Verifying Kneading Resilient Modulus of Soils with Backcalculated Values

K. P. GEORGE AND WAHEED UDDIN

The characterization of soils in terms of resilient behavior is gaining support because of its immediate application in the mechanistic analysis of pavements. This report investigates an alternative procedure of resilient testing using the U.S. Army Corps of Engineers gyratory testing machine (GTM). The development of the GTM test procedure, focusing on the simulating conditions of a moving load, is summarized. With consideration to specimen confinement in the mold, a revised equation for kneading resilient modulus (M_{rk}) is presented. The primary objective is to validate the GTM test procedure. For that purpose, six subgrade soils and three subbase materials are investigated using the GTM and the repeated load triaxial test, AASHTO T274-82. For in situ determination of resilient modulus of some of those soils (five samples only), Dynaflect and falling weight deflectometer deflections are obtained on finished pavements as well. Two computer programs—MODULUS and FPEDD1—were used to backcalculate the moduli of all of the layers.

The GTM moduli compare poorly with the triaxial moduli in this nine-soil comparative study, nor was there any correlation between M_{rk} and the backcalculated moduli when the latter was not corrected for nonlinear effects. However, the in situ modulus values (only five sites tested) agree with the GTM moduli with appropriate nonlinear correction, as programmed in FPEDD1. On the basis of the successful comparison with the in situ backcalculated moduli, it is concluded that GTM has the potential for resilient modulus characterization of subgrade soils.

The physical properties of subgrade soils are important parameters for designing, maintaining, and rehabilitating pavements. Traditional test procedures for characterizing subgrade soils are now replaced by resilient testing. For example, in the revised AASHTO guide (*1*), the resilient modulus (M_r) replaced the soil support value used in the previous editions of the guide.

The repeated load triaxial (RLT) test proposed for determining M_r (AASHTO T274-82) is relatively complex. Accordingly, highway agencies have sought alternative test methods. The diametral testing procedure, an alternative used in experiments by the Oregon Department of Transportation (*2*), was found adequate for cohesive soils, but it is not recommended for noncohesive soils. After a careful study of the literature review, this study was initiated 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, would be a viable alternative for resilient modulus testing. The GTM is described in a U.S. Army Corps of Engineers report (*3*).

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

OBJECTIVE AND SCOPE

The overall objective of this study was to verify the application of GTM in resilient testing of soils. To accomplish this, several subgrade soils (four fine-grained and five coarse-grained) were tested in the laboratory using both the conventional repeated load mode (AASHTO T274-82) and the GTM. First, the M_r from RLT is compared with the kneading resilient modulus (M_{rk}) for all the soils. To further substantiate the laboratory moduli, nondestructive testing (NDT) deflections [both Dynaflect and falling weight deflectometer (FWD)] of five pavements (soil samples from those five tested in the laboratory) were obtained for backcalculation. The subgrade moduli were backcalculated using two PC-based computer programs: MODULUS and FPEDD1. The backcalculated moduli were compared with the laboratory values to establish the reasonableness of the M_{rk} values, and, in turn, the feasibility of using GTM to estimate the resilient modulus of subgrade soils.

WHY GYRATORY TESTING MACHINE?

The GTM—a combination kneading compaction, "dynamic consolidation," and shear testing machine—simulates abrasion effects caused by repetitive stress and intergranular movement within the mass of material (subgrade, subbase, 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. Note that head G acts against roller bearing and is free to slip, while base H remains horizontal. A "gyratory 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 in Figure 1 shows gyratory motion or shear strain. The recording, referred to as a gyrograph, is a direct indicator of plasticity of the material being investigated.

By producing a uniform shearing action in the test specimen by a gyratory motion of the test mold, the apparatus is believed to simulate field compaction more closely than impact tamping, which is used in AASHTO and ASTM procedures. In an earlier Waterways Experiment Station study (*4*), good correlation was obtained between the gyratory-compacted densities and the densities of samples obtained from the test sections after traffic had been applied.

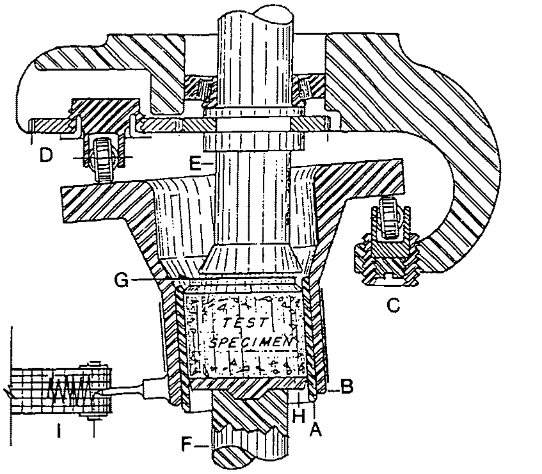

FIGURE 1 Schematic illustration of gyratory testing machine.

