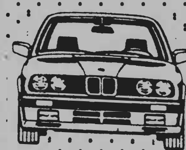
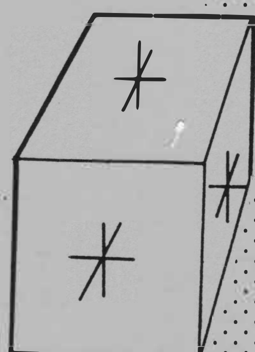
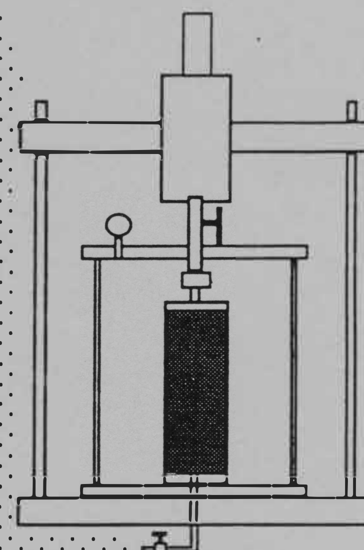
RESILIENT MODULUS OF SELECT AGGREGATE BASES AND THEIR CORRELATIONS WITH OTHER ENGINEERING PROPERTIES







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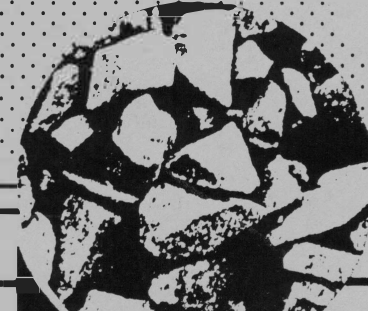

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16. ABSTRACT

Six most commonly encountered aggregate materials which are used as subbases/bases in Oklahoma are selected and tested under dynamic loading by using AASHTO designation T292-91I. A vibratory compaction method was successfully developed to prepare the 6 inch diameter and 12 inch long specimen at optimum moisture content. The gradation of the specimens met the ODOT 1988 specification Type A and Type B. Exploratory tests were carried out to assess the effect of varying gradations, compaction method, moisture content, specimen size, and testing procedures in the RM. Statistical correlations were established between RM and CBR, between RM and cohesion and friction angle, and between RM and E.

For a given gradation, the Resilient Modulus values of the six aggregate types at the same bulk stress are relatively close; the influence of gradation and compaction method on RM values were less significant compared to the effects of moisture content and the stress state; the T294-92I testing procedure gave higher resilient moduli than those obtained by using the T292-91I testing procedure; the RM values for 4" specimens were higher than those for 6" specimens; and the best correlations exist between the cohesion and friction angle and the RM values.

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DISCLAIMER

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SUMMARY

In order to improve the reliability of pavement design and enhance pavement performance, the American Association of State Highway and Transportation Officials (AASHTO) in 1986 proposed a new testing procedure to characterize primarily subgrade materials, and by extension aggregate pavement layers, that accounts for the repetitive load due to the moving vehicular traffic. The property that describes this behavior of subgrade materials is called the Resilient Modulus (RM). However, laboratory testing of RM is time consuming and requires special equipment. Therefore, it is desirable to establish relationships between RM and other index properties (namely, plasticity index, California bearing ratio (CBR), Elasticity (E), cohesion and friction angle) that are relatively easy and inexpensive to determine. This is also in line with the AASHTO proposal that agencies using the design guide establish such correlations.

In this study, and in cooperation with the Oklahoma Department of Transportation six most commonly encountered aggregate materials which are used as subbases/bases in Oklahoma are selected and tested under dynamic loading by using AASHTO designation T292-91I. A vibratory compaction method was successfully developed to prepare the 6" diameter (12" in length) aggregate specimen at optimum moisture content. The gradation of the specimens met the ODOT 1988 specification Type A and Type B. Exploratory tests were carried out to assess the effect of varying gradations, compaction method, moisture content, specimen size, and testing procedures in the RM. Statistical correlations were established between RM and CBR, between RM and cohesion and friction angle, and between RM and E.

For a given gradation, the Resilient Modulus values of the six aggregate types at the same bulk stress are relatively close; the influence of gradation and compaction method on RM values were less significant compared to the effects of moisture content and the stress state; the T294-92I testing procedure gave higher resilient moduli than those obtained by using the T292-91I testing procedure; the RM values for 4" specimens were higher than those for 6" specimens; and the best correlations exist between the cohesion and friction angle and the RM values.

CHAPTER 1

INTRODUCTION

The quality of pavement design is greatly dependent upon the accuracy and manner in which the material properties are evaluated and used in the analysis. Traditionally, the testing and evaluation of properties of aggregates and of aggregate layers have been conducted in a static manner that does not simulate the repetitive nature of the actual loads imposed by moving vehicular traffic (Laguros and Zaman <1>). Furthermore, the repeated load applications due to moving traffic induce repeated deformations that would cause cracking of pavement structure (Pezo et al. <2>). To correct this discrepancy and in order to improve the reliability of pavement design and enhance pavement performance, the American Association of State Highway and Transportation Officials (AASHTO) proposed a new testing procedure in 1986 <3>. The property that describes this behavior of materials is called the Resilient Modulus (RM) and defined as the deviatoric dynamic stress (due to the moving vehicular traffic) divided by the resilient (recoverable) strain. This holds true because the major component of deformation induced into a pavement structure under traffic loading is not associated with plastic deformation or permanent deformation, but with an elastic or resilient deformation (Robnett and Thompson <4>). Thus, the Resilient Modulus is considered to be a required input for determining the stress-strain characteristics of pavement structures subjected to traffic loading. However, laboratory testing of RM is time consuming and requires special equipment. Therefore, it is desirable to establish relationships between RM and other index properties (namely, cohesion, friction angle, elasticity and California bearing ratio (CBR)) that are relatively easy and inexpensive to determine.

Most of the previous studies have been concerned with subgrade cohesive soils and have not adequately addressed RM of subbase and base aggregates. Inasmuch as coarse and fine aggregates are used quite extensively in pavement construction in Oklahoma, their RM characteristics appropriately became the center of a study, whose additional objectives is to correlate the resilience response data to that of standard index properties such as cohesion, friction angle, elasticity and CBR. This is also in line with the AASHTO proposal that agencies using the design guide establish such correlations.

CHAPTER 2

LITERATURE REVIEW

The bulk of literature deals with soil materials. Nevertheless, the principles involved apply equally well to aggregate materials. Therefore, this chapter is written with that view in mind. A comprehensive literature search was conducted at the University of Oklahoma by Laguros and Zaman <1> which was submitted to the Oklahoma Department of Transportation (ODOT) in June 1991. Their study included: testing procedures previously developed and/or currently used by various agencies, and the correlation between RM and other engineering properties. This chapter also focuses on the testing parameters which may affect the RM results such as testing procedure used, confining pressure applied, the selection of dynamic wave form and the number of repetitions. In addition, the typical RM values established for granular materials and those suggested by various agencies are presented.

2.1 CONCEPT OF RESILIENT MODULUS

The successful selection of pavement thickness relies mainly on a proper characterization of the load-deformation responses of the pavement materials. When subgrade soils are subjected to repeated loads due to moving vehicle traffics, they undergo deformation. Laboratory results indicated that part of this deformation is resilient or recoverable (ε_r), while other is permanent or plastic (ε_p), as presented in Fig. 2-1. The property that describes this behavior of subgrade materials is called the Resilient Modulus (RM), defined as the deviatoric dynamic stress σ_d divided by resilient strain ε_r. Simply stated, RM represents a relationship between the applied stress (due to the moving vehicular traffic) and the elastic or resilient strain or

$$RM = \sigma_d / \varepsilon_r \tag{2-1}$$

2.2 TESTING PROCEDURE

Frequently, the Resilient Moduli for subgrade materials are determined either from cyclic triaxial tests or non-destructive pavement evaluation tests, the falling weight deflectometer test being a prime example. The non-destructive test is an easier way to obtain RM values, but it has a drawback in that the thickness of the layer needs to be precisely known in the back-calculation (Cosentino and Chen <5>). Further limitations for this method are : (1) relatively small loading magnitudes; (2) accessibility to construction site; (3) an already existing pavement structure; and (4) favorable weather (Pezo <2>). In contrast, the cyclic triaxial tests are performed under carefully controlled conditions. Most researchers agree that cyclic triaxial testing is more appropriate for design, while the field falling weight deflectometer tests are more appropriately used in the evaluation of existing pavement structures (Pezo <2>). The cyclic triaxial test may be costly and time consuming but is the most logical and is commonly used by researchers (Uzan <6>; Thompson and Smith <7>).

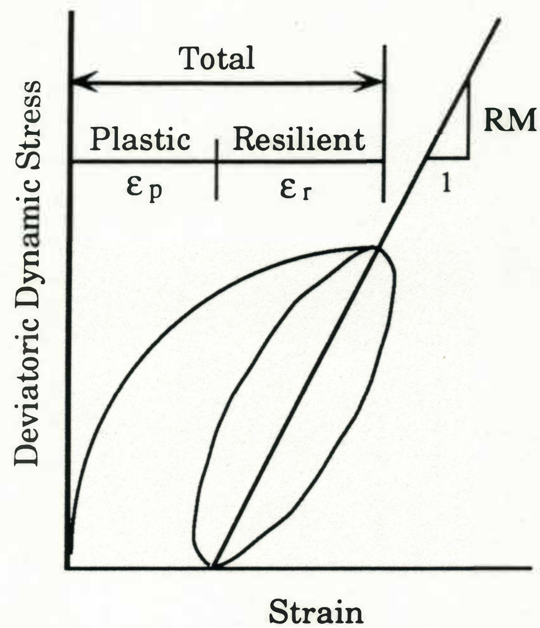

Fig. 2-1 Determination of the Resilient Modulus (RM)

The procedure for determination of RM has not yet been standardized, however, guidelines are given in AASHTO Test Method T274-82 <3>, T292-91I <8>, T294-92I <9> and Asphalt Institute <10>. Table 2-1 shows a comparison between T292-91I, T294-92I and Asphalt Institute testing procedures for granular soils. The basic differences among those testing procedures are particularly in terms of : (1) sample conditioning prior to testing; (2) number of loading cycles; and (3) applied stress magnitudes. The different testing procedures may result in different RM values and hence differences in the design of the pavement. Therefore, it is very important to investigate the effects on RM due to the different testing procedures.

Since its introduction, AASHTO T274-82 <3> has been the target of widespread criticisms. The main criticism for T274-82 is that the required testing procedure is too severe such that the specimen may fail in the conditioning stage. Consequently, researchers question the validity of and need for such an extensive process. For instance, Vinson <11> has documented his unsatisfactory experience with AASHTO T274-82, he reported that the T274-82 requires that all specimens be heavily conditioned prior to the actual test; by then, he argued the sample may have undergone a substantial variety of stress states for both cohesive and cohesionless soils. Also, Ho <12> stated that the conditioning stage, as suggested by T274-82, was very severe for many of their soils. For these reasons, various transportation departments have developed their own testing procedures such as Florida, New York, Illinois and South Dakota. Basically, the test procedures adopted by them are <u>similar</u> to AASHTO T274-82, except for some factors pertaining to sample condition, load magnitude and load application sequences.

Table 2-1 Comparison of Different Testing Procedures

AASHTO (T292-91I <8>) σ_C (psi)	σ_d (psi)	No. of Cycles	AASHTO (T294-92I <9>) σ_C (psi)	σ_d (psi)	No. of Cycles	Asphalt Institute <10> σ_C (psi)	σ_d (psi)	No. of Cycles
20*	15*	1000*	15*	15*	1000*	2*	3*	200*
20	10	50	3	3	100	2*	6*	200*
20	20	50	3	6	100	2*	9*	200*
20	30	50	3	9	100	2	6	200
20	40	50	5	5	100			
15	10	50	5	10	100			
15	20	50	5	15	100			
15	30	50	10	10	100			
15	40	50	10	20	100			
10	5	50	10	30	100			
10	10	50	15	10	100			
10	20	50	15	15	100			
10	30	50	15	30	100			
5	5	50	20	15	100			
5	10	50	20	20	100			
5	15	50	20	40	100			
3	5	50						
3	7	50						
3	9	50						

The notations used above as $\sigma_1 = \sigma_C + \sigma_d$

Where σ_1, σ_C, and σ_d are major principle stress, chamber confining pressure and deviator stress, respectively.

* The load sequence constitutes sample conditioning, that is, minimizing the effects of initially imperfect contact between the end platens and the test specimen.

2.3 CONFINING PRESSURE

It is easier to obtain RM values using the Asphalt Institute testing procedure because it requires only one confining pressure (2 psi); consequently, it gives only one RM value at the stated confining level. However, the RM response for the aggregate types considered is mainly dominated by the confining pressure applied (Rada and Witczak <13>). Thus, in order to better characterize granular materials, RM tests under a range of confining pressures expected within the subgrade or base/subbase are desirable. The AASHTO procedures (T274-82, T292-91I and T294-92I) uses a variety of confining pressures and deviatoric dynamic stresses; therefore, the data comprises a set of RM values corresponding to the state of bulk stress.

Khedr <14> argued that the constant confining pressure test, as the methods mentioned above, has the drawback of not simulating the in situ condition in which lateral pressure (confining pressure in this case) changes simultaneously with vertical pressure due to traffic load. Also, the constant confining pressure test instead of cycled confining pressure may over estimate RM values (Allen <15> and Khedr <14>). In contrast, Thompson <16> reported that for practical purposes, the triaxial resilient moduli are similar under both constant and variable confining pressures. Barksdale et al. <17> also reported that when a wheel load passes over an element of pavement structure, there is a simultaneous increase in both the major and minor principal stresses. However, only the variation in the major principal stress is considered essential in resilient modulus testing. Furthermore, it is important to know that part of the AASHTO testing procedures (T292-91I and T294-92I) requires bulk stresses θ (defined by $\theta = \sigma_1 + \sigma_2 + \sigma_3$) as high as 80-100 psi; these appear much higher than the stress prevailing in the field (Jackson <18>; Ho <12>). As reported by Thompson and Smith <7>, 20 psi is a representative bulk stress value in the mid-depth of the granular base (for a 3" AC surface and 12" granular base).

The same specimen can be used to measure the RM over a wide range of stress levels (Thompson <16>), and the stresses can be applied in any order, with the caveat that the repeated stress states are not greater than approximately 60% of the ultimate shear strength of the material.

2.4 DYNAMIC WAVE FORM AND NUMBER OF REPETITIONS

Seed and McNeill <19> made one of the earliest attempts to duplicate the stress-state history by considering the actual variation in vertical stress on a soil element at a depth of 27 inches below the surface of the pavement at the Stockton test track, shown in Fig. 2-2. They did not use the actual form of the vertical stress that was observed due to the limitations of their test equipment; rather, they chose a square wave (see Fig. 2-2). Terrel et al. <20> also studied the influence of the shape of wave pulse on the total and resilient strains induced in an asphalt treated base material. They found that either the triangular or the sinusoidal stress pulse produces similar effects on the resilience characteristics of the materials, and concluded that a square vertical stress pulse is a reasonable approximation of the actual conditions within a pavement structure.

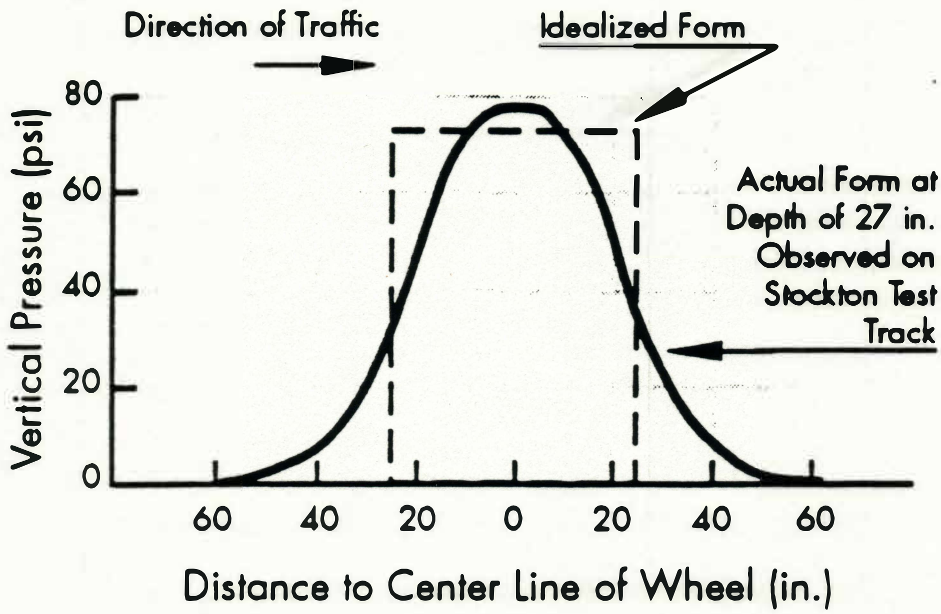

Fig. 2-2 Changes in Stress on Soil Element Caused by a Moving Load, as Shown by Seed and Mcneill <19>

To simulate the traffic load, AASHTO designation T292-91I suggested that the triangular and rectangular wave forms be applicable for RM testing of granular subgrade soils and base/subbase materials. A fixed cycle duration of between 0.1 and 1.0 seconds and a fixed cycle duration of between 1.0 and 3.0 seconds is specified by T292-91I. Further, for the granular specimen, a minimum 0.9 second period of relaxation between the end and the beginning of consecutive load repetitions is required by T292-91I.

To determine the number of repetitions necessary to reach the stable permanent deformation, AASHTO (T292-91I <8>) suggests comparing the recoverable axial deformation at the twentieth and fiftieth repetitions. If the difference is greater than 5%, an additional 50 repetitions are necessary at that stress state. Thompson <16> reported that for granular materials, the RM response after a limited number of load repetitions (100 or so) is representative of the response determined after several thousand repetitions because generally granular material will achieve a stable permanent deformation after 100 load repetitions. It has also been reported by Allen <15>, Hick <21> and Khedr <14> that the response of granular materials is fairly steady and stable after approximately 100 cycles of constant dynamic loading because the rate of permanent strain accumulation decreases logarithmically with the number of load repetitions.

Resilient Modulus is only minimally affected by variations in stress pulse duration. In fact , Kalcheff and Hicks <22> demonstrated that if the stress pulse is rapidly applied, and then sustained; the RM is the same as that obtained from a rapidly applied and released short duration stress pulse of the same magnitude.

2.5 TYPICAL RM VALUES

A comprehensive evaluation of nonlinear (stress-dependent) resilient modulus test results on granular material was conducted by Rada and Witczak <13>. A total of 271 test results obtained from 10 different research agencies were used in their study. They found that an expression appears to exist for all granular materials in the form of Eq. 2-2

$$RM = K_1 \theta^{K_2} \tag{2-2}$$

Six unique K_1 and K_2 relations were established for six different granular material types (silty sands, sand-gravel, sand-aggregate blend, crushed stone, lime rock, and slag). Their findings are summarized in Table 2-2.

May and Witczak <23> have established the relations for layer thickness and RM for sections of US-1, Interstate 695 and MD-97. The RM of the lower layer was determined in the laboratory by stress-dependent RM testing. The configuration of cross-section and their corresponding RM are presented in Fig. 2-3 and Table 2-3, respectively. Table 2-4 shows the typical values of K_1 and K_2 as suggested by the AASHTO design guide 1986 <24>. The New York DOT conducted an RM study on sand and the values obtained ranged from 6400-27,200 psi (Seim <25>)

Table 2-2 Summary of K_1 and K_2 for Aggregate Types (Rada and Witczak <13>)

	K_1			K_2		
Material	Mean	Standard Deviation	Range	Mean	Standard Deviation	Range
Silty Sands	1,620	780	710 to 3,830	.62	.13	.36 to .8
Sand-Gravel	4,480	4,300	860 to 12,840	.53	.17	.24 to .8
Sand-Aggregate Blend	4,350	2,630	1,880 to 11,070	.59	.13	.23 to .82
Crushed Stone	7,210	7,490	1,705 to 56,670	.45	.23	-.16 to.86
Limerock	14,030	10,240	5,700 to 83,860	.4	.11	.0 to .54
Slag	24,250	19,910	9,300 to 92,360	.37	.13	.0 to .52
All	9,240	11,225	710 to 92,360	.52	.17	-.16 to.86

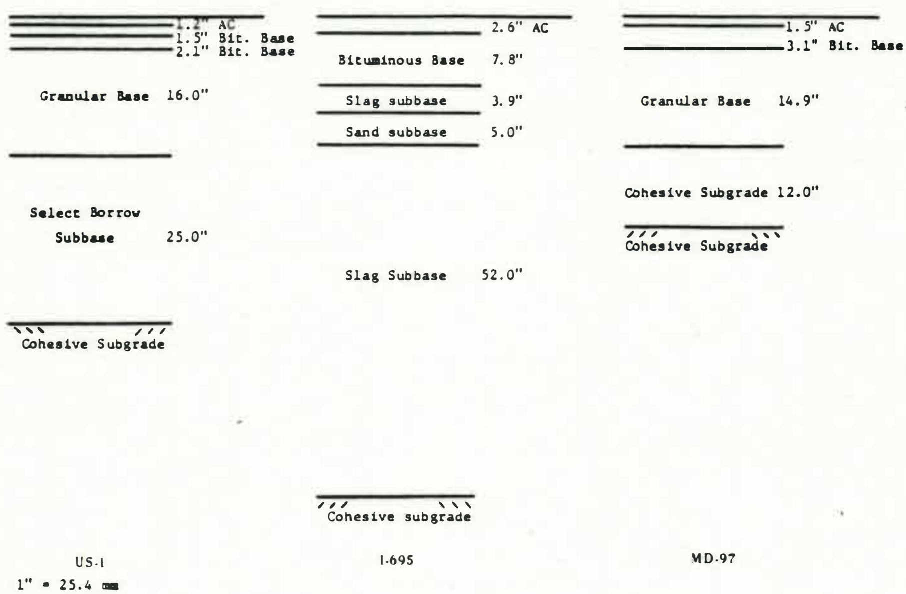

Fig. 2-3 Average Cross-Sections of Maryland Pavement (May and Witczak <13>)

Table 2-3 Resilient Modulus Relationships from Laboratory Testing of Lower Layer Materials (May and Witczak <23>)

Pavement Section	Layer	K_1**	K_2**
US-1	Granular	4,886	.239
	Subbase	2,632	.426
	Subgrade	5,796	-.696
I-695	Slag subbase 1	3,378	.520
	Sand subbase	3,683	.517
	Slag subbase	4,856	.487
	Subgrade	25,929	-.0309
MD-97	Granular Base	8,787	.365
	Subgrade 1	16,333	-0.345
	Subgrade 2	13,035	-0.180

** $RM = K_1 \theta^{K_2}$

Table 2-4 Typical Values for K_1 and K_2 for Unbound and Subbase Materials (AASHTO Guide 1986 <24>)

Layer	Moisture Condition	K_1	K_2
Base			
	Dry	6,000-10,000	.5-.7
	Damp	4,000-6,000	.5-.7
	Wet	2,000-4,000	.5-.7
Subbase			
	Dry	6,000-8,000	.4-.6
	Damp	4,000-6,000	.4-.6
	Wet	1,500-4,000	.4-.66

CHAPTER 3

SPECIMEN PREPARATION AND RESILIENT MODULUS TESTING

3.1 MATERIAL ORIGIN AND THEIR ENGINEERING INDEX

Six most commonly encountered aggregate bases in Oklahoma were selected in cooperation with the Oklahoma Department of Transportation (ODOT). The six types of aggregates include three limestones, one sandstone, one granite and one rhyolite. Table 3-1 shows the rock types, quarry names, county legal descriptions and geologic formations of the aggregate base samples (Hixon <31>). The locations of these quarries are presented in Fig. 3-1. It may be noted that the symbols in the Table 3-1 and Fig. 3-1 will be used in the following chapters to represent the aggregate origin. For example, rs, ark, qupa, bor, mer, wr are used to represent the aggregates from Comanche, Cherokee, Creek, Choctaw, Johnston, Murray Counties, respectively. Fig. 3-2 shows the open-face 125 feet thick sandstone (Choctaw County) after blasted by the dynamite and transported by a truck to the crusher. Figs. 3-3 and 3-4 show the stock-pile with mixing fine particle and without mixing fine particle, respectively. Figs. 3-5 and 3-6 show the sampling of type A aggregate from Choctaw and Murray Counties, respectively.

The engineering properties (liquid limit, plasticity index, maximum dry density (MDD), optimum moisture content (OMC), specific gravity (SG),

1 Comanche County (rs) Limestone
2 Cherokee County (ark) Limestone
3 Creek County (qupa) Limestone
4 Choctaw County (bor) Sandstone
5 Johnston County (mer) Granite
6 Murray County (wr) Rhyolite

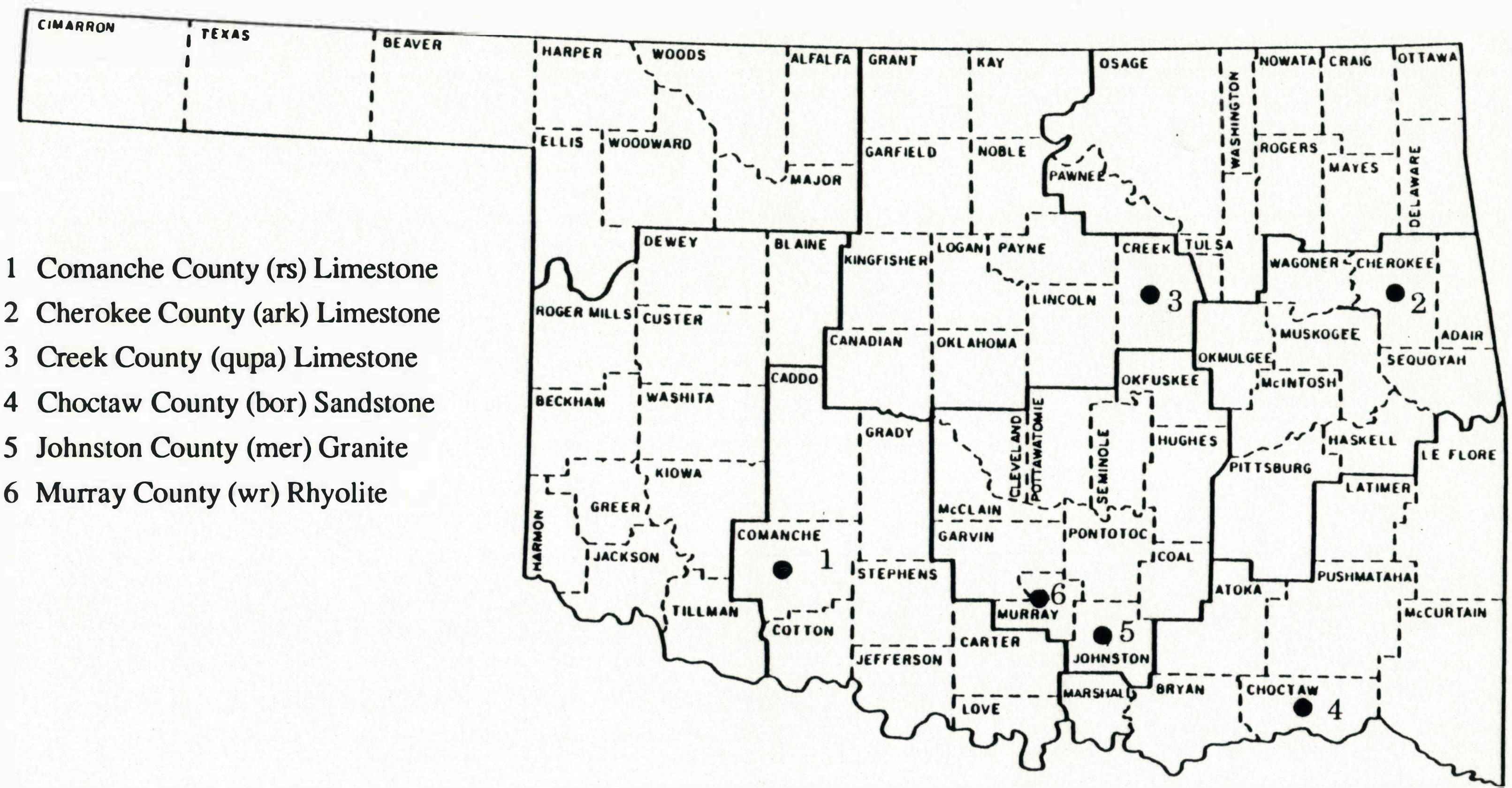

Fig. 3-1 Sources of Aggregate Base/Subbase Materials

Fig. 3-2 Open-Face 125 feet Thick Sandstone (Choctaw County)

Fig. 3-3 Stock-Pile With Mixing Fine Particles (Choctaw County)

Fig. 3-4 Stock-Pile Without Mixing Fine Particles (Choctaw County)

Fig. 3-5 Photographic View of Sampling Type A Aggregate (Choctaw County)

Fig. 3-6 Photographic View of Sampling Type A Aggregate (Murray County)

Table 3-1 Aggregate Sources (Hixon <31>)

Material	County	Company	Location	Formation
Limestone	Comanche (rs)	Dolese at Richard Spur	Sec. 31, T 4N, R 11W	Kindblade
Limestone	Cherokee (ark)	Arkhola at Zeb	N1/2, Sec.11,T15N,R 21E	Pitkin
Limestone	Creek (qupa)	Quapaw	Sec. 35&36, T 18N, R 7E	Pawhuska
Sandstone	Choctaw (bor)	American Rock Inc.	Sec. 13, T 5S, R 18E	Jackfork
Granite	Johnston (mer)	Meridian at Mill Creek	Sec. 20, T 2S, R 5E	Troy Granite
Rhyolite	Murray (wr)	Western Rock	Sec. 16, T 1S, R 1W	Colbert Porphyry

cohesion, friction angle and CBR) for these types of aggregates are evaluated. Most of these tests are repeated at least twice to insure the reproducibility of test data/results. The summary of the test results is presented in Table 3-2. The convention triaxial compression tests are performed to obtain cohesion and friction angle of the materials (aggregates) and the details are given in the Sec. 3.6. The CBR testings and results are presented in the Chapter 4.

