COMMONWEALTH OF KENTUCKY
DEPARTMENT OF HIGHWAYS
FRANKFORT

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ADDRESS REPLY TO
DEPARTMENT OF HIGHWAYS
MATERIALS RESEARCH LABORATORY
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COMMISSIONER OF HIGHWAYS

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P. 3. 1.

MEMO TO: D. V. Terrell
Director of Research

The attached report, "Re-Evaluation of the Kentucky Flexible Pavement Design Criterion," is the result of a study requested by the office of the State Highway Engineer. The Research Division made a comprehensive study in 1947 and 1948, "Investigation of Field and Laboratory Method for Evaluating Subgrade Support in the Design of Highway Flexible Pavements." The 1948 report recommended a method of flexible pavement design using the laboratory California Bearing Ratio test and equivalent wheel loads to arrive at a flexible pavement thickness. Revisions involving refinements in predicting traffic and additional curves for higher traffic volumes and equivalent wheel loads have been added since 1948.

Gross load limits were changed from 42,000 pounds to 59,600 pounds by the 1956 Legislature. The use of 4-axle semi-trailer type vehicles has increased greatly since the change in the gross load limit. Traffic designs have been changed from 10 years to 20 years.

The flexible pavement design criteria has been in use since 1948, and a variety of projects involving a range of design variables were available for performance evaluation. Visual performance data, rutting measurements, pavement deflections and pavement openings were used to evaluate the performance of pavements.

Waterbound macadam type base was found to be susceptable to infiltration of subgrade soil. Clay type soil in granular base results in a loss of the load supporting or load distribution value and may result in rutting or failure. A dense-graded aggregate base was opened and no subgrade infiltration was observed. It is believed that a dense-granular type base will not be affected by subgrade infiltration.

The design curves in Fig. 20 represent the recommended thickness for subgrade and traffic conditions.

This report was sponsored by the Flexible Pavement Design Committee of the Highway Research Board, and was presented in oral form to Session 36 of the 1959 Board meeting.

Respectfully submitted,

W. B. Drake

W. B. Drake
Associate Director of Research

WBD:dl
Enc.
cc: Research Committee
Bureau of Public Roads (3)

Commonwealth of Kentucky
Department of Highways

RE-EVALUATION OF THE KENTUCKY FLEXIBLE PAVEMENT DESIGN CRITERION

by

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and
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Highway Materials Research Laboratory
University of Kentucky

January, 1959

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ABSTRACT

RE-EVALUATION OF THE KENTUCKY FLEXIBLE PAVEMENT DESIGN CRITERION

Prior to 1948, the criterion in Kentucky for designing the thickness of bituminous pavements was based upon a modified laboratory CBR and the 1942 curves developed by the California Department of Highways. In 1948, the Materials Research Laboratory reported: "An Investigation of Field and Laboratory Methods for Evaluating Sub-grade Support in the Design of Highway Flexible Pavement." Included in that report as a recommended method of thickness design for use in Kentucky was a set of curves based upon an empirical relationship between minimum laboratory CBR and observed pavement performance. These five curves accounted for traffic groups up to 10,000,000 EWL's. Since that time six additional curves have been included in the design charts for EWL groups up to 320,000,000. These additional curves were determined by extrapolation of the results from the 1948 study. Early in 1957, an evaluation of the design method was undertaken. The basis for this re-evaluation was a statistical comparison of actual pavement performances with the designed life as anticipated or predicted by the design curves currently in use. On this basis, projects were selected, design records assembled, performances surveyed, and the data analyzed. Selected pavements which had been designed by the method developed in the 1948 study were checked for performance by visual survey, by roughness measurements, by measurements of rutting, by measurements of loaded-deflection with the Benkelman Beam, and by opening pavements for observation and sampling. Flexible base types studied included waterbound macadam, bituminous concrete, granular dense-graded aggregate and combinations. Laboratory evaluation on basis of bearing tests were made.

1. The visual survey established a range of performance.

2. Road roughness measurements were related to CBR but no attempt was made to draw design curves from this data since it could be greatly affected by factors not related to structural design.

3. Pavements opened for inspection revealed permanent deformation in the upper layers of the system as well as intrusions of subgrade in waterbound base courses.

4. An alternate method of design based on limiting deflection under load was developed from the Benkelman Beam measurements. Curves drawn from this data indicate a need for a slightly greater thickness than provided by the 1948 curves.

INTRODUCTION

Pavement design engineers are charged with the responsibility of determining the thickness and types of pavement courses necessary to support millions of vehicle-passes, intense loads, and to withstand extreme weather conditions. Most soils are inadequate for direct service of this type; and so pavements of differing thicknesses, depending on the supporting ability of the soil and the amount of anticipated traffic, are needed to distribute the loads and to confine and protect them. Pavement design engineers are, in fact, charged with more far-reaching responsibilities in the sense that thicknesses must be adequate but not excessive. It is this rather tedious balance between economy and pavement-sufficiency that guides the engineers and constitutes the general basis for any thickness-design criterion. Criteria of design are semi-empirical and semi-theoretical. In theory they involve boundary applications of stresses on layered, semi-infinite masses. Often these stresses are either indeterminate or obscure, and therefore theory must be compensated by empiricisms. Basically, of course, empiricisms are founded on experience and experiment. In this sense, each road that is designed and built is, in part, an experiment or test of the design system used. Thus, a statistical analysis of the performance histories of a large number of pavements with regard to design-parameters, i.e. bearing capacity of the soil, traffic, and pavement thickness, should provide a reliable derivation of a design criterion and should likewise reveal any need for modifications or re-adjustments in a criterion so derived and used.

Prior to 1949, the criterion in Kentucky for designing the thicknesses of flexible pavements was based upon a modified laboratory California Bearing Ratio (CBR) and the 1942 curves developed by the California Department of Highways(1). In 1948, the Materials Research Laboratory, in a report on "An Investigation of Field and Laboratory Methods for Evaluating Sub-grade Support in the Design of Highway Flexible Pavements" (2), recommended a similar method of thickness design for use in Kentucky and included a set of five curves based upon empirical relationships between EWL's, minimum laboratory CBR, and the observed performance of Kentucky pavements (Fig. 1).

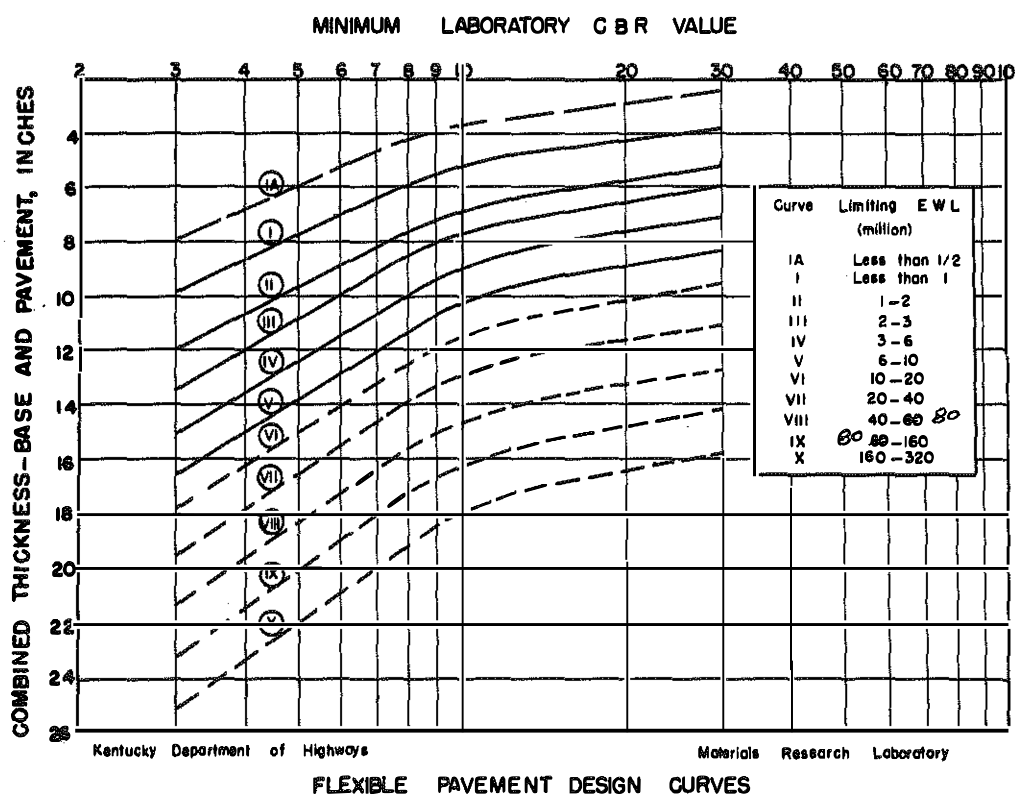

Fig. 1: Kentucky Flexible Pavement Design Curves. Curves I through V proposed in 1948; IA and VI through X added by extrapolation, 1954.

These five curves accounted for traffic groups up to 10,000,000 EWL's (Equivalent 5,000-lb. wheel loads, two directions). Since that time, six additional curves have been included in the design charts and cover EWL-groups up to 320,000,000. These additional curves were determined partly by extrapolation of the results from the 1948 study. This series of eleven curves, with some modification in methods of evaluating traffic, has been used by the Department to design flexible pavements during the past ten years. Early in 1957, the Research Division was requested to evaluate the effectiveness of the extrapolated curves as well as the original five curves and to determine if the curves should be further revised in any way or if factors heretofore not considered in the design of pavement thicknesses should now be taken into account.

Logically, of course, the basis for this re-evaluation would have to be a statistical study of actual pavement performances, accumulated EWL's, and subgrade CBR's. On this basis, projects were selected, design records assembled, performances surveyed, the data analyzed, and recommendations offered for revising the present design chart.

PRELIMINARY STUDIES

Selection of Projects for Study

The first criterion for selecting the projects to be studied was that the pavement must have been designed by the method recommended in the 1948 study. It was desired that the pavements be of high-type bituminous construction and have been in service as long as 1 year. The records were studied and a list of all eligible projects, meeting these requirements, was obtained. From a list of some 100 sections of road built since 1948, projects were selected so as to be distributed over the state as well as possible. Most of the major soil and geologic areas of the state were represented, and projects were selected so that available traffic groups were represented. An attempt was also made to select projects so that all of the more common base materials would come under study. Projects 1 mile or less in length and those not having sustained sufficient traffic were eliminated. Thus, curve revisions and bridge approaches were excluded. Projects involving large areas of salvaged pavement were also excluded. On the basis of these criteria, 70 projects representing 388.7 miles of Kentucky's flexible pavements were selected for study (See Fig. 2 and Table 1). Of these 70 projects, 57 were considered eligible, from the records available, for statistical analysis.

The Fayette-Madison County project and the Johnson-Lawrence County project, pavements studied in the 1948 investigation and not actually designed by the method currently under study, were included so as to provide extended distributions of projects.

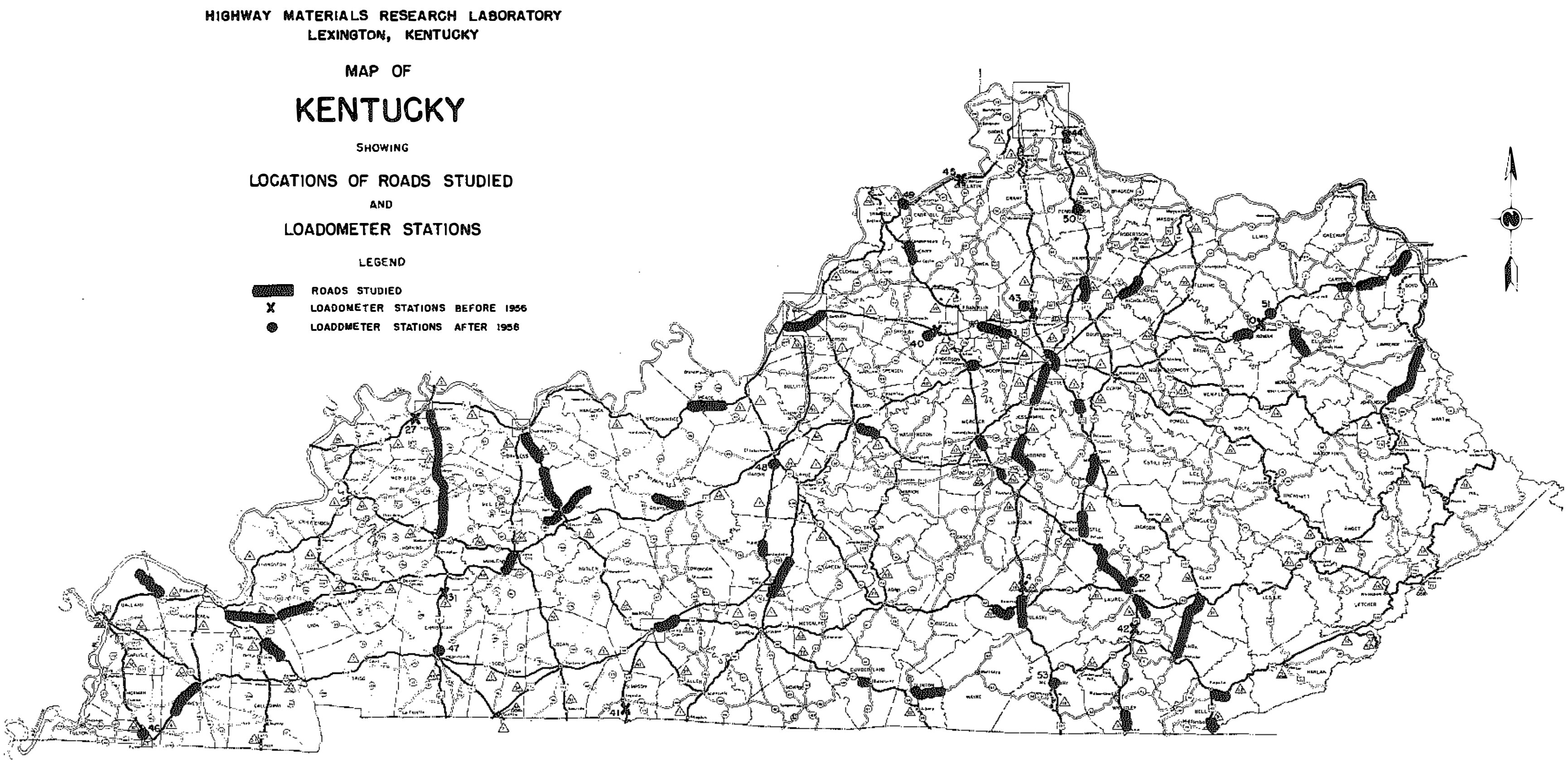

Fig. 2: Map Showing Locations and Distribution of Projects Studied.

Design Data

From the Division of Design, the design thickness of each pavement component for all projects was obtained and recorded. These values were then compared with the values as recorded on the plans and adjusted accordingly. When available, the design EWL and design CBR were also obtained. CBR's for most of the projects were gathered from soil reports on file in the Design Division and Materials Division. All relevant design data are given in Table 1 of the Appendix.

Evaluation of Traffic

Traffic data were obtained from the Division of Planning. For most of the projects, the ADT* for each year, from the time the

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* ADT - Average Daily Traffic, two directions

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project was completed through 1957, was recorded. Also available were weight data and vehicle classification counts for each year, from and including 1951, for the ten permanent loadometer stations located over Kentucky (Tables 8 and 9, Appendix). With this information, it was possible to calculate the EWL's which had passed over the pavements (Table ~~10~~ 2, Appendix) and thus to study the traffic history of each project.

By comparing the actual EWL value with the designed 10-year EWL's, a "traffic age" or "service age" for the projects could be determined (Table 2, Appendix). Thus, if the actual EWL's at any age exceeds the anticipated EWL's at that age, this would indicate that traffic has increased more rapidly than anticipated and that the service-age of the pavement exceeds its chronological age.

For computations of EWL's in the 1948 study, the only type of data available for the entire life of all roads studied was average yearly 24-hour traffic counts. Loadometer data were available from 10 permanent loadometer stations for the period between 1942 and 1947. During 1947, by the aid of temporary loadometer stations, loadometer data were obtained for all roads studied. Thus, where applicable, EWL's were computed from actual loadometer data. However, since many of the roads were built before 1942, it was necessary to project the trends in traffic and distribution factors, evident in the 1942-1947 data from each of the 10 permanent stations, to a year somewhat beyond the earliest construction date of any road studied. On this basis, the trends of each of the 10 stations were projected backwards to 1934. Then, for the year 1947, a ratio of EWL's to total vehicles per year was calculated for each road and each of the 10 permanent loadometer stations. On the basis of these ratios, similarity between a particular loadometer station and a particular road was established. Thus, the trends in traffic distribution where lacking on a particular road were calculated from a typical or similar loadometer station. These trends in distribution, when applied to the average yearly 24-hour traffic counts, provided a cumulative total of EWL's which was considered to be the total EWL's on each road since its construction or last resurfacing. The EWL's calculated in this manner were correlated empirically with other design parameters (CBR's, pavement thicknesses, and pavement conditions); and the best fitting curves, so derived, were adopted as the criterion for design.

None of these traffic data was tested for statistical reliability; and since the period involved the war-years, it was suspected that these data were unsuitable for predicting future traffic trends. Alternatively, it was assumed that truck traffic, in percent of existing ADT, would double in 10 years*. Thus, if it is also assumed that EWL's

*An example of the method of estimating 10-yr. design EWL's for all roads included in this study is given in the Appendix. In 1954, the method was revised to a 20-yr. estimated design EWL basis wherein traffic volume projection factors, vehicle classification factors, and axle and weight distribution factors are used in the computation. Examples of this method are also given in the Appendix.

would increase in direct proportion to the volume of truck traffic, the accumulation of EWL's at any age throughout the 10-year period, expressed in percent of the 10-year estimate, could be described theoretically by:

$$\%\text{ of 10-yr. estimated EWL} = 6.67x + .333x^2$$
$$\text{where } x = \text{chronological age in years}$$

The equation above describes the "theoretical curve", curve No. 1, shown in Fig. 3. Curve No. 2, a locus of points determined by the least squares method, represents calculated actual accumulations of EWL's at all ages for all roads which were designed and built according to the 1948 criterion and for which traffic data were sufficiently complete to be included in this re-evaluation study.

While there is wide variance among the data (standard deviation = $\pm$ 67.64%); the average or trend shows close agreement with the

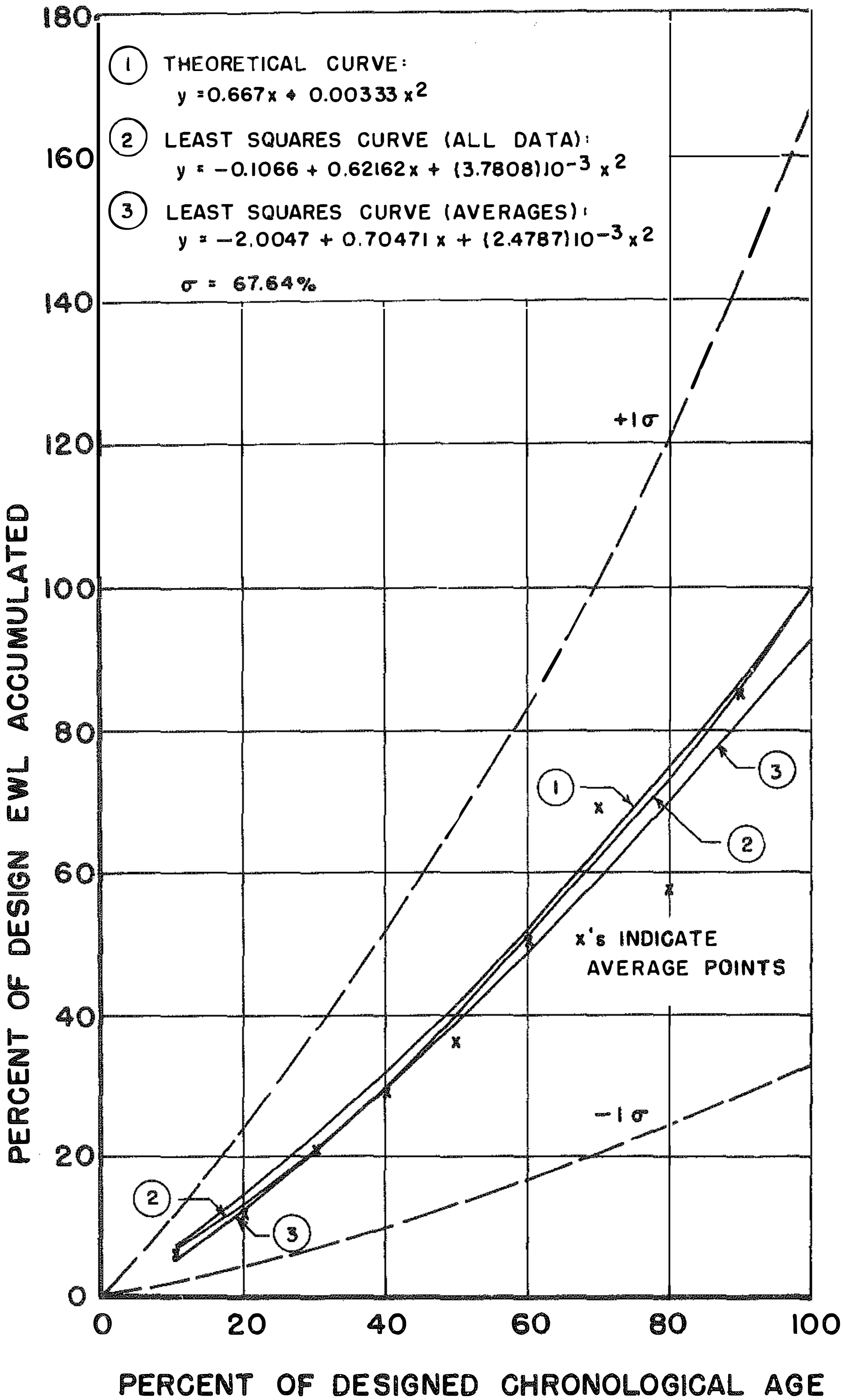

Fig. 3: Graph Illustrating Statistical Relationship Between Percent of Designed EWL Accumulated and Percent of Designed Chronological Age.

theoretical curve. To this extent, it may be said that actual accumulations of EWL's have closely paralleled the predicted accumulations and that, on the average, "traffic age" or "service-age" has closely paralleled chronological age. On the other hand, extreme variations in the percentage of accumulated EWL's at a particular chronological age, expressed as the 99.9% confidence limit, would be equivalent to $\pm$ 3 standard deviations or approximately $\pm$ 200%. Expressed on a 75% confidence limit basis, the extreme deviations, of course, would not exceed $\pm$ 1.15x67.64%. It may be similarly stated, therefore, that 15.9% of the roads accumulated traffic at a rate 1.68 times greater than the predicted rate. Likewise, 15.9% of the roads would reach 100% of their designed traffic age within 68% or less of their designed life-expectancy.*

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* To be precise, statistically speaking, we could have used the mean square error rather than the variance since the ratio estimates used involve a slight bias. However, the bias would be negligible in comparison with the variance and can safely be ignored.

