Resilient Testing of Soils Using Gyratory Testing Machine

K. P. GEORGE

Characterization of soils in terms of resilient behavior is gaining support because of its ready application in mechanistic analysis of pavements and in designing soil-structure systems. Resilient modulus (M_r) determination, using AASHTO T274-82, has been generally viewed as a complex and time-consuming test. An alternative procedure using the U.S. Army Corps of Engineers Gyratory Testing Machine (GTM) is investigated. Study shows that GTM, developed originally for the design of bituminous mixtures and subsequently used for density control of base and subgrade soils, is likely to be a feasible alternative for resilient modulus testing. The development of the GTM test procedure is described, with special focus on simulating conditions resulting from a moving load. The stress path of GTM loading is compared with that under a passing loaded vehicle to show that the GTM simulates field stress conditions. With due consideration to sample confinement in the mold, a revised equation for kneading resilient modulus, M_{rk}, is derived. For validating the test procedure, six subgrade soils and three subbase materials are investigated using the GTM and the repeated load triaxial device, and the results are analyzed with respect to material characteristics as well as test variables. Fines content, uniformity coefficient, and stress state (bulk stress) are shown to affect the M_{rk} of soils. The test variables investigated include the stress state of the sample and moisture content during compaction, the latter showing very little effect on the modulus. The important role of sample gyration (shear stress reversal) on kneading modulus is illustrated by the test results. Also included is a regression model for predicting kneading resilient modulus.

In the revised *AASHTO Guide* (*1*) the resilient modulus, M_r, was selected to replace the soil support value used in the previous editions of the guide. Resilient modulus is defined to include the recoverable part of the strain only, that is,

$$M_r = \frac{\sigma_d}{\varepsilon_{ar}} \tag{1}$$

where

σ_d = deviator stress = $\sigma_1 - \sigma_3$ = principal stress difference, and
ε_{ar} = resilient (recoverable) axial strain.

The repeated load triaxial (RLT) test proposed for determining M_r (AASHTO T274-82) is relatively complex; accordingly, highway agencies have sought other test methods. Diametral testing procedure, an alternative used in experiments by the Oregon DOT (*2*), has been found adequate for use with cohesive soils but is not recommended for use with noncohesive soils. After a careful study of the literature review, the researcher initiated this study to assess whether the U.S. Army Corps of Engineers Gyratory Testing Machine (GTM), developed originally for the design of bituminous mixtures and later used successfully for density control of base and subgrade soils, is a feasible alternative for resilient modulus testing. The GTM is described elsewhere (*3*).

Civil Engineering Department, The University of Mississippi, University, Miss. 38677.

OBJECTIVE AND SCOPE

The overall objective was to develop the GTM to perform resilient testing of soils. A basic requirement was that the test should capture the stresses/strains resulting from a moving load. The extent to which sample confinement in the GTM mold affects the kneading resilient modulus was investigated. The result is a revised equation for the kneading resilient modulus. The validity of the GTM test in characterizing subgrade soils was also investigated.

In developing the test procedure, the researcher instrumented the conventional GTM equipment to accommodate repeated loads and to sense and simultaneously record the stress and deformation in the sample. Compaction stresses as well as wheel load stresses in typical pavement subgrade were analyzed for selecting stress state and gyration angle in the GTM sample. Stress paths of both traversing load and the GTM test sample were prepared, a procedure that helped the author to select the test parameter. The test parameters were validated by performing kneading resilient modulus testing on a range of soils: six subgrade soils and three subbase materials. Using this data base, the author derived and substantiated a statistical model for predicting M_{rk}. Note that resilient modulus determined in GTM is designated as "kneading resilient modulus," M_{rk}.

GTM

The GTM, a combination kneading compaction, "dynamic consolidation," and shear testing machine, is a rather realistic simulator of abrasion effects caused by repetitive stress and intergranular movement within the mass of material (pavement or base) in a flexible pavement structure. Figure 1 is a schematic side view section of the gyrating mechanism. Mold A, containing a test specimen, is clamped in position in the flanged mold chuck B. Vertical pressure on the test specimen is maintained by upper ram E and lower ram F, acting against head G and base H, respectively. Head G acts against roller bearing and is free to slip, while base H is fixed. A gyratory

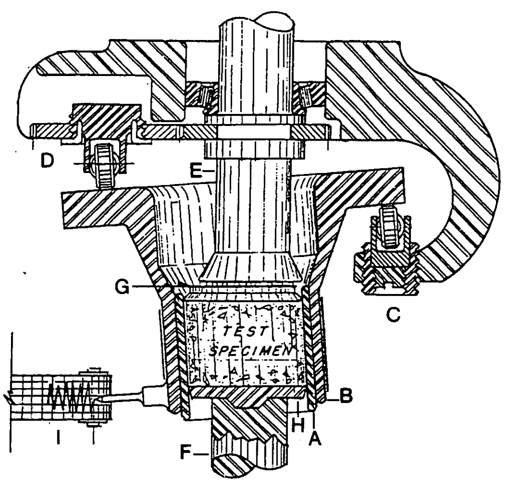

FIGURE 1 Gyratory testing machine.

motion is imparted to mold chuck B by rollers C and D as they travel around the flanged portion of the chuck. Roller C is adjustable in elevation to permit setting any desired gyratory angle (degree of shear strain). The recording mechanism I, Figure 1, shows gyratory motion or shear strain. This recording, referred to as a gyrograph, is a direct indicator of plasticity. A detailed explanation of this aspect of the machine (i.e., the gyrograph recording) is beyond the scope of this discussion except to point out that it will predict any instability that might result in either the pavement base or subgrade caused by the development of excess pore pressure.

How GTM Simulates Passing Wheel Loads

For GTM to be a viable test device, it is imperative that the stress state in the GTM sample simulate the passing of a loaded vehicle. When a moving load traverses a road, the subgrade experiences transient displacements, as shown in Figure 2. The shear stress reversal, when a moving load approaches the element in relation to when leaving the element, needs to be simulated in the test procedure.

