Comparative Study of Selected Nondestructive Testing Devices

MARIO S. HOFFMAN AND MARSHALL R. THOMPSON

An extensive program of flexible-pavement nondestructive testing (NDT) was conducted by the Illinois Department of Transportation (IDOT) in cooperation with the University of Illinois. Different (mostly in-service) pavements were tested. Conventional granular base and stabilized base material sections were studied. NDT devices used were the IDOT road rater model 2008, the Benkelman beam, and the falling-weight deflectometer. An accelerometer was used to measure surface pavement deflections under moving trucks. The main findings of the comparative NDT program are reported. Deflection-basin parameters for structural pavement evaluation are defined. Correlations and comparisons between the different devices are presented. Overall, the falling-weight deflectometer is the best NDT device for simulating pavement response under moving wheel loads. The road rater, because of its harmonic loading without rest periods and the static preload, induces pavement deflections lower than those achieved with the falling-weight deflectometer and moving wheel loads. Finally, the quasistatic loading in Benkelman beam measurements induces the highest pavement deflections.

There is general agreement among pavement engineers and researchers that pavement surface deflection-basin measurements provide valuable information for structural evaluation. Pavement deflections, however, are highly dependent on loading mode (vibratory, impulse, vehicular) and magnitude. The ideal responses for structural pavement evaluation are thus surface deflections under moving design loads. One of the main goals of this study was to determine the nondestructive testing (NDT) equipment and procedure that best simulate pavement response under moving loads.

With that goal in mind, the Illinois Department of Transportation (IDOT) in cooperation with the University of Illinois has developed an extensive flexible-pavement evaluation method based on the interpretation of measured surface deflections (1,2). The first stage of the program included the collection of 11 000 deflection measurements over a period of three years on different (mostly in-service) pavements.

Throughout the deflection data collection, different selected NDT devices and methods were used. These included the Benkelman beam (BB), IDOT road rater (RR), falling-weight deflectometer (FWD), and an accelerometer implanted in the pavement's surface to measure deflections under different moving trucks at varying speeds.

The different types of pavements tested in the program (see Table 1) included the following:

1. Conventional flexible pavements: asphalt-concrete (AC) surface over granular bases and subbases;
2. Stabilized pavements: AC surface over a stabilized base including cement aggregate mixture (CAM), pozzolanic aggregate mixture (PAM), and bitumen aggregate mixture (BAM);
3. Surface treatments: a nominal asphalt and chips covering over a granular base; and
4. Test sections: selected flexible sections in Loop 1 of the American Association of State Highway Officials (AASHO) Test Road in Ottawa, Illinois.

Except for the sections on Loop 1 of the AASHO Test Road, all the sections are in-service pavements. The effects of the following factors on surface deflections were investigated: RR load and driving frequency, FWD load, seasonal effects, vehicle weight and speed, and loading mode--quasistatic (BB), steady-state (RR), impulse (FWD), and vehicular (moving trucks). In addition to the deflection data collection, most pavement sections in the program were sampled for subsequent laboratory testing.

This paper presents the main findings of the comparative study of selected NDT devices. A detailed description of this study and a full summary of the results have been presented elsewhere (1). An extensive literature review of other deflection studies has also been given elsewhere (1,2).

NDT EQUIPMENT AND PROCEDURES

To facilitate the direct comparison of the different NDT devices, soil type and existing pavement conditions were kept uniform. Measurements were made on 20 points in each 100-ft-long section of pavement. The same 20 points (10 on each traffic lane, 10 ft apart) were tested at different times during a three-year period.

Benkelman Beam

A truck that had an 18-kip rear axle with dual tires (70-80 psi tire pressure) was used in the BB rebound method. The rebound deflection of the pavement was measured when the truck moved away from the testing point at creep speed.

Road Rater

The RR (model 2008) used in the study is an electrohydraulic vibrator capable of generating harmonic loads of up to 8 kips (peak to peak) at driving frequencies between 6 and 60 Hz. When the vibrator is set over the testing point, a static preload of 5 kips is applied through the 12-in-diameter circular loading plate. The desired peak-to-peak load is then generated at the preselected driving frequency, and peak-to-peak deflections are recorded with velocity transducers (geophones). The IDOT RR has four deflection sensors, located at the center of

Table 1. Description of pavement sections.

Section	Cross Section	Subgrade Classification (AASHTO)
Bement	Asphalt concrete, 4 in; field-mixed soil cement, 6 in	A-7-6(24)
Coffeen	Asphalt concrete, 3 in; lime-fly ash, 10 in	A-4(5)
Deland	Surface treatment; granular base B, 8 in	A-7-6(21)
Hillsboro	Asphalt concrete, 6.3 in; seal treatment, 2 in; crushed gravel, 6 in	A-7-6(18)
Midlothian "A"	Asphalt concrete, 5 in; lime-fly ash, 6 in; gravel, 10 in	A-6(14)
Midlothian "B"	Asphalt concrete, 7 in; gravel, 7 in; lime-fly ash, 6 in	A-6(10)
Monticello	Asphalt concrete, 3.5 in; plant-mixed CAM, 8 in	A-6(8)
Neoga North	Asphalt concrete, 5.5 in; BAM, 7 in	A-7-6(18)
Neoga South	Asphalt concrete, 2.5 in; BAM, 7 in	A-6(8)
Pana	Asphalt concrete, 5.5 in; BAM (MC-800), 10 in	A-7-6(16)
Sherrard	Asphalt concrete, 4 in; crushed stone, 14 in	A-4(6)
Viola	BAM (HFE-300), 9 in	A-6(9)
AASHO-845	Asphalt concrete, 3 in; crushed stone, 6 in; sandy gravel, 8 in	A-6(6)
AASHO-872	Asphalt concrete, 5 in; crushed stone, 6 in	A-6(6)
AASHO-874	Asphalt concrete, 5 in; crushed stone, 6 in; sandy gravel, 16 in	A-6(6)

the loading plate and 1, 2, and 3 ft away from the center.

Falling-Weight Deflectometer

The FWD is a deflection-testing device operating on the impulse-loading principle. A mass is dropped from a preselected height onto a footplate that is connected to a baseplate by a set of springs. The baseplate (12 in in diameter) is placed in contact with the pavement surface over the testing point. By varying the drop height, the impulse load was varied from 2 to 11 kips. The duration of the impulse loading ranges from 30 to 40 ms.

FWD deflections are measured with velocity transducers (geophones). One of these sensors is located at the center of the loading plate. Two additional sensors are movable and can be placed at any desired distance away from the center of the plate. During this testing program, the FWD sensors were placed 1, 2, and 3 ft away from the center of the loading plate, the same spacing used for the RR.

Figure 1. Deflection-basin characterization.

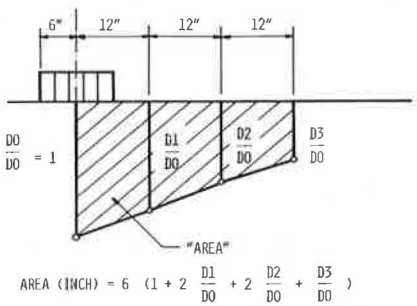


Figure 2. BB versus RR coefficients of variability.

