EVALUATION OF RIGID PAVEMENTS BY NONDESTRUCTIVE TESTS

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This paper presents a vibratory nondestructive evaluation procedure as applied to rigid pavements. It is restricted, however, to a comparison between a measured deflection field around a loaded plate and a predicted deflection field obtained by using elastic properties of layers (gathered by nondestructive tests) in a radial symmetric finite-element program on a test section. Because these two deflection fields do not agree in their magnitudes due to the low strain level created by the vibrator during the determination of the elastic properties of the layers, two methods to correct the modulus of the subgrade material (determined by low-intensity vibration tests) were investigated. The first method uses the information obtained by a plate load test; the second method uses laboratory repeated load test results (developed by the University of Kentucky). These methods were applied in a simplified form to a test section. By utilizing the finite-element method, the predicted deflection field with the corrected modulus was compared with the measured deflection field. Further studies required to make this evaluation procedure a useful tool in solving practical problems are outlined.

• THERE are several hundred airfields in the United States that are being used extensively by military and commercial aircraft. Most of these airfields have been in existence for quite some time. Some of these airfields require strengthening because of the heavy gear loads of the latest aircraft. Determination of airfield strengthening requirements is made by studying the current condition of the airfield and evaluating its load-carrying capacity. Thus, pavement evaluation can be defined as a study to determine the suitability of the pavement to support repeated loads of known magnitude.

There are several types of pavement evaluation. Grouped into four categories, they are as follows:

1. Visual evaluation—detect cracks, pumping, soft spots, and so forth;
2. Surface evaluation—skid resistance, drainage, and so forth;
3. Strength evaluation—study the load-carrying capacity; and
4. Environment effects and repeated load effects.

This paper is restricted to the study of strength evaluation only.

Currently, there are semi-empirical methods for evaluating a pavement. For instance, according to the Bureau of Yards and Docks, the load required to cause a 0.15-in. deflection would be the limiting load that could be applied on a flexible pavement (1), or if the stresses, computed by Westergaard's analysis, in a rigid pavement are less than the permissible stresses for concrete, the pavement would be safe to carry that load. Similar criteria on limiting radius of curvature are also available in the literature (2). However, in order to use these methods, support tests (determination of CBR or coefficient of subgrade reaction or plate load tests, etc.) are required. This, in turn, would result in closing down the runway or taxiway for a considerable

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period of time. Hence, a rapid method of evaluating the pavement by nondestructive methods is urgently needed. This paper explores one such method, namely, the vibratory nondestructive procedure.

When a vibrator operates on the surface of a layered system, it generates elastic waves that travel as surface, compression, and shear waves. A major portion of the energy is transmitted as a surface wave. Dispersion curves can be obtained for the medium by determining the velocity of this surface wave and by studying its change with the frequency of the vibrator. This paper is concerned with the dispersion of the surface Rayleigh waves. By studying the dispersion phenomenon, it is possible to obtain the elastic properties of the various layers (3).

The objectives of this study were as follows:

1. Estimate the elastic properties of layered systems by steady-state vibration testing;
2. Predict the deflection basin under a statically loaded plate by using the estimated layer properties;
3. Compare the predicted deflection basin with an experimentally measured deflection basin; and
4. If the measured deflection basin and the predicted deflection basin are radically different from each other, state the causes for disagreement and suggest procedures for correcting the properties of layered systems to make the deflection basins agree.

These objectives serve to develop a method of predicting the deflection basin under a given load, which is the first step in developing a rational method of pavement evaluation.

EXPERIMENTAL PROCEDURE

A low-intensity dynamic force was generated with an electrodynamic vibration generator by means of a power amplifier driven by an oscillator. The vibration generator operates between 100 and 10,000 Hz. Another mechanical vibrator generating about 250 lb of dynamic force and operating between 15 and 50 Hz was used to obtain the subgrade elastic property.

The vibration energy introduced into the layered system is dissipated primarily as a surface wave. Two accelerometers were located at a known distance apart on the surface of the layered medium. The outputs of these accelerometers were fed into a phase computer and display unit through a dual-channel tracking filter. The phase difference observed was used to compute the phase velocity of the surface wave. The results were plotted as a relation between the phase velocity, v, and the wavelength, Such a relation is called a dispersion curve.

