Influence of Testing Procedure and LVDT Location on Resilient Modulus of Soils

LOUAY N. MOHAMMAD, ANAND J. PUPPALA, AND PRASAD ALAVILLI

Several transportation agencies use the AASHTO-recommended resilient modulus (M_r) as the fundamental parameter in the mechanistic analysis and design of pavements. At present there are several types of dynamic testing devices calibrated to measure resilient modulus. The repeated triaxial device is popular because of its repeatability, reliability, and ease of operation. Testing procedures and internal deformation measurements play a crucial role in testing. A statistically designed experiment was used to compare the influence of these factors. Two testing procedures, AASHTO T-294-1992 and AASHTO T-292-1991, were evaluated, and two separate measurement systems inside the triaxial cell were used to measure the axial deformations. Both cohesive and granular soils were tested. The influence of testing procedures and measurement systems are presented in the form of normalized factors, which are discussed with respect to the test variables and confining and deviatoric stresses. The testing procedure appears to influence test results for sand specimens, possibly due to the variation in magnitudes of testing stresses. The measurement system has a greater influence on clay specimens due to a combination of several factors such as soil fabric, stress dependency behavior, and end friction effects. The type of soil, testing procedures, and location of the internal, linear variable differential transformers and their influence on the regression model constants are discussed and graphically presented.

In 1986 AASHTO recommended the use of resilient modulus as a fundamental property for characterizing highway materials in the mechanistic design of flexible pavements (*1*). Many state transportation agencies use empirical procedures involving soil support, California bearing ratio, and *R*-values for estimating the resilient modulus. These approaches do not adequately represent the response of pavement materials to the dynamic loading to which they are exposed under actual service conditions. Therefore dynamic testing methods are needed to determine realistic resilient modulus values.

The resilient modulus (M_r) is defined as the ratio of deviatoric stress to recoverable axial strain and is presented in the following equation:

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

where σ_d is the deviatoric stress and ϵ_r is the resilient strain.

Most of the recent research on resilient modulus testing is concentrated on the use of various dynamic testing equipment for determining the resilient modulus (*2,3*), influence of soil characteristics (*3–9*), and instrumentation effects on M_r (*9–16*). This research has contributed significantly to understanding of the resilient properties of soils.

AASHTO has recommended several procedures (T-274, T-292, and T-294) for determining resilient modulus of subgrade soils. The most recent procedure, T-294, is a modification of the old procedures and was published by AASHTO in the interim specifications in 1992 (*17*). Since their introduction, all the above procedures have been subjected to criticism and discussion. For example, many investigators, questioned the need for extensive sample conditioning (*6,11,14*). Their investigations showed that severe conditioning may result in disturbance to the soil sample and sometimes may result in the breaking of samples during testing. However it was reported by Houston et al. (*11*) that conditioning is needed to eliminate the plastic strains before obtaining measurements for determining the resilient modulus. Other reasons for this conditioning are given in the Conditioning and Testing Procedure section of this paper.

The sequence of applying the confining pressure and deviatoric stress to the specimen in AASHTO T-292-1991 has raised many concerns (*12,13*). The new protocol, T-294-1992, is a modified version of the sequence of stresses of T-292-1991. This protocol is more conducive to testing and does not have any sudden jumps in test stresses from one sequence to another. Furthermore some transportation agencies and organizations have adopted their own testing procedures based on investigations conducted on locally available soils (*14*). The procedure influence is investigated by conducting resilient tests on two different soils using two AASHTO procedures (T-292 and T-294).

The location of linear variable differential transformers (LVDTs) on the specimen for resilient displacement measurements is a crucial element in this investigation. Two locations, the middle and the end of the specimen, were selected for this study. The results of the tests using all these variables are presented in the form of coefficients or multipliers.

OBJECTIVE AND SCOPE

The main objective of this paper is to develop an understanding of the influence of AASHTO T-292-1991 and T-294-1992 testing procedures on measured resilient modulus. The testing was conducted on both cohesive and cohesionless soils (A-7 silty clay and A-3 sand). Another objective is to investigate the influence of the location of internal LVDTs on the specimen in measuring the resilient deformations. To achieve these objectives, an extensive resilient modulus testing program was initiated at the Louisiana Transportation Research Center (LTRC). As part of this study, fully automated test software, data acquisition, and equipment control were also developed.

EXPERIMENTAL SETUP

Loading System and Data Acquisition

An MTS model 810 closed-loop servo-hydraulic material testing system was used. The major components of this system are an ana-

Louisiana Transportation Research Center, Louisiana State University, 4101 Gourrier Avenue, Baton Rouge, La. 70808.

log controller, a load frame, a hydraulic actuator, and a function generator. The loading system consists of a load frame and a hydraulic actuator. Fully automated test software for equipment control and data acquisition was developed to perform tests and acquire, analyze, and present data. These units and software are described in detail elsewhere (*13*).

Measurement Systems

In existing testing procedures, LVDTs are placed outside the chamber for measuring displacements. This external measurement system is easy to install and provides a simplified procedure to externally rezero the initial LVDT readings without having to remove the chamber of the cell. However the influence of external LVDT location is significant on M_r results due to the nonuniform strain distributions at the ends, the result of end friction effects as well as instrumental and system compliance errors (*2*). One suggestion is to use internal LVDTs in the place of external LVDTs to minimize these errors (*2*). It should be mentioned that T-294 uses an external LVDT system and T-292 uses both external and internal LVDT systems. However, due to the reasons mentioned, internal LVDT systems were selected for this study. The internal system is subjected to fewer system compliance errors than the externally mounted LVDT system because the internal system is mounted directly on the specimen.

One system is used to measure deformations with respect to ends, whereas the other is used to measure the deformations at the middle one-third of the specimen (Figure 1). These systems are hereafter referred to as end and middle systems, respectively. The LVDTs of the middle system have a full-scale stroke of ±3.05 mm (±0.12 in.) with a nonlinearity of ±0.00762 mm (±0.0003 in.). The LVDTs of the end system have a full-scale stroke of ±6.35 mm (±0.25 in.) with a nonlinearity of ±0.0158 mm (±0.000625 in.).

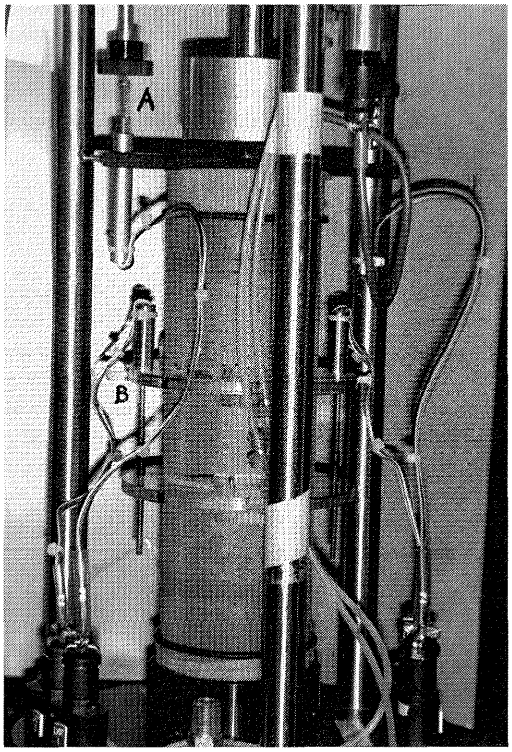


FIGURE 1 Specimen with LVDTs: (*a*) end system and (*b*) middle system.