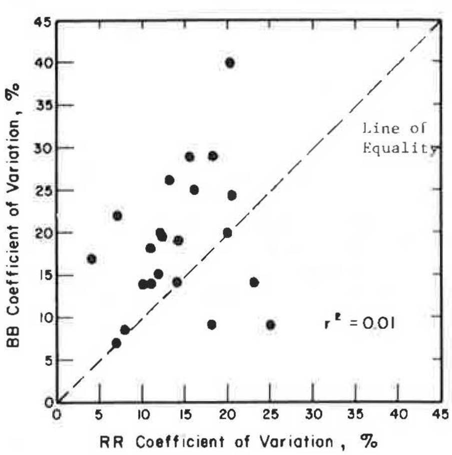


Table 2. Variables in stepwise multiple-regression analysis of BB and RR deflections.

Variable Considered	Range of Values Max	Range of Values Min
BB: Benkelman-beam deflection (mils)	194.0	8.0
RR: road rater deflection[a] (mils)	99.0	5,8
F1: RR shape factor 1	1.68	0.15
F2: RR shape factor 2	1.83	0.20
Area: RR deflection-basin area (in)	31.4	15.7
T: surface pavement temperature (°F)	115	64
Tac: thickness of AC layer (in)	8.0	0.0
Tgr: thickness of granular layer (in)	14.0	0.0
Tsta: thickness of stabilized layer (in)	10.0	0.0
Sta: Flag = 1 if stabilized layer in section; 0 if not		
Seas: Flag = 1 if spring measurement; 0 if other		

[a] 8-kip peak-to-peak load; 15-Hz driving frequency.

Accelerometer Measurements

An accelerometer was implanted in the surface of selected AASHO Test Road (Loop 1) sections (sections 845, 872, and 874) to measure deflections under moving trucks and under the FWD's loading plate. The accelerometer was placed in a hole 2 in in diameter by 2 in deep in the outer wheelpath. The single wire coming off the accelerometer was buried in a slot 1 in deep and 3/8 in wide sawed perpendicular to the direction of travel.

The following trucks were used in the testing:

Truck	Rear-Axle Weight (lb)
Light	5 100
Medium	9 000
Heavy	18 000

The trucks distributed the rear-axle weight shown through a single-axle, dual-wheel configuration. Truck speeds ranged from 8 to 30 mph.

DEFLECTION-BASIN CHARACTERIZATION

The deflection basin, measured with the RR and FWD, is characterized as follows:

1. D0, centerline plate deflection;
2. D1, D2, D3, surface deflections at 1, 2, and 3 ft, respectively;
3. Deflection-basin area, parameter combining all measured deflections in basin (see Figure 1), defined as Area (in) = 6 (1 + 2D1/D0 + 2D2/D0 + D3/D0); and
4. F1, F2, basin shape factors (dimensionless), F1 = (D0 - D2)/D1 and F2 = (D1 - D3)/D2.

The deflection-basin area ranges from a calculated practical minimum of 11 in (Boussinesq approximation) to a maximum of 36 in (maximum by definition). Also, the area increases with increasing pavement stiffness (2). The basin shape factors are analogous to a derivative of the deflection-basin curves, representing the variation of surface deflection with lateral distance from the centerline. In general, stiff pavements have lower shape factors (1).

RESULTS

Comparison of BB and RR Deflections

The comparative study between BB and RR deflections was performed on 12 different in-service pavement sections. Nine of the sections were tested twice at different times of the year for a total of 21 cases. The RR followed the BB at routine RR testing conditions of 8-kip peak-to-peak load and 15-Hz driving frequency. The same 20 points per section were evaluated at each testing date. The following results were obtained:

1. Without exception, mean BB deflections were higher than mean RR deflections; the ratio of mean BB and mean RR deflections ranged from 1.1 to 5.8.
2. The variability of the BB deflections was generally larger than that of the RR (18 of 21 cases). The mean coefficient of variation with the BB data was 19 percent and that of the RR data was 14 percent (Figure 2). There is no linear correlation between the BB and RR coefficients of variation

Table 3. Analyses of BB and RR deflections: stepwise regressions.

							Other Variables									
Group	N	Dependent Variable	R^{2a}	SEE (mils)	CV (%)	Constant a_0	RR a_1	F1 a_2	F2 a_3	Area a_4	T a_5	Tac a_6	Tgr a_7	Tsta a_8	Sta a_9	Seas a_{10}
All data	418	BB	0.78	20.2	39	153	-0.47	-	84.2	-	0.43	-	-5.6	-3.2	-42.7	-20.2
Stabilized sections	200	BB	0.72	3.3	17	113	0.90	-36.5	-	-2.9	-	-	-	-1.29	-	-
Remainder of data	218	BB	0.66	23.5	29	680	-0.44	-156.4	66.3	-22.9	0.53	4.15	-7.15	-	-	-15.4

Note: Equation of the following form: $BB = a_0 + a_1 RR + a_2 F1 + \ldots + a_{10}$ Seas.
[a]Significant at 1 percent level.

Table 4. BB versus RR deflections.

Group	N	R^{2a}	SEE (mils)	CV (%)	Regression Equation
All data	418	0.38	33.7	65	BB = 14.3 + 1.53 RR
Stabilized sections	200	0.66	3.7	19	BB = 2.6 + 1.27 RR
Remainder of data	218	0.07	38.5	47	BB = 61.7 + 0.58 RR

[a]Significant at 1 percent level.

for a given section (Figure 2).

3. If we assume that the inherent variability is constant for a given section, BB testing errors must be greater than RR testing errors.

Different correlations between BB and RR deflections were attempted in the study. A section-by-section linear correlation of BB versus RR deflections (20 points per section) resulted in coefficients of determination (R^2) ranging from 0.05 to 0.85. In 12 of 21 cases, the R^2-values were significant at the 1 percent level. In three cases, R^2-values were significant at the 5 percent level, and in the remaining six cases there was no significant linear correlation between BB and RR deflections.

The addition of other variables permitted the use of stepwise multiple-regression analyses (see Table 2). Deflection data were combined in different groups: (a) all data (418 pairs of observations), (b) stabilized section (200 pairs), and (c) remainder of data (218 pairs). Tables 3 and 4 summarize the regression equations developed in the study for the different groups. While the coefficients of determination (R^2) of all three groups are significant at the 1 percent level, only the stabilized-section group has a standard error of estimate (SEE) acceptable for predictive purposes.

Overall, it does not seem that BB deflections can be reliably predicted from RR deflections. The study (1) concluded that the loading-mode effects of the BB (quasistatic) and the RR (vibratory with static preload) deflections are not predictable based solely on statistical models. A treatment of loading-mode effects on pavement response, far beyond the scope of this paper, has been given elsewhere (2).

Comparison of RR and FWD Deflections

Two types of tests were performed in the comparative studies of RR and FWD deflections: (a) routine and (b) load and frequency sweeps. In the routine test, the RR was operated at an 8-kip peak-to-peak load and 15 Hz, and the FWD was operated at 8 kips. The FWD followed the RR over the preselected 19 stations per test section (one station had been eliminated for coring and sampling). In the load and frequency-sweep tests, the RR was operated at peak-to-peak loads of 2, 4, 6, and 8 kips and driving frequencies between 6 and 30 Hz at 2-Hz intervals for each load. Following the RR, the FWD was operated at loads ranging from 2 to about 11 kips.

