

Research Report  
UKTRP-84-21

EVALUATION OF FULL-DEPTH  
ASPHALTIC CONCRETE PAVEMENTS

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16. Abstract  <p>This study was initiated to verify a newly-developed set of design curves for full-depth asphaltic concrete pavements. Quality control during construction was checked using nuclear density testers, Benkelman beams, and a Road Rater. During the course of this study, an analysis system was developed to interpret the dynamic deflections as measured by the Road Rater and was confirmed by elastic theory. The thickness design curves were verified within the accuracy of construction variations.</p> <p>Rut depths measured in 1979 were analyzed in terms of potential rut depth resulting from consolidation under traffic due to lack of obtaining 100-percent of Marshall density during compaction immediately after paving. Potential rutting was calculated as the decimal equivalent of the quantity of 100 percent minus percent compaction times the layer thickness and the results accumulated from the surface downward.</p> <p>Advances in technology under this study have led to greater advances under succeeding studies.</p> <p>Traffic was monitored using automatic traffic counters, manual classification/volume counts, and weigh-in-motion scales installed in the pavement.</p>					
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## INTRODUCTION

In 1968 a revision of the 1959 Kentucky thickness design curves for asphaltic concrete pavements included a new set of curves for full-depth asphaltic concrete (1). Study KYHPR-70-49, "Full-Depth Asphaltic Concrete Pavements," was initiated to verify those full-depth structural designs developed from elastic theory by:

1. determining the mechanical response of an in-service experimental pavement to applied static and dynamic loads and
2. observing the pavement's performance under traffic over an extended period.

The experimental pavement was constructed on US 60 in Boyd County, Kentucky, and was completed in November 1971. Table 1 lists the design CBR's, design thicknesses, and station termini of experimental and control sections. Upon completion, cores were taken in each section, in-place CBR tests made, and a report issued summarizing all test data obtained during monitoring of the construction (2).

To monitor performance under traffic, the pavements were subjected to deflection testing by several methods, rut depths were measured, and roughness tests were made. Pavement temperatures were measured by thermistors installed at one site, and surface measurements were made during deflection testing at various test sites. Equipment to measure solar radiation was installed at the thermistor site to provide data so correlations, if any, between solar radiation and distributions of pavement temperatures could be evaluated. Pavement fatigue calculations required a knowledge of vehicle classifications, their volumes, and total vehicular loads within each classification. Thus, a weigh-in-motion system was installed at a location near the thermistor site.

## PROCEDURES AND PROBLEMS

### PAVEMENT TEMPERATURE AND SOLAR RADIATION MEASUREMENTS

Station 211+00 was selected as the site for the thermistors, they were installed at 51-mm (2-inch) increments throughout the 457-mm (18-inch) depth of asphaltic concrete. The thermistors were connected to a strip-chart recorder and monitored for approximately three years. By then approximately half of the thermistors had failed and the system was dismantled. Data recorded the first year were analyzed and observed temperature distributions verified predicted distributions using the procedure developed from data recorded in Maryland, New York, and Arizona (3, 4, 5).

A pyrometer to measure solar radiation was mounted on top of the cabinet housing the recorders for the thermistors and pyrometer. The recorder for the pyrometer developed mechanical problems shortly after installation. Calibration became such a problem that the recorded patterns could be considered valid only in terms of shape but not magnitude. The pyrometer failed during the second year. Thus, all attempts to correlate pavement temperature distributions with recorded solar radiation were abandoned.

### WEIGH-IN-MOTION INSTALLATION

Station 222+00 was selected for a weigh-in-motion system. The system was installed during July 1974 by removing approximately the top 127 mm (5 inches) of asphaltic concrete from the 457-mm (18-inch) section. Frames and

TABLE 1. PAVEMENT DESIGN AND CORED THICKNESSES

STATION		ASPHALTIC CONCRETE THICKNESS (INCHES)		DESIGN CBR
FROM	TO	DESIGN	CORE	
128+00	155+33	14.0	14.8	9
155+33	182+66	12.0	12.0	9
182+66	210+00	10.0	10.3	9
210+00	245+00	18.0	17.4	3
245+00	285+00	16.0	16.9	3
285+00	321+00	14.0	13.7	3
321+00	347+50	18.0	18.2	3
347+50	373+50	16.0	17.5	3
373+50	399+50	14.0	16.3	3
399+50	425+68	6.5*	6.8*	3

\*Control Section -- Asphaltic concrete on 19 inches of dense-graded aggregate base  
 1 inch = 25.4 millimeters

load-bearing pads were positioned using a jig frame and anchored into place with a fast-setting high-early cement and sand mix. Transducers were put into the frames and wires were inserted through copper tubing and connected to the recorder and transducers. Air in the copper tubing was purged using nitrogen gas, and the system was pressurized to approximately 34,000 pascals (5 psi) to prevent moisture from entering the system and causing electrical shorts. An area next to the shoulder was paved to provide a parking stand for a van housing the monitoring and recording equipment. Equipment in a van consisted of a console for operating and monitoring the system, a teletype printer used to obtain a hard copy of the axleloads on a vehicle and for calibrating procedures, a computer to receive and store all data, and a computer tape recorder compatible with a main-frame computer. The operator had the capability of entering the body style of the truck through one set of buttons and the axle configuration by another set when the system was on either manual or automatic mode of operation. When the system was placed on manual mode and the vehicle traversed the scale platform, the wheel load to the nearest kip was illuminated so the operator could check for improper data, which could be erased prior to recording. When the system was on automatic mode, the teletype and the illuminated wheel-load panel were by-passed and the data was recorded directly onto magnetic tape.

When the system was in the calibration mode, the operator used the teletype printer and adjustment controls to maintain calibration. A single-unit two-axle dump truck was loaded such that the four-tired rear axle carried 80 kN (18 kips). The truck was stopped with the rear axle on the scales, allowing the operator to calibrate the scales to represent a static condition. The truck was driven over the scales at various speeds and different positions across the lane to determine the variation in response patterns under a known load.

#### PROBLEMS WITH WEIGH-IN-MOTION SYSTEM

The original design of the pads, which supported the load cells of the transducer plate, proved to be difficult to grout properly and the pads would become detached from the foundation under traffic. The problem was partially solved by boring a hole through the plate so that either the grout could be squeezed up through the hole or so an epoxy grout could be forced through the hole to fill any void under the pad.

The original surface plates were rectangular and proved to be difficult to keep from rocking. Adjustment screws in the sides of the frame used to keep the plates from moving laterally had a tendency to bind the plates in the vertical direction and had to be monitored closely and adjusted frequently. The manufacturer redesigned the surface plates to a triangular configuration, which also required the frames to be modified by welding in spacer bars. Later installations of the modified frames were twisted and bounced out of the pavement by the heavy loads. Inspection revealed that welds between the spacer bars and side walls of the frame had failed. The frames were redesigned to provide for "J" bolts as anchor bolts when placed in the concrete foundation. Because of the repeated and frequent failure of the transducers, the system was abandoned prior to installation of the redesigned frames.

The weigh-in-motion system appeared to function adequately so long as normal traffic existed. The manufacturer's specifications called for the transducers to withstand a 133.3-kN (30-kip) axleload. The system recorded several axleloads of approximately 145 kN (33 kips), but the transducers in

the scale pit failed shortly thereafter. On one occasion, one or more load cells were shattered into many pieces.

On one occasion, lightning hit the system and damaged the computer circuits.

Drivers quickly learned they could veer to the right so the right tires of the vehicle would either partially or totally miss the right-hand transducer, making it necessary to void that data. Eventually that problem lessened after the drivers realized the unit was not being used as an enforcement tool.

The designer of the transducer was requested to prepare a new design to withstand loads known to exist on some trucks. As many as four configurations were investigated, but none proved to be satisfactory. Thus, the system was abandoned until improvements in transducer design might be available. It was recognized that future installations and installing methods required significant alterations. The concrete foundation for the entire installation should extend for at least 6.1 meters (20 feet) in length and be designed as a structural slab containing reinforcing steel. Heavier frames and anchor bolts will be required. The approach must be flat and smooth to reduce dynamic oscillations of the truck.

Weigh data were recorded onto a magnetic tape and sometimes displayed by the teletype printer. Technical difficulties in processing recorded weigh-in-motion data at the computer facility became severe enough to warrant abandoning analyses by computer. By 1978 the computer program would not run and analyses could no longer be completed.

#### WEIGH-IN-MOTION DATA

In spite of all the problems, valuable information was obtained from the weigh-in-motion system. The data provided vehicle classification counts, ranges in axle spacings, vehicle speed, and proved to many that the existing axleloads were much more severe than thought and that axles within the same group of axles were not equally loaded (6, 7). Axle spacing and vehicle speed data were not required for this study and detailed analyses were not made.

#### TRAFFIC DATA

In addition to the data collected by the weigh-in-motion system, traffic counters were installed at various locations throughout the project. Volume and lane distribution data were obtained using portable electronic traffic counters at four different locations (two eastbound and two westbound). Volume data were obtained for both inner and outer lanes for all sites. Data were recorded on punched paper tapes that were retrieved and processed by computer. Data were processed to obtain total vehicles per hour and were further summarized according to vehicles per day, week, month, and year.

Periodic classification counts were made manually. Counts were obtained for five different years at selected locations -- the intersection of US 60 and KY 716, US 60 at the intersection of a side road west of the entrance to the cemetery adjacent to the former city limits, and the weigh-in-motion installation. Manual counts at the weigh-in-motion site served to verify classifications recorded when the scale system was placed in the automatic mode. Information relating to vehicle classification is required to calculate accumulated pavement fatigue. Volume data were used to determine lane distribution relationships and traffic growth trends that also were required in the calculation of accumulated pavement fatigue.

Data from the weigh-in-motion system illustrated the need for developing a set of load equivalency factors for the steering axle and for tridems. Equivalency factors had not been developed for the steering axles at the AASHO Road Test, and tridem axle arrangements were not used at the AASHO Road Test. Adjustments to load equivalency factors also were required for the four-tired single axle and tandem axles (8). Steering axleloads at the AASHO Road Test were thought to be sufficiently low and thus insignificant in terms of fatigue. Analysis of weigh-in-motion data using elastic layer theory indicated a greater effect from front axles (particularly for heavily loaded vehicles). The Chevron N-layer computer program (9) was used to develop the load equivalency factors for both the steering axle and tridem groups.

#### DEFLECTION TESTING

A Road Rater was purchased for monitoring construction and long-term performance of this project. Road Rater deflections were obtained on the subgrade just prior to paving and after each lift of asphaltic concrete at preselected sites throughout the project (2). Upon completion of construction, Benkelman beam tests also were conducted so as to permit the correlation of static and dynamic deflections obtained by the two pieces of equipment (10). In October 1973 a series of tests were conducted simultaneously using the Road Rater, Benkelman beam, and a Dynaflect from Ohio State University (11).

Construction scheduling and restrictions prevented all test sites from being tested prior to placement of the next layer of asphaltic concrete. At least three days were required for a layer to cure sufficiently to prevent the feet of the Road Rater from leaving indentation marks on the surface. Deflection measurements also were obtained at selected locations using the Road Rater each spring and fall from 1972 through 1979.

#### CONSTRUCTION COMPACTION

Density tests were made on each layer of asphaltic concrete after compaction had been completed.

At the time of construction of this project, vibratory rollers were being introduced to the construction scene. This project was used as a test pavement to evaluate the performance of the roller (12).

### PERFORMANCE

#### VEHICLE CLASSIFICATION

A summary of the results of the classification surveys is presented in Figures 1 through 6. Trends of the number of a given vehicle classification as a portion of the total volume are shown as a function of time. It is noted that the proportions of various vehicle styles do vary with time, indicating a change in vehicle usage. A directional analyses of traffic volumes showed there was an apparent equal distribution of volumes by direction. Evaluation of the data for lane distributions are summarized in Figure 7. Data from manual classification counts also were evaluated to determine lane distribution and were supportive of lane distribution characteristics determined from data obtained with portable traffic counters. Lane distribution factors for various vehicle classifications had been developed previously (13) from data obtained at sites on Kentucky interstate routes and were essentially the same as observed on US 60.

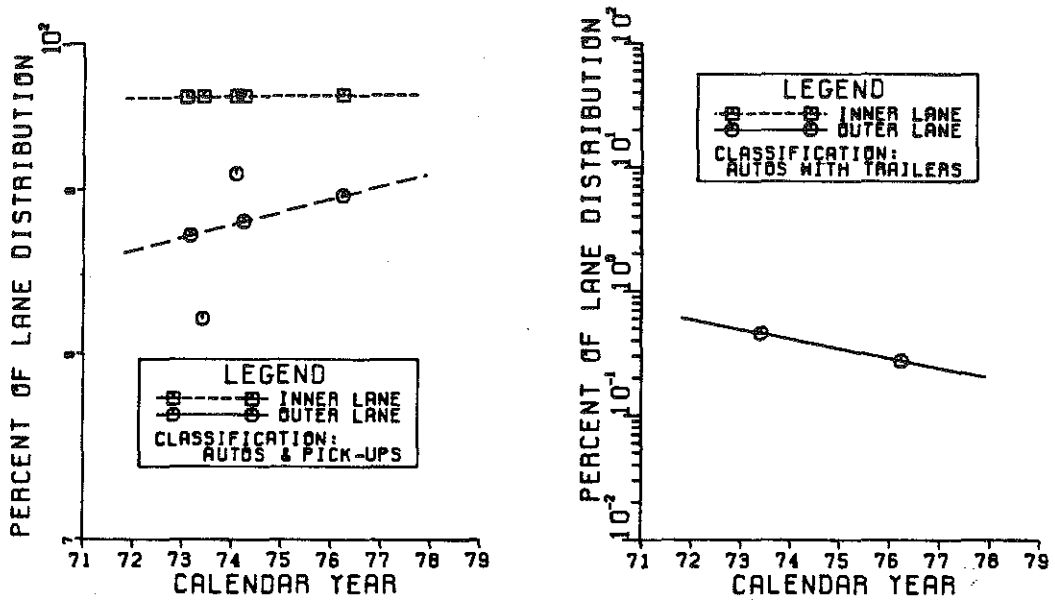


Figure 1. Percent Vehicle Classification by Lane versus Time -- Autos, Pickups, and Autos with Trailers.

