Laboratory Investigation of Flexible-Pavement Response by Using Transfer Functions

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Transfer-function theory was applied to examine the behavior of flexible highway pavements. A numerical computation was used to derive pavement response functions from impulse testing of three-layer models of flexible pavements. Analytical computations formed the basis for calculating deflections that result from static and repeated loads. It was hypothesized that more-significant parameters than are commonly used could be obtained under controlled laboratory conditions. By using nonlinear regression, the response functions were approximated by a mathematical model to include these parameters. The derived model was used as the input for the analytical computations. The adequacy of the developed model was verified by comparing predicted and measured deflections. A silty-sand subgrade, a crushed-aggregate base, and an asphalt-concrete surface were the components of the three-layer systems. Model pavements of two different surface-course thicknesses were tested statically and dynamically at three different stress levels. To permit the tests to be performed inside a constant-temperature room, the maximum possible size adopted for the model pavements was 0.8 x 0.8 x 0.6 m (32.5 x 32.5 x 23.25 in). Test temperatures were 10°, 24°, and 38°C (50°, 75°, and 100°F). Deflections were measured at five locations. It was shown that time-dependent behavior of flexible pavements can be represented by response functions. The parameters in these functions are regarded as descriptors of pavement characteristics and have the capability of predicting pavement response.

The pursuit of a rational method for the design and evaluation of pavements requires considerable knowledge of pavement response behavior to realistic loadings. Current mathematical models for prediction of pavement response and evaluation of its behavior generally evolve from classical (elastic and viscoelastic) theories. These models incorporate geometric aspects and material properties within a system of equations that, in turn, are solved so as to satisfy selected boundary conditions (1). In both elastic and viscoelastic theories, mathematical expediency has led to idealization of the constitutive relations. However, any predictive model must include in situ (global) material properties. Parameters or functions capable of describing time- and temperature-dependent material properties of pavements subjected to static or dynamic loads must be obtained.

The purpose of this paper is to evaluate pavement behavior based on transfer-function theory that has been applied successfully to the solution of several problems in mechanical and electrical engineering (2-5). Swami, Goetz, and Harr (6) sought to characterize bituminous mixtures under a variety of test conditions. They concluded that an asphalt concrete at a constant temperature could be uniquely represented by a transfer function.

Boyer and Harr applied transfer functions to airfield pavements and showed that the time-dependent behavior of a flexible pavement could be represented by time-dependent transfer functions (7).

This investigation extends the transfer-function concepts to flexible highway pavements. More significant parameters, or indicators, than are usually hypothesized are defined and determined under controlled laboratory conditions.

The procedure is depicted in the flowchart of Figure 1. A known impulse load, which corresponded to the signature left by a truck tire on a pavement system, was applied to a pavement system. The resulting output deflections were measured as functions of time at several locations on the pavement surface. The time-dependent input and the corresponding outputs were analyzed by using transfer-function theory to give response functions; a computer subroutine CONVLI was used (8).

A mathematical model for the response functions was evolved by using a computer program labeled NONLINR. This model was computed analytically with step and repeated-load input functions. Computer programs STALOD and REPLOD facilitated the respective calculations. The adequacy of the developed mathematical-experimental model was evaluated by comparing predicted responses due to static and repeated loads with experimentally measured values. The effects of temperature, surface-course thickness, and spatial location on the dynamic parameters and the predictions were examined.

THEORETICAL ANALYSIS

Transfer-Function Approach

A dynamic system can be represented by a transfer function [G(s)] defined as the ratio of the Laplace transform of the output [O(t)] to the Laplace transform of the input [I(t)]. By this definition,

$$G(s) = \bar{O}(s)/\bar{I}(s) \tag{1}$$

where

$O(s) = L[O(t)]$,
$I(s) = L[I(t)]$,
s = Laplace transform parameter,
L = Laplace transformation, and
t = time variable.

Consider an impulse load input of peak value F_p that acts as a point $x = x_0 = 0$ on a pavement. The resulting deflection responses at points $x = x_1$, $x = x_2$, ..., $x = x_i$ along a line normal to the load represent the output functions.

By using the definition of Equation 1, the transfer function at a point $x = x_i$ on the pavement that is caused by the input load at $x = x_0$ can be written as

$$G_{x_i}(s) = \bar{O}_{x_i}(s)/\bar{I}_{x_0}(s) \tag{2}$$

where

$\bar{O}_{x_i}(s)$ = Laplace transform of output response at $x = x_i$,
$\bar{I}_{x_0}(s)$ = Laplace transform of input load at $x = x_0$, and
$G_{x_i}(s)$ = transfer function between an operational output response at $x = x_i$ and the operational input at $x = x_0$.

Thus, the transfer function characterizes the pavement system between points $x = x_0$ and $x = x_i$ and provides a body concept for the pavement. This is in contrast to the point concept inherent in classical elastic and viscoelastic theories.