The GTM, originally a bituminous-mixture laboratory compactor, is modified in this research program to accommodate repeated load application. The soil specimen, confined in mold A, is subjected to a repeated load stress through the lower ram F and shear stress reversal through gyratory motion. 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 is 0.2 Hz, as is the gyratory displacement. Figure 2 shows the position of the top face of the specimen as the roller rotates through one full cycle (360 degrees). Employing finite element computations, the senior author has shown that the shear stress in the sample undergoes nearly sinusoidal variation (*5*,*6*).

In a recent study, George (*6*) analyzed the stress state, especially the stress reversal in the underlying pavement material, and concluded that repeated load GTM has the potential to simulate the moving load traversing a road. Not only are the vertical stresses cycled, but the shear stresses undergo sinusoidal variation as the GTM specimen is gyrated during resilient testing. Further evidence of similarity was presented by comparing the stress paths of three different loadings: (a) a GTM sample gyrated at 0.1 degree and subjected to a cyclic load, (b) a stress state resulting from a 9-kip single tire load, and (c) a repeated load triaxial sample subjected to a cyclic load as per AASHTO T274. The resemblance of the GTM stress path to the field stress path is construed as further evidence of GTM resilient testing's ability to better simulate the field conditions than the RLT counterpart.

Based on the foregoing discussion, three features favoring the use of GTM for resilient modulus can be cited as follows:

1. The GTM is capable of performing the compaction and resilient testing in one pass without having to extrude the specimen from the mold and subsequently mount the specimen for resilient testing.
2. The GTM avoids specimen extrusion and remounting for further testing averting any possible sample disturbance.
3. The GTM has the added capability of inducing stress reversal in the specimen during resilient testing.

Resilient Modulus Testing Using GTM

Previously the primary use of GTM has been to determine compaction characteristics of road materials and compaction, plasticity, and shear characteristics of bituminous mixtures. Consequently, the repeated load gyratory test procedure envisioned in this study had to be developed and standardized. Because sample compaction is performed in the GTM, a compaction procedure is conveniently combined with the repeated load test. The compaction pressure and the gyration angle are chosen to simulate the stress state of the soil material during field densification and to attain a unit weight representative of the ultimate in-place condition after extensive traffic load application. Based on the results of numerous trials, it is recommended that granular soils be compacted at 345 kPa (50 psi) compaction pressure and 0.5 degree gyration angle, whereas fine-grained soils should be compacted with the same gyration angle but at an elevated pressure of 482 kPa (70 psi).

Because 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 traffic loading. Because of the need to compact the sample at pressures greater than those called for in resilient modulus testing, a 2 hr waiting period (allowing for specimen rebound) is also programmed into the testing procedure. George (*5*) lists a

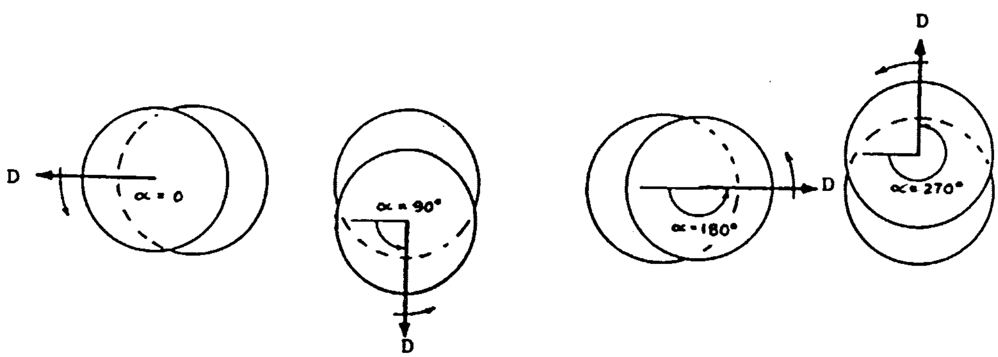

FIGURE 2 A constant rotating displacement (D) applied to top of specimen produces a gyratory motion (for 5 sec) during gyratory shear test.

step-by-step procedure of the test along with the test parameters adopted in this study.

