# Re-Evaluation of Kentucky Flexible Pavement Design Criterion

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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 Subgrade Support in the Deisgn 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.

● 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, that is, 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 Subgrade 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)

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

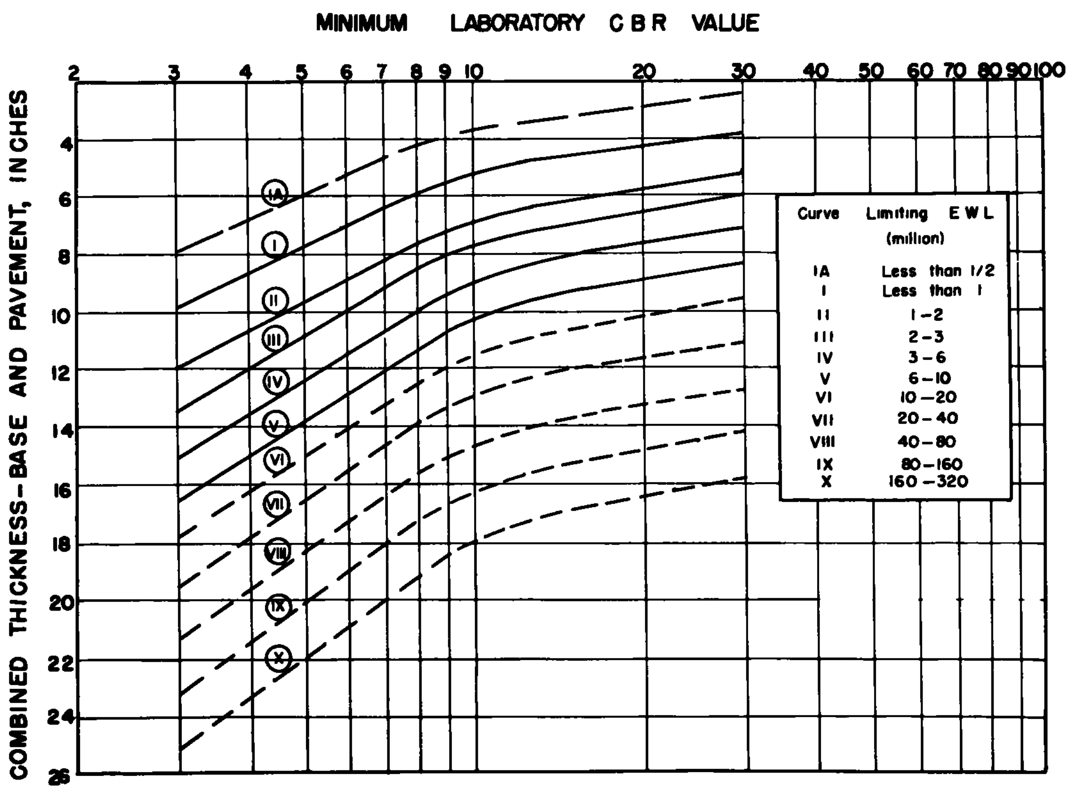



Figure 1. Kentucky flexible pavement design curves. Curves I through V proposed in 1948; IA and VI through X added by extrapolation, 1954.

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 one 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 one 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). 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.

### 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 and Materials Divisions.

### Evaluation of Traffic

Traffic data were obtained from the Division of Planning. For most of the projects, the ADT (Average Daily Traffic, two directions) for each year, from the time the 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. With this information, it was possible to calculate the EWL's which had passed over the pavements and thus to study the traffic history of each project.

By comparing the actual EWL value with the designed 10-yr EWL's, a "traffic-age" or "service-age" for the projects could be determined. Thus, if the actual EWL's at any age exceed 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-hr 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

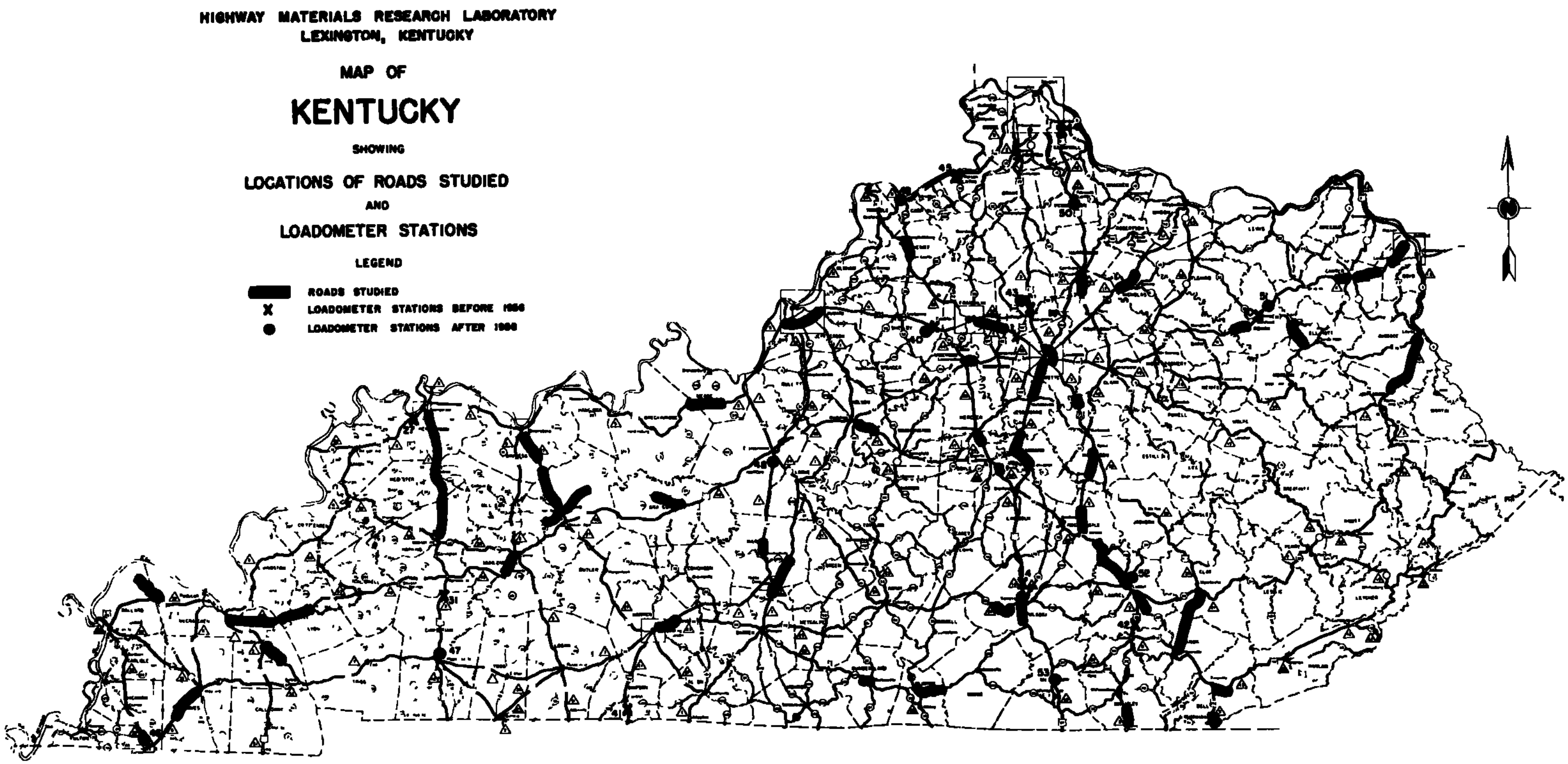



Figure 2. Locations and distribution of projects studied.

