adequate for comparative design studies but, if a pavement design is required for a highway project, nonlinear characterization of the granular layer is highly desirable. The findings generally confirm those obtained by Dehlen and Monismith (12).

It is not the purpose here to discuss the implications for asphalt-mix design of the results plotted in Figure 11, but we feel that an explanation for the decrease in design thickness with increase in bitumen content is required. Increasing the bitumen content in a mix decreases the stiffness of that mix and increases its fatigue resistance. Reduced mix stiffness means that the tensile strain developed in the bottom of the asphalt will increase, and an increased design thickness may therefore be expected in order to compensate for this. However, for the case illustrated in Figure 11, the improvement in fatigue performance and thus the increased allowable strain is much more significant than is the stiffness loss; hence a reduced layer thickness results. It must be emphasized that reduction in thickness with increased binder content will only be achieved for structures in which fatigue is the critical parameter and will not necessarily be achieved for all these structures.

SUMMARY AND CONCLUSIONS

1. It is essential to include a failure criterion in a system for nonlinear analysis of granular materials.
2. In conventional pavement structures the potentially high modulus of a granular material generally cannot be realized because, when it is highly stressed, the material approaches a failure stress state with a consequent reduction in modulus.
3. Nonlinear characterization of unbound granular layers is required for accurate designs, particularly when thin asphalt surfacings are contemplated.
4. In pavement structures that have asphalt layers of about 300 mm, the required thickness of the asphalt layer is not greatly affected by the thickness of the granular layer.

ACKNOWLEDGMENT

The work on which this paper is based was conducted at the University of Nottingham during a period when one of us (A.F. Stock) was employed as a senior research assistant. The project was undertaken as part of a contract with the Asphalt and Coated Macadam Association. We are grateful for the advice of P. S. Pell and the provision of facilities by R. C. Coates. The assistance of the staff of the Cripps Computing Centre is also gratefully acknowledged.

REFERENCES

1. S. F. Brown, P. S. Pell, and A. F. Stock. Application of Simplified Fundamental Design Procedures for Flexible Pavements. Proc., 4th International Conference on the Structural Design of Asphalt Pavements, Univ. of Michigan, Ann Arbor, Vol. 1, 1977, pp. 327-341.
2. R. G. Hicks. Factors Influencing the Resilient Response of Granular Materials. Univ. of California, Ph.D. thesis, 1970.
3. J. R. Boyce, S. F. Brown, and P. S. Pell. Resilient Behaviour of a Granular Material Under Repeated Loading. Proc., Australian Road Research Board, Vol. 8, 1976, pp. 8-9.
4. E. Smith. Prediction of Equivalent Granular Base Moduli Incorporating Stress Dependent Behavior in Flexible Pavements. Univ. of Maryland, College Park, M.S. thesis, 1977.
5. C. L. Monismith, H. B. Seed, F. G. Mitry, and C. K. Chan. Predictions of Pavement Deflections from Laboratory Repeated Load Tests. Proc., 2d International Conference on the Structural Design of Asphalt Pavements, Univ. of Michigan, Ann Arbor, 1967, pp. 109-140.
6. E. W. Brooker and H. O. Ireland. Earth Pressures at Rest Related to Stress History. Canadian Geotechnical Journal, Vol. 2, No. 1, Feb. 1965, pp. 1-15.
7. T. W. Lambe and R. V. Whitman. Soil Mechanics. Wiley, New York, 1969.
8. J. R. Boyce. Behaviour of a Granular Material Under Repeated Loading. Univ. of Nottingham, Ph.D. thesis, 1976.
9. J. H. Maree. Discussion of paper, South African Mechaniste Pavement Design Procedure, by R. N. Walker, W. D. O. Paterson, C. R. Freeme, and C. P. Marais. Proc., 4th International Conference on the Structural Design of Asphalt Pavements, Univ. of Michigan, Ann Arbor, 1977.
10. M. G. F. Peutz, H. P. M. Van Kempen, and A. Jones. Layered Systems Under Normal Surface Loads. HRB, Highway Research Record 228, 1968, pp. 34-45.
11. D. Leech and W. D. Powell. Levels of Compaction of Dense Coated Macadam Achieved During Pavement Construction. U. K. Transport and Road Research Laboratory, Crowthorne, Berkshire, England, TRRL Rept. 619, 1974.
12. G. L. Dehlen and C. L. Monismith. Effect of Nonlinear Material Response on the Behavior of Pavements Under Traffic. HRB, Highway Research Record 310, 1970, pp. 1-16.

Publication of this paper sponsored by Committee on Flexible Pavement Design.

Analysis of In Situ Granular-Layer Modulus from Dynamic Road-Rater Deflections

PAUL A. D'AMATO AND MATTHEW W. WITCZAK

The major objective of this study was to investigate the ability of elastic-layer theory coupled with nonlinear dynamic modulus tests to predict pavement deflections in a way comparable to dynamic road-rater deflection measurements on three highway sections in Maryland. It was found that theoretically predicted deflections were two to four times the measured road-rater deflections for all three pavement sections studied at all four road-rater sensor locations and for all times throughout the year in which road-rater deflections were made. To obtain equality between predicted and measured deflections, the granular-layer modulus was adjusted by using a K_1-factor. A linear log-log relationship was evident when the K_1-adjustment factor was plotted versus measured surface deflections. It was concluded that the current laboratory method of modulus characterization underestimates the modulus of a granu-

lar material and that an increase in surface deflection results in a decrease of the granular-layer modulus. An investigation was undertaken to determine whether the effect of shear strain was responsible for the decrease in granular-layer modulus. It was found that the granular-layer modulus decreases with increasing shear strain and that shear strain is proportional to surface deflection. The former finding substantiates other research dealing with the dynamic response of granular soils. The study of the K_1-factor and modulus adjustment also led to the development of equations that relate measured surface deflection to in situ granular-layer modulus. For two of the pavements analyzed, predicted granular-layer modulus showed very good agreement with the modulus required for equal computed and measured deflections.

The use of nondestructive deflection measurements has long been recognized as a valuable indicator of a pavement's structural characteristics. Early test-road studies that investigated flexible pavements include the WASHO Road Test (1) in 1952 and the AASHO Road Test (2) begun in 1958. Both these road tests developed correlations of Benkelman-beam deflections with pavement performance.

In recent years, research efforts have also been directed toward the development of deflection-based procedures that could be used to determine in situ moduli of pavement component layers. Determination of layer moduli allows critical stresses and strains to be evaluated by multilayer theoretical approaches. From these critical parameters, estimates by various techniques of remaining pavement life can be used for overlay and rehabilitation purposes.

STUDY OBJECTIVE

This study, part of an overall study of remaining life of flexible pavements, examined three Maryland highway sections in detail (3). Samples of each of the component layer materials were obtained for all of these flexible pavements, and appropriate routine and dynamic laboratory modulus tests were conducted (4,5). In addition, surface-deflection basin measurements on the same pavement sections were determined by means of a road-rater loading device by the Maryland State Highway Administration (MSHA).

The first objective of this study was to assess how well theoretically predicted deflections that used laboratory-determined dynamic moduli along with multilayer theory compared with observed measured deflections obtained from road-rater measurements. Inherent to the multilayer analysis, nonlinear behavior of both granular material (base, subbase) and fine-grained subgrade soils was accounted for. The results of this comparison showed that the computed deflections for the pavement model were not in agreement with observed behavior.

The second consideration hence involved adjusting the pavement model so that the pavement structure, elastic theory, and road-rater deflection measurements agreed. The modulus of the granular base layer was selected for this adjustment because the results of previous research indicated that this was possible (6-8). Relationships for the determination of in situ granular-layer modulus were developed by using the adjusted granular-layer modulus and corresponding road-rater deflections.

Table 1. Input for pavement routes in elastic-layer study.