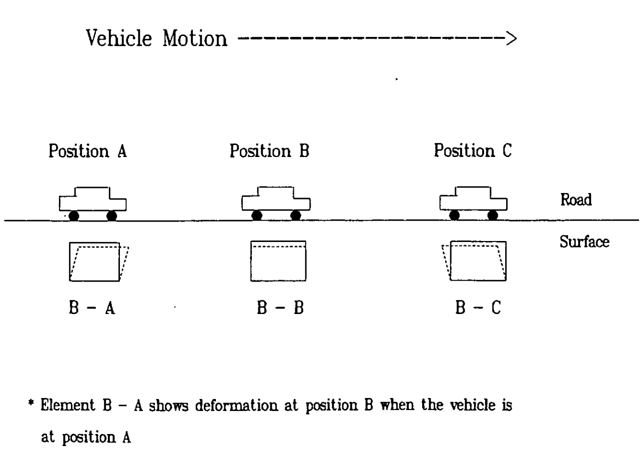

FIGURE 2 Stress reversal at Position B as the vehicle traverses from A to C.

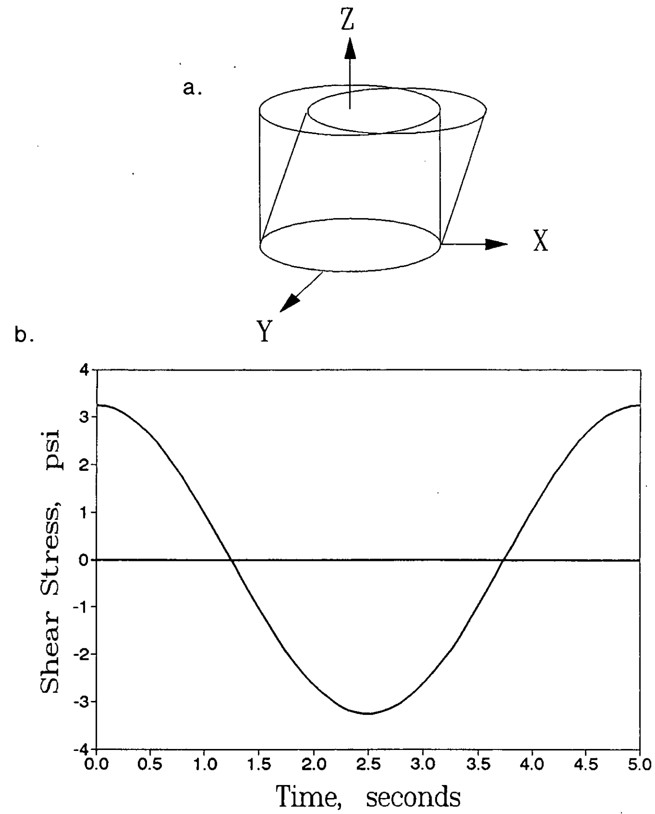

FIGURE 3 (*a*) Coordinate references system used in the finite element analysis. (*b*) Variation of τ_{zx} (with initial deflection in the *xz*-plane) during one revolution.

Stress State in GTM Sample

In the GTM test, the soil sample is confined in a steel mold and is subjected to a repeated axial stress as well as reversal of shear stresses. Whereas the axial repeated stress is applied at a frequency of 1 Hz with a 4-sec rest period, the frequency of the roller carriage and, in turn, gyratory displacement is 0.2 Hz. In other words, when the roller carriage rotates through one full cycle (2π radians) the shear stresses τ_{xz} and τ_{yz} at any point undergo nearly sinusoidal variation as shown in Figure 3. This plot is compiled from a finite element analysis of a sample (with the tacit assumption of elastic behavior of GTM sample) laterally deformed by 0.1 degree and gyrated with a frequency of 0.2 Hz. The GTM can be programmed to simulate the transient stresses generated under a moving load.

Comparison of Stress Paths

The stress path in *p-q* space of the GTM sample is compared with that of the passing vehiclular load in Figure 4. Note $p = ⅓ (\sigma_1 + \sigma_2 + \sigma_3)$, known as the mean normal stress, and $q = ½ (\sigma_1 - \sigma_3)$, designated as the deviatoric stress. Also shown in Figure 4 is the stress path generated in a conventional repeated load triaxial test. Typical stress values used to graph the three stress paths are as follows:

1. GTM sample gyrated at 0.1 degree and subjected to a cyclic load pulsating between 10 and 20 psi,

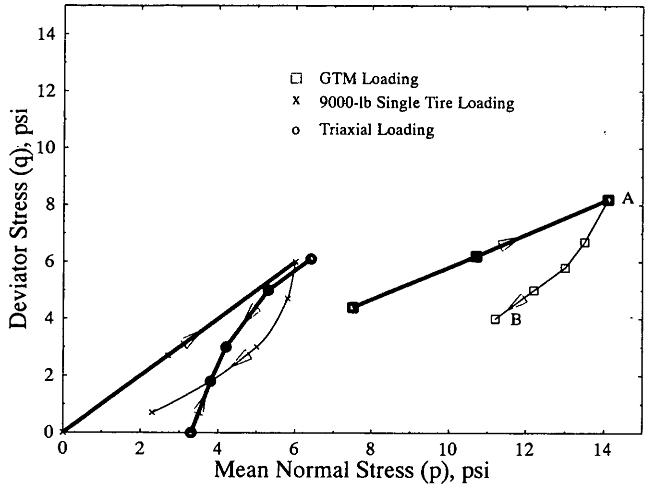

FIGURE 4 Stress paths for three load modes: (*a*) repeated loss stress pulse 10 psi/20 psi in the GTM, (*b*) 9000-lb single-tire load on 9-in.-thick pavement, and (*c*) triaxial sample repeated load stress pulse 3 psi/15 psi. Note: Point A on GTM stress path corresponds to peak load, about 1 sec after loading starts, and *B* to 1.6 sec after loading starts.