As specified in the ODOT Standard Specifications for Highway Construction <26>, the aggregate base material passing the No. 40 sieve shall conform to the following :

(1) Plasticity index shall not exceed 6;

(2) Liquid limit shall not exceed 25; and

(3) The blending of separate aggregates will be permitted to produce an aggregate mixture meeting the above requirements providing no individual aggregate has a plasticity index in excess of 8.

For all six aggregate types, the LL and PI (as presented in the Table 3-2) meet the above requirements. The AASHTO designation T180-90D <27> is used to determine the MDD and OMC, as specified by ODOT specification 1988 <26>. It can be observed from Table 3-2 that the values for maximum density are slightly higher than the values reported by Coffman et al. <28> (range from 136 to 140 pcf) and Hicks <21> (range from 137 to 144.8 pcf) because in this study the specimens are prepared at a gradation (gradation II, in Table 3-4) that almost reaches the optimum, as shown in Fig. 3-7.

Table 3-2 Summary of Index Properties

County	Material	LL (%)	PI	Maximum Dry density (pcf)	Optimum Moisture (%)	SG
Comanche (rs)	Limestone	16	1	150	5.6	2.66
Cherokee (ark)	Limestone	16	1	149	5.2	2.64
Creek (qupa)	Limestone	15	NP*	151	5.5	2.78
Choctaw (bor)	Sandstone	14	NP*	147	5.9	2.53
Johnston (mer)	Granite	15	NP*	146	5.4	2.62
Murray (wr)	Rhyolite	16	NP*	150	6.0	2.72

* NP denotes nonplastic material

3.2 SIEVE ANALYSIS AND GRADATION ADOPTED

After collecting the aggregate sample from the quarry, the aggregate particles are first oven dried for two days and then a sieve analysis is performed. Table 3-3 shows the results from the sieve analysis for these six aggregate types. The minimum weight of test sample for nominal maximum size square opening 1.5 inch is 33 lb or 12 kg, as suggested by the AASHTO designation T27-84 <30>. It is observed from Table 3-3 that the gradation varies from type to type. In order to meet the ODOT 1988 specification <26> and to ensure the same gradation for each specimen among types, a gradation curve is desirable to be selected for specimen preparation. Also, gradation of aggregate materials can be an important factor when comparing RM values. The selected gradation curves employed in this study and the gradation required by ODOT <26> are presented in the Table 3-4 and Figs. 3-7 and 3-8. It may be noted that the gradation for the 4 inch specimen is slightly different than that of the 6 inch specimen because the larger diameter specimens can accommodate bigger particles. Also, in order to consider the percentage of coarse materials in the field, the gradation for 4 inch specimen is obtained by modifying the gradation requirement for 6" specimen. This is achieved by increasing the percentage of particles retained on sieve No's. 1/2 inch and 3/8 inch. In Table 3-4 and Figs. 3-7 and 3-8, the gradation curves I, II, III are used for the 4 inch, 6 inch (type A), and 6 inch (type B), respectively.

Table 3-3 Sieve Analysis of the Aggregate

Passing % Sieve Size	Type of Aggregate: Comanche	Cherokee	Creek	Choctaw	Johnston	Murray
1.5"	100	100		100	100	100
1"	75.6	94.6	100	89.2	92.2	88.6
3/4"	54.1	85.1	96.3	72.3	79.8	68.4
1/2"	41.3	68.5	85.8	54.6	65.5	35.6
3/8"	36.5	56.3	48.0	42.7	57.6	23.3
#4	30.3	36	26.7	27.7	43.0	4.4
#40	2.8	5.4	12.2	11.6	10.1	0.8
#200	0.5	0.9	3.6	1.6	3.4	0.2

Table 3-4 Gradations Required by the Oklahoma Department of Transportation and Those Used in the Present Study

	Percent Passing				
	ODOT		Presently Used		
Passing % Sieve Size	Type A	Type B	Gradation (I)	Gradation (II)	Gradation (III)
1.5"	100	40-100			100
1"				100	83
3/4"	40-100	30-75	100	82	67
1/2"			86	68	55
3/8"	30-75	25-60	70	55	44
#4	25-60	20-50	47	44	31
#40	8-26	7-22	21	17	12
#200	4-12	3-10	6	6	4

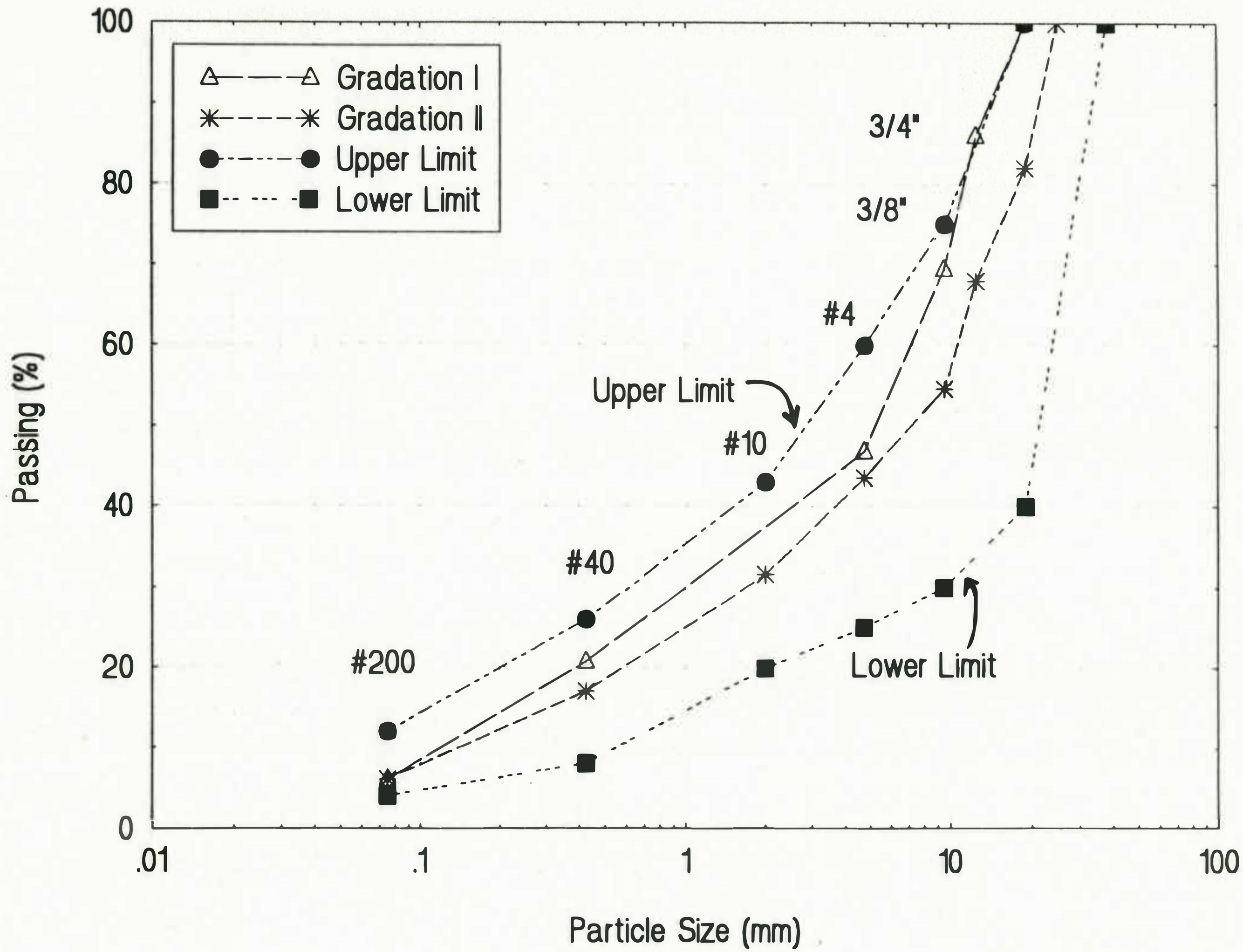

Fig. 3-7 ODOT Requirement and Those Used in this Study (Type A)

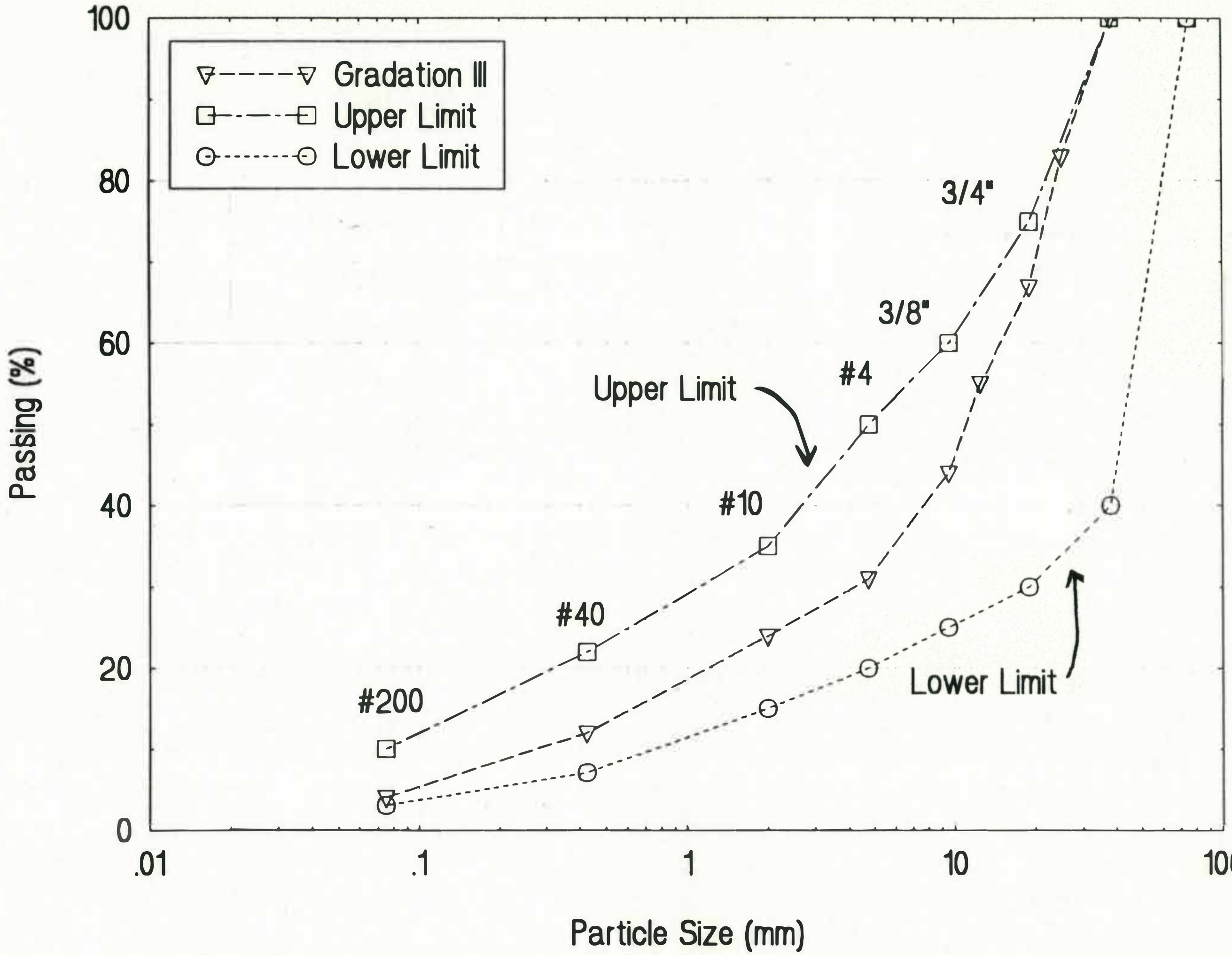

Fig. 3-8 ODOT Requirement and Those Used in this Study (Type B)

3.3 TRIAXIAL SPECIMEN PREPARATION

While method of compaction is important for fine-grained soils because of soil structure considerations, the primary factor affecting the stiffness characteristics of granular materials is water content (degree of saturation). Accordingly, any method of compaction which produces the desired dry density is suitable. Vibratory compaction has, for example, been used successfully by Hick <21>. ODOT specification <26> and AASHTO designation (T294-92I <9>) suggest that one find the OMC and MDD for the given aggregate types by using T180-90D; then, use the OMC and 95 % of the MDD for specimen preparation. The sample dry density and MC should not differ by more than 3% of the in situ dry density and 1% point of the in situ MC (T294-92I <9>).

It has been observed that using the energy level (55,986 ft-lb/ft^3) specified in the AASHTO designation T180-90D <27> as the compaction method to prepare the aggregate specimens for RM testing causes the breakage of particles. For example, the 1/2 inch particle is reduced in size by an average of 19%; as much as 23% reduction in particle having 3/8 inch size is found to occur due to compaction. A more recent AASHTO publication, the interim method of test for RM of unbound granular base/subbase materials (T294-92I and T292-91I) suggests that for the granular-type soil it is desirable to use a vibratory compaction method to prevent the breakage of particles. Exploratory tests conducted in this study indicate that by using vibratory compaction the maximum dry density values are reduced by 9.4% for 4 inch diameter specimen (9.25 inch in length) compared to those obtained by using T180-90D <27> which is required by the Oklahoma Department of Transportation (ODOT) specifications <26>. However, for the 6 inch specimens (12 inch in length) vibratory compaction method and the T180-90D <27> give nearly the same density values. The effects of changes in maximum dry density do not affect RM values significantly as compared to changes caused by stress level and moisture content. Soil structure effects on RM are generally unimportant for the granular type soils as compared to effects due to a change in moisture content and confining pressure; this is well documented in the literature (Hicks <21>; Rada and Witczak <13>). Thus, it might be advisable to use vibratory compaction as the method to prepare the RM specimens, particularly for the 6 inch specimens because the densities are very close for both compaction methods.

A split mold with provisions to apply a desired amount of vacuum, so as to fit the membrane tightly with the inner surface of the mold, was designed and fabricated in this study. The mold is found to be very useful in providing stability to the specimen and in transporting the specimen to the loading frame with minimum disturbance. The sample preparation equipment consists of the following components : (1) vibrating table; (2) vacuum pump; (3) sample mold; and (4) other accessories. The vibrating table is made up of a 30 inch * 30 inch square steel plate with a thickness of 0.25 inch. The plate rests on four steel springs so as to ensure uniform vibrations provided by an electromagnetic vibrator. The whole assembly rests on four- 1.5 inch * 1.5 inch * 0.25 inch angle iron legs for stability as shown in Fig. 3-9. The vibration of the table is controlled by a controller with a maximum vibrating speed of 3600 vibrations per minute (VPM). The sample mold was fabricated from a steel pipe and was cut into two equal halves (as presented in Fig. 3-10) to enable disassembling the mold under vacuum. This feature is essential for granular type samples. The internal diameter of the completed molds are 4 inch and 6 inch, having a wall thickness of approximately 0.25 inch. The length of the completed sample is 9.25 inches (for 4 inch diameter) and 12 inches (for 6 inch diameter). The base of the molds are firmly bolted onto the vibrating table to avoid movement of the mold during vibration, as shown in Fig. 3-9. A vacuum pump is used to provide the required suction to stretch the membrane around the wall of the mold so as to aid in the compaction of the specimen. Also, the vacuum pump provides stability to the specimen while transferring it from the mold to the triaxial cell of the RM testing apparatus.

The compaction method developed essentially involved a trial-and error adjustment in the weight of aggregate per layer, the number of compacted layers, and the vibrating period for each layer to produce specimens of the required densities. The specimens are prepared in ten layers of approximately 1600 grams of aggregate per layer. A steel rod is used to enhance the effectiveness of compaction. The vibrating time is approximately 30 seconds per layers for the first 8 layers and 4 minutes per layer for the last 2 layers. It was observed that the method mentioned above gives more uniform specimens than the specimens prepared at equal vibrating times for each layer in which the bottom layer is more dense due to vibrating times accumulating from bottom to top.

Fig. 3-11 shows a photograph view of one of the sample preparation steps involving vibration of the mold and compaction of the specimen in layers. Fig. 3-10 shows the completed specimen after being extracted from the split-mold.

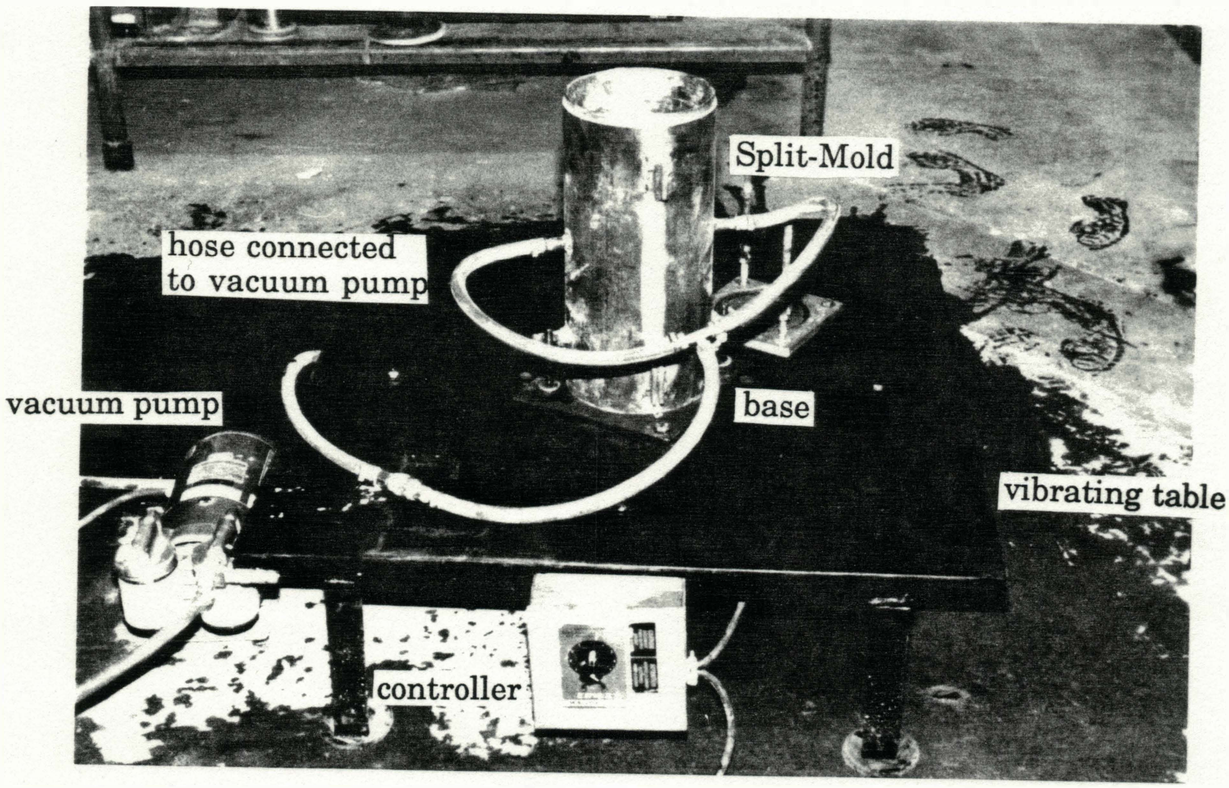

Fig. 3-9 Apparatus for the Resilient Modulus Specimens Preparation

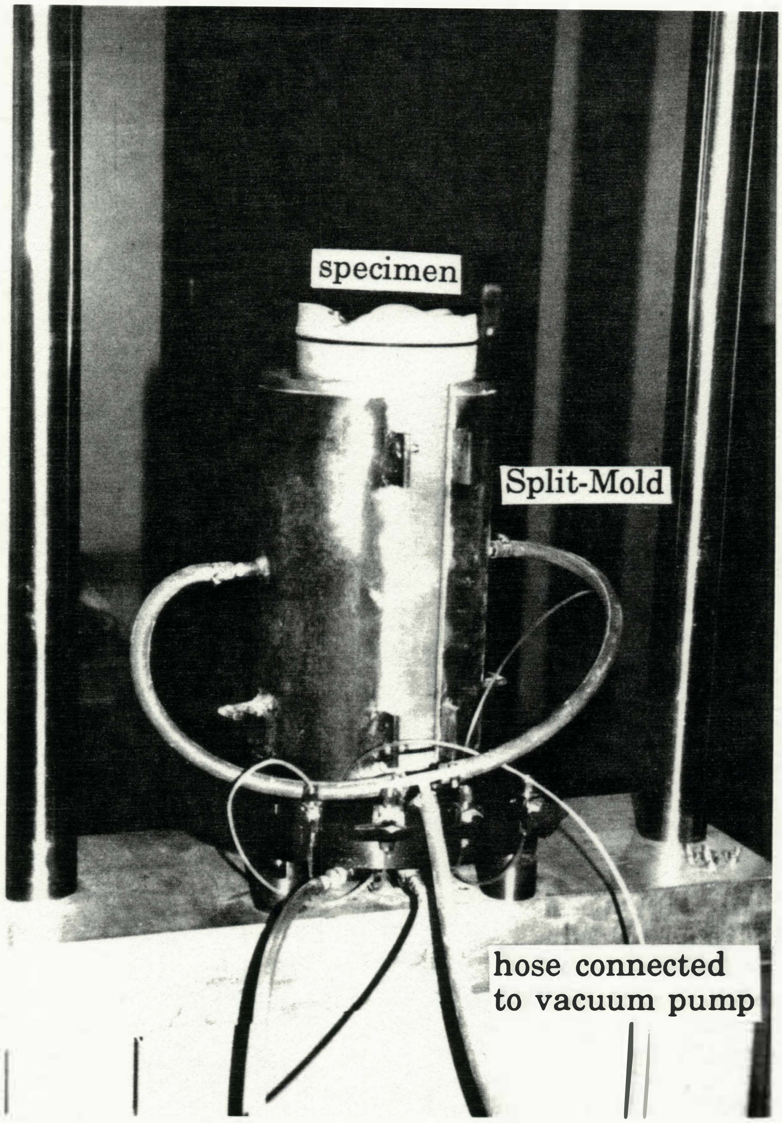

Fig. 3-10 Completed Specimen Ready to be Extracted from the Split-Mold

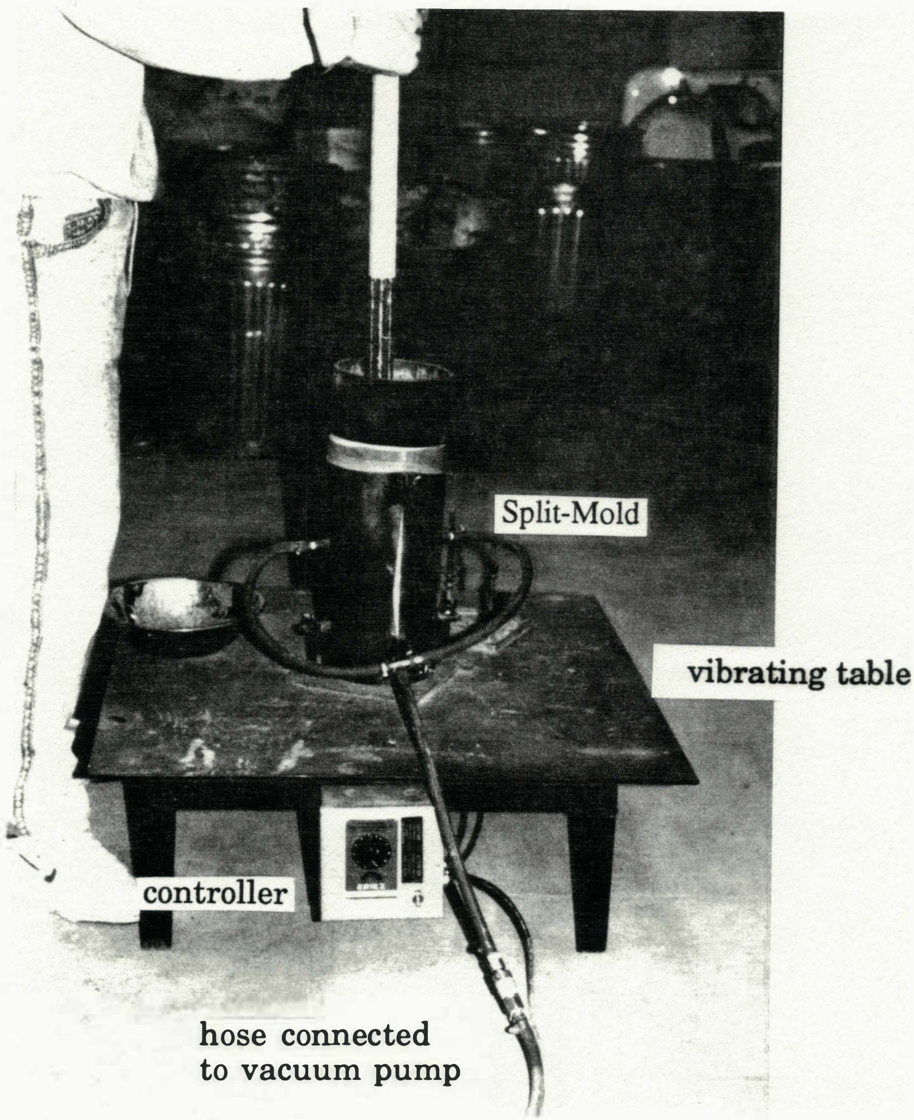

Fig. 3-11 Specimen Preparation by Using Assembled Split-Mold and Vibrating Table

3.4 EQUIPMENT AND ITS SETUP FOR RM TESTING

The load frame, triaxial cell, pressure gauge, load cell, and the overall set-up used in this study are shown in Fig. 3-12. The MTS 458.20 MicroConsole and servo-controller, along with the Microprofiler, provides an excellent facility to apply various types of cyclic loading (haversine, rectangular, triangular, etc.) in a very efficient and accurate manner. A 5 kip capacity load cell mounted inside the triaxial chamber and attached to the loading piston is used to monitor the actual deviatoric force. The 458.20 MicroConsole can use this 5 kips load cell either as a 5 kip or as a 0.5 kip load cell by simply changing the cartridge. The Microprofiler is programmed to conduct a test under desired loading. The shape and the amplitude of the cyclic loading waveform are continuously monitored by an oscilloscope. A data acquisition system was developed to record the signals emitted by the transducers. A data acquisition board, DT2801 (from Data Translation Inc.) was mounted inside an Zenix computer. This computer was used to host the data acquisition board, which converts the analog signal to digital data for all the transducers. Thus, the test data (load and displacement) is electronically collected and stored by this computer. Tests can be conducted either in a stress-controlled mode or a strain-controlled mode. The air pressure is used as the confining medium instead of water because the latter may get into the specimen even through tiny leaks or breakage of the membrane. Also, because transducers are located inside the triaxial chamber and air pressure is easy to operate and available in most laboratories. Therefore, air is used as the cell fluid to provide confinement to the test sample. An air pressure gauge was installed onto the triaxial cell to measure the confining pressure, as shown in Fig. 3-12. The

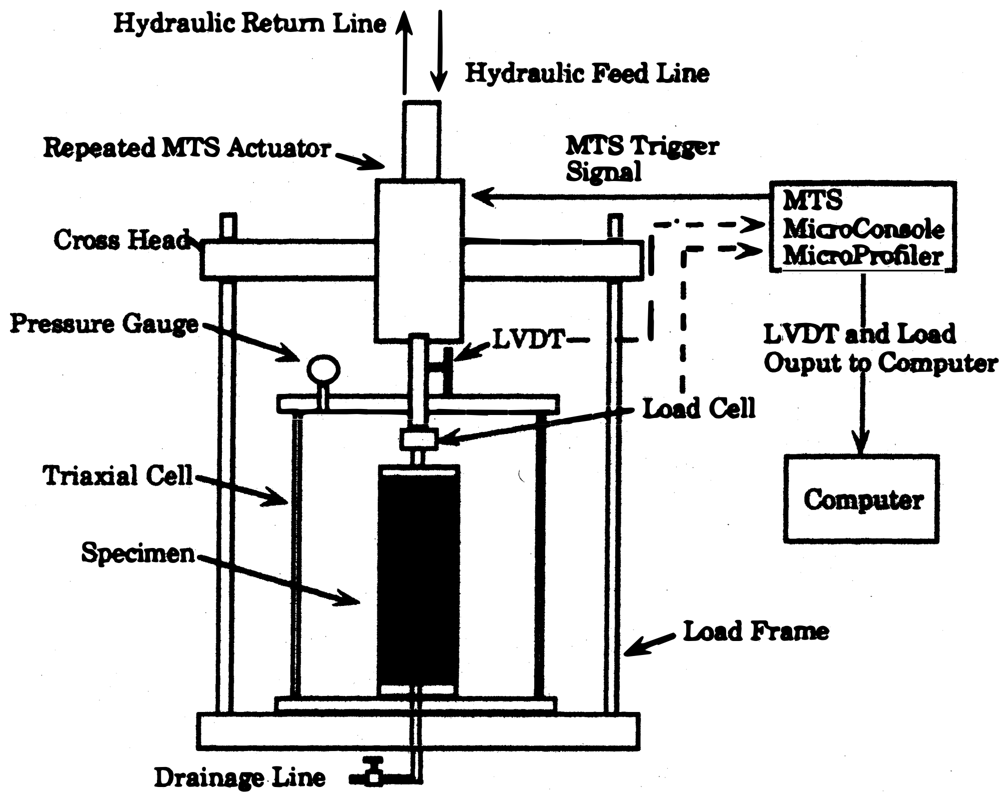

Fig 3-12 Flow Diagram of the Test Set Up for Resilient Modulus Testing

advantages of the referenced system is that the load cell is housed within the triaxial cell to allow in-vessel load measurement and to overcome the detrimental effects of friction due to the push rod. Also, the quality of test results is generally improved by monitoring the in-vessel load and confining pressures.