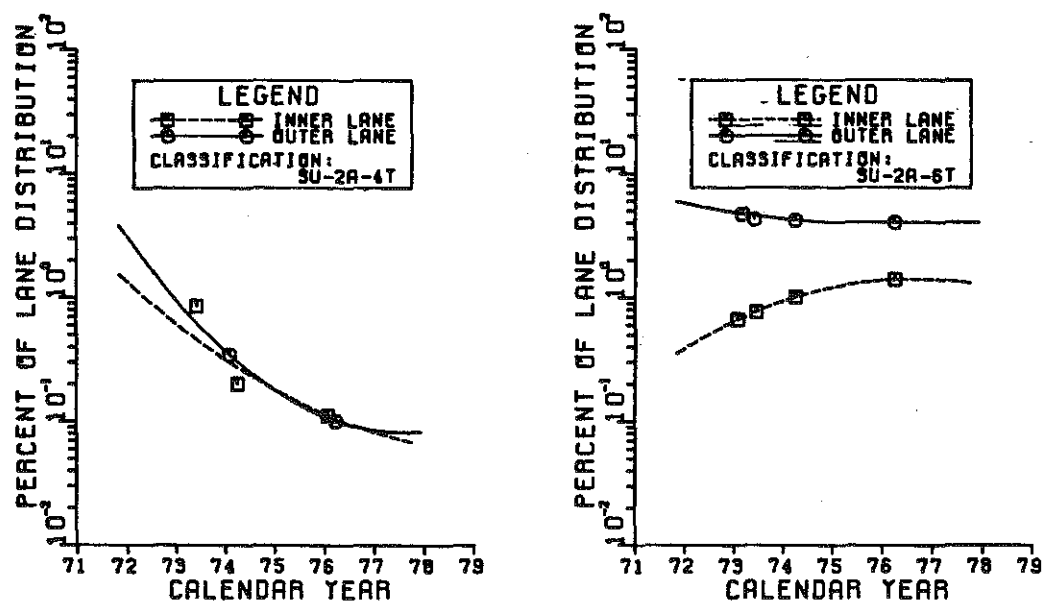


Figure 2. Percent Vehicle Classification by Lane versus Time -- Single-Unit Two-Axle Four Tires and Single-Unit Two-Axle Six Tires.



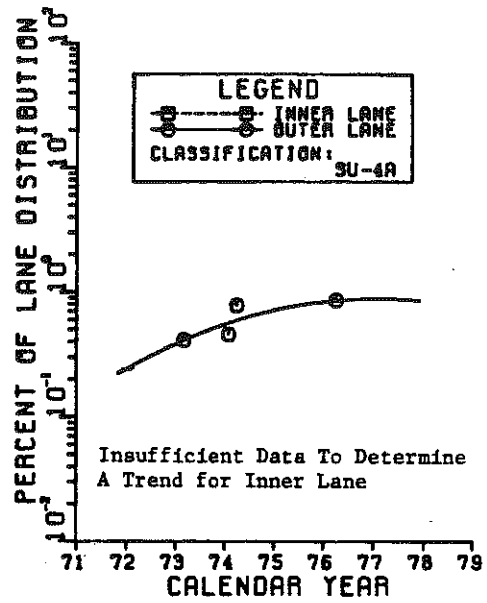
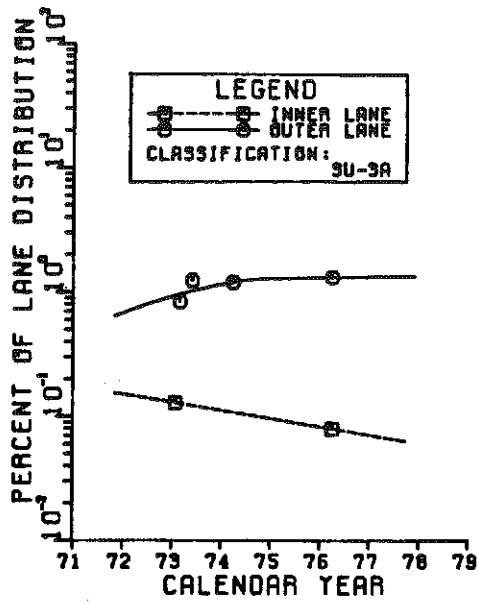


Figure 3. Percent Vehicle Classification by Lane versus Time -- Single-Unit Three Axles and Single-Unit Four Axles.

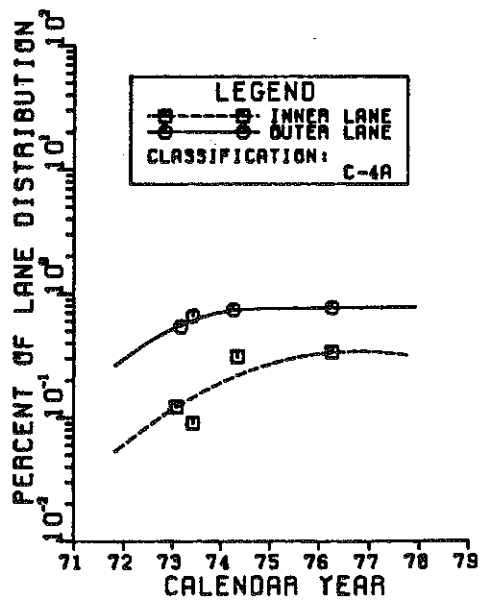
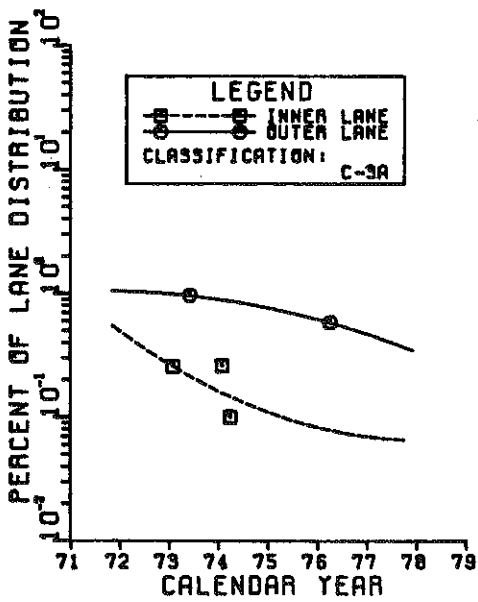


Figure 4. Percent Vehicle Classification by Lane versus Time -- Combination Three-Axle Semi-Trailer and Combination Four-Axle Semi-Trailer.

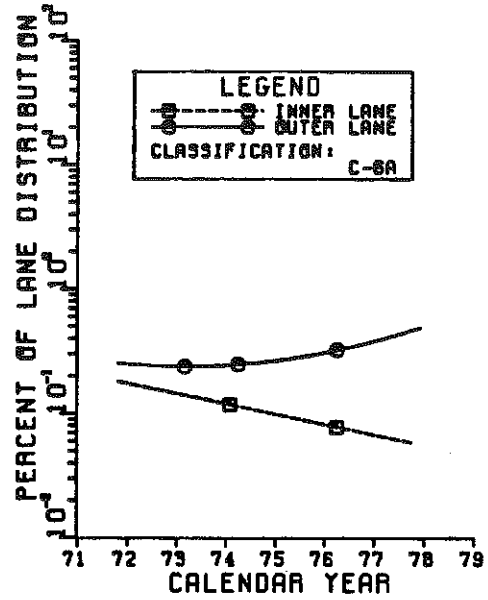
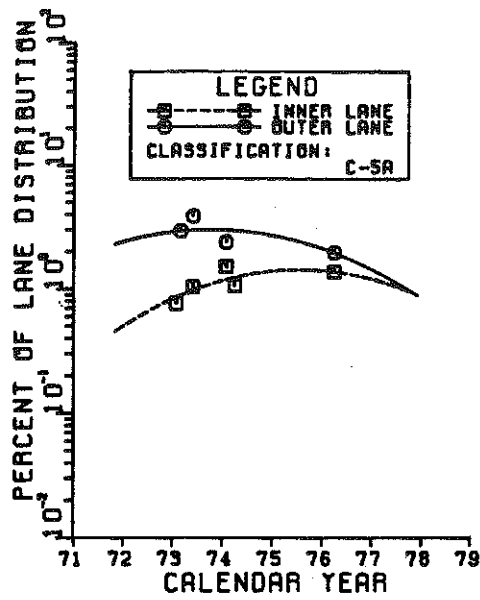


Figure 5. Percent Vehicle Classification by Lane versus Time -- Combination Five-Axle Semi-Trailer and Combination Six-Axle Semi-Trailer.

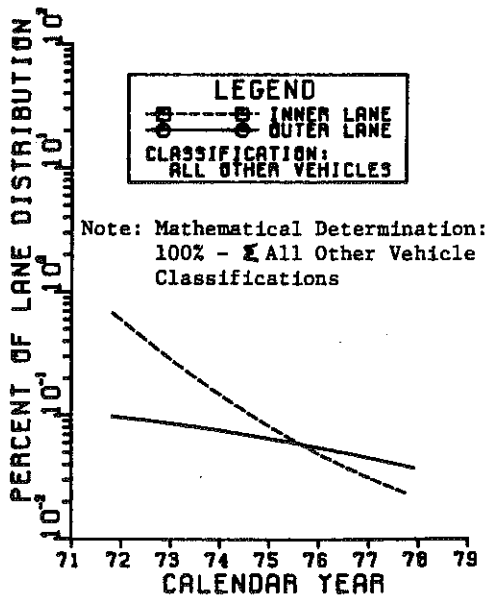
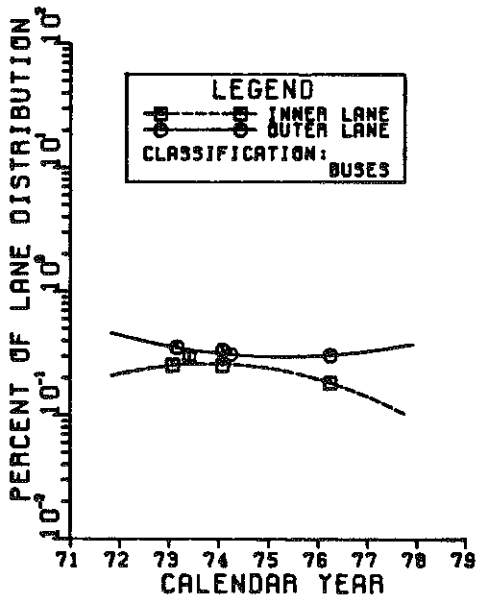


Figure 6. Percent Vehicle Classification by Lane versus Time -- Buses and Other.

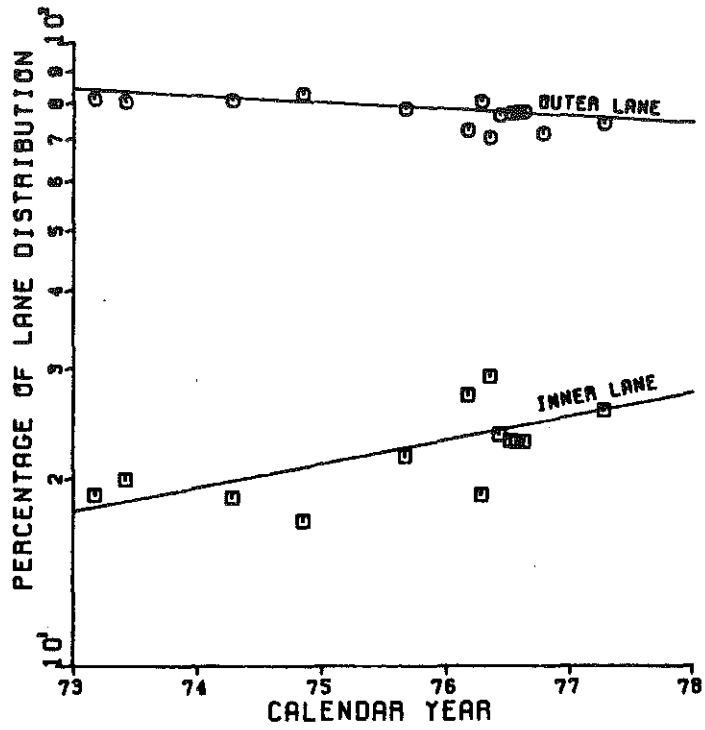


Figure 7. Total Vehicles by Lane versus Calendar Year.

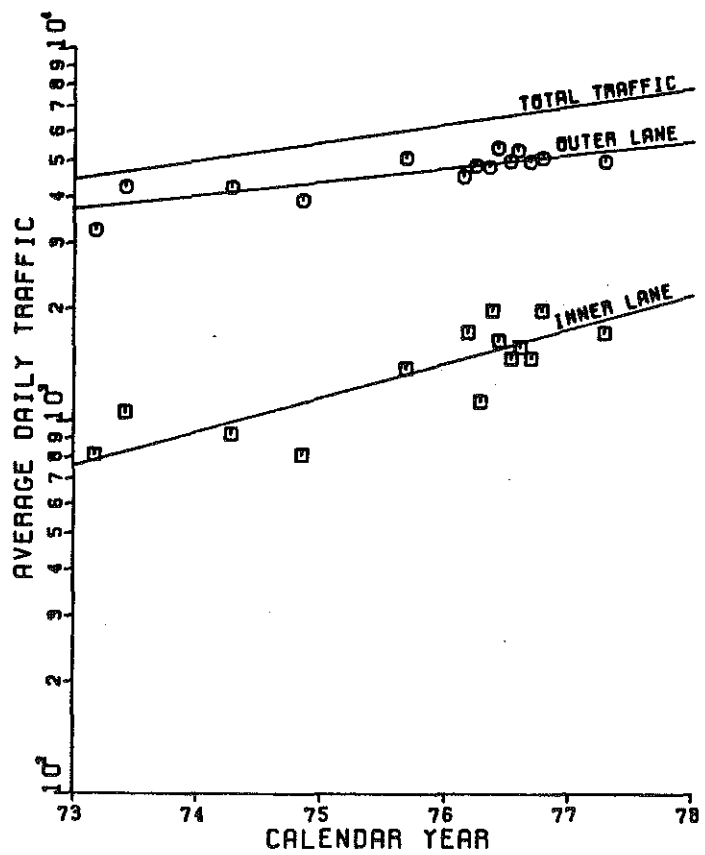


Figure 8. Average Daily Traffic versus Calendar Year.

Volume data obtained from portable traffic counters were combined on an annual basis to determine average annual daily volumes. The trends relating volume growth (traffic growth) with time are presented in Figure 8.

#### DAMAGE FACTORS

Research in Kentucky (14, 15, 16, 17) have indicated that work and energy principles may be used as the basis for the designs of flexible pavements and have been utilized to evaluate pavement performance. Work and energy principles also have been applied to the development of traffic load equivalency factors or "damage factors." Load equivalency factors normally are used to express pavement damage in terms of the damage attributable to an 80-kN (18-kip) axleload. Thus, damage factors are used to convert actual loadings for various vehicle styles and weights to equivalent 80-kN (18-kip) axleloads (EAL's).

