.1	12.9 (High)	Not Significant	Significant	Not Significant
	4.1	5.0 (Low)	Not Significant	Significant	Significant
	4.1	12.9 (High)	Not Significant	Significant	Significant
	2.1	5.0 (Low)	Significant	Significant	Significant
	2.1	12.9 (High)	Significant	Significant	Significant
NCHRP 1-28a	6.1	8.2 (Low)	Not Significant	Not Significant	Standard
	6.1	18.2 (High)	Not Significant	Not Significant	Standard
	4.1	7.8 (Low)	Not Significant	Not Significant	Standard
	4.1	17.8 (High)	Not Significant	Not Significant	Standard
	2.1	7.4 (Low)	Not Significant	Not Significant	Standard
	2.1	17.4 (High)	Not Significant	Not Significant	Standard

As shown, no statistical difference was found for the NCHRP 1-28a procedure between the standard seating load of 20 percent of the confining pressure in comparison to values of 5 and 10 percent. Using a percentage of the confining pressure over a wide range has no to little effect on the measured resilient modulus. AASHTO T 307 uses a seating load that is a percentage of the cyclic load and this is important in terms of the measured resilient modulus, especially at the lower confining pressures. As shown, there is a statistical difference in the results for many of the stress states in comparison to the AASHTO T 307 standard contact load. However, there is no statistical difference in the test results between 10 and 20 percent confinement as the seating pressure for AASHTO T 307 for both the A-7-6 soil and GAB material, as shown in Figures 7 and 8 – similar to the results from the NCHRP 1-28a procedure.

Based on the test results for this factor, deviation from the prescribed contact load under the NCHRP 1-28a test protocol has an insignificant effect on the amount of variability in test results within and between laboratories. One potential change to the test procedure for AASHTO T 307 is to revise the seating or contact load to a percentage of the confining pressure so that slight deviations of this parameter has little to no effect on the measured resilient modulus between and within laboratories. There is a bias or statistical difference, however, between the resilient modulus between the two test protocols for the same stress state for the same seating load (see Figures 1 and 2). This difference is discussed in the following paragraphs in terms of changing the AASHTO T 307 standard seating load to the standard value stated in the NCHRP 1-28a procedure.

Figures 9 and 10 show the resilient modulus measured by AASHTO T 307 and NCHRP 1-28a at equivalent stress states for the standard seating loads designated by each test protocol. The t-value for these results is 1.314, while the t-critical value for an alpha value of 0.10 is 1.86. Thus, there is no statistical difference in results (see Figure 10). If the seating load defined in AASHTO T 307 is revised to the use of a percentage of the confining pressure, there would be a statistical difference or bias in results between the two test protocols as illustrated in Figure 11.

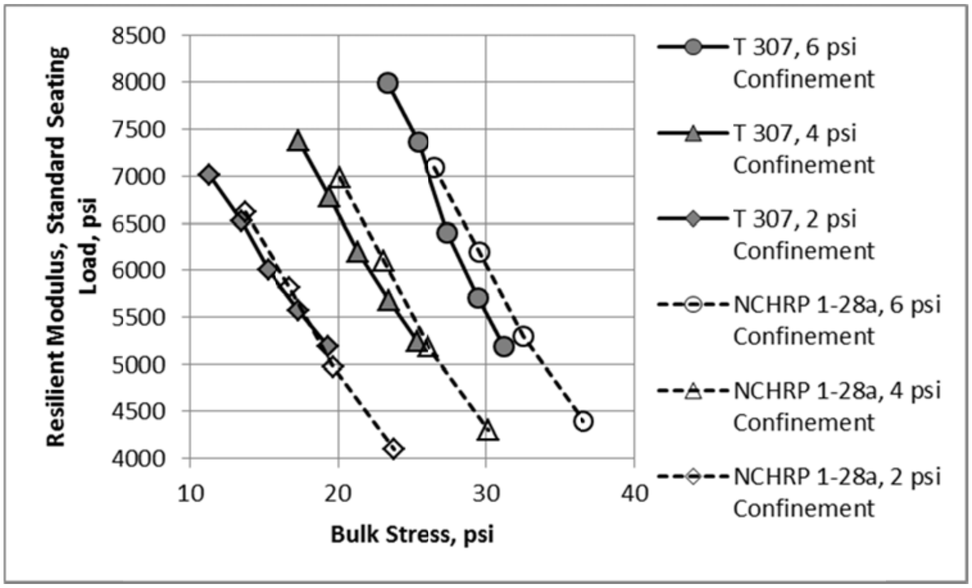

Figure 9. Resilient Modulus Measured in Accordance with AASHTO T 307 and NCHRP 1-28a Procedures Using the Standard Seating Loads Designated by each Procedure for the A-7-6 Soil

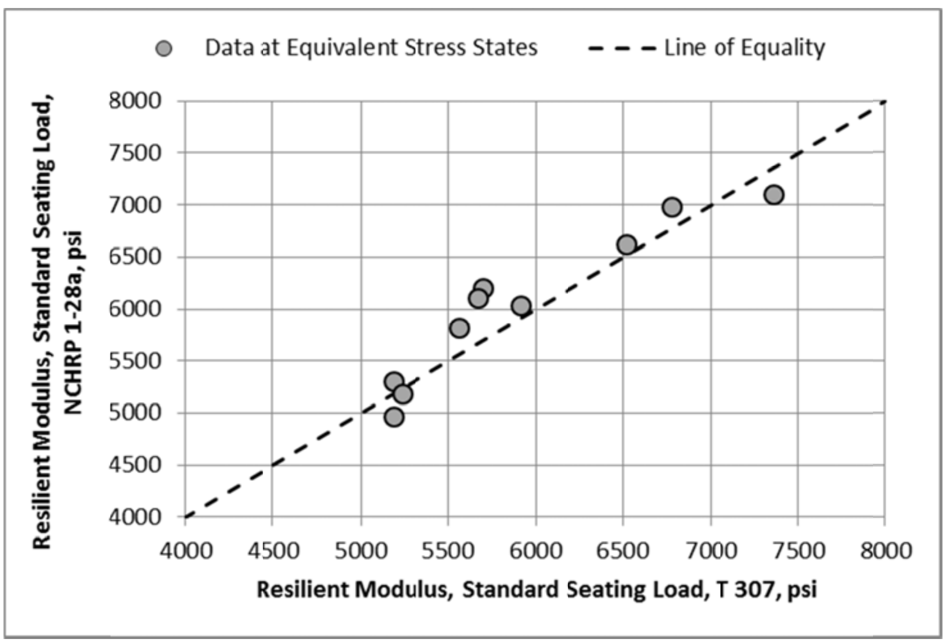

Figure 10. Comparison of Resilient Modulus at Equivalent Stress States between AASHTO T 307 and NCHRP 1-28a Using the Standard Seating Loads Designated by each Procedure

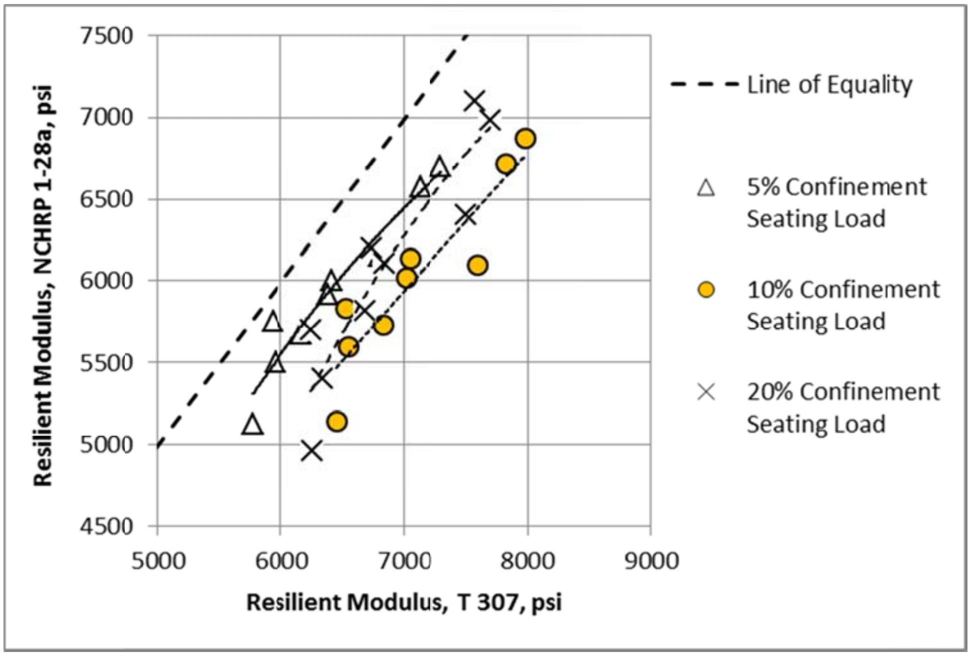

Figure 11. Comparison of Resilient Modulus between AASHTO T 307 and NCHRP 1-28a for the Same Seating Load

More importantly, a change in the seating load would require a modification by the manufacturer in some of the data acquisition systems. As such, it was decided to retain the standard seating load designated by each procedure in measuring resilient modulus for the precision and bias part of this study. This decision minimizes the bias between the two test procedures, but results in a slight increase in variation of the measured resilient modulus for AASHTO T 307 and little to no increase in variation of results for the NCHRP 1-28a procedure. This difference in variability, however, will be much less than the difference in resilient modulus to cause a change in the distress predictions or required pavement structure predicted by the MEPDG software, as presented in the literature review report (see Chapter 2 and Appendices A and B).

3.4 Factor 2: Curing or Mellowing Time

As noted above, three curing or mellowing times were used in the testing program to determine if variation in time between sample preparation and testing causes a significant deviation in the resilient modulus (see Table 6). Curing or mellowing time is the additional time to allow the soil and aggregate base material particles to absorb water. The three curing times were: 0-day or the test specimen was prepared and tested within the same day; 1-day cure or the material was mixed, allowed to "mellow" and then compacted and tested 1-day after adding water; and 2-days cure or the material was mixed, allowed to "mellow" and then compacted and tested 2-days after adding water.

The curing times were selected based on practical requirements and previous test results documented in the literature review report for this study (see Chapter 2 and Appendix A). Experience of the authors suggests that more consistent results are obtained in testing plastic soils and highly absorptive aggregates when water is added and the soil/material is allowed to "mellow" for a couple of days and then compacted and tested.

Figures 12 to 15 shows the resilient modulus values measured by the Alpha laboratory in accordance with AASHTO T 307 and NCHRP 1-28a procedures for the A-7-6 soil and GAB material for different curing times. Figures 16 and 17 include a comparison of curing time on resilient modulus measured by the Alpha and Beta laboratories for the A-7-6 soil, while Figures 18 and 19 provide the same comparison but for the GAB material. As shown, curing time does make a difference for the A-7-6 soil and that difference was exhibited by the Alpha and Beta laboratories (see Figures 16 and 17). The resilient modulus values measured on the 2-day cure test specimens were about 500 psi higher than for the 0-day cure.

For the GAB material, the Alpha laboratory found no difference between the curing time for the same test method, while the Beta laboratory measured an insignificant but slightly lower resilient modulus for the 2-day cure test specimens (see Figure 19). It is important to note, however, there is a significant difference in the resilient modulus measured for the GAB material between AASHTO T 307 and NCHRP 1-28a test methods at very high bulk stresses for the same conditioning times (see Figure 14).

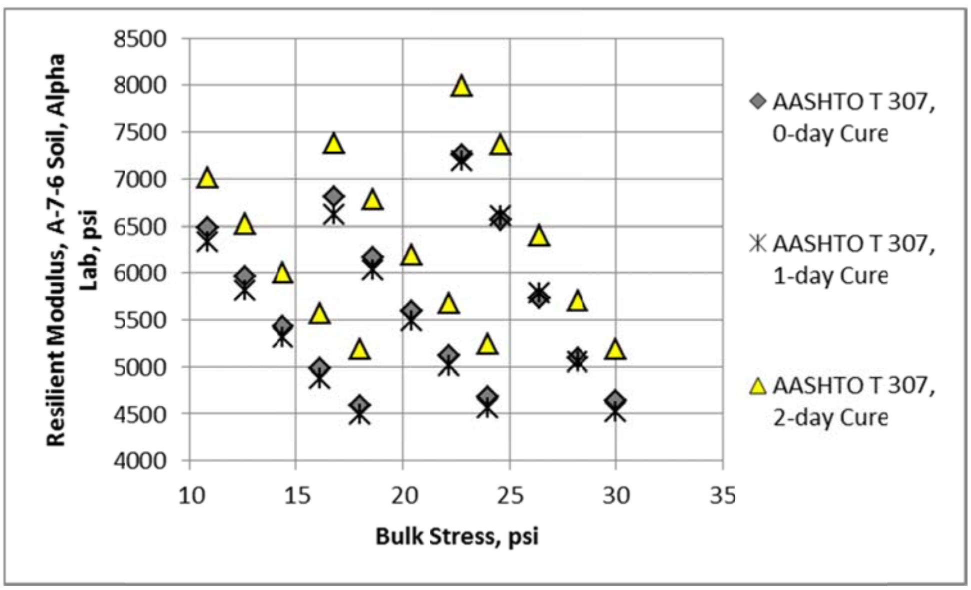

Figure 12. Resilient Modulus for the A-7-6 Soil Measured by the Alpha Laboratory for different Curing Times, AASHTO T 307

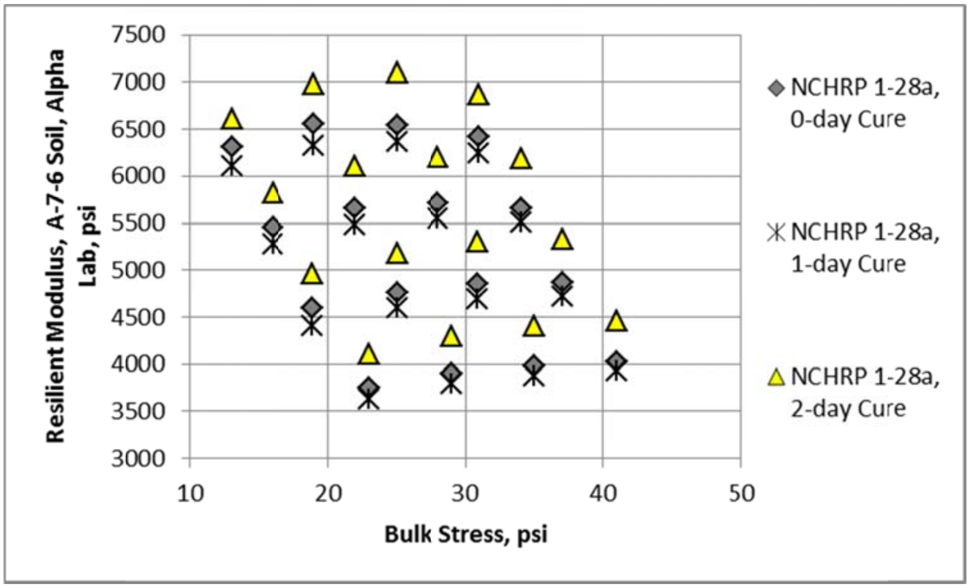

Figure 13. Resilient Modulus for the A-7-6 Soil Measured by the Alpha Laboratory for different Curing Times, NCHRP 1-28a

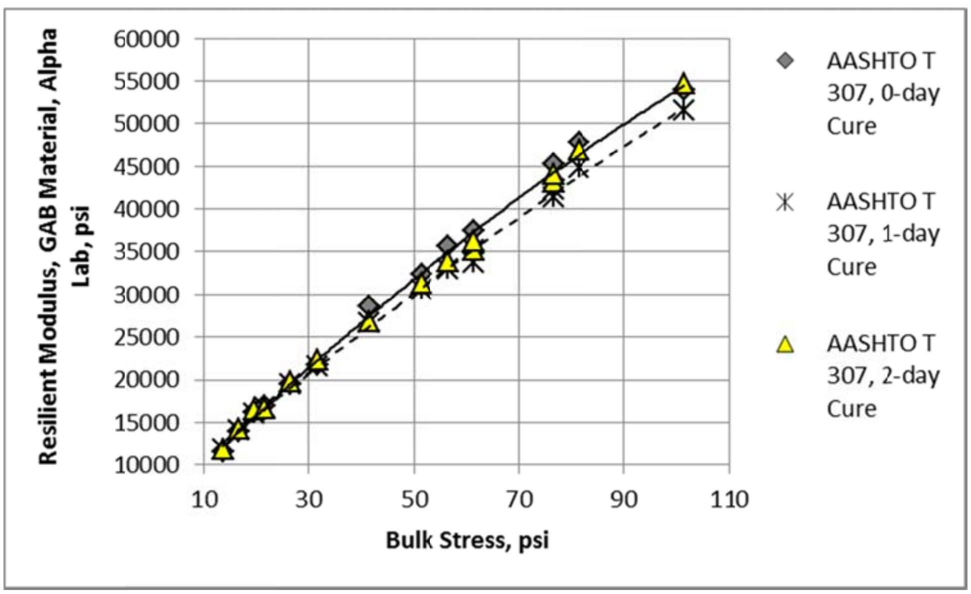

Figure 14. Resilient Modulus for the GAB Material Measured by the Alpha Laboratory for different Curing Times, AASHTO T 307

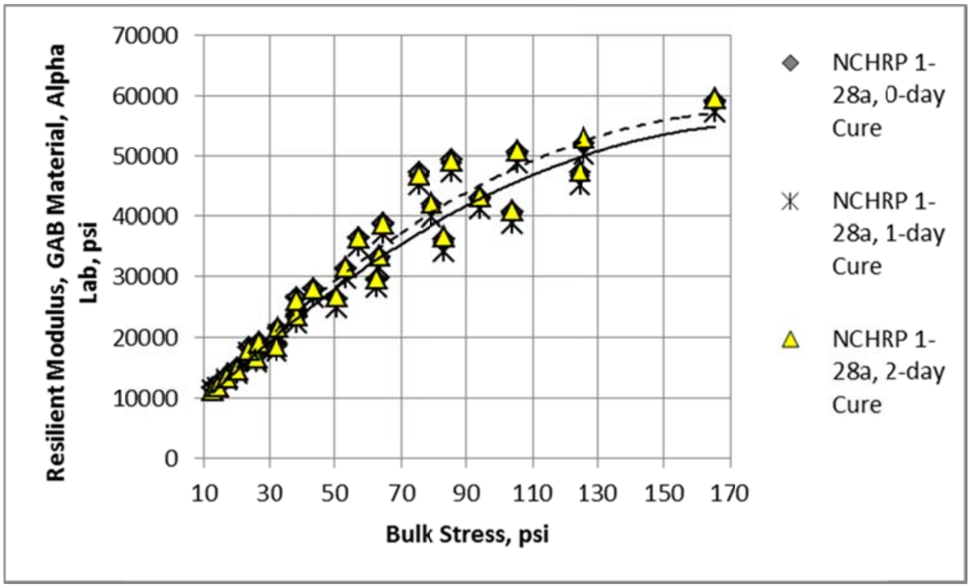

Figure 15. Resilient Modulus for the GAB Material Measured by the Alpha Laboratory for different Curing Times, NCHRP 1-28a

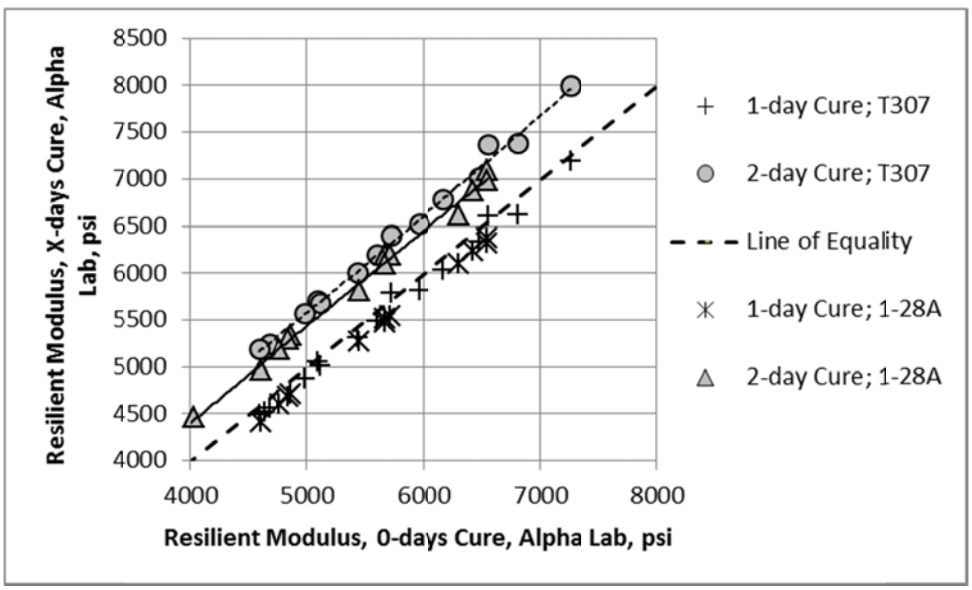

Figure 16. Effect of Curing Time on Resilient Modulus at Equivalent Stress States for the A-7-6 Soil, Alpha Laboratory

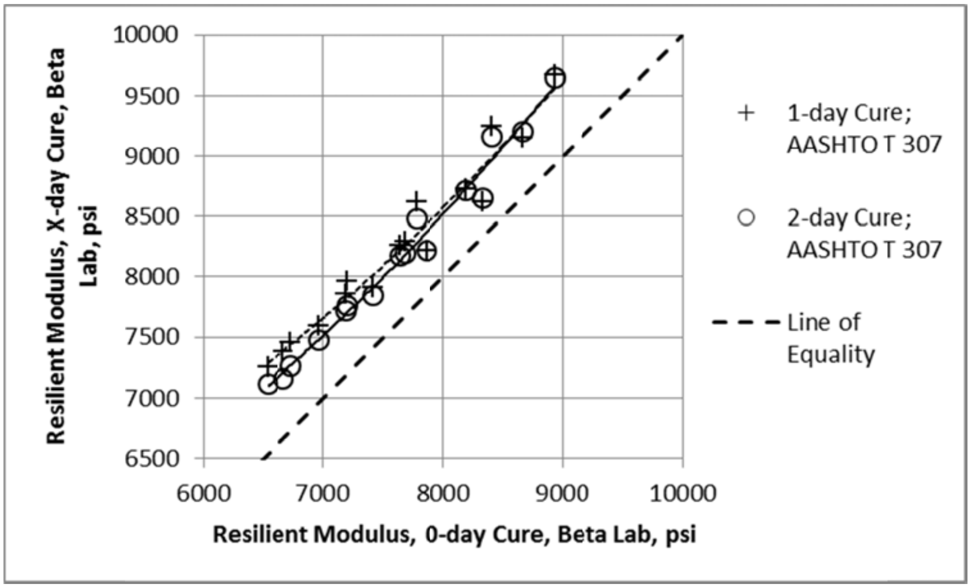

Figure 17. Effect of Curing Time on Resilient Modulus at Equivalent Stress States for the A-7-6 Soil, Beta Laboratory

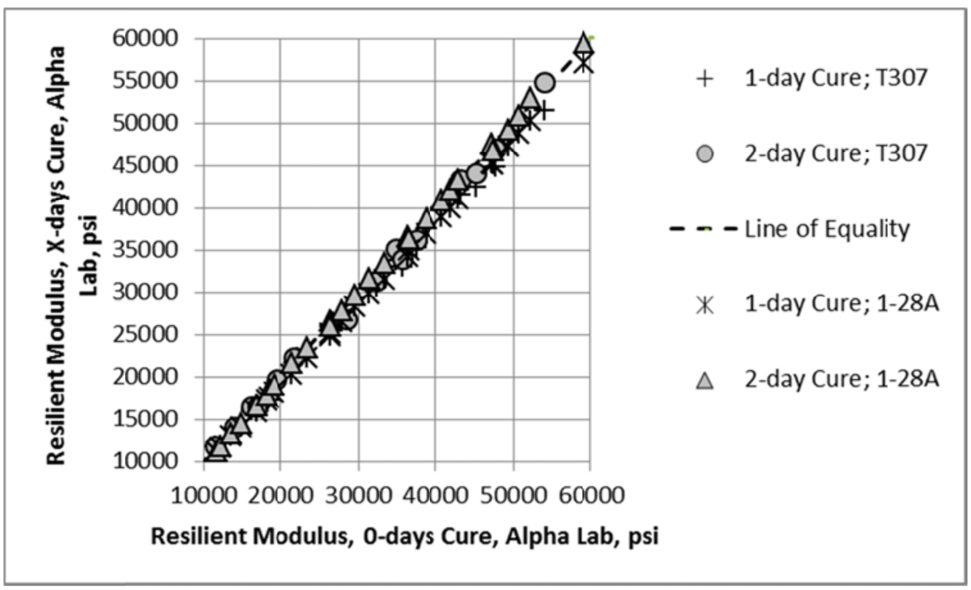

Figure 18. Effect of Curing Time on Resilient Modulus (Equivalent Stress States) for the GAB Material, Alpha Laboratory

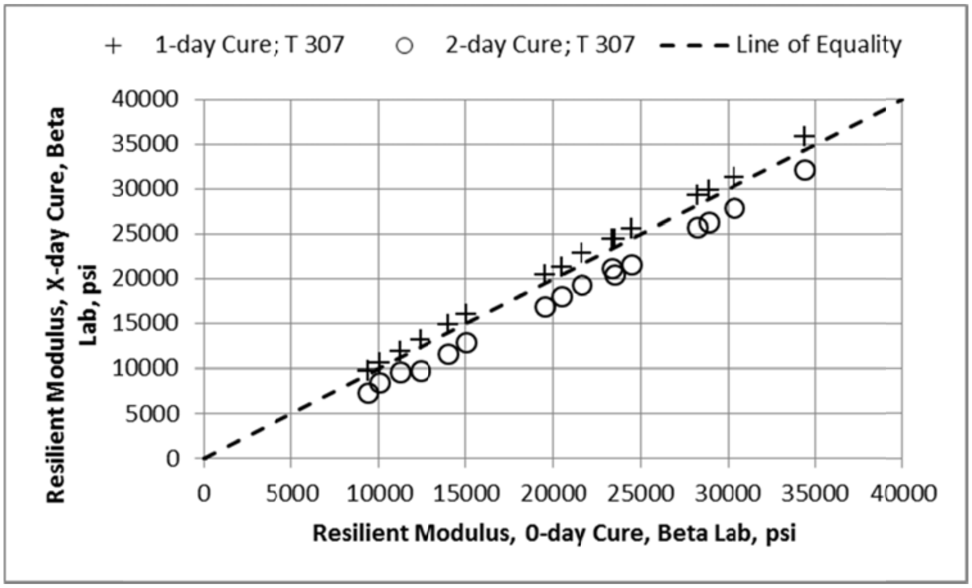

Figure 19. Effect of Curing Time on Resilient Modulus (Equivalent Stress States) for the GAB Material, Beta Laboratory

Figures 20 and 21 are bar-charts showing the average resilient modulus ratio for both materials and test procedures across all stress states. The resilient modulus ratio is defined as the resilient modulus measured on test specimens with 0-day curing divided by the resilient modulus measured on test specimens with 1-day or 2-days curing. As summarized, differences were exhibited between the Alpha and Beta laboratories (see Figures 16 to 21).

The t-test and paired t-test were used to determine if the resilient modulus values were statistically different or indifferent between use of the different curing or mellowing times for different stress states. The curing time of 0-days was used as the baseline for the null hypothesis. Tables 8 and 9 summarize the effect of curing time variation in terms of whether there is a significant effect on resilient modulus of the A-7-6 soil for the Alpha and Beta laboratories, respectively. Table 10 summarizes the effect of curing time variation on the resilient modulus of the GAB material. The GAB material included in the ruggedness test program is a low absorptive material, so little difference in test results was expected based on previous experience.

As shown, 2-days curing time did result in a statistical difference from 0-days curing time for both laboratories and test procedures for the A-7-6 soil, while no statistical difference was found for the GAB material. The statistical difference for the A-7-6 soil, however, is not considered a practical difference in many of the resilient modulus test results based on the sensitivity analyses reported in the literature.

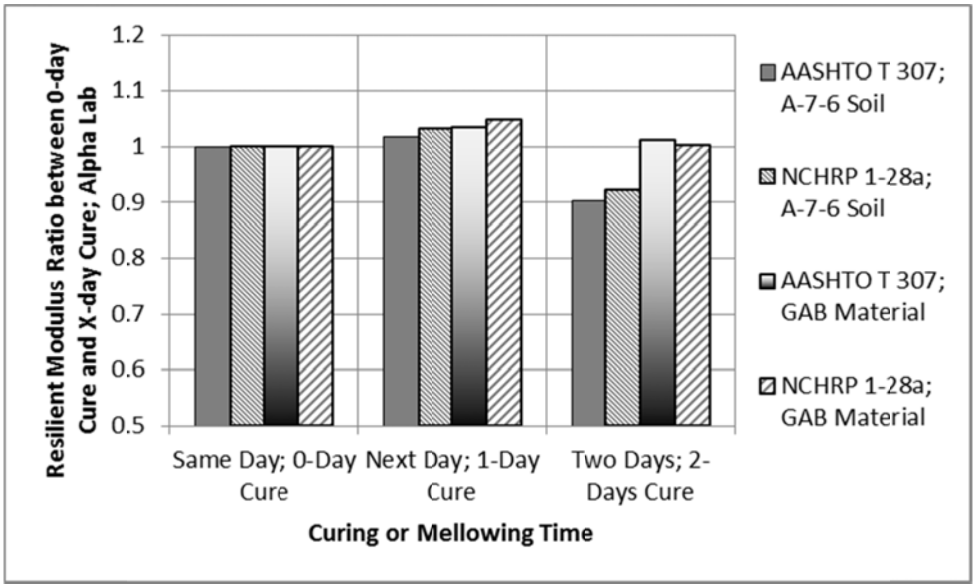

Figure 20. Effect of Curing Time on Average Resilient Modulus Ratio (Equivalent Stress States) for the A-7-6 Soil and GAB Material, Alpha Laboratory

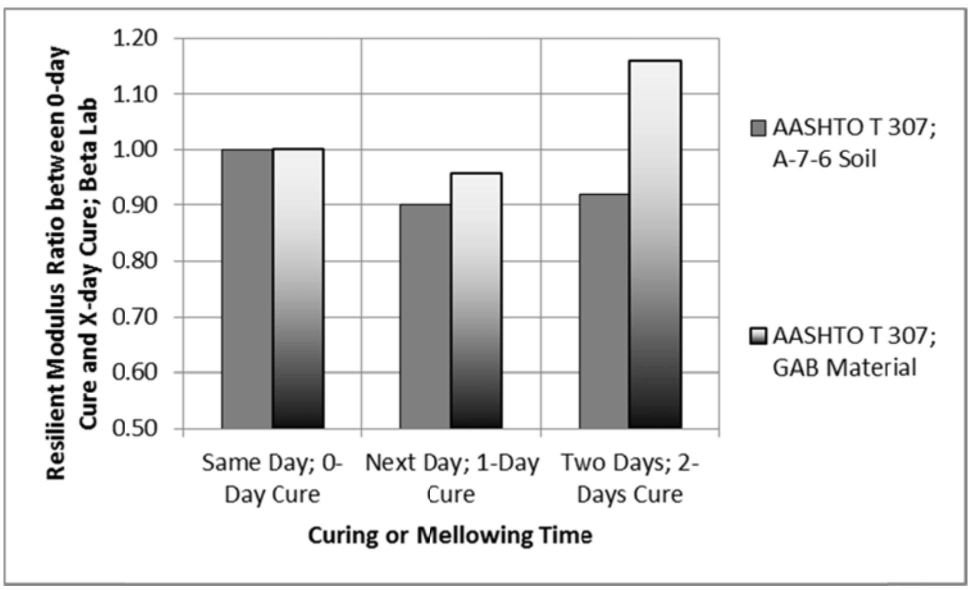

Figure 21. Effect of Curing Time on Average Resilient Modulus Ratio (Equivalent Stress States) for the A-7-6 Soil and GAB Material, Beta Laboratory

Table 8. Summary of Curing Time on Resilient Modulus for the A-7-6 Soil, Alpha Laboratory

Test Protocol	Confining Pressure, psi	Cyclic Stress Level, psi	Curing or Mellowing Time		
			0-Days	1-Day	2-Days
AASHTO T 307	6.1	4.5 (Low)	Baseline	Not Significant	Significant
	6.1	9.8 (High)	Baseline	Not Significant	Significant
	4.1	4.5 (Low)	Baseline	Not Significant	Significant
	4.1	9.8 (High)	Baseline	Not Significant	Significant
	2.1	4.5 (Low)	Baseline	Not Significant	Significant
	2.1	9.8 (High)	Baseline	Not Significant	Significant
NCHRP 1-28a	6.1	6.7 (Low)	Baseline	Not Significant	Significant
	6.1	12.6 (High)	Baseline	Not Significant	Significant
	4.1	6.7 (Low)	Baseline	Not Significant	Significant
	4.1	12.7 (High)	Baseline	Not Significant	Significant
	2.1	6.7 (Low)	Baseline	Not Significant	Significant
	2.1	12.6 (High)	Baseline	Not Significant	Significant

Table 9. Summary of Curing Time on Resilient Modulus for the A-7-6 Soil, Beta Laboratory

Test Protocol	Confining Pressure, psi	Cyclic Stress Level, psi	Curing or Mellowing Time		
			0-Days	1-Day	2-Days
AASHTO T 307	6.1	4.0 (Low)	Baseline	Significant	Significant
	6.1	10.0 (High)	Baseline	Significant	Significant
	4.1	4.0 (Low)	Baseline	Significant	Significant
	4.1	10.0 (High)	Baseline	Significant	Significant
	2.1	4.0 (Low)	Baseline	Significant	Significant
	2.1	10.0 (High)	Baseline	Significant	Significant

Table 10. Summary of Curing Time on Resilient Modulus for the GAB Material, Beta Laboratory

Test Protocol	Confining Pressure, psi	Cyclic Stress Level, psi	Curing or Mellowing Time		
			0-Days	1-Day	2-Days
AASHTO T 307	3	7.4 (Moderate)	Baseline	Not Significant	Not Significant
	5	11.4 (Moderate)	Baseline	Not Significant	Not Significant
	10	21.2 (Moderate)	Baseline	Not Significant	Not Significant
	15	16.3 (Moderate)	Baseline	Not Significant	Not Significant
	20	21.2 (Moderate)	Baseline	Not Significant	Not Significant
NCHRP 1-28a	3	7.9 (Moderate)	Baseline	Not Significant	Not Significant
	6	14.5 (Moderate)	Baseline	Not Significant	Not Significant
	10	23.2 (Moderate)	Baseline	Not Significant	Not Significant
	15	19.3 (Moderate)	Baseline	Not Significant	Not Significant
	20	25.3 (Moderate)	Baseline	Not Significant	Not Significant

Figures 22 and 23 include a comparison of the resilient modulus measured at equivalent stress states and confining pressures on test specimens tested by the Alpha laboratory between AASHTO T 307 and NCHRP 1-28a for the different curing times. As shown, there is no statistical difference in resilient modulus measured in accordance with AASHTO T 307 and NCHRP 1-28as for any of the curing times used for the A-7-6 soil and GAB materials.

Figures 24 and 25 include a comparison between the resilient modulus measured between the Alpha and Beta laboratories for the same test specimen condition for the AASHTO T 307 procedure. As shown, the resilient moduli measured by the Beta laboratory are about 1,500 to over 2,000 psi higher than measured by the Alpha laboratory for the A-7-6 soil (see Figure 24). Conversely, the Beta laboratory measured resilient modulus about 30 percent lower than the Alpha laboratory for the GAB material (see Figure 25).

The important observation is that the difference in resilient modulus measured between the Alpha and Beta laboratories is significantly greater than the different curing times. This difference or bias in resilient modulus magnitude is believed to be related to sample preparation and/or location of LVDTs, which will be addressed during the precision and bias part of the study.

3.5 Factor 3: Moisture Content

Both AASHTO T 307 and NCHRP 1-28a resilient modulus test methods limit the tolerance from the optimum water content to 0.5 percent for preparing test specimens. As noted above, four moisture contents from the optimum water content were used in the testing program to determine if variation in the water content causes a significant deviation in the resilient modulus (see Table 1). The four water contents were: 0.5 percent above and below the optimum water content and 1.0 percent above and below the optimum value. These water contents were selected based on practical requirements and previous test results documented in the literature review report for this study (see Chapter 2 and Appendix A).

Figures 26 and 27 show the resilient modulus values measured by the Alpha and Beta laboratories in accordance with AASHTO T 307 for the A-7-6 soil test specimens prepared using different water contents. Figures 28 and 29 provide a comparison of the resilient modulus measured on test specimens compacted to the optimum water content and specimens compacted at other water contents. The resilient modulus in Figures 28 and 29 were measured by the Alpha laboratory in accordance with AASHTO T 307 and NCHRP 1-28a for the A-7-6 soil and GAB material, while Figure 30 provides the same comparison of results for the A-7-6 soil measured by the Beta laboratory. An important observation between from Figures 28 to 30 is that the Beta laboratory measured much higher resilient modulus values for the A-7-6 soil in comparison to the Alpha laboratory. This difference will be addressed during the precision and bias part of the study.

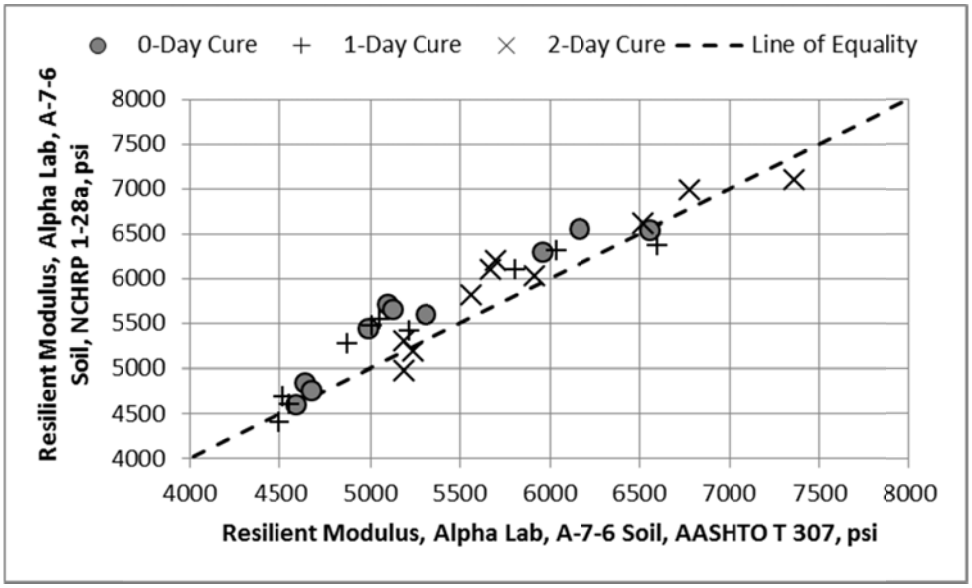

Figure 22. Comparison of Resilient Modulus Measured by the Alpha Laboratory in Accordance with AASHTO T 307 and NCHRP 1-28a for Different Curing Times, A-7-6 Soil

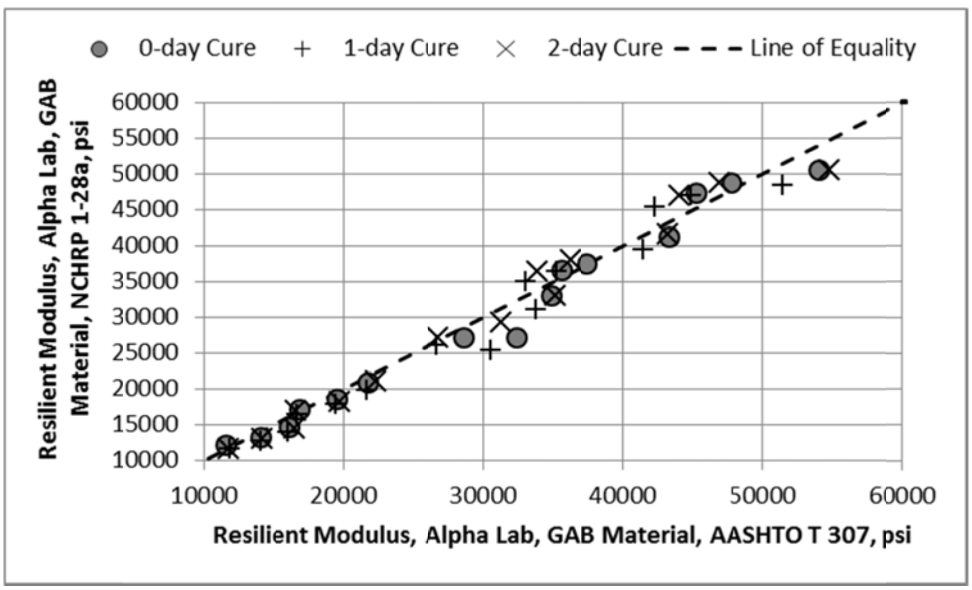

Figure 23. Comparison of Resilient Modulus Measured by the Alpha Laboratory in Accordance with AASHTO T 307 and NCHRP 1-28a for Different Curing Times, GAB Material

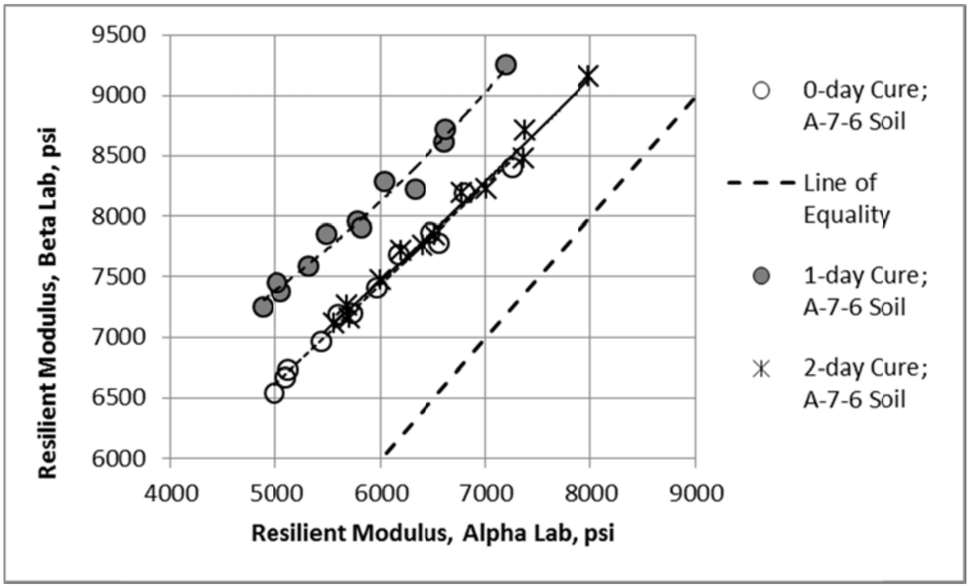

Figure 24. Comparison of Resilient Modulus Measured by the Alpha and Beta Laboratories in Accordance with AASHTO T 307 for Different Curing Times; A-7-6 Soil

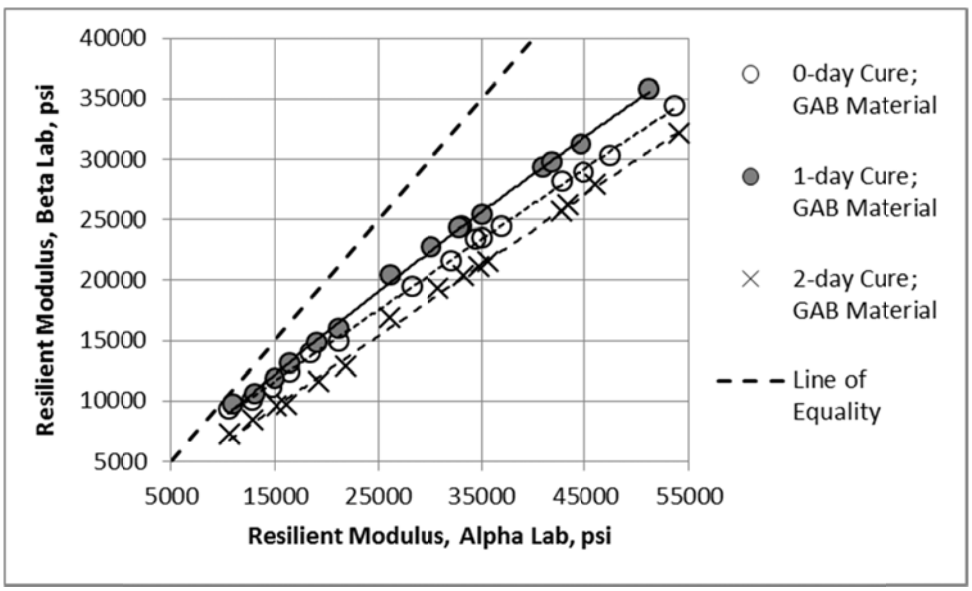

Figure 25. Comparison of Resilient Modulus Measured by the Alpha and Beta Laboratories in Accordance with AASHTO T 307 for Different Curing Times; GAB Material