Specimen Preparation

Tests were conducted on two locally available soils: a blasting sand and a silty clay. Properties of these soils are given in Table 1. Specimens tested were 71.1 mm (2.8 in.) in diameter and 142.2 mm (5.6 in.) in height. This produces a height-to-diameter ratio of 2, which is required in this type of testing to reduce end effects due to friction. Both types of soil specimens were compacted close to the optimum water content–dry density combination.

Quality Assessment and Control

The influence of sample preparation on the target design water content and density was examined for each soil type. No significant difference in densities were observed among samples for each soil type, indicating that similar specimens were tested in each category. For sand specimens, the influence of fine migration due to compaction and testing procedures was examined. After testing, the sand specimen was carefully removed, cut into two slices, and dried for grain size distribution tests. Results of these tests are shown in Figure 2, including the results for untested sand. No significant variation in grain size distributions was observed, indicating that the compaction procedures did not result in any fine migration, layering, or crushing of the aggregates.

The moisture migration check is important for fine-grained cohesive soils because variations in moisture content could result in variations in partial saturation of the specimen. This partial saturation induces suction pressures that cause some confinement, which in turn increases the total strength of the specimen and thereby influences the final M_r values. Moisture migration was therefore checked by measuring the water content of different slices of a freshly prepared silty clay specimen before testing. The moisture content of these slices varies between 20.8 and 21.6 percent, which implies that moisture migration was not present in these samples.

TABLE 1 Properties of Soils Tested

PROPERTY	BLASTING SAND	SILTY CLAY
Specific Gravity, G_s	2 .75	2.6
Optimum Density (kN/m^3)	17.5	16.0
Optimum Moisture Content (%)	12.0	21.2
Plasticity Index (PI)	--	22
AASHTO Classification	A-3	A-7

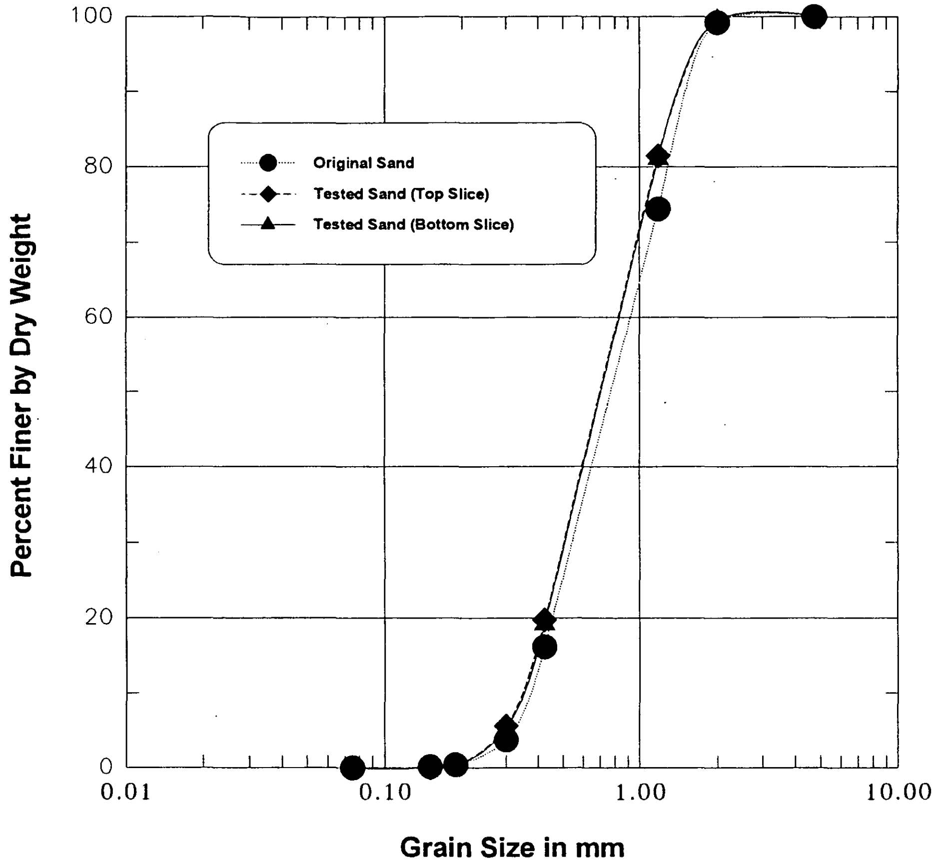


FIGURE 2 Fine migration check in sands.

Conditioning and Testing Procedure

The tests on both soil samples were performed at the confining and deviatoric stress levels recommended in AASHTO T-292-91 and T-294-92. Few additional testing stresses were added to T-294 and T-292 testing sequences for sands. These stresses are selected to fit in between the provided successive testing sequences.

The samples were first conditioned by applying 1,000 repetitions of a specified deviatoric stress. Conditioning eliminates the effects of specimen disturbances due to sampling, compaction, and specimen preparation procedures, and also aids in minimizing the effects of imperfect contacts between end platens and the specimen. Once the conditioning is completed, the specimen is subjected to different stress sequences of confining and deviatoric stresses. One hundred cycles were used for each sequence. The stress sequence was selected to cover the expected in-service range that a pavement or subgrade material experiences as a result of traffic loading.

Figure 3 presents the stress sequences of both AASHTO procedures in the form of bar charts for both soils, representing the maximum amount of deviatoric stress applied to each specimen at each confining pressure. It should be noted that granular soils under T-292 were subjected to a higher variation of stresses in each testing sequence than those under T-294.

Cohesive soil samples were subjected to lower magnitudes of stresses than granular samples. The stress sequence for clays under T-294 shows that they are subjected to a higher confining pressure in the beginning of a testing sequence (42 kPa) and a lower confining pressure in the end (0 kPa). It is well known that this type of phenomenon will cause over-consolidation of the clays and may result in strengthening of the specimen, which may give higher M_r values at lower confining stresses. The stress sequence for clays under T-292 depicts only one set of confining pressure (21 kPa).

All tests were conducted with a haversine-type loading waveform with a peak load equivalent to the specified deviatoric stress. The loading period and the relaxation periods were 0.1 and 0.9, respectively.

EXPERIMENTAL DESIGN

A statistically designed experiment was used to examine the influence of the testing procedure and LVDT location. The number of samples (n) for each soil type can be computed as follows:

$$n = (z_{\alpha/2}\sigma/e)^2 \qquad (2)$$

where

$z_{\alpha/2}$ = upper $\alpha/2$ critical value for the standard normal distribution,
σ = population standard deviation, and
e = error in estimation.