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

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 |

Figure 3. Sample calculation for estimating 10-yr EWL's.

or similar loadometer station. These trends in distribution, when applied to the average yearly 24-hr 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. (An example of the method of estimating 10-yr design EWL's for all roads included in this study is given in Figure 3. 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 given in Figure 4.) Thus, if it is also assumed that EWL's would increase in direct proportion to the vol-

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 Item (7) | 5205 | 0.09 | 5295 | 872,809 | 1 | 872,809 |
| $5\frac{1}{2}$-$6\frac{1}{2}$ | | 4.732 | 0.13 | 4862 | 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 |

Figure 4. Sample calculation for estimating 20-yr EWL's.

ume of truck traffic, the accumulation of EWL's at any age throughout the 10-yr period, expressed in percent of the 10-yr estimate, could be described theoretically by:

percent of 10-yr estimated EWL = $6.67x + 0.333x^2$

where x = chronological age in years.

The equation above describes the "theoretical curve," curve No. 1, shown in Figure 5. 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.

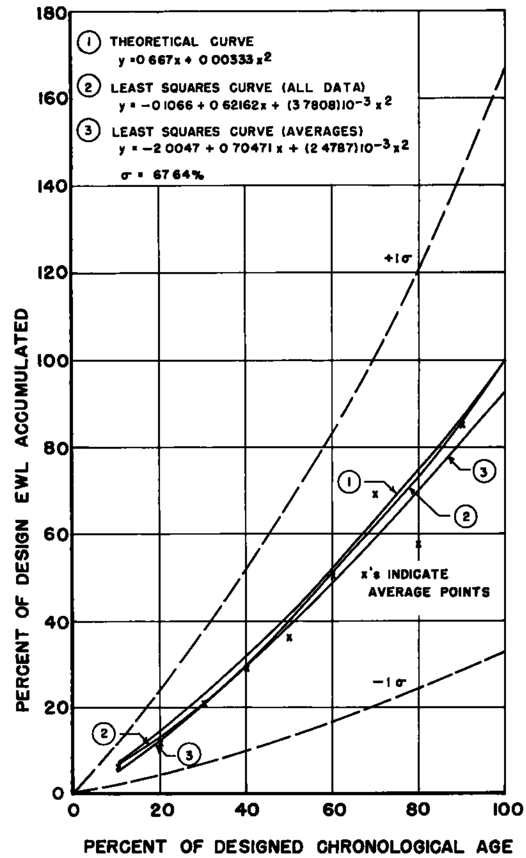



Figure 5. Statistical relationship between percent of designed EWL accumulated and percent of designed chronological age.

While there is wide variance among the data (standard deviation = ± 67.64 percent); the average or trend shows close agreement with the 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 percent confidence limit, would be equivalent to ± 3 standard deviations or approximately ± 200 percent. Expressed on a 75 percent confidence limit basis, the extreme deviations, of course, would not exceed ± 1.15 x 67.64 percent. It may be similarly stated, therefore, that 15.9 percent of the roads accumulated traffic at a rate 1.68 times greater than the predicted rate. Likewise, 15.9 percent of the roads would reach 100 percent of their designed traffic-age within 68 percent or less of their designed life-expectancy. To be precise, statistically speaking, the mean square error could have been used 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 percent of the roads in this series may vary between 68 and 168 percent 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-yr 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-yr design and a 20-yr design, assuming that the 20-yr estimate of EWL's exceeds the 10-yr estimate by a factor of 2, would be about 1½ in.

## PERFORMANCE SURVEYS

### 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—longitudinal, 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.

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

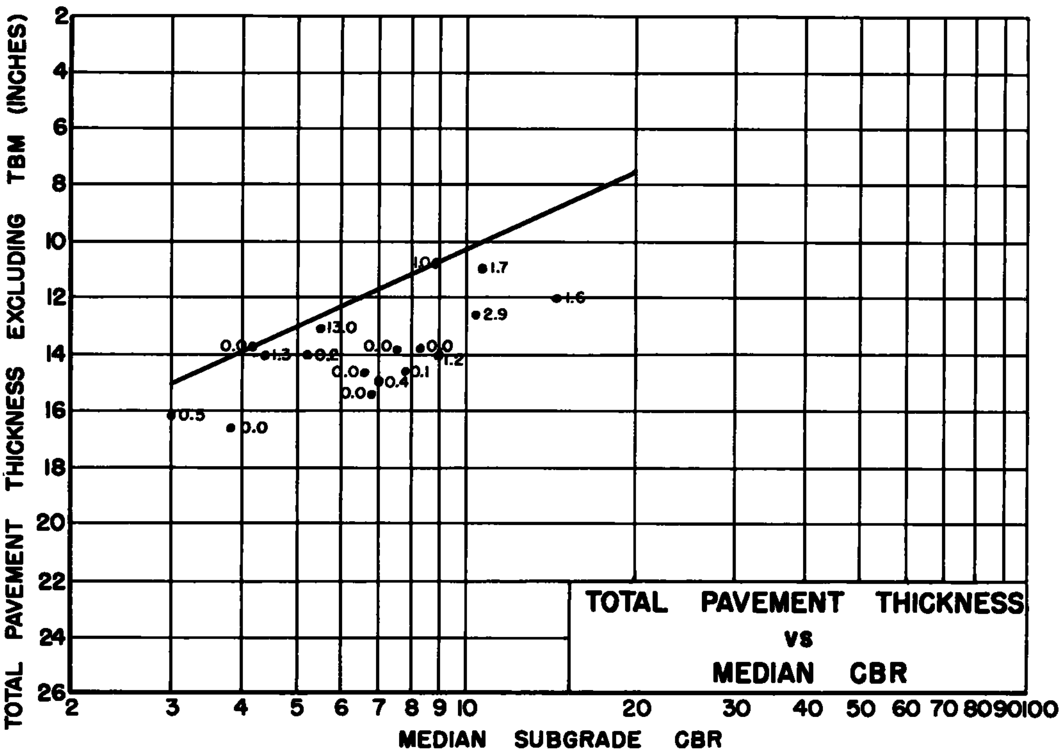



Figure 6. Pavement thicknesses vs median subgrade CBR's for all projects in which the accumulated EWL's fell within the limits of Traffic Group IV, 3 to 6 million.

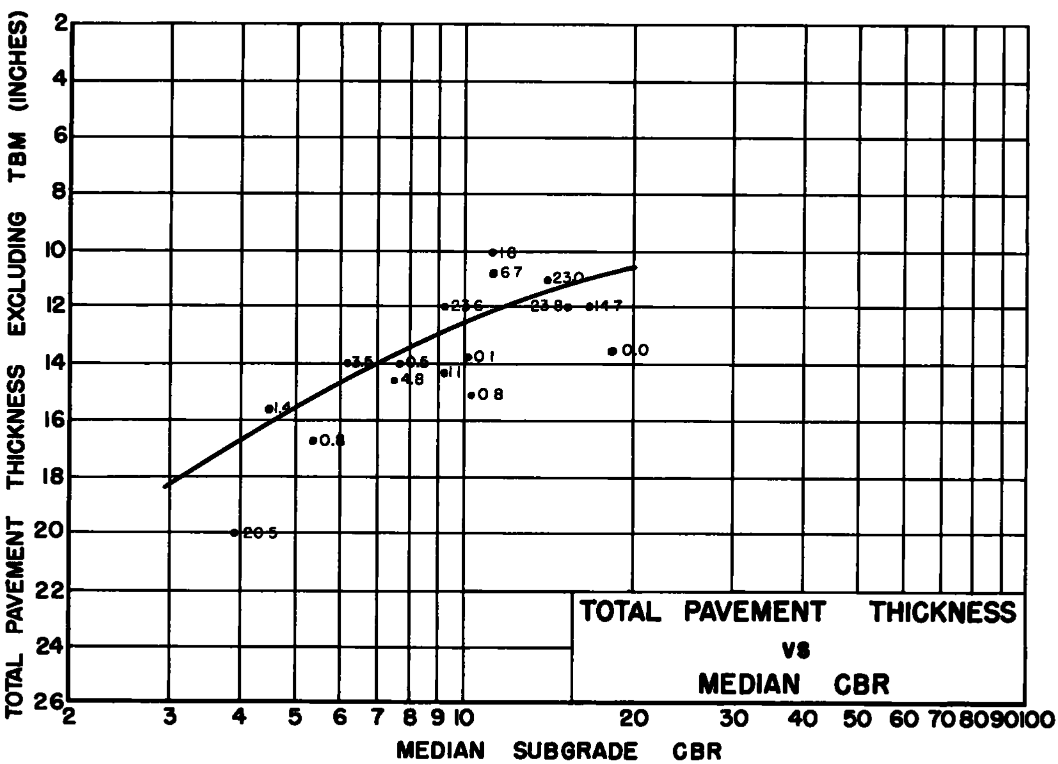



Figure 7. Pavement thicknesses vs median CBR's for all projects in which accumulated EWL's fell within Traffic Group VI.