Routine RR and FWD Test Results

Table 5 summarizes the results of the RR and FWD routine tests for the five sections in the study. The results show the following:

1. Mean RR and FWD centerplate deflections (D0) are different. The hypotheses that the mean deflections are equal are rejected in all five cases (95 percent confidence level).
2. The mean FWD deflection-basin areas are statistically different (α = 0.05) and consistently lower than the RR areas.
3. The mean basin shape factors (F1 and F2) are statistically different (α = 0.05) and consistently larger for the FWD.
4. The FWD variability, expressed by the coefficient of variation (CV), is larger than the RR variability of most values in Tables 3 and 4.

Considering that both devices applied an 8-kip load, both loading plates were 12 in in diameter, and deflections were measured with geophones at the same basin locations, then the differences in Table 4 can be mainly attributed to the loading-mode effects: vibratory with the RR and impulse loading with the FWD.

Correlations Between RR and FWD

Despite the difference in surface deflections between RR and FWD, they were highly correlated. Figure 3 shows the correlation between FWD and RR centerplate deflections (D0), and Figure 4 shows the correlation between FWD and RR deflection-basin areas.

The regression equation relating FWD and RR deflections is as follows:

$$\Delta_{FWD}\ (\text{mils}) = -3.40 + 1.21\ \Delta_{RR}\ (\text{mils}) \tag{1}$$

The coefficient of determination (R^2) is 0.94 (significant at the 1 percent level), and SEE is 3.23 mils. Note that up to a 15-mil deflection, RR deflections are larger than FWD deflections. Above the 15-mil RR deflections, the FWD induced larger deflections than the RR did (Figure 3).

The regression equation relating FWD and RR deflection-basin areas is as follows:

$$\text{Area}_{FWD}\ (\text{in}) = -7.59 + 1.19\text{area}_{RR}\ (\text{in}) \tag{2}$$

Table 5. Summary of FWD and RR deflections.

Section	Date	Pavement Temperature (°F)	Measurement Device	D0 Deflection (mils)	D0 SD (mils)	D0 CV (%)	D1 Deflection (mils)	D1 SD (mils)	D1 CV (%)	D2 Deflection (mils)	D2 SD (mils)	D2 CV (%)	D3 Deflection (mils)	D3 SD (mils)	D3 CV (%)
Bement	10/17/79	83	RR	14.92	2.59	17.40	11.91	1.66	13.90	9.51	0.91	9.60	7.61	0.70	9.20
			FWD	12.93	2.91	22.50	9.96	1.66	16.70	7.20	0.77	10.60	5.22	0.45	8.60
Deland	10/17/79	67	RR	40.50	3.53	8.70	24.08	2.25	9.30	14.45	0.76	5.30	11.16	0.50	4.50
			FWD	43.47	5.34	12.30	19.07	2.98	15.60	7.55	0.24	3.20	5.33	0.19	3.50
Monticello	10/17/79	60	RR	14.39	2.55	17.70	11.39	1.75	15.30	8.87	0.79	8.90	7.25	0.42	5.80
			FWD	12.04	3.00	24.90	8.74	1.52	17.40	6.16	0.72	11.70	4.29	0.31	7.30
Sherrard	10/19/79	83	RR	16.24	0.61	3.70	11.61	0.57	4.90	7.71	0.30	3.90	4.45	0.23	4.20
			FWD	17.85	0.77	4.30	10.69	0.53	5.00	6.20	0.38	6.20	4.18	0.15	3.70
Viola	10/19/79	65	RR	28.21	3.92	13.90	14.11	2.11	14.90	8.00	1.37	17.20	6.16	1.12	18.20
			FWD	34.64	4.37	12.60	13.18	2.03	15.40	5.72	0.92	16.00	3.77	0.80	21.10

Section	Date	Pavement Temperature (°F)	Measurement Device	Shape Factor F1 Value	F1 SD	F1 CV (%)	Shape Factor F2 Value	F2 SD	F2 CV (%)	Area Deflection tion Basin (in)	Area SD (in)	Area CV (%)
Bement	10/17/79	83	RR	0.44	0.10	22.10	0.45	0.12	26.30	26.53	1.58	5.90
			FWD	0.56	0.16	29.30	0.65	0.18	28.40	24.81	2.34	9.50
Deland	10/17/79	67	RR	1.08	0.05	4.90	0.89	0.13	14.80	19.10	0.46	2.40
			FWD	1.89	0.15	7.70	1.82	0.41	22.50	14.12	0.55	3.90
Monticello	10/17/79	60	RR	0.47	0.13	26.70	0.46	0.15	31.60	26.18	1.91	7.30
			FWD	0.65	0.17	26.10	0.71	0.14	19.70	23.46	2.15	9.20
Sherrard	10/19/79	83	RR	0.74	0.06	8.00	0.80	0.07	8.60	22.30	0.70	3.10
			FWD	1.09	0.12	11.10	1.05	0.07	7.00	18.78	0.88	4.70
Viola	10/19/79	65	RR	1.44	0.14	9.50	1.01	0.20	20.30	16.72	0.76	4.50
			FWD	2.23	0.36	16.30	1.64	0.16	9.80	13.22	0.95	7.20

Notes: RR at 8000-lb peak-to-peak load, 15 Hz, 19 stations per section.
FWD at 8000-lb ±5 percent, 19 stations per section.

Figure 3. Correlation between FWD and RR deflections.

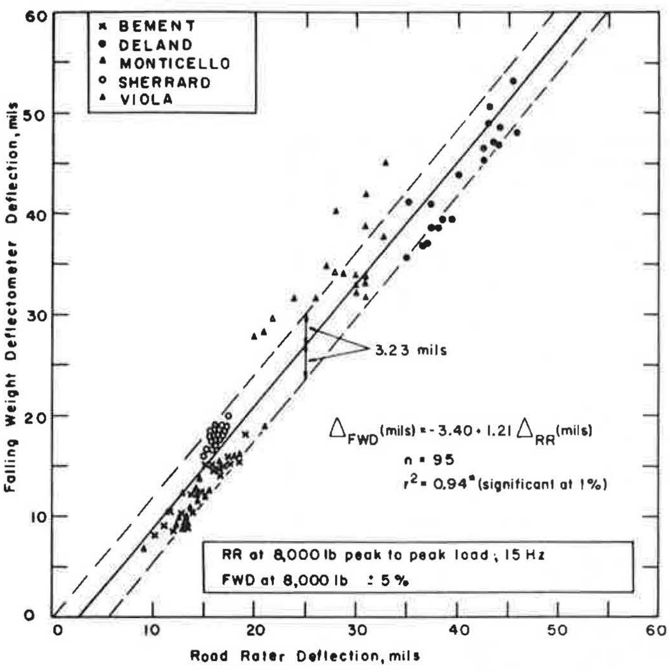
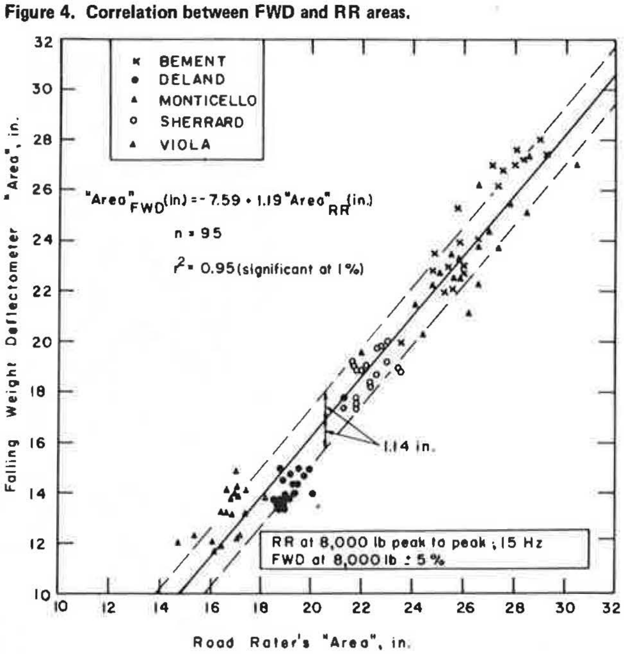