Traffic vs. Pavement-Life

Since the only parameters considered in the present design criterion are CBR's, pavement-thicknesses, and EWL's predicted for a chosen number of years in the future, it is implied thereby that a pavement would have a designed life-expectancy comparable to the number of years for which the EWL's were predicted. Hence, the variations evident in actual accumulations of EWL's should have an analogous effect on actual pavement-life statistics. While terminal-life statistics are not available for this study, it may be surmised from variations in traffic alone that the service-life of 68% of the roads in this series may vary between 68% and 168% of their so-called designed life-expectancy or between 6.8 and 16.8 years.

Actual average life and survivor statistics (3)(4) should provide helpful insight into this aspect of the problem and should also provide a test of the validity of the design system. For instance, if the EWL's were accurately predicted for a 10-year period and the average life of the pavements proved to be 18 years, it would have to be concluded that the thicknesses were excessive and that the design curves were unrealistic. The design system would seem equally unrealistic, of course, if the EWL's were accurately predicted for 18 or 20 years and the average life from survivor statistics proved to be only 9 or 10 years. Likewise, it can be seen from the present design curves (Fig. 1) that the difference in thickness between a 10-year design and a 20-year design, assuming that the 20-year estimate of EWL's exceeds the 10-year estimate by a factor of 2, would be about 1-1/2 inches.

PERFORMANCE SURVEY

Visual Inspection

Visual inspections of the various projects were made in the summer of 1957. To aid in evaluating pavement condition, each project was inspected throughout its entire length, and all evidences of distress were noted as to type, extent, and location. Conditions recorded included cracking of all kinds -- logitudinal, alligator, hairline -- and skin and structural patching. Wavy sections, any signs of slides, fill settlement, as well as any adverse or unusual drainage conditions were noted. Numerous measurements of rutting were taken on each project in order to obtain an indication of the extent of permanent deformation in the wheel tracks. In order to reduce the notes taken during the visual inspection to a numerical value, the lengths of wheel track showing longitudinal cracking, alligator cracking, skin patching, and structural patching were summed for each project and tabulated as a percent of the total length of wheel track in the project (Table 3).

Unfortunately the only traffic groups represented by enough samples to permit a cursory correlation of pavement condition with CBR and thickness were Groups IV and VI. For projects in traffic Group IV, a plot of thickness vs CBR, with the percent of pavement failed noted by each point, is shown in Fig. 4. Here again, there was not a sufficient number of failed pavements to clearly define a relationship, and the straight line represents an approximation of the required thickness assuming that the excessively failed pavement (13 percent) and the two adequate pavements falling on this line are near or below the critical thickness.

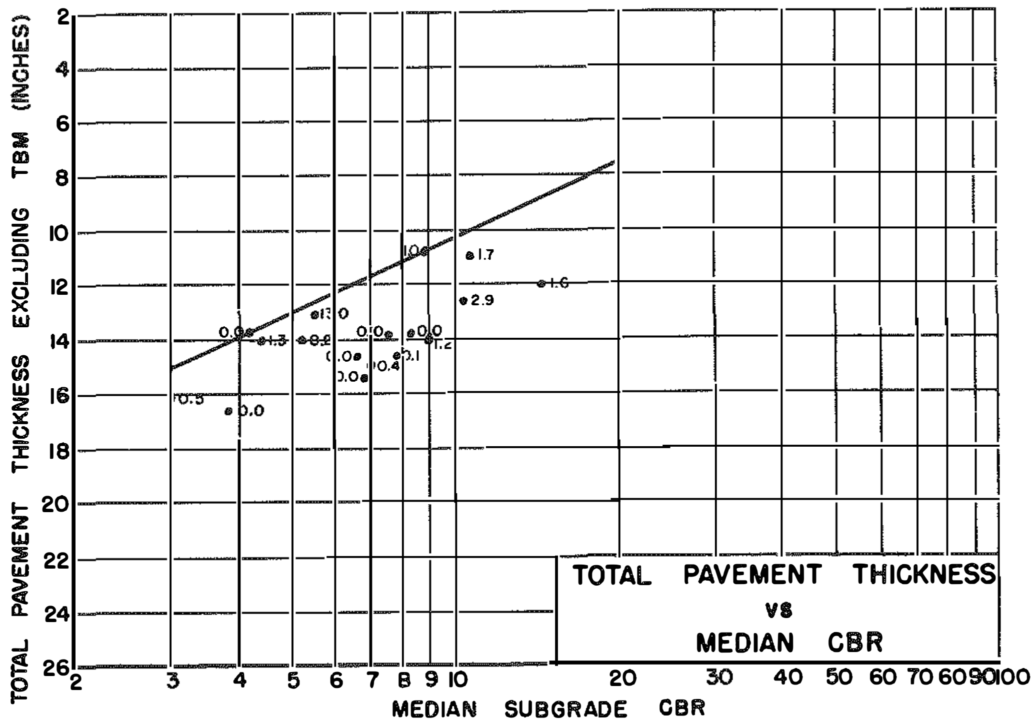

Fig. 4: Plot of Pavement Thicknesses vs Median Subgrade CBR's For all Projects in Which the Accumulated EWL's Fell Within the Limits of Traffic Group IV, 3 to 6 Million.

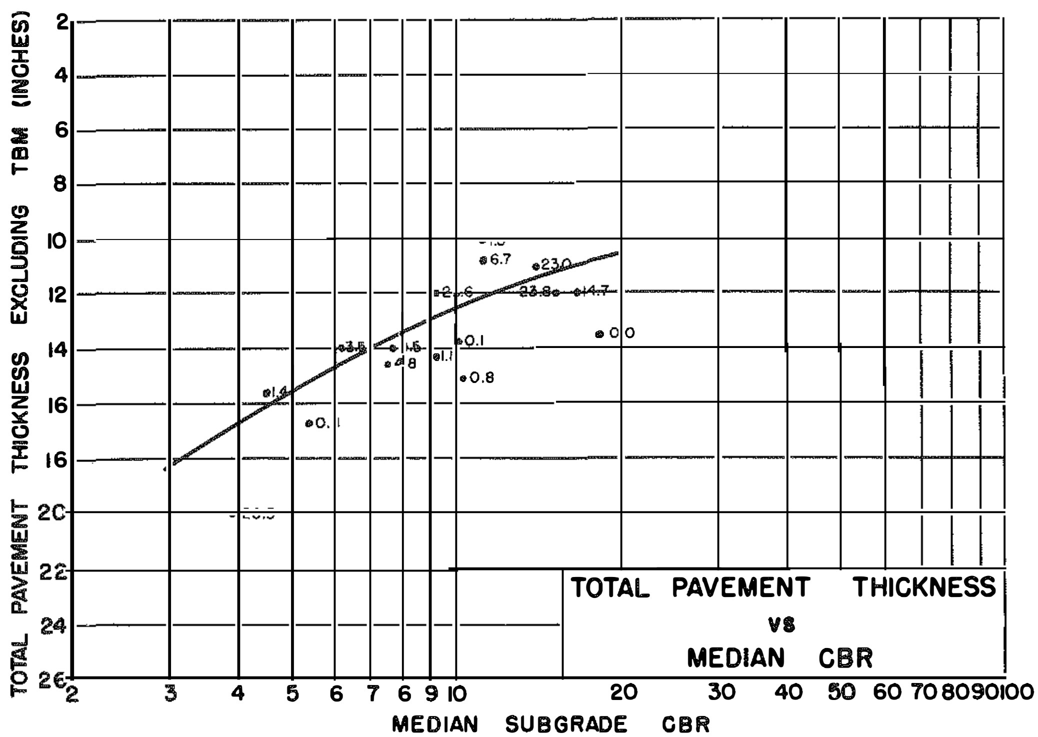

Fig. 5: Plot of Pavement Thicknesses vs Median CBR's for all Projects in Which Accumulated EWL's Fell Within Traffic Group VI.

Data for projects in Traffic Group VI plotted in the same manner are shown in Fig. 5. Here better control for the curve was provided by nearly equal numbers of failed and unfailed pavements in this group. The two excessively failed pavements, shown below the curve at CBR values of approximately 16 and 17, were on the same route (different projects); and performance may have been affected by other factors. Placing these points above the curve would require flattening the curve more sharply at CBR 10.

In Fig. 6, the curves in Figs. 4 and 5 are shown superimposed upon the original design curves and may indicate a need for slight revision of the design chart.

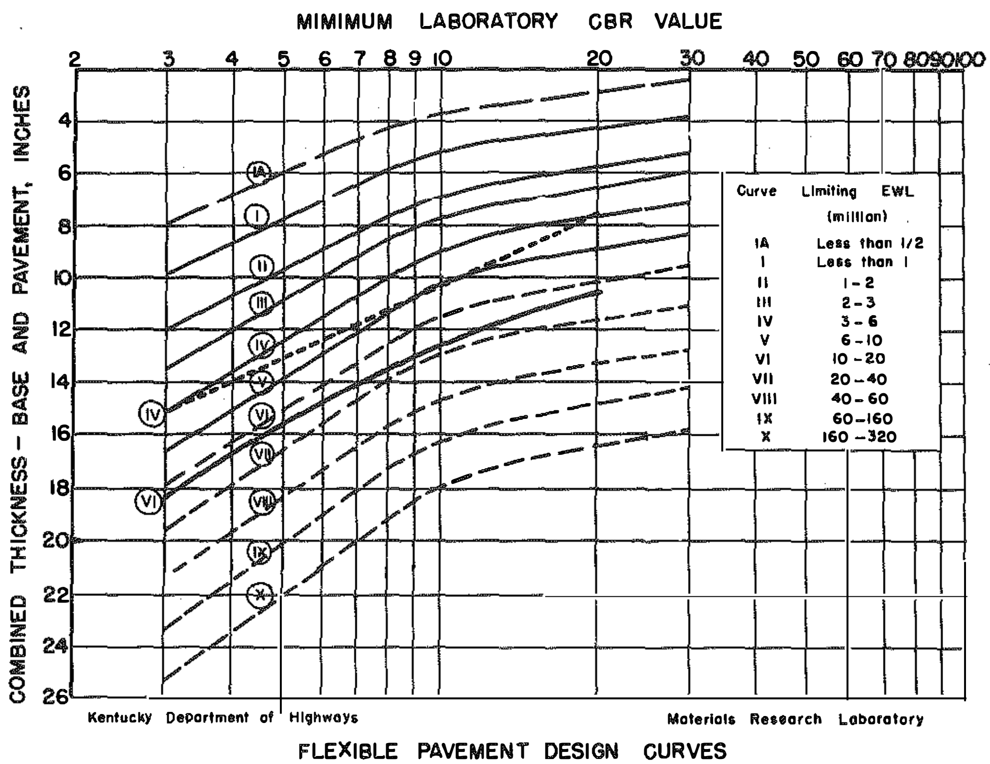

Fig. 6: Present Kentucky Design Chart Showing Trend Lines From Figs. 4 and 5 Superimposed.

Rutting

Rutting measurements were made by laying a straight-edge transversely across a traffic lane and measuring the maximum deviation (See Fig. 7). This measurement is not entirely rutting in the strict sense because a portion of the deformation may be the result of upheaval between the wheel tracks as illustrated in Fig. 8. However, throughout this report the term "rutting" implies the total deviation from a straight edge. Measurements were made at more or less random intervals. From these measurements, a simple arithmetic mean of all values was computed for each project. These average values are summarized in Table 3 of the Appendix.

At first it was thought that rutting was the result of consolidation either in the pavement courses or in the subgrade. Any extreme rutting would then be considered an advanced stage of failure extending into the subgrade. However, from information obtained by opening selected pavements showing medium to extreme rutting, it was noted that, on the average, only 4% of the rutting occurred within the bituminous layers while 72% occurred within the granular base courses. These percentages are based upon comparisons of the thicknesses of the layers within and outside the wheel tracks. This indicated that the original thoughts concerning rutting were in error and that water-bound macadam is more highly susceptible to consolidation or movement under traffic than previously suspected. The densities of the WBM obtained while opening the pavements do not indicate any great degree of consolidation in most cases; thus, the deformations must result primarily from particle rearrangement and movement and must be the combined result of upheaval and subsidence.

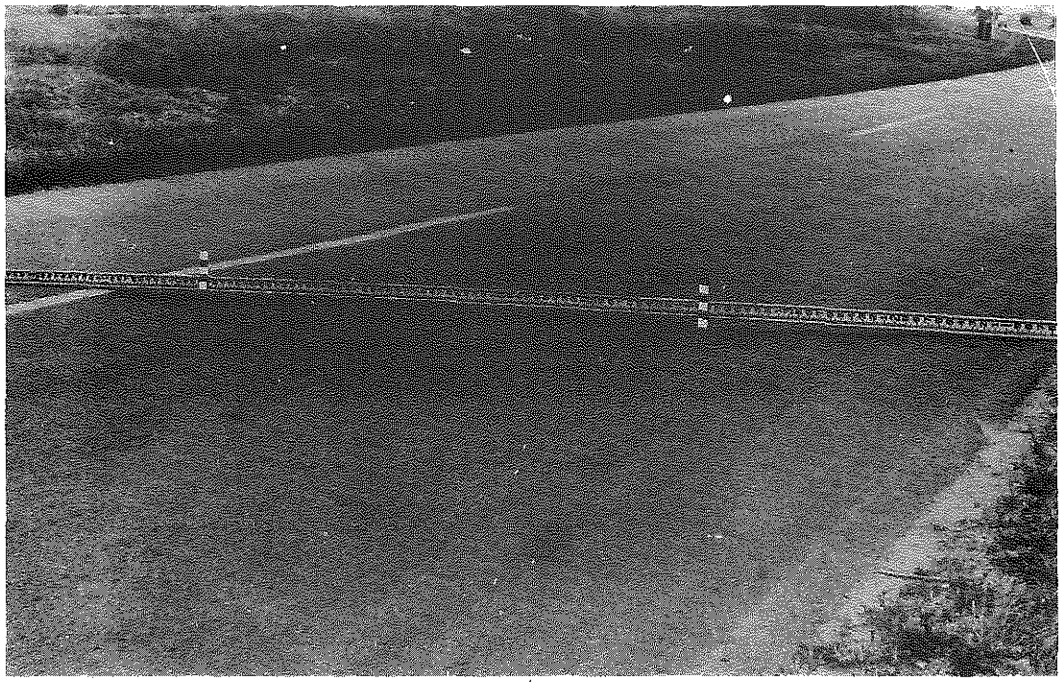

Fig. 7: Photograph Illustrating Rutting of Pavement Within Wheel Tracks as Deviation from a Straight-edge.

Fig. 8: Photograph Illustrating Extreme Rutting and Upheaval.

From what has been said, it might be expected that rutting would increase with total pavement thickness and with traffic. These general trends are also indicated by Fig. 9. However, the implied increases in rutting with increased pavement thicknesses are considered to be in the nature of a paradox and should be more properly interpreted as indicating that the conditions causing rutting are more critical in the thicknesses designed for high intensities of traffic.

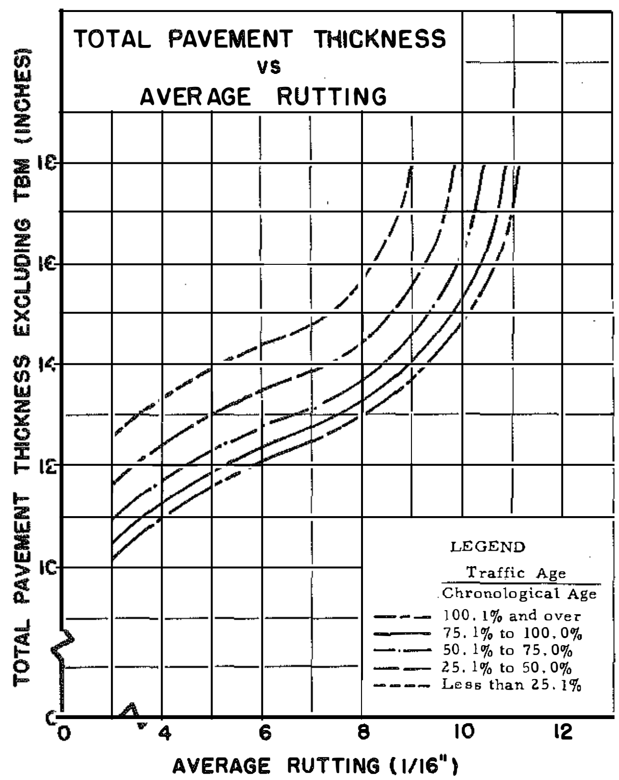

Fig. 9: Generalized Apparent Relationship Between Thickness and Rutting, According to Traffic-Age.

Road Roughness

With information from the field condition survey available, the traffic lane which exhibited the most distress was selected for an evaluation of roughness by the triaxial acceleration method reported in 1955 (5). The only deviation from the reported procedures was in evaluating the roughness records. The following method was used in determining the roughness of a road in terms of change in acceleration, sometimes referred to as "jerk". To obtain average acceleration, a compensating polar planimeter was used to measure the area under the vertical acceleration curve representing the length of pavement under consideration. Since the recorder chart was driven at a pre-set speed of 1/4 in./sec., each inch of chart length representing an elapsed time of 4 seconds, and the galvanometer sensitivity pre-set to 2 inches per g, it was possible to resolve the total area beneath the curve into g sec. (1 sq. in. = 2 g sec.); and:

$$\text{Area (in sq. in.)} \times 2g \text{ sec.} = \text{Total g sec.}$$

$$\frac{\text{Total g sec.}}{\text{Total Time}} = \text{Avg. g}$$

(Total time = 4 x length of chart considered, in inches)

By careful measurement of many charts, the average frequency of the acceleration wave was found to be 5 cycles/in. or 5/4 cps., giving a period of 0.8 sec./cycle. Since "jerk" is described as da/dt; average "jerk" would be:

$$\frac{\text{Average a}}{\text{Average t}} = \frac{A}{1.6L}$$

t = average period per acceleration cycle,
0.8 sec./cycle.

The vertical acceleration wave was analyzed by dividing the curve into short lengths of particular interest and determining the average "jerk" for that length using the above equation. To obtain an average "jerk" value for the entire project a weighted average was calculated. The average values are recorded in Table 3 of the Appendix.

In reviewing the roughness values it was noted that there is a general tendency for roughness to increase with increased rutting. However, in certain instances, it was noted that rutting could be rather uniform throughout a project and still result in good riding qualities provided that the vehicle remained in the wheel tracks. The curve in Fig. 10 indicates that roughness decreases as the bearing capacity of the subgrade increases.

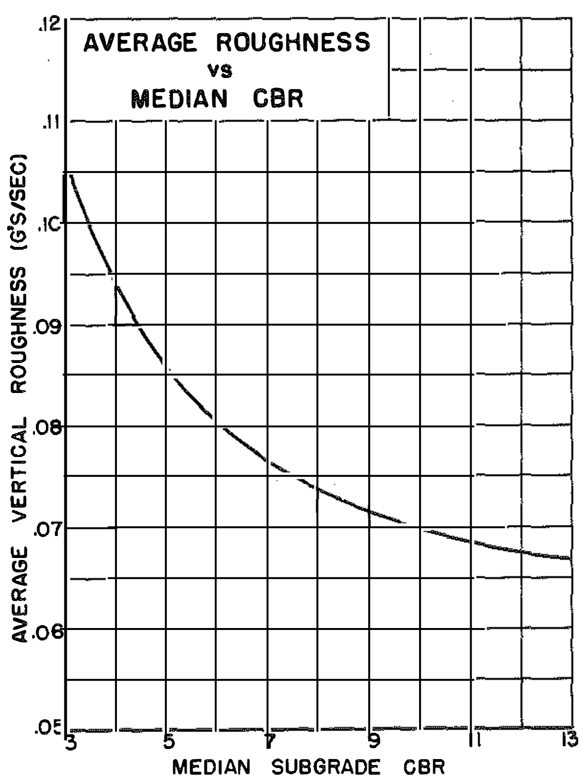

Fig. 10: Generalized Apparent Relationship Between Average Roughness Values and Median CBR of the Subgrades.