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 Figure 6. 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.

Data for projects in Traffic Group VI plotted in the same manner are shown in Figure 7. 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 Figure 8, the curves in Figures 6 and 7 are shown superimposed upon the original design curves and may indicate a need for slight revision of the design chart.

## Rutting

Rutting measurements were made by laying a straightedge transversely across a traffic lane and measuring the maximum deviation (see Fig. 9). 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 Figure 10. However, throughout this report the term "rutting" implies the total deviation from a straightedge. Measurements were made at more-or-less random intervals. From these measurements, a simple arithmetic mean of all values was computed for each project.

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

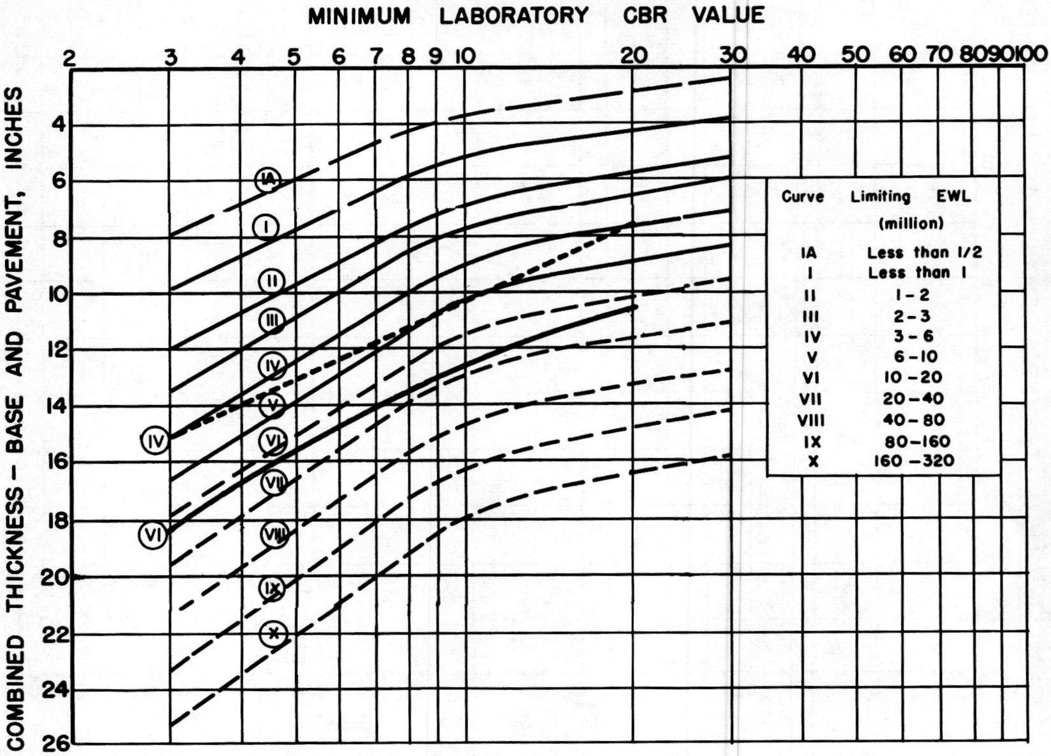



Figure 8. Present Kentucky design chart showing trend lines from Figures 6 and 7 superimposed.

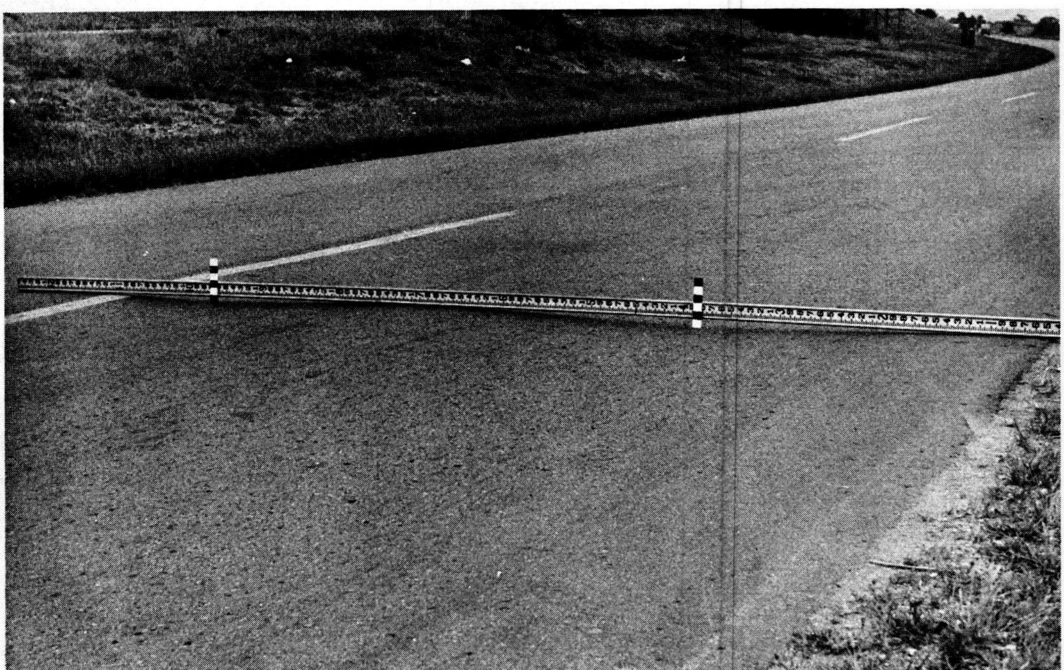


Figure 9. Rutting of pavement within wheel tracks as deviation from a straightedge.

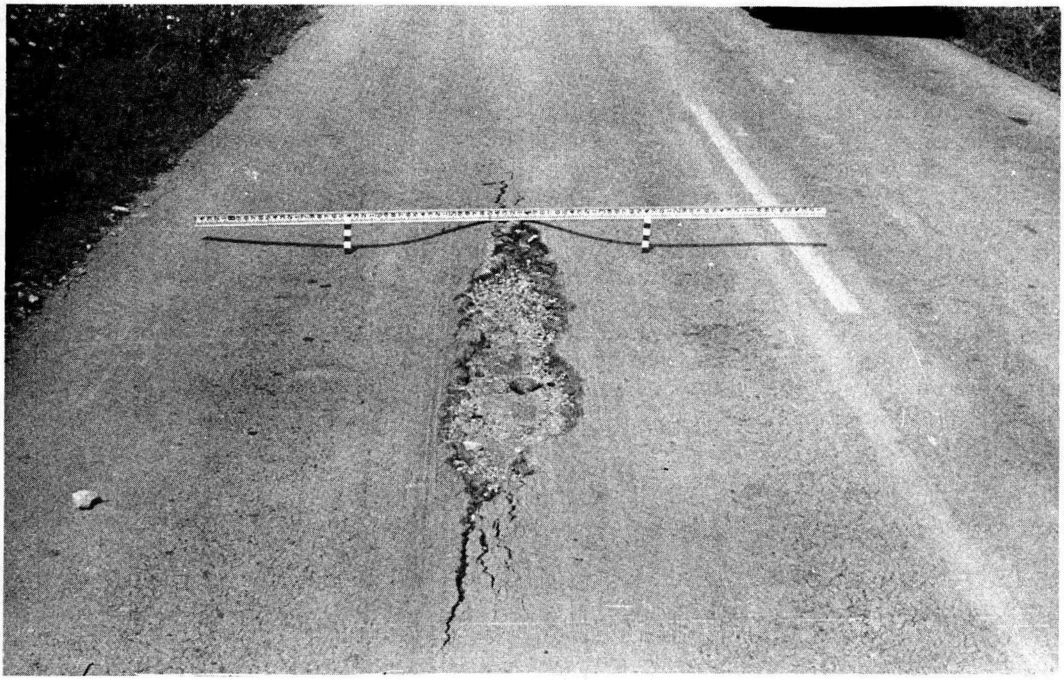


Figure 10. Extreme rutting and upheaval.