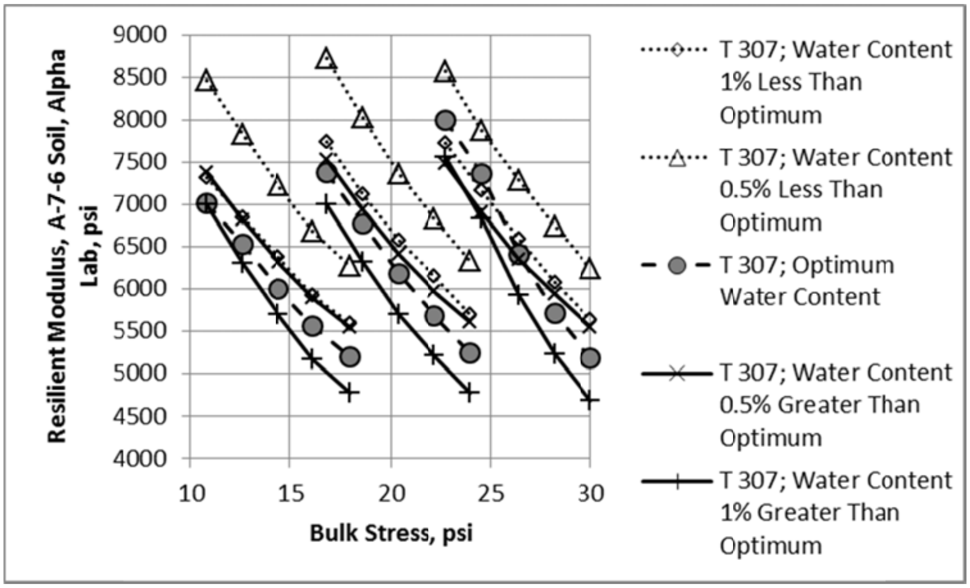

Figure 26. Resilient Modulus Measured on the A-7-6 Soil for Different Water Contents; Alpha Laboratory

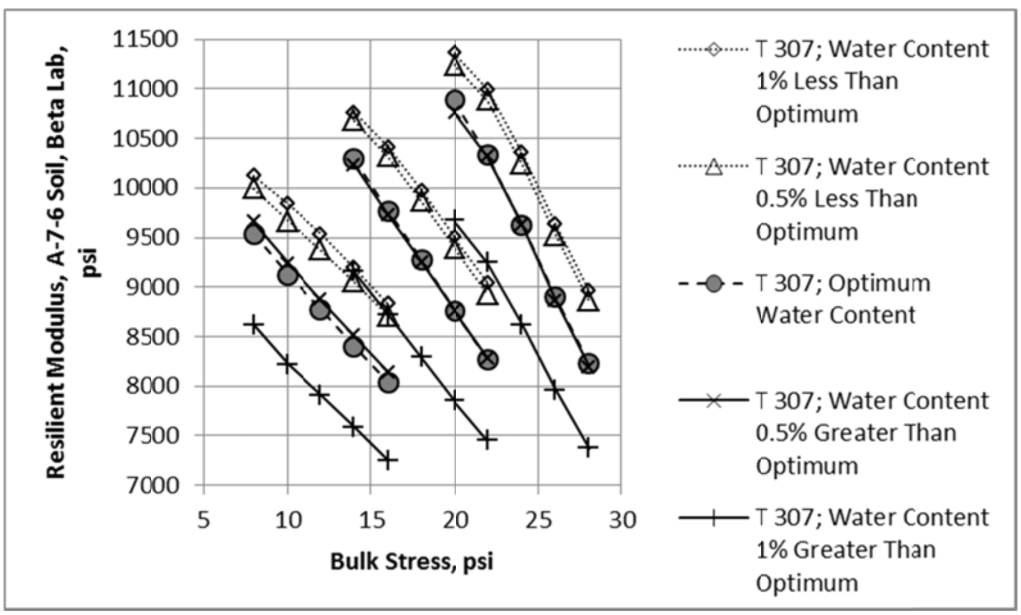

Figure 27. Resilient Modulus Measured on the A-7-6 Soil for Different Water Contents; Beta Laboratory

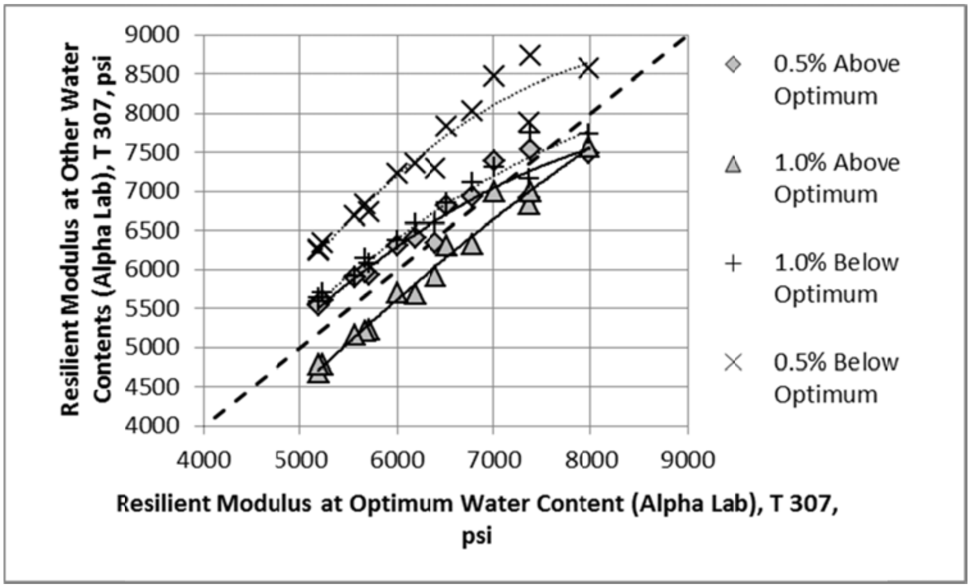

Figure 28. Effect of Water Content on Resilient Modulus Measured in accordance with AASHTO T 307 for the A-7-6 Soil, Alpha Laboratory

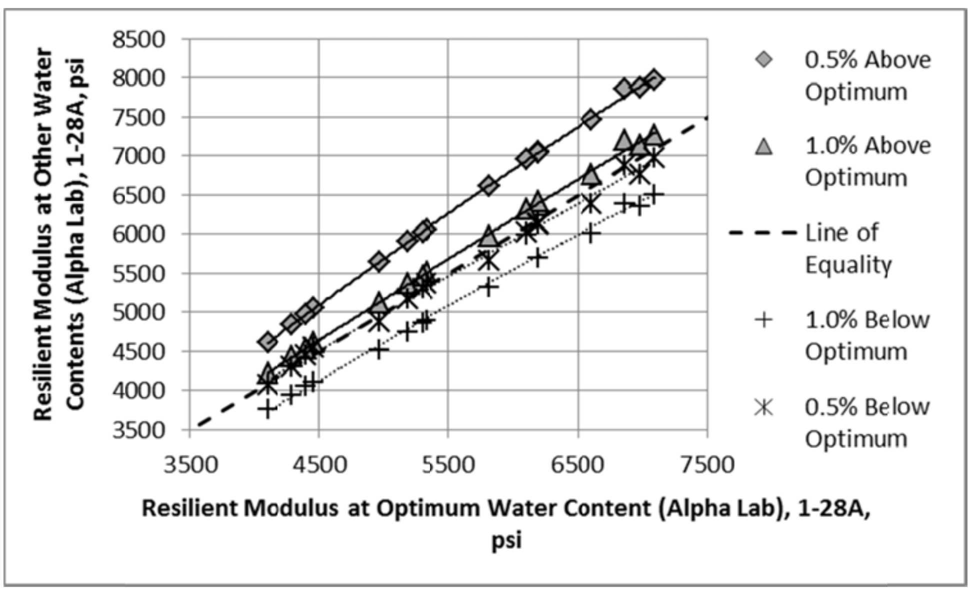

Figure 29. Effect of Water Content on Resilient Modulus Measured in accordance with NCHRP 1-28a Procedures for the A-7-6 Soil, Alpha Laboratory

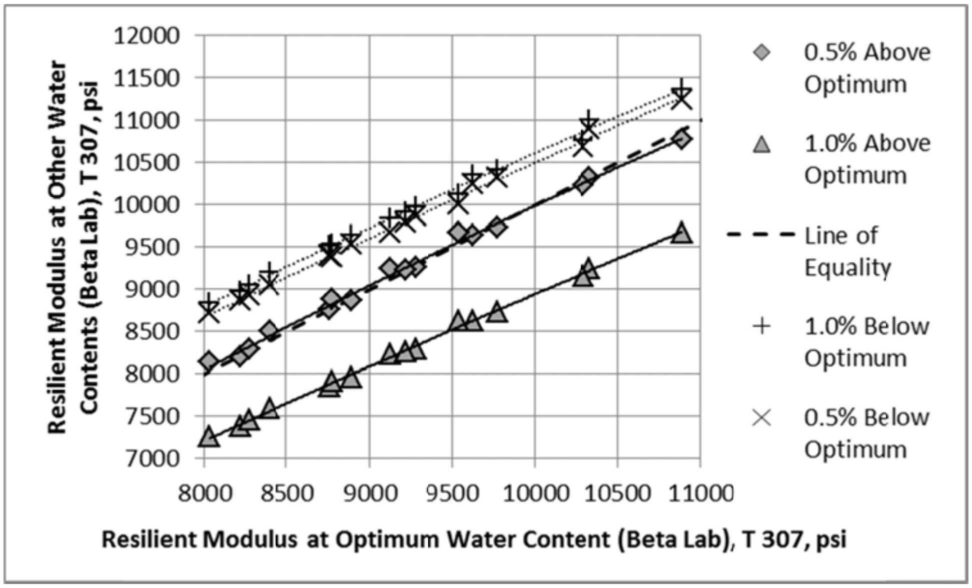

Figure 30. Effect of Water Content on Resilient Modulus Measured in Accordance with AASHTO T 307 for the A-7-6 Soil, Beta Laboratory

Figures 31 to 34 illustrate the effect of varying water contents from the optimum value on the average resilient modulus ratio resulting from the Alpha and Beta laboratories.[3] As shown, water content is a significant factor in terms of resilient modulus. The important observation from these results is that the effect of water content on resilient modulus in comparison to the optimum water content varies between the laboratories and different test procedures. This difference in resilient modulus magnitude is believed to be related to sample preparation, which was not addressed in the ruggedness study.

The t-test and paired t-test were used to determine if the resilient modulus values were statistically different or indifferent between the different water contents for different stress states. The optimum water content was used as the baseline for the null hypothesis. Tables 11 and 12 summarize the effect of water content variation in terms of whether there is a significant effect on resilient modulus for the Alpha and Beta laboratories, respectively. As shown, water content did result in a statistical difference from the optimum value for both laboratories and test procedures but for different conditions or cells within the testing matrix. Although there is a difference, the difference is not considered a practical difference, relative to the difference in results between the Alpha and Beta laboratories.

[3] Resilient modulus ratio is the resilient modulus measured on test specimens prepared at the optimum water content divided by the resilient modulus measured on test specimens prepared at the other water contents.

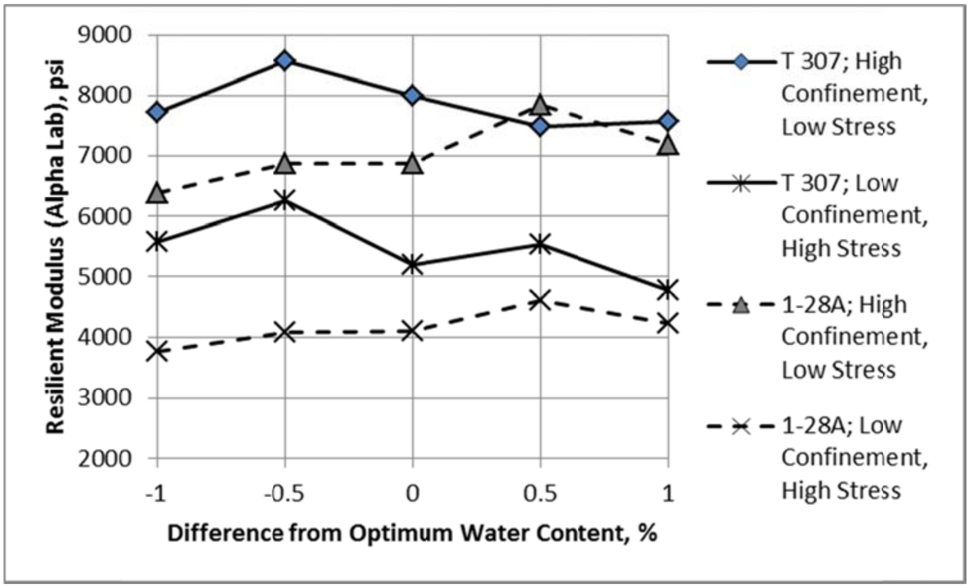

Figure 31. Effect of Water Content on Resilient Modulus at Equivalent Stress States; Alpha Laboratory

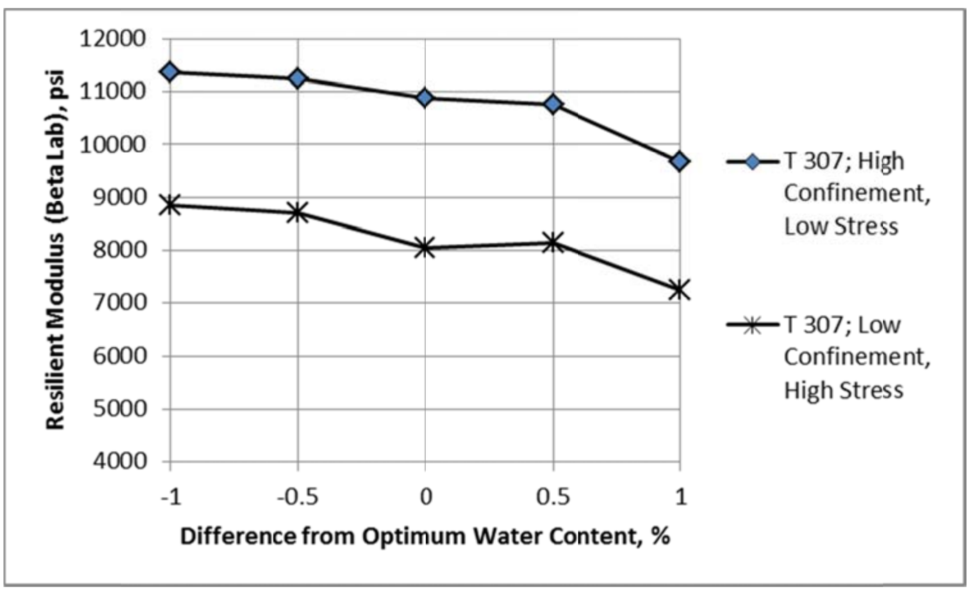

Figure 32. Effect of Water Content on Resilient Modulus at Equivalent Stress States; Beta Laboratory

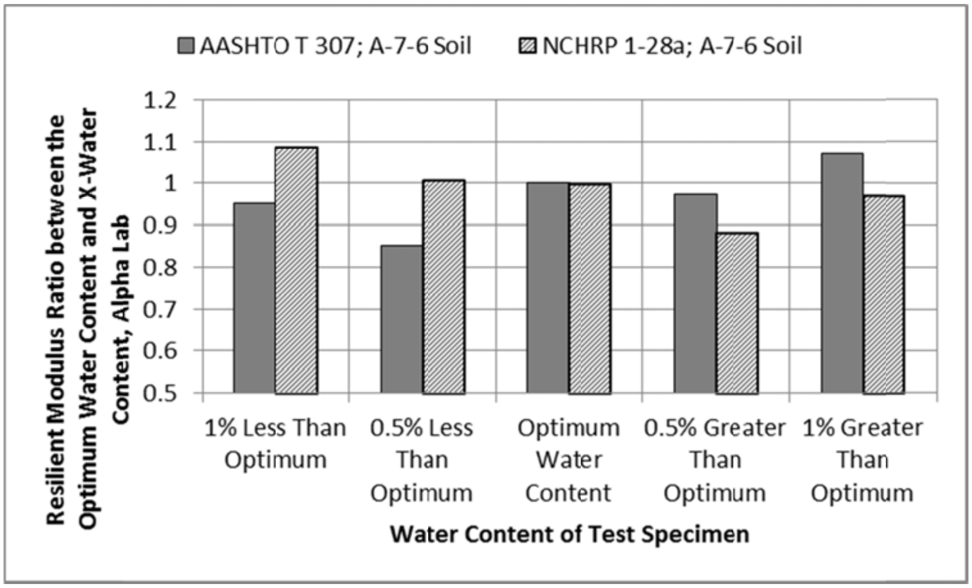

Figure 33. Effect of Water Content on Resilient Modulus Ratio; Alpha Laboratory

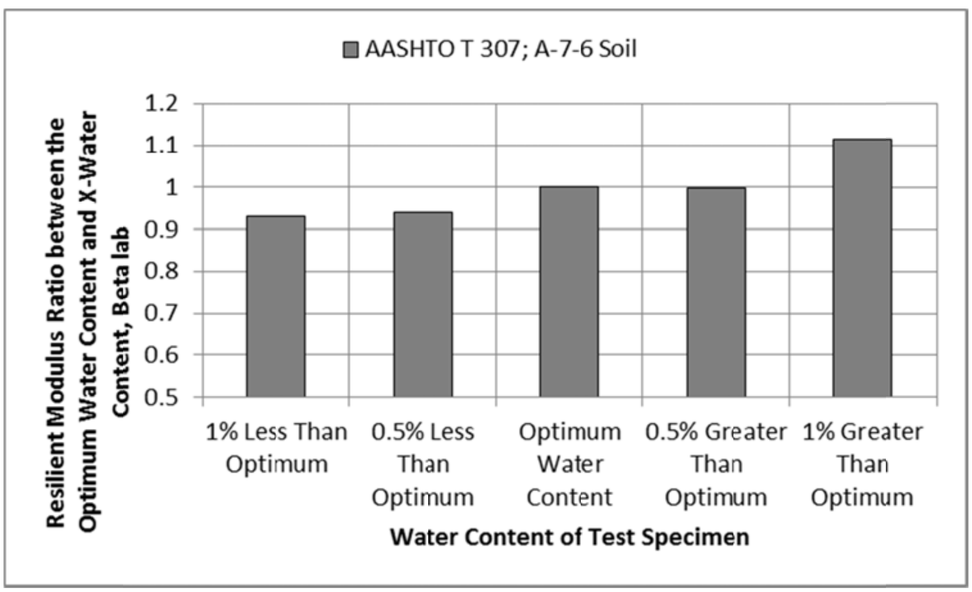

Figure 34. Effect of Water Content on Resilient Modulus Ratio; Beta Laboratory

Table 11. Summary of Water Content Effect on Resilient Modulus, Alpha Laboratory

Test Protocol	Confining Pressure, psi	Cyclic Stress Level	Water Content				
			1.0% Below	0.5% Below	Optimum	0.5% Above	1.0% Above
AASHTO T 307	6.1	6.3 (Low)	Indifferent	Indifferent	Baseline	**Different**	**Different**
	6.1	11.7 (High)	Indifferent	Indifferent	Baseline	**Different**	**Different**
	2.1	6.3 (Low)	Indifferent	**Different**	Baseline	Indifferent	**Different**
	2.1	11.7 (High)	**Different**	**Different**	Baseline	Indifferent	**Different**
NCHRP 1-28a	6.1	6.7 (Low)	**Different**	Indifferent	Baseline	**Different**	Indifferent
	6.1	12.6 (High)	**Different**	Indifferent	Baseline	**Different**	Indifferent
	2.1	6.7 (Low)	**Different**	Indifferent	Baseline	**Different**	Indifferent
	2.1	12.6 (High)	**Different**	Indifferent	Baseline	**Different**	Indifferent

Table 12. Summary of Water Content Effect on Resilient Modulus, Beta Laboratory

Test Protocol	Confining Pressure, psi	Cyclic Stress Level	Water Content				
			1.0% Below	0.5% Below	Optimum	0.5% Above	1.0% Above
AASHTO T 307	6.1	4.0 (Low)	**Different**	Indifferent	Baseline	Indifferent	**Different**
	6.1	10.0 (High)	**Different**	Indifferent	Baseline	Indifferent	**Different**
	4.1	4.0 (Low)	**Different**	Indifferent	Baseline	Indifferent	**Different**
	4.1	10.0 (High)	**Different**	Indifferent	Baseline	Indifferent	**Different**
	2.1	4.0 (Low)	**Different**	**Different**	Baseline	Indifferent	**Different**
	2.1	10.0 (High)	**Different**	**Different**	Baseline	Indifferent	**Different**

Figure 35 includes a comparison between the resilient modulus values measured in accordance with the AASHTO T 307 and NCHRP 1-28a test procedures for the A-7-6 soil test specimens prepared with different water contents. As shown, the resilient modulus values are the same or indifferent when measured at the optimum water content, but deviate when the water content is different from the optimum value. In fact, a deviation of only 0.5 percent in water content resulted in significantly different resilient modulus values in comparison to the values measured at the optimum water content. This is a reason to keep the water content tolerance as tight as possible. Trying to keep the moisture content of the test specimen closer to within plus or minus 0.5 percent of the optimum value, however, is considered impractical. Thus, the standard tolerance of plus and minus 0.5 percent of the optimum water content was used within the precision and bias testing part of the study.

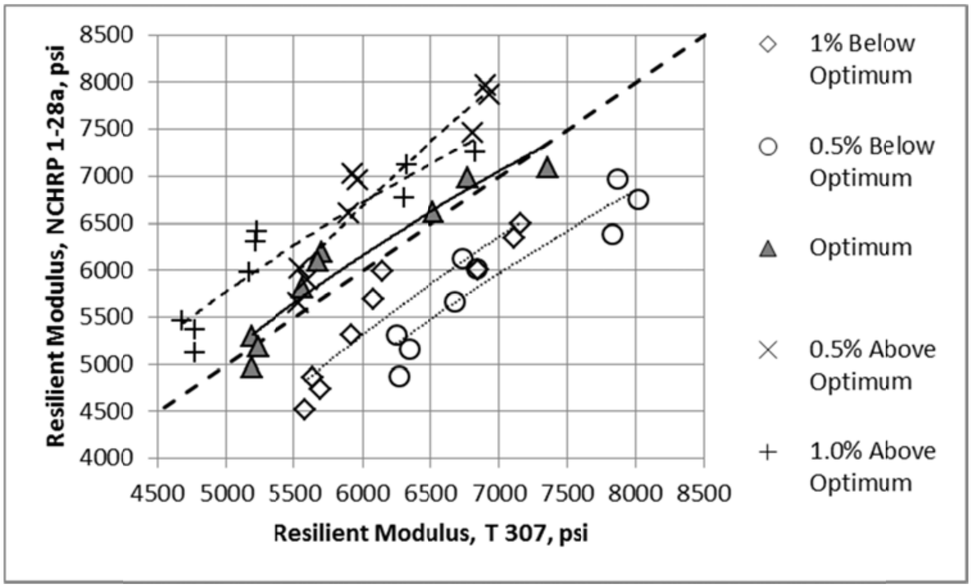

Figure 35. Comparison of Resilient Modulus for Varying Water Contents: AASHTO T 307 and NCHRP 1-28a

Figures 36 and 37 show a comparison between the resilient modulus measured between the Alpha and Beta laboratories for the same test specimen conditions and equivalent stress states and confining pressures for the AASHTO T 307 procedure. As shown, the resilient moduli measured by the Alpha laboratory are about 35 percent higher than measured by the Beta laboratory. This difference between the Alpha and Beta laboratories is significantly greater than the difference in resilient modulus measured at the different water contents used in the test program.

3.6 Summary of Observations

The intent of the ruggedness test program was to identify the allowable tolerance of some test parameters for which there is a minimal effect on the measured resilient modulus. One of the important observations from this effort, however, was that the difference in measured resilient modulus between the Alpha and Beta laboratories was significantly greater than any difference between the resilient modulus measured over the range of the selected test parameters included in the study. To reiterate some important observations from the roughness study:

1. Deviation from the standardized seating load specified in AASHTO T 307, based on a percent of the confining pressure, resulted in a significant difference in resilient modulus, while different seating loads from the standard identified in NCHRP 1-28a did not result in a significant difference in results.

2. Different curing or mellowing times resulted in a significant difference in resilient modulus for the A-7-6 soil, while no significant difference in results was exhibited for the GAB material. The GAB material is considered a low absorptive material. It is expected that a difference in results will be exhibited for higher absorptive aggregates used as a granular base material based on previous experience of the authors.
3. Significantly different resilient moduli were measured by the Alpha and Beta laboratories, as the water content deviated from the optimum value. As such, the optimum water content was specified for the precision and bias testing with a tolerance of 0.5 percent. The use of a tighter tolerance than 0.5 percent from the optimum value is considered impractical.
4. The difference in results between AASHTO T 307 and NCHRP 1-28a is insignificant at equivalent stress states and confining pressures for the different curing times and standard seating loads. In addition, no significant difference was exhibited in resilient modulus measured at the optimum water content. However, the difference in resilient modulus measured in accordance with AASHTO T 307 and NCHRP 1-28a test methods increased as designated below:
    a. As the moisture content deviated above and below the optimum value for the A-7-6 soil, the difference in resilient modulus measured between AASHTO T 307 and NCHRP 1-28a increased. As such, test specimens should be prepared and compacted to 95 percent of the maximum dry density and optimum water content for the precision and bias testing.
    b. As the seating load deviated from the standard value specified in AASHTO T 307, the difference in resilient modulus measured between AASHTO T 307 and NCHRP 1-28a increased. Deviation from the standard seating load specified in the NCHRP 1-28a test method did not result in a significant change in the results. As such, the standard seating load included in AASHTO T 307 and NCHRP 1-28a test methods were used in the precision and bias testing.
    c. No difference in test results between AASHTO T 307 and NCHRP 1-28a was exhibited for the different curing times used in the ruggedness test program.
5. The difference in test results between the testing laboratories is much greater than the difference in results between any of the test specimen and/or testing conditions. Typical variations in water content and density, seating load, and curing time do not explain the difference in results between the Alpha and Beta laboratories. Two differences between the Alpha and Beta laboratories are: (1) testing machine differences (Instron and Interlaken equipment), and (2) location of the LVDTs off the specimens (LVDTs located outside and inside the testing chamber). As these two parameters were not investigated within the ruggedness test program, the difference or larger variability exhibited will be reflected in the precision of the test methods – discussed and defined in Chapter 4.

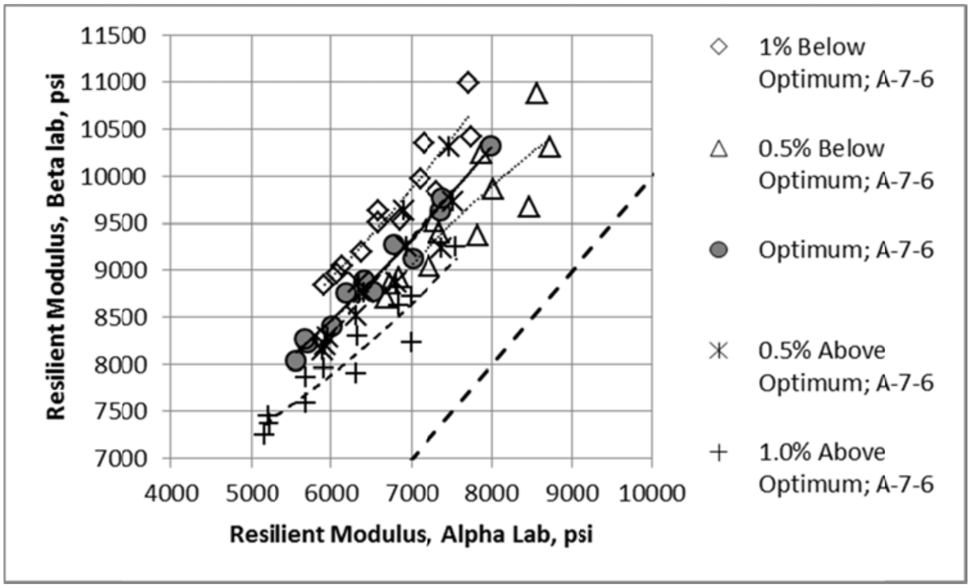

Figure 36. Comparison of Resilient Modulus between the Alpha and Beta Laboratories; A-7-6 Soil

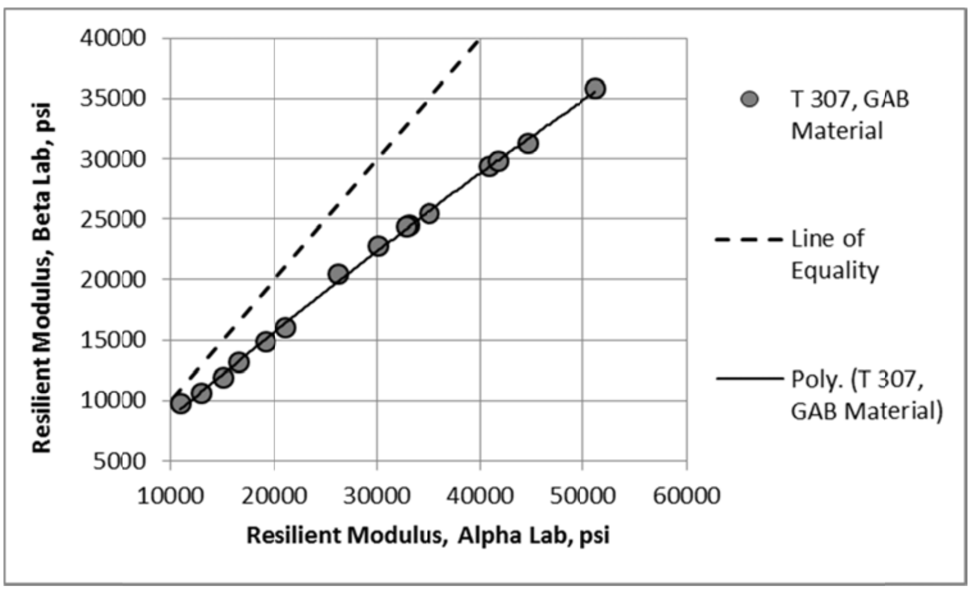

Figure 37. Comparison of Resilient Modulus between the Alpha and Beta Laboratories; GAB Material

CHAPTER 4 PRECISION AND BIAS TESTING

The purpose of the round robin testing plan was to determine the precision of measuring resilient modulus in accordance with AASHTO T 307 and NCHRP 1-28a, and determine if there is a bias between the two test methods. This chapter overviews the precision and bias testing plan and data analyses in determining the precision of the two test methods.

4.1 Precision Testing Factorial

A three-tiered full factorial was originally designed for the round robin test program. The three-tiers of the factorial were: test method, test or measurement system, and type of material. The remainder of this section discusses each tier of the testing factorial.

4.1.1 Test Method

Two test methods were included in the round robin or precision and bias test program: AASHTO T 307 and NCHRP 1-28a. The majority of the agencies contacted, as well as the majority of the PFS participating agencies use AASHTO T 307 on a routine basis. Table 2 summarized the agencies and the resilient modulus test method routinely performed by the agency, while Table 13 summarizes the laboratories that were contacted to participate in the study.

Table 13. Summary of Laboratories Initially Participating in the Round Robin Test Program

Laboratory	PFS Participating Laboratory?	Evaluated Using RD-02-034?	Test System	Test Procedure
Boudreau Engineering*	**Yes**	**Yes**	Instron	T-307
Burns Cooley Dennis*	**Yes**	**Yes**	Interlaken	T-307/1-28a
Alabama DOT*	**Yes**	**Yes**	Instron	T-307
Colorado DOT	**Yes**	No	IPC	T-307
Florida DOT*	**Yes**	No	Instron	T-307
Ground Engineering*	No	No	Instron	T-307
Idaho DOT*	No	No	GeoComp	T-307
Minnesota DOT	**Yes**	No	MTS	1-28A
North Carolina DOT*	No	**Yes**	Instron	T-307
Southern Poly Institute	No	No	Interlaken	T-307/1-28a
Texas DOT	**Yes**	No	MTS	T-307
Virginia DOT*	**Yes**	**Yes**	Instron	T-307
Virginia Transportation CIR	No	No	GeoComp	T-307
Wyoming DOT*	**Yes**	No	Interlaken	T-307

** Laboratories that participated in the precision and bias test program to determine the precision of AASHTO T 307. As shown, over half of the laboratories participating in the precision and bias test program were evaluated using RD-02-034.*

All laboratories were initially asked to measure the resilient modulus in accordance with both AASHTO T 307 and NCHRP 1-28a. As summarized in Table 2, however, only two of the participating laboratories have performed NCHRP 1-28a on other projects. Asking laboratories to perform the resilient modulus using a different procedure than routinely used by that agency was expected to decrease the precision of the test procedure or increase the standard deviation in the results. Thus, it was decided to request the participating laboratories to perform the resilient modulus test using the method routinely used to measure the resilient modulus.

4.1.2 Resilient Modulus Test Systems

Five test systems have been used to measure resilient modulus: Instron, Interlaken, MTS, GeoComp, and IPC. Only four of the test systems, however, are included in the PFS participating laboratories—none of the PFS laboratories have the GeoComp device or test system. Thus, other laboratories were contacted to participate in the precision and bias testing.

The experimental test plan was originally designed so that two laboratories with the same type of test system would be used to test each material for a total of ten laboratories. Laboratories were initially identified so that one of the two laboratories with the same test system was a commercial or university laboratory and the second a State highway agency laboratory. This “ideal” full factorial was impossible to fill.

All laboratories that participated in the precision and bias study were asked to complete the start-up evaluation process in accordance with the RD-02-034 guidelines. Some of the laboratories decided not to participate because of this request (see Table 13). As noted in Table 13, just over half of the participating laboratories in the precision and bias testing were evaluated using RD-02-034. In summary, six of the participating laboratories routinely use the Instron device, two use the Interlaken device, and only one uses the GeoComp device.

4.1.3 Material/Soil Types

Four soil or material types were included in the experimental test plan and were selected to cover the range of soils from stiff to soft. Two of these soils were defined as Type 1 or cohesionless coarse-grained soils (a granular aggregate base and an A-2-4 soil), and two as Type 2 or low and moderate to high plasticity cohesive fine-grained soils (an A-4 and A-7-6 soil). One of the Type 2 soils selected, however, turned out to be a Type 1 soil: A-2-4. Table 14 provides the soil and material types included in the experimental plan.

Basic soil properties and information was summarized and sent to each laboratory for preparing the test specimens. The data and information included details on sample preparation and compaction to determine the optimum volumetric condition, AASHTO classification, specific gravity, gradation, Atterberg limits, maximum dry density, and optimum moisture content (see Appendix C).

4.1.4 Sampling Soils

Sampling was a key issue so unbiased samples were packaged and shipped to different laboratories. Soils and materials were sampled from stockpiles. All materials were combined and split into equal samples by BCD or Boudreau laboratories (the Alpha and Beta laboratories included in the ruggedness test program discussed in Chapter 3).

Table 14. Types of Materials/Soils included in the Round Robin Test Program

Material or Soil Type		Sample Location	Material Description
Aggregate Base	Crushed Stone	Georgia; Martin-Marietta	Georgia crushed granite defined as GAB and used as an aggregate base for both flexible and rigid pavement construction.
Coarse-Grained Soils	A-2-4	Georgia; J.D. Stevens Pit	Silty sand with small amounts of gravel sampled from a pit.
	A-2-4 (selected as an A-4 soil)	Auburn, Alabama; NCAT	Low plasticity clayey sandy soil with amounts of gravel and larger aggregate particles; this soil had higher amounts of coarse-grained soil and turned out to be an A-2-4.
Fine-Grained Soils	A-7-6	Mississippi	High plasticity Mississippi river delta clay; sampled and processed from cuts within a construction project.

The National Center for Asphalt Technology (NCAT) A-2-4 material (selected as an A-4 soil) and the Georgia A-2-4 soil contained some gravel and larger aggregate particles. The larger particles were scalped from the prepared samples shipped to specific laboratories. This limited the amount of material for the precision testing and reduced the number of participating laboratories for testing these two soils. The following describes the sample preparation procedures for providing the soils to the participating laboratories which is dependent on test specimen size.

- Laboratories using 2.8-inch diameter by 5.6-inch height test specimens: Individual air-dried samples of 1,800 grams each were packaged in zip lock bags, marked, and shipped to the participating laboratories. The individual soil packages were randomly selected for shipping to a specific laboratory for test specimen preparation, compaction, and testing. The individual laboratories added the proper amount of water to achieve the optimum water content to compact the test specimens to 95 percent of the maximum dry unit weight (see section 4.1.5 of this chapter).

- Laboratories using 4.0-inch diameter by 8.0-inch height test specimens: Bulk air-dried soil samples of about 55 pounds each were marked and shipped to each laboratory. It was the responsibility of each participating laboratory to split the bulk sample into 5 equal and representative sample sizes for preparing the 5 replicate test specimens. Each laboratory added the proper amount of water to achieve the optimum water content to compact the test specimens to 95 percent of the maximum dry unit weight (see section 4.1.5 of this chapter).

- GAB 6.0-inch diameter test specimens: Air-dried bulk samples were marked and shipped to each laboratory with the capability for testing these larger sample sizes. It was the responsibility of each laboratory to split the bulk sample into 5 each and representative parts, add sufficient water to achieve the optimum water content, and compact the test specimens to 95 percent of the maximum dry unit weight (see section 4.1.5 of this chapter).

4.1.5 Test Specimen Condition

DOTs have different compaction specifications for different unbound materials and soils and use different compactive efforts in preparing or determining the optimum water content and maximum dry unit weight of the soil. The test plan did not consider these different conditions for each agency participating in the study. Thus, the testing plan was designed based on using one compactive effort or one set of specimen conditions (water content and dry density) for each soil type. That condition was the optimum water content and maximum dry unit weight as defined by AASHTO T 180 for aggregate base materials and AASHTO T 99 for all subgrade soils. It was most important that each participating laboratory achieve the correct (within tolerance) target density and moisture content. The optimum water content and maximum dry density for each soil are included in Table 15. The test specimens were compacted at the optimum water content and at a target dry density of 95 percent of the maximum dry density.

Determining the resilient modulus at the optimum water and maximum dry unit weight is consistent with the condition of the materials used in determining the default resilient modulus values included in the MEPDG. The MEPDG resilient modulus default values for flexible pavements are also included in Table 15. As shown, the MEPDG default optimum water contents are lower than the values for the soils included in this study while the MEPDG default maximum dry densities are higher, with the exception of the crushed stone base material.

Table 15. Test Specimen Condition for Precision and Bias Testing

Material or Soil Type		Study Target Values		MEPDG Default Values		
		Optimum Water Content, %	Maximum Dry Density, pcf	Optimum Water Content, %	Maximum Dry Density, pcf	Resilient Modulus, psi
Aggregate Base	Crushed Stone, A-1-a	6.0	144.0	7.4	127.2	30,000
Coarse-Grained Soils	A-2-4	10.6	120.2	9.0	124.0	16,500
	A-2-4 (selected as an A-4 soil)	14.6	110.1	11.8	118.4	15,000
Fine-Grained Soils	A-7-6	25.1	94.8	22.2	97.7	13,000

4.1.6 Test Specimen Replication

The experimental plan was originally designed using five replicates for each soil type and test method. Some previous studies have included the use of two to three replicates, but those are considered too few based on the COV that have been reported in the literature (Groeger and Bro, 2005; Boudreau and Wang, 2003; and unpublished studies). More importantly, five replicates were used because any anomaly in measuring the resilient modulus from one specimen will reduce the overall effect of the final results. Thus, five test specimens were prepared and tested within each cell. The resilient moduli were measured at each stress state and the individual values reported, as well as the mean and standard deviation for all replicate test specimens.

4.1.7 Testing and Test Specimen Instructions

Testing and experimental instructions specifying details for measuring and reporting the resilient modulus were provided to each participating agency. The instructions were based on the results from the ruggedness test program discussed in Chapter 3.

4.1.8 File Naming Process

The following specimen title convention was used: laboratory code, soil type, replicate number, test method, and test system. In addition to the core data submittal requirement, the following information was requested for the series of tests:

- Triaxial cell manufacturer, model number, approximate date placed into service.
- Porous stone supplier, physically measured diameter and thickness.
- Latex membrane supplier, physically measured thickness.
- Load cell manufacturer, model number, capacity.
- LVDT manufacturer, model number, capacity.
- A statement regarding calibration of the load cell and LVDTs (internal quality assurance procedures similar to AASHTO R 18 require calibrations on a specific cycle, and if an outside calibration company was used to perform the calibration). It was important that the dynamic verification system be completed prior to the precision and bias testing.

4.2 Experimental Hypotheses

Each laboratory participating in the study was asked to perform the repeated load resilient modulus test in accordance with these tests and tolerance deviations defined from the ruggedness test plan explained in Chapter 3. The test results from the laboratories listed in Table 13 for the soils identified in Table 14 provided the comparative data to determine the precision of AASHTO T 307 and NCHRP 1-28a test methods. In addition, the resilient modulus data were used to determine whether a bias or consistent difference exists between AASHTO T 307 and NCHRP 1-28a. However, only a few agencies use NCHRP 1-28a; most agencies use AASHTO T 307 (see Tables 2 and 13; only about 15 percent of the agencies use NCHRP 1-28a). As such, the difference in measured resilient modulus between AASHTO T 307 and NCHRP 1-28a was based on results from the ruggedness test program. This section of Chapter 4 provides a brief discussion of the experimental hypotheses for evaluation bias, while a latter section of this chapter provides more detailed discussion on the data analyses to evaluate the different hypotheses.

The resilient modulus test results measured using LTPP P46 and included in the LTPP database are expected to be more similar to those obtained from AASHTO T 307. NCHRP 1-28a, however, is perceived to be a more precise test, while AASHTO T 307 is considered a more practical test being run on a production basis. It is hypothesized that the NCHRP 1-28a method will result in higher resilient modulus values than the AASHTO T 307 method, because of potential stress and confinement effects. Thus, one question to be answered as part of the testing plan: is there a significant difference in resilient modulus and precision between the two test methods relative to predicting service life and determining pavement layer thicknesses? The hypothesis for this part of experimental plan was:

- The NCHRP 1-28a test method results in a higher and more precise estimate of resilient modulus as compared to AASHTO T 307. This hypothesis was evaluated using the roughness testing data, as discussed in Chapter 3.

The experimental plan was also designed to evaluate the precision of the test method using different test systems, but not the bias of the test method. Bias can only be addressed between the two test methods, as noted above. To determine the bias of a test method, the "true value" needs to be known or measured by another method. The true value for each soil was unknown, so the true bias could not be determined. The results from the test program were used to determine the resilient modulus of the soil and/or aggregate base of thin and thick (less stiff and very stiff) pavement structures to determine the in-place resilient modulus for a specific condition. These results were compared to the input level 3 default resilient modulus values included in the MEPDG. This comparison resulted in determining if a bias exists between the values used in the global calibration process to those measured by either of the test methods. The null hypothesis for this part of experimental plan was:

- The NCHRP 1-28a test method results in resilient modulus values that are indifferent to the default values included in the MEPDG.

- The AASHTO T-307 test method results in resilient modulus values that are indifferent to the default values included in the MEPDG.

4.3 Comparison of Test Results

Appendix D provides a graphical illustration of the resilient modulus test results for all of the materials tested. All participating laboratories measured the resilient modulus for the Mississippi A-7-6 soil and NCAT A-2-4 soil (selected as an A-4 soil). Some of the laboratories did not measure the resilient modulus for the Georgia A-2-4 coarse-grained soil, because of equipment related issues that occurred during the test program and/or the time required for testing all soils. Only three laboratories measured the resilient modulus of the GAB material. Many of the participating laboratories did not have the larger triaxial cells needed for the larger aggregate size.

Figures 38 to 48 graphically compare the resilient moduli measured between the different laboratories. As shown, reasonable trends exist between the resilient moduli reported from the different laboratories, except for laboratory C and I. Tables 16 to 19 include the average resilient modulus for two stress states for the four soils/materials included in the precision and bias test plan. Figures 42 to 49 are graphs that simply illustrate the difference of results between the different laboratories for the lower stress state of each material. The standard deviation of test results for the individual laboratories are also shown or included in each figure.