Relationships of damage factors versus time (Figure 9) for average statewide loadings have been determined previously (6, 7). In general, those relationships were developed using data reported in statewide W-4 tables from the Truck Weight and Vehicle Classification Studies for the period 1959 to 1978. The trends were modified to reflect observations of vehicle weight data obtained from the weigh-in-motion facility for the various vehicle classifications. Figures 8 and 9 summarize the relationships used in the calculation of accumulated pavement fatigue (Figure 10) for US 60 in Boyd County. The range of loadings associated with those relationships is summarized in Table 2.

#### PAVEMENT FATIGUE

The total equivalent axleloads associated with any vehicle classification may be determined by the summation (for all weight categories within a given classification) of the product of the number of vehicles in the weight category and the damage factor for that weight category. The total fatigue (for a specific time interval) may be determined by the summation of total fatigue for all vehicle classifications. The total accumulated fatigue is the sum of the fatigue for all time intervals.

The number of vehicles of a specific classification (and for a specific time interval) is determined as the product of the classification proportion from Figures 1 through 6 and the total number of vehicles from Figure 8. Figure 8 presents average daily traffic for each lane. The total vehicles for a given lane may be determined as the product of percent lane distribution (Figure 7) and the total number of vehicles for both lanes (Figure 8).

The calculation of fatigue for a particular vehicle type may be summarized by the following equation:

$$\text{EAL} = \text{ADT} \times \text{Time Interval} \times \text{Percent Lane Distribution (decimal fraction)} \\ \times \text{Percent Classification (decimal fraction)} \times \text{Damage Factor for} \\ \text{Vehicle Classification}$$

The total accumulated pavement fatigue is obtained by summing over all time periods and all vehicle classifications. Total accumulated pavement fatigue is summarized in Figure 10.

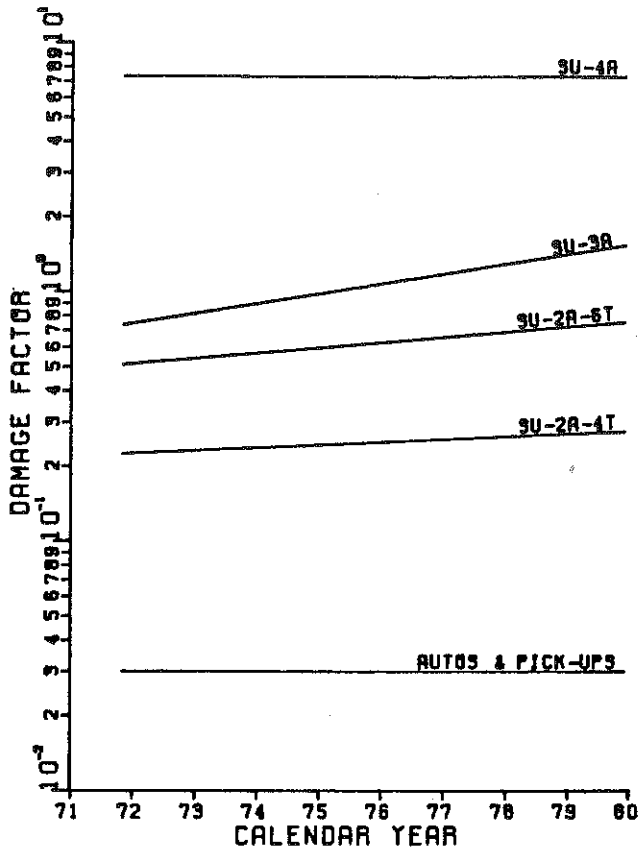
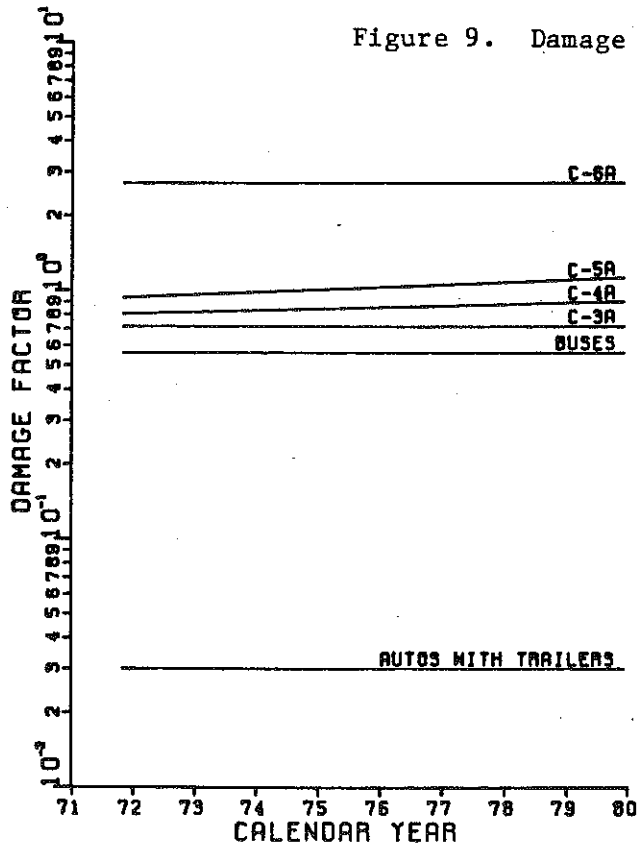


Figure 9. Damage Factor versus Calendar Year.



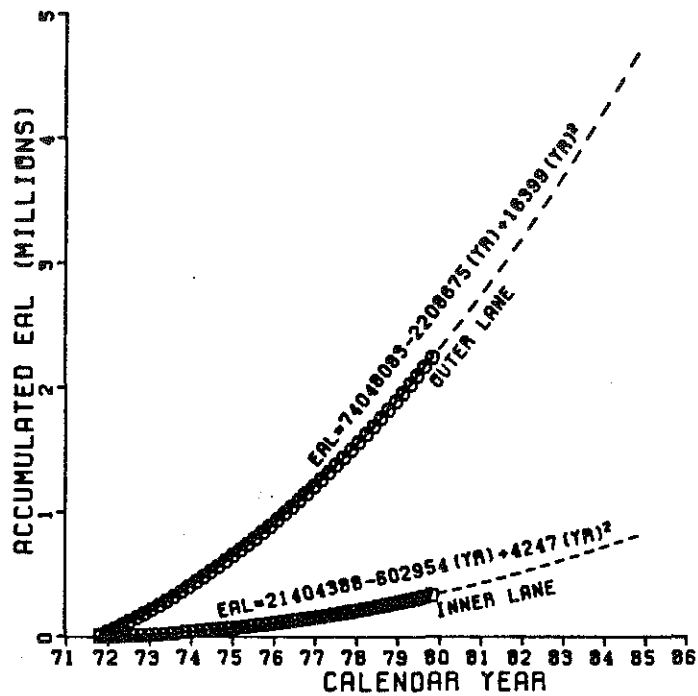


Figure 10. Accumulated 80-kN (18-kip) Equivalent Axleloads versus Calendar Year.

TABLE 2. AVERAGE LOADINGS ASSOCIATED WITH FATIGUE ESTIMATES

VEHICLE TYPE	GROSS LOADINGS (POUNDS)
Single Unit, Two Axles, Six Tires	16,000 - 26,000
Single Unit, Three Axles	42,000 - 62,000
Single Unit, Four Axles	92,000
Buses	26,000
Combination, Three Axles	40,000
Combination, Four Axles	54,000 - 60,000
Combination, Five Axles	76,000 - 85,000
Combination, Six Axles	110,000

## DEVELOPMENT OF PAVEMENT DETERIORATION CURVES

### PAVEMENT DEFLECTION MEASUREMENTS

Specific details regarding procedures for evaluation of pavement deflection measurements have been detailed previously (10, 11, 14, 15, 18 through 21). Resultant expressions of pavement structural condition involves a combination of estimated subgrade modulus and an effective thickness of asphaltic concrete (3.31-GPa (480-ksi) modulus). Such combinations are derived by evaluations of both the shape and magnitude of the measured deflection bowl matched with theoretically determined deflections.

### DEFLECTION TESTS

All pavement sections in the US 60 project were tested with the Benkelman beam and the Road Rater. A correlation had been developed previously (10) and was used to interpret test results obtained using Benkelman beams in terms of results obtained from tests using the Road Rater.

Behavior of asphaltic concrete pavements are dependent upon the temperature distribution throughout the asphaltic concrete. The appropriate factor used to adjust deflections to an equivalent value at a reference temperature depends upon the particular piece of test equipment. Benkelman beam testing required a different set of adjustment factors (3) than those required for a Road Rater (14, 15).

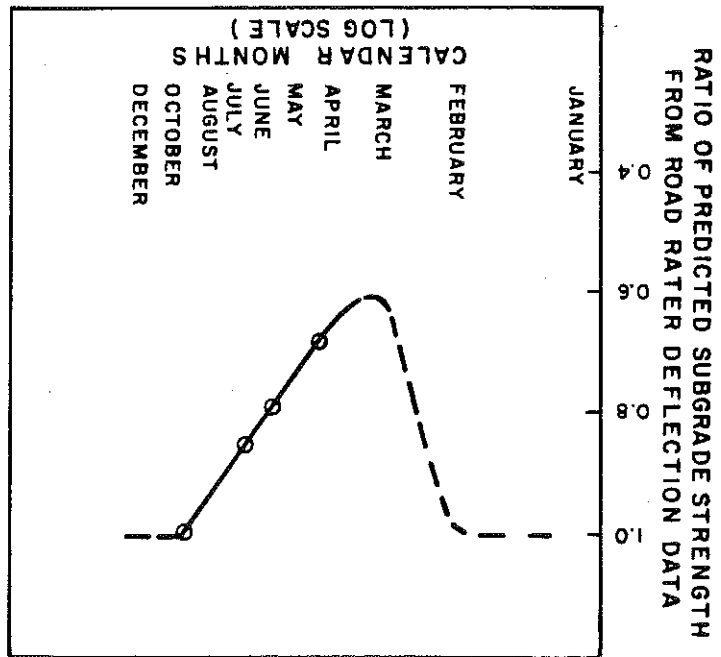
All pavement deflections were adjusted to 21.1 C (70 F) by estimating the temperature distribution in the asphaltic concrete pavement (3, 4, 5). Deflection data obtained in this study also were used to evaluate the seasonal variation of subgrade moduli, as shown in Figure 11 (13). Therefore, deflections could be adjusted to equivalent spring-time conditions. Having adjusted measured deflections for temperature and subgrade conditions as a function of the time of year, methods were developed to express the behavior of the pavement as that of a thickness of reference-quality asphaltic concrete at a given subgrade modulus.

### NORMALIZATION OF PAVEMENT BEHAVIOR

Direct comparisons of estimates of the structural conditions of the pavement sections are somewhat meaningless since expressions of pavement behavior involve a combination of subgrade modulus and effective thickness of reference-quality asphaltic concrete (3.31-GPa (490-ksi) modulus); there are an infinite number of combinations (but only a few are reasonable) that result in a theoretical deflection bowl that matches the measured deflection bowl. Therefore, it was necessary to normalize the results of pavement deflection evaluations to obtain meaningful comparisons.

Mean and 80th-percentile (weakest tail) subgrade moduli were determined for each design section. Theoretical relationships of No. 1 Sensor Road Rater deflection versus subgrade modulus were used to develop a relationship between the No. 1 Sensor deflection and pavement thickness for a constant 80th-percentile subgrade modulus for each design section. Measured No. 1 Sensor deflections were used as input into those relationships to determine an adjusted effective asphaltic concrete thickness in combination with the 80th-percentile subgrade modulus for each section. Adjusted effective asphaltic concrete thickness data were combined by design section and the statistical mean was determined. Those mean adjusted effective thicknesses of asphaltic concrete were divided by the maximum effective asphaltic

Figure 11. Adjustment Factor for Subgrade Modulus as a Function of Seasonal Variation.





concrete thickness for each section to normalize effective thicknesses. Normalized values (for outside lanes only) were plotted versus accumulated 80-kN (18-kip) equivalent axleloads (Figures 12 through 21). A similar analysis was conducted for data obtained for the inner lanes but were not used since somewhat lesser levels of fatigue were observed and the levels of accumulated EAL's were significantly lesser than for the outside lanes.

Data presented in Figures 12 through 21 were combined to develop pavement deterioration curves as illustrated in Figure 22, a plot of thickness versus EAL for three levels of normalized effective thickness for each design section or constructed thickness. Those relationships were used to refine the position of the lines representing the various constructed thicknesses into a family of curves for a constant level of EAL. The relationships were used further to develop the pavement deterioration curves in Figure 23, representing 51-mm (2-inch) increments of pavement thickness.

In Figure 23, the family of curves intersect the zero normalized effective thickness line at a range of EAL from 6,500,000 to 20,000,000, corresponding to 250 and 457 mm (10 and 18 inches) of asphaltic concrete. It also does not represent total deterioration of the pavement structure since there is still considerable structural worth associated with a mass of unbound or granular material. However, it does represent projected total deterioration of the layer as asphaltic concrete.

Design EAL's were estimated using the Kentucky flexible design curves, the design thickness, and design CBR's (2). Table 3 is a summary of back-calculated design EAL. Information presented in Table 3 indicate the minimum expected design life for any pavement section is 2,500,000 EAL's and is associated with the 262-mm (10.3-inch) full-depth asphaltic concrete constructed on a CBR 3 subgrade. A review of Figures 12 through 21 indicate relatively low levels of structural deterioration associated with this level of fatigue. The greatest level of structural deterioration (approximately 40 percent deterioration or less) is associated with the 262-mm (10.3-inch) full-depth asphaltic concrete. Figure 10 illustrates the accumulation of 2,500,000 EAL's that may be associated with 8.5 years service life for this pavement section. Figures 24 through 33 illustrate trends in roughness index or associated present serviceability index for each pavement section. Present serviceability indices corresponding to 8.5 years and 2,500,000 EAL's are generally greater than 3.0 and are greater than 2.5 for all sections. Therefore, it may be concluded that thickness design versus repetitions of 80-kN (18-kip) EAL's are consistent with observed pavement performance estimated on the basis of pavement roughness and associated present serviceability index.

#### CURING TIME

Asphaltic concrete gains strength (Young's modulus increases) with time (11). The time to "cure" completely is related to the total thickness of the asphaltic concrete. Analyses of Road Rater deflection data indicated "curing" took two to three years, depending upon the thickness of the asphaltic concrete.

Deflections decreased with additional thicknesses of asphaltic concrete when sufficient time elapsed before construction of the succeeding layers (11). For those sites where another layer was constructed before the previous layer had sufficient time to cure, the magnitude of the deflections changed very little with the addition of the new lift; but eventually the deflections decreased, as was expected. Pavement deterioration curves in Figures 12 through 22 also reflect these observations.

