Resilient Moduli of Aggregate Materials: Variability Due to Testing Procedure and Aggregate Type

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The variability of resilient modulus (RM) values is investigated for six aggregate materials that are commonly used in Oklahoma as subbases or bases. Six RM tests each under identical conditions for two aggregate types and three tests each for four aggregate types were performed according to the AASHTO T292–91I procedure to investigate the variability of the test results. All specimens were prepared at the same gradation that meets the Oklahoma Department of Transportation specifications for Type A materials. The effects of testing procedures on RM values were investigated for two selected aggregate types by conducting six tests each using the AASHTO T292–91I and T294–92I testing procedures. The variability of test data and consistency of the RM values are investigated within a statistical framework. The variability of RM values obtained ranges from 19 to 26 percent in terms of maximum coefficient of variation (MCOV). The results indicate that consistently higher RM values are obtained when using the T294–92I testing method than when using the T292–91I testing method, and the degree of increase depends on the type of aggregate. The variability of RM values due to testing procedure was found to be higher than that due to aggregate source. One aggregate type experienced a lower MCOV when using T294–92I than when using T292–91I, but the other aggregate type exhibited an opposite trend. Overall the RM values range from 41.3 to 261.8 MPa, depending on bulk stress, material type, and testing method, and they are slightly lower than those reported in the literature.

A reliable and economic design of pavement thickness relies on several factors, among which a proper characterization of the load-deformation response of the pavement materials is extremely important. AASHTO proposed a new pavement design procedure in 1986 (*1*) that incorporates the resilient modulus (RM) to properly describe the behavior of pavement materials subjected to moving traffic. Until now, however, no standardized test method has been adopted for measuring RM values. Since its introduction the T274–82 testing procedure (*2*) has been the target of widespread criticism (*3*), including the criticism that the required loading conditions are too severe and therefore a specimen may fail in the conditioning stage. For example, in documenting unsatisfactory experience with AASHTO T274–82, Vinson (*4*) stated that the heavy conditioning of the sample specimens, as required by T274–82, may cause different levels and types of stresses for both cohesive and cohesionless soils. Also, Ho (*5*) observed that the conditioning stage, as suggested by T274–82, was very severe for many soils. For these reasons various departments of transportation (DOTs), including those in Florida, New York, Illinois, and South Dakota, have developed their own testing procedures. A review of these methods reveals that they are similar to AASHTO T274–82 except for some factors pertaining to sample conditioning, load magnitude, and load application sequences (*6*). In 1991 AASHTO modified the T274–82 testing procedure and came up with the T292–91I testing procedure (*7*). In 1992 AASHTO adopted the Strategic Highway Research Program test method of determination of RM for soils and unbound aggregates, now known as T294–92I (*8*). The different testing procedures may result in different RM values. Also, the AASHTO *Guide for Design of Pavement Structures* (*1*) incorporates reliability in the design equation, which requires an understanding of the material strength variance. Therefore it is important to investigate the effects of pertinent factors on RM and the variability of the RM values due to the different testing procedures.

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The need for nonlinear characterization of granular base/subbase in the pavement structure has received attention in recent years (*9,10*). The associated material parameters that are evaluated either in the field (e.g., falling weight deflectometer test) or in the laboratory (e.g., cyclic triaxial test) involve some degree of difficulty. For example Parker (*9*) stated that the granular base/subbase is the most difficult paving material to characterize because the modulus is sensitive to the state of stress and there may be influential seasonal variations. Rada et al. (*10*) reported that the results for the unbound granular base and subbase materials are considerably more variable than those for other layered materials.

In this paper the variability of RM test data is investigated for six different aggregate materials under repeated dynamic loading by using the AASHTO T292–91I testing procedure. Two of the aggregate types were selected for RM testing by using T294–92I, and the results were compared with those obtained with T292–91I as well as with those reported by various agencies. Research is under way to investigate the variability of the RM test results under various moisture contents (soaked and dried conditions) and the effect of stabilizing agents (fly ash, lime, and cement).

RESILIENT MODULUS CONCEPT

Design of roadway pavements relies on proper characterization of the load-deformation responses of the associated materials from base, subbase, and subgrade. Subgrade soils undergo deformation when subjected to repeated loads from moving traffic. Laboratory results indicate that part of this deformation is resilient or recoverable (ϵ_r) and part is permanent or plastic (ϵ_p). The property that describes this behavior of subgrade materials is RM, defined as the deviatoric dynamic stress σ_d divided by the resilient strain ϵ_r:

$$RM = \sigma_d/\epsilon_r \quad (1)$$

The basic differences among test methods T274–82, T292–91I, and T294–92I are sample conditioning before testing, number of loading cycles, and applied waveform and sequence. Test methods T292–91I and T294–92I were used in this study and their comparisons are summarized in Table 1. The testing parameters, such as the stresses applied and their duration and the selection of dynamic wave form and the number of repetitions, may affect RM values, as discussed in the following sections.

