Overlay Recommendations for I 64: Rowan, Carter, and Boyd Counties

Gary W. Sharpe* Herbert F. Southgate†

*Kentucky Department of Transportation
†Kentucky Department of Transportation

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MEMORANDUM TO:  Henry Bennett
Acting State Highway Engineer
Acting Chairman, Research Committee

SUBJECT:  Research Report 551, "Overlay Recommendations for
I 64: Rowan, Carter, and Boyd Counties;" KYHPR-75-77;
HPR-PL-1(16), Part II; KYHPR-76-79: HPR-PL-1(16),
Part II; KYP-75-68: HPR-PL-1(16), Part III.B.

This report serves to document the assumptions and procedures used in the development of the overlay
thicknesses recommended for I 64 in Rowan, Carter, and Boyd Counties. The recommended overlay
thicknesses were transmitted by memorandum to Mr. A. R. Romine on December 27, 1979 (reproduced in
Appendix A).

Pavement management techniques were employed in the development of the plan and strategies, and the
construction certainly falls within the scope of the "R-R-R" program. Overlays which have come under
"R-R-R" concepts are I 75 in northern Kentucky, the CRCP section of I 71, and I 65 south of Bowling
Green. I 64 in Clark and Montgomery Counties was overlaid using Interstate funding because it was built
before 1964. FHWA participated in the overlay costs to extend the design life to 1984.

I 64 from the Carter-Boyd County line to the West Virginia State line has been subjected to heavy usage
by coal-haul trucks. These trucks were not anticipated in the original design. Whereas the pavement has not
endured the number of years anticipated, it has done its designed duty. It’s service life expired around
1977. Sections in Carter County also have been subjected to coal-haul traffic and are approaching the end
of their fatigue life for the existing subgrade conditions. I 64 certainly falls in the category of heavy
cohal-haul routes along with the Daniel Boone and Mountain Parkways and US 23.

On August 19, 1980, a memorandum report was advanced pertaining to “Shoulders: Function, Design
Criteria, and Strategies, Maintenance; Truck Rests,” which suggested the retro-fitting construction of
truck rest stops at the tops of selected hills. They might be included during the construction of the overlay
of I 64.

Respectfully submitted,

[Signature]
John H. Havens, Director
Division of Research

HFS: gws
c: Research Committee
A method was developed in 1978 for designing asphaltic concrete overlays for flexible pavements. This procedure utilizes Kentucky's theoretical flexible pavement design curves, estimates of pavement fatigue from traffic data, estimates of subgrade strengths, and estimates of the structural capacity of the existing pavement. Overlay designs were recommended for the asphaltic concrete pavement sections of I-64 in Rowan, Carter, and Boyd Counties. Road Rater deflection data were used to estimate the subgrade strength and the effective worth of the pavement structures. Pavement rutting and roughness were also considered in determining the final overlay thicknesses.
Overlay Recommendations for I 64:
Rowan, Carter and Boyd Counties, Kentucky

KYHPR-75-77; HPR-PL-1(16), Part II
KYHPR-76-79; HPR-PL-1(16), Part II
KYP-75-68; HPR-PL-1(16), Part III B

by
Gary W. Sharpe
Senior Research Engineer

and
Herbert F. Southgate
Chief Research Engineer

The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Bureau of Highways nor of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

August 1980
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Introduction

On August 24, 1978, the Division of Research was requested by the State Highway Engineer's office to review the condition of the asphaltic concrete pavement on Interstate 64 in Rowan, Carter, and Boyd counties. Visual observations indicated areas of extensive cracking and excessive rutting. Cores obtained by the Division of Materials indicated that the depths of cracking were deeper than normally expected. The cores were stored by the Division of Maintenance and were available for inspection. Based on these observations the Division of Research was requested to perform additional testing and evaluation.

Road Rater tests and rutting measurements were performed on September 25-27, 1978; Mays Ride Meter tests were conducted on September 9-10, 1979; and initial analyses of Road Rater data were completed early in 1979. Refinements in the procedure for interpreting dynamic deflections indicated it would be prudent to re-evaluate the Road Rater test data and compare the results. These comparisons are discussed briefly in this report.

Evaluations of Road Rater, rutting, and roughness data were used to assign "as-is" or "present-worth" parameters to the pavement as input into an overlay design procedure. Overlay designs were prepared and submitted by memo (December 27, 1979) to the Assistant State Highway Engineer for Operations (APPENDIX A).

Collection and Analysis of Data

Development of a Fatigue History from Traffic Data

Estimates of AADT were determined from AADT maps for each year that data were available. A 50 percent directional split was assumed. Plots of one-direction AADT versus calendar year were developed for each section between interchanges. Lane distribution factors and vehicle classification (style) percentages were obtained.

Pavement fatigue history was expressed in terms of 18-kip equivalent axle loads. Vehicle weight groupings were determined from W-4 Tables. AASHTO damage factors for the various load groups were determined and used with the data to determine the average damage per vehicle for each year from 1959 to 1976. Average damage per vehicle may be expressed by the following relationship:

\[ DF = \frac{\sum (N \times F)}{\text{number of weighed vehicles per classification}} \]

(1)

in which \( DF \) is the average damage per vehicle for classification \( i \); \( N \) is the number of axles weighed in a weight category; \( m \) is the number of weight categories, \( j \), in W-4 Tables; and, \( F \) is the damage factor for the type of paving material, axle configuration and axle load. For this analysis, \( F \) was obtained from the AASHTO Interim Guide for flexible pavements, \( P(t) = 2.5 \) and \( SN = 5.0 \). Equations were fitted to plots of average damage per vehicle versus calendar year for each vehicle classification. These equations were used in developing the fatigue history.

AASHTO damage factors were used instead of Kentucky damage factors because current research using strain energy principles had indicated the Kentucky factors might be too severe, and because of the international acceptance of the AASHTO factors. Strain energy factors appear to be somewhere between the AASHTO factors and the Kentucky factors.

The predominant coal-hauling vehicles are single-unit, three-axle vehicles (SU-3A), and five-axle, combination vehicles (C-5A). Single-unit, four-axle vehicles are also used in transporting coal, but were not included in this analysis due to lack of data. AASHTO factors for a three-axle group
were not available, and strain energy factors were not usable while these analyses were in progress.\textsuperscript{5-6} Also, very little data were available relative to the distribution of SU-4A vehicles in the traffic stream.

It was assumed that five percent of the SU-3A vehicles were more heavily loaded than the average vehicles from the W-4 tables. Energy-based damage factors were used for the loads associated with those trucks. These factors were then adjusted to AASHTO equivalent factors by multiplying by 1.15.\textsuperscript{4} A damage factor (from strain energy) of 14.63 per vehicle was used for the five percent of the SU-3A vehicles which were more heavily loaded. The remaining 95 percent were assigned AASHTO damage factors according to calendar year from the plots of average damage per vehicle versus calendar year. A similar analysis was used for five-axle combination vehicles; the strain energy damage factor for C-5A vehicles was 5.96 per vehicle. As before, this value was adjusted to an AASHTO equivalent.

Equivalent 18-kip axle loads (EAL) were calculated by the following relationship:

\[
EAL = 365 \times \text{AADT} \times \sum (C \times DF \times LD), \quad (2)
\]

in which AADT is one-directional AADT; \(n\) is the maximum number of vehicle classifications; \(C\) is the classification count/total number of vehicles counted or a proportion of vehicles that are of a given style or classification; \(DF\) is the average damage per vehicle per classification; and, \(LD\) is the lane distribution. It was assumed that trucks are operated 365 days per year. Plots of accumulated total EAL were developed for each section of interstate highway between interchanges. These values were used in designing the required overlays. Plots of estimated fatigue history are presented in APPENDIX B.