2. Stress state resulting from a 9-kip single tire (100 psi tire pressure) traversing a pavement 9 in. thick underlain by a subgrade (E = 10,000 psi), and
3. RLT sample subjected to a confining pressure of 3 psi and subsequently subjected to a cycle load between 3 and 15 psi.

In graphing the GTM stress path, the gyration-induced axial and shear stresses are taken into account. The unloading stress path for the field stress state is qualitative at best. In graphing the unload paths it is premised that for a given vertical stress, the horizontal stress will be larger during unloading than during the original loading. Although not conclusively demonstrated, there are indications that during cyclic loading and unloading the lateral stress ratio alternates approximately between K_o and $1/K_o$, where K_o is the coefficient of earth pressure at rest (*4*). As a compromise, however, the unloading lateral stress is assumed equal to the low value of cyclic stress (but not less than $K_o\sigma_{1p}$), a tacit assumption made to graph the GTM unload stress paths in Figure 4. Note that σ_{1p} is the high value of cyclic stress.

When one compares the stress paths of the three loading states, the slope of the GTM stress path resembles the field stress path, even though their magnitudes are different. In addition, in both cases the loading and the unloading paths are homologous, a fact that gives credence to the belief that the GTM is able to simulate the stress reversal phenomenon. The research recommends that, in future GTM tests, the stress level be lowered, which was not feasible in the current setup.

DEVELOPMENT OF RESILIENT MODULUS TESTING USING GTM

The gyratory repeated load test procedure envisioned in this study had to be developed and standardized. Because sample compaction is performed in the GTM, a compaction procedure must be in place before the repeated load test can be conducted. The compaction pressure and the gyration angle are chosen to simulate the state of the soil material during field densification, for example, roller compaction. Since the resilient behavior of a soil is controlled by stress state, among other factors, the stress levels during modulus testing should correspond to those anticipated under the passing of a loaded vehicle. The test procedure calls for compacting coarse and fine grain soils, respectively, at 50 and 70 psi vertical pressure. Regardless of soil type, the gyration angle is set at 0.5 degrees. Because the resilient test pressure is lower than the compacting pressure, a 2-hr waiting period to allow for the sample rebound is also programmed in the test procedure. Resilient deformation is measured at three levels of cyclic pressure: 10 psi/20 psi, 15 psi/25 psi, and 25 psi/35 psi, all at the 0.1-degree gyration angle, which then will be followed by a 10 psi/20 psi pressure test at 0-degree gyration angle. A step-by-step procedure of the test adopted in the study is given elsewhere (*5*).

Instrumentation of GTM

The GTM, modified as an integral part of this study, not only compacts the soil to a user input density (AASHTO T99 density, for example) but also performs repeated load test in the samples confined in the steel mold. Samples 4 in. in diameter and 2.5 in. high are used in the GTM for resilient testing. The equipment modifications were extensive and focused in two areas—repeated load testing and data acquisition:

1. A system to measure specimen height, gyration angle, ram pressure, and sample deformation, both compression and rebound, during repetitive load pulse;
2. A three-channel strip chart recorder to record any three of the four attributes in Item 1; and
3. A computer-based controller and data acquisition system and the associated software for data reduction.

Equation for Kneading Resilient Modulus

In the GTM, the sample, after being compacted in the mold, is subjected to a stress pulse, with a peak value smaller than the compaction pressure. The implication of this large compaction stress is that the sample tends to retain large lateral stress and continues to do so during the loading and unloading segments of the repeated stress pulse. Accordingly, a tacit assumption is made that the frictional resistance in the mold during loading and unloading is identical. An expression for the recoverable strain, ε_{ir}, is as follows:

$$\varepsilon_{ir} = \frac{(1 + \nu)(1 - 2\nu)}{E(1 - \nu)} \left(\sigma_{ir} - \frac{2}{3}\frac{h}{R}f\right) \tag{2}$$

where

ν = Poisson's ratio,
E = modulus (psi),
σ_{ir} = rebound stress in the axial direction (psi),
h = height of sample (in.),
R = radius of sample (in.), and
f = fully mobilized frictional resistance (psi).

TABLE 1 Soil Characteristics

Soil No.	Location Hwy/County	Passing #200 Sieve, %	Atterberg Limits LL	Atterberg Limits PI	Proctor Test Data Maxm. Density, lb/cu.ft	Proctor Test Data Optimum Moisture %	Soil Classification AASHTO/Unified	Poisson's Ratio
2	US98/Forrest & Perry	19	0	NP	122.4	10.4	SP-SM/A-3	0.25
3	MS7/Yalobusha	26	22	4	120.2	11.9	SM-SC/A-2-4	0.30
4	US49/Sunflower	70	32	13	116.9	15.1	CL/A-6(7)	0.35
5	US49/Sunflower	89	40	18	110.3	15.7	CL/A-6(16)	0.35
6	US61/Coahoma	97	70	39	97.5	23.0	CH/A-7-5(45)	0.40
7	US78/Benton & Union	51	26	7	123.3	11.5	ML-CL/A-4(1)	0.30
8*	US98/Forrest & Perry	23	0	NP	122.9	10.7	SM/A-2	0.25
9*	MS7/Yalobusha	12	0	NP	111.7	10.8	SP-SM/A-2	0.25
10*	US98/Forrest	10	0	NP	119.9	9.5	SP-SM/A-3	0.25

*subbase material
1 lb/cu.ft = 0.157 kN/m^3

The maximum frictional resistance is developed at the bottom of the mold where the slippage is maximum with zero resistance at the top where the deformation is negligible.