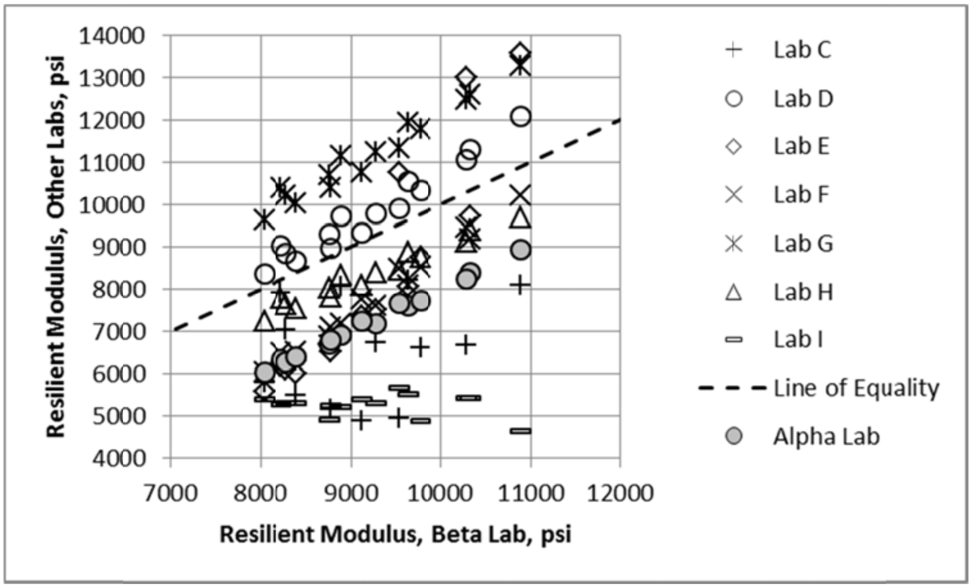

Figure 38. Resilient Moduli Reported from all Laboratories for the A-7-6 Soil

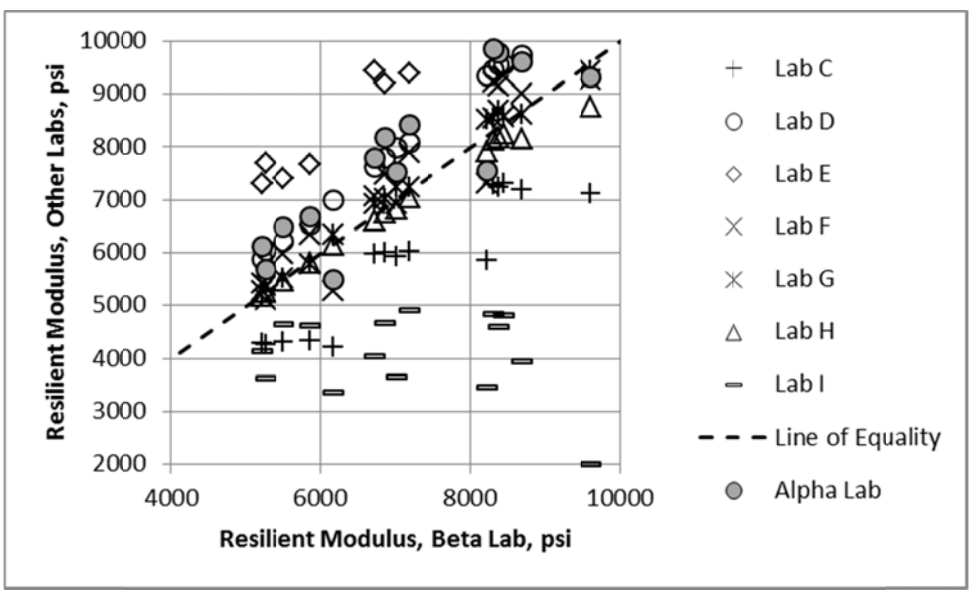

Figure 39. Resilient Moduli Reported from all Laboratories for the NCAT A-2-4 Soil

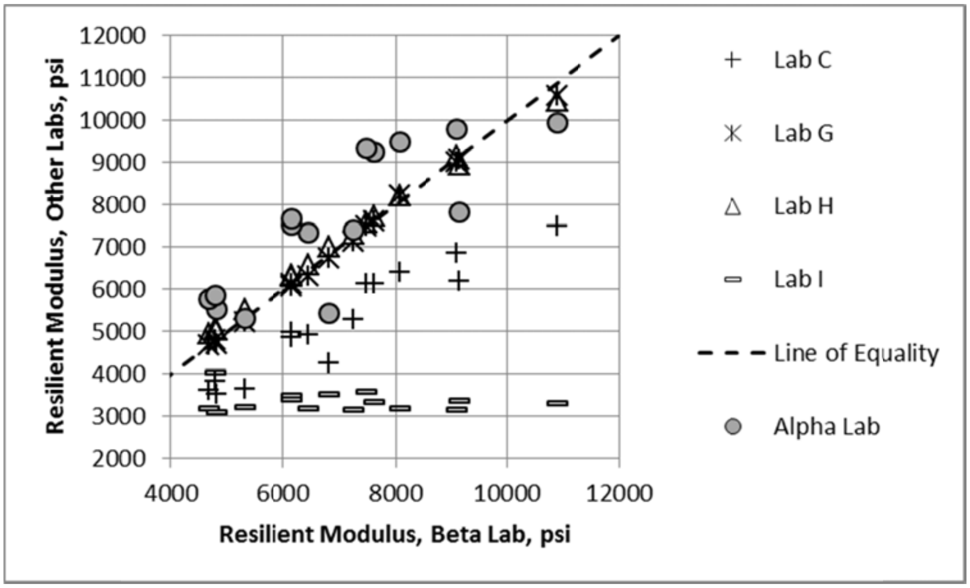

Figure 40. Resilient Moduli Reported from all Laboratories for the Georgia A-2-4 Soil

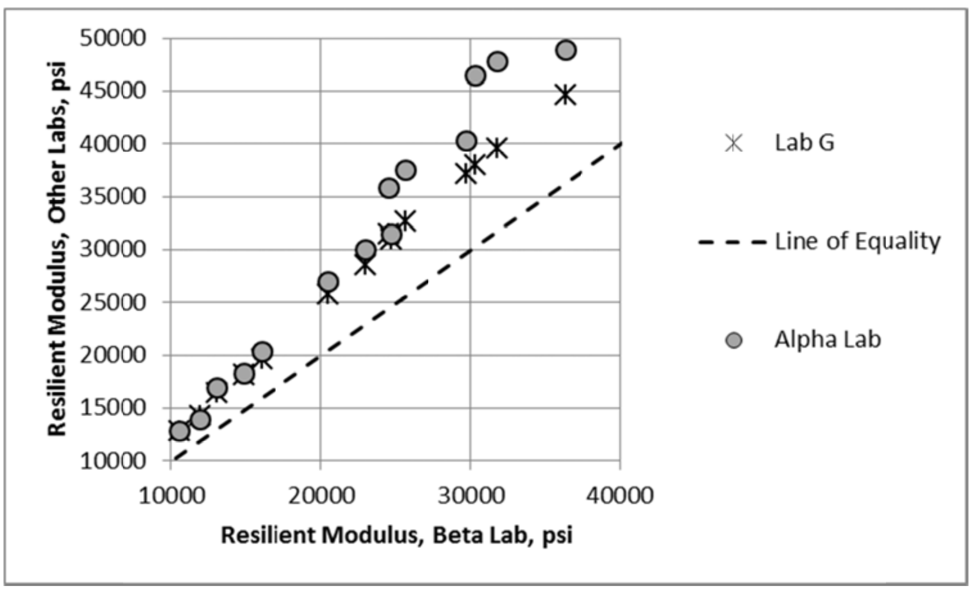

Figure 41. Resilient Moduli Reported from all Laboratories for the GAB Material

Table 16. Resilient Modulus Reported for the A-7-6 High Plastic Clay Soil

Laboratory Identification	Average Resilient Modulus, psi		Coefficient of Variation, %	
	Stress State 1	Stress State 2	Stress State 1	Stress State 2
A	8,928	6,035	9.00	9.33
B	10,887	8,036	5.73	6.73
C	8,103	5,786	2.44	2.52
D	12,068	8,354	4.83	5.58
E	13,569	5,589	7.53	5.22
F	10,219	6,020	7.72	10.02
G	13,274	9,638	5.28	7.85
H	9,695	7,241	3.64	3.68
I	4,633	5,383	14.30	6.97
Stress state 1 is 6 psi confinement and 2 psi vertical stress, while stress state 2 is 2 psi confinement and 10 psi vertical stress.				

Table 17. Resilient Modulus Reported for the A-2-4 NCAT Low Plasticity Soil

Laboratory Identification	Average Resilient Modulus, psi		Coefficient of Variation, %	
	Stress State 1	Stress State 2	Stress State 1	Stress State 2
A	9,313	6,677	2.01	2.91
B	9,602	5,854	3.60	2.41
C	7,099	4,325	4.68	3.56
D	10,459	6,525	3.82	3.19
E	22,406	7,656	9.68	10.53
F	9,258	6,337	8.70	6.02
G	9,431	5,788	4.15	3.39
H	8,763	5,795	2.73	1.94
I	3,359	4,746	31.76	8.76
Stress state 1 is 6 psi confinement and 2 psi vertical stress, while stress state 2 is 2 psi confinement and 10 psi vertical stress.				

Table 18. Resilient Modulus Reported for the A-2-4 Georgia Soil

Laboratory Identification	Average Resilient Modulus, psi		Coefficient of Variation, %	
	Stress State 1	Stress State 2	Stress State 1	Stress State 2
A	9,943	5,861	3.83	3.03
B	10,879	4,788	6.98	2.75
C	7,481	3,825	3.46	2.94
D	NA	NA	NA	NA
E	NA	NA	NA	NA
F	NA	NA	NA	NA
G	10,555	4,793	3.57	2.55
H	10,441	5,054	3.10	3.88
I	3,296	4,022	19.54	2.22
Stress state 1 is 6 psi confinement and 2 psi vertical stress, while stress state 2 is 2 psi confinement and 10 psi vertical stress.				

Table 19. Resilient Modulus Reported for the Crushed Stone Aggregate Base

Laboratory Identification	Average Resilient Modulus, psi		Coefficient of Variation, %	
	Stress State 1	Stress State 2	Stress State 1	Stress State 2
A	20,200	48,807	0.86	0.68
B	16,065	36,349	5.72	4.18
C	NA	NA	NA	NA
D	NA	NA	NA	NA
E	NA	NA	NA	NA
F	NA	NA	NA	NA
G	19,554	44,601	4.89	5.51
H	NA	NA	NA	NA
I	NA	NA	NA	NA
Stress state 1 is 5 psi confinement and 15 psi vertical stress, while stress state 2 is 20 psi confinement and 40 psi vertical stress.				

Based on an analysis of the test results some important observations were made in estimating the single operator and multi-laboratory precision of the test methods. These observations relate to a significant deviation or difference for one of the laboratories, different stress sensitivities measured, the difference between the locations of the LVDTs (within versus outside chamber deformation measurement locations), and the dependency of the standard deviation of resilient modulus on the average resilient modulus value.

- Laboratory I consistently measured significantly lower resilient modulus than any of the other laboratories. Figures 50 and 51 illustrate the resilient modulus for Laboratory I, while Figures 52 and 53 include an illustration of results from Laboratory H and is typical of the other laboratories. An analysis was completed to determine if laboratory I was considered an outlier or anomaly. The average resilient modulus for each soil was more than three standard deviations from the mean value of all other laboratories. As such, laboratory I was considered an outlier and was excluded from all analyses to determine the precision and bias of the test methods. In addition, the effect of the confining pressure was not found in comparison to the other laboratories. This suggests an issue with confinement during the test, based on the experience of the authors. It is expected there was an issue with the seals of the membrane such that little to no confinement was exhibited during testing. Yau and Von Quintus used this type of response in evaluating the resilient modulus test results stored within the LTPP database (2002). As such, the results from Laboratory I were excluded from the data analysis to determine the precision of the test methods.

- Laboratory E measured an extremely high resilient modulus for the NCAT A-2-4 soil at the lower bulk stresses, while the measured resilient modulus from laboratory E are similar to the resilient moduli reported by the other laboratories for the higher bulk stresses. This results in higher stress sensitivity than measured by the other laboratories. In addition, laboratory E consistently measured higher stress sensitivity for the soils with the higher optimum water contents (Mississippi A-7-6 and NCAT A-2-4; see Table 15) than the other laboratories. Figure 54 is a bar chart showing the difference in resilient modulus measured between two stress states for the soils included in the study (resilient modulus measured at the highest confinement/lowest vertical stress minus lowest confinement/highest vertical stress). The reason for this difference is unknown. Thus, laboratory E is considered an anomaly or outlier relative to the high confinement and lower deviator stresses.

[The list of observations continues after Figure 54.]

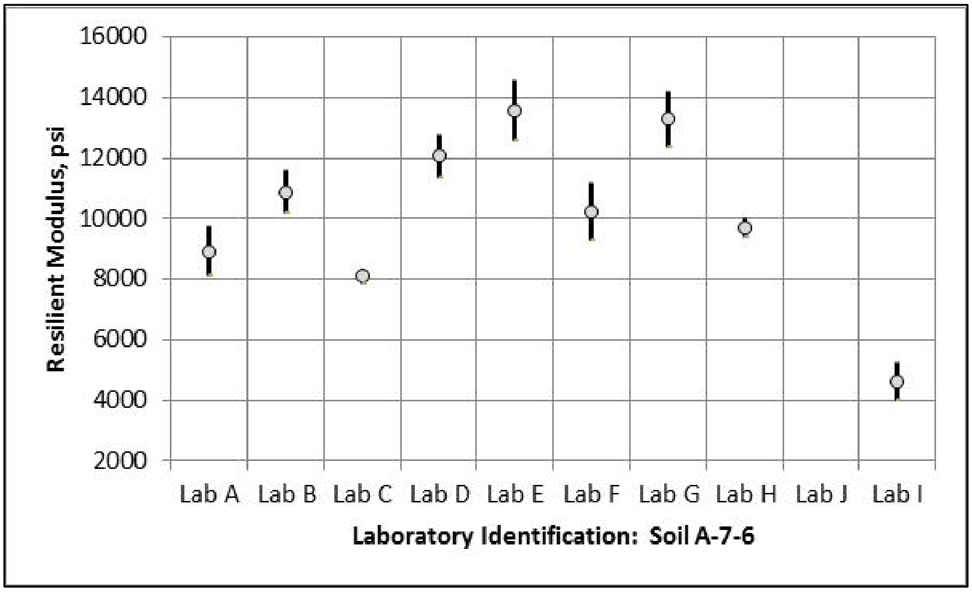

Figure 42. Difference in Resilient Modulus for the A-7-6 Soil; All Participating Laboratories

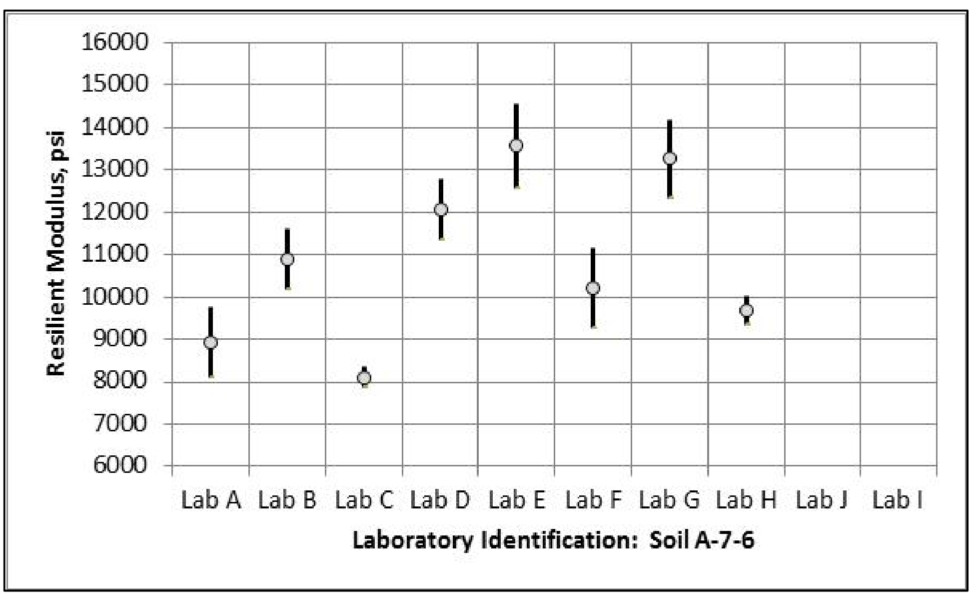

Figure 43. Difference in Resilient Modulus for the A-7-6 Soil; Excludes Laboratory I

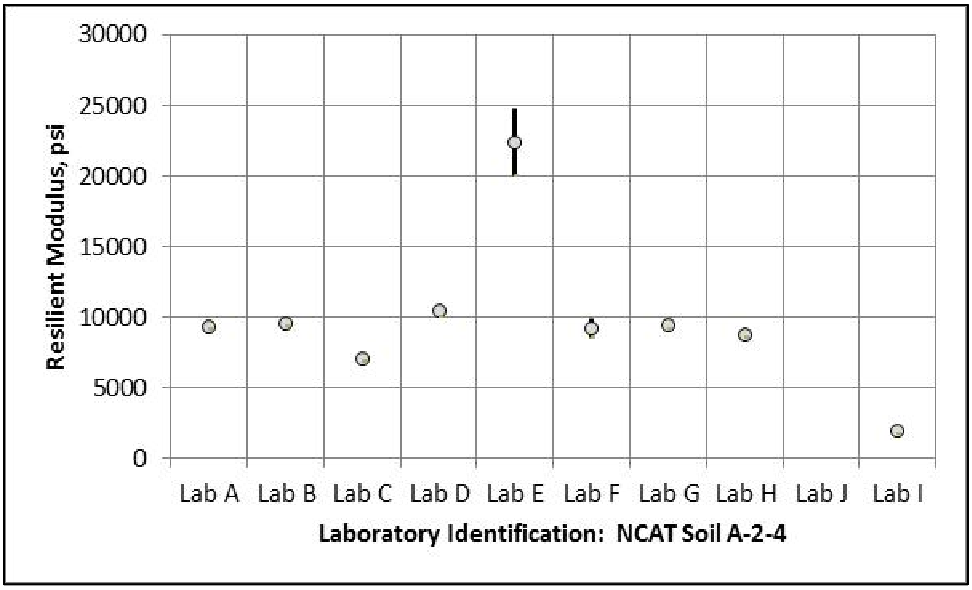

Figure 44. Difference in Resilient Modulus for the NCAT A-2-4 Soil; All Participating Laboratories

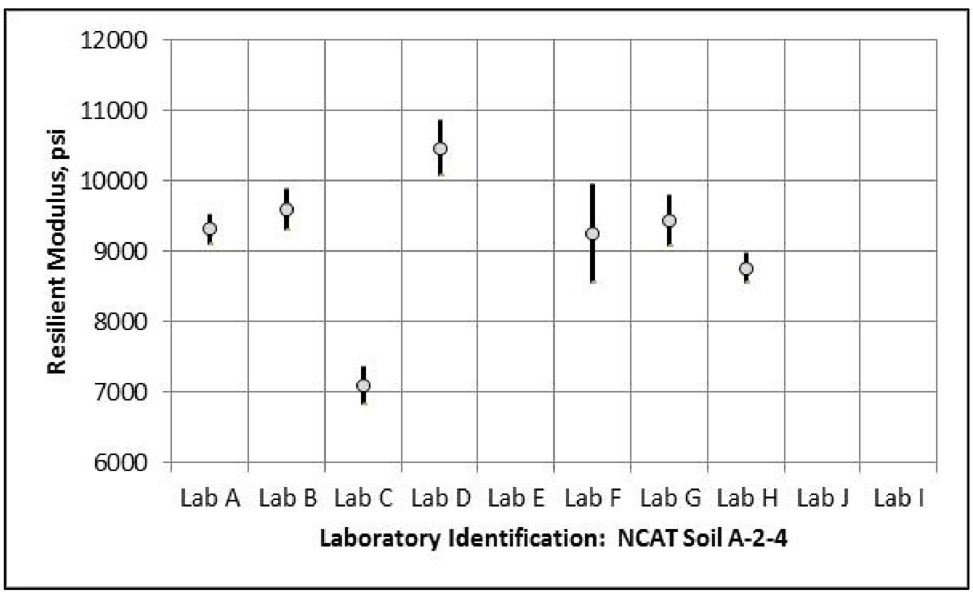

Figure 45. Difference in Resilient Modulus for the NCAT A-2-4 Soil; Excludes Laboratories E and I

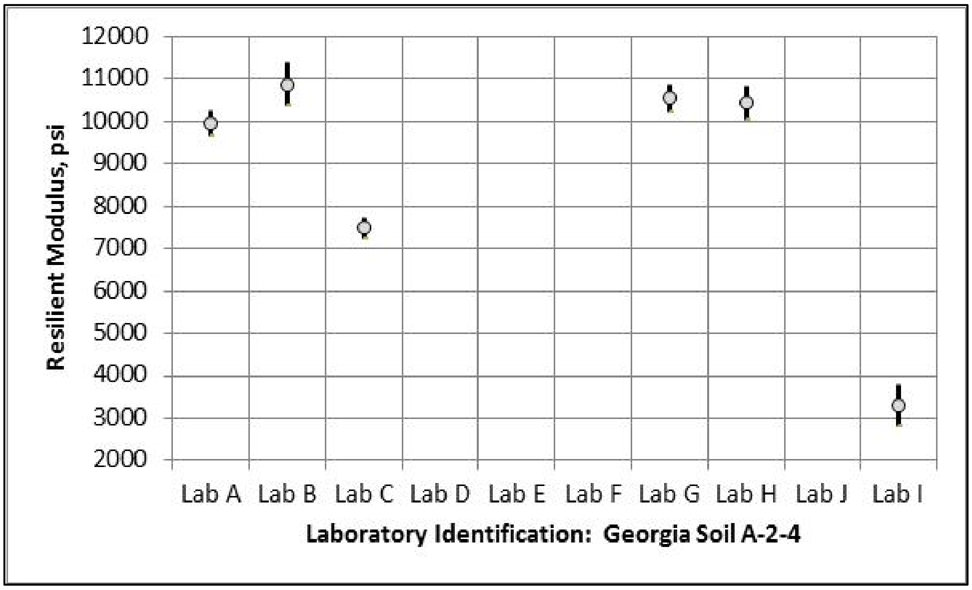

Figure 46. Difference in Resilient Modulus for the Georgia A-2-4 Soil; All Participating Laboratories

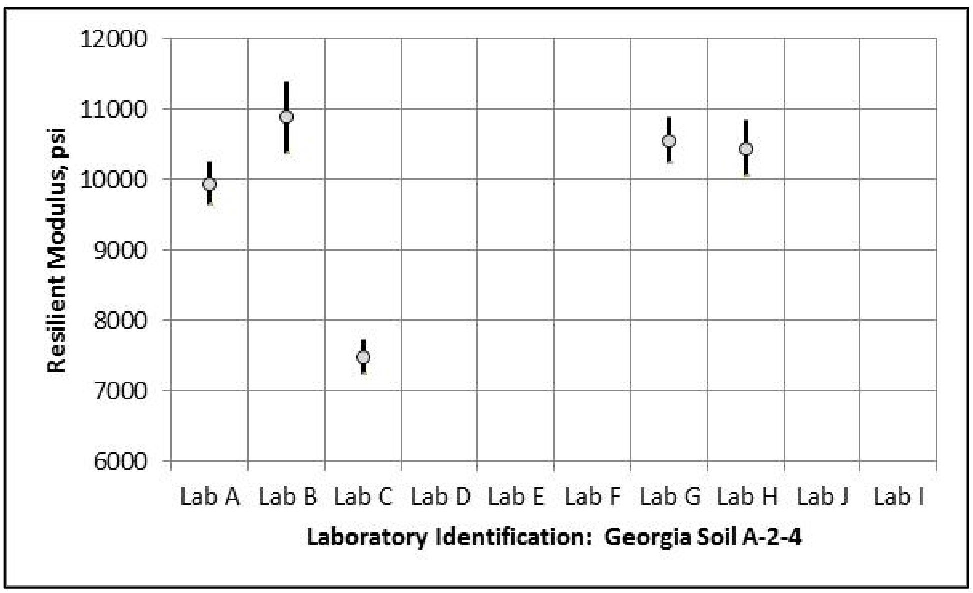

Figure 47. Difference in Resilient Modulus for the Georgia A-2-4 Soil; Excludes Laboratory I

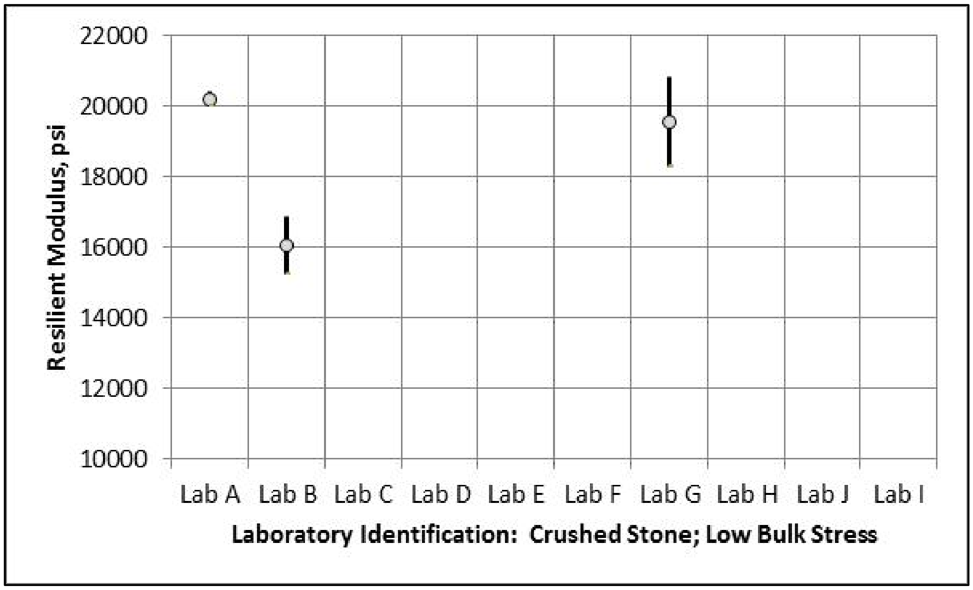

Figure 48. Difference in Resilient Modulus for the Crushed Stone Base Material; All Participating Laboratories, Low Bulk Stress Level

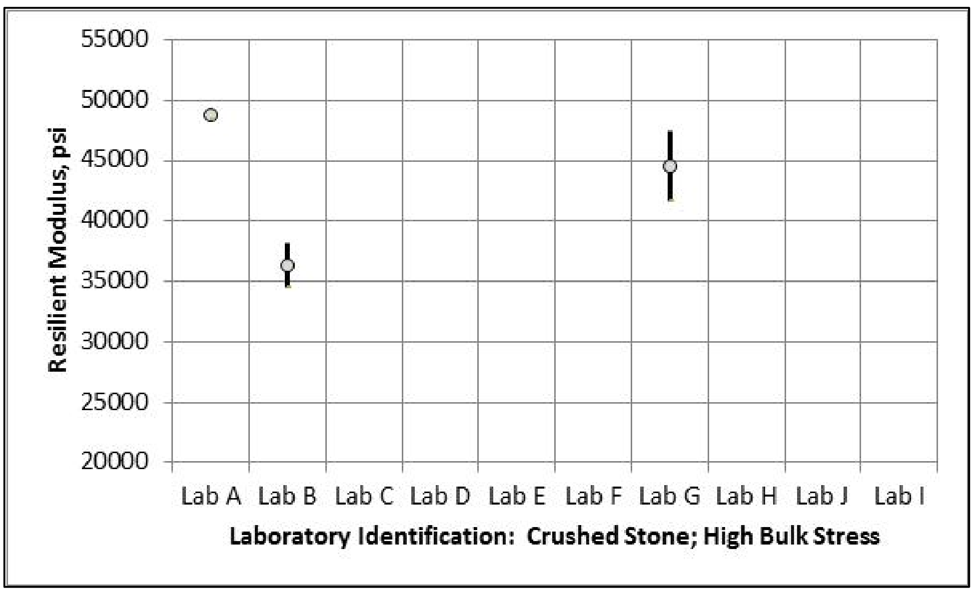

Figure 49. Difference in Resilient Modulus for the Crushed Stone Base Material; All Participating Laboratories, High Bulk Stress Level

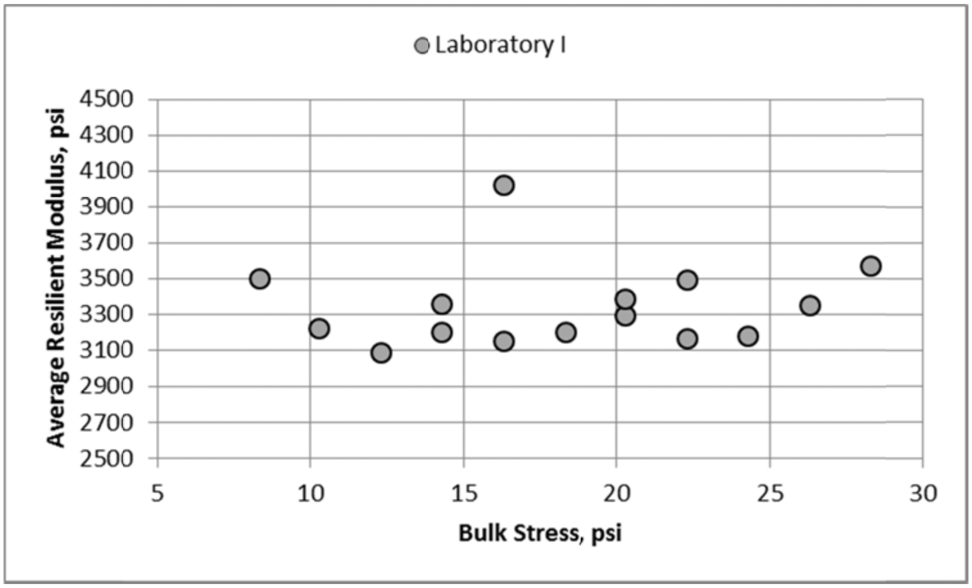

Figure 50. Resilient Modulus Values Measured by Laboratory I as a Function of Stress State for the Georgia A-2-4 Silty Sand Soil

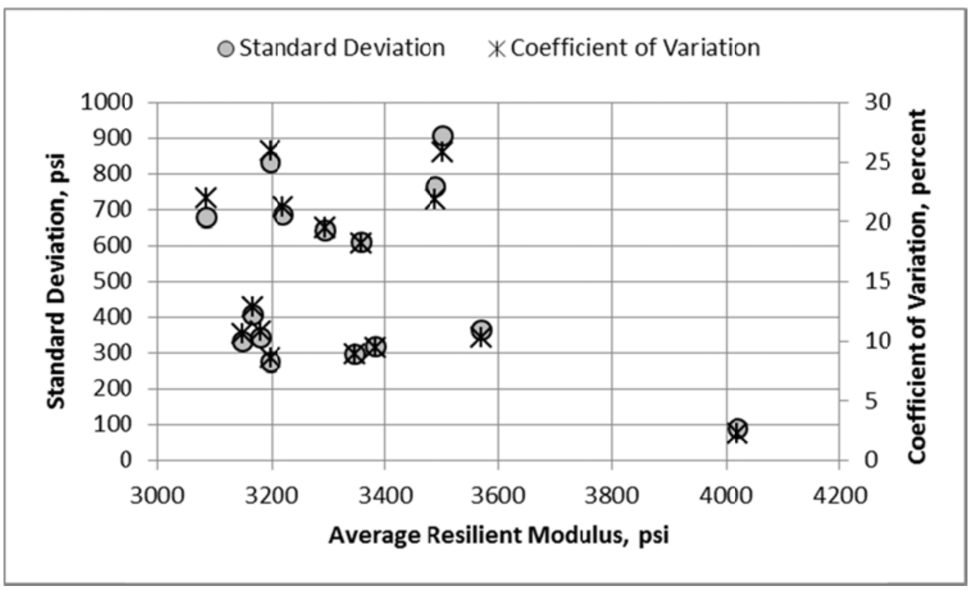

Figure 51. Variation of Resilient Modulus Values for Different Stress States as Measured by Laboratory I for the Georgia A-2-4 Silty Sand Soil

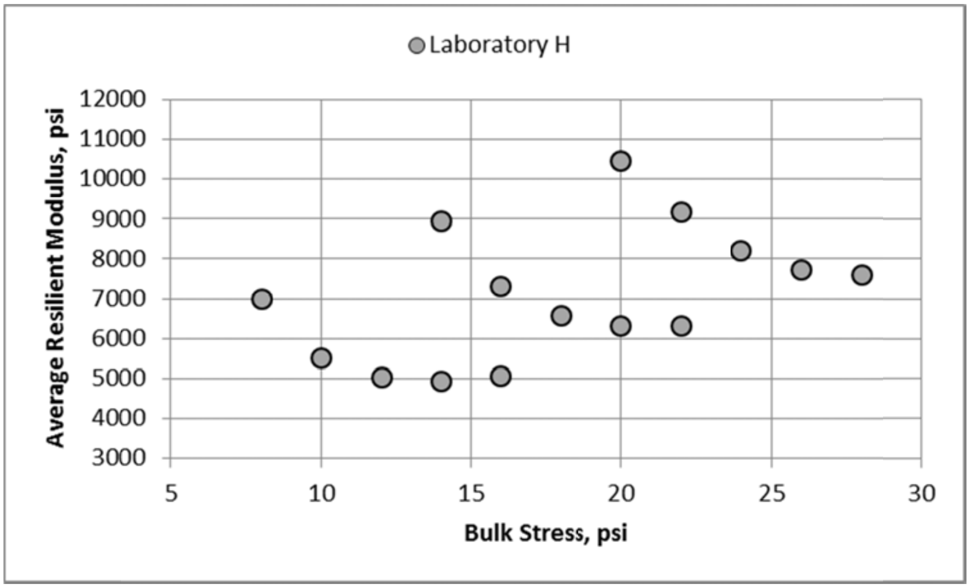

Figure 52. Resilient Modulus Values Measured by Laboratory H as a Function of Stress State for the Georgia A-2-4 Silty Sand Soil

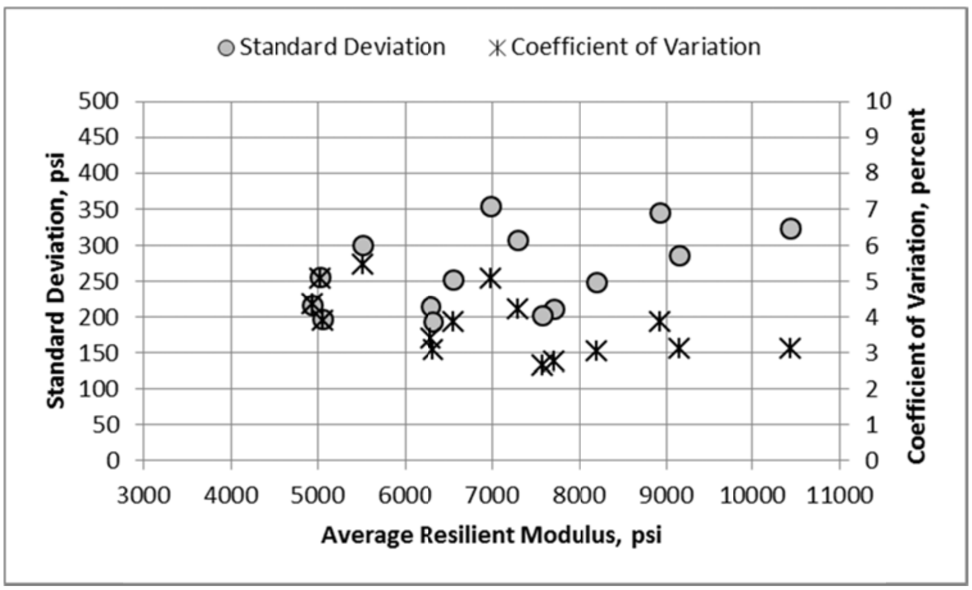

Figure 53. Variation of Resilient Modulus Values for Different Stress States as Measured by Laboratory H for the Georgia A-2-4 Silty Sand Soil

Note: The difference along the y-axis in Figure 54 equals the resilient modulus measured at highest confinement & lowest vertical stress minus the resilient modulus measured at lowest confinement & highest vertical stress.

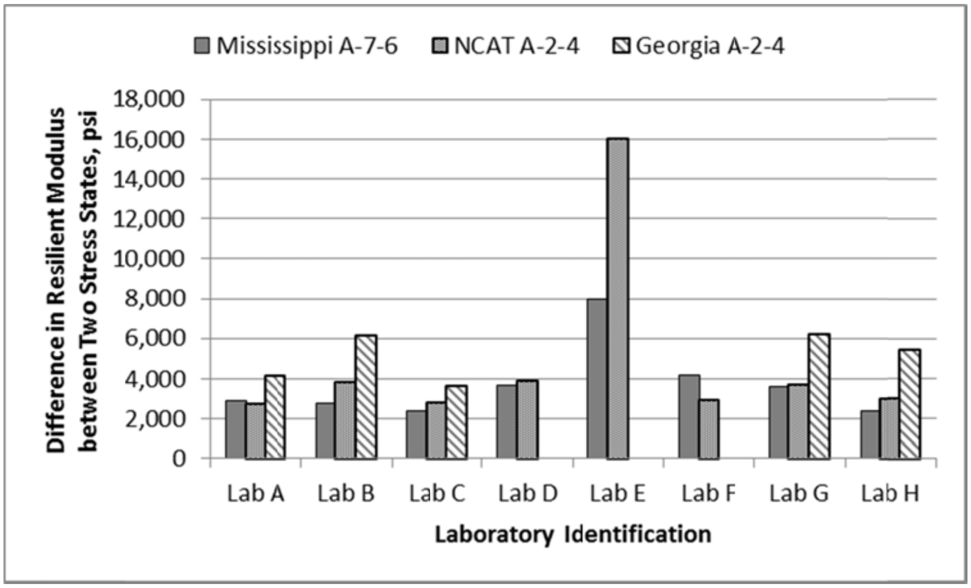

Figure 54. Average Difference in Resilient Modulus between Two Stress States

- Figures 55 and 56 show the average resilient modulus values for the soils, excluding laboratory I for the stress state with the lower confinement and higher deviator stress level. The standard deviation of test results for the laboratories are also shown or included in Figures 55 and 56. As shown, laboratory C also measured significantly lower resilient modulus values for the embankment soils than for the other laboratories. Laboratories C and I used different test systems. Laboratory C, however, was not considered an anomaly or outlier. Conversely, laboratories D and E exhibited the higher resilient modulus values measured for the soils (see Tables 16 and 17). Laboratories C, D, and E all used the same test system.

- The other important observation from Figures 55 and 56 is that the Mississippi A-7-6 high plasticity clay soil tested at the optimum water content has a much higher resilient modulus than for the other A-2-4 lower plasticity soils tested at their optimum water content. This observation or test result is just the opposite of the global default resilient modulus values included in the MEPDG (see Table 15), but is consistent with the test results from many other studies conducted by the authors.

- Another significant difference found in the test data was related to the location of the LVDTs in measuring deformation but only for the highly plastic clay soil (A-7-6 soil). Figures 57 and 58 illustrate the difference in resilient modulus as measured by different locations of the LVDTs: outside versus inside the triaxial cell or testing chamber. As shown, the resilient modulus measured on the A-7-6 soil from LVDTs located outside the testing chamber are consistently greater than for the LVDTs located inside the testing chamber. Figures 57 and 58 also include a comparison of the resilient modulus measured on the A-2-4 soils between the LVDTs located inside and outside the testing chamber. As shown, the difference was much smaller for the coarse-grained, lower plasticity soils. Thus, the location of the LVDTs were kept separated in evaluating the precision of the test methods for the A-7-6 soil, while for the coarse-grained, lower plasticity A-2-4 soils all results were combined.

- The final observation from the test results is that the standard deviation of the resilient modulus for the GAB material is dependent on the average resilient modulus. This dependency is illustrated in Appendix D and in Figure 59 for one of the laboratories. As shown, the standard deviation is resilient modulus dependent, while the coefficient of variation (COV) is fairly uniform or constant between the different magnitudes of resilient modulus. As such, the coefficient of variation was used to estimate the standard deviation for different magnitudes of resilient modulus values for the processed GAB material. Conversely, the standard deviation of the resilient modulus for the high and low plasticity soils was not typically dependent on the average resilient modulus (see Figure 60).

- The test results were grouped by different factors to try and explain some of the differences between the different laboratories.

    - The following lists the average resilient modulus values for the different soils for the two primary test devices or equipment included in the precision and bias part of the study. As shown, no consistent difference in results was identified. In fact, there was as much variability or difference in results for the same device as between the different devices. Thus, the different equipment does not explain the variation in test results between laboratories.

Equipment (Number of Labs)	Soil (Stress State 1)			Soil (Stress State 2)		
	A-7-6; Mississippi	A-2-4; NCAT	A-2-4; Georgia	A-7-6; Mississippi	A-2-4; NCAT	A-2-4; Georgia
Instron (6)	11,266	9,071	9,839	6,766	5,657	4,615
Interlaken (3)	9,574	9,286	9,943	6,028	6,507	5,861

    - The following lists the average resilient moduli for the different soils for the two test specimen sizes or diameters included in the precision and bias part of the study. As shown, no consistent difference in results was identified. In fact, there was as much variability or difference in results for the same test specimen diameter as between the two specimen sizes. Thus, specimen size does not appear to explain the variation in test results between laboratories.

