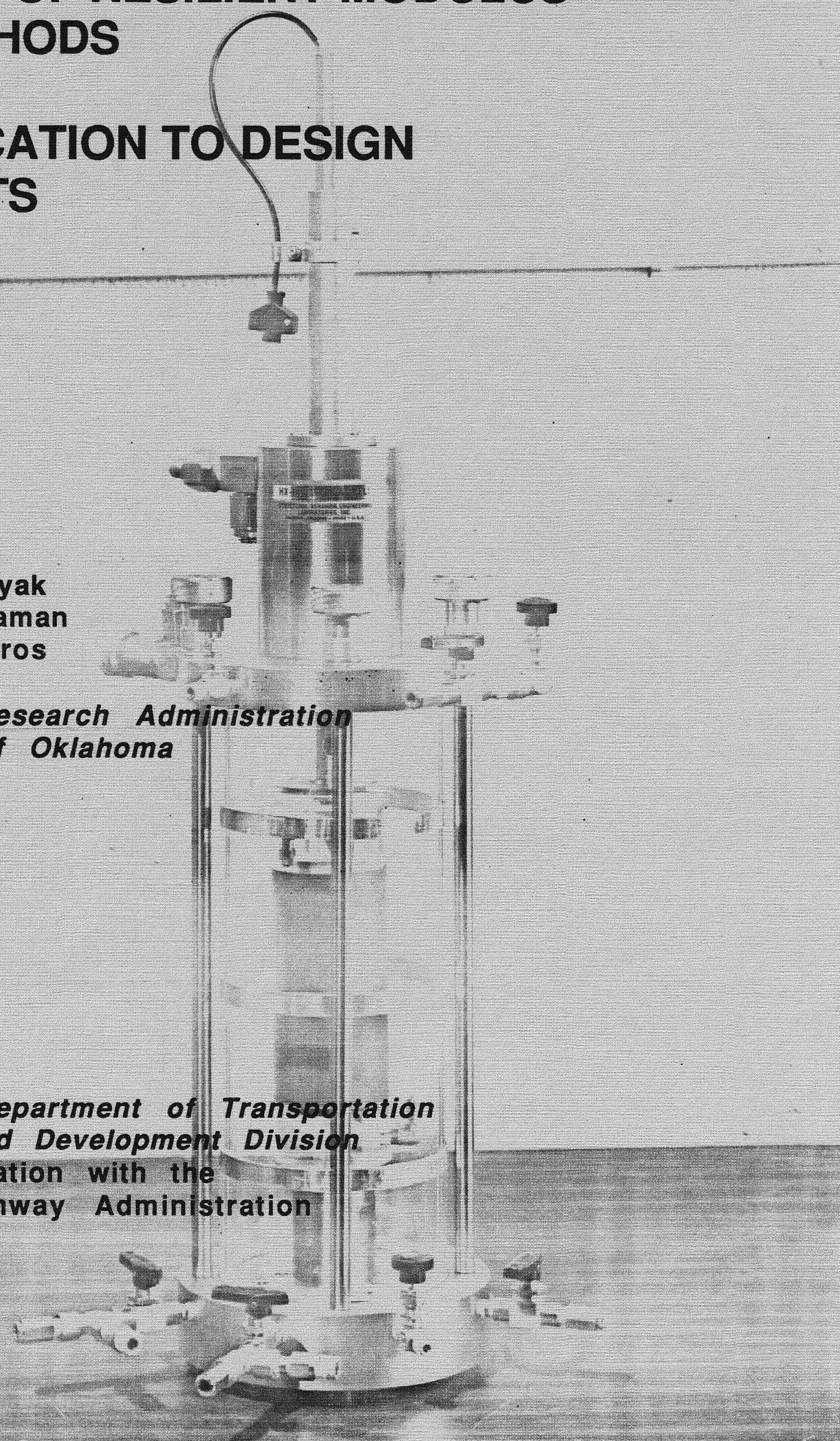
ASSESSMENT OF RESILIENT MODULUS TESTING METHODS AND THEIR APPLICATION TO DESIGN OF PAVEMENTS



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16. ABSTRACT

Resilient modulus (RM) is an important property of subgrade soils that accounts for repetitive loads due to vehicular traffic. Since AASHTO recommended its use in pavement design in 1986, various transportation agencies have devised procedures for testing and evaluation of RM. A comprehensive literature search was conducted in this study with two objectives in mind: (i) to obtain information on current practices pertaining to RM testing of subgrade soils; and (ii) to compile information pertaining to the collective experience of various agencies in correlating RM with other engineering soil properties.

Practices adopted by different transportation agencies in testing RM are not identical; some follow AASHTO guidelines, while others differ. The differences are centered around deviator stress, rate of loading, confining stress, moisture-density relationship, specimen preparation and stress sequence. The well known relationship between RM and CBR, proposed by AASHTO, does not correlate well for many soils. Efforts have been made by various researchers to correlate RM with other factors including clay, silt and organic carbon contents, plasticity index, liquid limit, group index, compressive strength, initial elastic modulus and confining pressure. Very limited efforts have been directed toward understanding the RM characteristics of bonded materials and aggregate bases.

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Pavement design, resilient modulus, subgrade soils, literature survey

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DISCLAIMER

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

SUMMARY

In cooperation with the Oklahoma Department of Transportation (ODOT) and Federal Highway Administration (FHWA), the School of Civil Engineering and Environmental Science (CEES) at the University of Oklahoma (OU) has embarked on a series of studies entitled "Technology Transfer." The purpose is to determine and present the state-of-the-art and the latest technological development in a specific research area for possible implementation.

The report presented herein is entitled "Assessment of Current Resilient Modulus and Their Application to Design of Pavements." To obtain the latest available information on this topic, an extensive literature survey was conducted through several computerized data bases including NTIS and HRIS. Also, the library resources of the local FHWA offices, and those of the University of Oklahoma were used.

The dual purposes of the study were: 1) to obtain information on current practices pertaining to resilient modulus testing of soils, and 2) to compile information pertaining to the collective experience of various transportation agencies in correlating resilient moduli with various soil parameters such as CBR, plasticity index, water content, liquid limit, compressive strength and clay content.

Resilient modulus of subgrade soils play an important role in pavement design. The AASHTO Guide for Design of Pavement Structures [1] recommends its use. A triaxial testing wherein

the load application is repetitive, is being used to determine resilient modulus according to AASHTO specifications [2] for both cohesive and cohesionless subgrade soils. Practices adopted by various transportation agencies for determining RM may follow the AASHTO guidelines or they may differ. The differences are centered around deviator stress, rate of loading, confining stress, moisture-density relationship, specimen preparation and stress sequence.

Various factors can influence the resilient modulus of subgrade soils. Some of the important factors include deviator stress, confining stress, rate of loading, moisture content, CBR value and method of compaction adopted for specimen preparation. Hicks [25], Thompson et al. [52,56], Thornton et al. [55,56] have reported that RM increases with increasing frequency of load application, increasing confining pressure and increasing density. On the other hand, reduction in RM is caused by increased moisture content. For cohesionless soil deviator stress has very little effect on RM value.

It was recognized that RM testing is difficult to conduct, keeping this view AASHTO [1] in 1986 suggested a correlation between RM and CBR values, for quick evaluation of RM. Brodsky [11] could not validate the relationship between RM and CBR, suggested by AASHTO. Similarly Thornton [55,56] could not establish and relationship between Rm and CBR for Arkansas soils. Other correlations have also been developed, which related RM with water content, plasticity index, confining pressure, deviator stress, percent clay traction. To this end, Thompson et al. [52,54] developed equation for Illinois soils that involved such factors as clay content, silt content, organic carbon content, plasticity index, liquid limit and group index. However, such relationship may not be applicable to other soils. Testing methods for determining RM have been suggested by various transportation agencies.

Chapter 1

INTRODUCTION

1.1 BACKGROUND

Subgrade soils play a major role in the design, construction and performance of roadway pavements. One of the important deficiencies of the current pavement design methods in evaluating the properties of subgrade soils is that they do not properly account for the repetitive nature of loading imposed by moving vehicular traffic. In an attempt to remedy this deficiency or at least partly overcome it, AASHTO [1] recommended in 1986 that the "Resilient Modulus" (RM), instead of the "Modulus of Subgrade Reaction" (K), be used to characterize the subgrade soil for pavement design purposes. Since then many studies [3,5,17,18,24-26,52-58] have been conducted and reported in the literature on various aspects of RM including testing, establishing the influence of various factors and correlations with other soil properties such as California Bearing Ratio (CBR) strength parameters, Atterberg limits, etc.

This report presents a critical reviews of the state-of-the-art of the resilient modulus testing methods and applications. An attempt is made to compile a list of the transportation agencies and their experience with resilient modulus testing and application and to identify the important and critical areas of research on RM that would benefit the State in terms of improving pavement design, performance, design life and economics.

1.2 OBJECTIVES

This study encompasses two main objectives: (i) A critical review of the current resilient modulus testing methods for subgrade soils, and (ii) Compilation of information pertaining to the collective experience of various transportation agencies and research centers, in correlating RM values with conventional soil properties such as CBR, moisture content, plasticity index and clay content. A cursory evaluation of the research on bonded materials, such as stabilized soils, is also attempted.

1.3 STUDY TASKS - ORGANIZATION OF THE REPORT

To accomplish the aforementioned objectives the following tasks were identified and pursued:

Task 1: Review of literature

Task 2: Survey of selected transportation agencies

Task 3: Analysis of information gathered

The information obtained and the findings thereof are organized and divided into the following items:

(1) Resilient modulus (RM) and its role in pavement design;

(2) Testing methods to determine RM;

(3) Influence of various factors on RM values;

(4) Summary of various studies correlating RM with conventional soil properties;

(5) Conclusions and recommendations.

Chapter 2

IMPORTANCE OF RESILIENT MODULUS IN PAVEMENT DESIGN

2.1 RESILIENT MODULUS (RM)

When subgrade soils are subjected to repeated loads due to moving vehicles, they undergo deformation or strain. Simulated laboratory tests [17,18,52-56] have shown that a part of this deformation is recoverable or elastic, while the other part is permanent or plastic.

Simply stated, RM represents a relationship between the applied stress (due to the moving vehicular traffic) and the elastic or resilient strain. In other words, the resilient modulus can be defined as the applied stress divided by the elastic strain. Thus, conceptually it is similar to the modulus of elasticity or Young's modulus (E), which is widely used in engineering to characterize the elastic behavior of a material. It should, however, be noted that there is a basic difference between E and RM. Determination of E is usually based on a simple monotonic loading of an unloading-reloading sequence and it does not account for the cyclic loading effects. The RM, on the other hand, is based on the concept of repeating/cyclic loading.

The resilient modulus is determined from various methods, the most common being the repeated load triaxial compression test. Typical response obtained from a repeated load test on a cylindrical specimen is shown in Fig. 2.1, in terms of deviator stress and axial strain. Deviator stress is defined as the

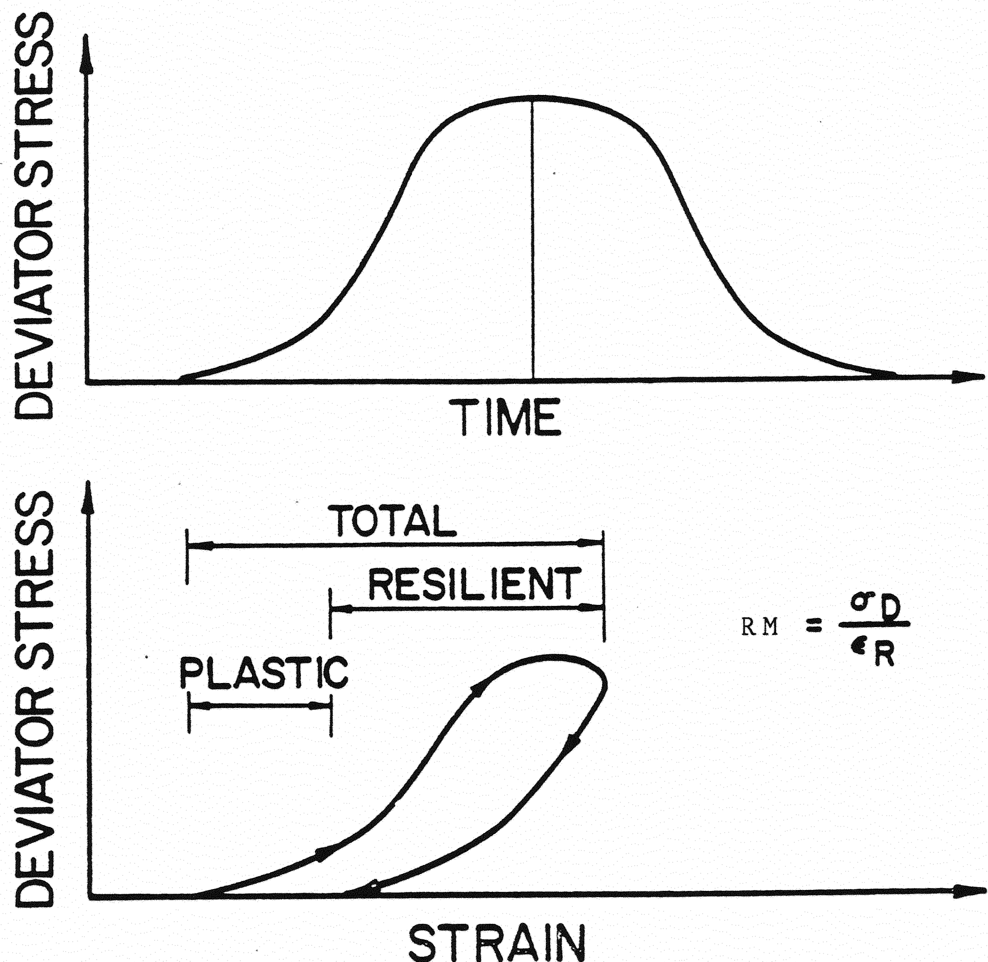


Figure 2.1: Typical Repeated Load Response (National Highway Institute) [52,53,55]

difference between the axial stress and the confining pressure. It is observed that with the application of load, the deviator stress increases and so does the axial strain. With the reduction in stress, strain is found to reduce. Upon complete withdrawal of stress, the total strain is not completely recovered. Total strain is expressed as a combination of the plastic (or permanent) and the resilient (or elastic) strain. The latter is important because the design of pavement is based on this strain [1,2,17,47,53,55,60].

2.2 IMPORTANCE OF RM IN PAVEMENT DESIGN

Use of resilient modulus (RM) in pavement design is important for various reasons as discussed below:

1. The 1986 AASHTO [1] recommendations to use RM recognizes the fact that vehicular loads acting on the pavement are repetitive in nature. This is a significant improvement compared to the commonly used assumptions in which traffic loads are treated as "static" and an impact factor is introduced to account for their dynamic nature. While the impact factor does modify the magnitude of the load(s), it does not change the subgrade soil properties. In conventional pavement design, subgrade soil properties are evaluated using monotonic loading. Thus, introduction of RM addresses one of the fundamental issues that vehicular loads are indeed cyclic.

2. Evaluation of RM is based on the recoverable or elastic strain and not the total strain. Since only elastic strains induce stresses in a material (i.e., inelastic or plastic strains do not induce any stress), it is logical to use elastic strain, instead of total strain, in determining soil modulus. Experimental observations have shown that the major component of deformation or strain induced by moving loads is elastic and not permanent [1,17-20,52-56]. However, improperly designed pavements may result in excessive plastic or permanent strains (e.g., rutting in asphalt concrete pavements).
3. Evaluation of RM accounts for loading frequency and rate that are similar to the conditions that exist in the field. Thus, the resulting elastic modulus (i.e., RM) is more representative of how the subgrade soil would behave under actual traffic loading. Further, sample conditioning, deviatoric stress magnitude and other details make RM testing more closer to actual loading of soil by moving vehicles [1].
4. Use of RM makes it easier to employ a mechanistic approach in analyzing cracking, rutting and faulting response of pavements.
5. It has been recognized internationally as a method for characterizing materials for use in pavement design and evaluation.
6. Techniques are available for estimating the RM properties of various materials in place using non-destructive tests.

Chapter 3

METHODS FOR DETERMINING RESILIENT MODULUS OF SUBGRADE SOILS

3.1 INTRODUCTION

Prior to World War II, the design of pavements was basically empirical, guided by experience, soil classification and pavement response due to static loads. The plate load and CBR tests were very widely used. With more experience and increase in traffic composition and volume, it was found that the results of static load tests are not reliable enough to evaluate the behavior of roadway pavement under moving vehicular traffic loading. As a remedial measure repeated load tests gradually replaced the static tests. These tests were helpful in developing procedures for determining RM of the subgrade soils in the laboratory.

Determining RM in the laboratory involves a complex procedure. No standard testing method has yet been developed to account for all the important factors which influence RM for a given soil. A significant amount of research is currently underway to standardize RM testing methods. A summary of the present state-of-the-art on RM testing is presented in this chapter. The following four basic repeated load test methods are commonly used to determine RM of a material:

(i) Direct tension [58]

(ii) Beam flexure (bending or rotating cantilever) [58,60]

(iii) Repeated load indirect diametral tension test [39,47,48,58]

(iv) Repeated triaxial compression test [2-6,9,17,21, 24,26,36-38,40-45,48,49,51,53-55,57,58,60-62].

Out of these four tests, diametral indirect tension and triaxial compression tests are simpler, more practical and economical for determining RM of pavement materials including certain types of subgrade soils. The indirect diametral tension test is generally used for bonded materials like asphalt concrete, stabilized base and shale, while the triaxial compression test appears to be more applicable to unbonded materials [18,19,38,39,49,55,59,60].

Various agencies have carried out repeated load triaxial tests to determine the RM of subgrade soils. The approach used in these tests was either the same as that suggested by the AASHTO T-274-82 or the tests somewhat differed in terms of sample conditioning, loading frequency, deviator stress magnitude, number of cycles and other details. A detailed review of the differences in testing procedures adopted by various agencies is included in Section 3.5. The transportation agencies (DOTs) which played a leading role in these efforts include Florida [26], New York [49], Illinois [17], Tennessee [18], Oregon [39]. Additionally, the Asphalt Institute [3], the University of Arkansas [37,55] and the U.S.D.A. Forest Service [13] have reported their findings on this important topic.

3.2 DIAMETRAL INDIRECT TENSION TEST

The RM of bonded stabilized materials, e.g., asphalt concrete, is usually determined using the diametral indirect tension test or repeated load triaxial tests. However, the latter is generally preferred because of its simplicity and cost effectiveness [37,40,52-54,58].

In an indirect diametral tension test, a light pulsating load is applied through a load cell across the vertical length of a cylindrical specimen, which causes a horizontal deformation across its diameter. These lateral deformations are measured by linear variable differential transformers (LVDTs). The applied load is of a particular duration and frequency. Duration and frequency of the applied cyclic load are controlled accurately using an electro-hydraulic servo controller. The magnitude of cyclic load (P) and resulting deformations are recorded. The resilient modulus (RM) is determined by the following equation [47,58]:

$$RM = \frac{P}{Ht}(\nu + 0.27) \qquad (3.1)$$

where t = thickness of specimen, H = total recoverable horizontal deflection, ν = Poisson's ratio.

Referring to Equation 3.2, ν can be expressed as

$$\nu = \frac{-3.5 - 0.27(V/H)}{.063 + (V/H)} \qquad (3.2)$$

in which V = total recoverable vertical deflection. Note that the Poisson's ratio (ν) is considered here as negative, which is consistent with its actual definition. However, a positive sign is generally used for ν for convenience.

3.2.1 Salient Features of the Test Proposed by ASTM D4123-82 [47,58]

The American Society of Testing and Materials (ASTM) has suggested a standard method for determining the RM of a bituminous mixture (ASTM D4123-82). This method is essentially based on the concept of the diametral indirect tension.