After Equation 2 has been rewritten, it follows that

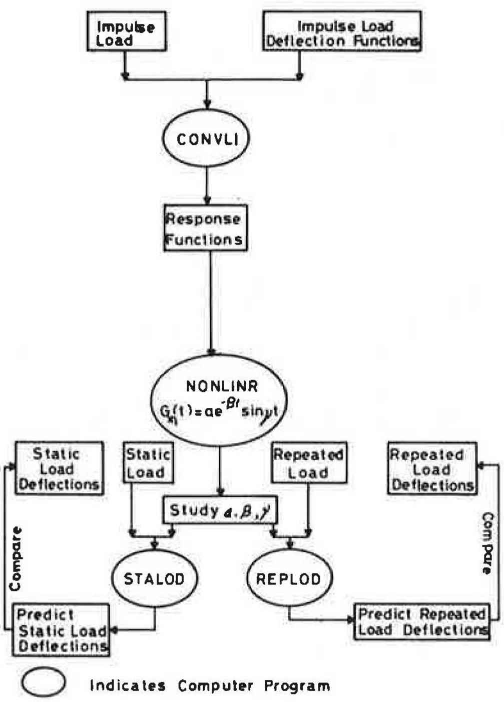
$$\bar{O}_{x_i}(s) = G_{x_i}(s)\bar{I}_{x_0}(s) \tag{3}$$

Equation 3 can be expressed in the time domain by using the computation of two functions (5):

$$O_{x_i}(t) = \int_0^t G_{x_i}(\tau) I_{x_0}(t-\tau) d\tau \tag{4}$$

where τ is the dummy time variable. The function $G_{x_i}(t)$ is the inverse transform of the transfer function $G_{x_i}(s)$ and is called the impulse response or the response function of the system.

Figure 1. Procedure for pavement evaluation.



The finite-difference solution (7) of Equation 4 to yield $G_{x_i}(t)$ gives Equation 5, provided that $I_{x_0}(t)$ and $O_{x_i}(t)$ are available as discrete values:

$$G_{x_i}(k) = \left[O_{x_i}(k) - \sum_{j=1}^{k-1} G_{x_i}(j) I_{x_0}(k-j+1)\Delta t_{x_i} \right] \Big/ I_{x_0}(1)\Delta t_{x_i} \qquad (5)$$

where j, k = dummy variables that represent discretized units and Δt = time interval.

Form of Response Function

The experimental basis discussed here to obtain the response function is the impulse test. An impulse-load input of known peak value was applied for a short time to the system for which the response function was sought. Both the impulse input force and the output deflections were recorded as functions of time. By using a modified version of the computer program CONVLI (8), the computation of response functions in conjunction with Equation 5 resulted in vector quantities of the response functions. These values were then approximated by a mathematical function that employed a nonlinear least-squares curve-fitting method (computer code NONLINR). A study of the response functions suggested the equation

$$G_{x_i}(t) = \alpha \exp(-\beta t) \sin\gamma t \qquad (6)$$

where

$G_{x_i}(t)$ = response function,
t = time, and
α, β, and γ = parameters of the response function.

Prediction of Step-Load Response

Consider the step-function input $F_{x_0}(t)$ applied at a point x_0 on the pavement:

$$F_{x_0}(t) = 0 \quad \text{for} \quad t = 0$$
$$F_{x_0}(t) = F_0 \quad \text{for} \quad t > 0 \qquad (7)$$

where F_0 = magnitude of the static compressive force. If the corresponding output time function at $x = x_i$ is $y_{x_i}(t)$, substitution of Equations 6 and 7 into Equation 4 yields

$$y_{x_i}(t) = \int_0^t F_{x_0}(t-\tau)\alpha \exp(-\beta\tau) \sin\gamma\tau \, d\tau \qquad (8)$$

Prediction of Repeated-Load Response

A haversine repeated load applied at $x = x_0$ is defined as

$$F_{x_0}(t) = 0 \quad \text{for } t = 0$$
$$F_{x_0}(t) = (F_0/2)[1 - \cos(2\pi/p)t] \quad \text{for } t > 0 \qquad (9)$$

where p = period of loading.

Substitution of Equations 6 and 9 into Equation 4 gives the repeated-load response $Y_{x_i}(t)$ at $x = x_i$:

$$Y_{x_i}(t) = (F_0/2) \int_0^t \alpha \exp(-\beta\tau) \sin\gamma\tau [1 - \cos(2\pi/p)(t-\tau)] \, d\tau \qquad (10)$$

The detailed solutions of Equations 8 and 10 and the computer programs STALOD and REPLOD, respectively, may be found elsewhere (9).

EXPERIMENTAL METHOD

To investigate the developed theory of pavement behavior, controlled laboratory testing of three-layer models of flexible pavements was conducted.

Scope of Testing

A silty-sand subgrade, a crushed-aggregate base 88.9 mm (3.5 in) thick, and an asphalt-concrete surface course were the components of the three-layer systems. Surface-course thicknesses of 25.4 and 50.8 mm (1 and 2 in) were used. Static, impulse, and cyclic loadings were considered. Contact pressures of 103, 206, and 412 kPa (14.93, 29.86, and 59.72 lbf/in^2) were applied over a circular loading plate of 101.6-mm (4-in) diameter at the center of the pavements. The tests were carried out in a constant-temperature room at 10°, 23.9°, and 37.8°C (50°, 75°, and 100°F).

Materials and Preparation of Pavements

The model pavements were contained in a wooden box reinforced with steel angles. To conduct the tests inside the constant-temperature room, the maximum possible size of the box had to be 0.8 x 0.8 x 0.6 m (32.5 x 32.5 x 23.25 in) deep. The silty-sand subgrade was compacted into 20 lifts by using an air hammer. The average dry density as determined by the sand-cone method (AASHTO T191) was 1848 kN/m^3 (115.5 lbf/ft^3), which was 101 percent of the standard AASHTO T99. The moisture content averaged 13.9 percent, which was close to the optimum moisture of 13.7 percent.

