LABORATORY AND FIELD EVALUATIONS AND CORRELATIONS OF PROPERTIES OF PAVEMENT COMPONENTS

by

David L. Allen
Chief Research Engineer

R. Clark Graves
Research Engineer

and

L. John Fleckenstein
Research Investigator

Kentucky Transportation Center
College of Engineering
University of Kentucky

in cooperation with
Transportation Cabinet
Commonwealth of Kentucky

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David L. Allen, R. Clark Graves, and L. John Fleckenstein

Kentucky Transportation Center
College of Engineering
University of Kentucky
Lexington, KY 40506-0043

Kentucky Transportation Cabinet
State Office Building
Frankfort, KY 40622

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This report documents the selection and sampling of 21 long-term pavement performance testing sites. Seven of those sites were also chosen to be a part of the LTPP portion of the Strategic Highway Research Program (SHRP). All sites were sampled in accordance with SHRP protocol and procedures.

The report includes three years of visual distress data and pavement deflection data obtained by a Road Rater. There were wide variabilities in backcalculated subgrade moduli and asphaltic concrete moduli within each 500-foot test section.

Laboratory resilient modulus tests were performed on undisturbed samples of the subgrade and asphaltic concrete cores. Again, there was wide variability between laboratory resilient moduli and backcalculated moduli from field deflection.

A number of models were developed to permit estimation of pavement distresses with time, AADT's, or ESAL's. In addition, relationships were developed between laboratory and field data.

It is recommended that longer-term data be obtained, to refine the models and to make their estimations more accurate. The models developed in this study can estimate the service history of a pavement.

Key Words:
- backcalculate
- performance models
- resilient modulus
- Road Rater

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EXECUTIVE SUMMARY

The technology for collection, evaluation, and interpretation of pavement deflection measurements has been evolving at a rapid rate for the past decade. Similarly procedures for evaluation of pavement materials in the laboratory have also progressed significantly. The technology associated with deflection testing equipment and laboratory testing equipment has also evolved in a similar fashion.

Early techniques for evaluation of paving materials typically involved static or near static testing and analyses procedures. More recently, field evaluation techniques have involved dynamic testing and analyses. Dynamic testing equipment may be generally grouped into two broad categories: vibratory testers and impulse testers. Vibratory testers involve such commercially available devices as the Dynaflect and Road Rater. Impulse testing devices typically involve some type of falling weight deflectometer apparatus whereby an impulse load is applied to the pavement by dropping a mass some fixed distance.

Deflection measurements from dynamic loading have been used in Kentucky since 1971 for evaluation of pavements. Early evaluation procedures typically involved the use of relative comparisons of deflection measurements from one location to another. Those evaluation procedures were very successful in terms of locating weak areas relative to strong areas. Later, research and development activities involved the use of deflection analyses to backcalculate effective pavement layer moduli. Elastic layer theory has been used to model deflection measurements obtained from the Road Rater and other deflection testing devices. Procedures have been developed whereby modelled theoretical deflections may be used to backcalculate effective pavement conditions for both asphaltic concrete and Portland cement concrete pavements.

Researchers in Kentucky have not attempted to directly correlate backcalculated modulus obtained from field deflections measurements with moduli values obtained from performance of standard laboratory procedures on laboratory specimens. This study was initiated to attempt to relate laboratory values of moduli to backcalculated modulus values. The general objectives of the study were as follows:

1. To develop correlations of pavement material properties backcalculated from field deflections measurements with properties of pavement materials obtained from laboratory testing and analyses.

2. To review literature and conduct necessary evaluations to determine whether other procedures (other than layer elastic theory) for modeling pavement deflections result in more appropriate correlations of backcalculated properties with laboratory defined properties.

3. To determine relationships between deflections (and change in deflection
It was originally intended to collect field data from 50 to 100 locations distributed throughout the state. However, the Study Advisory Committee concluded that many sites would require more manpower, time, and funds than were available; therefore, the number of study sites was reduced to 20.

In 1987, the United States Congress passed the Surface Transportation Act. Included as a part of that act was 150 million dollars for the Strategic Highway Research Program (SHRP). A large portion of SHRP was to be the long-term performance monitoring of thousands of sections of in-service highways located throughout the nation. This was referred to as the LTPP portion of SHRP. Sections that were to be monitored were referred to as GPS (General Pavement Studies) sections.

To assist SHRP in choosing these GPS sections, the states were to submit a list of possible candidate sections for consideration. A master list of Kentucky pavements was assembled and sent to SHRP for their use. SHRP narrowed the list of possible candidates, and returned this shortened list along with a data form on which to assemble specific test data for that individual candidate section. Ultimately, seven sites were chosen in Kentucky to be a part of the national LTPP-GPS program.

The Study Advisory Committee concluded that these seven sites should be included as a part of this study. An additional 13 sites were then chosen (using the same criteria as those used in choosing the GPS sections of the LTPP program of SHRP) to be monitored in an identical fashion as the national LTPP sites.

The Kentucky Transportation Center obtained cores at each 500-foot long test site, in 1989. In addition, moisture samples of the DGA and the subgrade at the 100- and 400-foot marks were obtained. Two CBR tests were performed at both of these areas. Shelby tubes were also collected where suitable subgrade was present. No trenching operations were conducted by the Kentucky Transportation Center. SHRP excavated a 4-foot by 4-foot test pit at each asphaltic concrete site. Nuclear density tests were performed on the DGA and/or subgrade at each site. Bag samples were obtained of the subgrade and the DGA. Jar moisture contents were also obtained. Shelby tubes or standard penetration tests were obtained at each site. A 20-foot auger boring was also performed at each site on the shoulder of the road to verify bedrock depth.

The Kentucky Transportation Center conducted visual distress surveys for 1989, 1990, and 1991. Each site was inspected for any sign of surface distress, such as rutting; transverse, longitudinal, and/or alligator cracking; spalling; ravelling; bleeding; and pumping. Each site was video taped, and surface distress was
The Kentucky Transportation Center performed deflection tests using both the FWD and the Road Rater. In this report, Road Rater deflections were compared using a 600-pound load with Road Rater deflections obtained using other load levels. Also, FWD deflections obtained using a 6,000-pound load are compared with FWD deflections obtained using other load levels. Finally, correlations were made between FWD deflections and Road Rater deflections.

Correlations between load levels of the FWD are less variable than are correlations between load levels of the Road Rater. This indicates that FWD data have less experimental error than do Road Rater Data.

Sensor No. 1 of the FWD yields a disproportionately larger deflection reading than does Sensor No. 1 of the Road Rater (when the different load magnitudes are scaled to the same magnitude). This is partially due to the location of the sensors, and partially due to the nonlinear behavior of the pavement structure.

The correlation between the No. 4 sensors on the two testing devices is very poor. This is primarily due to the large variation (scatter) in the data from the No. 4 sensor of the Road Rater.

Until further research is conducted, it appears that correlations between the two devices should be performed on a sensor-by-sensor basis and not by combining the data from the first four sensors into one data set and performing correlations on the entire data set.

Deflection data obtained from the Road Rater were used to backcalculate subgrade moduli. Values of subgrade moduli varied widely within the 500-foot length of the test section. Backcalculated values of subgrade moduli using deflections from the 600-pound loads were generally higher than values calculated from the 1,200-pound and 1,800-pound loads. From this, it may be concluded that a 1,200-pound load is likely to yield the most consistent results when using the Model 400 Road Rater. An in-depth study has not been conducted to confirm and explain this result; however, it would appear the 600-pound load may not produce deflections that are sufficient to be within the appropriate operating range of the sensors. The 1,800-pound load may be sufficient to produce extraneous machine vibrations that the sensors are reading.

It appears subgrade modulus may be somewhat related to rainfall pattern, with a nine- to 12-month lag. This apparent relationship developed from the data obtained from both testing devices, and therefore, does not appear to be machine related.

In general, subgrade modulus values backcalculated from the FWD are greater
than the values calculated from Road Rater data.

A second objective of this study was to compare backcalculated field moduli values with modulus values determined from laboratory tests. Resilient modulus tests were performed on subgrade samples collected from the sites that had soil subgrades. The testing standard used was AASHTO T-274. There is a reasonably good correlation between the backcalculated field modulus from the Road Rater and the laboratory modulus ($R^2 = 0.66$). A relationship was developed to relate field modulus to laboratory modulus.

There is little or no correlation between laboratory measured subgrade moisture content and resilient modulus. There is no correlation between density of laboratory samples and resilient modulus. There was an inverse relationship between field CBR values and laboratory resilient modulus.

There was a very large variability in backcalculated AC modulus for each 500-foot section. It appears the variability was greater for thicker pavements. This may indicate the Road Rater is not capable of inputting sufficient energy into the pavement to deflections that are within the range of precision of the deflection sensors.

The data developed in this study may assist the designer in assessing the reliability of design.

There is a large variability between the backcalculated AC modulus and the laboratory resilient modulus. This was expected, in view of the variability of the backcalculated field AC modulus.

A simple method was developed to convert from backcalculated AC modulus at pavement test temperature to the standard reference temperature of 70°F.

The rutting model developed under another study and referenced in this study predicted reasonably well rutting on all of the test sites. As more long-term data become available, this model will be refined. The average error of prediction was slightly over 1/16 inch. The model appears to predict equally well at all levels of ESAL’s.

Age is a better predictor of longitudinal cracking and transverse cracking than are ESAL’s or AADT’s.

The change in Rideability Index with time may be estimated from the data obtained in this study. The use of these models may be limited because of the small sample size from which they were developed.

The models developed in this study may be used to estimated the various modes of distress and to estimate the change in parameters with time; however, it
is critical that more long term data be obtained, to refine these models and to make estimations that are more accurate.

It is apparent more sophisticated modeling techniques would enhance predictive capabilities. It is recommended that further work be performed in this area to develop these models.

It is recommended that the data base that has been developed to date on these test sites be used as a basis for continuing to monitor and record behavior. It is recommended that these sites continue to be monitored annually (as is currently being done). From this long-term data, it will be possible to continue to develop and refine the models developed in this study. The refined models will more accurately predict the service life of all pavements in the future.

As a part of the process of refining these models, and continuing implementation a computer program will be developed that will include all the models in one interactive program. The designer or pavement manager will then be able to estimate the complete service history of a proposed new design or the remaining service history of an in-service pavement.
INTRODUCTION

The technology for collection, evaluation, and interpretation of pavement deflection measurements has been evolving at a rapid rate for the past decade. Similarly procedures for evaluation of pavement materials in the laboratory have also progressed significantly. The technology associated with deflection testing equipment and laboratory testing equipment has also evolved in a similar fashion.

Early techniques for evaluation of paving materials typically involved static or near static testing and analyses procedures. Examples include: unconfined compression testing, California Bearing Ratio tests, plate load testing, and deflection testing using the Benkelman beam. Early pavement design and performance experience was based almost entirely upon this type of information, and this information is still the primary basis for pavement design and performance evaluations.

More recently, field evaluation techniques have involved dynamic testing and analyses. Dynamic testing equipment may be generally grouped into two broad categories: vibratory testers and impulse testers. Vibratory testers involve such commercially available devices as the Dynaflect and Road Rater as well as custom developed devices such as the vibratory devices developed by the Army Corps of Engineers and the Federal Highway Administration. Impulse testing devices typically involve some type of falling weight deflectometer apparatus whereby an impulse loading is applied to the pavement by dropping a mass some fixed distance to develop an impulse load. The general concept behind any of the dynamic testers is to develop a loading application simulating or perhaps duplicating typical traffic loadings applied to in-service pavements.

Recent research activities have involved comparisons of static analyses versus dynamic analyses for both field and laboratory evaluations. The observed variations have resulted in considerable debate regarding appropriate criteria for pavement design and also evaluation of pavement quality control.

Deflection measurements have been used in Kentucky since 1971 for evaluation of pavements. Early evaluation procedures typically involved the use of relative comparisons of deflection measurements from one location to another. Those evaluation procedures were very successful in terms of locating weak areas relative to strong areas. Later, research and development activities involved the use of deflection analyses to back-calculate effective pavement layer moduli. Elastic layer theory has been used to model deflection measurements obtained from the Road Rater and other deflection testing devices. Procedures have been developed whereby modelled theoretical deflections may be used to backcalculate effective pavement conditions for both asphaltic concrete and Portland cement concrete pavements.

Research has demonstrated that deflection measurements may vary from
theoretically calculated deflections (using elastic layer principles) because of nonlinear stress dependent characteristics associated with asphalt-bound materials, granular bases, and subgrades. Project analyses have indicated that laboratory analyses of pavement material properties may be correlated with backcalculated pavement material properties.

Researchers in Kentucky have not attempted to directly correlate backcalculated modulus obtained from field deflections with moduli values obtained from performance of standard laboratory procedures on laboratory specimens. This study was initiated to attempt to relate laboratory values of moduli to backcalculated modulus values. The general objectives of the study are as follows:

(1) To develop correlations of pavement material properties backcalculated from field deflections measurements with properties of pavement materials obtained from laboratory testing and analyses.

(2) To review literature and conduct necessary evaluations to determine whether other procedures (other than layer elastic theory) for modeling pavement deflections result in more appropriate correlations of backcalculated properties with laboratory defined properties.

(3) To determine relationships between deflections (and change in deflection with time) and laboratory determined properties of pavement components (and changes in properties with time).

BACKGROUND

Backcalculation Methods

There are several methods of determining subgrade strength and effective structural condition of pavements through backcalculation. Lytton (1) discusses several methods of backcalculation, including some historical methods, microcomputer methods, impulse and response analysis methods, and systems identification methods.

The concepts of deflection basin area and basin shape factors have been documented by Thompson (2). This method uses regression equations to calculate nonlinear resilient layer moduli from deflection basin area, shape factors, and center deflection. This method was developed for use in the absence of expensive computer analysis.

Hossain and Zaniewski (3) discuss several methods that have been used for evaluation deflection basins. They also evaluated deflection basins using an exponential curve of the form $Y=Ae^{BX}$, where $Y$ is the deflection (mils), and $X$ is the radial distance from the center of the load. Coefficients $A$ and $B$ are functions of the
pavement structure. It was found that the degree of fit of the equation was useful for judging the ability of the deflection bowl to be evaluated by a deflection matching technique.