\textbf{Road Rater Testing and Data Analyses}

Random number tables were used to select test sites.\textsuperscript{7} Five sites per mile were tested. The total number of tests in any given section was determined by multiplying the length of the section in miles by five tests per mile. The sections from which test sites were selected are presented in Table 1. The same test sites were used for both the east- and westbound lanes. Road Rater and rutting measurements were obtained in both wheel tracks in the outside (shoulder) lane. Road Rater data were evaluated using methods developed by the Division of Research.\textsuperscript{8-12}

For this investigation, subgrade moduli were estimated by two procedures. Figure 1 illustrates the theoretical relationships between Road Rater deflections and subgrade moduli. One method uses the No. 2 sensor reading (Figure 1),\textsuperscript{8-11} while an alternate method uses the No. 1 sensor reading and the No. 1 projected deflection to predict subgrade strength. The No. 1 projected deflection, an empirical evaluation of Road Rater deflection data, involves extrapolating a straight line through the magnitudes of the deflections of the No. 2 and No. 3 sensors when log deflection is plotted versus distance from the load head. Extrapolation of the line to the position corresponding to the No. 1 sensor results in the No. 1 projected deflection:

\[
\text{No. 1 projection} = \exp (2 \log \text{No. 2 deflection} - \log \text{No. 3 deflection}). \quad (3)
\]

\textbf{Table 1. Randomly Selected Test Sections.}

<table>
<thead>
<tr>
<th>Milepoint to Milepoint</th>
<th>Description</th>
<th>Length (Miles)</th>
<th>Number of Sites</th>
</tr>
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<tbody>
<tr>
<td>146.2</td>
<td>Begin AC to Rowan-Carter County line</td>
<td>2.465</td>
<td>12</td>
</tr>
<tr>
<td>148.665</td>
<td>Rowan-Carter County line to KY 2 interchange</td>
<td>7.600</td>
<td>38</td>
</tr>
<tr>
<td>156.625</td>
<td>KY 2 interchange to US 60 interchange</td>
<td>5.187</td>
<td>26</td>
</tr>
<tr>
<td>161.452</td>
<td>US 60 interchange to KY 1 &amp; 7 interchange</td>
<td>10.156</td>
<td>51</td>
</tr>
<tr>
<td>180.812</td>
<td>Carter-Boyd County line to US 60 interchange</td>
<td>0.557</td>
<td>3</td>
</tr>
<tr>
<td>181.369</td>
<td>US 60 interchange to Kentucky 180 interchange</td>
<td>4.100</td>
<td>21</td>
</tr>
<tr>
<td>185.469</td>
<td>KY 180 interchange to US 23 interchange</td>
<td>5.255</td>
<td>26</td>
</tr>
<tr>
<td>190.724</td>
<td>US 23 interchange to West Virginia state line</td>
<td>0.783</td>
<td>4</td>
</tr>
</tbody>
</table>
The slope of the semilog line (secant line), the difference in magnitude between the No. 1 projected deflection and the No. 1 deflection, and the magnitudes of all deflections are indicative of the shape of the deflection bowl.

For a given pavement structure, asphaltic concrete modulus, and subgrade modulus, there is a difference between the No. 1 projected deflection and the No. 1 deflection for theoretical deflections. There will also be a difference between these values for field-measured deflections. Normally, the differences between the No. 1 projected deflection and the No. 1 deflection for both theoretical and field measurements are similar. Slab deterioration may be suggested when field measurements indicated a No. 1 deflection greater than the No. 1 projected deflection, and the difference is greater than the difference for theoretical deflections. A foundation problem, or lack of supporting capability, may be indicated by increased magnitudes of all field deflections and the difference between the No. 1 projection and the No. 1 deflection greater than normally expected for the magnitudes of the measured deflections.

A plot of No. 1 projected deflections versus No. 1 deflections in log-log form may be used to identify variations in the pavement structure (Figure 2). The solid lines (left side of Figure 2) show the theoretical relationships of No. 1 projected deflections and No. 1 deflections for a constant structure and asphaltic concrete modulus. Subgrade modulus varies along the line. The variation in position of the theoretical line due to changes in the deflections by ± one unit (2.54 x 10^{-4} mm or 1.0 x 10^{-5} inches) on the Road Rater meters and the associated change in calculated No. 1 projected deflection is indicated by two dashed lines. The zone inside these lines represents a normal variation due to reading the meters of the Road Rater.

The solid line on the right side of Figure 2 represents the theoretical relationship between Road Rater No. 1 deflections and subgrade modulus. Two different points are shown in Figure 2. The “x” points represent data points which would be suspected of having problems in the bound layers (from the No. 1 projected deflection versus No. 1 deflection relationship). The “o” points represent points suspected of having foundation or supporting layer problems.

The “x” points (Figure 2) have a No. 1 deflection higher than would be theoretically expected for the given values of the No. 2 and No. 3 sensors and the corresponding No. 1 projected deflections. This might be visualized as the pavement folding about the point of application of the load. Since the No. 1 deflection is higher than would be theoretically expected, it is necessary to adjust the No. 1 deflection to a theoretical value which matches the measured No. 2 and No. 3 deflections. The adjusted No. 1 deflection is now used to predict subgrade modulus. The predicted subgrade modulus is plotted versus the measured No. 1 deflection and compared to the theoretical relationship of Road Rater No. 1 sensor deflection versus subgrade moduli. The point will plot above the theoretical line, indicating behavior weaker than the reference conditions. The behavior may be expressed in terms of reduced asphaltic concrete modulus or a reduced thickness of asphaltic concrete at reference conditions. In terms of overlay design, effective behavior expressed as a reduced thickness is more meaningful.

The “o” points (Figure 2) have a No. 1 deflection lower than would be theoretically expected for the measured values of the No. 2 and No. 3 deflections and associated No. 1 projected deflection. In this situation, the deflection bowl is very “broad” and “flat” and representative of a problem in the foundation or supporting layers. The theoretical relationship of No. 1 projected deflection versus No. 1 deflection is used in combination
with the No. 1 projected deflection based on the measured No. 2 and No. 3 deflections to determine an adjusted No. 1 deflection. The adjusted No. 1 deflection will have a greater magnitude than the measured No. 1 deflection and will be compatible with the measured No. 2 and No. 3 deflections and associated No. 1 projected deflection. When the predicted subgrade strength (E₃) (from the No. 1 sensor deflection) is plotted versus the adjusted No. 1 deflection, the expression of pavement behavior is in terms of a predicted subgrade strength and a reduced thickness of reference quality materials.

A statistical analysis of all pairs of predictions for subgrade moduli indicated the two procedures for predicting subgrade strength were very closely related. The procedure using the combination of No. 1 deflections and the No. 1 projected deflections consistently predicted a subgrade modulus which was 82 percent of the modulus predicted when using the No. 2 deflections. Figure 3 illustrates this relationship.

The effective behavior of a pavement may be expressed in terms of a predicted subgrade modulus and an effective thickness (any combination of asphaltic concrete and dense-graded aggregate) which match the measured deflection behavior. The effective thickness may also be expressed as an effective full-depth thickness of asphaltic concrete, but this is not always meaningful. There also may be some combinations of thicknesses which are not reasonable representations of pavement behavior. For this analysis, the effective thickness is determined by assuming the thickness of dense-graded aggregate equal to the "design" or constructed thickness and determining the thickness of "reference" asphaltic concrete having a theoretical deflection bowl which matches the measured Road Rater responses. Reference conditions for Road Rater testing are as follows:

Figure 2. Illustration of procedures to estimate subgrade strength and the effective pavement structure.
Figure 3. Correlation of two procedures for estimation of subgrade strength.

1. Test frequency = 25 Hz,
2. Reference temperature = 70 degrees F (21.1 degrees C) mean pavement temperature,
3. Reference asphaltic concrete modulus = 1,200 ksi (8.27 GPa) at 70 degrees F (21.1 degrees C) and 25 Hz, and
4. Reference asphaltic concrete modulus = 480 ksi (3.31 GPa) at 70 degrees F (21.1 degrees C) and static conditions (Benkelman beam testing = 0.5 to 1.0 Hz).

At present, the concept of using the constructed thickness of dense-graded aggregate as a constant and expressing pavement condition in terms of an effective (behavioral) thickness of asphaltic concrete of a quality equal to reference conditions seems to be most appropriate. APPENDIX C contains examples of the three basic variations in Road Rater behavior. Photographs corresponding to each type of behavior are also presented.

Strip charts of estimated subgrade modulus versus milepoint and effective thickness versus milepoint are presented in APPENDIX D. The evaluation sheets and field data are on file at the Division of Research. These strip charts were used in the selection of design parameters.