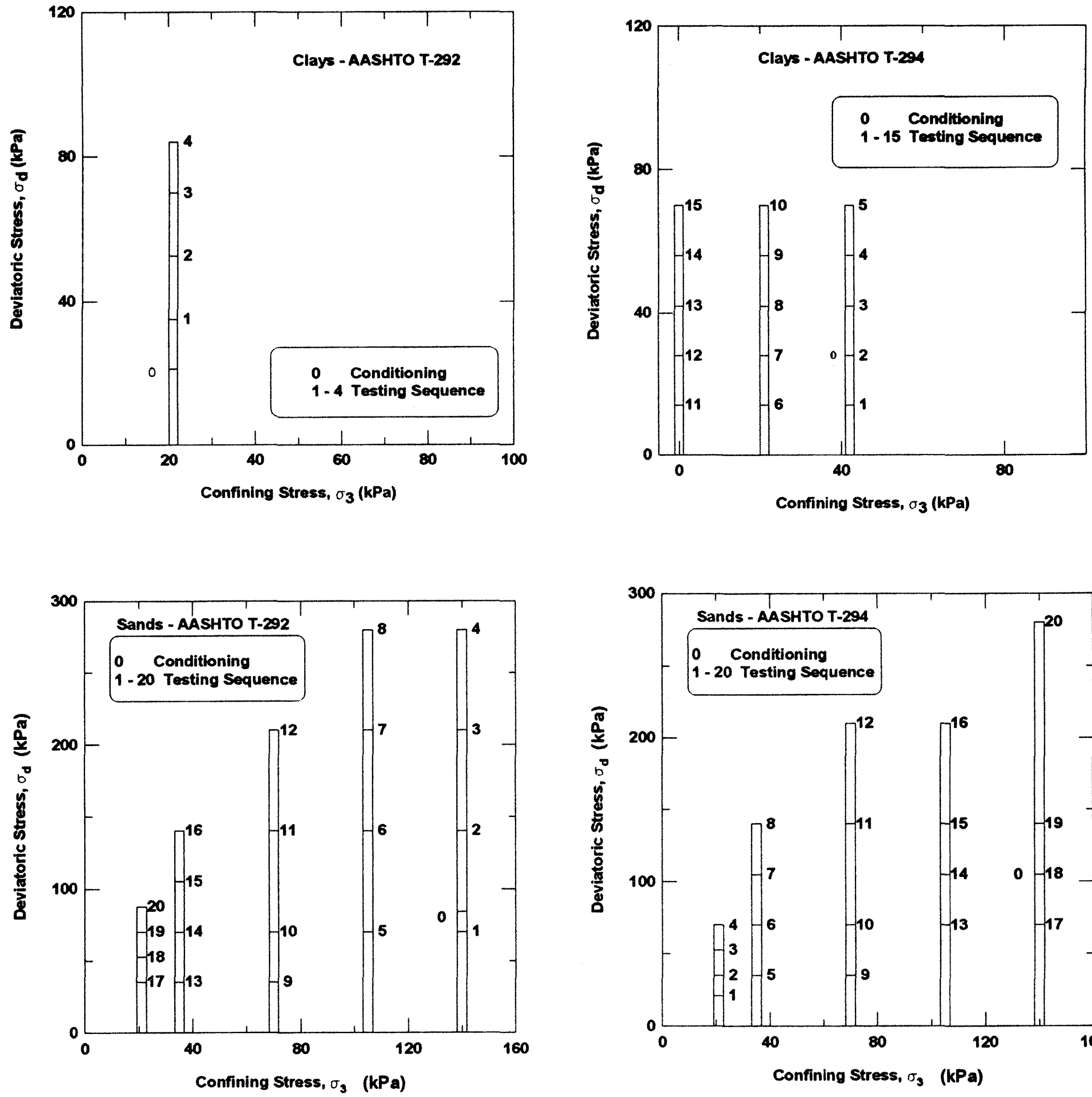


FIGURE 3 AASHTO procedures for sands and clays.

The variables σ and e were initially unknown, and in such a case statisticians have established that a sample size of 30 would define the pattern of the variation of the variable. Thus 30 specimens were used in the preliminary testing phase, which involved testing both soils under the T-292 procedure. The test results were statistically analyzed using the analysis of variance procedure. A multiple comparison procedure with a risk level of 5 percent was performed on the means. The independent variables are assumed to have populations with normal distributions. In addition operator errors are insignificant because all tests were conducted by a single operator. The number of samples for the subsequent testing based on the statistics obtained from the first-phase 30 sample test results total approximately 7, however 10 samples were selected and tested to produce more reliable results.

ANALYSIS OF RESULTS

The test results were analyzed using the analysis of variance procedure provided in the Statistical Analysis System (SAS) program. Tables 2 and 3 present the mean resilient modulus (in MPa) of the test results of sands and silty clay samples, respectively, along with standard deviation (σ), coefficient of variation (C_v), and test repeatability. The coefficient of variation, an indicator of the variation of the results, lies between 0.9 and 10.3 for sands and 5.10 and 24.0 for clays. Even though the range is wider for clays, most of the results have C_v values around 10. These variations are insignificant and therefore the test results are considered repeatable as per ASTM C670. Higher C_v values were obtained for samples tested under T-292 because the samples under T-292 were subjected to a wider

TABLE 2 M_r Results from Blasting Sand

S3	Sd	AASHTO Procedure								AASHTO Procedure							
		T−292				T−294				T−292				T−294			
		MRE	SD	CV	R	MRE	SD	CV	R	MRM	SD	CV	R	MRM	SD	CV	R
21	21					159.2	4.7	3.0	Y					189.0	8.6	4.6	Y
21	35	153.0	8.4	5.5	Y	164.2	6.7	4.1	Y	189.0	12.6	6.7	Y	193.2	10.7	5.5	Y
21	52.5	162.1	10.1	6.2	Y	173.6	5.6	3.3	Y	195.0	11.1	5.7	Y	206.5	6.1	2.9	Y
21	70	168.7	9.7	5.8	Y	180.1	4.3	2.4	Y	200.9	11.6	5.8	Y	212.1	8.7	4.1	Y
21	87.5	173.6	10.9	6.3	Y					205.5	11.9	5.8	Y				
35	42	168.0	9.6	5.7	Y	209.0	4.9	2.4	Y	194.6	12.1	6.2	Y	250.6	10.3	4.1	Y
35	70	183.8	9.6	5.3	Y	220.5	6.1	2.8	Y	217.0	9.5	4.4	Y	259.0	7.4	2.9	Y
35	105	199.5	10.1	5.0	Y	220.5	4.3	2.0	Y	231.7	9.9	4.3	Y	255.5	9.6	3.8	Y
35	140	206.5	12.3	6.0	Y	219.0	3.0	1.4	Y	237.3	13.9	5.8	Y	254.5	11.3	4.4	Y
70	35	226.1	11.9	5.3	Y	283.9	4.8	1.7	Y	269.9	16.6	6.1	Y	345.8	18.7	5.4	Y
70	70	238.0	10.1	4.3	Y	293.3	3.3	1.1	Y	275.8	12.9	4.7	Y	341.6	16.5	4.8	Y
70	140	257.6	10.0	3.9	Y	302.6	3.7	1.2	Y	295.4	11.5	3.9	Y	345.8	12.3	3.6	Y
70	210	274.1	9.6	3.5	Y	295.4	3.0	1.0	Y	309.8	11.8	3.8	Y	336.7	11.0	3.3	Y
105	70	287.0	12.0	4.2	Y	337.6	6.2	1.8	Y	336.7	21.4	6.4	Y	386.4	18.4	4.8	Y
105	105					346.5	4.6	1.3	Y					394.1	16.0	4.1	Y
105	140	304.5	12.4	4.1	Y	353.5	3.7	1.0	Y	347.2	19.3	5.6	Y	399.7	14.3	3.6	Y
105	210	317.8	11.6	3.7	Y	356.3	3.3	0.9	Y	358.4	17.6	4.9	Y	403.2	14.4	3.6	Y
105	280	327.3	10.7	3.3	Y					365.4	14.6	4.0	Y				
140	70	352.5	17.8	5.1	Y	380.8	4.8	1.3	Y	423.5	43.6	10.3	Y	445.9	26.1	5.9	Y
140	105					392.0	4.9	1.2	Y					448.7	23.2	5.2	Y
140	140	366.8	16.5	4.5	Y	396.2	3.3	0.8	Y	424.9	35.2	8.3	Y	456.1	22.4	4.9	Y
140	210	369.6	13.9	3.8	Y					420.0	28.1	6.7	Y				
140	280	372.4	11.8	3.2	Y	406.0	4.4	1.1	Y	417.9	21.9	5.3	Y	465.5	26.6	5.7	Y