To minimize the wall friction effect, the mold is lightly oiled; accordingly, $f \rightarrow 0$. Therefore, Equation 2, upon being transposed, becomes

$$E \text{ or } M_{rk} = \frac{\sigma_{ir}}{\varepsilon_{ir}} \frac{(1 + \nu)(1 - 2\nu)}{(1 - \nu)} \tag{3}$$

Recognizing how important is Poisson's ratio in Equation 3, the author undertook a review of the literature using the following recommended values: sand, 0.25; sandy/silty/clay, 0.35; and clay, 0.40. This recommendation is based on a statistical analysis of the data assembled from the literature, the details of which are presented elsewhere (*5*). On the basis of the findings of this analysis, an appropriate Poisson's ratio is assigned to each soil. They are given in Table 1.

GYRATORY RESILIENT MODULUS TESTING

Experimental Test Program

Six different subgrade soils covering a wide range of soils in the state of Mississippi were selected for resilient modulus determination. Three Class C subbase materials were also included in the testing program. All of the nine soil materials have been used recently in pavement construction. Dynaflect deflections were obtained on five of these pavements at various stages of construction, making it possible to backcalculate the in situ modulus of each layer. Table 1 gives the index properties and classification symbols of nine soil materials. A range of different gradations is represented, as indicated in Table 1.

The experiment design called for performing three series of testing: Three or more samples from each soil at optimum moisture and AASHTO T99 (standard Proctor) density composed Series 1. Series 2 is similar to Series 1 except that the samples are compacted to "equilibrium density." Equilibrium density is accomplished in "fully compacted condition" of the material when subjected to kneading pressure at the respective vertical stress and gyration angle. A material attains fully compacted condition provided the next 100 revolutions cause an increase in density of 1 lb/ft^3 or less (*6*). How moisture affects M_{rk} is studied by testing a third series in which each soil is tested at AASHTO density and moisture above and below optimum moisture.

The effect of state of stress in the sample on M_{rk} is investigated by including a range of stresses in the testing program. Accordingly, M_{rk} values are determined in all of the samples at three stress pulses: 10 psi/20 psi, 15 psi/25 psi, and 25 psi/35 psi. Gyration angle is set at 0.1 degree for all stress combinations listed. Following the combinations mentioned, the gyration angle in each test is brought to zero and tested at 10 psi/20 psi stress pulse. The latter tests simply determine the confined resilient modulus of the material for the purpose of comparison. When switching from one stress pulse to the next, the sample is conditioned (40 cycles) at the desired setting before recording the data in the ensuing 10 cycles.

Data Analysis

A brief discussion of error analysis, including the statistical methods used to compile the data, is presented first, followed by a discussion of the results.

Uncertainty Analysis of Experimental Data

The total error of M_{rk} determination comprises bias error and precision error. A regimented calibration schedule of the LVDT as well as the pressure transducers has helped to minimize the bias error. What remains is the random error or the precision error. On the basis of numerous repeated measurements, appropriate values of precision limits have been chosen: ± 1 psi on pressure measurement and ± 0.00016 in. on deformation measurement. Using the data reduction equation (Equation 3), the uncertainty in each M_{rk} determination is calculated to be $\pm 2{,}400$ psi. This value is judged to be high considering that M_{rk} values generally fall in the range of 5,000 to 20,000 psi. The experimental design, calling for a minimum of three replicated samples with three or more observations of each sample, has the objective of reducing the test variation to tolerable levels.

Data Reduction Procedure

In the main experiment (Series 1 and 2), not less than three samples were tested, with three or more observations of each sample. In other experiments, the testing program was scaled down both in number of samples and number of observations. The data reduction starts with applying Chauvenet's criterion for statistically rejecting "wild" readings from the sample measurements of three or more. Following this procedure, an analysis of variance (ANOVA) provides a basis for estimating the variation within groups as opposed to variation between groups. The *F*-test is subsequently used to test the hypothesis that all of the groups (three or more in the first and second test series) do not belong to the same population. In the event that the hypothesis is accepted, a technique known as least significant difference is used to isolate the outlier group, if any, from the sample population. Data from each soil are scrutinized in two steps (as detailed above) and then analyzed. The results are discussed in the following sections.

Discussion of Kneading Resilient Modulus Results

To what extent M_{rk} of soils is affected by soil characteristics or test variables is discussed in the ensuing sections.

Soil Gradation

The kneading resilient modulus increases with (*a*) a decrease in the finer fraction of soil (PF, passing No. 200 sieve), (*b*) an increase in uniformly coefficient (UC) of soil (see Table 2), and (*c*) an increase in UC/PF ratio. Column 7 of Table 2 is obtained by averaging the M_{rk} values of not less than three samples, each providing four kneading moduli at 10 psi/20 psi stress pulse. That the resilient modulus in fine-grained soils decreases with fines content has been observed in previous RLT studies, for example, Drumm et al. (*7*).

Moduli at AASHTO and Equilibrium Densities

The summary results in Table 2 indicate that the M_{rk} values at equilibrium densities are for the most part (except in Soils 3 and 5) lower than those at the AASHTO densities despite the fact that equilibrium densities exceed their AASHTO counterpart, though by only one or two units. The decrease in modulus may be traced, in part, to the relatively large number of gyratory revolutions (see Column 6 of Table 2) required to attain the so-called equilibrium density. It may be that soils subjected to repetitious shear deformation tend to become plastic or show strain softening behavior. Therefore, any attempt to attain higher densities by reworking the soil with accompanying large displacement should be discouraged.

Modulus Influenced by Stress State

The effect of stress state on M_{rk} is investigated in the second series of tests, where samples tested at 10 psi/20 psi are retested at higher stress levels 15 psi/25 psi and 25 psi/35 psi while keeping the cyclic deviator stress constant at 10 psi. Typical results of M_{rk} versus bulk stress, θ, given in Table 3 indicate that M_{rk} increases as expected with θ for all the soils, including both cohesionless and cohesive soils.