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 percent of the rutting occurred within the bituminous layers while 72 percent 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 waterbound 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.

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 Figure 11. 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

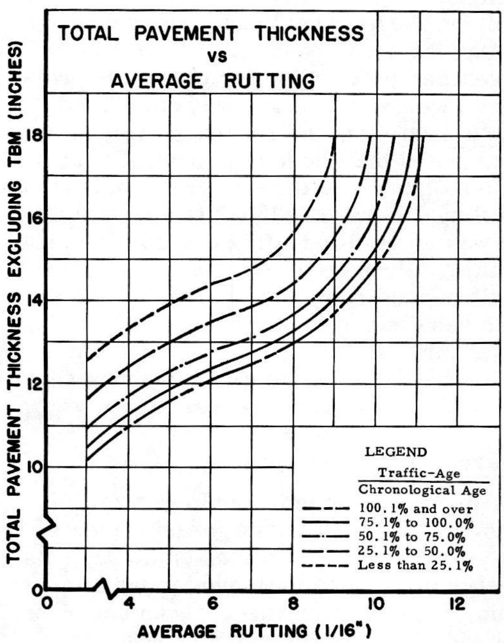



Figure 11. Generalized apparent relationship between thickness and rutting, according to traffic-age.

conditions causing rutting are more critical in the thicknesses designed for high intensities of traffic.

## 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 ¼ in. per sec, each inch of chart length representing an elapsed time of 4 sec, and the galvanometer sensitivity pre-set to 2 in. per g, it was possible to resolve the total area beneath the curve into g sec (1 sq in. = 2 g sec); and:

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

$$\frac{\text{Total g sec}}{\text{Total time}} = \text{Avg g}$$

(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 per in. or 5/4 cps, giving a period of 0.8 sec per 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 per 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.

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 Figure 12 indicates that roughness decreases as the bearing capacity of the subgrade increases.

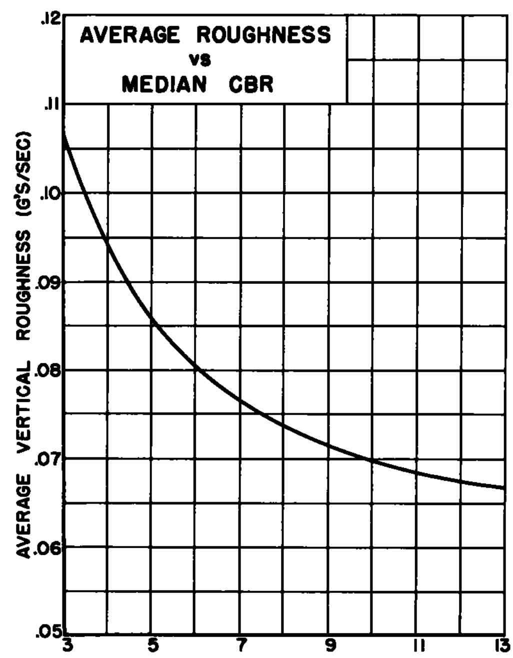



Figure 12. Generalized apparent relationship between average roughness values and median CBR of the subgrades.

## 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

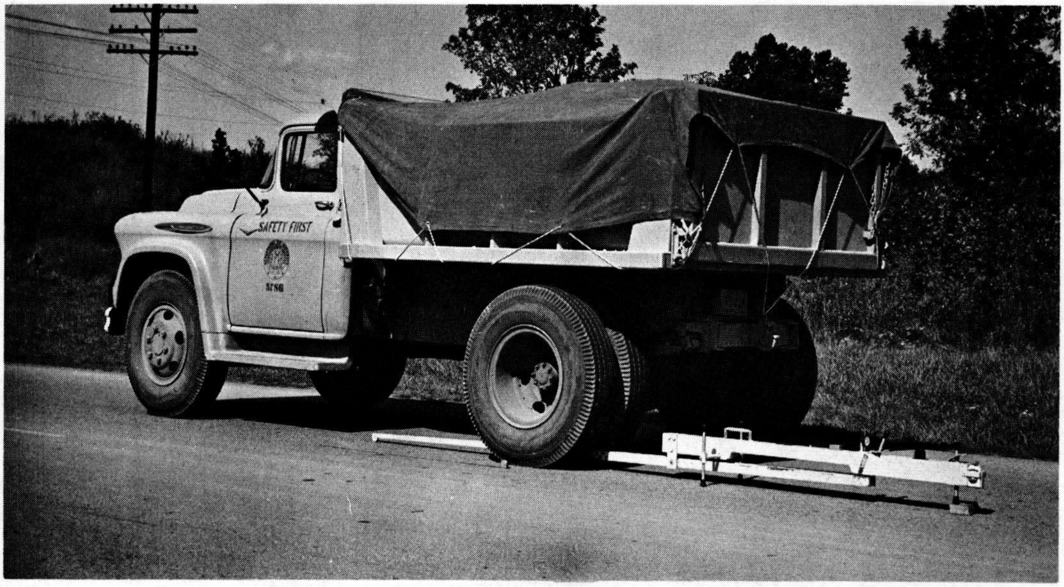


Figure 13. Benkelman Beam in-place for measuring pavement deflection under 18,000-lb single axle.

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. 13). 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 lb 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. 14). 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.

Most of the measured deflections occurred over about a 3-ft span. Deflection increased slowly until the wheel was within 12 to 18 in. of the probe and then increased rapidly to the maximum. This condition was more pronounced in the thinner pavements. Thick pavements and pavements having one or more courses of bituminous base deflected less and over a wider area of pavement surface. Least deflections were measured on pavements made up of full depth bituminous concrete. For the fall series of measurements, pavements with standard waterbound macadam bases averaged 0.028 in. of deflection. Minimum design pavements having one 3- or 3½-in. thickness of waterbound base averaged 0.048 in. of deflection while pavements having 3 in. or more of bituminous base averaged 0.015-in. deflection.

For the spring 1958 series of measurements, pavements with waterbound bases averaged 0.033-in. deflection, the minimum design pavements with waterbound bases averaged 0.078-in. deflection, and pavements with bituminous bases averaged 0.016 in. This increase in deflection during the spring was expected since the influence of sub-

Figure 14. Benkelman Beam in-place for measuring pavement deflection under 32,000- or 36,000-lb loads on tandem axles.

grade moisture would logically be greater during this period. The three locations measured during both periods on pavements having their total depth made up of bituminous concrete averaged 0.003 in. less deflection for the spring measurements. This decrease might be attributed to the lower temperatures at the time of spring testing.

The average difference between inner wheel track and outer wheel track deflections for the fall measurements was only 0.0003 in.; however, the outer wheel tracks averaged 0.005 in. more deflection than 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 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 Figure 15. Pavements marked unsatisfactory were showing patching or cracking at or near the point measured. The curve best 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, Figure 16 relates permissible 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. 17). 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 Figures 16 and 17 and were plotted as shown in Figure 18.