Test Specimen Diameter (Number of Labs)	Soil (Stress State 1)			Soil (Stress State 2)		
	A-7-6; Mississippi	A-2-4; NCAT	A-2-4; Georgia	A-7-6; Mississippi	A-2-4; NCAT	A-2-4; Georgia
2.8-inches (5)	11,196	9,677	9,943	6,500	6,513	5,861
4.0-inches (4)	10,490	8,724	9.839	7,675	5,441	4,615

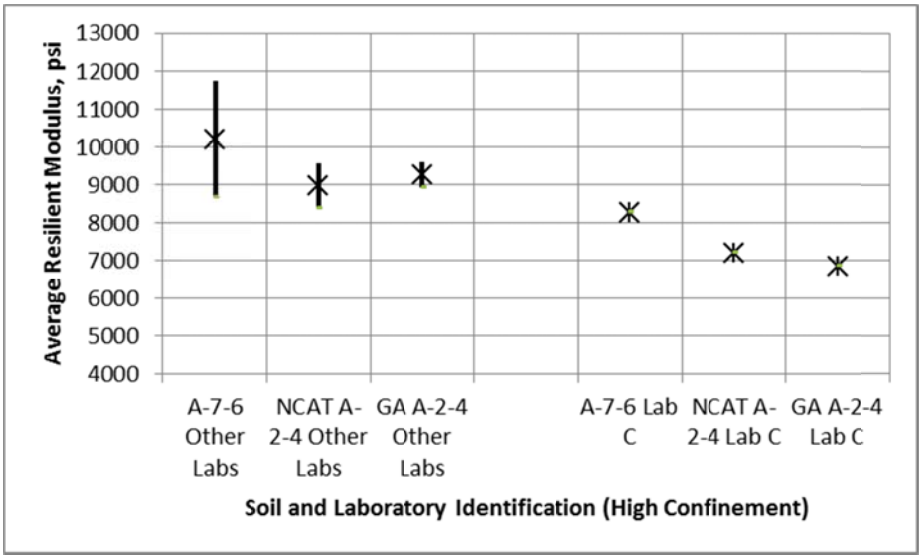

Figure 55. Average Resilient Modulus Measured at High Confinement (6 psi) and 4 psi Vertical Stress for the Different Embankment Soils

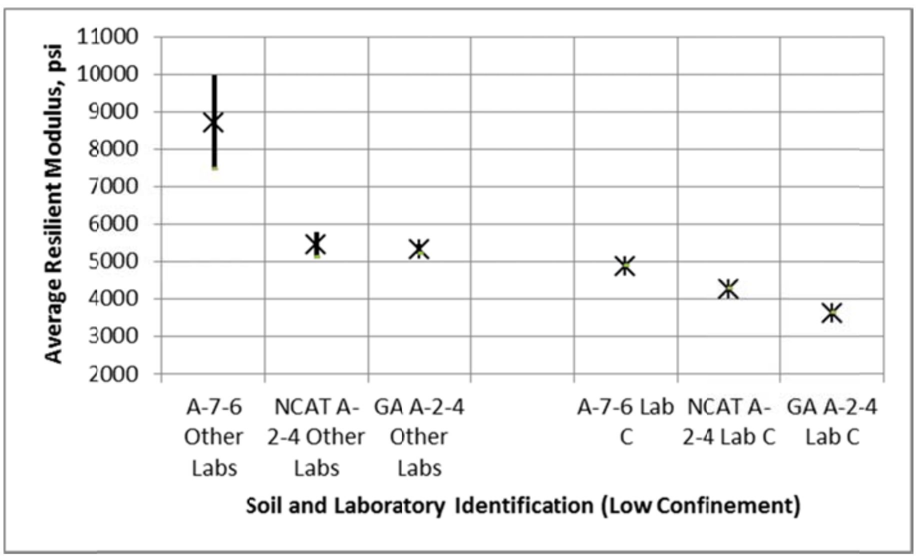

Figure 56. Average Resilient Modulus Measured at Low Confinement (2 psi) and 4 psi Vertical Stress for the Different Embankment Soils

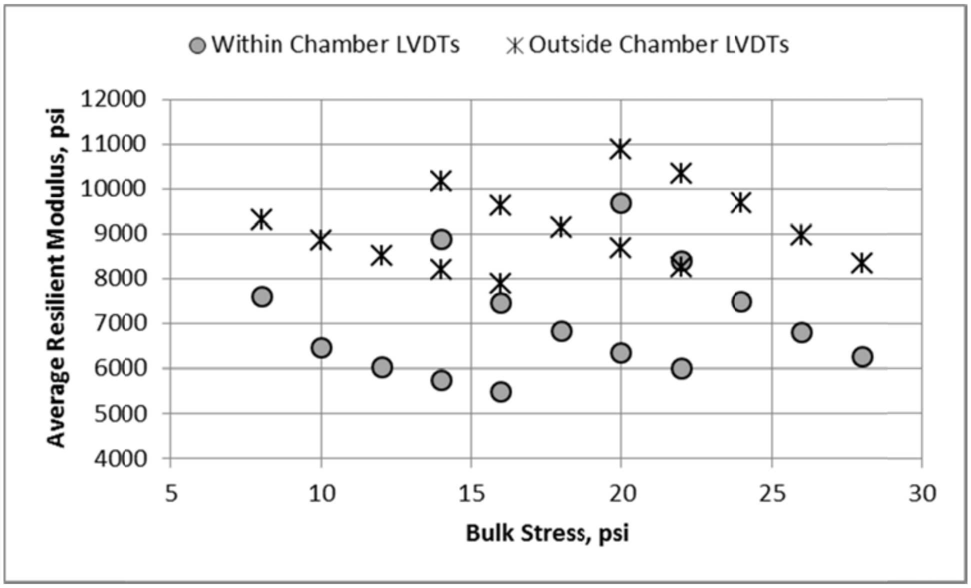

Figure 57. Comparison of Resilient Modulus Measured with different Locations of the LVDTs between Two Laboratories

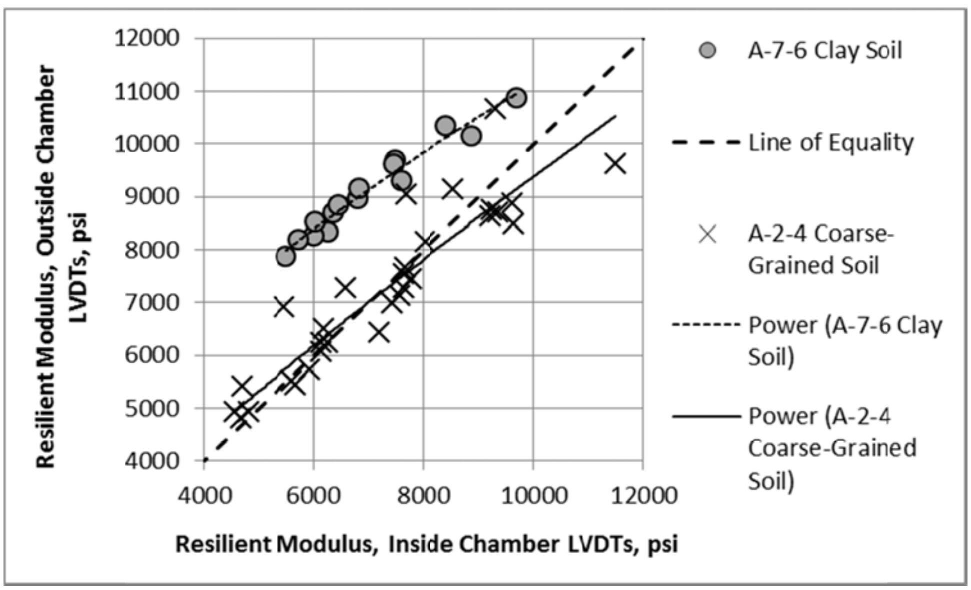

Figure 58. Comparison of Resilient Modulus Measured with different Locations of the LVDTs between All Participating Laboratories

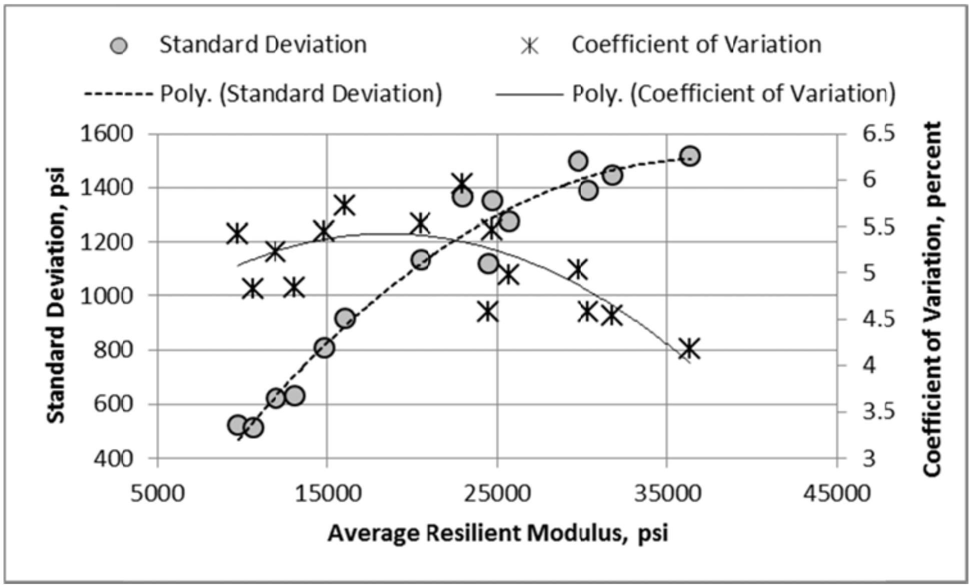

Figure 59. Single Operator Standard Deviation and Coefficient of Variation over a Range of Resilient Moduli; Laboratory B, GAB Material

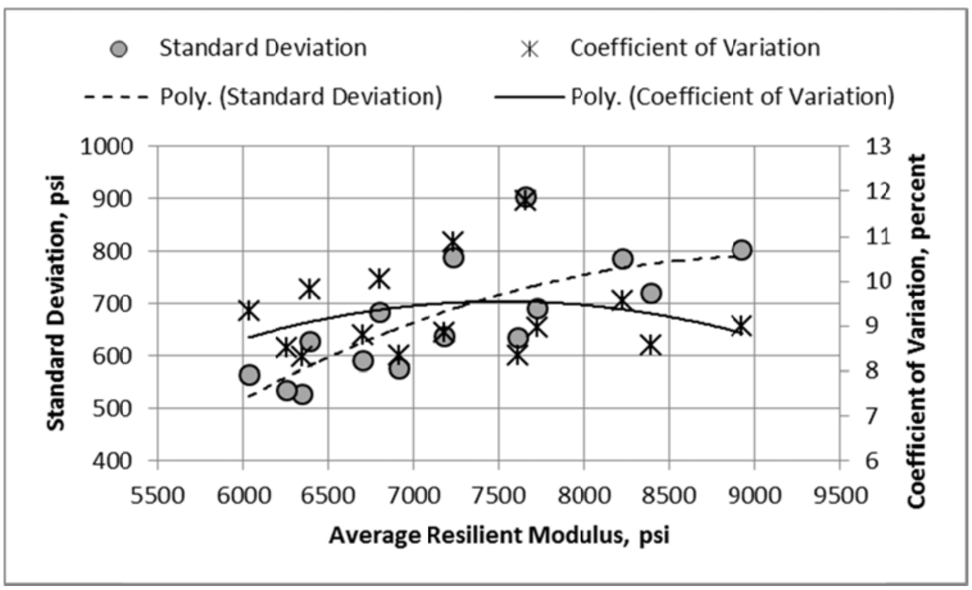

Figure 60. Single Operator Standard Deviation and Coefficient of Variation over a Range of Resilient Moduli; Laboratory A, A-7-6 Soil

4.4 Data Analyses: Determination of Precision and Bias

4.4.1 Precision of Resilient Modulus Test

For a precision statement, a one–sigma limit **(1S)** is used to denote the estimate of the standard deviation of the population of the test method. This limit is usually established for a single operator and for multi–laboratory precision. Another expression found in precision statements is the acceptable range of results, called the "difference two–sigma limit" **(D2S).** This index indicates a maximum acceptable difference between two results obtained on test portions of the same material. It is the difference between two individual test results that would be equaled or exceeded in the long run in only 1 case in 20:

$$D2S = 2\sqrt{2}(1S) \tag{6}$$

An insufficient number of laboratories measured the resilient modulus in accordance with NCHRP 1-28a to define its precision. As such, the test results from the ruggedness test program were used to estimate the precision of the NCHRP 1-28a test method in relation to AASHTO T 307. The following restates the hypothesis related to the precision of the test methods noted at the beginning of Chapter 4 under Section 4.2.

- Hypothesis 1: The NCHRP 1-28a test method results in a higher and more precise estimate of resilient modulus as compared to AASHTO T 307. This hypothesis was evaluated using the roughness test data, as discussed in Chapter 3. This hypothesis was rejected based on the results from the Alpha laboratory that routinely uses both test methods.

Hypothesis 1 was evaluated over different stress states to determine if stress state is an important factor in the single operator precision of the resilient modulus tests. The precision of the test methods, in terms of the standard deviation, is dependent on stress state for the GAB material and less dependent on stress state for the A-7-6 and A-2-4 soils (see Figures 59 and 60 and Appendix D). As such, the following question was evaluated: Is the difference significant in terms of the MEPDG software predictions? The answer to this question is discussed in Chapter 5 using the methodology suggested for determining the resilient modulus as an input to the Pavement ME Design software.

Table 20 summarizes the precision in terms of a single operator and multiple laboratories using the coefficient of variation calculated from the test results, excluding those laboratories that were identified as exhibiting anomalies or outliers. Table 21 summarizes the precision of the test but based on the measured standard deviations from the test results, excluding those same laboratories with anomalies or outliers. The average coefficient of variation for each soil is included in Table 22.

4.4.2 Bias of Test Method

Significant differences were found between the resilient moduli reported by some of the laboratories, as noted above. Laboratories C, E, and I had significantly different resilient moduli. The reason for this difference, however, was not identified or attributed to anyone particular factor and/or test device. Thus, additional work within this area is needed to clearly explain the reason for these differences. It is expected that the location of the LVDTs and handling/test specimen preparation procedures may be contributors or confounding factors within the precision and bias testing plan.

Table 20. Precision of Resilient Modulus Test Using the Calculated Coefficient of Variation

Material or Soil Type	Magnitude of Resilient Modulus, psi	Standard Deviation, 1S, psi		Precision, D2S, psi	
		Single Operator	Multiple Laboratories	Single Operator	Multiple Laboratories
Aggregate Base; GAB and Crushed Stones	10,000	420	1,300	1,188	3,677
	20,000	840	2,600	2,376	7,354
	30,000	1,260	3,900	3,564	11,031
	40,000	1,680	5,200	4,552	14,708
Low Plasticity, Coarse and Fine-Grained Soils; A-2-4 and A-4	5,000	210	594	410	1,160
	10,000	420	1,188	820	2,319
	15,000	630	2,758	1,230	3,479
High Plasticity, Fine-Grained Soils A-7-6	5,000	325	865	919	2,447
	10,000	650	1,730	1,838	4,893
	15,000	975	2,595	2,758	7,340

Table 21. Precision of Resilient Modulus Test Using the Measured Standard Deviation

Material or Soil Type	Magnitude of Resilient Modulus, psi	Standard Deviation, 1S, psi		Precision, D2S, psi	
		Single Operator	Multiple Laboratories	Single Operator	Multiple Laboratories
Aggregate Base; GAB and Crushed Stones	10,000	420	1,300	1,188	3,677
	20,000	840	2,600	2,376	7,354
	30,000	1,260	3,900	3,564	11,031
	40,000	1,680	5,200	4,552	14,708
Low Plasticity, Coarse and Fine-Grained Soils; A-2-4 and A-4	NA	259	600	733	1,697
High Plasticity, Fine-Grained Soils A-7-6	NA	544	1,475	1,539	4,172

Table 22. Average Coefficient of Variation for the Soils included in Precision and Bias Test Program

Material	Coefficient of Variation, %	
	Single Laboratory	Multiple Laboratories
A-7-6; Fine-Grained High Plastic Soil	6.5	16.7
A-2-4; Coarse-Grained Low Plastic Soil	4.2	8.2
GAB; Processed Aggregate Base	4.2	13.0

As noted above, an insufficient number of laboratories measured the resilient modulus in accordance with NCHRP 1-28a to determine any bias with AASHTO T 307. As such, results from the ruggedness test program were used to evaluate differences between the AASHTO T 307 and NCHRP 1-28a test results. The following summarizes the hypotheses noted at the beginning of Chapter 4 under Section 4.2.

- Hypothesis 2: The resilient moduli measured in accordance with AASHTO T 307 are different from those measured in accordance with NCHRP 1-28a. This hypothesis was evaluated using the roughness test data, as discussed in Chapter 3. The hypothesis was rejected based on the results from the Alpha and Beta laboratories that routinely use the test methods.

The difference in results between AASHTO T 307 and NCHRP 1-28a was found to be insignificant at equivalent stress states and confining pressures for the standard seating loads and test specimens tested at optimum conditions. However, as the moisture content deviated above and below the optimum value for the A-7-6 highly plastic soil, the difference in resilient modulus measured between AASHTO T 307 and NCHRP 1-28a increased.

- Hypothesis 3: Resilient moduli measured in accordance with the NCHRP 1-28a test method results in resilient modulus values that are indifferent to the default values included in the MEPDG (see Table 15). This hypothesis was rejected. The design resilient modulus for the GAB material, A-7-6 fine-grained highly plastic soil, and A-2-4 coarse-grained low plasticity soil were lower than the default value included in the AASHTO Pavement ME Design software (see Tables 16 to 19). In addition, resilient moduli measured in accordance with the AASHTO T-307 test method were lower than the default values included in the AASHTO Pavement ME Design software. Thus, the following question was evaluated: Is the difference between the default and measured resilient moduli significant in terms of the MEPDG software predictions and required layer thicknesses? The answer to this question is discussed in Chapter 5.

CHAPTER 5 APPLICATION AND USE OF RESILIENT MODULUS TEST

This chapter provides guidance on using resilient modulus data for pavement design and is grouped into three sections. The first section focuses on determining the input value or design resilient modulus to the Pavement ME Design software from the processed resilient modulus data. The second section provides examples using the procedure outlined in the first section to demonstrate determination of the design resilient modulus. The third section uses the precision of AASHTO T 307 given in Chapter 4 to determine if the range of resilient moduli result in a different layer thickness for different subgrade soils and aggregate base materials relative to alligator cracking and rutting predicted by the Pavement ME Design software.

5.1 Determination of Design Resilient Modulus

Some agencies, as part of their pavement design guidelines, use the average resilient modulus from the values measured at all stress states in accordance with AASHTO T 307. Use of the average value will overestimate the design resilient modulus for coarse-grained soils and crushed stone aggregate base materials. Conversely, the average resilient modulus from all stress states will underestimate the design value for fine-grained materials. An average resilient modulus from all stress states should not be used as the design or input value to the MEPDG software.

The procedure used to determine the input resilient modulus to the Pavement ME Design software is referred to and referenced in the MEPDG Manual of Practice and in National Highway Institute (NHI) Course #134064, Introduction to Mechanistic-Empirical Based Procedures. This is an iterative procedure that follows the material characterization guidelines recommended for use by Von Quintus et al for ME-based methods using linear elastic layer theory (1979, 1995, 2001, and 2005). This procedure is considered a quasi-input level 1 for the MEPDG. However, only a few sensitivity studies and design methods used by agencies in their day-to-day practice have followed this procedure.

The steps to determine the inputs for the unbound layers (aggregate base materials and subgrade soil) using repeated load resilient modulus tests are listed and defined below. These steps are in accordance with the MEPDG Manual of Practice and procedure recommended for use by Von Quintus and Killingsworth (1997), as well as in the final report for NCHRP project 1-37A for both flexible and rigid pavements.

1. Use the trial pavement cross section or structure to calculate the at-rest stress state from the overburden pressures for the aggregate base layer, embankment, and/or subgrade. The at-rest stress state for the aggregate base layer and embankment are determined at their ¼ depth, while the at-rest stress state for the subgrade is determined 18 inches into the subgrade. These material characterization depths are explained by Von Quintus et al in comparing laboratory resilient modulus values to backcalculated elastic layer modulus values (1997 and 1998). These depths are debatable but were selected for estimating the c-factor included in the 1993 AASHTO Design Guide, as well as in the MEPDG Manual of Practice (2008).

2. Start with the subgrade or lowest unbound layer and move upward in the pavement structure to establish the design resilient modulus for all unbound material layers using a linear elastic layer program for calculating layer responses or stresses at the locations defined in step 1. Assume the resilient modulus for the unbound layers above which the design resilient modulus is being estimated.

3. For the design truck axle load and season, calculate the load-related vertical and horizontal stresses using a linear elastic layered program to be consistent with the Pavement ME Design pavement response program. The load-related stresses are calculated at the material/soil characterization depths listed above (see step #1) over a range of resilient modulus values for the unbound layer in question. The range of resilient modulus values should be based on the range of values measured in the laboratory. The HMA modulus should be representative of the summer months and the heavier truck axle loads should be used because this represents the more critical condition for cracking and rutting of flexible pavements – the summer months and heavier axle loads control the rutting and fatigue or alligator cracking (bottom-up cracking) damage.

4. Calculate the at-rest horizontal and vertical stresses from overburden at the same critical points or locations in the unbound layers used to calculate the load-related stresses. The at-rest vertical pressure is calculated using equation 7, while equation 8 is used to calculate the at-rest horizontal stresses.

$$p_0 = \left(D_{HMA}\gamma_{HMA} + D_{Base}\gamma_{Base} + D_{Soil}\gamma_{Soil}\right) \tag{7}$$

$$p_1 = p_0 \quad \text{and} \quad p_2 = p_3 = p_0 K_0 \tag{8}$$

Where:

p_0 = At-rest vertical or overburden pressure from the layers above a specific point.
p_2, p_3 = At-rest horizontal stress.
K_0 = At-rest earth pressure coefficient.
D_{HMA} = Thickness of the asphalt concrete layers.
D_{Base} = Thickness of the unbound aggregate base and/or embankment layers. If determining the at-rest stresses in the unbound base layer, the point or depth into the base is ¼ of its thickness (see step 1).
D_{Soil} = Point for computing at rest stress state in subgrade, 18 inches.
γ_{HMA} = Average in place density of the asphalt concrete layers.
γ_{Base} = Average in place wet density of the unbound aggregate base and/or embankment layers.
γ_{Soil} = Average in place wet density of the subgrade soil.

5. Superimpose the at-rest and load-related stresses in the vertical and horizontal directions. In other words, add the at-rest and load-related vertical stresses, and add the at-rest and load related horizontal stresses.

6. Perform repeated load resilient modulus tests for the appropriate soil and material included in the trial design. Pool the test results to determine the stress-sensitivity of the soil/material. For thick pavements, the at-rest stress state normally controls determination of the design resilient modulus so the use of 12 or 15 stress states is usually not needed. This item will be discussed in the next section of this chapter relative to the examples used to determine the design resilient modulus.

7. Superimpose the total stress state versus resilient modulus calculated with linear elastic layer theory and the repeated load resilient modulus values versus stress state measured in the laboratory. The stress-state at which the elastic modulus and laboratory resilient modulus are equal is the value to be used in the Pavement ME Design software for quasi-input level 1.

8. Check the design resilient modulus determined for the lower unbound layers to be sure it is the same, as previously determined. This step can be an iterative process to determine a stable design resilient modulus. Based on the author's experience, one-iteration is typically needed to determine the subgrade design resilient modulus because the at-rest stress state from the overburden pressure normally controls the total stress state. The thinner pavement structures usually require multiple iterations because the load-related stresses are more important, especially in the aggregate base layers.

5.2 Examples Demonstrating Determination of Design Resilient Modulus

To demonstrate application of the above procedure, two soil types (an A-7-5 fine-grained soil and an A-1-b coarse-grained soil [see Figure 61 and Figure 62]), and two types of aggregate base materials (a high quality crushed stone and a lower quality crushed gravel [see Figure 63 and Figure 64) were used. The results from repeated load resilient modulus tests for both the soils and aggregate base materials were extracted from the LTPP database. As shown, all of the soils and base materials exhibit stress-hardening characteristics.

Figure 65 includes some examples of test results for typical aggregate base materials that have been used by the Corp of Engineers in comparison to those extracted from LTPP. The Corp of Engineers tests were conducted in the early 1970's, and are significantly higher than the two sets of resilient modulus data extracted from the LTPP database.

Two traffic levels were used to result in a thin and thick HMA pavement structure to illustrate the effect of overburden and total stress state on resilient modulus. Pavements with surface treatments were excluded from the demonstration, because the MEPDG procedure is not applicable to these types of low volume pavements. Table 23 provides a summary of the truck traffic and other inputs to determine the design layer thicknesses for a combination of the repeated load resilient modulus test results for the aggregate base materials and subgrade soils.

The linear elastic layer program entitled "WINJULEA" was used to calculate the load-related stresses for different combinations of soils and aggregate base layers for a conventional flexible pavement. For the load-related stress computations, typical elastic moduli representative of the summer months and heavier axle loads were used.

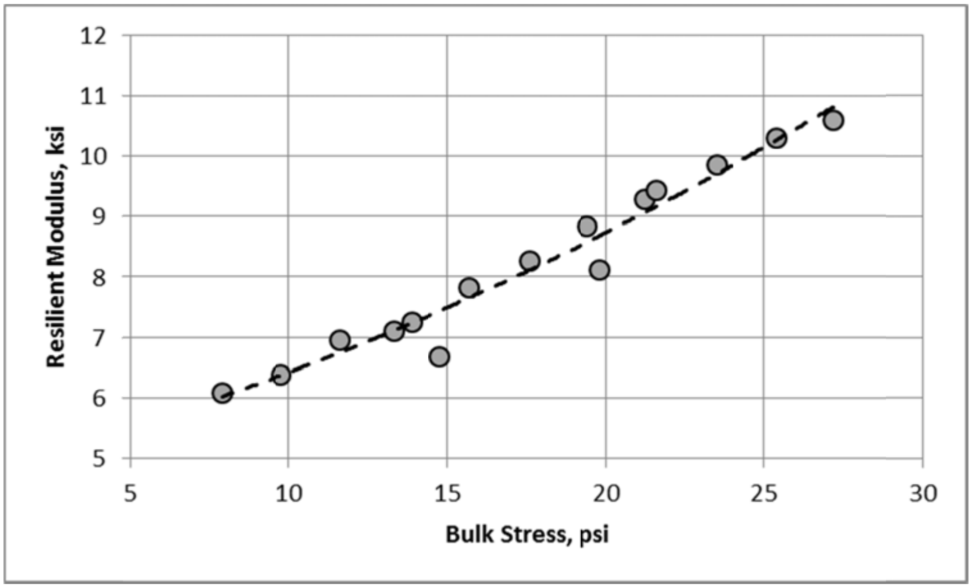

Figure 61. A-7-5 Fine-Grained Clay Soil; Repeated Load Resilient Modulus Tests Extracted from the LTPP Database

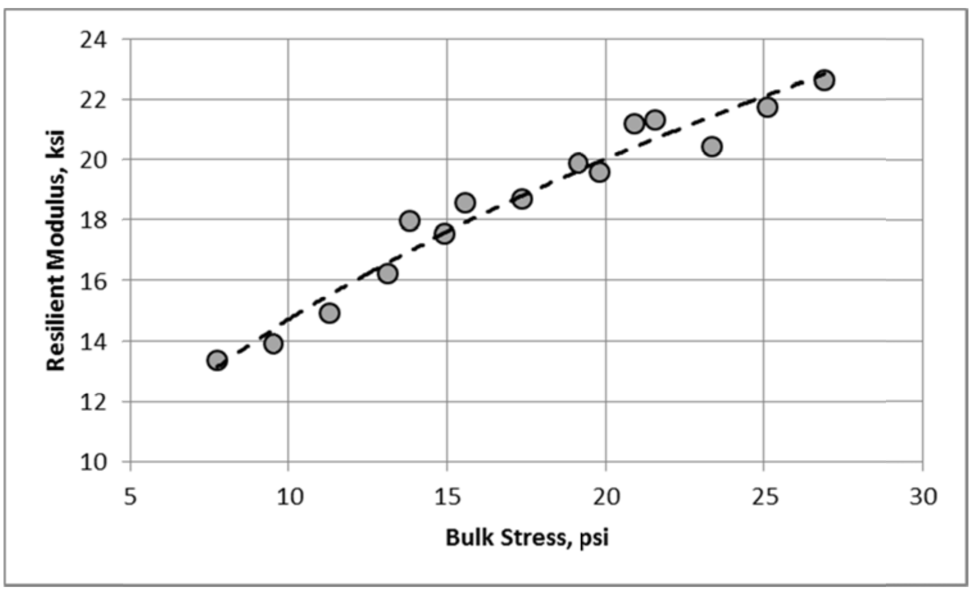

Figure 62. A-1-b Coarse-Grained Sandy Soil; Repeated Load Resilient Modulus Tests Extracted from the LTPP Database

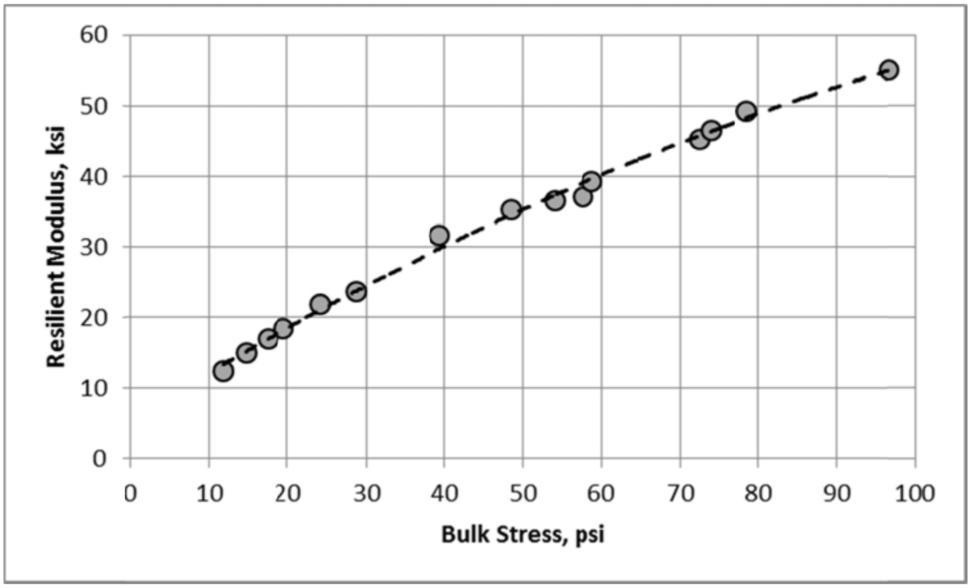

Figure 63. Crushed Gravel Aggregate Base; Repeated Load Resilient Modulus Tests Extracted from the LTPP Database

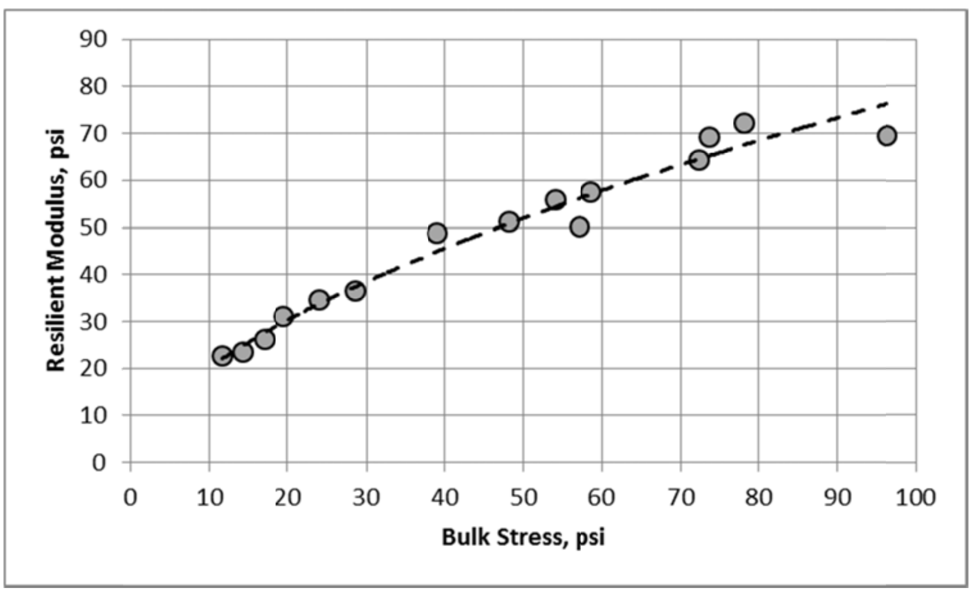

Figure 64. Crushed Stone Aggregate Base; Repeated Load Resilient Modulus Tests Extracted from the LTPP Database

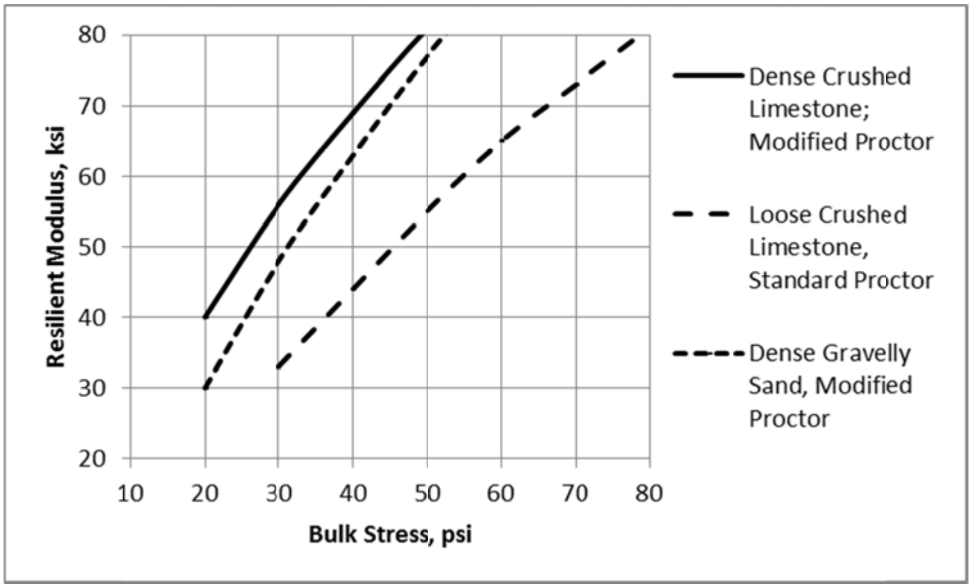

Figure 65. Some Typical Resilient Modulus Test Results from the Corp of Engineers

Table 23. Selected Inputs to Pavement ME Design Used for the Design Examples

Input Parameter	Input Value	
	Low Truck Traffic	High Truck Traffic
Reliability Level	75 percent	90 percent
Alligator Cracking Design Criteria	15 percent	5 percent
Average Annual Daily Truck Traffic	500	3000
Truck Traffic Classification Group	14	1
Total Number of Trucks over 20-year Design Period	2,300,000	13,400,000
Asphalt Performance Grade	PG 64-22	PG 70-22
All other inputs values were constant between each of the different runs with the exception of the resilient modulus values for the different aggregate base materials and subgrade soil combinations.		

The steps listed above were used to estimate the design resilient modulus, which are graphically illustrated in Figures 66 through 69. Figures 66 through 69 show the laboratory resilient modulus test results (Figures 61 through 64) combined or superimposed with the computed load-related stress states below the pavement surface at the depths defined in step 1. The stress state at which the laboratory resilient modulus and elastic modulus for the computed stress state are equal is the design resilient modulus. The following defines the design resilient modulus for each combination of layers and materials included in the examples.

- Figure 66 illustrates the design resilient modulus for the A-7-5 fine-grained subgrade soil for the same stress state used in the laboratory and computed with WINJULEA for the thin and thick HMA pavement structure.
    - A-7-5 design resilient modulus for the thin pavement = 6,100 psi.
    - A-7-5 design resilient modulus for the thick pavement = 6,300 psi.

- Figure 67 illustrates the design resilient modulus for the A-1-b coarse-grained subgrade soil for the same stress state used in the laboratory and computed with WINJULEA for the thin and thick HMA pavement structure.
  - A-1-b design resilient modulus for the thin pavement = 13,000 psi.
  - A-1-b design resilient modulus for the thick pavement = 13,300 psi.
- Figure 68 illustrates the design resilient modulus for the crushed aggregate base material at the same stress state used in the laboratory and computed with WINJULEA for the thin and thick HMA pavement structure.
  - Crushed gravel base design resilient modulus for the thin pavement = 27,000 psi.
  - Crushed gravel base design resilient modulus for the thick pavement = 12,000 psi.
- Figure 69 illustrates the design resilient modulus for the crushed stone base material at the same stress state used in the laboratory and computed with WINJULEA for the thin and thick HMA pavement structure.
  - Crushed stone base design resilient modulus for the thin pavement = 21,000 psi.
  - Crushed stone base design resilient modulus for the thick pavement = 43,000 psi.

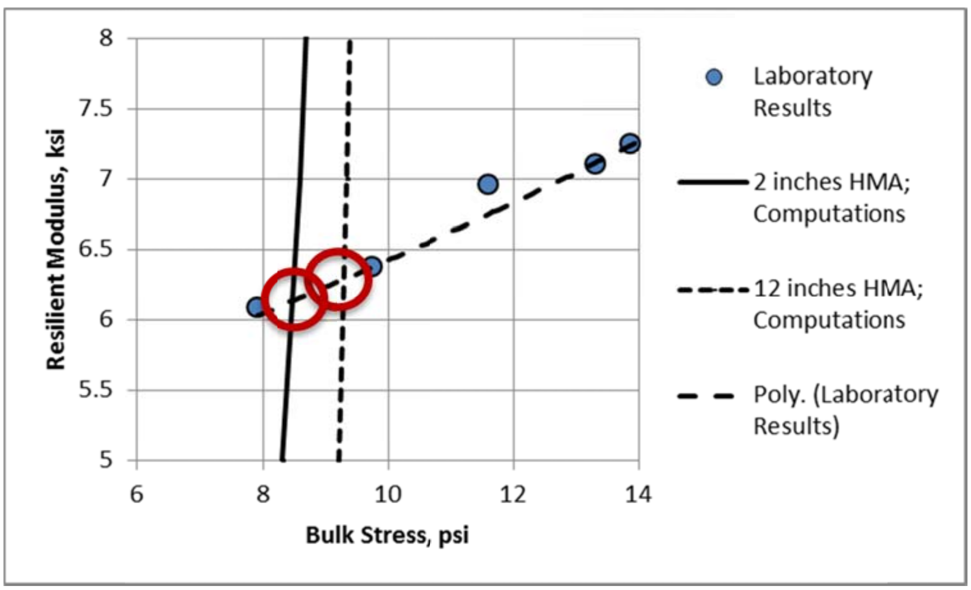

Figure 66. Graphical Determination of the Design Resilient Modulus for the A-7-5 Fine-Grained Clay Soil

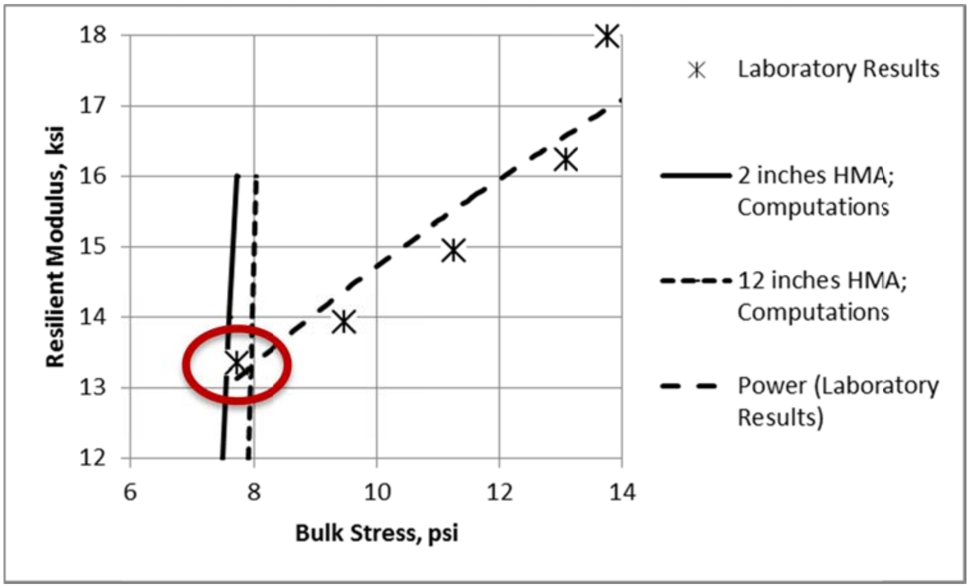

Figure 67. Graphical Determination of the Design Resilient Modulus for the A-1-b Coarse-Grained Sandy Soil

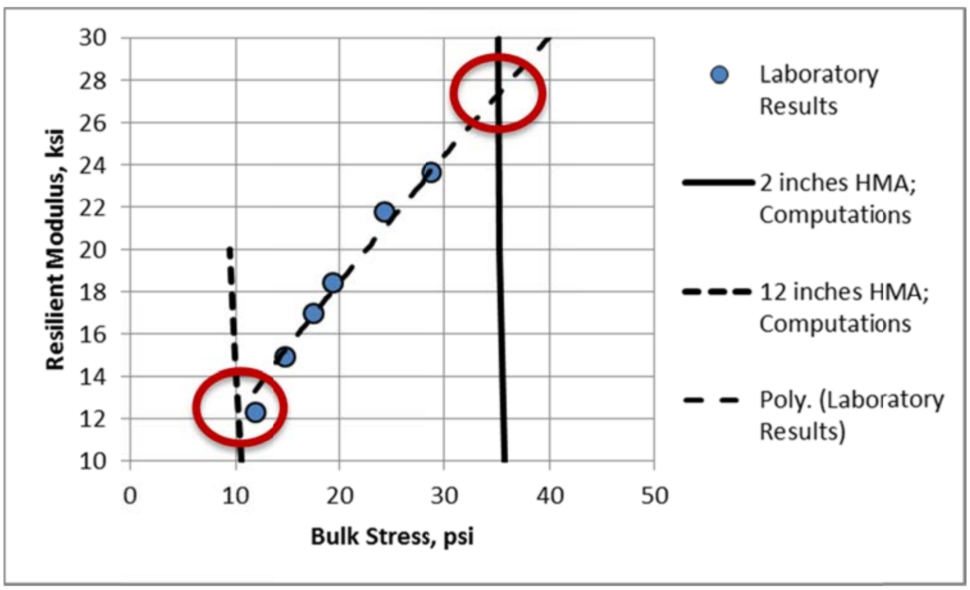

Figure 68. Graphical Determination of the Design Resilient Modulus for the Crushed Gravel Aggregate Base

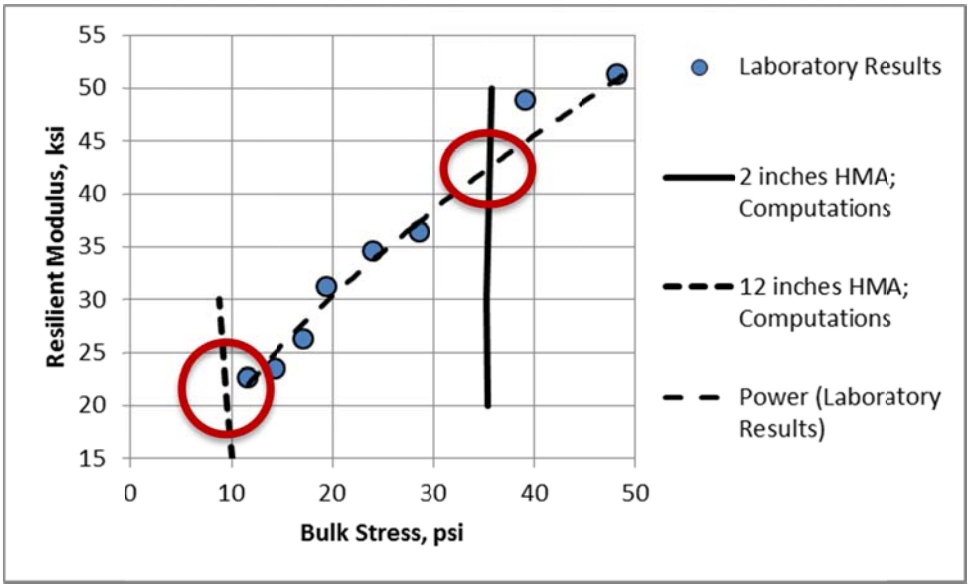

Figure 69. Graphical Determination of the Design Resilient Modulus for the Crushed Stone Aggregate Base

Table 24 summarizes the design resilient modulus for each combination of subgrade soils and aggregate base layers. The thickness of the aggregate base layer was determined to result in the highest resilient modulus considering the in place stress states using the adjacent layer modulus ratio criteria included in the MEDPG Manual of Practice. Figure 70 is a graphical representation of this criterion that was originally developed by the Corp of Engineers. Table 25 summarizes the final layer thickness derived for each condition using the Pavement ME Design software.

Table 24. Design Resilient Modulus Values for the Different Base Materials and Soils

Type of Soil	Type of Base	Design Resilient Modulus, psi			
		Subgrade Soil		Aggregate Base	
		2 inches (Low Traffic)	12 inches (High Traffic)	2 inches (Low Traffic)	12 inches (High Traffic)
Coarse-Grained; A-1-b (Fig. 62)	Crushed Gravel (Figure 63)	13,000 (see Fig. 67)	13,300 (see Fig. 67)	27,000	12,000
	Crushed Stone (Figure 64)	13,100	13,300	43,000	21,000
Fine-Grained; A-7-5 (Fig. 61)	Crushed Gravel (Figure 63)	6,100 (see Fig. 66)	6,300 (see Fig. 66)	27,000 (see Fig. 68)	12,000 (see Fig. 68)
	Crushed Stone (Figure 64)	6,200	6,200	42,000 (see Fig. 69)	21,000 (see Fig. 69)

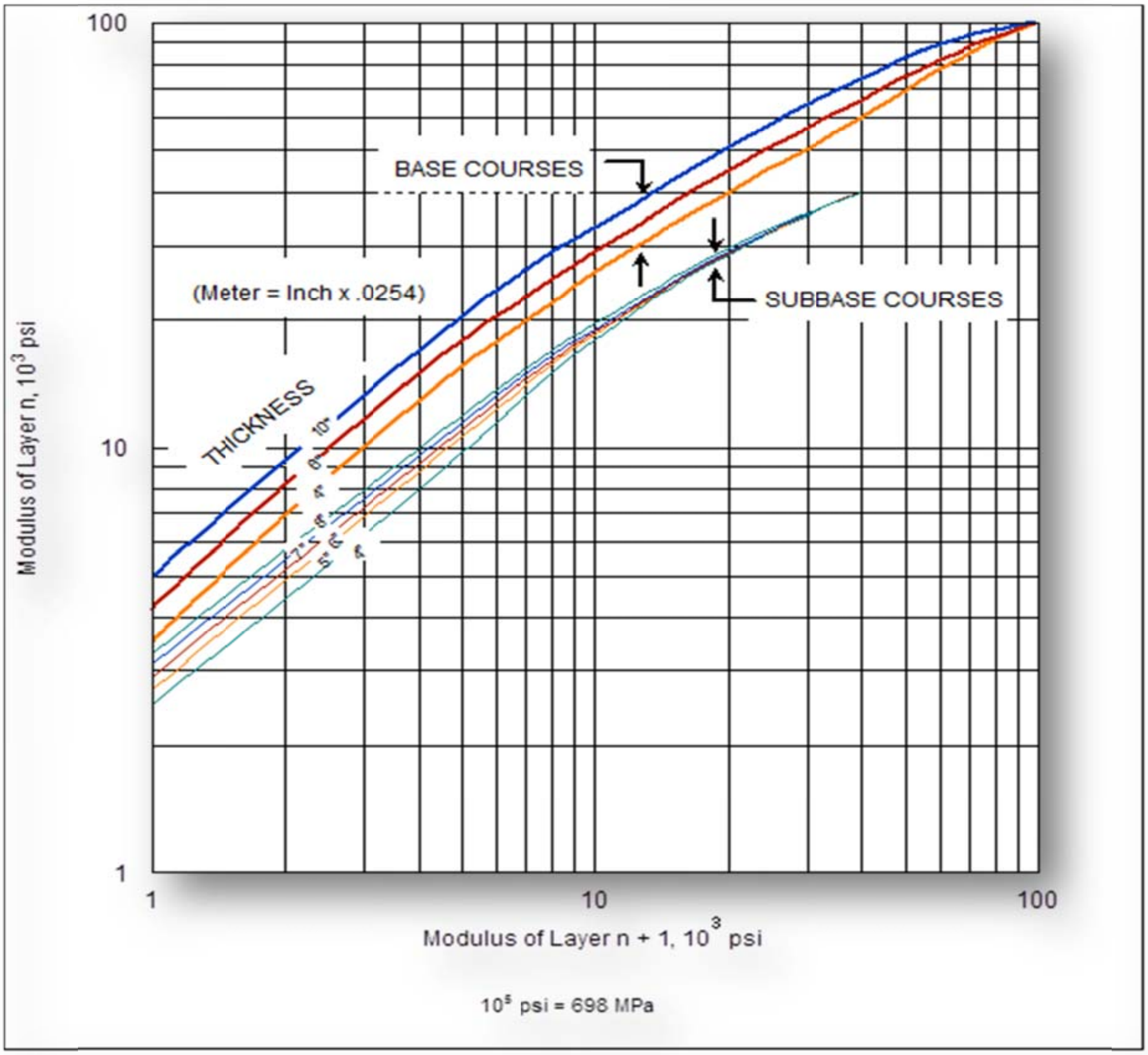

Figure 70. Limiting Modulus Criteria of Unbound Aggregate Base and Subbase Layers (Baker and Brabston, 1975)

An important observation from these examples is the stress state at which the laboratory resilient modulus and computed elastic modulus become equal are much lower than most of the repeated axial stresses and confining pressures specified in AASHTO T 307, even for the thin HMA pavement (for example; compare Figure 61 to Figure 66 and compare Figure 64 to Figure 69).