Since the internal LVDT's clamped to the test specimen increases the variation of results due to the sample having an outer membrane that can slip. Thus, in this study, the external LVDT setup has been selected. However, it may be noted that using an external LVDT is assuming that the movement of piston represents the axial deformation experienced by the test specimen.

A fixed cycle duration of 1.8 seconds is selected in this study to provide a 0.6 second loading duration and 1.2 seconds relaxation between the end and beginning of consecutive load repetitions. An oscilloscope is used to monitor the applied cyclic loading so as to achieve the desired rectangular wave form by adjusting the gain controller in the MicroConsole.

3.5 TESTING PROGRAM

The parameters involved in this study and the testing program are presented in the Table 3-5. A total of seven major exploratory tests (A,B,C,D,E,F,G) including : different testing procedures (AASHTO designations T292-91I and T294-92I), size of specimens (4 inch and 6 inch diameter), compaction method (hammer vs. vibratory table), moisture content and gradations (I, II and III) were conducted. The aggregate samples which came from the Quapaw Company, located in Creek County, Oklahoma have

Table 3-5 Test Program Used in this Study

	Type of Tests						
	A	B	C	D	E	F	G
Testing Procedures	T292-91I	T292-91I	T292-91I	T292-91I	T292-91I	T292-91I	T294-92I
Compaction	T180-90D	T180-90D	Vib	Vib	Vib	Vib	Vib
Size of Specimen (inch)	4	6	4	6	6	6	6
Gradation Curve	I	II	I	II	I	III	II
Average Density (pcf)	152.6	151.5	143.8	150.8	144.2	147.3	150.8

been investigated in great detail, including test types A- F and saturated tests under type D test conditions. For all six aggregate types (Comanche, Cherokee, Creek, Choctaw, Johnston, and Murray Counties), at least three RM tests were performed under type D test conditions. The aggregate type having the most consistent results on test type D (lowest standard deviation) is selected for type G tests. The details of gradations I, II and III are given in Table 3-4 and Figs. 3-7 and 3-8. The effects of specimen size and gradation, degree of saturation, aggregate types, and testing procedures on RM values are presented in the following sections.

3.5.1 Specimen Size and Gradation

The most commonly used specimen sizes for RM testing are 4 inch and 6 inch diameter samples. Since the 6 inch specimen can accommodate larger size particles, it is preferred for aggregate type material. The RM values for the 6 inch specimen are more realistic from the gradation and particle size considerations in the field, but on the other hand it is easier to prepare and conduct tests on the 4 inch specimen. For the same level of compaction energy applied as stated in T180-90D, the 4 inch specimen has slightly higher average dry densities (in Table 3-5 column 2) than those determined from 6 inch specimen (in Table 3-5 column 3) because of gradation differences between the two sizes.

In Fig. 3-13, it is observed that for a given bulk stress level, the RM values for 6 inch specimens (tests type B and D) are usually lower than those for the 4 inch specimen (test types A and C). At low stress levels (less than 20 psi), however, the differences are rather small. The higher RM values for 4 inch

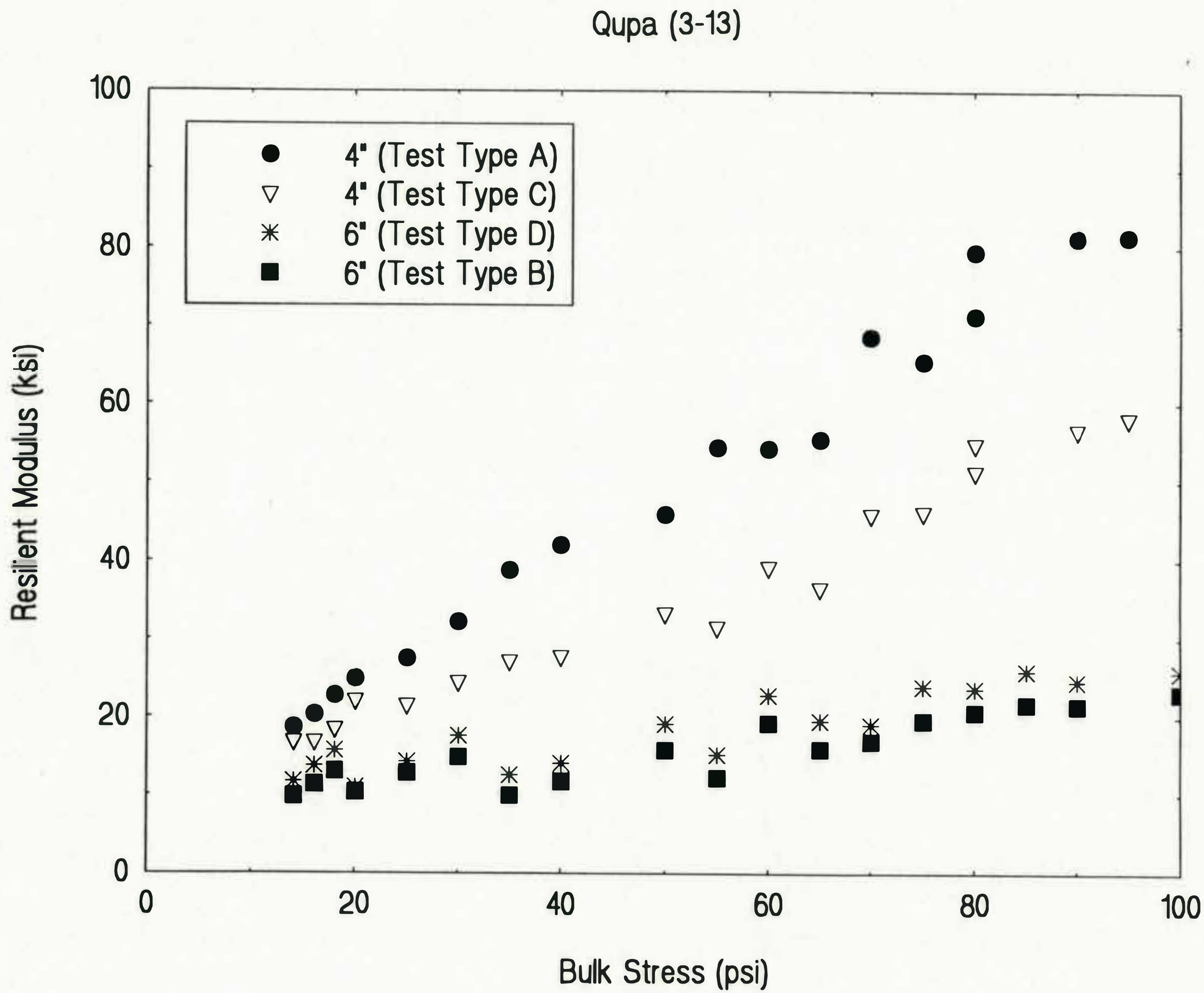

Fig. 3-13 Effects of the Compaction on the Resilient Moduli for 4" and 6" Specimens for the Aggregate from Creek County (Limestone)

specimen may be attributed to the different gradations used for different sizes of specimens. Indeed, the RM values are increased by using gradation I for the 6 inch specimen (test type E) compared to those using gradation II for the same specimen type (test type D), as shown in Fig. 3-14. By using the vibratory compaction method, the 4 inch specimen always have lower dry densities, compared to those of 6 inch specimen at the same level of moisture content. However, if gradation I for 6 inch specimen is used, the dry density for 6 inch specimen becomes similar to those for 4 inch specimen (average being approximately 143.8 pcf). Therefore, dry density is dominated by the gradation used, rather than the specimen size. For same specimen size (4 inch), the specimens have higher RM values than those prepared by the vibratory compaction method as observed from Fig. 3-13. This is due not only to the specimens having higher densities, but also to the residual compressive stress developed from compaction. Uzan <6> found that a residual stress of 1 to 2 psi may develop due to compaction. However, owing to the similar densities, the effect of compaction on RM is minimum for the 6 inch specimens, as shown in Fig. 3-13 (test types B and D).

In all cases, the 4 inch specimen has higher RM values than those for 6 inch specimen. It is believed that the preparation of specimens simulates natural geological materials (such as coal and rock) which exist in the field. As an analogy, since a smaller natural geological specimen (e.g., coal or rock) contains fewer defects and discontinuities, it exhibits higher strength (Evans et. al. <32>, Peng <33>)

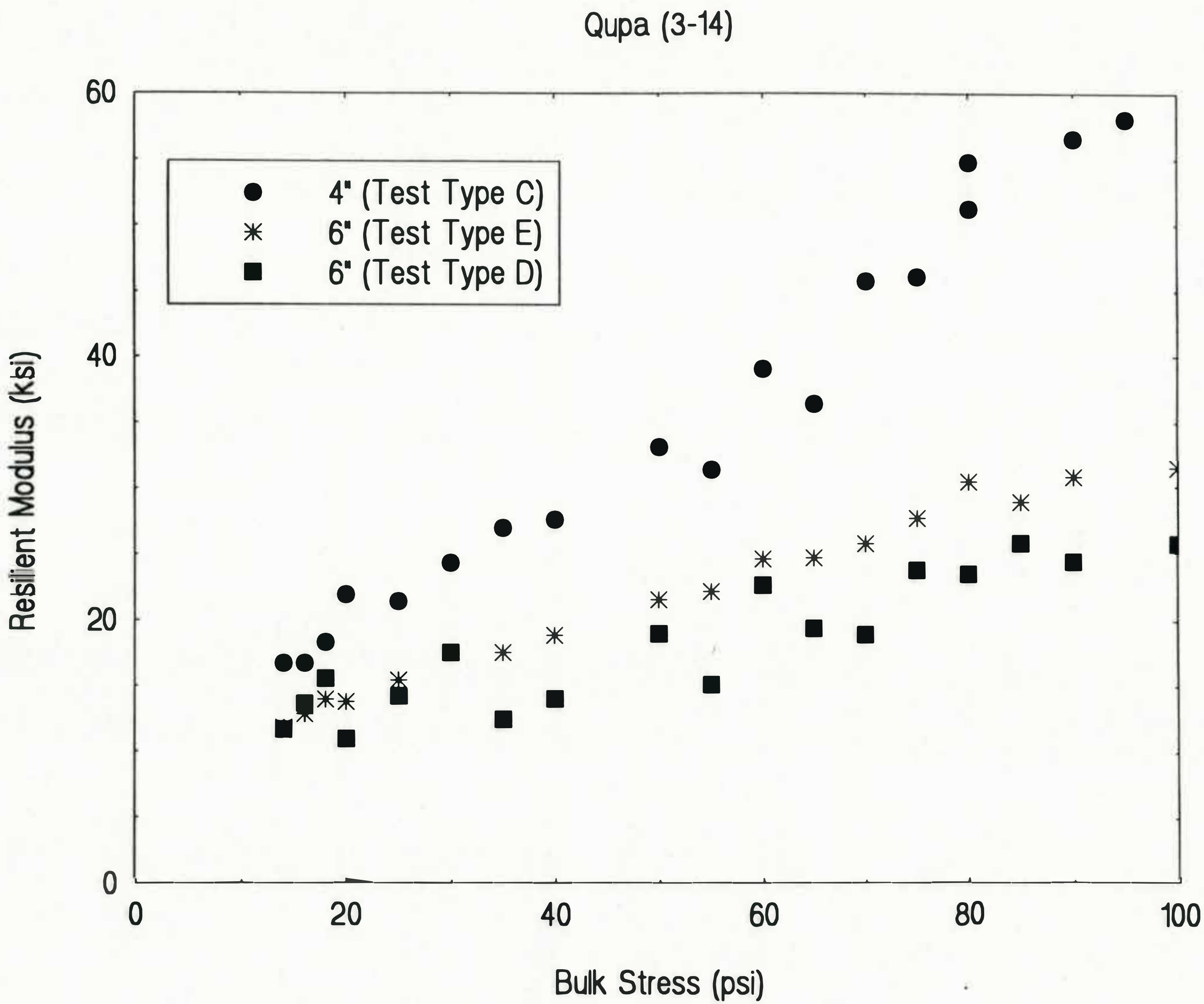

Fig. 3-14 Effects of the Gradation (I and II) on the Resilient Moduli for the Aggregate from Creek County (Limestone)

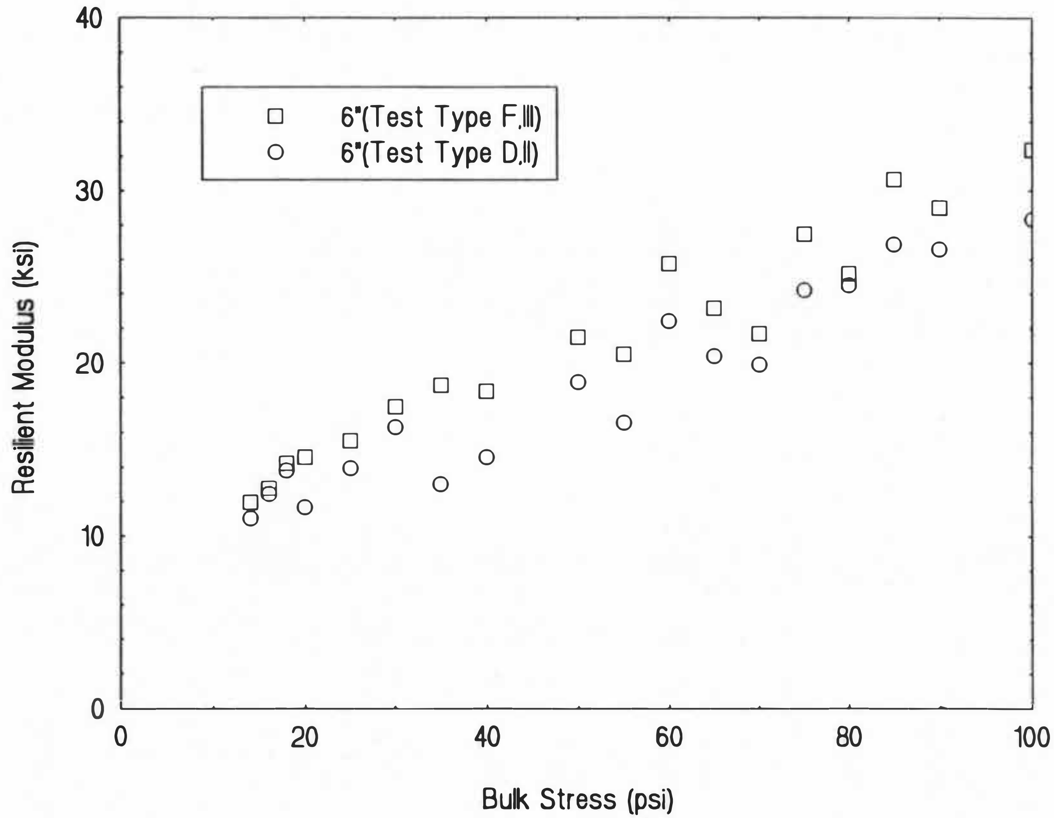

Fig. 3-15 Effects of the Gradation (II and III) on the Resilient Moduli for the Aggregate from Creek County (Limestone)

The comparison of RM values for gradations II and III (test types D and F, respectively) is presented in Fig. 3-15. As observed from the Fig. 3-15, the gradation III produces a slightly higher RM values than those for gradation II. However, the effect on RM due to gradation is less significant (< 10%).

3.5.2 Degree of Saturation

In order to simulate the wet season in the field, the specimens are prepared at optimum moisture content and maximum dry density and then immersed into a water tank for 7 days. The vibratory compaction method is used to prepare both 4 inch and 6 inch specimens. It is found that soaking compacted specimens are more realistic in terms of simulation of actual field conditions than preparing the specimens at moisture contents higher than the optimum. All RM tests in this study are conducted under drained conditions. Hicks <21> performed experiments under undrained conditions; static and transient pore pressures were measured throughout the tests. As the number of repeated loads increased, pore water pressure developed and weakened the specimen. An attempt is made in this study to investigate the possibility of conducting RM tests under undrained conditions but specimens failed during the conditioning stage (around 250-300 cycles) due to the development of excess pore pressure resulting from cyclic loading. It may be noted that this condition probably does not occur in a pavement, but it indicates the propensity of a reduction in the modulus when the pavement is saturated (Hicks <21>; Das <34>). Also it indicates that under the field conditions, granular soils are likely to undergo drainage if the rate of load application is moderate.

Fig. 3-16 shows that for both 4 inch and 6 inch soaked specimens the RM values decrease, as expected. The 4 inch specimen experienced a higher degree of strength loss due to soaking. It might be due to the degree of saturation being higher for 4 inch specimen than those for 6 inch specimen. The 4 inch specimen has a lower density and have more free void spaces than those for 6" specimen; this is confirmed by experimental observation (4" specimen increase by weight 1.9 %, while 6" specimen increase only 0.4 % due to soaking). In view of Fig. 3-16, the difference of the RM values for soaked and non-soaked specimens at the lower bulk stress is rather small compared to those at the higher bulk stress. One of the reasons may be attributed to the fact that the RM tests are performed under drained conditions; that is, the moisture content of soaked specimens at the lower bulk stress levels will be similar to the non-soaked specimens because the increased free water during soaking is drained out. It may be noted that the lower bulk stresses are applied at the last stage of the AASHTO testing procedure. Also, moisture content may be approximately the same due to the long duration of the test and the confining pressures applied.

3.5.3 Aggregate Types

The 6" specimen can accommodate larger size particles which are more realistic, from the gradation considerations in the field. Also, the compaction method used has minimum effects on RM for 6" specimen as evident from Sec. 3.5.1 and reported by Rada and Witczak <13>. Therefore, the test type D condition (in Table 3-5) are selected to investigate effects on RM values due to different aggregate types. The summary of regression constants K_1 and K_2

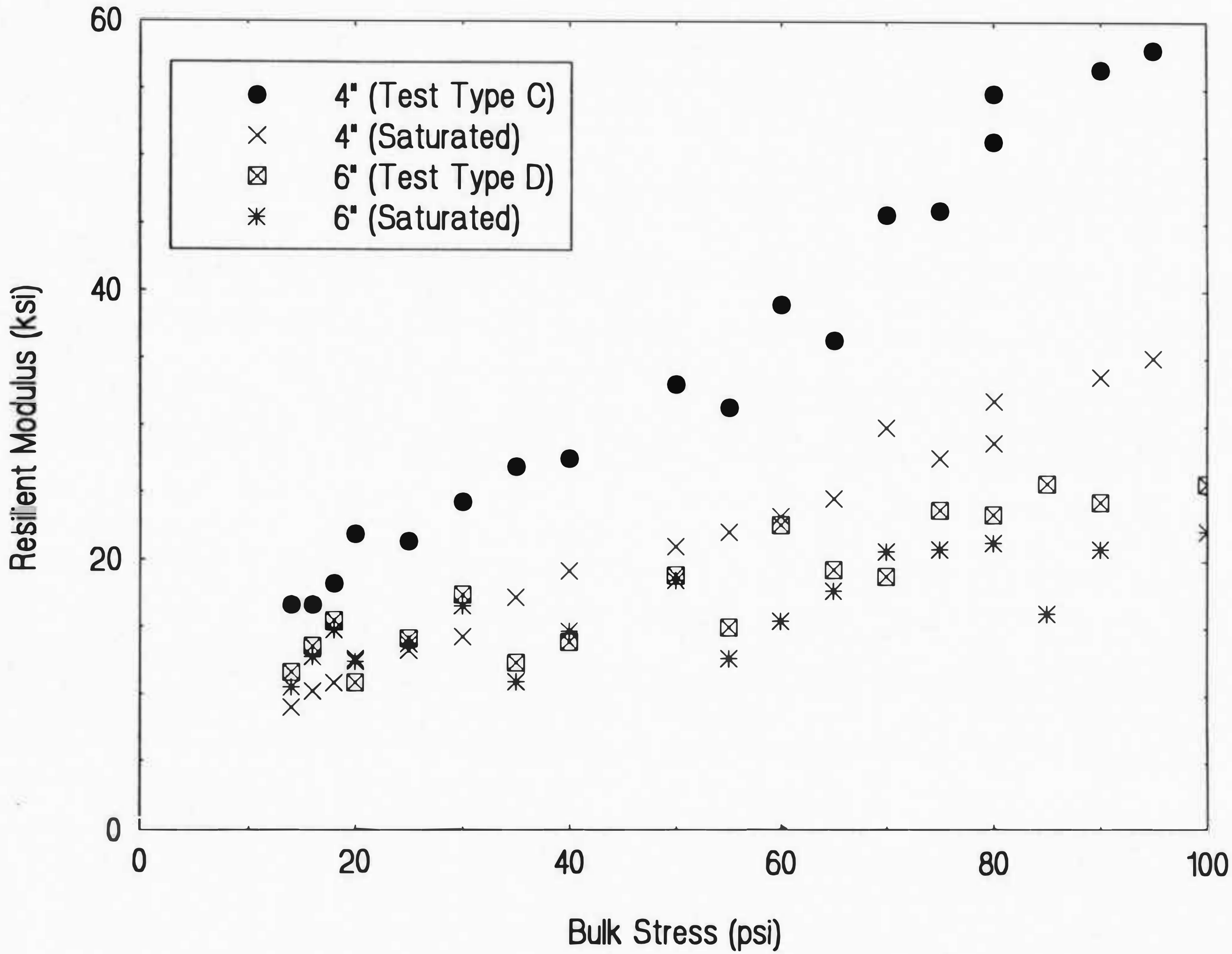

Fig. 3-16 Effects of the Saturation on the Resilient Moduli for the Aggregate from Creek County (Limestone)

(by using Eq. 2-2) for these six aggregate types is presented in the Table 3-6. It may be noted that at least three RM tests have been conducted for each aggregate type and the three most consistent results are presented in the Table 3-6 and Figs. 3-17 to 3-22. The detail of RM results in terms of bulk stress for each aggregate type are given in the Tables 3-7 to 3-12. For the sake of comparison, the average (mean) RM values for each aggregate types are grouped together and presented in Fig. 3-23. Thompson <16> reported that for a given gradation and for either crushed or uncrushed materials, the source (limestone, sandstone, granite, etc) is not a significant factor in terms of RM. Later, Thompson and Smith <7> stated that the resilient modulus properties of the various aggregates are similar. The type of aggregates base material (crushed stone/gravel) has a limited effect (10%) on RM (Thompson and Smith <7>). This phenomena is confirmed in this study and reflected in the Fig. 3-23.