Applied Stress and Duration

Dynamic response of aggregate-based materials can be significantly influenced by the applied confining pressure (*11*). To better characterize such materials, it is desirable to evaluate RM tests under a wide range of confining pressures expected within the subgrades or bases/subbases. The AASHTO procedures (T274–82, T292–91I, and T294–92I) use a variety of constant confining pressures and dynamic deviatoric stresses; therefore, the test data comprise a set of RM values corresponding to the different bulk stresses. However Khedr (*12*) argued that tests involving constant confining pressure do not simulate the in situ conditions properly because the lateral pressure (confining pressure in this case) changes simultaneously with vertical stresses caused by traffic loading. In addition, tests involving the constant confining pressure instead of the cyclic confining pressure may lead to an overestimation of the RM values (*12,13*). In contrast Thompson (*14*) reported that for practical purposes the triaxial RMs are similar under constant and variable confining pressures. TRB (*15*) 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 testing.

The AASHTO testing procedures (T292–91I and T294–92I) require bulk stresses θ (defined by $\theta = \sigma_1 + \sigma_2 + \sigma_3$) as high as 551.2 to 689.0 kPa (80 to 100 psi). These values appear to be much higher than the stresses prevailing in the field (*3,5,16*). As reported by Thompson and Smith (*17*), 137.8 kPa is a representative value of the bulk stress in the mid-depth of a granular base with a thickness of 30.48 cm below a 7.62-cm asphalt concrete surface.

RM is only minimally affected by variations in stress pulse duration. In fact, Kalcheff and Hicks (*18*) demonstrated that the RM values are not greatly influenced in a case in which the stress pulse is applied rapidly and then sustained compared with the case in which the stress pulse is applied rapidly and released for a short duration, provided that the magnitude of the stress pulse is equal in both cases.

TABLE 1 Comparison of Testing Procedures

AASHTO (T292-91I [5])			AASHTO (T294-92I [6])		
σ_c kPa	σ_d kPa	No. of Cycles	σ_c kPa	σ_d kPa	No. of Cycles
137.8*	103.4*	1000*	103.4*	103.4*	1000*
137.8	68.9	50	20.7	20.7	100
137.8	137.8	50	20.7	41.3	100
137.8	206.7	50	20.7	62.0	100
137.8	275.6	50	34.5	34.5	100
103.4	68.9	50	34.5	68.9	100
103.4	137.8	50	34.5	103.4	100
103.4	206.7	50	68.9	68.9	100
103.4	275.6	50	68.9	137.8	100
68.9	34.5	50	68.9	206.7	100
68.9	68.9	50	103.4	68.9	100
68.9	137.8	50	103.4	103.4	100
68.9	206.7	50	103.4	206.7	100
34.5	34.5	50	137.8	103.4	100
34.5	68.9	50	137.8	137.8	100
34.5	103.4	50	137.8	275.6	100
20.7	34.5	50			
20.7	48.2	50			
20.7	62.0	50			

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

σ_c and σ_d denote chamber confining pressure and deviator stress, respectively.

The conversion factor (1 psi = 6.89 kPa) was used in this Table.

Dynamic Waveform 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 in. below the surface of the pavement at the Stockton test track. Because of the limitations of their test equipment they did not use the actual form of the vertical stress that was observed; instead they chose a square wave. Terrel et al. (*20*) also studied the influence of the shape of the wave pulse on the total and resilient strains induced in an asphalt-treated base material. They found that the triangular and the sinusoidal stress pulses produce similar effects on the resilience characteristics of the materials. It was also concluded that a square vertical stress pulse is a reasonable approximation of the actual conditions within a pavement structure.

AASHTO T292–91I suggests that the triangular and rectangular wave forms are applicable to RM testing of subgrade soils and base/subbase materials for simulating traffic loading. However T294–92I recommends that the haversine-shaped load pulse with a 0.1-sec load followed by a 0.9-sec rest period be used for both soil and granular materials. T292-91I specifies a fixed load duration of between 0.1 and 1.0 sec and a fixed cycle duration of between 1.0 and 3.0 sec.