Mahoney, Coetzee, Stubstad, and Lee (4) used six separate techniques to evaluate layer moduli for five test sites in Washington State. The backcalculation techniques and results of the laboratory moduli, and the overlay thicknesses were generally in agreement but with some notable exceptions.

Zhou et al. (5) have developed a microcomputer program based on equivalent thickness and Boussinesq theory. The program can quickly and accurately backcalculate layer moduli.

It is generally known the backcalculated moduli values are dependent upon temperature, frequency, and load levels. Southgate et al. (6) developed a method for correcting moduli values for temperature and frequency. Germann and Lytton (7) have also published a method for correcting moduli values for temperature and frequency. In addition, the same authors reported a method for correcting for load level using stress-strain data that had been converted to hyperbolic functions.

Resilient Modulus

Elliott and Thornton (8) concluded in their research that the resilient modulus is a fundamental material property and that it provides a measure of the load-induced stress-strain behavior of the subbase (soil and granular subbase layers). This behavior in turn governs the response of a load on the pavement system. They also concluded that resilient modulus should not be the only judging or controlling property when evaluating a soil or granular material since resilient modulus provides no means for evaluating rutting (permanent deformation). They also pointed out that there are several factors that affect the resilient modulus causing it to vary seasonally throughout the pavement life. Therefore, it is difficult to select one single soil resilient modulus for design.

They also concluded that, on paper, the AASHO Guide selection procedure is straightforward, but the estimation of seasonal moisture variation involves a lengthy amount of testing. They suggest that a more practical approach is to test at a single period, most closely representing average conditions on a yearly basis, such as late spring.

Vinson (9) discuss the work of several researchers and practitioners to provide a sound understanding of the fundamentals of resilient modulus testing. They concluded that if the tests are properly conducted on specimens that represent in-situ conditions, and that tests will provided resilient modulus values which then may be used in the mechanistic pavement design approach.
Monismith (10) presents a summary of the stiffness characteristics of a number of materials comprising pavement sections. He also stresses the importance of insuring that traffic loading conditions and environmental influences be considered when selecting the actual stiffness values for pavement design and analysis. In addition one must insure that the stress versus deformation characteristics of the laboratory specimens are representative of the materials in situ.

Thompson and Robnett (11) did an extensive study on soils in the state of Illinois. Significant correlations and relations were developed between resilient behavior and factors such as static strength, degree of saturation and modulus data. The regression regressions that they developed may be used to predict the probable resilient properties of a soil. The soils were limited to fine grain soils that are encountered in pavement construction in the state and are similar to other soils in other areas of the country.

Bell (12) compared laboratory compaction devices based on their ability to produce mixtures with engineering properties similar to those produced in the field. The mobile steel wheel simulator most closely simulates field compaction but will not be broadly implemented because of its inability to produce specimens of the varied geometries which will be required by AAMAS. The Texas gyratory compactor and the California-type kneading compactor most generally do an acceptable job in simulating field compaction. The Marshall impact hammer does not adequately simulate field compaction.

The cited studies indicate that there are several factors and variables that must be taken into consideration prior to conducting laboratory testing to determine the resilient modulus of a specimen.

FIELD DATA SITES

It was originally intended to collect field data from 50 to 100 locations distributed throughout the state. However, the Study Advisory Committee concluded that many sites would require more manpower, time, and funds than were available; therefore, the number of study sites was reduced to 21.

In 1987, the United States Congress passed the Surface Transportation Act. Included as a part of that act was 150 million dollars for the Strategic Highway Research Program (SHRP). A large portion of SHRP was to be the long-term pavement performance (LTPP) monitoring of thousands of sections of in-service highways located throughout the nation. These sections that were to be monitored were referred to as GPS (General Pavement Studies) sections.

To assist SHRP in choosing these GPS sections, the states were to submit a list of possible candidate sections for consideration. SHRP published a set of criteria for
selecting pavement sections to be included in the LTPP-GPS portion of SHRP. Their project matrix included such variables as general ranges of subgrade strength, traffic volumes, pavement thicknesses and types, new pavements, and those having only one overlay during its history. Other factors that were to be considered in choosing candidate sections included the following:

- sites should be relatively moderate in grade and alignment
- should be at least 0.5 mile between discontinuities
- should not have lanes added or have been widened
- should be no older than 1970
- should have uniform traffic movement over section
- original surfaces should not have been ground or milled
- should not have more than one overlay
- should not have received a seal coat within one year
- AADT times the percent of trucks should be greater than 200 per day.

A master list of Kentucky pavements was assembled and sent to SHRP for their use. SHRP narrowed the list of possible candidates, and returned this shortened list along with a data form on which to assemble specific test data for that individual candidate section. The Division of Materials searched their files for the appropriate test data and a final list was prepared and resubmitted to SHRP. SHRP then selected the final sites according to their needs on a national basis and on the available Kentucky data included in that final list. Ultimately, seven sites were chosen in Kentucky to be a part of the national LTPP program.

The Study Advisory Committee concluded that these seven sites should be included as a part of this study. An additional 13 sites were then chosen (using the same criteria as those used in choosing the GPS sections of the LTPP program of SHRP) to be monitored in an identical fashion as the national LTPP sites. Table 1 lists the 20 sites, and Figure 1 is a map of their locations. (The site that was listed as No. 10 was deleted before the study began; therefore the total number appears to be 21 because No. 10 is missing.) Appendix A gives a detailed description of each site, along with a photograph at each location.

It should be noted that Site No. 4 (Pennyrrile Parkway) has been overlaid since the beginning of this study, and apart from data obtained in the first two years, no further data will be collected from this site.
FIELD DATA COLLECTION AND SURVEYS

Asphalt Cores

Kentucky Transportation Center obtained cores at each test site, in 1989. The cores were obtained within the 500-foot test section, 100 feet and 400 feet from the start of the test section. Four, six-inch cores were obtained at the 100-foot and at the 400-foot location at each site. The only exception was KY 4 where the cores were taken +50 and -50 feet outside the 500 foot section. The cores were all obtained in the right wheel path in a circular type pattern.

SHRP obtained cores outside the 500-foot section at both ends. On the asphaltic concrete sites, SHRP took a total of fourteen 4-inch cores, two 6-inch cores, and six 12-inch cores. SHRP also removed a 1-foot by 1-foot block sample from a test pit. A sample of SHRP's coring pattern for the asphaltic concrete sites is shown in Figure 2.

On the jointed plain concrete pavement sites, SHRP took a total of twelve 4-inch cores, two 6-inch cores, and six 12-inch cores. A sample of SHRP's coring pattern for the jointed plain concrete sites is shown in Figure 3.

Subgrade Samples and Trenching

Kentucky Transportation Center took moisture samples of DGA and the subgrade at the 100- and 400-foot marks. Two CBR tests were performed at both of these areas. In addition, Shelby tubes were obtained where suitable subgrade was present. No trenching operations was conducted by the Kentucky Transportation Center.

SHRP excavated a 4-foot by 4-foot test pit at each asphaltic concrete site. Nuclear density tests were performed on the DGA and/or subgrade at each site. Bag samples were obtained of the subgrade and the DGA. Jar moisture contents were also obtained. Shelby tubes or standard penetration tests were obtained at each site. A 20-foot auger boring was also performed at each site on the shoulder of the road to verify bedrock depth.

Visual Distress Surveys

Kentucky Transportation Center conducted visual distress surveys for 1989, 1990, and 1991. The sites were each inspected for any sign of surface distress, such as rutting; transverse, longitudinal, and/or alligator cracking; spalling; ravelling; bleeding; and pumping. Each site was video taped, and surface distress was photographed. Surface distress of each site has been plotted (Appendix B).
Deflection Testing

The Kentucky Transportation Center performed deflection test using both the FWD and the Road Rater. In 1989, deflection test were obtained at each site at the 100-foot and the 400-foot mark. In 1989, 19 sites were tested. At seven of the sites tested with the Road Rater, the 1,200-pound load only was used. An applied load of 9,000 pounds was used with the FWD at the same seven sites. At the other 12 sites tested with the Road Rater, the load was varied (600 pounds, 1,200 pounds, and 1,800 pounds). The load from the FWD was also varied using loads of 6,000, 9,000, 12,000 and 15,000 pounds. FWD and Road Rater tests were performed at mid slab on rigid pavements 100-feet and 400-feet into the 500-foot test section.

SHRP tested the same sites in 1989 but only with the FWD. SHRP varied the applied loads at each site. At flexible pavement sites, SHRP applied four different range of loads of 6,000, 9,000, 12,000 and 16,000 pounds. The tests were conducted in the outer-wheel path at the test pits, at the mid lane every 25 feet, and at the outer wheel path every 25 feet.

At rigid pavement sites, SHRP applied three different ranges of loads of 9,000, 12,000 and 16,000 pounds. The tests were conducted 1) outer wheel path at the test pits 2) mid-lane at the mid-panel 3) pavement edge at the corner 4) pavement edge at mid-panel 5) outer wheel path at the joint (Table 2).

In 1990 and 1991, Kentucky Transportation Center conducted their FWD testing according to SHRP standards. Two additional test sites were also added in 1990.

Temperature Gradient Measurements

SHRP conducted a series of temperature gradient measurements at each site. Two locations near the test section were selected as representative of the sun-exposure conditions at the site. Measurements were obtained at periodic time intervals. The measurements were made at the surface, mid slab, and the bottom. (Figure 4).
Description of Road Rater

The Model 400B Road Rater was manufactured by Foundation Mechanics, Inc. of El Segundo, California. The testing head on the Road Rater is mounted on the front bumper of a heavy-duty pick-up truck and consists of a vibrating mass weighing 160 pounds, which impulses the pavement. The forced motion of the pavement is measured by velocity sensors normally located at 5.25 inches, 12 inches, 24 inches, and 36 inches from the center of the test head (Figure 5). The vibrating mass is suspended by a system of rubber bellows. A second set of bellows housed in the test head distributes the dynamic load equally to two "feet". Frequency of the vibrator may be chosen from preselected frequencies of 10, 20, 25, 30, and 40 Hz. The vibrating mass is lowered to the pavement by a hydraulic system. Optimum and resonant frequencies are a function of the location of the test head, the dynamic force, the location of the load-bearing "feet", and the pavement structure. The dynamic force is a function of frequency and amplitude of vibration and the pressure in the hydraulic system.

To determine the appropriate frequency or frequencies at which to perform the Road Rater test, the response of a pavement to the vibrating mass of the Road Rater was determined for several full-depth asphaltic concrete pavements and conventional three-layer pavements. Resonant frequencies of the total pavement structure were usually multiples of approximately 7 Hz. The thickness of the asphaltic concrete layer appeared to cause the resonant frequency to shift 1 or 2 Hz at the 21 and (or) 28 Hz normal resonant frequencies. Resonance at these frequencies was indicated by oscillations of the meter's needle as opposed to the normally "rock steady" behavior. In all cases, the meter response remained steady at 25 Hz, which was chosen as the reference frequency. A frequency of 25 Hz and an amplitude of vibration of 0.06 inch results in a peak to peak dynamic force of 600 pounds. Once the dynamic force is set for a given frequency and amplitude, the other preset frequencies will vary the amplitude of the vibrating mass such that the dynamic force remains constant for all of the pre-selected frequencies. The composite loading consists of a static load of 1,670 pounds with a dynamic force of 600 pounds, peak to peak, oscillating about the static load. The loading is transmitted to the pavement by two "feet" symmetrically located on either side of a beam extending ahead and supporting the sensors.

The dynamic loading (sine wave) of the Road Rater may be approximated by a square wave, so that the maximum or root-mean square value of the square wave is equal to 1/√2 times the peak value of the sine wave. For short time periods, this is representative of a steady-state condition which approximates a static load (see Figure 6). The peak to peak loadings of the Road Rater are 1,882 pounds and 1,458 pounds. From symmetry, the loads on each "foot" of the test head are equal to 941 pounds and 729 pounds. The dynamic deflection is defined by:
\[ D_{\text{TOTAL}} = (D_{941} - D_{729}) \times 2 \]

where

\( D_{941} \) and \( D_{729} \) represent the deflections calculated by a layer elastic computer program from the peak loading conditions.

**Description of Falling Weight Deflectometer**

The Falling Weight Deflectometer which was utilized during this study is a JILS-20, also manufactured by Foundation Mechanics, Inc. The basic operation of the FWD consists of lifting a weight to a given height above the pavement and dropping it onto a spring-buffer system. The spring-buffer system transfers the load to the pavement over a duration of approximately 20 milliseconds. The load applied to the pavement and the peak deflections at various radial distances from the center of the load are measured.

The JILS-20 FWD has a load capacity of 2,500 to 25,000 pounds. The deflections are measured at intervals of 0, 12, 24, 36, 48, 60, and 72 inches from the center of the loading plate (Figure 7). The load is adjusted by varying both the drop height and the amount of weight dropped. The deflections at the radial locations are calculated from the outputs of velocity transducers. The operation of the FWD and the calculation of the deflections are carried out using a personal computer contained in the tow vehicle. All data collected (peak deflection at each radial distance and peak load) are stored on the personal computer for processing.

**Summary of Backcalculation Procedures**

Backcalculation procedures were described in some detail in Reference 20. Much of the information described under this section in this report is taken directly from that reference.

There has been considerable use of elastic theory and dynamic testing to estimate layer moduli. Since 1971 in Kentucky, Road Rater deflections have been used as indicators of the characteristics of individual layer components of the pavement structure.

In many backcalculation procedures, it is assumed that the thickness of all layers below the asphaltic concrete have remained as constructed. It is also assumed that fatigue and deterioration reduce the effective thickness of the asphaltic concrete to some equivalent thinner thickness of good quality material. In existing pavements, the thickness of the dense graded aggregate (DGA) is assumed to have remained as constructed. The other variables which influence the behavior of the pavement are the effective thickness of the asphaltic concrete and the strength of the subgrade.
Using elastic theory, a relationship relating subgrade modulus and Road Rater deflection may be developed for a given structural section and constant asphaltic concrete modulus and variable DGA modulus. The modulus of the DGA is assumed to vary with the modulus of the layers which confine it. Therefore, for a constant AC modulus and variable subgrade modulus, the DGA modulus must vary as well (17). The equation relating pavement deflection and subgrade modulus may be expressed as follows:

\[
\log(\text{delta}) = (K) \cdot \log(E_{\text{sub}}) + L
\]

where:

\[
\begin{align*}
\text{delta} &= \text{Road Rater Deflection (in)}, \\
K &= \text{Slope of the log-log Line}, \\
L &= \text{Constant, and} \\
E_{\text{sub}} &= \text{Elastic Modulus of the Subgrade (psi)},
\end{align*}
\]

Both K and L are dependent upon the asphaltic concrete thickness and DGA thickness, they may be described by third degree polynomials. The development of this equation is given in detail in (16).