According to Figure 18, pavement thicknesses should be increased in proportion to

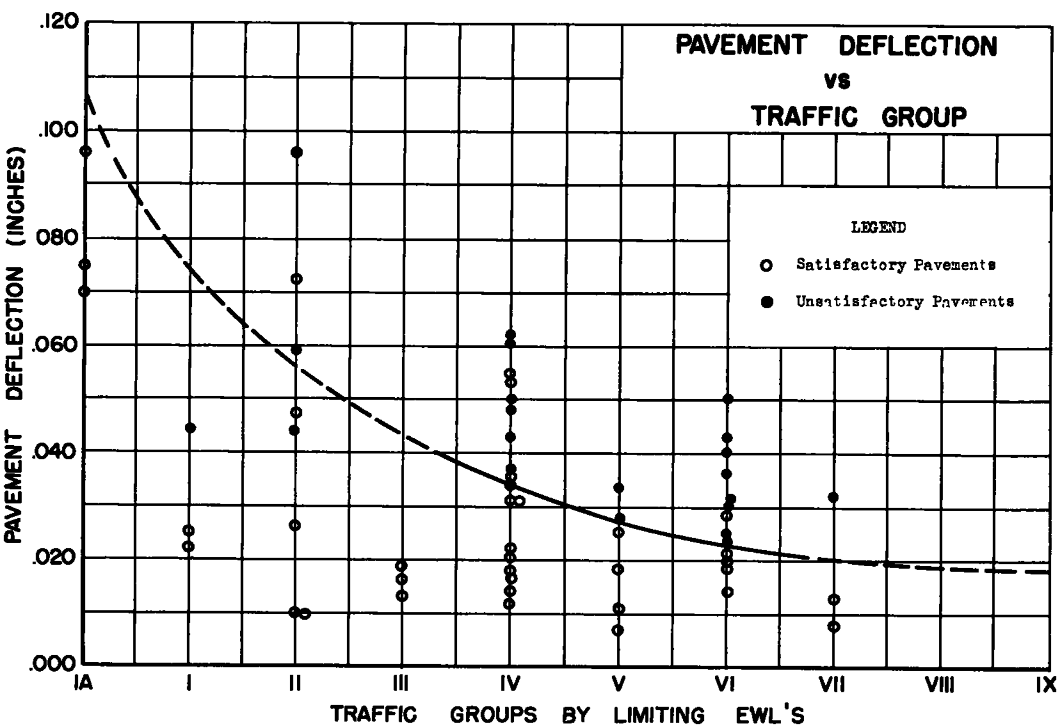



Figure 15. Pavement deflections, 18,000-lb single axle, obtained from both satisfactory and unsatisfactory pavements plotted according to traffic groups corresponding to accumulated EWL's.

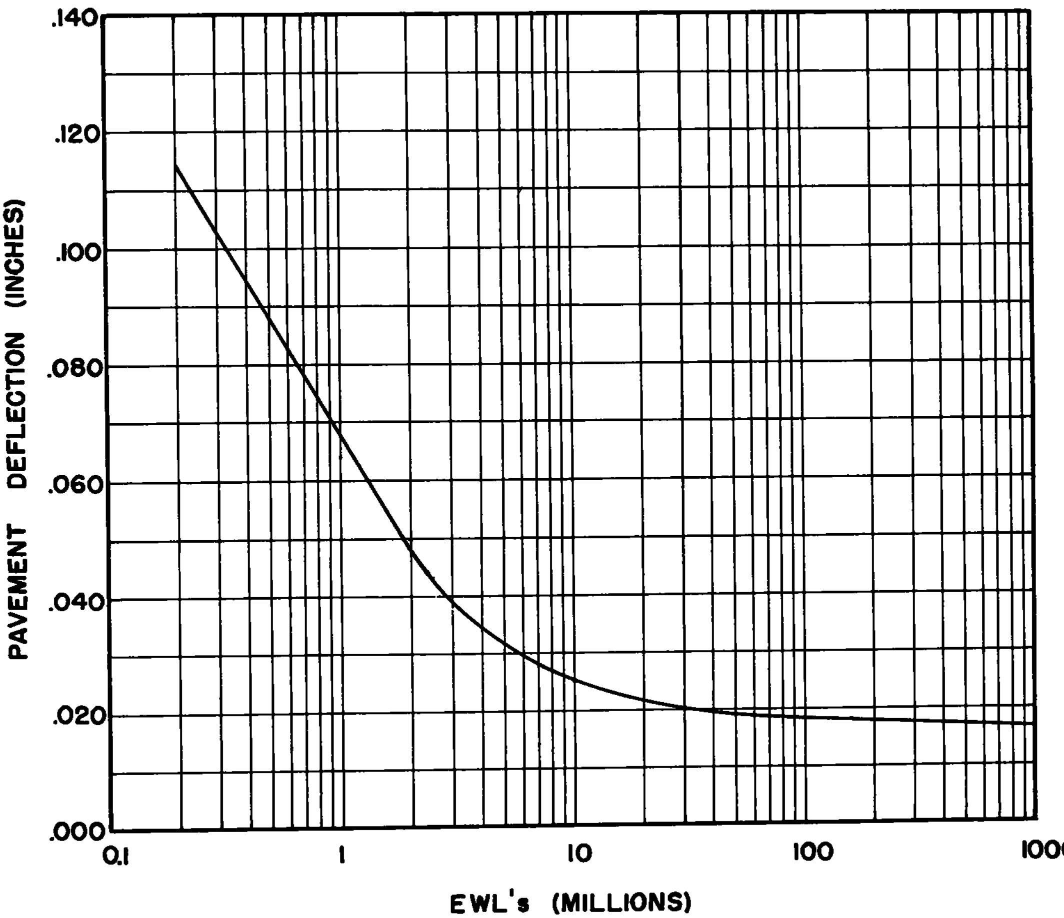
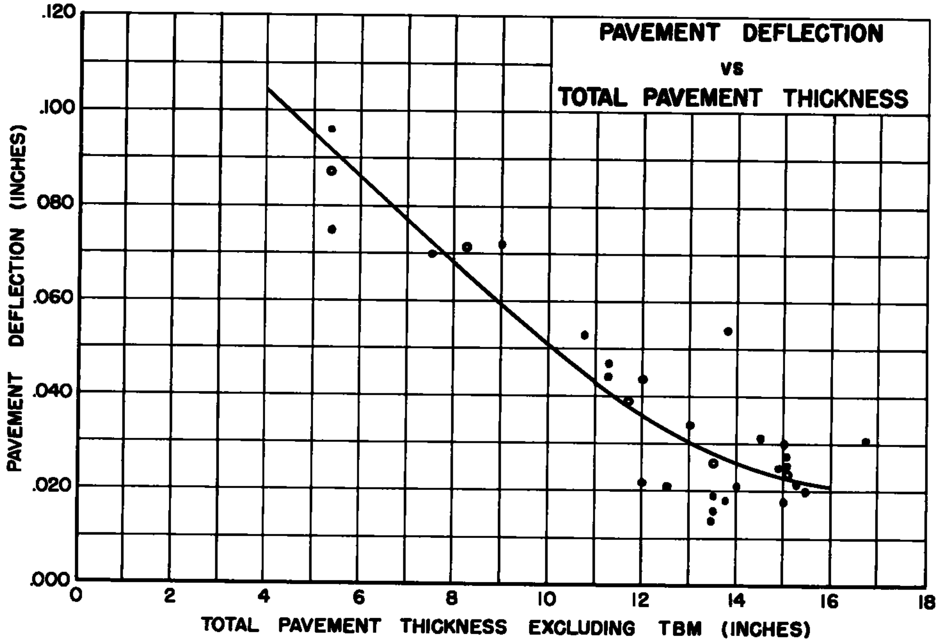



Figure 16. Pavement deflections as obtained in Figure 15, plotted according to the logarithm of the mid-point value of the respective EWL groups.