Some of the salient features of the indirect tension tests are listed below [47,58].

(a) Temperature: According to ASTM recommendations, the tests can be conducted at three different temperatures, namely 41, 77 and 104°F. The specimen should remain at the specified temperature during testing.

(b) Loading Frequencies: The test may be conducted at one or more loading frequencies; usually frequencies of 0.33, 0.5 and 1.0 Hz are recommended for each temperature.

(c) Specimen Size: Specimen should be at least 2 in. long, with a diameter of 4 in. for aggregates up to 1 in. maximum size. A length of at least 3 in. and a minimum diameter of 6 in. are required for aggregates up to 1.5 in. size.

(d) No. of load repetitions: The number of load repetitions required vary between 50 and 200.

(e) Load Duration: Recommended load duration is from 0.1 to 0.4 sec. Load duration of 0.1 sec. is generally adopted because it is representative of transient pavement loading due to moving vehicles.

(f) Magnitude of Load: The recommended loading range is based on the tensile strength of the material. Loading should be such that it will induce a stress of at least 10-50% of tensile strength. In absence of tensile strength data, a load range of 4-200 lb./in. of specimen length is recommended.

3.3 REPEATED TRIAXIAL COMPRESSION TEST PROPOSED BY AASHTO T-274-82

3.3.1 General

The RM of cohesionless base course materials or cohesive subgrade materials can be determined by using a repeated load triaxial compression test method. Various transportation organizations have adopted their own procedures for finding the RM of subgrade soils. The fundamental aspects of most of these methods are based on the procedure suggested by AASHTO (T-274-82) [2,23,26,42,49]. Sometimes sample preparation and conditioning stress sequence, load duration, and moisture content vary among different transportation organizations. These aspects are discussed in detail in Section 3.5 of this report. The important features of repeated triaxial compress test suggested by AASHTO (T-274-82) are discussed below. It may be noted that AASHTO is currently reviewing revised procedure for RM testing.

3.3.2 Salient Features of the Test Suggested by AASHTO T-274-82 [2,18,38,42,49,54,55]

1. Scope and Summary of the Test Procedure

Procedure for preparing and testing untreated subgrade soils for determining RM has been discussed in detail by AASHTO [2]. This method is applicable to undisturbed natural soils and compacted subgrade as well as disturbed samples prepared by compaction in the laboratory.

In this test a repeated axial deviator stress of fixed magnitude, duration and frequency is applied to a carefully prepared and conditioned cylindrical specimen. During the deviator stress application, the specimen is subjected to a static confining pressure by means of triaxial pressure chamber. The axial strain induced by repeated loads is measured and used to determine the RM.

2. <u>Apparatus</u>

The equipment is similar to most triaxial testing equipment, except that it has a larger pressure chamber to facilitate the placement of internally mounted load and deformation measuring system. Deformations are measured with the help of linear variable differential transformers (LVDTs).

3. <u>Specimen Size</u>

Specimen length should not be less than two times its diameter, the minimum specimen diameter being the larger of 2.8 inches or six times the largest particle size of soil.

4. <u>Compaction Methods</u>

(a) <u>Cohesive soils</u>: Soils which exhibit sufficient cohesion to permit handling or specimen may be compacted by kneading or static loading method.

(b) Granular Soil: Cohesionless granular soils are usually compacted by using a split mold and vibrating method.

5. Moisture-Density Relationship

(a) Cohesive Soils: The moisture-density relationship for cohesive subgrade soils can be determined as follows:

Specimens are compacted to the field moisture content and dry density. Two criterias are satisfied: (i) the cohesive subgrades compacted in the field at a water content corresponding to less than 80 percent saturation, and (ii) thereafter it maintains the water content close to that used during construction. But, if there is a possibility of subsequent increase of in-service moisture content, then specimens are compacted at in-service moisture content. If the subgrade is compacted at a water content greater than the 80% saturation in the field, then test specimens are prepared using a water content larger than that used in the field.

In the absence of information regarding service condition, the moisture-density relationship is established according to the procedure suggested by AASHTO T-99.

(b) Granular Soil: If the field moisture content and densities are known, the laboratory test specimen can be compacted according to the in-service water content and density. If adequate field information is not available, the moisture-density relationship can be found out by using the procedure outlined in AASHTO T-99.

6. <u>Testing of Specimen</u>

The laboratory testing is usually designed to simulate the behavior of subgrade soils supporting a pavement system that is subjected to moving vehicular loading. Sample preparation, conditioning, stress sequence and magnitude are important factors for testing.

The sample conditioning and stress sequence for testing are different for cohesive soils and granular soils. Sample conditioning is done to eliminate the effects of the interval between compaction and loading and also to eliminate the effects of interval between initial loading and reloading.

(a) <u>Cohesive Soil</u>: The stress sequence for sample conditioning and testing of an undisturbed and compacted specimen of cohesive subgrade soils are summarized in Table 3.1. These values are recommended by AASHTO T-274-82. The stresses are applied in the same sequence as given in Table 3.1. Although these values are suggested by AASHTO, many transportation agencies have adopted their own standards to meet their needs. These modifications are discussed in Section 3.5.

b. <u>Granular Soil</u>: For granular soils, sample conditioning and testing stress sequences are shown in Table 3.2. This procedure is used for both saturated and unsaturated specimens of cohesionless soil. These values are suggested by AASHTO [2], however, there are some transportation agencies which have adopted their own standard for loading, as discussed in Section 3.5.

Table 3.1 Conditioning and Testing Stress Sequence for Cohesive Soil as per AASHTO T-274-82 [2,26,49]

Serial No.	Confining Stress (psi)	Deviator Stress (psi)	Remarks
1	6	1, 3, 5, 7.5, 10 (200 repetitions)	Stress Sequence for sample conditioning
2	6	Decrease to 1 (200 reps.)	Record recovered deformation
3	3	1 (200 rep.)	Yes
4	0	1 (200 rep.)	Yes
5	6	2 (200 rep.)	Yes
6	3	2 (200 rep.)	Yes
7	0	2 (200 rep.)	Yes
8	6	4 (200 rep.)	Yes
9	3	4 (200 rep.)	Yes
10	0	4 (200 rep.)	Yes
11	6	8 (200 rep.)	Yes
12	3	8 (200 rep.)	Yes
13	0	8 (200 rep.)	Yes
14	6	10 (200 rep.)	Yes
15	3	10 (200 rep.)	Yes
16	0	10 (200 rep.)	Yes

Table 3.2 Conditioning and Testing Stress Sequence for Granular Soils (AASHTO T-274-82) [2,26,49]

Serial No.	Confining Pr. (psi)	Deviator Stress (psi)	Remarks
1	10*	10 (200 repetitions)*	For saturated specimen, the drainage valves at the base of specimen to the back pressure reservoir is opened
2	10	10 (200 rep.)	
3	20, 10, 5, 3, 1	20 (200 rep.)	Done at each confining pr. in the given order
4	15	1 (200 rep.)	Record vertical deformation
5	15	2 (200 rep.)	Yes
6	15	5 (200 rep.)	Yes
7	15	10 (200 rep.)	Yes
8	15	20 (200 rep.)	Yes
9	10	1, 2, 5, 10, 20 (200 rep.)	Yes
10	1	1, 2, 5, 10 (200 rep.)	Yes

* Some low density granular specimens may fail in the process of cyclic loading. Appropriate confining pressure and deviator stress levels would have to be selected for such cases.

3.4 COMPARISON OF DIAMETRAL AND TRIAXIAL REPEATED LOAD TRIAXIAL TEST

3.4.1. Introduction

A comparison of the test results obtained from diametral and triaxial repeated load tests is presented in this section. This comparison is based on a study carried out by the Oregon Department of Transportation [38]. The tests were conducted on subgrade soils obtained from two sites as described below:

- Wilamette Valley, Salem Parkway. The soil types at this site were primarily clayey and silty sand, which were classified as A-7-6 and A-4, respectively, according to AASHTO classification.
- Central Oregon, US-97. Soil type at this site was volcanic pumice material that was classified as A-1-6.

The basic properties, (e.g., liquid limit, plasticity index, maximum dry density, optimum water content, etc.) of these soils were obtained from standard tests. The test designations as well as the properties obtained are given in Table 3.3 and 3.4. The important features of the testing program are briefly discussed.

3.4.2. Repeated Load Triaxial Test

Important features of the repeated load triaxial tests carried out by Oregon D.O.T. are as given below [2,38].

(a) Sample preparation: The samples were prepared at water contents above and below optimum moisture content and at a maximum dry density obtained from the AASHTO Compaction Test

Table 3.3 Material Properties, Standard Indicator Tests by Oregon D.O.T. [38]

% Passing

Particle Size	Salem - Parkway		US - 97
	Subgrade* 1 (A-7-6)	Subgrade* 2 (A-4)	Subgrade (A-1-b)
38.1 mm (1-½")			100.0
25.4 mm (1")			99.8
19.0 mm (3/4")			98.2
12.7 mm (1/2")			95.8
9.5 mm (3/8")			91.1
6.4 mm (1/4")			87.6
4.75 mm (1/4")	100	100	87.6
2.00 mm (No. 10)	99.9	99.9	66.1
0.425 mm (No. 40)	98.9	99.7	32.1
0.175 mm (No. 60)	96.2	99.5	26.4
0.074 mm (No. 200)	73.1	33.1	17.3
Liquid Limit, % (AASHTO T-89)	48	23	NP
Plasticity Index, % (AASHTO T-90)	20	NP	NP
AASHTO Soil Classification	A-7-6	A-4	A-1-b
Maximum Density (pcf) (AASHTO T-99)	90.45	107	45**
Optimum Water Content, % (AASHTO T-99)	25	18	60**

1 KN/m^3 = 6.369 pcf

* subgrade:
1 = clayey soil (AASHTO classification A-7-6)
2 = silty soil (AASHTO classification A-4)

** used for testing

Table 3.4 In-Place Material Properties Oregon D.O.T. [38]

Location and Material	IN PLACE			
	Water Content, (%) Subgrade*		Density, pcf Subgrade*	
	1	2	1	2
Salem - New Parkway - Subgrade	23.5	14.2	93.1	103.9
US - 97 - Subgrade	76.1		+	

+ No in-place tests were conducted.

* subgrade:
1 - Clayey soil (AASHTO classification A-7-6)
2 - Silty soil (AASHTO classification A-4)

Table 3.5 Stress Level Sequence and Stress Ratios used for Repeated Load Testing of Untreated Soils [38]

Confining Pressure (psi)	2	4	6	8
Stress Ratio	Deviator Stress (psi)			
1.5	1.0	2.0	3.0	4.0
2.0	2.0	4.0	6.0	8.0
2.5	3.0	6.0	9.0	12.0
3.0	4.0	8.0	12.0	16.0
3.5	5.0	10.0	15.0	20.0

(T-99) and at 100% and 95% of maximum dry densities, such that they simulate the field conditions approximately.

(b) Specimen size: The specimen used for repeated triaxial test was 4 in. (10.4 cm.) in diameter and 10 in. (25.4 cm.) in height.

(c) Load: Load duration of 0.1 sec. at a rate of 30 repetitions per minute was chosen.

(d) Sample conditioning: The specimens were conditioned by applying 200 repetitions at a maximum confining pressure and minimum deviator stress, then increasing the deviator stress every 200 repetitions keeping the confining pressure constant, until 1,000 repetitions and maximum deviator stress were achieved.

(e) Stress sequence for test data: RM for the subgrade materials was determined over a range of stress. The stress level sequence and stress ratio used in the tests are given in Table 3.5 [38].

3.4.3. Repeated Load Diametral Test

The equipment and procedure used for the repeated load diametral test are similar to those described in ASTM D-4123-82 [4] for bituminous mixtures. The important features of this test are summarized below [38,47,58].

(a) Sample preparation: Samples were prepared in the same way as were done for the repeated load triaxial test.

(b) Specimen size: Specimens were 10.4 cm. (4 in.) in diameter and 6.4 cm. (2.5 in.) in height.

(c) Specimen conditioning: Samples were conditioned by the same stresses as for the repeated triaxial tests.

(d) Stress sequence of testing: Testing stress sequence was same as given in Table 3.5.

3.4.4. Comparison of the RM Values Obtained from Different Tests

The plots of RM values and deviator stress for Subgrade 1 (A-7-6) and Subgrade 2 (A-4) are shown in Figs. 3.1 - 3.3. For Subgrade 3 (A-1-6) the plots are shown in Figs. 3.4 - 3.6 in terms of RM and sum of principal stresses. The notable features are as follows:

(a) Subgrade 1 (A-7-6) and Subgrade 2 (A-4)

It is clearly observed from Figs. 3.1 - 3.3 that triaxial RM increased with an increase in confining pressure and decreased with increasing deviator stress. The RM was minimum when the deviator stress was maximum.

RM values obtained from diametral tests increased with increasing confining pressure and increased slightly with increasing deviator stress, however, this trend was not consistent.

(b) Subgrade 3 (A-1-6 for US-97 Project)

As shown in Figs. 3.4 - 3.6, the triaxial and diametral RM increase with increase in the sum of principal stresses. It can also be seen that at low level of stress, the diametral RM is higher than the triaxial RM, but at higher stress level, no particular trend was detected.

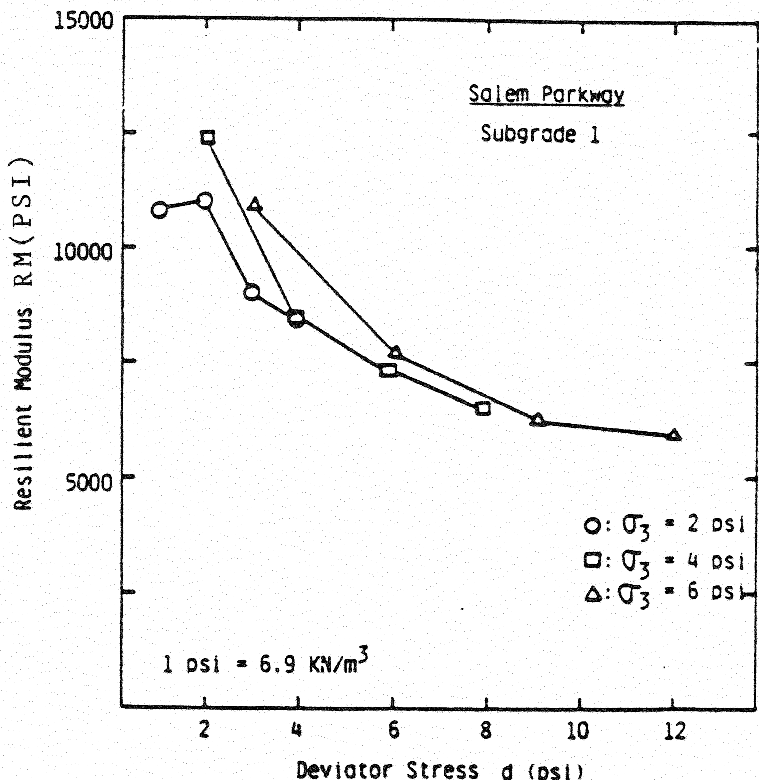


Figure 3.1: Triaxial Resilient Modulus vs. Deviator Stress
Salem Parkway Project
Subgrade 1, 95% Compaction
25% Water Content [38]

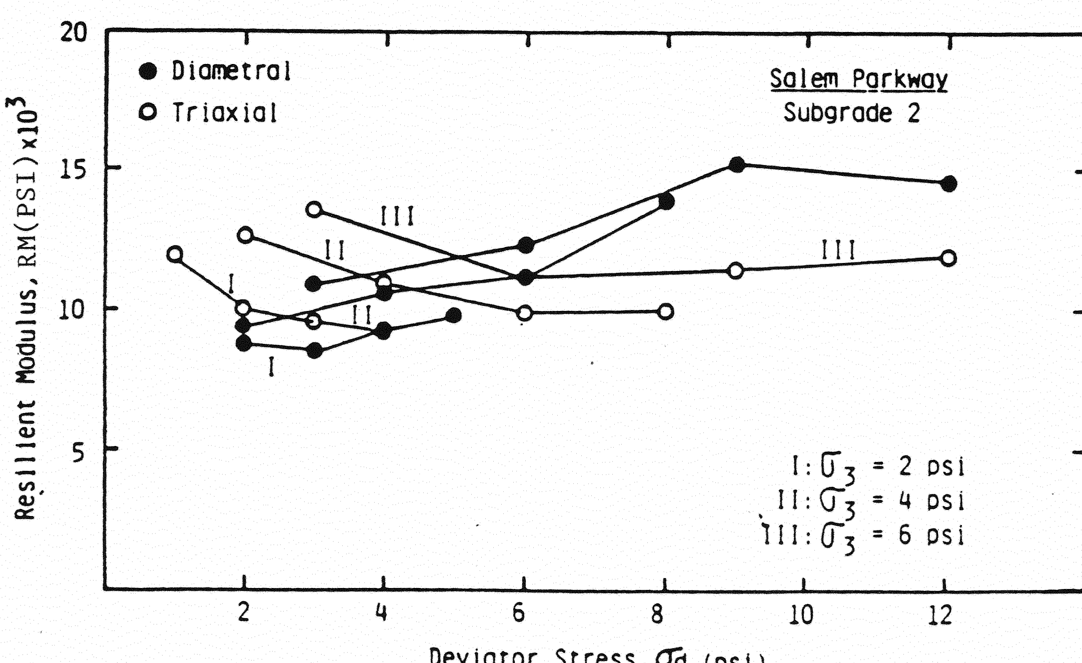


Figure 3.2: Comparison of Triaxial and
Diametral Resilient Modulus Results
Salem Parkway Project
Subgrade 2, 95% Compaction
14% Water Content [38]

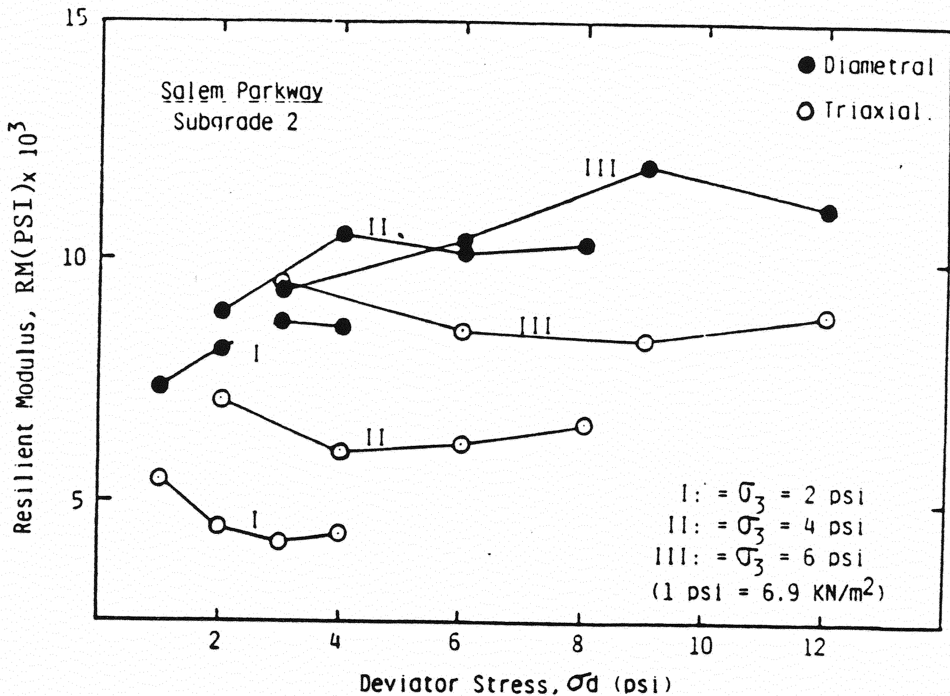


Figure 3.3: Comparison of Triaxial and Diametral Resilient Modulus Results Salem Parkway Project Subgrade 2, 95% Compaction 40% Water Content [38]