3.5.4 Testing Procedures

In this study, the testing procedures suggested by the AASHTO T292-91I <8> and T294-92I <9> are investigated. The aggregate type from Choctaw County has the most consistent results on type D tests (lowest standard deviation), and is selected for type G tests. The basic difference between test types D and G is the testing procedure. The testing procedures for test types D and G are AASHTO T292-91I and AASHTO T294-92I, respectively. The AASHTO T292-91I testing procedure starts with a higher confining pressure and deviatoric dynamic stress and ends with lower confining pressure and deviatoric dynamic stress which is opposite to the T294-92I testing procedure. The T294-92I testing procedure gives a higher resilient moduli than those obtained by using T294-92I testing procedure, as shown in Fig. 3-24. In both

Table 3-6 Summary of K_1 and K_2 for Six Aggregate Types

County	Material	K_1 (psi)	Mean	SD*	K_2	Mean	SD*
Comanche (rs)	Limestone	4151 3908 2168	3409	1082	.3918 .3683 .5825	.4475	.1175
Cherokee (ark)	Limestone	2283 4685 7213	4727	2465	.5017 .3472 .2882	.3808	.1133
Creek (qupa)	Limestone	4449 4317 3494	4087	518	.3698 .3858 .4180	.3912	.0246
Choctaw (bor)	Sandstone	1388 1691 1427	1502	165	.5309 .5847 .5734	.563	.0284
Johnston (mer)	Granite	2041 2366 2102	2170	173	.5242 .4350 .4889	.4827	.0449
Murray (wr)	Rhyolite	2747 2417 3099	2754	341	.4338 .4949 .4612	.4633	.031

* SD denotes the standard deviation

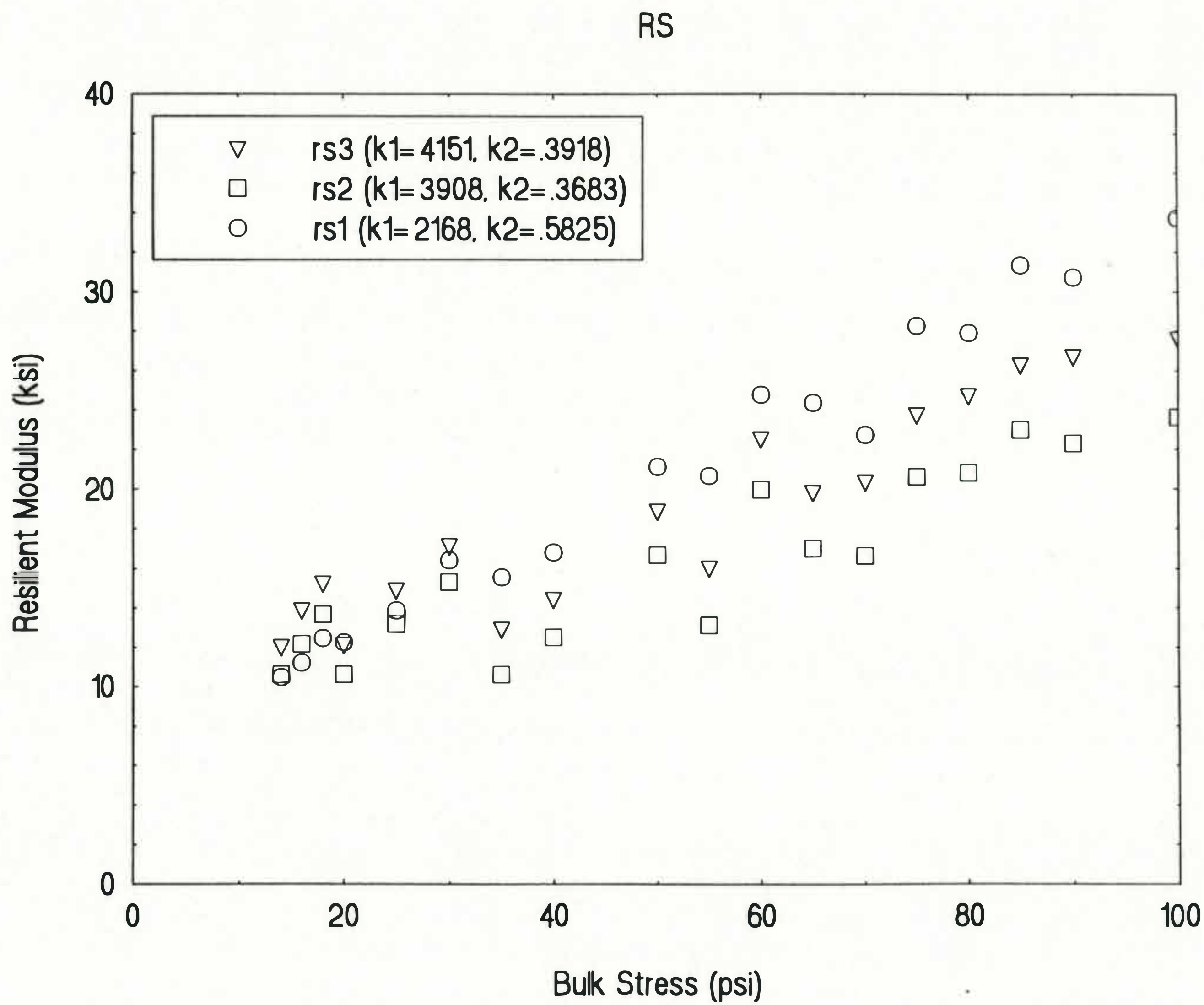

Fig. 3-17 Resilient Moduli for the Aggregate from Comanche County (Limestone)

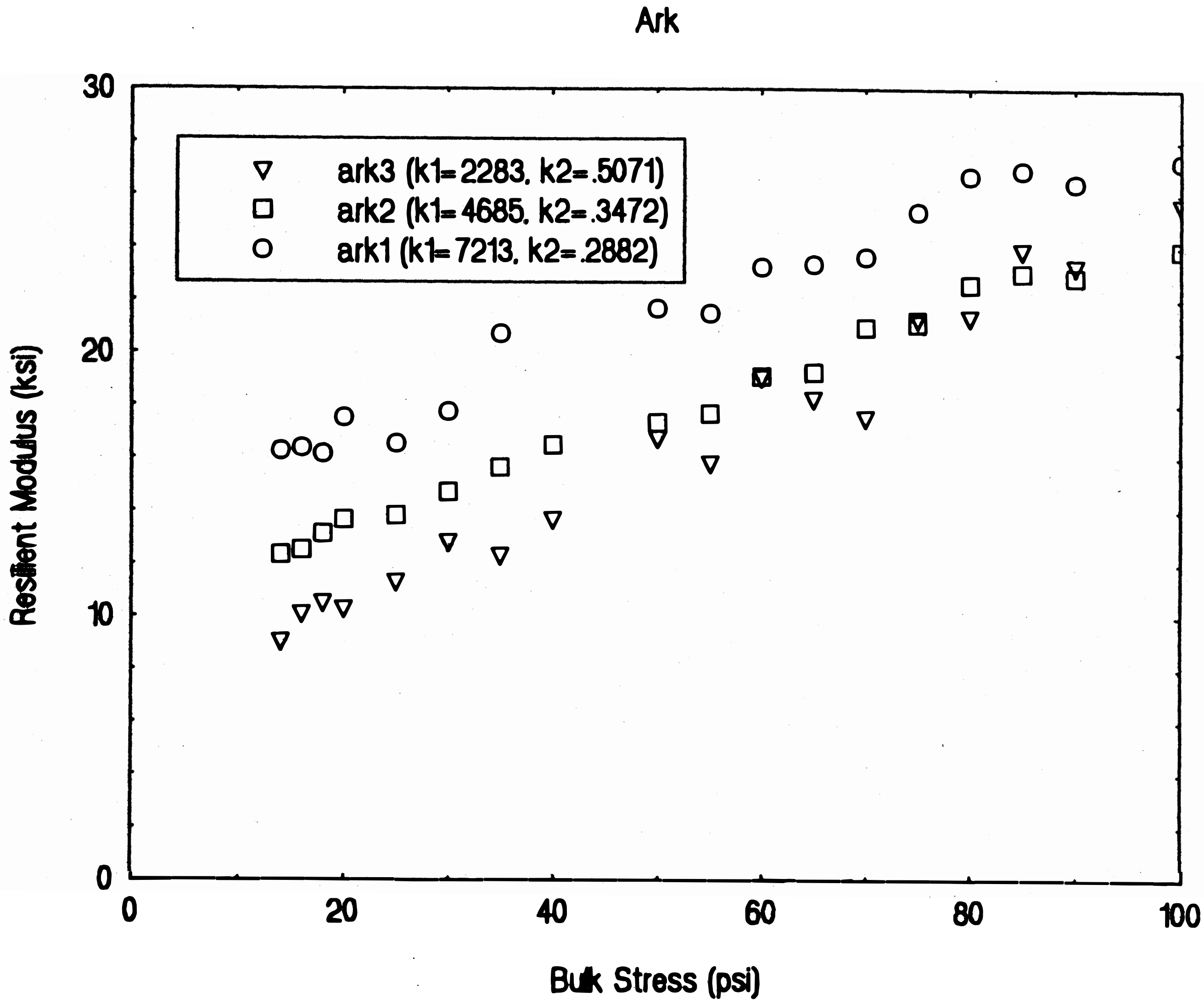

Fig. 3-18 **Resilient Moduli for the Aggregate from Cherokee County (Limestone)**

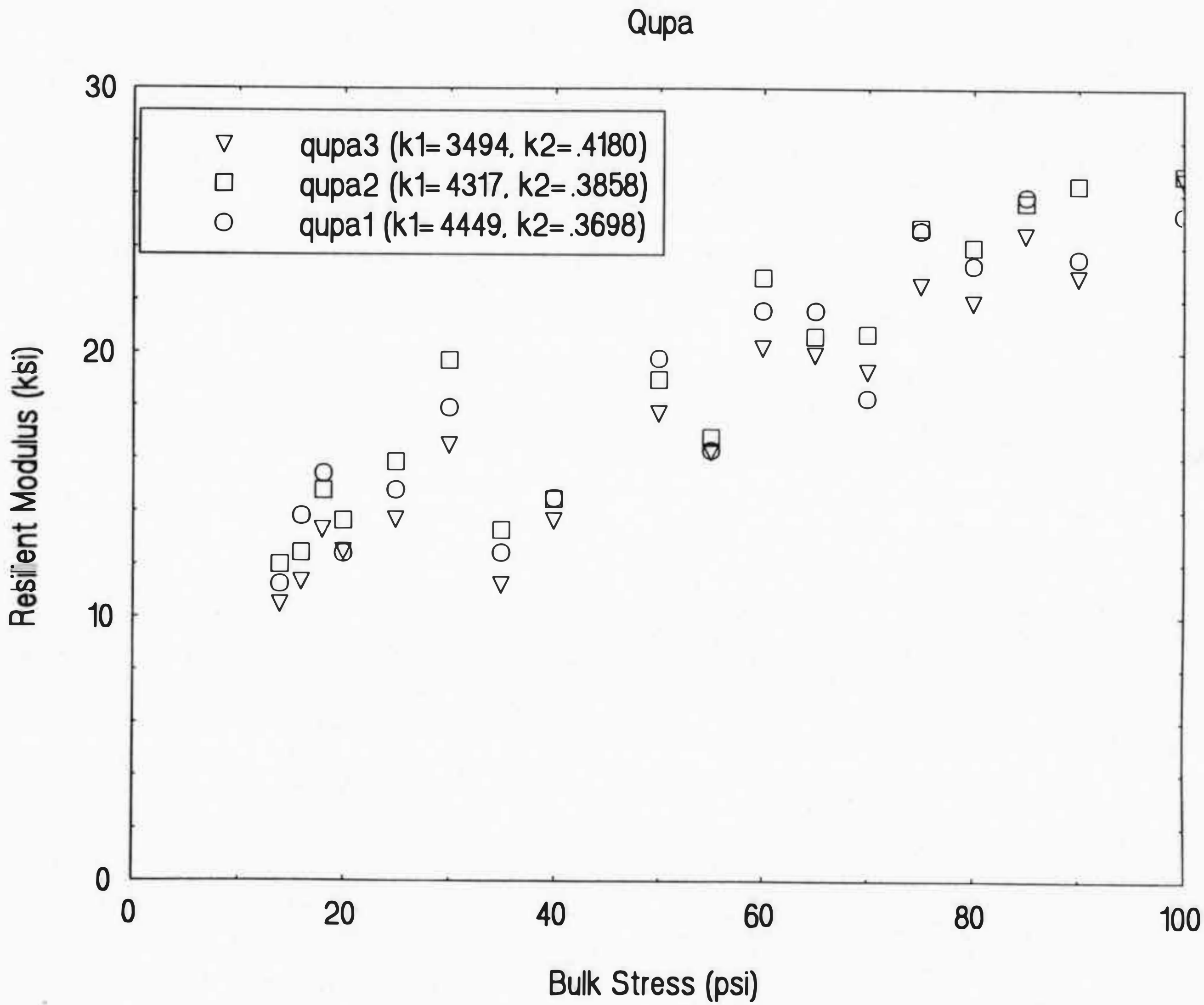

Fig. 3-19 Resilient Moduli for the Aggregate from Creek County (Limestone)

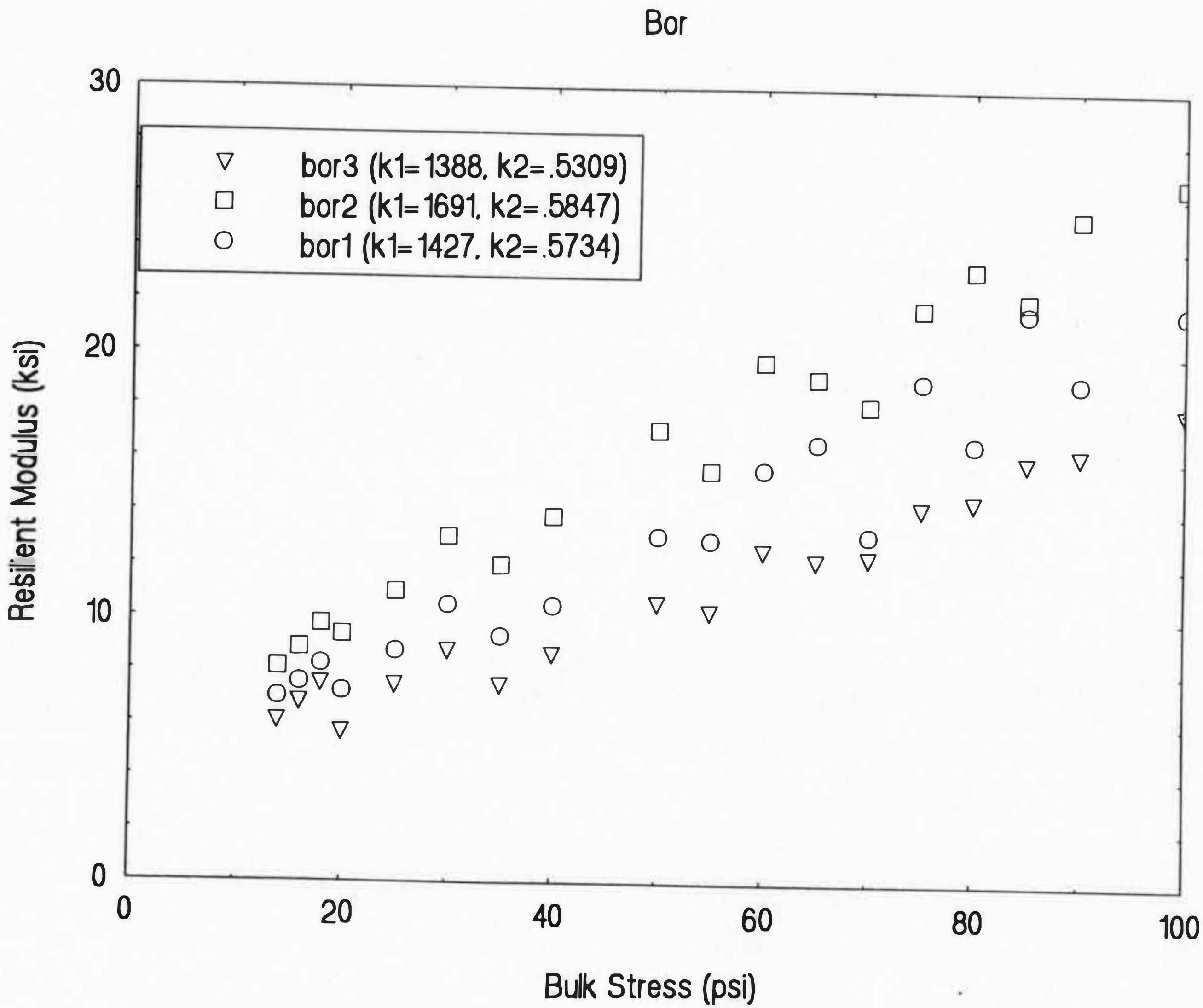

Fig. 3-20 Resilient Moduli for the Aggregate from Choctaw County (Sandstone)

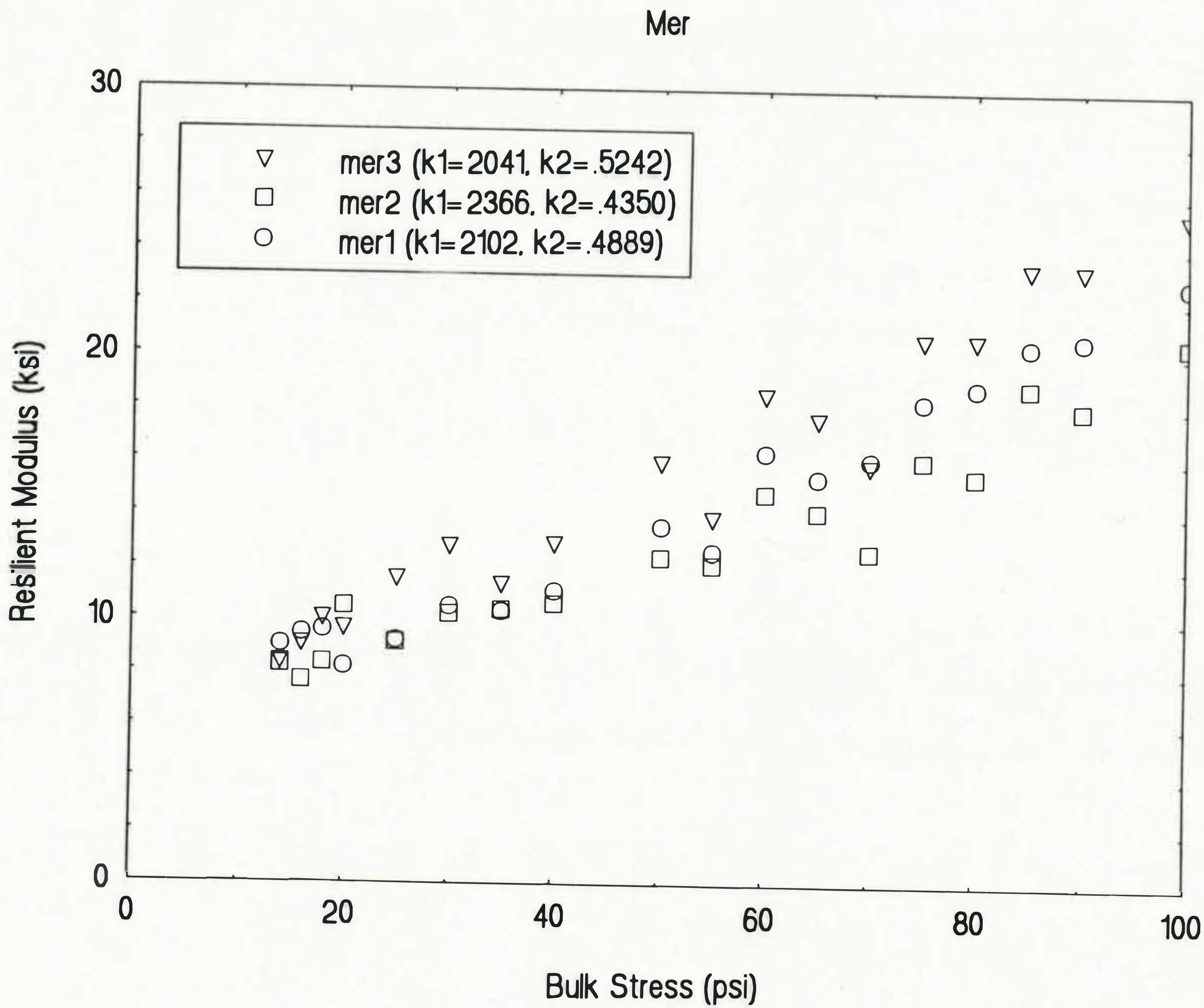

Fig. 3-21 Resilient Moduli for the Aggregate from Johnston County (Granite)

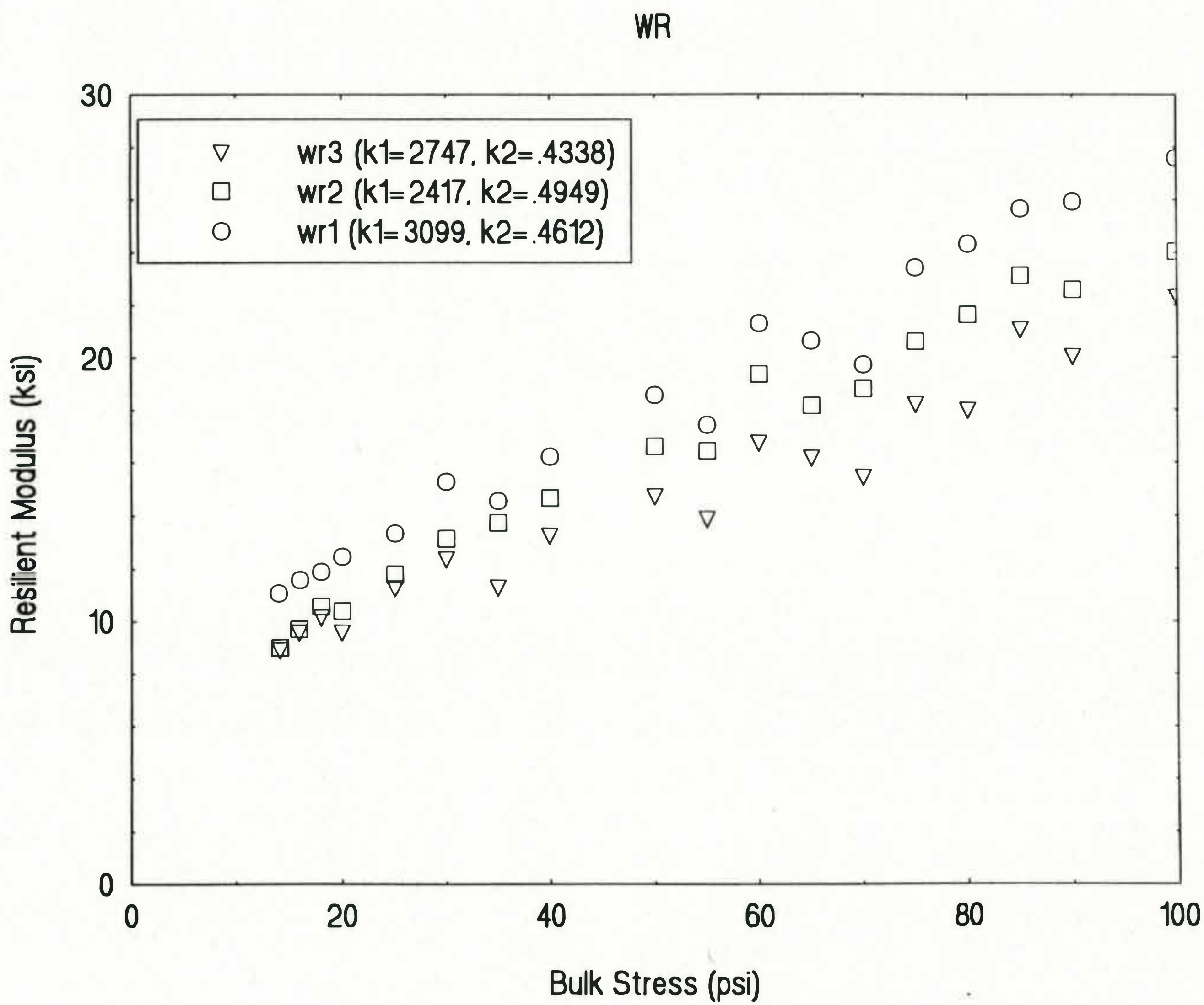

Fig. 3-22 Resilient Moduli for the Aggregate from Murray County (Rhyolite)

Table 3-7 Resilient Moduli for Aggregate from Comanche County (Limestone)

Bulk stress (psi)	Test 1 (ksi)	Test 2 (ksi)	Test 3 (ksi)	Mean (ksi)
70	22.75	16.64	20.38	19.92
80	27.93	20.81	24.77	24.5
90	30.73	22.28	26.73	26.58
100	33.73	23.61	27.62	28.32
55	20.64	13.12	16.04	16.6
65	24.38	16.99	19.82	20.4
75	28.28	20.59	23.77	24.21
85	31.35	23.0	26.33	26.9
35	15.55	10.59	12.94	13.03
40	16.8	12.5	14.46	14.59
50	21.12	16.68	18.88	18.89
60	24.79	19.94	22.57	22.43
20	12.25	10.63	12.16	11.68
25	13.87	13.16	14.91	13.98
30	16.39	15.30	17.15	16.28
14	10.48	10.64	12.04	11.05
16	11.22	12.17	13.91	12.43
18	12.47	13.69	15.29	13.82

Table 3-8 Resilient Moduli for Aggregate from Cherokee County (Limestone)

Bulk stress (psi)	Test 1 (ksi)	Test 2 (ksi)	Test 3 (ksi)	Mean (ksi)
70	23.61	20.95	17.5	20.69
80	26.68	22.55	21.34	23.52
90	26.44	22.85	23.27	24.19
100	27.26	23.91	25.62	25.60
55	21.46	17.65	15.79	18.30
65	23.33	19.24	18.24	20.27
75	25.30	21.08	21.28	22.55
85	26.92	23.0	23.88	24.60
35	20.71	15.63	12.3	16.21
40	24.42	16.45	13.65	18.17
50	21.65	17.32	16.75	18.57
60	23.22	19.07	19.06	20.45
20	17.53	13.63	10.28	13.81
25	16.52	13.79	11.31	13.87
30	17.74	14.67	12.81	15.07
14	16.25	12.32	9.03	12.53
16	16.37	12.51	10.08	12.99
18	16.14	13.1	10.52	13.25

Table 3-9 Resilient Moduli for Aggregate from Creek County (Limestone)

Bulk stress (psi)	Test 1 (ksi)	Test 2 (ksi)	Test 3 (ksi)	Mean (ksi)
70	18.36	20.8	19.44	19.53
80	23.43	24.12	22.07	23.21
90	23.7	26.48	23.03	24.4
100	25.34	26.75	26.6	26.23
55	16.31	16.79	16.3	16.47
65	21.69	20.71	20.05	20.82
75	24.73	24.85	22.71	24.1
85	25.93	25.72	24.62	25.42
35	12.47	13.33	11.33	12.38
40	14.54	14.49	13.75	14.26
50	19.85	19.06	17.83	18.91
60	21.69	22.93	20.31	21.64
20	12.46	13.68	12.56	12.9
25	14.84	15.9	13.77	14.84
30	17.97	19.77	16.58	18.11
14	11.31	12.03	10.58	11.31
16	13.86	12.48	11.43	12.59
18	15.4	14.77	13.38	14.52

Table 3-10 Resilient Moduli for Aggregate from Choctaw County (Sandstone)

Bulk stress (psi)	Test 1 (ksi)	Test 2 (ksi)	Test 3 (ksi)	Mean (ksi)
70	13.22	18.11	12.42	14.58
80	16.67	23.29	14.52	18.16
90	19.03	25.27	16.32	20.21
100	21.68	26.49	17.92	22.03
55	12.99	15.62	10.33	12.98
65	16.67	19.15	12.28	16.03
75	19.03	21.8	14.27	18.37
85	21.68	22.13	16.04	19.95
35	9.3	11.99	7.5	9.6
40	10.48	13.84	8.69	11
50	13.12	17.15	10.63	13.63
60	15.65	19.74	12.64	16.01
20	7.24	9.38	5.74	7.45
25	8.76	11.0	7.5	9.09
30	10.5	13.06	8.8	10.79
14	7.02	8.13	6.12	7.09
16	7.57	8.85	6.85	7.76
18	8.25	9.77	7.55	8.52

Table 3-11 Resilient Moduli for Aggregate from Johnston County (Granite)

Bulk stress (psi)	Test 1 (ksi)	Test 2 (ksi)	Test 3 (ksi)	Mean (ksi)
70	16.05	12.59	15.85	14.83
80	18.81	15.46	20.62	18.3
90	20.62	18.0	23.29	20.64
100	22.74	20.49	25.26	22.83
55	12.57	12.07	13.86	12.83
65	15.36	14.08	17.61	15.68
75	18.26	16.05	20.64	18.32
85	20.38	18.83	23.34	20.85
35	10.29	10.34	11.34	10.66
40	11.05	10.58	12.86	11.5
50	13.51	12.35	15.94	13.93
60	16.33	14.77	18.53	16.54
20	8.18	10.47	9.65	9.43
25	9.18	9.1	11.54	9.94
30	10.47	10.18	12.77	11.14
14	9.0	8.23	8.37	8.53
16	9.43	7.64	9.06	8.71
18	9.58	8.32	10.04	9.31

Table 3-12 Resilient Moduli for Aggregate from Murray County (Rhyolite)

Bulk stress (psi)	Test 1 (ksi)	Test 2 (ksi)	Test 3 (ksi)	Mean (ksi)
70	19.74	18.84	15.49	18.02
80	24.33	21.66	18.06	21.35
90	25.93	22.6	20.09	22.87
100	27.6	24.04	22.36	24.67
55	17.46	16.45	13.91	15.94
65	20.66	18.18	16.23	18.36
75	23.45	20.63	18.27	20.78
85	25.68	23.14	21.12	23.31
35	14.54	13.72	11.33	13.2
40	16.24	14.65	13.28	14.72
50	18.59	16.63	14.77	16.66
60	21.32	19.39	16.82	19.18
20	12.45	10.39	9.62	10.82
25	13.32	11.79	11.3	12.14
30	15.28	13.12	12.39	13.6
14	11.09	8.99	8.96	9.68
16	11.58	9.71	9.62	10.3
18	11.90	10.57	10.2	10.89