The modulus of elasticity of asphaltic concrete varies as a function of both temperature and frequency of loading (18). The thickness design procedures for Kentucky are based on a modulus of asphaltic concrete of 480,000 psi at 0.5 Hz and a temperature of 70 °F. Therefore, these reference conditions have been utilized for the analysis of RR data. Since the RR tests are conducted at a constant loading frequency of 25 Hz, a reference modulus at this frequency must be selected. A reference modulus of 1,200,000 psi at 70 °F has been determined to represent the 480,000 psi modulus at 0.5 Hz.

When field measurements are made, the pavement temperature and time of day are recorded. This information in addition to the five-day mean air temperature history are needed to calculate the mean pavement temperature (19). The relationship of asphaltic concrete modulus, frequency of loading, and temperature may be expressed as follows;

\[
\log(\text{E}_{\text{AC}}) = (A + B(T_p) + C(T_p)^2) + [(D + E(T_p) + F(T_p)^2) \cdot \log(\text{Hz})],
\]

Both K and L are dependent upon the asphaltic concrete thickness and DGA thickness, they may be described by third degree polynomials. The development of this equation is given in detail in (16).
where:

\[ E_{AC} = \text{Mean Asphaltic concrete modulus}, \]
\[ T_p = \text{Mean Pavement Temperature (degrees Fahrenheit)}, \]
\[ H = \text{Loading Frequency in Hertz}, \]
\[ A = 6.763855405, \]
\[ B = -0.0072846915, \]
\[ C = -0.0001108391, \]
\[ D = -0.1741191221, \]
\[ E = 0.0074997275, \]
\[ F = -0.0000180328. \]

An adjustment procedure has been developed to adjust field deflections to a reference temperature (70 °F) and modulus (1,200,000 psi) (3). The adjustment procedure uses ratios of deflections at the reference conditions to deflections resulting from an array of various asphaltic concrete moduli and pavement thicknesses. The relationship between asphaltic concrete modulus, pavement thickness, and adjustment factor may be expressed as:

\[
\log(AF_j) = \left[ \log(AC) \cdot (H_1 \cdot E_{AC}^3 + H_2 \cdot E_{AC}^2 + H_3 \cdot E_{AC} + H_4) \right] * \\
\left[ M_1 \cdot E_{AC}^3 + M_2 \cdot E_{AC}^2 + M_3 \cdot E_{AC} + M_4 \right]
\]

where:

\[ AF_j = \text{Adjustment Factor for Sensor } j, \]
\[ j = \text{Road Rater Sensor Number}, \]
\[ E_{AC} = \text{Mean Asphaltic Concrete Modulus}, \]
\[ AC = \text{Asphaltic Concrete Thickness}, \]
\[ H_1, H_2, H_3, H_4, M_1, M_2, M_3, \text{ and } M_4 = \text{Regression Constants (16)}. \]

These two relationships are used to adjust field deflections to equivalent deflections at the reference conditions.

Pavements generally exhibit distresses which may be grouped into three categories. The first is deterioration of the asphaltic concrete slab, the second is the loss of support of the subgrade, and the third is a combination of the two. Any of these problems will cause the pavement to have decreased structural capacity.

A method of determining the type of distress was developed using deflections which have been calculated using elastic theory. The deflections are plotted as radial distance from the load versus log of the RR deflection. A semi-log line is then projected through the magnitudes of the No. 2 and No. 3 deflections to the location
of the number 1 deflection. This procedure is know as the 2-3 Projected approach. This line may be represented by the following equation.

\[
\log(\text{No. 1}_{\text{Projected}}) = 2 \times \log(\text{No. 2}) - \log(\text{No. 3})
\] (4)

In addition, another relationship was developed relating the No. 1 projected deflection to the actual No. 1 deflection for theoretical structures. This equation is developed across a range of subgrade moduli from 6,000 to 60,000 psi, with constant structural section and asphaltic concrete modulus. This equation may be expressed as follows:

\[
\log(\text{No. 1}) = M \times \log(\text{No. 1}_{\text{Projected}}) + B,
\] (5)

where:
\[
M = \text{Slope of the line, and} \\
B = \text{Intercept.}
\]

For a given combination of layer moduli representing a pavement structure, there is a unique theoretical deflection bowl. For this theoretical deflection bowl, there is a difference between the No. 1 projected deflection and the actual No. 1 deflection. This also holds true for deflections obtained in the field with the Road Rater. Normally, these differences for both theoretical and field deflections are similar.

Equations 4 and 5 may be used to determine the portion of the pavement which is distressed. Equation 4 is used to calculate the projected No. 1 deflection from the field data. This value is then input into Equation 5 to determine the corresponding theoretical No.1 deflection for this structure. A comparison of theoretically calculated No. 1 and Field No. 1 is an indicator of which portion of the pavement structure may be distressed.

If the theoretically calculated No. 1 deflection is less than the actual measured No. 1 deflection, then the asphaltic concrete is in a weakened condition (Condition 1). If the calculated deflection is greater that the measured No. 1 deflection, then the subgrade or the portion of the structure below the asphaltic concrete is weak (Condition 2).

Two parameters are needed as input into the current overlay design procedure. These parameters are the elastic modulus of the subgrade and the structural worth of the existing asphaltic concrete (effective thickness). The calculation of the effective thickness and in-place subgrade modulus are dependent on the results of the above comparison. The type of distress will determine if actual field No. 1 deflections or theoretical No. 1 deflections, calculated from the projected field No. 1 deflection, are used to calculate the effective asphaltic concrete thickness and subgrade modulus.
In Condition 1, the calculated deflection is less than the actual deflection. This indicates a weak AC layer; therefore, the calculated displacement would be the proper deflection had the AC been in good condition. Since the weaker AC causes a bending of the deflection bowl, No. 2 and No. 3 deflections are representative of the subgrade condition. Therefore, the theoretically calculated No. 1 deflection is used in Equation 1 to calculate the test point subgrade modulus. The actual field No. 1 deflection is used to calculate the effective thickness.

In Condition 2, the calculated deflection is greater than the actual No. 1 deflection, therefore the subgrade is in a weakened condition. In this condition, more damage will result in the asphaltic concrete layer and it will deteriorate more quickly. To overcome this weakened condition, a thicker overlay is needed. To accomplish this, the subgrade modulus is calculated using the actual No. 1 deflection and the calculated No. 1 (higher deflection) is used to calculate the effective thickness.

The calculation of effective thickness is achieved using interpolation over a range of deflections calculated from structures of different AC thicknesses. The coefficients of Equation 1 are calculated from a matrix of AC thicknesses ranging from the design thickness down to a minimum thickness. Using Equation 1, solved for the subgrade modulus and the No. 1 deflection value chosen by Condition 1 or 2, the subgrade modulus of the test point may be calculated. This subgrade modulus is used in the matrix of equations, generated for different AC thicknesses, to calculate the corresponding No. 1 sensor deflection. The No. 1 sensor deflection, determined by Condition 1 or 2 for effective thickness calculation is used to interpolate the effective thickness from the matrix of calculated No. 1 deflections. This thickness is the effective thickness of the asphaltic concrete portion of the pavement structure.

The Kentucky Transportation Cabinet currently uses the procedure outlined above. The procedure is outlined in Research Report UKTRP-84-9, "Structural Evaluation of Asphaltic Concrete Pavements" (4).

A study just completed by the Kentucky Transportation Center involved the development of new backcalculation models. Each model utilizes the same mathematical concept of matching theoretically calculated deflection bowls with deflection bowls measured in the field. The procedure is an iterative process. Theoretical deflection bowls, calculated from various combinations of layer moduli, are systematically compared to the field deflection bowls. The square root of the sum of the squared differences (least square) between the theoretical and field deflections for all sensors is calculated. The structure which gives the minimum least square is selected as the in-situ structure.

The theoretical deflections are calculated using the Chevron N-Layer linear elastic computer program. The procedure for modeling the load configuration of the Road Rater was reported by Southgate (19). The same load configuration was used in that study.
The theoretical deflections were developed using a matrix of variable material thicknesses and elastic layer moduli. In the flexible pavement model a thickness ratio was utilized instead of requiring a set of DGA thicknesses. This ratio is defined as the asphaltic concrete thickness divided by the total pavement thickness, asphaltic concrete and dense graded aggregate. This is the same ratio which is utilized in Kentucky's pavement design procedure. Ratio's of 1.0 (full depth asphaltic concrete), 0.5, and 0.33 were used in that study. As mentioned earlier, the modulus of the dense graded aggregate is assumed to be a function of the confining layers of asphaltic concrete and subgrade.

The matrix of structures used to develop the model covers a wide range, but not all possibilities. Therefore to calculate deflections which were not included in the database, an interpolation procedure must be used. Lagrangian interpolation (21) was utilized to determine the deflections for structural sections which were not calculated by the database, but are within the ranges of the database. This method of interpolation is valid for an infinite number of data points.

The Lagrangian interpolating formula may be expressed as follows,

\[ P(x) = y_0 L_0(x) + y_1 L_1(x) + \ldots + y_n L_n(x) \]  \hspace{1cm} (6)

where each \( L_i(x) \) is expressed as

\[ L_i(x) = \frac{(x-x_0)\ldots(x-x_{i-1})(x-x_{i+1})\ldots(x-x_n)}{(x_i-x_0)\ldots(x_i-x_{i-1})(x_i-x_{i+1})\ldots(x_i-x_n)} \]  \hspace{1cm} (7)

The \( P_n(x) \) is the deflection desired for the value of \( x \), where \( x \) may be layer modulus or layer thickness. For interpolation of the database, the \( y_n \) would represent the deflections and \( x_n \) would represent the layer moduli or thicknesses corresponding to the deflections that are being used for interpolation.

A personal computer-based program was written to perform the interpolation across each database. The program was developed using Microsoft QuickBASIC Version 4.5 (22). This program performs all necessary interpolations and least squares calculations necessary to calculate the matching deflection bowls.

The least squares calculation may be represented by the following equation:
where: $D_{1r}$, $D_{2r}$, $D_{3r}$, $D_{4r}$ are the field deflections and $D_{11}$, $D_{21}$, $D_{31}$, $D_{41}$ are the theoretically calculated deflections.

The methodology of the model is to interpolate across the database for the known parameters and then perform iterative calculations on the remaining variables. Normally, the structural cross section is known; therefore, the database is interpolated for layer thickness directly. The remaining unknowns, elastic moduli, are calculated using an iterative procedure.

To provide better relationships for interpolation, the common logarithms of the values have been utilized. The use of a logarithmic relationship provides more accurate interpolation.

The following input information is needed to perform the calculations: layer thicknesses (AC and DGA), field deflections, test date and time of test, and temperature information (surface pavement temperature and 5-day mean air temperature).

Once the structural cross section of the pavement is known, the database may be interpolated for these thicknesses. The database is interpolated first for the thickness ratio (thickness of DGA/total pavement thickness). The relationship of log of the thickness ratio versus deflection is interpolated for the known thickness ratio. This creates a new database which contains data only for structures having the same thickness ratio. The new database is then interpolated for the AC thickness using the relationship of log (AC thickness) versus log (deflection). This database contains deflections for a matrix of structures for various AC and subgrade moduli, at the field AC and DGA thickness.

The modular ranges for the database used in the model are AC moduli of 50,000; 500,000; and 2,000,000 and subgrade moduli of 3,000, 12,000; and 60,000, respectively. For determining the theoretical bowl which will best match the field bowl, deflections at other layer moduli within these ranges must be calculated. The Lagrangian coefficients of $L_{i}(x)$ are calculated for both the AC and subgrade moduli. Coefficients are calculated for AC moduli on increments of 50,000 psi from 50,000 to 2,000,000 psi. For subgrade moduli, coefficients are calculated on 1,000-psi increments from 1,000 to 100,000 psi.

The process of determining the best fit deflection bowl based on the AC and subgrade modulus is an iterative process. First an AC modulus is assumed, based on a relationship of log (AC modulus) versus log (deflection). The current database is then interpolated for AC modulus. This provides a database which is a function of subgrade modulus and pavement deflection. The deflections are calculated for
different subgrade moduli based on the relationship of \( \log(\text{subgrade modulus}) \) versus \( \log(\text{deflection}) \). The subgrade modulus is varied in 1,000 psi increments from 1,000 to 100,000 psi. At each subgrade modulus, the field and theoretical bowls are compared and the least square calculation, Equation 8, is performed.

If the least square is less than the current minimum least square, then the AC modulus and subgrade modulus are stored as the best fit modular values for the given deflections. This iteration process is carried out for AC modulus increments previously outlined. A total of 4,000 different modular combinations are tested for the best fit theoretical deflection bowl. The combination of layer moduli, representing the smallest least square is assumed to be the best fit layer moduli for the input deflections.

The moduli selected by the previous procedure are utilized to determine another range of moduli with smaller increments. The range for the AC modulus calculations is set as 50,000 psi above the value calculated in the previous step to 50,000 psi below this value. The AC modulus is then varied in 10,000-psi increments. The same iterative procedure is conducted, using the same increments of subgrade modulus, and refined values of AC modulus and subgrade modulus are determined. These values are then selected as the best fit layer moduli, representing the measured field deflections. The deflections have not been adjusted for temperature; therefore, these moduli are determined at the prevailing pavement temperature.

All backcalculated moduli values in this report were determined using the new models recently developed at the Kentucky Transportation Center and reported in Reference 20.

**CORRELATION OF ROAD RATER AND FALLING WEIGHT DEFLECTOMETER**

As stated previously, deflection tests were performed on these sites simultaneously using the Model 400B Road Rater and the JILS-20 falling weight deflectometer. In this section of this report, Road Rater deflections are compared using a 600-pound load with Road Rater deflections obtained using other load levels. Also, FWD deflections obtained using a 6,000-pound load are compared with FWD deflections obtained using other load levels. Finally, correlations are made between FWD deflections and Road Rater deflections.