Figure 4. Correlation between FWD and RR areas.



where the coefficient of determination is 0.95 and SEE is 1.14 in (Figure 4).

From the point of view of evaluation of structural pavement, pavements are stiffer when evaluated from RR data. The stiffening effect of the RR can be attributed to the static RR preload and the harmonic loading without rest periods (2). A summary of regression relations between FWD and RR deflection-basin parameters is given in Table 6.

RR and FWD Load and Frequency-Sweep Tests

Figure 5 shows results of typical RR and FWD load and frequency-sweep tests. The left-hand side of Figure 5 shows the variation of centerplate RR deflections with load and driving frequency. The right-hand side shows the variation of RR and FWD centerplate deflections with load. Numerous data similar to those in Figure 5 were gathered in the study; they are summarized elsewhere (1).

Figure 5 shows that

1. The different sections show a distinct RR peak deflection at different driving frequencies;
2. For a given section, the driving frequency corresponding to the peak RR deflection is the same for all peak-to-peak loads;
3. For the Monticello section, RR and FWD center-plate deflections agree at a driving frequency of 22-24 Hz for all load magnitudes; and
4. For the Sherrard and Deland sections, the FWD induces larger deflections than the RR at all loads and driving frequencies.

Table 6. Correlations between FWD and RR deflections.

Dependent FWD Variable	A	B	R^2 [a]	SEE	Mean FWD Value	Mean RR Value
D0 (mils)	-3.40	1.21	0.94	3.23	24.19	22.85
D1 (mils)	1.68	0.72	0.92	1.13	12.24	14.62
D2 (mils)	3.98	0.27	0.54	0.64	6.57	9.71
D3 (mils)	2.69	0.25	0.48	0.55	4.56	7.52
Area (in)	-7.59	1.19	0.95	1.14	18.88	22.17
F1	-0.15	1.73	0.93	0.19	1.29	0.84
F2	0.03	1.57	0.72	0.26	1.16	0.72

Notes: FWD (variable) = A + B x RR (variable), Area = 6(D0 + 2D1/D0 + 2D2/D0 + D3), F1 = (D0 - D2)/D1, F2 = (D1 - D3)/D2.
RR at 8-kip peak-to-peak load and 15-Hz driving frequency; FWD at 8 kips ±5 percent; N = 95.

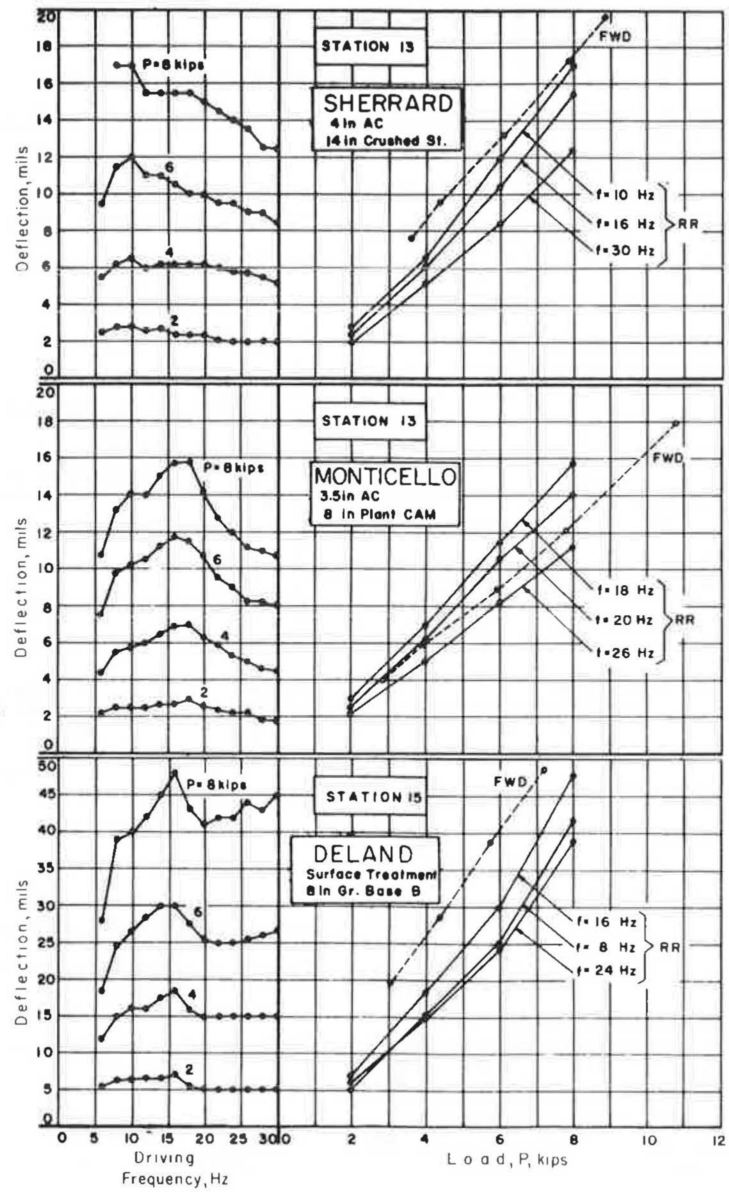
[a]Significant at 1 percent level.

To investigate the erratic response of different pavements to RR and FWD load and frequency tests, it is necessary to use pavement models capable of incorporating time or rate-of-loading variables. Hoffman used dynamic and viscous pavement models to that effect (3). That study concluded that it is theoretically difficult to predict pavement response under one loading mode based on the response measured under a different loading mode, i.e., impulse response (FWD) from vibratory response (RR). The main reason for that discrepancy is that the derived pavement parameters are dependent on loading mode and device (3).

Accelerometer Test Results

An accelerometer was used to check the FWD data-ac-

Figure 5. RR versus FWD deflections.



quisition system and generate deflection data under moving trucks at varying speeds. All tests were performed on selected AASHO Test Road sections (Loop 1).