To determine the number of repetitions necessary to reach the stable permanent deformation, AASHTO T292–91I suggests comparing the recoverable axial deformation at the 20th and 50th repetition. If the difference is greater than 5 percent, an additional 50 repetitions are necessary at that stress state. Thompson (*14*) reported that for granular materials the RM response after a limited number of load repetitions, such as 100, is representative of the response determined after several thousand repetitions because generally granular materials will achieve a stable permanent deformation after about 100 load repetitions.

MATERIAL ORIGIN AND ENGINEERING INDEX PROPERTIES

Six aggregate types used in Oklahoma as bases/subbases were selected for this study: three limestones, one sandstone, one granite, and one rhyolite. The engineering properties—liquid limit (LL), plasticity index (PI), maximum dry density (MDD), optimum moisture content (OMC), specific gravity (SG), cohesion (C), friction angle (ϕ) and California bearing ratio (CBR)—for these six types of aggregates were evaluated. A summary of the test results is presented in Table 2. It may be noted that following the repeated triaxial testing, static triaxial compression tests were performed to

TABLE 2 Summary of Index Properties

County	Material	LL (%)	PI	MDD g/cm³ (pcf)	OMC (%)	SG	C kPa (psi)	ϕ (°)	CBR
Comanche	Limestone	16	1	2.40 (150)	5.6	2.66	124 (18)	41	67
Cherokee	Limestone	16	1	2.39 (149)	5.2	2.64	96.5 (14)	45	132
Creek	Limestone	15	NP*	2.42 (151)	5.5	2.78	124 (18)	43	116
Choctaw	Sandstone	14	NP*	2.35 (147)	5.9	2.53	82.7 (12)	46	284
Johnston	Granite	15	NP*	2.34 (146)	5.4	2.62	75.8 (11)	46	226
Murray	Rhyolite	16	NP*	2.40 (150)	6.0	2.72	110.2 (16)	46	150

* NP denotes nonplastic material

obtain the cohesion (C) and friction angle (ϕ) of the material (aggregate). The repeated triaxial tests served as a "conditioning" of the sample for triaxial compression tests that could be imposed by moving vehicles. Thompson and Smith (*17*) reported that the shear strength of an unconditioned specimen does not represent the strength of an in-service compacted granular base material subjected to traffic loading.

TRIAXIAL SPECIMEN PREPARATION

Although the 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) (*7,11*). Accordingly any method of compaction that produces the desired dry density is suitable. Vibratory compaction, for example, has been used successfully by Hicks (*21*) and Laguros et al. (*22*) and recommended by AASHTO T292-91I and T294–92I. AASHTO T294–92I suggests using the OMC and MDD for a given aggregate type in accordance with T180–90D (*23*), then using the OMC and 95 percent of the MDD for specimen preparation.

A split mold was designed and fabricated for this study with provisions to apply a desired amount of vacuum to fit the membrane tightly on the inner surface of the mold. The internal diameter of the completed mold is 15.24 cm, the thickness of the wall is approximately 0.635 cm, and the height of the finished specimen is 30.48 cm. The base of the mold is firmly bolted onto the vibrating table so the mold will not move during vibration. A vacuum pump provides the required suction to stretch the membrane around the wall of the mold to aid in the compaction of the specimen.

The compaction method involves a trial-and-error adjustment in the weight of aggregate materials per layer, the number of compacted layers, and the vibrating period for each layer to produce specimens of the desired densities. On the basis of this trial-and-error approach a suitable sample preparation procedure was devised. With this method the specimens are prepared in 10 layers having approximately 1600 g of aggregate mixes per layer. A steel rod is used to enhance the effectiveness of compaction. The vibrating time is approximately 30 sec per layer for the first eight layers and 4 min per layer for the last two layers. This method yields specimens more uniform than those prepared by using equal vibrating times for each layer, in which case the bottom layer becomes more dense as a result of vibrating times accumulating from bottom to top. All specimens investigated were compacted to 95 percent of maximum dry density at optimum moisture content as determined from T180–90D, as shown in Table 2.

To meet the Oklahoma DOT 1988 specifications (*24*) and to ensure consistent gradation for each specimen among various aggregate types, a gradation curve was selected for the purpose of sample preparation. Gradation of aggregate materials can also be an important factor when comparing the RM values. The selected gradation curve used in this study and the gradation required by ODOT is presented in Figure 1.