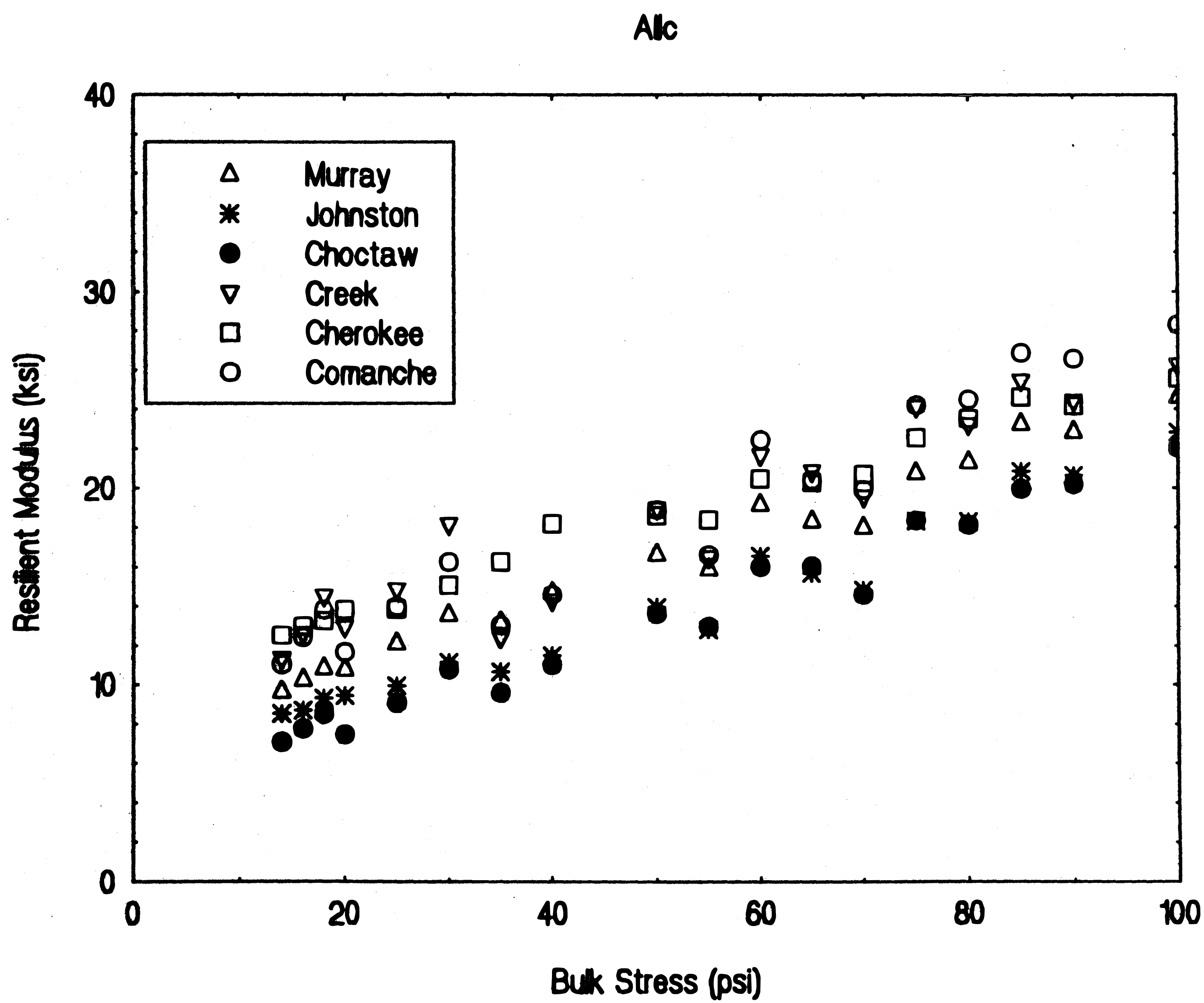

Fig. 3-23 Comparison of Average Resilient Moduli for Six Aggregate Types

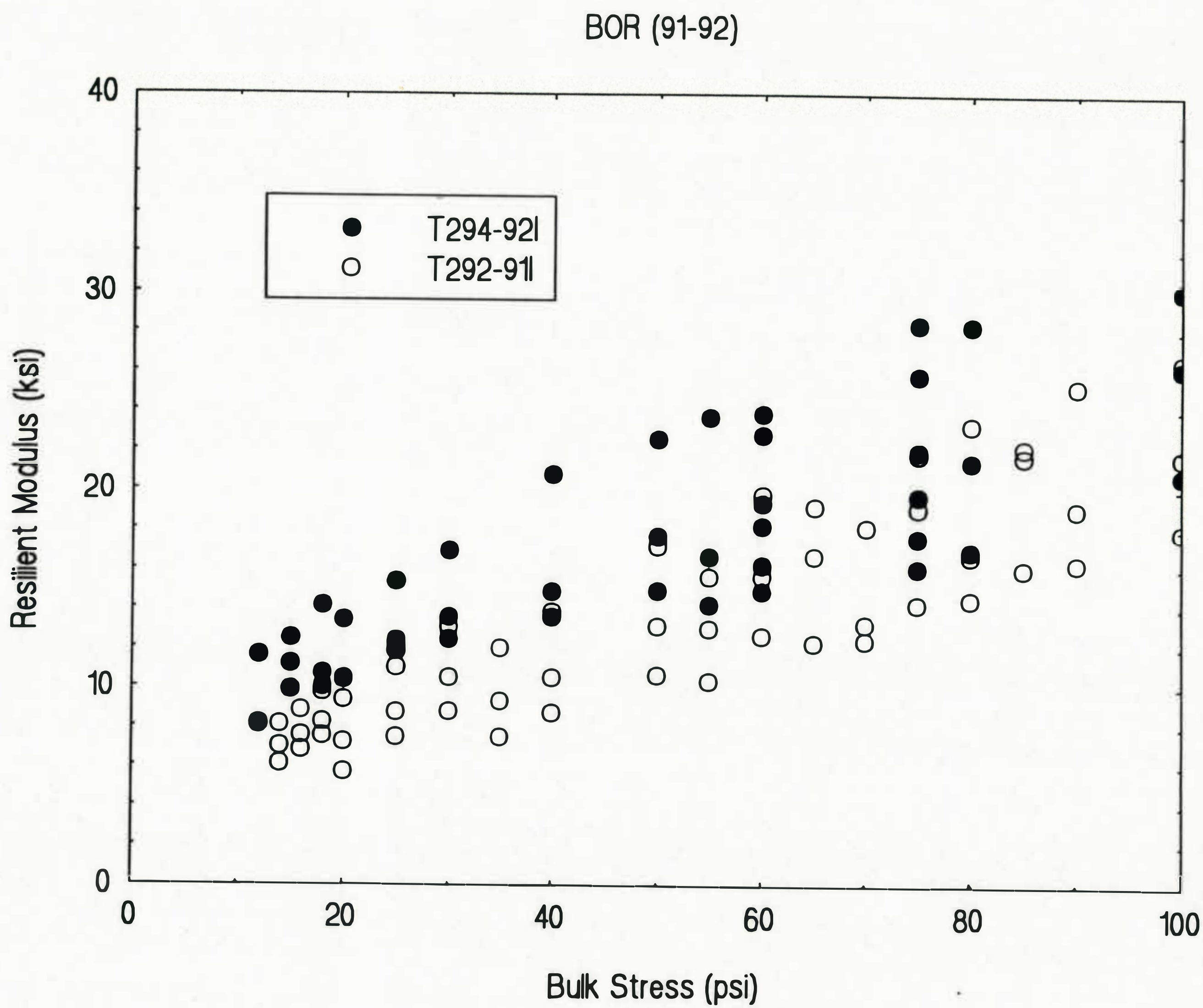

Fig. 3-24 Effects of the Testing Procedures (AASHTO T292-91I and T294-92I) on the Resilient Moduli

cases (test types D and G), three RM tests are performed under identical conditions, except for the stress application sequence (see Table 2-1 for detail). The fact that the T294-92I testing procedure yields higher resilient moduli may be attributed to the cyclic stress having a stiffening effect on the specimen structure because the stress application sequence begins from low to higher in the testing procedure T294-92I.

3.6 TRIAXIAL COMPRESSION TESTS

After the repeated triaxial testing, the static triaxial compression tests are performed to obtain the cohesion and friction angle of the material (aggregate). The repeated triaxial tests serve as a "conditioning" of the triaxial compression as imposed by the moving vehicles. Thompson and Smith <7> reported that the shear strength of unconditioned specimens does not represent the strength of an in service compacted granular base material subjected to traffic loading. They found that this strength increase varies from 34 to 217 percent, induced by the dynamic stress repetitions. However, the number of repetitions and the magnitude of the dynamic stress required to reproduce the field conditions is not completely understood or finalized at present.

An attempt is made to investigate the effect on shear strength of materials due to different specimen sizes, compaction method, and dry densities; the aggregate from Creek County is used for this purpose. The confining pressures used and maximum stresses obtained for 4" and 6" specimens are presented in Table 3-13. It is observed that the dry density and compaction method has minimal effects, if any, on maximum failure stresses. Also, for the the same level of confining pressures, the maximum failure stresses are similar for both 4" and 6" specimens. The minimal effect on shear strength due to variation of specimen size, compaction method, and dry density may attribute to the effect of conditioning (1900 cyclic repetitions), stiffening and strengthening specimens.

The conventional triaxial compression test results for six aggregate types are presented in Table 3-14. The Mohr circles are drawn based on the data presented in Table 3-14, and the obtained shear strength parameters (cohesion (intercept) and friction angle (slope)) are presented in the Figs. 3-25 to 3-27 and Table 3-14. The stress and strain curves obtained from conventional triaxial compression (CTC) tests are used to determine the initial tangent modulus (Young's Modulus, E), as shown in the Figs. 3-28 to 3-33 and the last column of Table 3-14.

Table 3-13 Comparison of Triaxial Compression Data for Different Specimen Sizes and Compaction Methods

Size of Specimen (in.)	Compaction Method Used	Dry Density (pcf)	Confining Pressure (psi)	Maximum Stress(σ_1) (psi)
4	T180-90D	153.1	10	132.8
4	T180-90D	150.3	10	145.2
6	Vibratory	151.0	10	134.2
6	Vibratory	150.0	10	132.5
4	T180-90D	153.6	15	151.4
4	Vibratory	143.1	15	154.0
6	Vibratory	150.8	15	151.7
4	Vibratory	145.4	20	179.1
6	Vibratory	150.0	20	183.0

Table 3-14 Triaxial Compression Data for Different Aggregate Types

County	Material	Confining Pressure (psi)	Maximum Stress (psi)	C (psi)	ϕ (degree)	Young's Modulus (ksi)
Comanche (rs)	Limestone	10	130	18	41	24.1
		15	153			25.4
		20	172			27.8
Cherokee (ark)	Limestone	5	86.1	12	44	22.6
		10	120.9			20.0
		15	144.7			23.3
Creek (qupa)	Limestone	10	133	18	43	23.5
		15	160			27.9
		20	187			34.6
Choctaw (bor)	Sandstone	5	112.2	12	46	22.7
		10	166.8			24.4
		15	186.2			27.1
Johnston (mer)	Granite	5	107.3	11	46	20.2
		10	134.7			23.1
		15	151.2			25.3
Murray (wr)	Rhyolite	5	120.89	18	45	20.5
		10	142.68			22.4
		15	175.24			25

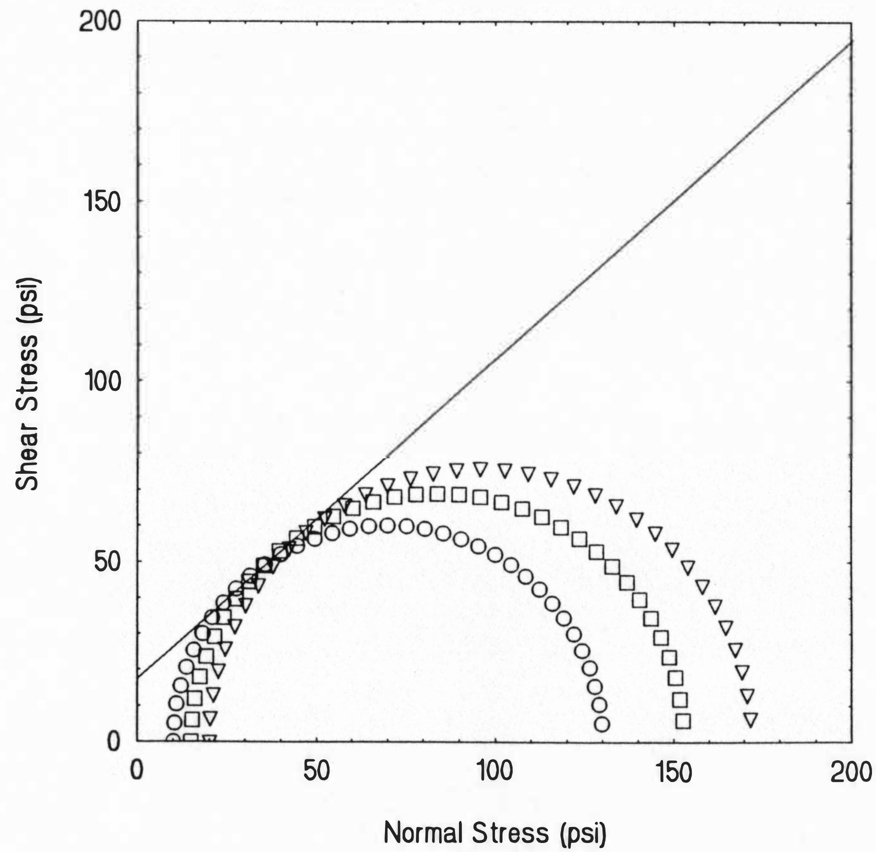

Fig. 3-25 **Mohr Diagram for the Aggregate from Comanche County (Limestone) with Cohesion = 18 psi and Friction Angle = 41 degree.**

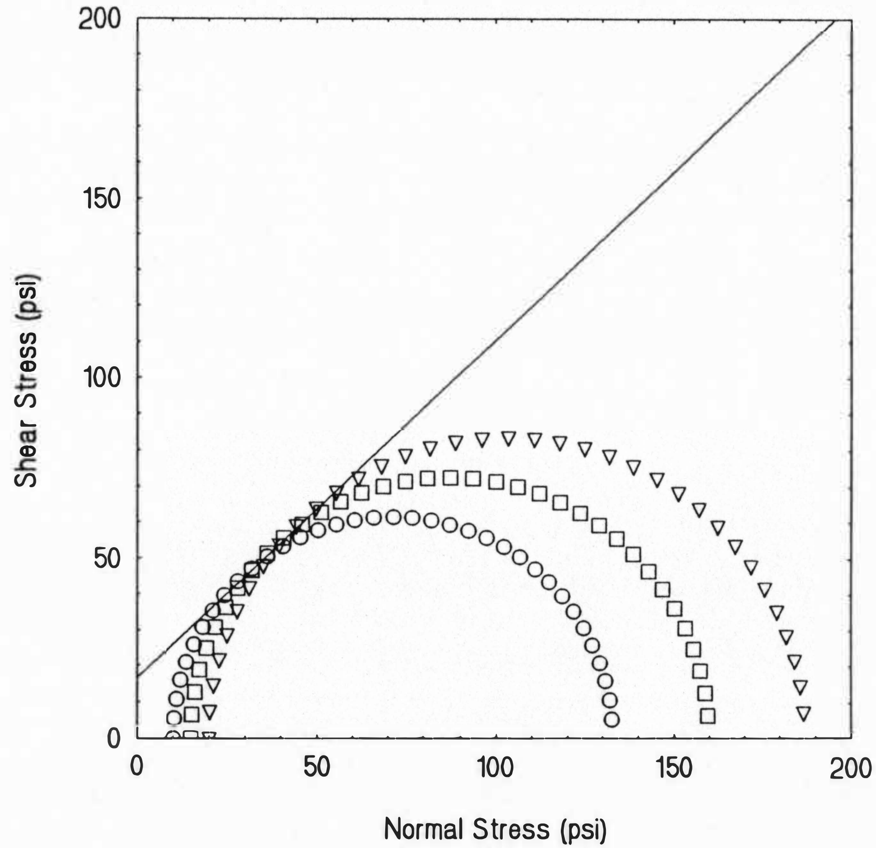

Fig. 3-26 **Mohr Diagram for the Aggregate from Creek County (Limestone) with Cohesion = 18 psi and Friction Angle = 43 degree.**

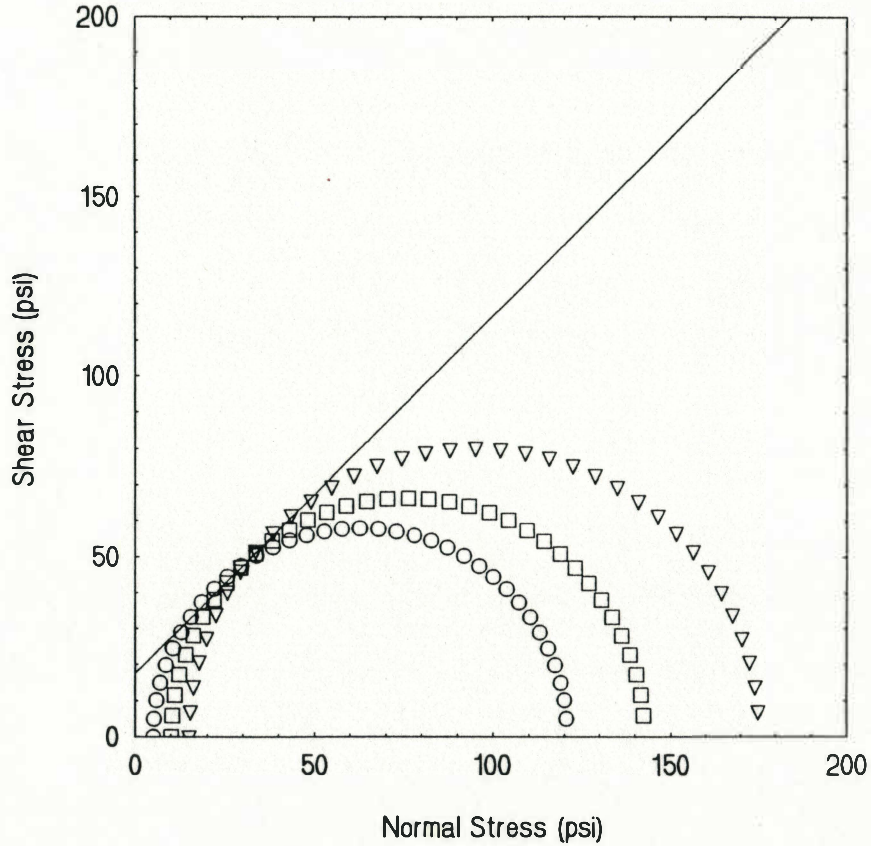

Fig. 3-27 Mohr Diagram for the Aggregate from Murray County (Rhyolite) with Cohesion = 19 psi and Friction Angle = 44 degree.

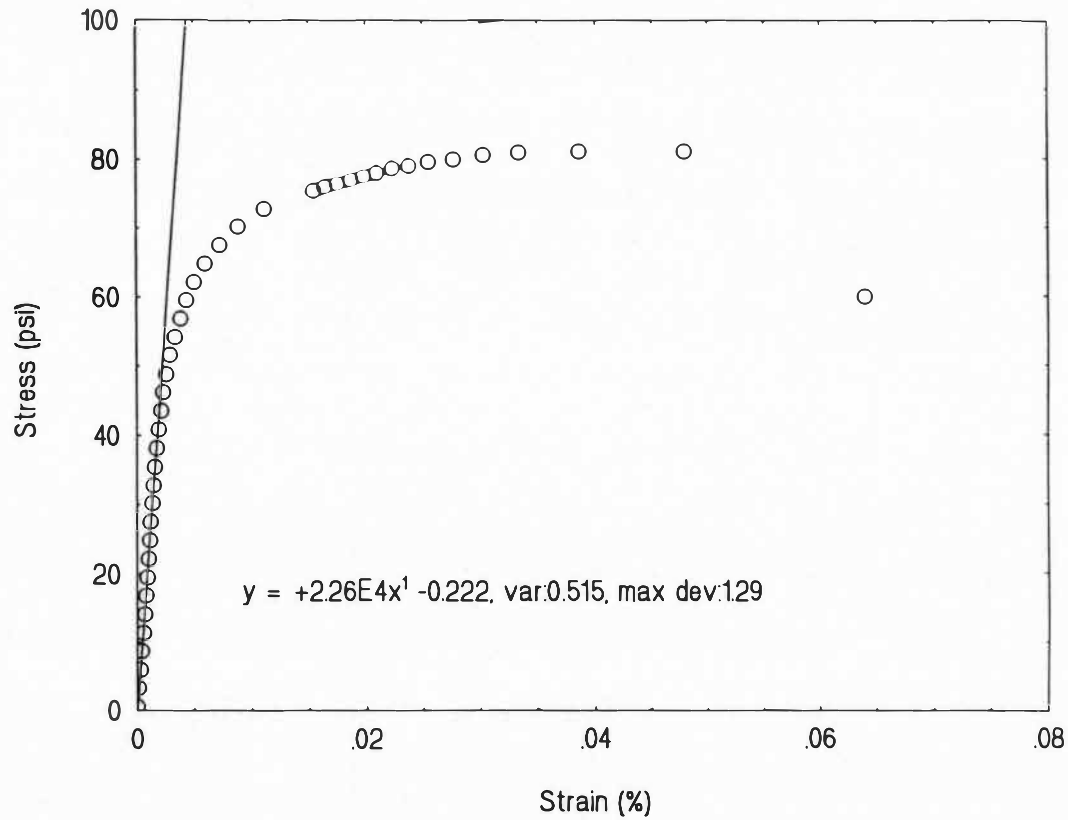

Fig. 3-28 **Stress-Strain Diagram Obtained from Conventional Triaxial Compression (CTC) Test for Aggregate from Cherokee County (Limestone) at Confining Pressure 5 psi.**

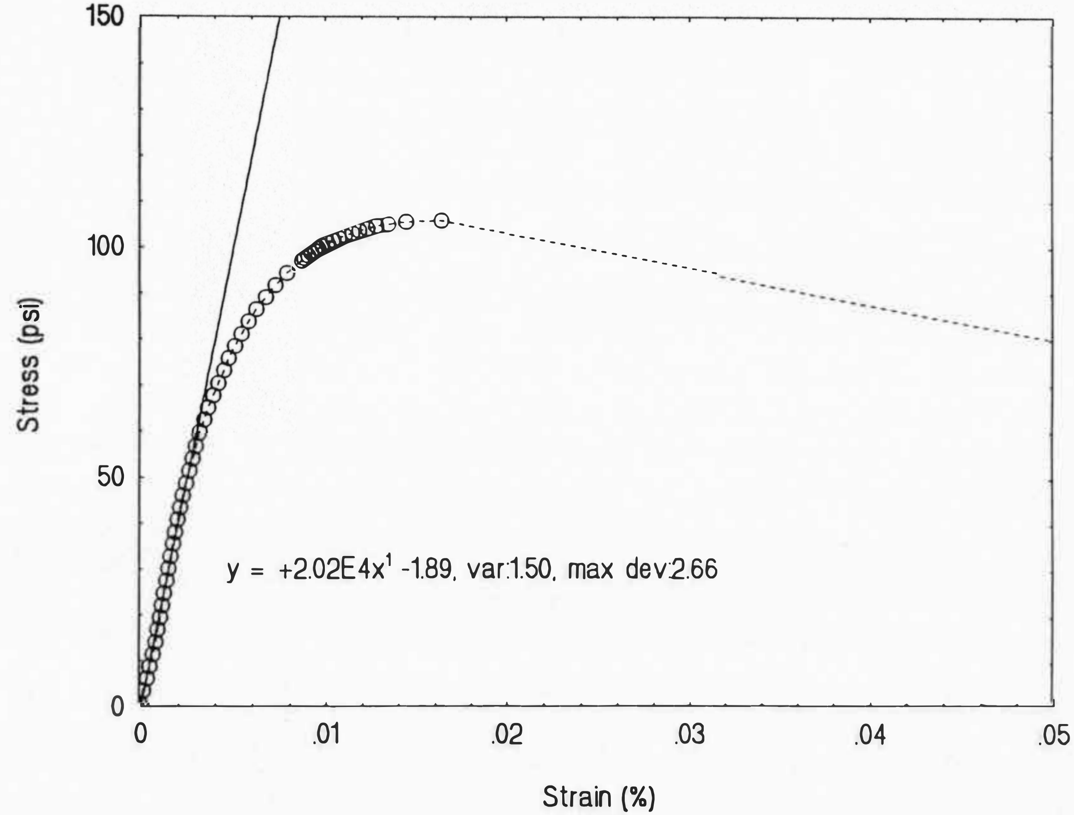

Fig. 3-29 **Stress-Strain Diagram Obtained from a Conventional Triaxial Compression (CTC) Test for the Aggregate from Johnston County (Granite) at Confining Pressure 5 psi.**

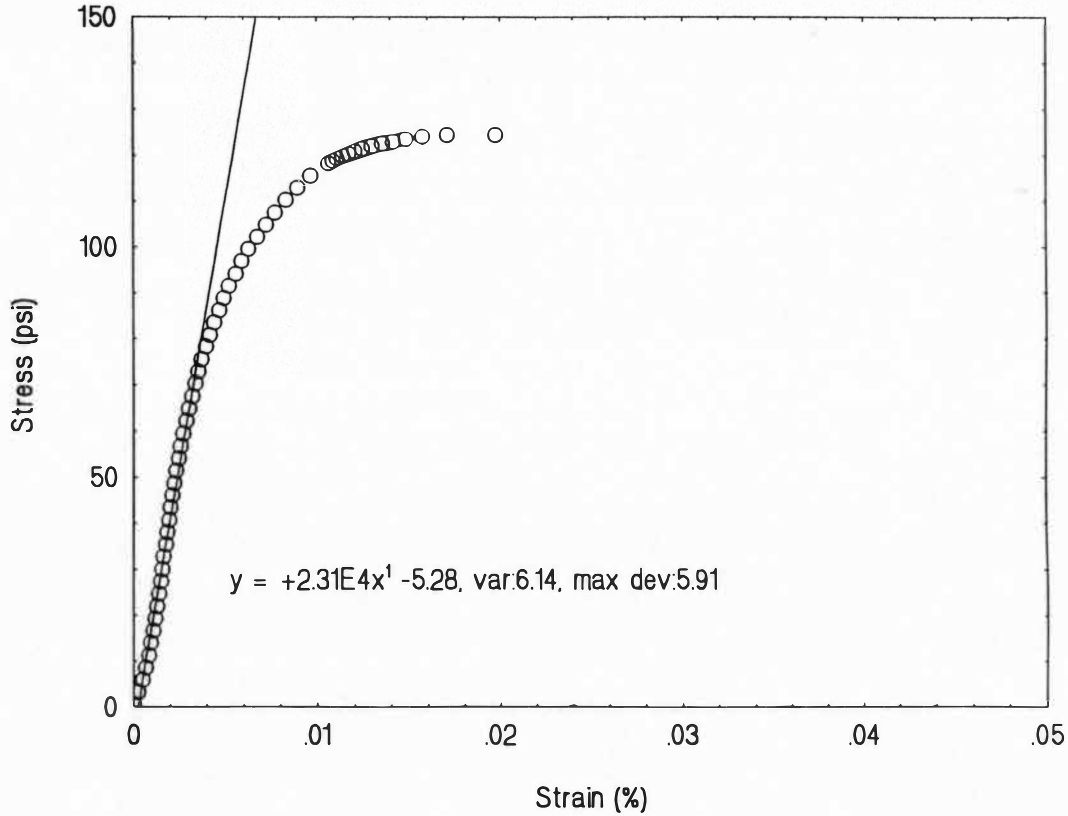

Fig. 3-30 Stress-Strain Diagram Obtained from a Conventional Triaxial Compression (CTC) Test for the Aggregate from Johnston County (Granite) at Confining Pressure 10 psi.