When the experiments were performed in a very short time, a sweep oscillator was used to drive the vibrator through a power amplifier producing a signal of slowly varying frequency. The outputs from the two accelerometers were recorded on a multichannel tape recorder. The data were later reduced by using a high-speed digitizing technique and a Calcomp plotter (4). Figure 1 shows the setup, and Figure 2 shows the apparatus used for obtaining the dispersion curves.

TEST SECTION

The test section consisted of a 10-ft thick processed clay soil of low plasticity (plasticity index = 15) placed at a CBR of 8 to 12, a 6-in. thick compacted gravel base course, and a 12-in. thick surface layer of portland cement concrete with ¾-in. aggregate.

VIBRATION TESTS

High-Frequency Tests

High-frequency vibration tests were performed on this test section. One accelerometer, located 3 in. from the vibrator, was used as a reference. The second accelerometer was placed at various distances from the first in increments of approximately

6 in. The phase difference at a set frequency obtained by the outputs of the two accelerometers was converted to the phase velocity. Several tests were performed at various frequencies, and the dispersion curve was plotted. Figure 3 shows the dispersion curve obtained by the high-frequency vibration tests.

Low-Frequency Tests

Low-frequency vibration tests were performed with a DEGEBO type vibrator (5) using one force level of 250 lb. The reference accelerometer was located at a distance of 1 ft from the center of the vibrator, and the moving accelerometer was located at various distances in increments of approximately 3 ft. Figure 3 also shows the dispersion curve for the low-frequency tests.

INTERPRETATION OF VIBRATION TEST RESULTS

The method of interpreting dispersion curves has been discussed extensively in earlier literature (6, 7). At very high frequencies (very short wavelengths), the dispersion curve gives the properties of the surface layer. The primary mode is the flexure mode; however, it is also possible that the compression mode (8) may be detected in some cases. Thus, it appears according to the theory of Lamb (8) that point A (Fig. 3) would correspond to the Rayleigh wave velocity in concrete. The Rayleigh wave velocity is approximately equal to the shear wave velocity in any material. In concrete, if we assume a Poisson's ratio, ν, of 0.2, the Rayleigh wave velocity, V_r, is related to the shear wave velocity, V_s, by

$$V_r = 0.91 \ V_s$$

On the other hand, at very long wavelengths, the dispersion curve would give the properties of the subgrade. Thus, point E on dispersion curve DE would give the shear wave velocity of the subgrade. Region BC of the curve shows a considerable scatter of results.

The assigned values of wave velocities and moduli of elasticity of the various layers were as follows:

1. Rayleigh wave velocity of surface layer, 7,800 ft/sec;
2. Modulus of elasticity of surface layer (assuming $\nu = 0.2$), 5.5×10^6 psi;
3. Shear wave velocity of subgrade material, 880 ft/sec; and
4. Modulus of elasticity of subgrade material (assuming $\nu = 0.45$), 56,000 psi.

The vibration method failed to recognize the presence of the intermediate layer (base course) and to give the information pertaining to Poisson's ratios.

INFLUENCE OF BASE COURSE PROPERTIES

The vibration method is only useful in obtaining the elastic properties of the materials. Because it did not give any information on the properties of the base course, it became necessary to determine the influence of the base course on stresses and displacements. A computer study was thus undertaken to qualitatively assess the relative importance of parameters in a three-layered pavement system. An axisymmetric finite-element program, WIL67 (9), developed at Berkeley was used. A standard problem was conceived. The pavement system contained a 12-in. thick surface course, a 6-in. thick base course, and a 498-in. thick subgrade. A 12-in. diameter area at the center of the slab was assumed to be loaded with a uniform pressure, and the influence of variations of the following parameters was studied: modulus of subgrade layer, E_3; modulus of base course layer, E_2; modulus of surface layer, E_1; Poisson's ratio of subgrade layer, ν_3; Poisson's ratio of base course layer, ν_2; and Poisson's ratio of surface layer, ν_1.

Maximum computed displacements at the center of the loaded area and the radial tensile stresses at a depth of 10.8 in. (inside the surface layer) are given in Table 1.