The simultaneous measurement of FWD deflections with the accelerometer and the FWD centerplate sensor showed almost identical results (Figure 6). The agreement indicates that both measuring techniques provide reliable results.

Accelerometer outputs were used to generate acceleration, velocity, and deflection signals under moving trucks (Figure 7) and blows of the FWD (Figure 8). Note in Figures 7 and 8 the vertical and horizontal scales for signal amplitude and time duration, respectively. From additional data similar to Figures 7 and 8, the study (1) concluded that (a) truck signals have a longer duration than FWD signals; typical truck "pulse" durations at 50 mph were estimated at 120 ms whereas FWD pulses are of the order of 30 ms; and (b) as an approximation, truck signals start at the edge of the deflection basin zone of influence. Thus, the stiffer the pavement, the longer the equivalent truck pulse duration.

Pulse duration is relevant to the theoretical treatment of pavement response under different loading modes (3). Bohn and others (4) performed

Figure 6. FWD versus accelerometer deflections.

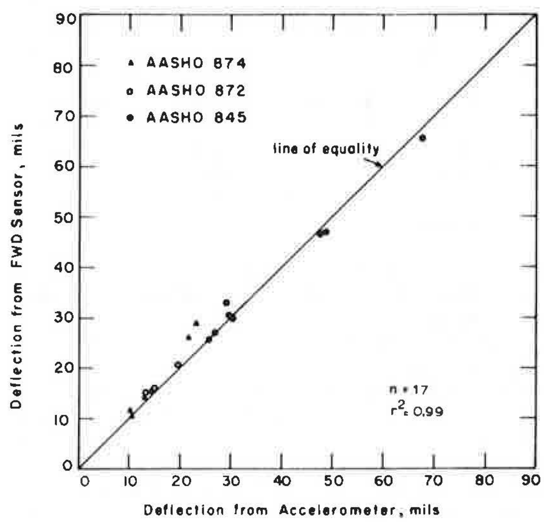


Figure 7. Acceleration, velocity, and deflection signals under moving trucks.

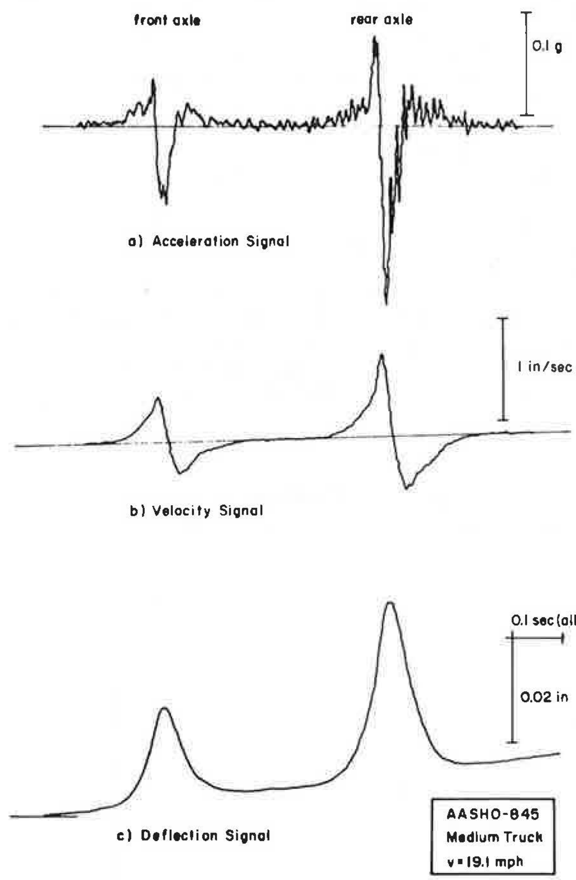
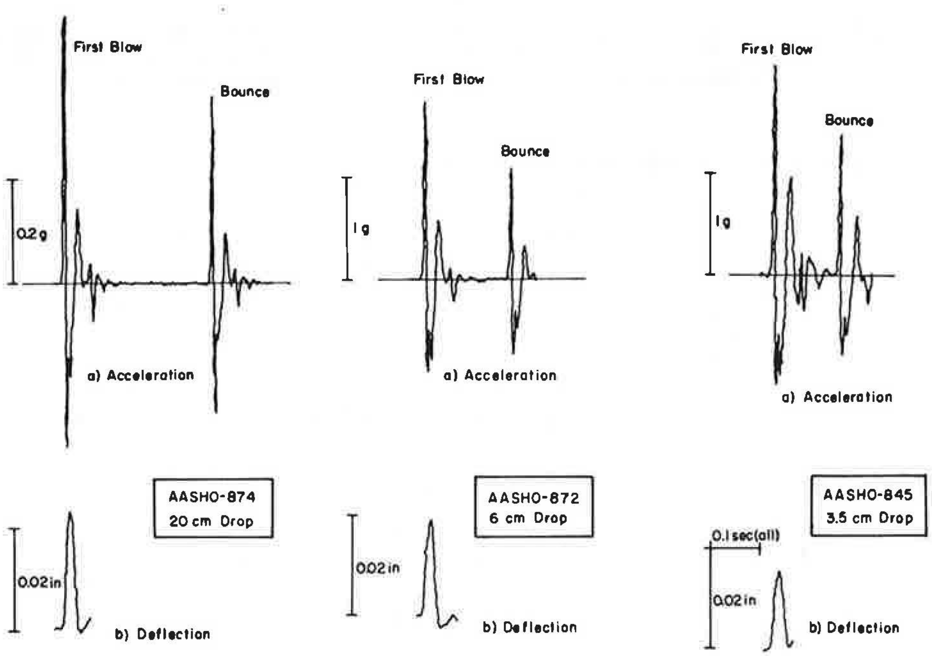


Figure 8. Typical FWD acceleration and deflection signals.

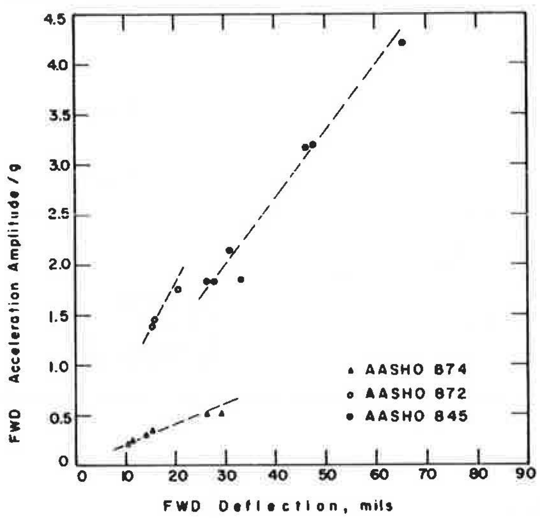


numerous moving-truck and FWD deflection measurements, and they also concluded that moving-truck signals were much longer than FWD deflection signals.