The base-course material was crushed-limestone aggregate conforming to Indiana specifications size 53. Tests performed according to T99 and T193 gave, respectively, an optimum density of 2022.4 kN/m^3 (126.4 lbf/ft^3) at an optimum moisture of 6.2 percent and a soaked California bearing ratio of 51.3 percent. The aggregate was compacted to a total thickness of 88.9 mm in three lifts by means of the air hammer. The surface was leveled with a vibrator.

The bituminous mixture used for the surface course was prepared from crushed limestone and sand blended according to Indiana type-B gradations. Asphalt cement was used that had a 6 percent (by weight of aggregate) penetration value of 60-70. The surface course was compacted and leveled by the air hammer and vibrator for a predetermined time. The density obtained for the 25.4-mm asphalt concrete was 2396.8 kN/m^3 (149.8 lbf/ft^3). The in-place void

Figure 2. Generation of impulse.

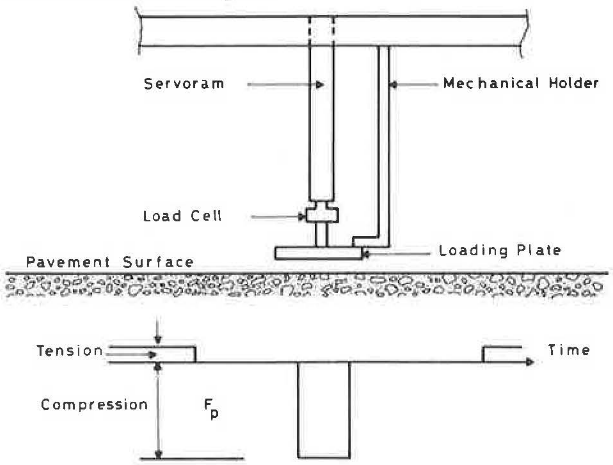
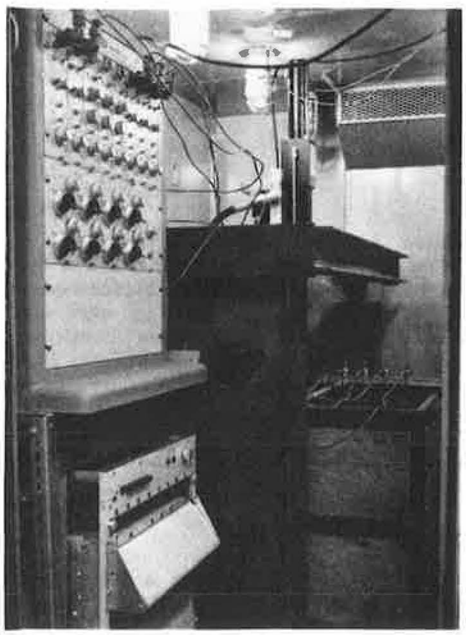


Figure 3. Overall view of test setup.



content corresponding to this density was calculated to be 4.6 percent. Hveem tests on laboratory-mix specimens yielded a unit weight of 2408 kN/m^3 (150 lbf/ft^3), Hveem stability of 48.1, and air voids of 4.2 percent.

On completion of the tests on the pavement that had 25.4-mm surfacing, the surface-course thickness was increased to 50.8 mm.

Instrumentation

Instrumentation was devised that would record the time-dependent input and output functions. A crucial part of the instrumentation technique was to devise a system capable of applying an impulse force of desired magnitude and duration and of detecting the surface motion at any required location on the pavement surface.

The equipment consisted of a steel loading frame coupled to a Material Testing System (MTS) electrohydraulic actuator. A mechanical holder was devised to hold the Servoram while the MTS console was being programmed in tension prior to application of a compressive square wave. The generation of an impulse by this technique is illustrated schematically in Figure 2.

Figure 4. Closeup view of test setup.

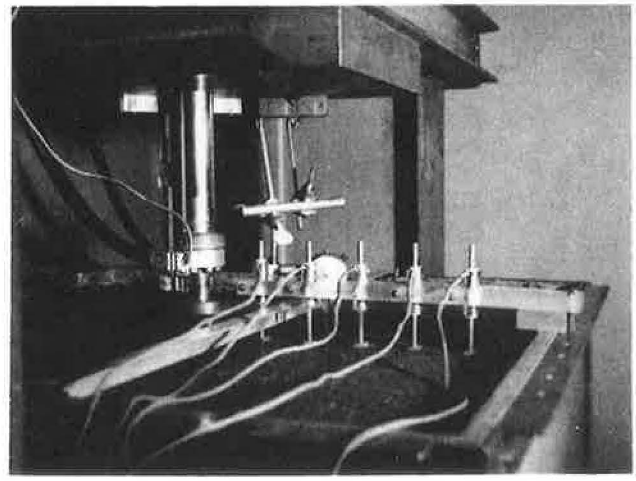


Table 1. Summary of data coding.

Surface-Course Thickness		Test Temperature		Input Function		Load Intensity[a]	
Code	mm	Code	°C	Code	Loading	Code	kPa
1	25.4	L	10	I	Impulse	1	103
2	50.8	M	23.9	S	Static	2	206
		H	37.8	R	Repeated	3	412

Note: 1 mm = 0.04 in; t°C = (t°F - 32)/1.8; 1 kPa = 0.145 lbf/in^2.