Table 1 shows that only three parameters (E_1, E_3, and ν_3) are necessary to determine the magnitude of displacement, of which the vibration method could only provide two. For radial stresses, E_1 and E_3 are the most important parameters. A reasonably good value of Poisson's ratio for the soil can be assumed based on experience (Poisson's ratio varies within a small range from about 0.30 to 0.45 for soils). Because the moisture content was high, the Poisson's ratio assumption of 0.45 for the subgrade seemed justified.

LOAD TESTS AND MEASUREMENT OF DEFLECTION BASIN

Two circular plates, one 6 in. in diameter and one 12 in. in diameter, were used to obtain the deflection basin. The total load applied was 30,000 lb. The test arrangement is shown in Figure 4. The deflections at 11 points on each of the 3 tangential lines at 1, 2, and 3 ft from the center of the loaded area were measured by dial gauges. Also, the deflection of the plate was measured at two points on the plate itself. These deflections were measured from 35-ft long steel beams fixed at one end and supported on rollers at the other end. These beams, as well as the loading frame, spanned the entire cross section of the test section. Average deflections of several loading tests are shown in Figures 5 and 6 for the two plates.

COMPUTED DEFLECTION BASIN

Because Table 1 showed that the base course modulus variation played a very minor role in computed deflections, a modulus of 100,000 psi was selected for the base course. Poisson's ratio was set at 0.3.

The deflection basin was computed with the axisymmetric finite-element program by using the moduli of the three layers along with their assumed values of Poisson's ratio. The only difference between the assumed computation model and the real situation was that the clay layer was taken to be 500 in. thick for purposes of computation, whereas in the test section it was only 120 in. thick and underlaid by an in situ silty sand material. Because the stresses induced by the surface load at depths greater than 120 in. are negligible, the associated displacements are also small, and hence the assumption of the existency of clay in place of silty sand would not cause significant error.

The computed displacements at various points are also shown in Figures 5 and 6. In these figures E_c and E_s are the moduli of the concrete and subgrade respectively.

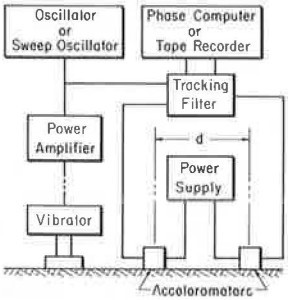
COMPARISON OF COMPUTED AND MEASURED DEFLECTION BASINS

By studying the agreements between the measured and computed deflection basins (Figs. 5 and 6), one will discover that there are two distinct regions in the displacement field. They are as follows:

1. Region A (high shear strain). The computed deflections are extremely small as compared to measured deflections. Such a region exists directly under the loaded plate or very close to the edge of the plate.
2. Region B (moderate shear strain). The computed deflections are smaller than the measured deflections by approximately one-half to one-third of the measured deflection.

The existence of these regions can be understood by the strain fields in the computations. There is a relatively steep strain gradient at the edge of the plate compared to that at a distance of 3 ft from the plate. The strain gradient decreased as the distance from the center of the plate increases. Such a decrease is rather slow in concrete pavements compared to asphalt pavements. The shear modulus of material like soil depends on the strain level in the material. For instance, the shear modulus obtained by using the pulse technique is much larger than the shear modulus obtained by using the conventional triaxial test. The pulse technique gives the modulus at very small strains, whereas the results of the triaxial test give the modulus at relatively large strains. The moduli obtained vary by several orders of magnitude. Similarly, the strain level in vibratory nondestructive tests is extremely small, whereas the strain level under a loaded plate is relatively large. This is probably the reason why the

Figure 1. Schematic of test setup.



Notes:
1. $\phi°$ = phase difference for distance d
2. One wavelength = 360·d/ϕ
3. Phase velocity = f·360·d/ϕ
4. Frequency, f, is changed to obtain dispersion curves

Figure 2. Vibration test equipment.

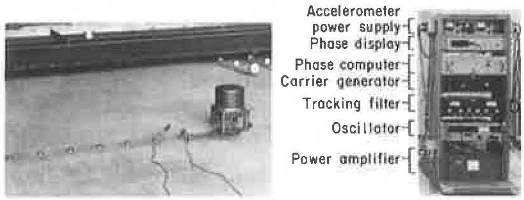


Figure 3. Vibration test results.