EQUIPMENT SETUP FOR RM TESTING

The load frame, triaxial cell, pressure gauge, load cell, and the overall setup used in this study are shown schematically in Figure 2 (*top*). A 5-kip (2 270-kg) load cell mounted inside the triaxial chamber and attached to the loading piston is used to monitor the applied deviatoric load. The MicroProfiler is programmed to conduct a test under the desired loading. A data acquisition software system reads and stores the desired data in the form of voltages that are emitted from the transducers. Air is used as the cell fluid and confining medium instead of water because water might get into the specimen through tiny leaks or breakage of the membrane. An air pressure gauge installed onto the triaxial cell measures the confining pressure, as shown in Figure 2 (*top*). The advantage of this 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

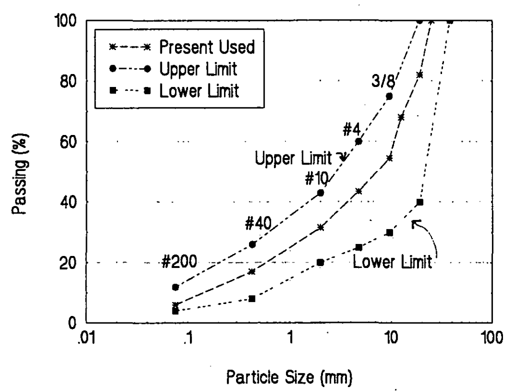

FIGURE 1 Gradation required by Oklahoma DOT and used in this study.

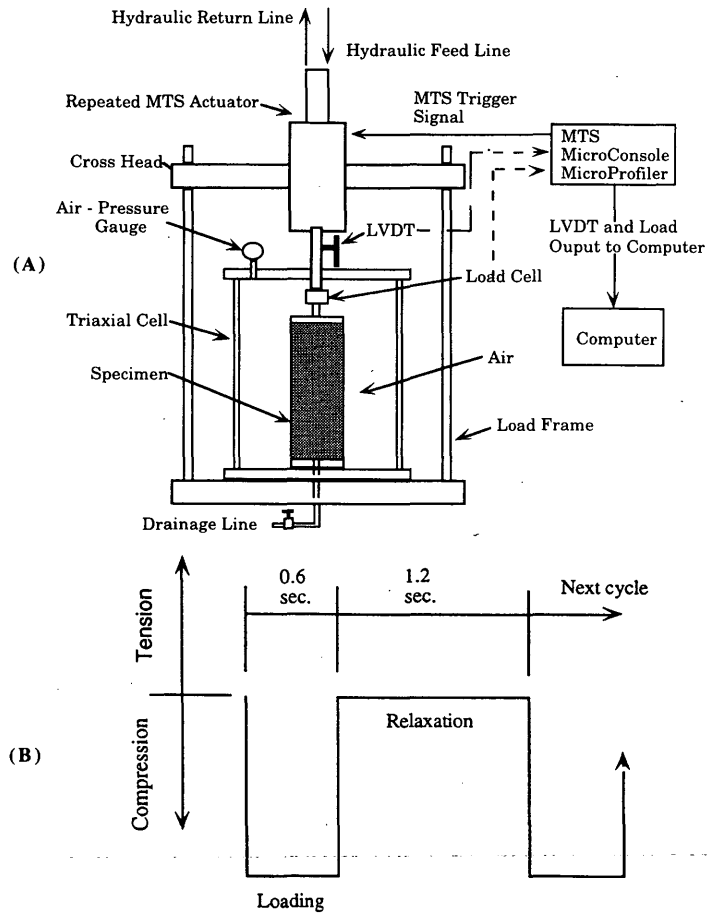

FIGURE 2 Test setup for RM testing (*top*) and rectangular wave (*bottom*) used in this study.

caused by the push rod. The quality of test results is generally improved by monitoring the in-vessel load and confining pressures.

A fixed cycle duration of 1.8 sec was selected in this study to provide a 0.6-sec loading duration and 1.2 sec of relaxation between the end and beginning of consecutive load repetitions, as shown in Figure 2 (*bottom*). An oscilloscope is used to monitor the applied cyclic loading to achieve the desired rectangular waveform by adjusting the gain controller in the MicroConsole.