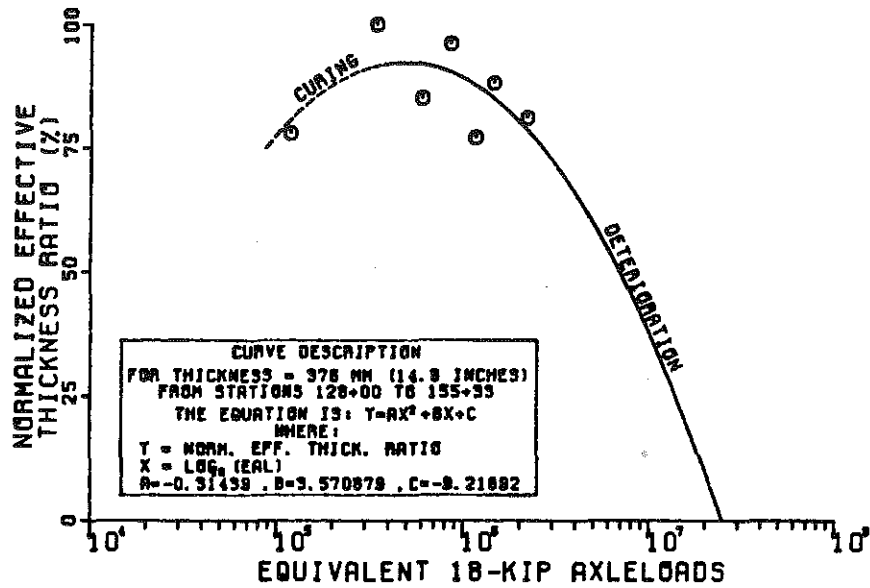


Figure 12. Normalized Effective Thickness versus Accumulated 80-kN (18-kip) Equivalent Axleloads for 376 mm (14.8 inches) of Asphaltic Concrete -- Station 128+00 to 155+33.

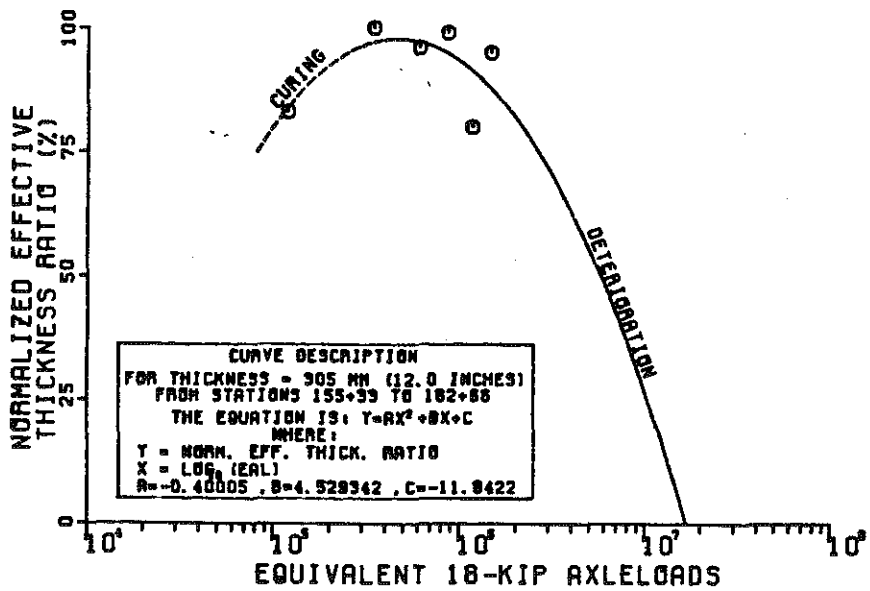


Figure 13. Normalized Effective Thickness versus Accumulated 80-kN (18-kip) Equivalent Axleloads for 305 mm (12.0 inches) of Asphaltic Concrete -- Station 155+33 to 182+66.

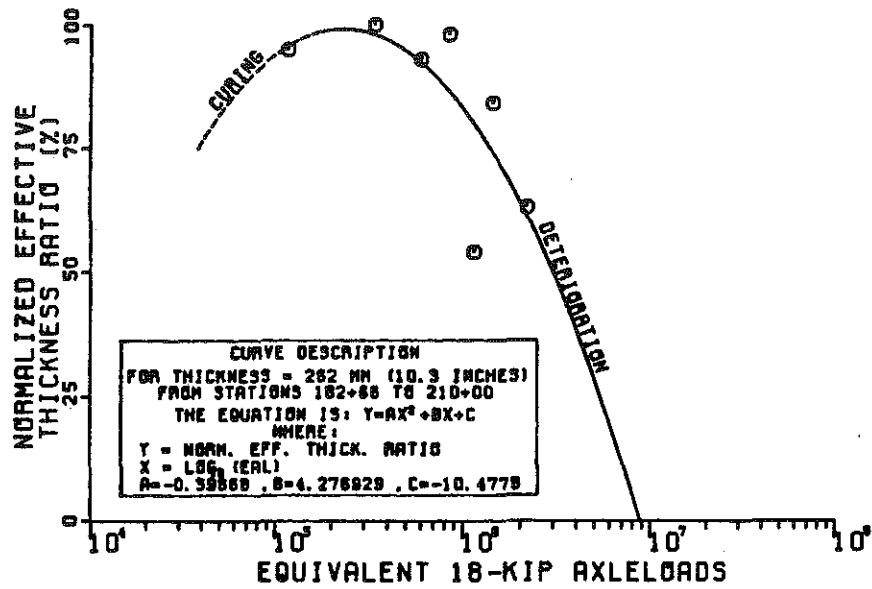


Figure 14. Normalized Effective Thickness versus Accumulated 80-kN (18-kip) Equivalent Axleloads for 262 mm (10.3 inches) of Asphaltic Concrete -- Station 182+66 to 210+00.

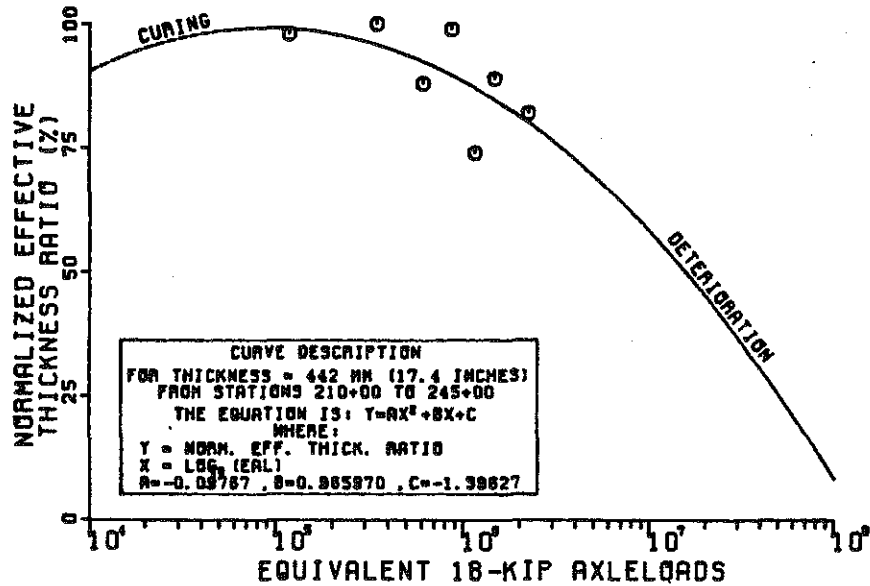


Figure 15. Normalized Effective Thickness versus Accumulated 80-kN (18-kip) Equivalent Axleloads for 442 mm (17.4 inches) of Asphaltic Concrete -- Station 210+00 to 245+00.

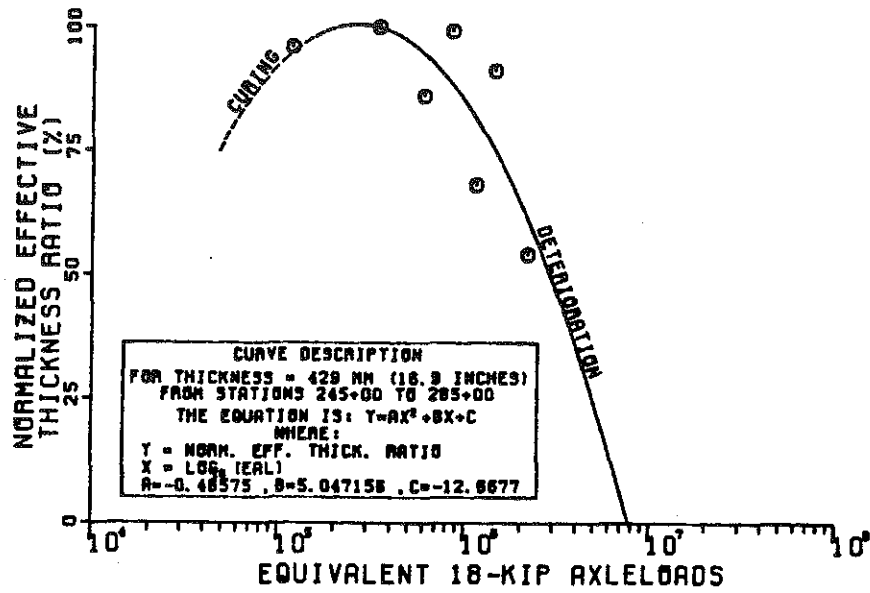


Figure 16. Normalized Effective Thickness versus Accumulated 80-kN (18-kip) Equivalent Axleloads for 429 mm (16.9 inches) of Asphaltic Concrete -- Station 245+00 to 285+00.

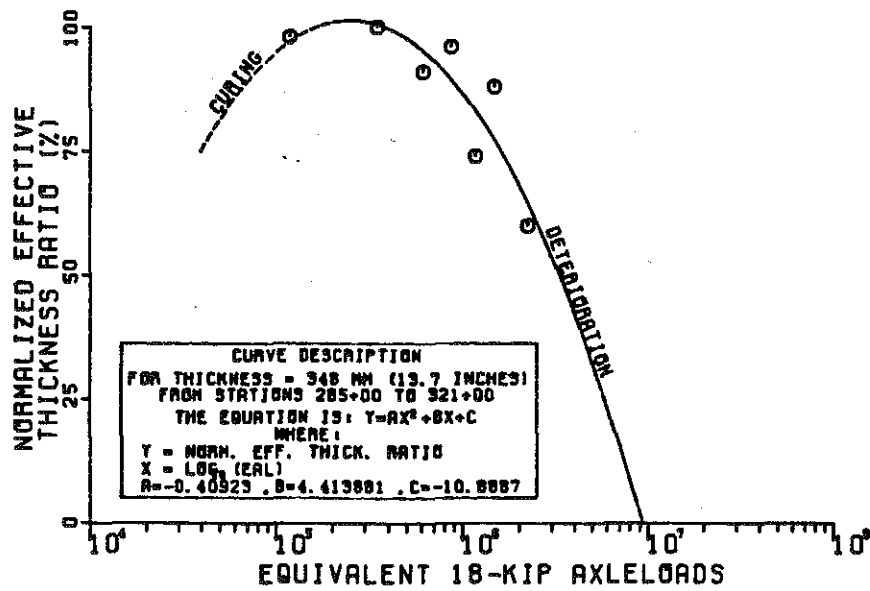


Figure 17. Normalized Effective Thickness versus Accumulated 80-kN (18-kip) Equivalent Axleloads for 348 mm (13.7 inches) of Asphaltic Concrete -- Station 285+00 to 321+00.

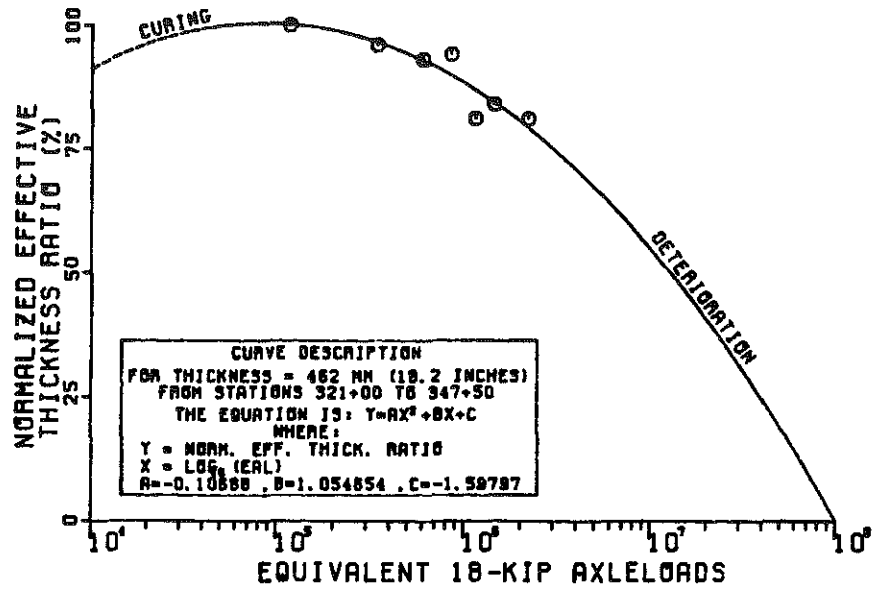


Figure 18. Normalized Effective Thickness versus Accumulated 80-kN (18-kip) Equivalent Axleloads for 462 mm (18.2 inches) of Asphaltic Concrete -- Station 321+00 to 347+50.

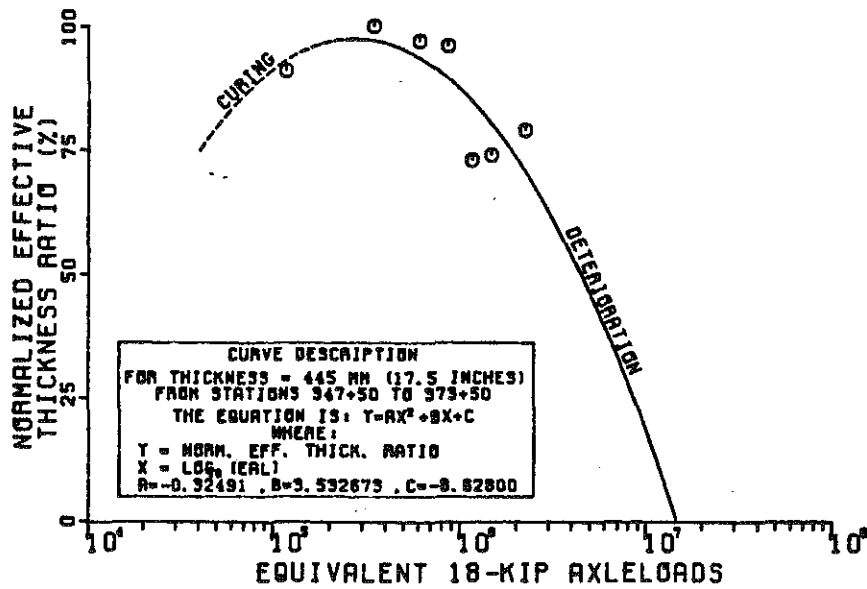


Figure 19. Normalized Effective Thickness versus Accumulated 80-kN (18-kip) Equivalent Axleloads for 445 mm (17.5 inches) of Asphaltic Concrete -- Station 347+50 to 373+50.