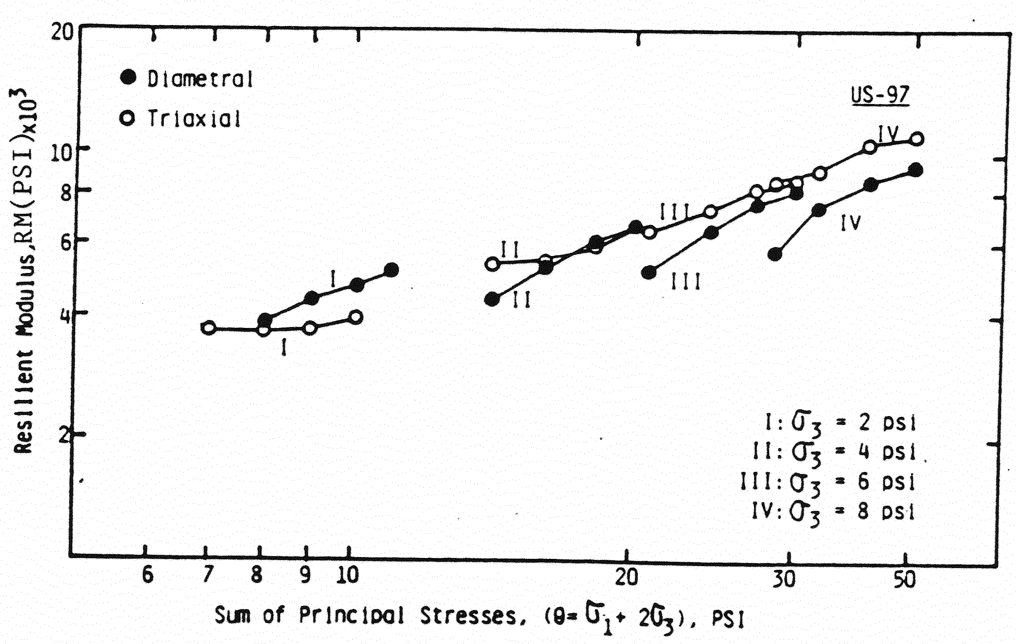


Figure 3.4: Comparison of Triaxial and Diametral Resilient Modulus Results US-97 Project Subgrade Soil, 95% Compaction 40% Water Content [38]

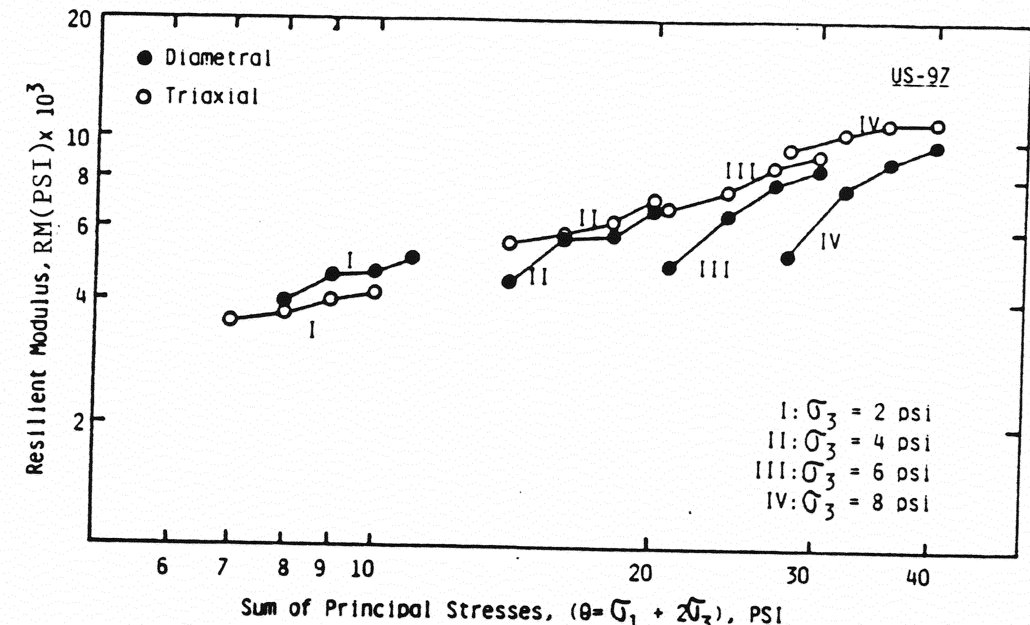


Figure 3.5 Comparison of Triaxial and
Diametral Resilient Modulus Results
US-97 Project
Subgrade soil, 95% Compaction
60% Water Content [38]

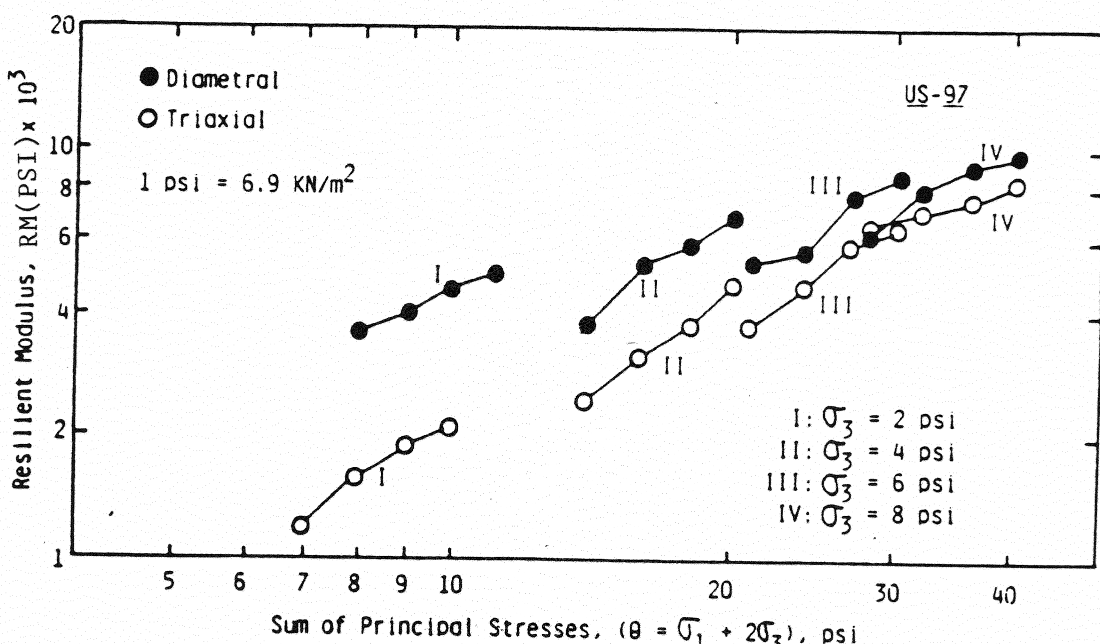


Figure 3.6 Comparison of Triaxial and
Diametral Resilient Modulus Results
US-97 Project
Subgrade Soil, 95% Compaction
80% Water Content [38]

(c) For all subgrade soils, the RM increased with an increase in the level of compaction, but decreased with an increase in the water content.

(d) Values obtained for RM and Poisson's ratio in the two tests exhibited large difference due to nonlinear and heterogeneous behavior of soil.

(e) Triaxial repeated load test is relatively straightforward to conduct, it is more versatile and is commonly used compared to diametral test. It gives repeatable results if conducted carefully. The repeated load diametral test is well established for treated materials, but for untreated materials, particularly cohesionless soils, the results obtained tend to be variable. Also, it requires high skill and knowledge of equipment being used.

3.4.5. Conclusions

RM values obtained from both tests are different and inconsistent, however, results obtained from diametral resilient tests were more variable than those from repeated triaxial tests. From these results, it can be inferred that the relationship between modulii obtained using both devices is non-unique and is dependent upon the testing equipment and testing procedures. Also, these relationships may not be applicable to other projects, due to a wide variation in soil type.

3.5 WORK DONE BY VARIOUS AGENCIES

Various agencies have carried out repeated load tests to determine the RM of subgrade soils. The test procedures adopted by them were either similar to AASHTO T-274-82 [2] (Section 3.2) or ASTM [4] (Section 3.3), except for some factors pertaining to sample conditioning, load applications, stress sequence, moisture condition. Presented in this part is work done by various transportation agencies, with an emphasis on the difference in equipment, compaction methods and other parameters.

3.5.1. Florida D.O.T. [21]

The Florida D.O.T. carried out repeated triaxial tests on two granular subgrades (20849-S,20891-S) and one base material (21077-S). These samples were tested using three methods, namely, AASHTO T-274-82 [2], draft ASTM method and a modified method in which conditioning stresses were applied statically.

Salient features of these tests are discussed below:

1. Equipment: An MTS electro-hydraulic closed loop test system was used to conduct all tests.
2. Types of Soils Tested: Two different soil types were used in this study, stabilized subgrade soil (20849-S,20891-S) and limerock (21077-S) wherein the stabilized subgrade consisted of A-2-4 or A-3 silty sand or fine sand mixed with shell or limerock. The basic properties of these soils are presented in Table 3.6.
3. Sample Preparation: Sample size of 4-inch diameter and 8-inch height were prepared by compacting the soil in a 4-inch diameter and 8-inch high mold. Compaction was done at about 1% dry of optimum moisture content.

Table 3.6 Properties of Soil as per Tests Conducted by Florida D.O.T. [26]

Properties	SOIL TYPE		
	Dark Grey Sand with Limerock 20849-5	Grey Sand with Shell 20891-S	Limerock 21077-S
LBR*	40	45	148
Max. Density	112.5 pcf	116.0 pcf	116.8 pcf
Optimum Moisture Content	10.5%	10.5%	12.3%

* LBR (Limerock Bearing Ratio) = 1.25 CBR

Table 3.7 Sample Conditioning Stresses for AASHTO T-274-82 Tests Conducted by Florida D.O.T. [26]

Confining (psi)	Deviator Stress (psi)	No. of Repetitions
5	5	200
5	10	200
10	10	200
10	15	200
15	15	200
15	20	200

4. <u>Deformation Measurements</u>: Two internal LVDTs were mounted on clamps on middle half of the specimen. Two external LVDTs were also mounted on the piston to record deformations of 8-inch long sample.

5. <u>Testing Methods Adopted</u>: Three testing methods, namely AASHTO T-274-82, ASTM draft and a modified method, were adopted. All these methods are given below:

   a) <u>AASHTO T-274-82</u>: In this method different stress conditioning levels were applied with 200 repetitions for each stress condition. The testing and sampling conditioning stress sequence for cohesionless and cohesive soils were the same as discussed in Section 3.3.2 and are summarized in Table 3.7.

   b) <u>The Draft ASTM Method</u>: In this method only one conditioning stress was applied with 1,000 repetitions for both cohesive and cohesionless soils. The sample conditioning and testing stress sequence was shown in Table 3.8.

   c) <u>Modified Method</u>: In this method the sample was subjected to static conditioning stresses, that are equal to the testing stresses for 3 minute cycles, before repetitive loads up 10,000 cycles were applied. The sample conditioning and testing stress sequence are summarized in Table 3.9.

6. <u>Test Results</u>: The results for subgrade 20849-S soil presented in Fig. 3.7 - 3.9 in the form of log-log plot of RM and sum of principal stresses. The second subgrade

Table 3.8 Conditioning and Testing Stress Sequence as per ASTM Method (Draft) [26]

Test Conditions	Confining Stress (psi)	Deviator Stress (psi)	Number of Repetitions
Sample Conditioning	6	1	1000
Testing	6	1,2,5,10	200 Each
Testing	3	1,2,5,10	200 Each
Testing	1	1,2,5,10	200 Each

Note: Stress sequence are same for both cohesionless and cohesive soils.

Table 3.9 Conditioning and Testing Stress Sequence for the Modified Triaxial Method [26]

Confining Pressure (σ_3) (psi)	Deviator Stress (σ_d) (psi)	No. of Repetitions
1	2	10,000
2	2	10,000
2	4	10,000
5	2	10,000
5	5	10,000

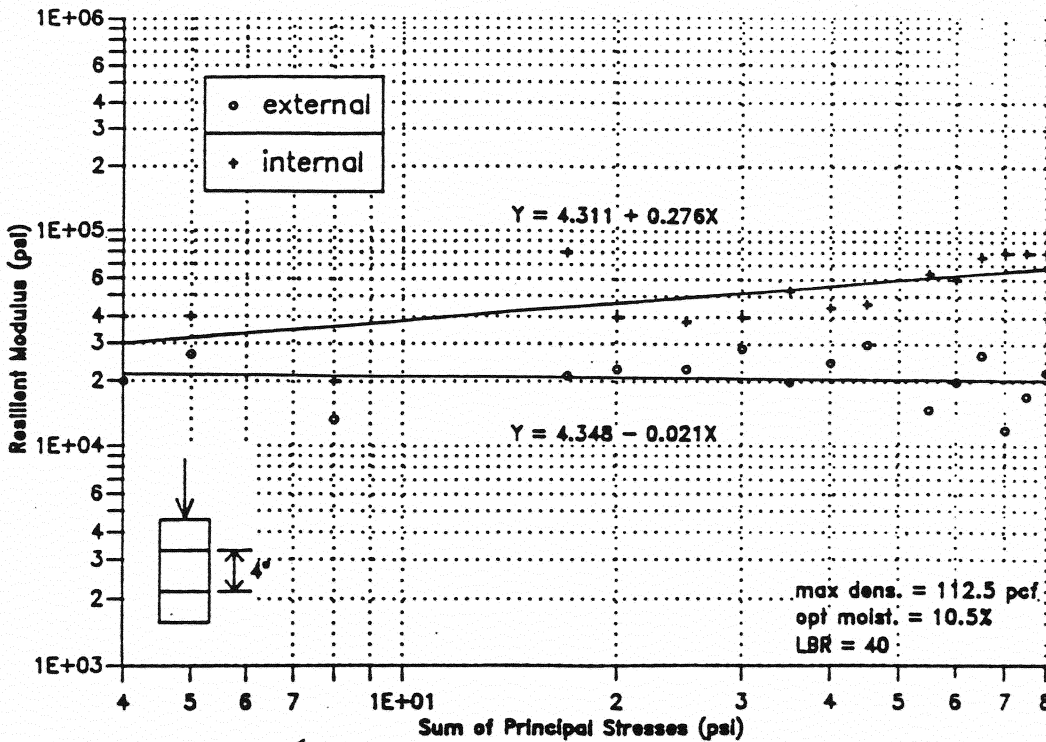


Figure 3.7: Plot of Resilient Modulus vs. Sum of Principal Stresses for Dark Grey Sand with Limerock,
Obtained by Obtained by AASHTO Method, Florida D.O.T. [26]

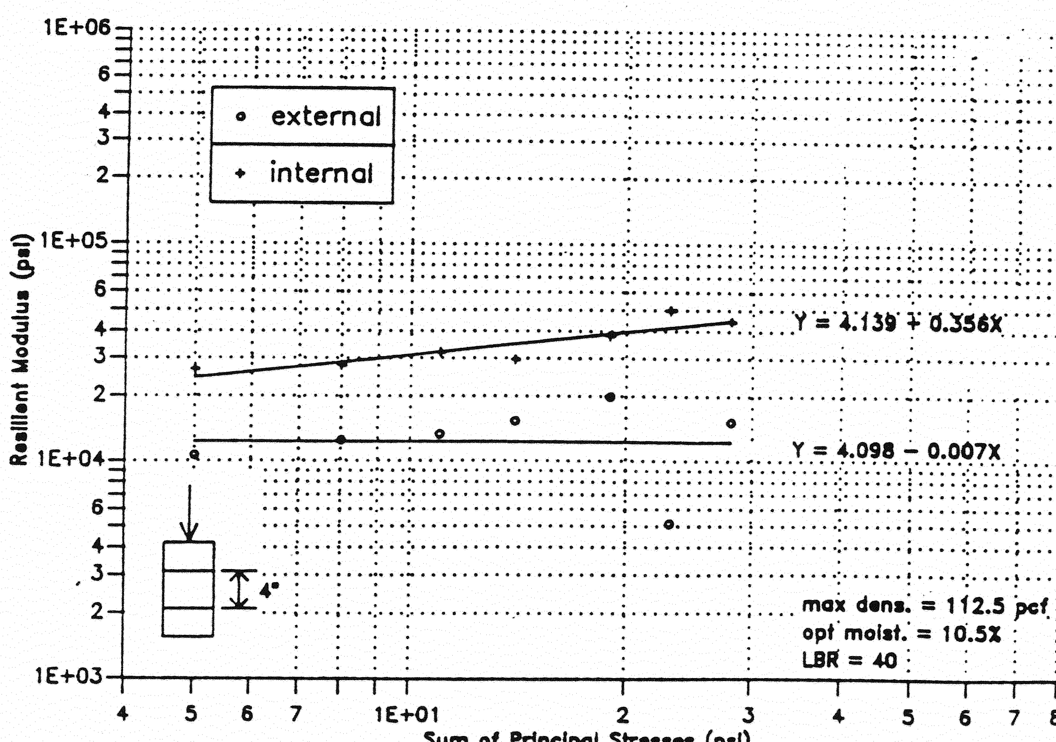


Figure 3.8: Plot fo Resilient Modulus vs. Sum of Principal Stresses for Dark Grey Sand with Limerock Obtained by ASTM Method, Florida D.O.T. [26]

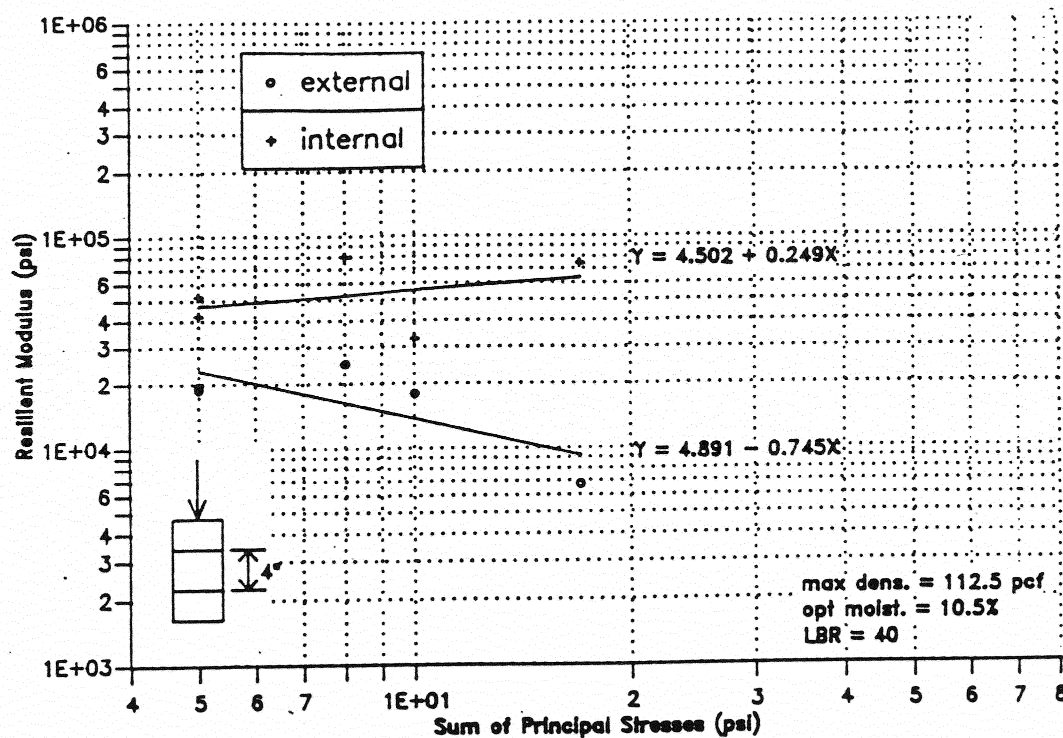


Figure 3.9: Plot of Resilient Modulus vs. Sum of Principal Stresses die Dark Grey Sand with Limerock Obtained by Modified Method, Florida D.O.T. [26]