Equation for Kneading Resilient Modulus

After being compacted to the specified density, the specimen in the GTM is subjected to a stress pulse, with the peak value smaller than the compaction pressure. With the tacit assumption of nearly zero wall friction, due to a light greasing of the mold, an equation for the resilient modulus (M_{rk}) is derived as follows (*5*):

$$M_{rk} = \frac{\sigma_r}{\varepsilon_r} \frac{(1 + \nu)(1 - 2\nu)}{(1 - \nu)} \quad (1)$$

where

σ_r = rebound stress in axial direction;
ε_r = recoverable strain; and
ν = Poisson's ratio.

EXPERIMENT DESIGN

Six subgrade soils covering a 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 and FWD 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 presents the index properties and classification symbols of nine soil materials. A range of gradations is represented, as indicated in Table 1.

The experiment design called for three series of testing. The first series comprised three or more specimens from each soil, at optimum moisture and AASHTO T99 (standard Proctor) density, tested in accordance with the AASHTO T274. The second series included specimens from the same soils at optimum moisture and density tested in a repeated gyratory machine. All of the latter specimens were compacted at 0.5 degree gyration angle and tested at gyration angles of 0.1 degree and 0.0 degrees. Five field sites, where the soil samples 2–5 and 10 have been obtained, were subjected to NDT deflection testing using both the Dynaflect and the FWD. The NDT was the third phase (series) of tests programmed in this investigation.

DISCUSSION OF RESULTS

For both the RLT and GTM devices, the deformations measured during repeated loading were corrected to account for machine compliance and seating errors. Failure to do so would result in apparent low resilient moduli. In the RLT tests, the deformation was sensed outside the triaxial cell, and, therefore, the measured deformations included seating errors, which can be significant at times. To correct the measurements for the seating error, the linear variable differential transformer rebound deflection registered at 14 kPa (2 psi) deviatoric stress is subtracted from the respective deformation at higher stress levels. A sample calculation illustrating this correction procedure is provided by George (*5*). However, in GTM a compliance curve was prepared, which is simply a plot of pressure versus deformation ascertained while loading a steel cylinder (relatively stiff compared with that of the soil) between the upper and lower load plungers. A correction was applied to the soil specimen deformation, commensurate with the pressure in the specimen.

Both RLT and GTM resilient tests were conducted on three replicated specimens with three or more observations on each specimen. Outliers for each soils were scrutinized using Chauvenet's criterion (*7*) before combining the results of each soil specimen to arrive at the representative resilient modulus.

TABLE 1 Soil Characteristics

Soil No.	Location Hwy/County	Passing #200 Sieve, %	Atterberg Limits LL	Atterberg Limits PI	Proctor Test Data Maxm. Density kN/m³	Proctor Test Data Optimum Moisture %	Soil Classification AASHTO/Unified	Poisson's Ratio	Lateral Stress Ratio, K_o[a]
2	US98/Forrest & Perry	19	0	NP	19.2	10.4	SP-SM/A-3	0.25	0.33
3	MS7/Yalobusha	26	22	4	18.9	11.9	SM-SC/A-2-4	0.30	0.43
4	US49/Sunflower	70	32	13	18.4	15.1	CL/A-6(7)	0.35	0.54
5	US49/Sunflower	89	40	18	17.3	15.7	CL/A-6(16)	0.35	0.54
6	US61/Coahoma	97	70	39	15.3	23.0	CH/A-7-5(45)	0.40	0.67
7	US78/Benton & Union	51	26	7	19.4	11.5	ML-CL/A-4(1)	0.30	0.43
8[b]	US98/Forrest & Perry	23	0	NP	19.3	10.7	SM/A-2	0.25	0.33
9[b]	MS7/Yalobusha	12	0	NP	17.5	10.8	SP-SM/A-2	0.25	0.33
10[b]	US98/Forrest	10	0	NP	18.8	9.5	SP-SM/A-3	0.25	0.33

a $K_o = \frac{\nu}{1 - \nu}$

b subbase material
1 kN/m³ = 6.37 lbf/ft³

TABLE 2 RLT Resilient Modulus of Nine Soils Compared with Those of Other Researchers