S3 : Confining Pressure (in kPa)
Sd : Deviatoric Pressure (in kPa)
MRE : Resilient Modulus from End Measurement System (in MPa)
MRM : Resilient Modulus from Middle Measurement System (in MPa)
SD : Standard Deviation
CV : Coefficient of Variation
R : Repeatability
Y : Indicates Test is Repeatable as per ASTM C670.

variation of stresses from test to test, which might have resulted in some change in the structure of the specimen.

Influence of Testing Stresses

The influence of confining and deviatoric stresses on the moduli of sands and clays is depicted in Figure 4. The trends represented in this figure are similar to those obtained in previous investigations (*5,6,8,11*). Granular materials exhibit an increase in M_r value with an increase in confining and deviatoric stresses. This is attributed to the dilatational characteristics and stiffness properties of the soils (*13*). Higher confining pressures tend to resist the dilatational behavior during shearing, which results in lower axial strain measurements and subsequently higher M_r values. For cohesive materials, however, M_r decreases with an increase in deviatoric stress. This is attributed to the pore pressure development, which increases with an increase in deviatoric stress and also in the number of cycles (*11*). This development of pore pressures results in a decrease in effective stresses and in the overall strength of the specimen. Therefore lower M_r values were obtained.

The traditional break in the curve with deviatoric stress as seen in cohesive specimen results was not observed during this study. Silty clay materials tested have significant strength even under unconfined conditions, and this is probably the reason for clays not displaying the breaking behavior at a certain deviatoric stress.

TABLE 3 M_r **Results from Silty Clay**

S3	Sd	Measurement System								Test Procedure
		End				Middle				
		MR	SD	CV	R	MR	SD	CV	R	
42	14	243.2	23.0	9.5	Y	293.4	42.2	14.4	Y	
42	28	216.9	14.2	6.5	Y	266.4	26.9	10.1	Y	T−294
42	42	195.9	10.1	5.1	Y	234.9	24.8	10.6	Y	
42	55	179.8	10.1	5.6	Y	214.2	26.1	12.2	Y	
42	69	152.7	10.0	6.1	Y	194.4	27.3	14.1	Y	
21	14	204.3	17.9	8.8	Y	265.9	33.5	12.6	Y	
21	28	186.2	9.9	5.3	Y	244.7	31.6	12.9	Y	T−294
21	42	171.7	11.6	6.8	Y	226.1	28.7	12.7	Y	
21	55	157.7	11.3	7.2	Y	204.5	27.9	13.7	Y	
21	69	145.9	12.5	8.6	Y	187.3	26.0	13.9	Y	
21	35	171.9	17.6	10.2	Y	240.6	57.5	23.9	Y	
21	52	158.4	18.9	11.9	Y	213.5	44.1	20.7	Y	T−292
21	69	141.7	19.6	13.8	Y	180.3	35.2	19.5	Y	
21	86	127.4	19.8	15.5	Y	154.8	31.9	20.6	Y	
0	14	161.5	15.5	9.6	Y	257.2	22.9	8.9	Y	
0	28	141.4	14.6	10.3	Y	223.3	26.8	12.0	Y	
0	42	129.9	13.8	10.6	Y	206.8	23.1	11.2	Y	T−294
0	55	122.4	13.0	10.6	Y	189.1	23.9	12.6	Y	
0	69	116.8	14.6	12.5	Y	172.9	23.9	13.8	Y	

S3 : Confining Pressure (in kPa)
Sd : Deviatoric Pressure (in kPa)
MR : Resilient Modulus (in MPa)
SD : Standard Deviation
CV : Coefficient of Variation
R : Repeatability
Y : Indicates Test is Repeatable as per ASTM C670.

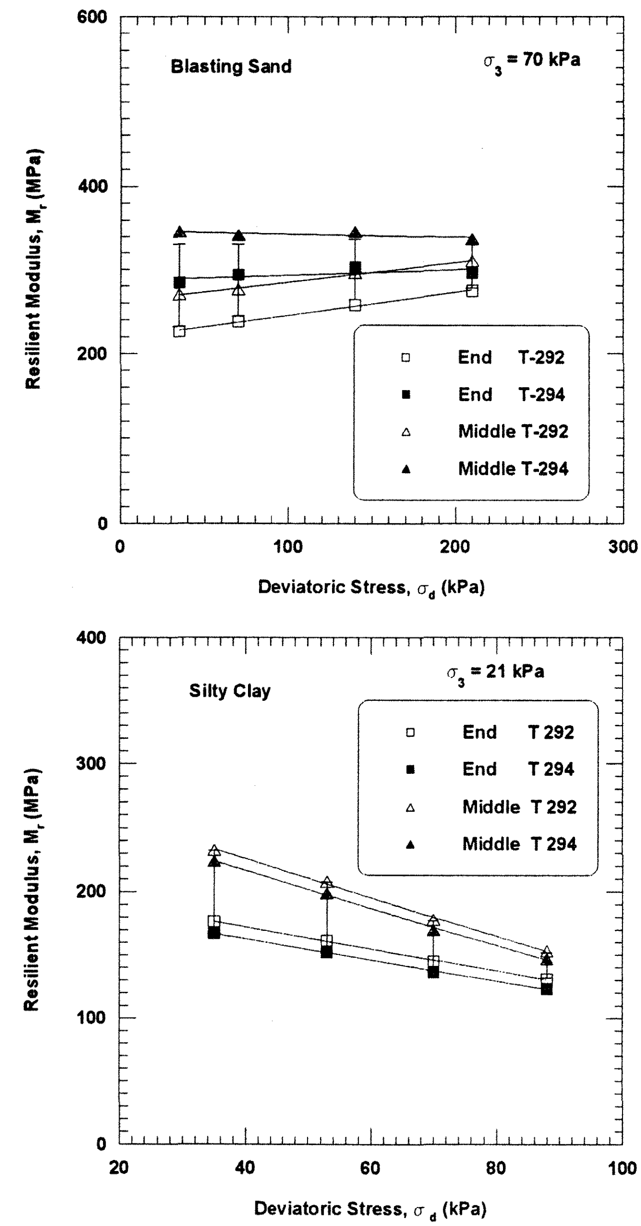


FIGURE 4 Influences of test stresses on M_r values of both soils (*13*).