**Road Rater Deflections**

Figures 8 through 11 show the relationship between deflections for a 1,200-pound and 1,800-pound load as compared to a 600-pound load for the Road Rater (for all sensors). A regression analysis was performed on all the data, and the regression
lines are shown in those figures. The regression analysis between the 1,200-pound load and the 600-pound load yielded a better \( R^2 \) for all four sensors than did the regression analysis between the 1,800-pound and 600-pound loads. The \( R^2 \) values for the 1,200-pound load ranged from a low of 0.720 at Sensor No. 2 to a high of 0.902 at Sensor No. 1. The \( R^2 \) values for the 1,800-pound load ranged from 0.620 to 0.803 for Sensors No. 4 and No. 1, respectively. Although attempts were not made to confirm this, it is suspected the scatter in the 1,800-pound data is partly due to the sensors detecting extraneous machine vibrations because the 1,800-pound load is approaching the upper limit of the capacity of the Model 400B Road Rater.

If the deflection response were linear, the slopes of the regression lines in Figures 8 through 11 should be proportional to the ratio of the 1,200-pound and 1,800-pound loads to the 600-pound load. This means the slopes of the 1,200-pound regression line and 1,800-pound regression line should be 2.0 and 3.0, respectively. The largest slope for the 1,200-pound load was 2.257 (Sensor No. 3). This is an increase of 12.9 percent over what would be expected for a true linear response. The largest slope for the 1,800-pound load was 4.145 (Sensor No. 4) which is an increase of 38.2 percent over a linear response.

**FWD Deflections**

Figures 12 through 14 compare the relationship between deflections from the FWD at 9-kip, 12-kip, and 16-kip loads with a 6-kip load for three sensor locations. The comparisons are made at the center of the load (0 inches), at 36 inches, and at 60 inches (Sensors No. 1, No. 4, and No. 6, respectively). There are good correlations at all three locations and at all three load levels. From the regression analysis performed on all the data, the minimum \( R^2 \) at the center of the load was 0.900 (16-kip load). The minimum \( R^2 \) at the 36-inch location was 0.863 (12-kip load), and the minimum at 60 inches was 0.789 (16-kip load).

The degree of apparent nonlinearity of the FWD deflections with increasing load was considerably less than with the Road Rater. If the deflection response were linear, the slope of the regression lines in Figures 12 through 14 should be 1.5, 2.0 and 2.67 for the 9-kip, 12-kip, and the 16-kip loads, respectively. The largest deviation from those ideal slopes was at the 36-inch location using the 16-kip load. The slope was 3.002. This is 12.6 percent greater than a true linear response.

**Comparison of Road Rater and FWD**

Figures 15 through 18 compare deflections from the Road Rater at 1,200 pounds to deflections from the FWD at 9,000 pounds. Deflections from Sensors No. 1 through No. 4 on both machines are compared. Some error is introduced in this comparison as the comparable sensor numbers are not at the exact same distance from the center of the load. The greatest difference is for Sensor No. 1. As shown
in Figure 15, Sensor No. 1 is 5.25 inches from the center of load in the Road Rater, while Sensor No. 1 is at the exact center of load in the FWD. The dashed lines in those figures are true regression lines. The solid lines are regression lines that have been forced through zero. The ratio between load magnitudes of 9,000 pounds for the FWD and 1,200 pounds for the Road Rater is 7.5; therefore the slopes of the true regression lines in Figure 15 through 18 should approximate that ratio. The slope of the true regression line in Figure 15 for the No. 1 sensors for the two machines is 10.961 — a difference of 46 percent. Much of this difference may be attributed to the difference in distances from the center of the load.

The distances from the center of the load for Sensors No. 2 and No. 3 are reasonably close to the same magnitude (see Figures 5 and 7). The slope of the true regression line for Sensor No. 2 is 7.845, and for Sensor No. 3 the slope is 7.530. These values are close to the expected ratio.

The relationship shown in Figure 18 is between the No. 4 sensors of the two devices. Clearly, there is a very poor correlation between the two data sets. The slope of the true regression line is only 2.01 (instead of 7.5) and the $R^2$ value is only 0.196. A statistical analysis on each data set shows the Road Rater data have considerably more variation than does the FWD data. The coefficient of variation for the Road Rater data is 58.2 percent, while the FWD data had a coefficient of variation of less than half that of the Road Rater at only 24.6 percent. It is suspected that the 1,200-pound load of the Road Rater produces such small deflections at the No. 4 sensor that the noise-to-signal ratio is very high for that sensor. The noise-to-signal ratio is the ratio of background electrical noise and ambient vibrations to the electrical signal produced by the velocity transducer. It should be noted that all of the deflections from the Road Rater are less than 0.4 mil. This is at the extreme low end of the range of the 80-mil (40 mils) velocity transducer that is used. These two facts would make the deflections from the No. 4 sensor of the Road Rater less reliable and would undoubtedly result in greater variation in the readings.

Because FWD loads are considerably greater than Road Rater loads, it would be expected that greater nonlinear response would be exhibited by FWD deflections than by Road Rater deflections (assuming the pavement materials are nonlinear materials); however, the converse is true. It must be concluded, therefore, that some of the apparent nonlinear behavior is related to the testing devices and not to the pavement structure. There are two possible reasons. The first reason, mentioned earlier, is the 1,800-pound load on the Road Rater is approaching the upper limit of the machine. This may be producing greater vibrations of the machine that are being detected by the sensors, and the amplitude of these vibrations could be superimposed on the amplitude of vibrations produced by the pavement response, producing larger than true readings. A second reason may be the steady-state nature of the Road Rater vibrations. The higher loads from the Road Rater may be producing some temporary pore pressures in the dense-grade aggregate base and/or the subgrade, which may be causing partial liquefaction, reducing the shear strength, and producing higher deflections.
The sensors of both devices can be read to the nearest 0.01 mil. Therefore, the least error that either machine could detect at each sensor would be 0.01 mil (this is equivalent to the residual of error between the true value of deflection and that measured by the machine). In matching measured deflection bowls to theoretical deflection bowls, the least sum of squares of the residuals is used to determine the "best-fit" bowl. The minimum sum of squares of the residuals for the Road Rater would be 0.01 mil multiplied by 0.01 mil multiplied by the four sensors, which is equal to 0.0004. For the FWD, the minimum sum of squares of the residuals would be equal to 0.0007. In many occasions, field deflection bowls are matched to theoretical deflection bowls using sums of squares of residuals that are smaller than 0.0004 when using the Road Rater. This simply says that the Road Rater cannot distinguish between any two cross sections of pavement represented by two theoretical deflection bowls that yield a sum of squares of residuals that is less than 0.0004. This same statement may be made concerning the FWD when the difference between the sum of squares of residuals of two theoretical deflection bowls is less than 0.0007.

This situation often occurs when using the Road Rater to test very thick or very stiff pavements. Under these conditions, the sensors are being used at the extreme lower end of their sensitivity.

FIELD AND LABORATORY DATA ANALYSIS

Subgrade Analysis

Deflection data obtained from the Road Rater were used to backcalculate subgrade moduli using methods previously published by Sharpe, Southgate, and Deen (6, 13, 14, 15, 16, 17) and by Graves and Allen (20). Table 3 lists the backcalculated values for subgrade moduli for 14 of the LTPP sites for the year of 1990. The calculated values are shown for 600-pound, 1,200-pound, and 1,800-pound loads. This same information is plotted in Figures 19 through 23. Also, Table 4 and Figures 24 through 27 lists and graphically display this same information for the year of 1991. Only average values for backcalculated subgrade moduli were available for 1989, and these will be discussed later.

Values of subgrade moduli varied widely within the 500-foot length of the test section. Backcalculated values of subgrade moduli using deflections from the 600-pound loads were generally higher than values calculated from the 1,200-pound and 1,800-pound loads. Figures 28 and 29 show the average backcalculated subgrade modulus and coefficient of variation for each test section for 1990 and 1991, respectively. Information from these figures indicates the coefficient of variation is generally less when using a 1,200-pound or 1,800-pound load. This is further illustrated in Figures 30 through 32. Figures 30 and 31 are scatter graphs of the coefficient of variation for 1,200 pounds and 1,800 pounds versus the coefficient of variation for 600 pounds. Most of the points in both figures fall below the line of equality, indicating a higher value for the coefficient of variation for the 600-pound
loads. Figure 32 is a scatter graph of the coefficient of variation for the 1,800-pound loads versus the coefficient of variation for the 1,200-pound loads. These data points are scattered more closely about the line of equality; however, more points are above the line, indicating a slightly greater coefficient of variation for the 1,800-pound loads. From this, it may be concluded that a 1,200-pound load is likely to yield the most consistent results when using the Model 400 Road Rater. An in-depth study has not been conducted to confirm and explain this result; however, it would appear the 600-pound load may not produce deflections that are sufficient to be within the appropriate operating range of the sensors. The 1,800-pound load may be sufficient to produce extraneous machine vibrations that the sensors are reading. Because the 1,200-pound loads tend to yield more consistent results, most of the analysis of the subgrade will use the values from this load.

Tables 3 and 4, along with Figures 19 through 29 show a high variability of subgrade modulus within a 500-foot section. The coefficient of variation ranged as high as 65 percent for the 1,200-pound load (Interstate 71 -- 1991 data). Assuming a designer were attempting to design an overlay for a particular pavement, this variability in backcalculated modulus would present a problem as to what value of modulus to use for design. This problem may be approached statistically, and at the same time, a degree of reliability may be determined.

The 1,200-pound data in Tables 3 and 4 were normalized by site to the maximum backcalculated modulus for that site. For example, the 1,200-pound normalized data for US 119 in Pike County (1991 data) would appear as follows:

<table>
<thead>
<tr>
<th>SECTION (FEET)</th>
<th>BACKCALCULATED MODULUS (PSI x 1000)</th>
<th>NORMALIZED DATA</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>44</td>
<td>0.76</td>
</tr>
<tr>
<td>25</td>
<td>33</td>
<td>0.57</td>
</tr>
<tr>
<td>50</td>
<td>51</td>
<td>0.88</td>
</tr>
<tr>
<td>75</td>
<td>40</td>
<td>0.69</td>
</tr>
<tr>
<td>100</td>
<td>51</td>
<td>0.88</td>
</tr>
<tr>
<td>125</td>
<td>45</td>
<td>0.78</td>
</tr>
<tr>
<td>150</td>
<td>39</td>
<td>0.67</td>
</tr>
<tr>
<td>175</td>
<td>52</td>
<td>0.90</td>
</tr>
<tr>
<td>200</td>
<td>41</td>
<td>0.71</td>
</tr>
<tr>
<td>225</td>
<td>45</td>
<td>0.78</td>
</tr>
<tr>
<td>250</td>
<td>51</td>
<td>0.88</td>
</tr>
<tr>
<td>275</td>
<td>42</td>
<td>0.72</td>
</tr>
<tr>
<td>300</td>
<td>58</td>
<td>1.00</td>
</tr>
<tr>
<td>325</td>
<td>42</td>
<td>0.72</td>
</tr>
<tr>
<td>350</td>
<td>37</td>
<td>0.64</td>
</tr>
<tr>
<td>375</td>
<td>51</td>
<td>0.88</td>
</tr>
<tr>
<td>400</td>
<td>41</td>
<td>0.71</td>
</tr>
</tbody>
</table>

The normalized data from all sites and for both years (1990 and 1991) were combined.
into a single data file, and were then sorted in descending order (from 1.00 to 0.00). This was a total of 393 data points. The percentage of data points in each 10 percent increment of the maximum backcalculated subgrade modulus was determined as shown in the following listing:

<table>
<thead>
<tr>
<th>DATA RANGE (% OF MAXIMUM MODULUS)</th>
<th>PERCENT OF DATA POINTS IN THE RANGE</th>
<th>ACCUMULATIVE DISTRIBUTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>0% – 10%</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>10% – 20%</td>
<td>0.25</td>
<td>0.25</td>
</tr>
<tr>
<td>20% – 30%</td>
<td>0.76</td>
<td>1.01</td>
</tr>
<tr>
<td>30% – 40%</td>
<td>2.78</td>
<td>3.79</td>
</tr>
<tr>
<td>40% – 50%</td>
<td>4.56</td>
<td>8.35</td>
</tr>
<tr>
<td>50% – 60%</td>
<td>10.13</td>
<td>18.48</td>
</tr>
<tr>
<td>60% – 70%</td>
<td>20.76</td>
<td>39.24</td>
</tr>
<tr>
<td>70% – 80%</td>
<td>24.30</td>
<td>63.54</td>
</tr>
<tr>
<td>80% – 90%</td>
<td>19.75</td>
<td>83.29</td>
</tr>
<tr>
<td>90% – 100%</td>
<td>9.37</td>
<td>92.65</td>
</tr>
<tr>
<td>100%</td>
<td>7.34</td>
<td>100.00</td>
</tr>
</tbody>
</table>

Subtracting the numbers in Column 3 of the previous listing from 100 percent yields the percent reliability of design. The results of that calculation are shown in Figure 33. From the information in Figure 33, the designer may determine the magnitude of subgrade modulus to use, depending on the desired level of reliability. For example, if the designer wished to design at a reliability level of 80 percent, a subgrade modulus value that was approximately 51 percent of the maximum backcalculated subgrade modulus should be used.

As stated previously, one of the objectives of this study was to determine the change in various pavement parameters with time. Table 5 lists the change in the average subgrade modulus for each section over a period of three years (Road Rater data). Figure 34 shows the same results graphically. Generally, there was only a small increase in subgrade modulus, or in many cases, there was a decrease in subgrade modulus from 1989 to 1990. The subgrade modulus increased from 1989 to 1991 for all sections except one (KY 11, Lee County).

Table 6 and Figure 35 also shows the change in backcalculated subgrade modulus from the FWD for all sections (for the years of 1989 to 1991). It is significant to note that, in general, the subgrade modulus was less in 1990 than in 1989 and 1991. It appears that this phenomenon was not machine related as both testing devices yielded similar results.