[a]Applied to a plate 101.6 mm (4 in) in diameter, these pressures amount, respectively, to 0.8, 1.7, and 3.3 kN.

Loads were applied by the MTS machine and measured with a FLIU 4.4-kN (1000-lbf) capacity Strainsert load cell mounted between the hydraulic actuator and the loading plate. The output of the load cell was recorded with a Brush Mark 280 two-channel recorder.

Five Sanborn Linearsyn linear variable differential transformers (LVDTs) were used to sense the deflections. One LVDT was located 82.6 mm (3.25 in) from the center of the loading plate. The other four were spaced 63.5 mm (2.5 in) apart along the same radial line. The outputs were amplified and recorded by an eight-channel Sanborn recorder.

The LVDTs were fixed to the pavement surface by screwing one end of a brass extension rod into the LVDT core and the other end into a brass plate 19 x 12.7 x 3.2 mm (0.75 x 0.5 x 0.125 in) thick. The plate was then glued to the appropriate location on the pavement surface by using Eastman 910 adhesive. The LVDTs were supported by aluminum channels and Plexiglas beams connected to the top angles of the box. Figure 3 shows the overall view of the test setup inside the constant-temperature room; a closeup of the LVDTs is show in Figure 4.

Test Procedures

Impulse-load magnitudes of 0.8, 1.7, and 3.3 kN (187.5, 375.0, and 750.0 lbf) were applied for about 0.16 s. The load input and the output responses formed the data for determination of the response functions. Static compressive forces of the same magnitudes were applied and held until the deflections were almost constant with time. This was found to be of the order of a few minutes. The repeated-load tests were conducted at a rate of 15 cycles/min for 250 cycles. The data coding shown in Table 1 summarizes the tests that were performed. For example, one series of tests might be represented by the code "series 1LI1".

Figure 5. Input-load and output-deflection functions: data for series 1HI1.

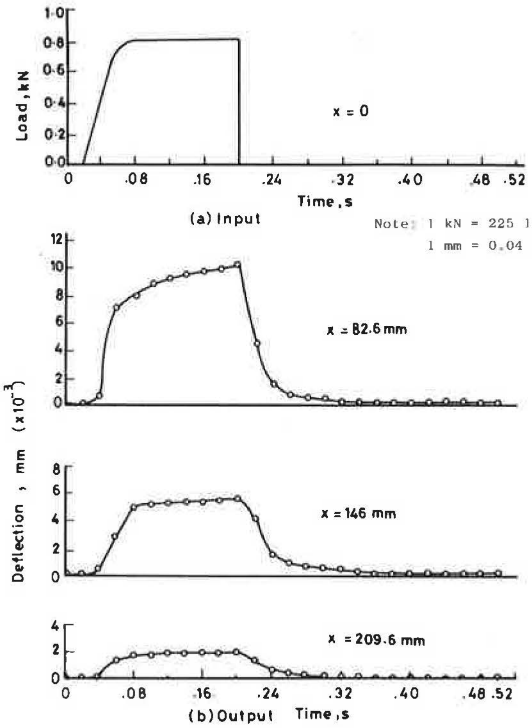


Figure 6. Input-load and output-deflection functions: data for series 1HI3.

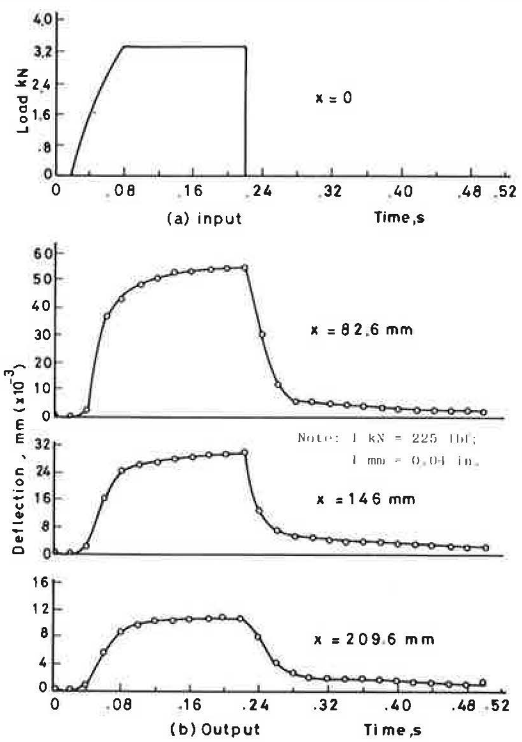


Figure 7. Response functions: data from series 1HI1.