Another observation from these examples and comparison the design resilient modulus values for the base and subgrade soils, the input level 3 default values included in the Manual of Practice are for thick flexible pavements and independent of pavement thickness. Table 26 compares the input level 3 default resilient modulus values to the resilient modulus design values for these examples. As shown, there can be substantial differences between the default values and design resilient moduli. This is why repeated load resilient modulus testing is important for pavement design and in building an agency's materials library, as a minimum.

Table 25. HMA Surface and Aggregate Base Layer Thicknesses for 20-Year Design Examples

Type of Soil	Type of Base	Layer Thickness, inches			
		Low Traffic Volume		High Traffic Volume	
		HMA	Aggregate Base	HMA	Aggregate Base
Coarse-Grained; A-1-b (Figure 62)	Crushed Gravel (Figure 63)	3	5 (Construction Platform)	9	5 (Construction Platform)
	Crushed Stone (Figure 64)	3	5 (Construction Platform)	8.5	7
Fine-Grained; A-7-5 (Figure 61)	Crushed Gravel (Figure 63)	3	8	10	6
	Crushed Stone (Figure 64)	3	8	9	8

Table 26. Design Resilient Moduli Compared to the MEPDG Default Values for the Soils and Materials included in the Examples

Type of Soil or Base Material	Resilient Modulus, psi		
	MEPDG Default Value	LTPP Laboratory Tests Results	
		Thin Pavement	Thick Pavement
A-7-5 Soil	10,000	6,100	6,300
A-1-b Soil	18,000	13,000	13,300
Crushed Gravel	25,000	27,000	12,000
Crushed Stone	30,000	43,000	21,000

5.3 Sensitivity of Layer Thickness Relative to Precision of AASHTO T 307

As reported in Chapter 2, the COV for multiple laboratories and operators varies between 15 to 25 percent and for single operators that value is around 10 to 15 percent. The COVs from the precision and bias testing were summarized in Section 4.4.1. In summary, the COVs for the single operator and multiple laboratories reported within this study are lower than those reported in the literature.

Based on the sensitivity studies already completed (see Appendix B), it has been widely reported that the flexible pavement distress transfer functions are more sensitive to changes in resilient modulus than for the rigid pavement distress transfer functions. Thus, the remainder of this section provides a brief summary to determine if a change in resilient modulus using the D2S precision values (see Tables 20 and 21) would have a significant impact on the predicted amount of rutting and alligator cracking in flexible pavements causing a change in the HMA layer thickness.

The pavement structure used in these computations with the Pavement ME Design software are consistent with conventional pavement structures where the resilient modulus of the unbound layers continually increase towards the surface of the pavement structure. To evaluate the change in resilient modulus of the subgrade soils, full-depth and conventional pavement structures were used. For determining the minimum change in the resilient modulus of aggregate base materials, the subgrade resilient modulus was held constant between different modulus values of the same aggregate base using a layer modulus ratio of 2 (see Figure 70). In addition, the calibration coefficients reported in the appendices of the AASHTO 2010 MEPDG Local Calibration Guide were used for these examples.

Figure 71 summarizes the change in resilient modulus of the subgrade layer causing a change in the thickness of HMA. Table 27 and Table 28 summarize the change in resilient modulus of the subgrade and aggregate base that would have an impact of less than 0.25 and 0.50 inches of HMA, respectively. The difference is shown in terms of the magnitude of the change and as a percentage of the design resilient modulus. As shown, the change in resilient modulus to cause a change of $\pm$ 0.25 inches of HMA thickness are greater than the 1S values reported in Tables 20 and 21 for a single operator. Similarly, the change in resilient modulus to cause a change of $\pm$ 0.50 inches of HMA are greater than the 1S values reported in Tables 20 and 21 for multiple laboratories. This brief evaluation suggests the precision of the resilient modulus test methods for a single operator defined from the precision and bias tests included in this study will not result in significant deviation of the pavement structure.

The D2S values reported in Tables 20 and 21 were used to evaluate whether those differences would require a change in the HMA thickness. Differences from the design resilient modulus that are slightly lower than the D2S values will cause a significant change in the HMA layer thickness for some of the soils, which is applicable to the A-7-6 soil. In other words, differences between the resilient modulus measured between two laboratories less than the D2S value for an A-7-6 soil would result in a substantial change in the pavement structure. The D2S value for the GAB material represents the value at which substantial differences in the pavement structure would be required for values greater than D2S. This is considered reasonable because differences in resilient modulus between two laboratories greater than the D2S values suggest a difference in resilient modulus. Differences in resilient modulus requiring a substantial change in pavement structure for the A-2-4 soil are greater than the D2S values reported for this soil.

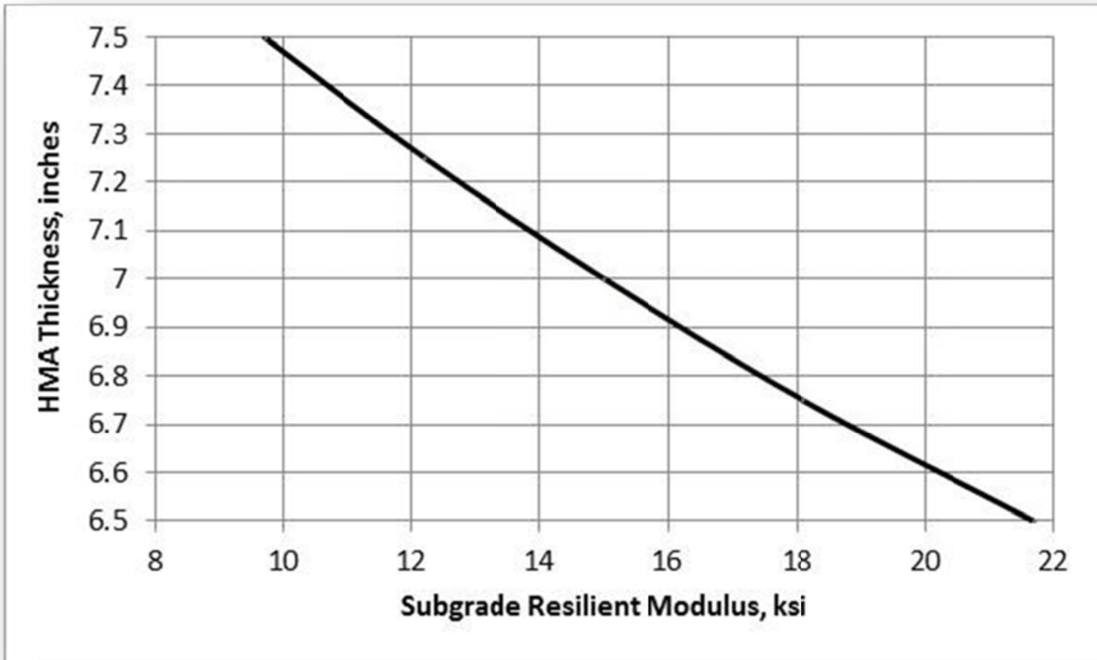

Figure 71. Graphical Determination of the Maximum Change in Resilient Modulus of the A-1-b Soil Requiring Less Than 0.5 inches Change in HMA Thickness

Table 27. Maximum Change in Resilient Modulus Resulting in Less Than 0.25 Inches of HMA

<table>
<tr><th colspan="3" rowspan="2">Unbound Material Type</th><th rowspan="2">Mean Resilient Modulus, psi</th><th colspan="2">Change in Resilient Modulus Requiring Less than 0.25 inches of HMA.</th></tr>
<tr><th>Magnitude, psi</th><th>Percent of Mean Value</th></tr>
<tr><td rowspan="2">Subgrade or Embankment</td><td>Weak, Fine-Grained Soil</td><td>A-7-5</td><td>5,000</td><td>900</td><td>18</td></tr>
<tr><td>Strong, Coarse-Grained Soil</td><td>A-1-b</td><td>15,000</td><td>2,800</td><td>19</td></tr>
<tr><td rowspan="2">Aggregate Base Layers</td><td colspan="2">Lower Quality Pit Run Gravel</td><td>20,000</td><td>4,400</td><td>22</td></tr>
<tr><td colspan="2">High Quality Crushed Stone</td><td>45,000</td><td>6,000</td><td>13</td></tr>
</table>

Table 28. Maximum Change in Resilient Modulus Resulting in Less Than 0.5 Inches of HMA

<table>
<tr><th colspan="3" rowspan="2">Unbound Material Type</th><th rowspan="2">Mean Resilient Modulus, psi</th><th colspan="2">Change in Resilient Modulus Requiring Less than 0.5 inches of HMA.</th></tr>
<tr><th>Magnitude, psi</th><th>Percent of Mean Value</th></tr>
<tr><td rowspan="2">Subgrade or Embankment</td><td>Weak, Fine-Grained Soil</td><td>A-7-5</td><td>5,000</td><td>1,600</td><td>32</td></tr>
<tr><td>Strong, Coarse-Grained Soil</td><td>A-1-b</td><td>15,000</td><td>5,300</td><td>35</td></tr>
<tr><td rowspan="2">Aggregate Base Layers</td><td colspan="2">High Quality Crushed Stone</td><td>20,000</td><td>8,500</td><td>43</td></tr>
<tr><td colspan="2">Lower Quality Pit Run Gravel</td><td>45,000</td><td>12,000</td><td>27</td></tr>
</table>

CHAPTER 6 CONCLUSIONS AND RECOMMENDATIONS

This chapter provides a summary of the findings reached from the ruggedness and precision and bias test programs, as well as the activities recommended to improve on the precision of the test procedure.

6.1 Conclusions

The following lists some of the more important findings or conclusions reached based on an analysis of the resilient modulus test data from nine laboratories. The test results from two of these laboratories were excluded from the data to determine the precision of AASHTO T 307.

- The average coefficient of variation for each soil and material was found to be lower than the values reported in the literature. Thus, substantial improvements have been made to the AASHTO T 307 and NCHRP 1-28a test methods. The single operator coefficient of variation varied from about 4 to 8 percent, while the multiple laboratory coefficient of variation varied from 8 to 17 percent for the different soils.

- The precision of the test methods, in terms of the standard deviation, is dependent on magnitude of the resilient modulus or stress state for the GAB material and less dependent on stress state for the A-7-6 and A-2-4 soils. The 1S and D2S values for a single operator and multiple laboratories were defined within this study.

- For the A-7-6 soil, differences between the resilient modulus measured between two laboratories less than the D2S value will cause a substantial change in the pavement structure, while the D2S value for the GAB material represents the value at which substantial differences in the pavement structure would be required for values greater than D2S. For the A-2-4 soil, differences in resilient modulus requiring a substantial change in pavement structure are greater than the D2S values reported for this soil. These results are considered reasonable because differences in resilient modulus between two laboratories greater than the D2S values suggest a difference in resilient modulus.

- The difference in results between AASHTO T 307 and NCHRP 1-28a was found to be insignificant at equivalent stress states and confining pressures for the standard seating loads and test specimens tested at optimum conditions. However, as the moisture content deviated above and below the optimum value for the A-7-6 highly plastic soil, the difference in resilient modulus measured between AASHTO T 307 and NCHRP 1-28a increased. Previous documents have reported there is a significant difference in results measured in accordance with AASHTO T 307 and NCHRP 1-28a.

- One potential change to the test procedure for AASHTO T 307 is to revise the seating or contact load to a percentage of the confining pressure so that slight deviations of this parameter has little to no effect on the measured resilient modulus between and within laboratories. If the seating load defined in AASHTO T 307 was revised to the use of a percentage of the confining pressure, there would be a statistical difference in results between the two test protocols.

- The design resilient modulus for the GAB material, A-7-6 fine-grained highly plastic soil, and A-2-4 coarse-grained low plasticity soil were lower than the default value included in the AASHTO Pavement ME Design software. More importantly, there can be substantial differences between the default values and design resilient moduli. The design resilient modulus is dependent on the pavement structure, especially for aggregate base materials, while the MEPDG default values are constant relative to structure. This is why repeated load resilient modulus testing is important for pavement design and in building an agency's materials library, as a minimum.

6.2 Recommendations

The ruggedness and precision test programs clearly demonstrated significant differences in results between the laboratories themselves. Although not confirmed, some of the differences in results were believed to be attributed to the location of the LVDTs and sample preparation techniques. As such, it is recommended that FHWA sponsor a series of training sessions for measuring the resilient modulus, similar to the ones developed for improving on measuring the dynamic modulus of HMA mixtures using the Asphalt Mixture Performance Test (AMPT).

These training sessions should focus on test specimen preparation and equipment setup to minimize the difference in results in the future. It is also recommended that agencies be encouraged to expand their materials libraries to include resilient modulus testing of aggregate bases in support of the MEPDG. In the interim, it is recommended that in reporting the resilient modulus test results, the location of the LVDTs be clearly identified in the report and the methods used to prepare and compact the test specimens be documented.

APPENDIX A EFFECT OF SOIL/MATERIAL PROPERTIES ON RESILIENT MODULUS

Changing the physical properties of the soil sample has an impact on the resilient modulus values. Several studies have been conducted to understand the correlation of soil physical properties and the resilient modulus. This section focuses on the findings from those studies. Table 29 is an overall listing on the studies reviewed and summarizes particular details of the materials/soils and test equipment included in each study. (Table 29 is included at the end of Appendix A.)

A.1 Soil Suction, Moisture Content, Density

A study by Gupta et al. (2007) on the resilient behavior of unsaturated soils indicated the effect of density on resilient modulus was relatively minor once the specimens were compacted to within 5 percent of the maximum dry unit weight or 98 to 103 percent of the standard Proctor density. The study also characterized the effects of soil suction on shear strength and resilient modulus of four soils representing different regions in Minnesota. The resilient modulus values increased with increasing suction. Results from the study also suggest that the influence of relative compaction on resilient modulus-soil suction relationship is insignificant because specimens were compacted near the optimum moisture content. It was also observed higher increases in resilient modulus occurred for small decreases in water content. More importantly, a strong relationship was observed between the resilient modulus based on internal displacement measurements in comparison to external displacement measurements for all soils.

Another study by Zapata et al. (2009) also focused on the relationship between soil suction and resilient modulus. The study presented a feasibility study on cohesionless granular base material, where soil suction was controlled during drained and measured during undrained resilient modulus tests. A fully integrated and automatically controlled system capable of applying repeated cycles of a haversine-shaped load pulse was used for the resilient modulus test during this study at Arizona State University. A typical granular base (designated as AB material and used by the Maricopa County Department of Transportation) was chosen for the testing program. When comparing drained and undrained conditions, no difference in the resilient modulus of the material was observed at lower degrees of saturation, suggesting that pore water pressure buildup further reduces the resilient modulus of the material under undrained conditions. The study concluded that soil suction has a significant effect on the resilient modulus of the material (increase in soil suction results in a corresponding increase in the resilient modulus of unbound materials, and vice versa) and should be incorporated in the next generation of models.

Cary and Zapata (2011), expanded the 2009 study and found that specimens with higher soil suction (lower degrees of saturation) yielded higher resilient modulus values, and vice versa. A typical granular base material used in the State of Arizona (designated as class GB), and a subgrade soil commonly encountered in the Phoenix Valley (designated as class SG) were chosen for the testing program. Sieve analysis, Atterberg limits, specific gravity, compaction or density-water content relationships, and soil water characteristics curve (*SWCC*) tests were performed. The study demonstrated the improvement of resilient modulus predictions using the proposed soil suction dependent resilient modulus model over the predictions obtained by using the MEPDG software to assess environmental effects on the resilient response of unbound materials (granular base and subgrade soil). The new model developed from this study was an enhanced version of the Universal Model by incorporating soil suction as a fundamental stress state variable.

A related study was conducted by Kung et al. (2006) on resilient modulus and plastic strain of unsaturated cohesive soils. Residual lateritic soil and pulverized mudstone were used for testing in this study. These soils were classified as A-7-6 and A-6. The laboratory data from the study indicated that an increase in soil suction resulted in a decrease in deformation and an increase in resilient modulus.

Another study by Zaman and Khoury (2007) focused on evaluating the effect of post-compaction moisture content on the resilient modulus of selected soils in Oklahoma. The soils were selected to represent a wide variation of soil types in Oklahoma (identified as: Burleson, Binger, Kirkland, Port, Minco, Sandy soil, Kingfisher, and Renfrow). The resilient modulus tests were performed on specimens compacted and subjected to a wetting and drying process. Results showed that the resilient modulus exhibited a hysteric loop with moisture variations. The resilient modulus values due to wetting are lower compared to the corresponding values after drying. It was also found that the initial compaction moisture content followed by drying or wetting affect the hysterics loop of both SWCC and the resilient modulus-moisture variation curve. Resilient modulus was found to increase as soil suction increased; however, the specific increase varied from one soil to another.

A paper by Ping et al.(2002) summarized a comparison study of experimental results from the FWD, field rigid plate bearing load, and laboratory resilient modulus tests on granular subgrade materials in flexible pavements. Granular soils from selected flexible pavement sites throughout Florida were tested for this research program funded by Florida DOT. In this study, for the laboratory triaxial resilient modulus test program, each type of soil was tested under the in situ moisture and density condition and optimum compacted condition. The results indicate the average laboratory resilient modulus values at optimum compaction conditions were 1.1 times higher than the average laboratory determined resilient modulus values at in situ conditions. One of the reasons cited for this observation is that the average dry density at optimum compacted conditions was slightly higher than the average field measured in situ dry density. It was also found the FWD back-calculated modulus was about 1.6 times higher than the laboratory resilient modulus for granular materials, or a c-factor of 0.63. This value is almost identical to the value of 0.62 which was derived by Von Quintus and Killingsworth from the LTPP sites (1995). No significant relationship was observed between the laboratory resilient modulus and the modulus of elasticity from the plate bearing load tests. The authors observed a trend of increasing laboratory resilient modulus with increasing plate secant modulus.

Ping and Sheng (2012) conducted another study to estimate the resilient modulus from basic soil properties. The two common types of granular subgrade soils found in Florida were used in this study, and included: a fine sand (A-3 soil) and silty/clayey sand (A-2-4 soil). Results from the study showed the physical properties (such as; moisture content, dry unit weight, and percent of fines) had a greater effect on the resilient modulus of A-2-4 soils than for A-3 soils.

Elias and Titi (2006) conducted a study to estimate the resilient modulus of various Wisconsin soils from basic soil properties. A laboratory testing program was conducted on 17 soils of the more common ones found in Wisconsin. The laboratory testing for resilient modulus indicated the effect of increased moisture content significantly reduces resilient modulus. The authors used the resilient modulus constitutive equation adopted by NCHRP Project 1-37A in developing the MEPDG procedure. A comprehensive statistical analysis was applied to develop correlations between basic soil properties and the coefficients of the resilient modulus constitutive equation. The analysis resulted in good correlations between the coefficients of the constitutive equation and soil properties. To evaluate the relationships, the regression equations developed in this study were compared with the regression equations developed by Yau and Von Quintus using data in the LTPP database. It was concluded by the authors the LTPP models did not yield good results compared with the regression equations proposed for use by this study. As previously stated, Yau and Von Quintus reported the LTPP-based regression equations will result in significant error. Reasons given by Elias and Titi for the better results were the differences in the test procedures, test equipment, sample preparation, and other testing conditions.

Soliman et al. (2009) evaluated the sensitivity of the resilient modulus to the variation in the physical properties of the Manitoba soils. For this study, the authors collected six soil samples to represent three types of soil: sandy silt, sandy clay, and high plastic clay. For each soil sample, four moisture contents were selected to evaluate the sensitivity of resilient modulus to the variation in moisture content and dry density. The project also investigated the effect of using two different methods for measuring soil deformation on the value of the resilient modulus. The first method consisted of two LVDTs mounted directly to the middle third of the specimen, thus eliminating the end effects. The gauge length of these LVDTs was 101.6 mm (4 inches). The second method consisted of two LVDTs mounted on the top loading plate with a gauge length of 203.2 mm (8 inches).

Soliman et al calculated two values for the soil resilient modulus (2009). The first resilient modulus value was calculated from the recoverable strain measured by the on sample LVDTs. The second resilient modulus value was calculated from the recoverable strain measured by the end LVDTs. The resilient modulus values for the last 5 cycles were averaged. The results indicated the resilient modulus values calculated from the measurements of on-sample LVDTs were higher than the resilient modulus values calculated from the measurements of off-sample LVDTs. It was found that the effect of using different measuring systems (on-sample LVDTs versus off-sample LVDTs) on resilient modulus values decreased with the increase of the cyclic stress for this type of soil. This finding is opposite what was found for some of the ASTM comparisons previously mentioned: off-sample LVDTs based resilient modulus were higher and more variable than for on-sample LVDTs based resilient modulus.

A.2 Surface Texture, Aggregate Angularity

Pan et al. (2006), in their research focused on investigating the influence of aggregate angularity and surface texture properties on the resilient behavior of unbound granular materials expressed by a nonlinear, stress-dependent resilient response model. The resilient responses of the 21 aggregate specimens were studied by analyzing the characteristics of the three regressed model parameters K1, K2, and K3 in terms of the three components of the Witczak–Uzan Universal model. The authors found the bulk stress component primarily controlled the stress dependency of the modulus by positively contributing to higher resilient modulus through increased confinement. In contrast, the octahedral shear stress component tended to decrease resilient modulus by causing dilation effects. They also found that both aggregate angularity and surface texture had significant correlations with the resilient modulus of the unbound aggregate specimens. It was observed that as the angularity or surface roughness increased, the modulus typically increased. Their study also indicated that angularity played a more important role through better interlock needed to increase the stiffness of the specimen under confinement, and that increasing angularity as well as surface texture through particle contact frictional resistance helped to reduce the dilative effect of the shear stress that caused a reduction in the resilient modulus.

A.3 Seasonal Variations, Source Lithology

A laboratory investigation, by Lin Li et al (2010)., of seasonal variations in resilient modulus of Alaskan base course materials. Four different types of D-1 aggregate materials were tested within this study. The study concluded increases in moisture content significantly reduces resilient modulus. It was observed when there are an insufficient amount of fines to fill the voids between gravels and sands, the resilient behavior of the material will not be significantly affected. Within the scope of this study (i.e., fines content range from 3.15 to 10 percent), impact of fines content on resilient modulus was unclear. The results from the study also indicated that when the soil is frozen, temperature is a primary factor affecting resilient modulus of the granular material. The study also concluded that during seasonal change, the resilient behavior of the soil is significantly affected by freeze-thaw action. In a very dense soil the volume might increase due to freeze-thaw, making the soil structure slightly looser and leading to a reduction in resilient modulus.

Another study, by Peng Li et al (2011), focused on the materials and temperature effects on the resilient response of asphalt-treated Alaskan base course materials (asphalt-treated base (ATB) and foamed asphalt-treated base (FATB). In this study, granular base course materials, typically known as D-1 materials, were collected from three regions of Alaska: southeast region, central region, and northern region. The study concluded that the resilient modulus of both ATB and FATB exhibited stress-state dependent properties. Generally, the resilient modulus increased with the increase of confining pressure (σ_3) and deviator stress (σ_d) classifying it as a stress-hardening material. It was observed that the resilient modulus of ATB was affected more by the change of confining pressure, while loading amplitudes (deviator stress) played a more important role for FATBs. The resilient modulus of ATBs also increased with a decrease in temperature. Compared with FATB, ATB was found to be more sensitive to a change in temperature. It was also reported the lower binder content produced higher resilient modulus for ATB due to higher compaction efforts applied to ATB specimens to reach the target 6 percent air void content. As for FATB, the resilient modulus at 2.5 percent residual binder content was the highest compared with those at 1.5 and 3.5 percent residual binder contents.

A study by Eggen and Brittnacher focused on investigating the range of load-carrying capability, in terms of resilient modulus, of crushed aggregate base materials in Wisconsin. The study also determined how variables, such as physical characteristics, material type, source lithology and regional factors influence resilient modulus. The authors tested 37 aggregate sources and statistically analyzed the results to look for correlations between resilient modulus and these variables, and more importantly, to determine if they could be used to predict resilient modulus. To accomplish the study objectives, the authors selected samples from groupings based on the origin of the materials. The results indicated that the resilient modulus test results did not differ significantly between the quarry and pit groups and that carbonate quarries generally gave significantly higher resilient modulus values than the Precambrian and felsic-plutonic quarries. In addition, the resilient modulus values did not differ significantly among the carbonate quarry groups or the sand and gravel pits. It could be noted that changing gradation of base course from a given source affected resilient modulus test results, but in an inconsistent pattern. The authors also found that certain physical parameters influenced resilient modulus in some of the geologic subsets. Overall, test results suggest no strong correlation to physical properties when many sources located over a wide geographical area with significant geologic diversity were considered.

A.4 Stress and Strain Characteristics

A study was conducted by Davich et al. (2005) to compare the small strain modulus and resilient modulus of pavement foundation materials in the context of resilient modulus testing. During this study, resilient modulus, ultimate shear strength, dielectric permittivity, and shear and compression wave speed values were determined for 36 soil specimens created from six soil samples. The hyperbolic model used in the study was able to accurately model the strain-dependent modulus reduction of a soil using only the small-strain modulus, friction angle, and cohesion as input parameters. The authors saw a potential for pavement inspectors to this relation to relate the results of the small-strain testing instruments used in the field to the laboratory-measured resilient modulus values used in pavement design. The finding is similar to the one reported by Von Quintus et al for acceptance of unbound aggregates and soil layers as part of the NCHRP 10-65 study (2009). The study also showed an increase in the modulus and strength of soils tested increased as their moisture contents decreased.

Titi et al. (2004) conducted a study to evaluate the resilient modulus of Wisconsin soils. Test results of four soils (A-2-4 and A-6) were covered in this study. These soils were subjected to different tests to determine their resilient modulus, physical properties, and compaction characteristics. The results indicated that the resilient modulus decreased with the increase in deviator stress and the decrease in confining stress for soil samples prepared under optimum moisture content and maximum dry weight (defined as stress softening). It was noted that for silty sand, the decrease of confining pressure resulted in lower resilient modulus. The effect of deviator stress was found to be more significant on the clay soil than on silty sand.

Peijun Guo and John Emery (2011) conducted a research study to highlight the importance of strain level associated with applied stresses in predicting resilient modulus. A granular base material (designed as aggregate LM4T from crushed limestone with designed particle size distribution) and a coarse sand (designated a Sand L derived from crushed limestone) were tested in this study. The results from the study indicated that the influence of strain level on the resilient modulus of unbound granular materials is significant. It was also observed that the resilient modulus may be 40 percent higher than that obtained from standard resilient modulus testing when relative amplitudes of cyclic and static axial stress were changed. The study also recommended that static stresses and resilient strains associated with applied cyclic stresses be taken into account in order to have improved prediction for the resilient modulus under general stress states.

A study by Anochie-Boateng and Tutumuler (2010), on the resilient behavior of three types of oil sands, showed that the resilient moduli of the oil sand materials were generally higher at 20°C than at 30°C. The three types of oil sand materials used in this study were obtained from Suncor Energy, Inc. and Syncrude Canada Ltd. Oil sand mines in Canada. Of the three resilient modulus models used in this study, i.e., K-theta, Witczak-Uzan, and the MEPDG, K-theta model appeared to give better predictions of resilient moduli for all the three oil sands. Stronger correlation coefficient values for the modified K-theta model indicated that the model can perform well on predicting resilient modulus of oil sand materials with similar characteristics for road construction. It was observed that fairly good resilient modulus predictions obtained for the modified K-theta model could not be repeated for the modified Witczak-Uzan and the MEPDG.

In their study, Malla and Joshi (2006) established prediction models for subgrade support (resilient modulus) values for typical soils in New England. The study used data extracted from LTPP Information Management System (IMS) database for 300 test specimens from 19 states in New England and nearby regions in the U.S. and 2 provinces in Canada.

Prediction equations were developed using SAS® for six AASHTO soil types viz. A-1-b, A-3, A-2-4, A-4, A-6, and A-7-6 and USCS soil types Coarse Grained Soils and Fine Grained Soils found in New England region to estimate resilient modulus. The results indicated that the predicted and laboratory measured resilient modulus values matched reasonably well for the soils considered. No definitive conclusion could be drawn based on relationship between laboratory resilient modulus values and FWD backcalculated elastic modulus from the LTPP test data. However, in general, it was observed that FWD backcalculated elastic modulus values were greater than the laboratory determined modulus values for the same soil type – similar to the findings from other authors.

A.5 Degree of Saturation, Gradation, Atterberg Limits

Nazzal et al. (2010) conducted a study to develop an efficient methodology for estimating resilient modulus values of subgrade soils for use in designing pavement structures. The field sampling program included obtaining subgrade soil samples from different sections along 10 pavement projects within the state of Louisiana (identified as: LA333, LA347, U.S.171, LA991, LA22, LA28, LA344, LA182, LA15, and LA652). These sections covered the common subgrade soil types found in Louisiana (A-4, A-6, A-7-5, and A-7-6 soil types). The authors conducted a regression analysis on the data to develop resilient modulus prediction models. The validity of the correlation equations was based on the regression equations developed by Yau and Von Quintus from the LTPP database. It was found the value of resilient modulus regressed coefficients was significantly affected by the deviation of moisture content from the optimum moisture content. The significance of this effect, however, was dependent on soil type. A significant difference was observed between the resilient modulus coefficients predicted by the LTPP models and those measured in this study. While the measured resilient modulus values of A-4 and A-6 soils and those recommended by the MEPDG were found to be significantly different, the difference was insignificant for soil types A-7-5 and A-7-6.

A study was conducted by Bennert and Maher (2005), on recycled asphalt pavement (RAP) and recycled concrete aggregate (RCA) to evaluate their potential use as base and subbase materials. Base and subbase materials were sampled from three regions in New Jersey and evaluated under the following performance tests: permeability (falling and constant head conditions), triaxial shear strength, cyclic triaxial loading, CBR and resilient modulus. The results suggested inclusion of RCA provided the largest CBR, highest resilient modulus, and lowest permanent deformation values. Though the increase in percent RAP added led to an increase in the resilient modulus, it also led to an increase in the accumulated permanent deformation from the cyclic triaxial test. Another study by Mohamed Attia and Magdy Abdelrahman (2011) on the effect of state of stress on the resilient modulus of base layer containing RAP also indicated that RAP material had higher resilient modulus than typical base aggregate. It was observed that RAP blends were less sensitive to bulk stress and more sensitive to confining pressure compared to aggregate base materials. Dong and Huang (2013), in their study on laboratory evaluation on resilient modulus and rate dependencies of RAP used as unbound base material, found that at ambient temperature (25°C), the resilient modulus of unbound RAP was higher than limestone and gravel with the same gradation and compaction condition.

Hodek and Mayrberger (2007) conducted a study to determine whether the dynamic stiffness of an unbound pavement base course, represented by a lab specimen, of a 4G gradation varies significantly over the acceptable gradation limits and a broad range of degrees of saturation. Michigan DOT's 4G gradation concept is intended to provide an open graded pavement course that allows for greater permeability or lower field saturation levels, than "densely graded" pavement courses. During this study, the stiffness of each compacted unbound granular material (natural gravel, crushed dolomite, slag, and recycled crushed concrete) was measured by the resilient modulus. The results indicated that the stiffness was dependent on material type, ratio of fine to coarse aggregate within the specification, and moisture content. It was observed that as the ratio of fine to coarse aggregate increased, the moisture content increased and the material's compacted stiffness decreased.

Xiao et al. (2011) on establishing correlations between aggregate source properties and aggregate resilient modulus found that aggregate shape properties played an important role in the resilient behavior of unbound aggregate materials. Their study demonstrated that resilient modulus values at a given stress state decreased with increased Flat and Elongated ratio and decreased angularity index values. Another study by Ghabchi et al. (2013) also demonstrated the dependency of resilient modulus on aggregate gradation, texture, and angularity. The study indicated that the denser gradations resulted in higher stability, in terms of resilient modulus. It was also reported the presence of more angular particles and rougher surface texture in limestone helped in interlocking and, therefore, provided a strong aggregate structure. This in turn led to higher resilient modulus values compared with those of the sandstone aggregates within the same gradation.

A study was conducted by Richardson et al. (2008) on resilient modulus of Missouri soils and unbound granular base materials. The test results included resilient modulus data from 27 common soils out of the 99 Missouri soil associations and from five unbound granular base materials. The results indicated that the material source and fines content are highly significant in resilient modulus. It was also noted that as the change in percent saturation increased, the percent change (loss) in resilient modulus increased. This study was followed by another study by Richardson and Lusher (2009), which focused on determining what effect a change in the Type 5 aggregate base gradation specification would have on the resilient modulus of an aggregate. To investigate the proposed gradation specification change, an experimental gradation was devised which followed the lower bounds of the proposed gradation specification on the #4, #30, and #200 sieves, and approximated the as-delivered gradations of two aggregate formations previously tested for the Missouri DOT on the 3/8, ½, ¾, and 1 inch sieves, making it a relatively open-graded material. Resilient modulus values in this study were higher compared to the previous study. This finding was a result of a lower degree of saturation present in the relatively open-graded specimens of this study in comparison to the dense-graded specimens examined in the previous study.

K.P. George (2004), in his study on prediction of resilient modulus from soil index properties, concluded that top five soil index properties influencing resilient modulus included moisture content, degree of saturation, material passing #200 sieve, plasticity index and density. The soils tested in this study were selected to provide a general representation of typical subgrade soils in Mississippi. Eight different subgrade soils from nine different sections were tested. The results indicated that water content or its surrogate percent saturation was the most significant soil property, followed by material passing the #200 sieve and plasticity index. The resilient modulus was negatively correlated with water content/ degree of saturation, positively associated with percent passing the #200 sieve, and had a mixed trend with plasticity index. The decrease of resilient modulus with increase in density was a suspect finding. It was found that although the density moderately affected the resilient modulus prediction in fine soil, it had a major role in resilient modulus prediction of coarse soils.

Khoury et al. (2010) investigated the stability and permeability of aggregate bases (limestone and sandstone) in Oklahoma. Three types of aggregates from Oklahoma were used in this study. In Phase 1, aggregates were collected from Anchor Stone quarry located at Owasso, Tulsa County. In Phase 2, two additional aggregates, Dolese from the Hartshorne quarry in Pittsburg County and Martin Marietta from Sawyer in Choctaw County, were collected. The results of the study indicated that resilient modulus of all aggregate types and gradations increased with the increase in confining pressure. It was also observed that the resilient modulus of specimens for a particular gradation decreased with the increase in fines over the gradation envelope. The lower limit of each gradation was found to produce higher resilient modulus values compared to those at the upper limit. The field data indicated that traffic-induced compaction of aggregate bases resulted in an increase in resilient modulus values, more so in dense graded aggregates.

A study by Andrei et al. (2009) demonstrated the effect of moisture and degree of saturation on unbound materials. In their study, moisture was found to have an impact on the resilient modulus of unbound materials, especially on plastic subgrade-type soils (A-2-4 and A-2-6). It was observed that for these materials, a 3 to 5 percent increase in moisture content from optimum conditions resulted in a 50 to 70 percent reduction in resilient modulus. The drying of the test specimens resulted in a significant increase in resilient modulus, in some cases ten-fold. It was also reported: compared to the plastic subgrade-type materials, non-plastic base-type materials (A-1-a) were less affected by moisture. The compactive effort was also found to play a significant role in the variation of resilient modulus with degree of saturation.

Drumm et al. (1997) conducted a study on Tennessee subgrade soils. It was found that all soils exhibited a decrease in resilient modulus with an increase in saturation, but the magnitude of the decrease in resilient modulus was found to depend on the soil type. The AASHTO A-7-5 and A-7-6 soils had the largest resilient modulus values at optimum, but were found to exhibit a greater reduction in resilient modulus with post compaction water content than A-4 and A-6 soil classifications. The soils with the highest resilient modulus for optimum conditions were found to experience the greatest decrease with saturation. Based on these findings, the researchers proposed a method for correcting the resilient modulus for increased degree of saturation.

Yau and Von Quintus (2002) conducted a comprehensive review and evaluation of the resilient modulus test data measured on pavement materials and soils recovered from the LTPP test sections. The study found that sampling technique (auger versus test pit samples) had an effect on the resilient modulus test results for the uncrushed gravel, crushed stone base/subbase materials, and subgrade soils. However, no significant difference was found between the augered and test pit samples of the other base/subbase materials. For the subgrade soils, sampling technique (Shelby tubes versus auger samples) had the most effect on the clay soils. Sampling technique was found to have little to no effect on the sand base/subbase materials and sand soils. The researchers also observed that the physical properties correlated to resilient modulus varied between the different materials and soils. The liquid limit, plasticity index, and the amount of material passing the smaller sieve sizes were found to be important as related to the resilient modulus for the lower strength unbound aggregate base/subbase materials, while the moisture content and density were important as related to the higher strength materials.

Unfortunately, few if any of these studies identified allowable deviations in the water content-density or degree of saturation which does have a significant effect on resilient modulus.

Table 29. Maximum Change in Resilient Modulus Resulting in Less Than 0.5 Inches of HMA

Authors	Supporting DOT/ Organization	Material Type	Test Protocol Used	Compaction Method	Design Value Used
Lin Li et al.	Alaska		AASHTO T 307-99		
Peng Li et al.	Alaska	Granular Base	AASHTO T 307-99		
Andrei et al.	Arizona	Plastic subgrade-type soils	NCHRP 1-28A	Standard and Modified Proctor	
Dennis, Jr. and Bennett	Arkansas	Granular base and subgrade	Spectral Analysis of Surface Waves (SASW) method		
Attoh-Okine and Wiredu	Delaware	Granular Subgrade	Indirect Procedure – Through correlation between CBR and basic soil tests	Standard Proctor	
Ping and Sheng	Florida	Granular materials (A-3 and A-2-4 soils)	AASHTO T307-99 and AASHTO T 292-91I		
Ping et al.	Florida	Subgrade soils (A-3 and A-2-4)	AASHTO T292-91I and AASHTO T307-99	Modified Proctor	*Mr* value obtained at a deviator stress of 34.5 kPa (5.0 psi) under the confining pressure 13.8 kPa (2.0 psi)
Ayithi et al.	Florida	Limerock and Granite Base			
Watson et al.	Georgia	Subgrade soils (A-2-4)	AASHTO T 307-99	Modified Proctor	Mr values obtained at a cyclic deviator

Table 29. Maximum Change in Resilient Modulus Resulting in Less Than 0.5 Inches of HMA

Authors	Supporting DOT/ Organization	Material Type	Test Protocol Used	Compaction Method	Design Value Used
					stress of 37.9 kPa (5.5 psi) and a confining stress of 13.8 kPa (2 psi)
El-Badawy et al.	Idaho	Subgrade soils	AASHTO T-294		
Ceylan et al.	Iowa	Subgrade soils (select, class 10/suitable, and unsuitable)	AASHTO T 307-99		Mr results without zero confining stress conditions (standard test procedure) for three types of soil with OMC conditions and one type of aggregate with 10% moisture condition
Rusell and Hossain	Kansas	Subgrade soils	AASHTO T-274-82		Average – Bilinear soils Subjective estimate – Non-bilinear soils
Yang and Wu	Louisiana	Granular base and subgrade.	N/A		

Table 29. Maximum Change in Resilient Modulus Resulting in Less Than 0.5 Inches of HMA

Authors	Supporting DOT/ Organization	Material Type	Test Protocol Used	Compaction Method	Design Value Used
Soliman et al.	Manitoba Infrastructure and Transportation	silty sand, sandy clay, and high plastic clay	NCHRP 1-28A	Standard Proctor	
Zapata et al.	Maricopa County, AZ	Granular Base	NCHRP 1-28A		
Hodek and Mayrberger	Michigan	Natural gravel, crushed dolomite, slag, and recycled crushed concrete.	AASHTO T 307-99	Standard Proctor	
Davich et al.	Minnesota	Granular	LTPP P-46		
Gupta et al.	Minnesota	Subgrade	NCHRP 1-28A	Standard Proctor	Mr at a bulk stress (σ_b) of 83 kPa and octahedral shear stress (T_{oct}) of 19 kPa.
Xiao et al.	Minnesota	Class 3, 4, 5, and 6 unbound aggregate base and granular subbase materials	NCHRP 1-28A		
Attia and Abdelrahman	Minnesota	RAP and base material	NCHRP 1-28A	Standard Proctor	
Richardson et al.	Missouri	Unbound granular base	AASHTO T 307-99	Standard Proctor	
Richardson and Lusher	Missouri	Unbound granular base	AASHTO T 307-99	Standard Proctor	
K.P.George	Mississippi	Unbound Subgrade	AASTHO TP46	Standard Proctor	
Malla and Joshi	New England	Subgrade	AASHTO T 307-99		

Table 29. Maximum Change in Resilient Modulus Resulting in Less Than 0.5 Inches of HMA

Authors	Supporting DOT/ Organization	Material Type	Test Protocol Used	Compaction Method	Design Value Used
	Transportation Consortium				
Janoo et al.	New Hampshire	Subgrade soils (including marine clay)	AASHTO TP 46	Standard Proctor	Use of effective Mr, calculated from relative damage (u_f)
Bennert and Ali Maher	New Jersey	RAP, RCA, and their blends with the base material	AASHTO TP46-94	Standard Proctor	
Ghabchi et al.	Oklahoma	Limestone and Sandstone aggregate base	AASHTO T 307-99	Standard and Modified Proctor	
Khoury et al.	Oklahoma	Limestone and Sandstone aggregate base	AASHTO T 307-99	Standard and Modified Proctor	Mr values were calculated at a deviatoric stress of 6.0 psi (41.4 kN/m^2) and a confining pressure of 4.0 psi (27.6 kN/m^2)
Hossain et al.	Oklahoma	Limestone and Sandstone	AASHTO T 307-99	Standard Proctor	Mr values calculated using average material constants obtained from regression modeling

Table 29**. Maximum Change in Resilient Modulus Resulting in Less Than 0.5 Inches of HMA**

Authors	Supporting DOT/ Organization	Material Type	Test Protocol Used	Compaction Method	Design Value Used
Zaman and Khoury	Oklahoma	Subgrade sandy and clayey soils	AASHTO T 307-99	Standard Proctor	
Drumm et al.	Tennessee	Subgrade soils (A-4, A-6, A-7-5, A-7-6)	AASHTO T274	Standard Proctor	
Zhou et al.	Tennessee	Silty and Clayey soils	AASHTO T 307-99	Standard Proctor	
Hossin	Virginia	Subgrade Soils	AASHTO T 307-99	Standard Proctor	MEPDG Level 3 design values of resilient modulus for Virginia soils were determined using average regression coefficients at confining and deviator stresses of 2 and 6 psi respectively.
Eggen and Brittnacher	Wisconsin	Crushed aggregate base	SHRP P46	Standard Proctor	
Elias and Titi	Wisconsin	Subgrade	AASHTO T 307-99	Standard Proctor	
Titi et al.	Wisconsin	Clayey and Silty Soils	AASHTO T 307-99	Standard Proctor	
Titi and English	Wisconsin	Subgrade soils (A-4, A-6,	AASHTO T 307-99	Standard	Average Mr

Table 29. Maximum Change in Resilient Modulus Resulting in Less Than 0.5 Inches of HMA

Authors	Supporting DOT/ Organization	Material Type	Test Protocol Used	Compaction Method	Design Value Used
		and A-7 (A-7-5 and A-7-6)		Proctor	values minus one standard deviation (μ-σ) on the wet category and confining pressure of 4 psi
Pan et al.		Granular	AASHTO T 307-99		
Cary and Zapata		Granular Base	NCHRP 1-28A		
Guo and Emery		Granular Base	AASHTO T307-99.		
Anochie-Boateng and Tutumluer		Oil Sand	AASHTO T 307-99		
Qian et al.		Subgrade (75% Kansas River sand and 25% kaolin)	Cyclic plate loading tests		
Nazzal and Mohammad		Subgrade (A-4, A-6, A-7-5, and A-7-6 soil types).	AASHTO T 307-99		
Dong and Huang		Unbound Reclaimed Asphalt Pavement (RAP), crushed limestone and crushed gravel	AASHTO T 307-99	Modified Proctor	
Kung et al.		Subgrade soils (Residual lateritic soil and pulverized mudstone)	AASHTO T 292-91		
Yau and Von Quintus		Coarse-grained and fine-grained base/subbase/subgrade materials			

APPENDIX B SENSITIVITY OF RESILIENT MODULUS USING THE AASHTO PAVEMENT ME DESIGN SOFTWARE

Appendix B includes a summary on the sensitivity of pavement distress to changes in resilient modulus that has been documented in the literature. Table 30 is a summary of the sensitivity studies that have been completed for both flexible and rigid pavements. Most of the past work included determining the impact of varying the elastic modulus of the unbound layers on the predicted distresses, and excludes the interaction with other layers and stress sensitivity. Appendix B is grouped into two parts related to the sensitivity analyses of distress to resilient modulus: flexible and rigid pavements.