Pavement Section	Layer	Thickness (cm)	Modulus Relationship (M_R)
US-1	Asphalt concrete		
	Surface	3.1	
	Binder	4.8	
	Base	5.3	
	Granular base		$4\,886\theta^{0.239}$
	1	13.5	
	2	13.5	
	3	13.7	
	Subbase		$2\,632\theta^{0.426}$
	1	31.8	
	2	31.8	
	Subgrade		$57\,962\sigma_d^{-0.696}$
MD-97, section 1	Asphalt concrete		
	Surface	3.8	
	Base	7.9	
	Granular base		$8\,787\theta^{0.365}$
	1	12.4	
	2	12.7	
	3	12.7	
	Subgrade		
	A		$16\,333\sigma_d^{-0.345}$
	B		$13\,035\sigma_d^{-0.180}$
MD-97, section 2	Asphalt concrete		
	Surface	3.8	
	Base	7.9	
	Granular base		$8\,787\theta^{0.365}$
	1	12.4	
	2	12.7	
	3	12.7	
	Subgrade B		$13\,035\sigma_d^{-0.180}$
I-695	Asphalt concrete		
	Surface	6.6	
	Base	19.8	
	Subbase		
	1	9.9	$3\,378\theta^{0.520}$
	2	12.7	$3\,683\theta^{0.517}$
	3A-E	24.5[a]	$4\,856\theta^{0.487}$
	Subgrade		$25\,925\sigma_d^{-0.309}$

Notes: 1 cm = 0.39 in.
Regression equations developed in U.S. customary units (1 lbf/in² = 0.145 kPa).
[a]Five layers, each 24.5 cm thick.

METHOD OF ANALYSIS

The study was conducted on pavement sections of MD-97, I-695, and US-1. For each of these pavement sections, layer thicknesses, material type, and modulus characterizations were ascertained in the remaining-damage portion of the research (4). Table 1 summarizes the required pavement input necessary for the theoretical study of elastic layers. The asphalt modulus relationships were obtained from Shell nomographs and are functions of pavement temperature and load frequency. The temperature within each asphalt layer for each road-rater test date was evaluated from temperature measurements of the pavement surface and relationships developed by Southgate (9). In addition, all unbound granular and subbase and the subgrade soils for all pavements were characterized by the nonlinear (stress-dependent) resilient modulus tests.

Figure 1 illustrates the actual road-rater loading and measuring system and the equivalent system used in the theoretical analysis. The equivalent systems were necessary because the elastic-layer program used is based on circular loads rather than on the actual rectangular foot pads on the road rater. The road rater applies a static preload of 6.0 kN (1350 lbf) and has a variable peak-to-peak dynamic load capability (formulated in U.S. customary units) defined by:

$$F_{p\text{-}p} = 32.704 f^2 D \quad (1)$$

where

$F_{p\text{-}p}$ = peak-to-peak dynamic force (lbf),
f = load frequency (Hz), and
D = peak-to-peak dynamic displacement or amplitude (in).

The load and contact pressure used in the theoretical study for the equivalent-circular-area loading condition (Figure 1) for the various combinations of f and D used by MSHA to obtain measured road-rater deflections are shown in Table 2.

The theoretical elastic-layer model used in the analysis was the Chevron NLAYER program. Because this program is capable only of solving for states of stress, strain, and deflection due to a single wheel load, superposition principles for total deflection at each sensor location were used.

Road-rater deflections were measured on six test dates for each pavement section under investigation. These measurements were made in 1975 and 1976 and encompassed all seasons. For each test date, deflection measurements

Figure 1. Actual and equivalent road-rater loading geometry.

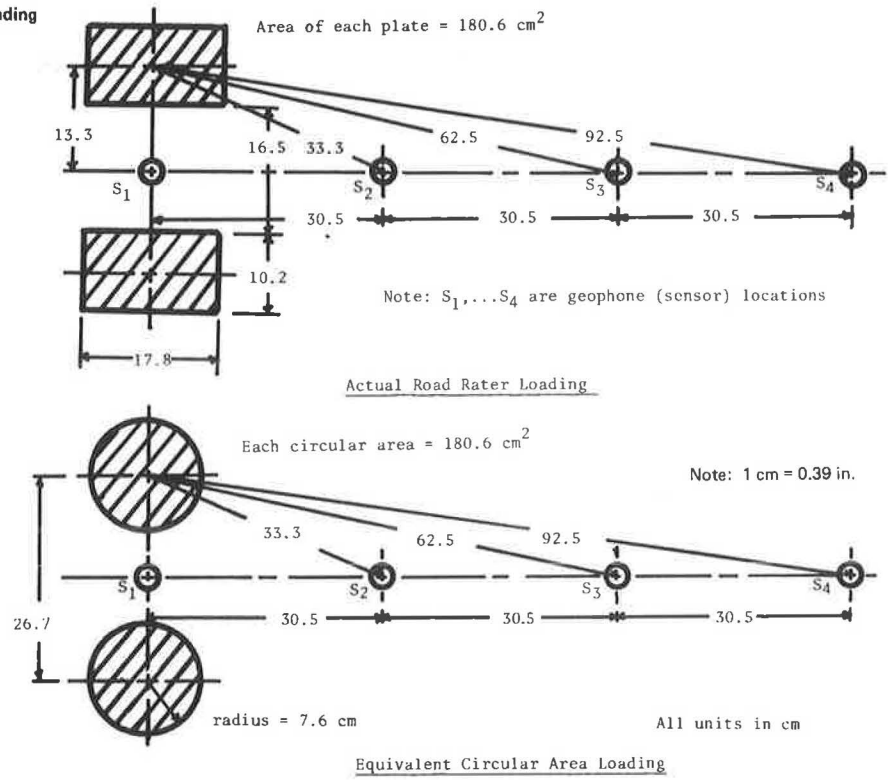


Table 2. Road-rater dynamic-load summary.

Frequency (Hz)	D (mm)	F_{p-p} (kN)	F_{p-p} Each Circular Load (kN)	Contact Pressure (kPa)
16	1.47	2.16	1.08	59.8
25	1.47	5.27	2.64	146.1
25	0.96	3.45	1.73	95.7
25	1.98	7.09	3.55	196.4

Note: 1 mm = 0.04 in; 1 kN = 224.8 lbf; 1 kPa = 0.145 psi.

were recorded at 161-m (0.1-mile) intervals. The length of each pavement test section ranged from 427 m (1400 ft) to 610 m (2000 ft).

SUBGRADE-LAYER STUDY

Before the main deflection comparison study was undertaken, several preliminary studies were conducted to establish the sensitivity of these items to the overall predictive deflection analysis. One such consideration was an analysis to determine the minimum number of subgrade layers necessary to model each pavement section. This was necessary because the moduli of the subgrades were characterized by nonlinear models. Thus, as the stress induced by the road rater decreases with depth into the subgrade, the modulus constantly changes. As a result, a study was undertaken to establish how many sublayers within the subgrade, if any, were necessary before an equilibrium deflection was reached.

The results of this analysis are summarized as the percentage of change in predicted deflection between three-layer and one-layer subgrade systems.

Highway Route	Percentage Change for Sensor Locations S1	S2	S3	S4	Avg
US-1	0.3	0.5	0.8	3.3	1.2
MD-97					
Section 1	1.8	2.3	3.6	5.4	3.3
Section 2	3.0	3.8	6.7	9.8	5.8
I-695	7.6	5.5	4.6	6.8	6.1
Avg	3.2	3.0	3.9	6.3	4.1

Three 30.5-cm (12-in) sublayers were used within the subgrade in the three-layer system. As can be seen, the percentage of change due to subdividing the subgrade into three layers is quite small, especially at sensor locations 1, 2, and 3. For all combinations considered, the average percentage of change is less than 5 percent. As a result of this study, it was concluded that a one-layer subgrade system was sufficiently accurate to predict deflections with the road-rater-induced loads (stresses) and that it was not necessary to subdivide the subgrade layer for the deflection analysis.