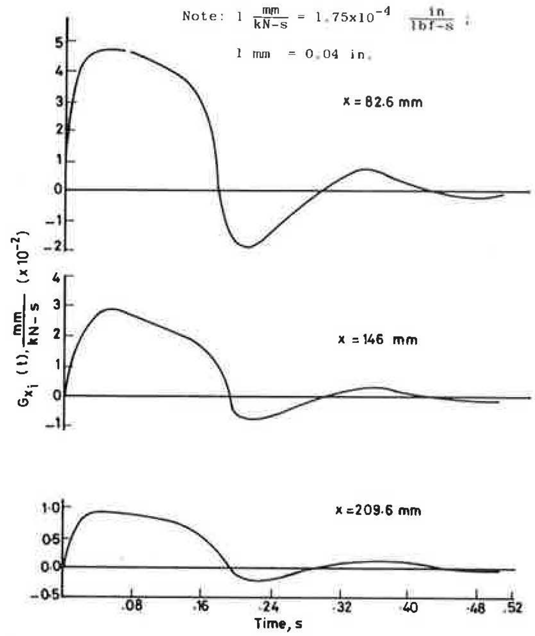
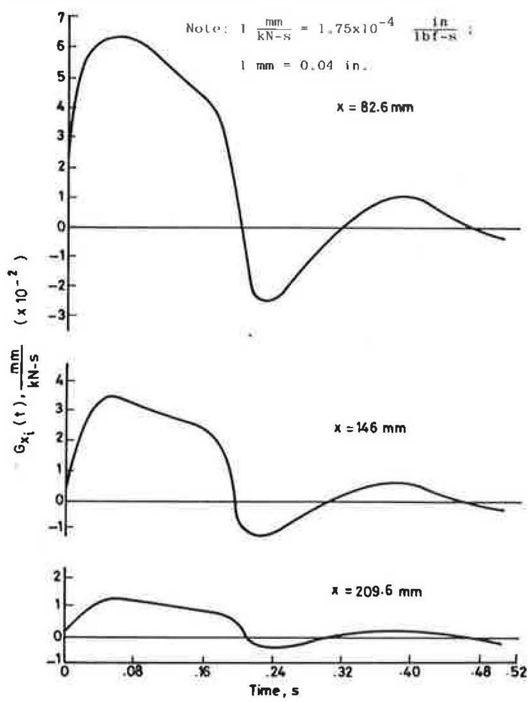


Figure 8. Response functions: data from series 1HI3.



RESULTS

Response-Function Parameters

Typical plots of the inputs and outputs from impulse tests are shown in Figures 5 and 6, and the corresponding response functions are illustrated, respectively, in Figures 7 and 8. It is seen that the response functions reach a peak and then oscillate with time. In general, the peak occurred in the time band of 0.04-0.08 s. The general shape of the response functions and the corresponding results agreed with those obtained from mechanical models (10-12). However, when mechanical models are used to describe pavement behavior, the parameters must first be defined, usually in the form of differential equations, and no reliable method now exists by which to determine them. The consistency of the response functions obtained in this study appears to validate the hypothesis that a determinable relationship exists between input and output of a pavement system.

Table 2 summarizes four response-function parameters for a surface-course thickness of 25.4 mm and a test temperature of 37.8°C (100°F). It is evident from this table that these parameters do not change significantly as the peak impulse load is changed. These results are quite similar to those found by Boyer and Harr in prototype testing of an airfield pavement (7).

Typical values of the α-, β-, and γ-parameters of Equation 6 are shown in Table 3, in which it is indicated that, in general, the parameters for a given test combination do not vary appreciably as the magnitude of load changes. In a few cases, distinct variability was evident in the α-values. It is possible that this discrepancy may be reduced if more parameters are included in the response-function model. Continuation of this research is already under way by using the following proposed equation:

$$G_{x_i}(t) = \alpha \exp(-\beta t) \sin\gamma t + \delta \cos\epsilon t \quad (11)$$

where α, β, γ, δ, and ε are parameters. Preliminary analysis indicated that the squared correlation coefficient (R^2) was improved.

Parameters α, β, and γ are measures of the peak of the response function $G_{x_i}(t)$ and the variability of $G_{x_i}(t)$ with time. As such, α can be regarded as representing the stiffness characteristics of a pavement system, whereas β and γ reflect the damping characteristics of the system. Pavement materials that have large α-values will provide less resistance to imposed loads.

From the plots of α versus temperature shown in Figure 9, it is observed that temperature plays an important role in the response function of pavement systems. Since flexible-pavement components (asphalt concrete in particular) are thermoplastic, they yield increased response for increases in temperature and hence increased α-values.

Figure 9 indicates that when x = 82.6 mm (3.25 in) and 146 mm (5.75 in) the value of α decreases with increases in surface-course thickness at all temperatures. However, at x = 209.6 mm (8.25 in) α increases slightly with increase in thickness. This trend reflects the slablike action of the system in which a thicker pavement responds more at remote locations than does a thinner one. The value of α decreases with increasing values of x; thus the attenuation of the magnitude of α with spatial location is depicted.

The interaction between surface-course thickness and temperature is exhibited by the shape of the curves in Figure 9. For the 25.4-mm surface, the bottom layers (the response of which constitutes most of the total deflection) are less affected by temperature changes than is the surface course, and hence the rate of increase in α decreases with increasing temperature (convex curves). In the case of the thicker pavement, increasing temperature raises the rate of increase in α (concave curves) due to the temperature susceptibility of asphalt concrete. Note that this trend is practically negligible at x = 209.6 mm, at which the effect of load is small.

Table 2. Comparison of response functions.