B.1 HMA Pavements

The MEPDG predicts five distresses for flexible pavements, which include: longitudinal cracking (assumed to be surface initiated or top-down cracking), alligator or area cracking (assumed to be bottom initiated or bottom-up cracking), rutting, thermal cracking, and roughness as measured by the International Roughness Index (IRI).

A comprehensive study was conducted by Schwartz et al. (2011) to evaluate the sensitivity of pavement performance predicted by the MEPDG to the values of the design inputs. The overall objective of this NCHRP funded research was to determine the sensitivity of the performance predicted by the MEPDG to variability of the design input values for flexible and rigid pavements. The evaluation was performed through an initial triage, extensive one-at-a-time sensitivity analyses, and comprehensive global sensitivity analyses. During this study, global sensitivity analyses was carried out for five pavement types under five climate conditions and three traffic levels. The results have been summarized in Table 31.

The following subsections provide an overview on how each predicted distress is affected by changes in resilient modulus of unbound aggregate base layers, embankments and subgrade soils.

B.1.1 Longitudinal Cracking

The MEPDG Manual of Practice suggests that longitudinal or top-down cracking not be used to make changes to the design strategy. Thus, the following summarizes the results from a few studies related to the effect of resilient modulus on predicted longitudinal cracking.

- Fernando et al (2007) Florida DOT sensitivity study to develop a database for calibrating the MEPDG transfer functions. As part of this study, two pavements, one flexible and one rigid, representative of typical Florida pavement, environmental, and traffic conditions, were used in the sensitivity analyses of this study. The flexible pavement was a four layer structure comprised of an asphalt concrete layer, limerock base, stabilized subgrade, and sand subgrade. The results indicated that for HMA sections, the top-down cracking model was sensitive to subgrade resilient moduli as the higher subgrade modulus resulted in greater lengths of longitudinal cracks; assumed to be surface initiated cracking.

e 30. **Maximum Change in Resilient Modulus Resulting in Less Than 0.5 Inches of HMA**

rting T/ ncy	Year	Material Type/ Property	Flexible Pavements					JPCP			CRCP
			Longitudinal Cracking	Alligator Cracking	Thermal Cracking	Rutting	IRI	Transverse Joint Faulting	Mid-Slab Cracking	IRI	Punch-out
ch ina	2010	Subgrade	-	H	-	H	H	N	N	N	-
da	2007	Subgrade	H	-	-	-	-	L	L	L	-
and	2007	Subgrade	-	VL	-	VL	-	-	-	-	-
		Ground Water	-	VL	-	VL	-	-	-	-	-
sota	2007	Subgrade	H	L/H*	L/H*	L	-	VL	VL	-	-
na	2012	Subgrade	-	M	-	M	-	-	-	-	-
		Ground Water	-	L	-	L	-	L	M	L	-
ado	2013	Subgrade[1]	-	-	-	-	-	-	-	-	-
nsin	2008	Subgrade	-	L/N	L/N	M	M	L	L	L	-
o	2009	Subgrade	N	M	N	M	L	H	H	M	-
uri	2009	Subgrade	-	M	N	L	H	H	H	M	-
		Ground Water	-	L	L	L	L/N	L	L	L/N	-
a	2005	Subgrade	VH	L	N	L/N	L/N	-	-	-	-
h	2005	Subgrade	L	M	-	M	-	L	M	L	-
		Ground Water	-	L	-	L	-	-	-	-	-
gan	2005	Subgrade	M	L	N	H	H	VL	H	H	-
RP	2011	Subgrade	M	M	N	VH	-	M	M	-	M
sota	2010	Base		H		M	N				
o	2011	Subgrade	M	H	M	H	M	-	-	-	-

- High Impact, M - Moderate Impact, L - Low Impact, VL - Very Low Impact, N - No Impact
million ESAL's
ubgrade type was not performed

Table 31. Maximum Change in Resilient Modulus Resulting in Less Than 0.5 Inches of HMA

HMA Pavement Inputs: Subgrade/Base Properties	Level of Sensitivity for Flexible Pavements				
	HMA Rutting	Total Rutting	Alligator Cracking	Longitudinal Cracking	Thermal Cracking
Resilient Modulus	NS	VS	S	S	NS
Poisson's Ratio	NS	NSNS	NS	NS	NS
Soil-Water Characteristic Curve	S	S	S	S	NS
Permeability	NS	NS	NS	NS	NS
Compacted/Uncompacted Layer	NS	NS	NS	NS	NS
[1]VS = very sensitive, S = sensitive, NS = non-sensitive.					

- Minnesota DOT MEPDG implementation study by Cochran et al (2007) focused on the results of the evaluation of default inputs, identification of deficiencies in the software, sensitivity analysis, and comparison of results to the expected limits for typical Minnesota site conditions. A wide range of pavement design features (e.g. layer thickness, material properties, etc.) were considered and the outcome was the effects of different parameters on predicted pavement distresses. For flexible pavements, the sensitivity analysis focused on two traffic levels: 10 and 1 million ESAL's. Climate, HMA thickness, asphalt binder grading, gradation of the HMA, base thickness, subbase thickness, and subgrade type were the parameters considered for the sensitivity analysis. For 10 million ESALs, changing the subgrade from A3 to A6 reduced the prediction of the longitudinal cracking. For 1 million ESALs, changing the subgrade from A3 to A6 increased the longitudinal cracking.

- Iowa MEPDG implementation study: Coree et al. (2005) evaluated the relative sensitivity of MEPDG to HMA properties, traffic, and climatic conditions based on field data from two existing Iowa flexible pavement systems (US-020 in Buchanan County and I-80 in Cedar County). The results indicated that the type of subgrade (or level of resilient modulus) was extremely sensitive towards longitudinal cracking.

- Utah MEPDG implementation study by Darter et al. (2005) indicated that as the subgrade modulus decreased, tensile strain at the bottom of the HMA layer as well as vertical strain at the top of the subgrade increased. The longitudinal cracking again peaked for an intermediate level of subgrade support and was lower for the stiffer and softer support, possibly due to the tensile strain calculated at the surface of the HMA. Overall, the longitudinal cracking was found to be less sensitive to subgrade type/modulus.

- A study was conducted by Buch et al. (2008) to highlight the evaluation of the current performance models for jointed plain concrete pavement (JPCP) and HMA pavements for the state of Michigan. The sensitivity analyses involved: a preliminary sensitivity using one variable at a time, and a detailed analysis using a full factorial to capture some of the interactions. Both analyses reflected the local design and construction practices in Michigan. Preliminary sensitivity investigation involved preparing a short-list of significant variables. The abbreviated variables were further refined based on engineering judgment and local practices while levels of the significant variables were selected based on the local design practices. In the detailed analysis, the full factorial multivariate analyses were conducted to highlight both main and interaction effects between input variables on pavement performance. In case of flexible pavements, the subgrade material was found to have appreciable impact on longitudinal cracking.

B.1.2 Alligator Cracking

- Schwartz study (2007) to transition Maryland's State Highway Administration (SHA) from current flexible pavement design procedures to the MEDPG procedure. The sensitivity analyses performed as part of this investigation had two stated objectives: 1) comparison of pavement designs from MEPDG and the current MDSHA pavement design procedure, and 2) to study the sensitivity of the MEPDG parameters for a better understanding of the program and to gather information on calibration and other implementation needs. Only rutting and fatigue cracking were used for comparison in study. The pavement layers were comprised of a 6.7 inch asphalt concrete layer, an 18.6 inch base layer, and a subgrade. A climatic location in Alabama, Arizona, Maryland, South Dakota, and Washington State were used to get a sample of different temperatures and precipitation levels. The ground water table location was studied at depths of 3, 7, and 15 feet in the central Maryland location, and the overall sensitivity was found to be negligible. The three different subgrade soil types studied were: an A-7-6 clay soil, an A-5 silty soil, and an A-2-4 sandy soil, and the results indicated an increase in subgrade resilient modulus resulted in a decrease in fatigue cracking. It was noted that the MEPDG is able to evaluate some kind of effect on the pavement performance due to the change of subgrade soil type, despite the rutting and fatigue cracking outputs not showing any significant conclusions.

- Minnesota DOT MEPDG implementation study: Cochran et al. (2007) indicated that for 10 million equivalent single axle loads (ESAL's), the soil type was found to have a positive correlation with alligator cracking, and changing the subgrade type from A3 to A6 increased alligator cracking. The use of coarse mix gradation instead of fine increased alligator cracking. For 1 million ESAL's though, the soil type had a positive correlation with alligator cracking. If subgrade type A6 instead of A3 was used the alligator cracking increased.

- Arizona DOT MEPDG Implementation: Darter et al (2012) conducted a comprehensive sensitivity study of the new HMA and new JPCP transfer functions to help validate the local Arizona model predictions. For the flexible pavements, the subgrade type had a small effect on alligator cracking; with the coarse grained subgrade materials (A-2-4) performing slightly better than the weaker, fine grained (A-6) materials.

- Wisconsin DOT MEPDG Implementation study: Mallela et al conducted a sensitivity analysis as part of the implementation process (2008). For the HMA pavements, the subgrade type and subgrade improvement technique was found to have low to no impact on alligator cracking.

- Ohio DOT MEPDG Implementation: Mallela et al (2009) completed a sensitivity analysis as part of the implementation planning process to determine: (1) level of importance of each data item needed for pavement design/analysis using the MEPDG, and (2) formalize strategies for data collection activities. The effect of subgrade type on performance was determined by simulating a new HMA pavement constructed over a fine-grained (A-7-5) and coarse grained (A-1-b) soil foundation. The results indicated that as the subgrade modulus decreased, the tensile strain in the bottom of the HMA layer and vertical strain at the top of the subgrade increased. In general, it was noted that the lower the subgrade type/modulus, the higher was the alligator fatigue cracking.

- Iowa DOT implementation study: Coree et al (2005) indicated that the type of subgrade (M_r) was in the between low sensitive and insensitive range with regards to alligator cracking, transverse cracking, smoothness/IRI and rutting.

- Utah DOT MEPDG implementation study: Darter et al (2005) indicated that for HMA pavements, the lower the subgrade type/modulus the higher was the alligator fatigue cracking.

- Michigan DOT study: Buch et al (2008) completed a study to highlight the evaluation of the current performance models for JPCP and HMA pavements. Results indicated that in case of flexible pavements, the subgrade material had minimal impact on fatigue cracking.

- Tennessee DOT: Another study was conducted by Zhou et al (2013) on the seasonal resilient modulus inputs for 14 Tennessee soils and their effects on asphalt pavement performance. Among these 14 soils, 3 were silty soils and 11 are clayey soils. The resilient moduli of the 11 clayey soils were evaluated under three different moisture contents: optimum water content and two higher water contents. The results showed a negative correlation between soil resilient modulus and moisture content. It was also observed that the considerations of the seasonal variation of subgrade resilient modulus due to the moisture change will decrease estimations of pavement fatigue life. The fatigue life of both low volume and heavy volume pavements were found to be greatly influenced by the subgrade resilient modulus reductions due to moisture change.

- Minnesota DOT: A study was conducted by Attia and Abdelrahman (2010) on the variability in resilient modulus of Reclaimed Asphalt Pavement as base layer and its impact on flexible pavement performance. The study used the results of resilient modulus testing on one source of granular material—Minnesota Class 5 (Class 5)—and laboratory blends consisting of 50 percent RAP + 50 percent Class 5, 75 percent RAP + 25 percent Class 5, and 100 percent RAP material. On the basis of actual test results, the study compared the resilient modulus variability of RAP with that of unbound granular materials. To investigate the effect of variability in resilient modulus prediction as the result of model selection, the resilient modulus value was predicted using three models— K-θ model, MEPDG model, and Witczak model. The impact of resilient modulus variability on flexible pavement performance for RAP as opposed to granular material was investigated by using the MEPDG. The results indicated that the base layer resilient modulus variability had more impact on thin asphalt pavement than on thick. The alligator cracking was found to be more sensitive than rutting, to the change in M_r. The difference of the predicted resilient modulus under the two states of stress [(σ_3 = 5 psi and σ_d = 15 psi) and (σ_3 = 2 psi and σ_d = 4 psi)] reflected a difference of 10% to 500% on predicted alligator cracking and a difference of 3 to 7 years on the life expectancy of different pavement sections.

- Idaho Transportation Department: El-Badawy et al. (2011) conducted a study on prediction of subgrade resilient modulus for the implementation of the MEPDG in Idaho. For this study, historical geotechnical soil testing results were collected from Idaho Transportation Department materials reports and soil profile scrolls, and analyzed. Three MEPDG computer simulation runs were conducted as a part of a limited sensitivity analysis to investigate the influence of the subgrade modulus variation on the predicted pavement performance. The results indicated that the subgrade modulus affected the overall pavement performance. All the performance indicators, the subgrade rutting and consequently total rutting, both longitudinal and alligator fatigue cracking as well as the pavement smoothness, were affected. The alligator fatigue cracking was significantly influenced. It was also noted that as the foundation became stronger the rutting and fatigue cracking became less, leading to better pavement smoothness.

B.1.3 Transverse Cracking

- Wisconsin DOT: The MEPDG Implementation study completed by Mallela et al (2008) indicated the subgrade type and subgrade improvement technique had no impact on predicted transverse cracking.

- Iowa DOT: Results from Iowa implementation study by Coree et al. (2005) indicated type of subgrade (or resilient modulus value) was in the between low sensitive and insensitive range with regards to transverse cracking.

- Michigan DOT: The study by Buch et al. (2008) to highlight the evaluation of the current performance models for JPCP and HMA pavements indicated subgrade soil or material had no impact on transverse cracking.

B.1.4 Rutting

- Schwartz (2007) study indicated an increase in subgrade resilient modulus resulted in a decrease in rutting.

- Minnesota DOT: The results from the MEPDG implementation study by Cochran et al (2007) indicated the total rutting had a positive correlation with mix gradation and subgrade type, and was found to be larger when coarse mix gradation and an A6 soil were used.

- Arizona DOT MEPDG implementation: Darter et al (2007) indicated the subgrade type had a small effect on rutting; with the coarse-grained subgrade materials (A-2-4) performing slightly better than the weaker, fine-grained (A-6) materials.

- Wisconsin DOT: The MEPDG Implementation study by Mallela et al (2008) indicated the stiffer bases, subgrades, and improved layers resulted in a lower predicted rutting. Therefore, material stiffness as a whole can be considered to have a moderate impact on total predicted rutting.

- Ohio DOT: For the ODOT MEPDG implementation study by Mallela et al (2009), the sensitivity analysis indicated the lower subgrade modulus or finer-grained soils, the higher was rutting.

- Missouri DOT: A sensitivity analysis was carried out by Mallela et al. (2009) as a part of M-E design implementation in Missouri. The inputs to the MEPDG for each baseline case and their respective ranges to conduct the sensitivity analysis were determined from Missouri-specific information—i.e., a general range of values observed in Missouri pavement sections and as reported in relevant documents such as design reports, specifications, and databases. It was determined through the analysis that stiffer subgrades, and to some extent stiffer base layers, resulted in a lower predicted rutting. A limited sensitivity analysis was performed to determine the reasonableness of the locally calibrated HMA models. The results indicated that the subgrade type and modulus have a smaller impact on the predicted rutting. The depth to water table had no impact on total rutting for the conditions evaluated.

- Iowa DOT: The results from Iowa implementation study by Coree et al. (2005) indicated that the type of subgrade (or resilient modulus value) was in the between low sensitive and insensitive range with regards to rutting.

- Utah DOT MEPDG implementation study by Darter et al. (2005) indicated the lower the subgrade modulus, the higher the predicted rutting.

- Michigan DOT: The study by Buch et al. (2008) to highlight the evaluation of the current performance models for JPCP and HMA pavements indicated the subgrade material had maximum impact on rutting.

- Tennessee DOT: The study by Zhou et al (2013) on the seasonal resilient modulus inputs for 14 Tennessee soils and their effects on asphalt pavement performance indicated that the seasonal variation of subgrade resilient modulus due to the moisture change will increase the rutting depth in subgrade.

- Idaho IDT: The study by El-Badawy et al. (2011) on prediction of subgrade resilient modulus for the implementation of the MEPDG in Idaho indicated that the results indicated that the subgrade modulus affected the total rutting significantly.

B.1.5 Roughness or IRI

- Arizona DOT: The Arizona MEPDG implementation study by Darter et al. (2007) indicated the subgrade type had a small effect on IRI; with the coarse grained subgrade materials (A-2-4) performing slightly better than the weaker, fine grained (A-6) materials.

- Wisconsin DOT: The ME Design Implementation in Wisconsin by Mallela et al. (2008) indicated the stiffness of a stabilized subgrade layer had a moderate effect on IRI.

- Ohio DOT: For the ODOT ME design implementation by Mallela et al. (2009) the sensitivity analysis indicated the lower the subgrade type/modulus, the higher was the IRI.

- Missouri DOT: The sensitivity results of locally calibrated HMA models from Missouri ME design implementation by Mallela et al. (2009) indicated subgrade type had considerable impact on IRI.

- Iowa DOT: The results from Iowa implementation study by Coree et al. (2005) indicated that the type of subgrade (or resilient modulus level) was in the between low sensitive and insensitive range with regards smoothness/IRI.

- Utah DOT MEPDG implementation study by Darter et al. (2005) indicated the lower the subgrade type/modulus the higher was the IRI.

- Michigan DOT: The study by Buch et al. (2008) to highlight the evaluation of the current performance models for JPCP and HMA pavements indicated the subgrade material had maximum impact on IRI.

- Minnesota DOT: The study by Attia and Abdelrahman (2010) on the variability in resilient modulus of RAP as base layer and its impact on flexible pavement performance, indicated that the variability in resilient modulus caused by sample variability and model selection had no impact on the IRI for all materials; there was less than 1 year of variation in pavement life for both thin and thick HMA pavements.

B.2 JPC and CRC Pavements

The MEPDG predicts four distresses for rigid pavements, which include: mid-slab cracking and faulting for JPCP, punchouts for continuously reinforced concrete pavements (CRCP), and roughness as measured by IRI for both JPCP and CRCP.

Similar to flexible pavements, a comprehensive study was conducted by Schwartz et al. (2011) to evaluate the sensitivity of pavement performance predicted by the MEPDG to the values of the design inputs. The overall objective of this NCHRP funded research was to determine the sensitivity of the performance predicted by the MEPDG to variability of the design input values for flexible and rigid pavements. The evaluation was performed through an initial triage, extensive one-at-a-time sensitivity analyses, and comprehensive global sensitivity analyses. During this study, global sensitivity analyses was carried out for five pavement types under five climate conditions and three traffic levels. The results have been summarized in Tables 32 and 33. The following subsections provide an overview on how each predicted distress is affected by changes in resilient modulus of unbound aggregate base layers, embankments and subgrade soils.

Fernando et al. (2007) also conducted a comprehensive sensitivity analysis of rigid pavements as part of the Florida DOT MEPDG implementation study. The rigid pavement was a six layer structure with a JPCP slab, two existing asphalt concrete layers, limerock base, stabilized subgrade, and sand subgrade. The results indicated that the moduli of unbound materials as well as the modulus of subgrade reaction had minimal effect on PCC performance predictions.

Table 32. Maximum Change in Resilient Modulus Resulting in Less Than 0.5 Inches of HMA

JPCP Inputs: Subgrade/Base Properties	Level of Sensitivity for JPCP Distresses[1]	
	Faulting	Cracking
Stiffness or Resilient Modulus	S	S
Poisson's Ratio	NS	NS
Compacted/Uncompated Layer	NS	NS
[1]VS = very sensitive, S = sensitive, NS = non-sensitive.		

Table 33. Maximum Change in Resilient Modulus Resulting in Less Than 0.5 Inches of HMA

CRCP Inputs: Subgrade/Base Properties	Level of Sensitivity for CRCP[1]		
	Punch-Outs	Maximum Crack Width	Minimum LTE
Stiffness or Resilient Modulus	S	NS	NS
Poisson's Ratio	NS	NS	NS
Compacted/Uncompated Layer	NS	NS	NS
[1]VS = very sensitive, S = sensitive, NS = non-sensitive.			

B.2.1 Transverse Joint Faulting

- Minnesota DOT: Cochran et al. (2007) in the Minnesota MEPDG implementation study conducted sensitivity evaluation for rigid pavements. Two types of bases (class 5 and class3) and two types of subgrades (A-6 and A-3) were used for the analysis of foundation support. No significant difference was observed in faulting between the base-subgrade combinations. Unlike for the cracking analysis, however, subgrade strength drove faulting more than base strength (subgrade A-3 performed slightly better than A-6 regardless of the base material). In other words, an increase in subgrade modulus caused a decrease in faulting.

- The Arizona MEPDG implementation study by Darter et al. (2007) indicated the subgrade type had a significant effect on joint faulting. The coarse grained subgrade materials (A-2-4) showed lower joint faulting performance than the weaker, fine grained (A-6) soil.

- Wisconsin DOT: The sensitivity analysis of JPCP pavements during the MEPDG Implementation in Wisconsin by Mallela et al. (2008) indicated that the subgrade type had a small impact on the predicted faulting.

- Ohio DOT: The JPCP sensitivity results from ODOT implementation study by Mallela et al. (2009) indicated that subgrade type affected joint faulting considerably.

- Missouri DOT: Sensitivity analysis for rigid pavements by Mallela et al. (2009) for Missouri M-E design implementation indicated that subgrades with higher fines in sections located in wet climates resulted in higher faulting than coarse grained subgrades located in dry climate zones.

- Iowa DOT: A sensitivity analysis of large number of input parameters on the predicted pavement distresses was conducted by Coree et al. (2008) for two rigid pavement sections that were selected from the Iowa DOT Pavement Management Information System (PMIS). It was noted that unbound layer modulus fell in the range of sensitive to very insensitive towards faulting.

- Utah DOT: For the JPCP pavements analyzed during the Utah M-E design implementation study by Darter et al. (2005), the subgrade type was found to have a relatively small effect on predicted transverse joint faulting. It was noted that the softer the subgrade the more was the faulting predicted.

- Michigan DOT: The results of the sensitivity analysis in rigid pavements, from the study by Buch et al. (2008) to highlight the evaluation of the current performance models for JPCP and HMA pavements indicate the effect of subgrade type was found significant on joint faulting.

B.2.2 Transverse Cracking

- Minnesota DOT: The results of sensitivity analysis of rigid pavements for the Minnesota DOT MEPDG implementation study by Cochran et al. (2007) indicated a minor difference in cracking between the four possible combinations of supporting layer materials. The class 5 base performed better than the class 3 base regardless of the subgrade material.

- The Arizona MEPDG implementation study by Darter et al. (2007) indicated that for the rigid pavements, the subgrade type had a small effect on cracking.

- Wisconsin DOT: The sensitivity analysis on JPCP pavements during the ME Design Implementation in Wisconsin by Mallela et al. (2008) indicated that the source of the DGAB material (quarry versus gravel pits) and subgrade type showed a relatively minor impact on slab cracking.

- Ohio DOT: The JPCP sensitivity results from the Ohio DOT implementation study by Mallela et al. (2009) indicated that subgrade type affected slab cracking considerably.

- Missouri DOT: Sensitivity analysis for rigid pavements by Mallela et al. (2009) for Missouri ME design implementation indicated the subgrade type had a relatively larger impact on slab cracking. Stiffer subgrades resulted in larger cracking due to increased subgrade k-values that in turn resulted in higher curling stresses in the slab.

- The Iowa DOT MEPDG implementation by Coree et al. (2008) for two rigid pavement sections indicated that the unbound layer modulus fell in the range of sensitive to very insensitive towards smoothness/IRI.

- Utah DOT: For the JPCP pavements analyzed during the Utah M-E design implementation study by Darter et al. (2005), subgrade type (or resilient modulus) had a more significant effect on transverse cracking, the higher the modulus of the subgrade the larger was the amount of transverse cracking. This was due to increase in thermal curling stresses when the foundation becomes stiffer.

- Michigan DOT: The results of the sensitivity analysis in rigid pavements, from the study by Buch et al. (2008) to highlight the evaluation of the current performance models for JPCP and HMA pavements indicated that the effect of subgrade type was fairly insignificant on slab cracking.

B.2.3 Roughness or IRI

- The Arizona MEPDG implementation study by Darter et al. (2007) indicated that for the rigid pavements, the subgrade type had a significant effect on IRI. The coarse grained subgrade materials (A-2-4) showed lower IRI performance than the weaker, fine grained (A-6) soil.

- Wisconsin DOT: The sensitivity analysis on JPCP pavements during the ME Design Implementation in Wisconsin by Mallela et al. (2008) indicated that the subgrade type had a small impact on the predicted IRI.

- Ohio DOT: The JPCP sensitivity results from ODOT implementation study by Mallela et al. (2009) indicated that subgrade type had a moderate effect on IRI.

- Missouri DOT: A limited sensitivity analysis was performed by Mallela et al. (2009) to determine the reasonableness of the locally calibrated JPCP IRI mode for Missouri ME design implementation. It was noted that the subgrade soil type had moderate impact on predicted IRI.

- The Iowa DOT MEPDG implementation by Coree et al. (2008) for two rigid pavement sections indicated that the unbound layer modulus fell in the range of low sensitive to insensitive towards cracking.

- Utah and Michigan DOT: For the JPCP pavements, Darter et al. (2005) reported subgrade type was found to have a relatively small effect on the predicted IRI. Conversely, Buch et al. (2008) reported, for the state of Michigan, the effect of subgrade type on IRI was significant for JPCP and HMA pavements.

APPENDIX C PHYSICAL PROPERITES OF SOILS INCLUDED IN TEST PROGRAM

Appendix C includes a summary of the physical properties of the soils that were included in the ruggedness and precision and bias test programs. Table 34 includes the physical properties of the three soils and one crushed aggregate base material included in the test program.

Table 34. Physical Properties of the Soils and Materials included in the Precision and Bias Test Program

<table>
<tr><th colspan="2" rowspan="2">Physical Property</th><th colspan="4">Soil or Material Types or Identification</th></tr>
<tr><th>Georgia GAB, A-1-a</th><th>Alabama A-2-4</th><th>Georgia A-2-4</th><th>Mississippi A-7-6(75)</th></tr>
<tr><td colspan="2">Soil Type</td><td>Crushed Stone</td><td>Gravelly Silty Sand</td><td>Gravelly Silty Sand</td><td>Tan Clay</td></tr>
<tr><td colspan="2">Unified Soil Classification System</td><td>---</td><td>SC-SM</td><td>SM</td><td>CH</td></tr>
<tr><td rowspan="2">Atterberg Limits</td><td>Liquid Limit</td><td>---</td><td>26</td><td>---</td><td>86</td></tr>
<tr><td>Plasticity Index</td><td>NP</td><td>4</td><td>NP</td><td>70</td></tr>
<tr><td colspan="2">Compaction Method</td><td>AASHTO T 180, Method D</td><td>ASTM D698, Method B</td><td>ASTM D698, Method B</td><td>ASTM D698-00a, Method A, Standard</td></tr>
<tr><td colspan="2">Maximum Dry Density, pcf</td><td>144.0</td><td>120.2</td><td>110.1</td><td>94.8</td></tr>
<tr><td colspan="2">Optimum Water Content, percent</td><td>6.0</td><td>10.6</td><td>14.6</td><td>25.1</td></tr>
<tr><td rowspan="4">Sieve Size, % passing</td><td>3/8 in.</td><td></td><td>99.8</td><td>99.6</td><td>100</td></tr>
<tr><td>#4</td><td></td><td>81.2</td><td>81.8</td><td>100</td></tr>
<tr><td>#40</td><td></td><td>43.7</td><td>55.0</td><td></td></tr>
<tr><td>#200</td><td></td><td>29.4</td><td>23.7</td><td>95.9</td></tr>
<tr><td colspan="6">NP – Non Plastic</td></tr>
</table>

APPENDIX D RESILIENT MODULUS TEST RESULTS FROM THE PRECISION TESTS

Appendix D includes a graphical summary and presentation of all the resilient modulus test results from the precision testing plan. The test results included in Appendix D are grouped by material type: A-6-7 high plasticity clay from Mississippi, A-2-4 low plasticity fine-grained soil from the NCAT test track in Alabama (A-4 was reported in the NCAT test reports, but the soil samples turned out to be classified as an A-2-4), A-2-4 fine-grained soil from the Atlanta area in Georgia, and a good quality granular aggregate base satisfying the Georgia DOT material specifications. Two graphs are provided for each laboratory that submitted test results for a specific material or soil: one graph shows the resilient modulus test results for each confining pressure used as a function of bulk stress, and the second graph shows the COV and standard deviation from the five test specimens as a function of the resilient modulus magnitude for a single operator.

D.1 A-7-6 High Plasticity Clay

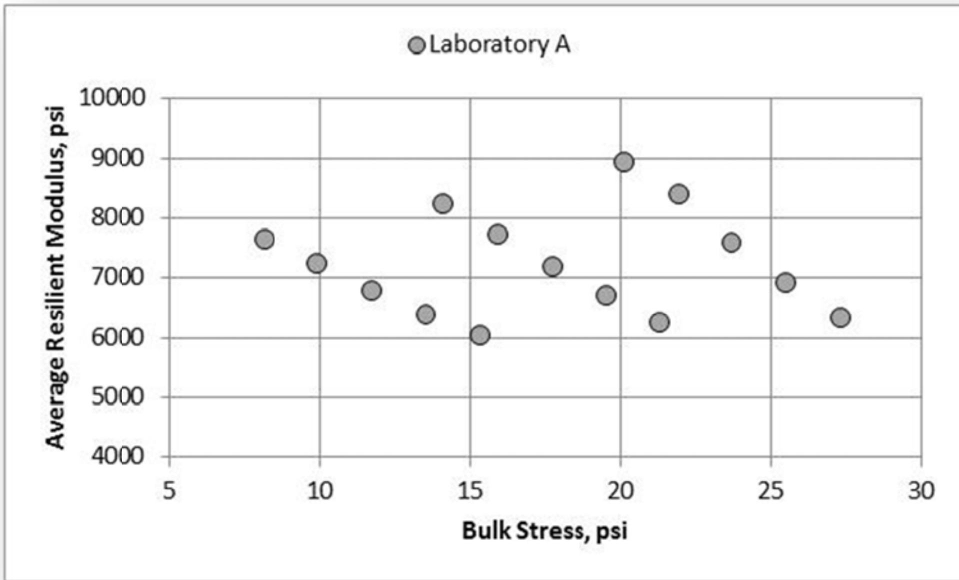

Figure 72. A-7-6 Soil, Laboratory A, Resilient Modulus

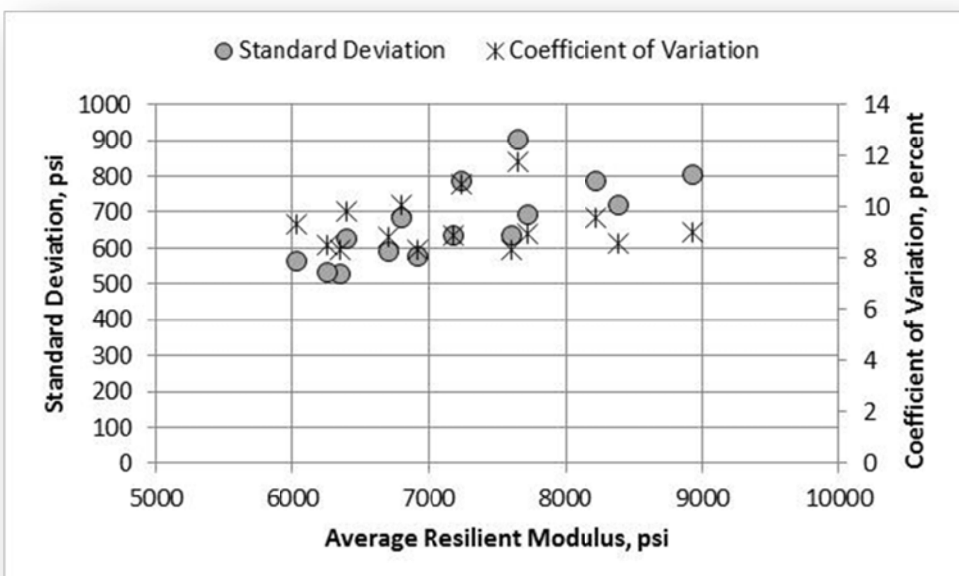

Figure 73. A-7-6 Soil, Laboratory A, Standard Deviation and Coefficient of Variation

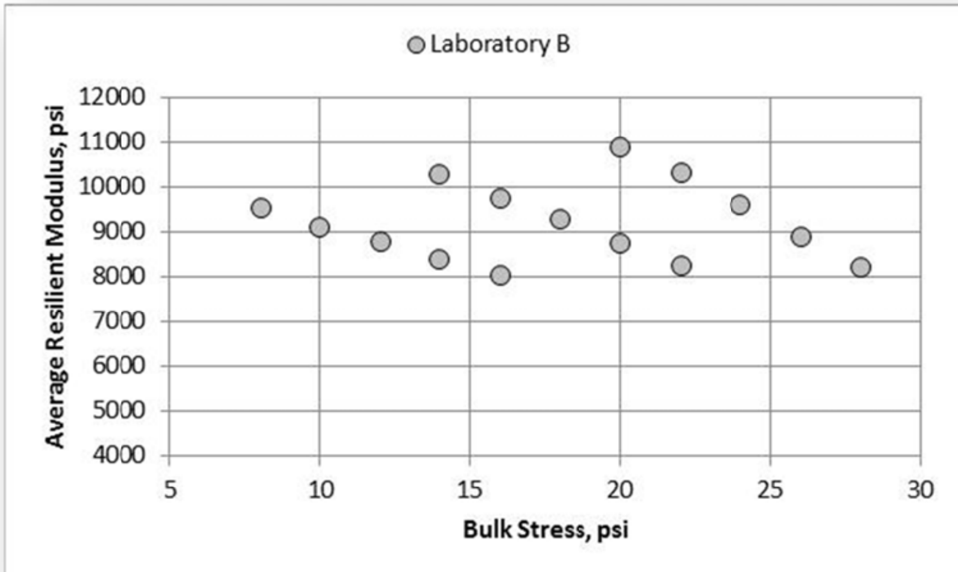

Figure 74. A-7-6 Soil, Laboratory B, Resilient Modulus

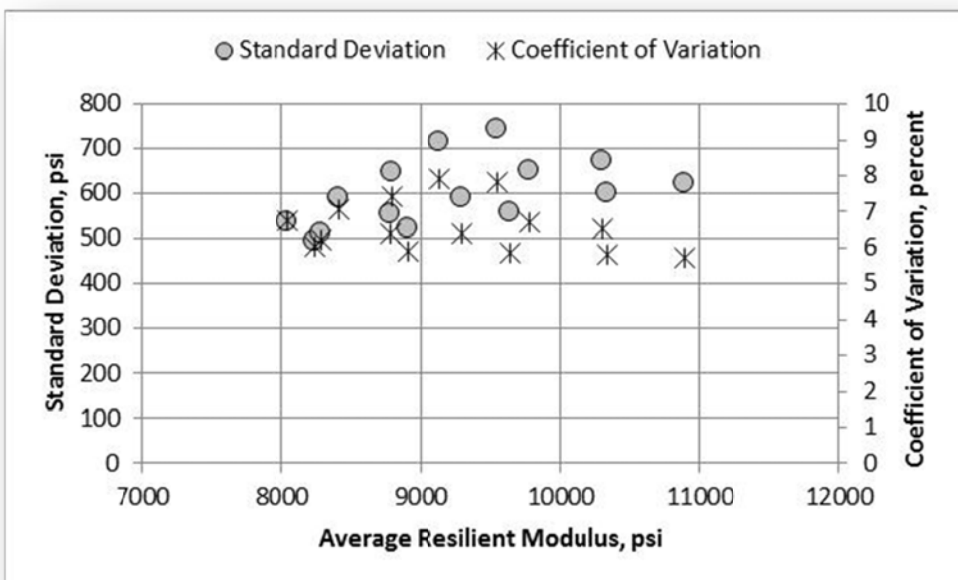

Figure 75. A-7-6 Soil, Laboratory B, Standard Deviation and Coefficient of Variation

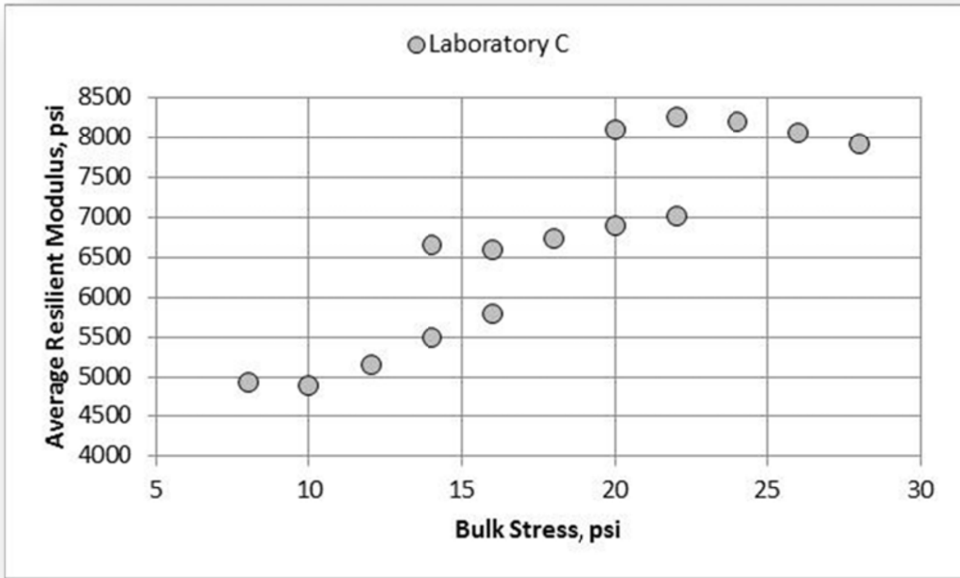

Figure 76. A-7-6 Soil, Laboratory C, Resilient Modulus

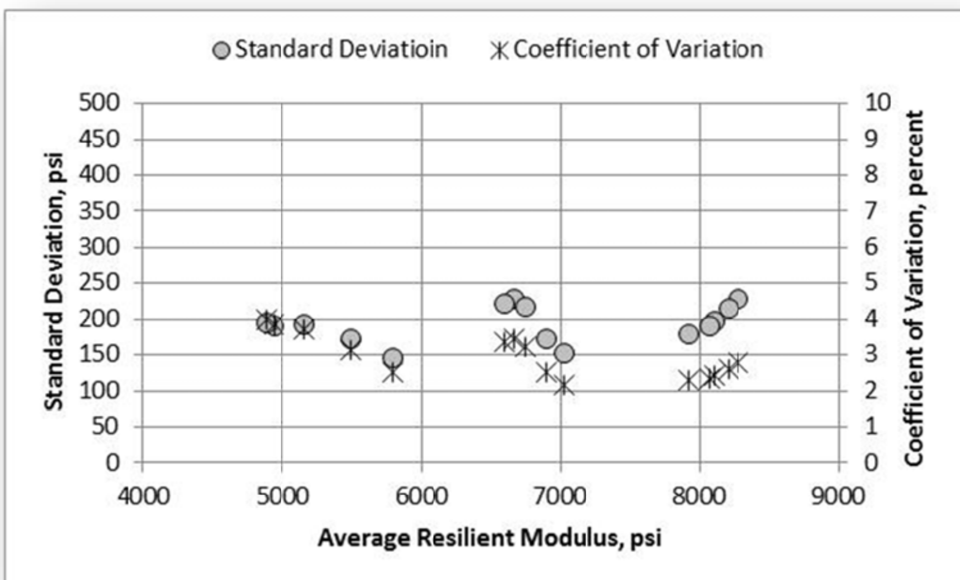

Figure 77. A-7-6 Soil, Laboratory C, Standard Deviation and Coefficient of Variation

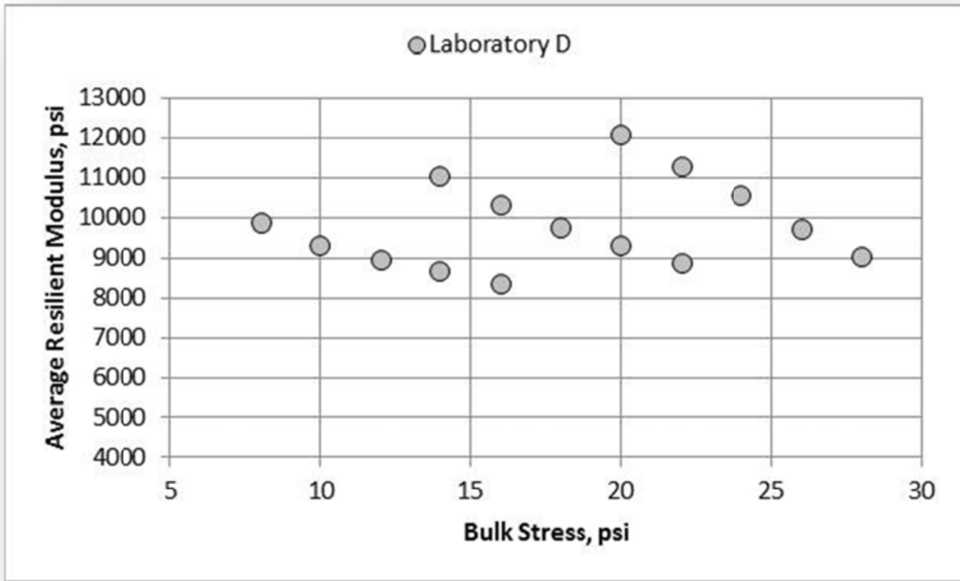

Figure 78. A-7-6 Soil, Laboratory D, Resilient Modulus

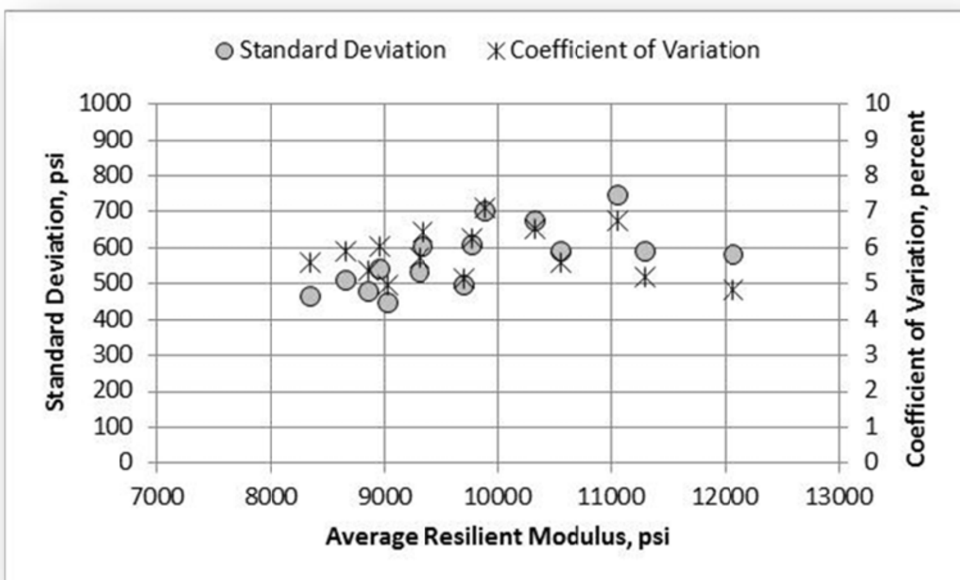

Figure 79. A-7-6 Soil, Laboratory D, Standard Deviation and Coefficient of Variation