STATIC PRELOAD STUDY

Research by the U.S. Army Engineer Waterways Experiment Station (10) that used large road vibrators, 22.25-89.0 kN (5 000-20 000 lbf), has shown that dynamic stiffness of a pavement (ratio of dynamic load to deflection) is dependent on the magnitude of the applied dynamic and static loads. As the static load of the vibrator is increased, the measured deflections due to the dynamic load have been found to decrease. Whether or not the computed surface deflections would decrease significantly on the inclusion of the 6.0-kN (1350-lbf) static load of the road rater was unknown.

To determine whether θ (bulk stress) due to static preload should be included in the deflection study, an analysis was conducted. Pavement deflections were computed for each pavement section after solving for stress-dependent layer moduli (by trial-and-error iteration) with and without the static preload.

Figure 2. Comparison of predicted pavement deflections and mean measured road-rater deflections by sensor location.

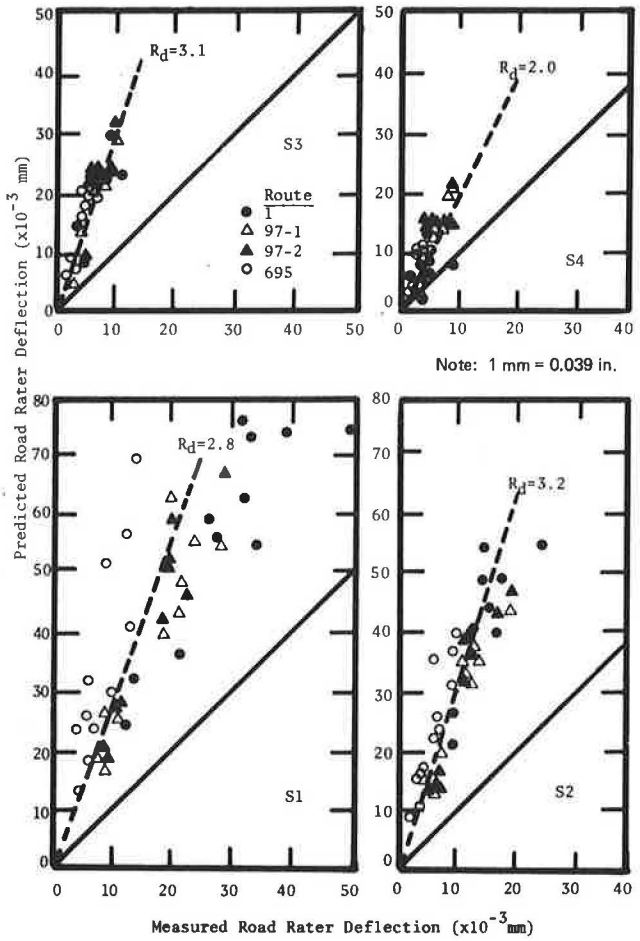
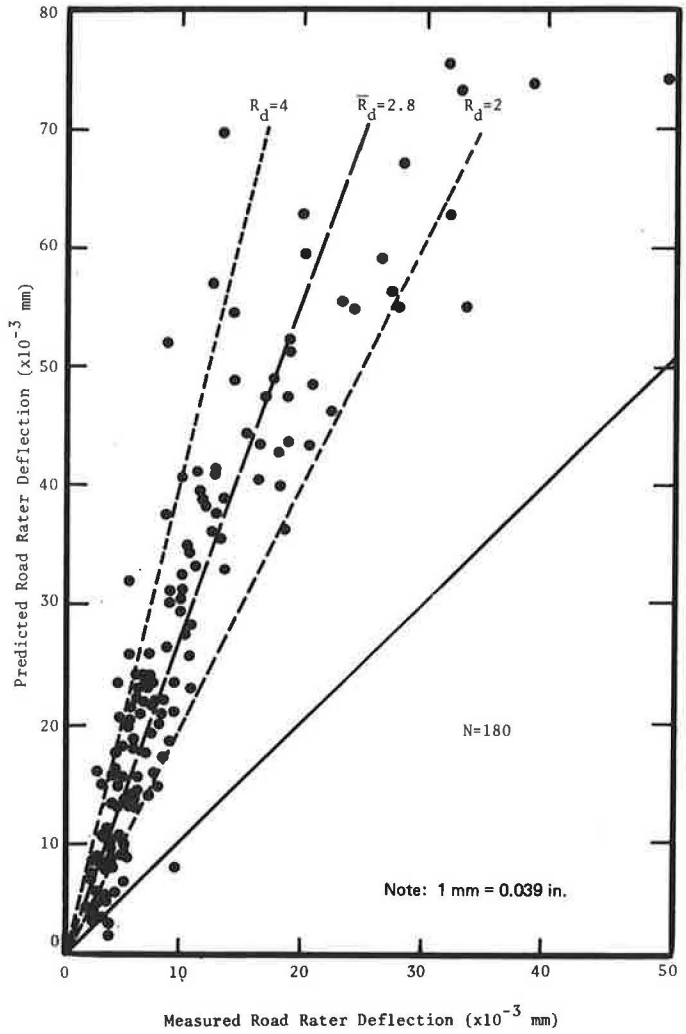


Figure 3. Comparison of predicted to measured deflections.



A summary of these results is shown below:

Highway Route	Percentage Change for Sensor Locations S1	S2	S3	S4	Avg
US-1	3.2	3.5	3.8	3.7	3.6
MD-97					
Section 1	5.0	5.3	3.3	0.0	3.4
Section 2	5.1	4.8	4.3	4.5	4.7
I-695	5.1	5.8	6.0	3.2	5.0
Avg	4.6	4.9	4.4	2.9	4.2

As shown, the percentage of change in deflections with and without the static preload bulk stresses is quite insignificant and again averages less than 5 percent. Accordingly, it was concluded that the influence on predicted deflections of incorporating the additional bulk stress increment due to the static preload of the road rater is negligible and for all practical purposes can be ignored.

DEFLECTION COMPARISON

In the theoretical deflection study, a total of 180 separate deflection predictions were made for each measured (road-rater) combination of pavement section, test date, and sensor location (3,5). In this analysis, the mean measured deflections (within a test section) were used together with the mean values of pavement layer thicknesses and mean layer modulus properties for the theoretical study. Hence all comparisons shown are between average measured deflections and average predicted deflections.

Figure 2 summarizes the comparisons between predicted (δ_p) and measured (δ_m) deflections by sensor locations. Figure 3 is a summary of the 180 data points investigated. The R_d value shown on these diagrams is the deflection ratio of predicted to measured deflections: $R_d = \delta_p/\delta_m$.

Based on this study, it is quite obvious that the deflections predicted by multilayer elastic theory (within a nonlinear iterative approach to layer modulus evaluation) are generally two to four times as large as those measured by the road rater. In general, it also appears that the deflection ratios for the thick asphalt pavement (I-695) were slightly higher than those for US-1 and MD-97, which are composed of more-conventional flexible-pavement [thin asphalt concrete (AC) and granular-base] structures.

K_1-INVESTIGATIVE STUDY

A study was then conducted to see whether the difference between predicted and measured deflections could be explained in a rational and logical manner. Assuming that the model selection of multilayer elastic theory is not the salient reason for the discrepancy, it is apparent that for the predicted (theoretical) deflections to be greater than the measured deflections the elastic modulus of a layer (or layers) characterized by laboratory tests and used as input into the theoretical model must be less than the in situ or apparent field-modulus response.

In a study conducted by Jones and Witczak (8) on San Diego test-road sections, computed pavement deflections

determined by means of laboratory-modulus relationships and multilayer theory were also found to be larger than measured surface deflections. In this study, two important conclusions were reached:

1. For test sections made of asphalt-treated base materials, the mean deflection-derived subgrade modulus of 13 728 kPa (20 000 psi) agreed well with the mean value of modulus predicted from the regression models developed from direct laboratory testing of field samples.
2. In the derivation of the subgrade modulus from deflection measurements, for the granular-base test sections, the comparison between the derived moduli and those determined from direct laboratory testing on field cores was very poor. For these types of bases, the stress-dependent function for the modulus of the base, determined from laboratory tests, tended to underpredict the in situ response of the granular material.