The reason for this behavior is not clearly understood. It is suspected that rainfall may be one of the factors that play a contributory role. Figure 36 is a plot of accumulated differences from normal rainfall for the years of 1987 through 1991. The years of 1987 and 1988 were very dry years as indicated by the negative values;
however, in 1989 rainfall was well above normal as the negative values began to decrease dramatically. It should be noted that subgrade moduli values generally decreased in 1990 after the rather prolonged period of excess rainfall in 1989. In 1990, rainfall decreased from the amount received in 1989 and remained near normal (as illustrated by the relatively flat portion of the curve in Figure 36). Subgrade moduli values increased in 1991 after a 12-month period of normal to slightly decreased rainfall amounts.

It would appear from the analysis that subgrade moduli values may be affected in a very general fashion by the amount of rainfall in the preceding nine-to-twelve-month period. It should be emphasized that this conclusion is based on only three years of data, and several years more of data will be necessary to confirm or deny this conclusion.

Figure 37 is a comparison of the backcalculated subgrade modulus between the FWD and the Road Rater. Generally, the FWD estimates a higher subgrade modulus than does the Road Rater.

Figure 38 shows the relationship between the average subgrade moduli values (calculated from the Road Rater) for 1990 versus 1989 (base year). Figure 39 shows the same relationship between 1991 and 1989. The \( R^2 \) values are reasonably good in each case, and the regression equation is shown on each figure. Plotting the regression coefficients from both equations in Figures 38 and 39 as a function of time, a generalized model may be developed to predict subgrade modulus for any future year (assuming the modulus is known for the base year of 1989). The equation for the generalized model is as follows:

\[
M_{\text{future}} = \left[ -2784(x) \right] + \left[ 0.382 + 0.324(x) \right] M_{89}
\]

(9)

where

- \( M_{\text{future}} \) = Modulus for the future year,
- \( M_{89} \) = Modulus for the base of 1989,
- \( x \) = difference in years between 1989 and future year.

Again, it should be emphasized that this model will undoubtedly be refined as more data become available. A similar model will be developed for the FWD as more data become available and are analyzed.

A second objective of this study was to compare backcalculated field moduli values with modulus values determined from laboratory tests. Resilient modulus tests were performed on subgrade samples collected from the sites that had soil subgrades. The testing standard used was AASHTO T-274. Table 7 lists the results of these tests, and Figure 40 is a graphical display of the same data. There is a reasonably good correlation between the backcalculated field modulus from the Road
Rater and the laboratory modulus ($R^2 = 0.66$). Using the relationship developed from the regression analysis on these data, the field modulus may be estimated from the following equation:

$$M_{\text{Field}} = (7.5 + 0.75M_{\text{Lab}}) \times (1000)$$  \hspace{1cm} (10)

where

$M_{\text{Field}} = \text{Field Modulus}$, and

$M_{\text{Lab}} = \text{Laboratory Modulus}$.

Figures 41 and 42 show the relationship between laboratory moisture content of the subgrade samples and resilient modulus, and the relationship between density of the samples and resilient modulus, respectively. There appears to be no correlation between these variables. This indicates that the type of soil is very important in determining strength as long as density and moisture content are near optimum conditions.

Figures 43 and 44 show the relationship between CBR values obtained from field tests and laboratory resilient modulus and backcalculated field modulus from Road-Rater data, respectively. Surprisingly, there appears to be an inverse relationship which (at present) cannot be fully explained. Although there are no data to confirm this, when performing the in-place CBR tests it has often been visually noted that the top two or three inches of the subgrade appear to be saturated. After removing these few top inches of the subgrade, the material below will be firmer and dryer.

When subgrade samples were tested in the laboratory, the top two or three inches of material that were extruded from the Shelby tubes were discarded, and only the firmer material was tested. The in-place CBR tests are often performed immediately at the interface between the dense-graded aggregate and the subgrade, thus testing at the location of the saturated material. Although permeability tests were not performed on the subgrade samples, the samples with higher resilient modulus values may have had lower permeabilities. If this were the case, then the higher strength materials probably would have been wetter near the top of the subgrade because of the inability of the water to penetrate deeper into the subgrade. This would have caused the CBR values to be lower for the high-strength materials. It is anticipated that this supposition will be tested in later research.

**Asphaltic Concrete Analysis**

Table 8 lists the backcalculated asphaltic concrete (AC) modulus for the flexible pavement sections for 1990. Figures 45 through 48 show the variation in moduli for each site as a function of distance from the start of each site for 1990. This same
information for 1991 is listed in Table 9 and in Figures 49 through 52. (It should be noted that only an average AC modulus was available for each site for the year of 1989.) There is a large amount of variability in the value of AC modulus within the

A number of factors may affect the variation of AC modulus within a short section. Cracking, water in the pavement, and decoupling of the AC slab from the dense-graded aggregate base are some of the more important factors. If the AC layer had decoupled from the dense-graded aggregate base, this will appear to make the AC layer "flop" causing a larger variation in the backcalculated modulus. It is suspected that thicker AC layers would be less susceptible to "flopping" than thinner AC layers. The data illustrated in Figure 53 would appear to help confirm this supposition. Except for one outlier, the thinner AC sections generally had higher coefficients of variations than did the thicker sections.

The data in Tables 8 and 9 may be used by the designer to determine a level of reliability for design in the same manner to that previously described for the subgrade modulus. AC modulus values in Tables 8 and 9 were normalized to the maximum backcalculated AC modulus for a particular site. These normalized values were then combined into a single data file and sorted in descending order. As in the case of the subgrade modulus, the percentage of data points in each 10-percent data range was determined. An accumulative distribution was calculated from 0 percent to 100 percent by 10-percent increments. Subtracting the accumulative distribution values from 100 yields the percent design reliability for AC modulus. This design reliability is shown in Figure 54. A close examination of Figures 33 and 54 shows that the reliability for AC modulus is not as great as that for subgrade modulus. For example, a designer wishing to use an 80-percent reliability for AC modulus could use an AC modulus value of only 23 percent of the maximum backcalculated AC modulus for a particular site (for subgrade modulus, 51 percent of the maximum backcalculated subgrade modulus could be used for a design reliability of 80 percent).

Reliability of design may be considered as the probability of a successful design. In other words, a design reliability of 80 percent is the same as saying the probability of a successful design is 80 percent. The combined probability of a successful design or combined design reliability of a particular design project is the product of design reliability of the subgrade modulus and the reliability of the AC modulus. The combined reliability is illustrated in Figure 55. For example, to achieve an overall design reliability of approximately 80 percent for a project, the designer must use a design reliability of 90 percent for subgrade modulus and a design reliability of 90 percent for AC modulus (or any combination of design reliabilities on the 80 percent line in Figure 55).

To determine the change in backcalculated AC modulus from 1990 to 1991, the average modulus for 1991 was plotted as a function of the backcalculated AC modulus from the Road Rater for 1990. This is shown in Figure 56. Performing a regression
analysis on that information yields a regression equation by which the change in AC modulus per year may be estimated:

\[ ACMOD_{\text{Future}} = [193.9 + (0.807 \times ACMOD_{\text{Present}})] \times t \times 1000 \]  \hspace{1cm} (11)

where

- \( ACMOD_{\text{Future}} \) = Modulus of AC in some future year,
- \( ACMOD_{\text{Present}} \) = Modulus of AC in present year, and
- \( t \) = Time in years between present and future year.

Resilient modulus tests were performed on cores of the AC obtained from each of the sites. Testing procedures used were according to ASTM D-4123 with the exception that the testing temperature was 75\(^\circ\) F (three temperatures are recommended -- 41\(^\circ\) F, 77\(^\circ\) F, and 104\(^\circ\) F). Tables 10 and 11 list the backcalculated field moduli corrected for temperature and frequency (method used to correct data previously reported in References 13 through 17) for the years of 1990 and 1991 and the laboratory resilient modulus. Figures 57 and 58 show the same information. Clearly the laboratory moduli are greater in almost every case. The laboratory test is performed on an intact sample while the field test measures the response of a much larger area, which may include cracks, decoupled layers, and other factors which could reduce the modulus.

Other researchers also have attempted to correlate backcalculated field modulus with resilient modulus obtained in the laboratory; Figure 59 is an example of an attempt by Lee, Mahoney, and Jackson (18). Their data show considerable variability about the line of equality --- indicating the difficulty correlating the field and laboratory moduli. Although there is considerable scatter in the data in this study, it is possible to estimate a relationship between the laboratory resilient modulus and the backcalculated field modulus. In Figure 60, the ratio of backcalculated field modulus from the Road Rater (uncorrected for frequency) to laboratory modulus (Column 2 divided by Column 6 in Tables 10 and 11) has been plotted as a function of pavement surface temperature. A regression analysis was performed on the data in that figure. The R\(^2\) value was 0.66. The resulting best-fit equation is as follows:

\[ \text{LOG}(\text{MOD}_R) = 1.476 - 0.0279(T) + 0.0000754(T)^2 \]  \hspace{1cm} (12)

where

- \( \text{MOD}_R \) = Ratio of field modulus to laboratory modulus, and
- \( T \) = Pavement surface temperature, \(^\circ\)F.

If the pavement surface temperature at which the field test was performed is known,
and if either the laboratory resilient modulus or the backcalculated field modulus is known, the other modulus value may be estimated using Figure 60.

The information in Figure 60 may be implemented in two ways. If a pavement designer is attempting to design an overlay but does not have deflection data from which to backcalculate an in-situ AC modulus, pavement cores may be obtained and resilient modulus tests at 75°F can be performed in the laboratory on those cores. The designer may then enter the chart in Figure 60 at 70°F (the standard reference temperature to which backcalculated moduli are converted) and determine the ratio between the field modulus and the laboratory resilient modulus. The laboratory resilient modulus is then multiplied by that ratio to yield an estimated backcalculated field modulus that could be used in design.

A second method of implementing the information in Figure 60 is to use the relationship developed from that data to provide a quick means of converting from backcalculated field modulus at test temperature to an estimated backcalculated modulus at the reference temperature of 70°F. This may be accomplished by use of the following equation:

\[
FM_{70} = \frac{FM_{\text{TEST}}}{(MOD_R)_{\text{TEST}}} \cdot [(MOD_R)_{70}]
\]  

(13)

where

- \(FM_{70}\) = Backcalculated modulus at reference temperature of 70°F,
- \(FM_{\text{TEST}}\) = Backcalculated modulus at pavement test temperature,
- \((MOD_R)_{\text{TEST}}\) = Modulus ratio calculated from the above equation at the pavement test temperature,
- \((MOD_R)_{70}\) = Modulus ratio calculated from the above equation at the reference temperature of 70°F.

Figure 61 is a plot of the ratio of backcalculated field modulus from the FWD to the magnitude of the laboratory resilient modulus as a function of pavement surface temperature. A regression analysis was also performed on these data with a resulting \(R^2\) of 0.51. The best-fit regression equation is given in Equation 13.

\[
\log(MOD_R) = 1.378 - 0.03072(T) + 0.000122(T)^2
\]  

(14)

where \(MOD_R\) and \(T\) are the same as defined in Equation 12. The results of Figure 61 may be implemented in the same manner as the results from Figure 60 (depending upon whether the Road Rater or the FWD is being used). It should be cautioned that neither Equation 12 nor Equation 14 should be used at temperatures below 40°F or above 140°F.

Figure 62 is a comparison of backcalculated AC modulus between the Road
Rater and the FWD for the years of 1990 and 1991. Both sets of data were corrected for temperature using Equations 12 through 14. It is clear that in most instances the backcalculated modulus from the Road Rater is greater than from the FWD. The two major reasons for this is that the Road Rater data were not corrected for frequency, and the nonlinear behavior of the pavement structures between the 1,200-pound load of the Road Rater and the 9,000-pound load of the FWD.

Pavement Performance Analysis

Results of the field distress surveys are plotted on the field distress sheets in Appendix B. Rutting, longitudinal cracking, transverse cracking, map cracking, alligator cracking, raveling, and any other distresses are noted on those sheets. Three years of distress data have been obtained. Raveling, map cracking, and alligator cracking were not prevalent at any site through 1991; therefore, no model could be developed for these distresses. Distress models were developed for rutting, longitudinal cracking and transverse cracking, and each is discussed separately below.

Rideability data were also obtained from the Pavement Management Branch of the Kentucky Transportation Cabinet and that information will be analyzed below.

There are four rigid pavement sections included in this study. It is clear from studying the distress sheets in Appendix B, for those sites, that little additional distress has occurred in those sections since the first survey was conducted (1989). Consequently, no distress analysis can be made on those sections until more long-term data are collected and more distress occurs.

Rutting

A flexible pavement rutting model was developed and published by Allen and Deen (19) in 1985. The model was based upon repeated-load testing of laboratory compacted samples. In that study, a permanent deformation equation was developed for asphaltic concrete, dense-graded aggregate, and soil subgrades. The general form of the equation for each component of the pavement section was as follows:

\[ e_p = C_0 + C_1(N) + C_2(N)^2 + C_3(N)^3 \]  

where
\[ e_p = \text{Permanent deformation} \]
\[ N = \text{Number of wheel passes or ESAL's} \]
\[ C_0, C_1, C_2, C_3 = \text{Constants whose value depends on stress levels, temperature, confining stress, moisture.} \]
This model was never compared to actual field performance except in one or two cases. In this study, that model has been used to predict rutting for those sites where information was available on accumulated ESAL's. There is reasonably good agreement between predicted and measured as shown in Figure 63. Figure 64 shows the difference (error) between predicted rutting and measured rutting as a function of accumulated ESAL's. The model appears to predict equally well at all levels of accumulated ESAL's. The absolute value of 14 of 18 data points in Figure 64 are less than 0.1 inch, indicating a good prediction. The average error in prediction for all 18 points was 0.074 inch (slightly greater than 1/16-inch).

Longitudinal Cracking

Longitudinal crack lengths were totalled for each site from the distress sheets in Appendix B (flexible pavements only). These totals were plotted as a function of accumulated ESAL's in Figure 65 and as a function of accumulated AADT's in Figure 66. There was a very poor correlation with both parameters. The \( R^2 \) value for accumulated ESAL's is only 0.28. The relationship was so poor for accumulated AADT's that no regression analysis was performed. The same longitudinal crack totals were plotted as a function of age in Figure 67. A reasonably good correlation resulted with an \( R^2 \) value of 0.71. From this, it must be concluded that age is a better predictor of longitudinal cracking than ESAL's.