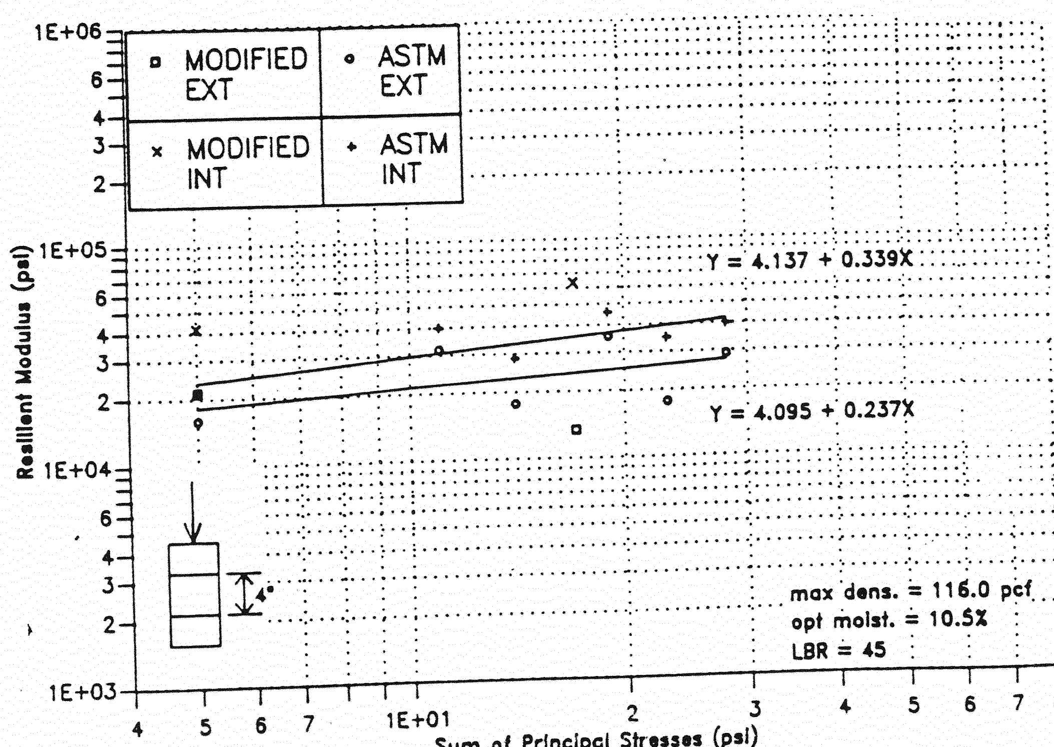


Figure 3.10: Plot of Resilient Modulus vs. Sum of Principal Stresses for Grey Sand with Shell Obtained by modified Method, Florida D.O.T. [26]

(20891-S) soil specimen could not be conditioned under AASHTO method, because the specimen failed during conditioning sequence itself. Therefore, only draft ASTM method and Modified method adopted for this soil. The results obtained from these two methods for the subgrade soil (20891-S) are presented in Fig. 3.10. Limerock samples were tested using AASHTO [2] and Modified method. The draft ASTM method was not performed because low confining pressure of this method were found inappropriate for Limerock. Results obtained for limerock are presented in Fig. 3.11.

7. <u>Discussion of Test Results</u>: It was found that AASHTO [2] procedure is inappropriate for subgrade materials (20849-S,20891-S) because the conditioning stresses are too severe. It was also seen from the Modified method that the RM of subgrade soils (20849-S,20891-S) generally appears to be independent of the number of load repetitions (Figs. 3.7 - 3.10). For limerock, however, results show an increase in RM with increase in number of repetitions (Fig. 3.11).

In all cases internal measurement resulted in higher RM values (lower resilient deformation), than external measurement. This was because external measurement include end effects. In all the three methods, for both subgrades (20849-S, 20891-S) and limerock, application of low deviator stress(1 & 2 psi) at high confining pressures, did not produce realistic RM values. For limerock samples, deviator stress of 1 and 2 psi often did not produce any measurable strains.

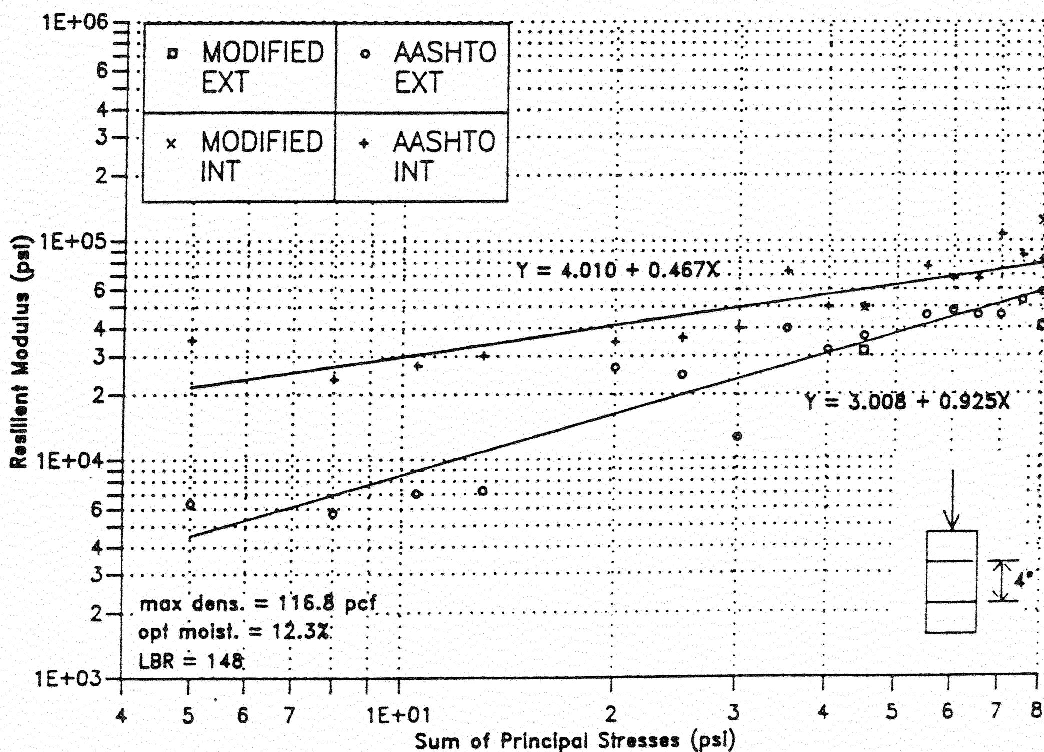


Figure 3.11: Plot of Resilient Modulus vs. Sum of Principal Stresses for Limerock Obtained by Modified Method, Florida D.O.T. [26]

3.5.2. New York D.O.T. [48]

The New York State Department of Transportation Soil Mechanics Bureau (SMB) suggested a procedure based on AASHTO T-274-82, but different conditioning and testing stress sequence for cohesive and cohesionless soil as given in Table 3.10 and Table 3.11, respectively [48,49].

Salient features of their testing procedure were as follows:

(a) Specimen Size

Testing was carried out on both cohesionless and cohesive soil specimen for cohesive soil the length to diameter ratio used was 2:1, which was in accordance with AASHTO [2]. For cohesionless soil specimen length to diameter ratio was chosen as 1.93:1, while AASHTO suggests this ratio to be 2:1.

(b) Specimen Compaction

All specimens were undisturbed specimen of 1.33 inch (3.375 cm.) diameter or were compacted in mold fabricated from a 3.375 cm. in diameter thin sampling tube in a manner similar to the standard or modified Proctor compaction tests. while AASHTO [2] prescribes compaction of specimen by vibratory, kneading or static methods.

(c) Specimen Saturation

The specimens were saturated in a similar manner as suggested by AASHTO [2].

(d) Load Duration

Load duration was chosen as 1.0 second for testing, while AASHTO specifies a duration of 0.1 second.

(e) <u>Deformation Measurements</u>

Deformations were measured by means of a single linear motion potentiometer (LMP). AASHTO suggests the use of LVDT for deformation measurements.

(f) <u>Stress Sequence</u>

For cohesionless specimens, the stress sequences that are in accordance with AASHTO [2], are shown in Table 3.10. The stress sequence for cohesive specimens (clay) is shown in Table 3.11. In this sequence, sample conditioning phase preceded the data collection phase at each deviator stress level. AASHTO specifies that each test begin with a specimen conditioning phase preceding the data collection phase where load and deflection are obtained.

(g) <u>Data Obtained</u>

The values of RM obtained from the above test procedure adopted by New York D.O.T. are presented in Tables 3.12 and 3.13. The tabulated values are average RM values. Although for individual stress conditions for each specimen RM ranged from 6,400 to 27,200 psi for cohesionless specimen and 1,200 to 21,700 psi for cohesive specimen. Average RM values obtained were about 13,600 psi for sand (cohesionless specimen) and 5,500 psi for clay (cohesive specimen).

From Table 3.14 it is clear that when deviator stress is kept constant, RM values vary substantially over the range of confining pressure. When confining pressure is kept constant and deviator stress is varied, the range of RM values obtained is also very large.

Table 3.10 Stress State Sequence for Cohesionless Specimen, Suggested by New York D.O.T. [48]

Cohesionless Specimen		
Phase	Deviator Stress (psi)	Confining Pr. (psi)
SC	5	5
SC	10	5
SC	10	10
SC	15	10
SC	15	15
SC	20	15
DC	1	20
DC	2	20
DC	5	20
DC	10	20
DC	15	20
DC	20	20
DC	1	15
DC	2	15
DC	5	15
DC	10	15
DC	15	15
DC	20	15
DC	1	10
DC	2	10
DC	5	10
DC	10	10
DC	15	10
DC	1	5
DC	2	5
DC	5	5
DC	10	5

Cohesionless Specimen		
Phase	Deviator Stress (psi)	Confining Pr. (psi)
DC	15	5
DC	1	1
DC	2	1
DC	2	1
DC	7.5	1
DC	10	1

SC = Specimen Condition
DC = Data Collection

Table 3.11 Stress State Sequence for Cohesive Specimen, Suggested by New York D.O.T. [48]

Cohesive Specimens		
Phase	Deviator Stress (psi)	Confining Pr. (psi)
SC	1	6
DC		6 3 0
SC	2	6
DC		6 3 0
SC	3	6
DC		6 3 0
SC	4	6
DC		6 3 0
SC	5	6
DC		6 3 0
SC	6	6
DC		6 3 0
SC	7	6
DC		6 3 0
SC	8	6
DC		6 3 0

Cohesive Specimens		
Phase	Deviator Stress (psi)	Confining Pr. (psi)
SC	9	6
DC		6 3 0
SC	10	6
DC		6 3 0
SC	11	6
DC		6 3 0
SC	12	6
DC		6 3 0

SC = Specimen Conditioning
DC = Data Collection

Table 3.12 Average RM Values for Cohesionless Specimen (Sand), Obtained by New York D.O.T. [49]

DEVIATOR STRESS psi	CONFINING PRESSURE (psi)				
	1	5	10	15	20
1	10,159	11,833	14,217	12,758	18,635
2	9,281	11,114	13,762	12,736	15,688
5	10,028	10,978	13,067	14,357	17,515
10	10,320	10,712	13,048	14,831	17,418
15		10,342	13,818	15,230	16,078
20				15,676	15,926

Note: Some RM values were not included in this table due to erroneous deflection data.

Table 3.13 Average RM values for Cohesive (Clay) Specimen, Obtained by New York D.O.T. [49]

DEVIATOR STRESS psi	CONFINING PRESSURE (psi)		
	0	3	6
1	7,178	6,619	6,086
2	9,933	10,386	10,258
3	7,012	7,011	7,261
4	4,530	4,470	4,775
5	3,035	3,406	3,441
6	2,907	3,073	3,255
7	3,068	3,539	3,219

It can be observed from Table 3.13 that trends of sand testing appear reversed with regard to clay. Varying deviator stress at constant confining pressure appears to have a significant influence on RM results of cohesive specimens. Varying confining pressure at constant deviator stress does not produce as large a range of RM values as with the sand testing.

3.5.3 Illinois D.O.T. [14]

The Illinois Department of Transportation fabricated its own equipment and has been involved in RM testing since 1984. The salient features of their testing procedure are discussed below [17,53]:

(a) Fine grained soil specimens were tested at the optimum moisture content and at 95% maximum dry density [17].

(b) Specimens were conditioned with 200 repetitions of 6 psi axial stress and 6 psi deviator stress. When soil is very fragile or soft, deviator stress may be reduced to 4 psi [17].

(c) After conditioning, the specimens were tested without lateral confining pressure ($\sigma_3 = 0$), followed by ten axial stress applications of 2, 4, 6, 8, 10, 14, 18 psi. Testing was sometimes terminated at 10 psi or lower due to excessive deformation of specimen.

(d) For each deviator stress the amount of recovered strain was determined and recorded. Modulus at 6 psi was chosen as the RM of the sample [17].

The agency also used falling weight deflectometer (FWD) and Dynatest 8002 to determine the subgrade RM. A drop force of 9,000 lbs. was used and the deflection was measured at 36 inches away from the center of the 12 inch diameter loading plate. This deflection was designated as D_3 [17,34].

The deflectometer deflections (D_3) were correlated with the RM for different types of materials. A summary of these correlations is given in Section 4.3.2.

3.5.4. South Dakota D.O.T. [11]

The South Dakota D.O.T. developed a procedure which is slightly different from AASHTO T-274-82 [2]. A total of 21 soil specimens from South Dakota were tested. Salient features of the modified procedure are given below [2,11]:

(a) The moisture content selected for 21 specimens varied between 20.3% and 34.7% by weight.

(b) Before beginning sample conditioning, the specimen was hydrostatically pressurized to 6.0 psi (41.4 kpa) for 300 seconds and then returned to atmospheric pressure. This technique allows the specimen to be tested for leaks throughout the testing procedure.

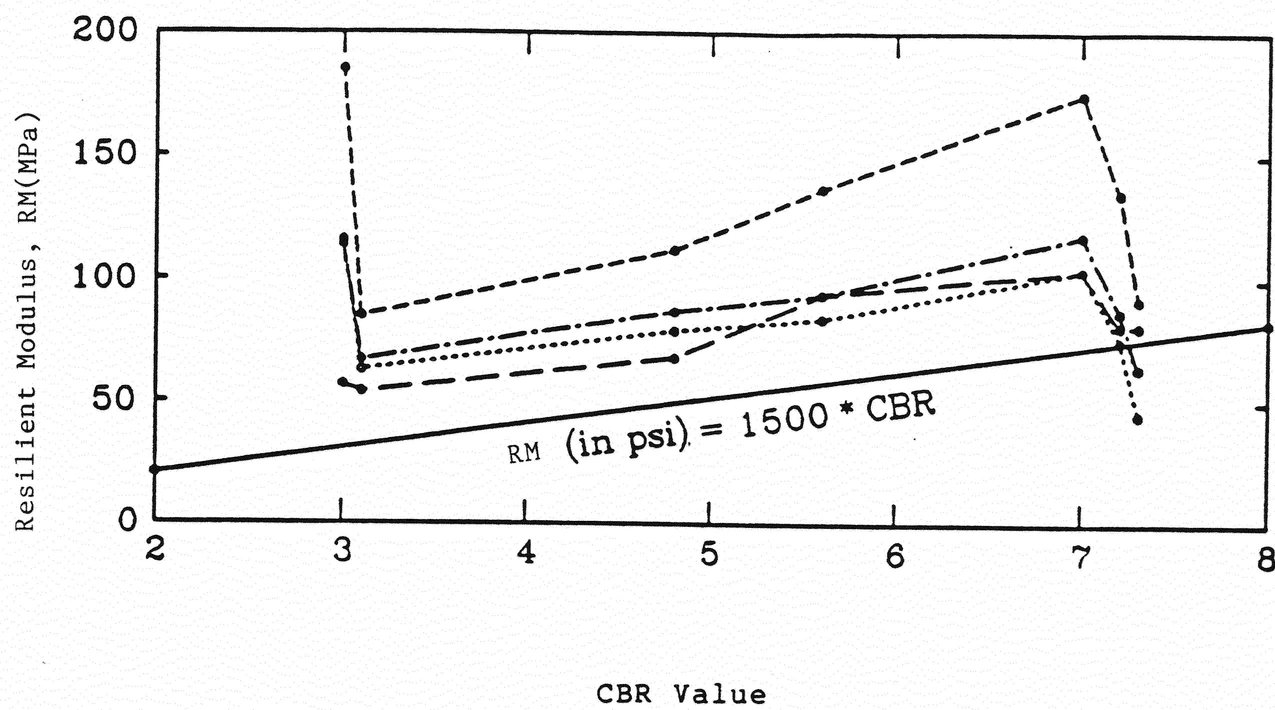
(c) After checking for jacket leaks, the specimen was conditioned by applying 200 cycles each of deviator stresses, 1 psi (6.9 kpa), 2 psi (13.8 kpa), 4 psi (27.6 kpa), 8 psi (55.2 kpa), and 10 psi (69.0 kpa). A complete testing sequence consisting of 15 sets of 200 loading cycles each was then applied. Beginning with 1 psi (6.9 kpa) deviator stress, 200 loading cycles were performed at confining pressures of 6 psi (41.4 kpa), 3 psi (20.7 kpa) and 0 psi. The process was then repeated for 2 psi (13.8 kpa), 4 psi (27.6 kpa), 8 psi (55.2 kpa) and 10 psi (69.0 kpa) deviator stress. Sample deformations were measured during sample conditioning and testing.

(d) Load duration was maintained at 0.1 seconds.

(e) The resilient moduli were calculated as per AASHTO T-274-82. Calculated RM exhibited significant scatter. Therefore, a second method was adopted for calculating RM. In this method, resilient moduli were calculated after 150 to 200 repetitions of deviator stress. Modulus values obtained for the last 50 cycles were averaged for each stress state, this average value was considered as the actual RM.

(f) The resilient moduli were compared with the CBR values. As shown in Fig. 3.12, the relation (RM = 1500 x CBR) suggested by AASHTO [1] did not fit the data. It was found that for the CBR values between 3.1 and 7.0, the correlation with CBR was relatively better.

(g) Based on these data, South Dakota DOT suggested some improvements to AASHTO T-274-82. These are: (1) eliminating the use of a vacuum pump for checking jacket leaks because such vacuum has a tendency to compact the specimen. South Dakota D.O.T. devised a procedure using a bubble chamber to check leak under pressure. This method is helpful in checking jacket leaks throughout the tests. Maximum stress that is acceptable as a lower limit during



— — — σ_3 = 41.4 KPa, σd = 6.9 KPa

---- σ_3 = 0 KPa, σd = 6.9 KPa

-·-·- σ_3 = 41.4 KPa, σd = 69.0 KPa

····· σ_3 = 0 KPa, σd = 69.0 KPa

—— As per AASHTO Guide (1986)

Figure 3.12: Resilient Modulus vs. CBR Value for Four Stress States [18]

cycling is also suggested for AASHTO T-274-82 because all loads cannot be removed from the specimen.

3.5.5. University of Arkansas [37,55]

A testing procedure to determine RM of cohesive subgrade soils was proposed by the research team at the University of Arkansas, Fayetteville. The method is applicable to disturbed samples prepared for testing by compaction in the laboratory. The important features are given below [21,37,55,56].