In essence, the results of this study suggest that it is the manner in which the unbound granular materials are characterized rather than the subgrade that may be responsible for the difference between predicted and measured deflections. Because of the similarities in relative results between the two studies, it was decided to investigate the granular material and its corresponding resilient-modulus characterization test (in the laboratory) to help explain the deflection deviations observed in the road-rater study.

In dynamic laboratory testing of unbound granular materials, the nonlinear constitutive equation for resilient modulus is given by

$$M_R = k_1 \theta^{k_2} \quad (2)$$

where k_1 and k_2 are laboratory-derived regression constants reflecting material type and physical properties and θ represents the first stress invariant (bulk stress), defined by the sum of the three normal stresses at a point (triaxial element).

Recalling that $\delta_p > \delta_m$, the above modulus expression can be adjusted by a factor, termed the K_1-factor, so that a new modular expression of the form shown below is introduced:

$$M_R = K_1 k_1 \theta^{k_2} \quad (3)$$

The unique K_1-factor required for $\delta_p = \delta_m$ ($R_d = 1.0$) was termed the K_1-factor at a 1.0 deflection ratio or $K_1(R_d = 1.0)$. The technique used to find $K_1(R_d = 1.0)$ for each measured deflection value (δ_m) was to repeat the theoretical study by using assumed values of K_1 and iterative stress solutions until the condition of $\delta_p = \delta_m$ was observed for each measured deflection value. For deflection values measured by sensors 2, 3, and 4, the deflection ratio (R_d) remained greater than unity even when a K_1-factor of 100 was tried. For these measured deflections, the study was terminated and $K_1(R_d = 1.0)$ remained undefined. Considering that $K_1(R_d = 1.0)$ was found for all sensor-1 deflections and that the Jones-Witczak study used deflections measured at the loading point (Benkelman beam), it was decided to consider only $K_1(R_d = 1.0)$ values derived from sensor-1 deflections for the remainder of the study.

In the Jones-Witczak study on the San Diego test sections, a linear relationship between log $K_1(R_d = 1.0)$ and log δ_m (measured deflection) was obtained from eight different flexible-pavement cross sections. In order to see whether the same analytical model was valid for this study, the relationship of the log of $K_1(R_d = 1.0)$ versus the log of δ_m was developed. Figure 4 shows the results of this study for the sensor-1 location. The correlation coefficients $r^2 = 0.26$ are obviously indicative of a poor fit when all three routes are analyzed together. Two separate relationships are shown on the plot. It should be recognized that the equation for the f = 25-Hz data is indicative of a road-rater load of 5.3 kN (1185 lbf), whereas the f = 16-Hz data are for a 2.16-kN (485-lbf) load. As such, although the correlations are poor, the data strongly suggest separation by magnitude of applied dynamic load. A further study of the data in Figure 4 showed that the I-695 data were significantly divergent from the US-1 and MD-97 relationship.

Figure 5 is the relationship developed for the same data shown in Figure 4 with the exception that the I-695 data (full-depth AC section) have been deleted from the analysis. The tremendous increase in the r^2-values from 0.26 to 0.87 and 0.90 signifies that a linear relationship between log $k_1(R_d = 1.0)$ and log δ_m is indeed valid from the flexible (granular-base) sections of US-1 and MD-97 identical to that found from the San Diego road test analysis. Because I-695 is a relatively thick asphalt-stabilized pavement, it appears that the results are not applicable to this pavement type based on the limited data available. However, at this time it cannot be unequivocally stated that a difference occurs between conventional flexible and thick stabilized sections. It must be noted that, because of the relatively thick asphalt section, a small error in the elastic modulus of the asphalt layer would have a significant effect on the theoretically predicted deflection initially computed.

Because the relationships developed appeared to be valid for pavement cross sections similar to the San Diego sections, this prompted a study of how the results of Figure 5 compared with the relationship developed from the Jones-Witczak San Diego study. Figure 6 illustrates these results. It is quite obvious that the results between studies are consistent, and this suggests that there is a family of log $K_1(R_d = 1.0)$-log δ_m lines that are functions of the applied load. By using Figure 6, Figure 7 was developed, which is assumed to be valid for flexible pavements (conventional granular base) and maximum deflection condition and warrants considerably more research because it may result in an extremely important development that would link theoretical to measured pavement deflections.

If Figure 7 was indeed found to be valid for all deflection combinations measured by load (P, δ_m), the required K_1-adjustment factor (for laboratory to field conditions) could be determined simply by using the measured road-rater deflection for a given road-rater load (frequency and amplitude).

The final interesting trend that was found from the K_1-investigation is shown in Figure 8. In this plot, log $K_1(R_d = 1.0)$ is seen to have an excellent correlation with the mean granular base modulus required to achieve a deflection ratio of $R_d = 1.0$. This relationship appears to be valid for all three routes studied (US-1, MD-97, and I-695) at both frequencies of the road rater as well as for the eight sections of the San Diego test road analyzed by Jones (8).

SHEAR STRAIN INVESTIGATION

Recent studies directed toward developing a better understanding of soil response under dynamic loading conditions have indicated that the shear modulus (G) of a granular soil is dependent on the level of shear strain. Idealized relationships have been developed that show that the shear modulus of a granular soil decreases with increasing shear strain (6), as shown in Figure 9. Of particular note in Figure 9 is the K_2-factor in the equation

$$G = 1000 K_2 (\sigma_m')^{1/2} \quad (4)$$

A similar equation would result if the elastic modulus (E) were substituted for the shear modulus. If the substitution is made, it becomes evident that the K_2-factor in the above equation is very similar in concept to the K_1-factor used in this study. Because of this important tie to dynamic response, it was decided to investigate the effect of shear strain on the $K_1(R_d = 1.0)$ values developed for US-1, MD-97, I-695, and the San Diego test-road sections.

Figure 4. K_1-factor at 1.0 deflection ratio versus mean surface deflection for all routes, sensor 1.

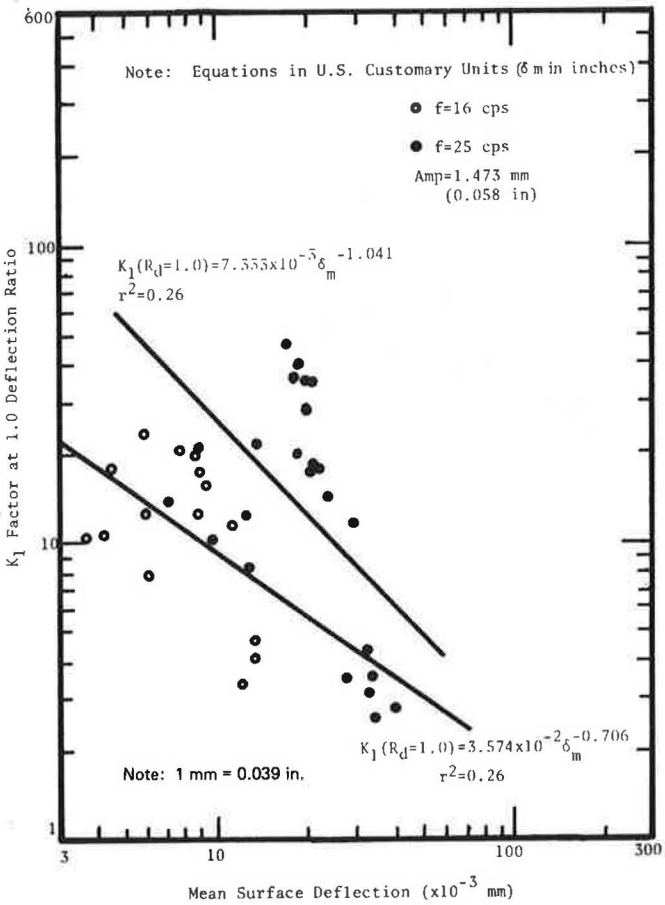


Figure 5. K_1-factor at 1.0 deflection ratio versus mean surface deflection for US-1 and MD-97, sensor 1.