		First Positive Peak		First Negative Peak	
x (mm)	F_p (kN)	Magnitude (mm/kN·s)	Time (s)	Magnitude (mm/kN·s)	Time (s)
82.6	0.8	0.0480	0.06	-0.0190	0.20
	1.7	0.0713	0.06	-0.0300	0.22
	3.3	0.0634	0.06	-0.0250	0.22
146.0	0.8	0.0297	0.04	-0.0075	0.22
	1.7	0.0369	0.04	-0.0094	0.24
	3.3	0.0343	0.04	-0.0118	0.22
209.6	0.8	0.0094	0.06	-0.0022	0.22
	1.7	0.0125	0.06	-0.0043	0.22
	3.3	0.0128	0.06	-0.0031	0.24

Note: 1 mm = 0.04 in; 1 kN = 225 lbf; 1 mm/kN·s = 1.75 x 10^{-4} in/lbf·s.

Table 3. Descriptive parameters in the response functions at 10 and 37.8°C.

Thickness (mm)	x (mm)	F_p (kN)	α (mm/kN·s)	β (1/s)	γ (1/s)	R^2 (%)
Response Functions at 10°C						
25.4	82.6	1.7	0.035 9	10.764	13.382	85.73
		3.3	0.059 4	10.551	15.848	87.48
		Avg	0.047 6	10.658	14.615	
	146.0	1.7	0.007 8	7.127	14.593	78.09
		3.3	0.020 6	8.784	17.285	86.79
		Avg	0.014 2	7.956	15.939	
	209.6	1.7	0.002 7	6.140	14.470	75.86
		3.3	0.004 1	8.195	15.716	86.32
		Avg	0.003 4	7.168	15.093	
50.8	82.6	0.8	0.030 8	12.879	12.336	92.39
		1.7	0.020 4	9.847	12.709	87.60
		3.3	0.030 8	8.785	16.018	84.34
		Avg	0.027 3	10.504	13.688	
	146.0	0.8	0.013 8	11.302	14.222	83.88
		1.7	0.009 7	7.517	12.988	83.16
		3.3	0.020 0	7.540	16.406	84.82
		Avg	0.014 5	8.786	14.539	
	209.6	0.8	0.004 0	9.610	12.598	65.68
		1.7	0.002 9	7.409	12.996	73.44
		3.3	0.008 9	8.792	16.824	83.86
		Avg	0.005 3	8.604	14.139	
Response Functions at 37.8°C						
25.4	82.6	0.8	0.011 05	8.953	16.968	84.62
		1.7	0.014 88	7.950	15.353	83.28
		3.3	0.015 03	8.633	15.334	84.30
		Avg	0.013 65	8.512	15.885	
	146.0	0.8	0.070 6	10.044	15.383	88.75
		1.7	0.012 65	12.506	12.335	82.95
		3.3	0.085 3	9.478	14.911	81.78
		Avg	0.094 1	10.676	14.210	
	209.6	0.8	0.025 2	10.523	15.061	88.57
		1.7	0.035 1	10.951	14.452	78.24
		3.3	0.036 9	10.659	12.977	85.20
		Avg	0.032 4	10.711	14.163	
50.8	82.6	0.8	0.010 02	11.273	14.046	83.11
		1.7	0.013 29	13.187	13.870	86.90
		3.3	0.011 05	12.960	12.700	80.87
		Avg	0.011 45	12.473	13.539	
	146.0	0.8	0.062 2	9.051	14.274	87.87
		1.7	0.063 3	8.779	12.825	89.93
		3.3	0.089 5	12.068	13.983	89.15
		Avg	0.071 7	9.966	13.694	
	209.6	0.8	0.026 1	8.202	13.798	84.81
		1.7	0.042 6	11.846	11.680	92.34
		3.3	0.038 2	11.081	13.015	93.19
		Avg	0.035 6	10.376	12.831	

Note: 1 mm = 0.04 in; 1 kN = 225 lbf; 1 mm/kN·s = 1.75 x 10^{-4} in/lbf·s.

Figure 9. Curves of α-parameter versus temperature.

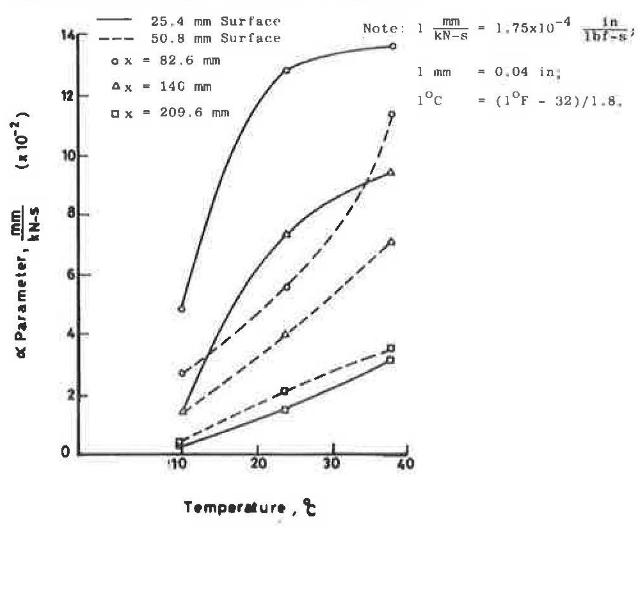


Figure 10. Typical predicted and measured static-load deflections, 25.4-mm surface, 37.8°C.