Pavement Deflections

In the late summer and early fall of 1957, Benkelman beam measurements were made at 50 locations on 20 projects. Deflections were measured in both the outside and inside wheel tracks under an 18,000-lb. axle load on dual tires. In order to evaluate the seasonal effect, deflection measurements were made again under the same conditions of loading in the spring of 1958 at the same locations previously visited as well as an additional 18 locations representing 11 other projects.

To obtain deflection readings, the probe beam was placed between the dual tires of the test vehicle so that the foot of the beam rested on the pavement 5 ft. ahead of the axle (See Fig. 11). The reference beam then rested on the pavement well back of the influence of the loaded wheels. As the test vehicle moved forward at creep speed, the probe foot deflected with the pavement, and the amount of deflection was read from an Ames dial. At each location, measurements were made until two consecutive readings were in agreement.

Also, in 1958, deflection measurements were made under a tandem axle loading of 32,000 lbs. at 8 locations on 5 projects. Two of these locations were also loaded with a 36,000-lb. tandem axle load and deflection measurements recorded.

Since the length of the probe beam on the Benkelman beam was designed for obtaining deflection measurements under single axles, modifications in the method of measuring were necessary. The probe beam was placed between the dual tires so that the foot rested on the pavement beneath the front axle (See Fig. 12). As the test vehicle moved ahead, the partial rebound between axles was noted, then the deflection was read as the rear axle passed the probe foot, and finally the complete rebound was read as the loaded vehicle moved well away from the setup. See Tables 4 and 5 (Appendix) for a listing of deflection measurements.

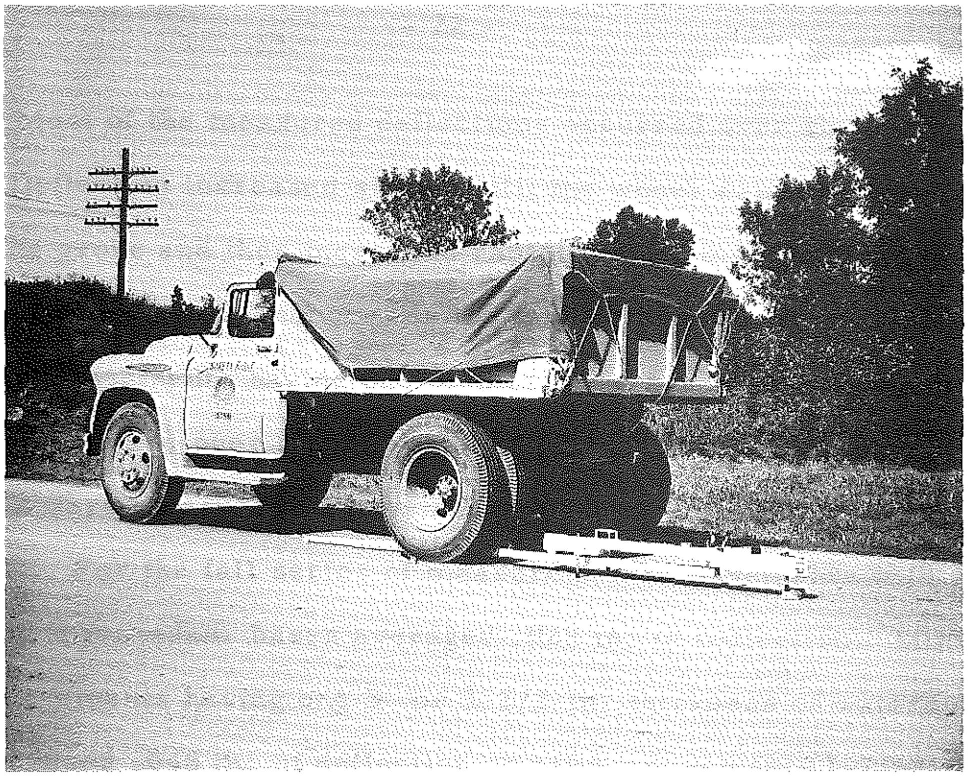

Fig. 11: Photograph Showing Benkelman Beam In-Place for Measuring Pavement Deflection Under 18,000-Lb. Single Axle.

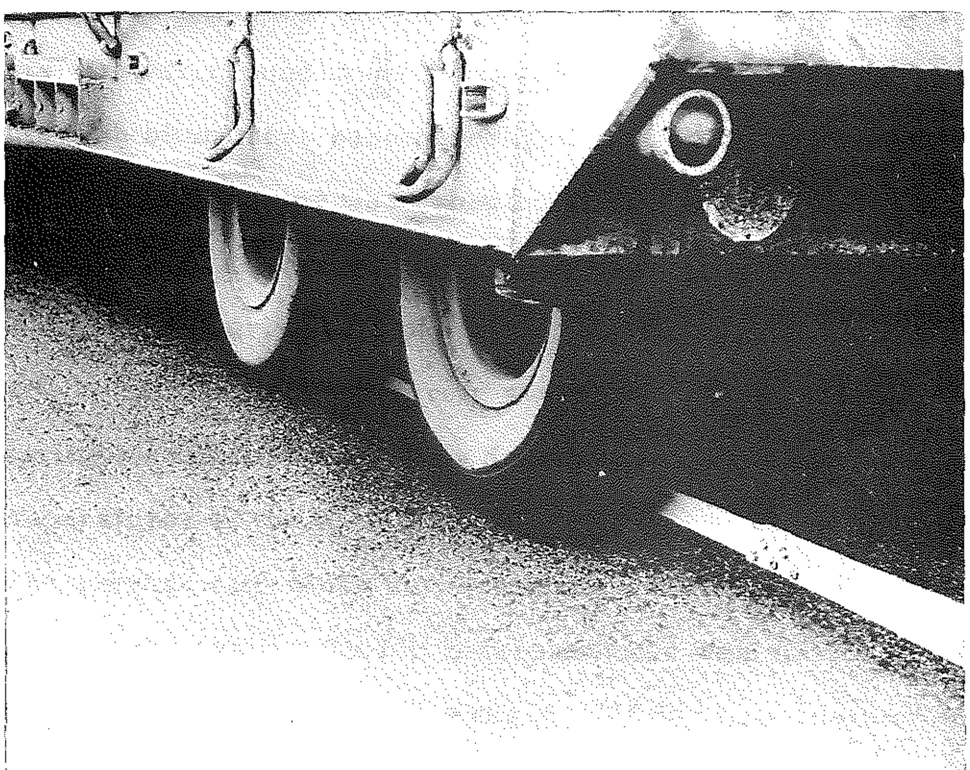

Fig. 12: Benkelman Beam In-Place for Measuring Pavement Deflection Under 32,000- or 36,000-Lb. Loads on Tandem Axles.

the inner wheel tracks during the spring measurements. This was probably due to greater susceptibility of the outer portion of the subgrade to climatic changes.

Four of the locations measured under 32,000-lb. tandem axle loadings were over waterbound bases, one was over combined waterbound and bituminous base, and three were over full depth bituminous bases. At creep speed over waterbound base, the tandem wheels acted independently. Rebound between the wheels was about one-half the maximum deflection and maximum deflection was 15.8 percent less than for the 18,000-lb. single axle. For the combined waterbound and bituminous bases, rebound was less than one-half the maximum deflection and maximum deflection was 15.3 percent less than for the 18,000-lb. single axle. For full depth bituminous bases, the tandem axles acted as a unit with no appreciable rebound between wheels. Maximum deflections for the 36,000-lb. tandem and 18,000-lb. single axle were equal. The lack of rebound between tandem wheels demonstrates the slab or beam action of bituminous concrete under the test conditions.

Deflections under a 36,000-lb. tandem axle loading were measured at two locations over a combined waterbound and dense-graded aggregate base. Rebound between the wheels was more than half the maximum deflection, and the maximum deflection was approximately equal to the maximum deflection under an 18,000-lb. single axle.

A plot of deflections according to traffic groups, with all points marked to distinguish between satisfactory or unsatisfactory pavements, is shown in Fig. 13. Pavements marked unsatisfactory were showing patching or cracking at or near the point measured. The curve best

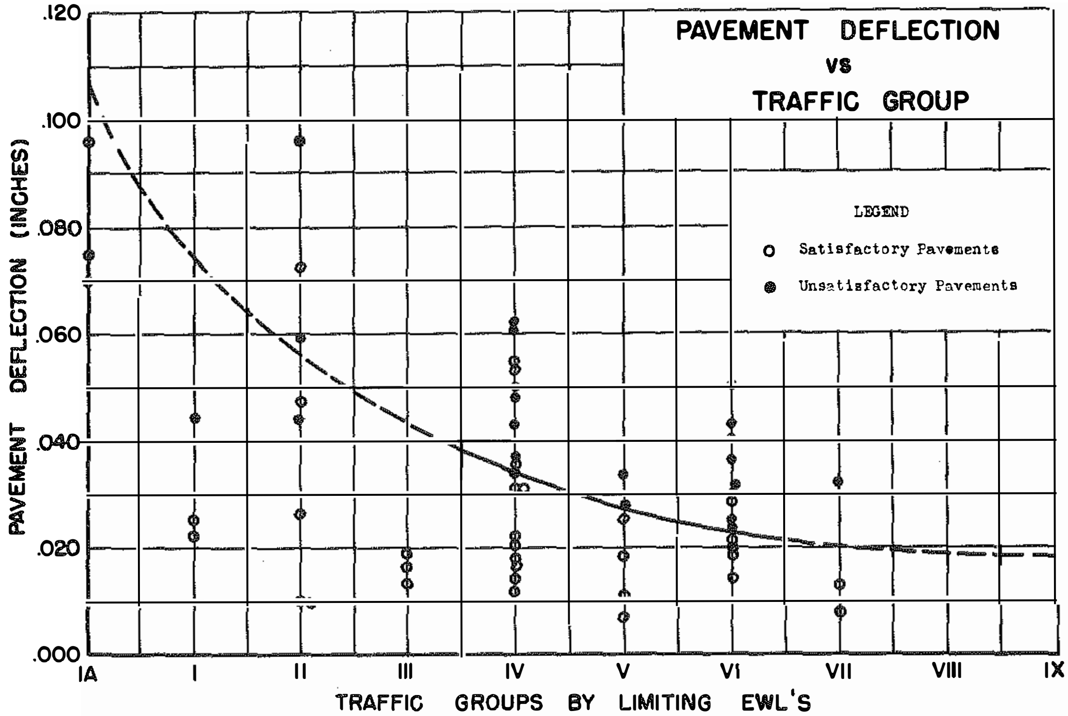

Fig. 13: Pavement Deflections, 18,000-Lb. Single Axle, Obtained From Both Satisfactory and Unsatisfactory Pavements Plotted According to Traffic Groups Corresponding to Accumulated EWL's.

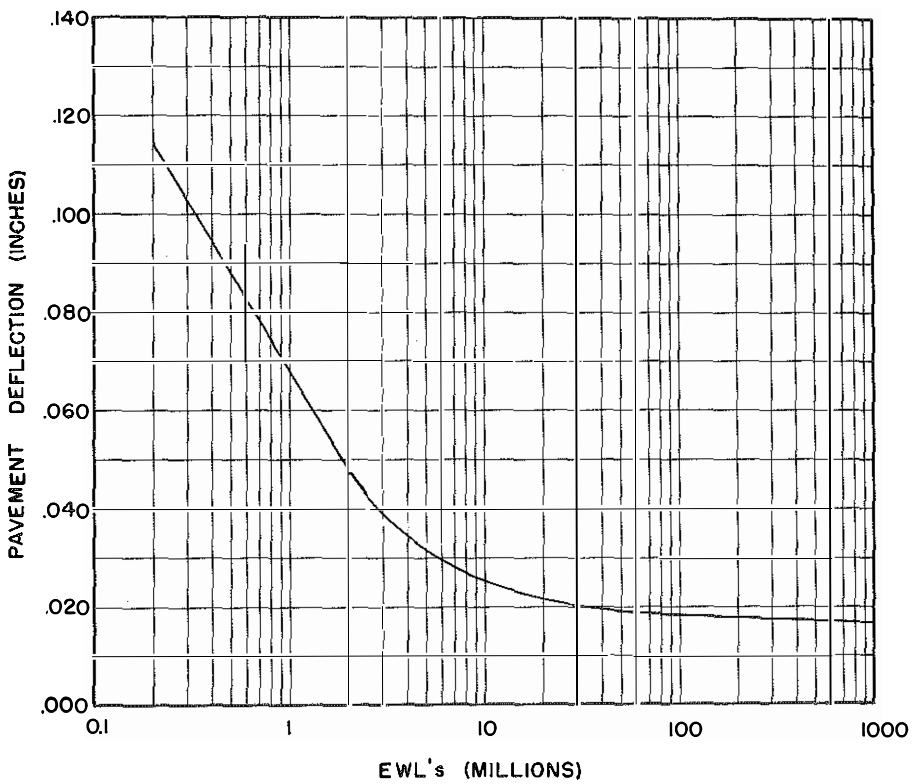

Fig. 14: Pavement Deflections as Obtained in Fig. 13, Plotted according to the Logarithm of the Mid-Point Value of the Respective EWL Groups.

separating satisfactory and unsatisfactory pavements implies a maximum deflection that can be tolerated by pavements in each traffic group. Deflection values were subsequently interpolated from this curve and plotted semi-logarithmically against the mid-points of the corresponding EWL group. Thus, Fig. 14 relates permissable deflections with EWL's. Independently of this apparent relationship, deflections taken in the spring of 1958 were plotted against the corresponding thicknesses of pavements that were adjudged satisfactory (Fig. 15). Spring measurements were used here in order to eliminate seasonal influences, and only satisfactory pavements were used in order to eliminate exaggerated deflections due to failed or weakened pavements. Here, also, a best-fitting curve was drawn, and a relationship between deflections and thicknesses appears to exist. Assuming these two relationships to be valid, to the extent that whatever hidden variables may be involved are either of minor influence or else vary only slightly, thicknesses and EWL's corresponding to the same limiting deflections were interpolated from Figs. 14 and 15 and were plotted as shown in Fig. 16.

According to Fig. 16, pavement thicknesses should be increased in proportion to the logarithm of the EWL's. This relationship appears to have been derived more-or-less independently of any parameter describing subgrade support. However, it is rather evident, since each pavement involved in the derivation was originally designed on the basis of a subgrade support parameter, that Fig. 15 must reflect a modal or prevailing subgrade CBR. Otherwise, the curve could not have been drawn. Therefore, while the relationship between thickness and log EWL may be of a general nature, the plot itself would be

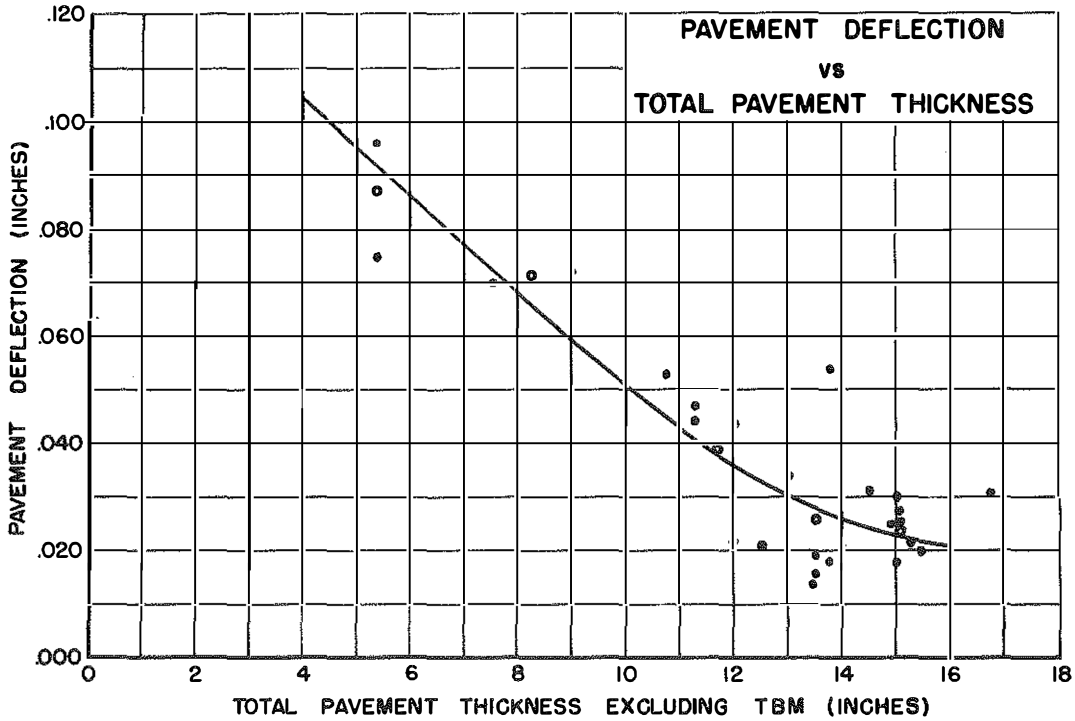

Fig. 15: Pavement Deflections Representing only Satisfactory Pavements Plotted According to Corresponding Pavement Thicknesses. The curve, as drawn, implies that deflections of equal or lesser magnitude would be within safe limits.

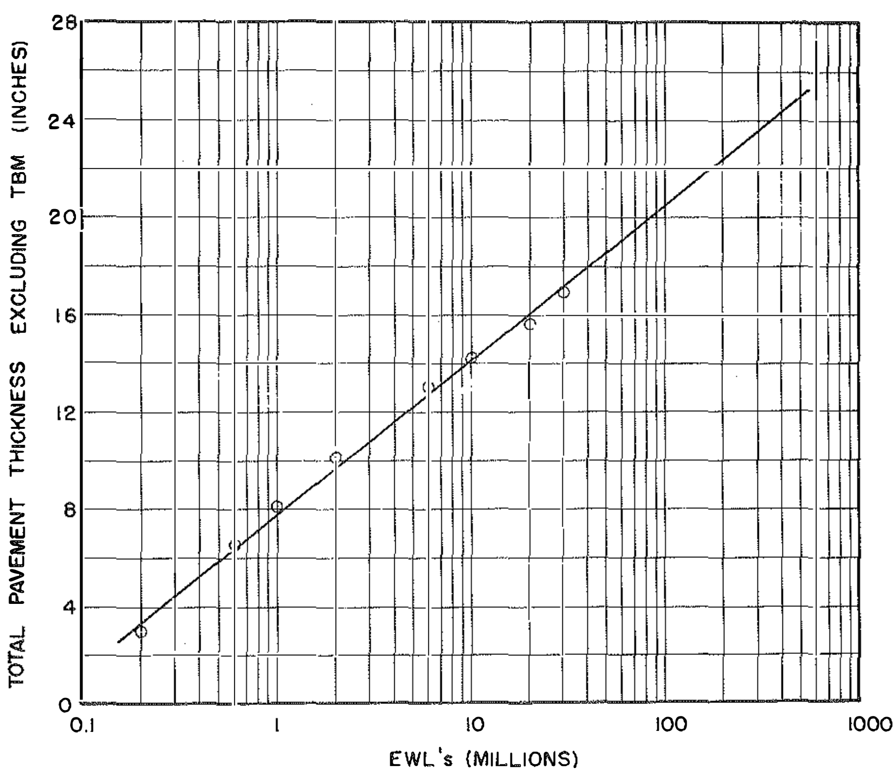

Fig. 16: Plot of Thicknesses and EWL's Interpolated from Figs. 14 and 15 for Corresponding Deflections.

significant only with respect to a particular CBR value which, in this case, should be very close to the average or median value of the group of roads involved or of the entire series.

To test the logic employed here, a cursory analysis of the frequency and distribution of project median CBR's was made; and itwas found that 90% of the CBR values from all data available fell within the range of 3 to 11. Within this range, the arithmetic mean was 7.1, and the average deviation was only 1.7. Thus, the assumption of a strong central tendency in CBR's seemed proper.

Taking 7.1 as the value most likely associated with Fig. 16, thicknesses for each of the EWL groups were interpolated from Fig. 16 and replotted at CBR 7.1 on the original design chart as shown in Fig. 17. Here the points tend to favor somewhat greater thicknesses than were required by the original curves. However, considering the fact that these points were derived on the basis of satisfactory pavements only (Fig. ~~14~~ 15), the points would naturally reflect safe design thicknesses but not necessarily the minimum design thicknesses. In any case, the derivation of these points provides a rather unique independent check upon the original curves as well as the revisions previously indicated in Fig. 6.