Soil Group (1)	Soil Number/ Classification (2)	Percent Passing #200 (PF) (3)	Atterberg Limits (4) LL	PI	RLT Resilient Modulus, kPa: Repeated Load Triaxial (5)	Carmichael & Stuart (8) (6)	Drumm et al. (9) (7)	Elliot et al. (10) σ_d = 45 kPa (8)
Coarse-Grained	10/A-3	10	0	NP	140,220[a]	155,670		
	9/A-2	12	0	NP	146,570[a]	150,080		
	2/A-3	19	0	NP	120,650[a]	148,570		
	8/A-2	23	0	NP	164,280[a]	146,290		
	3/A-2-4	26	22	4	123,200[a]	142,570		
Fine-Grained	7/A-4(1)	51	26	7	122,020[b]	111,890	27,580	49,360
	4/A-6(7)	70	32	13	92,860[b]	65,420	79,940	41,020
	5/A-6(16)	89	40	18	78,590[b]	28,400	74,800	47,640
	6/A-7-5(45)	97	70	39	114,510[b]	174,490	123,540	66,600

[a]Resilient modulus at bulk stress 275 kPa
[b]Resilient modulus at deviatoric stress 70 kPa and confining pressure 21 kPa
1 kPa = 0.145 psi

The resilient moduli of nine soil samples, which were determined using RLT, are listed in column 5 of Table 2. Because the resilient behavior of a soil sample is influenced by the applied confining pressure and the deviatoric stress, relations were sought between M_r and each of these stress variables. To remain consistent with the AASHTO recommendation, resilient modulus values are plotted with bulk stress ($\theta = \sigma_d + 3\sigma_3$) and cyclic deviatoric stress for the coarse- and fine-grained soils, respectively. As expected, the resilient moduli of coarse-grained soils increase with the bulk stress (see Figure 3). The effect of confining pressure on the resilient modulus showed a substantial modulus increase of as much as 40 percent when the confining pressure was increased from 35 to 70 kPa (5 to 10 psi). In fine-grained soils, resilient modulus is graphed against the cyclic deviatoric stress and the results show that M_r is decreased only slightly with the latter (Figure 4).

For comparison purposes, the moduli of soils tested in this research are predicted using empirical equations of other researchers. Columns 6–8 of Table 2 list resilient moduli calculated using the empirical equations of Carmichael and Stuart (*8*), Drumm et al. (*9*), and Elliot et al. (*10*), respectively. Recognizing that the experimental precision is ±16,536 kPa (±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 the modulus of Soil 7 agrees with that predicted by Carmichael's equation. The Drumm et al. equation is meant to predict only the modulus of fine-grained soils. 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 this study.

Gyratory Resilient Modulus

The gyratory modulus results are discussed in detail by George (*5,6*), with some specific results in Table 3. Columns 3 and

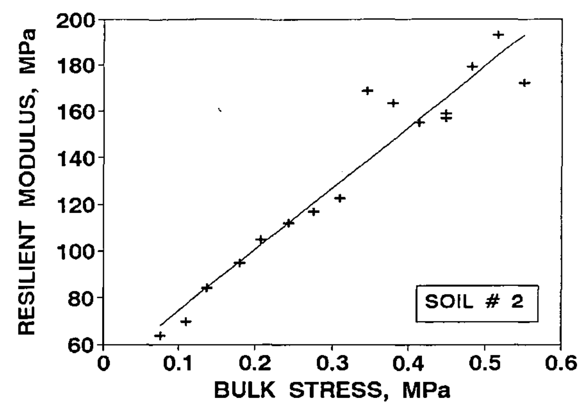

FIGURE 3 Resilient modulus related to bulk stress ($\theta = \sigma_1 + \sigma_2 + \sigma_3$).

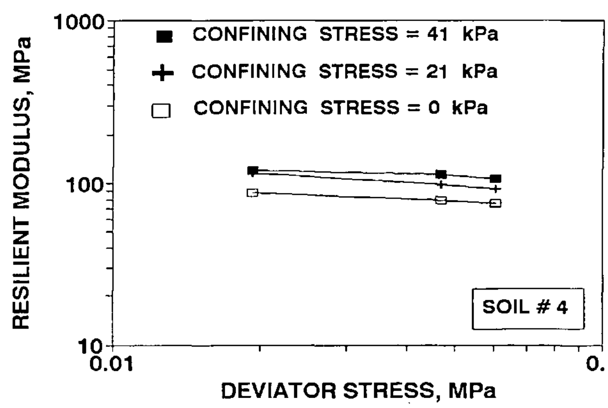

FIGURE 4 Resilient modulus related to deviatoric stress at different levels of confining stress (1 MPa = 145 psi).