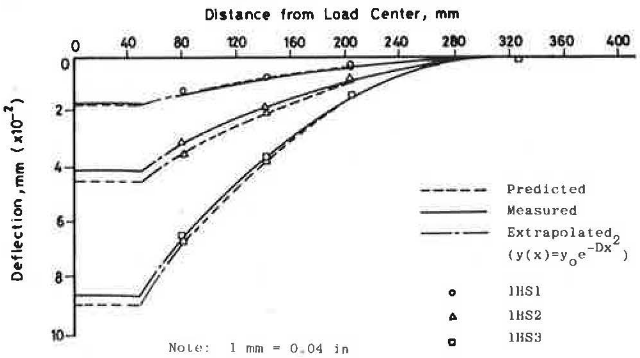
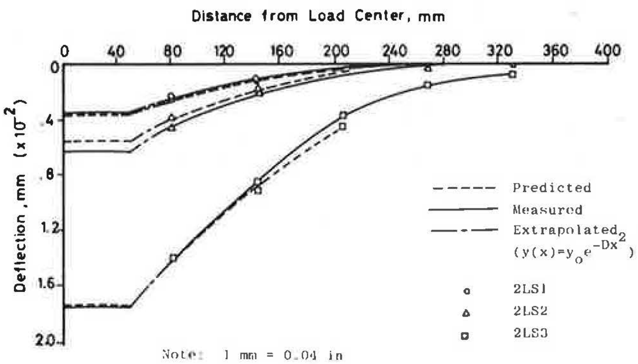


Figure 11. Typical predicted and measured static-load deflections, 50.8-mm surface, 10°C.



Examination of Table 3 reveals that changes in β and γ are small when temperature, thickness, or location change. This is possibly due to the fact that no significant change is observed in the shapes of the response-function curves. Furthermore, the slight variations in the values of these parameters are not critical, since they appear as exponential and sinusoidal functions, respectively, in the response-function model (Equation 6).

Pavement Response

Typical plots of predicted and measured deflections are shown in Figures 10 and 11 for static loads and in Figures 12 and 13 for repeated loads (total deformation). Close agreement is observed in almost all cases. The results indicate that static and repeated-load responses can be predicted by using the theory developed in this investigation.

For the application of the procedure to prototype flexible highway pavements, catalogs of response functions and pavement systems must be obtained by using, for example, a standard (or test) vehicle. Once sufficient information has been gathered, responses due to any stationary or moving vehicle can be computed by using computer programs STALOD and REPLOD, respectively.

When the deflection profiles of Figures 10-13 are plotted, the section from the innermost LVDT to the edge of the loaded plate was extrapolated by using the following equation (9, 13):

$$y(x) = y_0 \exp(-Dx^2) \qquad (12)$$

where

$y(x)$ = deflection at a distance x from the load center,
y_0 = maximum deflection of the deflected basin, and
D = a constant that reflects the attenuation of the deflected basin with x.

Figure 12. Typical predicted and measured repeated-load deflections, 25.4-mm surface, 23.9°C.

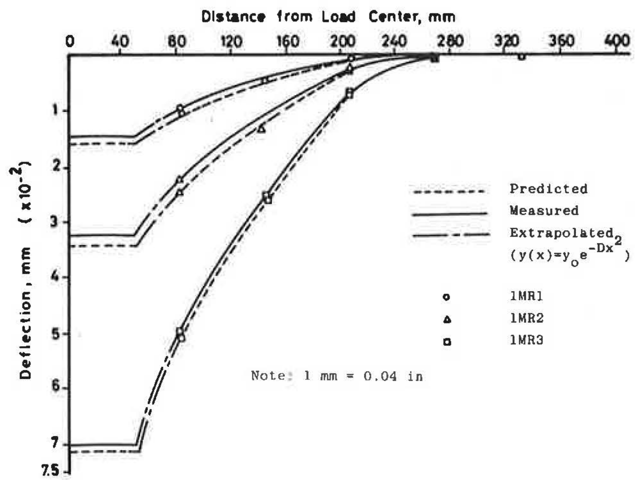
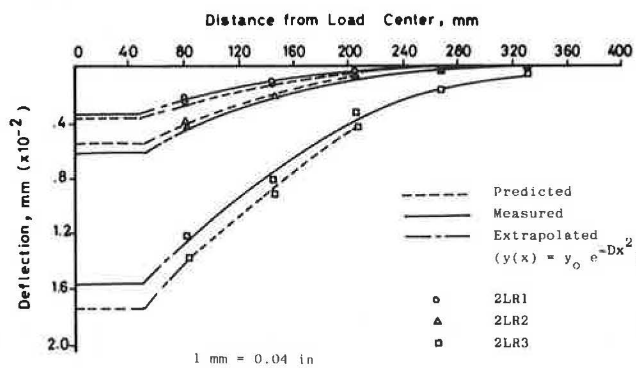


Figure 13. Typical predicted and measured repeated-load deflections, 50.8-mm surface, 10°C.



It may be observed in Figures 10-13 that the portion of the pavement surface under the loading plate was considered to have experienced constant displacement. This was a consequence of using a rigid loading plate, which was chosen to obtain a distinct trace of load duration during impulse tests.

SUMMARY AND CONCLUSIONS

Pavement parameters were determined from impulse-load tests and by using transfer functions. The derived parameters were used in conjunction with a formulated theory to predict static- and repeated-load deflections for various test conditions. Close agreement was observed between predicted and measured values.