Investigation of Rutting Within the Cross Section

Severe rutting had been noted on the eastbound, outside lane of the upgrade east of the Cannonsburg interchange in Boyd County. A trench was dug across the outside lane; measurements of cross-section layers were made; and photographs were taken. The memorandum reporting this investigation is included as a part of APPENDIX A, but only selected photographs, which were attached to the original memorandum, are reproduced herein.

Analysis of Rutting Data

Rut measurements were obtained by stretching a stringline across the pavement. The maximum rut in each wheel track was measured using a ruler or scale. The rut measurements were plotted on a strip chart of rut depth versus milepoint (APPENDIX D). This strip chart was used in developing the overlay designs and is a good indicator for estimating leveling course requirements.

A literature review indicated possible relationships between rutting and present serviceability index (PSI). One such relationship suggests that PSI and rut depth are inversely proportional. A pavement with a rut depth of 0.7 inch (17.8 mm) would have a PSI of approximately 2.5, while a pavement with a rut depth of 0.5 inch (12.7 mm) would have a PSI of 3.0 (Figure 4).

Figure 4. Relationship between measured rut depth and present serviceability index (PSI) developed by Lister and Addis.
An additional analysis was based on AASHO Road Test data. Photostatic copies of the microfilm data sheets were obtained. Measured rut depth and corresponding PSI values were taken, as well as initial PSI values. Measured depth was plotted versus PSI, and a line was fitted to the data (Figure 5). This relationship was similar to that developed by Lister and Addis. Differences are probably due to interpolation from the data sheets. Change in PSI expressed as a percentage \(\frac{\text{Original PSI} - \text{PSI at a given time}}{\text{Original PSI}} \times 100\) is plotted versus the measured depth in Figure 6. A second degree polynomial appears to be the best fit.

### Estimation of PSI from Mays Ride Meter Data

The Mays Ride Meter was used to survey sections of I-64 from milepoint 146.2 to milepoint 191.6 at the West Virginia line on September 9-10, 1979. Both eastbound and westbound, outside (shoulder) lanes were surveyed. Mays Ride Meter roughness values were determined for 1/20-mile increments. Mays roughness has been correlated with automobile roughness index (RI) for asphaltic concrete by the following equation:

\[
\text{AUTORI} = 4.22 \times \text{(Mays roughness)} + 78. 
\]  

Auto RI may be correlated with present serviceability index (PSI) by the following relationship:

\[
\text{PSI} = 4.65 - 0.003 \times \text{(Auto RI)}. 
\]  

Computed values for PSI corresponding to the same locations for Road Rater tests and rutting measurements were plotted versus milepoint to make strip charts similar to those developed for subgrade moduli, effective thickness, and measured rut depth (APPENDIX D).

### Seasonal Effects on Predicted Subgrade Modulus

In-place CBR or subgrade moduli are directly related to moisture in the subgrade. If the soil is saturated, or nearly so, its behavior will be similar to that of the soaked laboratory CBR. Normally a soaked or saturated condition is associated with the spring season. When new pavements have been tested with the Road Rater, spring tests have tended to indicate abnormal behavior while fall tests have indicated results more consistent with behavior normally expected from new pavements and match elastic theory. For these reasons, most Road Rater tests have been conducted in the summer and fall months. Abnormal behavior can be more readily defined when evaluating test data obtained during these seasons.

While summer and fall Road Rater tests have been best for pavement evaluation, design param-
eters should be based on a "weaker" or "weakest" condition. Therefore, it was necessary to make adjustments from a "fall" condition to a "spring" condition (expected to be the "weaker" or "weakest" condition).

Two three-layer pavement sections on US 60 in Boyd County were used to investigate seasonal effects. These sections consisted of 6.5 inches (165.1 mm) of asphaltic concrete on 12 inches (304.8 mm) of dense-graded aggregate and 6.8 inches (172.7 mm) of asphaltic concrete on 19 inches (482.6 mm) dense-graded aggregate. The first year after construction for which a complete set of data (spring, summer, and fall) was available was 1973. Subgrade moduli were predicted for each season. Data were available for April, May, June, and September. The September value was selected as the reference condition. Ratios of predicted subgrade moduli for other times to the September moduli were computed.

The April estimates of subgrade moduli were 67 percent of the September estimates (Figure 7). The relationship was extrapolated to cover a period of one year. Figure 7 indicates a factor of 0.6 to adjust from the strongest to the weakest condition, but Figure 7 is based on limited data. Additional research is needed to clearly define the relationships, even though they appear consistent with other research. In Pennsylvania, an adjustment factor of 0.5 was suggested. There the adjustment factor was applied directly to the measured deflections. When these adjustments were used to adjust Kentucky Road Rater deflections and the adjusted deflections were used to predict subgrade moduli, March moduli were approximately 50 percent of the November moduli.

![Figure 7](image)

Figure 7. Seasonal variation in predicted subgrade modulus from Road Rater deflections.
Comparison of Laboratory CBR Data and Predictions of CBR Using Road Rater Deflections

Laboratory CBR data were obtained from two sources. Copies of the soils laboratory reports (Division of Materials) were obtained from the Division of Design. Microfilm copies of the soil profile sheets from construction plans and copies of consultants’ reports were also reviewed. Soil samples were taken from the completed subgrade prior to paving. The stationing associated with the CBR data was converted to equivalent milepoints, making it possible to relate these values to those predicted by the Road Rater.

Road Rater estimates of subgrade strength are expressed in terms of moduli of elasticity. A literature review indicated that subgrade moduli of elasticity may be converted to an approximate CBR by dividing the subgrade modulus (in psi) by 1500. Experience has indicated this estimate is reasonably adequate for CBR values up to 20. Plots of CBR predicted from Road Rater data versus laboratory CBR were developed.

Laboratory CBR values represent the worst expected condition because the sample is soaked to saturation before testing. Predictions of subgrade strength from Road Rater data represent an “in-place” condition. Therefore, the Road Rater estimates represent a “fall” or dry condition inasmuch as the data were collected in early September 1978. If Road Rater estimates of CBR are plotted versus laboratory CBR corresponding to the same location, it would be expected that the CBR values from the Road Rater deflections would be greater than the corresponding laboratory CBR values. A plot of CBR predicted from fall Road Rater deflections versus laboratory CBR was developed to verify this expectation (Figure 8). Approximately 60 percent of the Road Rater estimates were greater than their laboratory counterparts. Inspection of the remaining data indicated that all but three of the outlying points were confined to one specific section of I 64.

In that section of I 64, the initial laboratory CBR values were much higher than in any of the others. However, Road Rater estimates of CBR showed little variation from section to section. Also, Road Rater estimates of CBR were considerably lower than the corresponding laboratory values. Laboratory CBR’s indicated the subgrade was of exceptionally high quality material when the section was built. However, 16 years later, the subgrade was behaving as a considerably weaker material and very similar to areas on either side of the section (APPENDIX D).

Shales are commonly used for road construction in this area. The question of the possible deterioration of shaly materials in the subgrade was considered. The following statements about the general geology of the area may provide a possible explanation for the measured behavior.

Laboratory CBR data obtained just prior to the paving of I 64 (1963 to 1964) indicated the following general bearing-capacity conditions: From the Carter-Boyd County line to milepost 182, CBR values were generally low (2 to 7). For the interval from milepost 182.5 to milepost 185, no CBR data were available. From milepost 185 to 189, CBR values were high (7 to 27). From milepost 185 to the Kentucky-West Virginia line, CBR values were low (2 to 7).

Road Rater estimates of the bearing capacities of these materials obtained recently
(September 1978) do not show these same trends. Bearing-capacity estimates for the entire Boyd County interval were relatively uniform (CBR 2 to 10), indicating the interval from milepost 185 to 189, which initially had high CBR values, no longer had superior bearing-capacity characteristics.

Bearing-capacity trends obtained from the initial laboratory CBR data show a good correlation with the local geology as presented in Figure 9; the area of higher CBR values corresponds to the area where the bedrock is the Monongahela and Conemaugh Formations. However, geologic columns prepared by the U.S. Geological Survey (Rush, Boltsfork, and Ashland Quadrangles) indicate there is a higher percentage of shaly materials in these formations than in the underlying Breathitt Formation. Boyd County data in the Kentucky Soils Data System were also checked to see what percentage of fine-grained materials were present in soils derived from these bedrock parent materials.\textsuperscript{24, 25}

Data contained two samples of soil thought to be derived from the Breathitt Formation, one from the Conemaugh, and two from the Monongahela. The particle-size analysis for these samples is shown in Table 2.