(a) Moisture content should be between 105% to 120% of the optimum moisture content. Dry densities between 92% and 98% of maximum density should be considered acceptable [37].

(b) All drainage valves leading to a specimen were kept open and a confining pressure of 3 psi was applied. Test was begun by applying 200 repetitions of deviator stress of magnitude 8 psi. These 200 repetitions were considered adequate for both the conditioning and testing phase. Recoverable or elastic deformations were recorded [37].

(c) The deviator stress was then decreased to 4 psi and fifty repetitions of this loading were applied for determination of RM.

3.5.6 University of Tennessee [18]

Drumm et al. [18] conducted a comprehensive series of RM tests on Tennessee soils at the University of Tennessee. A total of 11 soils were selected for testing.

The majority of soils tested were low plasticity silts and clays. Basic soil properties (such as liquid limit, plasticity limit, maximum dry density, optimum moisture content) of these soils were also determined from standard tests. Important properties of these soils are given in Table 3.14.

Salient features of the testing program are outlined below:

(a) Specimens were prepared in accordance with AASHTO Test Method T-274-82 [1] and the Tennessee D.O.T. (TN DOT, 1981) specifications.

(b) Based on observed unconfined shear strengths and the cohesive nature of the selected soils, the effects of confining stress were neglected and all tests were conducted at zero confining pressure.

(c) Specimens were conditioned by applying at least 100 cycles of load at 10 deviator stress levels. The repeated loads were applied at a frequency of 2 Hz.

(d) Resilient modulus values were expressed in terms of deviator stress using the concept of hyperbolic model. This correlation is given by Eq. 3.3.

$$RM = \frac{a + b(\sigma_d)}{\sigma_d} \quad For\ \sigma_d > 0 \qquad (3.3)$$

where, a and b are the material parameters, σ_d = deviator stress.

Table 3.14 Properties of Test Specimens, Obtained by University of Tennessee [18]

Specimens	Unit Weight γ (lb/ft^3)	Saturation S (%)	Unconfined Compressive Strength*	
			(psi)	(kPa)
A31-1 A31-1	105.0 106.0	70.7 72.6	63.3 63.9	436 440
B21-1 B21-2	100.3 102.1	80.5 78.6	68.8 64.8	474 447
C11-1 C11-2	113.1 112.7	81.2 84.5	30.9 32.3	213 223
D11-1 D11-2	83.3 83.2	86.7 83.4	28.7 33.5	198 230
E21-1 E21-2	97.2 98.6	84.8 79.8	67.7 72.1	467 497
E31-1 E31-2	98.9 96.8	92.8 87.5	45.6 36.2	314 249
F11-1 F11-2	105.7 106.4	80.0 80.8	53.5 44.0	369 303
H11-1 H11-2	109.9 109.9	77.3 83.0	62.6 62.3	431 429
H21-1 H21-2	114.2 115.2	81.8 83.9	39.7 51.9	274 358
J11-1 J11-2	82.9 81.4	97.2 93.7	27.3 27.3	188 188
J31-1 J31-2	89.5 86.3	92.9 91.9	46.0 53.0	317 365

* Unconfined Compressive Strength following repeated load testing.

(e) The resilient modulus was determined at different deviator stress levels. These results are presented in Figs. 3.13 and 3.14 for two types of soil. These figures gave the value of 'a' and 'b' for each soil type. The values of parameters 'a' and 'b' for each of eleven soils are provided in Table 3.15 along with the minimum value of resilient modulus (RM) min. and the breakpoint modulus RMi at deviator stress (σ_d) = 6 psi.

Equation 3.3, suggested by Drumm et al. [18], represents a non-linear relationship between resilient modulus and deviator stress.

Resilient modulus values obtained from Equation 3.3 were compared with the experimental data as shown in Figs. 3.15 and 3.16 for two different soils. It was found that the hyperbolic relationship given by Eq. 3.8 could be used to estimate the RM of Tennessee soils in a fairly accurate manner. This observation is particularly applicable for deviator stress exceeding about 3 psi.

3.5.7. Asphalt Institute [3]

The Asphalt Institute suggested a method for determination of resilient modulus of untreated fine grained soils using state of stress that approximately represents stress conditions in pavements due to moving wheel loads. The suggested method can be categorized as the repeated load triaxial test. The salient features of this method are given below [31]:

Table 3.15 Minimum and Breakpoint Resilient Modulus and Hyperbolic Model Parameters from Repeated Load Test Specimens

Specimen	Laboratory Data		Hyperbolic Model Parameters	
	RM (Min.) RM_{min} (ksi)	RM (Break Point) RM_i (ksi)	a	b
A31-1 A31-1	10.0 7.0	15.0 11.0	64.29 41.52	5.84 5.10
B21-1 B21-2	11.5 10.0	14.0 15.0	29.23 44.25	10.17 6.96
C11-1 C11-2	10.5 11.5	11.5 12.0	6.10 6.66	11.65 10.61
D11-1 D11-2	2.0 2.0	2.0 2.0	3.74 4.58	2.11 2.08
E21-1 E21-2	14.0 13.0	18.0 15.0	52.06 69.88	10.26 7.02
E31-1 E31-2	5.0 5.0	8.0 10.0	25.14 52.44	3.06 2.03
F11-1 F11-2	5.0 5.0	6.0 6.0	18.84 12.38	4.02 4.37
H11-1 H11-2	5.0 6.5	7.5 10.0	20.06 39.04	4.58 4.82
H21-1 H21-2	6.0 6.5	8.0 8.0	15.49 2.03	5.48 6.13
J11-1 J11-2	6.5 11.5	12.0 13.0	55.60 55.20	2.99 4.43
J31-1 J31-2	14.0 12.5	17.0 16.5	64.82 62.04	9.68 8.34

1 ksi = 6.89 MPa

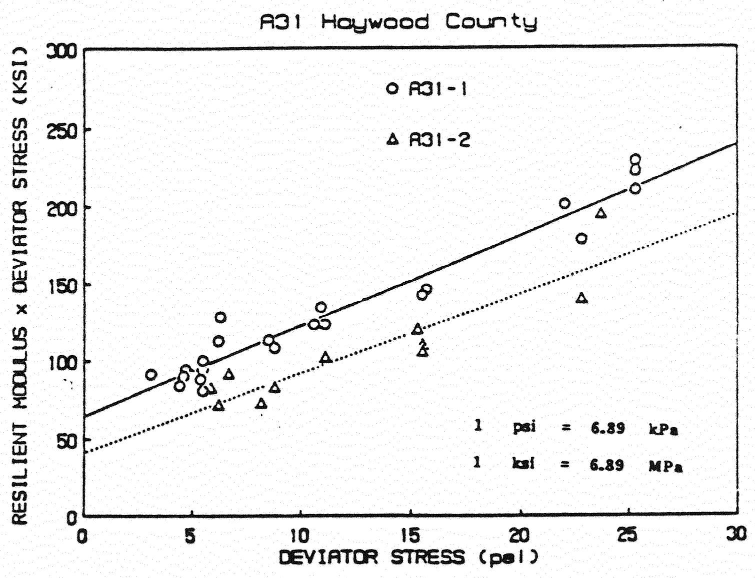


Figure 3.13: Resilient Modulus Data in Trasformed Coordinates Soil A31 [18]

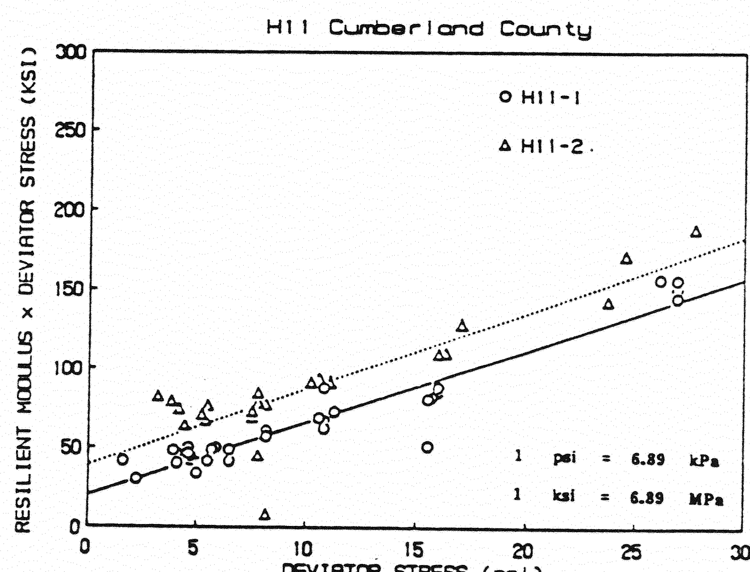


Figure 3.14: Resilient Modulus Data in Transformed Coordinates Soil H11 [18]

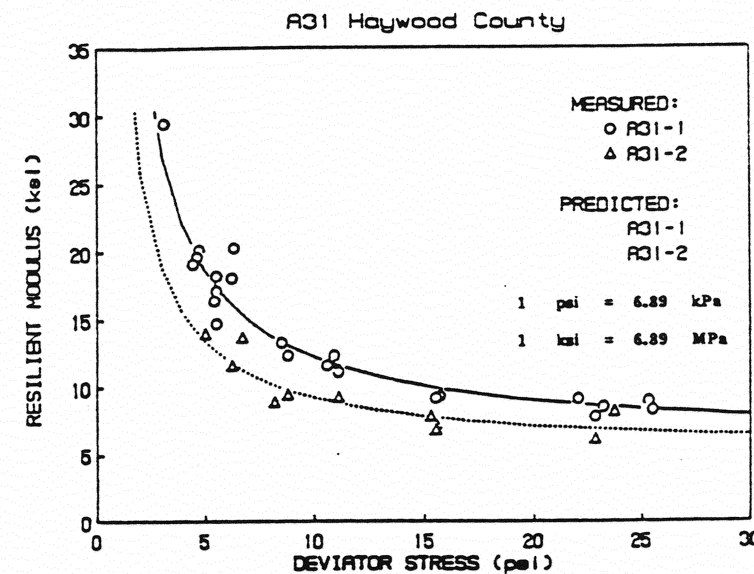


Figure 3.15: Comparison of Model and Measured Response Soil A31 [18]

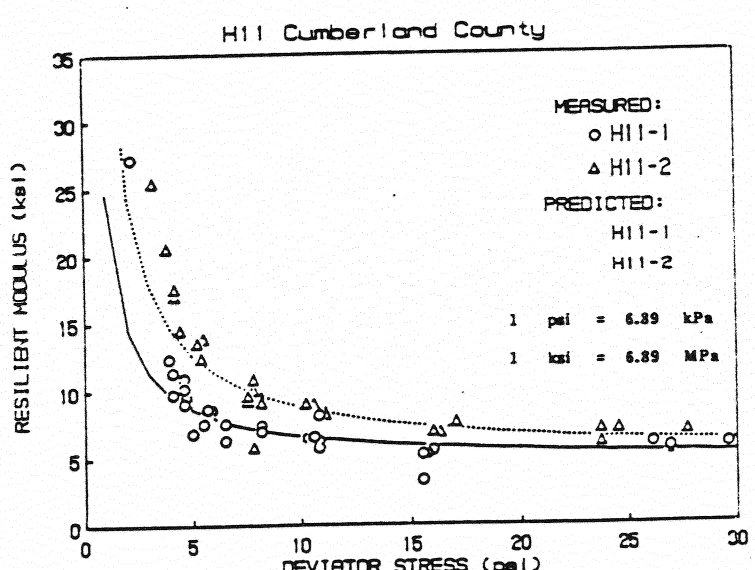


Figure 3.16: Comparison of Model and Measured Response Soil H11 [18]

(a) According to this method, resilient modulus can be evaluated using the following water contents [3]:

(i) optimum water content;

(ii) ±1% of optimum water content; and

(iii) approximately +2% or +3% of optimum water content.

(b) 200 repetitions of a deviator stress of magnitude 3 psi were applied in the beginning followed successively by 200 repetitions each at 6 psi and 9 psi. These stress sequences were used for conditioning the sample. Confining pressure was maintained at 2 psi [3].

(c) Deviator load was decreased to 6 psi and 200 repetitions of loads were applied. The deviator load and vertical recoverable deformations were recorded at or near 200th repetition [3].

3.5.8. U.S.D.A. Forest Service [13,35]

The United States Department of Agriculture Forest Service carried out repeated load triaxial testing in the laboratory in order to develop correlations for prediction of subgrade resilient modulus. Tests were carried out both on granular and cohesive soils. The AASHTO Test Method T-274-82 [2] was adopted in preparing specimens and conducting the tests. Two different compaction methods were adopted: (1) the maximum density at the optimum water content, and (2) one other density either on wet of optimum or dry of optimum [2,13].

The correlations given by this agency were verified for both granular and cohesive soils and were found to provide satisfactory resilient modulus values within the constraints considered in the analysis. The correlations developed by U.S.D.A. are discussed in Section 4.3.4.

Chapter 4

FACTORS INFLUENCING THE RESILIENT MODULUS OF SUBGRADE SOIL

4.1 INTRODUCTION

Many factors can influence the resilient modulus of subgrade soils. A review of these factors is presented in this chapter. Also included in the discussion are the correlations of resilient modulus with other conventional soil properties as well as factors which play an important role in the design of pavements [10,12,13,17,23-25,33,36,41,43,45,46].

4.2. INFLUENCE OF VARIOUS FACTORS ON THE RESILIENT MODULUS OF SOIL

The resilient modulus of fine grained soils and granular materials is stress dependent. Major factors influencing the RM of fine grained soils and granular materials are discussed below.

4.2.1. Fine Grained Soil

Behavior of cohesive soils under repeated loading display strain-softening resilient response. Several researchers attempted to establish relations between RM and magnitude of the cyclic deviator stress. A graphical representation of such a relationship is presented in Fig. 4.1 [17,23,52,55].

Recent experimental studies at the University of Illinois [23,52,53] have attempted to express RM in terms of stress dependent arithmetic model. It was observed that RM corresponding to a deviator stress of 6 psi (41.4 kpa) is a good

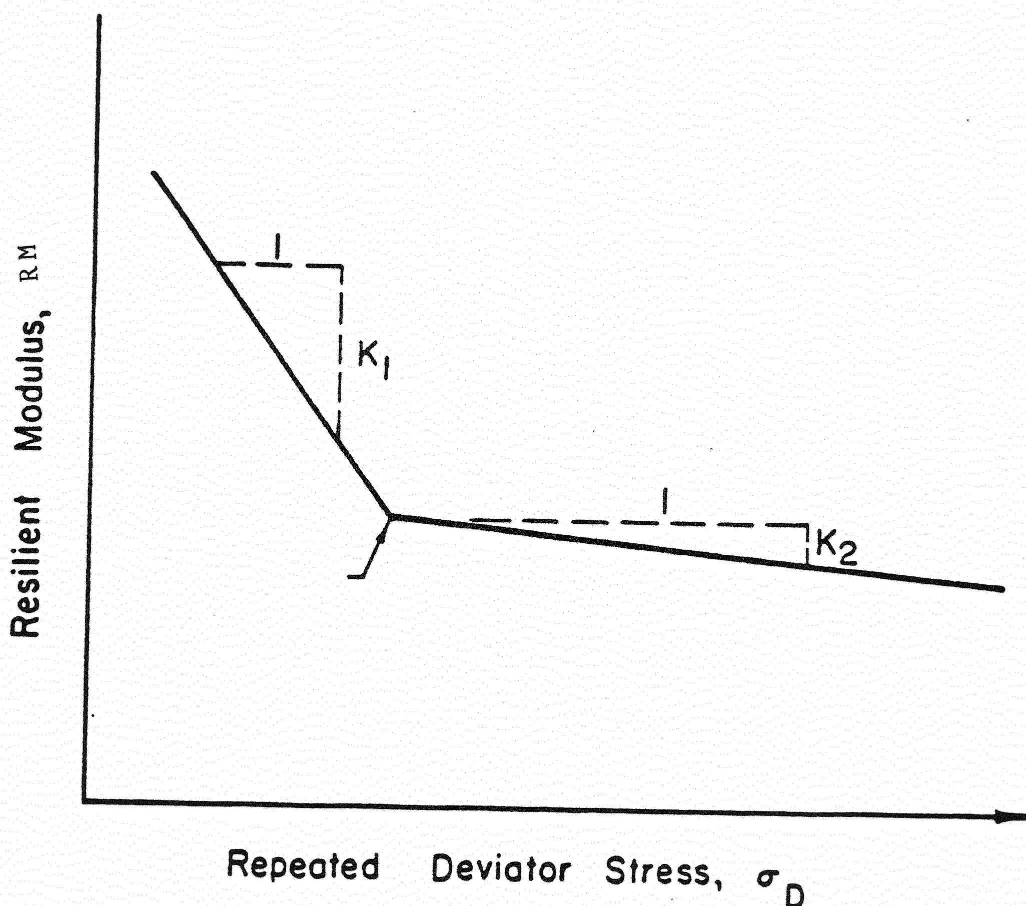


Figure 4.1: Arithmetic Model for Stress Dependent Resilient Behavior of Fine-Grained Soils [17,52,55]

indicator of the resilient behavior of soils. This was termed as breakpoint RM. In essence breakpoint RM is the modulus at which a significant change in slope occurs as shown in Fig. 4.1.

The following factors were found to influence the resilient response of fine grained soils [46,52,55].

(a) Soil Properties

Resilient response of fine grained soils can be significantly influenced by basic properties including liquid limit, plasticity index, group index, silt content, clay content, specific gravity and organic carbon content. Thompson et al. [52-55] found that RM modulus decreases with low plasticity (LL, PI), low group index, high silt content, low clay content, low specific gravity and high organic carbon content [54].

For fine grained Illinois soils the following correlation was proposed for using in conventional design of pavements [52]:

$$RM\ (OPT) = 4.46 + 0.98c + .119\ (PI) \tag{4.1}$$

where, RM (OPT) = Breakpoint resilient modulus (ksi) at AASHTO T-99 optimum moisture content and 95% compaction, c = less than 2 micron clay content (%), and PI = plasticity index (%).

(b) Degree of Saturation

Regression equations relating RM and degree of saturation were developed by Thompson [52,53]. It may be noted that these equations or correlations were different for 95% (AASHTO T-99) and 100% (AASHTO T-99) compactions. 100% compaction provided higher RM for a given degree of saturation as expected. The difference in RM value for 100% and 95% compaction was found to reduce at increased degree of saturation. Fig. 4.2 shows the behavior of RM at 100% and 95% compaction for the case of fine grained soil. A significant amount of scattering of data is observed for both cases, indicating that such correlations may not be very useful in terms of site-specific applications [7,13,17,35,41,50,52,55].

Fig. 4.3 shows the effect of compaction moisture content on resilient modulus [55] for these three types of soil. The RM of Wisconsin Loam Till (Fig. 4.3) shows a drop from 11 ksi (75.9 kpa) at the optimum moisture content to 4 ksi (27.6 kpa) at about 2% above optimum. It is clear from Fig. 4.3 that selection of an appropriate and representative water content can be a crucial factor in terms of short and long term applications; for short term moisture content, the undrained conditions would be more appropriate, while for long term moisture content, the drained condition would be appropriate [35,41,52,55].