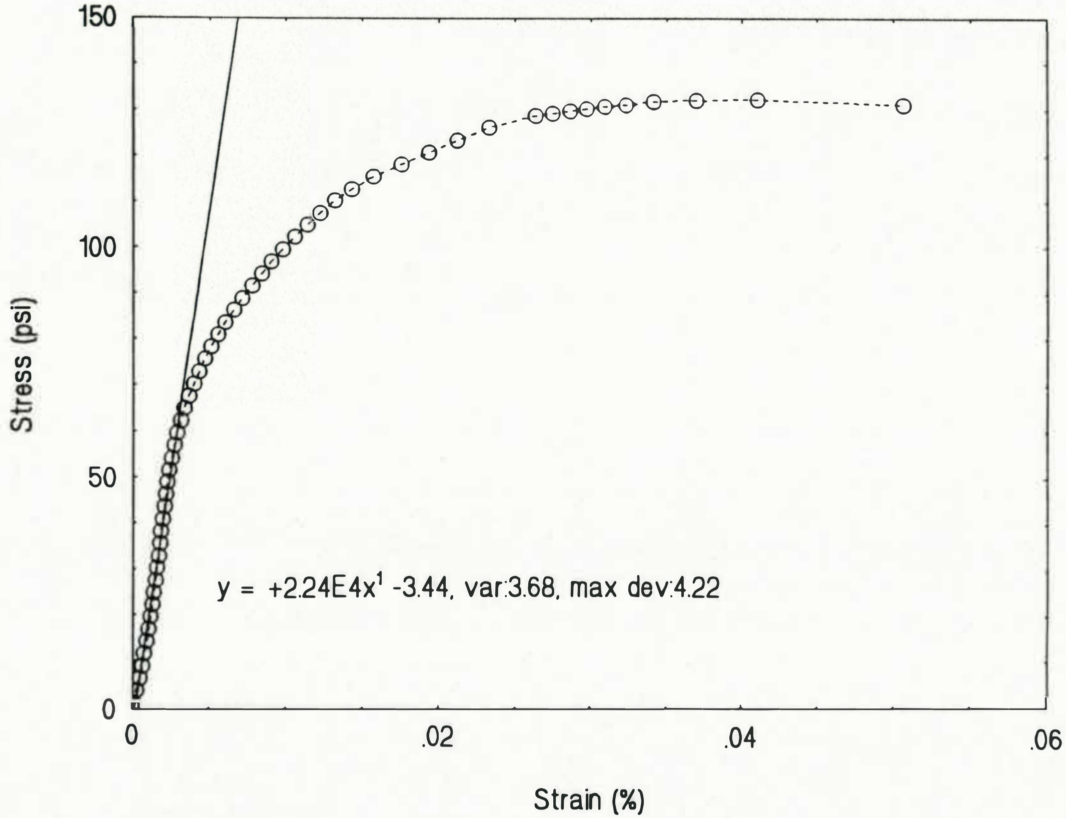

Fig. 3-31 **Stress-Strain Diagram Obtained from a Conventional Triaxial Compression (CTC) Test for the Aggregate from Murray County (Rhyolite) at Confining Pressure 10 psi.**

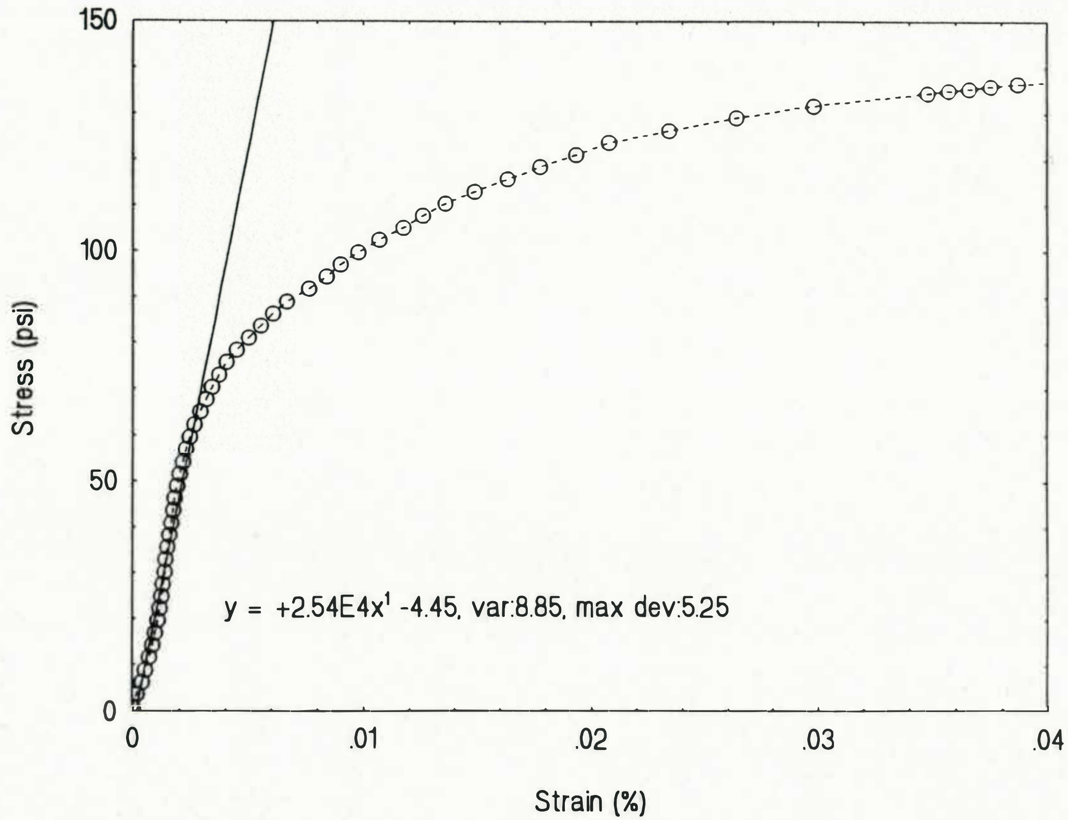

Fig. 3-32 Stress-Strain Diagram Obtained from a Conventional Triaxial Compression (CTC) Test for the Aggregate from Comanche County (Limestone) at Confining Pressure 15 psi.

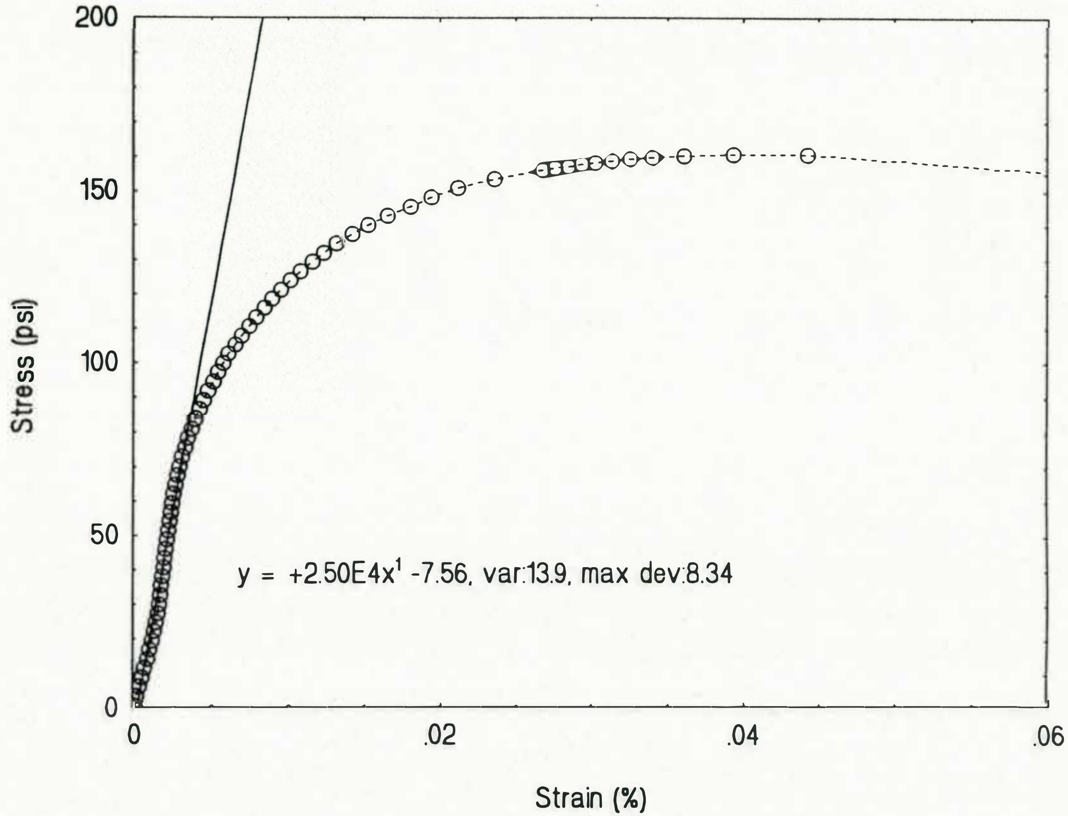

Fig. 3-33 Stress-Strain Diagram Obtained from a Conventional Triaxial Compression (CTC) Test for the Aggregate from Murray County (Rhyolite) at Confining Pressure 15 psi.

CHAPTER 4

STATISTICAL CORRELATIONS

Statistical correlations between RM and engineering index properties (namely, California bearing ratio (CBR), cohesion, friction angle, Elasticity (E)) are useful in practice because the engineering index properties are less difficult and inexpensive to evaluate. The RM values are neither intimately related to the PI of the granular materials nor to the conventional classification system used (such as the AASHTO and the Unified Classification Systems) (Laguros and Zaman <1>), therefore this correlation was not attempted. The possible correlation of CBR, cohesion, friction angle and E with RM are investigated and presented in the following sections.

4.1 CORRELATION WITH CBR

CBR is widely used as an indicator of the strength characteristics of subgrade soils and aggregates and such a relationship between RM may be useful in practice. The one developed by Heukelom and Klomp <35> is suggested by the AASHTO Design Guide 1986 <24>, which relates dynamic modulus of soils to CBR. The relation established takes the form:

RM (in psi) =1500 CBR (4-1)

and it resulted from extensive dynamic (wave propagation) field tests. The data from which this correlation was developed ranged from 750 to 3000 times CBR value. This relationship (Eq. 4-1) was proposed particularly for fine-grained soils with a soaked CBR of 10 or less. For unbound granular base/subbase materials, Rada and Witczak (1981) <13> proposed the following relationships :

Bulk Stress (θ) (psi)	RM (psi)
10	248*CBR
100	738*CBR

The AASHTO Design Guide 1986 <24> also suggests a series of relationships in terms of bulk stress to convert CBR to RM which are similar to the Rada and Witczak's study. These relationships are given in Table 4-1.

Table 4-1 Relationships between CBR and RM for Unbound Base/Subbase Granular Materials

Bulk Stress (θ) (psi)	RM (psi)
100	740*CBR
30	440*CBR
20	340*CBR
10	250*CBR

In this study, the same equipment for Resilient Modulus (RM) testing is the same as the one used to conduct the California bearing ratio (CBR) tests, but the piston attached to the load cell is modified as shown in Fig. 4-1. By using the same loading device and data acquisition system for both RM and CBR tests, it

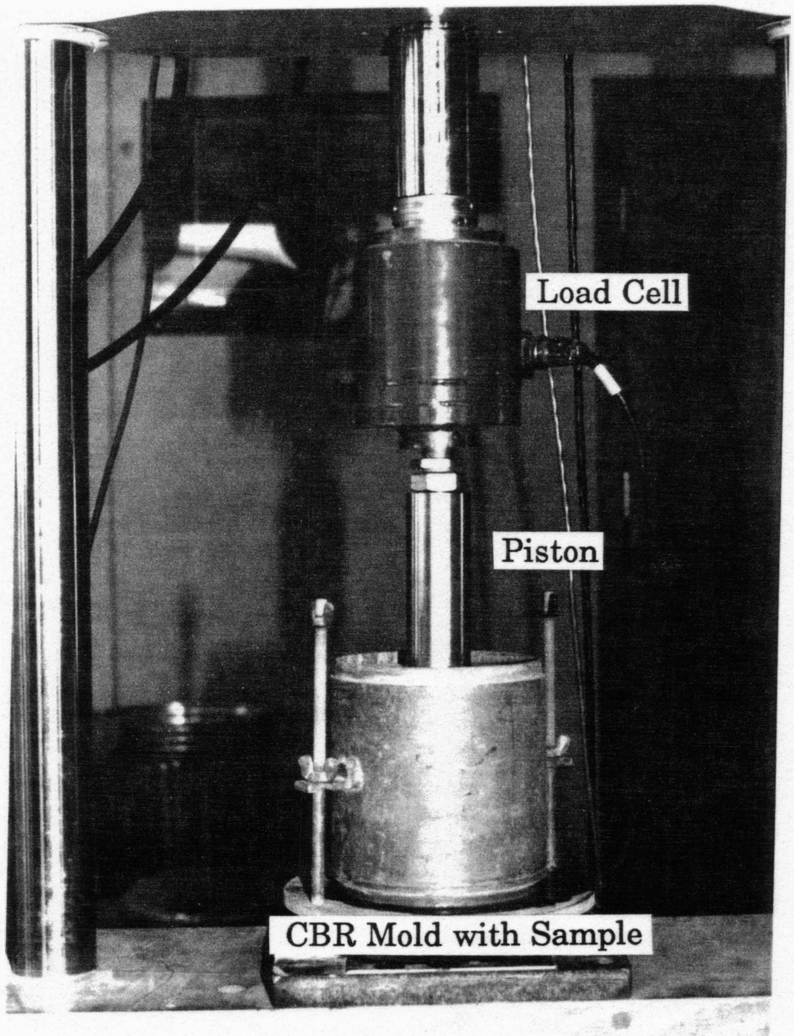

Fig. 4-1 Setup for California Bearing Ratio (CBR) Test

is expected that the experimental error due to different equipment can be eliminated which leads to better correlations between RM and CBR. All CBR tests are performed on soaked specimens following the testing procedure given by the AASHTO designation T193 <29>. The procedure in the AASHTO designation T193 <29> requires the determination of CBR values at piston penetrations of 0.1" and 0.2". If the CBR value determined at 0.2" is greater than that at 0.1", the test must be rerun. Also, AASHTO T193 <29> requires corrections to the CBR values based on the shape of the load-deformation curve. In this study to ensure test reproducibility, at least three CBR tests were performed for aggregates from six different Counties (Comanche, Cherokee, Creek, Choctaw, Johnston, Murray). All the specimens for CBR tests were prepared at the same gradation (gradation II, in Table 3-4) and compacted by using AASHTO designation T180-90D <27>.

The results obtained for soaked CBR at 0.1" and 0.2" are presented in the Table 4-2. It may be noted that some cases have only a CBR at 0.1" because the tests were discontinued due to the load applied exceeding 4500 psi (CBR value exceeds 300). The reason for having such high CBR values may attribute to the piston touching a big piece of aggregate; the load applied crushes the aggregate instead of punching the specimen which may lead to incorrect CBR values. The difficulty of determining CBR values is also reported by Rada and Witczak <13>. They stated that it is difficult to determine CBR because of unavoidable errors in determining the correct CBR value resulting from the extremely high sensitivity of moisture to CBR for most granular materials.

Table 4-2 CBR Values for Different Aggregate Types

County	Material	CBR 0.1"	CBR 0.2"
Comanche (rs)	Limestone	55	54
		60	80
		111	185
Cherokee (ark)	Limestone	92	131
		78	108
		116	158
Creek (qupa)	Limestone	95	122
		60	57
		65	116
Choctaw (bor)	Sandstone	340	*
		191	284
		302	*
Johnston (mer)	Granite	112	179
		218	274
		303	*
Murray (wr)	Rhyolite	101	163
		91	137
		**	**

* denotes the results are unavailable due to applied load exceeding 4500 psi

** denotes the results are not included in this table due to experimental error

The correlation between RM and CBR values were established by using the mean RM values in Tables 3-7 to 3-12 for the bulk stress at θ = 14, 20, 30 and 100 psi. The relationships are given in the form of

$$RM\ (\text{in psi}) = B * CBR \qquad (4\text{-}2)$$

where B is a variable and it is given in the Table 4-3. The CBR values used in the Eq. 4-2 and Table 4-3 are selected from the average values given in Table 4-2, values suspected to have experimental error have been excluded. The wide range of B among aggregate types indicates that the experimental error may occur in CBR tests and poor correlations may exist between RM. As observed from Fig. 3-23, the RM values are quite similar among aggregate types. However, the CBR values are quite different (see Table 4-2). This may be attributed to the specimen subjected to the way the load is applied (dynamic vs. static) and the resulting load-deformation characteristics are different. In the case of RM test, the specimen tends to bend and swell in the axial and radial directions when subjected to axial dynamic loading; however, in the CBR test due to confinement in the radial and the axial direction (bottom of specimen) the specimen is only allowed to swell and deform in one direction. Rada and Witczak <13> conducted an analysis of nearly 100 data sets and found that CBR values do not correlate well with RM values; particularly, for a given granular material where the RM values are stress dependent, unique correlations between RM and CBR do not seem to exist.

By referring to Table 4-3, it can be observed that all the values of B obtained are lower than those suggested by the AASHTO Design Guide 1986 <24>. To

Table 4-3 Variable B for Different Aggregate Types at Different Bulk Stresses (θ)

County	CBR	Variable B θ=100 (psi)	θ=30 (psi)	θ=20 (psi)	θ=14 (psi)
Comanche (rs)	67	423	243	174	165
Cherokee (ark)	132	181	106	96	88
Creek (qupa)	116	226	156	112	97
Choctaw (bor)	284	78	38	26	25
Johnston (mer)	226	101	50	42	38
Murray (wr)	150	164	91	72	65
Average	**132**	**193**	**96**	**82**	**74**

meet the values of B as suggested by the AASHTO Design Guide 1986 <24>, the CBR values need to be in the range of 22 to 41 (with a mean of 32.3, standard deviation 5.12) for the RM values obtained in this study. An attempt was made to find an average CBR value and the variable B in terms of bulk stresses so as to fit all the RM data. As shown in both Fig. 4-2 and Table 4-3, the average CBR value is found to be 132 and the values of B are 193, 96, 82 and 74 at θ=100, 30, 20 and 14 psi, respectively. Fig. 4-2 shows that the correlation with RM values does exit for the CBR value at or close to 132 which is a reasonable CBR value for the crushed aggregate bases/subbases. Also, as observed from the Fig. 4-2, the RM values obtained in this study are bounded in the CBR range of 100-160, provided the B constants are 193, 96, 82 and 74 at θ=100, 30, 20 and 14 psi, respectively.

4.2 CORRELATION WITH COHESION AND FRICTION ANGLE

Thompson (1989) <16> stated that Resilient Moduli of granular materials display more "generic" types of behavior and vary less than fine-grained soils. Gradation, shape/angularity/surface texture (crushed-uncrushed), and moisture content (especially for high fines content materials) influence of RM of granular materials. The magnitude of the repeated stress state (as expressed by the bulk stress θ) is the most dominating and significant factor (Thompson (1989) <16>). These phenomena are conformed and presented in the Figs. 3-17 to 3-22 which also attest to the Resilient Moduli increasing with bulk stress. This is similar to the shear stress increasing with normal principal stresses per the principles of the Mohr failure envelope. Thus, for a better correlation with RM value of the granular material, a model including the

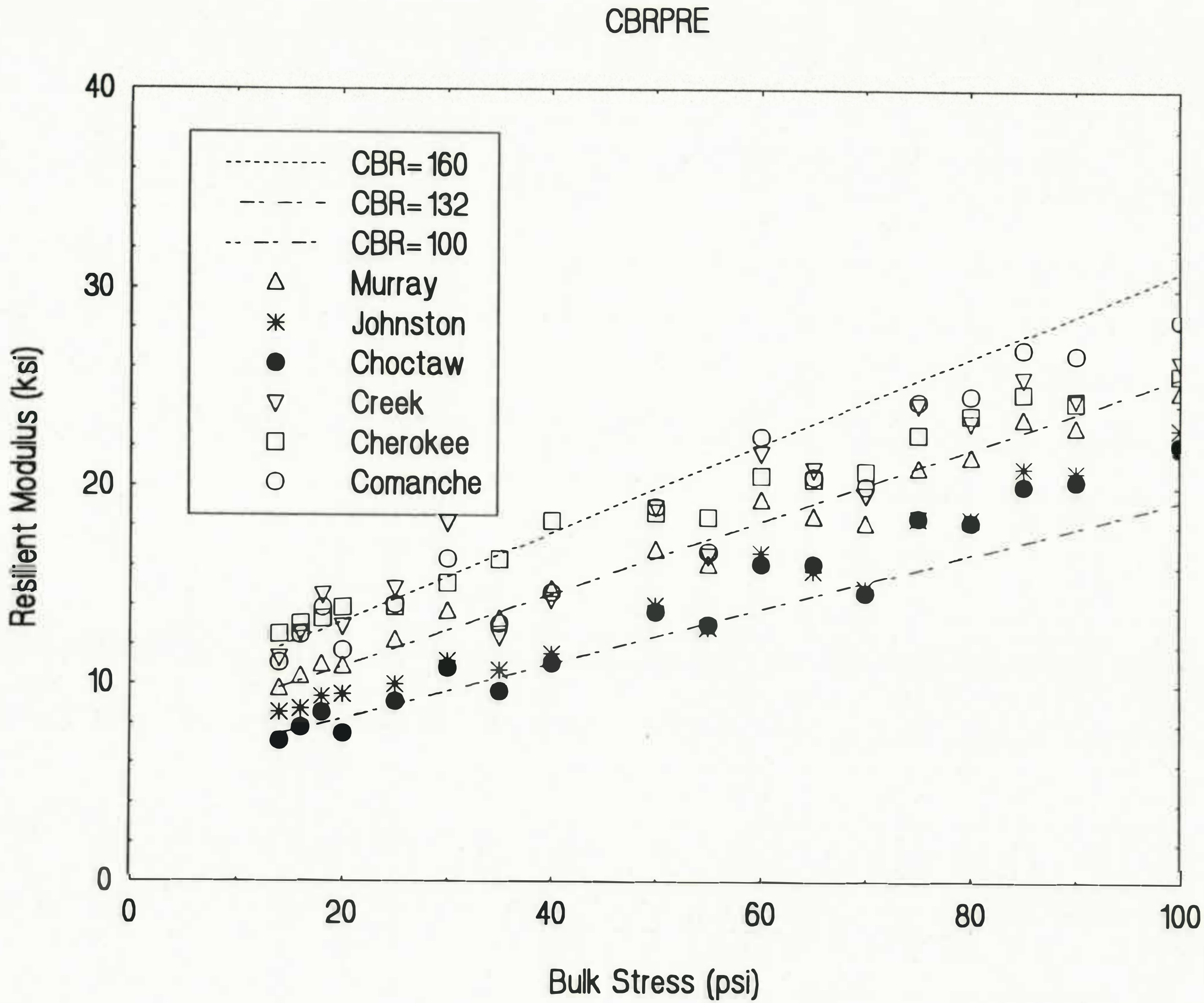

Fig. 4-2 Comparison of Model Prediction (By using Average CBR Values in Table 4-3 and the Relationship in Eq. 4-2) and Experimental Observed RM values

variables of stress state and moisture content variation is desirable. However, in this study due to insufficient RM data for the variation of moisture content, the variable of moisture content will not be included in the correlations. Therefore, a linear model relating cohesion (C) and friction angle (ϕ) with RM in terms of the major principal stress σ_3 and bulk stress θ is formulated and is given in the form of

$$RM \text{ (in psi)} = A_0 + A_1{*}C + A_2{*}\sigma_1{*}\tan\phi + A_3{*}\theta \tag{4-3}$$

Where $A_0 \sim A_3$ are the regression constants and θ is the bulk stress defined by $\theta = \sigma_1 + \sigma_2 + \sigma_3$.

The following numerical values of the regression constants are obtained :

A_0 .... 2860.94 psi

A_1 .... 275.0

A_2 .... 128.0

A_3 .... 118.0

The same C and ϕ values given in Table 3-14 are used in the prediction of six aggregate types. The comparisons between the experimental observations and the model predictions are presented in Figs. 4-3 to 4-8. In view of Figs. 4-3 to 4-8, in a few occasions for the same bulk stresses we have more than one RM value because the same bulk stress can have more than one combination of σ_1 and σ_3. The average (mean) of 3 RM values is compared with the model prediction and the percent difference for the given bulk stress as specified by AASHTO T292-91I <8> are presented in Table 4-4 through Table 4-9. The R^2

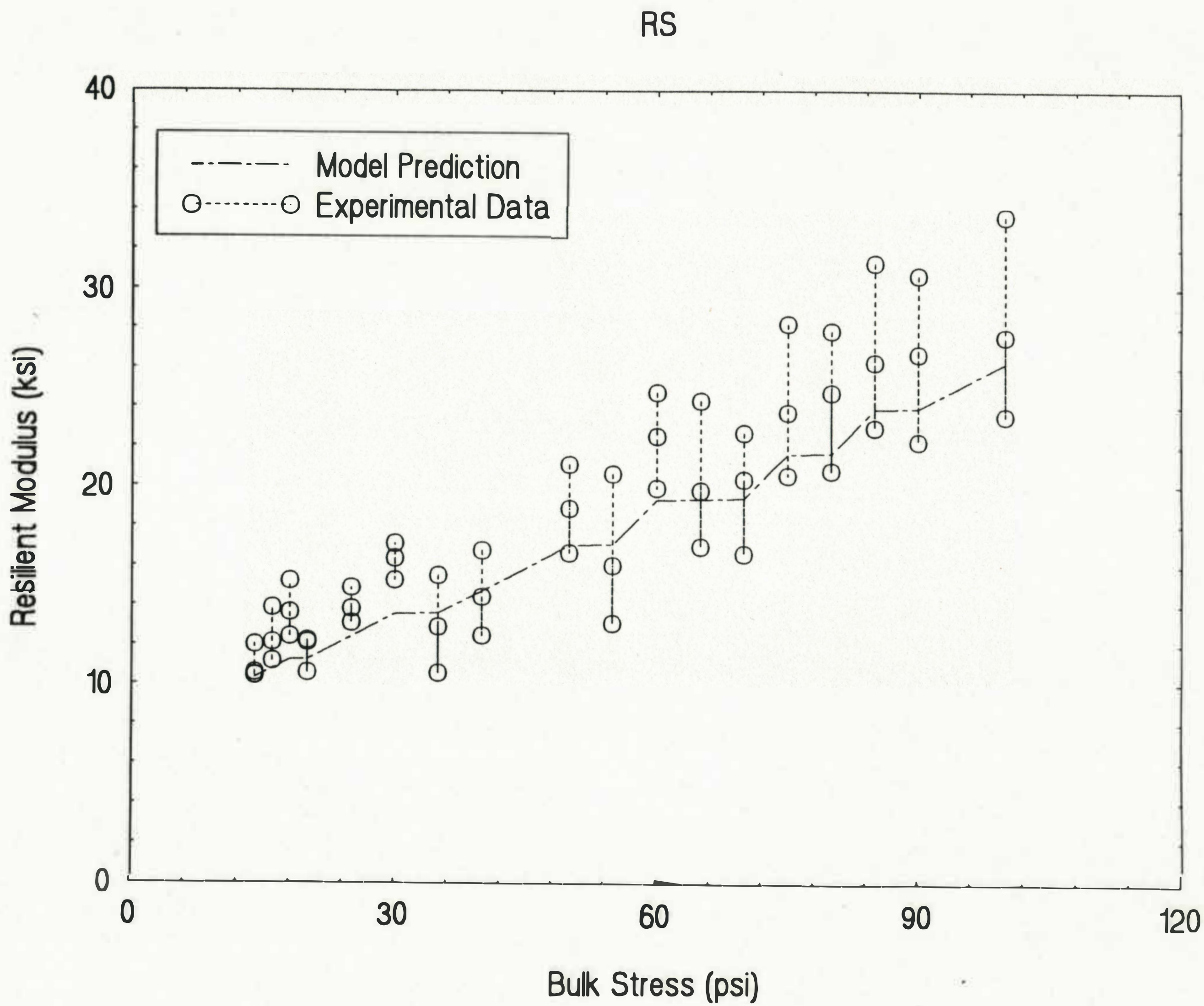

Fig. 4-3 Comparison of Model Prediction (Eq. 4-3) and Experimental Observed RM Values for Aggregate from Comanche County (Limestone)

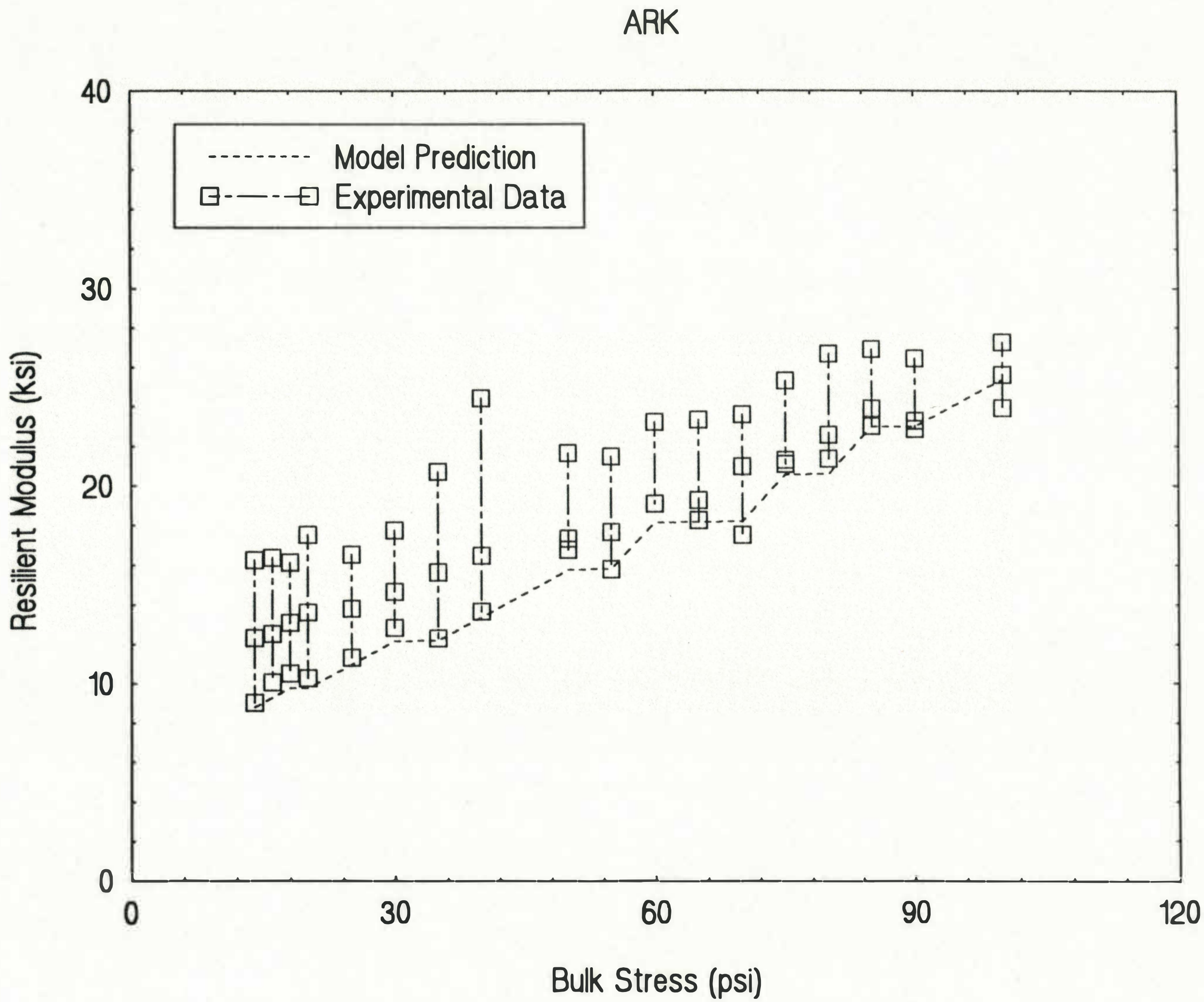

Fig. 4-4 Comparison of Model Prediction (Eq. 4-3) and Experimental Observed RM Values for Aggregate from Cherokee County (Limestone)