A greater percentage of fine-grained materials in the Monongahela-Conemaugh

![Figure 9. Sketch of geologic conditions for I 64 in Boyd County.](image)

<table>
<thead>
<tr>
<th>Sample</th>
<th>Parent Material</th>
<th>Percent by Volume</th>
<th>Percent by Volume</th>
<th>Percent by Volume</th>
<th>Percent by Volume Smaller Than</th>
</tr>
</thead>
<tbody>
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Table 2. Particle Size Gradations for Parent Materials in the Boyd County Region.
than in the Breathitt is indicated from both the geologic columns and the laboratory tests on soils derived from these bedrock materials. However, because a high percentage of shaly (fine-grained) materials is generally detrimental to slope stability or bearing capacity of an earth embankment, these data seem to conflict with the original CBR data.

In this area, the Breathitt Formation outcrops primarily in stream valleys, while the overlying Conemaugh is the bedrock of the adjacent upland areas. Interstate 64 follows streams (Big Run Creek west of KY 180, and Chadwick Creek east of milepost 187) in areas where the Breathitt Formation is the bedrock. Where I 64 crosses the intervening upland, the Monongahela-Conemaugh is the bedrock and is primarily exposed in cut sections. One possible explanation which takes into consideration both the original CBR data and the current Road Rater CBR estimates assumes that weathered materials from the upland cut sections were used as fill (embankment) material through most of the areas where the roadway follows the stream valleys. In the cut areas, excavation into the unweathered shales in the Conemaugh resulted in CBR values higher than in the adjacent fill areas where the more weathered Conemaugh materials had been placed. The current Road Rater CBR or bearing capacity determinations yielded more uniform values due to subsequent weathering of the Conemaugh shales in the cut areas.

A visual inspection of project sections indicated there were areas from milepost 185 to 189 where ditch lines had been clogged by rock slides in the cut areas (Figure 10). Accumulation of water in the subgrade could have accelerated the deterioration of the shaly materials. Clogged ditches were noted during the period of Road-Rater testing and also during subsequent trips through the area. No photographs of this condition were obtained until May 29, 1980, at which time it was noted that trapped water was deep enough and had been there long enough for colonies of tadpoles and frogs to be established. Visual inspection of the pavement areas adjacent to the clogged ditches indicated areas of excessive pavement distress (Figure 10). Clogged ditches could be the initial phase of problems in the foundation and supporting layers (APPENDIX C).
Selection of Design Parameters

The three primary variables that must be considered in the design of overlayment are "Design CBR," the effective or "behavioral" thickness (or worth of the existing pavement) and the "Design EAL," as determined from traffic volumes and vehicle classification distributions. Strip charts were developed (by plotting estimated subgrade modulus and effective thickness versus milepoint (APPENDIX D)) to locate natural breaks in the behavior of pavement sections. The natural breaks were merged with the breaks associated with intersections and associated changes in traffic volumes. This resulted in several "design" sections. Statistical analyses were used to evaluate the data within a given design section.

Within a given design section, a design CBR, a design effective thickness, a maximum rut depth, and a minimum present serviceability index (PSI) based on pavement roughness were selected. The design CBR was selected by calculating the mean CBR and then subtracting 1.5 times the standard deviation from the mean. This value was then multiplied by 0.6 to convert to a soaked or "spring" condition. The effective thickness was calculated by determining the mean effective thickness and subtracting 1.5 times the standard deviation. The maximum rut depth was estimated by adding 1.5 times the standard deviation of rut depth to the mean rut depth in each section. The minimum PSI was estimated by subtracting 1.5 times the standard deviation of PSI values in the section from the mean PSI of the section. The addition or subtraction of 1.5 times the standard deviation corresponds to the selection of an 87th percentile value. The multiplier 1.5 was arbitrarily selected and is based on engineering judgment.

Design EAL values were determined for four different traffic levels as follows: the year construction was completed plus 20 years, 1985, 1990, and 1996. Overlay designs were determined for each of these design levels.

Procedure for Determining Overlay Design

Road Rater data were used to determine the "effective" or "behavioral" worth of the pavements. The "effective thickness," determined from Road Rater data, was used as input into the overlay design procedure. In Figure 11, Curve "A" was created using the effective thickness of the dense-graded aggregate (unbound crushed-stone base) as the basic thickness. In this analysis, the effective thickness of dense-graded aggregate is assumed equal to the constructed thickness. The total thickness for various percentages of thickness of asphaltic concrete to the total thickness was determined from the following equation:

\[
\text{Total thickness} = \frac{(100 \times \text{DGA})}{(100 - (\text{AC}/\text{Total}) \times 6 \times 100)}
\]

where AC is the design thickness of asphaltic concrete; and, DGA is the effective thickness of dense-graded aggregate.

Road Rater data were used to determine the CBR value to be used as an input into the overlay design. The weakest in-place subgrade modulus for a design section was used. Statistical procedures discussed above were used to estimate the expected weakest condition.

With the selected design EAL and design CBR, charts in APPENDIX E were used to deter-
mine design thicknesses. These thicknesses were plotted versus percentage asphaltic concrete in the total thickness, as illustrated by Curve "B" of Figure 11. APPENDIX E also may be used to determine the design thicknesses for a pavement using new material.\textsuperscript{10-12, 21,27,28}

The total pavement thickness is determined by the intersection of Curves A and B in Figure 11. The overlay thickness is the difference between the total design thickness (existing pavement and overlay) and the effective thickness of the existing pavement and is determined from the following relationship:

\[
\text{Overlay thickness} = \text{Total Thickness} - \text{Total Equivalent Thickness},
\]

Total equivalent thickness is determined from either Road Rater or Dynaflect data, or by other means of estimating the effective worth of a partially deteriorated pavement.\textsuperscript{8-12, 27-29} Design calculations are on file at the Division of Research.

Table 3 summarizes the parameters used in the process and the overlays recommended; they were included in a memo (December 27, 1979) to A. R. Romine, Assistant State Highway Engineer for Operations, from J. H. Havens, Director of Research (APPENDIX A). Additional thicknesses were added when rutting was greater than the statistically expected maximum for that section. The patch thicknesses added were equal to the difference between the measured rut depth and the statistically expected maximum rut depth for that section. The expected, maximum rut depths are recorded in Table 3 and may be used to estimate required leveling. The procedure for estimating additional thicknesses due to excessive rutting was arbitrary and represents engineering judgment.

Patch thicknesses were also designed for those areas where the predicted subgrade strength was weaker than the selected "design" value. The Kentucky Design Curves (APPENDIX E) were used to determine the thicknesses of the patches using the same procedures used in designing the overlay thicknesses.\textsuperscript{1,10,11,21} Patch thickness is equal to the difference between the total thickness required for the "weaker" areas and the total thickness required for the selected design CBR. The locations and recommended patch thicknesses are presented in Table 4.

Design thicknesses from this analysis were compared to overlay designs determined by a separate analysis (1980 RRR) for the same areas and the same design year.\textsuperscript{28} Inasmuch as the design sections were different for the two design procedures, thicknesses were combined using a weighting procedure based on section lengths in miles and the thicknesses of the overlay for one section. When the weighted averages were compared, differences between the two design procedures varied from 0.5 inch (12.7 mm) to 1.0 inch (25.4 mm), depending on how the sections were combined and what structural significance was applied to the recommended 0.75-inch (19.1 mm) open-graded surface used in the 1980 RRR procedures.\textsuperscript{27,28}

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Note: If there is no value in the "To" column, the patch length is assumed to extend 100 feet either side of the milepoint.
Correlation of Predicted Behavior and Field Data

Attempts were made to empirically correlate the information predicted from the Road Rater deflection measurements with other pavement measurements, such as rutting and roughness index, and the associated estimates of present serviceability index (PSI). In general, the attempts were not successful. However, the data did indicate the existence of possible trends, but there was not sufficient data or control in testing procedures to verify the trends.