It is possible to estimate the amount of linear feet of cracking per mile in a flexible pavement by means of the relationship developed in Figure 67. That relationship is as follows:

\[
\text{Log}(C_{\text{Long}}) = [1.656 \times \text{Log(Age)} - 0.0156] \times [10.56] \tag{16}
\]

where

\[ C_{\text{Long}} = \text{Length of longitudinal cracking per mile.} \]

The previous relationship was developed for a 500-foot section. The constant 10.56 in Equation 16 converts the relationship to a 1-mile section.

Transverse Cracking

Transverse crack lengths were also totalled for each site by year and plotted as a function of accumulated ESAL's and accumulated AADT's (Figures 68 and 69). Clearly there is no correlation of accumulated ESAL's and accumulated AADT's with transverse crack length. Crack lengths were plotted as a function of age as shown in Figure 70. Although the correlation between transverse crack length and age is
not as good as between longitudinal crack length and age, there is a definite correlation ($R^2 = 0.53$). Transverse cracking per mile may be estimated from the regression equation in Figure 70 as follows:

$$\log(C_{\text{Trans}}) = [-0.285 + 0.182(Age)] \times [10.56]$$  \hfill (17)

where

$$C_{\text{Trans}} = \text{Length of transverse crack per mile.}$$

Rideability

Rideability Index values for all sites for years 1989 through 1992 are listed in Table 12. Figure 71 shows the change in Rideability Index as a function of age for flexible pavements (RI data from years 1989 through 1992). Clearly, the change in rideability cannot be estimated from age. These same rideability data were plotted as a function of accumulated AADT's in Figures 72 through 74. The dashed straight lines drawn through the data for each site in each of those figures were determined visually (not a regression line) to represent the RI behavior with accumulated AADT's for that site. The magnitude of the slopes tended to fall into three general categories. Those categories are (1) coal-haul roads carrying more than 50,000 tons of coal per year, (2) coal-haul roads carrying less than 50,000 tons of coal per year, and (3) non coal-haul roads. Each category is represented by the previously mentioned Figures 72 through 74. The calculated slopes of the straight lines for each site are as follows:

<table>
<thead>
<tr>
<th>COAL-HAUL (&gt;50,000 TONS/YEAR)</th>
<th>SLOPE</th>
</tr>
</thead>
<tbody>
<tr>
<td>US 23, Lawrence County</td>
<td>-0.382</td>
</tr>
<tr>
<td>US 119, Pike County</td>
<td>-0.123</td>
</tr>
<tr>
<td>Daniel Boone Pkwy., Clay County</td>
<td>-0.062</td>
</tr>
<tr>
<td>Mountain Pkwy., Powell County</td>
<td>-0.096</td>
</tr>
<tr>
<td>KY 80, Floyd County</td>
<td>-0.062</td>
</tr>
<tr>
<td>KY 11, Owsley County</td>
<td>-0.096</td>
</tr>
<tr>
<td>Average =</td>
<td>-0.137</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>COAL-HAUL (&lt;50,000 TONS/YEAR)</th>
<th>SLOPE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interstate 24, Marshall County</td>
<td>-0.038</td>
</tr>
<tr>
<td>Interstate 64, Carter County</td>
<td>-0.038</td>
</tr>
<tr>
<td>Western Kentucky Pkwy., Lyon County</td>
<td>-0.043</td>
</tr>
<tr>
<td>Average =</td>
<td>-0.040</td>
</tr>
</tbody>
</table>
NON COAL-HAUL

<table>
<thead>
<tr>
<th>Location</th>
<th>Slope (RI per year)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interstate 71, Henry County</td>
<td>-0.014</td>
</tr>
<tr>
<td>Cumberland Pkwy., Barren County</td>
<td>-0.018</td>
</tr>
<tr>
<td>KY 61, Hardin County</td>
<td>-0.035</td>
</tr>
<tr>
<td>KY 4, Fayette County</td>
<td>-0.008</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>-0.019</strong></td>
</tr>
</tbody>
</table>

The average slopes from the above list represent the change in RI per year for each of the three general categories. The RI for some future year can be estimated for any site in one of the three categories by the following equations (assuming a present RI value is known):

\[
\text{Coal-Haul (}>50,000 \text{ tons/year})
\]

\[
\text{RI}_{\text{FUTURE}} = \text{RI}_{\text{PRESENT}} - 0.137 \times (X),
\]

(19)

\[
\text{Coal-Haul (<50,000 \text{ tons/year})}
\]

\[
\text{RI}_{\text{FUTURE}} = \text{RI}_{\text{PRESENT}} - 0.040 \times (X),
\]

(20)

\[
\text{Non Coal-Haul}
\]

\[
\text{RI}_{\text{FUTURE}} = \text{RI}_{\text{PRESENT}} - 0.019 \times (X),
\]

(21)

where

\[
\text{RI}_{\text{FUTURE}} = \text{The desired RI of some future year,}
\]

\[
\text{RI}_{\text{PRESENT}} = \text{The present RI, and}
\]

\[
X = \text{Additional accumulated AADT's (in millions) from the present year to the desired year.}
\]

RI has also been plotted as a function of accumulated ESAL's in Figures 75 and 76 for those sites for which the data were available. Trend lines were visually drawn through the data in those figures, as indicated by the dotted straight lines (regression analysis was not performed). A trend line was not drawn through the data for Cumberland Parkway because the trend of the data was generally increasing. Slopes were determined for the trend lines, and are listed as follows:
There appears to be a significant relationship between slope of the RI versus accumulated ESAL's curve as calculated above and the magnitude of the backcalculated subgrade modulus (subgrade data from KY 4 was not available). This relationship is illustrated in Figure 77. A regression analysis was performed on the data in Figure 77. The resulting best-fit equation is as follows:

\[ \Delta RI = -0.8674 + 0.0449(M_{SUB}) - 0.00064(M_{SUB})^2 \]  

(22)

where

\( \Delta RI \) = Change in RI per 1,000,000 ESAL's, and
\( M_{SUB} \) = Backcalculated subgrade modulus (in thousands).

However, if the backcalculated subgrade modulus is not available, the RI for some future accumulated ESAL's may be estimated by using the above calculated average slope as follows:

\[ RI_{FUTURE} = RI_{PRESENT} - 0.190 * (X) \]  

(23)

where

\( X \) = Difference between present and future accumulated ESAL's (in millions).

Depending upon the information available, the designer or pavement manager may estimate a future RI value from Equations 9 through 11 (if only AADT's are available) or Equation 13 (if ESAL's are available). Equation 12 should be used to calculate future RI when subgrade modulus and ESAL's are available.

Figure 78 shows plots of RI as a function of accumulated AADT's for the four rigid sections. Trend lines were visually determined for each of the sites (dotted lines
in Figure 78). Slopes of the lines were calculated to determine the change in RI per 1,000,000 AADT's. The calculated slopes are as follows:

<table>
<thead>
<tr>
<th>Location</th>
<th>Slope</th>
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</thead>
<tbody>
<tr>
<td>Audubon Parkway, Daviess County</td>
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<tr>
<td>US 31W, Hardin County</td>
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<tr>
<td>Interstate 65, Bullitt County</td>
<td>-0.0059</td>
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<td>-0.0190</td>
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</table>

There appears to be no discernible correlation of these slopes with age or pavement thickness. Reliable accumulated ESAL's were not available for two of the sites; therefore, no attempts were made to correlate change in RI with ESAL's. Complete subgrade information also was not available, preventing a correlation with subgrade strength. Although a general model must wait until further information is available, the change in RI for the individual sites may be estimated from the following equation:

\[
\Delta R_{I,RIGID} = R_{I,PRESENT} - (S_{RIGID}) \times (X)
\]

where

\(\Delta R_{I,RIGID}\) = The change in RI for some future accumulation of AADT's,
\(R_{I,PRESENT}\) = Present RI,
\(S_{RIGID}\) = Slope for each individual site as calculated above, and
\(X\) = Additional accumulation of AADT's from present to future date (in millions).

**THE USE OF OTHER THEORIES FOR BACKCALCULATION**

The remaining objective of this study was to review the appropriateness of using theories other than layer elastic theory (such as finite elements, or nonlinear finite elements) to model pavement structures. Less effort has been expended in fulfilling this objective than in the other objectives. However, as noted in the section of this report entitled "Background," other researchers have used finite elements to model pavements, and the authors of Reference 1 have proposed a method for correcting for the nonlinear response of pavement materials. However, it appears that these methods may not be used routinely by most day-to-day practitioners.

During the course of this study, it was noted that some field deflection bowls did not match any theoretical deflection bowl very well. An example of a field deflection bowl that matches theory quite well is shown in Figure 79 (FWD data). Figure 80 displays a best-fit deflection bowl from KY 11 (Owsley County) that does
not match very well the theoretical deflection bowl calculated from layer elastic theory. It appears this is often the case when the field deflection bowls are very steep. (Note the difference in the magnitude of the deflections of the No. 1 sensors.) This indicates the possible inability of layer elastic theory to accurately model some situations.

An even more striking example is given in Figures 81 through 84. The data in those figures were obtained from FWD tests on a city street (not a part of this study). The street section was 2.0 inches of asphaltic concrete base, 8.0 inches of dense-graded aggregate, and a soil subgrade with an average in-situ CBR of slightly less than 3.0. The asphaltic concrete was badly deteriorated, and 100 percent of the surface had alligator cracking. The small squares in Figures 81 through 83 represent the experimental deflections, and the three lines represent predicted values from a linear finite element analysis using three values of modulus for the asphaltic concrete. In every case, an AC modulus of 50,000 pounds per square inch more closely approximately the experimental data. The AC was assumed in Figure 81 to behave as a solid, unbroken mass; the modulus of the dense-graded aggregate was assumed to be 75,000 pounds per square inch; the subgrade modulus was assumed to be 4,200 pounds per square inch. For these conditions, the linear finite element model did not predict well (particularly for the No. 1 and No. 2 sensors).

Figure 82 also shows the results of a linear finite element analysis. The assumed dense-graded aggregate and subgrade strengths were unchanged. However, in this analysis, the AC was assumed to be cracked. This was modeled by permitting the nodal points in the upper portion of the finite element grid to "uncouple" as shown in the example in Figure 84. It is clear that the more sophisticated modeling technique produced a more accurate prediction. The sum of squares of residuals for the cracked model was 12.8, and for the solid model it was 28.6 (using only the line representing an AC modulus of 50,000 pounds per square inch).

In Figure 83, the AC is assumed to be cracked, the subgrade modulus is held constant at 4,200 pounds per square inch, but the dense-graded aggregate modulus is reduced to 25,000 pounds per square inch. The prediction is very good.

It must be concluded that more sophisticated modeling would, in many situations, provide better predictive capabilities. More research should be performed in this area.

CONCLUSIONS

Correlations between load levels of the FWD are less variable than are correlations between load levels of the Road Rater. This indicates that FWD data have less experimental error than do Road Rater Data.
Sensor No. 1 of the FWD yields a disproportionately larger deflection reading than does Sensor No. 1 of the Road Rater (when the different load magnitudes are scaled to the same magnitude). This is partially due to the location of the sensors, and partially due to the nonlinear behavior of the pavement structure.

The correlation between the No. 4 sensors on the two testing devices is very poor. This is primarily due to the large variation (scatter) in the data from the No. 4 sensor of the Road Rater.

Until further research is conducted, it appears that correlations between the two devices should be performed on a sensor-by-sensor basis and not by combining the data from the first four sensors into one data set and performing correlations on the entire data set.

There is a high variability in subgrade modulus within each test site with coefficients of variability as high as 75 percent.

The 1,200-pound load generally yielded the most consistent results when using the Road Rater (lowest coefficients of variability). It is recommended that the 1,200-pound load be used for pavement analysis when using the Road Rater.

It appears subgrade modulus may be somewhat related to rainfall pattern, with a nine- to 12-month lag. This apparent relationship developed from the data obtained from both testing devices, and therefore, does not appear to be machine related.

In general, subgrade modulus values backcalculated from the FWD are greater than the values calculated from Road Rater data.

There is little or no correlation between laboratory measured subgrade moisture content and resilient modulus.

There is no correlation between density of laboratory samples and resilient modulus.

There was a very large variability in backcalculated AC modulus within each 500-foot section. It appears the variability was greater for thicker pavements. This may indicate the Road Rater is not capable of inputting sufficient energy into the pavement to deflections that are within the range of precision of the deflection sensors.

The data developed in this study may assist the designer in assessing the reliability of design (Figures 33, 54, and 55).

There is a large variability between the backcalculated AC modulus and the laboratory resilient modulus. This was expected, in view of the variability of the
backcalculated field AC modulus.

Equations 12, 13, and 14 (developed from Figures 60 and 61) provide a simple method of converting from backcalculated AC modulus at pavement test temperature to the standard reference temperature of 70° F.

The rutting model developed under another study and referenced in this study predicted reasonably well rutting on all of the test sites. As more long-term data become available, this model will be refined. The average error of prediction was slightly over 1/16 inch. The model appears to predict equally well at all levels of ESAL’s.

Age is a better predictor of longitudinal cracking and transverse cracking than are ESAL’s or AADT’s.

The change in Rideability Index with time may be estimated from the data obtained in this study. The use of these models may be limited because of the small sample size from which they were developed.

The models developed in this study may be used to estimated the various modes of distress and to estimate the change in parameters with time; however, it is critical that more long term data be obtained, to refine these models and to make estimations that are more accurate.

It is apparent more sophisticated modeling techniques would enhance predictive capabilities. It is recommended that further work be performed in this area to develop these models.

**RECOMMENDED IMPLEMENTATION**

It is recommended that the data base that has been developed to date on these test sites be used as a basis for continuing to monitor and record behavior. It is recommended that these sites continue to be monitored annually (as is currently being done). From this long-term data, it will be possible to continue to develop and refine the models developed in this study. The refined models will more accurately predict the service life of all pavements in the future.
REFERENCES


Table 1. List of Long-Term Pavement Performance Study Sites.