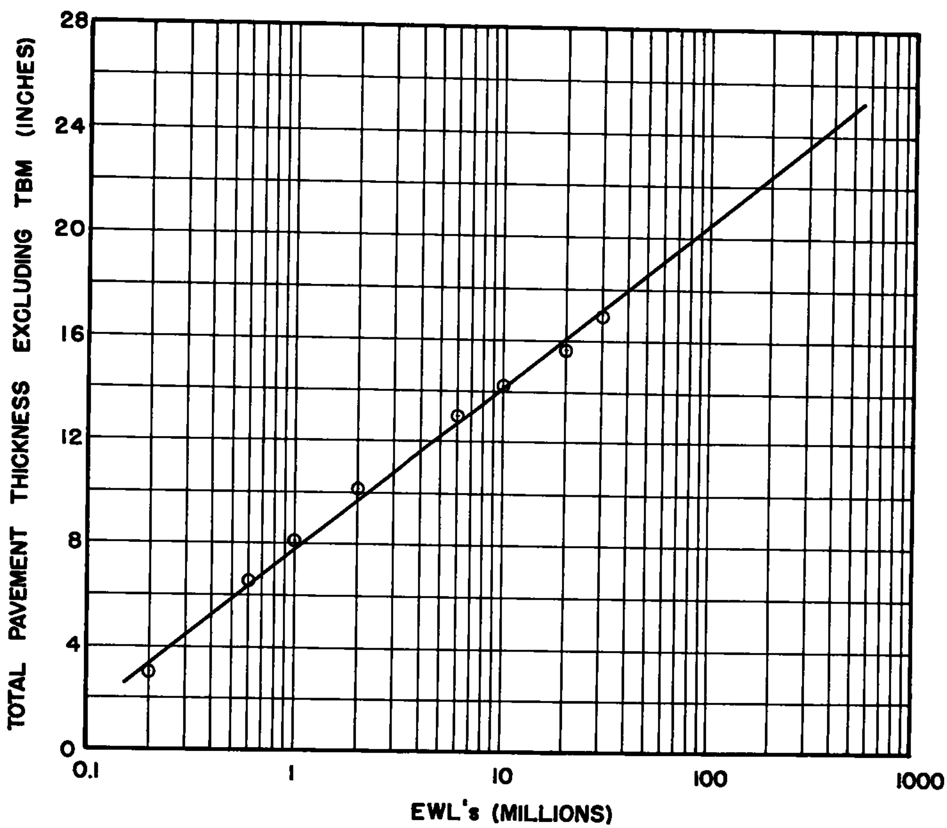
the logarithm of the EWL's. This relationship appears to have been derived more-or-less independently of any parameter describing subgrade support. However, it is rather evident, since each pavement involved in the derivation was originally designed on the basis of a subgrade support parameter, that Figure 17 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 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 distributio of project median CBR's was made; and it was found that 90 percent 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 Figure 18, thicknesses for each of the EWL groups were interpolated from Figure 18 and replotted at CBR 7.1 on the original design chart as shown in Figure 19. 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. 17), the points would naturally reflect safe design thicknesses but not necessarily




igure 17. Pavement deflections representing only satisfactory pavements plotted according to corresponding pavement thicknesses. The curve implies that deflections of equal or lesser magnitude would be within safe limits.




gure 18. Plot of thicknesses and EWL's interpolated from Figures 16 and 17 for corresponding deflections.

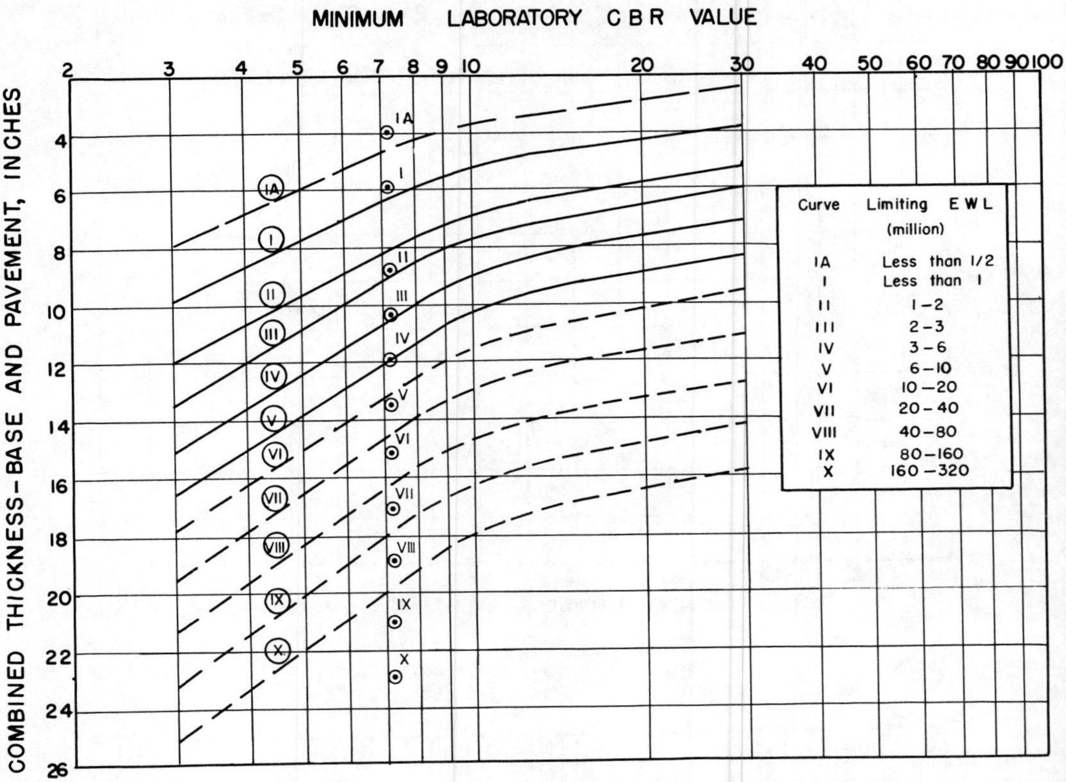



Figure 19. Present design chart showing thicknesses for each median point of the traffic volume groups, taken from Figure 18, plotted at an average CBR of 7.1.

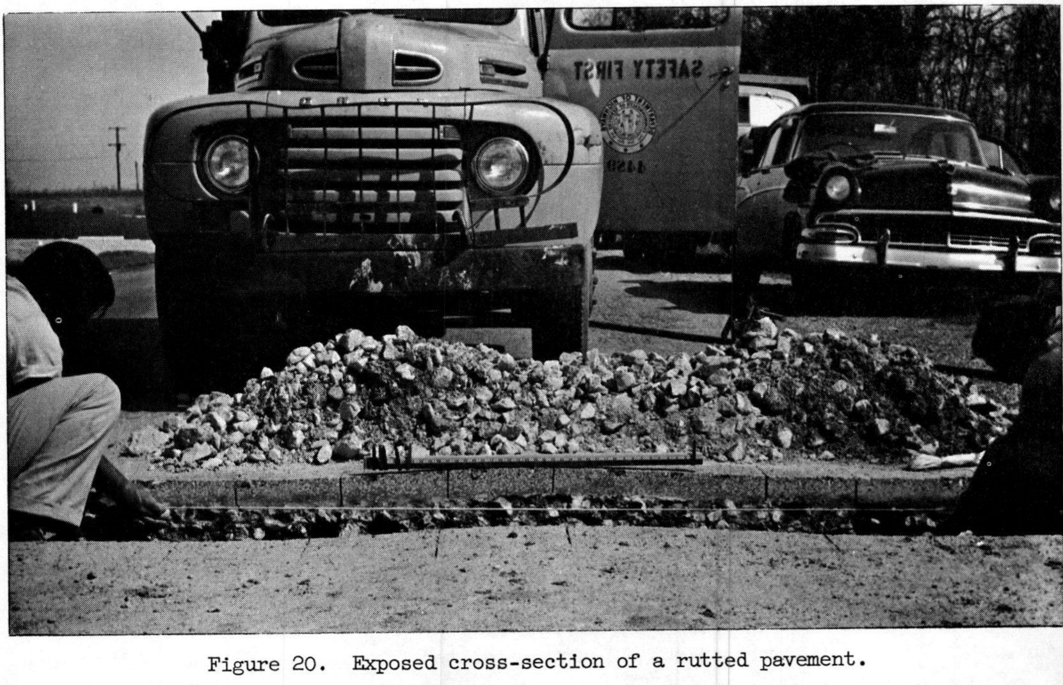



Figure 20. Exposed cross-section of a rutted pavement.