Effect of Moisture on M_{rk}

Modulus results of each soil at AASHTO dry density and at OMC, dry and wet of OMC, are given in Table 4. Whether the three groups of moduli are statistically different is indicated in the sixth column of the table. The results do not conclusively show that moisture significantly affects kneading resilient modulus of soils. This finding deviates from other results (*8,9*) indicating that moisture exerts a strong influence on modulus determined in the RLT device—on the dry side

TABLE 2 Kneading Resilient Modulus Related to Percent Passing No. 200 Sieve (Ascending Order)

Soil Group (1)	Soil No. (2)	Uniformly Coefficient (UC) (3)	UC/PF Ratio (4)	Density Mode, lb/cu.ft. (5)	Chuck Revolutions to Attain Density (average) (6)	Kneading Resilient Modulus, PSI (7)
Coarse-Grained	10	7	0.70	AASHTO, 119.9 Equilibrium, 121.3	96 200	15,780 10,610
	9	155	12.92	AASHTO, 111.7 Equilibrium, 112.3	215 125	16,610 15,470
	2	130	6.84	AASHTO, 122.4 Equilibrium, 126.3	80 216	16,150 14,170
	8	>110	4.78	AASHTO, 122.9 Equilibrium, 126.0	28 181	16,210 13,380
	3	160	6.15	AASHTO, 120.2 Equilibrium, 124.9	61 190	12,840 14,100
Fine-Grained	7	>38	0.75	AASHTO, 123.3 Equilibrium, 121.7	178 124	11,110 8,690
	4	>16	0.23	AASHTO, 116.9 Equilibrium, 116.6	140 108	9,110 8,220
	5	>7	0.08	AASHTO, 110.3 Equilibrium, 111.4	138 132	6,330 8,720
	6	>2	0.02	AASHTO, 97.5 Equilibrium, 98.3	50 123	7,630 7,130

1 lb/cu.ft = 0.157 kN/m^3
1 psi = 6.894 kPa

TABLE 3 Kneading Resilient Modulus Influenced by Stress State (Bulk Stress = $\sigma_1 + \sigma_2 + \sigma_3$)

Soil Group (1)	Soil No. (2)	Stress State, psi (3)	Bulk Stress, psi (4)	Kneading Resilient Modulus, PSI: Gyration Angle, 0.1 degree: AASHTO Density (5)	Gyration Angle, 0.1 degree: Equilibrium Density (6)	Gyration Angle, zero degree: AASHTO Density (7)	Gyration Angle, zero degree: Equilibrium Density (8)
Fine-Grained	4	10/20*	35	9110	8220	13890	14300
		15/25	50	11360	10450		
		25/35	80	11420	11370		
	6	10/20	35	7630	7130	13350	14050
		15/25	50	7500	7920		
		25/35	80	9020	9220		
Coarse-Grained	8	10/20	35	16210	13380	29860	30110
		15/25	50	16720	14590		
		25/35	80	18770	15850		
	9	10/20	35	16610	15470	28210	27390
		15/25	50	17430	15550		
		25/35	80	18760	17120		

*low pressure/high pressure during repeated load sequence
1 psi = 6.894 kPa

TABLE 4 Kneading Resilient Modulus Determined at Dry- and Wet-of-Optimum (Load Pulse 20 psi/10 psi)

Classification by Use (1)	Soil No. (2)	Kneading Resilient Modulus, psi/Compaction Moisture, %: Dry of OMC* (3)	OMC (4)	Wet of OMC (5)	Difference Between Moduli in Columns 3, 4 and 5 (6)
Subgrade	2	15460/9.4	16150/10.4	14250/11.4	not significant
	3	12910/10.9	12840/11.9	13630/12.9	not significant
	4	8590/13.1	9110/15.1	6890/17.1	not significant
	5	7110/13.7	6330/15.7	7800/17.7	not significant
	6	7420/21.0	7630/23.0	10630/25.0	significant
	7	5700/11.2	11110/13.0	5500/14.3	significant
Subbase	8	15170/9.7	16210/10.7	13470/11.7	not significant
	9	11650/9.8	16610/10.8	13120/11.8	significant
	10	12000/8.5	15780/9.5	13,000/10.8	significant

*Optimum moisture content
1 psi = 6.894 kPa

of optimum modulus increases significantly and vice versa. In discussing this phenomenon Fredlund et al. (*9*) used the concept of matrix suction as a surrogate for degree of saturation and reported that modulus increases with matrix suction. If the moisture in the GTM sample was varied over a large range, for example 20 to 30 percent of OMC, M_{rk} might have shown a similar trend, perhaps of a lesser order than what Fredlund et al. observed.

Why is the GTM modulus not significantly affected by minor variation in moisture, like the RLT modulus? One fundamental difference between GTM and RLT devices is that the sample is not gyrated in the latter (that is, no shear strain reversal). Because the GTM sample is kneaded during modulus testing, the air-water interface is continually changing; this action causes matrix suction to be relatively small. Accordingly, minor moisture changes have an insignificant effect on matrix suction and, in turn, on kneading modulus. Considering this result, one may expect moisture-related moduli variation to be minimal in the GTM.

Angle of Gyration (Shear Strain) on Kneading Resilient Modulus

The researcher recommended that GTM modulus tests be conducted at a 0.1-degree (1,700-μrad) gyration angle inducing cyclic shear stresses (τ_{zx} and τ_{zy}, Figure 3) at 0.2 Hz. To determine the effect of this cyclic shear stress on the modulus, the samples included in Series 1, 2, and 3 are also tested at 0-degree angle (no kneading), and the results are compared in Table 3 (Columns 5 and 7, and 6 and 8). The results are convincing that the modulus increases under a no-kneading condition (or simple confined compression). Depending on the soil type, the increase in confined modulus can be anywhere from 50 to 80 percent.

The effect of shear stress (strain) reversal on stiffness of soil and susceptibility to liquefaction has been well documented in the literature (*10*). Analyzing in situ results, May and Witczak (*11*) concluded that in situ effective modulus of granular material is a function of not only the stress state but also the magnitude of the shear strain induced in that layer by the surface loading. At low levels of shear strain, the effective in situ modulus is much larger than at higher levels of shear strain.