One set of comparisons did seem to indicate that, as the predicted subgrade strength decreased, the measured rut depth increased. However, there was considerable scatter and more research is needed to evaluate the relationship. Estimates of PSI from rutting measurements were compared to estimates of PSI from roughness measurements. Again, while there were indications of trends, there was too much scatter, and more study is needed.

Summary

The criteria and logic used in determining the overlay thicknesses for the asphaltic concrete pavement sections of Interstate 64 in Rowan, Carter, and Boyd Counties are presented in this report. Road Rater data, roughness data, and rutting data were considered. Road Rater deflections were used to predict subgrade strength and the current load-carrying capability of the existing pavements. Predictions of subgrade strength were compared to laboratory CBR values. When the predicted CBR values (from Road Rater deflections) were adjusted for seasonal effects, the predicted CBR's were (in most cases) within the range of laboratory CBR's. Laboratory CBR values calculated prior to paving were obtained from the Division of Materials. Additional CBR information was obtained from microfilm copies of the soil profile sheets on construction plans. It was noted in some situations that the CBR used for pavement design was greater than the laboratory CBR values obtained prior to paving. If the subgrade strength prior to paving was less than the subgrade strength used for design, some degree of premature failure might be expected. It is possible the design CBR had to be selected years before results of extensive laboratory soil testing were available.

Pavement sections from milepoint 146.2 to milepoint 171.6 were constructed with 12.0 inches (304.8 mm) dense-graded aggregate and 7.5 inches (190.5 mm) asphaltic concrete (6.5 inches (165.1 mm) base and 1 inch (25.4 mm) surface). The design CBR was 9. These conditions may be used in combination with the 1959 Kentucky Flexible Pavement Design Curves to estimate the design equivalent 5-kip wheel loads (EWL). The 1973 Kentucky Flexible Pavement Design Curves use EAL values (equivalent 18-kip axle load) as the expression of traffic loading; therefore, EWL values from the 1959 procedure, which were used in the initial design, were converted to EAL's used in the overlay designs. The design EAL's for milepoint 146.2 to milepoint 171.6 were 8.0 million.

The pavement from milepoint 180.8 to 191.5 was constructed with 14.0 inches (355.6 mm) dense-graded aggregate and 7.5 inches (190.5 mm) asphaltic concrete, 6.5 inches (165.1 mm) base and 1 inch (25.4 mm) surface on a subgrade of design CBR 5. The design EAL's associated with these conditions were 3.5 million.

Visible signs of pavement distress were noticed in all sections; however, sections in Boyd County showed more distress. Normally, pavement distress is a result of fatigue associated with the actual number of passes of an equivalent axle load (EAL). This appeared to be the situation for the Boyd County sections, as indicated by the estimates of fatigue history in APPENDIX B. The design EAL's for the section from the Boyd-Carter County line (milepoint 180.812) to the KY 180 interchange (milepoint 185.465) will be reached in 1981. The design EAL's from the KY 180 interchange (milepoint 185.465) to the US 23 interchange (milepoint 190.724) and from the US 23 interchange (milepoint 190.724) to the West Virginia line (milepoint 191.507) were reached in 1979 and 1977, respectively. On the other hand, the design EAL's for the sections in Rowan and
Carter Counties (MP 146.2 to MP 171.607) will not be reached until the early 1990's.

Road Rater deflections, rutting, and roughness generally seemed to indicate the Boyd County sections were in greater distress than the Rowan and Carter County sections. However, pavement behavior (from deflections, roughness, and rutting measurements) of the Rowan and Carter County sections did indicate a distressed condition and warrant an overlay although the initial design EAL will not be reached until the 1990's. Road Rater estimates of "spring" CBR values were in the range of 2 to 4 and were less than the initial design CBR of 9. Also, limited laboratory CBR data indicated the subgrade prior to paving was not of CBR 9 quality, which could account for signs of premature pavement distress. Yet, Road Rater estimates of "spring" CBR for the Boyd County sections were in the range of 2 to 4 and were similar to the laboratory CBR values and were closer to design CBR 5.

**Acknowledgements**

Special appreciation is expressed to Mr. Donald C. Newberry, Jr., for his efforts in the collection of the rutting measurements, to Mr. James L. Burchett, who developed the method of roughness data analysis, to his staff who collected the roughness data, and to Mr. William J. Pfalzer for his treatment of the geologic analysis.

**References**


12. Sharpe, G. W.; Southgate, H. F.; and Deen, R. C.; "Interpretations of Dynamic Pavement Deflections," Division of Research, Bureau of Highways, Kentucky Department of Transportation, to be issued.


17. Burchett, J. L.; and Rizenbergs, R. L., "Road Roughness Surveys in Kentucky," Division of Research, Bureau of Highways, Kentucky Department of Transportation, to be issued.


27. Southgate, H. F.; Newberry, D. C.; Deen, R. C.; and Havens, J. H.; "Resurfacing, Restoration and Rehabilitation of Interstate Highways: Criteria and Logic Used to Determine January 3, 1977, Needs and Estimates of Costs," Division of Research, Bureau of High-
ways, Kentucky Department of Transportation, Report 475, July 1977.


30. Drake, W. B.; and Havens, J. H.; "Kentucky Flexible Pavement Design Studies," The Engineering Experiment Station, College of Engineering, University of Kentucky, Bulletin No. 52, June 1959.
Appendix A.

Memorandums Relating to Overlay Design
Thicknesses for I 64 in Rowan, Carter and Boyd Counties, Kentucky
MEMO TO: A. R. Romine  
Asst. State Highway Engineer  
for Operations

FROM: Jas. H. Havens  
Division of Research

SUBJECT: I 64, Rowan, Carter, and Boyd Counties

RE: Mr. Romine's Memo; August 24, 1978

Attached is Mr. Southgate's memo report summarizing an extensive evaluation of the condition of pavements on I 64 in the named counties. Structural overlay requirements have been determined. Southgate is now preparing a more documented report.

We have observed rutting since the onset of heavy coal hauling on sections of the road. I believe the surface cracking preceded extensive hauling. I believe the cracking there -- as was observed in Clark and Montgomery Counties -- is not service related. That is to say: Surface cracking there and here was induced by the roller compacting the AC at the time of construction. Weathering and erosion have magnified them. Longitudinal cracking at the edges of the rutted wheel paths were load-induced and are service related. Heavy loads, together with structural inadequacies (from the beginning), have compounded the problem.

A cross section at MP 186.3 was exposed by trenching, November 20, 1978.

JHH/mm

Attachments
MEMORANDUM

TO:  Jason H. Havens, Director
      Division of Research

FROM:  Herbert F. Southgate
        Chief Research Engineer
        Division of Research

SUBJECT:  Pavement condition evaluation and overlay designs for I 64 in Rowan, Carter, and Boyd Counties.

Mr. Romine requested the Division of Research to evaluate I 64 from where the pavement changes from PCC to AC at MP 146.2 to the Grayson interchange at MP 171.6, and from the Boyd–Carter county line at MP 180.8 to the West Virginia state line at MP 191.6. Rutting measurements and Road Rater tests were made on September 25–27, 1978, on the basis of five tests per mile using random numbers from a 'random number table'. Mays Ride Meter tests were made on September 9–10, 1979.

The original design for the section of highway from MP 146.2 to MP 171.6 assumed a CBR of 9 and 8 million EAL's. The 1959 Kentucky curve cross matches these assumptions and requires a total pavement thickness of 19.5 inches. Thus the design was chosen to consist of 7.5 inches of AC and 12 inches of DGA. For the section of highway from MP 180.8 to MP 191.6, the original design assumed a CBR of 5 and 3.5 million EAL's. The 1959 Kentucky design curves require a total thickness of 21.5 inches and consists of 7.5 inches of AC and 14 inches of DGA.

The Road Rater data were adjusted for temperature variations, and the effective AC thickness, subgrade moduli, and condition of the general structure were determined for each test. Tests were noted that displayed either an abnormal AC or foundation problem. Plots were made of effective AC thickness versus milepoint and subgrade moduli versus milepoint for each direction. With these two plots and traffic changes due to interchanges, section lengths were determined and are shown in Tables A1 and A2.
### Table A1. Recommended Overlay Designs.