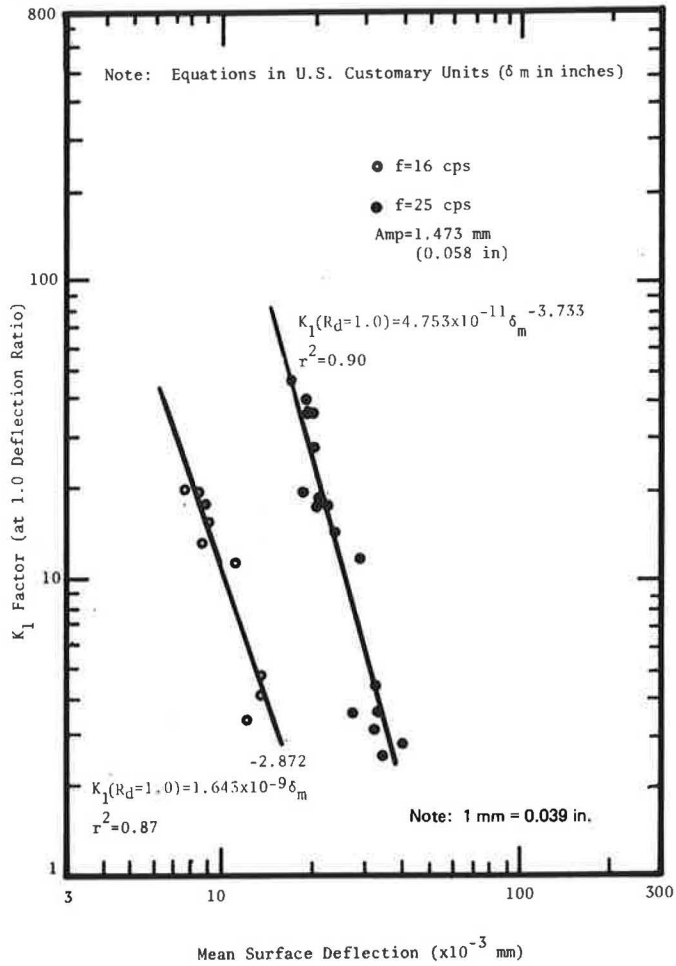
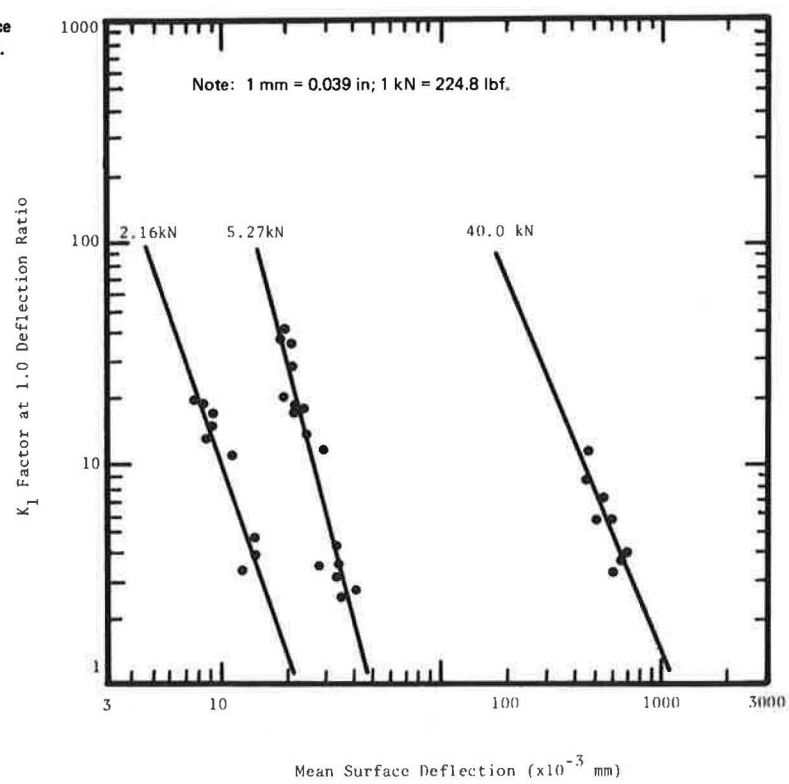


Figure 6. K_1-factor at 1.0 deflection ratio versus mean surface deflection for US-1, MD-97, and San Diego test-road sections.



The theoretically predicted maximum shear strain for each adjusted pavement model ($\delta_p = \delta_m$) was determined in the following manner:

1. The modulus values of all pavement layers required for $R_d = 1.0$ were input into the NLAYER program. The granular layer was modeled as a single layer with an average modulus.
2. Strains computed by the NLAYER program and Mohr circle-of-strain relationships were used to evaluate

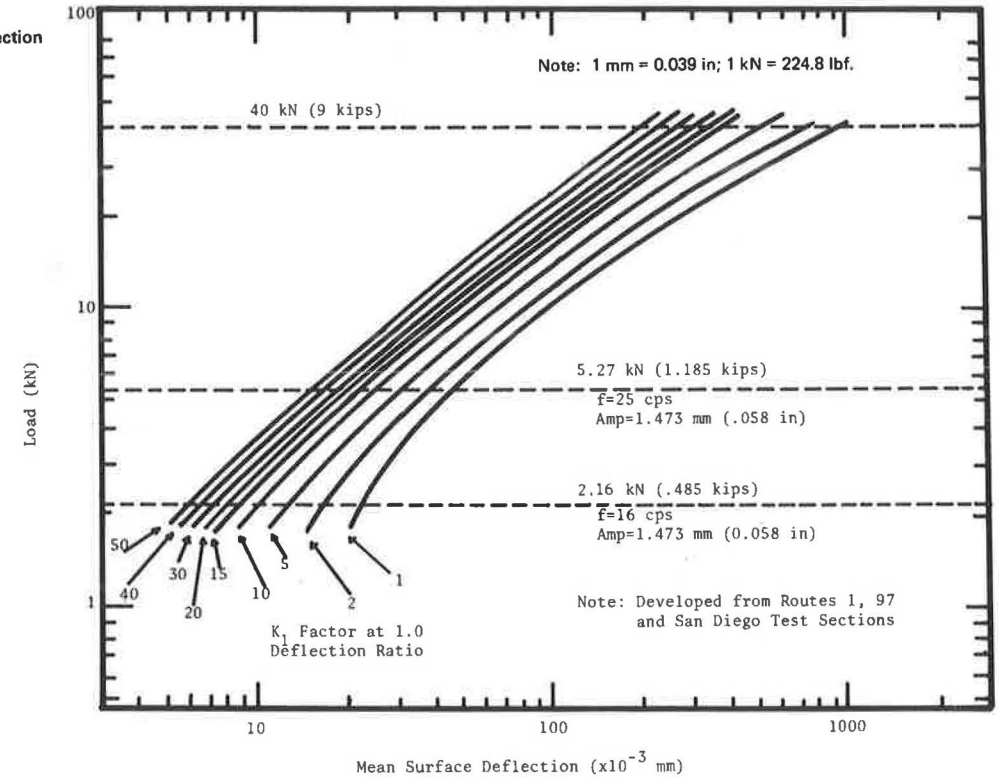


Figure 7. Load versus mean surface deflection for K_1-factors at 1.0 deflection ratio.