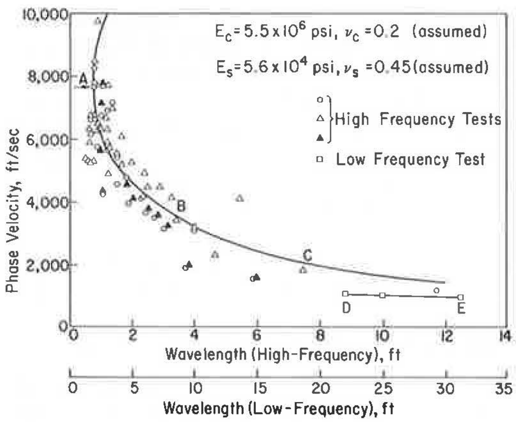


Figure 4. Load test and displacement measurement setup.

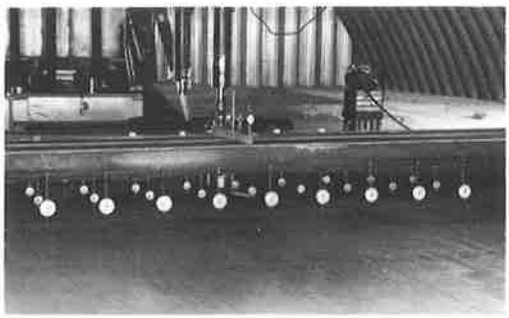


Figure 5. Computed and measured deflection fields (12-in. diameter plate).

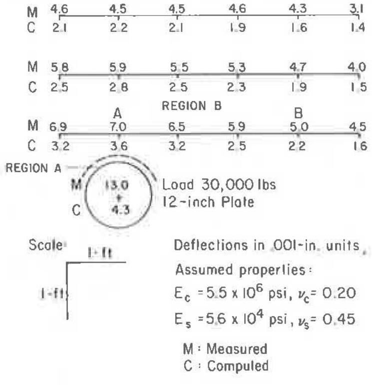


Figure 6. Computed and measured deflection fields (6-in. diameter plate).

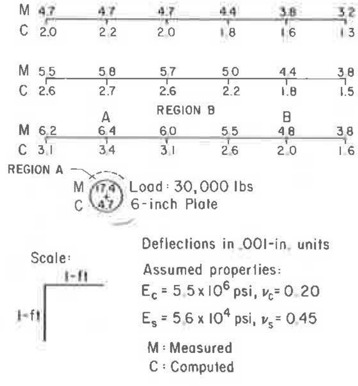


Table 1. Results of parametric study using finite-element model.

Run	System Parameters		Maximum Deflection, in.	Maximum Radial Tensile Stress at 10.8 in., psi	Variable	Remarks
	Modulus (E), psi	Poisson's Ratio (ν)				
1	$E_1 = 3.5 \times 10^6$	$\nu_1 = 0.2$	(a) 0.16383	(a) 157	E_3	Important Parameter
	$E_2 = 50{,}000$	$\nu_2 = 0.3$	(b) 0.00486	(b) 130		
	$E_3 =$ a) 4,000 b) 40,000 c) 400,000	$\nu_3 = 0.45$	(c) 0.00210	(c) 99		
2	$E_1 = 3.5 \times 10^6$	$\nu_1 = 0.2$	(a) 0.01109	(a) 161	E_2	Negligible Effect
	$E_2 =$ a) 2,000 b) 20,000 c) 200,000	$\nu_2 = 0.3$	(b) 0.00957	(b) 155		
	$E_3 = 10{,}000$	$\nu_3 = 0.45$	(c) 0.00892	(c) 131		
3	$E_1 =$ a) 10^5 b) 10^6 c) 3.5×10^6	$\nu_1 = 0.2$	(a) 0.04790	(a) 28.7	E_1	Important
	$E_2 = 50{,}000$	$\nu_2 = 0.3$	(b) 0.01699	(b) 117		
	$E_3 = 10{,}000$	$\nu_3 = 0.45$	(c) 0.00935	(c) 150		
4	$E_1 = 3.5 \times 10^6$	$\nu_1 = 0.2$	(a) 0.01437	(a) 148	ν_3	Important for Displacement
	$E_2 = 50{,}000$	$\nu_2 = 0.3$	(b) 0.01301	(b) 149		
	$E_3 = 10{,}000$	$\nu_3 =$ a) 0.15 b) 0.30 c) 0.45	(c) 0.00935	(c) 150		
5	$E_1 = 3.5 \times 10^6$	$\nu_1 =$ a) 0.05 b) 0.20 c) 0.35	(a) 0.00941	(a) 131	ν_1	Negligible
	$E_2 = 50{,}000$	$\nu_2 = 0.30$	(b) 0.00935	(b) 150		
	$E_3 = 10{,}000$	$\nu_3 = 0.45$	(c) 0.00905	(c) 169		