The decrease in resilient modulus with gyration angle can be attributed to the nonlinear constitutive stress-strain relationship of soils. The explanation offered here invokes the Duncan-Chang nonlinear stress-strain model (*12*). According to them, the modulus at any given deviatoric stress, $\sigma_1 - \sigma_3$, can be related to the initial tangent modulus, E_i, as follows:

$$E_t = 1 - \left[\frac{R_f(1 - \sin\phi)(\sigma_1 - \sigma_3)}{2c\cos\phi + 2\sigma_3\sin\phi}\right]^2 E_i \qquad (4)$$

where

$E_t = \frac{\Delta(\sigma_1 - \sigma_3)}{\Delta\varepsilon_1}$, modulus at deviatoric stress $\sigma_1 - \sigma_3$;

$R_f = \frac{(\sigma_1 - \sigma_3)_f}{(\sigma_1 - \sigma_3)_{ult}}$;

c = cohesion;

ϕ = angle of internal friction; and

E_i = initial tangent modulus.

In a confined sample, if the maximum principal stress is σ_1, the minimum principal stress, σ_3, is $K_o\sigma_1$, both of which will be changed (σ_1 increases and σ_3 decreases) owing to the shear stress induced by sample gyration. Modified by the deviator stress, $(\sigma_1 - \sigma_3)$ in the gyrated sample is larger than that in a simply confined state. The enhanced $(\sigma_1 - \sigma_3)$ as well as the smaller σ_3 decreases E_t in Equation 4, thus confirming the experimental observations.

Prediction of M_{rk} from Test Variables and Soil Characteristics

After testing a range of subgrade and subbase materials, it would be instructive to formulate a model for predicting M_{rk} of soils, preferably using basic soil characteristics and test variables. Not only will an equation prove useful to agencies in preliminary design calculations, but it also helps to delineate the most critical explanatory variables (in some sense, sensitivity analysis of the model) that determines kneading resilient modulus of soils.

The factors that affect resilient modulus may be divided into two categories: material and test variables. Important material variables include dry unit weight, degree of saturation/moisture content, aggregate gradation, uniformity coefficient, fines content, and Atterberg limits. Among the test variables, confining stress, number of loadings, and stress state/deviatoric stress have the greatest effect on resilient modulus. In the GTM sample, because confining stress, σ_c, is related to vertical stress ($\sigma_c \approx K_o\,\sigma_1$), the effect of σ_c can be only indirectly related to resilient modulus.

Using those variables, a regression equation with M_{rk} as the dependent variable is developed. A multivariate statistical analysis—specifically, a backward stepwise selection procedure—was adopted in which the independent variables were removed from the model one at a time starting with the variable with the least statistical significance. The resulting model, as in Equation 5, yields a coefficient of determination, $R^2 = 0.89$.

$$\log M_{rk} = 9.447576 - 0.007381\text{PF} + 0.022538\text{UC/PF} + 0.0046934\theta \quad (5)$$

where

M_{rk} = kneading resilient modulus (psi),

PF = percent finer than No. 200 sieve,

UC/PF = ratio of uniformity coefficient and percent fines, and

θ = bulk stress = $\sigma_1 + \sigma_2 + \sigma_3$,

in which σ_1, σ_2, and σ_3 = maximum, intermediate, and minimum principal stress (psi), respectively.

In view of the relatively high R^2, a recommendation is in order here that the model can provide a satisfactory prediction of resilient modulus of soils and warrant consideration in selecting preliminary input values in the revised AASHTO pavement design procedure.

Repeated Load Triaxial Test Results

For comparison, the nine soils were tested in triaxial mode (three or more samples for each soil) using the AASHTO T274 procedure. The averaged resilient modulus, after applying Chauvenet's criterion, is given in Table 5. As expected, coarse-grained soils exhibit larger values than the fine-grained counterpart. For comparison purposes, the moduli of soils tested in this research are predicted using empirical equations of other researchers. Columns 6, 7, and 8 of Table 5, respectively, give resilient moduli calculated using the empirical equations of Carmichael and Stuart (*13*), Drumm et al. (*7*), and Elliot et al. (*14*). Recognizing that the experimental precision is ±2,400 psi, the equations of Carmichael and Stuart predict the moduli of coarse-grained soils rather well. Of the four fine-grained soils, only Soil 7 modulus agrees with that

TABLE 5 RLT Resilient Modulus of Nine Soils Compared with Those of Other Researchers

Soil Group	Soil Number/ Classification	Percent Passing #200 (PF)	Atterberg Limits		RLT Resilient Modulus, psi			
			LL	PI	Repeated Load Triaxial	Charmichael et al. (13)	Drumm et al. (7)	Elliot et al. (14) σ_d = 8 psi
(1)	(2)	(3)	(4)		(5)	(6)	(7)	(8)
Coarse-Grained	10/A-3	10	0	NP	20,340*	22,580		
	9/A-2	12	0	NP	21,260*	21,770		
	2/A-3	19	0	NP	17,500*	21,550		
	8/A-2	23	0	NP	23,830*	21,220		
	3/A-2-4	26	22	4	17,870*	20,680		
Fine-Grained	7/A-4(1)	51	26	7	17,700**	16,230	4,000	7,160
	4/A-6(7)	70	32	13	13,470**	9,490	10,290	5,950
	5/A-6(16)	89	40	18	11,400**	4,120	10,850	6,910
	6/A-7-5(45)	97	70	39	16,610**	25,310	17,920	9,660

*Resilient modulus at bulk stress 40 psi
**Resilient modulus at deviatoric stress 10 psi and confining pressure 3 psi

predicted by the equation of Carmichael and Stuart. The equation of Drumm et al. is meant to predict modulus of fine-grained soils only. With the exception of Soil 7, the agreement is satisfactory. The Elliot et al. equation, which again is recommended for fine-grained soils, underpredicts the test values determined in the present study.