DISCUSSION OF RESULTS

Variability Due to Aggregate Types

The six aggregate types were selected, compacted, and tested at OMC and 95 percent of MDD under testing procedure T292–91I to evaluate the effect of aggregate source variation and to investigate the variability of the results. According to testing procedure T292–91I, the RM values (in psi) for granular materials can be conveniently reported by using the relationship given in Equation 2, which requires determination of the regression constants K_1 and K_2.

$$RM = K_1\, \theta^{K_2} \quad (2)$$

The constants K_1 and K_2 for these six aggregate types were obtained for every test and are presented in Table 3. Six RM tests were performed for the aggregates from Choctaw and Murray Counties and three RM tests each were performed for the aggregates from Comanche, Cherokee, Creek, and Johnston Counties. The standard deviation (SD) was also computed for each aggregate source and is given in Table 3. Table 3 shows that under testing procedure T292–91I some aggregate types experience a lower SD than other aggregate types. The ranges of K_1 and K_2 for untreated granular materials reported by several agencies are given by Shook et al. (*25*). Although the values of K_1 and K_2 obtained in this study are in the ranges reported by other agencies, it is inappropriate to make a direct comparison of the RM values by means of K_1 and K_2. Thus, in an effort to investigate the effects on RM values due to varying aggregate sources, the average (mean) RM values for each aggregate type are grouped together and presented in Figure 3. The details of the RM results in terms of bulk stress are given in Table 4, and the coefficient of variation (COV) is computed for each bulk stress and also presented in that table. An attempt was made to find the effect of confining stress on the RM values. Figure 4 shows that the RM values of all six aggregate types increased with increasing confining pressure, as expected. As shown in Table 4, the RM values obtained ranged from 51 to 195 MPa (7 to 28 ksi, values varying with

TABLE 3 Summary of K¹ and K² for Six Aggregate Types

County	Material	K_1 (psi)	SD*	K_2	SD*
Comanche	Limestone	4151	1082	.3918	.1175
		3908		.3683	
		2168		.5825	
Cherokee	Limestone	2283	2465	.5017	.1133
		4685		.3472	
		7213		.2882	
Creek	Limestone	4449	518	.3698	.0246
		4317		.3858	
		3494		.4180	
Choctaw	Sandstone	1388	165	.5309	.0295
		1691		.5847	
		1427		.5734	
		1498		.6073	
		2029		.5364	
		1440		.5533	
Johnston	Granite	2041	173	.5242	.0449
		2366		.4350	
		2102		.4889	
Murray	Rhyolite	2747	580	.4338	.056
		2417		.4949	
		3099		.4612	
		2160		.4769	
		1673		.5230	
		1652		.5949	

* SD denotes the standard deviation

bulk stress θ), which is in the range reported by May and Witczak (*26*) but lower than that suggested by the Asphalt Institute (*27*) for the design of flexible pavement [RM for untreated granular material vary from fewer than 103.4 MPa (15 ksi) to more than 344.5 MPa (50 ksi)].

Thompson (*14*) reported that for a given gradation (for either crushed or uncrushed materials) the source (limestone, sandstone, granite, etc.) is usually not a significant factor in terms of RM. Thompson and Smith (*17*) also observed that the RM properties of various aggregates are similar and the type of aggregates used as base courses of roadway pavement (crushed stone/gravel) has limited effect on the RM. In fact, as shown in Figure 3, the differences of the RM values due to variation of aggregate types are approximately in the range of 20 to 50 percent. Table 4 shows that the maximum coefficient of variation (MCOV) for these six aggregate types is 20.8 percent, which suggests that the source of aggregate has some effect on the RM values.

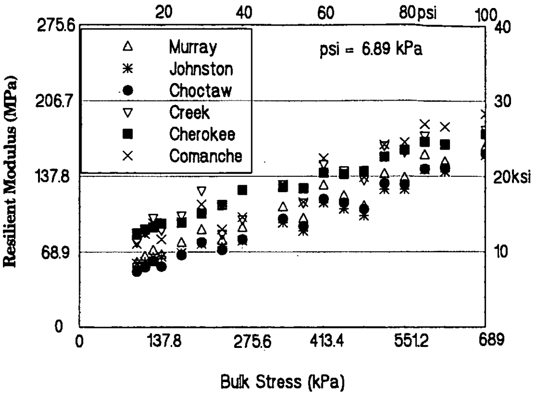

FIGURE 3 Comparison of average RMs for six aggregate types using AASHTO T292–91I.