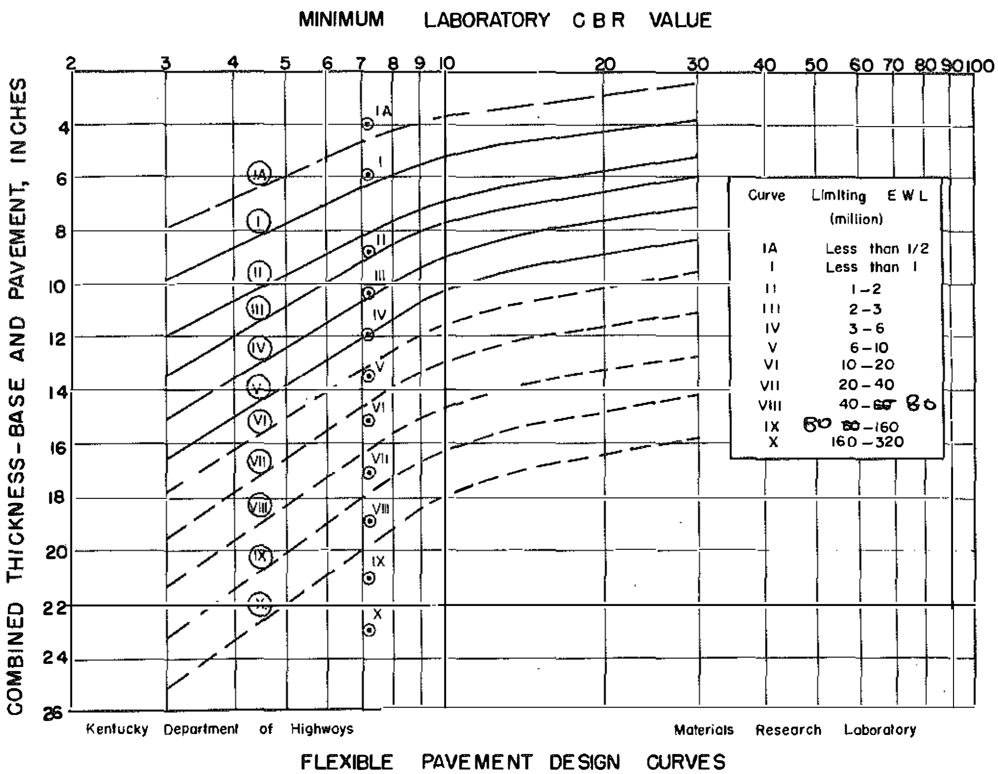

Fig. 17: Present Design Chart Showing Thicknesses for each Median Point of the Traffic Volume Groups, Taken from Fig. 16, Plotted at an Average CBR of 7.1.

Pavement Openings

In order to investigate the extent to which rutting, evident at the surface, penetrated the different layers of the pavement, eight locations on seven projects were opened to expose a cross-section to full view. An eighth pavement not originally scheduled for study was opened (in Bullitt County) in order to examine the performance of a different type of granular base material with regard to subgrade infiltration. The Bullitt County base was dense-graded aggregate (DGA).

To open the pavements, a pavement saw with an 18-in. diamond blade was used. An opening approximately 30 in. wide was made across the full width of a traffic lane. The saw was used to cut through the top layers of the pavement while the granular base materials were carefully removed by hand so that the layers could be separated and studied. Samples were obtained from the bituminous layers and returned to the laboratory for density determinations (by weighing in air and in water). These samples were taken from the wheel tracks as well as from between the wheel tracks. In-place density tests were made on the different layers of granular base by the calibrated sand method. Subgrade densities were obtained by both the rubber balloon method and the sand method (See Table 6, Appendix). Sufficient measurements were made so that the extent of rutting in most of the pavement components could be noted (See section on Rutting).

Disturbed samples from the layers of granular base and from the subgrade were returned to the laboratory for other testing, the results of which are presented in Table 7 of the Appendix. It may be noted that no significant difference in density occurred between samples taken from the wheel tracks and those taken between the wheel tracks.

This was particularly true of the surface and binder courses but less so of the lower portions of the pavement.

It was observed that much subgrade material had penetrated the WBM base courses as much as 10 inches in some places (See Figs. 18 and 19.) This indicates that the insulation or subbase courses normally used in waterbound base construction in Kentucky has not performed properly and is not fulfilling its intended function, which is to protect the WBM courses from infiltration of soil and subgrade material. Observations made in this investigation indicate that soil in the WBM courses is a result of improper rolling during construction or as a result of traffic action. In those instances where the penetration of soil was rather uniform across the section, infiltration appears to have been caused by construction rolling while the subgrade was wet. In other instances, greater penetration within the wheel tracks indicates that the clay or soil was forced up by traffic. Naturally, some loss of strength of the affected WBM courses would be expected; however, the degree of this loss and its equivalent in terms of reduced pavement thickness could not be determined.

Fig. 18: Photograph Showing Exposed Cross Section of a Rutted Pavement.

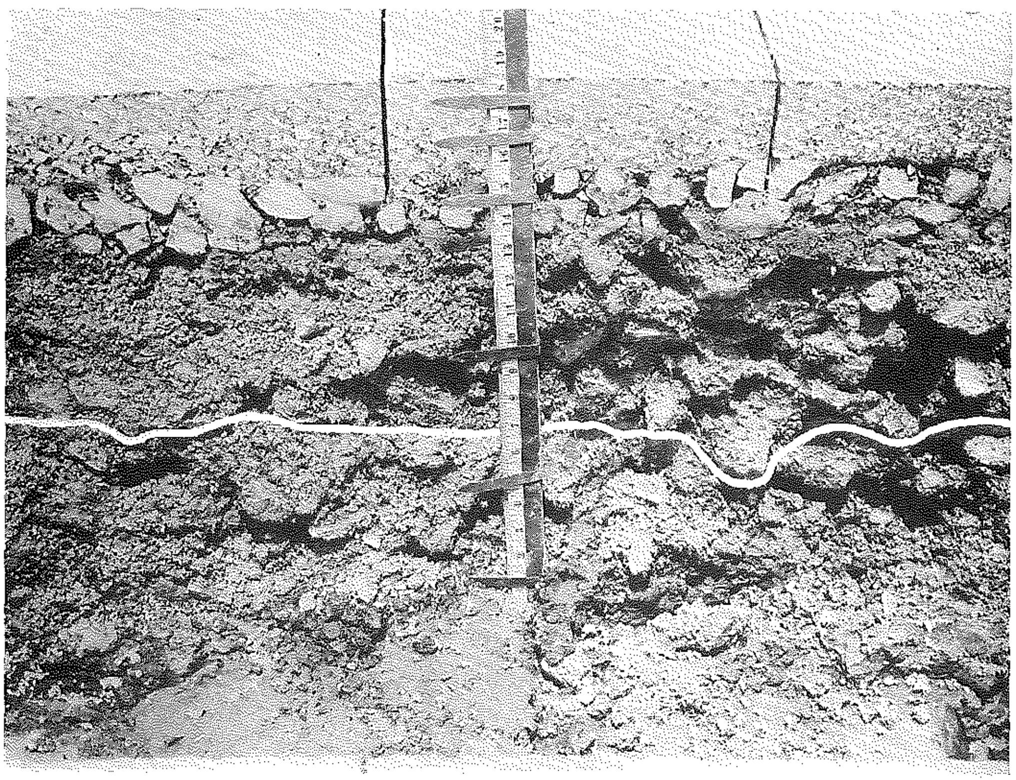

Fig. 19: Photograph Showing Exposed Cross Section of a Rutted Pavement. Markers indicate the thickness of pavement layers. Demarcation line shows the height to which subgrade soil had intruded into the WBM base.

SUMMARY

This re-evaluation of the Kentucky flexible pavement design criterion has emphasized some recognized shortcomings of pavement design systems in general and has further clarified some opinions concerning needed revisions in the present flexible pavement design.

Traffic evaluation based upon summations of equivalent-wheel loads does take into account both volumes and weights of traffic. The projected service-life of a flexible pavement designed by this method is dependent upon the accuracy of the traffic projections. The original 10-yr. basis of predicting traffic has been revised to a 20-yr. basis, and the report indicates that the 20-yr. traffic projections may be reasonably valid. The average value for each volume system analyzed is close to the projected traffic value.

The need for an adequate method of rating pavement performance is recognized. The four methods used here are advocated only as being a combination that can be used. The visual rating while probably the oldest and soundest method is usually open to more criticism than some of the others. Visual ratings were the basis for selection of locations for load deflection measurements and pavement openings. Design curves for two traffic volume groups were prepared from the visual performance ratings.

The roughness measurements taken by the triaxial acceleration method, though difficult to analyze on a project basis, undoubtedly have basic significance with regard to over-all pavement adequacy. The data appear to correlate with the visual performance rating.

Load-deflection measurements were used in the analysis of adequate pavement thickness for average subgrade support on various traffic volume groups. Those points indicated a need for revision of thickness.

Pavement openings were used to examine the layered system of selected rutted pavements. The openings permitted the determination of the extent of rutting in each layer of the pavement. The majority of the pavements studied were constructed using water-bound macadam base and 7 of the 9 locations opened were constructed with layers of WBM base. Of the pavements opened, it was noted that 72 percent of rutting was confined to the layers of WBM base while 4 percent was localized in the bituminous courses. Only 24 percent of the rutting penetrated the pavement structure to the subgrade. It appears that one of the greatest shortcomings of WBM type base is its susceptibility to subgrade infiltration. Clay subgrades tend to fill the voids in the base and to lubricate the stone and cause rutting.

Clay subgrade can be forced into the base during construction by extensive rolling over a wet subgrade. Water bonding itself can provide the moisture for the subgrade softening. Where the infiltration of subgrade does not vary through the cross section and is at the same elevation in the wheel tracks as elsewhere, it appears that the infiltration occurred at the time of construction.

Traffic can pump subgrade soil into the voids of WBM. If traffic is the motivating force, the height of infiltration would normally be greater in the wheel tracks. In the majority of the locations opened the infiltration was to a uniform elevation, and it is deduced that the subgrade soil was rolled into the base by construction equipment.

Dense-graded aggregate base is less susceptibile to damage from subgrade infiltration and lubrication. Present Kentucky specifications require the moisture to be added to the stone in a plant-mix operation, thereby eliminating the possibility of over-wetting the subgrade at that time. Dense-graded aggregate type base having considerably less voids than the average waterbound macadam is a much better insulation against subgrade infiltration.

The flexible pavement design curves shown in Fig. 20 represent the combination of the data from the 1948 study, revisions to 1957, and the results of the various approaches presented in the present investigation. These curves require a somewhat greater total thickness of pavement in the lower CBR range. The curves have been extended to a CBR value of 2, primarily to emphasize the need for subgrade improvement or stabilization of soils with CBR values of less than 3. It is still recommended that soils with CBR of less than 3 not be used for subgrade. The curves have been extended to CBR 100 to permit the use of the curves for subbase or local granular materials. The thicknesses have been reduced for CBR values of over 20.

Present Departmental policies regarding the types of base materials and relative course thicknesses for the various Highway systems appear to be sound.

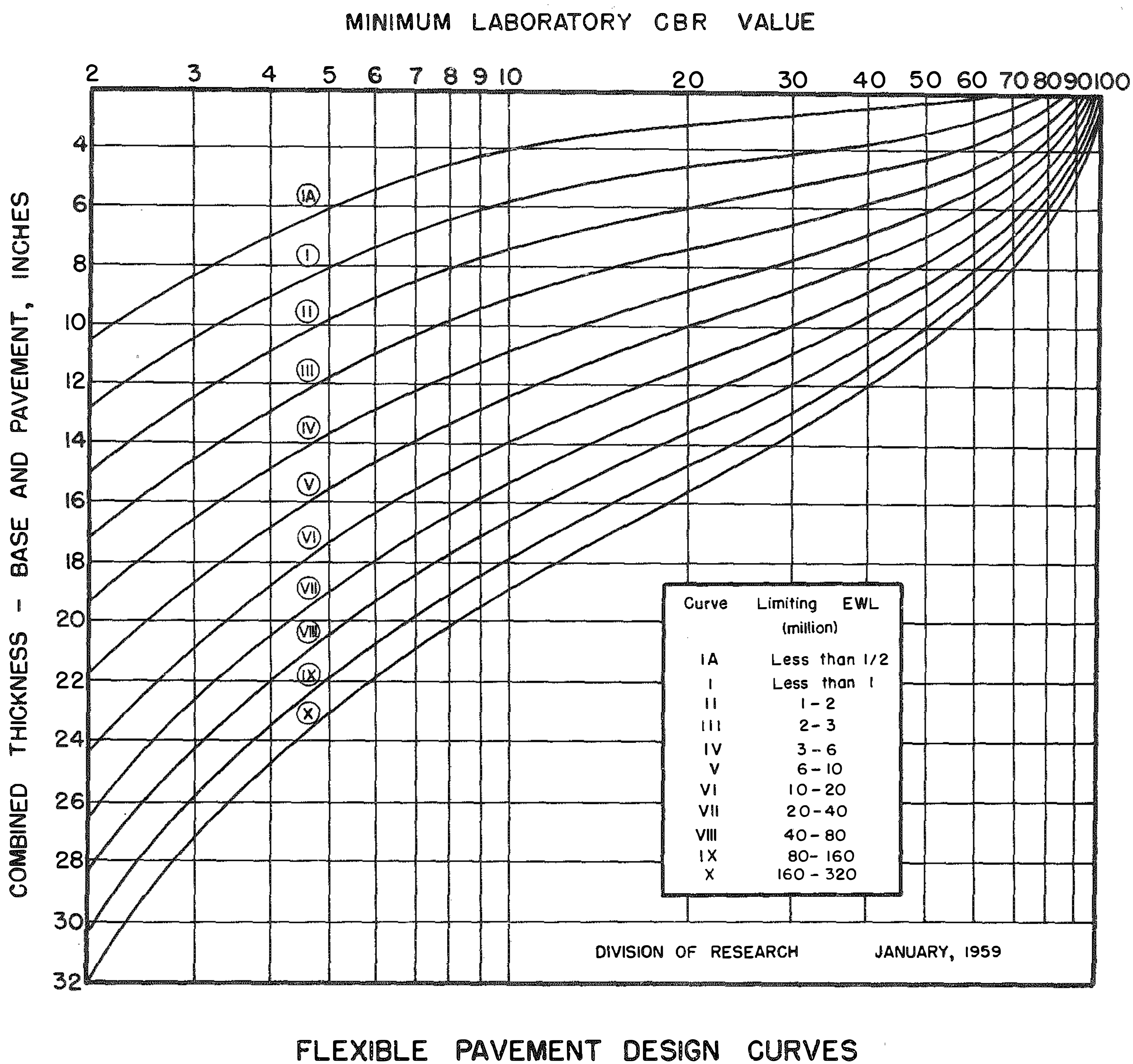

Fig. 20: Revised Flexible Pavement Design Curves.

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Mr. Loren H. Strunk, formerly Research Engineer and Head of the Bituminous Section, supervised the laboratory testing of base materials and conducted certain phases of the performance surveys.

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APPENDIX

COUNTY ——— ROAD NAME ——— ROUTE NO. ———

PROJECT LIMITS ——— PROJECT NO. ———

LOADOMETER STATION REFERENCE *State Average, 1957: Volume Group 3000-3999*

(1) Per Cent of Trucks *15.4*

(2) Axles per Truck *2.557*

(3) Average 24 hour Traffic *3640*

(4) Av. 24 hour Truck Traffic = (1) x (3) *561*

(5) Av. Yearly increase, 10 yr. period = $\frac{(4)}{2}$ *280*

(6) Av. 24 hr. Truck Traffic for 10 yr. period = (4) + (5) *841*

(7) Av. Axles per Truck for 10 yr. period = (2) + 0.05 . . . . *2.607*

(8) Total Axels in 10 years = (6) x (7) x 10 x 365 = ... *8002,578*

(1) Axle Load (tons)	(2) Total Axles	(3) % of Total Axles from L. Sta.	(4) Plus Correct.	(5) Corrected % of total Axles (3) + (4)	(6) Total Axles by Wt. Class (2) x (5)	(7) Calif. Factors	(8) EWL for two directions (6) x (7)
4½-5½	*8002,578*	*5.205*	0	*5.205*	*416,534*	1	*416,534*
5½-6½	*"*	*4.732*	0	*4.732*	*378,682*	2	*757,364*
6½-7½	*"*	*4.732*	1.25	*5.982*	*478,714*	4	*1,914,856*
7½-8½	*"*	*4.574*	0.85	*5.424*	*434,060*	8	*3,472,480*
8½-9½	*"*	*4.101*	1.50	*5.601*	*448,224*	16	*7,171,584*
9½-10½	*"*	*1.261*	0.35	*1.611*	*128,922*	32	*4,125,504*
10½-11½	*"*	*0.158*	0	*0.158*	*12,644*	64	*809,216*
11½-12½	*"*	*0*	0	*0*	*0*	128	*0*
TOTAL EWL for 10 year period (two directions)							*18,667,538*

Fig. IA: Sample Calculation for Estimating 10-Yr. EWL's.

TRAFFIC VOLUME GROUP 3000+

COUNTY ____________ ROAD NAME ____________ ROUTE NO. ______

PROJECT LIMITS ____________ PROJECT NO. ______

LOADOMETER STATION REFERENCE *State Average 1957, Volume Group 3000-3999*

(1) Per Cent of Trucks *15.4*

(2) Average Axles per Truck *2.557*

(3) Average 24 Hour Traffic *3640*

(4) Average 24 Hour Truck Traffic = (1) x (3) *561*

(5) Average 24 Hour Truck Traffic at End of 10 Year Period = 1.465 x (4) . *822*

(6) Average Axles per Truck at End of 10 Year Period = (2) + 0.19 . . . . *2.747*

(7) Total Axles in 20 Years = (5) x (6) x 365 x 20 . . . . *16,483,650*

(A) Axle Load (Tons)	(B) Total Axles (7)	(C) % of Total Axles From Load Sta.	(D) Correction	(E) Corrected % of Total Axles (C) + (D)	(F) Total Axles by Weight Class (B) x (E)	(G) EWL Factor	(H) EWL for Two Directions
$4\frac{1}{2}$-$5\frac{1}{2}$	*Same as 16,483,650*	*5.205*	0.09	*5.295*	*872,809*	1	*872,809*
$5\frac{1}{2}$-$6\frac{1}{2}$		*4.732*	0.13	*4.862*	*801,435*	2	*1,602,870*
$6\frac{1}{2}$-$7\frac{1}{2}$		*4.732*	0.27	*5.002*	*824,512*	4	*3,298,048*
$7\frac{1}{2}$-$8\frac{1}{2}$		*4.574*	0.15	*4.724*	*778,688*	8	*6,229,504*
$8\frac{1}{2}$-$9\frac{1}{2}$		*4.101*	0.11	*4.211*	*694,127*	16	*11,106,032*
$9\frac{1}{2}$-$10\frac{1}{2}$		*1.261*	0.05	*1.311*	*216,101*	32	*6,915,232*
$10\frac{1}{2}$-$11\frac{1}{2}$		*0.158*	0.00	*0.158*	*26,044*	64	*1,666,816*
$11\frac{1}{2}$-$12\frac{1}{2}$		*0*	0.00	*0*	*0*	128	*0*
TOTAL EWL for 20 year period (two directions)							*31,691,311*

Fig. IIA: Sample Calculation for Estimating 20-Yr. EWL's.