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<td>12.00</td>
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<td>9.00</td>
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<td>9.00</td>
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<td>Design EAL (million)</td>
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<td>8.00</td>
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<td>8.00</td>
<td>8.00</td>
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<td>EAL for &quot;Plus 20 Years&quot; (million)</td>
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<td>5.0</td>
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<td>5.0</td>
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#### 1970 Road Rate Evaluation

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<td>Mean Ride (Maximum)</td>
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#### Overlay Designs

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<td>4.00</td>
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<td>10.40</td>
<td>10.40</td>
<td>10.40</td>
<td>10.40</td>
<td>10.40</td>
</tr>
<tr>
<td>Less Original AC Thickness</td>
<td>7.50</td>
<td>7.50</td>
<td>7.50</td>
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<tr>
<td>AC Deficiency in Original Design</td>
<td>2.90</td>
<td>2.90</td>
<td>2.90</td>
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<td>2.90</td>
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<tr>
<td>Prevented Serviceability Index</td>
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<td>0.00</td>
<td>0.00</td>
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<tr>
<td>Mean Ride (Maximum)</td>
<td>2.50</td>
<td>2.50</td>
<td>2.50</td>
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<tr>
<td>Measured Pothole Depth</td>
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<td>0.45</td>
<td>0.45</td>
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Table A2. Location and Recommended Thickness for Construction of Structural Patches on I 64 in Rowan, Carter and Boyd Counties.

<table>
<thead>
<tr>
<th>Eastbound Milepoint</th>
<th>Thickness</th>
<th>Westbound Milepoint</th>
<th>Thickness</th>
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<tr>
<td>From</td>
<td>To</td>
<td>From</td>
<td>To</td>
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<tr>
<td>147.8</td>
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<td>153.5</td>
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<td>1.25</td>
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<tr>
<td>155.3</td>
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<td>148.1</td>
<td>148.3</td>
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<td>157.0</td>
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<td>149.5</td>
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<td>157.4</td>
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<td>151.0</td>
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<td>158.8</td>
<td>2.50</td>
<td>157.0</td>
<td>157.5</td>
</tr>
<tr>
<td>159.2</td>
<td>1.00</td>
<td>159.4</td>
<td>2.25</td>
</tr>
<tr>
<td>159.5</td>
<td>1.50</td>
<td>161.3</td>
<td>0.50</td>
</tr>
<tr>
<td>164.0</td>
<td>0.50</td>
<td>162.1</td>
<td>162.3</td>
</tr>
<tr>
<td>170.6</td>
<td>0.50</td>
<td>165.6</td>
<td>0.50</td>
</tr>
<tr>
<td>182.3</td>
<td>1.50</td>
<td>169.5</td>
<td>2.00</td>
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<td>184.0</td>
<td>1.25</td>
<td>182.7</td>
<td>183.1</td>
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<td>184.9</td>
<td>1.00</td>
<td>186.3</td>
<td>0.50</td>
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<td>185.2</td>
<td>2.00</td>
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<td>187.9</td>
<td>1.25</td>
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<td></td>
</tr>
<tr>
<td>188.9</td>
<td>1.25</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: If there is no value in the 'To' column, the patch length is assumed to extend 100 feet either side of the milepoint.

Noting that the Road Rater tests were made in the fall and that the laboratory CBR test value yields the weakest condition which should correspond to spring conditions, a satellite study was made using data from the two three-layer sections of US 60 research pavements in Boyd County. Five test series were made in 1973 which covered the period from early April to mid October. Analyses indicated that the early spring moduli were 0.6 times that of the fall moduli. A literature review revealed that the Pennsylvania Department of Transportation had determined that 0.5 was an appropriate value for their conditions. Thus, 0.6 was not only reasonable, but on the conservative side.

The divisions of Materials and Design had copies of the laboratory CBR tests for samples taken from the subgrade at the time of construction in Boyd County and for about 0.3 miles in Carter County. The Division of Design had reports from consulting engineering firms giving laboratory CBR values of samples taken during location studies. The microfilms of the final plans were searched for CBR data. The result showed that the laboratory CBR data versus milepoint had far more variation in values than did the Road Rater estimates when adjusted by 0.6. Furthermore, the Road Rater data were within the limits of the laboratory CBR data.
To determine what value should be chosen for design, statistical analyses were made for the predetermined section lengths for the Road Rater spring CBR, effective AC thicknesses as determined from the Road Rater evaluations, rutting measurements, and pavement serviceability index values as determined from the Mays Ride Meter tests. The standard deviations were determined and multiplied by 1.5 and subtracted from the mean for all these tests, except for rutting. For rutting, the value corresponding to 1.5 times the standard deviation was added to the mean to yield the higher rutting conditions. The resulting value was checked against the minimum or maximum measured value for reasonableness.

Inspection of Tables A1 and A2 and Figures A1 and A2 show that the estimated CBR for most of the sections were considerably less than the original design CBR. Figure A3 illustrates how a minor change in the design CBR affects the design EAL. The mean Road Rater spring CBR for the section of highway from MP 146.2 to MP 171.6 was approximately 4.4, and from MP 180.8 to MP 191.6 was 3.3, as compared to the original design CBR's of 9 and 5, respectively. The "as-built" pavement structure placed on the existing subgrade had the net result of reducing the expected EAL life. Figure A3 displays the relationship of CBR versus EAL, and reveals that the reduction in CBR from a value of 9 to 4.4 would reduce the expected EAL's from 8 million to 1 million for the section from MP 146.2 to MP 171.6.

Figure A1.
### Figure A2.

<table>
<thead>
<tr>
<th>MILEPOINT</th>
<th>146.2</th>
<th>150.9</th>
<th>156.5</th>
<th>159.1</th>
<th>162.1</th>
<th>165.1</th>
<th>172.7</th>
<th>175.6</th>
<th>178.2</th>
<th>181.2</th>
<th>185.9</th>
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<th>191.6</th>
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</thead>
<tbody>
<tr>
<td>MILE POINT</td>
<td>PCC</td>
<td>ROWAN, CARTER</td>
<td>KY-2 INTERCHANGE</td>
<td>OLIVE MILL WEST</td>
<td>US 60 INTERCHANGE</td>
<td>OLIVE MILL EAST</td>
<td>KY-187 INTERCHANGE</td>
<td>GRAYSON</td>
<td>CARTER-SOUTH COVE</td>
<td>US 60 INTERCHANGE</td>
<td>KY RD INTERCHANGE</td>
<td>US 277 INTERCHANGE</td>
<td>W. VAL. STATE LINE</td>
</tr>
</tbody>
</table>

### Figure A3.

![164, ROWAN, CARTER, BOYD COUNTIES](image)

**Design Condition from MP 146.2 to MP 171.6 (ROWAN & CARTER COUNTIES)**

**19.5°**

**Design Condition from MP 171.6 to MP 191.6 (BOYD COUNTY)**

**1959 KENTUCKY DESIGN CURVES**

**I 64 Westbound**
Likewise, the expected life for the section from MP 180.8 to MP 191.6 would have a reduction in expected life from 3.5 million EAL's to 1 million EAL's when the CBR is reduced from 5 to 3.3.

Traffic has been estimated using AADT maps, determining the percentage of the traffic for given vehicle classifications by procedures reported in research report 455. Traffic volumes were computed from interchange to interchange. Damage factors used were those used in the AASHTO design system. These factors are very nearly the same as those reported in report 455. Plots were made of EAL versus calendar year for each interchange-to-interchange length.

The Division of Research was requested to provide several overlay designs for each section according to different levels of EAL's. Thus, the levels chosen were:

1. twenty years after the section was opened to traffic,
2. 1985,
3. 1990, and
4. 1996.

The EAL's estimated for the section corresponding to 20 years after being opened to traffic were used to determine the AC thickness required with the existing DGA thickness. Subtracting the original AC thickness yields the deficient AC thickness at the time of construction. For other design years, subtracting the initial deficient thickness and the effective thickness yields the overlay thickness due to deterioration.