The comparative analysis suggests that the triaxial resilient moduli values of coarse-grained soils determined in this research are realistic, which cannot be said about those of fine-grained soils. The question now is how the RLT moduli compare with the GTM moduli. The M_{rk} values at 0.1-degree gyration angle are consistently lower than the respective RLT moduli (compare Column 7 of Table 2 with Column 5 of Table 5). One exception is where in the fine-grained soils the gyratory moduli agree with at least one of the predicted values (Column 6, 7, or 8 of Table 5). The 0-degree gyratory moduli of fine-grained soils show some agreement with the corresponding RLT moduli obtained in this study; however, in coarse-grained soils the gyratory moduli (at a 0-degree angle) exceed the RLT results by 5,000 to 6,000 psi. It is the sample confinement afforded by the steel mold that is responsible for the significant increase of moduli in coarse-grained soils, which seems to have no effect on fine-grained soils. Guided by this comparative analysis, the writer cautiously concludes that the GTM procedure includes the ingredients (for example, the sample stress state) that can yield a realistic evaluation of resilient properties of subgrade soils, especially fine-grained soils.

CONCLUDING REMARKS

This study is designed to develop the GTM for repeated load testing of subgrade soils. The specific requirement that the test parameters simulate subgrade stresses/strains under the passing of a loaded vehicle is given paramount consideration. Taking into account the sample confinement and possible wall friction in the mold, an equation has been derived for the kneading resilient modulus. Nine soil materials are tested for kneading resilient modulus, M_{rk}, as well as triaxial resilient modulus, M_r. The variation of M_{rk} with soil composition (texture), dry density, and the stress state are in agreement with reported results of repeated load triaxial device. M_{rk}, however, is very little influenced by fluctuations in compaction moisture. The fact that the resilient modulus is significantly affected by the angle of gyration (which is proportional to the induced shear strain) hints that for realistic modulus determination the test must simulate shear stress reversal, a condition associated with moving loads. Using the data base resulting from this study, a prediction model for samples compacted at AASHTO density is developed with the percentage of fines, the uniformity coefficient, and the bulk stress as the explanatory variables.

The gyratory modulus, M_{rk}, is now compared with the triaxial modulus, M_r, with the objective of authenticating the gyratory test. A cursory examination of the M_r values and those predicted by three different empirical models reveals that RLT tests yield reasonable modulus values in coarse-grained soils. However, the results in fine-grained soils are somewhat questionable. The kneading moduli in coarse-grained soils, though slightly smaller (69 to 92 percent) than the corresponding M_r values, are considered realistic. The kneading moduli of fine-grained soils, despite not showing any relation to the RLT values determined in this study, signal some agreement with either of two predicted triaxial moduli values. Relying on the overall agreement with the published results, the writer concludes that the GTM procedure includes the necessary ingredients that can yield a realistic evaluation of resilient properties of subgrade soils, especially fine-grained soils.

Although nine soils have been tested for M_{rk} values, their relative values are more valuable than the absolute value of each soil. One reason for this is that the Poisson's ratio plays a major role in M_{rk} calculation (see Equation 3), and that the Poisson's ratio adopted for each soil is an estimate at best. The height-diameter ratio's being less than 1 cannot be considered a major factor because the sample is perfectly confined in the rigid mold during testing. The wall friction effect on M_{rk} is minimized by lightly oiling the mold before sample preparation.

ACKNOWLEDGMENT

This report includes results of one phase of a study, *Resilient Modulus of Subgrade Soil Using Gyratory Testing Machine*, sponsored by the Mississippi State Highway Department and FHWA. Joe Sheffield and Alfred Crawley of the Mississippi State Highway Department helped to shape the project scope. A. S. Rajagopal supervised the testing phase, contributing immensely to this research program. The tests were performed by research assistants in the Civil Engineering Department.

REFERENCES

1. *AASHTO Guide for Design of Pavement Structures*. American Association of State Highway and Transportation Officials, Washington, D.C., 1986.
2. J. R. Montalvo, C. A. Bell, J. E. Wilson, and J. T. Scofield. *Application of Resilient Modulus Test Equipment for Subgrade Soils*. Report FHWA-OR-84-3. Oregon State University, Corvallis, 1984.
3. *Development of the Gyratory Testing Machine and Procedures for Testing Bituminous Paving Mixtures*. Technical Report 3-595. U.S. Army Engineer Waterways Experiment Station, Corps of Engineers, Vicksburg, Miss., 1962.
4. T. W. Lambe and R. V. Whitman. *Soil Mechanics*. John Wiley and Sons, New York, 1969.
5. K. P. George. *Resilient Modulus of Subgrade Soil Using Gyratory Testing Machine*. Final Report. Mississippi State Highway Department, Jackson, 1991.
6. *Gyratory Compaction Method for Determining Density Requirements for Subgrade and Base of Flexible Pavements*. Misc. Paper 4-494. U.S. Army Engineering Waterways Experiment Station, Corps of Engineers, Vicksburg, Miss., May 1962.
7. E. C. Drumm, Y. Boateng-Poku, and T. J. Pierce. Estimation of Subgrade Resilient Modulus from Standard Tests. *Journal of Geotechnical Engineering*, ASCE, Vol. 116, No. 5, 1990.
8. M. R. Thompson and Q. L. Robnett. Resilient Properties of Subgrade Soils. *Journal of Transportation Engineering*, ASCE, Vol. 105, No. 1, 1979.
9. D. G. Fredlund, A. T. Bergan, and P. K. Wong. Relation Between Resilient Modulus and Stress Conditions for Cohesive

Subgrade Soils. In *Transportation Research Record 642*, TRB, National Research Council, Washington, D.C., 1977.
10. A. Casagrande, *Liquefaction and Cyclic Deformation of Sands—A Critical Review*. Harvard Soil Mechanics Series No. 88. Harvard University, Cambridge, Mass., 1976.
11. R. W. May and M. W. Witczak. Effective Granular Modulus to Model Pavement Responses. In *Transportation Research Record 810*, TRB, National Research Council, Washington, D.C., 1981.
12. J. M. Duncan and C. Y. Chang. Nonlinear Analysis of Stress and Strain in Soils. *Journal of Soil Mechanics and Foundation Division*, ASCE, Vol. 96, No. 5, 1970.
13. R. F. Carmichael and E. Stuart. Predicting Resilient Modulus: A Study To Determine the Mechanical Properties of Subgrade Soils. In *Transportation Research Record 1043*, TRB, National Research Council, Washington, D.C., 1985.
14. R. P. Elliot, S. I. Thornton, K. Y. Foo, K. W. Siew, and R. Woodbridge. *Resilient Properties of Arkansas Subgrades*. TRC-94. Arkansas Highway and Transportation Research Center, 1988.