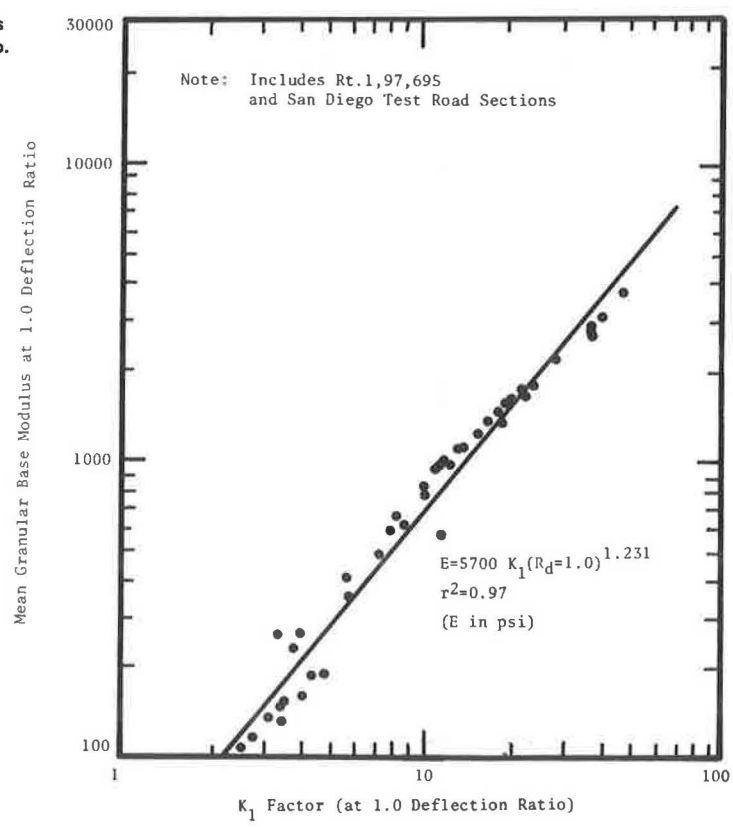


Figure 8. Mean granular base modulus versus K_1-factor at 1.0 deflection ratio.

the maximum shear strain at the middle of the granular layer.

The results of this study were used to develop the relationships shown in Figure 10. For each pavement loading, it can be seen that the $K_1(R_d = 1.0)$ value decreases with increasing shear strain. If we recall that $M_r = K_1 k_1 \theta^{k_2}$, an increase in shear strain [and corresponding decrease in $K_1(R_d = 1.0)$] would result in a decrease in the granular-base elastic modulus. This finding was considered to conceptually support previous findings that demonstrate a decrease in granular-soil shear modulus that results from increasing shear strain (Figure 9).

By comparing relationships shown in Figures 6 and 10, it is evident that the $K_1(R_d = 1.0)$ value decreases with respect to both increasing mean surface deflection and maximum shear strain. This would imply that maximum shear strain is proportional to mean surface deflection. A plot of maximum shear strain versus mean surface deflection is shown in Figure 11. A linear log-log relationship with a squared correlation coefficient of 0.97 results if the data points for I-695 are excluded from the analysis. For I-695, smaller shear strains occur at a particular surface deflection because of the relatively thick asphalt layer.

Figure 9. Shear moduli of sands at different relative densities.

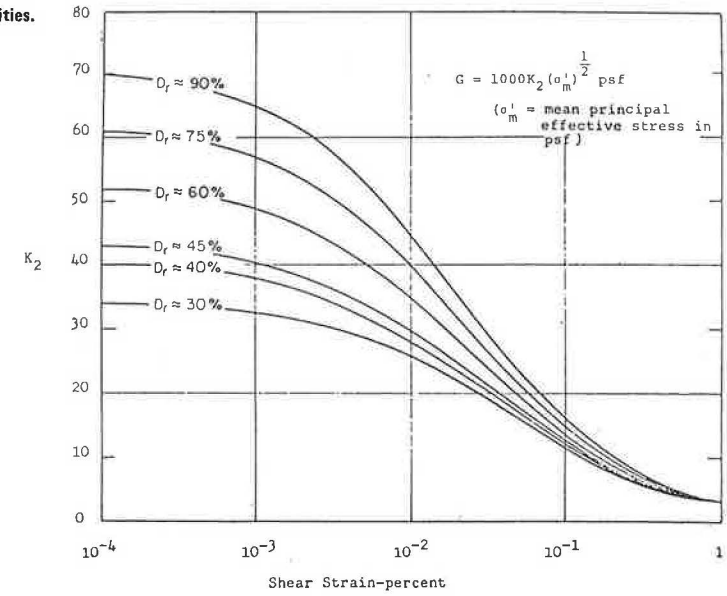
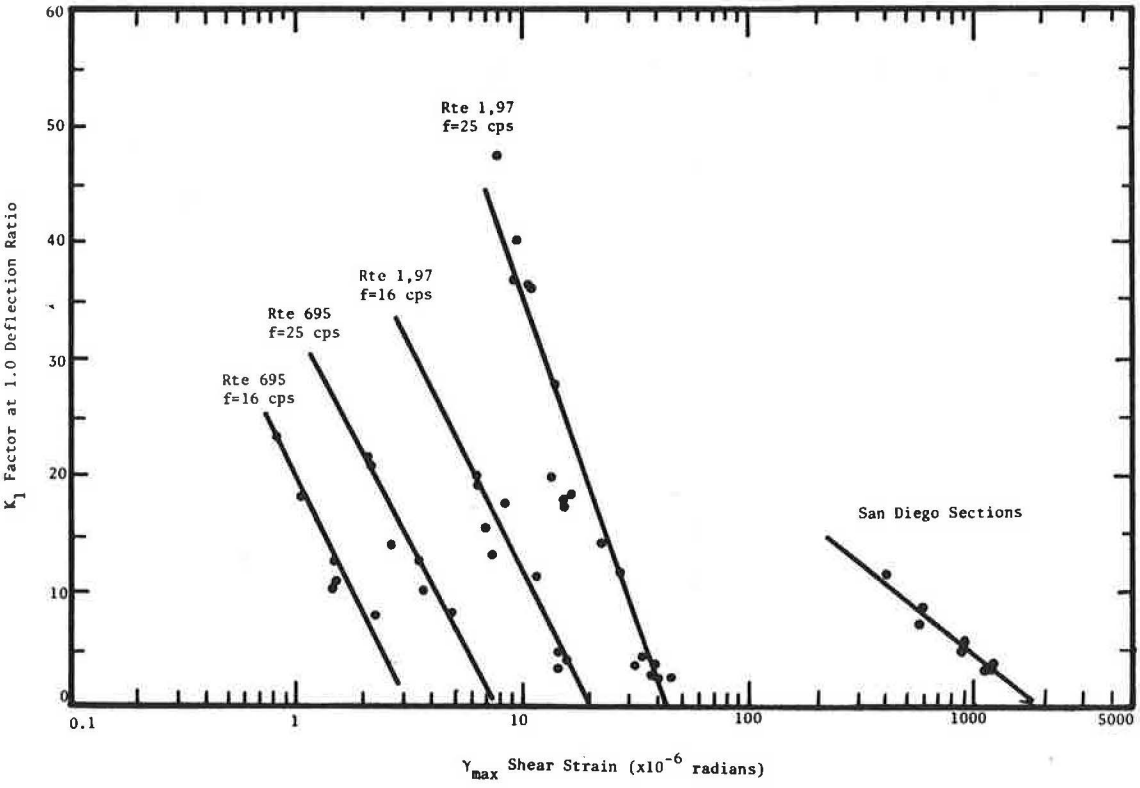


Figure 10. K_1-factor at 1.0 deflection ratio versus maximum computed shear strain.