Figure 7. Computed and measured deflection fields (12-in. diameter plate with modified subgrade modulus).

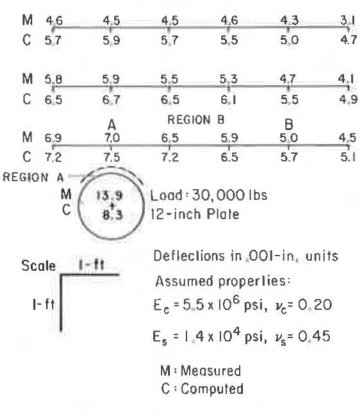
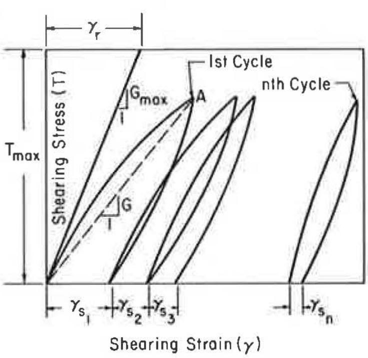


Figure 8. Pure shear stress-strain relation for repeated load tests.



computed displacement fields obtained by using the vibration modulus are much smaller than the measured displacement fields under the loaded plate.

There are two methods for correcting the modulus. One of them is empirical and requires a field test and measurement of the relative deflection between two points. The other utilizes the relation between shear modulus and shear strain on specimens compacted to the same water content and density as in the subgrade.

Method I

The measured relative deflection between points A and B in Figure 5 for the 12-in. diameter plate was 0.0020 in. Similarly, the measured relative deflection between points A and B in Figure 6 for the 6-in. diameter plate was 0.0016 in. These relative deflections occurred under the 30,000-lb load and were easily measured. The computations showed a completely different relative deflection. The computed relative deflection was 0.0014 in. between points A and B for both the 6- and 12-in. diameter plates. Because the deflection is a result of the deformation in the softer subgrade layer, its modulus for purposes of computation may be altered until the computed relative deflection matches exactly the measured deflection basin. Figure 7 shows the results for one of the loaded plate defelction basins by using a modified value of 14,000 psi for the subgrade modulus. The computed relative deflection was 0.0018 in., whereas the measured relative deflection was 0.0020 in. This tends to better match the entire measured deflection basin and the computed deflection basin. This modified modulus value would be more appropriate for computation of the deflection basin under a loaded plate than the modulus obtained by vibration testing. However, this procedure cannot be extrapolated to larger loads.

Method II

This method was developed recently by the University of Kentucky (10). Figure 8 shows the shear stress-strain relation for a continuous constant amplitude loading. Each cycle of loading and unloading is accompanied by a permanent set, γ_{s_n}. The maximum shear modulus obtained at the origin is denoted by G_{max}. For a point A on the loading curve in a load test, one should use G but not G_{max} if linear elastic theories are applied. However, the vibration test results only provide G_{max}. With this fact, a special procedure was developed to correct the modulus (10).

TESTS TO CORRECT ESTIMATED SUBGRADE MODULI

Briefly, the procedure consists of preparing a hollow specimen at the moisture content and density of in situ material and subjecting it, under triaxial conditions, to a torsional type of shear test. The amplitude and rate of loading can be varied. The loading and unloading curves are obtained with an x-y recorder.