From the results and analyses of the data and within the scope of this investigation, the following conclusions can be drawn:

1. The time-dependent behavior of a flexible pavement can be represented by a set of response functions $G_{x_i}(t)$, which are functions of time (Equation 6). It is possible to obtain these functions from impulse tests on the pavement.
2. Parameters in the response functions are believed to be descriptors of pavement behavior. These parameters are generally independent of type or magnitude of input load.
3. Temperature, surface-course thickness, and spatial location have their respective influences on the response functions. Increases in temperature increase the value of the α-parameter in the response functions, whereas

increases in surface-course thickness or in the distance from the load center decrease the α-value. The β- and γ-parameters do not seem to be affected appreciably by the above factors.

4. It is believed from the favorable agreement between calculated and measured responses that transfer-function theory appears to be capable of predicting static- or repeated-load deflections of flexible pavements.

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REFERENCES

1. C.B. Drennon and W.J. Kenis. Response of a Flexible Pavement to Repetitive and Static Loads. HRB, Highway Research Record 337, 1970, pp. 40-54.
2. T.M. Eggleston and C.W. Mathews. Application of Several Methods for Determining Transfer Functions and Frequency Response of Aircraft from Flight Data. National Advisory Committee for Aeronautics, TR-Rept. 1204, 1954, pp. 1-24. NTIS: NACA-TR-1204.
3. P.A. Crafton. Shock and Vibration in Linear Systems. Harper, New York, 1961.
4. J.H. Goldberg. Automatic Controls: Principles of Systems Dynamics. Allyn and Bacon, Boston, 1964.
5. T.K. Puchalka and A. Wozniak. Elements and Circuits for Automatic Control. Boston Technical Publishers, Cambridge, MA, 1968.
6. S.A. Swami, W.H. Goetz, and M.E. Harr. Time and Load Independent Properties of Bituminous Mixtures. HRB, Highway Research Record 313, 1970, pp. 63-78.
7. R.E. Boyer and M.E. Harr. Predicting Pavement Performance. Transportation Engineering Journal of ASCE, Vol. 100, No. TE2, May 1974, pp. 431-442.
8. R.E. Boyer. Predicting Pavement Performance Using Time-Dependent Transfer Functions. Purdue Univ. and Indiana State Highway Commission, Lafayette, IN, Joint Highway Research Proj. 32, Sept. 1972.
9. G.A. Ali. A Laboratory Investigation of the Application of Transfer Functions to Flexible Pavements. Purdue Univ., Lafayette, IN, Ph.D. thesis, Aug. 1972.
10. W. Heukelom. Analysis of Dynamic Deflections of Soils and Pavements. Geotechnique, Vol. 11, No. 3, Sept. 1961, pp. 224-243.
11. N.M. Isada. Detecting Variations in Load-Carrying Capacity of Flexible Pavements. NCHRP, Rept. 21, 1966.
12. N.M. Isada. Impulsive Load Stiffness of Flexible Pavements. Journal of the Soil Mechanics and Foundations Division of ASCE, Vol. 96, No. SM 2, March 1970, pp. 639-648.
13. G.Y. Baladi and M.E. Harr. Nondestructive Pavement Evaluation: The Deflection Beam. TRB, Transportation Research Record 666, 1978, pp. 19-26.

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Evaluation of Pavement in Florida by Using the Falling-Weight Deflectometer

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A method is presented by which mechanical properties of a pavement system can be determined by using nondestructive test methods that are now available. The ultimate goal is the establishment of rehabilitation criteria for existing flexible pavements that use purely analytical (as opposed to empirical) relationships. More specifically, the use of the falling-weight deflectometer (FWD) is discussed. Several sections of Interstate 75 in Florida were chosen in order to determine material characteristics of the pavement layers. These sections were also tested with the Dynaflect apparatus. Data developed from the FWD and Dynaflect deflections were accumulated and elastic moduli for the typical section were determined by using a computer program developed at the Florida Department of Transportation: in situ stress-dependent elastic moduli, four layers (ISSEM4). The elastic moduli were then compared with other test results, and a good correlation was indicated. How such mechanical properties may be used in an appropriate structural analysis to better locate and control distress parameters in the pavement system is outlined. Such analysis is possible from the knowledge obtained in situ of the various structural layers involved.

For many years the Florida Department of Transportation (FDOT) has used various deflection concepts to monitor both local and Interstate road networks. Generally, it has been found that deflection alone is not an adequate indicator of pavement performance or loss of serviceability. For example, many situations have been observed in which deflections remained low, even though significant load-associated pavement deterioration was visibly taking place.

Surface deflection may be interpreted as the sum of the vertical strains throughout each structural layer below. If a weakness should develop in one or more of these layers, it may not necessarily change the total deflection significantly; e.g., a relatively thin layer might contribute little change to the center measurement of a deflection-measuring device. A more-indicative measure of distress is thus necessary.

In order to further understand and evaluate pavement deterioration and ultimately recommend corrective rehabilitation and management strategies, it was decided to try to isolate problem areas in terms of which layer or layers were instrumental in the deterioration of Interstate 75 in northern Florida. Also, it was hoped that performance criteria based on derived material properties could be developed.

On the basis of work done in Europe, the falling-weight deflectometer (FWD) was chosen to carry out a layered-system (mechanistic) analysis of the pavement structure. Approximately 180 lane-km (110 lane-miles) of Interstate were tested and analyzed (see Figure 1).

FWD AND ASSOCIATED EQUIPMENT

Use of the FWD has been well documented elsewhere (1-3). Briefly, the basic idea behind the development of the FWD