Figure 9 shows the relationship between the ground acceleration amplitude g and centerplate deflection caused by FWD blows determined from accelerometer measurements. From Figure 9 it is observed that (a) FWD-imposed ground accelerations can reach values of up to 4 times g and (b) there are different relationships between acceleration and deflection for different sections. The first observation suggests that inertia effects under FWD blows should be significant and may need to be included in theoretical analyses (3). The study also concluded that vehicle-imposed accelerations were approximately one-tenth of the FWD accelerations. The discrepancy between FWD and vehicle-imposed accelerations suggests that a fixed-in-place NDT device cannot simulate the loading effect of a moving load. However, FWD and moving-truck deflections in this study and those reported by Bohn and others (4) compared favorably.

One explanation for the different relationships between acceleration and deflection amplitudes (Figure 9) is that FWD pulse duration changes for different pavements. From accelerometer signals FWD pulse durations were estimated to range between 24 and 56 ms (2).

Figure 9. FWD acceleration versus FWD deflection.



Deflections Under Moving Trucks

Figure 10 shows a typical relationship between deflection and vehicle speed for the AASHO Test Road sections tested in the study. Truck speed ranged from 8 to 30 mph.

Figure 10 shows no effect of vehicle speed on surface deflections. In fact, none of the nine deflection-speed relationships developed in the study (three per section per truck) showed a regression coefficient significantly different from zero. In other words, the mean deflection is indicative of the speed effect.

No generalizations are attempted based on the limited speed-effect data collected in the study. Bohn and others (4) performed speed-deflection studies at speeds between 6 and 38 mph (18-kip rear-axle truck) and reported no effect of speed on surface deflections. Other studies (5) reported a deflection decrease of 35 percent between creep and 40 mph. It is apparent that most of the deflection decrease with increasing speed takes place between creep speed and about 8-10 mph (1).

Comparison of Moving-Truck, RR, and FWD Deflections

Figure 11 shows the deflection-load relationship for moving trucks, the IDOT RR, and the FWD for the AASHO Test Road sections in the study. Average moving-truck deflections are shown since speed did not have a significant effect on deflections (see Figure 10). RR deflections are shown for the peak deflection frequency and the lowest deflection frequency. The following conclusions were drawn from these studies:

1. Peak RR deflections were consistently the lowest. At other frequencies RR deflections were even lower. At an 8-kip load level, RR deflections were about 25 percent lower than moving-truck deflections (average).
2. Moving-truck and FWD deflections were the closest. In two test sections, FWD deflections were slightly higher than moving-truck deflections. In the remaining section, moving-truck deflections were higher than FWD deflections. On the average, moving-truck and FWD deflections were in close agreement. Agreement between moving-truck and FWD deflections had been reported earlier (4).

The lower RR deflections can be attributed to the vibratory RR loading (without rest periods) and the RR's static preload. This loading mode produces a

Figure 10. Variation of moving-truck deflections with vehicle speed.

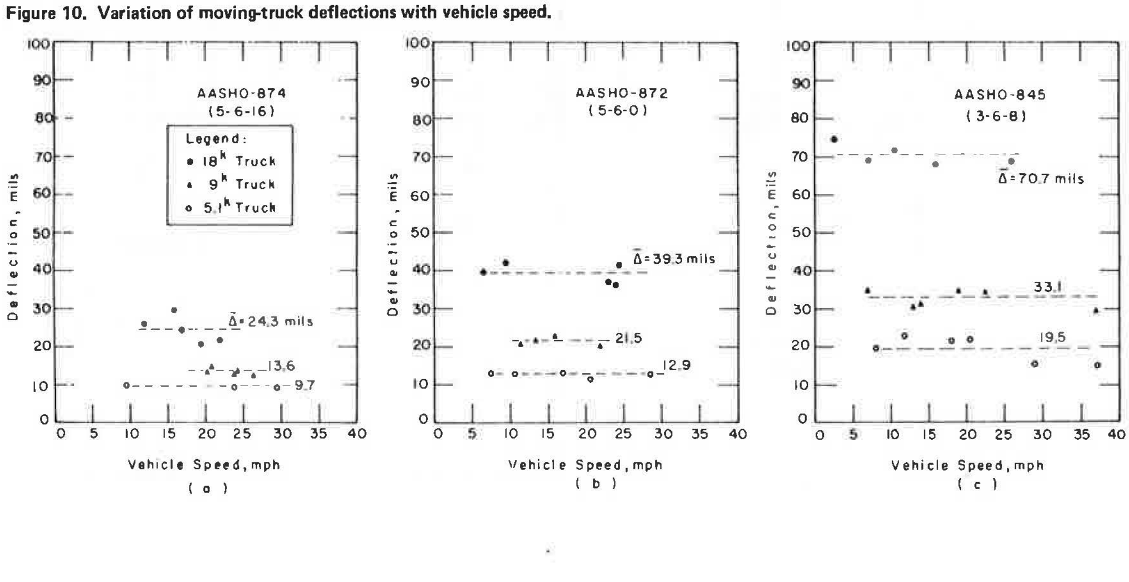
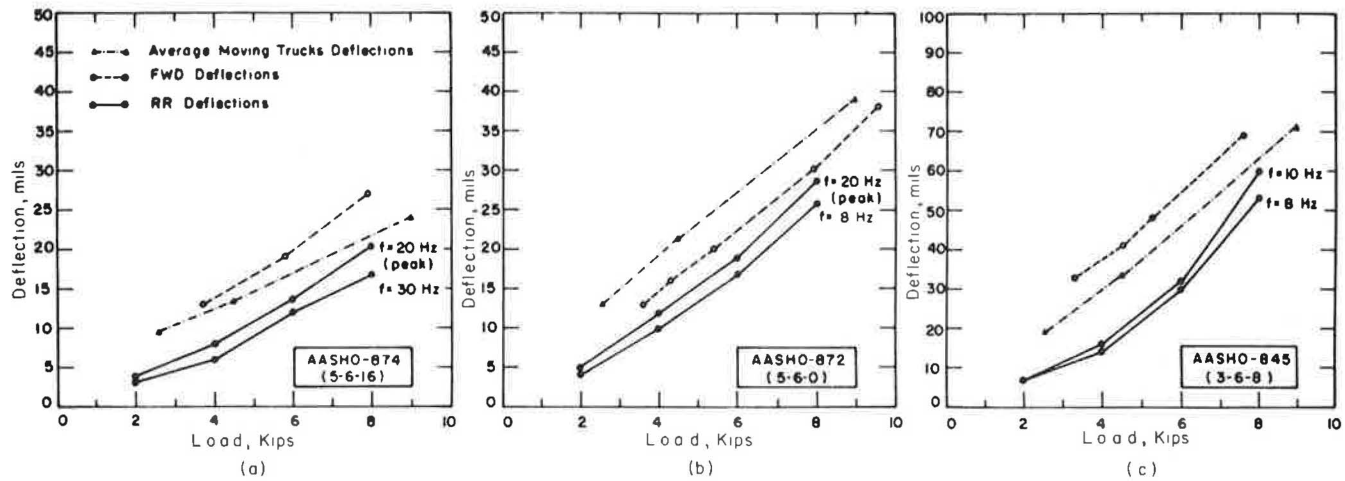


Figure 11. Variation of moving-truck, RR, and FWD deflections with load.



stiffening effect and thus lower pavement deflections. The study concluded that the FWD is the best NDT device to simulate pavement response (deflections) under moving loads.