TABLE I. PROJECT DESIGNATION, LOCATION, AND DESIGN DATA FOR ALL PAVEMENTS STUDIED

COUNTY	PROJECT	FROM	TO	PROJECT LENGTH (MILES)	THICKNESS OF PAVEMENT COMPONENTS (INCHES): TBM	INSULATION OR SUBBASE	GRANULAR BASE[1]	CLASS I BASE	CLASS I BINDER	CLASS I SURFACE	TOTAL THICKNESS	EXTENT OF DESIGN	ACCEPTANCE DATE	CBR DATA: DESIGN EWL (MILLION)	DESIGN	MEAN	MEDIAN	NO. OF CBR TESTS	% OF CBR'S ABOVE DESIGN
Anderson	US 62 F 208(4)	Near Fairfield St in Lawrenceburg (Sta 24+50)	Near Tyrone Bridge (Sta 133+31.1)	2.060	—	1	7 [2]	3	1½	1½	13 3/4	Entire project	12-18-53	6.0	6	10.8	10.0	8	75
Ballard-McCracken	Ky 473 AS 63(4)	Bandana (Sta 317+97.1)	Woodville (Sta 95+49.4)	9.420	4 [3]	—	4 [3]	—	1½	1	10 1/2	Entire project	1-7-54	—	—	—	—	—	—
Barren-Hart	US 31-E F 28(5) F 7(5)	Near Bear Wallow (Sta 10+00)	Canmer (Sta 646+01)	12.069	—	1	9½	—	1½	1½	13 1/2	Entire project	10-11-50	—	—	9.0	8.1	125	—
Bell	US 119 F 151(7) F 21(5)	Wasioto (Sta 37+72.8)	Page (Sta 320+50)	5.360	2	1	8	3	1½	1½	16 3/4	Entire project	1-11-56	23.4	6	7.3	8.2	7	69
Bell	US 25-E U 322(7)	Tannery Cut-off (Sta 543+00	US 25-E east of Middlesboro (Sta 649+00)	2.044	—	1	8	—	1½	1½	12	Entire project	3-14-50	—	—	8.1	7.0	18	—
Boyd	US 60 FI 8(4) F 1(4)	SCL of Ashland (Sta 25+71.3)	Approx. 1½ miles west of Cannonsburg (Sta 301+90)	6.750	—	1	10	—	1½	1½	14	Entire project	10-11-50	—	—	5.3	4.3	62	—
Boyd	US 60 FI 8(6)	Princess (Sta 689+00)	Cannonsburg (Sta 763+09)	1.318	2	1	4	5	1½	1¼	14 3/4	Entire project	11-12-52	(8.0)[4]	6	5.0	4.0	7	29
Boyle	US 150 F 244(4)	Revisions at Clarks Run and Ball Branch (Sta 22+50) (Sta 73+00)	(Sta 35+00) (Sta 98+50)	0.682	1½	1	—	7	1½	1¼	12 1/4	Entire project	7-17-52	(6.0)	8	6.2	6.1	4	25
Breckinridge-Meade	US 60 F 523(3)	Irvington (Sta 24+83.3)	Jct with Ky 64 (Sta 553+50)	10.731	1½	2	7	—	1½	1¼	13 1/4	Entire project	9-21-53	(4.0)	6	7.7	7.5	8	75
Carter	US 60 FI 4(4)	Near ECL of Grayson (Sta 159+75)	Near Stinson Creek (Sta 294+50)	2.574	2	1	11	—	1½	1½	17	Entire project	4-15-52	(15.0)	5	4.5	4.8	26	46
Carter	US 60 FI 4(6)	Near Stinson Creek (Sta 294+50)	Near Kilgore (Sta 499+42.5)	6.404	2 2	1 3	7 7	— —	1½ 1½	1¼ 1½	12 3/4 14 3/4	294+50 to 326+00 and 347+00 to 444+00 Remainder of project	8-5-53	(15.0) (15.0)	10 10	5.2 11.4	3.5 12.5	25 17	8 59
Carter	US 60 FI 13(5)	Approx. 12 miles east of Olive Hill (Sta 4+63)	WCL of Grayson (Sta 119+08)	2.148	—	1	11	—	1½	1½	15	Entire project	5-15-52	(30.0)	7	5.5	4.5	25	28
Clay	Ky 11 SP 72(4)	Knox County Line (Sta 705+88)	Jct with Ky 80 (Sta 1182+10)	8.816	3	1	3	—	1½	1¼	9 3/4	Entire project	2-9-54	(1.0)	5	4.5	4.6	3	33
Clinton	Ky 90 S 10(5)	Jct with Ky 55 at Ida (Sta 336+00)	Wayne County Line (Sta 245+66.4)	7.164	2	1	6½	—	—	1½	11	Entire project	12-18-53	(1.0)	5	6.7	7.0	9	67
Cumberland	Ky 90 F 116(10)	Jct with Ky 100 (Sta 5+20)	Jct with Ky 61 (Sta 162+03)	3.019	2	1	3½	5	1½	1¼	14 1/4	Entire project	3-23-56	(12.0)	8	8.1	7.8	24	46
Daviess	US 231 F 125(18)	End of PCC pavement in Owensboro (Sta 122+30)	Near Chapman's Store (Sta 397+68.2)	5.097	—	1	3½	2½	1½	1½	10	Entire project	12-14-49	—	—	11.0	11.5	35	—
Daviess-Ohio	US 231 F 125(19)	Near Pleasant Ridge (Sta 737+00)	1.055 miles north of Hartford (Sta 518+26.4)	11.128	— — — — —	1 1 1 1 1	7 8 10 11 12	— — — — —	1½ 1½ 1½ 1½ 1½	1½ 1½ 1½ 1½ 1½	11 12 14 15 16	737 to 764; 210 to 447 800 to 56 99 to 160; 171 to 210 764 to 800; 447 to 518 56 to 99; 160 to 171	11-27-50	— — — — —	— — — — —	11.6 16.0 8.3 8.3 6.1	10.9 12.5 9.0 7.0 3.0	38 9 17 18 10	— — — — —
Elliott-Rowan	Ky 173 S 288(5)	Jct with Ky 32 (Sta 1+00)	Jct with Ky 7 (Sta 565+63.7)	10.691	2	1	3	—	—	1¼	7 1/4	Entire project	11-17-55	1.0	6	—	—	—	—
Fayette	Relief Road UI 538(5)	Richmond Road (Sta 10+12)	Georgetown Road (Sta 317+67)	6.139	1½ 1½	1 1	6½ 10	— —	1½ 1½	1½ 1½	11 3/4 15 1/4	10+12 to 56+00 Remainder of project	1-7-54	(40.0) (40.0)	10 8	8.6 7.4	9.0 7.0	3 10	33 30
Fayette	Southern By-Pass RS 34-304-10	US 27 south of Lexington (Sta 62+81.1)	Rosemont Garden	1.041	2	—	4	4 [6,7]	1½	1½	12 3/4	Entire project	2-5-53	—	—	—	—	—	—
Fayette-Jessamine	US 27 F 524(4) F 524(5)	Nicholasville (Sta 45+50)	Lexington (Sta 476+24.4)	8.157	— — —	— — —	10 11 12	— — —	1½ 1½ 1½	1½ 1½ 1½	13 14 15	45+50 to 325+84.7 441+00 to 476+00 326+00 to 369+00 369+00 to 441+00	11-23-49	(15.0) (35.0) (35.0)	7 7 7	8.1 8.3 6.9	8.1 8.5 7.0	62 8 14	63 75 50
	Resurfacing									1 [5]		Entire project	9-2-54						
Fayette-Madison	US 25 FI 124(4)	Clays Ferry Bridge approaches(Sta 15+00)	(Sta 101+00) Fayette County side	1.004	—	1	—	7½	1½	1½	11 1/2	Entire project	12-12-46	—	—	—	—	—	—
	Resurfacing									1½		Madison County only	6-1-57						
Franklin-Woodford	US 421 F 326(22)	Near Midway (Sta 602+32)	Near Moore's Point (Sta 1024+00)	7.985	2 2	1½ 1	9½ 10	— —	1½ 1½	1½ 1½	15 15	Woodford County Franklin County	8-16-51	(20.0) (15.0)	7 6	6.7 6.1	5.9 5.7	53 31	36 42
	Resurfacing									1 [5]		Entire project	1-11-57						
Garrard	US 27 S 366(2)	NCL of Lancaster (Sta 142+00)	Near Camp Dick Robinson (Sta 356+50)	3.502	1½	1	10	—	1½	1½	15 1/4	Entire project	9-21-53	(20.0)	7	7.2	8.3	6	57

1. All granular bases are WBM unless otherwise noted.
2. One course of vibrated WBM.
3. Bank gravel.
4. Values in parenthesis estimated from pavement thickness and design CBR.
5. Rock asphalt.
6. CaCl treated.
7. DGA.

TABLE I. (Continued)

COUNTY	PROJECT	FROM	TO	PROJECT LENGTH (MILES)	TBM	INSULATION OR SUBBASE	GRANULAR BASE	CLASS I BASE	CLASS I BINDER	CLASS I SURFACE	TOTAL THICKNESS	EXTENT OF DESIGN	ACCEPTANCE DATE	DESIGN EWL (MILLION)	DESIGN	MEAN	MEDIAN	NO. OF CBR TESTS	% OF CBR'S ABOVE DESIGN
					THICKNESS OF PAVEMENT COMPONENTS (INCHES)									CBR DATA					
Garrard	US 27 F 525(4)	2.5 miles south of Camp Nelson (Sta 136+50)	1.5 miles south of Camp Dick Robinson (Sta 406+00)	5.104	— —	1 1	8 11	— —	1½ 1½	1½ 1½	12 15	299+00 to 361+00 Remainder of project	11-23-49	— —	— —	11.0 5.7	10.8 4.5	12 34	— —
Garrard	Resurfacing US 27 F 525(5)	1.0 mile north of Lancaster (Sta 174+50)	Toward Nicholasville (Sta 383+26)	1.837	—	1	7½	2½	1½	1 5. 1½	13 3/4	Entire project Entire project	10-29-56 12-2-55	13.0	10	—	—	—	—
Garrard	US 27 F 525(3)	Camp Nelson (Sta 10+00)	Toward Lancaster (Sta 136+50)	2.427	—	1	10	—	1½	1½	14	Entire project	7-12-48	—	—	—	—	—	—
Graves	Resurfacing US 45 F 146(19)	WCL of Mayfield (Sta 10+00)	1.5 miles north of Wingo (Sta 386+00)	7.075	—	1	8	3	1½	1 5. 1½	14 3/4	Entire project Entire project	10-29-56 9-12-56	35.5	10	7.6	6.6	56	20
Grayson	Ky 54 S 462(4)	NCL of Leitchfield (Sta 56+20.2)	Jct with Ky 105 (Sta 593+50)	10.748	2½ 6.	—	3½	—	—	1½	7 1/2	Entire project	12-18-53	(0.5)	6	6.9	5.5	6	50
Harrison	US 27 F 189(5)	Bridge at Cynthiana (Sta 7+95)	Near Lear (Sta 214+00)	4.094	1½	1	11	—	1½	1½	16 1/2	Entire project	4-16-51	(15.0)	5	7.2	5.7	43	49
Harrison	Resurfacing US 27 F 189(6)	Near Lear (Sta 207+00)	Near Bourbon County Line (Sta 286+40.4)	1.503	1½	1	7	—	1½	1½ 1½	12 1/4	All but north 2.2 miles Entire project	11-22-57 12-4-52	(5.0)	7	5.0	4.7	16	6
Hart	Resurfacing US 31-W FI 169(12)	1.75 miles north of Munfordville (Sta 1154+50.2)	Toward Elizabethtown (Sta 1312+14.0)	2.985	2	1	9½	—	1½	1½ 1½	15 1/2	All but south 1.0 mile Entire project	12-14-56 3-20-53	(35.0)	10	17.3	18.3	24	88
Henderson	US 41 F 526(9)	Near Robards (Sta 261+00)	McKinley Ave. in Henderson (Sta 750+50)	9.176	—	1	7	—	1½	1½	11	Entire project	2-24-50	(30.0)	—	14.5	14.1	93	—
Henderson-Webster	US 41 F 526(12)	Main St. in Sebree (Sta 503+25)	Near Robards (Sta 261+00)	7.593	1½	1	10	—	1½	1½	15 1/4	Entire project	9-9-53	(30.0)	8	10.4	10.1	77	70
Henry	US 421 S 552(1) SG 552(2)	1.2 miles south of Campbellsburg (Sta 337+81)	600' north of L&N RR (Sta 400+00)	1.177	—	2	7	—	1½	1½	11 3/4	Entire project	11-20-56	2.5	5	—	—	—	—
Henry	US 421 F 536(3)	Near NCL of New Castle (Sta 57+50)	1.2 miles south of Campbellsburg (Sta 337+81)	5.308	—	1	7½	—	1½	1½	11 1/4	Entire project	12-2-55	2.5	9	4.8	4.8	51	2
Hopkins	US 41 F 526(6)	Hanson (Sta 364+75)	Slaughters (Sta 622+58.2)	4.731	3	1	8	—	1½	1½	15	Entire project	12-1-48	(18.0)	10	13.9	15.3	50	66
Hopkins	US 41 F 526(7)	Madisonville (Sta 83+00)	Hanson (Sta 364+75)	5.264	— —	1 1	8 11	— —	1½ 1½	1½ 1½	12 15	130 to 280; 360 to 365 Remainder of project	8-4-50	(35.0) (35.0)	20 5	10.0 9.8	9.2 10.3	29 27	10 4
Jefferson	Inner Belt Line U 528(2)	Bardstown Road (Sta 500+20)	Breckinridge Lane (Sta 626+36)	2.389	—	1	8	—	1½	1½	12	Entire project	2-1-50	(20.0)	5	14.0	13.7	26	100
Jefferson	Inner Belt Line U 528(10)	Breckinridge Lane (Sta 606+00)	US 60 (Sta 715+89.1)	2.066	—	1	8	3	1½	1	14 1/2	Entire project	7-23-53	(30.0)	7	6.9	7.8	5	60
Jefferson	Inner Belt Line U 528(12)	US 31-W (Sta 0+29.6)	Peachtree St. (Sta 105+00)	1.942	1	2	7½	6	1½	1½	19 1/4	Entire project	4-12-57	(80.0)	6	10.5	11.0	10	60
Jefferson	Inner Belt Line U 528(14)	Peachtree St. (Sta 105+00)	Poplar Level Road (Sta 307+50)	3.025	—	2	7½	6	1½	1½	18 1/4	Entire project	4-12-57	(80.0)	—	9.2	9.2	11	—
Jefferson	Inner Belt Line U 528(16)	Poplar Level Road (Sta 307+50)	Bardstown Road (Sta 450+41.5)	2.670	1	2	7½	6	1½	1½	19 1/4	Entire project	4-12-57	(80.0)	8	9.6	8.5	5	80
Johnson	US 23 77 A(23)	Jct with US 460	Lawrence County Line	5.800	—	—	—	6½	1½	1¼	9 1/4	Entire project	10-26-42	—	—	7.0	8.0	3	—
Knox	Resurfacing Ky 11 S 72(3)	US 25-E at Barbourville (Sta 91+30.0)	Clay County Line (Sta 705+97.2)	11.936	2	—	3½	—	—	1 5. 1½	6 3/4	Entire project Entire project	9-21-53 12-6-51	—	—	—	—	—	—
Laurel	Ky 229 S 150(4)	US 25 at London (Sta 29+00)	Bridge at Laurel River (Sta 107+24)	5.345	3	1	3½	—	—	1½	9	Entire project	1-2-53	(1.0)	6	—	—	—	—
Laurel	US 25 FI 29(9)	Jct with Ky 80 (Sta 78+00)	Near Pine Lodge (Sta 212+50)	2.507	—	1	9½	—	1½	1½	13 1/2	Entire project	10-19-51	(35.0)	6	9.4	6.5	12	58
Laurel	US 25 FI 517(6)	Near Pine Lodge (Sta 212+50)	Rockcastle River (Sta 613+36)	7.590	—	1	10	—	1½	1½	14	Entire project	10-19-51	(30.0)	8	8.9	6.2	70	43
Lawrence	US 23 F 78(6)	Johnson County Line	1.9 miles south of Louisa	17.083	—	—	—	5½	1½	1¼	8 1/4	Entire project	11-10-41 (N 6.1 mi) 11-26-46 (S 11.0mi)	—	—	11.0	8.5	4	—
Livingstone-Lyon	Resurfacing US 62 F 530(6)	Kuttawa (Sta 6+00)	Kentucky Dam (Sta 476+36)	8.916	— — —	— — —	4 3. 12 3. 16 3.	— — —	2 2 2	1 5. 1 1 1	7 15 19	North 6.110 miles 150+00 to 195+00 6 to 150; 275 to end Remainder of project	9-21-53 7-6-53	— — —	— — —	— 7.8 8.8	— 6.8 7.8	— 12 4	— — —
Madison	US 25 FI 299(6)	Terrill (Sta 150+00)	Berea (Sta 590+52.2)	8.312	3 3	1 1	10 11½ 3.	— —	1½ 1½	1½ 1½	17 18 1/2	Remainder of project 196+00 to 377+00	12-28-51	(40.0) (40.0)	10 5	7.1 5.1	7.7 4.5	6 9	50 50
Marshall	US 68 F 163(9)	Near Fair Dealing (Sta 526+00)	US 68 north of Benton (Sta 941+90)	7.874	2½ 3. 2½ 3.	— —	7 3. 9 3.	— —	1½ 3. 1½ 3.	1½ 3. 1½ 3.	12 1/2 14 1/2	Remainder of project 790+00 to 896+00	2-14-51	(10.0) (10.0)	12 8	10.7 8.6	10.2 7.8	45 24	48 50

TABLE I. (Continued)

					THICKNESS OF PAVEMENT COMPONENTS (INCHES)									CBR DATA					
COUNTY	PROJECT	FROM	TO	PROJECT LENGTH (MILES)	TBM	INSULATION OR SUBBASE	GRANULAR BASE	CLASS I BASE	CLASS I BINDER	CLASS I SURFACE	TOTAL THICKNESS	EXTENT OF DESIGN	ACCEPTANCE DATE	DESIGN EWL (MILLION)	DESIGN	MEAN	MEDIAN	NO. OF CBR TESTS	% OF CBR'S ABOVE DESIGN
Marshall-McCracken	US 62 F 530(8)	Kentucky Dam (Sta 547+00)	Reidland, Jct with US 68 (Sta 1275+00)	13.692	2	—	12 [3]	2½	1½	1½	19 1/2	Entire project	1-29-57	—	—	11.1	9.7	13	—
Mercer	Ky 35 F 294(2)	Harrodsburg (Sta 237+33)	Toward Danville (Sta 65+00)	3.189	1½	1½	11	—	1½	1½	16 3/4	Entire project	2-9-54	(8.0)	4	6.8	6.8	3	67
Muhlenburg	US 62 F 40(6)	1.0 mile north of Greenville(Sta 6+50)	Central City (Sta 288+67.8)	5.277	1½ 1½ 1½	1 1 1	7 10 13	— — —	1½ 1½ 1½	1½ 1½ 1½	12 1/4 15 1/4 18 1/4	156 to 239; 253 to 271 Remainder of project 31 to 69; 85 to 120	8-25-53	— — —	— — —	10.2 8.0 5.2	8.8 7.5 3.8	19 19 16	— — —
Nelson	US 150 F 222(4)	1.0 mile east of Bardstown (Sta 29+00)	Eastward (Sta 365+50)	6.349	1½	1	9½	—	1½	1	14 1/2	Entire project	10-21-52	—	—	5.5	5.5	9	—
Nicholas	US 68 F 234(9)	2.0 miles north of Bourbon County Line (Sta 77+00)	Toward Maysville (Sta 396+00)	4.454	2 2	1 1	7 10	— —	1½ 1½	1½ 1½	13 16	346+00 to 396+00 Remainder of project	10-3-51	2.8 1.2	4 4	5.0 6.0	5.2 6.0	11 2	54 50
Ohio	Resurfacing Ky 69 SP 92-224	0.2 mile west of Jct with US 231 (Sta 16+32)	Jct with Ky 85 (Sta 376+75)	6.692	2	—	6 [7]	—	—	1 [5] 1½	9 1/2	Entire project Entire project	10-11-56 12-21-54	—	—	—	—	—	—
Ohio	Ky 69 S 473(2)	Hartford (Sta 0+00)	Sulphur Springs (Sta 589+00)	11.132	3½	1	6	—	1½	1	12	Entire project	10-20-53	(4.0)	8	—	—	—	—
Pulaski	Ky 80 SP 100-235(6)	Nancy (Sta 34+00)	Somerset (Sta 311+77)	5.261	2	—	7	—	—	2	11	Entire project	11-28-51	(10.0)	8	7.9	8.1	20	50
Pulaski	US 27 F 502(4)	Jct with Ky 80 (Sta 97+77.8)	Near Norwood (Sta 321+50)	3.942	—	1	7½	3	1½	1	14	Entire project	3-16-56	18.8	8	8.5	8.0	15	60
Pulaski	US 27 U 110(4)	US 27 south of Somerset (Sta 8+79)	Jct with Ky 80 (Sta 97+30.4)	1.732	1½	1	11	—	1½	1½	16 1/2	Entire project	8-7-51	(18.8)	5	5.4	4.4	7	29
Rockcastle	US 25 FI 70(6)	Bridge in Livingstone (Sta 753+50)	6.0 miles east of Mt. Vernon (Sta 909+62.3)	2.953	—	3	11	—	1½	1½	16 3/4	Entire project	9-20-54	(80.0)	11	5.6	5.3	28	11
Rockcastle	US 25 FI 88(6)	Mt. Vernon (Sta 43+23.1)	Roundstone (Sta 376+00)	6.226	—	2	12	—	1½	1½	17	Entire project	1-4-50	(40.0)	6	6.8	6.2	67	64
Rockcastle	US 25 FI 517(7)	Rockcastle River (Sta 504+88.8)	Livingstone (Sta 753+50)	4.716	—	1	16	—	1½	1½	20	Entire project	12-6-51	(30.0)	5	4.8	3.9	49	31
Rowan	US 60 FI 3(8)	0.8 mile east of Bluestone (Sta 1+50)	Licking River (Sta 718+60)	3.308	— —	1 1	8 11	— —	1½ 1½	1½ 1½	12 15	1+50 to 807+00 807+00 to 718+60	2-1-50	(25.0) (25.0)	16 2	10.0 8.1	8.0 7.4	7 16	14 100
Warren	US 31-W FI 113(5) FI 16(2)	ECL of Bowling Green (Sta 1014+00)	Jct with US 68 (Sta 1305+00)	5.588	—	1	7½	3	1½	1½	14 1/4	Entire project	3-16-55	(50.0)	10	8.9	9.1	89	38
Webster	US 41 F 526(10)	Slaughters (Sta 0+00)	5.0 miles south of Sebree (Sta 250+00)	4.739	3	1	8	—	1½	1½	15	Entire project	11-28-51	(18.0)	10	17.2	16.4	49	86
Webster	US 41 F 526(13)	5.0 miles south of Sebree (Sta 250+00)	Sebree (Sta 503+25)	4.750	2 2	1 1	7 11	— —	1½ 1½	1½ 1½	12 3/4 16 3/4	250+00 to 335+00 Remainder of project	3-20-53	(18.0) (18.0)	20 5	11.1 11.8	11.5 7.5	11 30	0 70
Whitley	US 25-W FI 23(16)	5.0 miles south of Williamsburg (Sta 357+50)	4.0 miles north of Jellico, Tenn. (Sta 191+00)	3.015	— —	1 1	7 11	— —	1½ 1½	1½ 1½	11 15	248 to 270; 336 to 357 Remainder of project	12-1-48	— —	— —	— —	— —	— —	— —