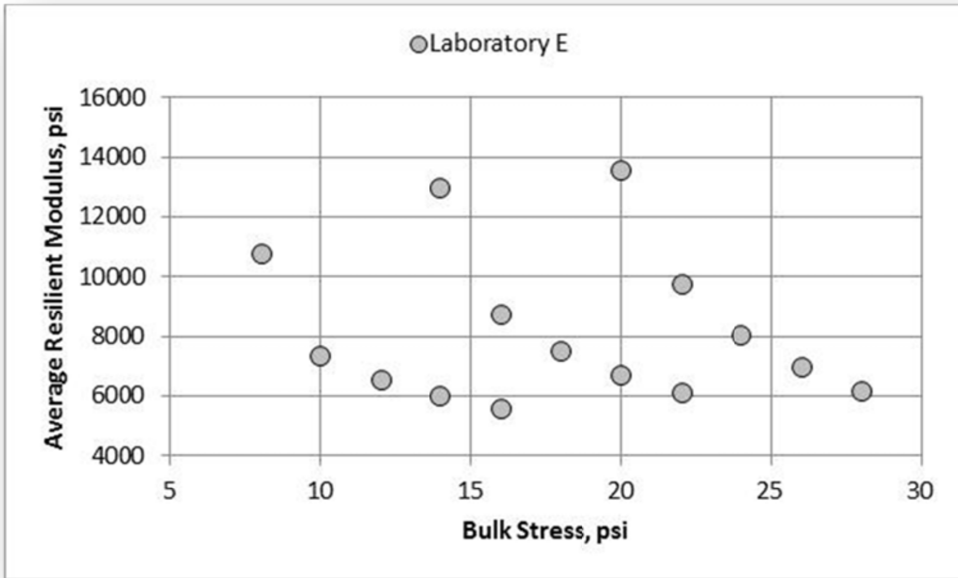

Figure 80. A-7-6 Soil, Laboratory E, Resilient Modulus

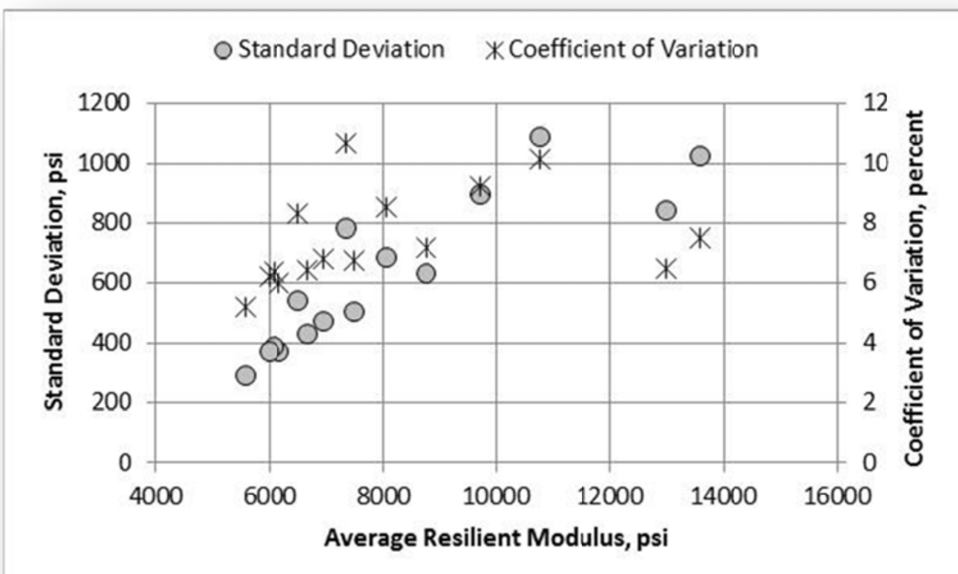

Figure 81. A-7-6 Soil, Laboratory E, Standard Deviation and Coefficient of Variation

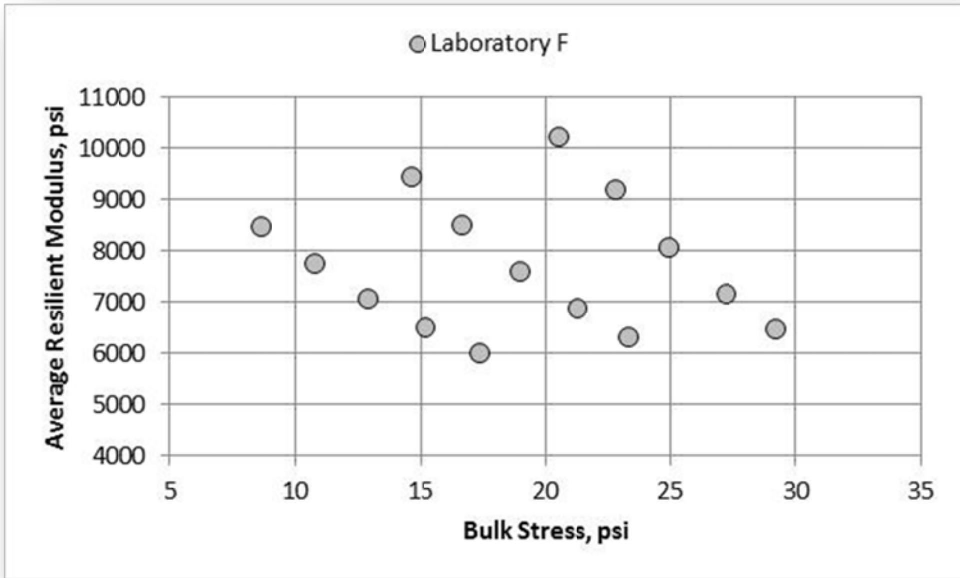

Figure 82. A-7-6 Soil, Laboratory F, Resilient Modulus

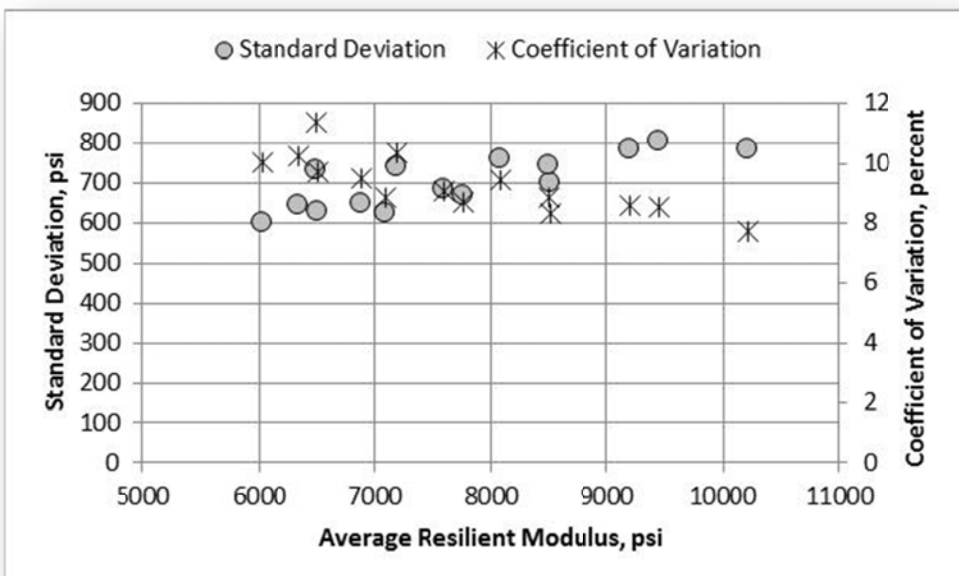

Figure 83. A-7-6 Soil, Laboratory F, Standard Deviation and Coefficient of Variation

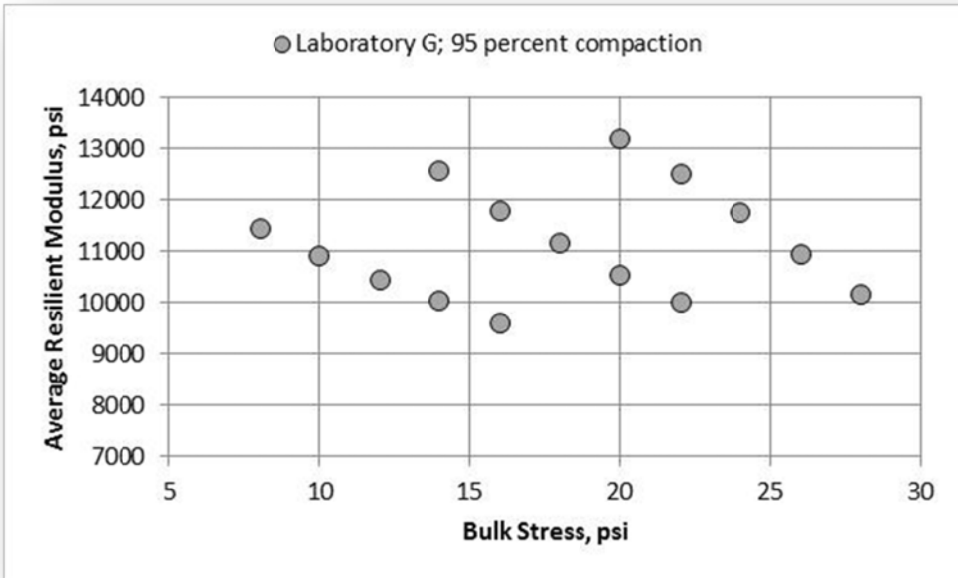

Figure 84. A-7-6 Soil, Laboratory G, 95 percent Compaction, Resilient Modulus

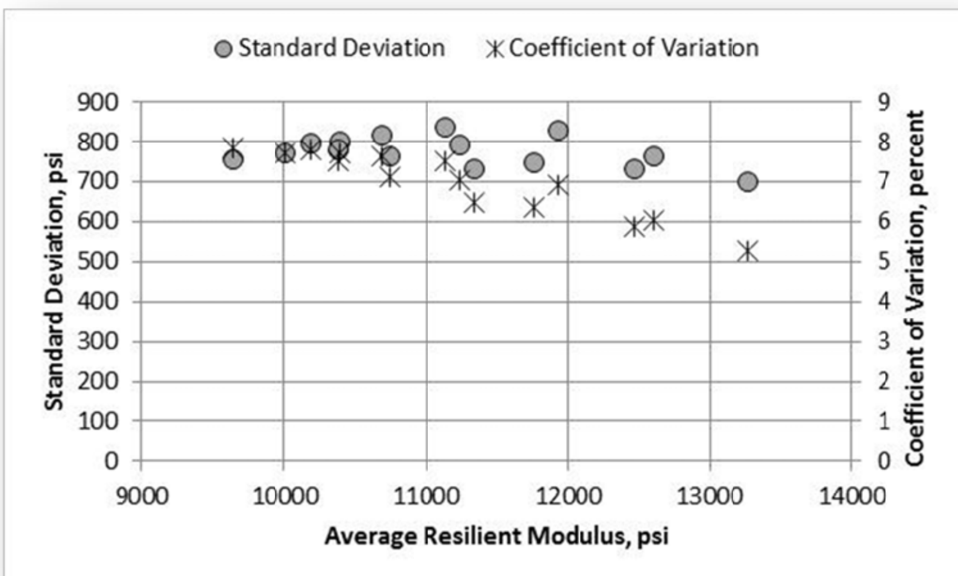

Figure 85. A-7-6 Soil, Laboratory G, 95 percent Compaction, Standard Deviation and Coefficient of Variation

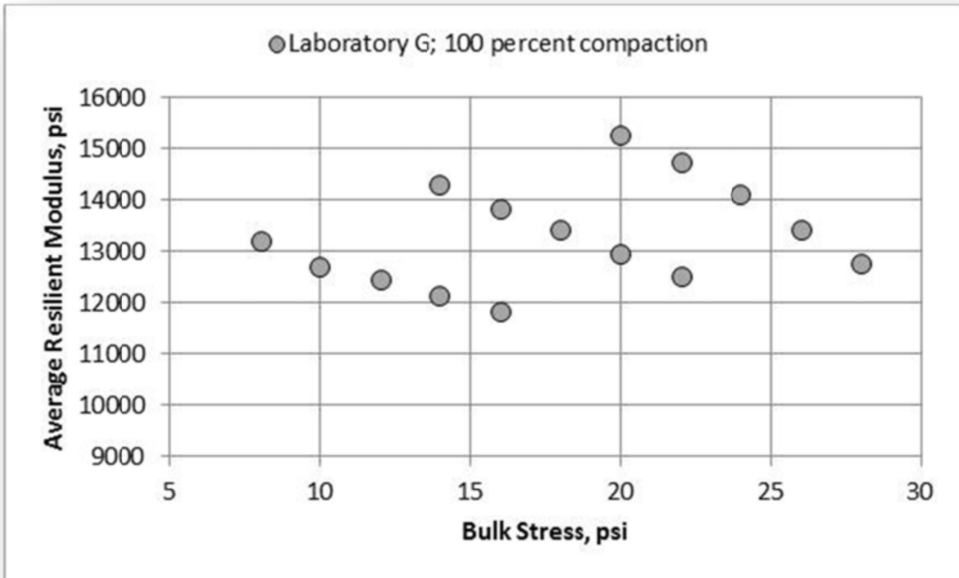

Figure 86. A-7-6 Soil, Laboratory G, 100 percent Compaction, Resilient Modulus

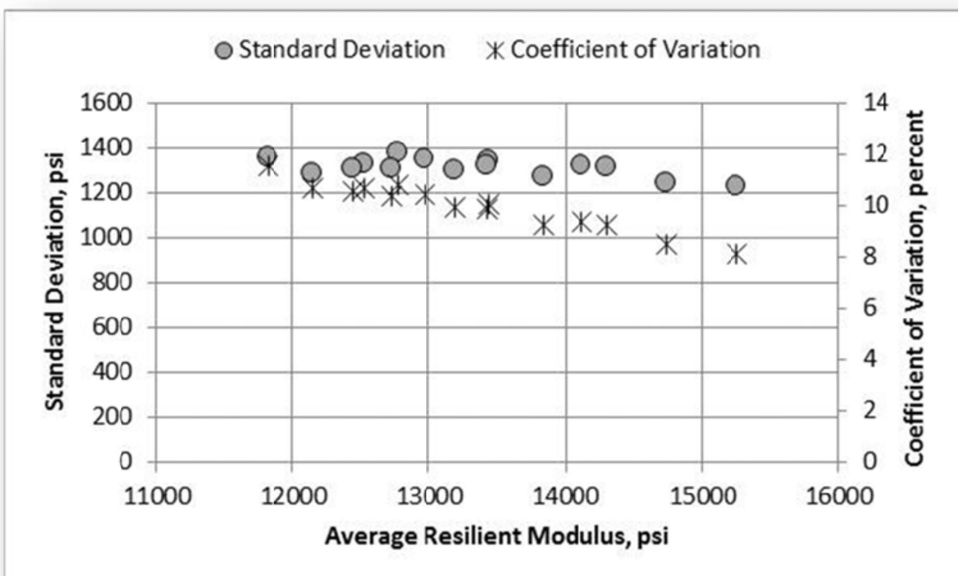

Figure 87. A-7-6 Soil, Laboratory G, 100 percent Compaction, Standard Deviation and Coefficient of Variation

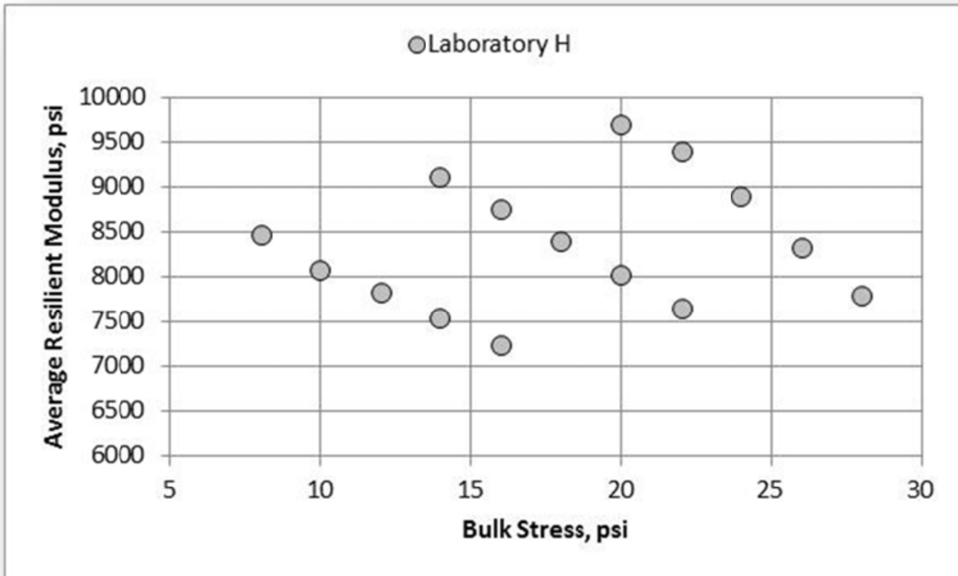

Figure 88. A-7-6 Soil, Laboratory H, Resilient Modulus

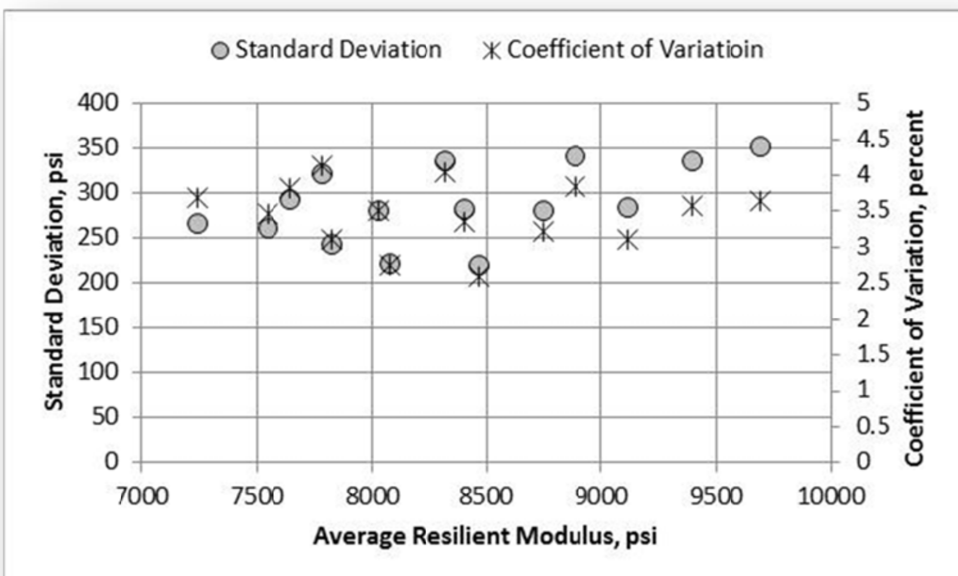

Figure 89. A-7-6 Soil, Laboratory H, Standard Deviation and Coefficient of Variation

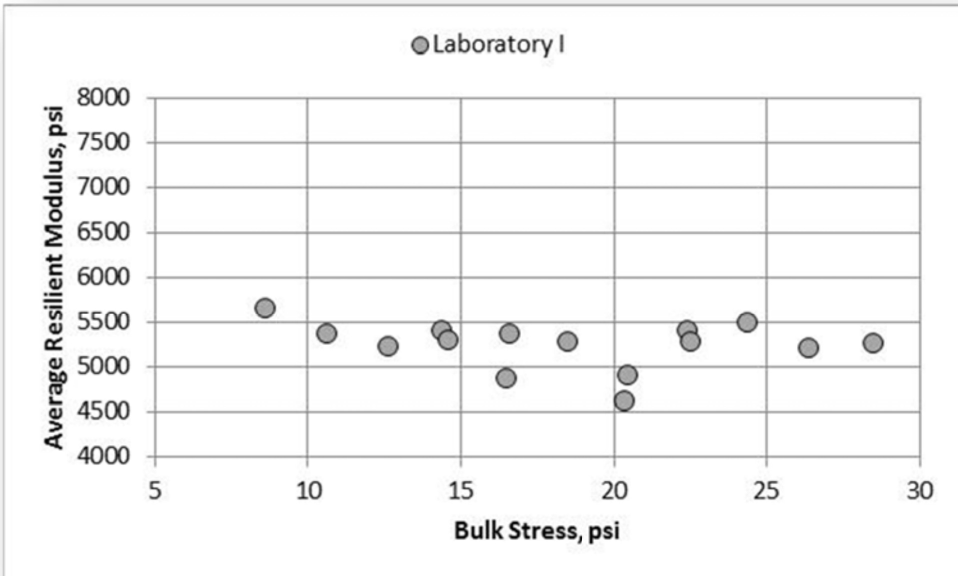

Figure 90. A-7-6 Soil, Laboratory I, Resilient Modulus

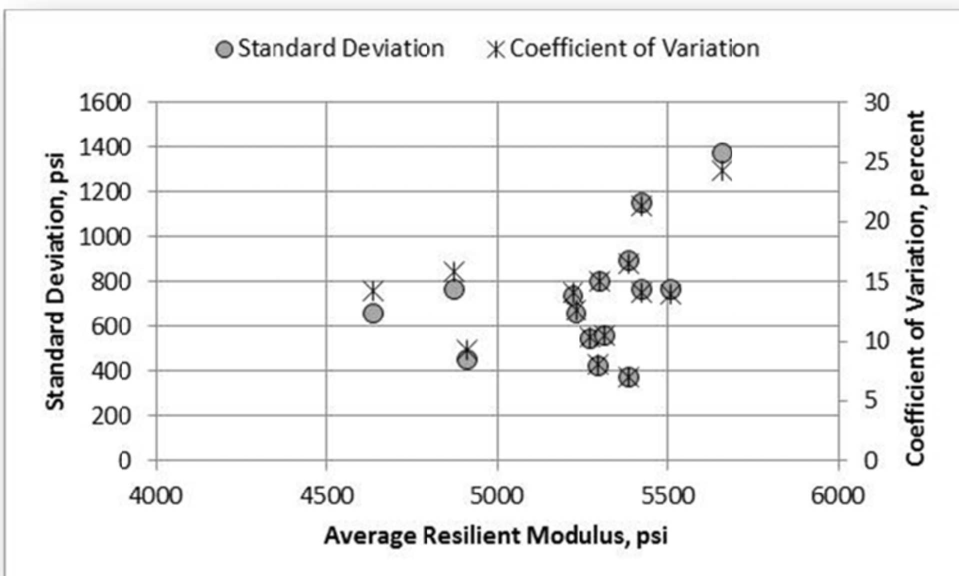

Figure 91. A-7-6 Soil, Laboratory I, Standard Deviation and Coefficient of Variation

D.2 A-2-4 Low Plasticity Fine-Grained Soil (NCAT Subgrade)

This soil was selected as an A-4 material, but was classified as an A-2-4.

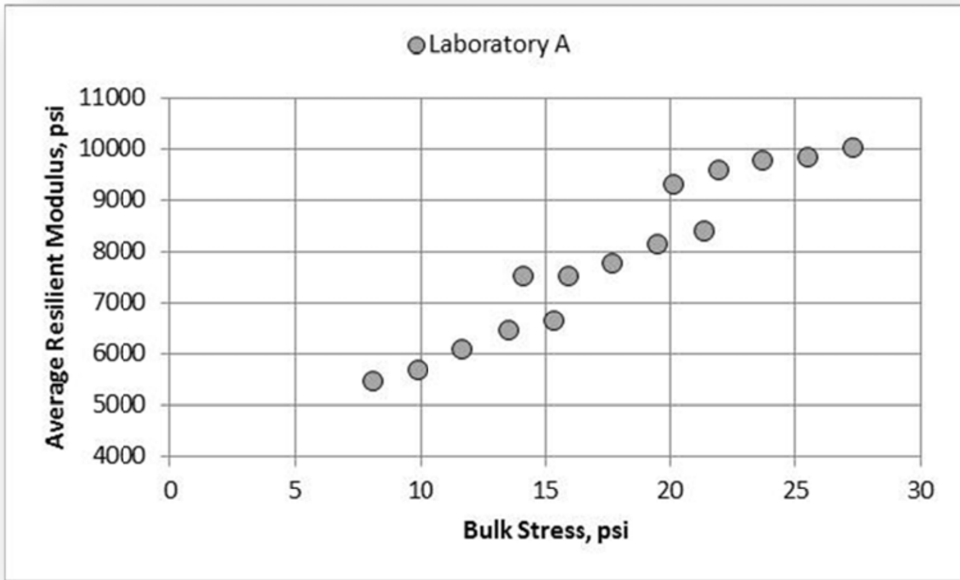

Figure 92. A-2-4 Soil (NCAT Subgrade), Laboratory A, Resilient Modulus

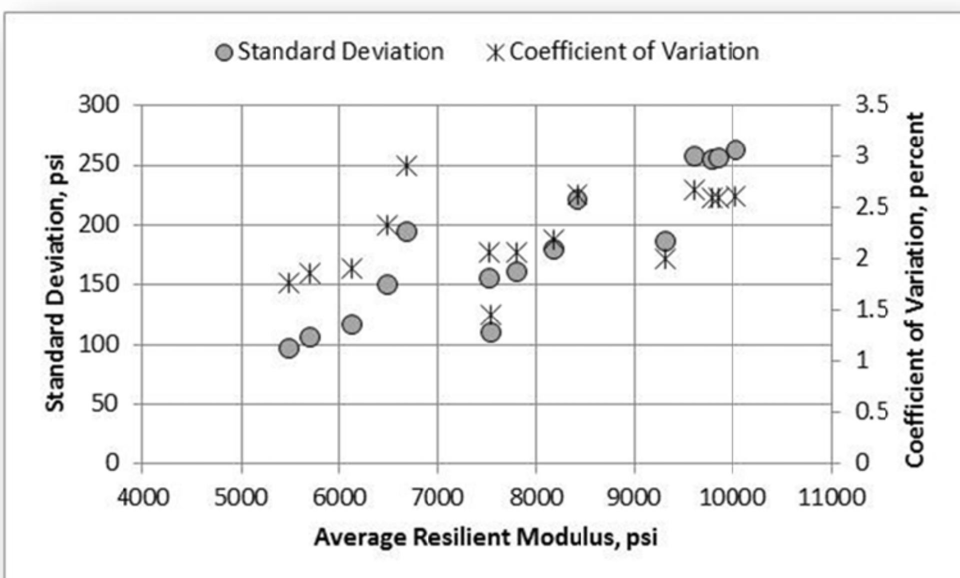

Figure 93. A-2-4 Soil (NCAT Subgrade), Laboratory A, Standard Deviation and Coefficient of Variation

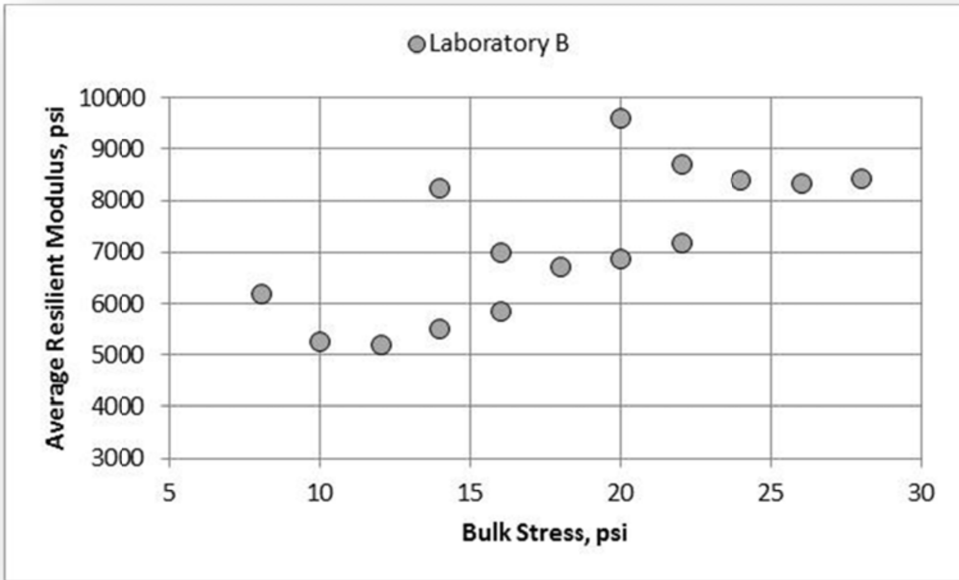

Figure 94. A-2-4 Soil (NCAT Subgrade), Laboratory B, Resilient Modulus

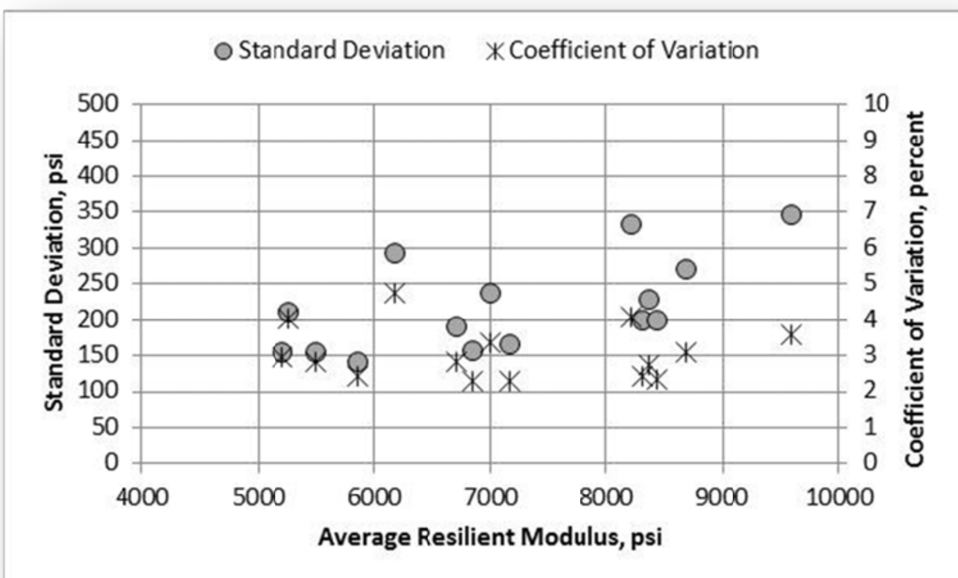

Figure 95. A-2-4 Soil (NCAT Subgrade), Laboratory B, Standard Deviation and Coefficient of Variation

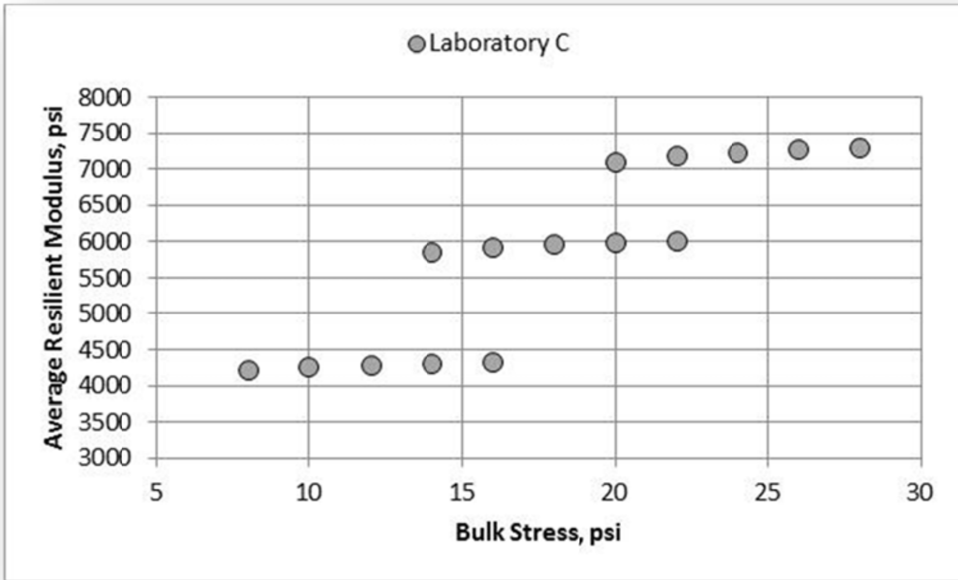

Figure 96. A-2-4 Soil (NCAT Subgrade), Laboratory C, Resilient Modulus

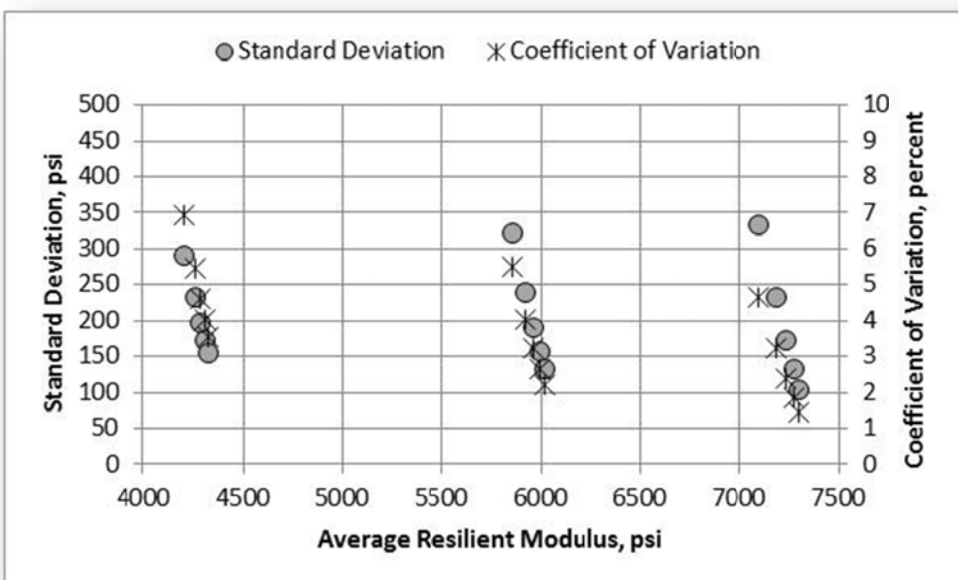

Figure 97. A-2-4 Soil (NCAT Subgrade), Laboratory C, Standard Deviation and Coefficient of Variation

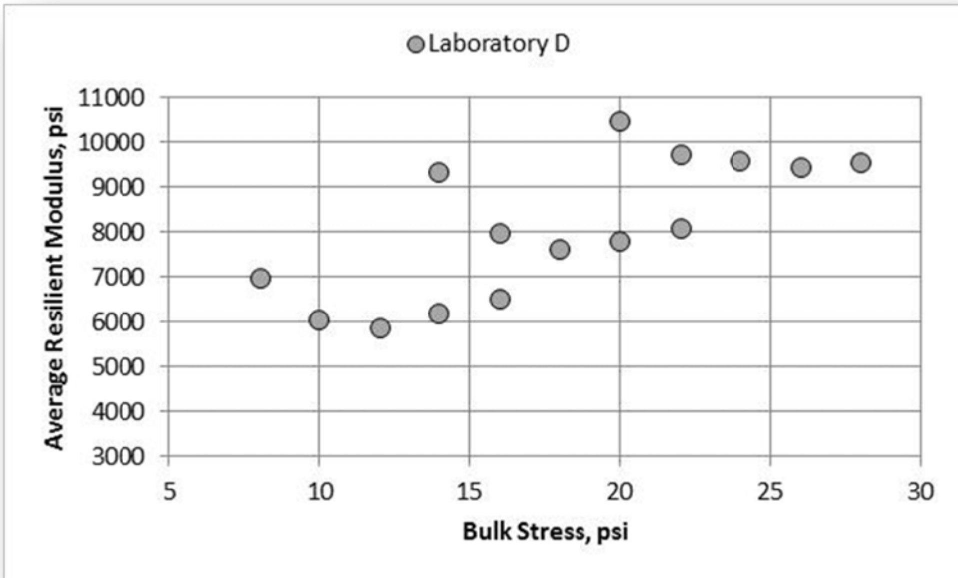

Figure 98. A-2-4 Soil (NCAT Subgrade), Laboratory D, Resilient Modulus

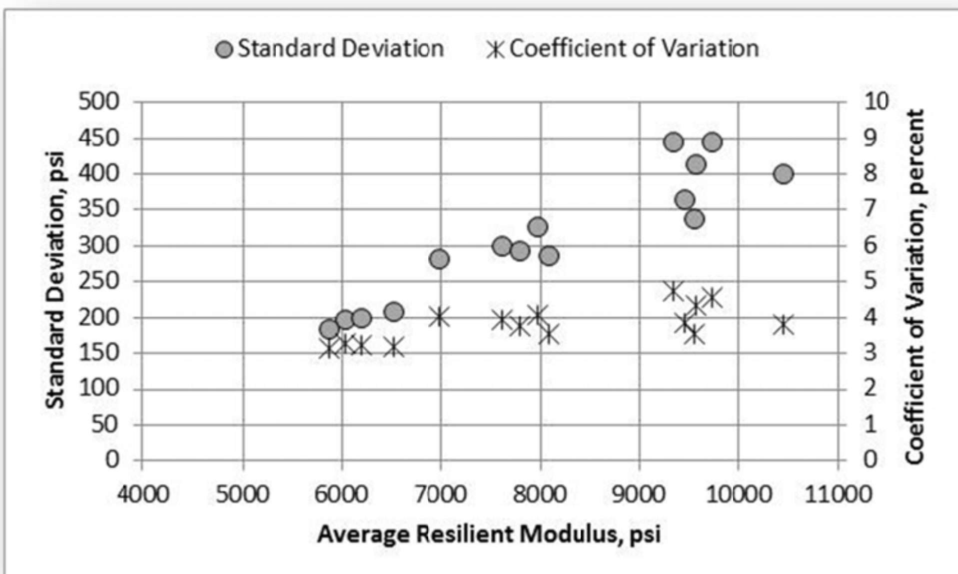

Figure 99. A-2-4 Soil (NCAT Subgrade), Laboratory D, Standard Deviation and Coefficient of Variation

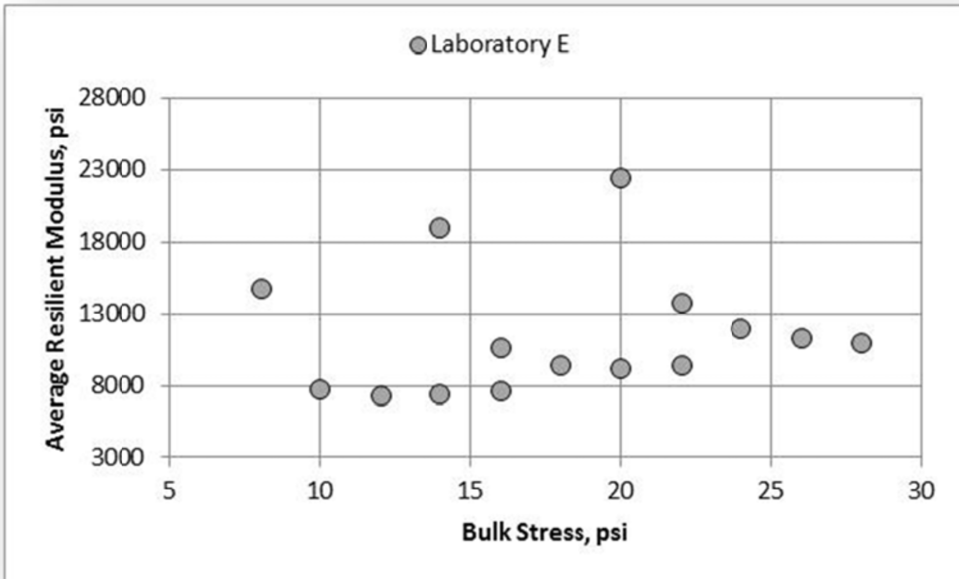

Figure 100. A-2-4 Soil (NCAT Subgrade), Laboratory E, Resilient Modulus

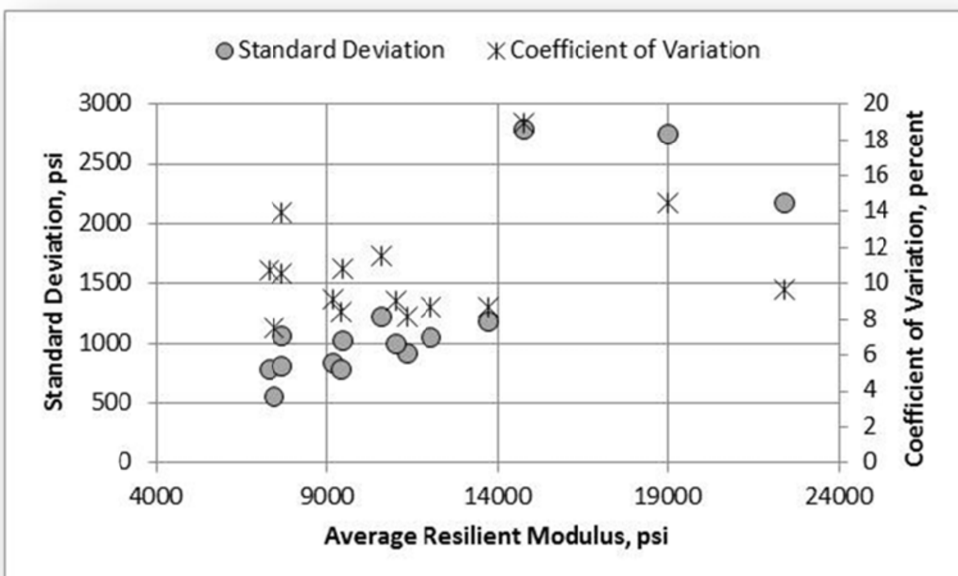

Figure 101. A-2-4 Soil (NCAT Subgrade), Laboratory E, Standard Deviation and Coefficient of Variation

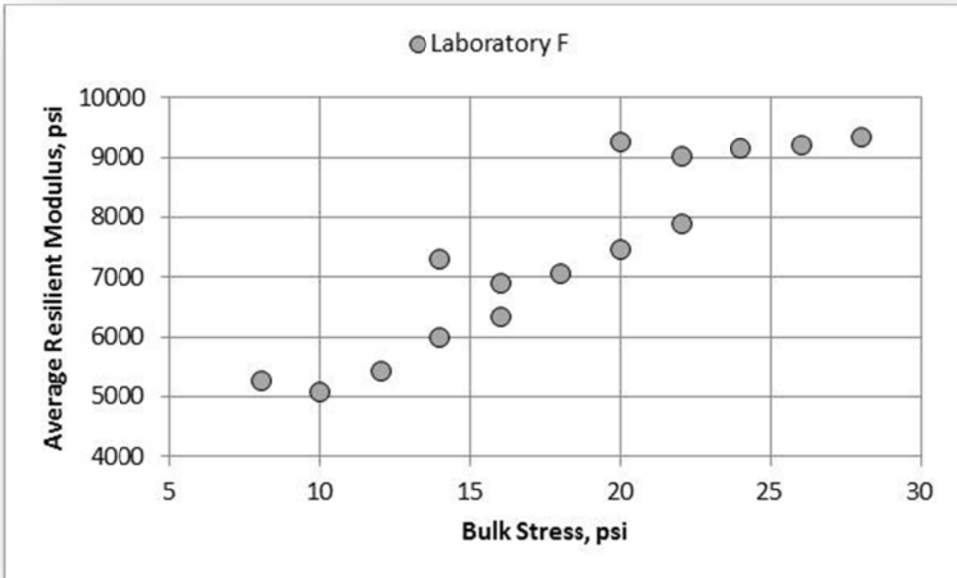

Figure 102. A-2-4 Soil (NCAT Subgrade), Laboratory F, Resilient Modulus

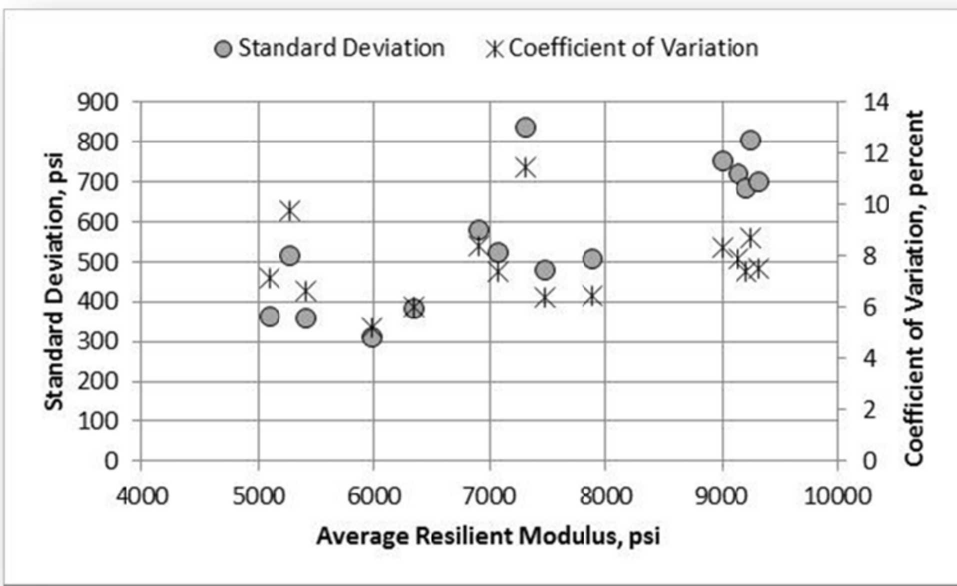

Figure 103. A-2-4 Soil (NCAT Subgrade), Laboratory F, Standard Deviation and Coefficient of Variation