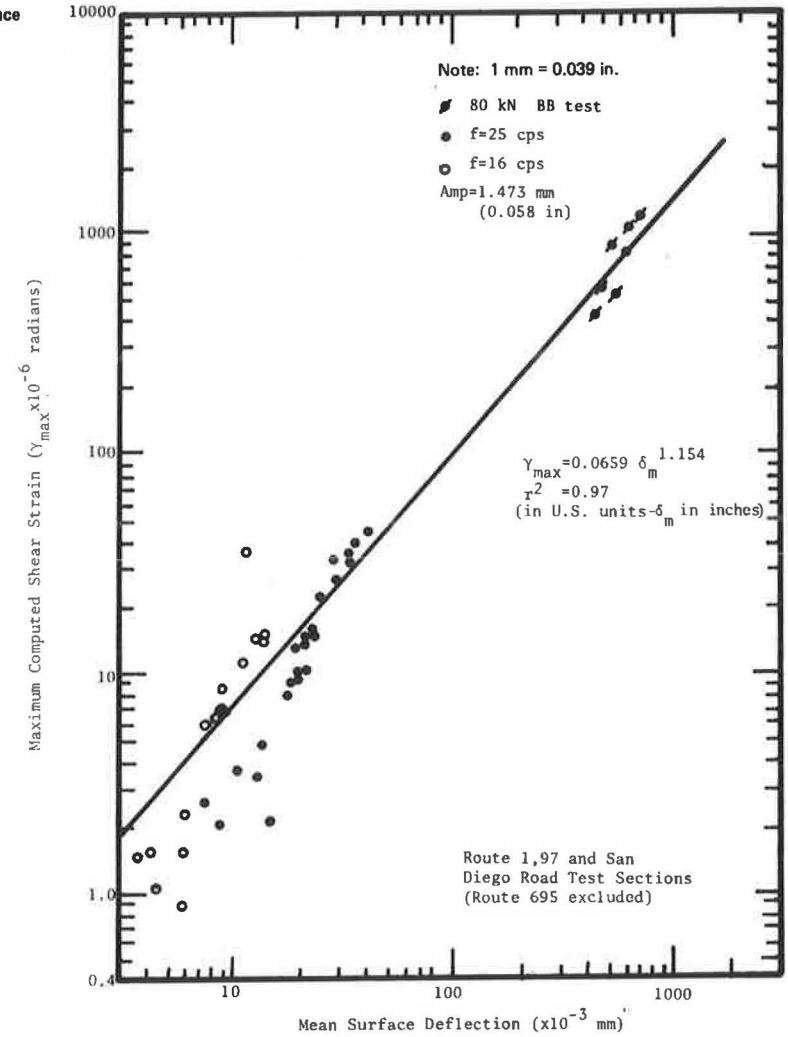


Figure 11. Maximum computed shear strain versus mean surface deflection.

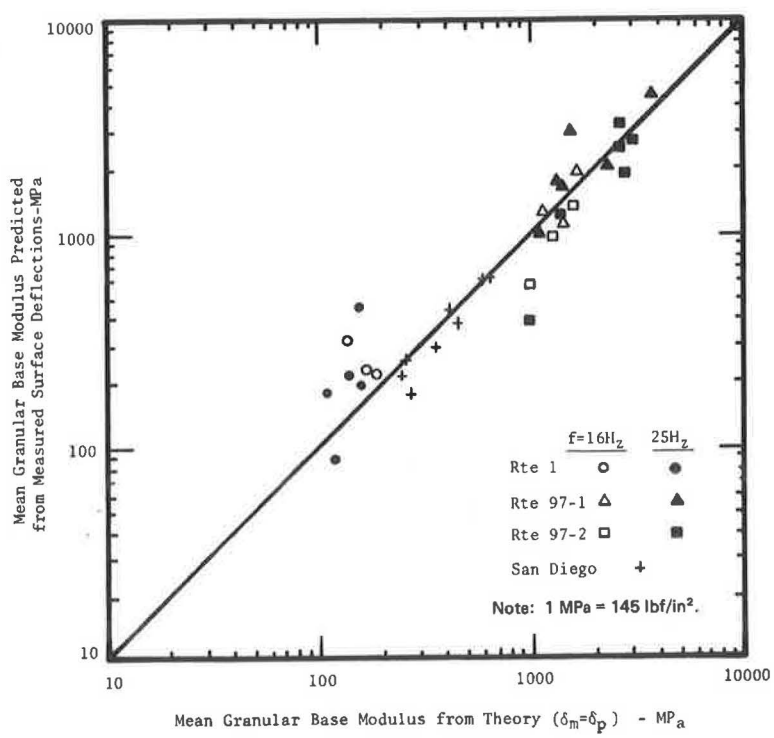


Figure 12. Relationship between mean granular base moduli predicted from measured deflections and moduli computed from theory.

Relationships shown in Figures 10 and 11 can be used to formulate a possible explanation for the decreased $K_1(R_d = 1.0)$ with increased surface deflection. Increased surface deflection implies an increased level of shear strain (Figure 11) and therefore decreased in situ granular-base modulus. This decreasing in situ modulus occurs simultaneously with increasing bulk stress. Hence, the K_1-factor necessary to characterize that modulus must also decrease.

ESTIMATE OF IN SITU MODULUS

The results shown in Figures 6 and 8 were grouped together to formulate a provisional procedure that can be used to estimate the in situ granular-base modulus from measured pavement deflections. From Figure 6, it can be observed that the relationship for a given load (P) is of the form

$$K_1(R_d = 1.0) = A_i \delta_m^{B_i} \quad (5)$$

where A_i and B_i are regression constants that depend on the load magnitude. For the three loads shown in Figure 6 the values of A_i and B_i are (1 kN = 224.8 lbf):

P(kN)	A_i	B_i
2.16	1.645×10^{-9}	-2.872
5.27	4.753×10^{-11}	-3.773
40.00	5.577×10^{-4}	-2.390

From Figure 8, the regression equation relating $K_1(R_d = 1.0)$ to mean granular-base modulus ($\bar{E}_{gb}$) is

$$\bar{E}_{gb} = CK_1(R_d = 1.0)^D \quad (6)$$

where C = 5700 and D = 1.231. Since $K_1(R_d = 1.0)$ is predicted by the measured deflection (δ_m) and $\bar{E}_{gb}$ is predicted by $K_1(R_d = 1.0)$, $\bar{E}_{gb}$ may be predicted directly from δ_m by

$$\bar{E}_{gb} = C(A_i \delta_m^{B_i})^D \quad (7)$$

Figure 12 is a summary comparison between the predicted $\bar{E}_{gb}$ found from the iterative-layer-theory solution and the computer program for deflection results of US-1, MD-97, and the San Diego test-road sections. The average percentage of error between modulus values was found to be 65 percent and 25 percent, respectively, for US-1 and MD-97 and 13 percent for the San Diego test-road sections. Although the general agreement is not perfect, the relationship is apparent, and further research should certainly develop a more accurate and applicable system. If this is accomplished, it is believed that a very significant step will have been taken toward the use of measured dynamic deflection data to predict the moduli of the subgrade layer. Such an objective is the major utility of eventually employing nondestructive testing devices for overlay and rehabilitation problems.

CONCLUSIONS

The following conclusions are presented, based on the data obtained in this study.