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<tr>
<th>STATE ID NO.</th>
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38
Table 2. Frequency and Location of Deflection Tests Per Site.

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<th>Location</th>
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<td>Test Pits</td>
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<td>P₁</td>
<td>ML (Mid Lane)</td>
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<tr>
<td></td>
<td>4.5</td>
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<td>OWP(Outer Wheel Path)</td>
<td>± Joint</td>
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<td>Load</td>
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<td>Load</td>
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NOTE: ⁽¹⁾ Maximum number of tests per lane
Figure 1. Location of Study Sites.
Sampling Point Locations After Test Section - Experiment 1
Asphalt Concrete over Granular Base

Figure 2. Sampling Point Locations - Asphalthic Concrete.
Sampling Point Locations After Test Section - Experiment 3
Jointed Plain Concrete Pavement

Figure 3. Sampling Point Locations - Jointed Plain Concrete.
Figure 4. Locations of Drill Holes for Temperature Measurements.
Figure 5. Location of Road Rater Sensors.

Figure 6. Conversion of Dynamic Sine Wave to Equivalent Square Wave for Road Rater.
Figure 7. Location of Falling Weight Deflectometer Sensors.
Figure 8. Correlation of Load Levels at Sensor No. 1 for Road Rater.
Figure 9. Correlation of Load Levels at Sensor No. 2 for Road Rater.
ROAD RATER MODEL 400B
LOAD COMPARISON, SENSOR 3

Figure 10. Correlation of Load Levels at Sensor No. 3 for Road Rater.
Figure 11. Correlation of Load Levels at Sensor No. 4 for Road Rater.
LOAD COMPARISON, 0-inches

Figure 12. Correlation of Load Levels at 0 Inches for FWD.
Figure 13. Correlation of Load Levels at 36 Inches for FWD.
Figure 14. Correlation of Load Levels at 60 Inches for FWD.
Figure 15. Correlation at Sensor No. 1 Between Road Rater and FWD.
Figure 16. Correlation at Sensor No. 2 Between Road Rater and FWD.
Figure 17. Correlation at Sensor No. 3 Between Road Rater and FWD.
Figure 18. Correlation at Sensor No. 4 Between Road Rater and FWD.
### TABLE 3. VARIATION OF SUBGRADE MODULUS FOR 1990

<table>
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<tr>
<th>Hypothetical District</th>
<th>Revenue Bonds &amp; Special Assessment</th>
<th>Revenue Bonds only</th>
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**VARIATION OF SUBGRADE MODULUS (1990 FIELD DATA, 600-LE RELOAD)**

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**VARIATION OF SUBGRADE MODULUS (1990 FIELD DATA, 1200-LE RELOAD)**

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**VARIATION OF SUBGRADE MODULUS (1990 FIELD DATA, 1800-LE RELOAD)**

<table>
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<th>Revenue Bonds only</th>
<th>Service Charges</th>
<th>Property Charges</th>
<th>Bond Charges</th>
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Figure 19. Variation of Subgrade Modulus - 1990 (KY 11, Lee County; US 119, Pike County; KY 80, Floyd County).
Figure 20. Variation of Subgrade Modulus - 1990 (KY 61, Hardin County; Interstate 64, Carter County; Interstate 24, Marshall County).
Figure 21. Variation of Subgrade Modulus - 1990 (Daniel Boone Pkwy., Clay County; Cumberland Pkwy., Barren County; AA Highway, Lewis County).
Figure 22. Variation of Subgrade Modulus - 1990 (Ky 11, Owsley County; US 23, Lawrence County; Western Kentucky Pkwy., Lyon County).
Figure 23. Variation of Subgrade Modulus - 1990 (Mountain Pkwy., Powell County; Pennyrile Pkwy., Webster County).
### TABLE 4. VARIATION OF SUBGRADE MODULUS FOR 1991

#### VARIATION OF SUBGRADE MODULUS (1991 FIELD DATA, 620-868 LOAD)

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<th>I-64</th>
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<th>Cumberland</th>
<th>AA</th>
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<th>OW</th>
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#### VARIATION OF SUBGRADE MODULUS (1991 FIELD DATA, 1230-868 LOAD)

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#### VARIATION OF SUBGRADE MODULUS (1991 FIELD DATA, 1800-868 LOAD)

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<th>MOUNT.PY</th>
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Figure 24. Variation of Subgrade Modulus - 1991 (KY 11, Lee County; US 119, Pike County; KY 80, Floyd County).
Figure 25. Variation of Subgrade Modulus - 1991 (KY 61, Hardin County; Interstate 64, Carter County; Daniel Boone Pkwy., Clay County).
Figure 26. Variation of Subgrade Modulus - 1991 (Cumberland Pkwy., Barren County; AA Highway, Lewis County; KY 11, Owsley County).
Figure 27. Variation of Subgrade Modulus - 1991 (US 23, Lawrence County; Mountain Pkwy., Powell County; Interstate 71, Henry County).
Figure 28. Mean Values and Coefficient of Variation of Subgrade Modulus - 1990.
Figure 29. Mean Values and Coefficient of Variation of Subgrade Modulus - 1991.
Figure 30. Coefficient of Variation of Subgrade Modulus for 1,200-Pound Load Versus Coefficient of Variation for 600-Pound Load.
Figure 31. Coefficient of Variation of Subgrade Modulus for 1,800-Pound Load Versus Coefficient of Variation for 600-Pound Load.
Figure 32. Coefficient of Variation of Subgrade Modulus for 1,800-Pound Load Versus Coefficient of Variation for 1,200-Pound Load.
Figure 33. Design Reliability for Subgrade Modulus.
Table 5. Variation of Subgrade Modulus with Time (Road Rater).

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<td>I-24</td>
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Figure 34. Average Subgrade Modulus by Site for Three Years (Road Rater).
Table 6. Variation of Subgrade Modulus with Time (FWD).

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Figure 35. Average Subgrade Modulus by Site for Three Years (FWD).
Figure 36. Cumulative Departure from Normal Rainfall from 1987 through 1991.
Figure 37. Comparison of Backcalculated Subgrade Modulus (Road Rater and FWD).
Figure 38. Subgrade Modulus for 1990 Versus Subgrade Modulus for 1989.
Figure 39. Subgrade Modulus for 1991 Versus Subgrade Modulus for 1989.

\[ R^2 = 0.89 \]

\[ y = 5059.91 + 1.03x \]
LABORATORY MODULUS VS. FIELD MODULUS

MOD = 7.5 + 0.75 MOD

Figure 40. Backcalculated Subgrade Modulus Versus Laboratory Resilient Modulus - 1989.
Figure 41. Laboratory Moisture Content Versus Laboratory Resilient Modulus.
RESILIENT MODULUS VS. LAB DENSITY

Figure 42. Laboratory Density Versus Laboratory Resilient Modulus.
Figure 43. Field CBR Values Versus Laboratory Resilient Modulus.
Figure 44. Field CBR Values Versus Backcalculated (Road-Rater) Modulus.
Table 7. Moisture Contents and Modulus of Subgrade (Laboratory and Field).

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<th>STATION</th>
<th>% MOISTURE</th>
<th>MODULUS (PSI)</th>
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Table 8. Variation of AC Modulus for 1990.

**VARIATION OF AC MODULUS (1990 CONVERTED FIELD DATA)**

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Figure 45. Variation of AC Modulus for 1990 (KY 11, Lee County; US 119, Pike County; KY 80, Floyd County).
Figure 46. Variation of AC Modulus for 1990 (KY 61, Hardin County; Interstate 64, Carter County; Interstate 24, Marshall County).
Figure 47. Variation of AC Modulus for 1990 (Daniel Boone Pkwy., Clay County; Cumberland Pkwy, Barren County; AA Highway, Lewis County).
Figure 48. Variation of AC Modulus for 1990 (KY 11, Owsley County; US 23, Lawrence County; Western Kentucky Pkwy., Lyon County).

VARIATION OF AC. MODULUS (1991 CONVERTED FIELD DATA)

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<th>KY-61</th>
<th>I-64 CAR</th>
<th>DAN. BOONE</th>
<th>CUMBERLAND</th>
<th>AA HW</th>
<th>KY11</th>
<th>OWSL</th>
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MEAN 1,138,194 357,435 658,566 497,772 618,388 546,054 530,000 453,501 446,284 248,127
STD 325,285 151,186 212,989 157,081 172,918 83,261 307,523 153,494 337,571 72,014
VARIANCE 1.058e+11 2.285e+10 4.536e+10 2.467e+10 2.990e+10 6.932e+09 9.457e+10 2.356e+10 1.139e+10 5.185e+09
CV (%) 28.6 42.3 32.3 31.6 28.0 15.2 58.0 33.8 75.6 29.0
Figure 49. Variation in AC Modulus for 1991 (KY 11, Lee County; US 119, Pike County; KY 80, Floyd County).
Figure 50. Variation in AC Modulus for 1991 (KY 61, Hardin County; Interstate 64, Carter County; Daniel Boone Pkwy., Clay County).
Figure 51. Variation in AC Modulus for 1991 (Cumberland Pkwy., Barren County; AA Highway, Lewis County; KY 11, Owsley County).
Figure 52. Variation in AC Modulus for 1991 (US 23, Lawrence County).
Figure 53. Coefficient of Variation for AC Modulus as a Function of AC Thickness.
DESIGN RELIABILITY FOR ASPHALT CONCRETE MODULUS

Figure 54. Design Reliability for AC Modulus.
Figure 55. Total Combined Reliability for a Particular Project.
BACKCALCULATED AC MODULUS
1990 VS. 1991

Figure 56. Backcalculated AC Modulus (1990 Versus 1991).
Table 10. Average Backcalculated AC Modulus for 1990.

(1990 field data, average mod. of sec.0.00-400.00)

<table>
<thead>
<tr>
<th>SITE</th>
<th>FIELD MODULUS (PSI)</th>
<th>FIELD TEMP. CONV.-&gt;10Hz (°F)</th>
<th>LABORATORY MODULUS SEC. TEMP. (PSI)</th>
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<td>I-64CARTER</td>
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Note: (*) : Incomplete data, not included in the plot below.

Figure 57. Backcalculated Field Modulus Versus Laboratory Resilient Modulus (1990).
Table 11. Average Backcalculated AC Modulus for 1991.

(1991 field data, average mod. of sec.0.00-400.00)

<table>
<thead>
<tr>
<th>SITE</th>
<th>FIELD MODULUS (PSI)</th>
<th>FIELD TEMP. CONV.-&gt;10Hz (°F)</th>
<th>LABORATORY MODULUS SEC. (PSI)</th>
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<td>I-71</td>
<td>2,000,000(*)</td>
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<td>1,203,584 A 75</td>
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</table>

Note: (*) : Incomplete data, not included in the plot below.

Figure 58. Backcalculated Field Modulus Versus Laboratory Resilient Modulus (1991).
Figure 59. Field Modulus vs. Laboratory Modulus (from Lee, Mahoney, and Jackson).
FIELD MODULUS/LAB. MODULUS
VERSUS TEMPERATURE (ROAD RATER)

Figure 60. Ratio of Field Modulus to Laboratory Modulus vs. Temperature (Road Rater).
FIELD MODULUS/LAB. MODULUS
VERSUS TEMPERATURE (FWD)

LOG Y = 1.378 - 0.03072(X) + 0.000122(X)^2
R^2 = 0.51

Figure 61. Ratio of Field Modulus to Laboratory Modulus vs. Temperature (FWD).
COMPARISON OF BACKCALCULATED AC MODULUS
FWD vs. ROAD RATER (TEMP.CORRECTED)

Figure 62. Comparison of Backcalculated AC Modulus from FWD and Road Rater.
PREDICTED RUTTING FROM MODEL VERSUS MEASURED RUTTING

Figure 63. Predicted Rutting vs. Measured Rutting.
Figure 64. Difference Between Predicted and Measured Rutting vs. Accumulated ESAL's.
LONGITUDINAL CRACKING
(Per 500-Foot Section)

\[ \log(Y) = 0.369 \times \log(X) + 1.3546 \]

\[ R^2 = 0.28 \]

Figure 65. Longitudinal Cracking as a Function of Accumulated ESAL's.
LONGITUDINAL CRACKING
(PER 500-FOOT SECTION)

Figure 66. Longitudinal Cracking as a Function of Accumulated AADT's.
LONGITUDINAL CRACKING
(PER 500-FOOT SECTION)

Figure 67. Longitudinal Cracking as a Function of Age.
Figure 68. Transverse Cracking as a Function of Accumulated ESAL's.
TRANSVERSE CRACKING
(PER 500-FOOT SECTION)

Figure 69. Transverse Cracking as a Function of Accumulated AADT's.
TRANSVERSE CRACKING
(PER 500-FOOT SECTION)

FEET OF CRACK

AGE (YEARS)

LOG(Y) = -0.285 + 0.182(AGE)

R^2 = 0.53

Figure 70. Transverse Cracking as a Function of Age.
Table 12. Rideability Index Values for All Sites.

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Figure 71. Change in Rideability Index as a Function of Age (Flexible Pavements).
RI VERSUS ACCUMULATED AADT'S
COAL HAUL ROADS > 50,000 TONS/YEAR

Figure 72. Riedability Index as a Function of Accumulated AADT's.
(Coal Haul Roads > 50,000 Tons/Year)
Figure 73. Rideability Index as a Function of Accumulated AADT's.
(Coal Haul Roads < 50,000 Tons/Year)
Figure 74. Rideability Index as a Function of Accumulated AADT's. (Non-Coal Haul Roads)
Figure 75. Rideability Index as a Function of Accumulated ESAL's (Flexible).
Figure 76. Rideability Index as a Function of Accumulated ESAL's (Flexible).
Figure 77. Change in Rideability Index as a Function of Backcalculated Subgrade Modulus.
Figure 78. Rideability Index as a Function of Accumulated AADT's (Rigid).
Figure 79. Comparison of Field and Backcalculated Deflection Bowls Showing Relatively Good Fit.

Figure 80. Comparison of Field and Backcalculated Deflection Bowls Showing a Relatively Poor Fit.
Figure 83. Finite Element Deflection Bowl Prediction Assuming Cracked Asphaltic Concrete and a Reduced Dense-Graded Aggregate Modulus.