Testing Procedure

The results on both sands and clays are presented in the form of a simplifying normalized factor, termed the procedure coefficient (PC). The PC is defined as the ratio of the M_r value obtained from the AASHTO T-294 procedure to that obtained from the AASHTO T-292 procedure. The T-292 procedure value was taken as the reference value to which the comparisons were made. In other words, the PC values represent the variation of M_r of the T-294 procedure with respect to the same from the T-292 procedure. The PC values for confining and deviatoric stresses are determined for each measurement system.

Figure 5 shows the results for sand specimens using both measurement systems. The PC values are as high as 1.28 at low confining stresses (35 to 105 kPa) and deviatoric stresses (less than 70 kPa) and are reduced to around 1.15 with the increase in these stresses. Both measurement systems produced similar results. At low confining stresses (35 to 105 kPa) and deviatoric stresses (less than 70 kPa), the previous sequence of the testing had a certain influence on the moduli, which resulted in higher PC values. This influence, however, is not observed at higher deviatoric stresses (greater than 70 kPa), which implies that test procedures have only minor influence on M_r values at these stresses. This is probably because the higher deviatoric stresses applied to the specimen will overcome the stress dependency effects due to previous testing stress sequences. Surprisingly for both measurement systems, however, lower PC values with an average value of around 1.08 are observed for the tests conducted at the lowest (21 kPa) and the highest (140 kPa) confining pressures. The lower values at higher confining pressures can be reasoned from the previous explanations, but cannot be explained in the case of lowest confining pressure (21 kPa) results. After additional examinations it can be assumed that one of the reasons for the lower values is that both procedures tested the samples at this confining stress, 21 kPa, either at the end of the testing, as in the case of T-292, or at the beginning, as with T-294, in which this test was preceded by conditioning at a high deviatoric stress (140 kPa). In both procedures, therefore, previous conditioning (T-294) and testing (T-292) stabilized the sample and reduced the stress dependency behavior to an extent beyond which the testing procedures did not contribute to any significant variation in the results.

Equation 3 is derived based on the results reported in Figure 5. This equation, which provides the procedure coefficients, is valid for both measurement systems and confining pressures of magnitudes 35, 70, and 105 kPa. For other confining pressures of 21 and 140 kPa, the coefficients remain constant for all deviatoric stresses and are around 1.08.

$$PC = 1.28 - 0.00115 * \sigma_d \quad (3)$$

where σ_d is the deviatoric stress in kPa.

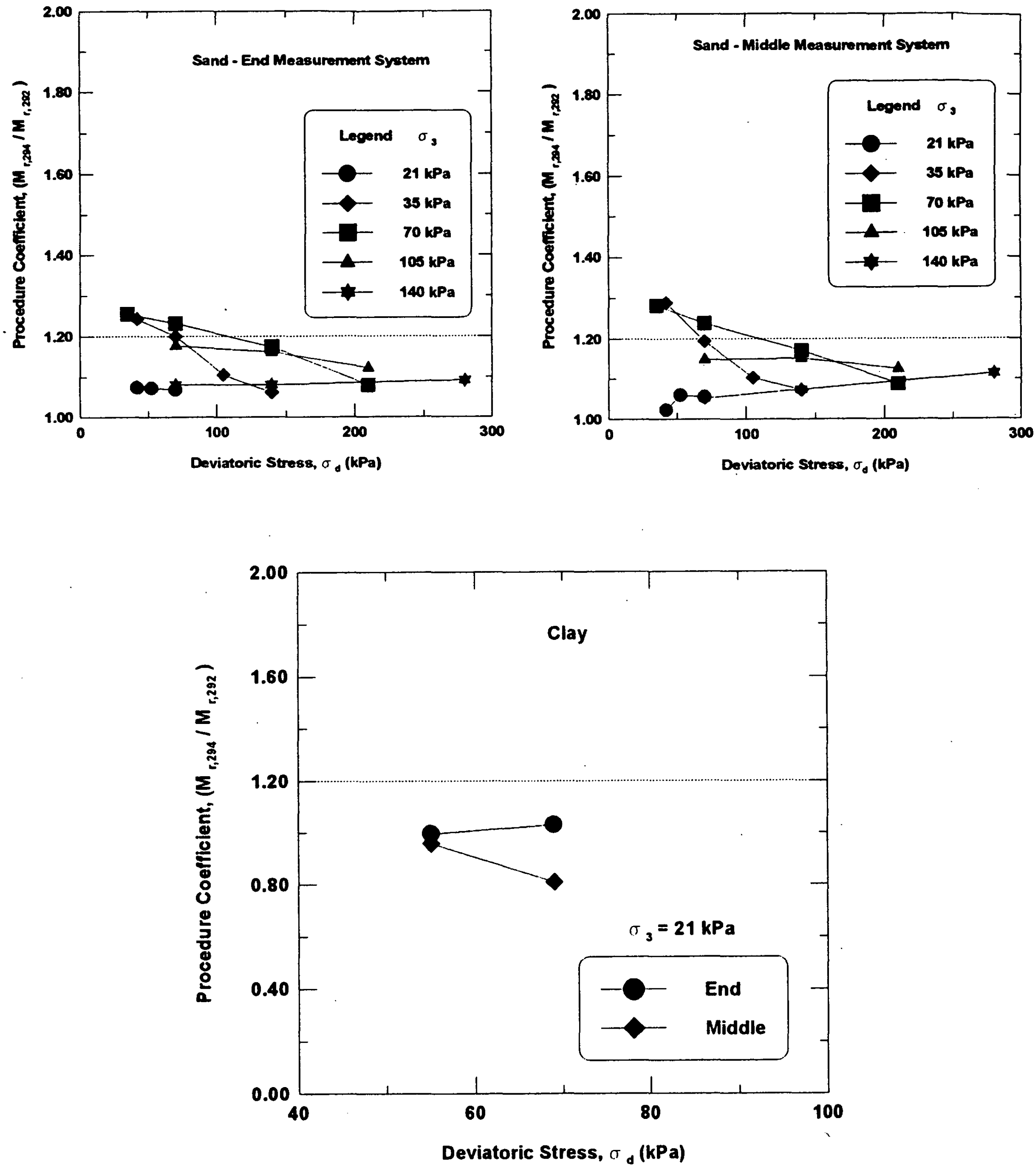


FIGURE 5 Procedure coefficients (sands and clays).