(c) Method of Compaction

Fig. 4.4 shows the effect of compaction (static and kneading methods) on RM values for different soils. The first soil specimens, Fayette B, were compacted at 95% density. While the second soil, Wisconsin loam till, was compacted at 100% density. It was observed that for both soils static compaction consistently produced higher values of RM. It was also observed that RM decreases with increase in deviator stress magnitudes as expected. The rate of decrease is more sensitive at lower values

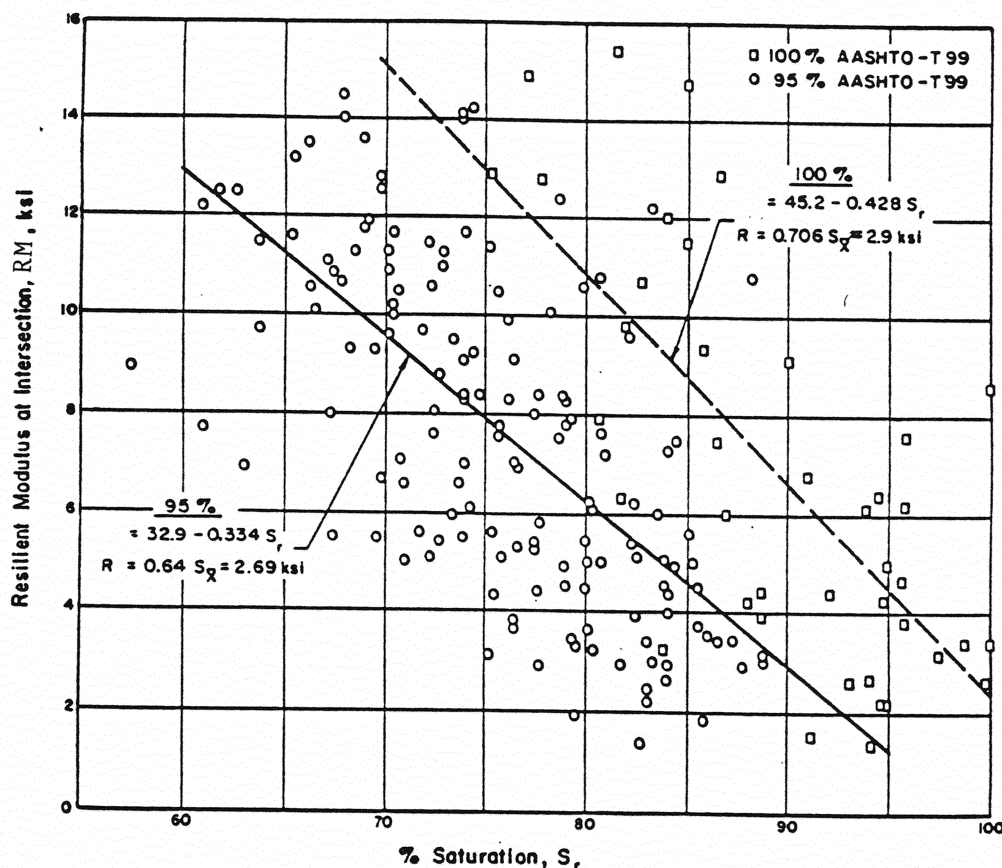


Figure 4.2: Resilient Modulus vs. % Saturation for Fine Grained Soils [52,53,55]

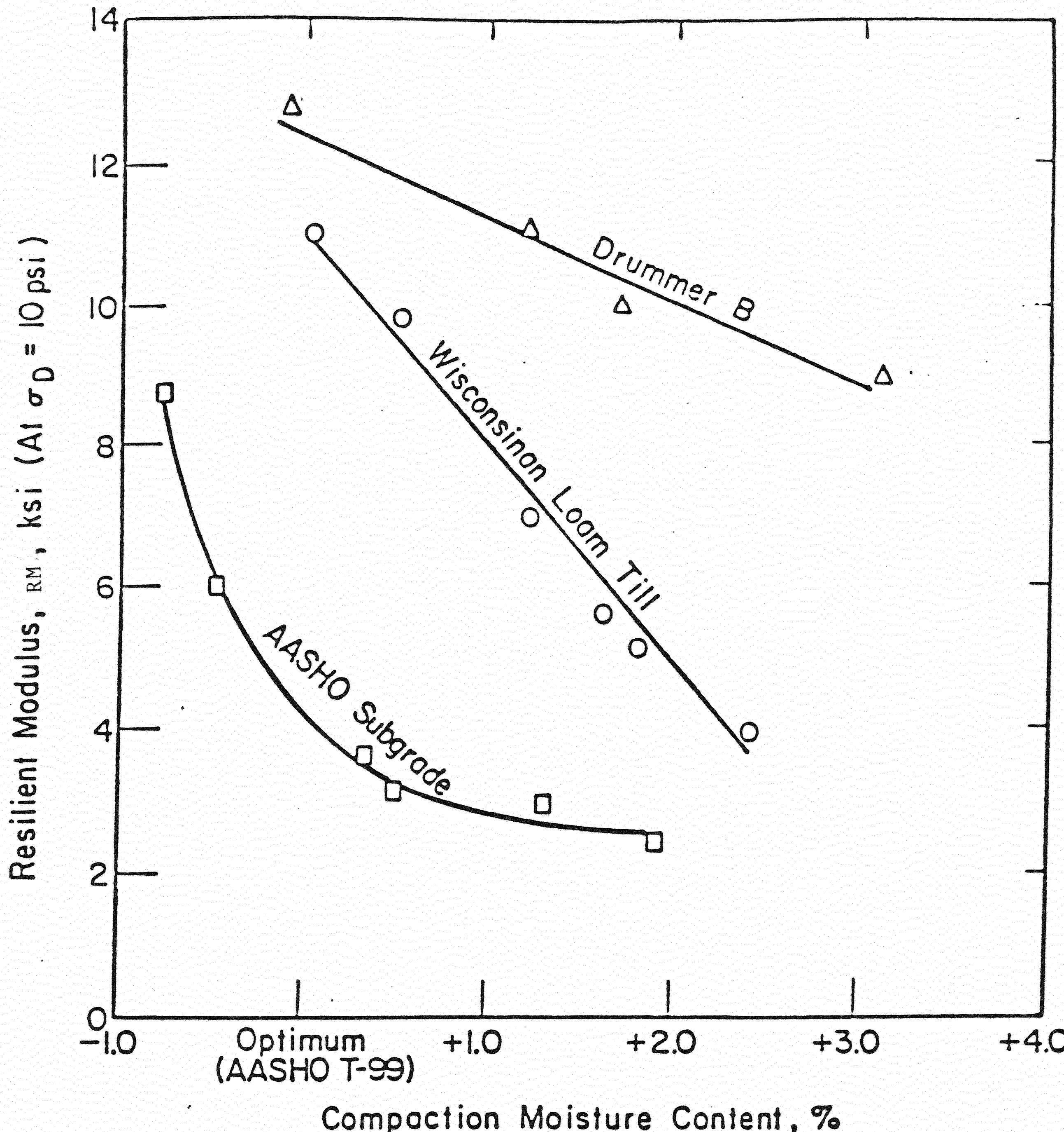


Figure 4.3: Relilient Modulus vs. % Saturation for Fine Grained Soils [52,55]

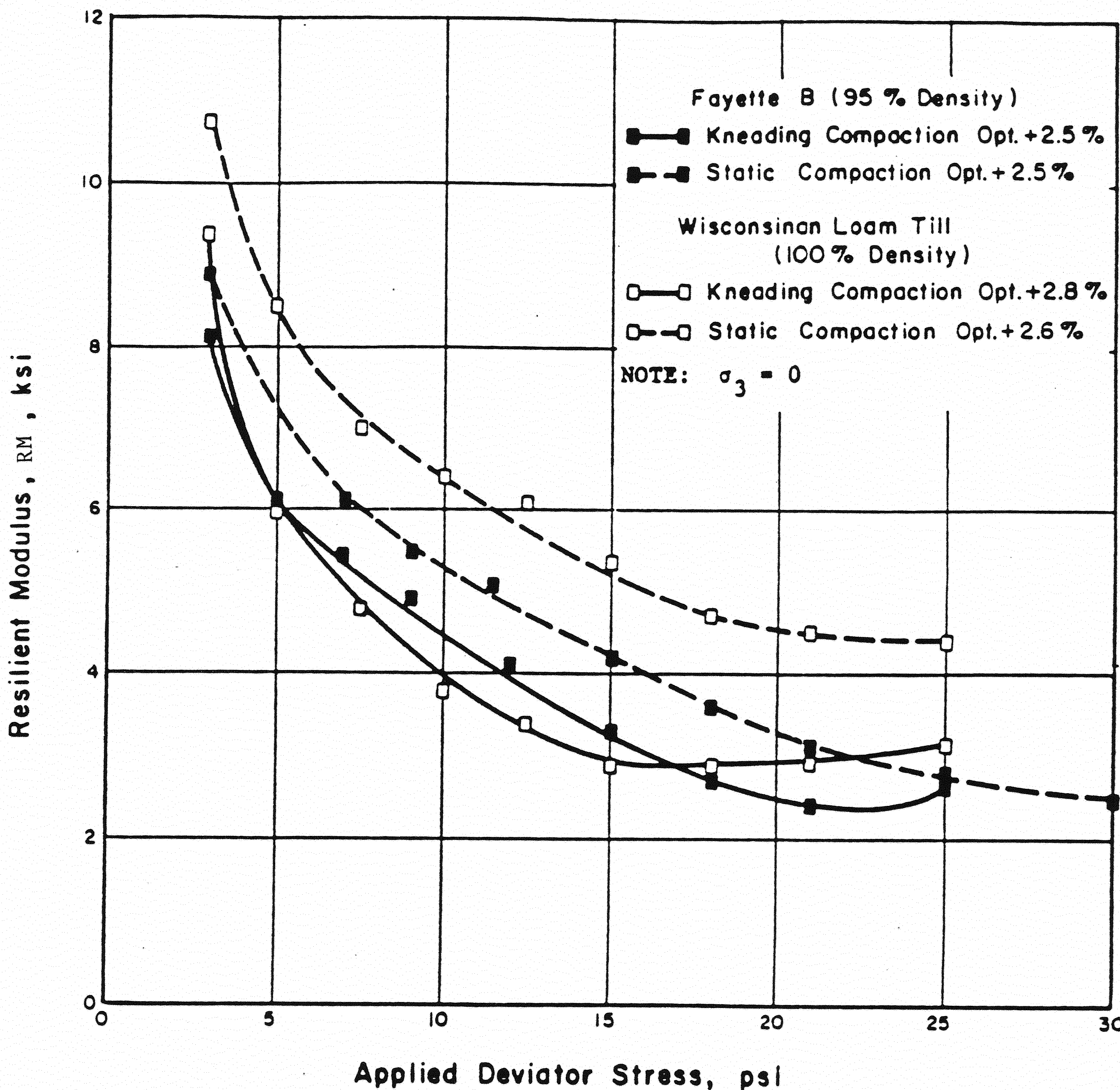


Figure 4.4: Effect of Density on Resilient Modulus of Specimens Compacted with Kneading Compactor [41,52,55]

of applied deviator stress and its effect gradually reduces with increasing deviator stress [41,52,53,54].

(d) Soil Classification Effects

Resilient modulus of a subgrade is not intimately related to soil classification systems, such as Unified, AASHTO and USDA. Classifying the soil according to AASHTO, Unified or USDA system emphasize different aspects of soil behavior and applications. For example, the AASHTO classification emphasizes on the suitability of soil for roadway application. However, classifying the soil according to these systems does not change any physical characteristics of the soil. Therefore, any suitable classification system can be adopted to place fine grained soils for resilient modulus testing [53].

(e) Effect of Compressive Strength

The University of Illinois [53,54] conducted a comprehensive experimental program for determining RM. A regression analysis was performed to obtain a relation between Q_u and RM [53,54].

$$RM = .86 + .307\ Q_u \tag{4.2}$$

where, RM = Break point resilient modulus, ksi (Fig. 4.1) and Q_u = Unconfined compressive strength, psi.

This information may be valuable from the view point of application because the unconfined compressive strength provides an indication of the in-situ conditions [41,52,54].

(f) Effects of Freeze-Thaw Cycles

Studies have shown [40,52,55] that the resilient modulus of fine grained cohesive soils is significantly influenced by cyclic freeze thaw action. A substantial increase in resilient deformations (i.e., reduced resilient moduli) has been reported by several researchers due to imposition of small number of freeze-thaw cycles [15,16,28,29,30,39,52,55].

A case illustrating the freeze-thaw cycles for Tama B soil (AASHTO Class A-7-6) is shown in Fig. 4.5 [52]. It is noted that one freeze-thaw cycle is sufficient to drastically reduce the resilient modulus of the soil [52,54]. It can be seen from Fig. 4.5 that percent drop in RM value for zero freeze-thaw cycle and 1 freeze-thaw cycle is around 55% and this drop is approximately constant at each deviator stress.

(g) <u>Effect of CBR Value</u>

CBR is widely used as an indicator of the strength characteristics of subgrade soils and such a relationship may be useful in practice. In general, subgrade with a higher CBR value will have higher RM [8,52-54,56]. The 1986 AASHTO Design Guide [1] suggested the following relationship between RM and CBR for fine grained soil [1,17]:

$$\text{RM (psi)} = 1500 \times \text{CBR} \qquad (4.3)$$

In a detailed study involving 15 Arkansas soils, Thornton [53,54] did not find any significant correlations between RM and CBR or R-value (the modulus of subgrade reaction). Similarly, Thompson and Robnett [53] tested eight Illinois soils. Fig. 4.6 shows CBR versus RM plot for these soils. The scattered data

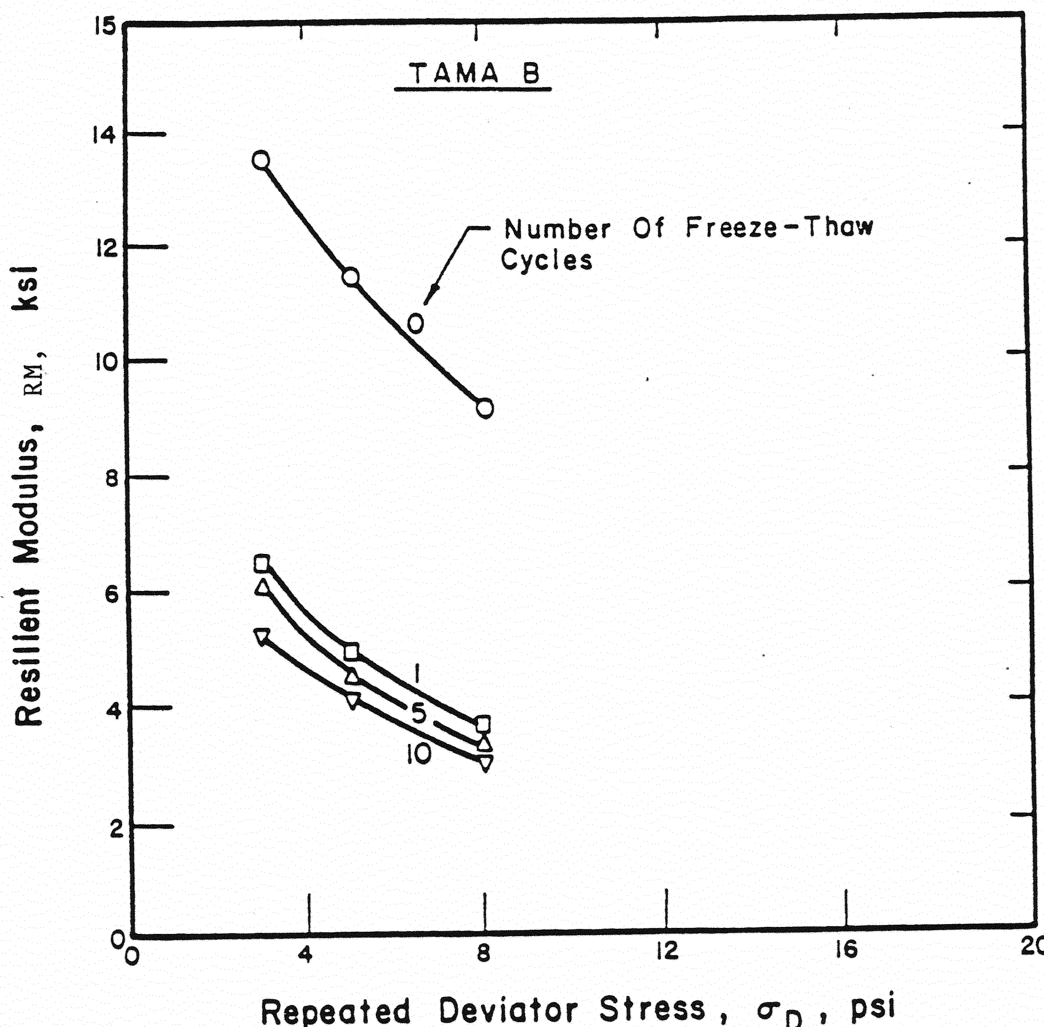


Figure 4.5: Influence of Cyclic Freeze-Thaw on the Resilient Modulus of a Fine Grained Soil (AASHTO A-7-6 [52,54]

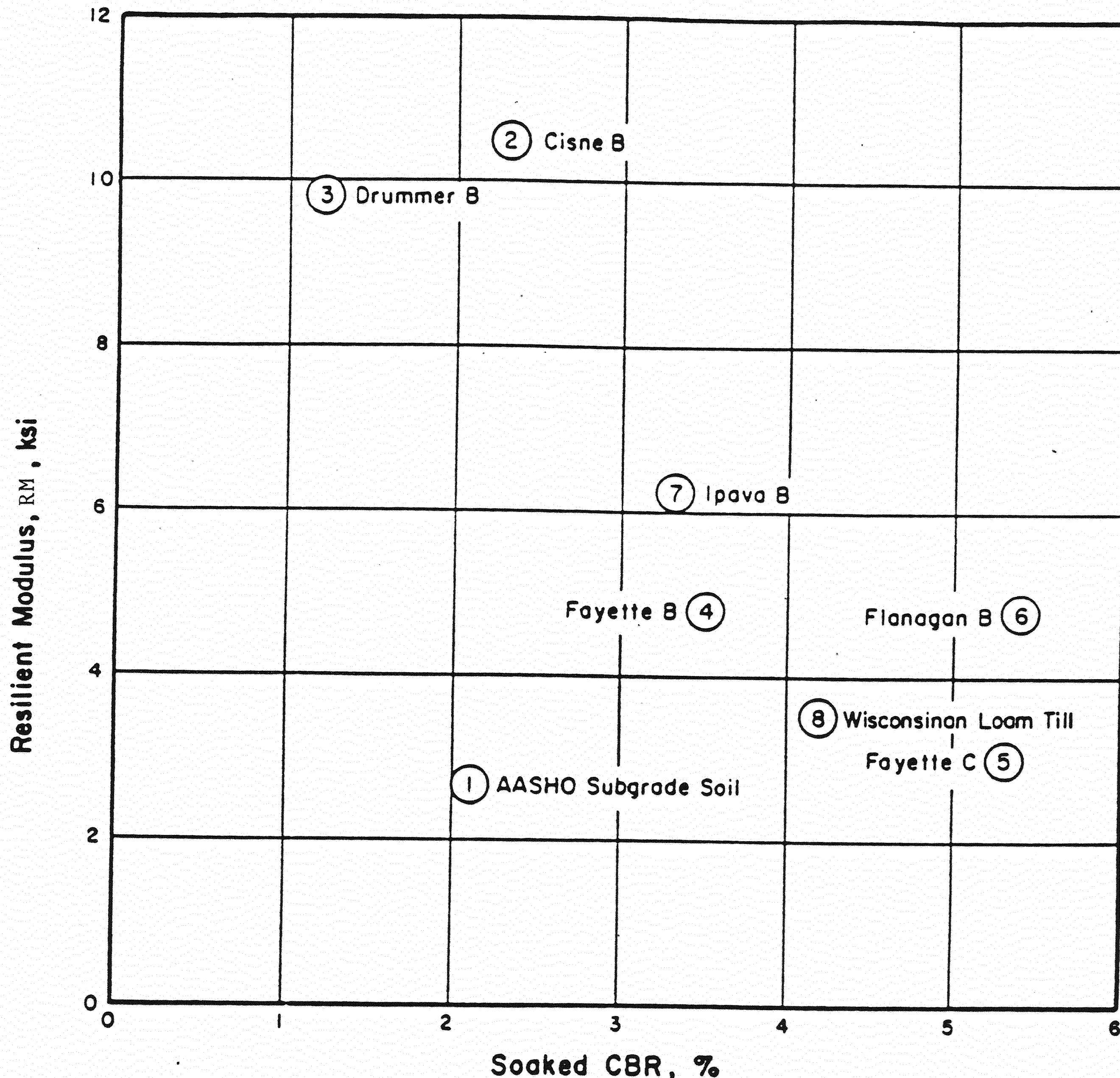


Figure 4.6: Plot of Soaked CBR and Resilient Modulus for Subgrade Soils [52,53,54]

demonstrates the weakness of any correlation between CBR and RM. Such inconclusive observations may be related to various factors related to sample conditioning and testing, procedures adopted as well as the questionable accuracy of test data. This indicates a need for further research in this important area [52,53].