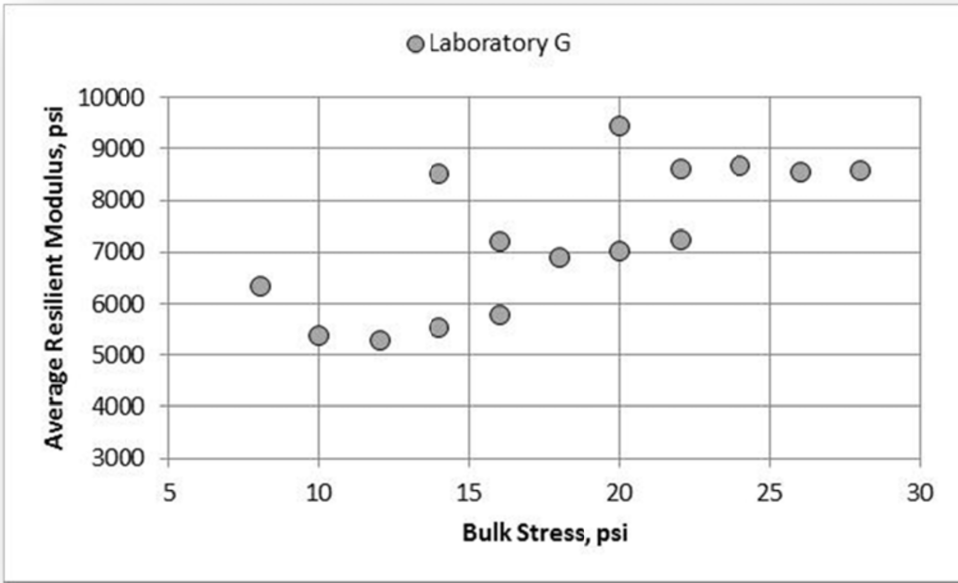

Figure 104. A-2-4 Soil (NCAT Subgrade), Laboratory G, Resilient Modulus

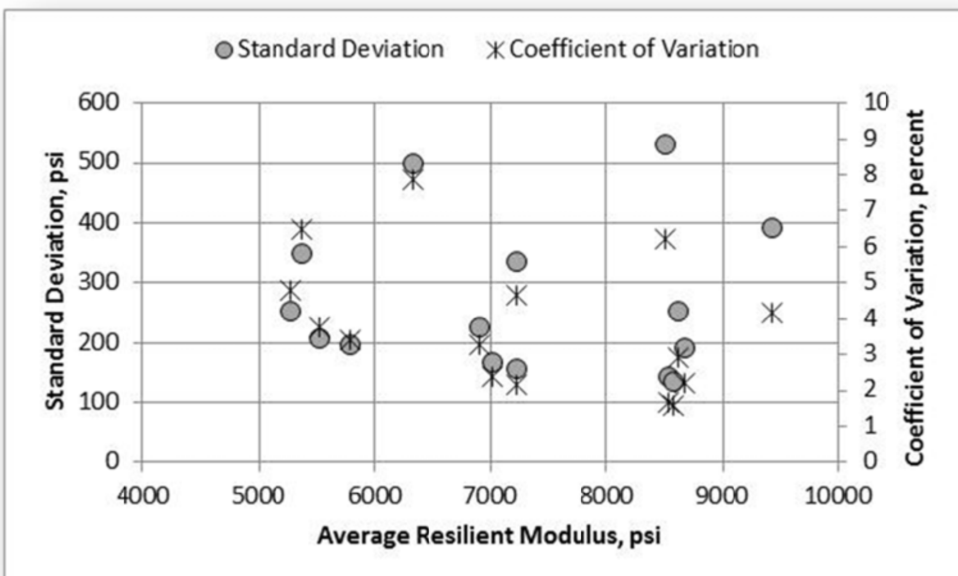

Figure 105. A-2-4 Soil (NCAT Subgrade), Laboratory G, Standard Deviation and Coefficient of Variation

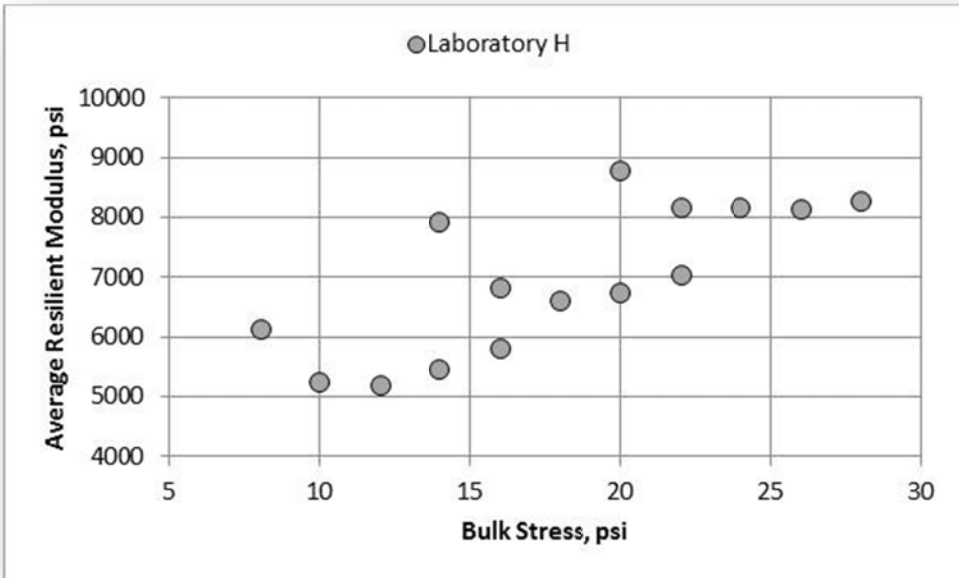

Figure 106. A-2-4 Soil (NCAT Subgrade), Laboratory H, Resilient Modulus

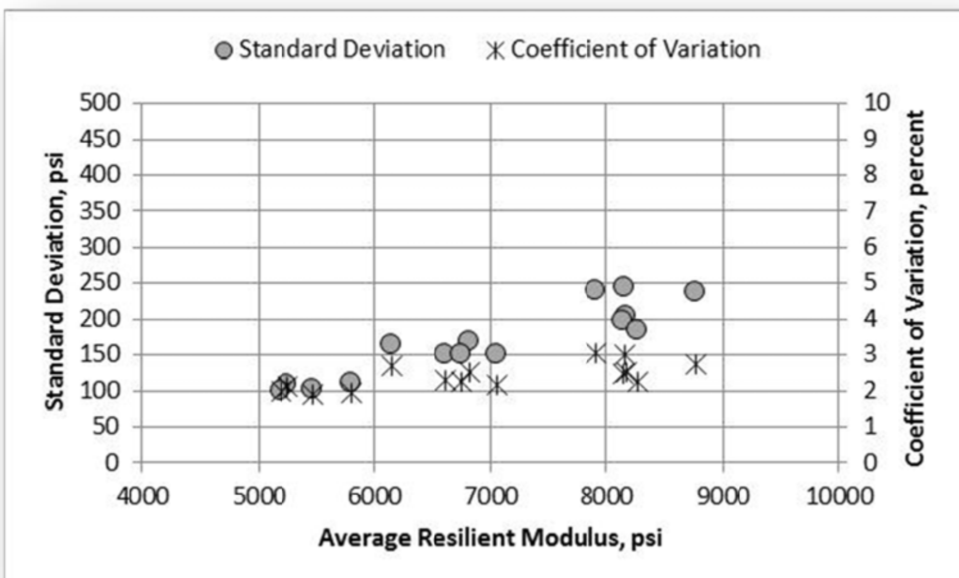

Figure 107. A-2-4 Soil (NCAT Subgrade), Laboratory H, Standard Deviation and Coefficient of Variation

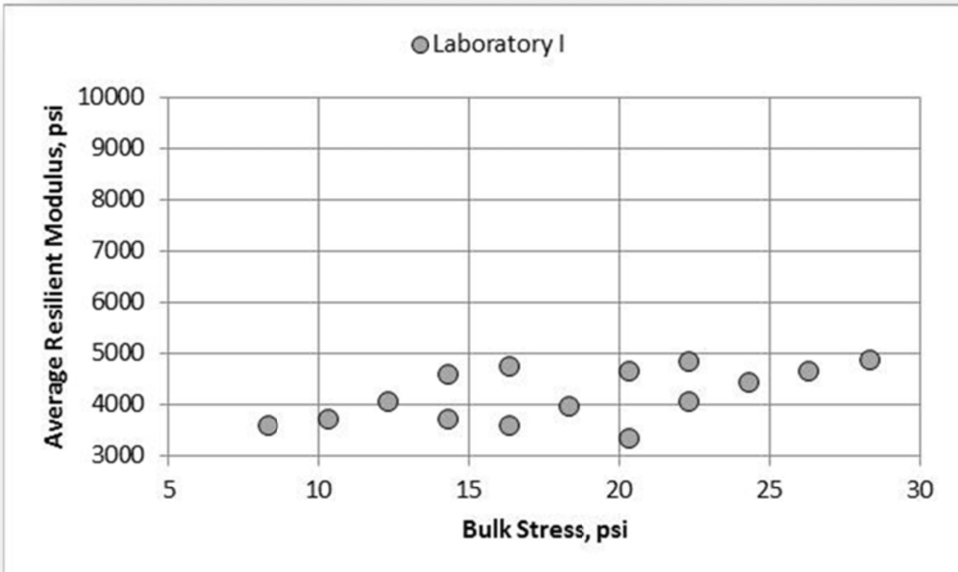

Figure 108. A-2-4 Soil (NCAT Subgrade), Laboratory I, Resilient Modulus

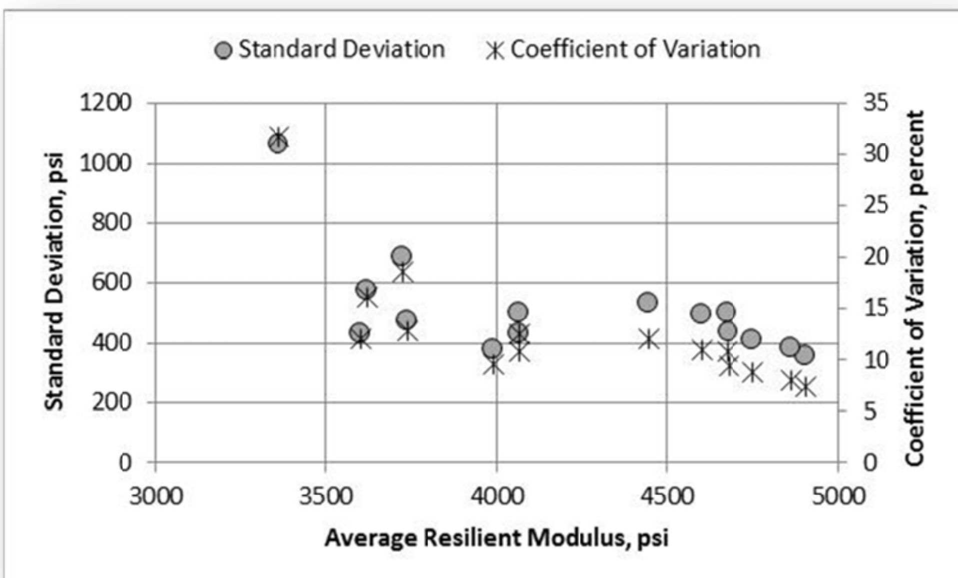

Figure 109. A-2-4 Soil (NCAT Subgrade), Laboratory I, Standard Deviation and Coefficient of Variation

D.3 A-2-4 Low Plasticity Soil with Varying Amounts of Silty Sand

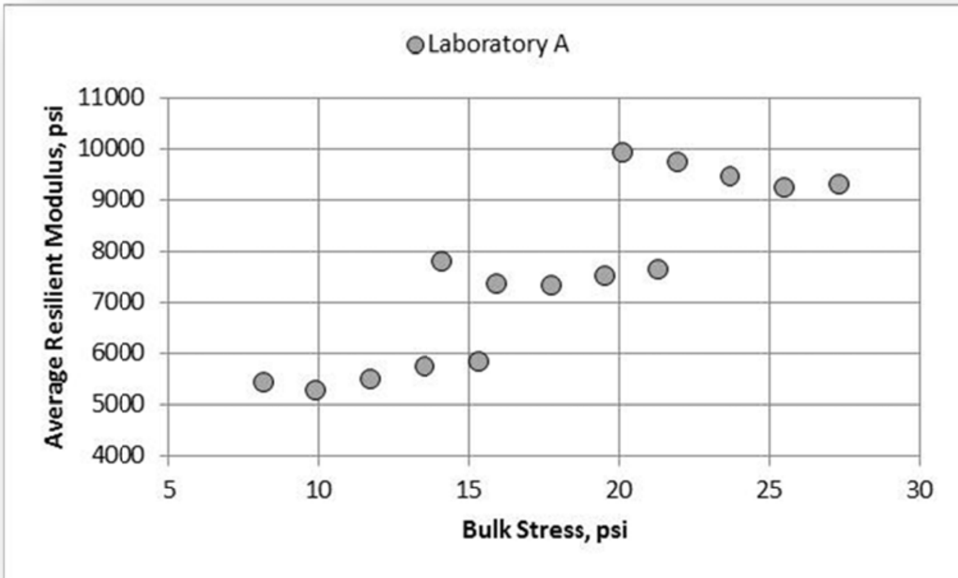

Figure 110. A-2-4 Soil (Low Plasticity), Laboratory A, Resilient Modulus

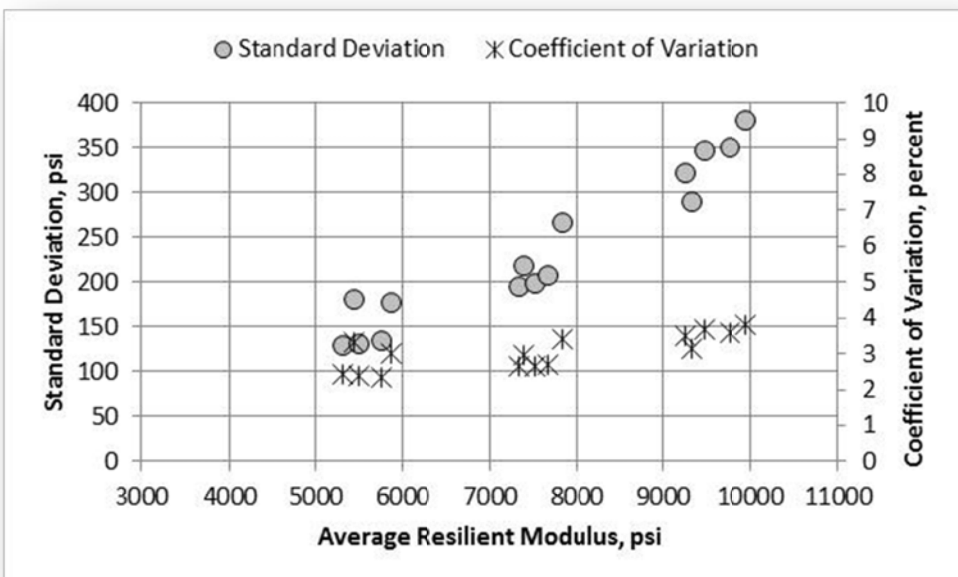

Figure 111. A-2-4 Soil (Low Plasticity), Laboratory A, Standard Deviation and Coefficient of Variation

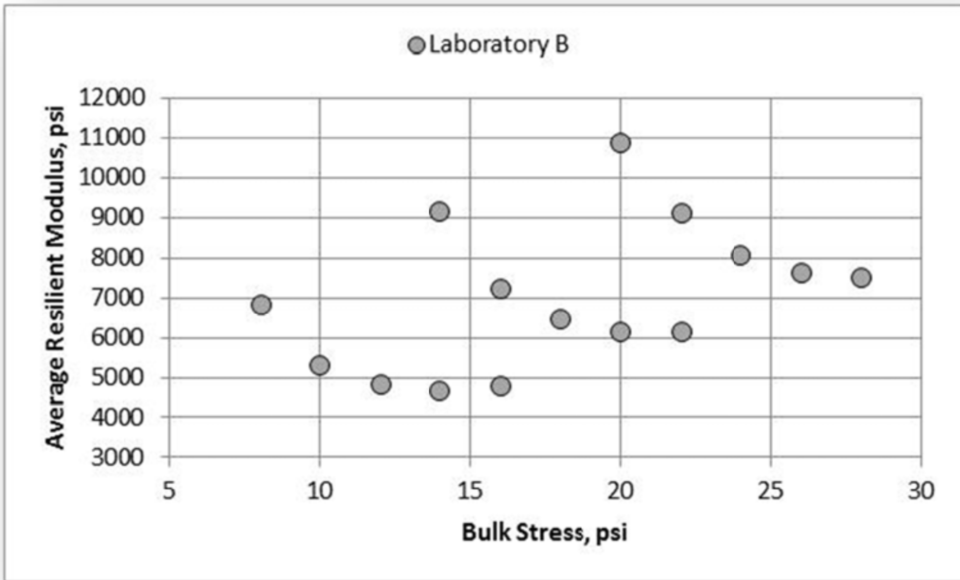

Figure 112. A-2-4 Soil (Low Plasticity), Laboratory B, Resilient Modulus

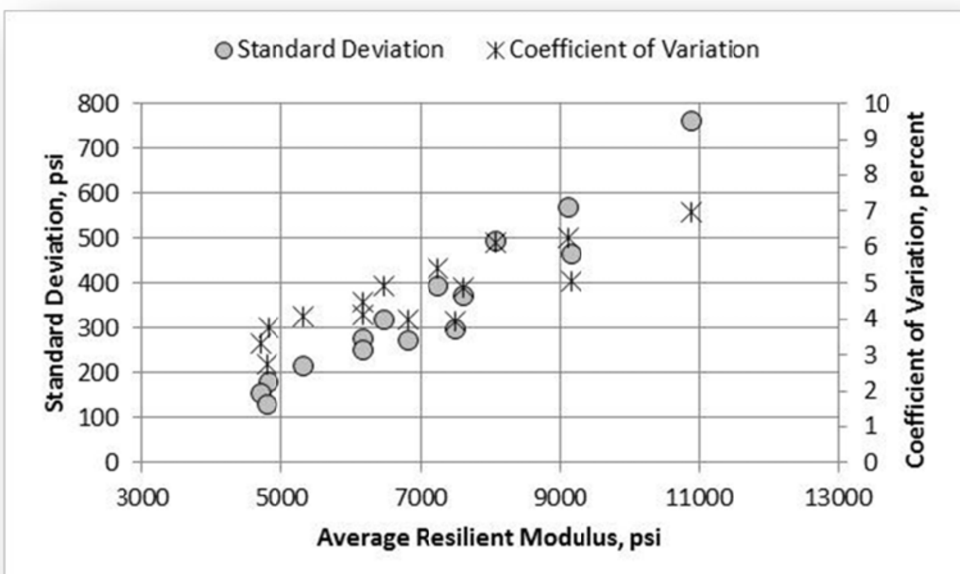

Figure 113. A-2-4 Soil (Low Plasticity), Laboratory B, Standard Deviation and Coefficient of Variation

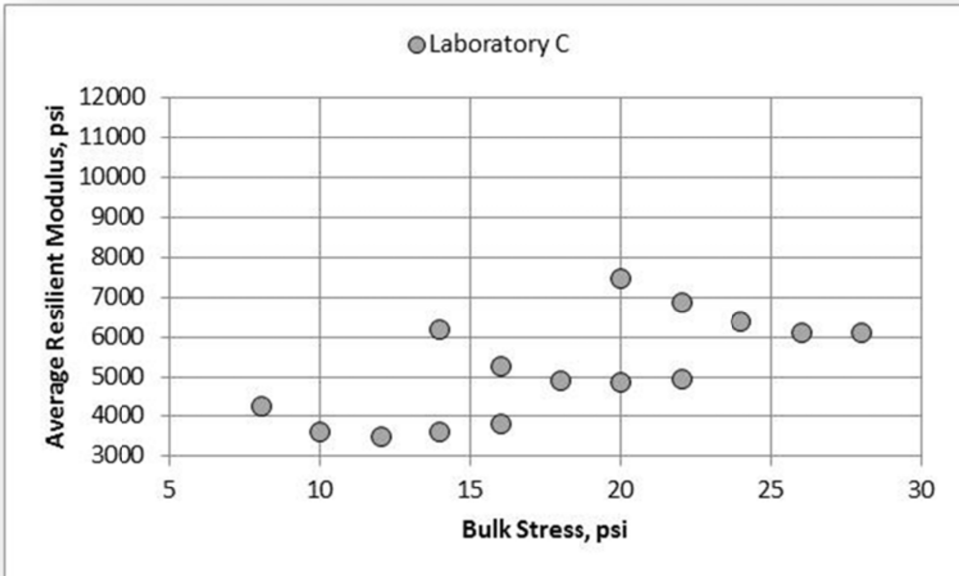

Figure 114. A-2-4 Soil (Low Plasticity), Laboratory C, Resilient Modulus

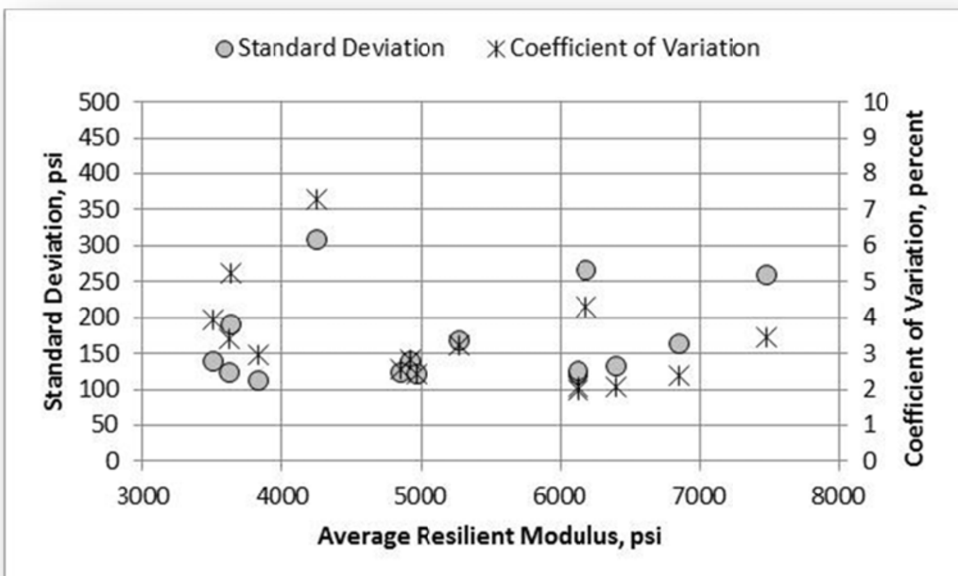

Figure 115. A-2-4 Soil (Low Plasticity), Laboratory C, Standard Deviation and Coefficient of Variation

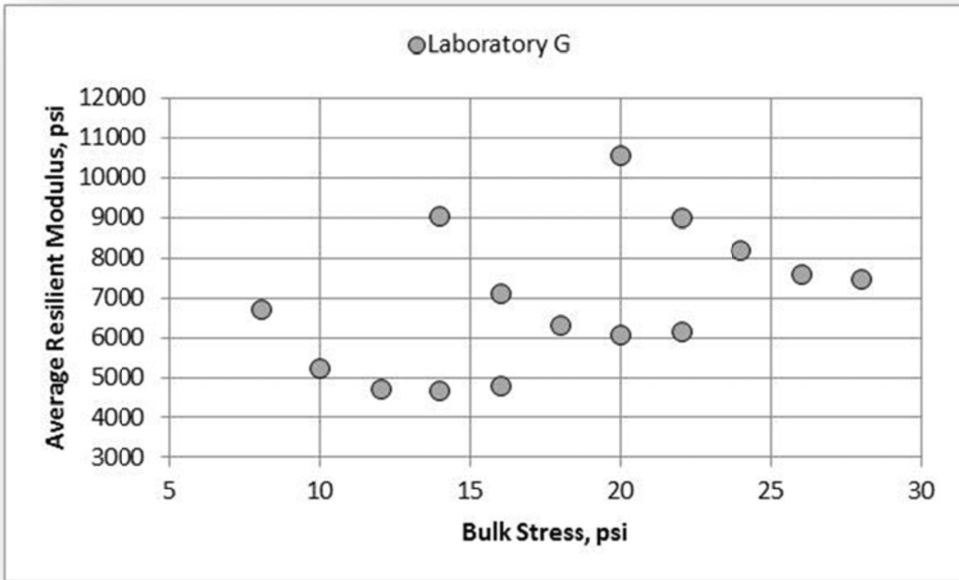

Figure 116. A-2-4 Soil (Low Plasticity), Laboratory G, Resilient Modulus

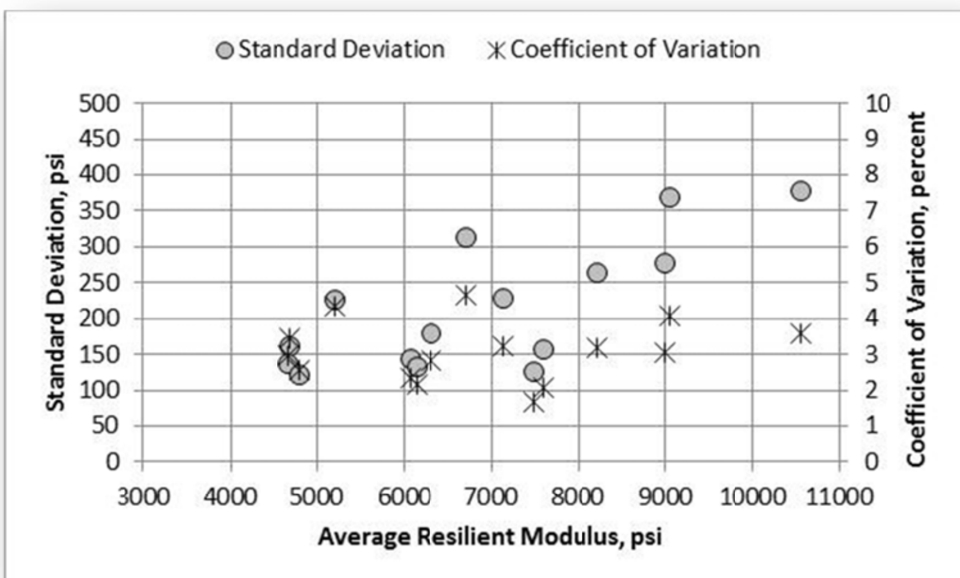

Figure 117. A-2-4 Soil (Low Plasticity), Laboratory G, Standard Deviation and Coefficient of Variation

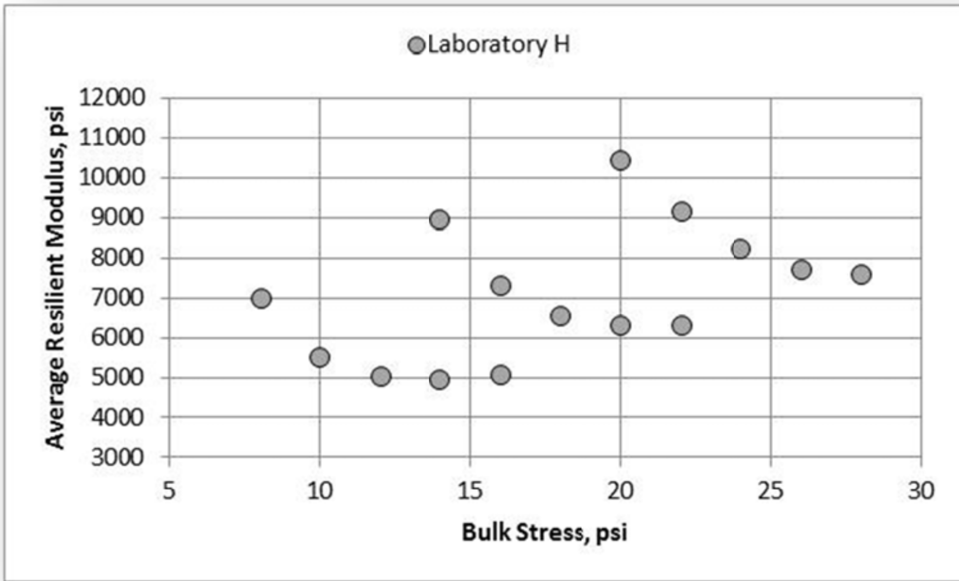

Figure 118. A-2-4 Soil (Low Plasticity), Laboratory H, Resilient Modulus

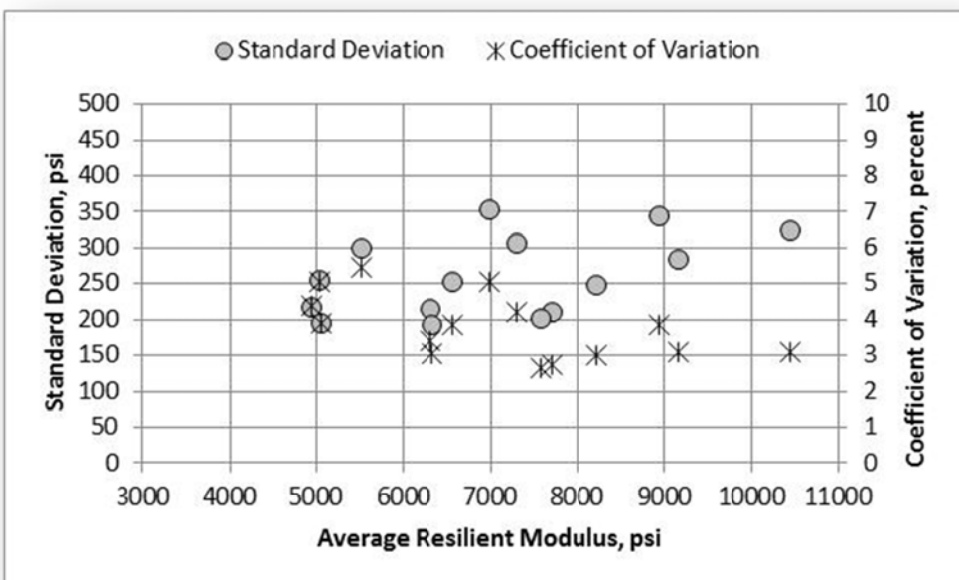

Figure 119. A-2-4 Soil (Low Plasticity), Laboratory H, Standard Deviation and Coefficient of Variation

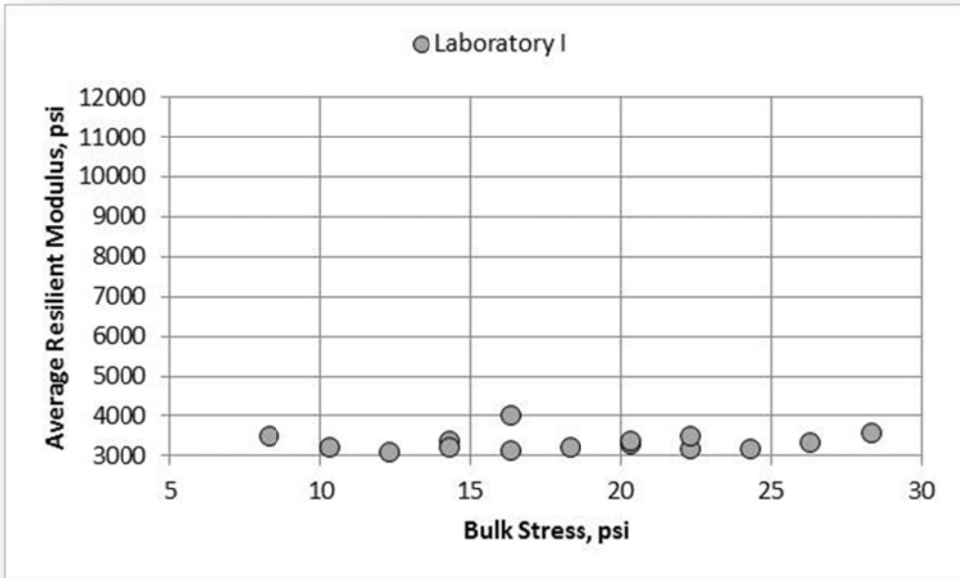

Figure 120. A-2-4 Soil (Low Plasticity), Laboratory I, Resilient Modulus

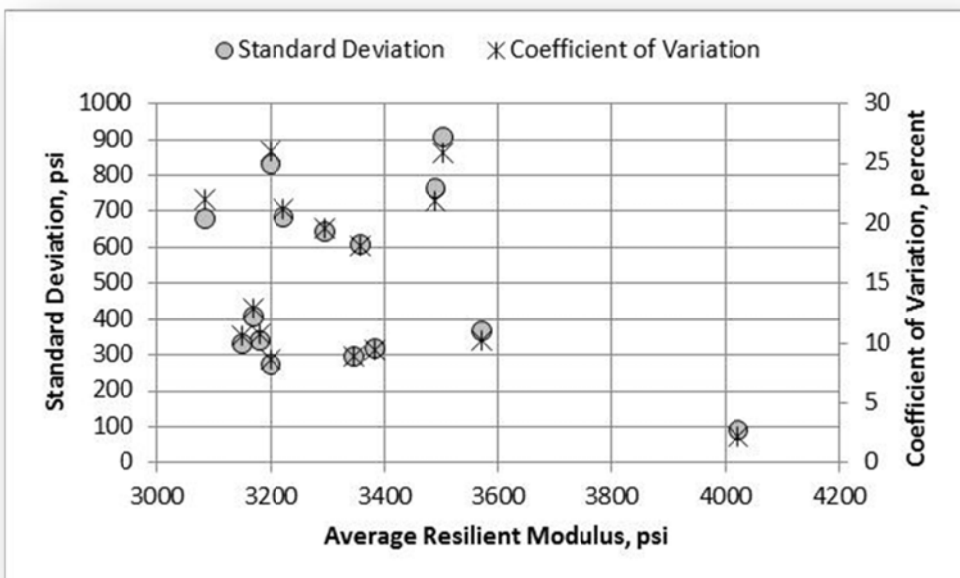

Figure 121. A-2-4 Soil (Low Plasticity), Laboratory I, Standard Deviation and Coefficient of Variation

D.4 Granular Aggregate Base: Crushed Stone

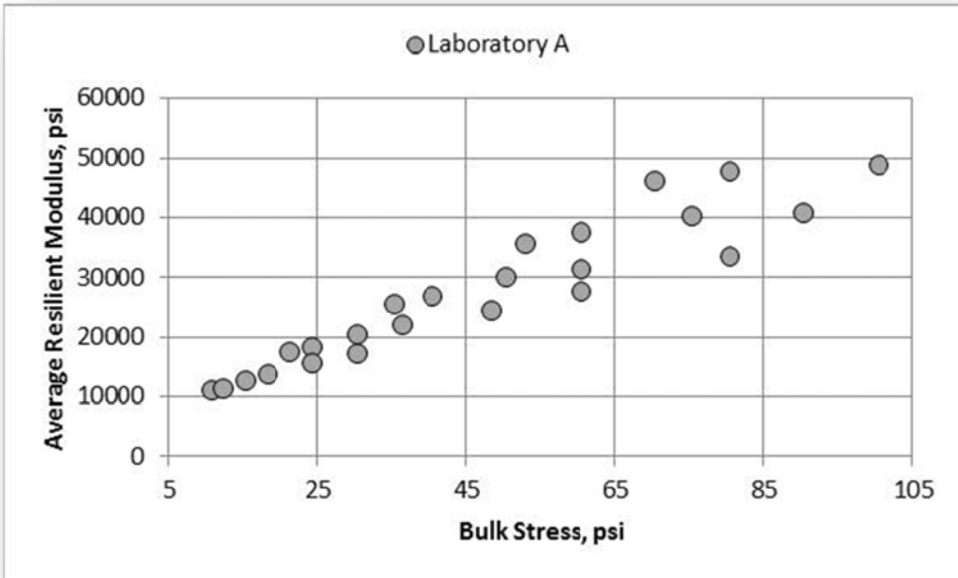

Figure 122. GAB Material, Laboratory A, Resilient Modulus

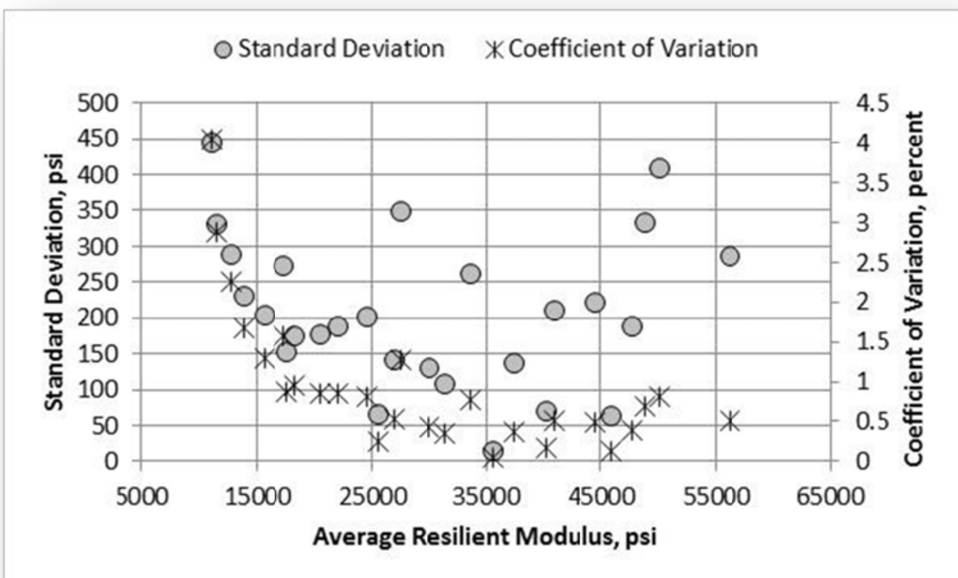

Figure 123. GAB Material, Laboratory A, Standard Deviation and Coefficient of Variation

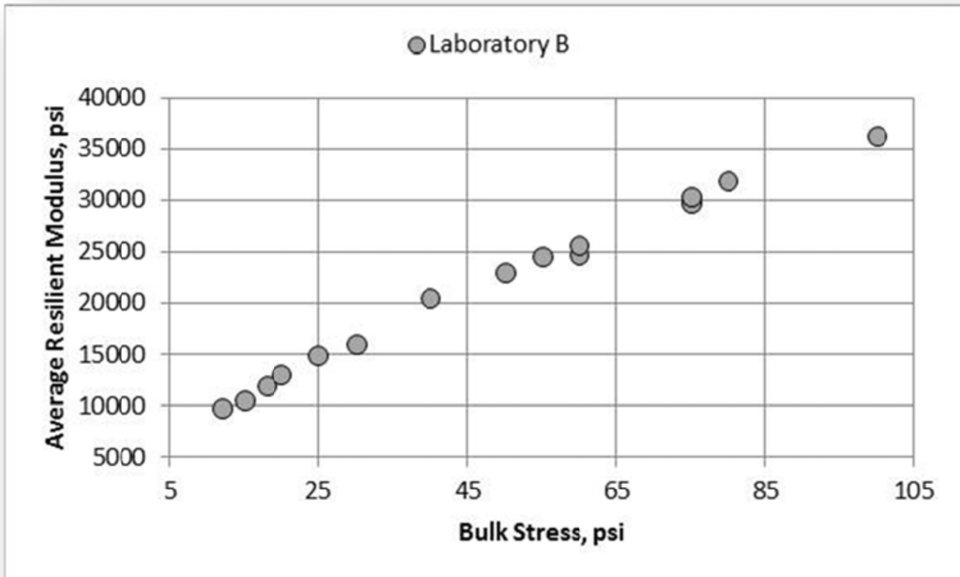

Figure 124. GAB Material, Laboratory B, Resilient Modulus

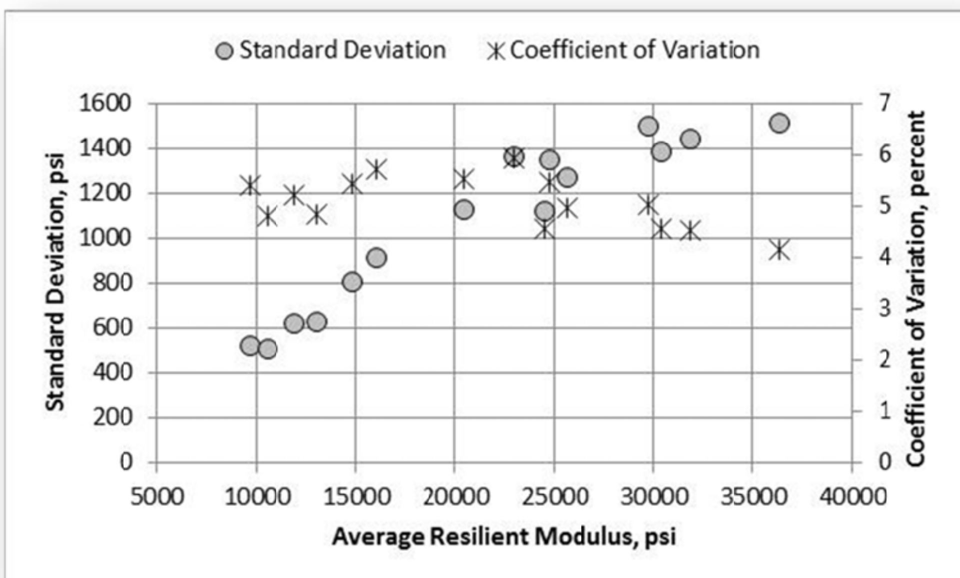

Figure 125. GAB Material, Laboratory B, Standard Deviation and Coefficient of Variation

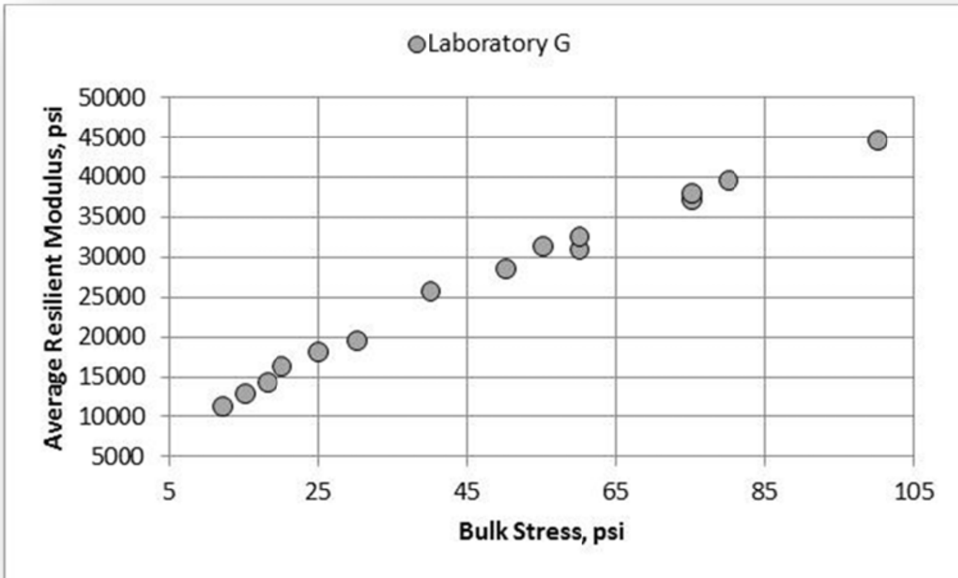

Figure 126. GAB Material, Laboratory G, Resilient Modulus

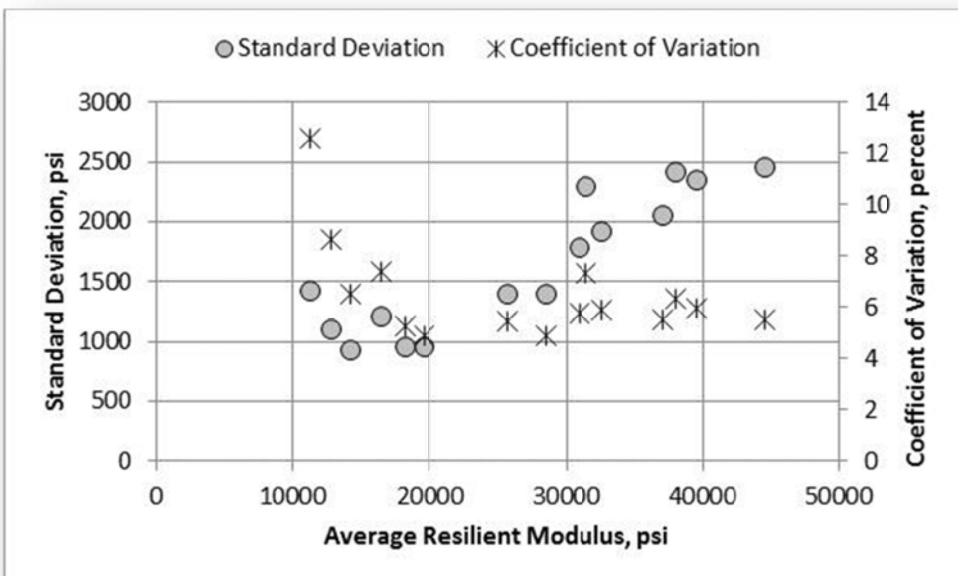

Figure 127. GAB Material, Laboratory G, Standard Deviation and Coefficient of Variation

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