TABLE 2. SUMMARY OF ACTUAL TRAFFIC AS DETERMINED BY TRAFFIC COUNTS

COUNTY	PROJECT	AVERAGE DAILY TRAFFIC (VEHICLES PER DAY) 1948	1949	1950	1951	1952	1953	1954	1955	1956	1957	DESIGN EWL	TOTAL ACTUAL EWL'S THROUGH 1957	CHRONOLOGICAL AGE IN YEARS	TRAFFIC AGE IN YEARS	REMARKS
Anderson	F 208(4)						1800	1900	2000	2050	2100	6,000,000	2,935,819	5	4.9	
Ballard-	AS 63(4)							650	665	670	675	—	171,357	4	—	
McCracken																
Barren-Hart	F 28(5)			1000	1100	1150	1250	1275	1200	1300	1400	—	2,277,801	8	—	
	F 7(5)															
Bell	F 151(7)									2500	2500	23,400,000	2,162,165	2	1.0	
	F 21(5)															
Bell	U 322(7)			4000								—	—	8	—	
Boyd	FI 8(4)			2300	2500	2650	3000	3750	4500	3200	3900	—	3,087,991	8	—	
	F 1(4)															
Boyd	FI 8(6)					2050	2091	2700	3200	3100	3040	(8,000,000)[1]	1,909,637	6	2.4	
Boyle	F 244(4)					3750	3900	3950	4000	3800	3600	(6,000,000)	7,523,949	6	12.5	
Breckinridge-	F 523(3)						1500	1550	1600	1675	1750	(4,000,000)	2,230,655	5	5.6	
Meade																
Carter	FI 4(4)					1670	2200	2750	3300	3450	3600	(15,000,000)	2,083,640	6	1.4	
Carter	FI 4(6)						1785	2293	2800	2950	3100	(15,000,000)	1,669,657	5	1.1	
Carter	FI 13(5)					1662	1780	2115	2450	2575	2700	(30,000,000)	1,593,981	6	0.5	
Olay	SP 72(4)							695	700	705	710	(1,000,000)	743,629	4	7.4	
Clinton	S 10(5)						550	625	700	775	850	(1,000,000)	205,654	5	2.3	
Cumberland	F 116(10)									1000	1050	(12,000,000)	874,189	2	8.7	
Daviess	F 125(18)		1650	1900	2600	2850	3000	3050	2800	3900	4500	—	10,407,664	9	—	
Daviess-Ohio	F 125(19)			1500	1700	1850	1915	1970	2000	2050	2100	—	4,579,400	8	—	
Elliott-Rowan	S 288(5)								620	710	800	1,000,000	138,004	3	1.3	
Fayette	UI 538(5)							3200	4005	4468	5500	(40,000,000)	9,513,732	4	2.4	10+12 to 56+00
								5000	6076	7878	11749	(40,000,000)	23,375,090	4	5.8	Remainder of project
Fayette	RS 34-304-10												—	4	—	
Fayette-	F 524(4)		3000	3200	3800	4300	4500	5000	6000	6500	7000	(15,000,000)	12,881,449	9	8.6	Jessamine County
Jessamine	F 524(5)		3000	3200	3800	4300	4500	5000	6000	6500	7000	(35,000,000)	12,881,449[2]	9[2]	—	Remainder of project
													4,302,095[2]	5[2]	2.9[2]	Jessamine County
													4,302,095[2]	5[2]	--	Remainder of project
Fayette-	FI 124(4)	2600	3245	3678	4145	3943	4020	5200	5650	5500	5400	—	30,065,608	12[2]	—	
Madison														11[2]	—	Madison County only
Franklin-	F 326(22)				2200	3200	3350	3450	3550	4150	4235	(20,000,000)	20,984,190	7	10.5	Woodford County
Woodford					2200	3200	3350	3450	3550	4150	4235	(15,000,000)	20,984,190[2]	7[2]	14.0	Franklin County
													13,416,528[2]	5[2]	6.7[2]	Woodford County
													13,416,528[2]	5[2]	8.9[2]	Franklin County
Garrard	S 366(2)						3000	3100	3200	3400	3600	(20,000,000)	5,283,036	5	2.6	
Garrard	F 525(4)		2100	2300	2000	3000	3500	3650	3800	3900	3950	—	8,214,869[2]	9[2]	—	
													5,009,095[2]	7[2]	—	
Garrard	F 525(5)								3200	3300	3400	13,000,000	3,503,837	3	2.7	
Garrerd	F 525(3)	500	2900	3000	3100	3000	3200	3350	3500	3650	3800	—	8,644,529[2]	10[2]	—	
													5,597,578[2]	8[2]	—	
Graves	F 146(19)									4300	4350	35,500,000	5,916,077	2	1.7	
Grayson	S 462(4)						300	310	320	360	400	(500,000)	138,911	5	2.8	
Harrison	F 189(5)				1800	4000	4100	4150	4000	4100	4200	(15,000,000)	4,214,916[2]	7[2]	2.8[2]	
													3,070,214[2]	6[2]	2.0[2]	All but north 2.2 miles
Harrison	F 189(6)					2225	2290	2320	1550	2600	2800	(5,000,000)	2,338,362[2]	6[2]	4.7[2]	
													1,256,670[2]	4[2]	2.5[2]	All but south 1.0 mile
Hart	FI 169(12)						5800	5850	5100	5250	5400	(35,000,000)	17,388,184	5	5.0	
Henderson	F 526(9)			700	1100	2300	3500	3750	4000	4750	5100	(30,000,000)	15,434,726	8	—	
Henderson-	F 526(12)						2950	3300	3650	4300	4650	(30,000,000)	11,955,706	5	—	
Webster																
Henry	S 552(1)									1225	1250	2,500,000	1,055,558	2	0.4	
	SG 552(2)															
Henry	F 536(3)								1200	1225	1250	2,500,000	1,258,611	3	5.0	
Hopkins	F 526(6)	350	75	453	775	2776	3000	3350	3850	4050	4200	(18,000,000)	14,741,344	10	—	
Hopkins	F 526(7)			677	1750	3400	3700	4000	4250	4750	5300	(35,000,000)	16,839,804	8	—	
Jefferson	U 528(2)									16250		(20,000,000)	—	8	—	
Jefferson	U 528(10)									9820		(30,000,000)	—	5	—	
Jefferson	U 528(12)									18363		(80,000,000)	—	2	--	
Jefferson	U 528(14)									23469	30029	(80,000,000)	23,479,023	2	2.9	
Jefferson	U 528(16)									23076		(80,000,000)	—	2	—	

1. Values in parenthesis estimated from pavement thickness and design CBR.
2. Up to time of resurfacing.

TABLE 2. (Continued)

COUNTY	PROJECT	AVERAGE DAILY TRAFFIC (VEHICLES PER DAY) 1948	1949	1950	1951	1952	1953	1954	1955	1956	1957	DESIGN EWL	TOTAL ACTUAL EWL'S THROUGH 1957	CHRONOLOGICAL AGE IN YEARS	TRAFFIC AGE IN YEARS	REMARKS
Johnson	77 A(23)				1500	1500	1600	1750	2230	2100	2125	—	--	—	—	
Knox	S 72(3)				475	500	510	550	600	750	800	--	1,107,629	7	--	
Laurel	S 150(4)						400	700	900	1100	1200	(1,000,000)	1,112,567	5	11.1	
Laurel	FI 29(9)				3900	5200	6200	7000	7500	7600	7700	(35,000,000)	24,600,076	7	7.0	
Laurel	FI 517(6)				2500	3050	3500	4700	5200	5500	5700	(30,000,000)	16,605,879	7	5.5	
Lawrence	F 78(6)				1500	1500	1600	1750	2230	2100	2125		--	--	--	
Livingstone-Lyon	F 530(6)						1750	2150	2450	3600	4000	—	6,028,322	5	—	
Madison	FI 299(6)				4410	3300	3350	4500	5100	5300	5450	(40,000,000)	16,851,911	7	4.2	
Marshall	F 163(9)				1600	1800	1830	1850	1880	1900	1925	(10,000,000)	3,086,579	7	3.1	
Marshall-McCracken	F 530(8)										3000	—	—	2	—	
Mercer	F 294(2)							2800	2900	3000	3094	(8,000,000)	3,603,824	4	4.5	
Muhlenburg	F 40(6)						3000	3100	3250	3400	3500	—	5,828,966	5	—	
Nelson	F 222(4)					1900	1950	2000	2100	2150	2000	—	3,573,263	6	—	
Nicholas	F 234(9)				800	1000	1275	1425	1600	2300	2500	1,192,060	3,068,187	7	—	346+00 to 396+00
					800	1000	1275	1425	1600	2300	2500	2,781,650	3,068,187	7	11.0	Remainder of project
													923,064[2]	5[2]	3.3[2]	346+00 to 396+00
													923,064[2]	5[2]	—	Remainder of project
Ohio	SP 92-224							575	590	625	650	—	645,572	4	—	
Ohio	S 473(2)						400	600	760	780	800	(4,000,000)	883,898	5	2.2	
Pulaski	SP 100-235(6)				2400	2700	2800	2900	3000	3100	3200	(10,000,000)	6,505,380	7	6.5	
Pulaski	F 502(4)											18,800,000	—	2	—	
Pulaski	U 110(4)					2300						(18,800,000)	—	7	—	
Rockcastle	FI 70(6)							5100	5250	5325	5375	(80,000,000)	12,173,323	4	1.5	
Rockcastle	FI 88(6)			3800	4000	4200	4300	4900	5400	5500	5550	(40,000,000)	20,443,023	8	5.1	
Rockcastle	FI 517(7)				2600	2700	3480	4500	5200	5225	5250	(30,000,000)	15,697,549	7	5.2	
Rowan	FI 3(8)			1500	1600	1575	1400	2000	2200	2325	2460	(25,000,000)	1,780,638	8	—	
Warren	FI 113(5) FI 16(2)								9000	9100	9200	(50,000,000)	18,862,057	3	3.8	
Webster	F 526(10)				350	2287	2600	3100	3600	3800	3900	(18,000,000)	12,534,236	7	—	
Webster	F 526(13)						2700	3125	3650	3825	4000	(18,000,000)	11,153,720	5	6.2	
Whitley	FI 23(16)	3100	2800	3100	3400	2900	3000	3800	4500	4650	4750	—	19,215,579	10	—	

TABLE 3. SUMMARY OF VISUAL SURVEY AND ROAD ROUGHNESS DATA

COUNTY	PROJECT	WHEEL TRACK DEFORMATION (1/16")						LANE OF ROAD ROUGHNESS	AVERAGE JERK	SPEED OF ROUGHNESS RUN (MPH)	EXTENT OF PAVEMENT DISTRESS (%)						
		W.B. OR N.B. LANE		E.B. OR S.B. LANE		AVERAGE VALUES					LONGITUDINAL CRACKING	ALLIGATOR CRACKING	SKIN PATCHING	STRUCTURAL PATCHING	TOTALS		
		OUTSIDE TRACK	INSIDE TRACK	INSIDE TRACK	OUTSIDE TRACK	ENTIRE PROJECT	LANE OF ROAD ROUGHNESS								MAJOR	MINOR	TOTAL
Anderson	F 208(4)	5.67	6.00	4.67	6.00	5.59	5.84	W.B.	.0672	55	0	0	0	0	0	0	0
Ballard-McCracken	AS 63(4)	4.00	1.25	1.00	1.25	1.88	2.63	N.B.	.0816	55	2.6	0.2	0	0.9	1.1	2.6	3.7
Barren-Hart	F 28(5) F 7(5)	3.20	2.40	2.80	3.80	3.05	2.80	N.B.	.0632	55	0	0	7.0	0	0	7.0	7.0
Bell	F 151(7) F 21(5)	1.60	2.00	4.80	1.40	2.45	1.80	N.B.	.0807	55	0	0	0	0	0	0	0
Bell	U 322(7)	11.80	7.20	5.50	5.00	7.38	9.50	N.B.	.1272	55	0	1.0	0	2.2	3.2	0	3.2
Boyd	FI 8(4) F 1(4)	4.50	1.75	2.50	2.75	2.88	3.13	W.B.	.0931	55	0.9	0	0	0.4	0.4	0.9	1.3
Boyd	FI 8(6)	3.00	2.50	3.00	3.00	2.88	2.75	W.B.	.0869	55	1.1	0	0	0	0	1.1	1.1
Boyle	F 244(4)	3.67	2.75	4.50	4.50	3.86	3.21	W.B.	.0697	55	0	0	0	0	0	0	0
Breckinridge-Meade	F 523(3)	5.43	3.71	4.14	3.57	4.21	4.57	W.B.	.0863	55	0	0	0	0.4	0.4	0	0.4
Carter	FI 4(4)	9.00	6.00	5.00	8.00	7.00	7.50	W.B.	.0909	55	0.7	0.2	0	0	0.2	0.7	0.9
Carter	FI 4(6) 12 3/4"	9.00	5.00	5.00	5.25	6.06	7.00	W.B.	.0859	55	0.1	0.6	0	0.2	0.8	0.1	0.9
	14 3/4"	2.00	2.00	6.00	3.00	3.25	2.00	W.B.	.0871	55							
Carter	FI 13(5)	5.33	9.00	5.00	8.67	7.00	6.84	E.B.	.0915	55	6.3	3.2	0	1.7	4.9	6.3	11.2
Clay	SP 72(4)	6.00	5.60	4.40	11.00	6.75	5.80	N.B.		45	0	0.6	0	19.0	19.6	0	19.6
Clinton	S 10(5)	3.00	1.40	1.80	4.80	2.75	3.30	E.B.	.0783	55	0	0	0		0	0	0
Cumberland	F 116(10)	2.75	2.00	1.25	2.75	2.19	2.00	E.B.	.0649	55	0	0	0	0	0	0	0
Daviess	F 125(18)	7.50	3.50	5.00	7.75	5.94	6.38	S.B.	.0802	55	1.5	0	0	0.3	0.3	1.5	1.8
Daviess	F 125(19) 11"	7.33	4.00	4.67	5.33	5.33	5.00	S.B.	.1205	55	0.3	0	1.2	0.2	0.2	1.5	1.7
	12"	9.00	13.00	8.00	10.00	10.00	11.00	S.B.	.1038	55	1.6	0	0	0	0	1.6	1.6
	14"	8.00	3.67	4.67	6.67	5.75	5.67	S.B.	.1084	55	1.0	0.2	0	0	0.2	1.0	1.2
	15"	7.50	6.50	6.50	9.00	7.38	7.75	S.B.	.1090	55	0	0	15.6	0	0	15.6	15.6
	16"	7.00	4.00	3.00	5.00	4.75	4.00	S.B.	.1094	55	0.5	0	0	0	0	0.5	0.5
Elliott-Rowan	S 288(5)	4.20	0.00	1.80	1.80	1.95	2.10	N.B.		40	2.4	0.9	0	1.7	2.6	2.4	5.0
Fayette	UI 538(5) 11 3/4"	8.00	8.00	8.00	9.00	8.25	8.50	S.B.		45	0	0	0	3.0	3.0	0	3.0
	15½"	9.00	9.00	7.75	10.75	9.13	9.25	S.B.		45	0	0	0	0.3	0.3	0	0.3
Fayette	RS 34-304-10	2.50	2.00	2.50	3.50	2.63	—	—	—	—	0	0	0	0	0	0	0
Fayette-Jessamine	F 524(4) 13"	5.00	2.25	5.00	1.75	3.50	3.38	S.B.	.0816	55	0	0	0	0	0	0	0
	F 524(5) 14"	3.00	2.00	6.00	2.00	3.25	4.00	S.B.	.0869	55	0	0	0	0	0	0	0
	15"	4.00	7.00	2.00	0.00	3.25	1.00	S.B.	.0805	55	0	0	0	0	0	0	0
Fayette-Madison	F 124(4)	2.50	3.00	2.40	2.80	2.70	2.60	S.B.	.0904	55	0	0	0	0	0	0	0
	(Truck Lane)			2.50	3.00												
Franklin-Woodford	F 326(22)	4.20	3.40	5.60	4.60	4.45	5.10	S.B.	.0789	55	0	0	0	0	0	0	0
Garrard	S 366(2)	3.67	4.00	6.00	6.00	4.92	3.84	N.B.	.0778	55	0	0	0	0	0	0	0
Garrard	F 525(4) 12"	—	—	—	—	—	—	N.B.	.0809	55	0	0	0	0	0	0	0
	15"	4.00	3.00	5.67	4.67	4.33	3.50	N.B.	.0969	55	0	0	0	0	0	0	0
Garrard	F 525(5)	—	—	—	—	—	—	N.B.	.0805	55	0	0	0	0	0	0	0
Garrard	F 525(3)	3.00	11.00	8.67	4.00	6.67	7.00	N.B.	.0721	55	0	0	0	0	0	0	0
Graves	F 146(19)	4.17	6.00	5.63	4.00	4.95	4.82	S.B.	.0799	55	0	0	0	0	0	0	0
Grayson	S 462(4)	4.88	3.13	10.13	12.75	7.72	11.44	E.B.	.0879	55	0	0.8	0	5.9	6.7	0	6.7
Harrison	F 189(5)	9.60	7.20	6.40	6.80	7.50	6.60	S.B.	.0821	55	0	0	0	0	0	0	0
Harrison	F 189(6)	13.00	8.00	13.00	14.00	12.00	13.50	S.B.	.0983	55	0	0	0	0	0	0	0
Hart	FI 169(12)	5.33	7.33	2.75	4.00	5.74	5.78	S.B.	.0906	55	0	0	0	0	0	0	0
	(Truck Lane)			7.00	8.00												
Henderson	F 526(9)	9.67	8.00	6.83	8.83	8.33	7.83	S.B.	.1257	55	0.1	10.0	7.7	5.2	15.2	7.8	23.0
Henderson-Webster	F 526(12)	8.00	5.40	7.20	9.20	7.45	8.20	S.B.	.0875	55	0.1	0.1	0	0	0.1	0.1	0.2
Henry	S 552(1) SG 552(2)	4.00	4.00	7.00	5.00	5.00	6.00	S.B.	.0897	55	0	0	0	0.3	0.3	0	0.3
Henry	F 536(3)	8.00	7.17	8.11	18.33	10.40	13.22	S.B.	.0936	55	0.1	0.4	0.5	6.8	7.2	0.6	7.8
Hopkins	F 526(6)	4.67	5.33	5.00	6.67	5.42	5.83	S.B.	.0877	55	2.3	10.2	6.8	4.5	14.7	9.1	23.8
Hopkins	F 526(7) 12"	5.00	4.67	3.00	5.00	4.42	4.00	S.B.	.1062	55	0.4	3.7	11.7	7.8	11.5	12.1	23.6
	15"	3.75	4.50	4.00	8.00	5.06	6.00	S.B.	.1044	55	0.1	0.7	0	0	0.7	0.1	0.8
Jefferson	U 528(2)	11.50	4.00	6.00	11.50	8.25	7.75	W.B.	.0811	55	9.3	0	0	0	0	9.3	9.3
Jefferson	U 528(10)	6.50	4.00	4.50	4.00	4.75	5.25	W.B.	.0728	55	3.2	0	0	0	0	3.2	3.2
Jefferson	U 528(12) OS Ln.	2.50	2.50	1.00	0.05	2.69	2.50	W.B.	.0526	55	0	0	0	0	0	0	0
	IS Ln.	4.50	3.50	4.00	3.00												
Jefferson	U 528(14) OS Ln.	2.00	2.00	2.00	2.00	2.53					0	0	0	0.1	0.1	0	0.1
	M. Ln.	2.00	4.00	2.67	2.00		3.00	W.B.	.0689	55							
	IS Ln.	3.67	3.33	1.67	3.00												
Jefferson	U 528(16) OS Ln.	3.00	2.00	1.50	2.00	2.56	2.50	W.B.	.0679	55	0	0	0	0	0	0	0
	IS Ln.	2.00	4.00	3.50	2.50												

TABLE 3. (Continued)