4.2.2. Granular Soils

Unlike fine grained soils, deformation behavior of granular soils are dependent upon the in situ stress (confining pressure). In general, an increase in confining pressure results in an increased RM. Repeated load testing is widely used to evaluate RM of granular soils. The so-called 'Theta Model' is frequently used to express the RM in terms of confining pressure [1,33,45,53,57]. The model is given by Eq. 4.4 [1,28,44,54,59].

$$RM = k_1 \, \theta^{k2} \qquad (4.4)$$

where, RM = resilient modulus, k_1 and k_2 = experimentally derived factors, θ = bulk stress $= \sigma_1 + \sigma_2 + \sigma_3 = \sigma_1 + 2\sigma_3$ (in case of triaxial test)

Fig. 4.7 shows a relationship between RM - θ for a sandy gravel (AASHTO A-1-a) [53].

Comprehensive studies have been conducted by various researchers on the repeated loading behavior of granular soils [10,25,33,45,53,57]. A summary of the important observations from literature is given below:

(a) <u>Number of Load Repetitions</u>

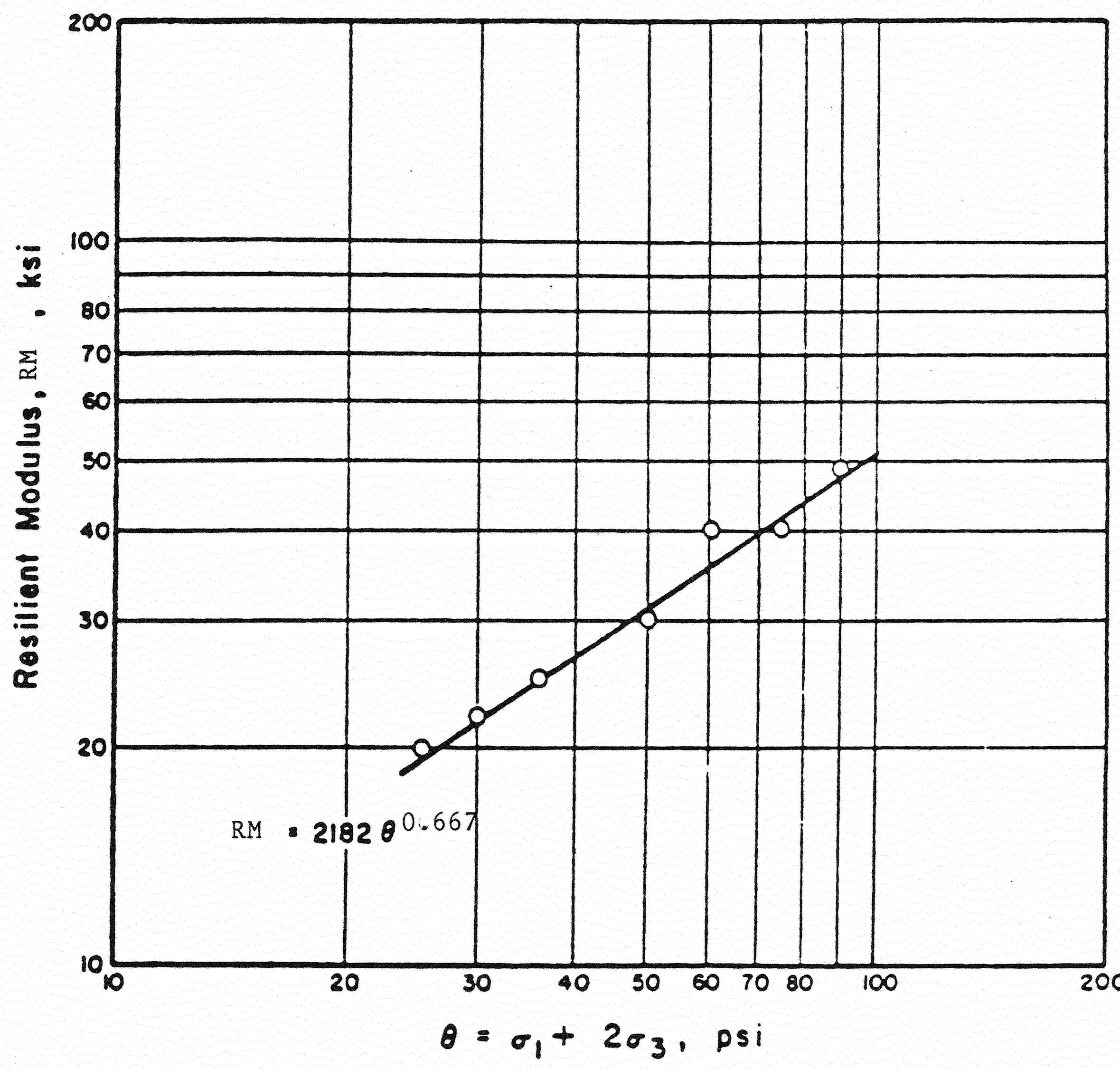


Figure 4.7: Resilient Modulus Relation for a Sandy Gravel [52,53,54]

The resilient response after a limited number of load repetitions (100 or more) is representative of the response determined after several thousand repetitions [40,53-55]. This is probably due to the fact that a granular soils specimen undergoes compaction for a limited number of load repetitions, at which time it reaches an optimum level of compaction. Beyond this density it is difficult to achieve further compaction.

(b) Stress Levels

Stresses can be applied in any order (repeated stress states are usually not greater than approximately 60% of the ultimate shear strength of a given soil material) [53]. The same specimen can be used to measure the resilient response over a wide range of stress levels [40,53,55].

(c) Stress Duration

RM is minimally affected by a variation in stress pulse duration. Monismith et al. [39,40] have shown that if a stress pulse is rapidly applied and then sustained, the resilient response is the same as that obtained from a rapidly applied and released short duration stress pulse of the same magnitude [40,53].

(d) Gradation

For a given gradation, crushed material provides increased RM. According to Thompson and Robnett [53,54], for a given gradation and nature of material (e.g., crushed, uncrushed, etc.), the source of soil (i.e., type of rock) does not significantly influence the RM values. This observation has not been verified by other researchers. RM of open graded aggregates tends to be somewhat lower than for conventional dense graded aggregates [53,55].

(e) Density

For a given soil, initial density has a limited impact on RM because the specimen gets compacted in the process of cyclic loading.

(f) Moisture Content

Increased moisture content tend to decrease RM. Moisture sensitivity will vary with voids between aggregates and with fines content in the voids [7,16,35,53].

Factors which influence RM of subgrade are summarized in Table 4.1 [10,12,13,17,23-25,33,36,41,43,56].

4.3 RELATIONSHIP OF RESILIENT MODULUS WITH VARIOUS PROPERTIES

4.3.1. AASHTO Design Guide (1986) [1]

The 1986 AASHTO Design Guide has suggested that the RM of fine grained soil can be estimated by Eq. 4.5 [1,18]:

$$RM\ (psi) = 1500 \times CBR \tag{4.5}$$

However, a number of research and transportation agencies have reported that this relationship may provide unreliable and misleading results [11,45].

4.3.2. Illinois D.O.T. [14]

The Illinois D.O.T. reported the following correlations for fine grained soils [14]:

Table 4.1 Factors Affecting RM of Subgrade Soils

Factor	Affect on Resilient Modulus (RM)
1. Stress duration	RM increases slightly when time of load application is reduced
2. Frequency	RM increases with increased frequency of load application
3. Grain size	RM moisture and density relationship is dependent on soil types
4. Saturation	RM decreases as a result of saturation
5. Confining Pr	Increase in confining pressure results in large increase in RM
6. Deviator stress	In granular soils, deviator stress level has little effect on RM so long as the sample has little plastic deformation

Table 4.2 Typical Resilient Property Data [43]

Granular Material Type	Number of Data Points	k_{1*} (psi)		k_2^*	
		Mean	Standard Deviation	Mean	Standard Deviation
Silty Sands	8	1620	780	0.62	0.13
Sand - Gravel	37	4480	4300	0.53	0.17
Sand - Aggregate Blends	78	4350	2630	0.59	0.13
Crushed Stone	115	7210	7490	0.45	0.23

$* \; RM = k_1 . \theta^{k2}$

(a) Relationship with clay percentage and plasticity index:

$$RM\ (OPT) = 4.46 + 0.098\ (\%\ clay) + 0.119\ (PI) \qquad (4.6)$$

where, RM (OPT) = resilient modulus at optimum water content (ksi), clay = particle finer than 2 micron, PI = plasticity index (AASHTO T-90).

(b) Relationship with unconfined compressive strength:

$$RM = 0.86 + 0.307\ Q_u \qquad (4.7)$$

where, RM = resilient modulus in ksi, Q_u = unconfined compressive strength in psi.

(c) Relationship with deviator stress:

Relationships were developed for fine grained soil (IPAVA-B) at two different moisture contents.

1. At water content 0.4% below optimum:

$$\text{Log } RM = 1.182 - 0.021\ \sigma_d \qquad (4.8)$$

2. At water content 1.8% above optimum:

$$\text{Log } RM = 1.029 - 0.037\ \sigma_d \qquad (4.9)$$

where, RM = resilient modulus in ksi, σ_d = deviator stress in psi.

The above relationships can be presented in a graphical form as shown in Fig. 4.8 [53].

(d) Illinois D.O.T.:

The Illinois D.O.T. also developed the following correlations based on the deflection measurements in the falling weight deflectometer tests [14].

1) Surface treatment plus granular base

$$RM\ (ksi) = 24.2 - 5.71\ (D_3) + 0.35\ (D_3)^2 \qquad (4.10)$$

2) Asphalt concrete (3+ inches) plus granular base

$$RM\ (ksi) = 25.0 - 5.25\ (D_3) + 0.29\ (D_3)^2 \qquad (4.11)$$

3) Asphalt concrete (any thickness) plus granular base (any thickness)

$$RM\ (ksi) = 24.1 - 5.08\ (D_3) + 0.28\ (D_3)^2 \qquad (4.12)$$

4) Full depth asphalt

$$RM\ (ksi) = 24.7 - 5.41\ (D_3) + 0.31\ (D_3)^2 \qquad (4.13)$$

5) Stiff pavement

$$RM\ (ksi) = 25.7 - 7.28\ (D_3) + 0.53\ (D_3)^2 \qquad (4.14)$$

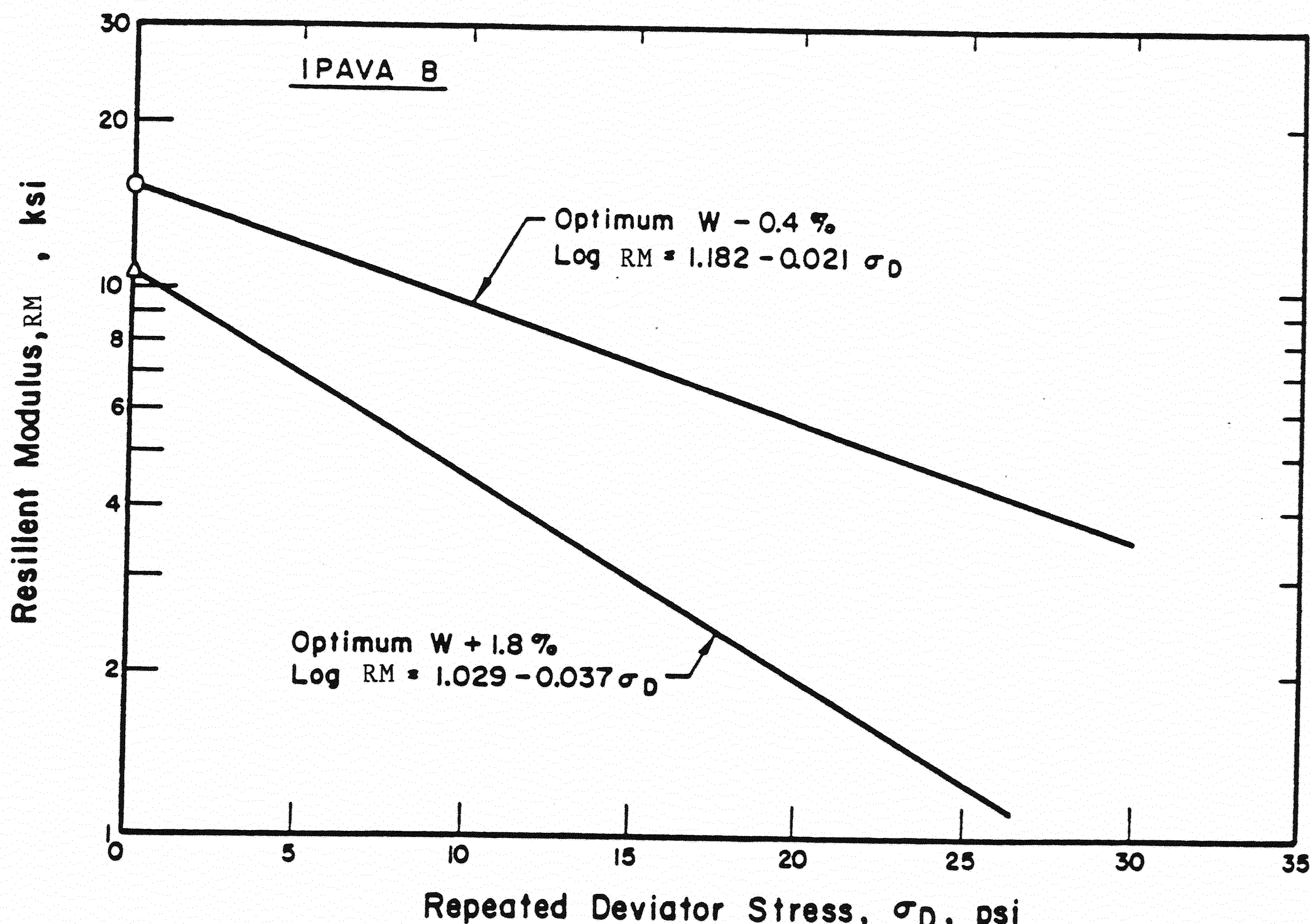


Figure 4.8: Semi-Log Model for Stress Dependent Resilient Behavior of a Fine-Grained Soil (AASHTO A-7-6) [25,52,53]

The variable D_3 in these equations represents the deflectometer deflection, as discussed in Section 3.5.3.

4.3.3 Maryland State Highway Administration

The Maryland State Highway Administration summarized typical resilient property data that were derived from available literature for granular soils. Their findings are summarized in Table 4.2. The relationship can be expressed in the form of Eq. 4.15.

$$RM = k_1\theta^{k2} \qquad (4.15)$$

Rada and Witczak [45,46] proposed a procedure to determine k_1 and k_2 as shown in Fig. 4.9. Also, shown in Table 4.3 are typical values of k_1 and k_2 as suggested by the 1986 AASHTO Guide [1,42-44].

4.3.4. U.S.D.A. Forest Service [9]

The U.S.D.A. Forest Service carried out a detailed study to develop models for predicting subgrade RM of cohesive and granular soils. These models are given below [9].

(a) Cohesive Soil

$$MR = 37.431 - 0.4566(PI) - 0.6179\ (\%W) - 0.1424\ (S200) + 0.1791(\sigma_d) - 0.3248\sigma + 36.422\ (CH) + 17.097\ (MH) \qquad (4.16)$$

where, CH = 1 for CH soil = 0 otherwise (for MH, ML or CL

Table 4.3 Typical Values for k_1 and k_2 for Unbound Base and Subbase materials ($RM = k_1 \theta^{k2}$)

Moisture Condition	k_1*	k_2*
(a) Base		
Dry	6,000 - 10,000	0.5 - 0.7
Damp	4,000 - 6,000	0.5 - 0.7
Wet	2,000 - 4,000	0.5 - 0.7
(b) Subbase		
Dry	6,000 - 8,000	0.4 - 0.6
Damp	4,000 - 6,000	0.4 - 0.6
Wet	1,500 - 4,000	0.4 - 0.6

* Range in k_1 and k_2 is a function of the material quality.

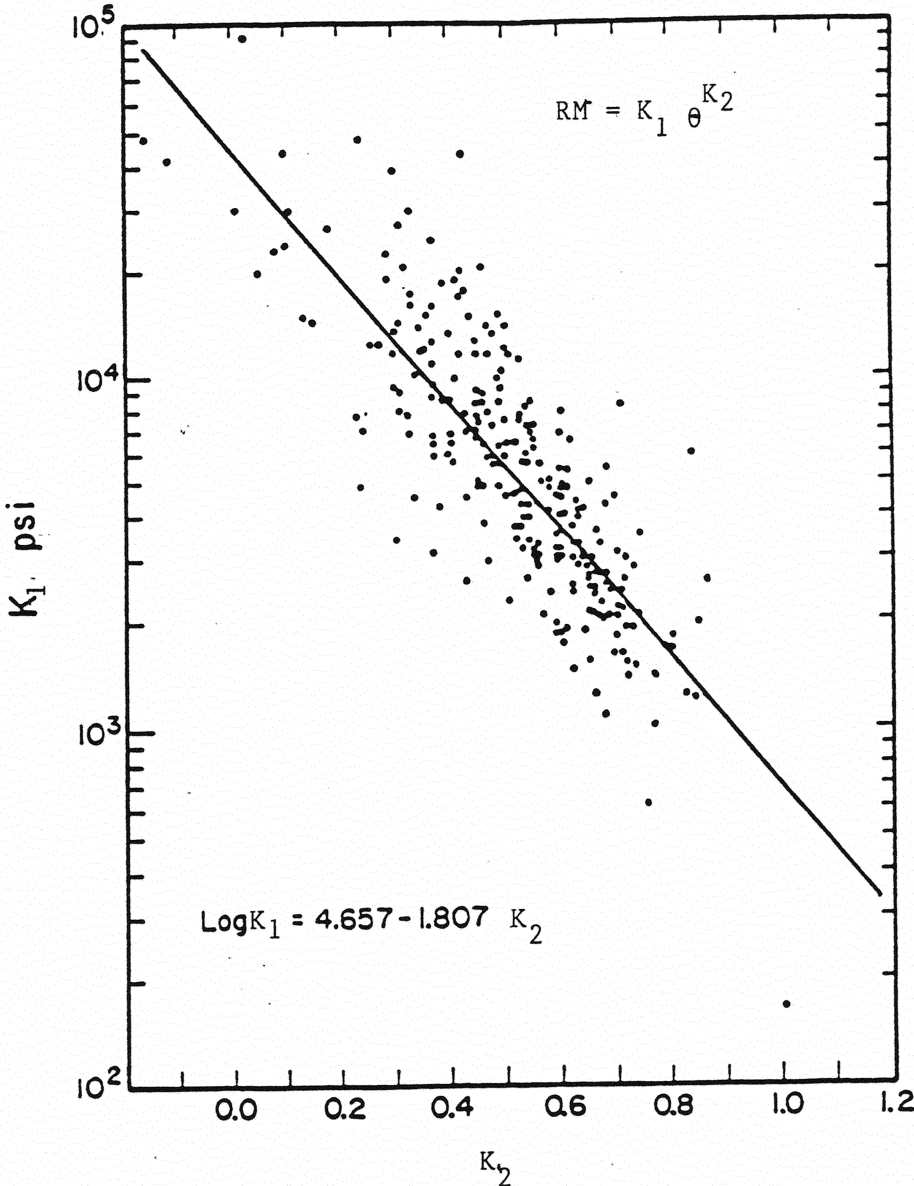


Figure 4.9: Relationship Between K_1 and K_2 Values for Granular Materials [45,52,56]

soil), MH = 1 for MH soil = 0 otherwise (for CH, ML or CL soil).