1. For small dynamic loadings (road rater), flexible-pavement models with a one-layer subgrade condition were found to be sufficient for accurate surface-deflection computations when the nonlinear subgrade response was incorporated into the analysis. Only a minor decrease in computed surface deflection was found when the subgrade was subdivided into three layers for the stress-dependent pavement study.
2. The 6.0-kN static preload of the road rater need not be considered for accurate modeling of the loading methodology. Results of the static preload study have shown that the inclusion of the static preload will reduce computed deflections by only a minor degree (4 percent).
3. Based on the comparison of computed and measured surface deflections for US-1 and pavement sections of MD-97 and I-695, deflections computed with elastic-layer theory, laboratory dynamic modulus testing, and a nonlinear iteration approach for both granular and subgrade layers are larger (by a factor of 2 to 4) than measured surface deflections from the road rater. Results of this study support previous research, which indicates that the major factor contributing to this discrepancy lies in the present characterization procedure for the resilient modulus of granular materials (i.e., $M_R = k_1 \theta^{k2}$).
4. It was found that the log $K_1(R_d = 1.0)$ versus log measured-surface-deflection (δ_m) relationship reported by Jones (8) for the San Diego test-road sections could also be applied to pavements analyzed in this study. This relationship was found valid for US-1 and MD-97, both of which have pavement cross sections similar to those studied by Jones [asphalt surface layer 7.5-13 cm (3-5 in) thick and conventional granular base].
5. The linear relationship between log in situ granular-layer modulus and log $K_1(R_d = 1.0)$ demonstrated by Figure 8 was observed for all pavement sections analyzed, regardless of dynamic load magnitude and asphalt layer thickness. This and the $K_1(R_d = 1.0)$ versus δ_m relationship were used to develop a provisional procedure for estimating in situ granular-layer modulus from dynamic road-rater deflections. It is concluded that this procedure has promise and that further research should certainly allow development of a more accurate and applicable system.
6. The resilient modulus of the granular layer was found to decrease with increasing level of shear strain. This finding was demonstrated by the linear log-log relationship of $K_1(R_d = 1.0)$ versus maximum shear strain (Figure 10) and substantiates other recent research dealing with the dynamic response of granular soils.
7. The effect of shear strain on the granular-layer modulus (theoretical model) is believed responsible for the increase in $K_1(R_d = 1.0)$ with increase in surface deflection. Increased surface deflection was found to increase the level of shear strain, thus decreasing the in situ modulus. This decreasing in situ modulus occurs simultaneously with increasing bulk stress (θ). Since $\delta_p > \delta_m$ and M_R is proportional to θ (i.e., $M_R = K_1 k_1 \theta k2$), the $K_1(R_d = 1.0)$ value must decrease as δ_m increases.

Although the results of this study demonstrate a problem in laboratory modulus characterization of granular materials, it should also be recognized that further research that involves a larger number of pavement sections and dynamic loads between those of the road rater and an 80-kN axle load needs to be conducted before final conclusions and a rational explanation of this phenomenon can be stated.

ACKNOWLEDGMENT

We are indebted to the Maryland State Highway Administration for providing financial and technical assistance necessary for this study.

REFERENCES

1. The WASHO Road Test: Part 1. HRB, Special Rept. 18, 1962.
2. The AASHO Road Test: Report 5—Pavement Research. HRB, Special Rept. 61E, 1962.
3. P.A. D'Amato. Analysis of In Situ Granular Layer Modulus from Dynamic Road Rater Deflections. Univ. of Maryland, College Park, M.S. thesis, 1977.
4. M.W. Witczak and K.R. Bell. Remaining-Life Analysis of Flexible Pavements. Proc., AAPT, Vol. 47, 1978.
5. M.W. Witczak. Determination of Remaining Flexible Pavement Life, Volume 1: Executive Summary. State Highway Administration, Maryland Department of

Transportation, Brooklandville, FHWA-MD-R-79-1, Oct. 1978. NTIS: PB-296562/2ST.
6. Seismic Analysis of Structures and Equipment for Nuclear Power Plants. Bechtel Power Corporation, San Francisco, CA, Rept. BC-TOP—4-A, Revision 3, Nov. 1974.
7. S.F. Brown. Repeated Load Testing of a Granular Material. Geotechnical Engineering Journal of ASCE, Vol. 100, No. GT7, July 1974, pp. 825-841.
8. M.P. Jones and M.W. Witczak. Subgrade Modulus on the San Diego Test Road. TRB, Transportation Research Record 641, 1977, pp. 1-6.
9. H.F. Southgate. An Evaluation of Temperature Distribution Within Asphalt Pavements and Its Relationship to Pavement Deflections. Kentucky Department of Transportation, Frankfort, Rept. HPR-1(3), Part 2, 1968.
10. J.W. Hall, Jr. Nondestructive Pavement Evaluation Machine. U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS, Prelim. Rept., July 1973.

Publication of this paper sponsored by Committee on Pavement Condition Evaluation.

Pavement Design for Permafrost Conditions: Structural and Thermal Requirements

LYNNE G. COWÉ FALLS AND RALPH HAAS

The existing Arctic road network is made up almost completely of gravel-surfaced secondary roads for which design, construction, and maintenance procedures are adequate. Proposed reconstruction and paving of the Alaska Highway in the next decade has raised several questions about the adequacy of pavement-design technology for permafrost areas. Because of the nature of permafrost terrain the problems of pavement design are twofold: provision of a structurally sound, smooth pavement to allow safe passage of vehicles during critical thaw periods and prevention of thermal degradation of the subgrade and right-of-way. Recent research has concentrated on the evaluation of new materials and design configurations that minimize subgrade thaw settlement. Research into the structural performance of pavements on permafrost has been minimal. Identification of the structural and thermal bases for pavement design in permafrost areas is a key requirement for the development of a design technology that includes economic analysis and evaluation. This paper examines the effects of environment, materials, and loading on the thermal and structural responses of insulated and conventional pavement designs on discontinuous permafrost. The vertical temperature and stress distribution for a range of feasible designs was analyzed by means of two computer programs. Dynamic traffic loading of the structures investigated did not produce excessive subgrade strains. However, the dead load of the structure contributed greatly to thaw consolidation of the subgrade. None of the designs completely prevented subgrade thaw. A trade-off between dead load of the structure and thermal protection of the subgrade was identified. This conclusion provides a new justification for the use of low-density insulating layers in pavements on unstable permafrost.

With the increased emphasis on northern construction, such as the jointly financed Canadian-U.S. project to reconstruct and pave the Alaska Highway, pavement-design technology needs to be able to consider permafrost conditions.

On continuous permafrost where the depth of annual thaw is shallow, paved surfaces have been constructed with reasonable success by using conventional design technology. However, the scale of the Alaska Highway project and the instability of the terrain that it traverses bring into serious question the adequacy of southern pavement-design methods applied to discontinuous permafrost. Although equilibrium with the subgrade has been reached in the 35-year existence of the Alaska Highway, differential settlements can be expected when additional construction (and in many cases realignment) again upsets the thermal balance. Although these settlements would be relatively easy to correct if the highway were to remain a gravel surface, patching and padding on an asphalt surface will be much more expensive. Current annual maintenance costs for the gravel-surfaced highway are about $6500/km (1). The annual maintenance cost of a badly distorted and cracked paved surface would be substantially higher.

The general purpose of this paper is to consider the design interaction between thermal and structural requirements for pavements on permafrost by using the Alaska Highway as a case history. There are three main objectives:

1. Review of the general design problems associated with discontinuous permafrost, the applicability of current design technology, and the use of experimental pavement designs;
2. Fundamentally based analysis of the structural and thermal responses of a range of feasible designs for a variety of environmental, loading, material, and other conditions; and
3. Evaluation of the boundaries within which structural and thermal trade-offs can be made within the factor space of 2.

Some of the terms used in this paper are defined as follows:

1. Permafrost: Soil or rock (or both) that has a mean annual ground temperature at or below 0°C for several years;
2. Active layer: Depth below the surface at which annual temperature fluctuations are not felt, i.e., depth of maximum thaw;
3. Continuous permafrost: Permafrost that has a mean annual ground temperature below -4°C--frozen ground is continuous (i.e., there are no breaks) and the active layer is shallow; and
4. Discontinuous permafrost: Permafrost that has a mean annual ground temperature between 0° and -4°C--the active layer is very deep and the existence of permafrost is widespread; vegetation and topography dictate the presence or absence of frozen ground.

DESIGN PROBLEMS, CURRENT TECHNOLOGY, AND EXPERIMENTAL DESIGNS

Problems of Pavement Design in Discontinuous Permafrost

In permafrost zones, where the mean annual ground temperature is at or below 0°C, thaw penetration has a greater effect on a pavement structure than does frost penetration. Because of the ice-rich nature of permafrost silts, clays, and organic soils, alteration of the surface characteristics causes an increase in the equilibrium ground temperature and a deepening of the active layer. Over the