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 Figure 8.

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. 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

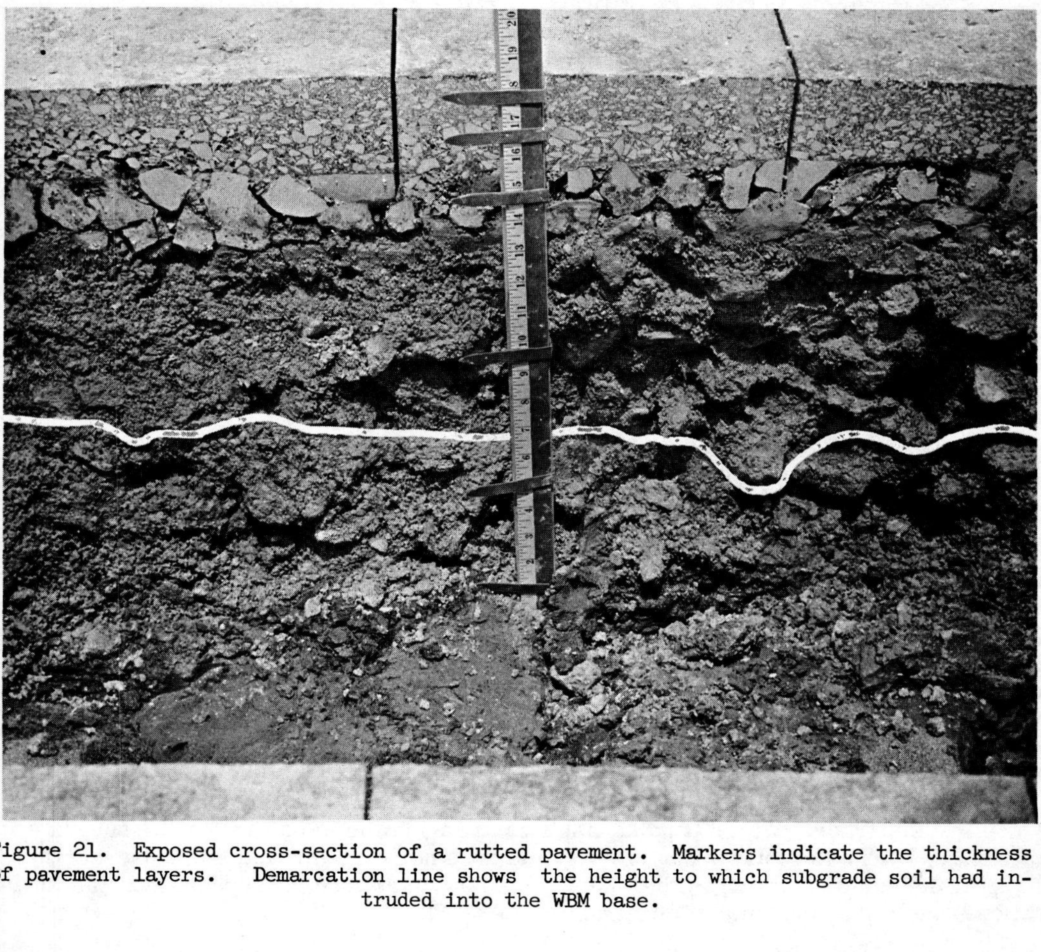


Figure 21. Exposed cross-section of a rutted pavement. Markers indicate the thickness of pavement layers. Demarcation line shows the height to which subgrade soil had intruded into the WBM base.

returned to the laboratory for other testing. 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 in. in some places (see Figs. 20 and 21). 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.

## 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 waterbound 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.

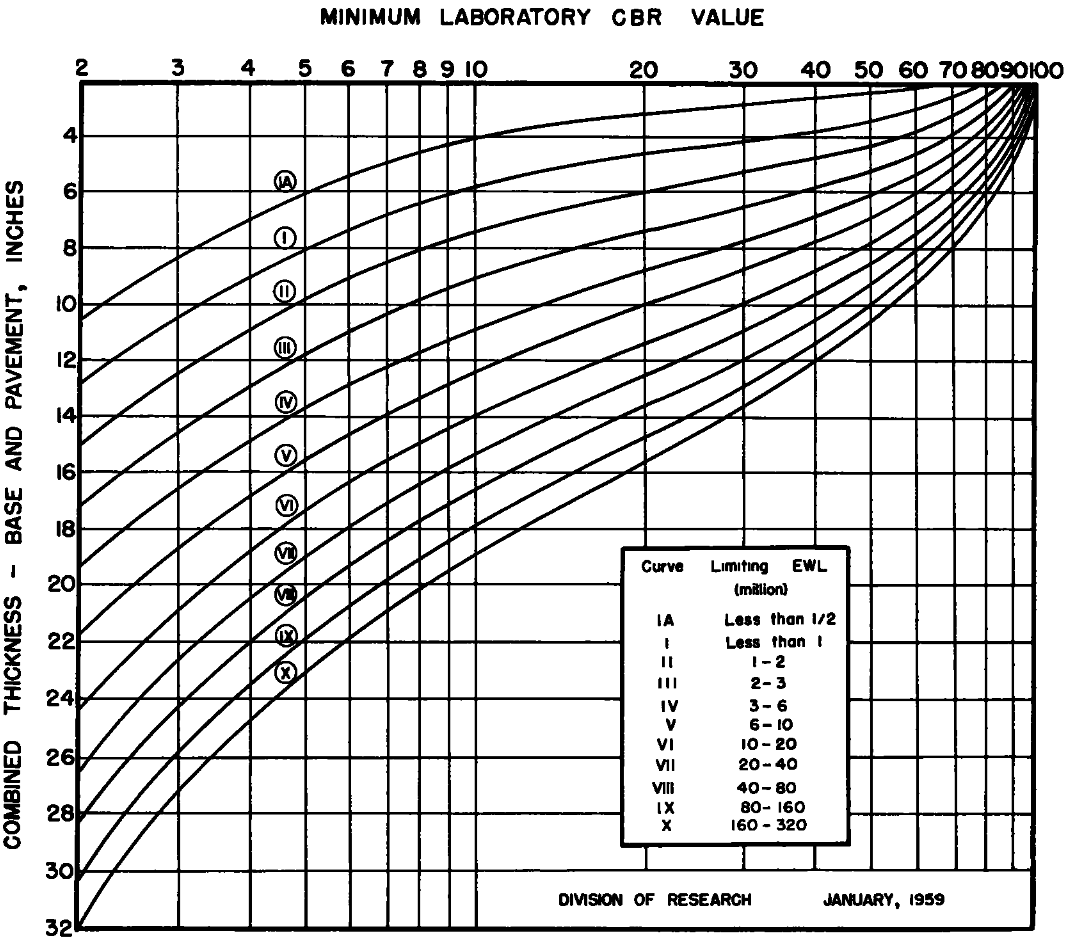



Figure 22. Revised flexible pavement design curves.

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 susceptible 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 Figure 22 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.

ACKNOWLEDGMENTS

The authors wish to acknowledge the cooperation and assistance of many individuals in the Kentucky Department of Highways: D. H. Bray, State Highway Engineer; A. O. Neiser, Assistant State Highway Engineer, who through his special interest in pavement design requested that the study be made and suggested specific approaches that should be investigated; George D. Aaron, Acting Director, J. R. Pulliam, Engineer of Traffic Surveys, J. J. Constantine, Head of the Statistical Section, Division of Planning J. A. Spears, Director of Maintenance; W. B. Carrington, Director, and S. T. Collier, Assistant Director, Division of Design; and J. A. Bitterman, Director, and O. R. Threlkeld, Soils Engineer, Division of Materials. The cooperation of the District Engineers and their Maintenance Assistants aided materially in the field operations.