Those areas which had very weak AC layer or foundation conditions were noted earlier. Special overlays were designed for those spots and the overlay thickness for that section was subtracted from this special overlay thickness to determine the required thickness. In addition, the rut depth was subtracted to yield the 'patch' thickness given in Table A2. The assumption has been made that a leveling course will be constructed to eliminate the surface rutting prior to constructing the overlays.

Thus, overlay thicknesses have been presented as two values in case there are two methods of funding to be used. The 'design overlay year' will have to be chosen by others.
MEMO TO: Jas. H. Havens, Director  
Division of Research  

FROM: D. C. Newberry, Jr.  
Chief Research Engineer  

SUBJECT: Rutting Investigations; I 64 and US 60.

December 8, 1978

In regard to our further concern about rutting in the wheelpaths on asphaltic concrete pavements, two additional sites were trenched (cross-sectioned) and analyzed. One site was at MP 186.227, eastward on I 64, Boyd County. The rutting there had progressed to 0.75 inches or more. The second site was on the experimental, full-depth (18 inches) asphaltic concrete near Ashland (US 60) M.P. 8.139, at Sumit. The rutting, as had been observed before, near Thousand Sticks on the Daniel Boone Parkway (also cross-sectioned)* appeared to have occurred in the form of shear in the upper five or six inches of the asphaltic concrete. The discovery of this manner of occurrence on the Parkway and the confirmation now of its typical pattern will have significant bearing on decisions and strategies employed in the design of pavement structures to carry heavy traffic. Labeled photographs are attached, and more detailed information follows.

Deflection tests were made with the Road Rater; density tests were made with the Seaman Nuclear Density Meter; and physical measurements were made from a string line at the surface.

The rutting was determined previously during a visit to the sites.** Rutting at the I 64 site was a maximum of 0.50 inches (12.7 mm) in the outer and 0.625 inches (15.88 mm) in

---

*Memorandum, September 5, 1978; File P.3.1; J. H. Havens to W. B. Drake; Subject: "Rutting, Asphaltic Concrete Pavements," with attachments.

**Memo to G. F. Kemper from A. R. Romine, August 24, 1978; Inspection of I 64; Rowan, Carter, and Boyd Counties.
the inner wheel track. Rutting at the US 60 site was a maximum of 1.19 inches (30.16 mm) in the outer and 1.125 inches (28.58 mm) in the inner wheel track. Both of these sites have a high volume of coal-truck-type traffic.

Photograph 1821-6 shows obvious cracking in the inner lane. This cracking resembles that examined on I 64 in Clark and Montgomery Counties in 1968.*** That portion of I 64 was overlaid in 1973. The cracking here and there extends only through the surface. The cracking now, as then, is believed to have been induced by rolling -- at the time of construction. Cracking is less obvious in the outer lane -- more especially in the wheel paths. There, those cracks (see Photo 1821-5) appear to have been healed by traffic; and close-spaced, tension cracks perpendicular to the wheel path indicate tractive displacement (shear) in the backward direction. This type of movement was observed at the Daniel Boone Parkway site and has been observed on I 75 at about MP 51 and northward. Lines were scribed onto the surface at the

***Memo report by D. C. Newberry; August 20, 1968; also: Unfinished Report; "An Investigation of Surface Cracking in a Bituminous Concrete Surface [I 64-5-(8)100];" Jas. H. Havens; February 1970; and photos made 3-9-72.

Figure A4. Photograph 1821-6 showing cracking in the surface layer, inner lane of US 60.
Figure A5. Photograph 1821-5 showing the surface of the outer lane of US 60; apparently surface cracking has been healed by traffic.

Figure A6. Photograph 1821-14 showing lines cut into the pavement surface.
Figure A7. Density and physical cross section of I 64, MP 186.227 (20 Nov 1978).

Figure A8. Density and physical cross section of US 60, MP 8.139 (21 Nov 1978).

I 64 site (see Photo 1821-14); they will be observed through the next warm season.

The results of the density and physical cross-section measurements are graphically displayed, and the graphs are attached hereto.

The Road Rater data are available but are not included here.

The data and photographs support the following observations: 1) the rutting is contained in the upper asphaltic concrete courses; 2) the I 64 cross section, measuring the depth of the various courses from a string line, indicated possible but slight rutting in the DGA base course.

Additional attention must be given to achievement of higher stabilities in the upper pavement courses to assure immunity against rutting. AC 20 or heavier asphalt cement should be used in the upper portion of the heavy duty pavements. In fact, the use of AC 40 may be indicated.
Figure A9. Photograph 931-7 showing the DGA base layer of a test section of I 64 in Boyd County, Kentucky.

Figure A10. Photograph 931-2 showing the AC surface layer of a test section of US 60 in Boyd County, Kentucky.
The trenching of I 64, using the earth saw which cuts dry, upon exposure of the DGA base layers revealed no indication of free water or muddiness anywhere. Photograph 931-7, of the I 64 cut, is a good view of the DGA layer.

Photograph 931-2, of the US 60 cut, exposes the 18-inch, (457 mm) asphalt concrete depth and the surface rutting there. No free-draining water was found in or around the full-depth section.

gd
MEMO TO: G. F. Kemper
State Highway Engineer

FROM: A. R. Romine, P.E.
Assistant State Highway Engineer for Operations

DATE: August 24, 1978

SUBJECT: Inspection of I-64
Rowan, Carter, and Boyd Counties

This is a follow up on the January 1978 report. Cores were taken in Boyd and Carter Counties only. The section of Carter from Grayson to the Boyd County line was in fair condition. We did not take cores on this section.

The attached sheet shows where the cores were taken and our observation of the depth of the cracking.

The cores were taken on Section 1, beginning at the Rowan County line (Milepoint 146.20) and extending to the junction of KY 1 at Grayson (Milepoint 171.60). Section 2 begins at the Boyd County line (Milepoint 180.812) and ends at the West Virginia state line (Milepoint 191.507).

The depth of the cracks were much deeper than we expected and this made the rate of deterioration much faster than most of us would expect.

This office recommends that the Division of Research be called upon to assist in evaluating the action we should take in restoring this 36-mile section of I-64. It would be highly desirable to schedule these sections of I-64 next year.

The cores are being stored by the Division of Maintenance if you would like to take a look at them. Please advise this office on how we should proceed with this needed project.

ARR/bh

Attachment
Mr. Bayes:

The cores you requested to be taken from the pavement on I 66 in Carter & Boyd county are listed as follows:

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<thead>
<tr>
<th>COUNTY</th>
<th>MILE POST</th>
<th>WHEEL TRACK</th>
<th>CORE NO.</th>
<th>AVERAGE DEPTH OF CRACKS</th>
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<td>155 + 1,050 ft.</td>
<td>inside &quot;</td>
<td>2</td>
<td>2 1/2&quot; 4&quot;</td>
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<tr>
<td></td>
<td>160 + 2,100 ft.</td>
<td>outside &quot;</td>
<td>3*</td>
<td>3 1/3&quot; 3&quot;</td>
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<tr>
<td></td>
<td>165 + 40 ft.</td>
<td>inside &quot;</td>
<td>4</td>
<td>3 3/4&quot; 3 3/4&quot;</td>
</tr>
<tr>
<td></td>
<td>168 + 170 ft.</td>
<td>inside &quot;</td>
<td>5</td>
<td>4 1/2&quot; 1 1/2&quot;</td>
</tr>
<tr>
<td>Boyd</td>
<td>181 + 2,700 ft.</td>
<td>inside &quot;</td>
<td>6</td>
<td>5 1/4&quot; 2 1/2&quot;</td>
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<tr>
<td></td>
<td>182 + 1,600 ft.</td>
<td>outside &quot;</td>
<td>7</td>
<td>6 1/4&quot; 3&quot;</td>
</tr>
<tr>
<td></td>
<td>185 + 0</td>
<td>outside &quot;</td>
<td>8</td>
<td>7 1/2&quot; 3 3/4&quot;</td>
</tr>
<tr>
<td></td>
<td>189 + 0</td>
<td>outside &quot;</td>
<td>9</td>
<td>8 1/4&quot; 4&quot;</td>
</tr>
<tr>
<td></td>
<td>190 + 0</td>
<td>inside WBL</td>
<td>10</td>
<td>9 1/2&quot; 4&quot;</td>
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<td>185 + 0</td>
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<td>11</td>
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<td>12</td>
<td>11 1/2&quot; 4 1/2&quot;</td>
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<tr>
<td></td>
<td>181 + 2,600 ft.</td>
<td>inside &quot;</td>
<td>13</td>
<td>12 3/4&quot; 7 3/4&quot;</td>
</tr>
<tr>
<td>Carter</td>
<td>168 + 0</td>
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<td>14</td>
<td>13 1/2&quot; 2&quot;</td>
</tr>
<tr>
<td></td>
<td>165 + 50 ft.</td>
<td>inside &quot;</td>
<td>15</td>
<td>14 1/2&quot; 2 1/2&quot;</td>
</tr>
<tr>
<td></td>
<td>160 + 50 ft.</td>
<td>inside &quot;</td>
<td>16</td>
<td>15 1/2&quot; 3&quot;</td>
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<tr>
<td></td>
<td>155 + 2,600 ft.</td>
<td>outside &quot;</td>
<td>17</td>
<td>16 1/2&quot; 1 1/2&quot;</td>
</tr>
<tr>
<td></td>
<td>150 + 1,000 ft.</td>
<td>outside &quot;</td>
<td>18</td>
<td>17 1/2&quot; 1 1/2&quot;</td>
</tr>
</tbody>
</table>