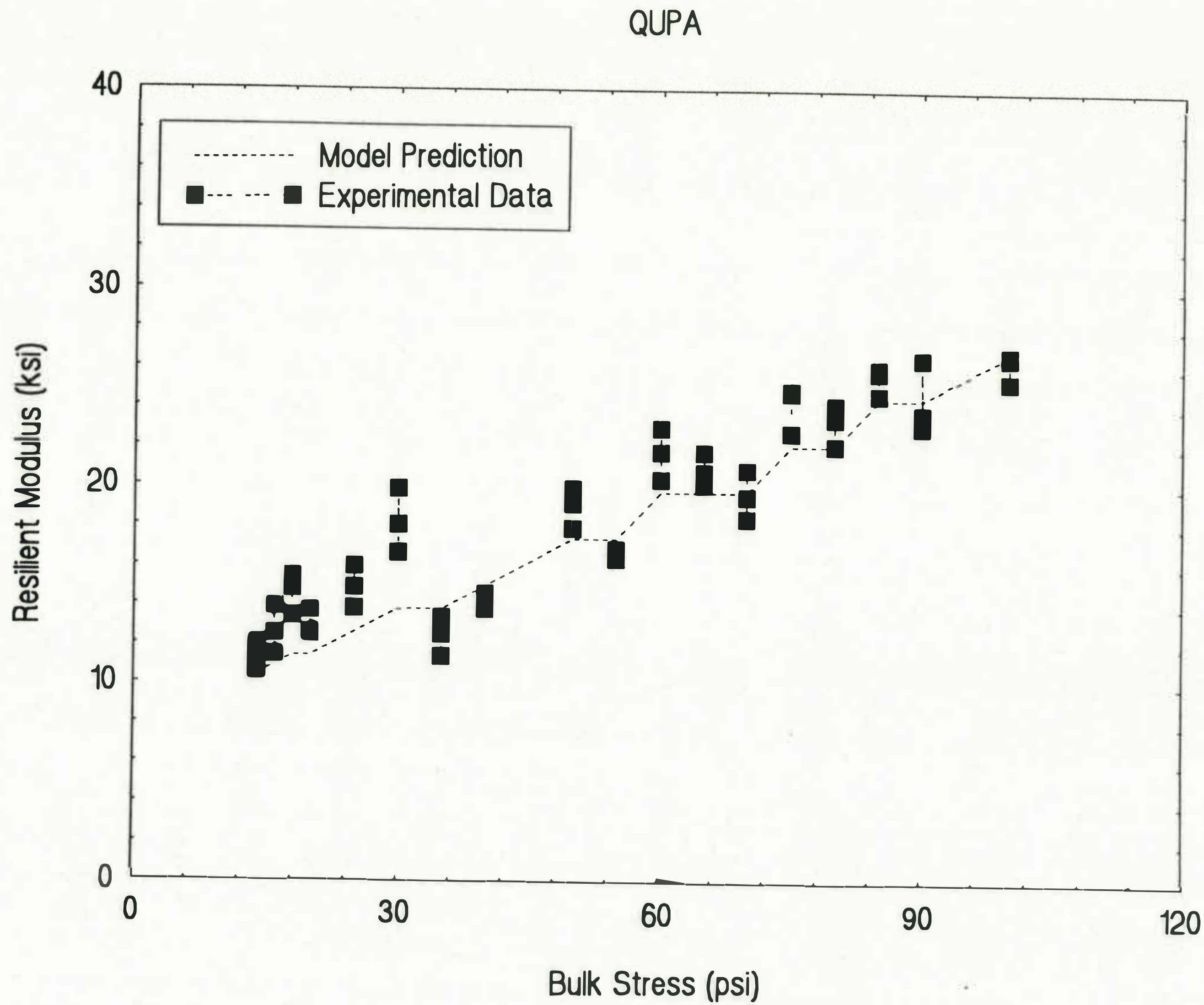

Fig. 4-5 Comparison of Model Prediction (Eq. 4-3) and Experimental Observed RM Values for Aggregate from Creek County (Limestone)

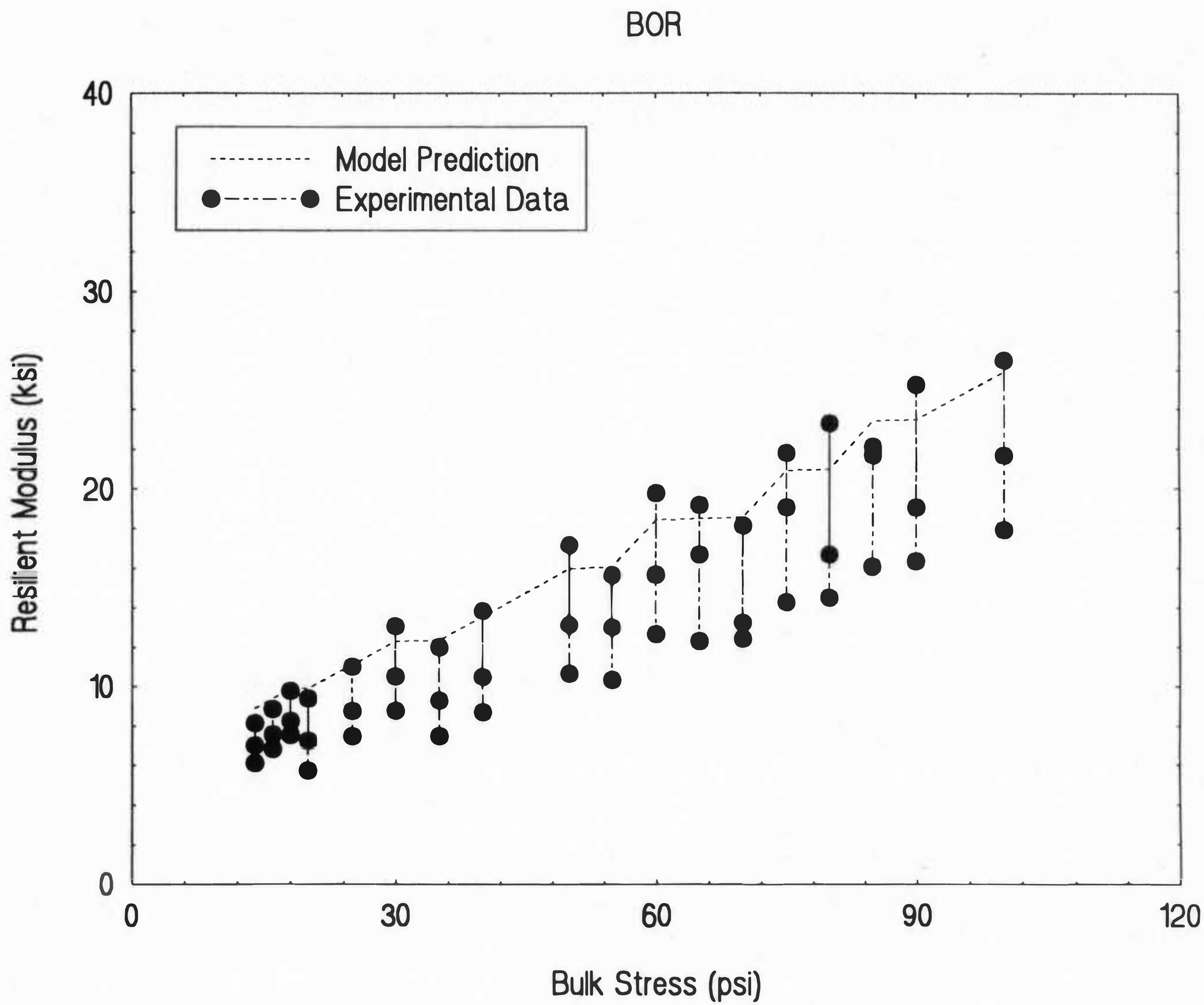

Fig. 4-6 Comparison of Model Prediction (Eq. 4-3) and Experimental Observed RM Values for Aggregate from Choctaw County (Sandstone)

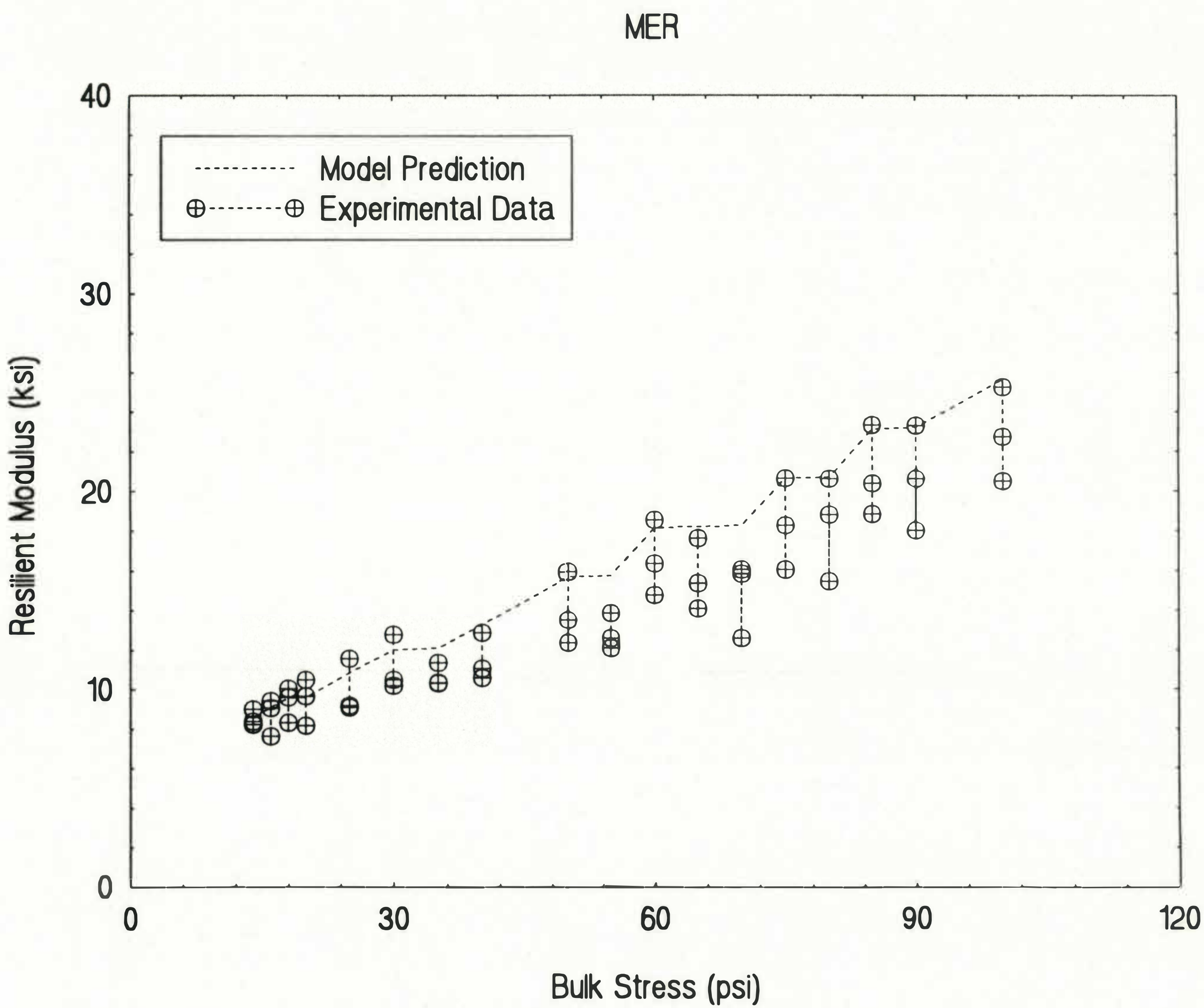

Fig. 4-7 Comparison of Model Prediction (Eq. 4-3) and Experimental Observed RM Values for Aggregate from Johnston County (Granite)

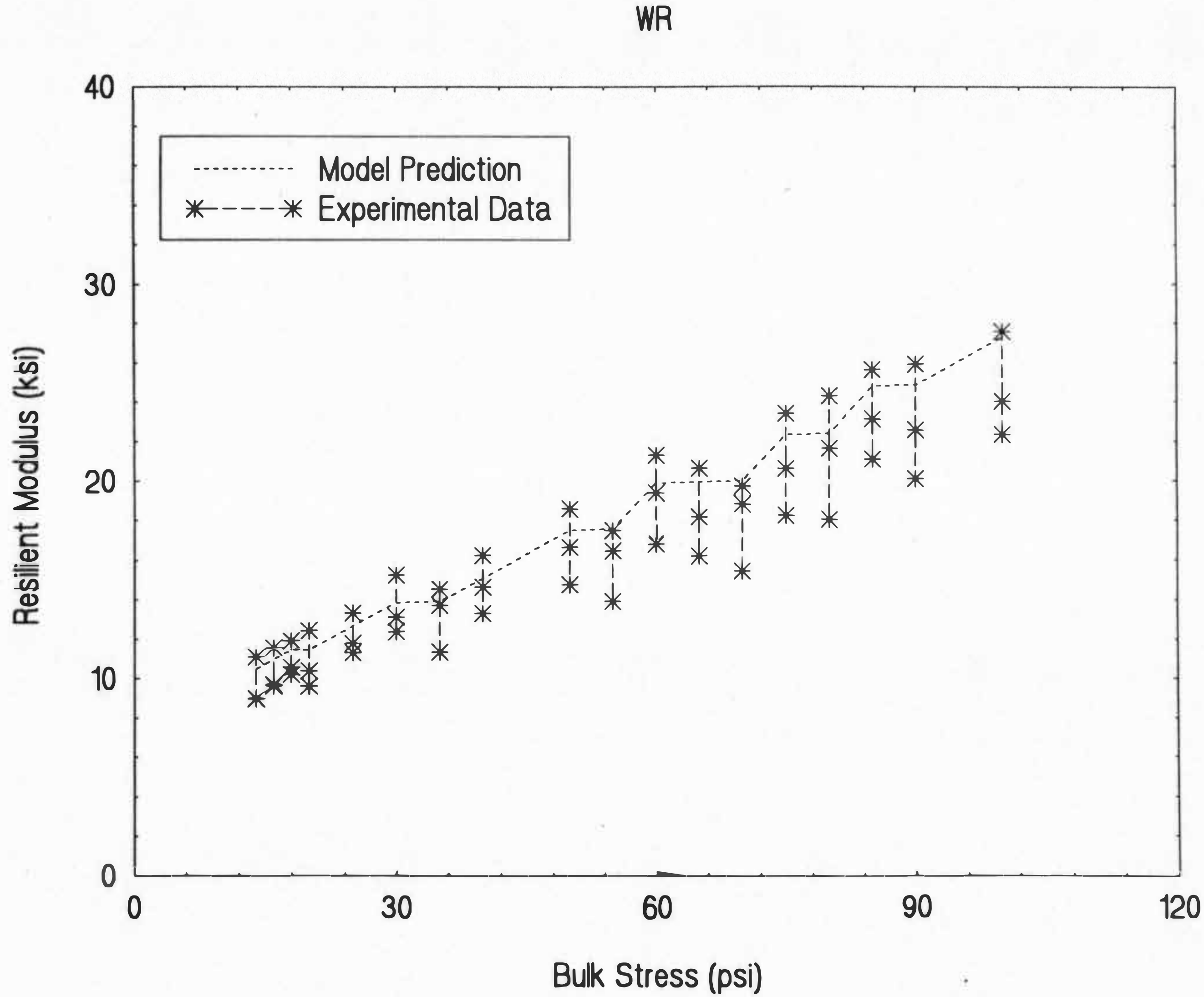

Fig. 4-8 Comparison of Model Prediction (Eq. 4-3) and Experimental Observed RM Values for Aggregate from Murray County (Rhyolite)

Table 4-4 Comparison of Experimental Data and Model Predictions (Eq. 4-3) for Aggregates from Comanche County

Bulk Stress (psi)	Test 1 (ksi)	Test 2 (ksi)	Test 3 (ksi)	Mean (ksi)	Predicted (ksi)	Difference (%)
70	22.75	16.64	20.38	19.92	19.41	3
80	27.93	20.81	24.77	24.5	21.7	11
90	30.73	22.28	26.73	26.58	23.99	10
100	33.73	23.61	27.62	28.32	26.29	7
55	20.64	13.12	16.04	16.6	17.08	3
65	24.38	16.99	19.82	20.4	19.38	5
75	28.28	20.59	23.77	24.21	21.67	11
85	31.35	23.0	26.33	26.9	23.96	11
35	15.55	10.59	12.94	13.03	13.61	4
40	16.8	12.5	14.46	14.59	14.76	1
50	21.12	16.68	18.88	18.89	17.05	10
60	24.79	19.94	22.57	22.43	19.34	14
20	12.25	10.63	12.16	11.68	11.28	3
25	13.87	13.16	14.91	13.98	12.43	11
30	16.39	15.30	17.15	16.28	13.58	17
14	10.48	10.64	12.04	11.05	10.35	6
16	11.22	12.17	13.91	12.43	10.81	13
18	12.47	13.69	15.29	13.82	11.27	18

Table 4-5 Comparison of Experimental Data and Model Predictions (Eq. 4-3) for Aggregates from Cherokee County

Bulk Stress (psi)	Test 1 (ksi)	Test 2 (ksi)	Test 3 (ksi)	Mean (ksi)	Predicted (ksi)	Difference (%)
70	19.45	20.95	17.5	19.3	18.13	6
80	21.97	22.95	21.34	21.95	20.55	6
90	21.77	22.85	23.27	22.63	22.96	1
100	22.45	23.91	25.62	23.99	25.38	6
55	17.67	17.65	15.79	17.04	15.74	8
65	19.22	19.24	18.24	18.9	18.16	4
75	20.84	21.08	21.28	21.07	20.57	2
85	22.17	23.0	23.88	23.02	22.99	0
35	17.05	15.63	12.3	14.99	12.15	19
40	16.5	16.45	13.65	15.53	13.35	14
50	17.83	17.32	16.75	17.3	15.77	9
60	19.12	19.07	19.06	19.08	18.19	5
20	14.43	13.63	10.28	12.78	9.76	24
25	13.60	13.79	11.31	12.9	10.97	15
30	14.61	14.67	12.81	14.03	12.17	13
14	13.38	12.32	9.03	11.58	8.8	24
16	13.48	12.51	10.08	12.02	9.29	23
18	13.29	13.1	10.52	12.3	9.77	21

Table 4-6 Comparison of Experimental Data and Model Predictions (Eq. 4-3) for Aggregates from Creek County

Bulk Stress (psi)	Test 1 (ksi)	Test 2 (ksi)	Test 3 (ksi)	Mean (ksi)	Predicted (ksi)	Difference (%)
70	18.36	20.8	19.44	19.53	19.65	1
80	23.43	24.12	22.07	23.21	22.03	5
90	23.7	26.48	23.03	24.4	24.4	0
100	25.34	26.75	26.6	26.23	26.77	2
55	16.31	16.79	16.3	16.47	17.28	5
65	21.69	20.71	20.05	20.82	19.66	6
75	24.73	24.85	22.71	24.1	22.03	9
85	25.93	25.72	24.62	25.42	24.41	4
35	12.47	13.33	11.33	12.38	13.73	11
40	14.54	14.49	13.75	14.26	14.92	5
50	19.85	19.06	17.83	18.91	17.29	9
60	21.69	22.93	20.31	21.64	19.67	9
20	12.46	13.68	12.56	12.9	11.36	12
25	14.84	15.9	13.77	14.84	12.55	15
30	17.97	19.77	16.58	18.11	13.74	24
14	11.31	12.03	10.58	11.31	10.42	8
16	13.86	12.48	11.43	12.59	10.89	13
18	15.4	14.77	13.38	14.52	11.37	22

Table 4-7 Comparison of Experimental Data and Model Predictions (Eq. 4-3) for Aggregates from Choctaw County

Bulk Stress (psi)	Test 1 (ksi)	Test 2 (ksi)	Test 3 (ksi)	Mean (ksi)	Predicted (ksi)	Difference (%)
70	13.22	18.11	12.42	14.58	18.4	26
80	16.67	23.29	14.52	18.16	20.9	15
90	19.03	25.27	16.32	20.21	23.41	16
100	21.68	26.49	17.92	22.03	25.91	18
55	12.99	15.62	10.33	12.98	15.96	23
65	16.67	19.15	12.28	16.03	18.47	15
75	19.03	21.8	14.27	18.37	20.98	14
85	21.68	22.13	16.04	19.95	23.48	18
35	9.3	11.99	7.5	9.6	12.28	28
40	10.48	13.84	8.69	11	13.53	23
50	13.12	17.15	10.63	13.63	16.04	18
60	15.65	19.74	12.64	16.01	18.54	16
20	7.24	9.38	5.74	7.45	9.85	32
25	8.76	11.0	7.5	9.09	11.1	22
30	10.5	13.06	8.8	10.79	12.35	15
14	7.02	8.13	6.12	7.09	8.87	25
16	7.57	8.85	6.85	7.76	9.37	21
18	8.25	9.77	7.55	8.52	9.88	16

Table 4-8 Comparison of Experimental Data and Model Predictions (Eq. 4-3) for Aggregates from Johnston County

Bulk Stress (psi)	Test 1 (ksi)	Test 2 (ksi)	Test 3 (ksi)	Mean (ksi)	Predicted (ksi)	Difference (%)
70	16.05	12.59	15.85	14.83	18.12	22
80	18.81	15.46	20.62	18.3	20.63	13
90	20.62	18.0	23.29	20.64	23.13	12
100	22.74	20.49	25.26	22.83	25.64	12
55	12.57	12.07	13.86	12.83	15.69	22
65	15.36	14.08	17.61	15.68	18.19	16
75	18.26	16.05	20.64	18.32	20.7	13
85	20.38	18.83	23.34	20.85	23.21	11
35	10.29	10.34	11.34	10.66	12.0	13
40	11.05	10.58	12.86	11.5	13.26	15
50	13.51	12.35	15.94	13.93	15.76	13
60	16.33	14.77	18.53	16.54	18.27	10
20	8.18	10.47	9.65	9.43	9.57	1
25	9.18	9.1	11.54	9.94	10.82	9
30	10.47	10.18	12.77	11.14	12.08	8
14	9.0	8.23	8.37	8.53	8.6	1
16	9.43	7.64	9.06	8.71	9.1	4
18	9.58	8.32	10.04	9.31	9.6	3

Table 4-9 Comparison of Experimental Data and Model Predictions (Eq. 4-3) for Aggregates from Murray County

Bulk Stress (psi)	Test 1 (ksi)	Test 2 (ksi)	Test 3 (ksi)	Mean (ksi)	Predicted (ksi)	Difference (%)
70	19.74	18.84	15.49	18.02	19.91	10
80	24.33	21.66	18.06	21.35	22.37	5
90	25.93	22.6	20.09	22.87	24.83	9
100	27.6	24.04	22.36	24.67	27.29	11
55	17.46	16.45	13.91	15.94	17.5	10
65	20.66	18.18	16.23	18.36	19.96	9
75	23.45	20.63	18.27	20.78	22.42	8
85	25.68	23.14	21.12	23.31	24.88	7
35	14.54	13.72	11.33	13.2	13.86	5
40	16.24	14.65	13.28	14.72	15.09	2
50	18.59	16.63	14.77	16.66	17.55	5
60	21.32	19.39	16.82	19.18	20.01	4
20	12.45	10.39	9.62	10.82	11.45	6
25	13.32	11.79	11.3	12.14	12.68	4
30	15.28	13.12	12.39	13.6	13.91	2
14	11.09	8.99	8.96	9.68	10.49	8
16	11.58	9.71	9.62	10.3	10.98	7
18	11.90	10.57	10.2	10.89	11.47	5

Table 4-10 R^2 for Different Aggregate Types By Using Eq. 4-3

County	Material	R^2
Comanche (rs)	Limestone	.7398
Cherokee (ark)	Limestone	.7314
Creek (qupa)	Limestone	.8345
Choctaw (bor)	Sandstone	.5374
Johnston (mer)	Granite	.7345
Murray (wr)	Rhyolite	.8240

values for these six aggregate types are presented in Table 4-10. To compare the overall model predictions vs. experimental data, the average RM for each aggregate type and the corresponding model predictions are grouped together and presented in the Fig. 4-9. It may be noted that the total number of curves in the Fig. 4-9 is 12 (six average RM curves from six aggregate types and six model predictions). The model fits the experimental data extremely well. Consequently, it may be advanced that the correlation of cohesion and friction angle with RM is better than that with CBR. A possible explanation is that deformation characteristics for the conventional triaxial compression test and RM test are more similar than those between the RM and CBR tests.

4.3 CORRELATION WITH ELASTICITY

By referring to Table 3-14, it is observed that the Elasticity (E) increases with confining pressures. The relationship between confining pressure (σ_3) and E for these six aggregate types can be expressed as

$$E \text{ (in psi)} = a_0 + a_1 * \sigma_3 \tag{4-4}$$

Where a_0 and a_1 are the material constants. The values for a_0 and a_1 for the six aggregate types are presented in the Table 4-11.

An attempt was made to find the correlation between E and RM in the form of

$$RM \text{ (in psi)} = (a_0 + a_1 * \sigma_3) * B_1 + B_2 * \theta \tag{4-5}$$

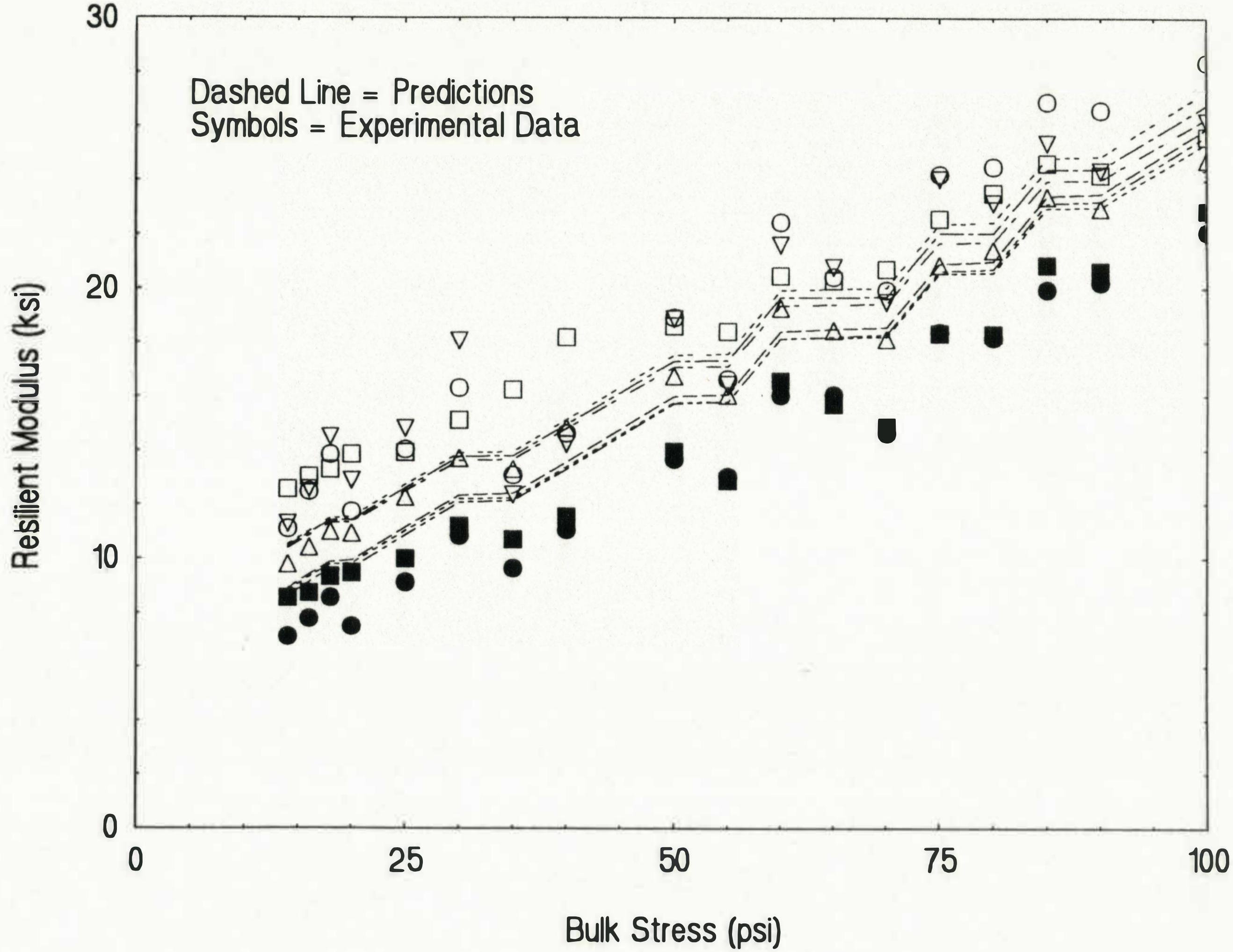

Fig. 4-9 Overall Comparison of Model Prediction (Eq. 4-3) and Average Resilient Moduli for Six Different Aggregate Types

Table 4-11 The Material Constants (a_0 and a_1) for Different Aggregate Types

County	Material	Confining Pressure (σ_3) (psi)	Young's Modulus (psi)	a_0	a_1
Comanche (RS)	Limestone	10	24,100	20.2	0.37
		15	25,400		
		20	27,800		
Cherokee (Ark)	Limestone	5	22,600	21.3	0.07
		10	20,000		
		15	23,300		
Creek (Qupa)	Limestone	10	23,500	12.0	1.11
		15	27,900		
		20	34,600		
Choctaw (Bor)	Sandstone	5	22,700	20.3	0.44
		10	24,400		
		15	27,100		
Johnston (Mer)	Granite	5	20,200	17.8	0.51
		10	23,100		
		15	25,300		
Murray (WR)	Rhyolite	5	20,500	18.1	0.45
		10	22,400		
		15	25,000		

where a_0 and a_1 have the same meaning as given in the Eq. 4-4. B_1 and B_2 are the regression constants. σ_3 and θ are the confining pressure and bulk stress, respectively.