Influence of Load on Surface Deflections

The influence of load on surface deflections was investigated with the RR and the FWD. RR loads ranged from 2 to 8 kips peak to peak and FWD loads varied from 2 to 11 kips. Load effects were analyzed by using the pavement-stiffness concept. Stiffness is defined as the load needed to cause a unit deflection (spring constant). The main findings of the study are as follows (Tables 7 and 8):

1. Most pavements showed a decrease in stiffness with increasing load (RR and FWD). A decrease in stiffness with increasing load means that the pavements soften (deflections increase faster than the load).
2. The ratio between the maximum and minimum pavement stiffness ranged from 1.0 to 2.5 (most values between 1.2 and 1.6) with the RR and from 1.03 to 1.26 (most values between 1.1 and 1.2) with the FWD. RR stiffness values were computed at the peak deflection frequency.
3. The deflection-basin area is less sensitive to load than centerplate deflections. RR (at a given driving frequency) and FWD areas varied within 10 percent for the range of loads considered. The deflection-basin area is thus a strong characterizing pavement parameter.

Influence of Driving Frequency on RR Deflections

The effect of driving frequency on RR deflections was investigated in frequency-sweep tests for driving frequencies varying from 6 to 30 Hz at 2-Hz intervals. The study indicated the following points:

1. Different sections show one distinct peak deflection at different driving frequencies (see Figure 5).
2. For a given section, the driving frequency at the peak deflection is roughly the same at different peak-to-peak loads.
3. The ratio between the peak deflection and the minimum deflection in the sweeps ranged from 1.3 to 3.9. Rigid pavements are more susceptible to driving frequency than are conventional flexible pavements.
4. The deflection-basin area reaches a peak at the same driving frequency as the peak deflections.

Table 7. Influence of load magnitude on RR deflections.

Section	Driving Frequency (Hz)	Stiffness at Peak-to-Peak Load Shown (kips/in) 2 Kips	4 Kips	6 Kips	8 Kips	Stiffness max / Stiffness min
Bement[a]	18	625	558	547	508	1.23
Deland[a]	16	296	245	224	197	1.50
Monticello[a]	18	723	597	549	522	1.39
Sherrard[a]	10	706	597	483	468	1.51
Viola[a]	8	400	343	281	267	1.50
AASHO-874	20	541	500	435	396	1.37
AASHO-872	20	400	333	275	275	1.45
AASHO-845	16[b]	250	200	182	100[c]	2.50

[a]Average of three stations.
[b]Not peak deflection frequency.
[c]Unstable readings at these conditions.

Table 8. Influence of load magnitude on FWD deflections.

Section	Stiffness at Load Shown (kips/in) 3 Kips	6 Kips	8 Kips	>8 Kips	Stiffness max / Stiffness min
Bement[a]	783	779	736	711	1.10
Deland[a]	175	172	170	-	1.03
Monticello[a]	677	647	639	617	1.10
Sherrard[a]	500	458	439	-	1.14
Viola[a]	316	276	261	261	1.21
AASHO-874	273	306	296	313	1.15[b]
AASHO-872	273	273	265	257	1.06
AASHO-845	100	110	110	-	1.10[b]

[a]Average of three stations.
[b]Increasing stiffness with increasing load.

5. The ratio between the maximum and minimum areas in a sweep never exceeded 1.36; most ratios were between 1.1 and 1.2.
6. The combined effects of load and frequency on pavement stiffness can be substantial. A typical flexible pavement with a stiffness of 1000 kips/in at a 2-kip peak-to-peak load and 8-Hz driving frequency can have a stiffness of only 500 kips/in or less at 8 kips and the peak deflection frequency. This illustrates the inconvenience of using low-load fixed-frequency NDT devices for structural pavement evaluation.

Seasonal Effects on Surface Deflections

Routine RR deflections (8-kip peak-to-peak load, 15-Hz driving frequency) were measured four to six

Table 9. Analysis of variance of RR deflections with testing date.

			Grand Total									
Section	N	F-ratio	Mean Δ (mils)	SD (mils)	CV (%)	Duncan Multiple-Range Test						
Bement	120	2.02	15.34	2.84	18.5	Group	Oct. 78	June 78	Oct. 79	May 78	Apr. 79	Sept. 79
						Mean Δ	14.32	14.51	14.92	15.34	15.63	16.89
Coffeen	99	37.87[a]	9.93	1.98	20.0	Group	Oct. 78	Sept. 78	Apr. 79	May 78	Aug. 79	
						Mean Δ	7.21	9.46	10.52	10.67	11.74	
Deland	110	34.32[a]	43.55	7.76	17.8	Group	Oct. 78	Oct. 79	Sept. 79	May 78	Apr. 79	June 78
						Mean Δ	33.76	40.50	41.42	47.30	50.40	51.25
Hillsboro	99	106.48[a]	24.98	6.86	27.5	Group	Oct. 78	May 78	Sept. 78	Apr. 79	Aug. 79	
						Mean Δ	14.08	25.16	25.55	27.00	33.11	
Midlothian "A"	139	20.19[a]	7.95	1.90	23.9	Group	Nov. 78	May 79	Sept. 79	Aug. 78	Sept. 77	June 78
						Mean Δ	5.60	6.95	7.80	8.32	8.58	9.79
Midlothian "B"	139	20.95[a]	14.08	2.06	14.6	Group	Nov. 78	May 79	Aug. 78	Sept. 79	Sept. 77	June 78
						Mean Δ	11.08	13.76	14.11	14.44	14.80	15.63
Monticello	120	14.39[a]	17.13	3.78	22.0	Group	Oct. 79	Oct. 78	May 78	Sept. 79	June 78	Apr. 79
						Mean Δ	14.39	14.91	16.24	16.93	19.36	20.66
Neoga "N"	80	61.11[a]	28.67	12.66	44.2	Group	Oct. 78	Aug. 79	Apr. 79	Aug. 78		
						Mean Δ	11.84	28.08	34.97	39.83		
Neoga "S"	73	122.96[a]	21.87	6.33	29.0	Group	Oct. 78	Aug. 78	Aug. 79	Apr. 79		
						Mean Δ	12.61	24.28	25.29	27.15		
Pana	100	251.11[a]	20.71	8.69	42.0	Group	Oct. 78	May 78	Apr. 79	Sept. 78	Aug 79	
						Mean Δ	9.24	16.71	18.41	25.71	33.50	
Sherrard	116	134.76[a]	21.05	7.66	36.4	Group	Nov. 78	Oct. 79	Sept. 79	May 78	July 78	May 79
						Mean Δ	13.23	16.24	19.47	20.41	22.18	36.03
Viola	78	117.41[a]	39.11	23.04	59.0	Group	Nov. 78	Oct. 79	Sept. 79	May 79		
						Mean Δ	21.16	28.21	32.26	73.90		

[a]Significant at $\alpha = 0.05$.

Table 10. Analysis of variance of RR basin area with testing date.