DISCUSSION

WAHEED UDDIN
Department of Civil Engineering, The University of Mississippi, University, Miss. 38677.

The use of Gyratory Testing Machine (GTM) for laboratory characterization of resilient modulus (M_r) of granular material and cohesive subgrade soils will provide a practical and relatively simple-to-use alternative to more complex triaxial testing procedures (AASHTO T274-82 and SHRP-LTPP Protocol P46-1989). The author and the sponsors of the research study are commended for taking initiative in this direction. The author's final report on this research study (*1*) also includes a comparison with the backcalculated moduli that further supports the GTM alternative.

Table 6 (*1*) gives the M_r values determined by the GTM method and the subgrade moduli backcalculated from the Dynaflect deflection basins measured at these test sites and analyzed by the MODULUS (*2*) and FPEDD1 (*3*) programs. It is interesting to note the following:

1. Both backcalculation programs give comparable results (see Columns 4 and 5 of Table 6). These backcalculated moduli are generated by the two computer programs by matching measured with theoretical deflections. The FPEDD1 program does not require any user input to generate seed moduli.
2. The backcalculated moduli (without correction for nonlinear behavior) are significantly higher, on the order of 80 to 200 percent, than the GTM resilient moduli. In the case of Soil 5, the backcalculated moduli are about three times higher than the laboratory GTM resilient modulus.
3. The final solution of the FPEDD1 program is a set of effective in situ moduli after correction for nonlinear behavior of both the unbound granular base/subbase layer and subgrade. These corrected moduli are shown in Column 3 of Table 6.
4. The corrected FPEDD1 moduli (Column 3) and the GTM resilient moduli (Column 2) compare very well; the difference is generally less than 10 percent. The only exception is Soil 5, for which the corrected FPEDD1 modulus is about 1.5 times higher than the GTM resilient modulus.

TABLE 6 Comparison of Laboratory Resilient Moduli with Backcalculated Moduli for Various Roadbed Soils (*1*)

Soil No.	M_r, psi (GTM, at 0.1°)	Dynaflect Backcalculated Moduli, psi		
		FPEDD1 Corrected	FPEDD1 Uncorrected	MODULUS
Column 1	2	3	4	5
10	15,780	17,980	28,000	28,230
2	16,150	15,820	24,750	25,030
3	12,840	12,940	25,820	24,420
4	9,110	8,360	16,370	16,000
5	6,330	9,280	17,630	17,330

Overall, these results indicate the strong need of applying correction to the backcalculated moduli for nonlinear behavior of unbound pavement layers and roadbed soils. The results further provide evidence that the self-iterative nonlinear algorithm of the FPEDD1 program is a practical approach to backcalculating effective in situ nonlinear moduli for these unbound pavement materials and soils.

The FPEDD1 and RPEDD1 programs apply correction for nonlinear behavior using the strain-softening models derived from earthquake engineering studies (*3*). The recent work at the University of Texas at Austin (*4*) supports this approach, in which normalized modulus versus strain relationships developed by conducting series of resonant column, torsional shear, and resilient modulus tests show approximately the same strain-softening pattern as that used in the FPEDD1 and RPEDD1 programs. Further laboratory work using repeated triaxial testing is in progress at The University of Mississippi, and falling weight deflectometer tests are also planned at these selected test sites. The MODULUS and FPEDD1 programs will be used again to analyze the deflection basins. The author is encouraged to present the result of these ongoing and planned studies to benefit the pavement community.

REFERENCES

1. K. P. George. *Resilient Modulus of Subgrade Soils Using Gyratory Testing Machine*. Final Report. Mississippi Highway Department, Dec. 1991.
2. Y. J. Chou and R. L. Lytton. Accuracy and Consistency of Backcalculated Pavement Layer Moduli. In *Transportation Research Record 1293*, TRB, National Research Council, Washington, D.C., 1991, pp. 72–85.
3. W. Uddin, A. H. Meyer, W. R. Hudson, and K. H. Stokoe II. Project-Level Structural Evaluation Based on Dynamic Deflections. In *Transportation Research Record 1007*, TRB, National Research Council, Washington, D.C., 1985, pp. 37–45.
4. D.-S. Kim and K. H. Stokoe II. Characterization of Resilient Modulus of Compacted Subgrade Soils Using Resonant Column and Torsional Shear Test. Presented at the 71st Annual Meeting of the Transportation Research Board, Washington, D.C., 1992.

AUTHOR'S CLOSURE

The discussant brings out a very significant aspect of backcalculating moduli values from deflection data. Citing the author's results, the discussant makes a valid observation that the backcalculated moduli of unbound materials, including roadbed soils, should be corrected for nonlinear behavior. That the kneading resilient moduli agree with the corrected backcalculated moduli gives credence to the gyratory resilient modulus test, in which the soil sample is subjected to shear stress reversal. What is important in the resilient modulus test would be to program the loading sequence in such a manner that both normal and shear stresses undergo reversal during each loading.

The opinions, findings, and conclusions expressed in this report are those of the author and not necessarily those of the Mississippi State Highway Department or FHWA. This does not constitute a standard, specification, or regulation.

Publication of this paper sponsored by Committee on Soil and Rock Properties.