Robert C. Deen, Research Engineer and Head of the Soils Section, presently on edu- cational leave of absence, was directly responsible for assembling the design data, the soil testing, and analysis and some phases of the performance survey.

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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## *Discussion*

S. M. FERGUS, Standard Oil Company—Messrs. Drake and Havens have prepared an excellent paper setting forth the design method for flexible pavement presently in use by the Kentucky Department of Highways and have presented a re-evaluation of that method based on a statistical study of actual pavement performance. As a result of the study, it is suggested that the present curves be redrawn to provide slightly greate thicknesses.

This paper should be of great interest to highway engineers everywhere since it pro vides usable data for design evaluation which takes into consideration not only the axle loads but also the number of axles using the pavement. In this respect it is unfortunat that the curves are designated for Equivalent Wheel Loads (EWL's) only, rather than i terms of particular axle loads.

A study of the data reveals that inherent in the method is the assumption that a com- putable relation exists between a certain number of coverages with one axle load and a different number of coverages with another axle load. This must be so since different combinations of axle load and vehicle number yield the same value for EWL. It can be shown that this relation is given by the equation

$$N_1 (2)^{P_1} = N_2 (2)^{P_2} \tag{1}$$

where $N_1$ and $N_2$ are the numbers of axles carrying respectively, the axle loads (in ton $P_1$ and $P_2$. In a given case, assuming that values for $P_1$, $P_2$, and $N_2$ are known, the equivalent number of coverages $N_1$ can be computed from the expression

$$N_1 = N_2 (2)^{P_2 - P_1} \qquad (2)$$

As an example, assume it is desired to know what effect one coverage with a 10-ton axle load would have in terms of a greater number $N_1$ of coverages with a 5-ton axle load. From Eq. 2

$$N_1 = (1)(2)^{10 - 5} = 32$$

Thus it is shown that one 10-ton axle load is assumed to produce as much wear on the pavement as 32 5-ton axle loads.

It is also evident that the California factor (f) used in the Kentucky design system may be computed from the expression

$$f = (2)^{P - 5} \qquad (3)$$

and that the EWL values for each class (2 directions) may be computed from

$$EWL = 2,000,000(2)^{P - 5} \qquad (4)$$

The relationships between these various values are shown in the following tabulation

| Axle Load | | California[1] Factor | EWL[2] Millions | EWL Class |
|---|---|---|---|---|
| Tons = P | Pounds | | | |
| 2 | 4,000 | 1/8 | 1/4 | IA |
| 3 | 6,000 | 1/4 | 1/2 | I |
| 4 | 8,000 | 1/2 | 1 | II |
| 5 | 10,000 | 1 | 2 | III |
| 6 | 12,000 | 2 | 4 | IV |
| 7 | 14,000 | 4 | 8 | V |
| 8 | 16,000 | 8 | 16 | VI |
| 9 | 18,000 | 16 | 32 | VII |
| 10 | 20,000 | 32 | 64 | VIII |
| 11 | 22,000 | 64 | 128 | IX |
| 12 | 24,000 | 128 | 256 | X |

[1] Computed from Eq. 3.
[2] Computed from Eq. 4.

It is thus shown that in each class the pavement is designed to accommodate 2,000,000 (2 directions) axle loads of the indicated weight or their equivalent in axle loads of other weights. If the Kentucky curves were to be designated by axle loads rather than by EWL's, it would be necessary for design purposes to determine the equivalent design load rather than the total EWL. This could be accomplished by means of the equation

$$N_1 (2)^{P_1 - 5} + N_2 (2)^{P_2 - 5} + N_3 (2)^{P_3 - 5} + \text{etc.} = 2,000,000(2)^{P_d - 5} \qquad (5)$$

Assuming that values for $N_1$, $N_2$, $N_3$, etc. and $P_1$, $P_2$, $P_3$, etc. are known, the value for $P_d$ can be determined. As a numerical example, take the case presented in Figure 3 of the subject paper. Taking values for N from column (6) and for P from column (1), the following equation is obtained

$$\begin{aligned} 2,000,000(2)^{P_d - 5} &= (416,534)(1) + (378,682)(2) + (478,714)(4) \\ &\quad (434,060)(8) + (448,224)(16) + (128,922)(32) \\ &\quad (12,644)(64) \end{aligned}$$

$$(2)^{P_d - 5} = \frac{18,667,538}{2,000,000} = 9.33$$

$$(P_d - 5) \log(2) = \log 9.33$$

$$(P_d - 5) = \frac{\log (9.33)}{\log (2)} = 3.22$$

$$P_d = 8.22 \text{ tons} = 16{,}440 \text{ lb}$$

The indicated curve is for an axle load of 16,000 lb or curve VI. This is the same as for an EWL of 18,667,538 as given in Figure 3.

For multiple-lane highways, in which the outer lane traffic is more numerous and heavier than that in the passing lane, it may become desirable to express the vehicle frequency in terms of axles per lane per day. In this case, Eq. 5 would become

$$n_1 (2)^{P_1 - 5} + n_2 (2)^{P_2 - 5} + n_3 (2)^{P_3 - 5} + \text{etc.} = \frac{2{,}000{,}000(2)^{P_d - 5}}{(2)\ (365)\ (y)}$$

where the lower case n's indicate that the frequency is in axles per lane per day and where y indicates the design life of the pavement in years.

Taking y = 10 and using again the values from Figure 3, but dividing each value by 7,300 which is the product of 2 times 365 times 10, the computations are

$$274(2)^{P_d - 5} = (57)(1) + (52)(2) + (66)(4) + (59)(8) + (61)(16) + (18)(32) + (2)(64)$$

$$(2)^{P_d - 5} = \frac{2{,}577}{274} = 9.4$$

$$P_d - 5 = \frac{\text{Log } 9.4}{\text{Log } 2.0} = \frac{0.97313}{0.30103} = 3.23$$

$$P_d = 8.23 \text{ tons} = 16{,}460 \text{ lb}$$

this checks the design axle load computed from Eq. 5.

Where the design life is 20 years, the computations are the same except that the values are all divided by 14,600 which is the product of 2 times 365 times 20. Taking the values from column F of Figure 4, the computations are

$$137(2)^{P_d - 5} = (60)(1) + (55)(2) + (56)(4) + (53)(8) + (48)(16) + (15)(32) + (2)(64)$$

$$(2)^{P_d - 5} = \frac{2{,}194}{137} = 16.0$$

$$P_d - 5 = \frac{\text{Log}16.0}{\text{Log } 2.0} = \frac{1.20412}{0.30103} = 4.0$$

$$P_d = 9.0 \text{ tons} = 18{,}000 \text{ lb}$$

The design load of 18,000 lb indicates Curve VII which is the same as for an EWL of 31,691,311, the value shown in Figure 4.

It may seem only an academic question whether the basis of design is in terms of EWL's or axle loads but since it is desirable that knowledge accumulated by engineers in one district should be readily understood by those in other districts and since most methods for determining the design thickness of flexible pavements are based on loading considerations, it would seem desirable to include that factor in Kentucky's curves also.

In any case, a vote of approval should go to the authors and to the Kentucky Department of Highways for an important contribution to flexible pavement knowledge.

W. B. DRAKE and JAMES H. HAVENS, Closure—The authors are grateful to Mr. Fergus for an enlightening and scholarly development which appears to enhance the significance of the EWL method of evaluating traffic as well as the axle load method. Since it is now possible to relate each to the other, interesting comparisons may be made among various design criteria, particularly those involving CBR and EWL's or axle loads as the independent parameters.

There is a slight inconsistency in the progression of EWL's used in the Kentucky design chart. However, Mr. Fergus has rounded this out in his calculations; and, for all practical purposes it seems that the progressions of both the California factor and EWL's should be considered as being truly geometrical.