Figure 5 also depicts the procedure variation on clays but does not provide enough information for discussion because, for testing on clays, both procedures have different confining pressure and deviatoric stress sequences. The only common test stresses are the confining pressure of 21 kPa and the deviatoric stress of 72 kPa. To determine another PC value, results from the deviatoric stress of 55 kPa in T-294 and 52 kPa in T-292 are assumed to be equivalent. The PC values of these two deviatoric stresses are calculated and are also shown in Figure 5. These coefficients from both measurements are around 1.0, except at the middle system, which has a value of 0.8 at 72 kPa deviatoric stress and 21 kPa confining pressure. Swelling phenomenon and stress dependency may have occurred for the specimens tested under T-294 at 21 kPa confining pressure as a result of a drop from the previous confining stress, which was 42 kPa. These phenomena appear to have more influence on middle measurement results, therefore, lower M_r and PC values are calculated by the middle measurement system at 72 kPa deviatoric stress. Overall the procedure variation on M_r values for clay specimens is not as significant as in the case of sands because the procedures for clay specimens do not have a wide range of testing stresses and the lower magnitudes of confining pressures (0 to 42 kPa range).

Measurement System

The influence of the measurement system is presented in the form of a measurement coefficient (MC), which is defined as the ratio of the resilient modulus or axial strain measured by the middle system to that measured by the end system. These coefficient values are determined for both procedures and test stresses. The coefficient can be used to convert the end measurement system results to more realistic middle measurement system results.

Figure 6 presents the variation of MC values of sands for both AASHTO procedures. The MC values range from 1.20 at lower confining and deviatoric stresses to 1.08 at higher confining and deviatoric stresses. The lower value is due to the perfect contacts between the end plates, porous stones, and the specimen ends at higher stresses. This is the reason that both measurement systems measured relatively similar values. An average measurement coefficient value of 1.14 is recommended for converting M_r values for the end system to M_r values for the middle system.

Figure 6 also presents the MC values obtained from results on clay specimens. The influence of the measurement system can be clearly seen from this figure. MC values ranging from 1.5 to 1.6 are observed for unconfined conditions. These significantly higher coefficients are due to the complex behavior of clay specimens that can result from specimen preparation, stress history caused by the stress sequence of the testing (note that T-292 shows only loading sequence and T-294 has both loading and unloading sequences),

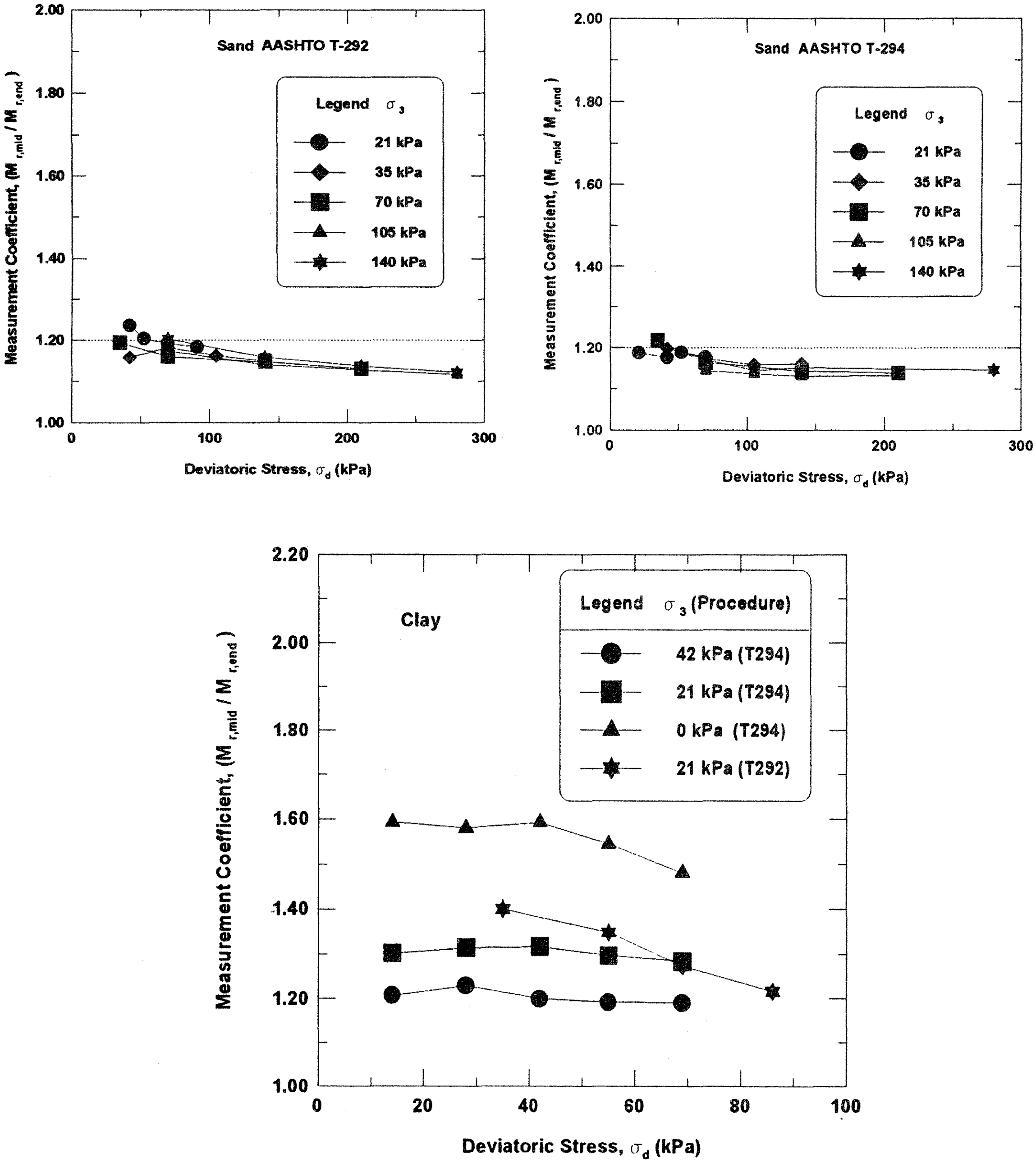


FIGURE 6 Measurement coefficients (sands and clays).

imperfect end contacts, and system compliance errors. Specimen preparation using a standard Proctor test may not produce the same soil fabric in all layers. The bottom layer is subjected to more blows or energy than the top layer, even though each layer is subjected to a similar number of blows. This, coupled with the variations due to test stress sequences that cause stress dependency behavior and errors due to instrumentation, will significantly influence the displacement measurements. The end system that measures the displacements over the full length of the specimen will be more influenced by these problems than the middle system. The end system therefore measured significantly higher displacements, which resulted in lower M_r values and higher measurement coefficients.

These MC values decrease with an increase in confining stress and, to some extent, with deviatoric stress. The MC values from both test procedures, which match at 21-kPa confining pressure, are compared in Figure 6. These values are similar and vary between 1.2 to 1.52, with most around 1.3.