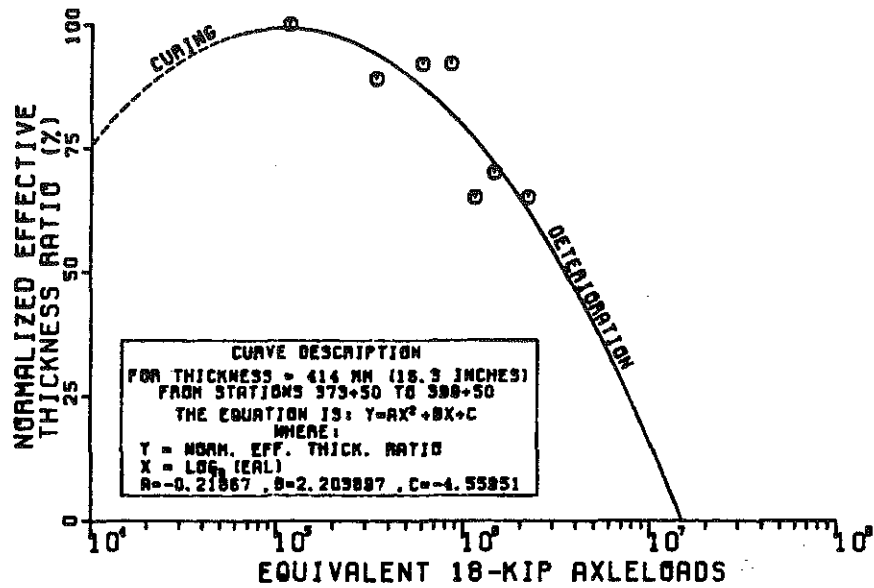


Figure 20. Normalized Effective Thickness versus Accumulated 80-kN (18-kip) Equivalent Axleloads for 414 mm (16.3 inches) of Asphaltic Concrete -- Station 373+50 to 399+50.

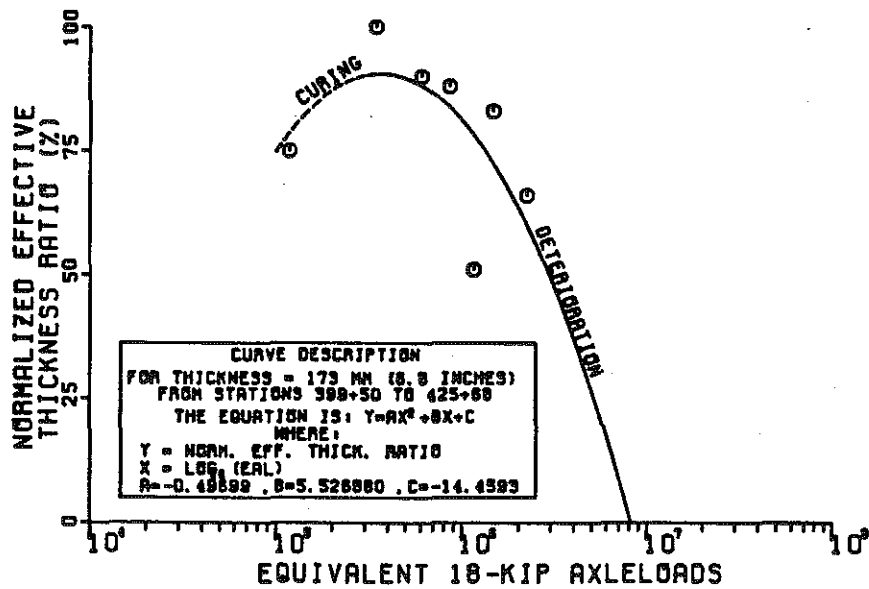


Figure 21. Normalized Effective Thickness versus Accumulated 80-kN (18-kip) Equivalent Axleloads for 173 mm (6.8 inches) of Asphaltic Concrete -- Station 399+50 to 425+68.

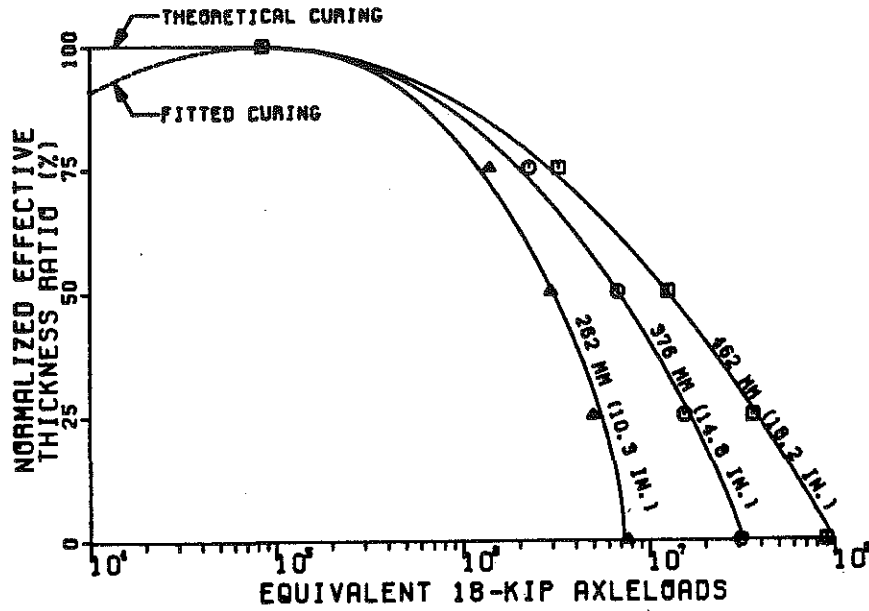


Figure 22. Normalized Effective Thickness versus Accumulated 80-kN (18-kip) Equivalent Axleloads for Constructed Thicknesses of Asphaltic Concrete.

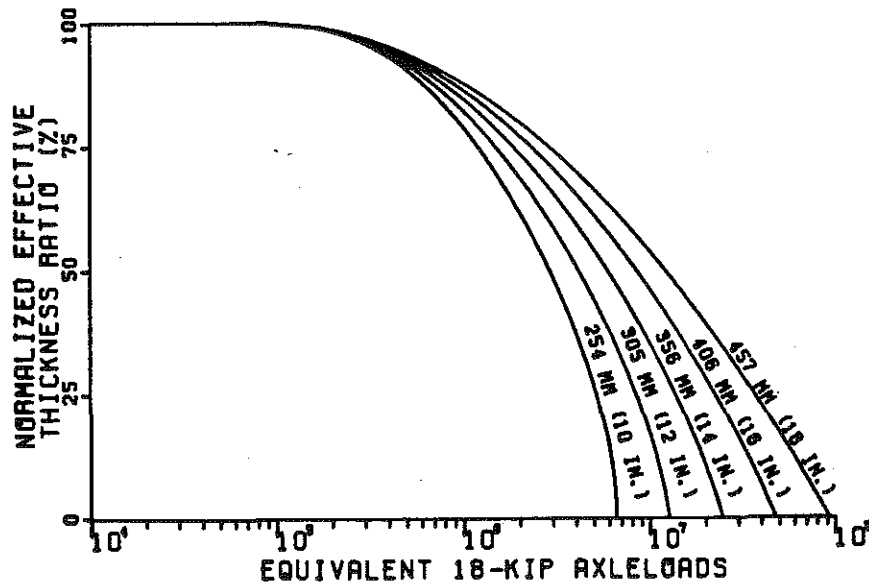


Figure 23. Pavement Deterioration for a Family of Asphaltic Concrete Thicknesses versus Accumulated Pavement Fatigue.

TABLE 3. ESTIMATED FATIGUE LIFE DETERMINED FROM DESIGN AND CORE THICKNESS

STATION		ASPHALTIC CONCRETE THICKNESS (INCHES)		DESIGN CBR	FATIGUE LIFE (MILLION 18-kip EAL)
FROM	TO	DESIGN	CORE		
128+00	155+33	14.0	14.8	9	30
155+33	182+66	12.0	12.0	9	7
182+66	210+00	10.0	10.3	9	2.5
210+00	245+00	18.0	17.4	3	25
245+00	285+00	16.0	16.9	3	22.5
285+00	321+00	14.0	13.7	3	4.5
321+00	347+50	18.0	18.2	3	37.5
347+50	373+50	16.0	17.5	3	25
373+50	399+50	14.0	16.3	3	17.5
399+50	425+68	6.5	6.8	3	2.75

THICKNESS = 376 MM (14.8 INCHES). FROM STATIONS 128+00 TO 155+33

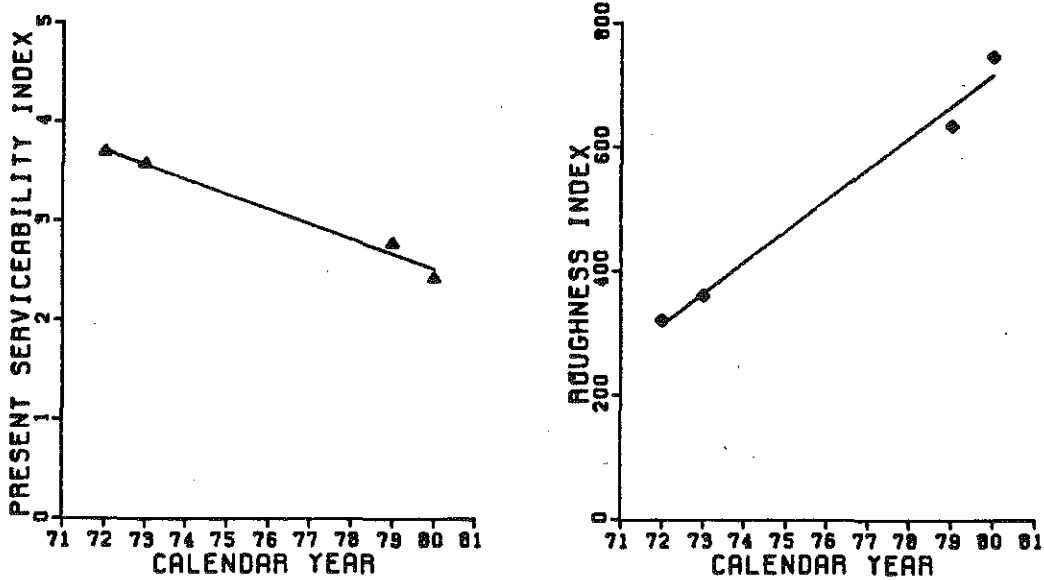


Figure 24. Percent Serviceability Index and Roughness Index versus Calendar Year -- 376 mm (14.8 inches) of Asphaltic Concrete -- Station 128+00 to 155+33.



THICKNESS = 305 MM (12.0 INCHES), FROM STATIONS 155+33 TO 182+66

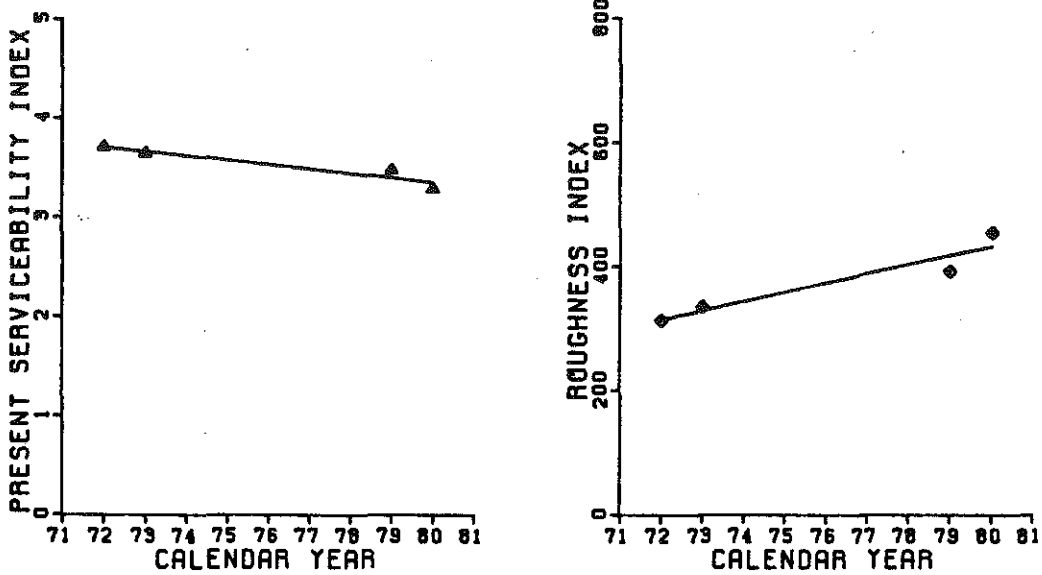


Figure 25. Percent Serviceability Index and Roughness Index versus Calendar Year -- 305 mm (12.0 inches) of Asphaltic Concrete -- Station 155+33 to 182+66.

THICKNESS = 262 MM (10.3 INCHES), FROM STATIONS 182+66 TO 210+00

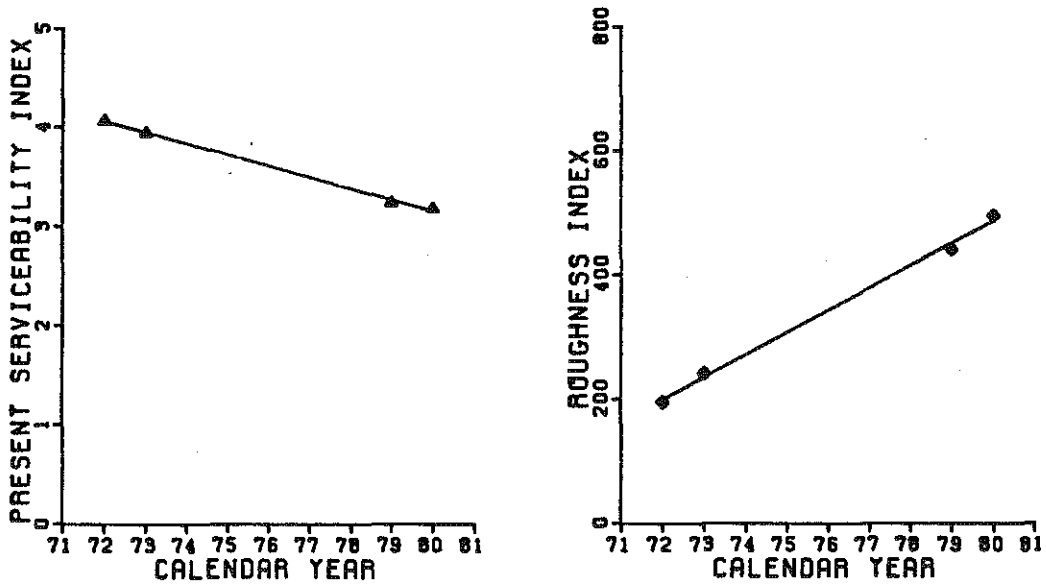


Figure 26. Percent Serviceability Index and Roughness Index versus Calendar Year -- 262 mm (10.3 inches) of Asphaltic Concrete -- Station 182+66 to 210+00.