Figure 84. Finite Element Grid Showing Deformed Shape of Cracked Asphalt Under FWD Loading.
Figure 81. Finite Element Deflection Bowl Prediction Assuming Uncracked Asphaltic Concrete.

Figure 82. Finite Element Deflection Bowl Prediction Assuming Cracked Asphaltic Concrete.
APPENDIX A

SITE DATA SHEETS
Travel Direction: South
Street/Road Type: Parkway
Mile Point: 2.9
Length: 500 feet
Pavement Surface Type: AC
Lane Width: 12 feet
Number of Travel Lanes: 4
Shoulder Width: 10 feet

Approximate Grade: -0.84%
Super Elevation: No

Layer Thickness
DGA: 6-inches
AC: 11.7-inches
AADT: 4,720 (1989)
Travel Direction: East
Street/Road Type: Interstate
Mile Point: 22.0
Length: 500 feet
Pavement Surface Type: AC
Lane Width: 14 feet
Number of Travel Lanes: 4
Shoulder Width: 8 feet

Approximate Grade: +1.2%
Super Elevation: No

Layer Thickness
DGA: 11-inches
AC: 7.5-inches
AADT: 13,820 (1989)
Travel Direction: East
Street/Road Type: Parkway
Mile Point: 20.0
Length: 500 feet
Pavement Surface Type: PCC
Lane Width: 12 feet
Number of Travel Lanes: 4
Shoulder Width: 10 feet

Approximate Grade: +0.34%
Super Elevation: No
Layer Thickness
DGA: 4-inches
PCC: 9-inches
Travel Direction: North
Street/Road Type: Parkway
Mile Point: 65.0
Length: 500 feet
Pavement Surface Type: AC/PCC
Lane Width: 12 feet
Number of Travel Lanes: 4
Shoulder Width: 12 feet

Approximate Grade: -0.47%
Super Elevation: No

Layer Thickness
DGA: 4-inches
PCC: 10-inches
AC: 7-inches
AADT: 6,760 (1989)
Travel Direction: East
Street/Road Type: Parkway
Mile Point: 9.17
Length: 500 feet
Pavement Surface Type: AC
Lane Width: 12 feet
Number of Travel Lanes: 4
Shoulder Width: 9.5 feet

Approximate Grade: +2.4%
Super Elevation: No

Layer Thickness
DGA: 0
AC: 15-inches
AADT: 4,090 (1989)
Travel Direction: North
Street/Road Type: State Primary
Mile Point: 32.7
Length: 500 feet
Pavement Surface Type: PCC
Lane Width: 12 feet
Number of Travel Lanes: 4
Shoulder Width: 9.5 feet

Approximate Grade: +0.88%
Super Elevation: No

Layer Thickness
DGA: 4-inches
PCC: 8.75-inches
AADT: 13,700 (1989)
Travel Direction: South
Street/Road Type: State Primary
Mile Point: 1.7
Length: 500 feet
Pavement Surface Type: AC
Lane Width: 12 feet
Number of Travel Lanes: 4
Shoulder Width: 10 feet
Approximate Grade: -0.59%
Super Elevation: No
Layer Thickness
DGA: 12-inches
AC: 9-inches
Travel Direction: North
Street/Road Type: Interstate
Mile Point: 106.2
Length: 500 feet
Pavement Surface Type: PCC
Lane Width: 12 feet
Number of Travel Lanes: 4
Shoulder Width: 10 feet

Approximate Grade: 0%
Super Elevation: No
Layer Thickness
DGA: 5.5-inches
PCC: 9-inches
AADT: 33,740 (1989)
LTTP SITE NO. 9
INTERSTATE 71
HENRY COUNTY, KENTUCKY

Travel Direction: North
Street/Road Type: Interstate
Mile Point: 25.0
Length: 500 feet
Pavement Surface Type: PCC
Lane Width: 12 feet
Number of Travel Lanes: 4
Shoulder Width: 10 feet

Approximate Grade: Sag
Super Elevation: No
Layer Thickness
DGA: 6-inches
AC: 6.5-inches
Broken PCC: 10 inches
AADT: 20,690 (1989)
Travel Direction: East
Street/Road Type: Interstate
Mile Point: 73.8
Length: 500 feet
Pavement Surface Type: PCC
Lane Width: 12 feet
Number of Travel Lanes: 4
Shoulder Width: 10 feet

Approximate Grade: +1.1%
Super Elevation: No

Layer Thickness
DGA: 6-inches
PCC: 10-inches
AADT: 17,260 (1989)
Travel Direction: East
Street/Road Type: State Primary
Mile Point: 3.5
Length: 500 feet
Pavement Surface Type: AC
Lane Width: 12 feet
Number of Travel Lanes: 4
Shoulder Width: 10 feet

Approximate Grade: +1.3%
Super Elevation: No

Layer Thickness
DGA: 12-inches
AC: 8.5-inches
AADT: 50,000 (1989)
Travel Direction: West
Street/Road Type: Interstate
Mile Point: 170.6
Length: 500 feet
Pavement Surface Type: AC
Lane Width: 12 feet
Number of Travel Lanes: 4
Shoulder Width: 10 feet

Approximate Grade: 0%
Super Elevation: No

Layer Thickness
DGA: 12-inches
AC: 12-inches
AADT: 10,700 (1989)
Travel Direction: West
Street/Road Type: State Primary
Mile Point: 2.8
Length: 500 feet
Pavement Surface Type: AC
Lane Width: 12 feet
Number of Travel Lanes: 2
Shoulder Width: 10 feet

Approximate Grade: 0%
Super Elevation: No

Layer Thickness
STABILIZATION: 6-inches, lime
DGA: 4-inches
AC: 11.0-inches
AADT: 2,000 (1989)
Travel Direction: North
Street/Road Type: 2-Lane
Mile Point: 11.14
Length: 500 feet
Pavement Surface Type: AC
Lane Width: 12 feet
Number of Travel Lanes: 4
Shoulder Width: 10 feet

Approximate Grade: -1.54
Super Elevation: No

Layer Thickness
DGA: 6-inches
AC: 8.75-inches
AADT: 2,100 (1989)
Travel Direction: West
Street/Road Type: Parkway
Mile Point: 30.8
Length: 500 feet
Pavement Surface Type: AC/PCC
Lane Width: 12 feet
Number of Travel Lanes: 4
Shoulder Width: 10 feet

Approximate Grade: +.189%
Super Elevation: No

Layer Thickness
DGA: 5-inches
Broken PCC: 8-inches
AC: 7.87-inches
AADT: 7,150 (1989)
Travel Direction: South

Street/Road Type: State Secondary

Mile Point: 13.3

Length: 500 feet

Pavement Surface Type: AC

Lane Width: 12 feet

Number of Travel Lanes: 3

Shoulder Width: 9 feet

Approximate Grade: -2.3%

Super Elevation: Yes

Layer Thickness

DGA: 6-inches

AC: 7.5-inches

AADT: 2,640 (1989)
Travel Direction: West
Street/Road Type: Parkway
Mile Point: 15.77
Length: 500 feet
Pavement Surface Type: AC
Lane Width: 12 feet
Number of Travel Lanes: 2
Shoulder Width: 8 feet

Approximate Grade: +1.07%
Super Elevation: No

Layer Thickness

DGA: 12-inches
AC: 7.5-inches
AADT: 5,000 (1989)
Travel Direction: East
Street/Road Type: State Primary
Mile Point: 6.5
Length: 500 feet
Pavement Surface Type: AC
Lane Width: 12 feet
Number of Travel Lanes: 4
Shoulder Width: 10 feet

Approximate Grade: Vert. Curve
Super Elevation: No
Layer Thickness
AC: 20.0-inches
AADT: 9,730
Travel Direction: South

Approximate Grade: Vert. Curve

Street/Road Type: State Primary

Super Elevation: No

Mile Point: 2.4

Layer Thickness

Length: 500 feet

DGA: 0

Pavement Surface Type: AC

AC: 11.5

Lane Width: 12 feet

AADT: 10,800

Number of Travel Lanes: 4

Shoulder Width: 10 feet
Travel Direction: North

Street/Road Type: State Primary

Mile Point: 17.5

Length: 500 feet

Pavement Surface Type: AC

Lane Width: 12 feet

Number of Travel Lanes: 4

Shoulder Width: 10 feet

Percent Grade: Vert. Curve

Super Elevation: No

Layer Thickness

DGA: 4-inches

57 Stone: 4-inches

AC: 13-inches

AADT: 5,740 (1989)
APPENDIX B

DISTRESS SURVEY SHEETS
STATE ID # 1
COUNTY  LYON
MILE POINT/STATION 2.9 WB
ROUTE  WESTERN KY PKEWY
PAVEMENT TYPE  AC

1989 —

1990 —

1991 —

1992 —

6" CORE HOLES

148
STATE ID # 2
COUNTY MARSHALL
MILE POINT/STATION 2200 EB
ROUTE I 24
PAVEMENT TYPE AC

1989 — ○
1990 — ●
1991 GREEN △
1992 — □
<table>
<thead>
<tr>
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<tbody>
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<td></td>
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</table>

**STATE ID # 3**

**COUNTY**: DAVIESS

**MILE POINT/STATION**: 20.0 EB

**ROUTE**: AUDUBON PKWY

**Pavement Type**: PCC

1989

1990

1991 **GREEN**

**Spalling**

**POD OUT**

**CORE HOLES**

**EDGE BROKEN AWAY**

<table>
<thead>
<tr>
<th>500'</th>
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<tbody>
<tr>
<td>450'</td>
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<tr>
<td>400'</td>
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<tr>
<td>State ID: 4</td>
</tr>
<tr>
<td>-------------</td>
</tr>
<tr>
<td>County: Webster</td>
</tr>
<tr>
<td>Mile Point/Station: 65.0 NB</td>
</tr>
<tr>
<td>Route: Pennyville Pkwy</td>
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<tr>
<td>Pavement Type: AC/PCC</td>
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<table>
<thead>
<tr>
<th>1989</th>
<th>1992</th>
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- 1990
- 1991 Green

<table>
<thead>
<tr>
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<table>
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- 450' |

- 400' |

- 350' |

<table>
<thead>
<tr>
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<thead>
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- Core Holes

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- 50' |

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151
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<td><img src="image3" alt="Diagram" /></td>
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**STATE ID # 5**

**COUNTY: BARREN**

**MILE POINT/STATION** 9.17 EB

**ROUTE** Cumberland Pkwy

**Pavement Type** AC

<table>
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<td>△</td>
</tr>
<tr>
<td>1992</td>
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**Diagram Details:**
- **BLEEDING**
- **SUNKEN 3/4'' RUT**
- **CORE HOLES**

**152**
STATE ID # 6
COUNTY HARDIN
MILE POINT/STATION 32.7 NB
ROUTE US 31 W
PAVEMENT TYPE PCC

1989 —
1990 —

CORE HOLE

CORE HOLES

CORE HOLE
STATE ID # 7
COUNTY HARDIN
MILE POINT/STATION 3.4 SB
ROUTE KY 61
PAVEMENT TYPE AC

1989 -
1990 -
1991 -
1992 -

150'
100'
50'
0'

500'
450'
400'
350'

154
STATE ID # 8

COUNTY: Bullitt

MILE POINT/STATION: 106.2 NB

ROUTE: I-65

Pavement Type: PCC

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1989 -
1990 -
1991 -
**STATE ID #** 11  
**COUNTY** FAYETTE  
**MILE POINT/STATION** 73.8 EB  
**ROUTE** I 64  
**PAVEMENT TYPE** PCC  
**1989 —**  
**1990 —**  
**1991 —**  

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**STATE ID # 12**

**COUNTY** FAYETTE

**MILE POINT/STATION** 3.5 EB

**ROUTE** KY 4

**Pavement Type** AC

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<th>400'</th>
<th>350'</th>
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<td>1/4</td>
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<td>3/8</td>
<td>1/4</td>
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</table>

**State ID #** 13

**County** Carter

**Mile Point/Station** 170.6 WB

**Route** I-64

**Pavement Type** AC

- 1989 — ○
- 1990 — □
- 1991 — △
- 1992 — □

159
STATE ID # 14

COUNTY LEWIS

MILE POINT/STATION 1493+40 WB

ROUTE AA HIGHWAY

Pavement Type AC

1989 —
1990 —
1991 —
1992 —

160
STATE ID # 15

COUNTY LEE

MILE POINT/STATION 11614 NB

ROUTE KY 11

PAVEMENT TYPE AC

1989 —
1990 —
1991 —
1992 —

161
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<th>LWG</th>
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</tbody>
</table>

**State ID #: 16**

**County: Powell**

**Mile Point/Station:** 30.8 WB

**Route: Mountain PKWY**

**Pavement Type: AC/PCC**

- **1989:** ○
- **1990:** ●
- **1991:** ▲
- **1992:** □

**Note:** CORE HOLE

**Hole in Pavement:**

**Core Holes:**

**Distance:**
- 350'
- 400'
- 450'
- 500'
STATE ID # 17
COUNTY Owsley
MILE POINT/STATION 13.3 SB
ROUTE KY 11
PAVEMENT TYPE AC

1989
1990
1991
1992

163
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<tr>
<th>LWP</th>
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<tr>
<td>![Diagram Image]</td>
<td>![Diagram Image]</td>
<td>![Diagram Image]</td>
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</table>

**STATE ID #** 18

**County** CLAY

**Mile Point/Station** 15.8 WB

**Route** Daniel Boone Parkway

**Pavement Type** AC

**Core Holes**

|------|------|------|------|------|

**Core Holes**

<table>
<thead>
<tr>
<th>Depth</th>
<th>164</th>
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**STATE ID # 19**

**COUNTY** FLOYD

**MILE POINT/STATION** 6.1 EB

**ROUTE** KY 80

**PAVEMENT TYPE** AC

1989 — ○
1990 — ●
1991 — △
1992 — □

165
<table>
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<th></th>
<th>LWP</th>
<th>RWP</th>
<th>350'</th>
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**STATE ID #: 20**

**COUNTY**: Pike

**MILE POINT/STATION**: 2.5 SB

**ROUTE**: US 119

**PAVEMENT TYPE**: AC

- 1989 - ○
- 1990 - ●
- 1991 GREEN - △
- 1992 - □

**MAP CRACKING AT CENTERLINE**