The following measurement coefficient equation for clays is derived from the results shown in Figure 6. The deviatoric stress is not taken into account in the equation because its influence on MC value is relatively insignificant.

$$MC = 1.52 * e^{-0.00594 * \sigma 3} \tag{4}$$

Regression Models

Regression models are used in the form of equations for predicting the moduli. The theta (θ) or the bulk stress and the deviatoric stress are used as predictors in these models on the basis of whether the soil is cohesionless or cohesive (*8,9,12*). These models were recommended in AASHTO T-292, T-294, and Strategic Highway Research Program Protocol P-46. The model can be expressed as

$$M_r = k_1 * \theta^{k2} \quad \text{granular soils} \tag{5}$$

$$M_r = k_3 * \sigma_d^{k4} \quad \text{cohesive soils} \tag{6}$$

where k_1 and k_2 (granular soils) and k_3 and k_4 (cohesive soils) are regression coefficients.

The regression coefficients were determined from the test results for both soils (Figure 7) and are given in Table 4. It is interesting to note that $k_2$and k_4, which are slopes of the lines in the respective models, appear to be mainly dependent on the type of soil tested and to some extent on conditioning and testing procedure. In the case of sands, the variation of k_2 obtained from both AASHTO procedures (0.49 to 0.47) is negligible; however, the same is not true in the case of clay soils. The k_4 values from both procedures are significantly different from one another because of the variations in the conditioning and testing procedures. The other constants, k_1 and k_3, which are intercepts in the figures, depend both on the testing procedures and on the measurement systems. As expected, higher k_1 and k_3 values are obtained for the middle system than for the end system because of higher measurements of resilient moduli. Figure 8 shows the influence of the type of soil, the testing procedure, and the measurement system on the regression coefficients.

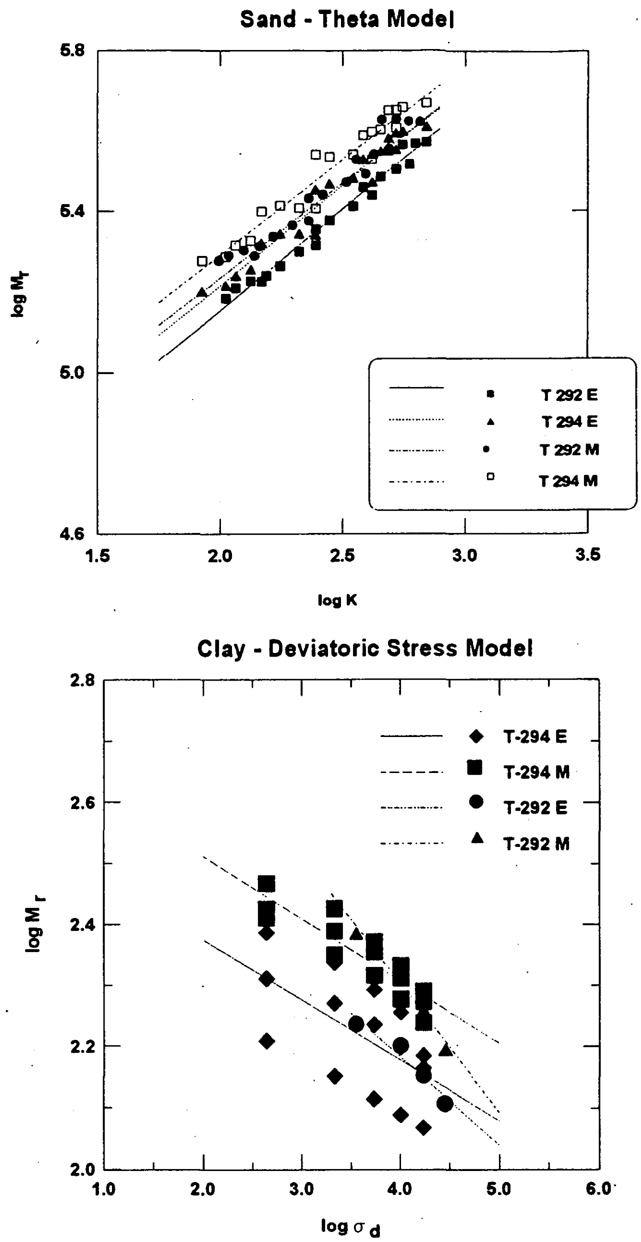


FIGURE 7 Regression models: sands (θ model) and clays (deviatoric stress model).

DISCUSSION OF RESULTS

The testing procedure influenced the resilient moduli of sands more significantly than those of clays. This can be attributed to the sequences involved in the conditioning and testing that make the specimens stress dependent and in some cases may cause some disturbance to the sample. The ranges of stresses for sands are also significantly larger than those for clays. T-294 is more conductive for testing sands because it has less variation in the deviatoric stress magnitudes in the successive test sequences (Figure 3). Moreover in the testing phase the deviatoric stress increases at each confining pressure. Therefore results from the T-294 procedure are unaffected by the specimen stress dependency phenomenon at higher stresses. The T-294 procedure for clays has both loading (deviatoric) and unloading (both deviatoric and confining stresses) phases. The range and magnitudes of confining stresses for both procedures for clays are significantly low, which may be the reason for clays not showing significant procedure variation.

TABLE 4 Regression Constants: θ and Deviatoric Stress Models

	Sand		Silty Clay	
Procedure & Measurement System	k_1	k_2	k_3	k_4
T-292E	4.15	0.49	2.75	-0.14
T-292M	4.30	0.47	3.15	-0.21
T-294E	4.23	0.49	2.57	-0.10
T-294M	4.35	0.47	2.71	-0.10

Note: E - End System; M - Middle System; T-292 and T-294 - AASHTO Procedures.

The following PC can be used to determine the moduli values of T-294 procedure:

$$M_{r,294} = PC * M_{r,292} \quad (7)$$

where

$PC = 1.08$ (sands—both measurement systems, $\sigma_3 = 21$ and 140 kPa; all σ_d values),
$PC = 1.28 - 0.0015 * \sigma_d$ (sands—both measurement systems, $\sigma_3 = 35$ to 105 kPa), and
$PC = 1.00$ for clays at all measurement systems and stresses; and 0.8 for $\sigma_3 = 21$ kPa and $\sigma_d = 72$ kPa for middle measurement system.

Because the resilient moduli values were computed based on a uniform state of stresses and strains, the middle internal measurement system will be the appropriate one to use. As explained earlier, however, the end internal measurement system is easier to use routinely than the middle system. Whenever end measurement systems are used, the measurement coefficients must be multiplied with the end measurement resilient moduli to get realistic resilient moduli that can be used in the design of flexible pavements. The measurement coefficient is presented for both soils in the following equation:

$$M_{r,\text{mid}} = MC * M_{r,\text{end}} \quad (8)$$

where

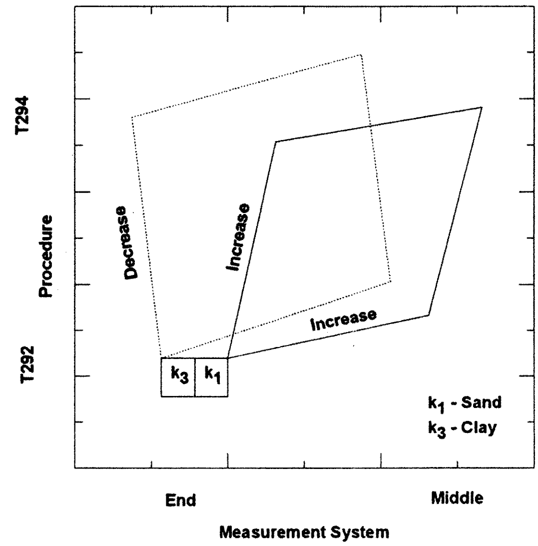
MC = 1.14 (sands—both procedures, for all σ_3 and σ_d values) and
MC = 1.41 $e^{-0.0365\,\sigma_3}$ (clays—both procedures).