THICKNESS = 442 MM (17.4 INCHES), FROM STATIONS 210+00 TO 245+00

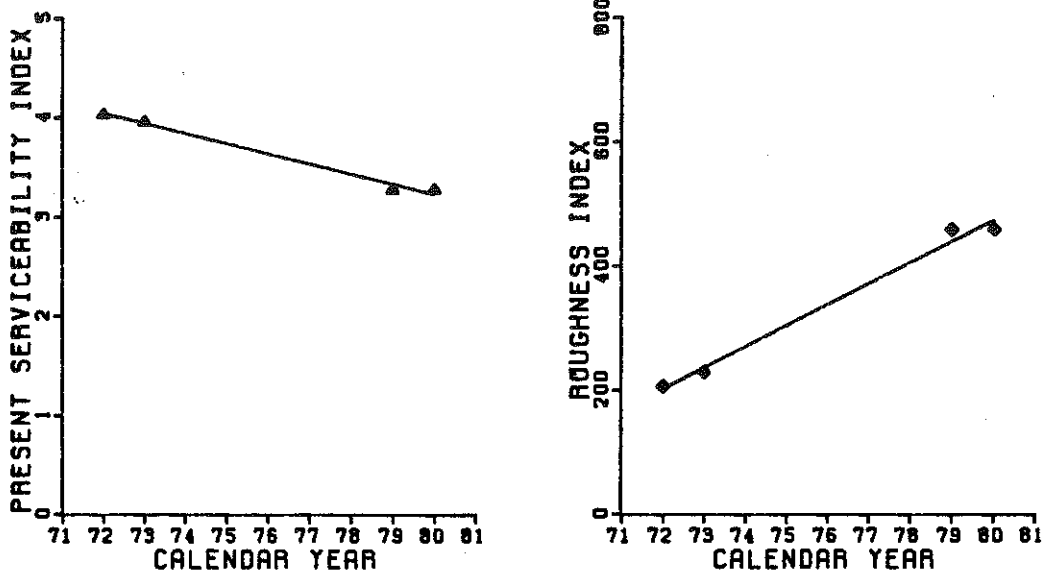


Figure 27. Percent Serviceability Index and Roughness Index versus Calendar Year -- 442 mm (17.4 inches) of Asphaltic Concrete -- Station 210+00 to 245+00.

THICKNESS = 429 MM (16.9 INCHES), FROM STATIONS 245+00 TO 285+00

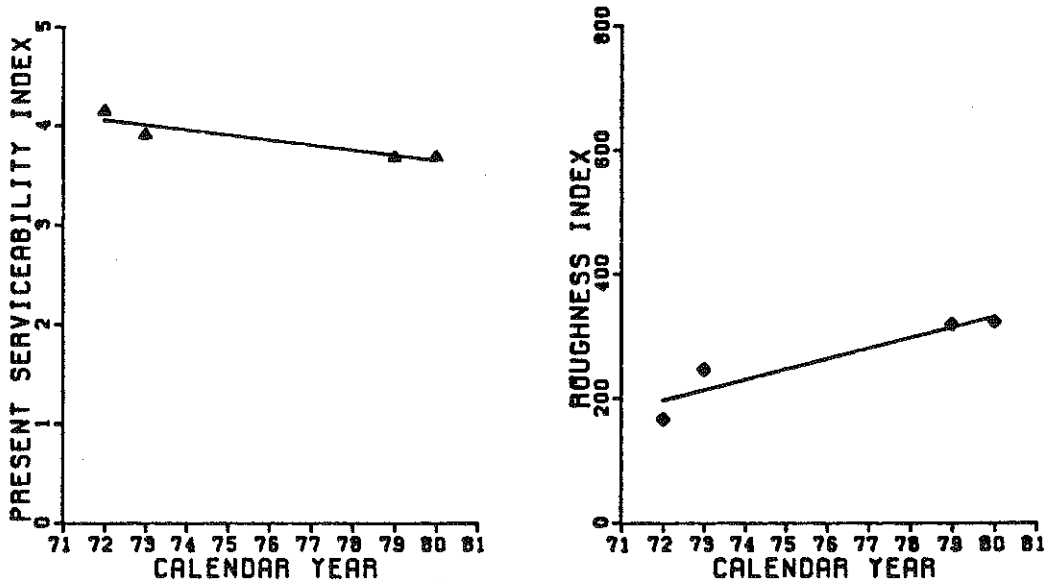


Figure 28. Percent Serviceability Index and Roughness Index versus Calendar Year -- 429 mm (16.9 inches) of Asphaltic Concrete -- Station 245+00 to 285+00.

THICKNESS = 348 MM (13.7 INCHES), FROM STATIONS 285+00 TO 321+00

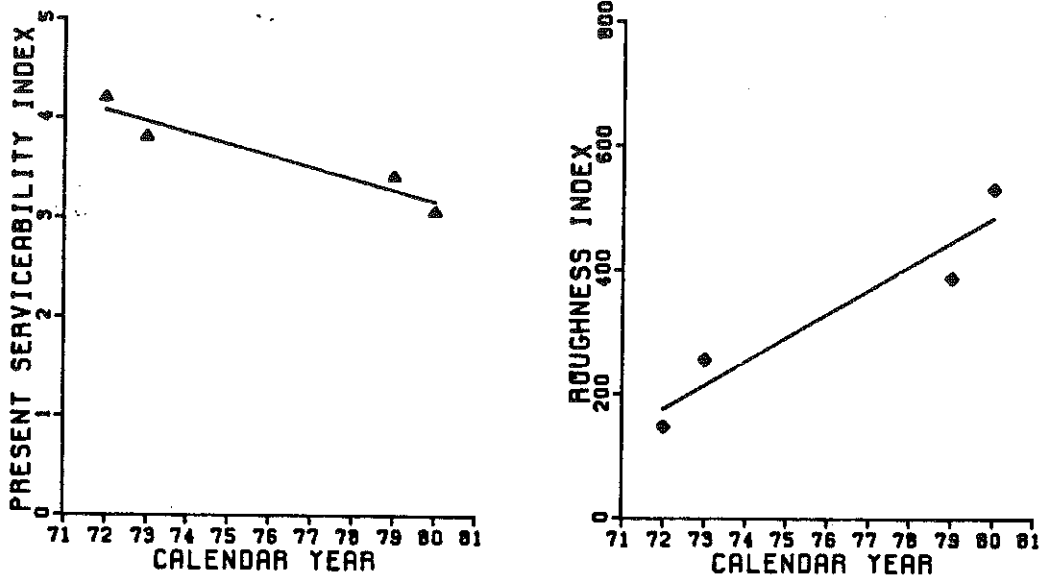


Figure 29. Percent Serviceability Index and Roughness Index versus Calendar Year -- 348 mm (13.7 inches) of Asphaltic Concrete -- Station 285+00 to 321+00.

THICKNESS = 462 MM (18.2 INCHES), FROM STATIONS 321+00 TO 347+50

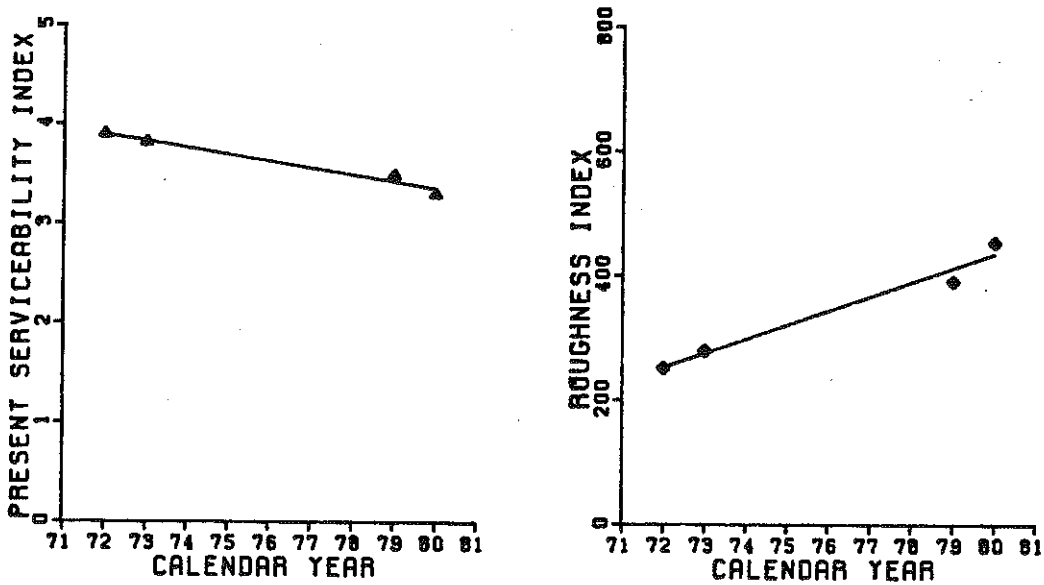


Figure 30. Percent Serviceability Index and Roughness Index versus Calendar Year -- 462 mm (18.2 inches) of Asphaltic Concrete -- Station 321+00 to 347+50.

THICKNESS = 445 MM (17.5 INCHES), FROM STATIONS 347+50 TO 373+50

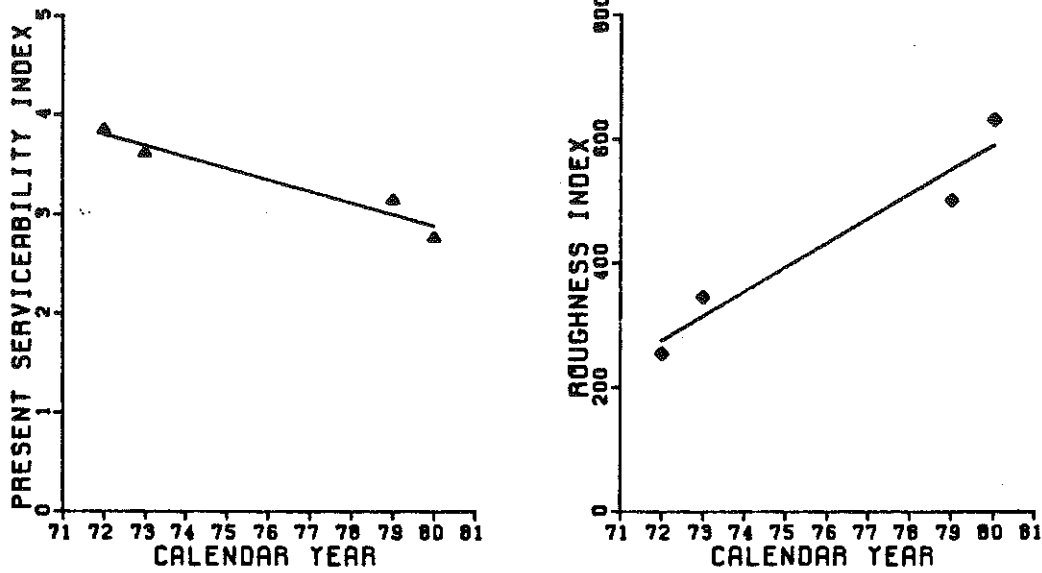


Figure 31. Percent Serviceability Index and Roughness Index versus Calendar Year -- 445 mm (17.5 inches) of Asphaltic Concrete -- Station 347+50 to 373+50.

THICKNESS = 414 MM (16.3 INCHES), FROM STATIONS 373+50 TO 399+50

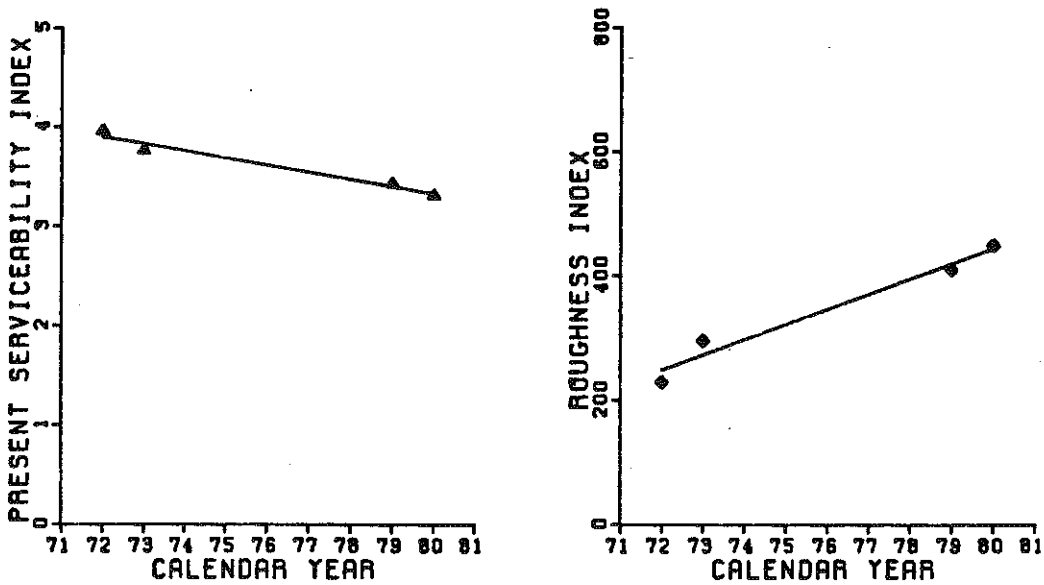


Figure 32. Percent Serviceability Index and Roughness Index versus Calendar Year -- 414 mm (16.3 inches) of Asphaltic Concrete -- Station 373+50 to 399+50.