TABLE 3 RLT and GTM Resilient Modulus Compared with Backcalculated Modulus

Soil Number	RLT Modulus, M_r, kPa	Kneading Modulus, kPa		Dynaflect Backcalculated Modulus, kPa		FWD Backcalculated Modulus, kPa	
		0.0 degree, M_{rko}	0.1 degree, M_{rk}	FPEDD1 (w/o correction)	FPEDD1 (w/correction)	FPEDD1	MODULUS 4.0
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
10[a]	140,220	211,650	108,790	198,750	121,270	235,020	229,570
2[a]	120,650	202,890	111,340	177,310	108,170	222,680	220,610
3[a]	123,200	161,460	88,520	178,000	78,390	221,300	199,240
4[b]	92,860	95,760	62,800	101,760	69,150	145,600	146,840
5[b]	78,590	86,380	43,640	114,230	76,870	148,840	149,600
6[b]	114,510	92,040	52,600				
7[b]	122,020	99,340	76,590				
8[a]	164,280	205,860	111,750				
9[a]	146,570	194,480	114,510				

[a]coarse-grained soil
[b]fine-grained soil
1 kPa = 0.145 psi

4, list the gyratory resilient moduli at 0.0 and 0.1 degree gyration angles, respectively. The variation of M_{rk} with soil composition (texture), and dry density and stress state agreed with the reported results of the repeated load triaxial device. M_{rk}, however, is influenced little by fluctuations in compaction moisture. Comparing Columns 3 and 4, the authors concluded that the modulus increases under no-kneading condition (designated as M_{rk0}). The fact that the resilient modulus is significantly affected by the angle of gyration (which induces shear strain) suggests that for realistic modulus determination, the test must simulate shear stress reversal, a condition inherent in the field under moving loads. Values for M_{rk}, which are determined at various cyclic stress levels, are normalized with respect to M_{rk} at 70 kPa/140 kPa (10 psi/20 psi) stress level and plotted against the corresponding volumetric stress in Figure 5. The intermediate and minor principal stresses, which are equal in the GTM sample, are estimated using the lateral stress ratio, K_0 ($K_0 = \nu/1 - \nu$). K_0, estimated from Poisson's ratio, is listed in Table 1. The gyratory modulus results for two coarse-grained and two fine-grained are shown in Figures 5(*a*) and 5(*b*), respectively. No significant change with volumetric stress was observed in either soil group. This result was somewhat different from what has been observed with the RLT data, as shown in Figure 3.

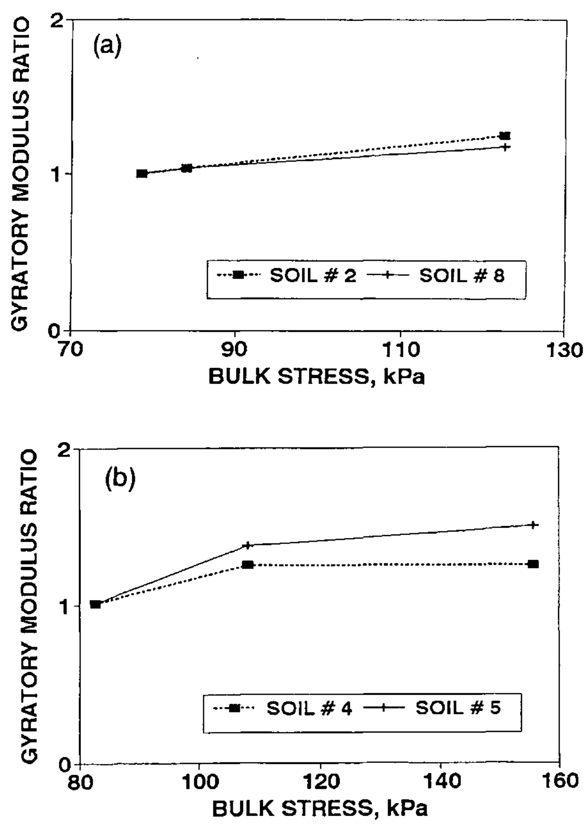

FIGURE 5 Gyratory shear modulus, normalized with respect to M_{rk} at 138/69 kPa plotted against volumetric stress: (*a*) coarse-grained soils and (*b*) fine-grained soils (1 kPa = 0.145 psi).