In order to study the variability of the test results from the same aggregate type, aggregates from Choctaw and Murray Counties were selected for further investigations. It is assumed that Equation 2 adequately represents the granular material behavior. Accordingly the overall K_1 and K_2 (including results from the six tests) was computed and the predicted RM values were evaluated based on Equation 2 and compared with the corresponding experimental values. The relative error between the prediction (*P*) and the experimental results (*T*) was evaluated. The maximum relative error (MRE) thus obtained was defined as follows:

$$MRE = \left\{ \begin{matrix} \left|\frac{T1-P}{P}\right|, \left|\frac{T2-P}{P}\right|, \left|\frac{T3-P}{P}\right| \\ \left|\frac{T4-P}{P}\right|, \left|\frac{T5-P}{P}\right|, \left|\frac{T6-P}{P}\right| \end{matrix} \right\}, \quad (3)$$

TABLE 4 Average RM for Six Aggregate Types Using AASHTO T292–91I

Bulk Stress (kPa)	Resilient Modulus (MPa): Coma. County	Cher. County	Cree. County	Choc. County	John. County	Murr. County	MRE (%)	COV (%)
482.3	137.1	142.6	134.4	108.2	102.0	110.9	24.4	14.2
551.2	168.8	161.9	159.8	130.2	126.1	136.4	17.7	12.5
620.1	183.3	166.7	168.1	145.4	141.9	150.9	21.0	10.0
689.0	195.0	176.4	180.5	158.5	157.1	164.7	22.7	8.5
379.0	114.4	126.8	113.7	92.3	88.2	99.2	27.0	14.1
447.8	140.6	139.9	143.3	113.7	108.2	119.9	17.0	12.1
516.8	166.7	155.7	166.0	131.6	126.1	139.9	19.7	11.9
585.7	185.3	169.5	175.0	144.7	144	157.1	25.6	10.3
241.1	89.6	111.6	85.4	71.0	73.7	79.2	27.7	17.3
275.6	100.6	125.4	98.5	80.6	79.2	90.9	24.0	17.7
344.5	130.2	128.2	130.2	99.2	95.8	109.6	17.1	13.9
413.4	154.3	141.2	148.8	117.1	113.7	129.5	22.8	12.5
137.8	80.6	95.1	88.9	55.8	64.8	66.1	26.4	20.4
172.3	96.5	95.8	102.0	66.1	68.2	77.2	21.3	18.6
206.7	112.3	104	124.7	77.9	76.5	88.9	36.4	20.0
96.5	76.5	86.1	77.9	51.0	58.6	59.3	33.8	20.2
110.2	85.4	89.6	86.8	55.1	59.9	64.8	30.8	20.8
124.0	95.1	91.6	99.9	60.6	64.1	70.3	38.2	21.4

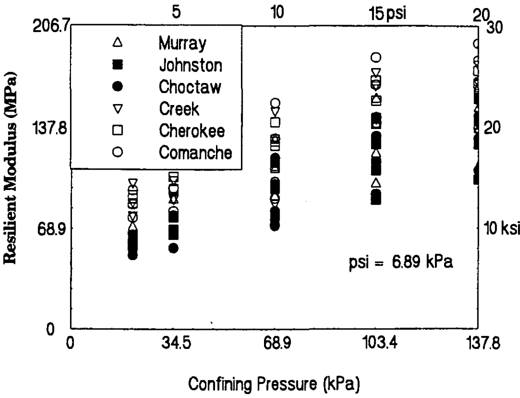

FIGURE 4 Effects of confining pressure on average RMs for six aggregate types using AASHTO T292–91I.

where $T1$, $T2$, $T3$, $T4$, $T5$, and $T6$ represent the experimental results from Tests 1, 2, 3, 4, 5, and 6, respectively. Since MRE fluctuates with bulk stress, a weighted maximum relative error (WMRE) becomes an important parameter in investigating the variability of the test results. Thus it is proposed that the calculations be performed in the form of

$$WMRE = \sqrt{\frac{\Sigma MRE^2}{n}} \quad (4)$$

It may be noted that here n is 18 for AASHTO testing procedure T292–91I and 15 for T294–92I because there are 18 and 15 RM values, respectively, for these two testing procedures. The computed WMRE for these two aggregate sources is presented in Table 5. The WMRE values of 28 percent and 26.4 percent were found for the aggregates from Choctaw and Murray Counties, respectively. The closeness of the WMRE values for these two aggregate types indicates that the testing procedure used gave consistent results.

In addition to the aforementioned comparisons, another attempt was made to determine the COV for the same aggregate source. The COV values were computed from the six RM tests along with the bulk stresses for the specimens from Choctaw and Murray Counties, as shown in Table 6. The maximum coefficient of variation (MCOV) for the aggregates from Choctaw and Murray Counties is presented in Table 5. The MCOV values were found to be 19.7 percent and 20.6 percent for the aggregates from Choctaw and Murray Counties, respectively.