(b) Granular Soil

$$\text{Log } M_R = [0.523 - 0.0225\ (\%W) + 0.544\ (\log \theta) + 0.173\ (SM) + 0.197\ (GR)] \qquad (4.17)$$

where, SM = 1 for SM soils = 0 otherwise; GR = 1 for GR soils (GM, GW, GC or GP) = 0 otherwise.

Other terms in the above two equations are described below. RM = resilient modulus (ksi); PI = plasticity index; %W = water content in percent; σ_3 = confining stress (psi); σ_d = deviator stress (psi); θ = bulk stress (psi); $(\sigma + 3\sigma_d)$; r = dry density (pcf); S200 = percentage passing No. 200 sieve; and SS = soil suction.

These models were adopted for use in the new Forest Service Surfacing Handbook (FSSH 7709.56a). Soils were tested for verification of these models and good correlations were observed. However, no independent verification of these equations have been attempted yet.

4.3.5. University of Tennessee [18]

Drumm et al. [18] conducted an extensive study on Tennessee soils. He found the relationship given by Eq. 4.18 to be valid for these types of soils.

$$RM = \frac{a + b(\sigma_d)}{\sigma_d} \qquad (4.18)$$

where, σ_d = deviator stress, a and b are the material parameters.

It may be noted that this relationship is inverse of the hyperbolic relationship frequently used to represent the non-linear stress-strain response of soil. It should be noted that Eq. 4.18 is not defined at $\sigma_d = 0$. Hence this relationship is valid for deviator stress greater than zero.

Chapter 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

This study focused on the state-of-the-art of resilient modulus (RM). On the basis of information obtained through literature search the following conclusions have been drawn:

1. No standard method has yet been developed to determine RM of subgrade soils and bonded materials from laboratory tests. However, the repeated load triaxial test is most widely used and a majority of transportation agencies follow the AASHTO T-274-82 method in conducting this test, with some modifications in terms of sample conditioning, loading frequency, deviator stress magnitudes and sequence, number of loading cycles and other details. Specific modifications have been often dictated by soil type, loading condition, compaction requirements and other practices.
2. RM determined from different tests can vary significantly. According to Oregon D.O.T., RM values obtained from diametral resilient tests usually show more variation and inconsistency than those obtained from repeated load triaxial tests.
3. Many factors can influence the RM of subgrade soils including degree of saturation, level and method of compaction, freeze-thaw cycle, clay content, plasticity index and compressive strength. For example: (i) RM is drastically reduced with an increase in the freeze-thaw

cycle but it increases with increasing compressive strength, (ii) for a given gradation, crushed material provides increased RM, (iii) increased moisture content, beyond OMC, causes a reduction in RM; however, the degree of variation depends on the voids formed in the fabric of the aggregates and fines, (iv) for granular soils, initial density has limited impact on RM because the specimen gets compacted in the process of cyclic loading, (v) RM is minimally affected by a variation in stress pulse duration, (vi) RM is not intimately related to the conventional soil classification systems used, (vii) static compaction leads to higher RM than kneading compaction, and (viii) RM decreases with increased deviator stress magnitude.

4. The 1986 AASHTO Design Guide has suggested a correlation between RM and CBR. According to a number of research and transportation agencies, this correlation may provide unreliable and totally misleading results.

5. RM can be correlated with other soil properties such as compressive strength, initial elastic modulus, plasticity index, clay and silt content, moisture content and deviator stress. However, the existing correlations are very much dependent upon soils type, moisture content, freeze-thaw cycles, confining pressure and loading. Thus, these correlations derived for one soil type and conditions may not be applicable for other soil types and conditions.

6. Application of RM in pavement design is becoming increasingly more important among the State D.O.T.s.

5.2 RECOMMENDATIONS

Based on the conclusions of the present study, the following recommendations can be made:

1. Since RM values can differ significantly depending upon the sample preparation, conditioning, loading and measurement techniques used, it is recommended that a study be conducted to devise a RM testing method that would be consistent with ODOT practices (e.g., compaction, moisture content, loading), material type (e.g., shale, stabilized/ unstabilized subgrade, asphalt, etc.) and other considerations (e.g., freeze-thaw cycle, moisture content during construction and in-service, etc.). Such a study should include the effect of response measurement techniques (e.g., in-vessel versus outside).
2. Since RM determination involves significant commitment of resources in terms of time and equipment, it may not be economically feasible to conduct such tests for small projects. In such instances, as well as for other projects and as a part of bid preparation, it would be highly desirable to establish correlations between RM and other physical and mechanical properties of subgrades. It is recommended that such a study be undertaken to establish the desired correlations for common subgrade soils in Oklahoma. The long-term economic benefits from such a study are expected to be significant.
3. No previous study has addressed resilient modulus testing of

aggregate bases and stabilized (e.g., lime, fly ash) soils. It is recommended that a study be conducted to investigate resilient behavior of such materials.

4. Although the use of RM is an improvement, the 1986 AASHTO method is a pseudo-mechanistic approach for the design of roadway pavements. It is recommended that an improved mechanistic design approach be developed for pavement design. Such study should consider both flexible and rigid pavements and should account for the pavement-vehicle interaction.

References

1. American Association of State Highway and Transportation Officials (1986), "AASHTO Guide for Design of Pavement Structures," Washington D.C.

a. American Association of State Highway and Transportation Officials (1986), AASHTO Designation T-274-82, "Standard Method of Test for Resilient Modulus of Subgrade Soils," Washington D.C.

3. Asphalt Institute, "Method of Test for Resilient Modulus of Soil." Laboratory Manual Series No. 11, Appendix C, College Park, MD.

3. American Association of State Highway and Transportation Officials (1986), AASHTO Designation T-274-82, "Standard Method of Test for Resilient Modulus of Subgrade Soils," Washington D.C.

4. American Society for Testing and Materials (March 1988), "Test Method for Resilient Modulus of Subgrade Soils." PEP Temporary No. 18-023, ASTM, Philadelphia, PA.

5. Bell, C. A., et al (January 1984), "Application of Resilient Modulus Test Equipment and Procedure of Subgrade Soil - Part 1: Material Testing," Oregon Department of Transportation, p. 90.

6. Bell, C. A., et al (January 1984), "Application of Resilient Modulus Test Equipment and Procedure of Subgrade Soil - Part 2: Application of Results in Pavement Analysis," Oregon Department of Transportation, p. 142.

7. Bibbens, R. F., Bell, C. A., and Hicks, R. G. (1984), "Effect of Season of Year on Pavement Response," Transportation Research Record No. 993, Transportation Research Board, Washington D.C., pp. 1-11.

8. Bindra, S. P., and Al-Sanad, H. A. (March 1984), "A Comprehensive Evaluation of Pavement Design Parameters," Institution of Engineers, Australia 11 National Circuit Barton A.C.T., 2600 Australia, pp. 78-82.

9. Blacklock, J. R. (July 1984), "Study of Engineering Material Properties Which Influence the Utilization of Marginal Aggregates: Final Report," Arkansas University, National Technical Information Service, Springfield, Virginia, p. 97.

10. Borden, R. H., Lowder, W. M., and Khosla, N. P. (May 1986), "Evaluation of Pavement Subgrade Support Characteristics by Dilatometer Test: Final Report," North Carolina State University, National Technical Information Service, Springfield, Virginia, p. 157.

11. Brodsky, N. S. (October 1989), "Resilient Modulus Measurement on Cohesive Soils," Geotechnical Special Publication No. 24, ASCE, New York, NY, pp. 15-30.

12. Brown, S. F., and Pappin, J. W. (1981), "Analysis of Pavement with Granular Bases," Transportation Research Record No. 810, Transportation Research Board, Washington D.C., pp. 17-23.

13. Carmichael, R. F. III, and Stuart, E. (1985), "Predicting Resilient Modulus: A Study to Determine the Mechanical Properties of Subgrade Soil (Abridgement)," Transportation Research Record No. 1043, Transportation Research Board, Washington D.C., pp. 145-148.

14. Chandra Shekhar Rao, T. S., Doddihal, S. R., and Pandey, B. B. (April 1983), "Repeated Load Test Set Up for Determining Plastic Deformation Characteristic of Subgrade Soils," Indiana Geotechnical Journal, Vol. 13, No. 2, pp. 103-113.

15. Cole, D., et al (July 1986), "Resilient Modulus of Freeze-Thaw Affected Granular Soil for Pavement Design and Evaluation. Part 1. Laboratory Tests on Soils from Winchendon," Cold Regions Research and Engineering Laboratory Department of the Army, p. 13.

16. Cole, D. M., Irwin, L. H., and Johnson, T. C. (1981), "Effect of Freezing and Thawing on Resilient Modulus of a Granular Soil Exhibiting Nonlinear Behavior," Transportation Research Record, No. 809, pp. 19-26.

17. Dharmrait, J. S. (March 28-30, 1989), "Illinois Experience with Resilient Modulus," Workshop on Resilient Modulus Testing," Oregon State University, Corvallis, Oregon, 012.

18. Drumm, E. C., Boateng-Poku, Y., and Pierce, T. J. (May 1990), "Estimation of Subgrade Resilient Modulus from Standard Test," Journal of Geotechnical Engineering, ASCE, Vol. 116, No. 5, pp. 774-789.

19. Elliott, R. P. and Thornton, S. I. (1988), "Resilient Modulus and AASHTO Pavement Design," Transportation Research Record No. 1196, Transportation Research Board, Washington D.C., pp. 116-124.

20. Elliott, R. P. and Thornton, S. I. (1988), "Simplification of Subgrade Resilient Modulus Testing," Transforation Research Record No. 1192, Transportation Research Board, Washington D.C., pp. 1-7.

21. Elliott, R. P., et al (November 1988), "Resilient Properties of Arkansas Subgrades: Final Report," Arkansas University, National Technical Information Service, p. 104.

22. Emery, J. (1982), "Resilient Moduli of Flexible Pavement Components," Multiscience Publication Limited, McGill College, pp. 157-180.

23. Fisher, J. A., et al (1984), "Kr - The Resilient Modulus of Subgrade Reaction," Transportation Research Record No. 954, Transportation Research Board, Washington D.C., pp. 1-10.

24. Fredlund, D. G., Bergan, A. T., and Wong, P. K. (1977), "Relation between Resilient Modulus and Stress Conditions for Cohesive Subgrade Soils," Transportation Research Board, No. , Transportation Research Board, Washington D.C., pp. 73-81.

25. Hicks, R. G., and Monismith, C. L. (1971), "Factors Influencing the Resilient Response of Granular Material," Transportation Research Record No. 345, Transportation Research Board, Washington D.C. pp. 15-33.

26. Ho Robert, K. H. (March 28-30, 1989), "Repeated Load Tests on Untreated Soils, a Florida Experience," Workshop on Resilient Modulus Testing, Oregon State University, Corvallis, Oregon.

27. Hoebedd, P. (1985), "Laboratory Study of and Test Road Experiments on Incineratory Residue at Sysav, Malmoe," National Swedish Road and Traffic Research Institute, p. 28.

28. Johnson, T. C., et al (February 1987), "Resilient Modulus of Freeze-Thaw Affected Granular Soils for Pavement Design and Evaluation, Part 3," National Technical Information Service, p. 40.

29. Johnson, T. C., et al (October 1986), "Resilient Modulus of Freeze-Thaw Affected Granular Soils for Pavement Design and Evaluation, Part 4," Field Validation Test at Albany County Airport, National Technical Information Service, p. 148.

30. Johnson, T. C., Berg, R. L., and Dimillo, A. (1986), "Frost Action Predictive Technique: An Overview of Research Results," Transportation Research Record No. 1089, Transportation Research Board, Washington D.C., pp. 147-161.

31. Johnson, E. G., and Hicks, R. G. (1987), "Evaluation of Effect of Uncrushed Base Layers on Pavement Performance," Transportation Research Record No. 1117, Transportation Research Board, Washington D.C., pp. 11-20.

32. Kalcheff, I. V, and Hicks, R. G. (November 1973), "A Test Procedure for Determining the Resilient Properties of Granular Material," Journal of Testing and Evaluation, Vol. 1, No. 6, pp. 472-279.

33. Khedr, S. (1985), "Deformation Characteristics of Granular Base Course in Flexible Pavements," Transportation Research Record No. 1042, Transportation Research Board, Washington D.C., pp. 131-138.

34. Khosla, P. N. and Ali, N. A. (1987), "Determination of Layer Moduli Using a Falling Weight Deflectometer," Transportation Research Record No. 1117, Transportation Research Board, Washington D.C., pp. 1-90.

35. Lary, J. A., and Mahoney, J. P. (1984), "Seasonal Effect on the Strength of Pavement Structure," Transportation Research Record No. 954, Transportation Research Board, Washington D.C., pp. 88-94.

36. Loffi, H., and Witzcak, M. W. (1985), "Dynamic Characteristic of Cement Treated Base and Subbase Materials," Transportation Research Record No. 1031, Transportation Research Board, Washington D.C., pp. 41-48.

37. "Method of Test for Resilient Modulus of Cohesive Subgrade Soil," Research Report, University of Arkansas.

38. Montalvo, J. R., Bell, C. A., and Wilson, J. E. (1984), "Comparison of Diametral and Triaxial Repeated Load Testing Techniques for Untreated Soil," Transportation Research Record No. 948, Transportation Research Board, Washington D.C., pp. 6-13.

39. Monismith, C. L. (March 28-30, 1989), "Resilient Modulus Testing: Interpretation of Laboratory Results for Design Purposes," Workshop on Resilient Modulus Testing, Oregon State University, Corvallis, Oregon.

40. Monismith, C. L., and Bergan, A. T. (1973), "Characterization of Subgrade Soils in Cold Regions for Pavement Design Purposes," Highway Research Record 431, Highway Research Board, Washington D.C., pp. 25-37.

42. Motan, E. S., Edil, T. B. (1982), "Repetitive Load Behavior of Unsaturated Soils," Transportation Research Record No. 872, Transportation Research Board, Washington D.C., pp. 41-48.

43. Proceeding of the Workshop on Resilient Modulus Testing (September 1989), Report No. FHWA-TS-90-031, NTIS, Springfield, Virginia, p. 728.

44. Pumphrey, N. D. Jr., Lentz, R. W. (1986), "Deformation Analysis of Florida Highway Subgrade Sand Subjected to Repeated Load Triaxial Test," Transportation Research Record No. 1089, Transportation Research Board, Washington D.C., pp. 49-56.

45. Rada, G., and Witczak, M. W. (1981), "Comprehensive Evaluation of Laboratory Resilient Modulus Results of Granular Material," Transportation Research Record No. 810, Transportation Research Board, Washington D.C., pp. 23-33.

46. Rada, G., and Witczak, M. W. (1982), "Material Layer Coefficient of Unbound Granular Materials from Resilient Modulus," Transportation Research Record No. 852, Transportation Research Board, Washington D.C., pp. 15-21.

47. Schmidt, R. J. (19), "A Practical Method for Measuring the Resilient Modulus of Asphalt Treated Mix," Transportation Research Record No. 404, Transportation Research Board, Washington D.C., pp. 23-32.

48. Scholz, T, Hick, R. G., and Scholl, L. (September 1989), "Repeatability of Testing Procedures for Resilient Modulus and Fatigue: Final Report," NTIS, Springfield, Virginia, p. 47.

49. Seim, D. K. (March 1989), "A Comprehensive Study on the Resilient Modulus of Subgrade Soils," New York Department of Transportation, Albany, New York.

50. Snedden, R. V. (November 1988), "Resilient Modulus Testing of 14 Nebraska Soils," Nebraska Department of Roads, p. 70.

51. Sweeve, G. T., and Galjaard, P. J. (1988), "Repeated Static Loading Triaxial Tests for Determination of Resilient Properties of Sands," Transportation Research Record No. 1192, Transportation Research Board, Washington D.C., pp. 8-15.

52. Thompson, M. R. (March 28-30, 1989), "Factors Affecting the Resilient Modulus of Soils and Granular Materials," Workshop on Resilient Modulus Testing, Oregon State University.

53. Thompson, M. R., and Robnett, Q. L. (March 28-30, 1989), "Resilient Properties of Subgrade Soils," Workshop on Resilient Modulus Testing, Oregon State University, Corvallis, Oregon.

54. Thompson, M. R., and Robnett, Q. L. (1976), "Effect of Lime Treatment on the Resilient Behavior of Fine Grained Soil," Transportation Research Record No. 560, Transportation Research Board, Washington D.C., pp. 11-20.

55. Thornton, S. I., and Elliott, P. R. (1989), "Resilient Modulus - What is it," University of Arkansas, prepared for Workshop on Resilient Modulus Testing, Oregon State University, Corvallis, Oregon, pp. 267-282.

56. Thornton, S. I. (1983), "Correlation of Subgrade Reaction with CBR, HVEEM Stabilometer of Resilient Modulus," Arkansas State Highway and Transportation Department, p. 126.

57. Uzan, J. (1985), "Characterization of Granular Material," Transportation Research Record No. 1022, Transportation Research Board, Washington D.C., pp. 52-59.

58. Vinson, T. S. (March 28-30, 1989), "Fundamental of Resilient Modulus Testing," Workshop on Resilient Modulus Testing, Oregon State University, Corvallis, Oregon.

59. Xiu-Xian, L., and Zhi-Hong, X. (September 16-18, 1986), "New Development in the Structural Design of Flexible Pavements Bearing Capacity of Roads and Airfields," Proceeding of 2nd International Conference, Plymouth, England.

60. Yapa, K., and Lytton, R. L. (December 1988), "A Simplified Mechanist Rut Depth Prediction Procedure for Low Volume Roads: Interim Report, NTIS, p. 145.

61. Yeh, S. T., and Su, C. K. (December 1989), "Resilient Properties of Colorado Soil: Final Report," NTIS, p. 75.

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16. ABSTRACT

Resilient modulus (RM) is an important property of subgrade soils that accounts for repetitive loads due to vehicular traffic. Since AASHTO recommended its use in pavement design in 1986, various transportation agencies have devised procedures for testing and evaluation of RM. A comprehensive literature search was conducted in this study with two objectives in mind: (i) to obtain information on current practices pertaining to RM testing of subgrade soils; and (ii) to compile information pertaining to the collective experience of various agencies in correlating RM with other engineering soil properties.

Practices adopted by different transportation agencies in testing RM are not identical; some follow AASHTO guidelines, while others differ. The differences are centered around deviator stress, rate of loading, confining stress, moisture-density relationship, specimen preparation and stress sequence. The well known relationship between RM and CBR, proposed by AASHTO, does not correlate well for many soils. Efforts have been made by various researchers to correlate RM with other factors including clay, silt and organic carbon contents, plasticity index, liquid limit, group index, compressive strength, initial elastic modulus and confining pressure. Very limited efforts have been directed toward understanding the RM characteristics of bonded materials and aggregate bases.

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