* Transverse crack across the width of road.

**Extraction test showed the percent of asphalt in the base as 5.26% and the surface as 5.91%.

*Width of cracks at surface varies from 1/8" to 3/8".*
**ROWAN-CARTER SECTION 1**

Typical as Constructed

<table>
<thead>
<tr>
<th>Description</th>
<th>Roughness Index</th>
<th>Skid No.</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mile Points</td>
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</tr>
<tr>
<td>146.20 to 154.2</td>
<td>E. Bound Outer 510</td>
<td>43</td>
<td>41.5</td>
</tr>
<tr>
<td>154.2 to 161.5</td>
<td>W. Bound Outer 540</td>
<td>40</td>
<td>40.0</td>
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<td>161.5 to 171.607</td>
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<td>36.6</td>
</tr>
<tr>
<td>161.5 to 171.607</td>
<td>W. Bound Outer 430</td>
<td>39</td>
<td>36.6</td>
</tr>
</tbody>
</table>

Roughness Average: 443. Rated Smooth: 443 - 30

5. Cracking developing in wheel paths. Width of cracks 1/8" - 1/4". Very little alligator cracking visible. Some of the wider cracks may be taking water in the 100 percent shale cuts and fills and the long fill section west of Grayson.

6. Rutting averages 1/4" to 1/2"
SECTION 2

Carter County Bituminous Surface 9.20 Miles

Beginning: KY 1 and KY 7 Under pass (Mile Point 171.607)
Ending: Boyd County Line (Mile Point 180.812)

SPECIAL NOTES:
2. Surface and shoulders good to very good condition.
3. Roughness Index and Skid No. by Research Division.

<table>
<thead>
<tr>
<th>Description</th>
<th>Roughness Index</th>
<th>Skid No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mile Point</td>
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<td></td>
</tr>
<tr>
<td>171.607 to</td>
<td>328 Smooth</td>
<td>37-Average</td>
</tr>
<tr>
<td>181.812</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

4. Rutting average approximately 1/4".
5. Very little cracking apparent.
6. There has been some wedging at some of the bridge abutments.
BOYD COUNTY

SECTION # 3

BOYD COUNTY SECTION - 70.3 MILES

1. Completed - and opened to traffic - 1965
2. Safety project in progress now (1978)
   Safety work consists of:
   (a) New type guardrail
   (b) Safety headwalls
   (c) Some fill flattened out
   (d) No shoulder or surface work

CONDITION OF SURFACE AT PRESENT TIME

   West Bound - 39
2. Roughness Index - 525 (10/77 Research Information)
3. The surface is badly cracked in wheel paths - some areas have developed alligator cracking
4. Average Rutting Depth - 1/4" - 1/2" 
5. Cracked surface - 30 - 40 percent
   Cracks are opened 1/8" to 1/4"
6. Surface Raveling - 10% or Less

Typical As Constructed
SECTION No. 3
Appendix B.

Graphs of Estimated Fatigue History
CARTER-BOYD COUNTY LINE TO US 60 INTERCHANGE TO KY 180 INTERCHANGE

MP 180.812 TO MP 181.369 TO MP 185.469

DESIGN CBR = 5
DESIGN EAL = $3.5 \times 10^6$
REACHED 1981
BEGINNING OF AC PAVEMENT TO KY 2 INTERCHANGE
MP 146.20 TO MP 156.265

DESIGN CBR = 9.0
DESIGN EAL = 8.0 x 10^6
REACHED 1995
KY 180 INTERCHANGE TO US 23 INTERCHANGE
MP 185.469 TO MP 190.724

DESIGN CBR = 5
DESIGN EAL = 3.5 x 10^6
REACHED 1979
KY 2 INTERCHANGE TO US 60 INTERCHANGE

MP 156.265 TO MP 161.452

DESIGN CBR = 9.0
DESIGN EAL = 8.0 \times 10^6
REACHED 1996
US 60 INTERCHANGE TO KY 1,7 INTERCHANGE

MP 161.452 TO MP 171.607

DESIGN CBR = 9.0
DESIGN EAL = 8.0 x 10^6
REACHED 1992
US 23 INTERCHANGE TO WEST VIRGINIA STATE LINE
MP 190.724 TO MP 191.507

DESIGN CBR = 5
DESIGN EAL = 3.5 × 10^6
REACHED 1977

ACUMULATED EQUIVALENT AXLE LOADS (EAL'S)

10^5
10^6
10^7
10^8

CALENDAR YEAR
Appendix C.
Examples of Road Rater Behavior
Figure C1. Example slab problem: Cracking and rutting (MP 168.2, westbound I 64).
Figure C2. Example foundation problem: Cracking and rutting (MP 191.15, westbound I 64).
Figure C3. Pavement in good condition with only very slight rutting (Carter-Boyd County line, I-64).
Appendix D.

Strip Charts Illustrating Measured and Predicted Pavement Condition
Figure D1. Strip charts illustrating CBR prior to paving.
Figure D2. Strip charts illustrating predicted CBR from Road-Rater deflection data.
Figure D3. Strip charts illustrating predicted effective thickness of asphaltic concrete from Road Rater deflection data.
Figure D4. Strip charts illustrating measured rut depths.
Figure D5. Strip charts illustrating predicted present serviceability index from Mays Ride Meter data.
Appendix E.
Kentucky Flexible Pavement Design Curves
DESIGN CURVES

PERCENT OF TOTAL THICKNESS COMPOSED
OF ASPHALTIC CONCRETE: 33%

MODULUS OF ASPHALTIC CONCRETE FOR
FULL-DEPTH DESIGN = 270 KSI
= 305 KSI
= 375 KSI

TOTAL THICKNESS, INCHES

REpetitions of 18-Kip AASHO Axleloads
DESIGN CURVES

PERCENT OF TOTAL THICKNESS COMPOSED
OF ASPHALTIC CONCRETE = 50%

MODULUS OF ASPHALTIC CONCRETE FOR
FULL-DEPTH DESIGN = 375 KSI

REpetitions of 18-KIP AASHO Axleloads
DESIGN CURVES

PERCENT OF TOTAL THICKNESS COMPOSED OF ASPHALTIC CONCRETE = 67%

MODULUS OF ASPHALTIC CONCRETE FOR FULL-DEPTH DESIGN = 375 KSI

REpetitions of 18-KIP AASHO AXLELOADS
DESIGN CURVES

PERCENT OF TOTAL THICKNESS COMPOSED
OF ASPHALTIC CONCRETE: 75%

MODULUS OF ASPHALTIC CONCRETE: FOR
FULL-DEPTH DESIGN = 375 KSI

REpetitions of 18-KIP AASHO AXLELOADS

TOTAL THICKNESS, INCHES
DESIGN CURVES
PERCENT OF TOTAL THICKNESS COMPOSED
OF ASPHALTIC CONCRETE = 100%
MODULUS OF ASPHALTIC CONCRETE FOR
FULL-DEPTH DESIGN = 375 KSI