THICKNESS = 173 MM (6.8 INCHES), FROM STATIONS 399+50 TO 425+68

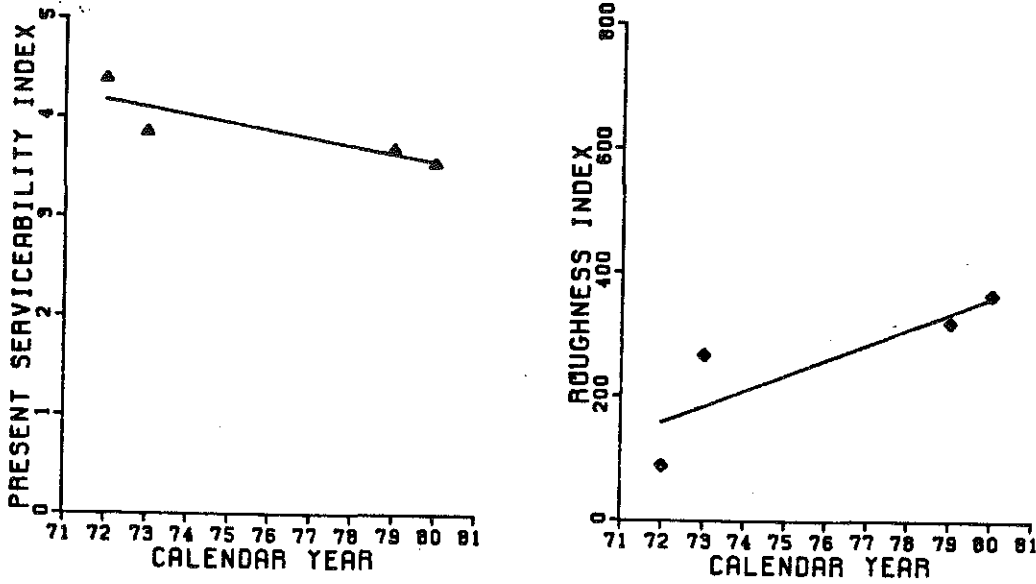


Figure 33. Percent Serviceability Index and Roughness Index versus Calendar Year -- 173 mm (6.8 inches) of Asphaltic Concrete -- Station 399+50 to 425+68.

## BEHAVIORAL THICKNESS

Procedures for evaluating and interpreting pavement deflections were evolutionary during the course of this study (11, 19, 20). Both the shape and magnitude of the deflection bowl, as a function of the constructed thickness of the asphaltic concrete pavement are utilized in current analyses. If measured deflections do not match theoretical deflections for the constructed thickness, a different thickness of pavement, having a reference modulus for the asphaltic concrete, is determined so as to match the measured deflection bowl. A modulus of elasticity that most closely approximates the behavior of asphaltic concrete in Kentucky has a reference value of 3.31 GPa (480 ksi) and, therefore, has been used as the reference modulus for evaluation procedures in Kentucky. In general, the behavioral thickness increased for the first few years because it took that long for the full potential modulus to develop. Thereafter, the behavioral thickness remained essentially constant for two or three years until heavy coal trucks became prevalent on US 60. The effects of fatigue began to show as the behavioral thickness started to decrease, as illustrated in Figures 12 through 21.

Interpretations only in terms of behavioral thickness can be misleading, however. The essential information desired is a combination of the behavioral thickness and the subgrade modulus. Pavements that have been deflection tested before and after placement of an overlay have been used to verify the concept of expressing the condition of an asphaltic concrete pavement as an equivalent thickness of asphaltic concrete on the original thickness of a crushed stone base resting on a subgrade of estimated modulus of elasticity.

## SUBSTITUTION RATIOS

Pavement designs of the US-60 experimental sections were based upon two subgrade CBR values. Each section had a corresponding 51-mm (2-inch) thicker section and thinner section, which resulted in varying substitution ratios (2). Ratios were dependent on CBR, thickness of asphaltic concrete, and certain variations in construction. For US 60, all three variables individually differed so widely and collectively that attempting to validate the original substitution ratios will require more accumulated fatigue and time.

## RUTTING

Shook and Kallas (22) performed a laboratory study relating modulus, percent asphalt cement in the mixture, and void content. A relationship (23) was developed to reflect the change in modulus as a function of degree of compaction and/or variations in asphalt cement content. Many construction projects have shown that a mean compactive effort of 95 percent of Marshall density is achieved typically. During the life of the AASHO Road Test, a 76-mm (3-inch) overlay was placed over a number of sections. A detailed study of the data for the Road Test sections revealed that 3.8-mm (0.15-inch) ruts were measured within 3 months after paving. Assuming the overlay at the AASHO Road Test was compacted to 95 percent, then  $0.05 \times 76 \text{ mm} = 3.8 \text{ mm}$  rut. This concept may explain, in part, the difference between the constructed and behavioral thicknesses for US 60.

Each pavement section was cored upon completion of construction (2) and the thickness of each layer measured. Those thicknesses were used to estimate the potential rutting. Nuclear density meters were used to monitor

compaction during construction. Due to construction schedules and downtime for the density meter, not all layers of asphaltic concrete were tested at all test locations. Therefore, only those stations for which compaction tests were available for all layers were selected for the rutting analysis. Compacted densities of the asphaltic concrete were recorded as a percent of the target density. Potential rutting was calculated by taking the difference between the target density and the achieved density, divided by 100, multiplied by the layer thickness, and accumulated for the total thickness. Table 4 contains representative examples of detailed data used to calculate the potential rutting.

For many test sites, the behavioral thickness was less than the cored thickness. Assuming the same subgrade moduli, the core thickness minus the accumulated potential rutting very nearly equaled the behavioral thickness. Table 5 indicates amazingly close agreement, considering that only one core was taken from each pavement design section.

Measured rutting data compared to potential rutting are presented in Table 6. The potential rut depth was accumulated, layer by layer, from the pavement surface downward until the total thickness at which the accumulated potential rut depth equaled the measured rut depth was obtained. That thickness was divided by the thickness of the asphaltic concrete and expressed as a percentage. The measured rut depth was converted to a percentage of the total potential rut depth. Figure 34 illustrates the relationship between the measured rut depth expressed as a percentage of the total potential rut depth and the thickness to 100-percent compaction expressed as a percentage of the total thickness of asphaltic concrete.

Measured rut depths exceeding the calculated potential rutting are likely related to other factors, which may include shearing and/or plastic flow. Trenches were cut in two pavement sections on US 60. Visual inspections indicated rutting to be restricted to the top one third of the pavement section. Thus, one hypothesis is that rutting greater than the calculated potential may be associated with plastic flow of the pavement structure as related to application of heavily loaded vehicles. Plastic flow may be related to low stability of the asphalt mixture. However data were not obtained in this study to definitely indicate such a conclusion.

In some instances, the surface temperature was below 93.3 C (200 F) when compaction efforts started. In one instance, the initial rolling temperature was 65.5 C (150 F) and decreased to 60 C (140 F) before compaction was completed. The number of passes of the rollers was inconsistent from station to station and section to section. As a result, compacted densities ranged from 82.9 percent to 102.0 percent of the target density. Thus, heavy truck loads probably completed, or partially completed, the compactive effort not obtained during construction.

Rutting measurements made in September 1979 are presented in the third column of Table 6. A leveling course was placed in the wheel tracks shortly thereafter to eliminate the rutting and prevent hydroplaning.

#### PAVEMENT ROUGHNESS AND SERVICEABILITY DATA

Pavement roughness data were obtained for four years between 1972 and 1979 using both the Surface Dynamics Profilometer and the Mays Ridemeter. Data from each device were converted to a roughness index (RI) as described in earlier research (24, 25, 26). Pavement roughness indices were converted to an estimate of pavement serviceability index (PSI). Specific methodologies for this conversion have been presented elsewhere (24, 25,

TABLE 4. RUTTING POTENTIALLY ATTRIBUTED TO DEGREE  
OF COMPACTION DURING CONSTRUCTION

STATION	LAYER		PERCENT COMPACTION (Note 1)	POTENTIAL RUTTING (INCHES) (Note 2)	ACCUMULATED RUTTING (INCHES) (Note 3)
	NO.	THICKNESS (INCHES)			
155+00	S	1.1	94.2	0.0638	0.0638
	4	4.9	93.2	0.3332	0.3970
	3	3.1	89.0	0.3410	0.7380
	2	3.1	94.7	0.1643	0.9023
	1	2.6	94.1	0.1534	1.0557
215+00	S	1.1	95.8	0.0462	0.0462
	5	2.8	90.4	0.2688	0.3150
	4	3.8	94.7	0.2014	0.5164
	3	3.0	91.9	0.2430	0.7594
	2	3.1	91.9	0.2511	1.0105
	1	3.6	92.6	0.2664	1.2769
264+00	S	1.1	95.8	0.0462	0.0462
	5	3.2	98.9	0.0352	0.0814
	4	3.9	86.2	0.5382	0.6196
	3	3.3	91.9	0.2673	0.8869
	2	3.5	89.0	0.3850	1.2719
	1	1.9	89.8	0.1938	1.4657
295+50	S	1.0	95.8	0.0420	0.0420
	4	3.0	91.9	0.2430	0.2850
	3	3.8	98.9	0.0418	0.3268
	2	3.1	94.7	0.1643	0.4911
	1	2.8	91.3	0.2436	0.7347
396+50	S	1.0	94.2	0.0580	0.0580
	4	4.1	90.4	0.3936	0.4516
	3	5.0	93.2	0.3400	0.7916
	2	3.2	96.1	0.1248	0.9164
	1	3.0	92.6	0.2220	1.1384
407+50	S	1.1	96.8	0.0352	0.0352
	2	2.9	89.0	0.3190	0.3542
	1	2.8	92.6	0.2072	0.5614

Note 1. Data from Ref 2

Note 2. Calculated as:  $(100 - \text{Percent Compaction}) \times$   
 $(\text{Layer Thickness}) / 100$

Note 3. Accumulated Potential Rutting = Sum starting at surface  
and extending downward through asphaltic concrete

1 inch = 25.4 millimeters



TABLE 5. CORE THICKNESS RELATED TO BEHAVIORAL THICKNESS AND POTENTIAL RUTTING

STATION	CORE THICKNESS (INCHES)	POTENTIAL RUTTING (INCHES)	CORE MINUS POTENTIAL RUTTING (INCHES)	BEHAVIORAL THICKNESS (INCHES)
135+00	14.8	0.86	13.9	14.0
155+00	14.8	1.06	13.7	13.3
160+00	12.0	0.83	11.2	11.3
215+00	17.4	1.28	16.1	16.0
262+00	16.9	1.01	15.9	15.7
264+00	16.9	1.47	15.4	15.5
271+00	16.9	1.17	15.7	15.7
279+50	16.9	1.39	15.5	15.5
295+50	13.7	0.73	13.0	12.9
299+00	13.7	0.61	13.1	12.0
306+50	13.7	0.75	12.9	13.0
359+00	17.5	1.11	16.4	16.3
360+00	17.5	1.03	16.5	16.3
376+50	16.3	1.22	15.1	15.0
396+50	16.3	1.14	15.2	15.2
403+50	6.8	0.33	6.5	6.5
407+50	6.8	0.56	6.2	5.8
410+00	6.8	0.61	6.2	6.0
414+00	6.8	0.83	6.0	5.8

1 inch = 25.4 millimeters

TABLE 6. RUTTING MEASUREMENTS TAKEN SEPTEMBER 1979  
IN INNER WHEELTRACK OF OUTER LANE

STATION	PAVEMENT THICKNESS (INCHES)	MEASURED RUT DEPTH--INNER WHEEL TRACK (INCHES)	MEASURED RUT AS PERCENTAGE OF POTENTIAL RUT	DEPTH TO FULL CONSOLIDATION AS PERCENTAGE OF TOTAL THICKNESS
135+00	14.8	0.750	87.0*	86.0
140+00	14.8	0.563	42.0	40.5
145+00	14.8	0.813	82.0	71.0
150+00	14.8	0.563	53.0	58.0
155+00	14.8	0.938	89.0	86.5
160+00	12.0	0.875	106.0	100.0+
215+00	17.4	0.625	48.9	52.0
262+00	16.9	0.688	67.8	75.1
264+00	16.9	0.688	46.9	53.5
268+50	16.9	0.813	69.3	85.0
279+50	16.9	0.688	49.6	69.1
289+50	13.7	0.625	93.7	97.0
295+50	13.7	0.688	93.6	96.1
299+00	13.7	0.625	103.0	100.0+
306+00	13.7	0.750	109.0	100.0+
306+50	13.7	0.750	100.0	100.0
307+00	13.7	0.750	86.1	85.0
360+50	17.5	0.375	60.6	62.9
376+50	16.3	0.625	51.4	31.9
396+50	16.3	0.750	65.9	58.2
403+50**	6.8	0.625	192.0	100.0+
405+50**	6.8	0.625	84.4	85.3
407+50**	6.8	0.563	100.3	100.0+
410+00**	6.8	0.625	102.3	100.0+
414+00**	6.8	0.500	60.3	55.4

\* See Table 5 for example of how these figures are calculated

\*\* 6.8 inches asphaltic concrete over  
19.0 inches dense-graded aggregate  
1 inch = 25.4 millimeters

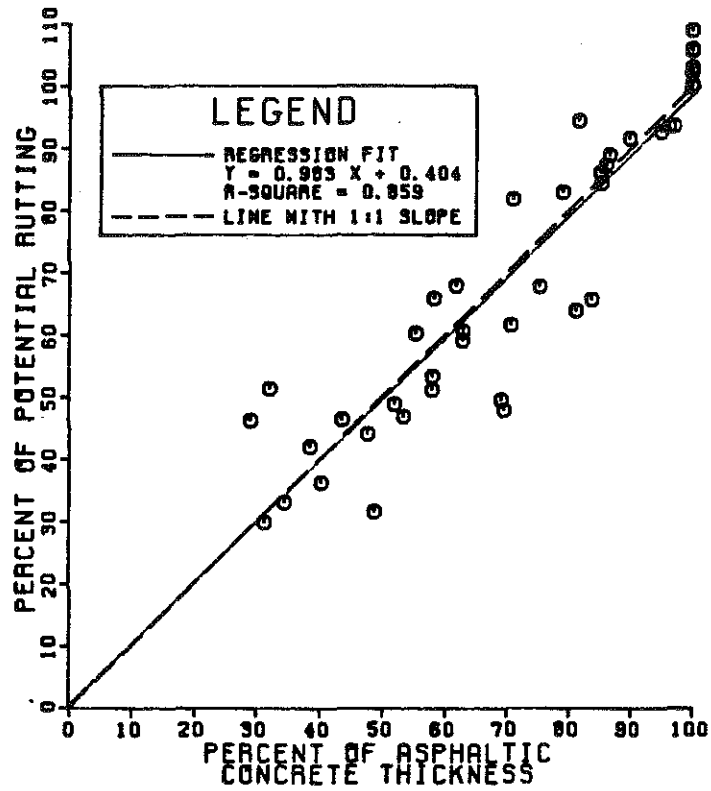


Figure 34. Measured Rut Depth as a Percentage of Potential Rut Depth due to Incomplete Compaction during Construction versus Depth from Surface for Fully Consolidated Asphaltic Concrete as Percentage of Total Thickness of Asphaltic Concrete.