COUNTY	PROJECT	WHEEL TRACK DEFORMATION (1/16") W.B. OR N.B. LANE OUTSIDE TRACK	W.B. OR N.B. LANE INSIDE TRACK	E.B. OR S.B. LANE INSIDE TRACK	E.B. OR S.B. LANE OUTSIDE TRACK	AVERAGE VALUES ENTIRE PROJECT	AVERAGE VALUES LANE OF ROAD ROUGHNESS	LANE OF ROAD ROUGHNESS	AVERAGE JERK	SPEED OF ROUGHNESS RUN (MPH)	EXTENT OF PAVEMENT DISTRESS (%) LONGITUDINAL CRACKING	ALLIGATOR CRACKING	SKIN PATCHING	STRUCTURAL PATCHING	TOTALS MAJOR	TOTALS MINOR	TOTALS TOTAL
Johnson	77 A(23)	4.00	2.00	2.00	3.00	2.75	3.00	N.B.	.0718	55							
Knox	S 72(3)	5.50	2.25	5.50	7.00	5.06	6.25	S.B.		45	0	8.8	0	2.1	10.9	0	10.9
Laurel	S 150(4)	5.00	8.25	7.60	10.00	7.71	6.63	N.B.	.1157	55	0	10.2	0.5	8.3	18.5	0.5	19.0
Laurel	FI 29(9)	7.00	5.80	7.00	5.86	6.42	6.43	S.B.	.0802	55	0	0	0	0	0	0	0
Laurel	FI 517(6)	9.17	9.17	4.83	5.83	8.75	9.84	N.B.	.1006	55	0.2	0.5	0.4	2.4	2.9	0.6	3.5
	(Truck Lanes)	10.00	11.00	12.00	8.00												
Lawrence	F 78(6)	2.25	1.50	2.25	1.25	1.81	1.88	N.B.	.0707	55							
Livingstone-	F 530(6) 7"	5.00	2.50	6.00	1.50	3.75	3.75	W.B.	.0641	55	0	0	0	10.3	10.3	0	10.3
Lyon	15"	6.25	5.50	5.75	9.75	6.81	5.88	W.B.	.0574	55	0	0	0	3.8	3.8	0	3.8
	19"	13.00	4.50	12.50	8.50	9.63	8.75	W.B.	.0655	55	0	0	0	24.6	24.6	0	24.6
Madison	FI 299(6) 17"	7.00	9.00	7.29	7.86	7.79	7.58	S.B.	.0690	55	0.3	0	0	0.2	0.2	0.3	0.5
	18½"	6.67	7.67	7.60	11.40	8.34	9.50	S.B.	0758	55	0.1	0	0	1.3	1.3	0.1	1.4
Marshall	F 163(9) 12½"	4.80	3.80	2.63	5.00	4.06	4.30	W.B.	.0697	55	0.1	0	1.4	1.4	1.4	1.5	2.9
	14½"	3.50	1.50	4.67	2.33	3.00	2.50	W.B.	.0661	55	0	0	0	0.1	0.1	0	0.1
Marshall-McCracken	F 530(8)	2.00	1.50	2.50	2.17	2.04	1.75	W.B.	.0546	55	0	0	0	0	0	0	0
Mercer	F 294(2)	6.67	6.67	10.40	13.00	9.19	6.67	N.B.	.0806	55	0	0	0	0	0	0	0
Muhlenburg	F 40(6) 12½"	6.67	6.50	5.50	6.00	6.17	6.59	W.B.	.0866	55	0	0.5	0	0.5	1.0	0	1.0
	15¾"	4.00	4.25	5.00	7.25	5.13	4.13	W.B.	.0855	55	0	0	0	0	0	0	0
	18¾"	7.50	6.00	6.50	12.00	8.00	6.75	W.B.	.1048	55	0	0	0	0	0	0	0
Nelson	F 222(4)	5.25	2.75	4.75	5.25	4.50	5.00	E.B.	.0973	55	6.5	3.1	0	3.4	6.5	6.5	13.0
Nicholas	F 234(9) 13"	3.50	4.00	1.00	2.50	2.75	3.75	W.B.	.0649	55							
	16"	2.20	4.60	3.75	3.00	3.39	3.40	W.B.	.0599	55							
Ohio	SP 92-224	1.60	1.00	0.60	1.60	1.20	1.10	S.B.	.0923	55	0	0	0	1.2	1.2	0	1.2
Ohio	S 473(2)	2.80	0.80	1.80	2.80	2.05	2.30	S.B.	.1178	55	0	0	0.2	10.1	10.1	0.2	10.3
Pulaski	SP 100-235(6)	5.60	4.60	4.80	4.60	4.90	4.70	E.B.	.0862	55	0	0	0	0	0	0	0
Pulaski	F 502(4)	5.50	3.75	2.75	6.50	4.63	4.63	S.B.	.0740	55	0	0	0	0	0	0	0
Pulaski	U 110(4)	5.00	4.33	6.00	5.67	5.25	4.67	N.B.	.0765	55	0	0	0	1.1	1.1	0	1.1
Rockcastle	FI 70(6)	6.00	6.25	2.75	3.50	5.08	4.56	S.B.	.1365	55	0.2	0	0	0.6	0.6	0.2	0.8
	(Truck Lane)			5.00	7.00												
Rockcastle	FI 88(6)	7.00	3.40	11.60	7.60	10.93	13.80	S.B.	.1087	55	2.3	0.4	0	0	0.4	2.3	2.7
	(Truck Lane)			16.00	20.00												
Rockcastle	FI 517(7)	17.40	11.60	5.40	6.40	10.20	14.50	N.B.	.0951	55	1.0	15.3	0.6	3.6	18.9	1.6	20.5
Rowan	FI 3(8) 12"	4.00	5.00	5.00	3.67	4.42	4.50	W.B.	.0869	55	0	0	0	4.6	4.6	0	4.6
	15"	5.00	6.67	4.00	4.00	4.92	5.84	W.B.	.0881	55	0.5	0	0	4.5	4.5	0.5	5.0
Warren	FI 113(5) OS Ln.	5.00	4.50	5.25	7.50	4.19	6.38	S.B.	.0780	55	1.0	0	0	0.1	0.1	1.0	1.1
	FI 16(2) IS Ln.	2.75	2.25	3.50	2.75												
Webster	F 526(10)	7.20	4.40	6.60	9.40	6.90	8.00	S.B.	.0870	55	7.1	1.1	0.4	6.1	7.2	7.5	14.7
Webster	F 526(13) 12 3/4"	10.00	5.50	9.50	12.00	9.25	10.75	S.B.	.0953	55	1.1	3.1	0	2.5	5.6	1.1	6.7
	16 3/4"	8.67	7.67	10.33	8.67	8.84	9.50	S.B.	.0872	55	3.0	0.1	0	1.7	1.8	3.0	4.8
Whitley	FI 23(16) 11"	10.00	9.00	6.33	7.67	8.25	7.00	S.B.	.0716	55	1.1	1.1	0	0.8	1.9	1.1	3.0
	15"	9.00	7.00	4.50	8.50	7.25	6.50	S.B.	.0972	55	0.1	0	0	0.1	0.1	0.1	0.2

TABLE 4. SUMMARY OF PAVEMENT DEFLECTIONS UNDER AN 18,000 POUND SINGLE AXLE LOAD

COUNTY	PROJECT	PAVEMENT* THICKNESS (INCHES)	BENKELMAN BEAM DEFLECTIONS (INCHES) FALL, 1957 INSIDE TRACK	FALL, 1957 OUTSIDE TRACK	FALL, 1957 AVERAGE	SPRING, 1958 INSIDE TRACK	SPRING, 1958 OUTSIDE TRACK	SPRING, 1958 AVERAGE	PAVEMENT CONDITION
Anderson	F 208(4)	13 3/4	.016	.010	.013	.016	.010	.013	Satisfactory
Barren-Hart	F 28(5)	13 1/2	—	—	—	.020	.018	.019	Satisfactory
	F 7(5)	13 1/2	—	—	—	.015	.016	.016	Satisfactory
		Average				.018	.017	.017	
Boyd	FI 8(4)	14	—	—	—	.010	.014	.012	Satisfactory
	F 1(4)								
Boyd	FI 8(6)	12 3/4	.007	.009	.008	.008	.012	.010	Satisfactory
		12 3/4	.007	.009	.008	.009	.011	.010	Satisfactory
		Average	.007	.009	.008	.009	.012	.010	
Boyle	F 244(4)	10 3/4	.012	.010	.011	.007	.007	.007	Satisfactory
		10 3/4	.012	.016	.014	.010	.012	.011	Satisfactory
		Average	.012	.013	.013	.009	.010	.009	
Carter	FI 13(5)	15	—	—	—	.036	.016	.026	Satisfactory
Elliott-Rowan	S 288(5)	5 1/4	—	—	—	.080	.114	.097	Satisfactory
		5 1/4	—	—	—	.051	.098	.075	Satisfactory
		Average				.066	.106	.086	
Fayette	RS 34-304-10	10 3/4	.020	.019	.020	.028	.028	.028	Satisfactory
		10 3/4	.012	.023	.017	.019	.025	.022	Satisfactory
		Average	.016	.021	.019	.024	.026	.025	
Fayette-Madison	FI 124(4)	11 1/2	.009	.010	.010	.007	.008	.008	Satisfactory
Garrard	F 525(4)	15	—	—	—	.032	.034	.033	Unsatisfactory
Garrard	F 525(5)	13 3/4	—	—	—	.015	.016	.016	Satisfactory
		13 3/4	—	—	—	.017	.014	.015	Satisfactory
		Average				.016	.015	.016	
Garrard	F 525(3)	14	—	—	—	.021	.028	.025	Satisfactory
Garrard	S 366(2)	13 3/4	—	—	—	.015	.020	.018	Satisfactory
		13 3/4	—	—	—	.033	.023	.028	Satisfactory
		Average				.024	.022	.023	
Graves	F 146(19)	14 3/4	.008	.014	.011	.012	.017	.015	Satisfactory
		14 3/4	.031	.029	.030	.035	.037	.036	Satisfactory
		Average	.020	.021	.021	.024	.027	.026	
Grayson	S 462(4)	7 1/2	.038	.032	.035	.077	.062	.070	Satisfactory
		7 1/2	.126	.168	.147	.180	.190	.185	Unsatisfactory
		Average	.082	.100	.091	.129	.126	.128	
Hart	FI 169(12)	13 1/2	.011	.014	.013	.014	.014	.014	Satisfactory
		13 1/2	.015	.026	.021	.016	.022	.019	Satisfactory
		Average	.013	.020	.017	.015	.018	.017	
Henry	F 536(3)	11 1/4	.058	.050	.054	.044	.045	.044	Unsatisfactory
		11 1/4	.042	.040	.041	.043	.050	.047	Satisfactory
		11 1/4	.077	.060	.069	.061	.056	.059	Unsatisfactory
		Average	.059	.050	.055	.049	.050	.050	
Hopkins	F 526(7)	12	.046	.036	.041	.032	.039	.036	Unsatisfactory
		12	.024	.026	.025	.051	.049	.050	Satisfactory
		Average	.035	.031	.033	.042	.044	.043	
		15	.022	.022	.022	.038	.048	.043	Unsatisfactory
		15	.028	.036	.032	.029	.031	.030	Satisfactory
		Average	.025	.029	.027	.034	.040	.037	
Laurel	S 150(4)	6	.060	.047	.054	.072	.072	.072	Satisfactory
		6	.061	.050	.056	.136	.096	.116	Unsatisfactory
		Average	.061	.049	.055	.104	.084	.094	
Laurel	FI 29(9)	13 1/2	.008	.011	.010	.012	.015	.014	Satisfactory
		13 1/2	.025	.041	.033	.030	.034	.032	Unsatisfactory
		Average	.017	.026	.021	.021	.025	.023	
Lawrence	F 78(6)	8 1/4	—	—	—	.107	.122	.115	Unsatisfactory
		8 1/4	—	—	—	.059	.117	.088	Unsatisfactory
		8 1/4	—	—	—	.019	.022	.021	Satisfactory
		Average				.062	.087	.075	
Livingstone-Lyon	F 530(6)	15	—	—	—	.012	.024	.018	Satisfactory
		15	—	—	—	.022	.031	.026	Satisfactory
		Average				.017	.028	.022	
Madison	FI 299(6)	15 1/2	.041	.041	.041	.038	.042	.040	Unsatisfactory
		15 1/2	.017	.023	.020	.020	.019	.020	Satisfactory
		15 1/2	.020	.023	.022	.021	.026	.024	Unsatisfactory
		Average	.026	.029	.028	.026	.029	.028	
		14	.004	.006	.005	.018	.023	.021	Satisfactory

*Excluding TBM.

TABLE 4. (Continued)

COUNTY	PROJECT	PAVEMENT THICKNESS (INCHES)	BENKELMAN BEAM DEFLECTIONS (INCHES) FALL, 1957 INSIDE TRACK	FALL, 1957 OUTSIDE TRACK	FALL, 1957 AVERAGE	SPRING, 1958 INSIDE TRACK	SPRING, 1958 OUTSIDE TRACK	SPRING, 1958 AVERAGE	PAVEMENT CONDITION
Marshall	F 163(9)	10	.030	.040	.035	.042	.082	.062	Unsatisfactory
		10	.018	.024	.021	.016	.026	.021	Satisfactory
		Average	.024	.032	.028	.029	.054	.042	
		12	.024	.030	.027	.041	.054	.048	Unsatisfactory
		12	.023	.026	.025	.031	.031	.031	Satisfactory
		Average	.024	.028	.026	.036	.043	.040	
Mercer	F 294(2)	15 1/4	.041	.049	.045	.041	.044	.043	Unsatisfactory
		15 1/4	.016	.022	.019	.020	.024	.022	Satisfactory
		Average	.029	.036	.032	.031	.034	.033	
Muhlenburg	F 40(6)	10 3/4	.056	.062	.059	.062	.062	.062	Unsatisfactory
		10 3/4	.045	.032	.039	.057	.048	.053	Satisfactory
		Average	.051	.047	.049	.060	.055	.058	
		13 3/4	.025	.028	.027	.063	.044	.054	Satisfactory
		16 3/4	.030	.032	.031	.032	.030	.031	Satisfactory
Nelson	F 222(4)	13	—	—	—	.034	.034	.034	Unsatisfactory
		13	—	—	—	.044	.056	.050	Unsatisfactory
		Average				.039	.045	.042	
Nicholas	F 234(9)	11	.067	.074	.071	.056	.063	.060	Satisfactory
		14	.028	.030	.029	.030	.039	.035	Unsatisfactory
Ohio	SP 92-224	7 1/2	.074	.071	.073	.114	.124	.119	Unsatisfactory
		7 1/2	.082	.070	.076	.070	.142	.106	Satisfactory
		Average	.078	.071	.075	.092	.133	.113	
Rowan	FI 3(8)	12	.022	.016	.019	.047	.041	.044	Satisfactory
		12	.025	.011	.018	.027	.016	.022	Satisfactory
		Average	.024	.014	.019	.037	.029	.033	
		15	.015	.013	.014	.026	.025	.026	Satisfactory
Whitley	FI 23(16)	11	.024	.026	.025	.032	.029	.031	Unsatisfactory
		15	.026	.024	.025	.027	.030	.029	Satisfactory
		15	.021	.024	.023	.025	.023	.024	Unsatisfactory
		Average	.024	.024	.024	.026	.027	.027	

TABLE 5. SUMMARY OF PAVEMENT DEFLECTIONS UNDER A 32,000 POUND TANDEM AXLE LOAD

COUNTY	PROJECT	PAVEMENT* THICKNESS (INCHES)	BENKELMAN BEAM DEFLECTIONS (INCHES) INSIDE WHEEL TRACK FRONT AXLE	INSIDE WHEEL TRACK BETWEEN AXLES	INSIDE WHEEL TRACK REAR AXLE	OUTSIDE WHEEL TRACK FRONT AXLE	OUTSIDE WHEEL TRACK BETWEEN AXLES	OUTSIDE WHEEL TRACK REAR AXLE	AVERAGES FRONT AXLE	AVERAGES BETWEEN AXLES	AVERAGES REAR AXLE	PAVEMENT CONDITION
Anderson	F 208(4)	13 3/4	.013	.008	.012	.009	.006	.009	.011	.007	.011	Satisfactory
Boyle	F 244(4)	10 3/4	.008	.007	.008	.008	.008	.008	.008	.008	.008	Satisfactory
		10 3/4	.010	.010	.010	.011	.011	.011	.011	.011	.011	Satisfactory
		Average	.009	.009	.009	.010	.010	.010	.010	.010	.010	
Fayette	RS 34-304-10	10 3/4	.026	.010	.026	.026	.012	.022	.026	.011	.024	Satisfactory
		10 3/4	.013	.007	.014	.021	.010	.021	.017	.009	.018	Satisfactory
		Average	.020	.009	.020	.024	.011	.022	.022	.010	.021	
Fayette**	RS 34-304-10	10 3/4	.027	.011	.027	.026	.011	.023	.027	.011	.025	Satisfactory
		10 3/4	.017	.007	.017	.026	.012	.026	.022	.010	.022	Satisfactory
		Average	.022	.009	.022	.026	.012	.025	.024	.011	.024	
Fayette-Madison	FI 124(4)	11 1/2	.007	.005	.007	.007	.005	.007	.007	.005	.007	Satisfactory
Mercer	F 294(2)	15 1/4	.033	.018	.033	.037	.019	.037	.035	.019	.035	Unsatisfactory
		15 1/4	.016	.004	.015	.021	.008	.021	.019	.006	.018	Satisfactory
		Average	.025	.011	.024	.029	.013	.029	.027	.012	.027	

* Excluding TBM.
** 36,000 pound tandem axle load.

TABLE 6. SUMMARY OF FIELD DENSITY MEASURMENTS

	IN PLACE DENSITIES OF PAVEMENT COMPONENTS (lbs. per cu. ft.)													
COUNTY	PROJECT	SUBGRADE SOIL					GRANULAR BASE (sand method)					BITUMINOUS CONCRETE (water displacement)		
		IWT[3.]	BWT[3.]	OWT[3.]	METHOD	M. C. (pct)	MATERIAL	COURSE	IWT	BWT	OWT	IWT	BWT	OWT
Boyle	F 244(4)	110 106	111 109	--- 102	Rainhart Sand	24.1	--- ---	--- ---	--- ---	--- ---	--- ---	153	151	153
Bullitt		--- 101	--- 100	--- 100	Rainhart Sand	23.3	DGA ---	Total ---	139 ---	130 ---	138 ---	146	144	144
Daviess	F 125(18)	112 103	117 116	111 109	Rainhart Sand	23.9	WBM ---	Total ---	146 ---	128 ---	130 ---	144	145	146
Fayette	UI 538(5)	104 98	104 104	108 91	Rainhart Sand	28.0	WBM WBM	Top Bottom	138 137	147 148	--- ---	145	147	147
Laurel	FI 517(6)	--- 98	--- 115	--- 108	Rainhart Sand	19.4	WBM WBM	Top Bottom	147 ---	148 ---	144 ---	147	148	148
Madison	FI 299(6)	113 95	115 108	113 109	Rainhart Sand	19.7	WBM WBM	Top Bottom	--- 118	137 145	148 ---	148	149	148
Mercer	F 294(2)	94 83	116 99	--- ---	Rainhart Sand	16.0	WBM WBM	Top Bottom	161 137	112 130	151 ---	149	149	147
Rockcastle[1.]	FI 88(6)	--- 111	121 111	118 113	Rainhart Sand	18.7	WBM WBM	Top Bottom	126 122	126 139	98 135	148	151	149
Rockcastle[2.]	FI 88(6)	--- ---	--- ---	117 114	Rainhart Sand	28.9	WBM WBM	Middle Bottom	159 123	136 137	138 116	145	148	148

1. Truck Lane
2. Passing Lane
3. IWT indicates inside wheel track.
   BWT indicates between wheel tracks.
   OWT indicates outside wheel track.

TABLE 7. SUMMARY OF DATA FROM BASE AND SUBGRADE SAMPLES

					STD. PROCTOR		GRAIN SIZE DISTRIBUTION						
COUNTY	PROJECT	SAMPLE IDENTIFICATION	LIQUID LIMIT	PLASTICITY INDEX	MAX. DEN.	OPT. M.C.	PERCENT SAND	PERCENT SILT	PERCENT CLAY	PERCENT COLLIODS	SPECIFIC GRAVITY	MIN. LAB. CBR	HRB CLASSIFICATION
Boyle	F 244(4)	Subgrade - Inside Wheel Track	31	10	106	18	25	56	19	0	2.71	28	A-4(8)
Daviess	F 125(18)	Subgrade - Inside Wheel Track	30	12	111	17	12	64	24	9	2.76	7	A-6(9)
		WBM - Outside Wheal Track	18	3			95*	3	2	0			
Fayette	UI 538(5)	Subgrade - Outside Wheel Track	49	23	99	26	26	44	30	12	2.79	8	A-7-6(15)
		WBM - Bottom Two Courses - Outside Wheel Track	21	5			92*	4	4	1			
Laurel	FI 517(6)	Subgrade - Outside Wheel Track	34	14	110	17	23	37	40	14	2.76	3	A-6(9)
		Subgrade - Inside Wheel Track	27	9	113	14	39	39	22	6	2.64	6	A-4(8)
		WBM - Bottom Course - Between Wheel Tracke	19	5			95*	3	2	0			
		WBM - Middle Oourse - Between Wheel Tracke	16	NP			95*	3	2	1			
Madison	FI 299(6)	Subgrade - Inside Wheel Track	30	12	113	17	19	49	32	12	2.79	7	A-6(8)
		WBM - Bottom Course - Outside Wheel Track	20	6			93*	3	4	1			
Mercer	F 294(2)	Subgrade - Between Wheel Tracks	29	11	118	14	25	42	33	15	2.81	3	A-6(8)
		WBM - Bottom Course - Inside Wheel Track	14	NP			96*	2	2	1			
		WBM - Middle Course - Outside Wheel Track	16	2			93*	3	4	1			
Rockcastle (Truck Lane)	FI 88(6)	Subgrade - Outside Wheel Track	31	14	112	16	23	52	25	5	2.70	13	A-6(9)
		WBM - Bottom Course - Outside Wheel Track	21	7			92*	4	4	1			
		WBM - Middle Course - Outside Wheel Track	18	5			97*	2	1	1			
Rockcastle (Passing Lane)	FI 88(6)	Subgrade - Outside Wheel Track	31	14	113	16	27	47	26	5	2.70	—	A-6(10)
		WBM - Bottom Course - Outside Wheel Track	15	NP			98*	1	1	0			
Bullitt		Subgrade - Between Wheel Tracks	35	16	108	19	19	40	41	10	2.75	5	A-6(10)
		DGA - Outside Wheel Track	17	NP			91*	6	3	1			

* Percent larger than silt size.

Note: The data for base material was obtained on that portion passing the number 40 sieve.