Backcalculated Moduli from In Situ Tests

To validate the laboratory moduli values, in situ moduli of five subgrade soils were determined from NDT deflection in conjunction with backcalculation procedure. Thirteen deflection tests were conducted in November 1991 on each of the five pavements using the Dynaflect. FWD tests at the same sites were more elaborate because deflection data were ascertained using four different loads: 26,688 kN, 40,032 kN, 53,376 kN, and 71,168 kN (6,000 lbf, 9,000 lbf, 12,000 lbf, and 16,000 lbf). FWD data were obtained in May, June, and October 1992, 6 to 11 months later than the Dynaflect tests. Asphalt surface temperatures were measured during both Dynaflect and FWD tests, but no temperature correction was applied to the results reported in Table 4. The mid-depth temperature of the asphalt surface during the FWD test is reported in Column 1 of Table 4. Moduli of the various layers were backcalculated for each FWD basin using MODULUS 4.0 and FPEDD1 programs, with the results presented in Table 4. The FPEDD1 program readings showed consistent values within the 13 adjacent sites. Note that in Soil (site) 3, the MODULUS program failed to give reasonable values at 6 of the 13 locations. Therefore, only seven results are included in Row 5 of Table 4. Although there is good agreement in the subgrade moduli calculated from both programs, the surface moduli (Column 4, Table 4) and base moduli (Column 5, Table 4) from FPEDD1 are more realistic than those calculated using MODULUS 4.0. Similar results have been reported by George (*5*) where Dynaflect data are used in conjunction with FPEDD1. The relatively small coefficient of variation of the layer moduli, especially with FPEDD1, reflects the robustness of the backcalculation procedure.

Because the force applied by Dynaflect is 4,448 kN (1,000 lbf), substantially smaller than the wheel load of 40,032 kN (9,000 lbf), a correction for the nonlinear constitutive relationship is recommended. FPEDD1 includes the required algorithm to account for the nonlinear behavior. An empirical relationship developed in earthquake engineering studies has been adopted for this purpose (*11*). The strain versus modulus relationship in FPEDD1 for nonlinear correction is described by Uddin et al. (*12*). Corrected and uncorrected backcalculated moduli from Dynaflect deflection basins are reported in Columns 6 and 5 of Table 3, respectively. Nonlinear correction is not required when using FWD. Accordingly, backcalculated modulus values in Columns 7 and 8 of Table 3 are not corrected.

A comparison of the uncorrected Dynaflect backcalculated moduli with those from FWD basins, (Table 3, Columns 5 and 7) reveals that the FWD values are consistently larger than their Dynaflect counterparts. From the point of view of nonlinear considerations, an opposite trend would have been more appropriate, that is, moduli from the heavier FWD (40,030 kN load) would be smaller than the Dynaflect moduli, where the load is only 4,448 kN (1000 lbf). Side-by-side FWD and Dynaflect tests were conducted on one site (Soil 3) in October 1992, and the backcalculated moduli showed the same trend, that FWD moduli (Row 5, Column 6, Table 4) were larger than their Dynaflect counterparts, 221 MPa versus 169 MPa. Similar results have been reported by Zhou et al. (*13*). The effect of the loading mode (impact versus steady-state vibratory) may have had a significant role in the measured deflection. This will be further investigated by dynamic analysis.

FEASIBILITY OF GTM FOR RESILIENT MODULUS TESTING

Now that resilient modulus values of nine soils have been determined by (a) employing repeated load triaxial test,

TABLE 4 Sample Results of Backcalculated Moduli Using FWD Data in Conjunction with Modulus 4.0 and FPEDD1

Soil/Test Temp./ Test Date (1)	Program Used (2)	Statistical Measure (3)	Hot Mix Surface Binder Modulus, MPa (4)	Base Modulus, MPa (5)	Subgrade Modulus, MPa (6)
2/27°C[a]/05-20-92	MODULUS 4.0	Mean/CV[d]	3790/(21%)	827[b]/(18%)	221/(4%)
	FPEDD1 (w/o correction)	Mean/CV[d]	1994/(16%)	1172[b]/(17%)	223/(4%)
10/27°C[a]/05-20-92	MODULUS 4.0	Mean/CV[d]	5854/(27%)	717[b]/(8%)	230/(2%)
	FPEDD1 (w/o correction)	Mean/CV[d]	2581/(26%)	1132[b]/(26%)	235/(3%)
5/24°C[a]/06-24-92	MODULUS 4.0	Mean/CV[d]	3399/(11%)	174[c]/(40%)	147/(8%)
	FPEDD1 (w/o correction)	Mean/CV[d]	2412/(16%)	260[c]/(26%)	146/(9%)
5/24°C[a]/06-24-92	MODULUS 4.0	Mean/CV[d]	3373/(11%)	128[c]/(32%)	150/(8%)
	FPEDD1 (w/o correction)	Mean/CV[d]	2221/(15%)	240[c]/(28%)	149/(8%)
3/20°C[a]/10-7-92	MODULUS 4.0[e]	Mean/CV[d]	3916/48%	2009[b]/72%	216/23%
	FPEDD1 (w/o correction)	Mean/CV[d]	2928/31%	1997[b]/31%	221/33%