26). Trends of average pavement roughness versus service life and also pavement serviceability versus service life by design section are presented in Figures 24 through 33 for each design section. A scale for accumulated equivalent axleloads (EAL's) may be superimposed on the service life scale, using information presented in Figure 10.

In Figures 24 through 33, the roughness index generally increases with increasing service life (or accumulated EAL's), and the pavement serviceability index decreases in a similar fashion. Linear models generally provided an acceptable description of the variation of the data. Roughness measurements expressed as a pavement serviceability index generally confirmed that the level of serviceability of 3.5 to 3.0 coincided with early visible signs of surface distress.

#### SUMMARY

The weigh-in-motion system purchased for this study would have been adequate in many traffic environments. The system, however, was not designed to withstand the heavy loads on US 60. Thus, the system was abandoned because of frequent equipment failures.

Solar radiation measurements were to have been correlated with measured pavement temperatures. Technical difficulties forced the abandonment of such analyses.

Measured temperature distributions verified the temperature estimation system (3, 4, 5) used in the methodology to evaluate the behavior of the pavement sections.

Construction procedures produced wide variations in the subgrade, the asphaltic concrete mixture, compacted densities within any given layer and between layers, and total thickness. Those variations may be related to variations in behavioral thicknesses and subgrade moduli. However, for the same subgrade moduli (design CBR), construction variations in compaction and asphalt content may have contributed to decreases in thickness (consolidation) under traffic loadings until it very nearly matched the behavioral thickness. Construction specifications should contain a minimum temperature for the compaction process. It is recommended that the temperature of the uncompacted asphaltic concrete should be no less than 105 C (220 F), and compaction should be completed before the temperature drops below 82.2 C (180 F).

Compaction tests using the nuclear density meter indicated the degree of compaction ranged from 82 to 102 percent of Marshall density. The degree of compaction obtained during construction can be used to calculate the potential consolidation (rutting) that may occur under traffic. Since the measured rutting was closely approximated by this measure of potential rutting, rutting on this project was probably the result of consolidation rather than displacement (shearing).

The first or lower layer of asphaltic concrete constructed was 51 mm (2 inches), 76 mm (3 inches), or 102 mm (4 inches), depending upon the test section. For those sections having a 51-mm (2-inch) first lift, the first lift moved vertically and formed a wave in front of the tires as loaded asphaltic concrete trucks backed to the paver while laying the second lift. Thus, the first layer may have been severely cracked and its assigned fatigue life virtually destroyed by construction traffic. The wave was still noticeable, but much less pronounced, on those sections having a 76-mm

(3-inch) first lift. No movement was observed on those sections having a first lift thickness of 102 mm (4 inches). Therefore, for full-depth pavements, it is recommended that the first lift have a compacted thickness of 102 mm (4 inches).

Traffic volume, classification counts, limited weight data from the weigh-in-motion system, and an updated and expanded set of damage factors for various axle configurations combined to verify the full-depth thickness design curves. Measured ruts and roughness measurements combined to add additional data in verifying the pavement serviceability rating system.

During construction of the pavement on US 60, vibratory rollers were new pieces of construction equipment. One vibratory roller was evaluated on this project and accepted for general use within the Commonwealth of Kentucky (12). Three important findings were 1) the vibrator should not be used on the final surface course, 2) the vibrator should not be used when climbing relatively steep grades such as crossovers where one lane is considerably higher in elevation than the other, and 3) the forward speed of the roller should be limited to minimize a rippled surface.

#### IMPLEMENTATION OF RESEARCH

This research project resulted in implementation of the following items.

Vibratory rollers were evaluated and accepted for general and routine use in Kentucky.

Dynamic deflections as measured by the Road Rater have been used to determine the behavioral thickness of asphaltic concrete pavements. Analyses of data from this study provided for the evaluation of pavement evaluation methodologies. The current analysis procedure has been proven over wide ranges of subgrade conditions, from thin to moderately thick pavement sections, for pavements with and without dense-graded aggregate bases, and from tests at the same locations "before and after" overlay construction.

Evaluation methods to interpret measured deflections and to quantify them in terms of behavioral thicknesses and estimated subgrade moduli resulted in their acceptance as a tool in the annual rehabilitation program within the pavement management program of the Kentucky Transportation Cabinet. City streets have been tested and evaluated. In Kentucky, significant savings or more effective utilization of funds have resulted either by placing thinner overlays than had been anticipated by more traditional approaches, or by requiring greater thicknesses than had been anticipated so as to prevent or minimize premature failures. In Tennessee, savings resulted when thinner overlays than had been anticipated were recommended.

Confirmation of the fatigue relationship for asphaltic concrete pavements has permitted the development of criteria used to develop thickness design methods for other pavement systems (for example, portland cement concrete pavements (27, 28), for asphaltic concrete pavements having a pozzolanic base (29), and for evaluating fragmented portland cement concrete as input to an overlay design). Results of those analyses eventually will permit the development of a rational overlay design method for composite pavements.

It has been shown that the observed behavior of pavements at the AASHO Road Test was a function of a variable level of serviceability (30), as

verified by some 40 years of Kentucky experience in pavement testing and thickness design. Thus, one thickness design nomograph may be developed to incorporate a variable level of serviceability as a function of design EAL. The Kentucky design curves do, in fact, incorporate this concept (13, 16, 17, 30).

Heavily loaded trucks with new axle configurations necessitated the reassessment of load equivalency relationships for steering axles, single axles, tandem axles, and tridem axle groups. The need to determine their effect upon the rate of accumulation of pavement fatigue was a direct result of analyses of weigh-in-motion data obtained in this study. Adjustment factors were developed to account for uneven load distributions on the axles within the tandem (6) and tridem axle assemblies (7). Confirmation of their effect was verified using data obtained from the Office of Planning, Federal Highway Administration.

Data from the AASHO Road Test were analyzed using elastic layer theory matched with observed behavior. Thus, the Kentucky flexible pavement thickness design curves have been confirmed by over 40 years of empirical testing, by recent dynamic testing methods, and by observed behavior at the AASHO Road Test, all of which have been confirmed by elastic layer theory.

Pavement behavior may be defined using the classical definition of "work." All components of strain or stress may be combined into one value having the same order of magnitude as any one component. Also, a direct correlation between any component of strain and "work strain" permits translation and transformation of past laboratory results and experience into a design system based upon "work."

The concept of "work" as a foundation for pavement design methods has been applied to the development of thickness design systems for both asphaltic concrete, portland cement concrete (27, 28), and pavement structures using pozzolanic materials (29). In the case of portland cement concrete, the concept of "work" provided the key to merging the design criteria of the Portland Cement Association and the American Association of State Highway and Transportation Officials into one criterion (27, 28). The concept of "work" is being used to develop criteria for composite pavement design and overlay design for fractured portland cement concrete pavements.

Implementation of research is sometimes difficult if not impossible to quantify in terms of monetary advantages. Improved construction techniques may certainly be expected to increase the life and/or improve the performance of a pavement section. Techniques for evaluation of pavements will permit more efficient assessment of the condition of deteriorating pavement sections but is almost impossible to quantify in terms of dollar benefits. Finally, expanded experience and the development of a pavement data bank is invaluable for future work in the areas of pavement management, performance, and evaluation.

#### FUTURE DEVELOPMENTS

Results from this study will permit the development of a rational overlay design procedure for composite pavements for asphaltic concrete over broken, or unbroken, portland cement concrete pavement and/or other non-asphaltic but bound base materials.

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APPENDIX

## DESCRIPTION OF DETERIORATION CURVE FAMILY

Effective thickness values were calculated based on Road Rater testing. These values were adjusted to reference conditions and presented as normalized ratios (maximum value set to unity). These ratios were plotted against the log of the accumulated equivalent 80-kN (18-kip) axleloads (EAL's) for the outside lanes. Quadratic curves were fitted to the data by regression analyses. To develop a family of deterioration curves, three curves were chosen as representative of the overall behavior. The selected curves were for the maximum, the minimum, and the mean thicknesses of full-depth asphaltic concrete. For each of the three thicknesses, values of EAL were calculated from the fitted curves for ratios equal to 0.00, 0.25, 0.50, and 0.75. Thickness versus log of EAL were plotted for each ratio, and lines were fitted by regression analyses to obtain a slope for each ratio (Figures A-1, A-2, A-3, and A-4). The slopes were not constant, but appeared to form a smooth curve when ratio was plotted versus slope (Figure A-5). The functional relationship of this curve approximated a trigonometric tangent function. A regression line was used to fit a plot of slope versus tangent of ratio times 90 degrees (Figure A-6). The family has the following equation:

$$\log(\text{EAL}) = f(\text{ratio}) + (T - 462)/(g(\text{ratio}))$$

in which :

$f(\text{ratio})$  = quadratic equation solution for the deterioration curve for thickness of 462 mm (18.2 inches),

$T$  = thickness of full-depth asphaltic concrete (mm), and

$$g(\text{ratio}) = 187.18774 + 144.39143 \tan (\text{ratio} * 0.5 * 3.14159).$$

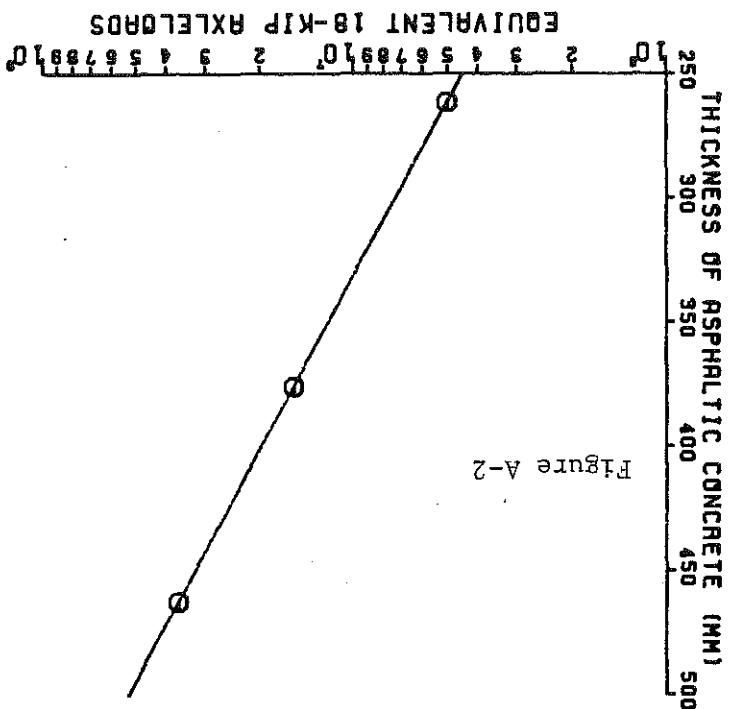


Figure A-2

NORMALIZED EFFECTIVE THICKNESS RATIO = 0.25  
 SLOPE = 299 MM / CYCLE

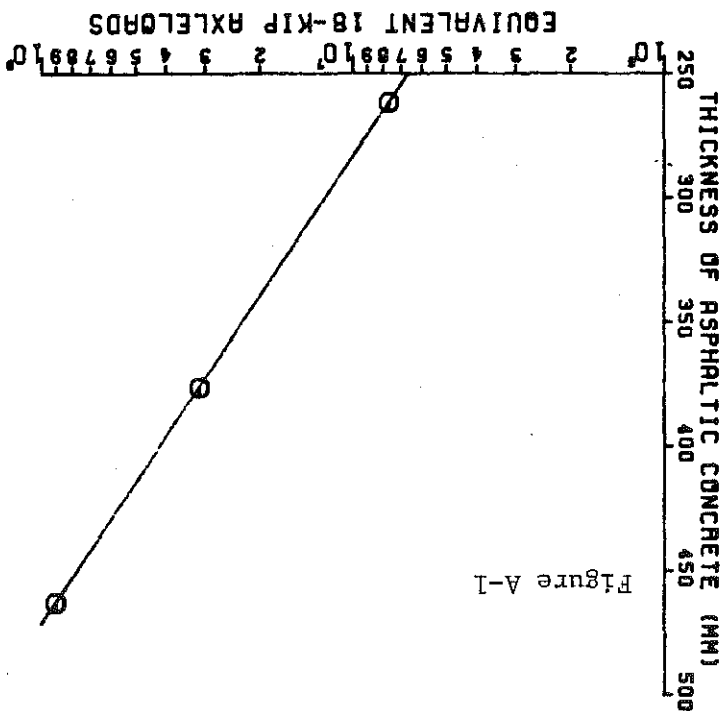
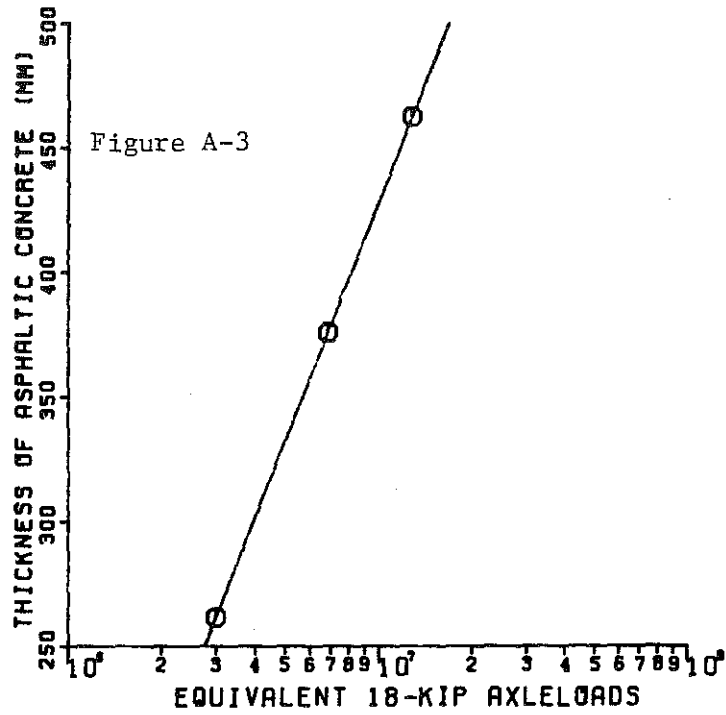


Figure A-1

NORMALIZED EFFECTIVE THICKNESS RATIO = 0  
 SLOPE = 187 MM / CYCLE

NORMALIZED EFFECTIVE THICKNESS RATIO = 0.50  
SLOPE = 319 MM / CYCLE



NORMALIZED EFFECTIVE THICKNESS RATIO = 0.75  
SLOPE = 544 MM / CYCLE