A batch of silty clay (used as subgrade material in the test section) was sent to the University of Kentucky, and the results of repeated load tests were published in a recent report (10). Because the subgrade material in the test section had a high moisture content, the data pertaining to high saturation levels were taken from that report and are presented here. Table 2 (taken from another publication, 10) gives the various conditions of the test and the corresponding E values (assuming $\nu = 0.45$).

It is interesting to note that the E_{max} obtained by laboratory tests at the University of Kentucky for the 60 to 90 percent saturation varies between 15 and 20,000 psi. Interpretation of the data from the vibration tests gave an E_{max} of 56,000 psi, which is rather high.

There are three possible reasons for the difference between the E_{max} values obtained by in situ vibration tests and the laboratory tests performed at the University of Kentucky. They are as follows:

1. The presence of the base course in the in situ tests has influenced the interpreted value of the elastic subgrade modulus;
2. The soil in situ could have built a structure because of the time lapse between construction and testing, whereas the tests on the compacted samples were conducted immediately after compaction; and

3. It has been found, by limited experimental data, that E_{max} obtained in the laboratory for stresses below 10^{-6} in./in. remains a constant. If such a statement is not valid for this soil, then the difference in the strain levels on laboratory samples and in in situ tests would account for some of the difference in the E_{max} values.

It is probably because of the first two reasons that a larger E_{max} was produced in the in situ tests. The effect of this difference will diminish when the data are normalized, as will be explained.

The data obtained by the University of Kentucky (10) were plotted in a nondimensional form from the first loading cycle as G/G_{max} versus γ/γ_r (Fig. 9, which is taken from another publication, 10) where γ_r is the reference strain shown in Figure 8. Based on repeated load tests, the best fit curves for samples between 10 and 1,000 cycles of loading and unloading and for first cycles of loading are shown in Figure 9. It was found by these tests that, by dividing the strain, which is already nondimensional, by another reference strain, γ_r (shown in Fig. 8), all data would fall on the two curves shown in Figure 9. The results shown in Figure 9 indicate that the modulus, which varies widely, depends entirely on the strain level. Repeated loads (up to 1,000 cycles) have a tendency to increase the modulus, as shown by the dotted line in Figure 9, until the fatigue effects (which are more pronounced in asphalt pavements) become predominant.

TEST RESULTS TO CORRECT MODULUS

To obtain the deflection basin by nondestructive tests, one must obtain additional information on the variation of shear modulus with the shear strain level. The maximum modulus obtained for the subgrade material by the in situ nondestructive tests will be higher than the value obtained on compacted laboratory samples. One could, therefore, question the applicability of using Figure 9 to correct the in situ modulus. It may be observed from Table 2 that the E_{max} values of the various specimens varied from 5,000 to 20,000 psi. Still, the normalization technique has brought all the results onto one curve shown in Figure 9 (for the first cycle of loading). Hence, the difference in E_{max} values between in situ and compacted laboratory samples will not play a significant role because of the normalization technique used.

Curves similar to Figure 9 are yet to be developed for base course material and for other soil types. The other important study yet to be made is to modify the finite-element programs to accept the modulus variation from point to point. Once these studies are made, an iteration technique may be developed as follows:

1. Based on vibration tests, the maximum moduli values determined by using in situ vibration tests are assumed for each layer, and the strain and deflection fields are obtained by using finite-element programs.
2. Based on data similar to that shown in Figure 9, the modulus of each element is modified depending on its strain level. With the modified modulus value, the new strain and displacement fields are obtained.
3. By iterating several times, the correct deflection profile is obtained for the loaded pavement.

This procedure is under active development at the Civil Engineering Research Facility (CERF), along with the cooperation of several agencies, under the sponsorship of the Air Force Weapons Laboratory (AFWL).

Because this procedure is in the developmental stage, the values given here are assumed to show the validity of the procedure. It is assumed that the properties of each layer will remain constant at each point in the layer but will depend on the average strain level at each layer. With $\gamma/\gamma_r = 1.25$ in the subgrade and $\gamma/\gamma_r = 0$ in the surface layer and base course, the modified moduli values for each layer would be as follows: surface layer, 5.50×10^6 psi; base course, 100,000 psi; and subgrade, $0.25 \times E_{max} =$ 14,000 psi.

Table 2. Data for tests on silty clay.