These factors are only recommended for the soils tested. Additional research needs to be conducted to develop such factors for other soils.

SUMMARY AND CONCLUSIONS

Several resilient modulus tests were conducted using two AASHTO procedures on both granular and cohesive soils (A-3 sand and A-7 silty clays). Two types of internal systems were used for displacement measurements. The following conclusions were obtained from this study:

1. Procedures have some influence on sands because of the differences in the stress sequences. The T-292 procedure causes more stress dependency and disturbance to the specimen than the T-294 procedure because of the sudden stress jumps of significant magnitudes.
2. For clays tested, the resilient modulus from both procedures is not significantly different. This is thought to be due to the smaller magnitudes of confining stresses used in these procedures. Data used for understanding the procedure variation of clays are not sufficient to provide meaningful conclusions.
3. Measurement systems have more influence on clays than on sands. This is attributed to changes in the fabric of the specimen due



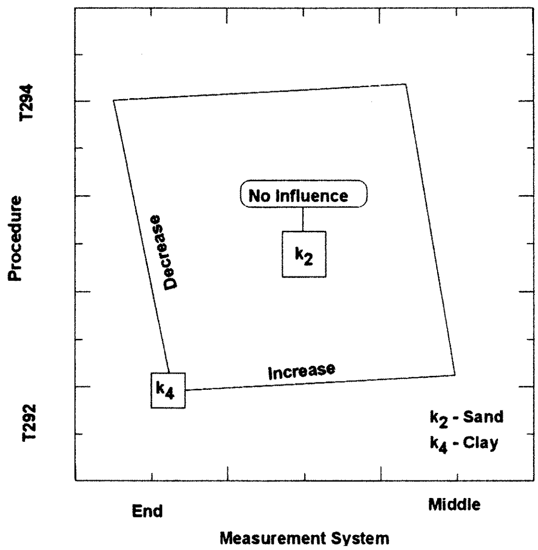


FIGURE 8 Regression model: a graphical picture about constants.

to the preparation procedures; the test stress sequence, which may have resulted in stress dependency behavior; the visco-elastic behavior of the clays; and the imperfect contacts at the ends of the specimen. Even though some of these problems are present, the primary reason for obtaining lesser measurement coefficients for sands is the perfect contacts between porous stones and the specimen ends. These perfect contacts may have occurred as a result of the higher magnitudes of the stresses at which the sands are tested.

4. A multiplier of 1.5 to 1.6 is recommended for M_r values of the end system to obtain M_r values at the middle system in an unconfined test on clays. An equation for the measurement coefficient of clays is also provided as a function of confining pressure. The same coefficient is approximately 1.12 for sands.

5. The theta and deviatoric stress models are used to determine the constants for both soils. The constants k_2 and k_4 depend mainly on the type of soil and to some extent on the testing stresses. Constants k_1 and k_3, however, depend on the measurement system and testing procedure. The middle system produced higher k_1 and k_3 values for both soils because of higher resilient modulus determinations.

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REFERENCES

1. *Guide for Design of Pavement Structures.* AASHTO, Washington, D.C., 1986.
2. Barksdale, R. D., et al. *Laboratory Determination of Resilient Modulus For Flexible Pavement Design.* NCHRP Report 1–28. Georgia Institute of Technology, 1990.
3. Kim, D. S., K. H. Stokoe. Characterization of Resilient Modulus of Compacted Subgrade Soils Using Resonant Column and Torsional Shear Tests. In *Transportation Research Record 1369,* TRB, National Research Council, Washington, D.C., 1992.
4. Sneddon, R. V. *Resilient Modulus Testing of 14 Nebraska Soils.* Project RES1(0099)P404. University of Nebraska, Lincoln, 1988.
5. Elliott, R. P., S. I. Thornton, K. Y. Foo, K. W. Siew, and R. Woodbridge. *Resilient Properties of Arkansas Subgrades.* Report FHWA/AR-89/004. Arkansas Highway and Transportation Research Center, University of Arkansas, Fayetteville, 1988.
6. Ho, R. Repeated Load Tests on Untreated Soils: A Florida Experience. *Proc., Workshop on Resilient Modulus Testing State-of-the-Practices,* Oregon State University, Corvallis, 1989.
7. Seim, D. A Comprehensive Study on the Resilient Modulus of Subgrade Soils. *Proc., Workshop on Resilient Modulus Testing State-of-the-Practices,* Oregon State University, Corvallis, 1989.
8. Thompson, M. Factors Affecting the Resilient Moduli of Soils and Granular Materials. *Proc., Workshop on Resilient Modulus Testing State-of-the-Practices,* Oregon State University, Corvallis, 1989.
9. Thompson, M. Resilient Modulus of Subgrade Soils. *Proc., Workshop on Resilient Modulus Testing State-of-the-Practices,* Oregon State University, Corvallis, 1989.
10. Baladi, G. Resilient Modulus. *Proc., Workshop on Resilient Modulus Testing State-of-the-Practices,* Oregon State University, Corvallis, 1989.
11. Houston, W. N., S. L. Houston, and T. W. Anderson. Stress State Considerations for Resilient Modulus Testing of Pavement Subgrade. Presented at 71st Annual Meeting of the Transportation Research Board, Washington, D.C., 1992.
12. Mohammad, L. N., A. J., Puppala, and P. Alavilli. Effect of Strain Measurements on Resilient Modulus of Sands. *In Special Technical Publication 1213: Dynamic Geotechnical Testing: Second Volume* (R. J. Ebelhar, V. P. Drnevich, and B. L. Kutter, eds.), ASTM, Philadelphia, Pa., 1994.
13. Mohammad, L. N., P. Alavilli, and A. J. Puppala. Data Acquisition System for Determining the Resilient Modulus of Soils. *In Geotechnical Special Publication 37: Advances in Site Characterization: Data Acquisition, Management and Interpretation,* ASCE, Dallas, Tx., 1993, pp. 27–41.
14. Pezo, R. F., G. Claros, and W. R. Hudson. An Efficient Resilient Modulus Testing Procedure for Subgrade and Non-Granular Subbase Materials. Presented at the 71st Annual Meeting of the Transportation Research Board, Washington, D.C., 1992.
15. Vinson, T. Fundamentals of Resilient Modulus Testing. *Proc., Workshop on Resilient Modulus Testing State-of-the-Practices,* Oregon State University, Corvallis, 1989.
16. Wilson, B. E., et al. Multiaxial Testing of Subgrade. *In Transportation Research Record 1278,* TRB, National Research Council, Washington, D.C., 1990.
17. *Interim Specifications for Transportation Materials and Methods of Sampling and Testing, Part II: Interim Test Methods.* AASHTO, Washington, D.C., 1992.

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