TABLE 8. DISTRIBUTION OF TRUCKS WITH RESPECT TO ADT BY YEARS

LOADOMETER STATION OR TRAFFIC VOLUME GROUP	FACTORS FROM LOADOMETER STATION DATA														TYPICAL VALUES**	
	1951		1952		1953		1954		1955		1956		1957			
	PERCENT TRUCKS	AXLES PER TRUCK	PERCENT TRUCKS	AXLES PER TRUCK	PERCENT TRUCKS	AXLES PER TRUCK	PERCENT TRUCKS	AXLES PER TRUCK	PERCENT TRUCKS	AXLES PER TRUCK	PERCENT TRUCKS	AXLES PER TRUCK	PERCENT TRUCKS	AXLES PER TRUCK	PERCENT TRUCKS	AXLES PER TRUCK
4	20.6	2.372	18.8	2.405	20.7	2.486	17.9	2.482	17.4	2.477	19.8	2.581				
10	21.4	2.152	16.7	2.176	13.8	2.310	14.7	2.259	12.9	2.343	15.2	2.358				
27	14.7	2.337	14.0	2.377	12.9	2.419	16.2	2.339	14.8	2.411	15.3	2.451				
31	28.9	2.438	26.1	2.554	28.9	2.675	23.8	2.695	22.9	2.748	25.3	2.841				
40	30.6	2.610	26.1	2.650	28.2	2.681	27.3	2.673	25.4	2.699	25.3	2.827	23.6	2.995		
41	29.5	2.579	27.1	2.651	26.2	2.655	25.6	2.746	24.2	2.816	31.3	2.895				
42	24.3	2.522	21.7	2.653	24.6	2.703	20.8	2.713	20.7	2.763	24.7	2.910				
43	24.6	2.602	25.2	2.619	25.7	2.706	23.1	2.733	22.6	2.728	24.4	2.933	24.6	3.087		
44	13.9	2.188	11.7	2.272	12.0	2.295	11.3	2.368	11.4	2.333	12.3	2.418				
45	16.8	2.615	18.0	2.670	17.1	2.728	15.6	2.741	16.7	2.773	19.5	2.837				
46											17.7	3.038	22.3	3.337		
47											20.9	2.846	23.7	2.970		
48											21.4	3.040	22.1	3.245		
49											19.3	2.997	18.5	3.152		
50											16.1	2.396	15.4	2.557		
51											14.6	2.482	15.3	2.646		
52											21.5	2.747	22.9	2.889		
53											23.5	2.645	21.8	2.745		
Averages	22.5	2.442	20.5	2.503	21.0	2.566	19.6	2.575	18.9	2.609	20.5	2.736	21.0	2.962		
0-399	--	--	--	--	--	--	--	--	--	--	--	--	--	--	8.7	2.046
400-999	--	--	--	--	--	--	--	--	--	--	--	--	--	--	12.9	2.131
1000-1999	--	2.375*	--	2.415*	--	2.460*	--	2.511*	--	2.570*	23.5	2.645	21.8	2.745	14.4	2.510
2000-2999	21.0	2.395	18.7	2.475	22.2	2.630	16.8	2.612	17.4	2.477	18.2	2.787	20.2	3.056	14.8	2.864
3000-3999	26.9	2.551	23.0	2.482	19.4	2.492	19.7	2.560	18.4	2.667	20.8	2.640	15.4	2.557	27.5	2.841
4000-4999	30.6	2.610	21.7	2.653	26.4	2.692	18.5	2.526	17.3	2.523	20.3	2.736	23.7	2.970	38.8	2.744
5000-over	14.7	2.337	20.1	2.514	12.9	2.419	27.3	2.673	23.1	2.731	22.7	2.871	22.9	3.043	13.4	3.004

* Extrapolated.
** Typical Values furnished by the Division of Planning, Kentucky Department of Highways.

TABLE 9. DISTRIBUTION OF TRUCK AXLES BY WEIGHT GROUPS

YEAR	LOADOMETER STATION OR TRAFFIC VOLUME GROUP	PERCENT OF TOTAL AXLES BY WEIGHT GROUP										
		UNDER 7000	7000 TO 9000	9000 TO 11000	11000 TO 13000	13000 TO 15000	15000 TO 17000	17000 TO 19000	19000 TO 21000	21000 TO 23000	23000 TO 25000	25000 TO 27000
1951	4	65.820	10.010	4.059	3.505	6.458	7.749	2.030	0.369	—	--	—
	10	80.309	7.499	2.013	4.362	2.685	2.125	0.336	0.671	—	--	—
	27		80.707*	3.751	4.180	3.859	5.788	1.286	0.322	0.107	--	—
	31	58.118	10.906	5.092	7.214	7.355	8.062	2.546	0.707	—	--	—
	40	53.970	12.332	5.596	7.482	7.543	6.995	4.562	1.277	0.182	0.061	—
	41	57.881	14.743	4.309	5.450	8.238	6.971	2.155	0.253	--	--	—
	42	61.720	11.932	4.461	3.112	4.979	5.913	6.120	1.452	0.311	--	—
	43		67.278*	3.466	4.995	5.607	8.053	9.072	1.427	0.102	—	—
	44	75.711	10.816	2.994	3.294	2.694	3.593	0.599	0.299	—	—	—
	45		70.149*	5.597	8.582	8.023	4.851	2.052	0.746	—	—	—
	Averages		74.990*	4.134	5.218	5.744	6.010	3.076	0.752	0.070	0.006	—
	2000-2999	64.764	11.339	3.870	5.325	5.470	5.739	2.773	0.703	0.102	--	—
	3000-3999	65.751	7.387	4.385	4.281	6.609	6.442	4.138	0.853	0.311	--	—
	4000-4999	55.760	10.542	5.596	7.482	7.543	6.995	4.562	1.277	0.182	0.061	—
	5000-over	65.938	14.769	3.751	4.180	3.859	5.788	1.286	0.322	0.107	—	—
1952	4	73.323	11.151	2.838	3.339	3.172	4.174	1.836	0.167	—	--	—
	10	84.525	7.892	1.230	1.947	1.844	2.152	0.410	—	—	--	—
	27		80.835*	5.641	5.178	4.250	2.782	0.850	0.309	0.155	--	—
	31	59.974	11.254	3.581	5.243	5.243	6.777	4.859	2.685	0.384	—	—
	40	54.763	12.513	4.245	4.138	5.750	8.436	9.350	0.645	0.160	--	—
	41	58.691	14.949	5.046	5.870	6.591	5.973	1.853	0.926	—	0.102	—
	42	60.101	11.619	5.730	3.696	9.981	7.024	1.479	0.370	—	--	—
	43		65.994*	3.650	4.035	5.764	7.973	12.008	0.480	0.096	—	—
	44	72.952	10.422	2.015	3.023	5.290	2.519	3.275	—	0.504	--	—
	45		76.961*	5.065	4.085	5.392	5.392	3.105	--	—	--	—
	Averages		76.792*	3.904	4.055	5.328	5.320	3.903	0.558	0.130	0.010	—
	2000-2999	67.237	11.772	3.375	3.923	4.774	4.716	3.269	0.713	0.222	--	—
	3000-3999	69.531	7.812	3.309	3.951	4.733	5.365	4.757	0.469	0.032	0.034	—
	4000-4999	60.317	11.403	5.730	3.696	9.981	7.024	1.479	0.370	—	--	—
	5000-over	60.504	13.552	4.943	4.658	5.000	5.609	5.100	0.477	0.158	--	—
1953	4	73.983	11.251	2.234	2.234	4.122	2.991	2.813	0.186	0.186	--	—
	10	80.723	7.537	2.348	3.949	3.308	1.601	0.427	0.107	--	--	—
	27		83.071*	5.709	4.429	2.265	2.854	1.476	0.098	0.098	—	—
	31	60.347	11.324	4.047	5.483	7.311	7.702	3.394	0.392	—	—	—
	40	53.329	12.186	4.496	4.770	5.811	5.976	11.897	1.425	0.055	0.055	--
	41	60.224	15.339	4.180	4.180	8.253	5.895	1.393	0.214	0.214	0.108	—
	42	57.185	11.055	4.425	6.096	6.981	9.342	4.425	0.295	0.098	0.098	—
	43		65.192*	4.259	3.604	6.224	8.518	10.975	1.228	--	—	—
	44	68.581	9.797	3.153	4.054	4.730	4.279	4.279	1.127	—	—	—
	45		70.304*	6.977	7.335	7.871	4.830	2.504	0.179	—	—	—
	Averages		75.143*	4.183	4.613	5.688	5.399	4.358	0.525	0.065	0.026	—
	2000-2999	64.445	11.284	4.419	5.017	6.434	5.174	2.903	0.252	0.062	--	—
	3000-3999	69.086	7.762	3.485	3.947	5.629	4.398	4.269	0.669	0.054	0.027	—
	4000-4999	56.244	10.634	4.461	5.433	6.396	7.659	8.161	0.860	0.077	0.077	—
	5000-over	67.869	15.202	5.709	4.429	2.265	2.854	1.476	0.098	0.098	—	—
1954	4	65.866	10.016	3.824	4.412	6.471	4.706	3.235	0.882	0.588	--	—
	10	80.402	7.507	1.849	2.276	2.276	3.414	2.134	0.142	—	—	—
	27		74.326*	5.816	7.376	5.532	3.262	2.411	1.277	--	--	—
	31	53.787	10.093	5.017	9.197	9.030	7.023	4.181	0.836	0.836	—	—
	40	52.919	11.919	4.808	4.921	5.769	8.428	11.143	0.735	0.113	--	—
	41	51.283	13.062	6.257	5.313	7.084	9.091	6.257	1.535	0.118	--	—
	42	52.104	10.073	6.061	6.061	6.173	11.672	6.958	0.786	0.112	--	—
	43		58.049*	5.587	5.303	7.102	9.754	13.258	0.852	0.095	—	—
	44	68.454	9.779	5.678	3.470	5.047	2.524	4.101	0.631	0.316	—	—
	45		66.667*	6.996	8.025	6.584	7.613	3.909	0.206	—	—	—
	Averages		67.423*	5.187	5.576	6.108	7.561	7.159	0.817	0.218	—	—
	2000-2999	60.655	10.620	5.410	6.219	6.528	6.160	3.572	0.544	0.294	—	—
	3000-3999	63.364	7.119	4.878	5.112	6.108	6.361	5.986	0.799	0.273	--	—
	4000-4999	57.362	10.845	5.939	6.719	5.853	7.467	4.685	1.032	0.056	--	—
	5000-over	52.356	11.727	4.808	4.921	5.769	8.428	11.143	0.735	0.113	--	—
1955	4	72.907	11.087	2.249	2.778	3.175	3.836	2.778	1.058	--	0.132	--
	10	81.954	7.652	1.422	2.845	2.845	1.860	0.985	0.328	—	0.109	—
	27		79.208*	5.831	5.061	4.620	2.970	1.980	0.330	—	—	—
	31	56.711	10.642	4.881	5.315	7.484	8.894	4.772	1.193	0.108	--	—
	40	51.304	11.723	4.737	5.315	6.355	6.239	12.652	1.444	0.173	--	0.058
	41	59.185	15.075	4.261	4.348	6.174	6.609	3.565	0.696	—	0.087	—
	42	57.425	11.101	4.442	6.238	7.561	6.522	4.820	1.701	0.093	0.095	—
	43		63.744*	3.870	4.976	6.951	8.531	10.742	1.027	0.159	—	—
	44	72.794	10.399	4.412	3.361	3.151	2.731	2.101	0.840	0.211	—	—
	45		75.138*	3.683	3.499	6.630	6.446	3.867	0.737	—	—	—
	Averages		72.762*	4.072	4.607	5.769	5.799	5.861	0.997	0.074	0.042	0.006
	2000-2999	71.479	12.515	2.249	2.778	3.175	3.836	2.778	1.058	—	0.132	—
	3000-3999	65.049	7.308	4.212	4.288	6.054	6.651	5.371	0.949	0.120	--	—
	4000-4999	68.142	12.883	3.838	4.085	4.546	3.813	2.177	0.451	—	0.065	—
	5000-over	53.740	12.037	4.590	5.777	6.958	6.381	8.736	1.573	0.133	0.048	—

* Under 9000.

TABLE 9. (Continued)

YEAR	LOADOMETER STATION OR TRAFFIC VOLUME GROUP	PERCENT OF TOTAL AXLES BY WEIGHT GROUP										
		UNDER 7000	7000 TO 9000	9000 TO 11000	11000 TO 13000	13000 TO 15000	15000 TO 17000	17000 TO 19000	19000 TO 21000	21000 TO 23000	23000 TO 25000	25000 TO 27000
1956	40	49.607	11.565	5.337	5.913	7.902	10.152	8.373	0.942	0.209	--	--
	43	49.509	9.278	5.263	8.118	10.883	8.296	7.583	0.981	--	0.089	--
	46	59.381	9.897	5.567	3.711	4.536	8.041	8.041	0.826	--	--	--
	47	48.319	9.349	5.777	7.038	9.664	11.450	7.983	0.315	0.105	--	--
	48	51.466	13.145	4.146	6.876	10.010	8.392	5.763	0.202	--	--	--
	49	47.327	11.881	5.545	10.890	10.297	6.337	7.129	0.594	--	--	--
	50	78.076	7.159	3.132	2.685	2.908	3.356	2.237	0.447	--	--	--
	51	79.321	5.493	2.262	2.100	4.523	3.393	2.262	0.646	--	--	--
	52	57.547	9.691	4.717	5.746	7.118	7.290	6.947	0.858	0.086	--	--
	53	69.419	7.645	3.058	3.670	6.728	4.587	3.670	1.223	--	--	--
	Averages	55.585	10.045	4.753	6.055	8.026	8.050	6.689	0.716	0.050	0.009	--
	1000-1999	69.419	7.645	3.058	3.670	6.728	4.587	3.670	1.223	--	--	--
	2000-2999	58.317	8.907	4.528	6.675	8.161	7.059	5.791	0.518	0.035	--	--
	3000-3999	63.793	8.219	4.198	5.402	6.896	5.826	4.910	0.714	--	0.045	--
	4000-4999	48.319	9.349	5.777	7.038	9.664	11.450	7.983	0.315	0.105	--	--
	5000-over	52.868	11.466	4.733	6.178	8.342	8.610	7.027	0.607	0.098	--	--
1957	40	53.785	12.100	5.011	5.064	5.171	6.503	8.635	3.305	0.159	0.267	--
	43	55.882	9.349	4.517	5.882	9.349	7.038	7.248	0.630	0.105	--	--
	46	56.305	14.032	8.881	11.190	6.927	2.131	0.178	--	0.178	--	0.178
	47	51.488	9.447	6.043	5.363	10.383	8.511	8.085	0.426	0.170	0.085	--
	48	45.184	11.534	4.518	5.886	9.572	12.188	9.869	1.249	--	--	--
	49	47.619	13.741	4.490	7.075	11.701	6.803	6.666	1.769	0.136	--	--
	50	68.612	6.625	5.205	4.732	4.732	4.574	4.101	1.261	0.158	--	--
	51	70.681	8.377	3.403	3.796	5.236	4.450	3.272	0.785	--	--	--
	52	53.182	11.675	5.779	7.064	8.757	6.538	5.196	1.751	0.058	--	--
	53	58.621	11.576	5.911	5.172	6.404	6.650	4.680	0.986	--	--	--
	Averages	56.136	10.846	5.376	6.122	7.823	6.539	5.793	1.216	0.096	0.035	0.018
	1000-1999	58.621	11.576	5.911	5.172	6.404	6.650	4.680	0.986	--	--	--
	2000-2999	57.622	11.374	5.323	6.986	8.303	5.106	4.341	0.796	0.105	--	0.045
	3000-3999	68.612	6.625	5.205	4.732	4.732	4.574	4.101	1.261	0.158	--	--
	4000-4999	51.488	9.447	6.043	5.363	10.383	8.511	8.085	0.426	0.170	0.085	--
	5000-over	50.717	11.770	5.103	6.005	7.833	8.410	7.900	2.102	0.072	0.089	--
Typical Values	0-399	89.274	3.761	2.635	1.692	0.754	0.754	0.943	0.189	--	--	--
	400-999	84.214	5.692	2.401	2.891	1.958	1.600	0.976	0.089	0.133	0.044	--
	400-999	82.216	3.162	2.106	2.503	1.976	1.843	2.106	3.032	0.790	0.132	0.132
	Haul Roads 1000-1999	81.359	4.791	2.983	2.945	2.927	2.200	1.398	1.007	0.280	0.095	--

TABLE 10. ACTUAL TRAFFIC ACCUMULATION EXPRESSED AS PERCENT OF DESIGN TRAFFIC

COUNTY	PROJECT	DESIGN EWL'S (MILLIONS)	ACCUMULATIVE PERCENTAGE OF DESIGN TRAFFIC BY CHRONOLOGICAL AGE									
			1 YEAR	2 YEARS	3 YEARS	4 YEARS	5 YEARS	6 YEARS	7 YEARS	8 YEARS	9 YEARS	10 YEARS
Anderson	F 208(4)	6.0	4.86	10.10	18.11	32.21	48.93					
Ballard-McCracken[1]	AS 63(4)											
Barren-Hart[1]	F 28(5) F 7(5)											
Bell	F 151(7) F 21(5)	23.4	4.41	9.52								
Bell[1]	U 322 (7)											
Boyd[1]	FI 8(4) F 1(4)											
Boyd	FI 8(6)	8.0	1.37	2.90	6.39	9.80	15.48	23.87				
Boyle	F 244(4)	6.0	22.71	42.17	70.38	83.62	109.06	125.40				
Breckinridge-Meade	F 523(3)	4.0	6.07	12.47	19.22	35.93	55.77					
Carter	FI 4(4)	15.0	0.60	1.45	3.35	5.23	8.60	13.89				
Carter	FI 4(6)	15.0	0.70	2.28	3.87	6.75	11.13					
Carter	FI 13(5)	30.0	0.29	0.64	1.37	2.07	3.33	5.31				
Clay	SP 72(4)	1.0	18.40	36.80	55.49	74.36						
Clinton	S 10(5)	1.0	3.56	7.62	12.11	17.08	22.56					
Cumberland	F 116(10)	12.0	3.32	7.28								
Daviess[1]	F 125(18)											
Daviess-Ohio[1]	F 125(19)											
Elliott-Rowan	S 288(5)	1.0	4.03	8.64	13.80							
Fayette	UI 538(5)	40.0	3.43	5.42	12.19	23.78						
		40.0	10.66	21.36	33.68	58.44						
Fayette[1]	RS 34-304-10											
Fayette-Jessamine	F 524(4) F 524(5)	15.0	5.27	10.75	17.45	21.99	28.50	39.25	48.81	64.57	85.70	
Fayette-Madison[1]	FI 124(4)											
Franklin-Woodford	F 326(22)	20.0	7.89	19.59	35.20	49.92	67.08	84.05	104.92			
		15.0	10.52	26.12	46.94	66.56	89.44	112.07	139.90			
Garrard	S 366(2)	20.0	3.26	8.26	12.08	18.26	26.42					
Garrard[1]	F 525(4)											
Garrard	F 525(5)	13.0	5.89	15.11	26.95							
Garrard[1]	F 525(3)											
Graves	F 146(19)	35.5	7.35	16.67								
Grayson	S 462(4)	0.5	1.47	3.00	4.60	6.36	27.78					
Harrison	F 189(5)	15.0	0.98	4.58	9.36	13.86	17.04	20.47	28.10			
Harrison	F 189(6)	5.0	6.01	14.01	21.44	25.13	31.51	46.77				
Hart	FI 169(12)	35.0	6.61	18.47	25.30	35.14	49.68					
Henderson[1]	F 526(9)											
Henderson-Webster[1]	F 526(12)											
Henry	S 552(1) SG 552(2)	2.5	1.93	4.00								
Henry	F 536(3)	2.5	8.12	27.66	50.34							
Hopkins[1]	F 526(6)											
Hopkins[1]	F 526(7)											
Jefferson[1]	U 528(2)											
Jefferson[1]	U 528(10)											
Jefferson[1]	U 528(12)											
Jefferson	U 528(14)	80.0	10.84	29.35								
Jefferson[1]	U 528(16)											
Johnson[1]	77 A(23)											
Knox[1]	S 72(3)											
Laurel	S 150(4)	1.0	2.61	7.10	12.91	56.84	111.26					
Laurel	FI 29(9)	35.0	6.19	10.89	20.51	32.25	43.64	56.69	70.29			
Laurel	FI 517(6)	30.0	4.63	7.85	14.19	23.38	32.59	43.61	55.35			
Lawrence[1]	F 78(6)											
Livingston-Lyon[1]	F 530(6)											
Madison	FI 299(6)	40.0	6.12	8.73	13.18	19.89	26.66	34.62	42.13			
Marshall	F 163(9)	10.0	2.50	5.36	8.33	11.38	14.56	22.14	30.87			
Marshall-McCracken[1]	F 530(8)											
Mercer	F 294(2)	8.0	10.75	19.46	34.52	45.05						
Muhlenberg[1]	F 40(6)											
Nelson[1]	F 222(4)											
Nicholas[1]	F 234(9)											
Nicholas	F 234(9)	2.8	1.84	7.52	14.91	23.32	32.97	66.90	109.58			
Ohio[1]	SP 92-224											
Ohio	S 473(2)	4.0	2.68	6.63	11.67	16.82	22.10					
Pulaski	SP 100-235(6)	10.0	7.11	14.56	23.26	32.17	43.87	56.33	65.05			
Pulaski[1]	F 502(4)											
Pulaski[1]	U 110(4)											
Rockcastle	FI 70(6)	80.0	3.74	7.06	11.06	15.22						
Rockcastle	FI 88(6)	40.0	5.19	10.74	14.06	19.90	27.[illegible]9	34.26	42.53	51.11		
Rockcastle	FI 517(7)	30.0	4.81	7.68	13.99	21.82	31.03	4[illegible].51	52.33			
Rowan[1]	FI 3(8)											
Warren	FI 113(5) FI 16(2)	50.0	8.44	20.38	37.72							
Webster[1]	F 526(10)											
Webster	F 526(13)	18.0	7.55	18.44	29.77	45.06	61.97					
Whitley[1]	FI 23(16)											

1 Incomplete traffic data.