			Grand Total									
Section	N	F-ratio[a]	Mean Area (in)	SD (in)	CV (%)	Duncan Multiple-Range Test						
Bement	120	8.00	27.12	1.88	6.9	Group	Sept. 79	Oct. 78	Oct. 79	May 78	June 78	Apr. 79
						Mean area	25.95	26.25	26.50	27.41	28.07	28.52
Coffeen	99	4.89	29.24	1.75	6.0	Group	Aug. 79	Sept. 78	Oct. 78	May 78	Apr. 79	
						Mean area	28.59	28.78	28.85	29.44	30.78	
Deland	110	30.13	18.40	1.24	6.7	Group	May 78	June 78	Oct. 78	Oct. 79	Sept. 79	Apr. 79
						Mean area	17.21	17.43	18.12	19.10	19.52	19.65
Hillsboro	99	112.81	26.90	2.29	8.5	Group	Aug. 79	Sept. 78	May 78	Apr. 79	Oct. 78	
						Mean area	24.64	25.01	26.31	28.74	29.91	
Midlothian "A"	140	62.25	24.61	2.19	8.9	Group	June 78	Aug. 78	Sept. 77	Sept. 79	Nov. 78	May 79
						Mean area	22.71	23.17	23.80	24.83	25.43	28.56
Midlothian "B"	140	84.63	23.91	1.60	6.7	Group	Sept. 77	Aug. 78	June 78	Sept. 79	Nov. 78	May 79
						Mean area	22.85	23.00	23.17	23.96	24.75	26.91
Monticello	120	5.2	25.85	1.91	7.4	Group	Sept. 79	Oct. 78	June 78	Oct. 79	May 78	Apr. 79
						Mean area	24.82	24.89	25.62	26.18	26.54	27.04
Neoga "N"	80	106.90	22.75	2.42	10.6	Group	Aug. 78	Aug. 79	Apr. 79	Oct. 78		
						Mean area	19.49	22.32	23.91	25.30		
Neoga "S"	73	43.50	25.15	1.65	6.6	Group	Aug. 78	Aug. 79	Oct. 78	Apr. 79		
						Mean area	23.47	24.58	26.25	26.92		
Pana	100	254.62	23.74	2.47	10.4	Group	Sept. 78	Aug. 79	May 78	Apr. 79	Oct. 78	
						Mean area	20.80	21.58	23.62	25.86	26.86	
Sherrard	116	63.34	20.88	1.33	6.4	Group	July 78	Sept. 79	May 79	May 78	Nov. 78	Oct. 79
						Mean area	18.80	20.47	20.57	21.51	21.71	22.30
Viola	78	15.19	16.61	1.05	6.3	Group	Sept. 79	Oct. 79	Nov. 78	May 79		
						Mean area	15.55	16.72	16.81	17.31		

[a]Significant at $\alpha = 0.05$.

times in a two-year period on the 12 primary in-service sections of the program. Tables 9 and 10 show the analyses of variance of RR deflections and area with date of testing, respectively.

In addition to the F-ratio, which indicates whether the deflections and area changed with testing date, Tables 9 and 10 include the results of the Duncan multiple-range test. In this test, means that are statistically equal (95 percent confidence level) are grouped together and are underlined. Tables 9 and 10 indicate the following findings:

1. Except for the Bement section (deflections), mean RR deflections and area did change with testing date;
2. Within the testing period, mean RR deflections for a given section changed by factors of 1.4-3.6;
3. Within the testing period, mean RR area for a given section changed by factors of 1.07-1.30;
4. Without exception, the lowest mean RR deflection was measured in the fall (October or November);
5. The highest mean RR deflection was obtained either in the summer or the spring; and
6. There is no specific time of the year when the area is the highest or lowest in any given section.

SUMMARY AND CONCLUSIONS

This paper presented the main findings and conclusions of a comparative study of selected NDT devices for the structural evaluation of Illinois flexible pavements. The devices used were the Benkelman beam, the road rater (model 2008), the falling-weight deflectometer, and an accelerometer to measure surface deflections under moving trucks. The different (mostly in-service) pavements tested were selected to reflect typical flexible pavement constructions over a wide variety of subgrade soils throughout Illinois. Comparisons and correlations (where applicable) between different NDT devices were presented. The following conclusions were drawn:

1. Overall, BB deflections cannot be reliably predicted from RR deflections.
2. RR deflections (8-kip, 15-Hz) and FWD deflections (8-kip) are significantly different (statistically) for all pavements tested. However, RR and FWD deflections and areas are highly correlated.
3. Surface deflections are highly sensitive to the RR load and driving frequency. Low-load fixed-frequency vibrators can overestimate the pavement stiffness by factors of 2 or more.
4. Overall, the FWD is the best NDT device to simulate pavement response under moving loads. The RR, because of its harmonic loading without rest periods and static preload, induces pavement deflections lower than those achieved with the FWD and moving loads.
5. FWD deflections and deflection-basin areas at a 9-kip load level or converted RR deflections and area at 8 kips and 15 Hz (by using the FWD-RR correlations proposed) are recommended for structural flexible-pavement evaluation.

ACKNOWLEDGMENT

This paper is based on the results of Project IHR-508: Load Response Characteristics of Flexible Pavements. Project IHR-508 is a cooperative study. Project planning and development were joint efforts of IDOT and the University of Illinois. IDOT (P. Dierstein, J.S. Dhamrait, and K.W. Wicks) assumed responsibility for all NDT testing. Various IDOT districts aided in the field sampling. The Soils Laboratory of the Bureau of Materials and Physical Research under the supervision of T.S. Lewis conducted routine soil tests. Specialized soils and materials tests were conducted by the University of Illinois. V.J. McDonald, Research Engineer at the University of Illinois, and the IDOT Instrumentation Development Group of the Physical Research Section were responsible for accelerometer data acquisition and data processing. Data reduction, structural modeling analyses, and the development of procedures for analyzing NDT data were activities conducted by the University of Illinois.

Special thanks are extended to the various IDOT districts, County Superintendents of Highways, and Township Commissioners who willingly agreed to provide test pavement sites and general aid.

IHR-508 was sponsored by the Illinois Department of Transportation (Division of Highways) and the Federal Highway Administration, U.S. Department of Transportation.

REFERENCES

1. M.S. Hoffman and M.R. Thompson. Nondestructive Testing of Flexible Pavements--Field Testing Program Summary. Univ. of Illinois at Urbana-Champaign, Transportation Engineering Series 31, Illinois Cooperative Highway and Transportation Research Program Series 188, June 1981.
2. M.S. Hoffman and M.R. Thompson. Mechanistic Interpretation of Nondestructive Pavement Testing Deflections. Univ. of Illinois at Urbana-Champaign, Transportation Engineering Series 32, Illinois Cooperative Highway and Transportation Research Program Series 190, June 1981.
3. M.S. Hoffman. Mechanistic Interpretation of Nondestructive Pavement Testing Deflections. Department of Civil Engineering, Univ. of Illinois at Urbana-Champaign, Ph.D. thesis, Sept. 1980.
4. A. Bohn, P. Ullidtz, R. Stubstad, and A. Sorenson. Danish Experiments with the French Falling Weight Deflectometer. Proc., 3rd International Conference on the Structural Design of Asphalt Pavements, Univ. of Michigan, Ann Arbor, Vol. 1, 1972, pp. 1119-1128.
5. AASHO Road Test. HRB, Special Repts. 61 and 73, 1962.

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