TABLE 5 Summary of WMRE and MCOV

County	WMRE(%)	MCOV(%)
Choctaw (T292-91I)	28.0	19.7
Murray (T292-91I)	26.4	20.6
Choctaw (T294-92I)	23.0	18.9
Murray (T294-92I)	30.2	26.0
Aggregate Source		20.8
Testing Procedure		
Choctaw		25.8
Murray		36.0

TABLE 6 MRE and COV for Aggregates from Choctaw and Murray counties Using AASHTO T292–91I

Bulk Stress	Choctaw		Murray	
(kPa)	MRE (%)	COV (%)	MRE (%)	COV (%)
482.3	27.8	14.8	35.4	20.6
551.2	25.5	19.7	23.3	18.0
620.1	27.4	18.7	24.0	14.7
689.0	25.9	17.1	25.2	12.9
379.0	31.2	14.8	30.0	17.6
447.8	25.6	16.8	18.2	13.7
516.8	22.3	17.1	22.7	12.5
585.7	27.2	16.0	26.3	10.9
241.1	35.6	17.4	29.7	18.8
275.6	30.7	17.8	25.5	16.0
344.5	25.3	19.2	19.1	11.9
413.4	29.5	18.9	24.7	10.8
137.8	32.4	19.3	25.8	17.4
172.3	22.1	16.6	20.4	12.9
206.7	29.6	17.6	26.2	11.8
96.5	28.6	14.6	33.8	17.3
110.2	24.2	12.5	30.8	14.6
124.0	28.6	13.1	26.7	11.9

Variability Due to Testing Procedures

An attempt was made to investigate the effects of the applied stress sequence on the RM values. AASHTO test procedures T292–91I and T294–92I were chosen for this purpose because the T292–91I testing procedure starts with a higher confining pressure and deviatoric dynamic stress and ends with a lower confining pressure and deviatoric dynamic stress, whereas the T294–92I procedure suggests the reverse order. To eliminate another unknown in the comparison, the rectangular waveform shown in Figure 2 (bottom) was used for both the T292–91I and T294–92I methods. Aggregates from Choctaw and Murray Counties were selected and tested under both methods using six tests each. The results presented in Figure 5 suggest that the T294–92I procedure gives RM values higher than those obtained from T292–91I for both aggregate types. The higher RMs yielded by the T294–92I testing procedure may be attributed to the cyclic stress having a stiffening effect on the specimen structure because the stress application follows the low-to-high sequence. The amount of difference in the RM values due to testing method varies with aggregate type as shown in Figure 6. For example, aggregates from Murray County experience a higher degree of increase (about 35 to 55 percent) when using T294–92I than the aggregates from Choctaw County (about 15 to 34 percent).

Because the same aggregates have been tested using different testing procedures, it is expected that in using other testing procedures (such as the procedure suggested by the Florida, New York, Illinois, and South Dakota DOTs) the RM values would also fall in the ranges obtained in this study. When both T292–91I and T294–92I were used, the RM values ranged from 41.3 to 206.7 MPa (6 to 30 ksi) and from 48.2 to 261.8 MPa (7 to 38 ksi) (depending on bulk stress) for aggregates from Choctaw and Murray Counties, respectively (refer to Figure 5). These values are close to the values reported by Elliott (*28*) [ranging from 96.5 MPa (14 ksi, at θ = 14

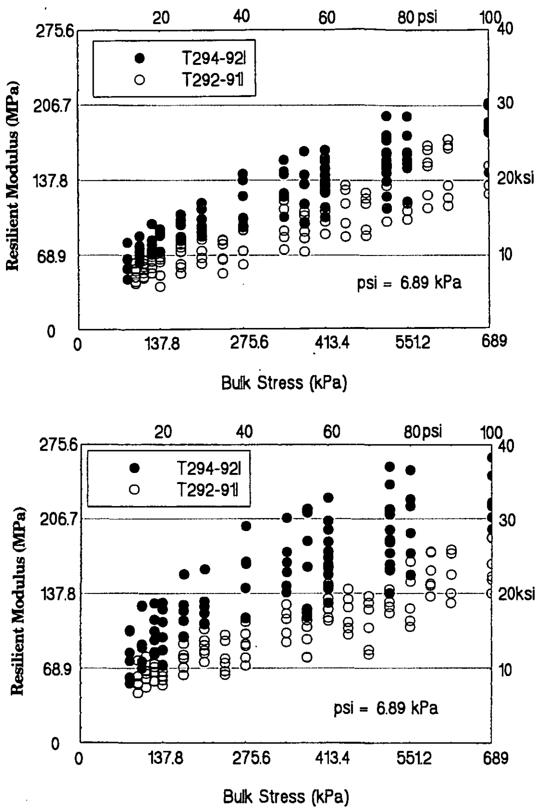

FIGURE 5 Effects of the testing procedure (AASHTO T292–91I and T294–92I) on RMs for the aggregates from Choctaw (*top*) and Murray (*bottom*) counties.

psi) to 255 MPa (37 ksi, at $\theta = 100$ psi) with a relationship of RM (in psi) $= 4{,}120 * \theta^{0.47}$] but are lower than the values suggested by Monismith (*29*). A design modulus of 248 MPa (36 ksi) was recommended by Monismith for the crushed stone base.