Test Number	Void Ratio	Saturation (percent)	Chamber Pressure (kg/cm^2)	Initial G_{max} (psi)	Initial[a] E_{max} (psi)
20	0.62	90	0.5	6,770	19,633
21	0.63	91	0.5	5,200	15,080
22	0.74	100	0.5	1,880	5,452
23	0.72	98	0.5	4,360	12,644
24	0.66	96	1.0	4,210	12,200
25	0.74	61	0.5	6,290	18,241

[a]Assuming ν = 0.45.

Figure 9. Normalized loading shear modulus versus normalized strain for silty clay.

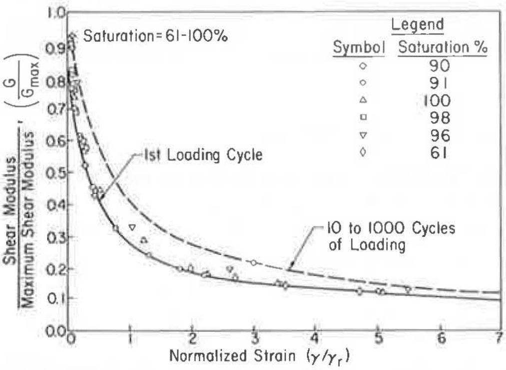
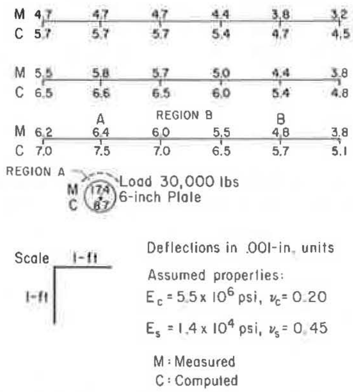


Figure 10. Computed and measured deflection fields (6-in. diameter plate with modified subgrade modulus).



COMPUTED AND MEASURED DISPLACEMENTS WITH MODIFIED SUBGRADE MODULUS

Figure 10 shows a comparison of the measured and computed displacement fields using the modified subgrade modulus and the 6-in. diameter plate. There is closer agreement in the computed and measured displacement fields close to the loaded area, particularly in region B. This study establishes the need for modifying the modulus depending on the strain levels. This procedure can be used not only for evaluation of pavements but also for developing a design technique when laboratory vibration moduli, G_{max} (based on resonant column test or pulse technique), are available for each of the layered materials.

Better agreement between computed and measured deflection at the loaded area itself could be obtained if the finite-element model were developed to accept the change in modulus with strain level at each element.

CONCLUSIONS

Results of the study substantiate the following conclusions:

1. The vibratory method of testing pavement provides information on the maximum modulus of the surface and the deep layer in a three-layered rigid pavement system.
2. The second base course layer plays a minor role in contributing toward the total deflection, and hence any reasonable modulus value may be assumed for this layer.
3. The vibratory method does not provide information on Poisson's ratio.
4. There are two distinct regions in the measured displacement field under thick rigid pavements: regions of high shear strain gradient and regions of moderate shear strain gradient.

5. Two procedures for correcting the modulus of the subgrade have been given. The first matches the relative deflections, and the second uses supplementary laboratory repeated torsion load tests on subgrade soil. The latter procedure is theoretically reasonable; the former procedure is only intuitive.

6. A method has been presented that would make the nondestructive method a practical procedure for estimating the deflection basin under a given load. In the absence of some of the information (like finite-element program accepting of inhomogeneity introduced by different strain levels), an approximate method can be used to predict the deflection basin. This method shows that the predicted deflections agree more closely with the measured deflection in region B when the modulus value for the subgrade is modified.

SUGGESTIONS FOR FUTURE STUDIES

The following suggestions for future studies are offered:

1. The existing finite-element programs should be modified to accept changes in modulus with changes in strain level;
2. Once the capability of predicting the entire deflection field under a loaded area is developed, a distress criterion in terms of strains or stresses should be developed to make the nondestructive procedure practical;
3. More soils should be tested to develop relations between the changes in modulus and the changes in strain level; and
4. This method has been tested on one rigid pavement test section and two flexible pavement test sections (11); however, it should be verified on actual airfields (such investigations are currently in progress).

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