[a]Mid-depth temperature of hot mix surface
[b]Lime-fly ash base course
[c]Granular material
[d]Coefficient of Variation
[e]Only seven out of 13 deflection bowls gave reasonable solution
1 MPa = 145 psi

(b) using GTM test at 0.1 and 0.0 degree gyration angles, and (c) using NDT in conjunction with the backcalculation procedure, the kneading resilient moduli, M_{rk} and M_{rk0}, may be compared with other values. The objective is to validate the gyratory modulus and, in turn, the applicability of the gyratory testing procedure for resilient modulus determination. At the outset, it should be remarked that a one-to-one comparison between GTM–M_{rk}/M_{rk0} and RLT–M_r should not be expected, for the reason that the states of stress in the respective specimens are far from similar. Because of its stress-dependency, the resilient modulus should be determined under a stress state, as close to the field loading conditions as possible. To evaluate GTM for resilient modulus testing, emphasis should be placed on comparing the GTM modulus with the in situ modulus, although a comparison with RLT–M_r is certainly desired. Four different comparative discussions will be presented as follows:

1. M_{rk} with M_r values,
2. M_{rk0} with M_r values,
3. M_r with in situ backcalculated modulus values (both Dynaflect and FWD), and
4. M_{rk}/M_{rk0} with backcalculated modulus values.

First, by comparing M_{rk} and M_r values in Columns 4 and 2, respectively, of Table 3, it can be seen that M_{rk} values are consistently lower than the M_r values. Between the two groups, fine-grained soils show pronounced reduction in modulus values in GTM at 0.1 degree gyration. The lower moduli in GTM may be attributed to the nonlinear constitutive relationship. As shown by George (*5*,*6*), fine-grained soils, being highly nonlinear, show a relatively large reduction in M_{rk}, which may be attributed to increased deviatoric stress (resulting from GTM-induced shear), a valid explanation for the pronounced decrease in moduli of fine grained soils.

Second, the 0.0 degree kneading modulus, M_{rk0}, does not show any one trend when compared with the corresponding M_r-values. As noted in Columns 3 and 2 of Table 3, the M_{rk0} of coarse-grained soils is higher than M_r, whereas in the fine-grained soils, they are equal or the M_{rk0} is slightly lower. An explanation of this result is that when coarse soils are tested under confinement (zero lateral strain), they exhibit a tendency to be stiff, whereas confinement plays only a minor role in cohesive soils. The question now arises why M_{rk0} values of fine-grained soils are slightly lower than the corresponding M_r values. It may be that because the clayey soils are plastic, they have "memory" to reflect the large (0.5 degree gyratory angle) shear strains imposed in the specimen during compaction. Note that the 0.0 degree gyratory test invariably followed the 0.1 degree test in the same specimen. The presence of residual deformation was corroborated by observing a nonzero angle in the test gyrograph when the chuck was set to read zero angle.

The third comparison is between the triaxial M_r values with the backcalculated values. In all of the five soils, the M_r values lie somewhere between the uncorrected and corrected in situ values in Columns 5 and 6 of Table 3, respectively. Note that in situ backcalculated values (both Dynaflect and FWD) are larger than the corresponding triaxial moduli. Similar results have been reported by other researchers (*14*) in that the backcalculated moduli are consistently larger than the triaxial counterpart by roughly 50 to 75 percent.

Fourth, the validity of M_{rk} or M_{rk0} was examined with the basic tenet that the backcalculated moduli form the basis for comparison. As indicated earlier, the moduli corrected for strain sensitivity are a better measure than the uncorrected values. As can be verified in Table 3 (Columns 6 and 4), those corrected in situ values of four soils are in good agreement with the GTM kneading modulus (M_{rk}), with deviations of +10, −3, −13, and +9 percentage points in soils 10, 2, 3, and 4, respectively. In Soil 5, a heavy clay, the corrected backcalculated value lies between the 0.0 degree and 0.1 degree kneading moduli. An angle of gyration smaller than 0.1 degree could have resulted in a value comparable to the corrected NDT value. Very limited tests with a 0.05 degree gyration have confirmed this contention. Evidenced by this result is the basic premise of this research: that resilient modulus testing conducted under stress reversal indeed has some merit. Coincidentally, the 0.0 degree kneading moduli of the five soils reasonably agree with the uncorrected in situ moduli from Dynaflect, but far exceed the corrected moduli, which are considered more realistic than the uncorrected values. Therefore, the researcher concludes that gyratory tests at 0.1 degree or smaller have the potential for resilient modulus characterization of subgrade soils.