Similarly, the COV values were computed along with bulk stress for the aggregates from Choctaw and Murray Counties on the basis of the results from six RM tests using the T294–92I procedure. The

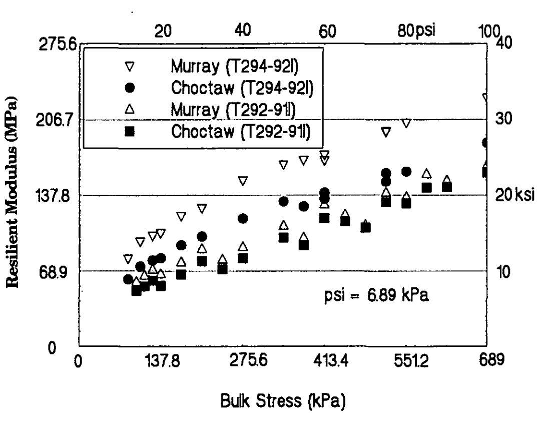

FIGURE 6 Comparison of average RM for different testing procedures (AASHTO T292–91I and T294–92I) and aggregate sources (Murray and Choctaw counties).

obtained WMRE and MCOV are given in Table 5. It was found that when using the T294–92I method the aggregates from Murray County experience a higher variability in the RM values (with WMRE 30.2 percent and MCOV 26 percent) than when using the T292–91I method (with WMRE 26.4 percent and MCOV 20.6 percent). In contrast, aggregates from Choctaw County exhibit a lower variability in the RM values (with WMRE 23.0 percent and MCOV 18.9 percent) when using T294–92I than when using T292–91I (with WMRE 28.0 percent and MCOV 19.7 percent). Thus, on the basis of the available data, it is premature to say that one test method is better than the other.

To investigate the variability of RM values due to testing procedure for the same aggregate source, six RM test data using T292–91I and six RM test data using T294–92I at the same level of bulk stress were grouped together and the COVs were computed. MCOVs of 25.8 percent and 36.0 percent were found for Choctaw County and Murray County aggregates, respectively, as evident from Table 5. Table 5 also shows that the variability of RM values due to the testing procedure was higher than the variability due to the aggregate source.

CONCLUSIONS

The AASHTO T292–91I testing procedures were used in this study to conduct the RM tests of six selected aggregate types and to investigate the variability of the test results. The variability of RM values due to testing procedures (T292–91I and T294–92I) and aggregate type was investigated. The following observations are made on the basis of the data obtained:

1. The RM values (ranging from 41.3 to 261.8 MPa, depending on bulk stress, material type, and testing method) obtained in this study are lower than those reported in the literature.
2. The variability of RM values obtained in this study ranges from 19 percent to 26 percent in terms of MCOV.
3. For a given gradation, the differences in RM values due to aggregate sources are found to be from 20 to 50 percent and the MCOV for these six aggregate types is 20.8 percent.
4. For both aggregate types investigated in this study, the T294–92I testing procedure yields higher RM values than those obtained by using the T292–92I testing procedure, possibly because the stress sequence in T294–92I has a stiffening and strengthening effect on the specimen structure as the stress level increases. The amount of increase in RM values due to testing method varies with the type of aggregate. For example, aggregates from Murray County exhibited a higher degree of increase (about 35 to 55 percent) when using the T294–92I testing method than the aggregates from Choctaw County (about 15 to 34 percent).
5. When using the T294–92I method, the aggregates from Murray County experience a higher variability in RM values (with WMRE 30.2 percent and MCOV 26 percent) than when using the T292–91I method (with WMRE 26.4 percent and MCOV 20.6 percent). In contrast, aggregates from Choctaw County exhibit a lower variability in RM values (with WMRE 23 percent and MCOV 18.9 percent) when using T294–92I than when using T292–91I (with WMRE 28 percent and MCOV 19.7 percent). Thus, on the basis of the available data, it is premature to say that one test method is better than the other.
6. The variability of RM values due to testing procedure was found to be higher than that due to aggregate source.

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