The regression constants were found to be $B_1 = 0.4098$ and $B_2 = 150.76$. To illustrate the comparison between model predictions (Eqs. 4-3 and 4-6) and experimental observations, the results are grouped together and presented in the Figs. 4-10 to 4-15. By referring to Figs. 4-10 to 4-15, it is found that the model prediction for Eq. 4-3 (correlation of cohesion and friction angle with RM) has a better agreement with experimental observations than those obtained by using Eq. 4-6 (correlation of Elasticity (E) with RM). The reason for that may be attributed to the difficulty in determining the initial tangent slope (E) which leads to inconsistent results among aggregate types. On the other hand, the determination of cohesion and friction angle is much easier from the Mohr diagrams which leads to more consistent results among aggregate types.

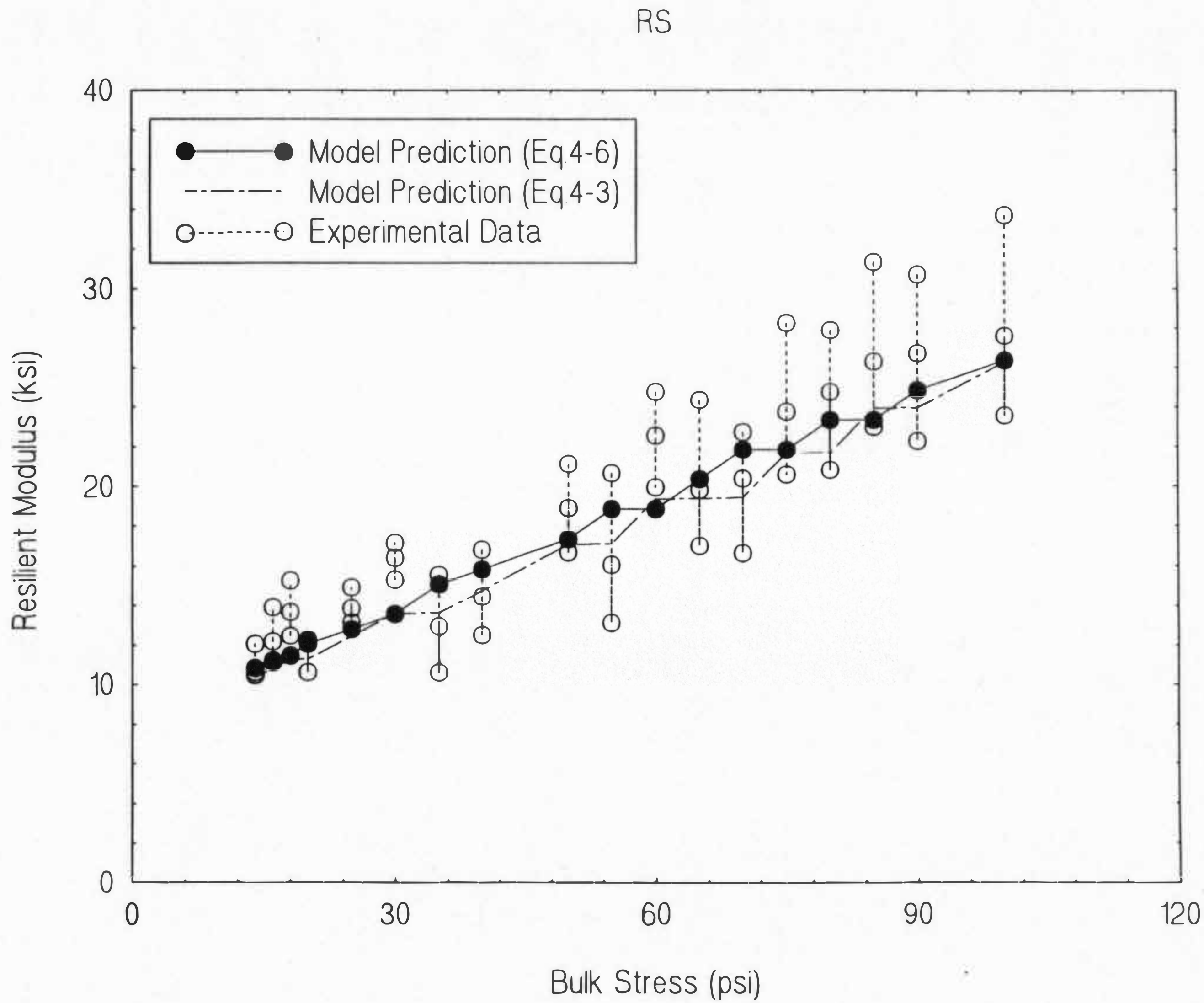

Fig. 4-10 **Comparison of Model Predictions (Eqs. 4-3 and 4-6) and Experimental Observed RM Values for Aggregate from Comanche County (Limestone)**

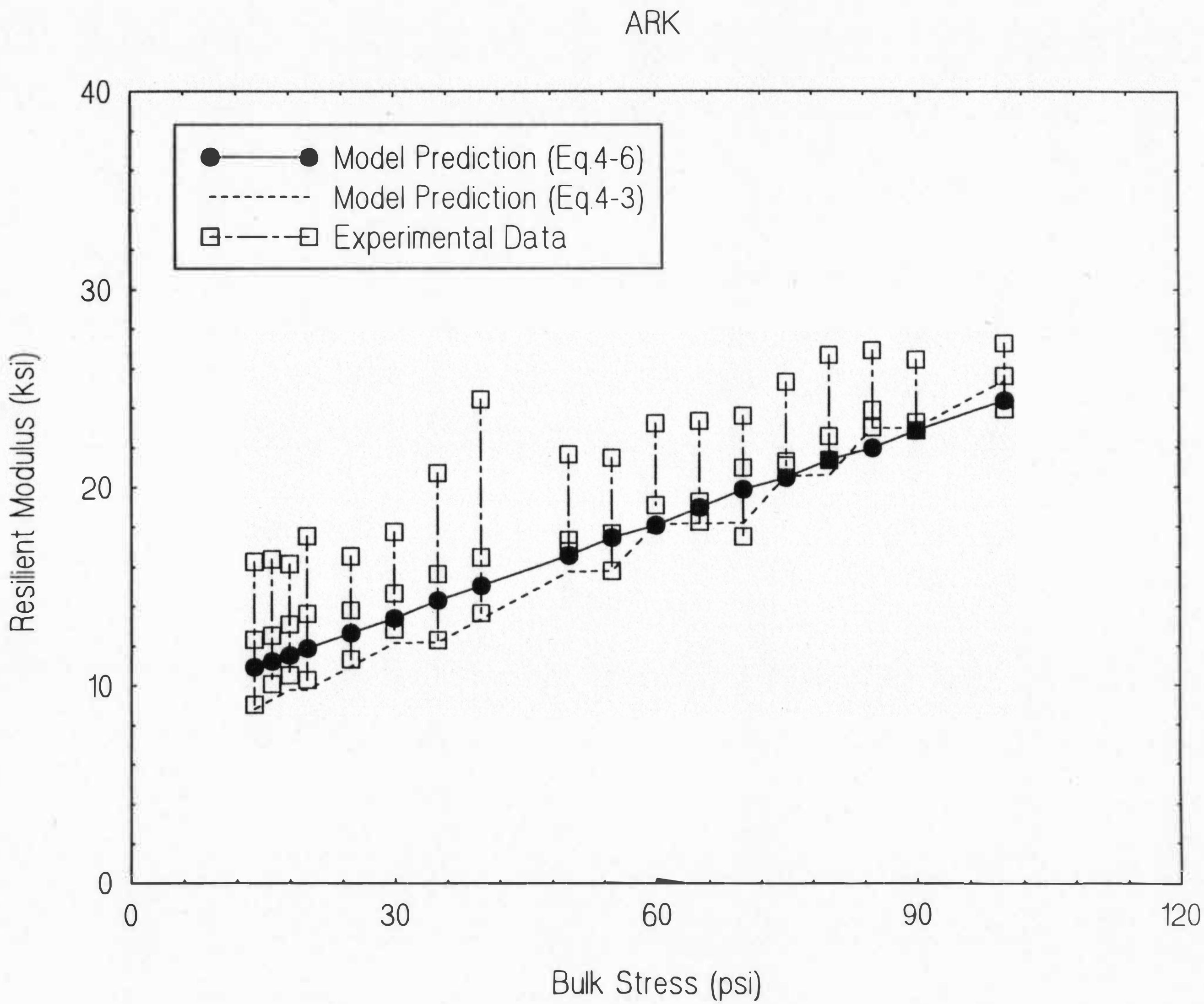

Fig. 4-11 **Comparison of Model Predictions (Eqs. 4-3 and 4-6) and Experimental Observed RM Values for Aggregate from Cherokee County (Limestone)**

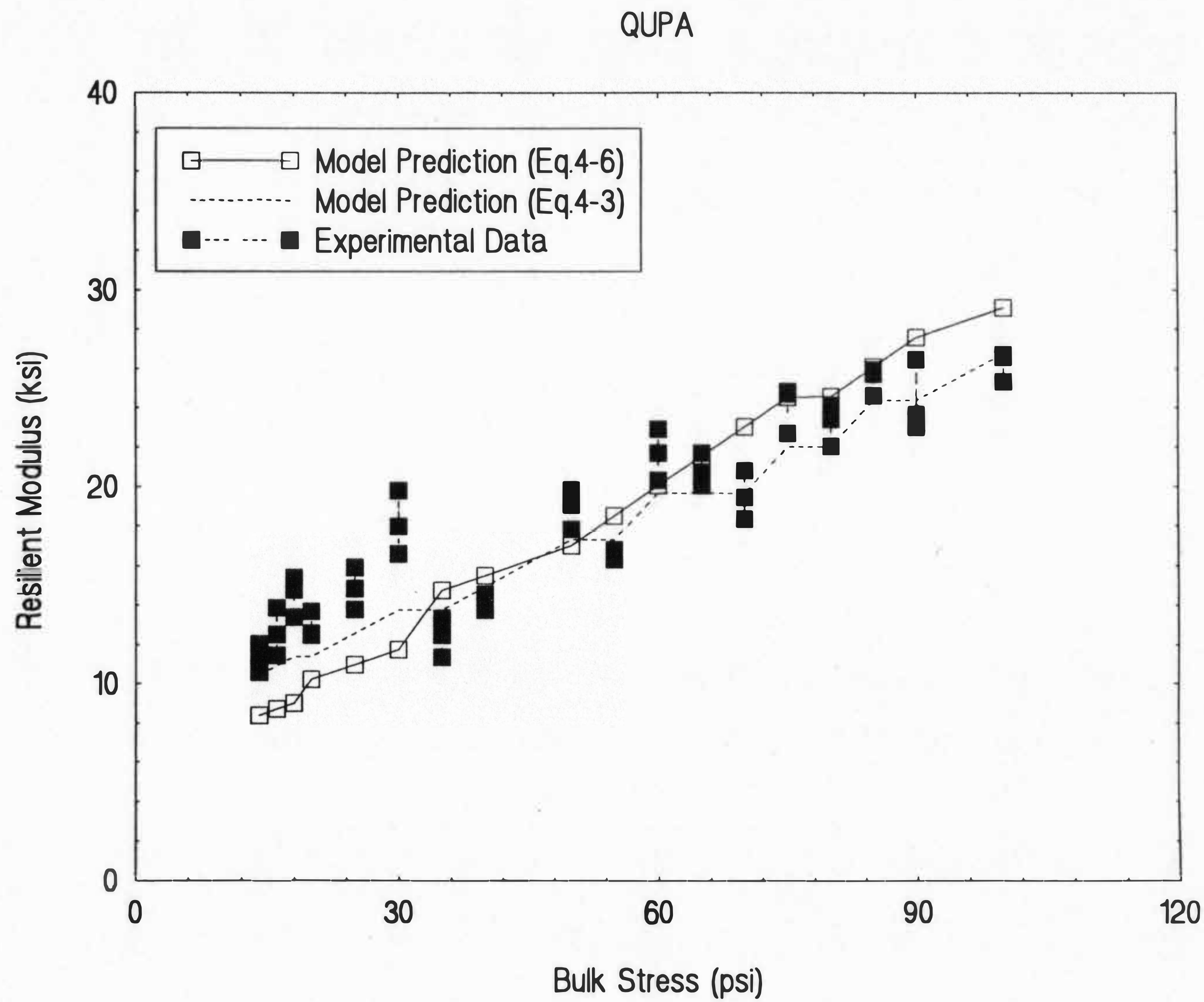

Fig. 4-12 Comparison of Model Predictions (Eqs. 4-3 and 4-6) and Experimental Observed RM Values for Aggregate from Creek County (Limestone)

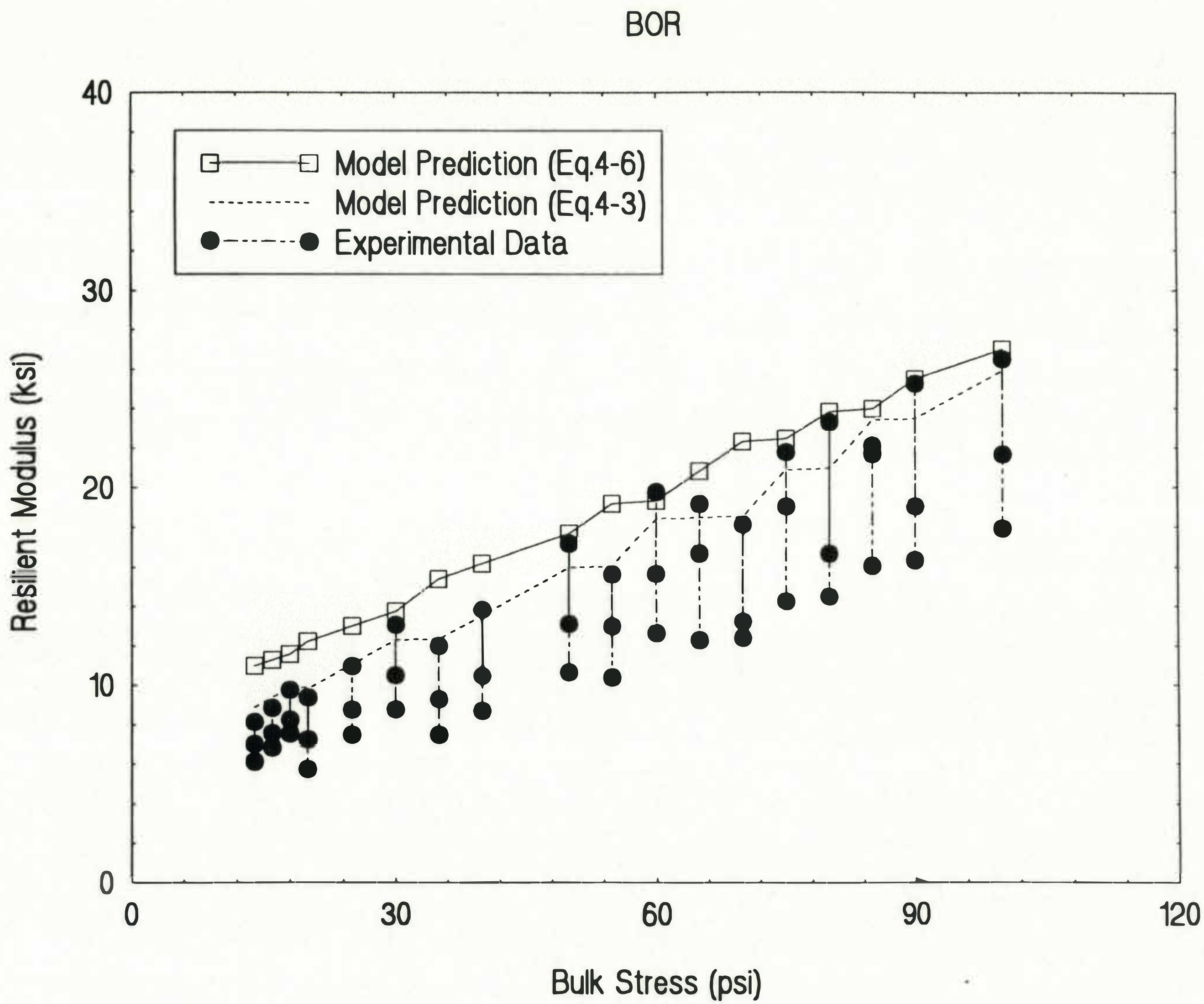

Fig. 4-13 Comparison of Model Predictions (Eqs. 4-3 and 4-6) and Experimental Observed RM Values for Aggregate from Choctaw County (Sandstone)

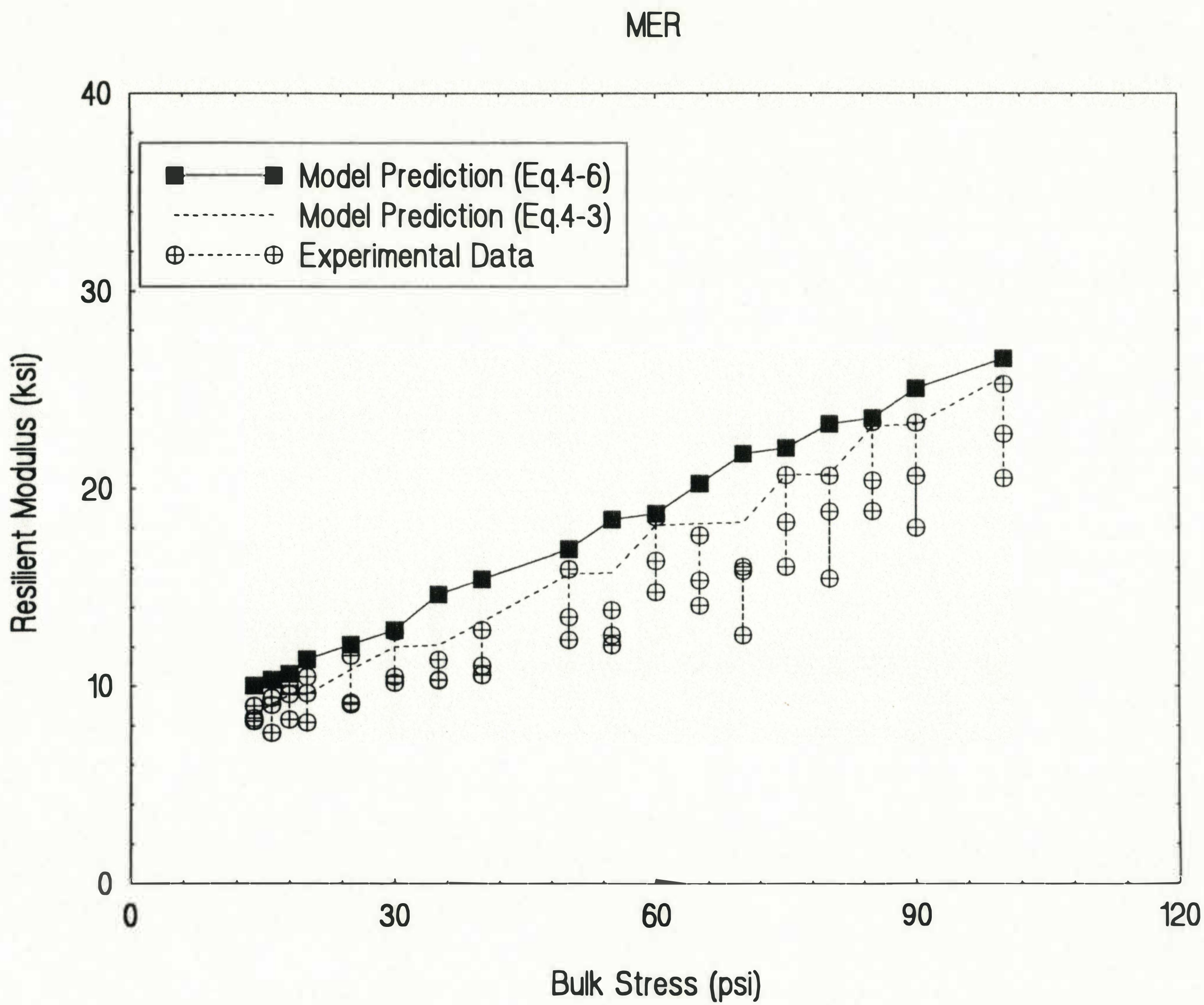

Fig. 4-14 Comparison of Model Predictions (Eqs. 4-3 and 4-6) and Experimental Observed RM Values for Aggregate from Johnston County (Granite)

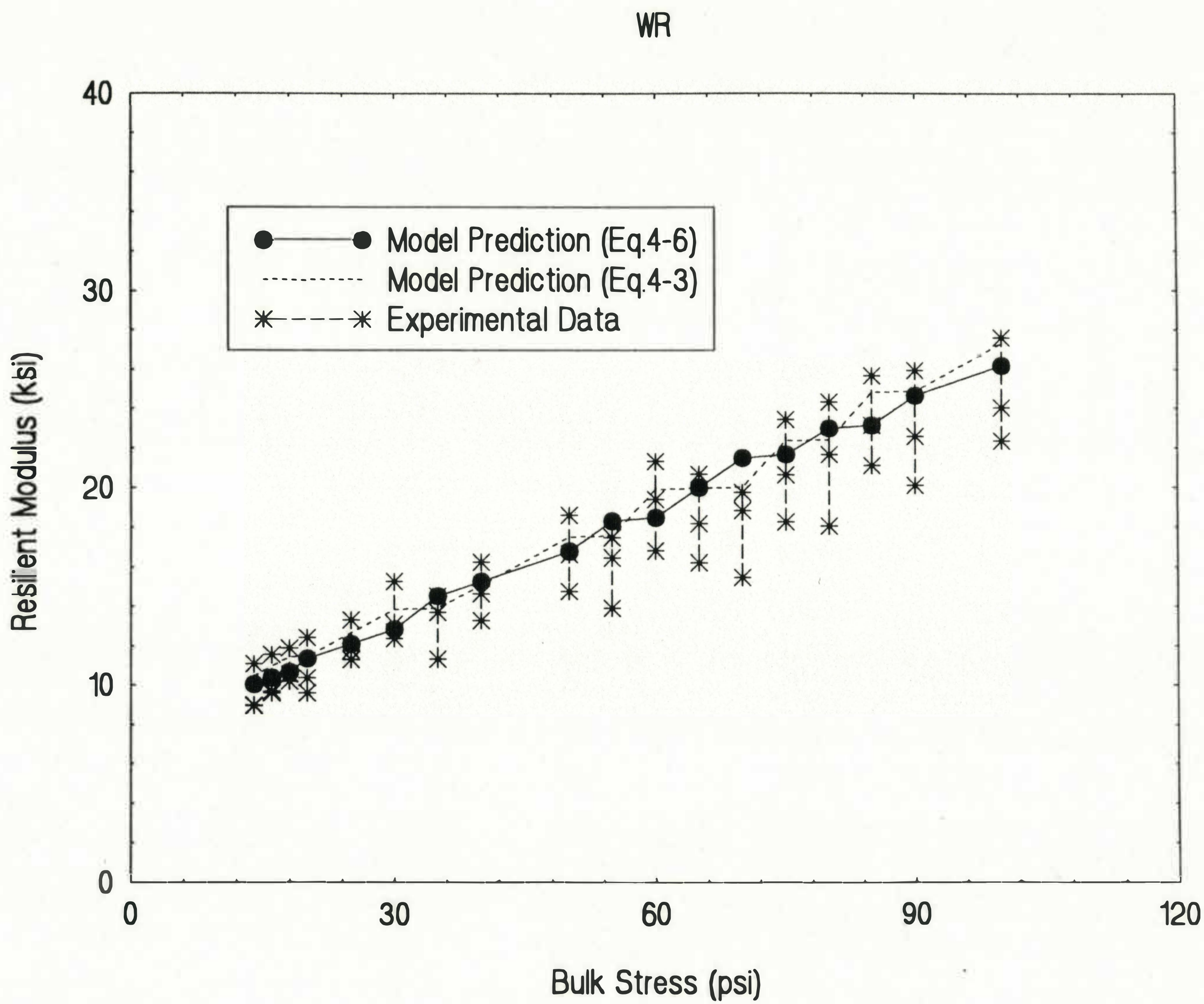

Fig. 4-15 Comparison of Model Predictions (Eqs. 4-3 and 4-6) and Experimental Observed RM Values for Aggregate from Murray County (Rhyolite)

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

The AASHTO T292-91I <8> and the AASHTO T294-92I <9> were used to conduct the Resilient Modulus tests of the six aggregate types. The effects of compaction method (hammer vs. vibratory table), testing procedures (T292-91I and T294-92I), specimen size, gradation and degree of saturation on RM values were investigated. The RM values were correlated with the CBR, Elasticity, and cohesion and friction angle of the material. Based on the data obtained the following observations are made:

1. The 4" specimens prepared according AASHTO T180-90D <27> had higher dry densities than those prepared by vibratory compaction and they yielded higher RM values. However, for 6" specimens both T180-90D and vibratory table compaction methods developed in this study produced similar densities. Also, it was found that the compaction method used had minimum effects on RM for 6" specimens.
2. The T294-92I testing procedure gave higher resilient moduli than those obtained by using the T292-91I testing procedure, possibly because the cyclic stress had a stiffening and strengthening effect on the specimen structure as the stress level increases from low to high.

3. In all cases, the RM values for 4" specimens were higher than those for 6" specimens.

4. Gradation influenced the density of the specimens, however, its influence on RM values was less significant compared to the effects of moisture content and the stress state.

5. The moisture content effects were of an exploratory nature. The RM values only for both 4" and 6" specimens appear to decrease due to saturation with the 4" specimens experiencing a higher degree of strength loss due to soaking.

6. For a given gradation, the resilient modulus values of the six aggregate types at the same bulk stress are relatively close.

7. The regression analysis demonstrated that it is possible to reliably determine the resilient modulus of aggregate through indirect methods that are easy and inexpensive. The best correlations exist between the RM values and the cohesion and friction angle.

8. The correlation of CBR with RM values obtained in this study showed a very general, but varying correlation. It is better to use the average CBR value and the corresponding B values of 193, 96, 82 and 74 at θ=100, 30, 20 and 14 psi, respectively; this is significantly lower than the values (740, 440, 340 and 250 at θ=100, 30, 20 and 10 psi, respectively) suggested by AASHTO Design Guide 1986 <24>.

5.2 RECOMMENDATIONS

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

1. To study the variations of cohesion, friction angle and elastic modulus due to the effects of conditioning.

2. To study the varying moisture contents in the range from slightly dry (compared to the optimum moisture content (OMC)) to fully saturated.

3. To study the RM values and matrix characteristics of stabilized aggregates (fly ash & cement) since such bases/subbases are likely to be used in the future in Oklahoma.

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