The gyratory resilient modulus test, with some modifications in the test apparatus, promises to be a viable test for resilient modulus determination. The gyratory angle setting provision needs to be modified, and the cyclic load duration should be adjusted to a fraction of a second. With these modifications, the GTM could be fine-tuned to suit the testing needs in pavement design.

SUMMARY AND CONCLUSIONS

The resilient modulus is a significant parameter in the design and rehabilitation of pavements. This parameter can be highly nonlinear and influenced by stress state, moisture content, soil type, and density. The resilient modulus most be determined under simulated traffic loading conditions. In this investigation, the resilient modulus was formulated from repeated load tests conducted in the laboratory using the triaxial device and the U.S. Army Corps of Engineers GTM and from using backcalculation techniques using Dynaflect and FWD deflection.

With the objective of investigating the feasibility of the GTM for resilient testing of subgrade soils, nine soils—five coarse grained and four fine grained—were tested in the laboratory. Dynaflect and FWD deflections from five of the nine pavement sites were also ascertained after the pavement structure was in place. The adequacy of the GTM procedure is judged by comparing the GTM resilient modulus values with those of the repeated load triaxial test AASHTO T274 and backcalculated moduli from NDT deflection.

The GTM modulus of coarse-grained soils is compared with the triaxial resilient modulus to note that the former values are 8 to 47 percent lower than the latter. The decrease is more pronounced in fine-grained soils. The same general trend (a lower GTM modulus) is observed with the backcalculated modulus with no correction applied for nonlinearity. Because the two independent backcalculation programs (MODULUS 4.0 and FPEDD1) identify nearly equal subgrade moduli, they support the credibility of the back-calculation techniques. FPEDD1, however, results in more realistic surface and base moduli. A comparison of Dynaflect and FWD backcalculated values reveals that the Dynaflect deflection basin can well characterize conventional flexible pavement structures. The in situ moduli adjusted for low stresses (Dynaflect load only 4,448 kN), as derived by FPEDD1 with a correction for nonlinear behavior, agree with the 0.1 degree kneading resilient modulus. Encouraged by this comparison, the authors recommend that the 0.1 degree GTM test be further explored and modified for possible use in the resilient characterization of subgrade soils.

ACKNOWLEDGMENTS

This report includes results of a study entitled Resilient Modulus of Subgrade Soil Using Gyratory Testing Machine, which was sponsored by the Mississippi State Highway Department and FHWA. Joe Sheffield and Alfred Crawley of the Mississippi State Highway Department were instrumental in developing the project scope. The tests were performed by graduate research assistants in the Civil Engineering Department, University of Mississippi.

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

REFERENCES

1. *AASHTO Guide for Design of Pavement Structure*, AASHTO, 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, Oreg., 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. *Gyratory Compaction Method for Determining Density Requirements for Subgrade and Base of Flexible Pavements*. Miscellaneous Paper 4-494. U.S. Army Engineer Waterways Experiment Station, Corps of Engineers, Vicksburg, Miss., 1962.
5. K. P. George. *Resilient Modulus of Subgrade Soil Using Gyratory Testing Machine*. Mississippi State Highway Department, Jackson, Miss., 1991.
6. K. P. George. Resilient Testing of Soils Using Gyratory Testing Machine. In *Transportation Research Record 1369*, TRB, National Research Council, Washington, D.C., 1992.
7. H. W. Coleman and W. G. Steele. *Experimentation and Uncertainty Analysis for Engineers*. John Wiley and Sons, Inc., New York, N.Y., 1989.
8. 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.
9. 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.
10. 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.
11. H. B. Seed and I. M. Idriss. *Soil Moduli and Damping Factors for Dynamic Response Analysis*. Report EERC 70-10. Earthquake Engineering Research Center, University of California, Berkeley, Sept. 1970.
12. 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.
13. H. Zhou, R. G. Hicks, and I. J. Huddleston. Evaluation of the 1986 AASHTO Overlay Design Method. In *Transportation Research Record 1215*, TRB, National Research Council, Washington, D.C., 1989.
14. M. S. Mamlouk, W. N. Houston, S. L. Houston, and J. P. Zaniewski. *Rational Characterization of Pavement Structures Using Deflection Analysis*. Report FHWA-AZ88-254, I, CART. Arizona State University, Tempe, 1988.

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